Guide to Road Design Part 4A

Unsignalised and Signalised Intersections

Austroads

Guide to Road Design Part 4A: Unsignalised and Signalised Intersections

Austroads

Sydney 2023

Guide to Road Design Part 4A: Unsignalised and Signalised Intersections

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Abstract

The Austroads *Guide to Road Design Part 4A: Unsignalised and Signalised Intersections* provides road designers and other practitioners with guidance on the detailed geometric design of all at-grade intersections (excluding roundabouts). This Part contains information for the design of signalised and unsignalised intersections. Guidance is provided on intersection sight distances, including approach sight distance, safe intersection sight distance, and minimum gap sight distance. Left and right-turn treatments are outlined including the incorporation of auxiliary lanes at intersections and the use and size of traffic islands.

Keywords

Signalised intersection, unsignalised intersection, sight distance, approach sight distance, auxiliary lanes, merge tapers, traffic islands, median openings, urban intersection treatments, rural intersection treatments, right-turn treatments, left-turn treatments, U-turn treatments.

ISBN 978-1-925451-73-3

Austroads Project No. SRD6288

Austroads Publication No. AGRD04A-23

Pages 193

Edition 3.2 published May 2023

Edition 3.2 contains editorial and technical changes to sections 2.2, 3.2.2, 3.2.3, 3.3, 3.4, 4.1, 5.2.2, 5.5, 6.1.4, 9, 9.2. It also contains new guidance in Section 8.2.5 for offset rural CHL treatments, Appendix A.5.7 for S-lane treatments and Appendix A.10 for EDD for truck acceleration lanes.

Edition 3.1 published February 2021

Edition 3.0 published June 2017

Edition 2.0 published October 2010

Edition 1.0 published August 2009

Publisher

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Austroads

About Austroads

Austroads is the peak organisation of Australasian road transport and traffic agencies.

Austroads' purpose is to support our member organisations to deliver an improved Australasian road transport network. To succeed in this task, we undertake leading-edge road and transport research which underpins our input to policy development and published guidance on the design, construction and management of the road network and its associated infrastructure.

Austroads provides a collective approach that delivers value for money, encourages shared knowledge and drives consistency for road users.

Austroads is governed by a Board consisting of senior executive representatives from each of its eleven member organisations:

- Transport for NSW
- Department of Transport and Planning Victoria
- Queensland Department of Transport and Main Roads
- Main Roads Western Australia
- Department for Infrastructure and Transport South Australia
- Department of State Growth Tasmania
- Department of Infrastructure, Planning and Logistics Northern Territory
- Transport Canberra and City Services Directorate, Australian Capital Territory
- Department of Infrastructure, Transport, Regional Development, Communications and the Arts
- Australian Local Government Association
- Waka Kotahi NZ Transport Agency.

Acknowledgements

The authors acknowledge the role and contribution of the Austroads Road Design Task Force Panel in providing guidance and information during the preparation of this Part, in particular, the project working group consisting of Albert Wong (Main Roads Western Australia), Bernard Worthington (Queensland Department of Transport and Main Roads) and Richard Fanning (Department of Transport and Planning Victoria).

Summary

The Austroads *Guide to Road Design Part 4A: Unsignalised and Signalised* Intersections (AGRD Part 4A) provides road designers and other practitioners with guidance on the detailed geometric design of all at-grade intersections (excluding roundabouts). However, some of the guidance in AGRD Part 4A may be appropriate for the design of approaches to roundabouts and is relevant to the design of ramp terminals where freeway ramps intersect with the minor road at an interchange.

AGRD Part 4A does not provide all the information that is necessary to design a satisfactory intersection and therefore, depending on the situation, should be used in conjunction with all other parts of the *Guide to Road Design*, in particular:

- *Part 4: Intersections and Crossings: General* (Austroads 2023b)
- *Part 4B: Roundabouts* (Austroads 2023c)
- *Part 4C: Interchanges* (Austroads 2023d).

Edition 3.2 of AGRD Part 4A contains editorial and technical changes to sections 2.2, 3.2.2, 3.2.3, 3.3, 3.4, 4.1, 5.2.2, 5.5, 6.1.4, 9, 9.2. It also contains new guidance in Section 8.2.5 for offset rural CHL treatments, Appendix A.5.7 for S-lane treatments and Appendix A.10 for EDD for truck acceleration lanes.

In addition, road designers should also refer to the Austroads *Guide to Traffic Management Part 6: Intersections, Interchanges and Crossings* (Austroads 2020a) which provides guidance on the traffic management aspects of intersection design and road users' requirements.

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1. Introduction

1.1 Purpose

The Austroads *Guide to Road Design* (AGRD) seeks to capture the contemporary road design practice of member organisations. In doing so, it provides valuable guidance to designers in the production of safe, economical and efficient road designs.

The Austroads *Guide to Road Design Part 4: Intersections and Crossings: General* (AGRD Part 4) (Austroads 2023b) provides guidance to road designers on the geometric design of road intersections and crossings and together with three other parts:

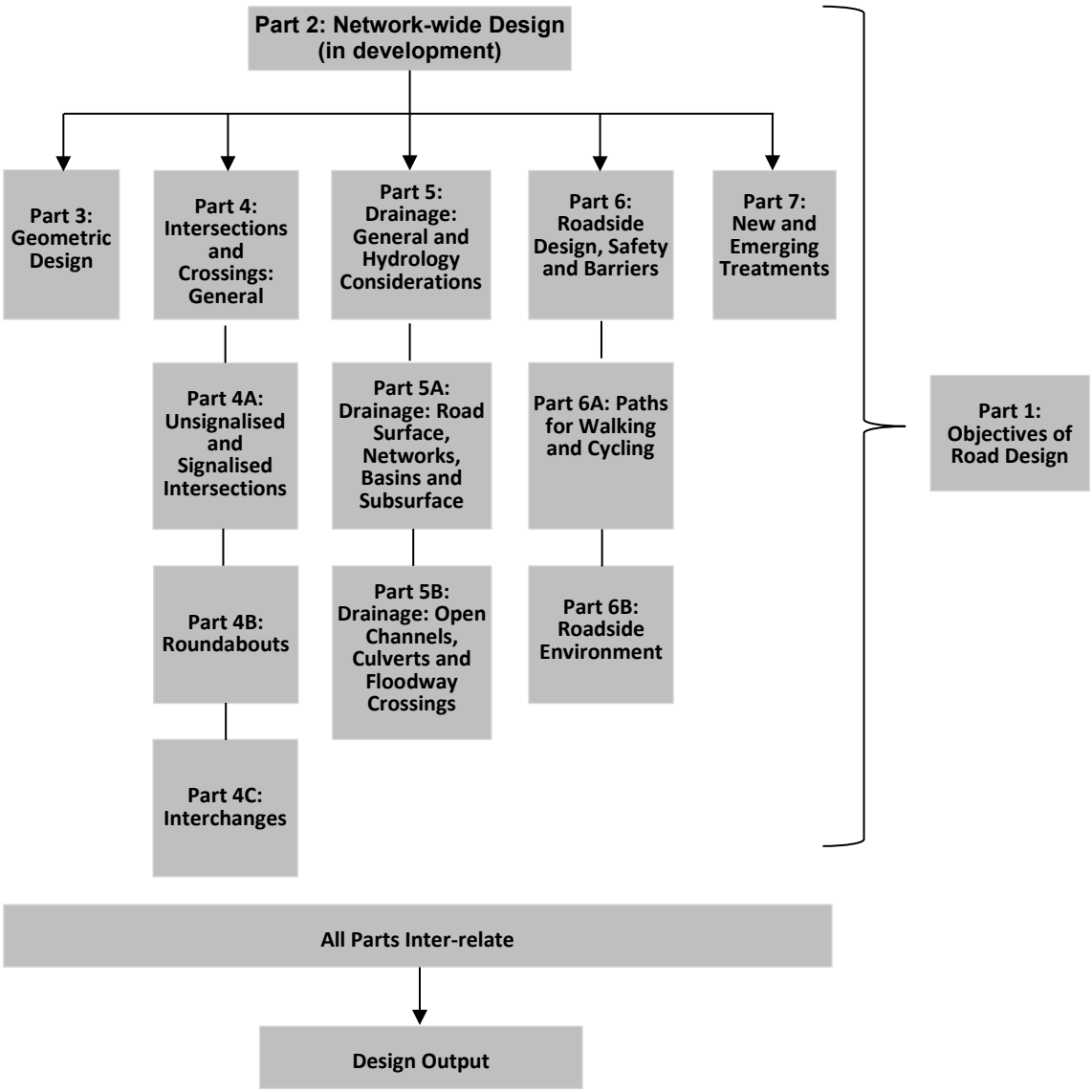
- *Part 4A: Unsignalised and Signalised Intersections* (AGRD Part 4A)
- *Part 4B: Roundabouts* (AGRD Part 4B) (Austroads 2023c)
- *Part 4C: Interchanges* (AGRD Part 4C) (Austroads 2023d).

AGRD Part 4A covers:

- information on the types of unsignalised and signalised intersections and their use
- an intersection layout design process and factors to be considered
- detailed geometric design requirements for various types of intersection.

Figure 1.1 shows Part 4A is one of seven parts that comprise the AGRD. Collectively these parts provide information on a range of disciplines including geometric design, drainage, roadside design, and geotechnical design, all of which may influence the location and design of intersections.

Figure 1.1: Flow chart of the Guide to Road Design



1.2 Scope of this Part

This Part is limited to the design of unsignalised and signalised intersections. Designers should be aware that there are other subject areas spanning the range of Austroads publications that may also be relevant to the design of intersections.

The Austroads *Guide to Traffic Management Part 6: Intersections, Interchanges and Crossings* (AGTM Part 6) (Austroads 2020a) should be regarded as a related document to this part of the AGRD, as it provides information on the types of intersections, the selection of an intersection and the location of intersections. AGTM Part 6 should be consulted when determining the appropriate type of intersection to be provided, and when considering the design of particular features from a traffic management and road user perspective.

Designers should refer to the Austroads *Guide to Road Design Part 1: Objectives of Road Design* (AGRD Part 1) (Austroads 2021a) for objectives in the design of intersections, interchanges and crossings.

The Austroads *Guide to Road Design Part 3: Geometric Design* (AGRD Part 3) (Austroads 2016a) provides specific details relating to geometric design principles and their application.

1.3 Design Criteria in Part 4A

AGRD Part 4 defines greenfield and brownfield sites within the context of road design, and suggests that in most cases the application of normal design domain (NDD) values will be suitable for both. However, it is also acknowledged that situations will arise where it may not always be practical or possible to achieve all the relevant NDD values (e.g. in constrained locations), in which case road agencies may consider the use of values outside of the NDD values.

The body of this Part contains NDD values that should be used for the design of all unsignalised and signalised intersections, including new intersections on new or existing roads, and modifications to existing intersections. Appendix A contains extended design domain (EDD) values that relate to intersection design, and through research and/or operating experience, have been found to provide a suitable solution in constrained situations. EDD values have only been developed for particular parameters, where considerable latitude exists within the NDD values. Using values outside of the normal design domain (NDD) requires evidence that adopting the design value will not compromise the safety performance.

Guidance on use of values outside of the design domain (i.e. outside of the NDD and EDD), is not provided in this Part. Designers should consult the delegated representative from the relevant road agency for advice and direction with respect to an appropriate standard when values within the design domain are not achievable.

In using this Part:

1. NDD values given in the body of this Part should be used wherever practical.
2. Design values outside of the NDD are only to be used if approved in writing by the delegated representative from the relevant road agency. The relevant road agency may be a state or territory road agency, municipal council or private road owner.
3. If using EDD values, the reduction in standard associated with their use should be appropriate for the prevailing local conditions. Generally, EDD should be used for only one parameter in any application and not be used in combination with any other minimum or EDD value for any related or associated parameters.
4. Designers should refer to the relevant jurisdiction for minimum treatments to be used for the particular class of road.

1.4 Intersection Safety and the Safe System Approach

Road designers should be aware that intersection design takes place in a broader context where designs are influenced by many factors, including cost and economic considerations. However, it is important that intersections should perform the intended function and operate as efficiently as possible, but it is paramount that intersections are designed to be as safe as possible for all road users.

For intersections the Safe System approach seeks to prevent fatal and serious injury crashes by ensuring that when vehicles collide at intersections, or errant vehicles run-off the road, they are afforded a forgiving roadside environment, while roadsides nearby are shielded with safety barriers. Using the Safe System approach inherently includes safer design for pedestrians.

An understanding of the types of crashes that occur at different types of intersections, and of the factors that contribute to crashes is essential in the development of effective designs and related road safety countermeasures. Intersection design must focus on safety for all road users with a focus on pedestrians and cyclists, as the most vulnerable, and creating safe and accessible environments for disabled transport users. Turner et al. (2009) also note that the primary treatments for pedestrian safety are physical separation and low speed environments.

Intersections also represent a particular road safety hazard for motorcyclists and so intersection layouts should be kept obvious, as simple as possible, and particular attention given to the following:

- Visibility between vehicles – motorcycles may be difficult to detect, and motorcycles may take longer to stop in some situations.
- Recognition of the layout – night-time visibility is critical for motorcyclists, and it is best to keep it simple.
- Parked vehicles may obscure a motorcycle.
- Identifying locations where gravel or debris may build up and provide travel paths that are clear of these areas.
- Surface treatments must have adequate skid resistance.
- Islands need to be lit because of the limited effectiveness of motorcycle headlights.
- Signal detector loops should be set to detect motorcycles.
- Zebra crossing and other markings at left-turn slip lanes must have adequate skid resistance as motorcyclists may lean travelling around the corner.

For more information on road design philosophy and principles refer to AGRD Part 1.

1.5 Grade Separation of Traffic Movements

The design of at-grade intersections, particularly those in urban situations, often requires traffic analysis to establish the number of traffic lanes and length of traffic queues that should be accommodated to achieve a satisfactory capacity and level of service in the design year. Guidance on the required analysis is available in the Austroads *Guide to Traffic Management Part 3: Transport Studies and Analysis Methods* (AGTM Part 3) (Austroads 2020d).

Generally, at-grade intersections can be designed to provide adequate capacity and safety. However, situations may arise when a particular traffic movement results in major traffic congestion or a road safety problem cannot be resolved through traffic management or at-grade treatment. In these cases, the road agency may choose to grade separate one or more movements. These treatments can involve layouts to suit local situations and traffic movements and typically have the normal features of an at-grade design (e.g. auxiliary lanes). To improve safety the treatment may result in the major road intersections comprising only left turns and auxiliary lanes.

2. Layout Design Process

2.1 Design Process

The design of an intersection involves operational and geometric requirements that are inter-related and determine the information that is presented on conceptual and functional design plans.

Table 2.1 provides a summary of some considerations in relation to the intersection layout design process.

Table 2.1: Considerations in the intersection layout design process

Design element	Key considerations	Reference
Provision for pedestrians	Consider pedestrian desire lines (e.g. the preferred route which is often the quickest or straightest) and the crossing width including any medians or kerb build outs to reduce crossing distances, to maximise the level of service (LOS) for pedestrians. Ensure that pedestrian sight distance requirements are met. Provide adequate storage and crossing points to meet current and future pedestrian demand at the crossing location.	*Section 3.3* *Pedestrian Planning and Design Guide* (NZTA 2009) AGTM Part 4 AGTM Part 6
Alignment of the approaches	Straight and flat alignments are ideal Where it is necessary for a road to change direction as it passes through an intersection the tangent points for the horizontal curve should be located on approaches some distance from the intersection (rather than have a sharp change in direction at the intersection) Align the side road to intersect the major road at 90° and a straight on the immediate approach Approaches with flatter angles may reduce the impact of a collision, however sight lines and the driver observation angle also need to be considered in adopting the approach angle	AGRD Part 3
Cross-sections	Provide the details with which the intersection layout must match on each leg (i.e. mid-block road space allocation) May include verges, paths, parking lanes, special use lanes, medians, public transport reservations (in the median or roadside)	AGTM Part 5 AGRD Part 3
Traffic lanes (approach and departure) required for satisfactory operation and safety	Number of through lanes and turn lanes are determined by traffic analysis Road function may require bus, transit or bicycle lanes, or tram lines Specify median width, where one is to be provided Provide adequate footpaths	AGTM Part 3 AGTM Part 6
Roadside areas	Consider drainage, roadside safety including the location and placement of safety barriers, road furniture and signage, utilities and lighting and paths Use of frangible fixtures, such as poles Environmental requirements Locate pits outside of the travel paths	AGRD Part 5 AGRD Part 5A AGRD Part 5B AGRD Part 6
Pavement area and the location and shape of median noses and kerb returns	Use an appropriate turning template or computer software for the design vehicle Use appropriate turning radii for templates and clearances to kerbs and other vehicles Plot the required pavement area and the location and shape of the median noses, turning lines and edge lines, other kerbs etc.; locate stop and give way lines	AGRD Part 4

Design element	Key considerations	Reference
Left-turn and right-turn treatments	Appropriate treatment determined by traffic analysis, consideration of road user requirements and safety Give way situations – high entry angle treatment Free-flow and signalised left-turns – design for an appropriate turning speed Check observation angles Provide a right-turn treatment based on traffic analysis – may require relatively wide median on the intersection approach Provide uniform pavement surface along the turning path	Section 8 Section 7
Turning lane lengths including tapers treatment to mid-block cross-sections	Determine the lengths of auxiliary lanes and draw the lanes with appropriate physical tapers for both left-turn and right-turn lanes, based on deceleration length, storage length (from traffic analysis), and queue lengths in adjacent through lanes	Section 5
Sight distance requirements	Approach sight distance, safe intersection sight distance, and minimum gap sight distance	Section 3
Parking limits	Define required parking limits in relation to statutory rules and traffic operation	AGRD Part 3 Australian Road Rules
Road markings	Location of linemarking, including stop lines, give way lines, marked pedestrian crossings, lane lines, turning lines, pavement arrows and symbols	AS 1742.2-2009 AGTM Part 10 *Manual of Traffic Signs and Markings (MOTSAM): Part 2: Markings* (NZ Transport Agency 2010b) in NZ
Other requirements	Design should also meet other requirements including public transport facilities, bus routes, trams, bicycle lanes etc. Locate stops (incorporate stops into traffic islands?)	AGRD Part 4

The design of a signalised intersection must also result in the production of a signal layout plan that shows the location and types of all the signals and the associated hardware and infrastructure. Table 2.2 summarises key aspects of a signal layout plan, some of which need to be considered in the geometric design of the intersection.

Table 2.2: Considerations in developing a signal layout plan that may influence geometric design

Design element	Key considerations	Reference
Location of signal poles	Mast arms and joint use poles have large foundations. Consider: • overhead and underground services • coordination of foundation locations with drainage pits and pipes • accommodation of signage on traffic signal poles • design vehicle and checking vehicle • size of traffic signal lanterns • location of road furniture • pedestrian and cyclist access • people with a visual and mobility impairment traffic signal phasing • clearance between opposing turns • traffic signal post location to minimise the likelihood of damage • frangibility of traffic signal posts • clearances to traffic signal posts • island size and median width • location of kerb ramps	AGTM Part 9
Signal aspects		AGTM Part 9
Location of crossing point	Provide good visibility of pedestrians for all vehicle movements Provide good design for tactile paving requirements Provide direct crossing point on the desire line and where possible reducing the crossing width	Section 3.3 AS/NZS 1428.4.1 NZTA (2015) NZTA (2009)
Locating the traffic signal controller	Controllers are expensive; choose locations where: • it is not vulnerable to run-off-road crashes • the maintenance technician has a clear view of traffic movements • the maintenance vehicle can be parked adjacent to the controller (desirable) • a power source is readily available	
Location of power conduits, power cable pits, detectors and detector pits	Where possible, locate power cable pits outside of pedestrian paths and storage areas Coordinate power conduit location with drainage and underground services	
Determining parking limits in relation to statutory rules and traffic operation	Queue lengths of left-turn movements	AGTM Part 3 *Australian Road Rules*
Add a schedule of signs relating to the signalisation of the intersection	Regulatory and warning signs are most important to operation Coordinate with proposed or existing signage on approaches	AS 1742.2-2009

2.2 Alignment of Intersection Approaches

Once the preferred intersection location, the general alignment of the intersecting roads, type of layout and form of control have been determined, the first step in the layout design process is to determine the detailed alignment of each leg of the intersection. In establishing the alignment of the approaches it is important to consider and minimise possible crash impact forces, should a crash occur.

Issues may arise and adjustments may have to be made to the layout during this part of the process, perhaps because original site assumptions or information were not entirely correct.

2.2.1 Horizontal Alignment

Road centrelines should be designed to intersect as close to 90° as possible so that driver observation angles to potentially conflicting vehicles are satisfactory. This is particularly important for older drivers who may have limited ability to turn their head and neck to observe potentially conflicting traffic (Austroads 2000).

The most appropriate horizontal geometric requirements for an intersection provide adequate sight distance for motorists on approach to the intersection as well motorists negotiating the intersection. Preferably, an intersection should consist of a minimum straight section of road, equivalent to two seconds of travel time at the design speed on both upstream and downstream sides. This provides approaching drivers with the best view of the intersection layout and other vehicles. Where this is not possible, it is desirable that any curved horizontal alignment for a through movement at the intersection is of a constant radius and that tangent points are located a substantial distance from the intersection. This practice is desirable at rural and urban sites.

The following considerations should be applied to the horizontal alignment of intersection approaches:

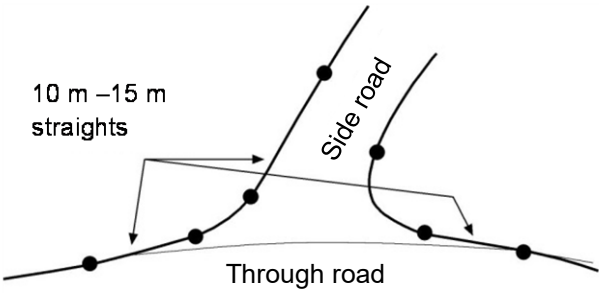
- A straight alignment on one approach should not be followed by a tight curve on the downstream exit from an intersection.
- Intersection on the outside of a small radius horizontal curve should be avoided, especially on curves with large deflection angle.
- Careful attention is required for the design of minor road legs of unsignalised intersections with high approach speeds as these legs record high crash rates.
- The use of over-run kerb profiles may be required to accommodate design or check vehicle paths,; however, careful attention must be paid to the location and height of kerb profiles on curved horizontal alignments at intersections

The available road reservation or other constraints at some urban sites may result in tangent points being located close to the intersection and a misalignment of lanes through an intersection. This in turn can adversely affect lane discipline through the intersection and therefore lanes may have to be delineated within the intersection (e.g. raised pavement markers). Where misalignments are unavoidable, the maximum amount of misalignment should be the lesser of:

- the lateral shift using a rate of 1.0 m/s, or
- half a lane width.

On curved horizontal alignments it is inevitable that reverse curvature will be involved in some turning movements and this can create difficulties, particularly with crossfall. In urban environments, a minimum 10 m to 15 m length of straight should be provided between reverse curves, as shown in Figure 2.1. This distance approximates one to two sec travel time and allows drivers to make any reverse steering manoeuvres necessary. Each movement should be checked with a turning path template and the length of straight increased as necessary.

Figure 2.1: Typical arrangement of reverse curves with a short straight used at urban sites



Source: Department of Main Roads (2006).

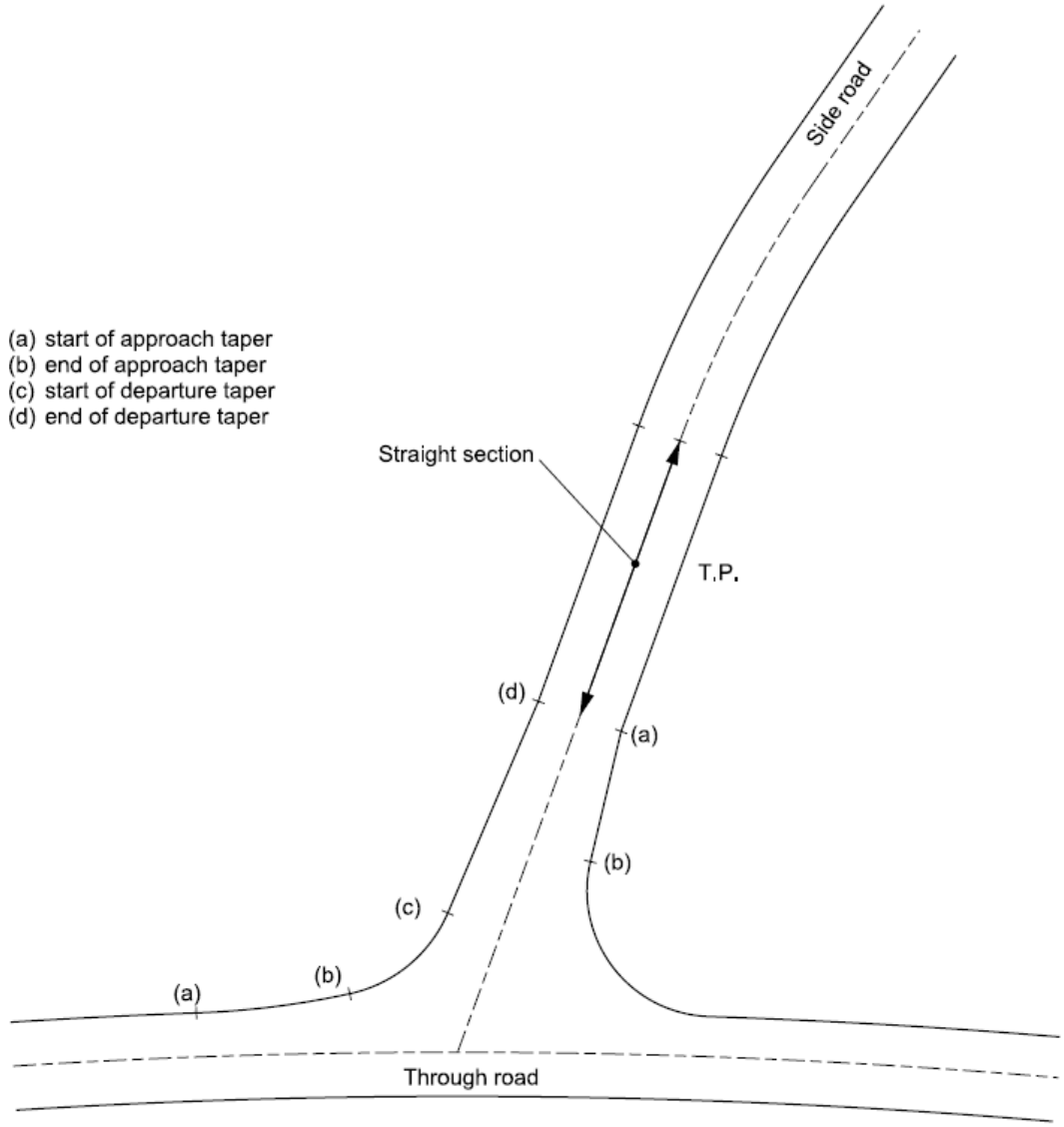
In rural situations a curved approach on a terminating leg requires a length of straight to allow for plan and/or crossfall transition. From a driver's perspective it is also most desirable to have a straight rather than curved alignment on the immediate approaches to intersections. A suitable length of straight ensures that sight distance lines occur above the road formation giving drivers the best opportunity to see the intersection and brake on a straight rather than on a curved alignment. In addition, it is often difficult for drivers to judge the location of intersections that are located at the end of horizontal curves.

In rural situations the road geometry on a side road can be used to progressively slow drivers so that they can safely give way or stop at the intersection. Reference should be made to the Austroads *Guide to Road Design Part 3: Geometric Design* (AGRD Part 3) (Austroads 2016a) for design parameters that are appropriate to a particular intersection approach.

The desirable requirements for rural sites are illustrated in Figure 2.2. If these requirements cannot be met the designer should consider a minimum treatment or realignment in order to increase the length of straight on the side 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 10 m.

Figure 2.2: Desirable minor road approach alignment at rural sites



Note: Straight section required to provide a transition between an approach curve and the intersection.

Source: Department of Main Roads (2006).

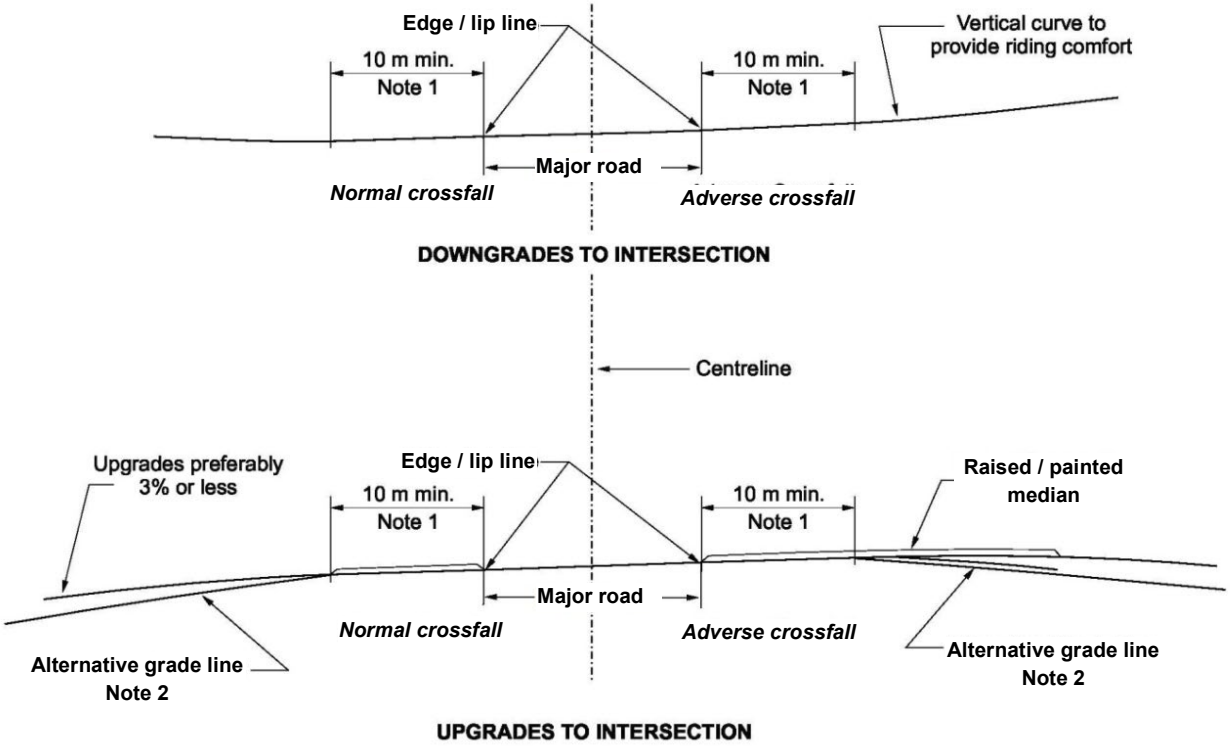
2.2.2 Vertical Alignment

A steep upgrade on the approach to an intersection generally results in both sight distance and operational problems. 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 at 3% or less for a minimum distance of 10 m from the lip of the channel or edge line (maximum of 4%). This limitation is required in order to facilitate acceleration and to improve sight distance. This situation is illustrated in Figure 2.3 and is more critical for upgrades than downgrades. If a significant number of heavy vehicles use the side road the distance provided should accommodate the design vehicle. A similar approach is recommended for downgrades on the minor road.

On downhill approaches of side roads to intersections, grades should desirably not exceed 3% with a maximum of 5% in order to limit the effect of steep grades on stopping distances. Alternatively, truck stopping sight distance and high friction surfaces or transverse grooving on the downhill approaches to such intersections should be provided. High friction surfacing typically has a life of 8–12 years (ARRB 2018). Maintenance of the selected treatment is critical to ensure effectiveness in assisting the stopping ability of a vehicle. Refer to the Austroads *Guide to Pavement Technology Part 3: Pavement Surfacings* (AGPT Part 3) (Austroads 2021c) for further information.

At T-intersections where it is not possible to maintain sight distance to the pavement on the minor road, short vertical curves (e.g. 5 m to 10 m long) may be used. These vertical curves should not encroach into the traffic lanes on the main road. In this situation it is desirable to provide a median in the minor road that extends over the crest to provide a cue to drivers that they are approaching an intersection. All minor roads that have priority through an intersection (i.e. at intersections between two minor roads) should be designed with vertical alignment standards which are consistent with the operating speed on the approaches.

Figure 2.3: Cross-section of major road showing grading options for minor road intersection approaches



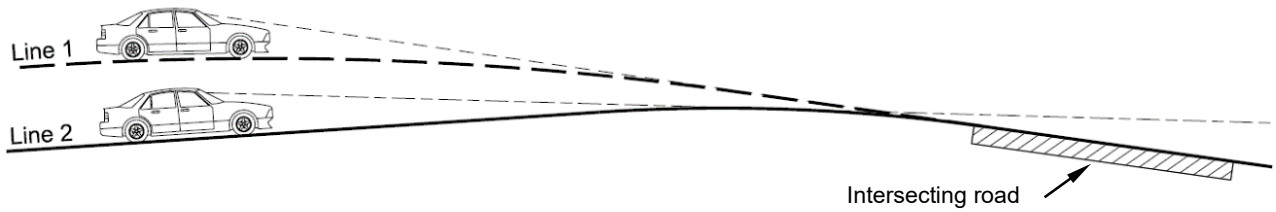
1 *Crossfall grade not to be exceeded for 10 m in approach to edge/lip line.*
2 *Maximum algebraic change of grade for alternative grade line 12%.*

Notes: ASD 1.1 m to 0.0 m to be provided to stop/give way line and median nose.
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.
Source: Department of Main Roads (2006).

Where there is a choice it is preferred that intersections are located away from horizontal curves. It is desirable that approach sight distance (ASD) is available (Section 3.2.1) to the road surface at all intersections. Unfortunately, locating a leg of an intersection on the back of a curve invariably results in drivers approaching from that direction not being able to see the intersection and its layout.

Where it is necessary to place a leg of an intersection on the outside of a superelevated curve it is therefore important to achieve the best possible sight distance outcome. The options are illustrated in Figure 2.4. The grading shown as line 1 is not generally practicable or favoured because of the extent and cost of the earthworks required to provide acceptable minimum sight distances. Where line 1 is not practicable, the preferred grading is line 2 comprising a uniform approach grade with a short vertical curve to join it to the crossfall of the major road. This also results in a relatively flat 'standing' area similar to the 10 m shown in Figure 2.3.

Figure 2.4: Sight distance to T-intersection



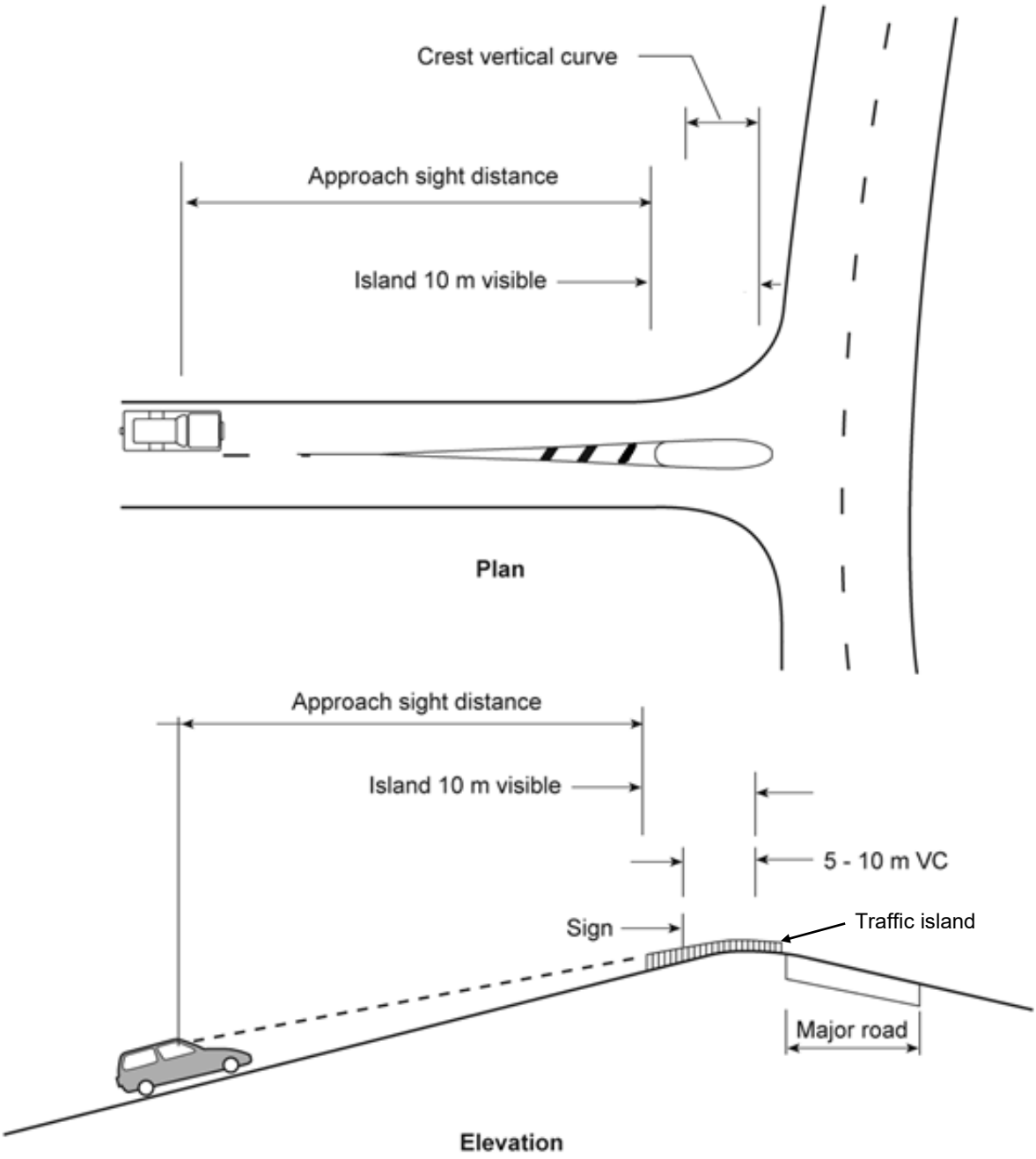
Source: Based on Department of Main Roads (2006).

In the case of line 2, an island is provided on the minor road approach with appropriate signing to warn approaching drivers of the intersection ahead.

The requirements illustrated in Figure 2.5 apply to designs based on line 2. It should be noted that:

- At least 10 m of the island should be visible to approaching drivers for a distance equal to the ASD for the 85th percentile operating speed on the approach.
- The island should, as far as possible, be directly in the line of sight of drivers for a distance equal to the ASD for cars.
- The island 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.
- The short vertical curve should not encroach onto the shoulder of the major road.
- The island should be visible for all approaching truck drivers from the appropriate truck stopping distance.

Figure 2.5: Sight distance to T-intersections on a major road curve



Source: Based on Department of Main Roads (2006).

2.2.3 Combined Horizontal and Vertical Curves

The coordination of horizontal and vertical curves is discussed in AGRD Part 3. Situations may occur where vertical and horizontal geometry must be coordinated at intersections, but it is preferable that intersections are located on straight and relatively flat sections of road.

2.2.4 Superelevation at or near Intersections

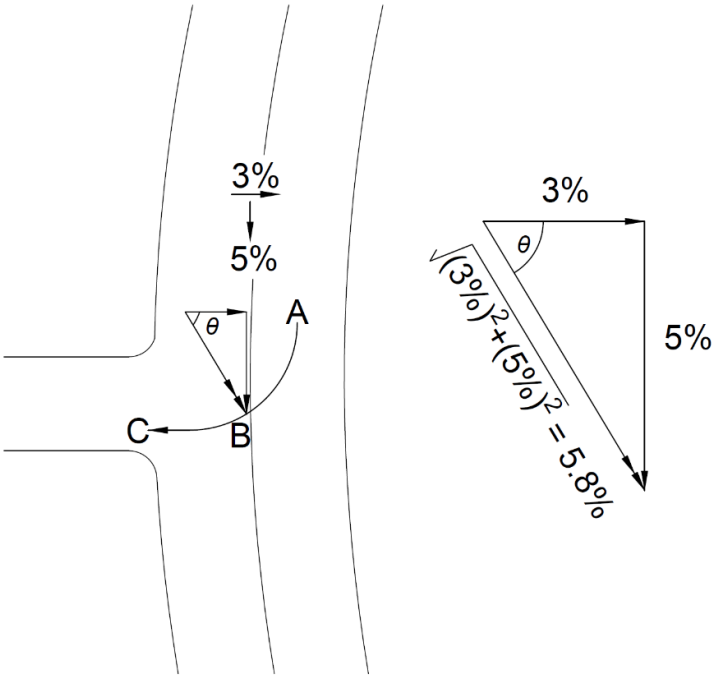
Superelevation at intersections is associated with horizontal curves that pass through the intersection. While intersections on the inside of small radius horizontal curves produce difficult observation angles for drivers, those on the outside of curves result in:

- greater difficulty for a driver in the side road to perceive the presence of the 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
- greater difficulty for a driver on the major road to perceive the location of the intersection due to the superelevation that is normally required on the major road horizontal curve.

Superelevation and changes in superelevation (i.e. crossfall) within an intersection can have a detrimental effect on driver and passenger comfort and vehicle stability, particularly for heavy vehicles. In general, the crossfall adopted for turning roadways where vehicles can turn at moderate speed should desirably not exceed +7% or –3%. For a turn executed at very slow speed (say < 10 km/h), the desirable maximum adverse crossfall (i.e. the vector sum) is –5%.

The maximum effective adverse crossfall for turning movements at intersections can be determined using vector diagrams. Figure 2.6 shows how a 3% crossfall and a 5% longitudinal fall can combine to result in a 5.8% adverse crossfall throughout a right turn. As effective adverse crossfall is defined as being perpendicular to the vehicle path, the location where the maximum resultant vector is at right angle to the vehicle path (point A to Point C) identifies the point where the maximum crossfall occurs, in this case at point B. Where a site is constrained and approach speeds are low a larger adverse crossfall may be considered under EDD (Appendix A).

Figure 2.6: Illustration of adverse crossfall for a right-turn movement



With respect to adverse crossfall within intersections it is desirable that there are no surprises for drivers and in particular the magnitude of the adverse crossfall should not increase markedly throughout the turning movement. Where longitudinal grades are significant (e.g. ≥ 5% in hilly areas) and trucks with high loads turn at the intersection, it may be necessary to construct a flatter area in the longitudinal grade in order to achieve satisfactory crossfalls for turning traffic. This requirement can also apply to left-turn movements. Designers may use the road surface contours to check/assess the crossfalls. Appendix B provides additional guidance on truck stability at intersections.

3. Sight Distance

3.1 General

It is fundamental to the safety of intersections that drivers approaching in all traffic streams are able to:

- recognise the presence of an intersection in time to slow down or stop in a controlled and comfortable manner
- see vehicles approaching in conflicting traffic streams and give way where required by law or avoid a crash in the event of a potential conflict.

Intersection safety performance is therefore largely dependent upon adequate sight distance in relation to both horizontal and vertical geometry for all drivers approaching and entering the intersection. Consequently, sight distance is a key consideration in the location and design of intersections.

A feature of intersections is that sight lines are often required at large angles to the user's normal viewpoint, and the driver of a vehicle may have to look through the side windows. In addition, the paths travelled are often curved, which means that drivers may find it more difficult to view other vehicles and estimate distances.

Large angles can be a significant issue for older drivers, particularly those who may have difficulty in turning their head and neck to detect the presence of conflicting vehicles (Austroads 2000). For new at-grade intersections where right of way is not restricted, the roadway should meet at a 90° angle to provide the best sight lines. For re-design of existing at-grade intersections where right of way is restricted, the roadway should meet at an angle of not less than 70°.

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 checked to ensure that they are not disrupted by natural objects such as trees, and structures such as fences, buildings and safety barriers.

Adequate sight distance at proposed intersections and remodelled intersections must be achieved when developing the horizontal and vertical alignments of new and upgraded roads and should be checked as the design proceeds through various iterations.

It is equally important that sight distance requirements are achieved at all pedestrian, cyclist and rail crossings.

3.2 Sight Distance Requirements for Vehicles at Intersections

The types of sight distance that must be provided in the design of all intersections include:

- approach sight distance (ASD)
- safe intersection sight distance (SISD)
- minimum gap sight distance (MGSD).

In addition to the above specific intersection sight distance requirements, stopping sight distance (SSD) in accordance with the Austroads *Guide to Road Design Part 3: Geometric Design* (AGRD Part 3) (Austroads 2016a) must be available at all locations through the intersection. This Part provides reaction times, longitudinal deceleration rates, vertical height parameters (e.g. driver eye height) for sight distance requirements for road design in general. Specific sight distance values for intersections are provided in the following sections.

3.2.1 Approach Sight Distance (ASD)

Provision of ASD for cars

ASD is:

- the minimum level of sight distance which must be available on the minor road approaches to all intersections to ensure that drivers are aware of the presence of an intersection
- also desirable on the major road approaches so that drivers can see the pavement and markings within the intersection and should be achieved where practicable.
  However, the provision of ASD on the major road may have implications (e.g. cost; impact on adjacent land and features) in which case SSD is the minimum sight distance that should be achieved on the major road approaches to the intersection and within the intersection.
- measured from a driver's eye height (1.1 m) to 0.0 m, which ensures that a driver is able to see any line marking and kerbing at the intersection. In some situations this may not be possible due to the vertical alignment. Section 2.2.2 provides some guidance on additional visual cues that can be incorporated in these situations.

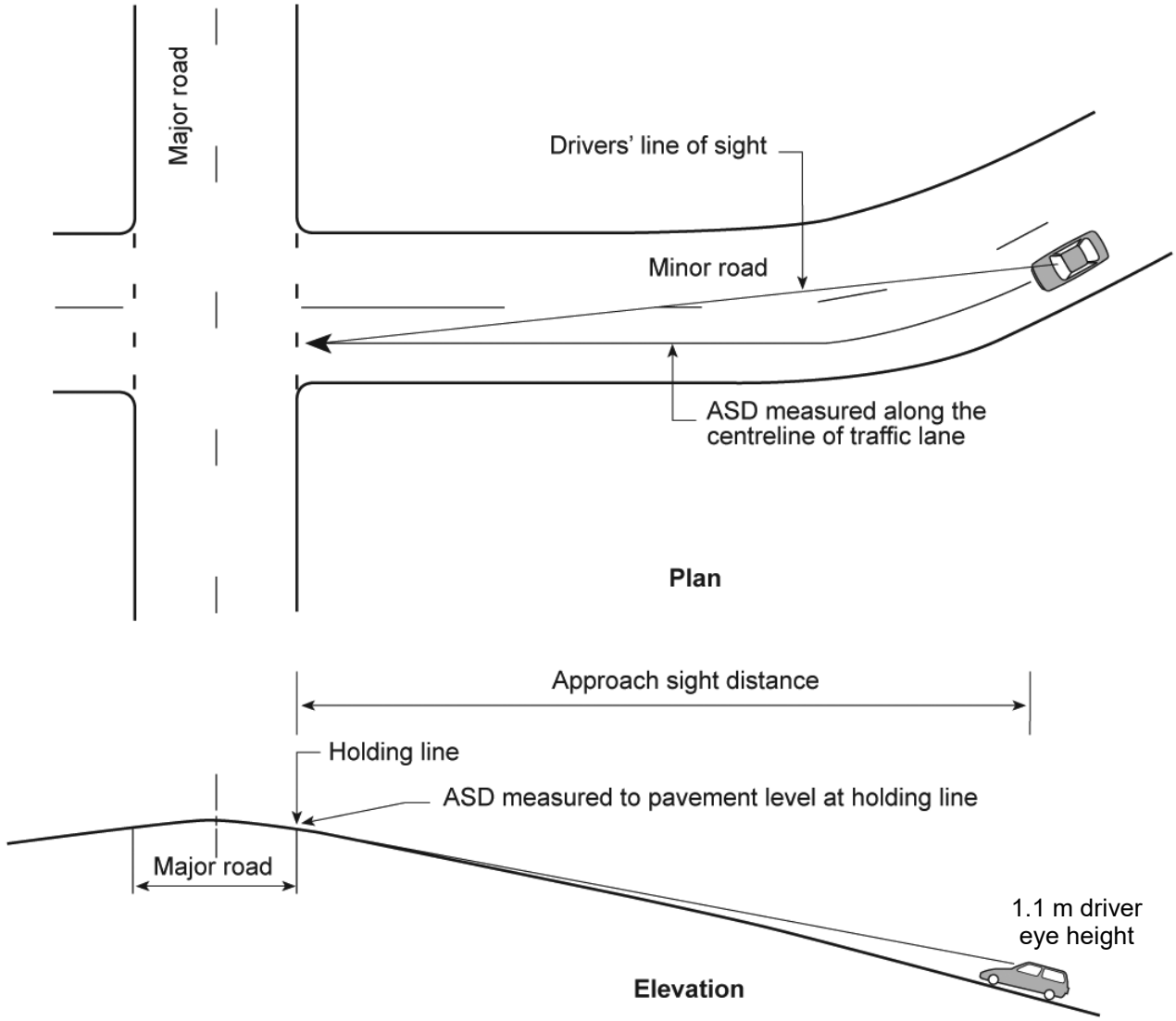
Equation 1 provides the formula for ASD and Figure 3.1 illustrates the application of ASD:

$$ASD = \frac{R_T \times V}{3.6} + \frac{V^2}{254 \times (d + 0.01 \times a)} \qquad 1$$

where

ASD	=	approach sight distance (m)
R_T	=	reaction time (sec), refer to *AGRD Part 3* (Austroads 2016a) for guidance on values
V	=	operating (85th percentile) speed (km/h)
d	=	coefficient of deceleration, refer to Table 3.1. Refer also to *AGRD Part 3,* Table 5.3 for further information on coefficient selection
a	=	a longitudinal grade in % (in direction of travel: positive for uphill grade, negative for downhill grade)

Figure 3.1: Application of approach sight distance (ASD)



Values for ASD are provided in Table 3.1 and correction factors for gradient are provided in Table 3.4.

Provision of ASD for trucks

The various sight distance requirements discussed above apply to cars. ASD for trucks should be provided at intersections to ensure that trucks approaching the intersection, at the 85th percentile operating speed of trucks, are able to stop safely. ASD for trucks on intersection approaches should be measured from truck driver eye height (2.4 m) to pavement level at the stop or holding line (0.0 m). Approach sight distances for trucks are numerically the same as the SSD values for trucks provided in AGRD Part 3.

Table 3.1: Approach sight distance (ASD) and corresponding minimum crest vertical curve size for sealed roads (S < L)

Design speed (km/h)	Based on approach sight distance for a car[1] h_1 = 1.1, h_2 = 0, d = 0.36[2]					
	R_T = 1.5 sec[3]		R_T = 2.0 sec		R_T = 2.5 sec	
	ASD (m)	*K*	ASD (m)	*K*	ASD (m)	*K*
40	34	5.3	40	7.2	–	–
50	48	10.5	55	13.8	–	–
60	64	18.8	73	24.0	–	–
70	83	31.1	92	38.9	–	–
80	103	48.5	114	59.5	–	–
90	126	72.3	139	87.3	151	104
100	151	104	165	124	179	146
110	–	–	193	171	209	198
120	–	–	224	229	241	264
130	–	–	257	301	275	344
Truck stopping capability provided by the minimum crest curve size[4]	h_1 = 2.4 m, h_2 = 0 m, d = 0.22					

1 *If the average grade over the braking length is not zero, calculate the approach sight distance (ASD) values using the correction factors in Table 3.4 (or use Equation 1) by applying the average grade over the braking length.*

2 *In constrained locations (typically lower volume roads, less important roads, mountainous roads, lower speed urban roads and tunnels), a coefficient of deceleration of 0.46 may be used. For any horizontal curve with a side friction factor greater than the desirable maximum value for cars (in constrained locations), use a coefficient of deceleration of 0.41. The resultant crest curve size can then be calculated using the relevant equations in AGRD Part 3 (Austroads 2016a).*

3 *A 1.5 sec reaction time is only to be used in constrained situations where drivers will be alert. Typical situations are given in Table 5.2 of AGRD Part 3. The general minimum reaction time is 2 sec.*

4 *This check case assumes the same combination of design speed and reaction time as those listed in the table, except that the 120 km/h and 130 km/h speeds are not used.*

Notes:

K is the length of vertical curve in metres for a 1% grade change.

Main Roads Western Australia has adopted a desirable minimum reaction time of 2.5 sec and an absolute minimum reaction time of 2.0 sec. A reaction time of 1.5 sec is not to be used in Western Australia.

Combinations of design speed and reaction times not shown in this table are generally not used.

Refer to AGRD Part 3 to determine the ASD for trucks around horizontal curves.

3.2.2 Safe Intersection Sight Distance (SISD)

SISD is the minimum sight distance which should be provided on the major road at any intersection. Designers should note that the object height for the application of SISD has been increased to 1.25 m (previously driver eye height was used i.e. 1.1 m) based on research by the Department of Main Roads (Lennie et al. 2008). The basis of the 1.25 m object height for cars is that this height is 0.2 m less than the 15th percentile height of passenger cars (1.45 m) as determined by the study.

Equation 2 provides the formula for SISD:

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

where

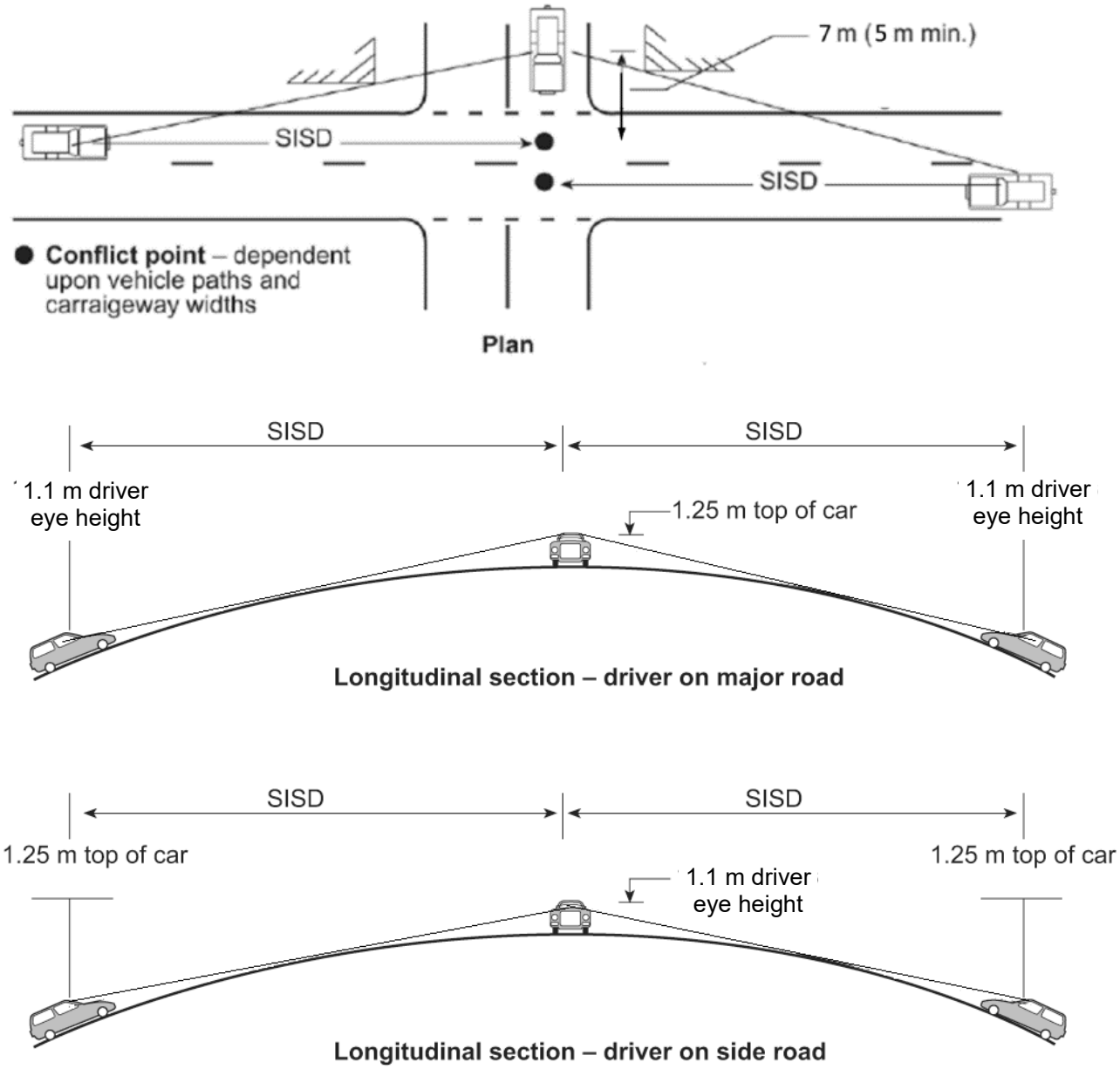
SISD	=	safe intersection sight distance (m)
D_T	=	decision time (sec) = observation time (3 sec) + reaction time (sec) – refer to *AGRD Part 3* (Austroads 2016a) for a guide to values
V	=	operating (85th percentile) speed (km/h)
d	=	coefficient of deceleration – refer to Table 3.3 and AGRD Part 3 for a guide to values
a	=	longitudinal grade in % (in direction of travel: positive for uphill grade, negative for downhill grade)

Designers should note that SISD:

- is measured along the carriageway from the approaching vehicle to the conflict point; the line of sight having to be clear to a point 7.0 m (5.0 m minimum) back along the side road from the conflict point
- provides sufficient distance for a driver of a vehicle on the major road to observe a vehicle on a minor road approach moving into a collision situation (e.g. in the worst case, stalling across the traffic lanes), and to decelerate to a stop before reaching the collision point. All possible conflict points arising from vehicles entering from the minor road should be assessed.
- is viewed between two points to provide inter-visibility between drivers and vehicles on the major road and minor road approaches
  It is measured from a driver eye height of 1.1 m above the road to points 1.25 m above the road, which represents drivers seeing the upper part of cars. Figure 3.2 illustrates the longitudinal section for the two cases representing inter-visibility; one for drivers on the major road and the second for a driver waiting in the minor road for an opportunity to enter the major road.
- assumes the driver on the minor road is situated at a distance of 7.0 m (minimum of 5.0 m) from the conflict point on the major road
  SISD allows for a 3 sec observation time for a driver on the priority legs of the intersection to detect a problem ahead (e.g. car from minor road stalling in through lane), plus the SSD.
- provides sufficient distance for a vehicle to cross the non-terminating movement on two-lane two-way roads, or undertake two-stage crossings of dual carriageways, including those with design speeds of 80 km/h or more
- should also be provided for drivers of vehicles stored in the centre of the road when undertaking a crossing or right-turning movement
- enables approaching drivers to see an articulated vehicle, which has properly commenced a manoeuvre from a leg without priority, but its length creates an obstruction.

Where practicable, designers should provide a larger sight distance than SISD. Values for SISD are given in Table 3.2 and corrections for grade are given in Table 3.4. Refer also to Table 3.3 for SISD check cases.

Figure 3.2: Safe intersection sight distance (SISD)



Source: Based on Department of Main Roads (2006).

Table 3.2: Safe intersection sight distance (SISD) and corresponding minimum crest vertical curve size for sealed roads (S < L)

Design speed (km/h)	Based on safe intersection sight distance for cars[1] h_1 = 1.1; h_2 = 1.25, d = 0.36[2]; Observation time = 3 sec					
	R_T = 1.5 sec[3]		R_T = 2.0 sec		R_T = 2.5 sec	
	SISD (m)	*K*	SISD (m)	*K*	SISD (m)	*K*
40	67	4.9	73	6	–	–
50	90	8.6	97	10	–	–
60	114	14	123	16	–	–
70	141	22	151	25	–	–
80	170	31	181	35	–	–
90	201	43	214	49	226	55
100	234	59	248	66	262	74
110	–	–	285	87	300	97
120	–	–	324	112	341	124
130	–	–	365	143	383	157

1 *If the average grade over the braking length is not zero, calculate the safe intersection sight distance (SISD) values using the correction factors in Table 3.4 (or use Equation 2) by applying the average grade over the braking length.*
2 *A coefficient of deceleration of greater than 0.36 is not provided in this table. The provision of SISD requires more conservative values than for other sight distance models (e.g. the stopping sight distance model allows values up to 0.46 in constrained situations). This is because there is a much higher likelihood of colliding with hazards at intersections (that is, other vehicles). Comparatively, there is a relatively low risk of hitting a small object on the road (the stopping sight distance model).*
3 *A 1.5 sec reaction time is only to be used in constrained situations where drivers will be alert. Typical situations are given in Table 4.2 of AGRD Part 3. The general minimum reaction time is 2 sec.*

Notes:
K is the length of vertical curve for a 1% change in grade.
To determine SISD for trucks around horizontal curves, use Equation 2 with an observation time of 2.5 sec.
Main Roads Western Australia have adopted a desirable minimum reaction time of 2.5 sec and an absolute minimum reaction time of 2.0 sec. A reaction time of 1.5 sec is not to be used in Western Australia.
Combinations of design speed and reaction times not shown in this table are generally not used.

Table 3.3: Safe intersection sight distances check cases

Minimum SISD capability provided by the crest vertical curve size[1]	Car at night[2]	d = 0.46, h_1 = 0.65 m, h_2 = 1.25 m, observation time = 2.6 sec (car headlight to top of car) d = 0.46, h_1 = 1.1 m, h_2 = 0.8 m, observation time = 2.5 sec (car driver eye height to car taillight)
	Truck	d = 0.24, h_1 = 2.4 m, h_2 = 1.25 m, observation time = 3.0 sec (truck driver height to top of car)
	Truck at night[2]	d = 0.29, h_1 = 1.05 m, h_2 = 1.25 m, observation time = 1.8 sec (commercial vehicle headlight to top of car) d = 0.29, h_1 = 2.4 m, h_2 = 0.8 m, observation time = 3.0 sec (truck driver eye height to car taillight)

1 *These check cases assume the same combination of design speed and reaction time as those listed in the table, except that the 120 km/h and 130 km/h speeds are not used for the truck cases.*
2 *Many of the sight distances corresponding to the minimum crest size are greater than the range of most headlights (that is, 120–150 m). In addition, tighter horizontal curvature will cause the light beam to shine off the pavement (assuming 3° lateral spread each way).*

Note: Designers should also refer to AGRD Part 3 for further information on the vertical height parameters.

Table 3.4: Grade corrections to ASD and SISD (cars)

Design speed (major road) (km/h)	Correction (m)							
	Upgrade				Downgrade			
	2%	4%	6%	8%	2%	4%	6%	8%
40	–1	–2	–2	–3	1	2	3	5
50	–1	–3	–4	–5	2	3	5	8
60	–2	–4	–6	–7	2	5	8	11
70	–3	–5	–8	–10	3	7	11	15
80	–4	–7	–10	–13	4	9	14	20
90	–5	–9	–13	–16	5	11	18	25
100	–6	–11	–16	–20	6	14	22	31
110	–7	–13	–19	–24	8	17	26	38
120	–8	–16	–22	–29	9	20	31	45
130	–10	–18	–26	–34	11	23	37	53

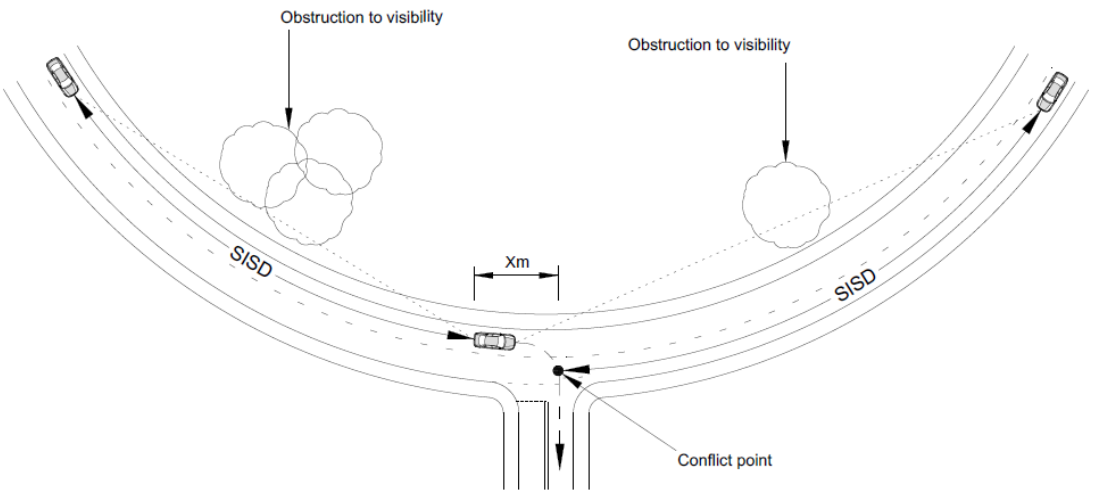
Note: This table to be used in conjunction with Table 3.2.

The SISD model should also be applied to the following cases to ensure that adequate visibility is provided between:

- vehicles approaching on the major road and vehicles turning right from the major road for basic right-turn (BAR) treatments (i.e. no right-turn lane provided)
  This is a similar requirement to the line of sight required between approaching major road vehicles and a stalled vehicle turning right from the minor road at all types of right-turn treatments.
- vehicles turning right from the major road and oncoming major road vehicles at all types of right-turn treatments, including those on divided roads.

The ability to achieve SISD in these cases could be influenced by the horizontal alignment, the vertical alignment, or a combined horizontal and vertical alignment. Figure 3.3 shows the application of the SISD model to an intersection on the outside of a horizontal curve.

Figure 3.3: Application of the SISD model for minor roads intersecting on the outside of horizontal curves



Note: X is the distance based on design vehicle storage length and turning path.

Source: Department of Main Roads (2006).

3.2.3 Minimum Gap Sight Distance

General

Minimum gap sight distance (MGSD)[1] is based on distances corresponding to the critical acceptance gap that drivers are prepared to accept when undertaking a crossing or turning manoeuvre at intersections. Typical traffic movements are shown in Figure 3.4 and Figure 3.5. Information on gap acceptance theory in relation to intersection capacity is provided in the Austroads *Guide to Traffic Management Part 3: Transport Studies and Analysis Methods* (AGTM Part 3) (Austroads 2020d).

MGSD is:

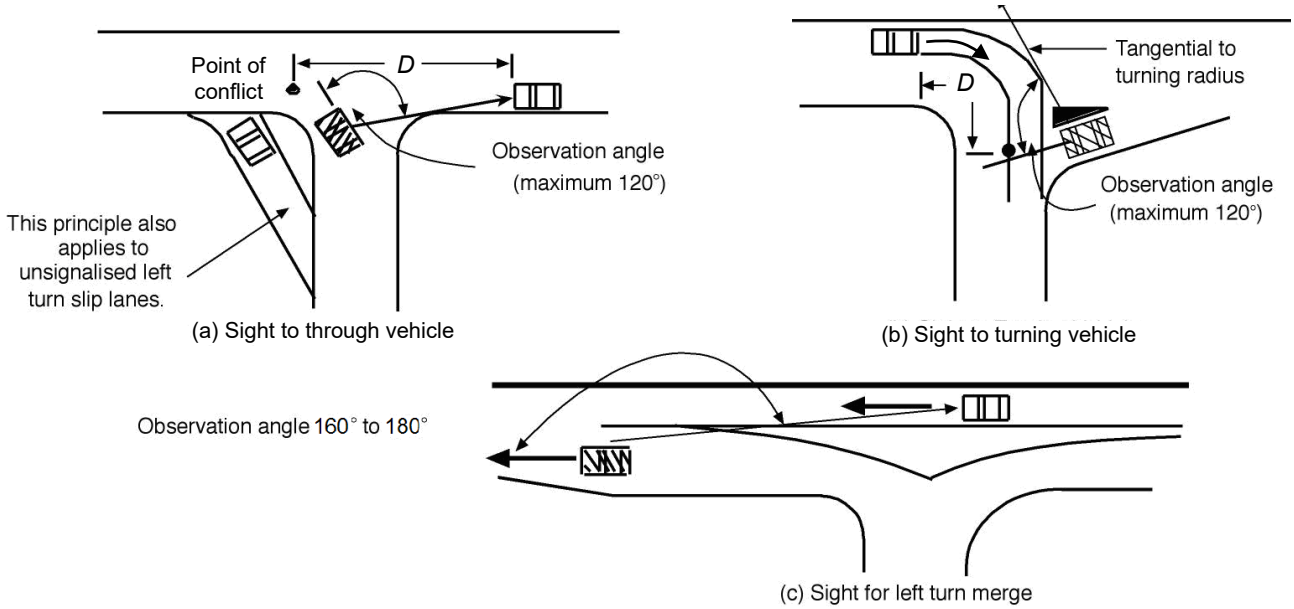
- shown as '*D*' in Figure 3.4 and Figure 3.5
- measured from the point of conflict (between approaching and entering vehicles) back along the centre of the travel lane of the approaching vehicle
- measured from a point 1.1 m (driver's eye height) to a point 0.65 m (object height – typically a vehicle indicator light) above the travelled way.

The MGSD required for the driver of an entering vehicle to see a vehicle in the conflicting streams in order to safely commence the desired manoeuvre is dependent upon the:

- length of the gap being sought (critical acceptance gap time t_a)
- observation angle to approaching traffic.

Figure 3.4 illustrates that for left turns the sighting angle is restricted to a maximum of 120º for a give way situation and 160° to 180° for a free flow left turn. The sighting angles are restricted to a maximum of 110° for right turns, and 170° to 180° for right-turn merges (Figure 3.5).

Figure 3.4: Sight distance requirements and angles for traffic turning left



Note: D is the minimum gap sight distance (MGSD).

Source: Department of Main Roads (2006).

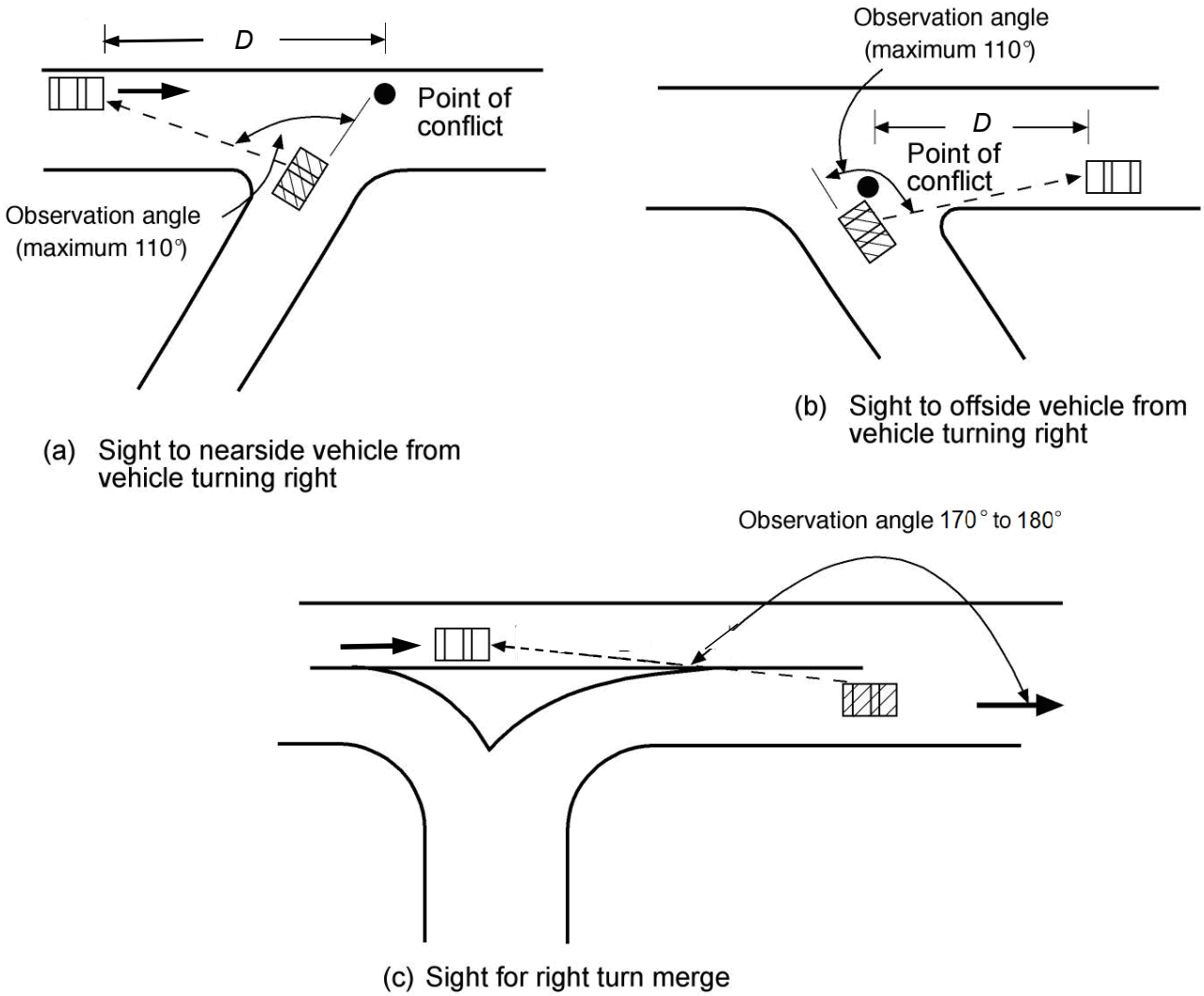
1 Minimum gap sight distance not used in Western Australia.

Critical acceptance gaps and follow-up headways

The critical acceptance gap time varies according to:

- the type of manoeuvre – left-turn/right-turn/crossing
- the width of carriageway – increased time required for greater widths
- whether the major road has a one-way or two-way traffic flow – increased time required to look both ways.

Figure 3.5: Sight distance requirements and angles for traffic turning right

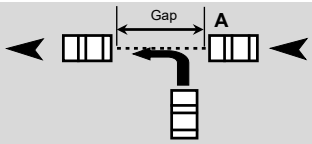
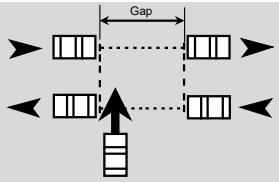
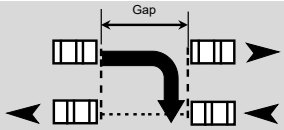
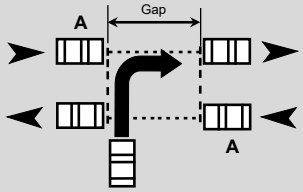
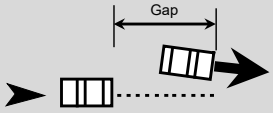


Note: D is the minimum gap sight distance (MGSD).

Source: Department of Main Roads (2006).

Table 3.5 shows critical acceptance gap times for various manoeuvres into, from and across various through carriageway widths for both one-way and two-way traffic. The corresponding distances are given in Table 3.6.

Table 3.5: Critical acceptance gaps and follow-up headways

Movement	Diagram	Description	t_a(1) (sec)	t_f(2) (sec)
Left turn		Not interfering with A Requiring A to slow	14–40 5	2–3 2–3
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 6 8 5 8 8	2 3 4 3 5 5
Right turn from major road		Across one lane Across two lanes Across three lanes	4 5 6	2 3 4
Right turn from minor road		Not interfering with A One way Two lane/two way Four lane/two way Six lane/two way	14–40 3 5 8 8	3 3 3 5 5
Merge		Acceleration lane	3	2

1 *t_a = critical acceptance gap (sec).*
2 *t_f = follow-up headway (sec).*

Notes:

For a description of the follow-up headway and its uses, refer to AGTM Part 3.

The critical acceptance gaps (t_a) listed are based on simple road layouts with an assumed 3.5 m wide lane and no median width to cross. Any geometric features that increase the crossing distance therefore require an increase in the values of ta to be applied. These factors include, a skewed crossing path, auxiliary turn lanes and bicycle lanes, wide centreline treatments and narrow medians. t_a values can be extrapolated for those given in Table 3.5 based on the specific conditions. For example, for a crossing manoeuvre at a two-lane/two-way road with an auxiliary lane, t_a could be assumed using the mid-point between a two-lane/two-way and a four-lane/two-way (e.g. t_a = 6.5).

Source: Department of Main Roads (2006).

Table 3.6: Table of minimum gap sight distances ('*D*' metres) for various speeds

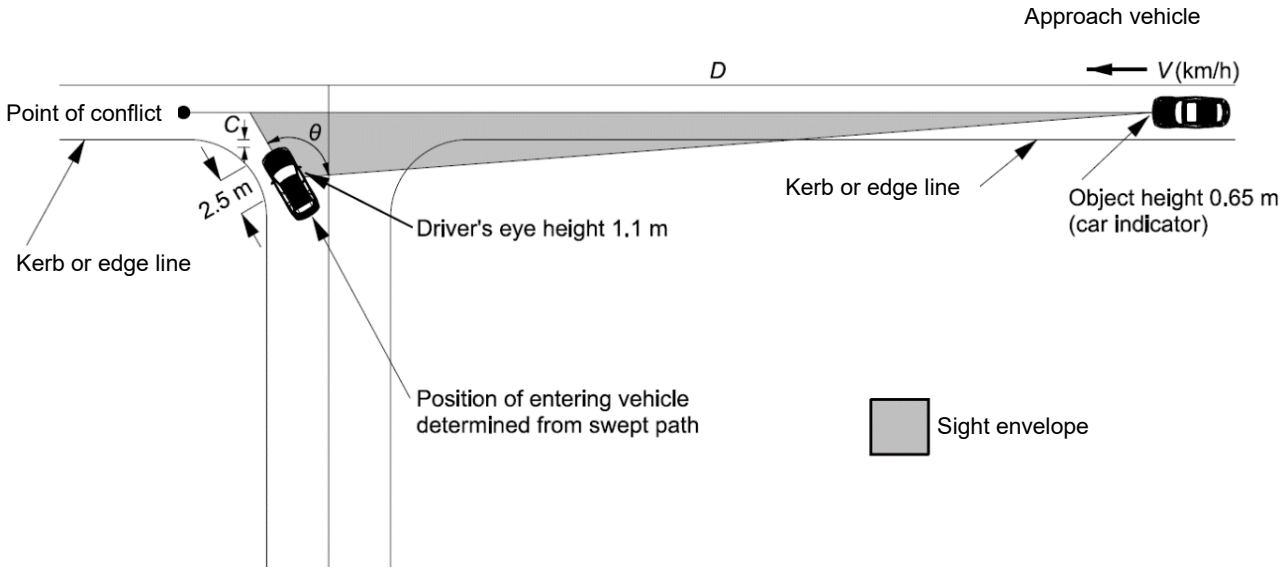
Critical gap acceptance time (t_a) (secs)	85th percentile speed of approaching vehicle (km/h)										
	10	**20**	**30**	**40**	**50**	**60**	**70**	**80**	**90**	**100**	**110**
4	11	22	33	44	55	67	78	89	100	111	122
5	14	28	42	55	69	83	97	111	125	139	153
6	17	33	50	67	83	100	117	133	150	167	183
7	19	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

Detailed sight distance requirements for left-turning drivers

Figure 3.6 illustrates the sight distance to a through vehicle from a vehicle turning left. Sight requirements for left turns depend on the direction of approaching traffic and right-of-way regulations. For drivers of vehicles entering a priority road, sight lines should be considered to:

- through vehicles approaching from the left and right
- turning vehicles on other approaches.

Figure 3.6: Sight distance to a through vehicle from a vehicle turning left



Notes:
Sight envelope:

- *Assess sight distance both horizontally and vertically within this envelope.*
- *For rural areas – there should be no obstructions to sight lines in this area.*
- *For urban areas – fixed objects should not cause entering vehicles to lose sight of approaching vehicles.*

Θ observation angle for new or reconstructed work maximum 120°.
C 0.5 m from kerb or edgeline projection or
1.0 m from stop or give way line.
D minimum distance travelled by approaching vehicle in 5 sec at design speed (V km/h).
Minimum distance travelled by approaching vehicle in 5 sec based on t_a for left-turn from Table 3.5.

Source: Department of Main Roads (2006).

The acceptable maximum observation angle of 120° is based on the visibility requirements from vehicles provided in Commentary 1.

3.3 Pedestrian Sight Distance Requirements

There are two key sight distance requirements at pedestrian crossing facilities: ASD and crossing sight distance (CSD). Figure 3.7 illustrates these two sight distance criteria.

ASD ensures that approaching drivers are aware of the presence of a pedestrian crossing facility. It is important that this line of sight is not obstructed as it 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.

CSD ensures that the pedestrian can see approaching traffic in sufficient time to judge a safe gap and cross the roadway. It also ensures a clear view for approaching drivers to sight pedestrians waiting to cross the roadway.

Pedestrian sight distance requirements are as follows:

- ASD should be provided between approaching vehicles (1.1 m eye height) and the surface of the roadway (generally 0 m or 0.1 m for a wombat crossing) at all formal, marked pedestrian crossings. Refer to Section 3.2.1 for guidance on how to calculate ASD.
- Crossing sight distance (CSD) should be provided between approaching vehicles (1.1 m eye height) and a pedestrian waiting to cross the road (waiting 1.6 m from the pavement edge or kerb line). The pedestrian eye height should be taken as 1.07 m which represents the lower bound of the range applicable to a person in an A80 wheelchair. CSD allows sufficient time for the pedestrian to cross the road, clear of any approaching traffic. CSD should be provided at crossings where the pedestrian does not have the priority or where the pedestrian does have the priority but must be sighted by approaching traffic in order for the approaching traffic to give way (e.g. a zebra crossing). It is also desirable that CSD be provided at crossings controlled by signals in case of signal failure.

CSD is calculated from the critical safe gap (in the traffic stream) and the speed of approaching traffic given by Equation 3:

$$CSD = t_c \times \frac{V}{3.6} \qquad 3$$

where

CSD	=	sight distance required for a pedestrian to safely cross the roadway
t_c	=	critical safe gap (sec) = (crossing length/walking speed) + 3 sec for pedestrian start up and end clearance time
V	=	85th percentile approach speed (km/h)

Notes:

- *Average walking speed is 1.2 m/s; however, there are pedestrians who may walk at different rates and designers need to consider the types of pedestrians and their likely walking speeds.*
- *The crossing length shall include the pedestrian set back (e.g. 1.6 m from pavement edge or kerb line).*
- *The 3 sec for pedestrian start up and end clearance time may not be achievable in constrained situations. A risk assessment should be undertaken if the 3 sec start up and end clearance time is omitted.*

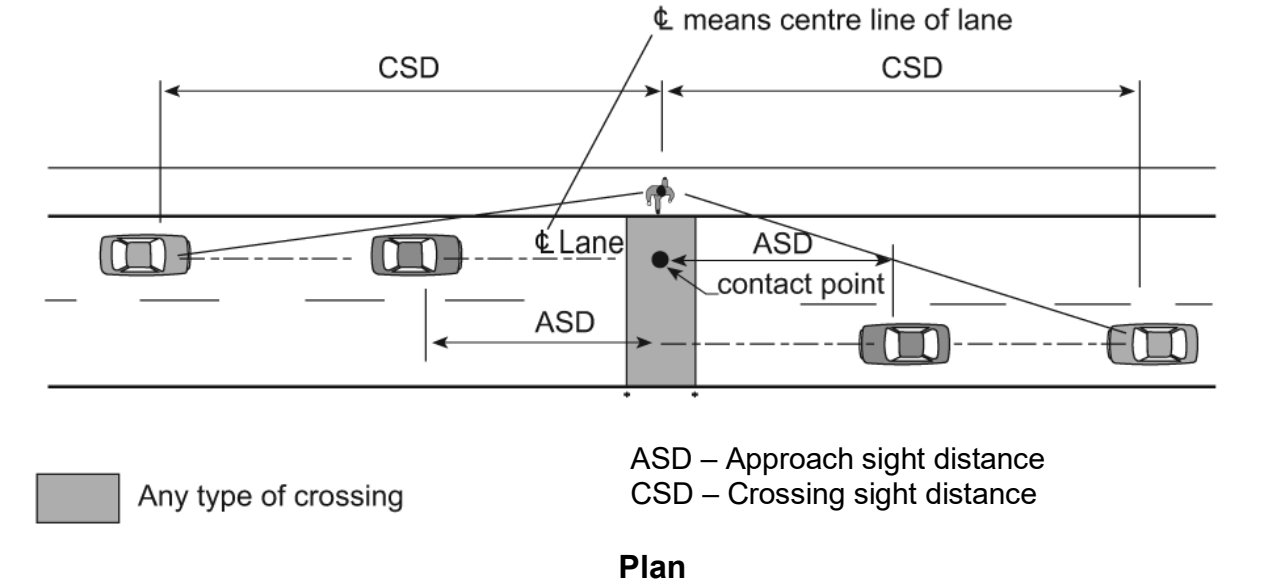
It is important that the line of sight for CSD is not impeded by any object such as:

- street furniture (e.g. poles, mailboxes, telephone booths, trees, decorative planters)
- parked vehicles.

Parked vehicles can cause 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 locations where there is a strong requirement by adjoining land uses to retain legal on-street parking, consideration should be given to extending the width of the footpath to improve the visibility of pedestrians.

Minor obstructions, such as posts, poles and tree trunks less than 200 mm diameter within the sight line may be ignored.

Figure 3.7: Sight distance at pedestrian crossings



Note: The pedestrian offset from the edge of the pavement or kerb line is 1.6 m for determination of the sight triangle.

Source: Department of Main Roads (2006).

In the urban context, it is often difficult to achieve the desired sight distances and there are instances where limiting sight distance for vehicles could be beneficial to achieving a safer and more attractive environment for pedestrians. The development of self-explaining roads (Ambros 2013) or similar local area traffic management (LATM) schemes (Austroads *Guide to Traffic Management Part 8: Local Street Management* (AGTM Part 8) (Austroads 2020b)) may include limiting sight distance to assist in reducing traffic speeds on the approach to intersections.

This approach may not be suitable in all situations and would only be recommended for consideration for local roads in the urban context. Maintaining good sight distance as per the following guidance should still be adopted on higher-volume urban intersections and in the rural context (Corben et al. 2005).

3.4 Sight Distance at Property Entrances

The Austroads *Guide to Road Design Part 4: Intersections and Crossings – General* (AGRD Part 4) (Austroads 2023b) provides guidelines that relate to property access in general. It also provides reference to a New Zealand planning policy manual that covers integrated planning and development of state highways including accessway standards and guidelines (NZ Transport Agency 2007).

Desirably, sight distances at accesses should comply with the sight distance requirements for intersections, i.e. that approach sight distance (ASD), safe intersection sight distance (SISD), and minimum gap sight distance (MGSD) are achieved. However, where this is not possible due to constraints, guidance is provided in Appendix A.3 to apply EDD for sight distance at domestic accesses. AS 2890.1 provides guidance for non-domestic accesses.

The criteria above often cannot be obtained at accesses on roadways with tighter horizontal and vertical alignments, or vegetation. For new roads comprising such geometry, minimum sight distances at accesses should comply with the following:

- minimum gap sight distance in Section 3.2.3
- safe intersection sight distance using values given under the extended design domain (EDD) criteria for sight distance at intersections (Table A 9 to Table A 14).

Obtaining ASD at domestic accesses is preferable but may not always be necessary due to the familiarity with their location of 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.

4. Types of Intersection and Their Selection

4.1 General

This section introduces the types of intersection, their features and the factors that may influence the selection of an appropriate type of intersection to suit a site or circumstances. Generally, these factors include traffic operation, road safety, and physical conditions at the site. Detailed guidance on the layout design of commonly used intersection treatments is provided in Section 7 and Section 8. For detailed information on the type and selection of an intersection, designers should refer to the Austroads *Guide to Traffic Management Part 6: Intersections, Interchanges and Crossings Management* (AGTM Part 6) (Austroads 2020a).

It is important that requirements for safe pedestrian movement and safe cycling should be considered in the development of all intersection layouts and facilities and should be provided where a current or future need exists. Where practicable, bicycle lanes should be continuous through intersections to avoid creating squeeze points due to bicycle lanes ending and forcing cyclists back into the motor vehicle lane.

Where no bicycle facility is provided, the shoulders of major rural roads should be of an appropriate width to cater for cyclists (Austroads *Guide to Road Design Part 3: Geometric Design* (AGRD Part 3) (Austroads 2016a)) and be carried through intersections so that cyclists do not have to travel along the traffic lane and be exposed to conflict with motor vehicles. It is suggested that all traffic islands should be set back at least 1.5 m from the edge of the traffic lane to facilitate safer cyclist movement through intersections.

On designated bicycle routes the shoulders should be sealed and a marked bicycle lane provided within intersections including the length of the auxiliary lane to ensure continuity of the facility.

4.2 Intersection Types

Intersections are generally defined by the type of turn treatments and the types of traffic islands provided. They are often also described as:

- unchannelised and unflared
- unchannelised and flared
- channelised (including roundabouts).

Flaring is a general term for the provision of additional lanes and/or tapers while channelisation is the provision of traffic islands. Flaring and channelisation may be applied to T-intersections or cross-intersections although cross-intersections are not favoured in high-speed situations.

Unchannelised and unflared intersections

This type of intersection is normally adequate where minor roads meet and where a major road intersects with a minor road and does not require turning lanes or traffic islands.

Unchannelised and flared intersections

Simple unchannelised intersections may be flared to provide additional through lanes or auxiliary lanes, such as speed-change lanes or passing lanes. Speed-change lanes allow left-turning or right-turning vehicles to reduce speed when leaving the through road without adversely affecting the speed of through traffic and permit through vehicles to pass another vehicle waiting to complete a turn at an intersection.

Channelised intersections

A channelised intersection is one where paths of travel for various movements are separated and delineated. Raised traffic islands, raised pavement markers, painted markings and safety bars can be used for channelisation.

The simplest channelisation on a major road involves a painted or raised island in the centre of a two-lane two-way road designed to shelter a stationary vehicle waiting to turn right and to guide through vehicles past the turning vehicle.

Channelisation applies to left-turning, right-turning, and crossing vehicles and consequently a particular intersection layout will have a combination of lanes and islands designed to cater for specific traffic movements within the intersection.

Channelisation utilises islands to ‘funnel’, direct and separate vehicles into the required paths through an intersection, and to shelter vehicles that are waiting or moving within an intersection. This gives rise to specific forms of channelised intersection such as staggered T-intersections, seagull treatments, wide median treatments and roundabouts that are provided to achieve particular design objectives.

Further information on the types of intersections is discussed in the AGTM Part 6.

5. Auxiliary Lanes

5.1 General

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 the near and/or off-side, and on the approach and/or departure. On the approach side they are designed on the basis of deceleration models and on the departure side models of acceleration are used.

The two main types of auxiliary lanes related to intersection design are turn lanes (i.e. deceleration and acceleration lanes) and auxiliary through lanes.

Conversion of through lanes into turning lanes should only be used in existing extremely constrained locations because of the poor crash history associated with such treatments. They are not to be used for the design of new intersections.

Conversion of an approach through lane of a multi-lane road into an exclusive right-turn or left-turn lane should be avoided as it may cause some through traffic to change lanes at the last moment, creating a potential for crashes, 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 should be erected informing drivers of what to expect. The signs should be supplemented by pavement arrows.

For guidance on determining the need for auxiliary lanes, refer to Austroads *Guide to Traffic Management Part 6: Intersections, Interchanges and Crossings Management* (AGTM Part 6) (Austroads 2020a).

5.2 Deceleration Lanes

5.2.1 Components of Deceleration Turn Lanes

The design of deceleration turn lane length is based on the performance of cars. It is generally accepted that a design based on the performance of trucks would not be cost-effective and that it is generally acceptable for trucks to commence deceleration in the through lane. However, consideration should be given to providing a longer deceleration lane in situations where there is a high volume of trucks turning.

The length of a deceleration lane will be governed by one or more of the following:

- deceleration from the approach speed to a stop
- deceleration from the approach speed to a turning speed
- additional length required for storage of vehicles queuing while waiting to turn
- diverge distance
- additional length to enable turning vehicles to enter the turn lane when vehicles are queued in an adjacent through lane.
- at the intersection of arterial roads, a longer deceleration length is provided when the volume of trucks is ≥ 10%.

Deceleration from the approach speed to a stop condition is applicable to:

- right turns from two-way roads (all cases)
- right turns from one-way roads where the turn is controlled by a stop sign, give way sign or a traffic signal
- a left turn to a stop, give way or signalised approach
- left-turn slip lanes without a protected acceleration lane on the departure.

Deceleration from the approach speed to a turning speed is applicable to unsignalised:

- right turns from a one-way priority road
- left turns with a protected acceleration lane on the departure
- left turns from a major road where there is no left-turn island (i.e. basic left-turn (BAL) and auxiliary left-turn (AUL) treatments).

The components of an auxiliary lane are shown in Figure 5.1. They apply to both left-turn lanes and right-turn lanes as described previously. The components comprise:

B = total length of auxiliary lane

D = deceleration length (m)

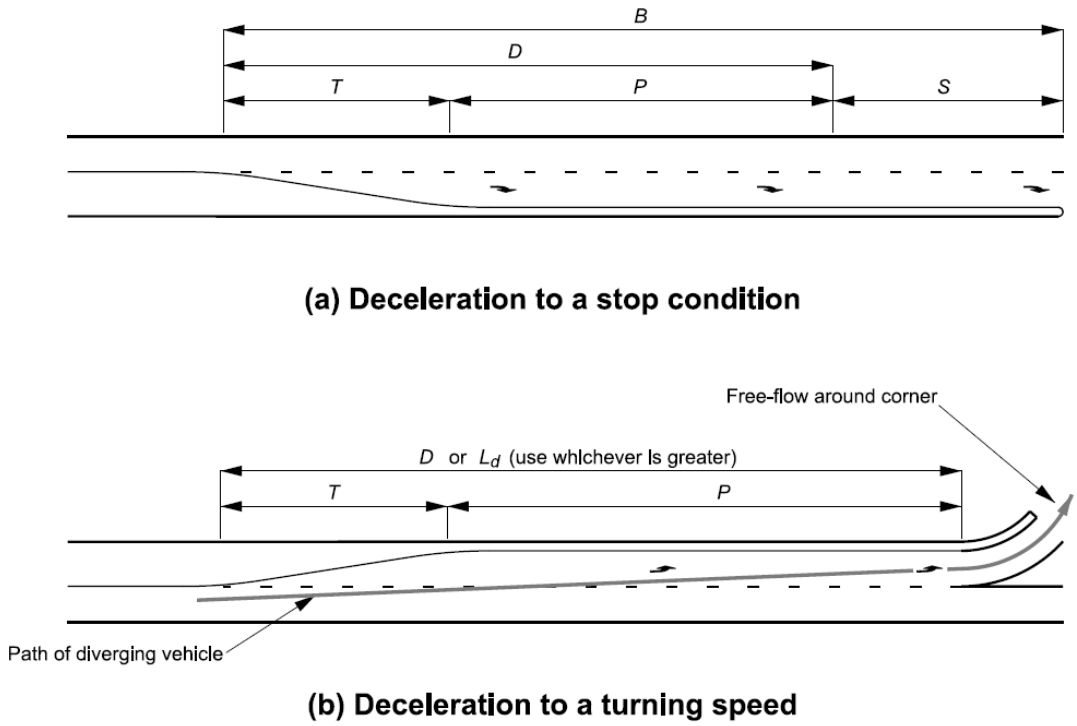
L_d = diverge length (m)

S = storage length (m)

T = physical lane taper length (m)

P = length of parallel lane for deceleration (m).

Figure 5.1: Components of a deceleration turning lane



Source: Based on Department of Main Roads (2006).

Total length of auxiliary lane (B)

The overall length of a deceleration auxiliary lane is determined by either the deceleration length plus storage length (*D* + *S*), or the diverge length depending on circumstances.

Deceleration length (D)

Where drivers are required to stop or give way as described previously, the overall length of deceleration lane will be determined by the deceleration distance for cars plus storage required for the queuing of vehicles ($D + S$) as shown in Figure 5.1(a). The deceleration length is determined from Table 5.2, the use of which is discussed below.

Diverge length (L_d)

When drivers enter turning lanes they will generally prefer to diverge from the through lane at a comfortable rate of lateral movement (i.e. 1.5 m/s) from the centre of the through lane to the centre of the auxiliary lane. In the case of free-flow left-turn lanes, where the driver can drive on a left-turn roadway at a particular speed (i.e. exit speed), the value of D may be relatively small in which case the driver is likely to diverge directly toward the left-turn roadway as shown in Figure 5.1(b). In some situations the diverge length (L_d) will exceed the deceleration length (D), in which case L_d determines the length of lane required (Table 5.2).

In some cases a design may provide for right-turners to leave the road at an 'exit speed'. Table 5.2 also shows values of L_d for two typical lane widths. However, L_d should be based on the distance that the diverging vehicle shifts sideways when undertaking the diverge manoeuvre and Equation 4 can be used to determine L_d in any given situation:

$$L_d = \frac{VY}{3.6S} \qquad 4$$

where

L_d	=	diverge length (metres)
V	=	design speed (km/h). For normal length of deceleration lanes, use the mean free speed of the through road, about numerically equal to the posted speed limit (km/h)
S	=	rate of lateral movement (1.5 m/sec)
Y	=	width of lateral movement (metres)

Where short length deceleration lanes are used, the design speed V may be decreased to the value used at the start of the taper. For example, channelised right-turn (short lane) (CHR(S)) and auxiliary left-turn (short lane) (AUL(S)) turn treatments are based on a 20% reduction in through road speed at the start of the taper. Therefore, the design speed V in these cases may be taken as 80% of the mean free speed of the through road.

Storage length (S)

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 computer programs. Unsignalised intersections, signalised intersections and roundabouts can be analysed. The analysis is usually undertaken by using computer software for which design traffic volumes and a preliminary intersection layout design are required as input.

Physical taper length (T)

The taper length (T) is the physical taper to be constructed at the entry to the lane. It does not represent the path likely to be driven by drivers entering the lane (L_d). It is important that the taper length is not too long to ensure that:

- the commencement of the auxiliary lane is well-defined
- drivers do not inadvertently enter the lane during inclement weather, a situation that is more likely where a deceleration lane is on a horizontal curve
- additional storage capability is provided at urban locations for those times when the 95th percentile queue length is exceeded.

Recommended physical taper lengths for various design speeds in rural and urban areas are shown in Table 5.1. However, in rural situations a shorter taper (e.g. 20 m to 30 m) can be used to provide clearer definition of turn lanes located on curves, and in urban situations a shorter taper (e.g. 10 m to 20 m) should be used to maximise storage in the turn lane.

Table 5.1: Length of physical taper *T* for a 3.5 m lane width

Design speed of approach (km/h)	Taper length *T* (m)
50	15
60	20
70	23
80	25
90	30
100	33
110	35

Note: Values of taper length are rounded.

The values in Table 5.1 assume a desirable lane width of 3.5 m. However, where a different lane width is provided Equation 5 should be used to compute the physical taper length:

$$T = \frac{0.33VW_T}{3.6} \qquad 5$$

where

T = taper length (m)

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

W_T = width of turn lane (m)

Length of parallel lane (P)

The parallel part of the deceleration length (P) is deduced from $D - T$.

5.2.2 Determination of Deceleration Turning Lane Length

Procedure

Table 5.2 shows the distances required for deceleration (including the physical taper) required for cars on a level grade and the diverge length required to change lanes, for a range of design speeds.

Table 5.2 should be used as follows:

- Where vehicles are required to stop or to give way the deceleration distance from the 'stop condition' column for a comfortable deceleration rate of 2.5 m/sec^2 should be used.
- The column for a maximum design deceleration rate of 3.5 m/sec^2 should only be used where it is impracticable to adopt the 'comfortable' rate. This usually involves situations where an intersection is located adjacent to a design constraint and it is not feasible to relocate either the intersection or the constraint (e.g. bridge abutment, bridge pier, utility that would be excessively expensive to modify or relocate), in order to achieve a deceleration lane length that provides for the 2.5 m/sec^2 rate.
- In situations where a turning vehicle does not have to stop or give way and is able to turn a corner at speed, less deceleration distance is required (shaded green and pink in Table 5.2). L_d should be used where it exceeds the value shown in the area shaded green.

Table 5.2: Deceleration distances required for cars on a level grade

Design speed of approach road (km/h)	Length of deceleration D – including diverge taper *T* (m)										Diverge length L_d[3] for lane widths (m)	
	Stop condition[1] (m)		Design speed of exit curve (km/h)[2]									
	0	0	20	30	40	50	60	70	80	90	3.5 m[4]	3.0 m[4]
	Comfortable 2.5 m/s^2	Maximum 3.5 m/s^2	Comfortable average rate of deceleration 2.5 m/s^2									
50	40	30	30	25	15						33	27
60	55	40	50	40	30	15					40	33
70	75	55	70	60	50	40	20				47	40
80	100	70	95	85	75	60	45	25			54	44
90	125	90	120	110	100	85	70	50	25		60	50
100	155	110	150	140	130	115	100	80	55	30	67	57
110	185	135	180	175	160	150	130	110	90	60	74	62

1 *Rates of deceleration are: 2.5 m/s^2 for comfortable deceleration; 3.5 m/s^2 is the maximum for design purposes.*
2 *Speed of exit curve depends on radius and crossfall (Figure 5.2).*
3 *Distance L_d assumes a lateral rate of movement of 1.5 m/s.*
4 *Example lane widths – use actual lateral shift distance of vehicle.*

Notes:
The pink shading indicates that the deceleration lengths given are greater than the diverge length. The length of the deceleration lane should be based on these values.
The green shading indicates that the diverge length is greater than the deceleration length. In these cases, the length of the deceleration lane should be based on the diverge length (the values shown in yellow shading).
Adjust for grade using Table 5.3.

Source: Department of Main Roads (2006).

For design speeds not listed in Table 5.2, the following formula (Equation 6) can be used to calculate the deceleration length for a stop condition:

$$D = \frac{V^2}{254d} \qquad 6$$

where

D = length of deceleration (m)

V = speed (km/h)

d = coefficient of deceleration

The coefficient of deceleration can be converted from the rate of deceleration using Equation 7:

$$d = \frac{a}{9.81} \qquad 7$$

where

d = coefficient of deceleration

a = rate of deceleration (m/s²) (refer to Table 5.2 for values)

Furthermore, for design speeds not listed in Table 5.2, Equation 8 can be used to calculate the deceleration length for a particular exit speed:

$$D = \frac{V^2 - U^2}{254d} \qquad 8$$

where

D = length of deceleration (m)

V = initial speed (km/h)

U = exit speed (km/h)

d = coefficient of deceleration

The deceleration distance determined from Table 5.2 or Equation 8 should be increased for a downgrade and may be reduced for an upgrade in accordance with Table 5.3.

Table 5.3: Correction to deceleration distance *D* for grade

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

Source: Department of Main Roads (2006).

Figure 5.2 is used to determine the design speed of the exit curve required in Table 5.2. The calculation method is shown below (Equation 9).

As discussed previously, the length of auxiliary lane should be the larger of the diverge length (L_d) or the deceleration length (D). The distances for L_d in Table 5.2 are based on a rate of lateral shift of 1.5 m/sec and lane widths of 3.5 m and 3.0 m. Where a different lane width applies, or a designer has a case to use a different rate of lateral shift, Equation 4 can be used to determine L_d.

Where a turning lane caters for a high percentage of heavy vehicles consideration may be given to increasing the length above that required for cars so that less interference to traffic flow occurs in the through lane as a result of trucks slowing. However, it is accepted that turning lanes should generally be designed for the deceleration of cars and that heavy vehicles may reduce speed in the through lane.

Turning speeds for various radii (R) and crossfall are calculated from the formula:

$$R = \frac{V^2}{127(e+f)} \qquad 9$$

where

R = curve radius (m)

V = speed (km/h)

e = superelevation (m/m)

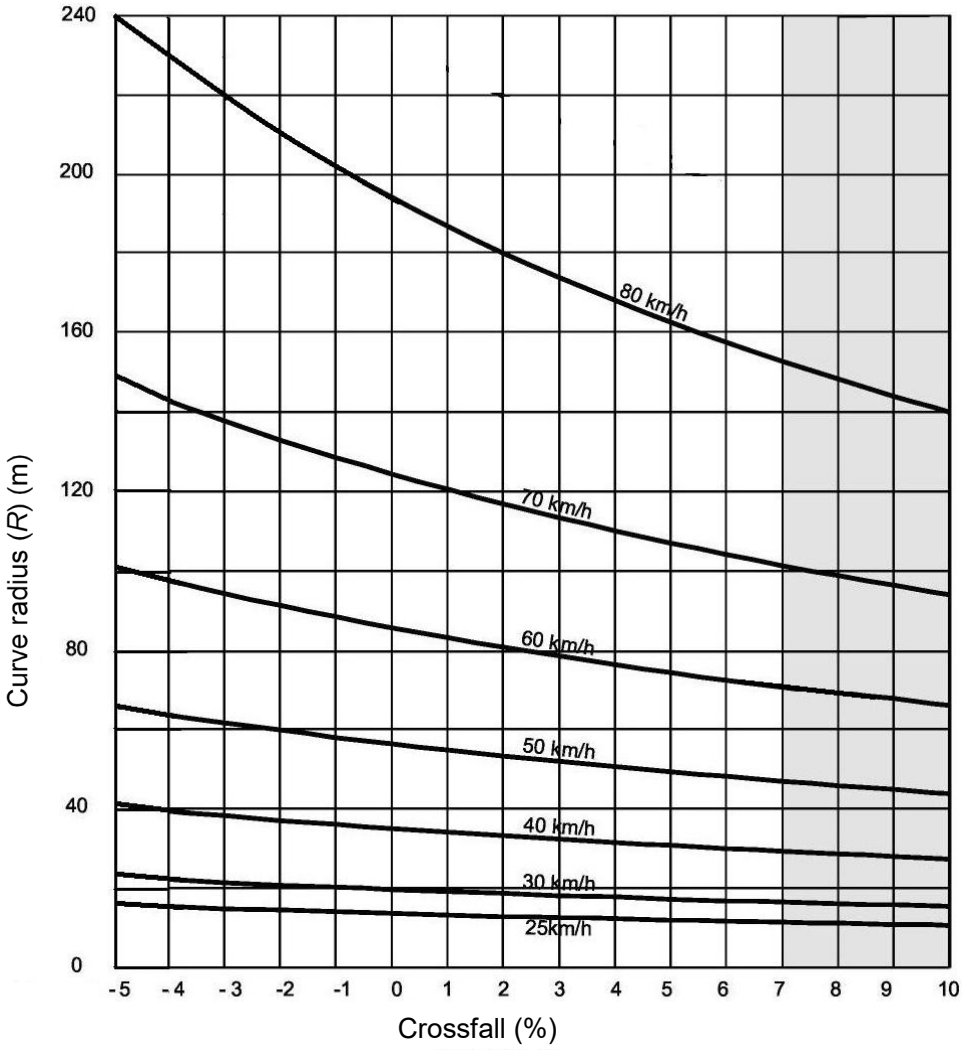
f = side friction factor between vehicle tyres and the pavement (refer to Table 5.4)

Table 5.4: Side friction factors

V (km/h)	*f* – maximum	*f* – desirable
25	0.36	0.30
30	0.36	0.30
40	0.36	0.30
50	0.35	0.30
60	0.33	0.24
70	0.31	0.19
80	0.26	0.16
90	0.20	0.13
100	0.16	0.12
110	0.12	0.12
120	0.11	0.11
130	0.11	0.11

Source: Derived from Department of Main Roads (2006) and AGRD Part 3.

Figure 5.2: Turning speeds for various combinations of radius and crossfall



Notes:
A maximum crossfall of 8% should be adopted for turning roadways.
Adverse crossfall should be as flat as possible consistent with drainage requirements and should not be steeper than –3%. The 'effective adverse crossfall' should not be steeper than –5%. (Effective crossfall is the vectorial sum of the longitudinal grade and crossfall).
Graphs are based on the maximum side friction factors shown in Table 5.4.

Source: Department of Main Roads (2006).

Practical application of the procedure

This section provides guidance on the factors that influence the length of a deceleration lane and the process of determining the length. However, situations may arise in practice where engineering judgement must be applied by the designer in determining an appropriate length.

High-speed rural and urban roads

On high-speed rural and urban roads (≥ 90 km/h) with moderate to high traffic volumes it is important for road safety that turning vehicles do not impede through traffic. It is therefore most desirable that turning vehicles (i.e. cars) do not decelerate in the through lane, and that the deceleration lane is long enough to cater for deceleration and storage.

Generally, in rural areas the storage length will not be large and the resulting length of lane (i.e. *D* + *S*) will be within practical limits. An exception is facilities on recreational routes which may periodically experience long queues. In these cases analysis should be undertaken to determine the queue length (*S*) likely to be experienced in the design year and the economic value of adopting a length that can accommodate the required storage.

Low to moderate-speed urban arterial road intersections

At major intersections on low to moderate speed urban arterial roads (< 90 km/h) it is desirable to provide for deceleration plus storage (*D* + *S*) for turning traffic. However, in situations where the road system is congested and queues are long it may not be practicable to provide for *D* + *S*. If this is the case designers should do everything practicable to at least accommodate the 95th percentile queue (i.e. storage length) within the turn lane as shown in Figure 5.3(a).

Another consideration at urban arterial road intersections is access to auxiliary lanes for turning vehicles. Many turning movements at signalised intersections are catered for by a 'leading' right-turn phase or a left-turn phase that runs concurrently with a right-turn phase from the intersecting road. A frustration occurs for some turning drivers when queues in the through lanes block access to the turning lanes and drivers are unable to utilise the green turn arrow (Figure 5.3(b)).

The inability of turning vehicles to access turn lanes can also adversely affect the capacity of an intersection and result in vehicles encroaching onto medians and causing maintenance issues.

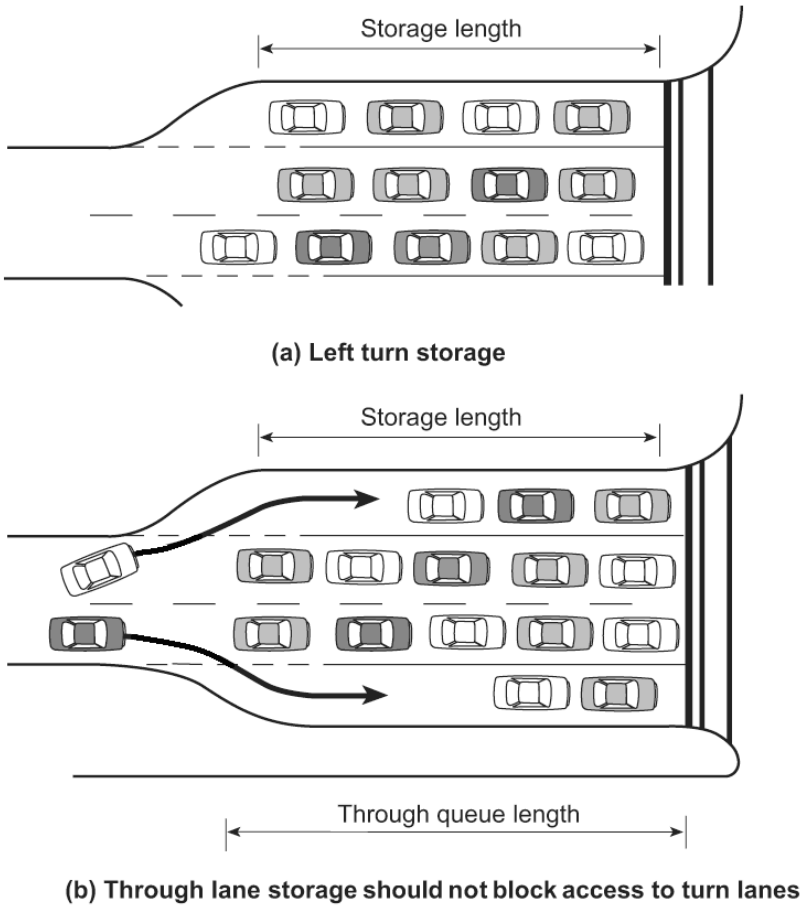
The length of turning lanes possible at major urban intersections is often influenced by the existence of physical constraints on achieving a greater length. The constraints may include:

- other intersecting roads
- a railway crossing on an approach
- a road bridge
- the abutment or pier of an overpass.

In all cases the costs and benefits of removing the constraint should be investigated to determine the most effective solution.

At intersections between low to moderate speed arterial roads (< 90 km/h) and collector roads it is desirable, because of the lower turning volumes, to provide a length equivalent to *D* + *S* for vehicles turning right and left from the major road.

Figure 5.3: Storage length and through lane queue blocking access to turn lane during red signal



Source: Based on Department of Main Roads (2006).

5.3 Acceleration Lane for Cars

5.3.1 General

While this section is concerned with the design of acceleration lanes provided for turning traffic, the principles may be applied to other road design situations such as acceleration distances required for drivers using ramp meters on freeway/motorway on-ramps.

The length of an acceleration lane is governed by the acceleration requirements from a stop or turning speed to the speed of the through traffic on the road being entered. The acceleration should occur wholly within the lane.

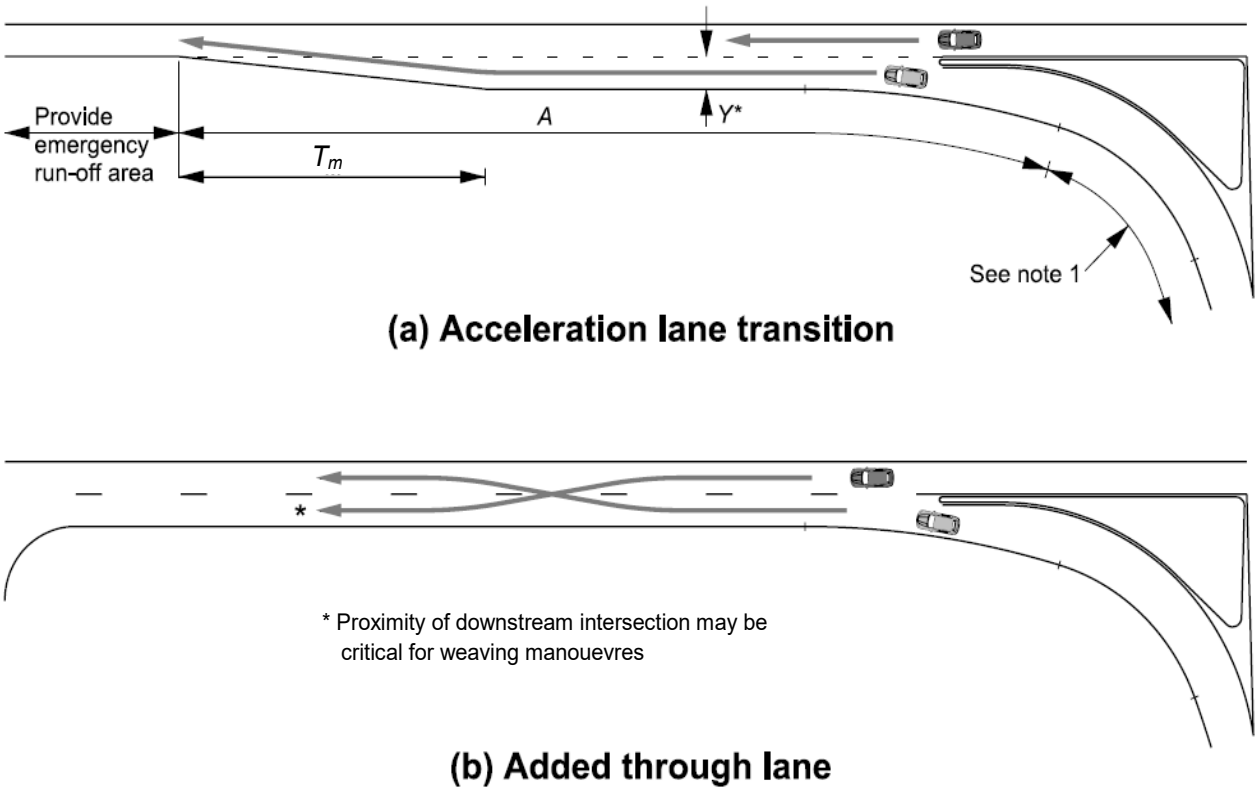
While the design of an acceleration lane is usually based on the performance of cars, situations may arise where the performance of trucks needs to be considered (Section 5.4).

An acceleration lane has two basic design requirements:

- acceleration length
- merge length.

Figure 5.4 shows a typical arrangement for an acceleration lane and an option where the lane continues as an additional through lane. This type of treatment may require the provision of a signalised crossing for pedestrians and cyclists to enable them to safely cross the turning roadway.

Figure 5.4: Options for auxiliary acceleration lanes



1 *See Figure 5.2 for turning speeds.*

Notes:
Y – width of lateral movement.*
A – acceleration lane length, see Table 5.5.

Source: Department of Main Roads (2006).

5.3.2 Acceleration Distance

The process to determine the minimum length of acceleration lane is:

- select the appropriate turning speed from Figure 5.2 for turn radius and crossfall
- using this turning speed, determine the overall acceleration lane length from Table 5.5 and adjust for grade using Table 5.6.

The values for the minimum length of an acceleration lane in Table 5.5 are based on the greater of:

- the length required to accelerate from the turning speed to the design speed of the road being entered
- the distance travelled in four sec by the driver using the acceleration lane (to enable the driver time to observe potentially conflicting vehicles in the right-side rear-view mirror, and to prepare to merge) plus the required merge length.

The values shown in Table 5.5 apply to both urban and rural conditions and are:

- for cars and include the merge taper
- based on the data from research studies on instantaneous acceleration rates of a typical passenger car.

Acceleration rates for a typical passenger car are provided in Commentary 2.

While it is desirable in most cases that the accelerating vehicle reaches the mean free speed of the adjacent through lane (about numerically equal to the posted speed limit) before merging, in some situations where the site is constrained and the volume in the through lane is low it may be acceptable to design for a speed decrement of 20 km/h within the merge area (i.e. a merging vehicle travelling at 80 km/h enters a traffic stream of vehicles travelling at 100 km/h).

Table 5.5: Length of acceleration lanes for cars on level grade

Design speed of road entered[1]	Length of acceleration lane *A* (m) (including length of merge taper)								4 sec travel (m)	Merge T_m (m)	Min. desirable length 4 sec + T_m[3]
	Design speed of entry curve (km/h)										
(km/h)	0[2]	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	500	450	320	120	110	230

1 *For the purpose of calculating acceleration lane lengths at intersections, the speed reached is usually made equal to the mean free speed. In the absence of local data it can be assumed that the mean free speed is approximately equal to the speed limit.*
2 *Length required where a vehicle accelerates from zero speed.*
3 *Minimum desirable values have been rounded.*

Notes:

- *Values in the non-shaded areas are based on the distance required to accelerate from the turning speed to the design speed of the road being entered.*
- *For values in the green-shaded areas adopt the minimum desirable length.*

Source: Roads and Maritime Services (2015) and Roads and Traffic Authority (2011).

Table 5.6: 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[(1)] 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	0.70					0.60				
60	0.70					0.60				
80	0.65					0.55				
100	0.60					0.50				
110	0.60					0.50				

1 *Ratio from this Table multiplied by length in Table 5.5 gives length of speed change lane on grade.*

Source: Roads and Traffic Authority (1999).

5.3.3 Merge Taper T_m

The merge length is the distance required for a vehicle to merge from the auxiliary lane into the adjacent through lane. The physical taper at the end of an acceleration lane should be equal to the desirable merge length (T_m). The taper is included in the overall acceleration length for an auxiliary lane shown in Table 5.5.

For an acceleration lane it is assumed that drivers are expecting that they will have to merge and can therefore comfortably merge at a lateral rate of 1.0 m/sec (basis used for T_m in Table 5.5). For most practical purposes the acceleration lane merge length T_m approximates the design speed (V km/h). However, if a lane width other than 3.5 m is proposed, or a different rate of lateral movement is considered to be appropriate, a specific value can be calculated for T_m using the formula in Commentary 3.

5.4 Acceleration Lanes for Trucks

The speed of heavy vehicles needs to be considered when designing acceleration lanes. For the design of new acceleration lanes it is preferable that the design heavy vehicle has sufficient length (refer to Table 5.7 and Table 5.8) to accelerate to a speed no less than 20 km/h below the mean free speed of the through road, particularly if the acceleration lane is on a dedicated heavy vehicle route.

As trucks require very long acceleration distances, often to an extent that is not possible to accommodate in practice, a speed differential between general traffic and heavy vehicles will usually have to be accepted at the point of merging.

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 and considered when designing the length of acceleration lanes.

If the speed of heavy vehicles at the merge is much slower than the speed of the through traffic (30 km/h to 40 km/h difference or more) consideration should be given to extending the length of the acceleration lane. If this cannot be achieved, refer to Appendix A.10 which provides alterative EDD treatments.

Table 5.7 provides a guide to the acceleration lane lengths that are 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 can be seen that, depending on gradient, the lengths are generally within practicable limits.

Table 5.7: Acceleration lane lengths (m) for semi-trailers to accelerate from rest to a specified speed on a level or downgrade

Downgrade (%)	Truck speed (km/h)				
	100	90	80	70	60
0	2400	1500	910	550	320
1	1400	940	640	410	250
2	970	700	500	330	210
3	760	560	400	280	180

Note: 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).

Source: Based on Austroads (2002).

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 or even a reasonable decrement below the speed of a through lane. Graphs in Commentary 4 provide speed profiles for a semi-trailer on various gradients (Austroads 2002) and reference to relevant computer software. Table 5.8 shows acceleration lengths scaled from the graphs for upgrades and various decrements relative to the through lane speed. Furthermore, it indicates that a decrement of 10 km/h or 20 km/h is generally not practicable in terms of an acceleration lane length where the through road speed is 100 km/h. See Commentary 4.

Table 5.8: Acceleration lane lengths (m) for semi-trailers to accelerate from rest to a speed on an upgrade

Upgrade (%)	Truck speed (km/h)						
	100	90	80	70	60	50	40
1	–	–	2000	890	480	230	100
2	–	–	–	–	890	320	130

Note: Dashes indicate that it is not practical to provide sufficient acceleration lane length for semi-trailers to reach the speeds indicated.

Source: Based on Austroads (2002).

5.5 Auxiliary Through-lane Design

Auxiliary through lanes may be used at signalised intersections as a means of increasing capacity. They are often introduced on the immediate approach to an intersection and terminated on the departure. It is desirable that the lane commences far enough in advance of the intersection to accommodate a queue length that will enable saturation flow to be maintained across the stop line for the duration of the green time. The queue can be estimated from the maximum green time allocated to the approach in the signal timings. For example:

- Assuming that the green time is 40 sec and that traffic is discharged across the stop line of the auxiliary lane at 2 sec headways (average) a total of 40/2 = 20 vehicles will be discharged during the green signal phase.
- The queue that can be discharged is therefore 20 x 8 m = 160 m which is a guide to the length of parallel lane that should be provided on the approach.

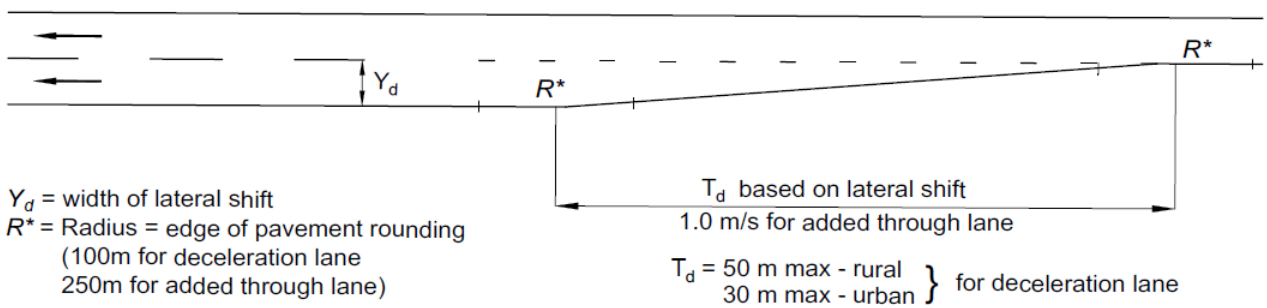
However, the signal analysis should be examined to assess the potential for the queue in the adjacent through lane to block access to the added short through lane.

It is also important that the departure lane is long enough to enable the capacity of the approach lane to be utilised. In order to satisfactorily discharge a queue similar to that described, it is suggested that the length of the parallel departure lane should be based on about 4 sec to 6 sec of travel time at the operating speed of the through lane plus a taper length.

For a through lane the physical diverge taper T_d on the approach should be based on a lateral rate of movement of 1.0 m/s. The merge taper on the departure T_m should be calculated on the basis of a lateral rate of movement of 0.6 m/sec which is more generous than the merge length associated with acceleration lanes. This should also be used where no run-out area is available. Refer to Equation 4 in Section 5.2.1 to calculate the taper lengths.

The development of the through lane can be seen in Figure 5.5.

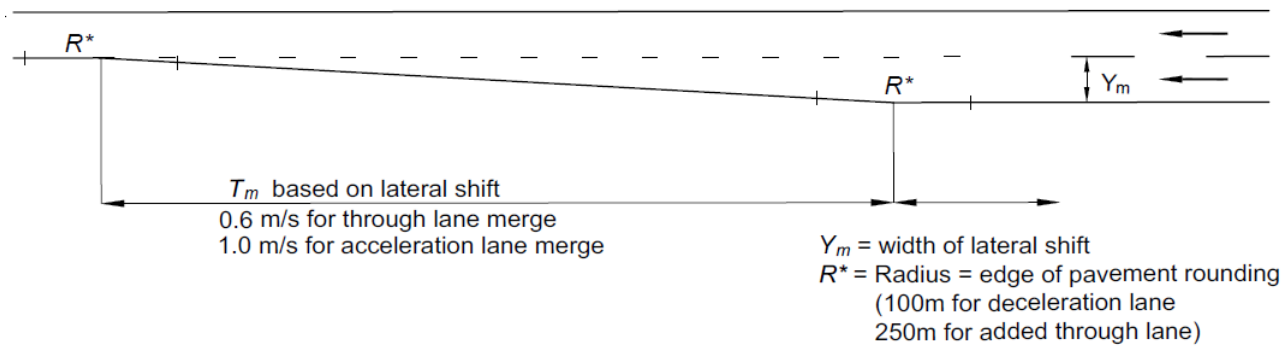
Figure 5.5: Development of auxiliary through lane approach



Source: VicRoads (2011).

On the departure side of the intersection, the auxiliary lane is merged as shown in Figure 5.6.

Figure 5.6: Merging of auxiliary lane



Source: VicRoads (2011).

5.6 Other Considerations

For safety reasons merges should never be installed:

- over a crest that has a sight distance less than the ASD or on the inside of a horizontal curve where the radius of the right-side lane line is less than
  - 185 m for a 60 km/h design speed
  - 330 m for an 80 km/h design speed
  - 515 m for a 100 km/h design speed
  - 620 m for a 110 km/h design speed.

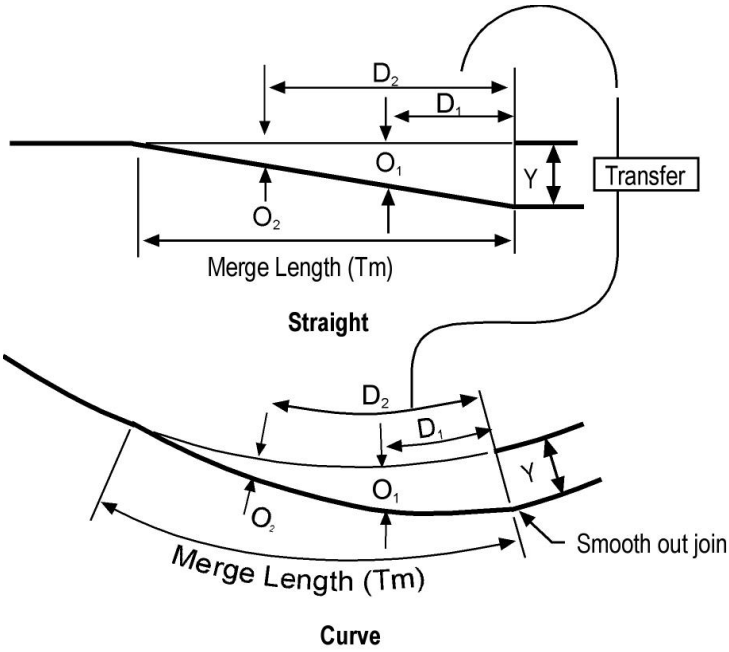
This is based on a 3.5 m lane width and a 2 sec gap. For 3.0 m wide lanes a 20% larger radius applies.

It is also important that:

- approach sight distance (ASD) is available at all points along the merge length to allow drivers to observe the linemarking with sufficient time to react
- merges should be avoided on horizontal curves
- merge transitions around horizontal curves be developed as for a straight alignment and transferred to the curved alignment by the distance and offset method
  This is shown in Figure 5.7.
- 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. In plane geometry, concentric circular arcs cannot be joined tangentially by a third circular arc, unless they are joined over exactly 180°.
  The curve within the merge length in the bottom half of Figure 5.7 is obtained by linear interpolation only; it is not a circular arc.

Where an intersection is located downstream of the end of the acceleration lane it is important to verify that sufficient weaving distance is available for drivers using the acceleration lane who wish to turn right at that intersection. The same applies to drivers in the through lane who may wish to turn left at that intersection (Figure 5.4(b)). This will require traffic analysis in accordance with the Austroads *Guide to Traffic Management Part 3: Transport Studies and Analysis Methods* (AGTM Part 3) (Austroads 2020d).

Figure 5.7: Procedure for plotting a merge on a curved alignment using offsets from a straight road



6. Traffic Islands and Medians

A traffic island is an area provided to separate and direct traffic. Medians are a dividing strip provided to separate traffic flowing in opposite directions. Traffic management aspects of traffic islands and medians are provided in Austroads *Guide to Traffic Management Part 6: Intersections, Interchanges and Crossings Management* (AGTM Part 6) (Austroads 2020a).

The width and type of median on a road leading into an intersection may influence its design. Designers should refer to the Austroads *Guide to Road Design Part 3: Geometric Design* (AGRD Part 3) (Austroads 2016a) for guidance on the design of medians in mid-block situations.

Traffic islands and medians can be raised, depressed, painted, or defined by contrasting material on the pavement. There are no numerical warrants for the provision of the various types of islands or medians at intersections; however, there are a number of considerations that assist in the decision. The requirement for a traffic island or median tends to be site specific and consideration may depend on a subjective assessment of turning volumes, the percentage of trucks and the road agency's knowledge of driver behaviour at other sites in the jurisdiction.

Painted medians and traffic islands do not have the same degree of physical control or conspicuity as raised treatments. Painted treatments are often used, particularly to define right-turn lanes, in constrained situations or where there are budgetary constraints.

Traffic islands and median noses at an intersection should be designed to suit the turning paths of the design vehicle and to maintain continuity of the major road through the intersection. The decision to use painted or raised medians and traffic islands often requires individual assessment: assessing the relative advantages and disadvantages of raised medians and islands listed in Table 6.1, compared to flush or painted treatments may assist in decision making.

Table 6.1: Relative advantages and disadvantages of raised islands and medians

Advantages	Disadvantages
A raised island or median: • provides improved conspicuity of the median and islands, particularly during inclement weather • physically restricts turning or crossing movements • can safely accommodate signposting, traffic signal hardware and road lighting • provides refuge areas for pedestrians • provides positive guidance for turning and through vehicles • can concentrate and direct rainfall runoff into channels and thus reduce the risk of hydroplaning • provides an opportunity for landscaping • assists in controlling vehicle speeds • approach sight distance may not be required to pavement surface.	A raised island or median: • 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 costs to cater for the concentrated rainfall runoff • could generate safety issues if struck by fast moving traffic • may require more space due to greater lane widths to cater for broken-down vehicles and the need for offsets to kerbs • may block access for through traffic 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) • may have traffic furniture which is prone to damage by errant and over-dimensional vehicles • is often a more costly treatment.

They should be designed and located with regard to the following considerations:

- The proper line of travel should be obvious and any changes in direction should be gradual and smooth.
- Approach sight distance (refer to Section 3.2.1), measured from 1.1 m to the pavement level for painted medians and 1.1 m to 0.1 m for raised medians, must be provided on the minor road and is desirable on the major road.
- Small islands with the inherent problem of low target value should be avoided.

Guidelines, on the use and restrictions on the use of types of kerb and channel are summarised in Table 6.2. Kerbs are an obstruction on the road (especially barrier kerbs), so the appropriate type of kerb should be highly visible and have properly designed and adequately maintained approach delineation (e.g. painted lines and marking, and raised pavement markers).

While it is recognised that there may be a desire to construct kerb and channel from materials other than concrete in order to satisfy heritage or urban design requirements, the kerb and channel on arterial roads should be smooth so that it does not damage vehicle tyres, and light in colour to assist roadside delineation (e.g. kerbs constructed of bluestone pitchers often have relatively sharp edges and offer poor delineation at night).

Generally, the kerb and/or channel should not be placed in front of safety barriers because it will adversely affect the operation of the barrier (Austroads *Guide to Road Design Part 6: Roadside Design, Safety and Barriers* (AGRD Part 6) (Austroads 2022)). Where the situation is unavoidable safety barriers should be placed as close to the kerb face as possible but not less than 200 mm.

Table 6.2: Use and restrictions on use of kerb and channel

Type	Use	Restriction on use
Semi-mountable	• Preferred for use on medians, traffic islands and outer kerb lines of all intersections, particularly where the speed zone is ≥ 80 km/h • Can be driven over by slow-moving vehicles passing a broken-down vehicle, where insufficient width is available on the road surface	
Barrier	Should be considered: • where it is essential to prevent vehicles from moving upon areas used by pedestrians, typically during on-street parking manoeuvres, but also at sharp left-turn kerb returns • as protection for traffic signal poles • in car parks • in shopping areas • when matching into council kerbing • under or close to a bridge barrier • where it is more suitable for drainage behind a safety barrier	Not recommended for use: • under guardrail on high-speed routes because the rail deflects on impact and the barrier kerb and rail combination may form a ramp to launch errant vehicles • on high-speed roads (i.e. > 80 km/h, as it is more likely to trip and overturn a vehicle which is out of control
Fully mountable	May be used: • on the leading nose of a median or traffic island in order to extend the island nose where space or funding is limited • for heavy vehicle over-run areas within an intersection • at the interface of indented bus bays with the adjacent traffic lane	• Use to delineate encroachment areas for heavy vehicles may not be supported in some jurisdictions
Channel	• Channel of semi-circular cross-section may be provided at the rear edge of the shoulder in some rural situations. • May be used along the edge of the through lane opposite a bus embayment	Not to be provided: • where vehicle wheels may become trapped in a semi-circular channel

6.1 Raised Traffic Islands and Medians

6.1.1 Raised Islands

Raised islands are preferred where there is a need to:

- physically control and direct traffic movement within an intersection (i.e. channelisation)
- control movements to or from property accesses in the vicinity of an intersection
- provide refuge for pedestrians and cyclists crossing the road
- locate traffic control devices in a prominent position for approaching drivers
- provide consistent treatments along a route.

Raised islands should be constructed of semi-mountable kerbs (Figure C5 1 in Commentary 5 for examples of kerb types). Barrier kerbs and other profiles are not favoured for use on islands. Depressed islands can also be outlined using kerbs, provided that adequate definition and delineation of the island can be achieved by other means (e.g. berm behind the kerb). See Commentary 5.

The size and shape of traffic islands vary according to site conditions and the design vehicles for various traffic movements, the need to accommodate pedestrians and/or cyclists, and roadside furniture within the island. In addition, clearances to the edge of traffic lanes are necessary based on the traffic speed in the adjacent lane (Table 6.5). Where street furniture is installed on a traffic island or median, it should be located clear of the swept path of the design and check design vehicle (refer to Austroads 2023a).

Apart from functional aspects, a key consideration is that the island should be conspicuous to drivers approaching at the operating speed of the approach road. Rural sites with few constraints will have relatively large islands (e.g. ≥ 100 m^2 for a splitter island on an important approach to an arterial road) whereas an unsignalised urban intersection may have a small island.

Where raised islands are used:

- Island noses should be offset from the edge of the adjacent traffic lane to provide additional clearance to the kerb to enhance comfort for approaching drivers and prevent any tendency for them to shy away from the kerb. As a general guide it is suggested that the island nose be offset by 0.2 m per 10 km/h of approach speed but this is not used by all jurisdictions. On narrow islands where an offset to the approach nose is not practicable a fully mountable nose may be provided, which requires a smaller off-set and nose radius than a kerb.
- ASD should be available to all island noses on the minor road.
- The radii at the ends or corners of islands will depend on the size of the island designed for the particular site.

6.1.2 Raised Medians

General

In this Part a median is considered to be any island that separates traffic travelling in opposite directions and includes short islands on the approaches to intersections that are referred to as splitter islands in some jurisdictions. Median islands also include islands in the centre of a major (priority) road that are provided on the approaches to intersections (i.e. CHL treatment).

Raised median islands are used on approaches to intersections primarily to separate opposing traffic streams, but also to warn drivers of the presence of an intersection, provide refuge for pedestrians, reduce the number of points of crossing conflict and to shelter right-turning vehicles.

Wherever practicable, intersections should be designed to provide ASD to the pavement markings at the intersection (e.g. holding line). However, where this cannot be achieved because of limited visibility to intersections that are located on crests or relatively tight curves, raised median islands in the major road can be used to improve driver perception of the intersection. In such cases the island nose should be designed to a length that carries it over the crest or around the curve to a point where it can be easily seen.

In high-speed areas, or where the intersection layout does not adequately mitigate the head-on collision risk, designers should consider supplementing the raised median with a barrier system.

When determining which barrier system to install, designers should consider the impact that this will have on the general readability for all movements at the intersection as well as the barrier's more obvious impact on the different sight distance requirements.

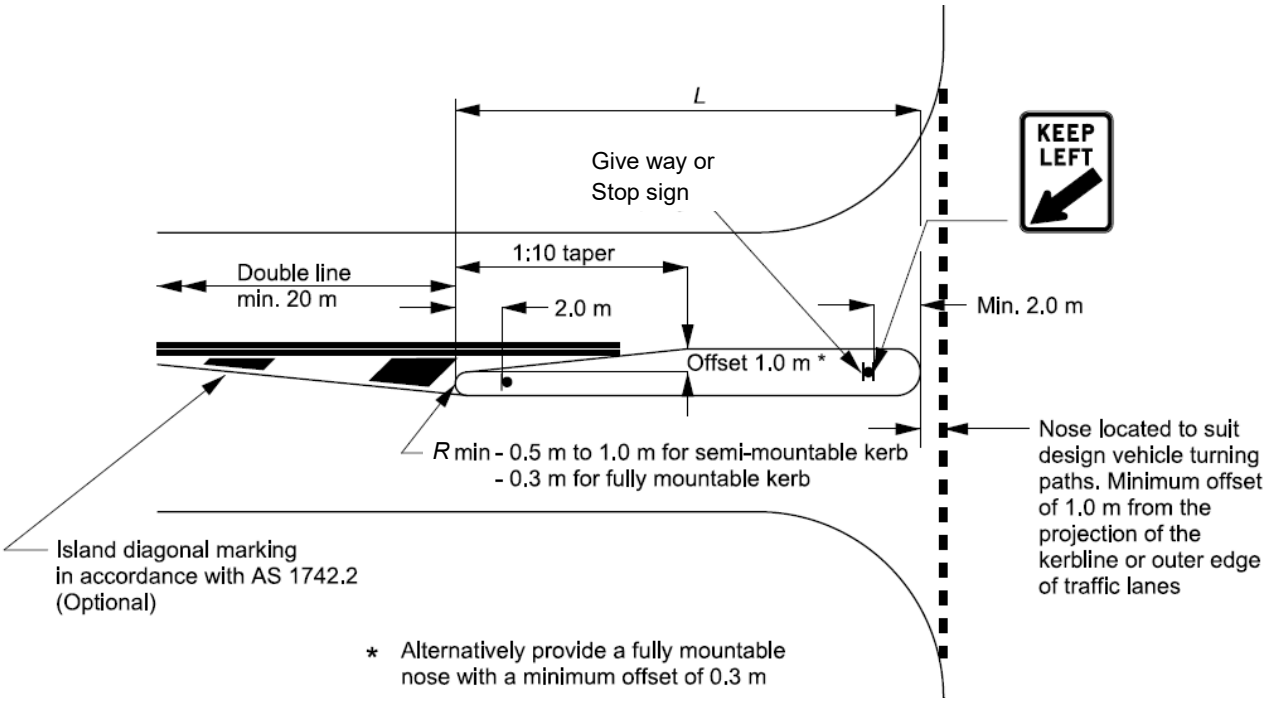
Raised median islands and medians should preferably have semi-mountable kerbs. However, kerbs are an obstruction on the road and so must be visible and should have approach geometry and delineation (e.g. nose offsets, pavement markings and raised reflective pavement markers). Consequently, there are requirements for the length, area and offset from the edge of lanes for medians.

The following design aspects should also be considered for medians and splitter islands at intersections:

- A length of painted diagonal markings and barrier line should precede the approach nose to alert drivers to the presence of the median island and to guide them past the nose.
- Any short lengths of kerbed median-island should be offset from the edge of the traffic lane.
- A median island in a side road should be set back from the prolongation of the through road (kerb or edge of traffic lane) to provide a clearance for major road vehicles and to assist heavy vehicle turning movements.

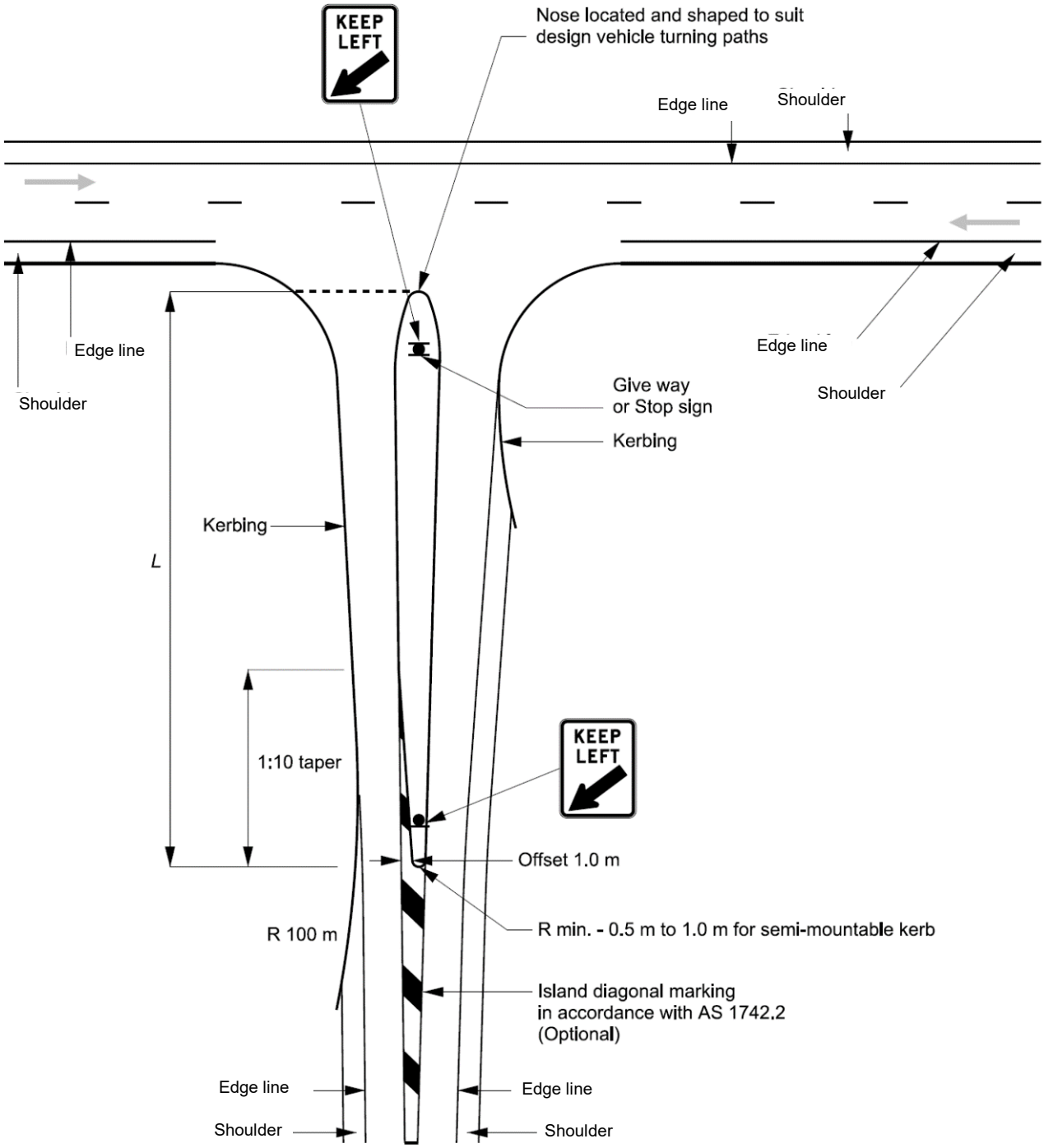
These aspects are illustrated in Figure 6.1 which shows a simple median island in a side road that is typical of urban situations and in Figure 6.2 which shows a rural example.

Figure 6.1: Example of a layout of a simple median island in an urban side road



Notes:
It is preferable that a barrier kerb is not used for traffic islands or medians.
Refer to Table 6.3 for L.
For widths of traffic islands or medians refer to Table 6.4.
Source: Based on Department of Main Roads (2006).

Figure 6.2: Example of a median island treatment at a rural unlit intersection



Notes:
Some jurisdictions may use a larger offset to the approach nose based on the speed of approaching vehicles.
For rural intersections or where the posted speed is more than 80 km/h, the minimum offset to the nose of the median island from the major roads through-lane should be the greater of the shoulder width or 1.0 m.
Refer to Table 6.3 for L.

Minimum dimensions

The minimum area required for a median island depends on the speed environment and site conditions and is influenced by other design requirements (e.g. provision for turning vehicles, sight distance) and the need to accommodate road users and road furniture.

Median islands on major roads where the median accommodates a right-turn lane treatment are usually large in order to develop the transitions for vehicles to pass to the left of the turn lane. On the other hand, median islands in local side streets may be relatively small. The geometry of median islands at rural intersections where two major roads intersect will generally be determined by the swept path of the design vehicle and the need to provide a conspicuous treatment on high-speed approaches. Consequently, islands can be large as shown in Figure 6.2.

As an initial guide, designers may adopt the minimum length of raised median islands shown in Table 6.3. However, the required length, width and shape of median islands should be derived from traffic and site characteristics. For example, where an intersection must be placed around a horizontal curve or over a vertical crest it is good practice to extend the island to the start of the horizontal curve or prior to the crest in order to provide additional warning to drivers that they are approaching an intersection. In addition, it is often necessary to increase the width and provide curved sides to match the required turning path of the design vehicle.

These widths are measured to the line of the kerb. The full median width should be maintained for a distance of at least 2.0 m each side of a pedestrian ramp crossing.

For simple applications Table 6.3 and Table 6.4 can be used to determine a minimum area for median (splitter) islands, for example:

- urban unsignalised – 8.0 m^2 for a median island in a minor side road (e.g. island with small sign 1.2 m wide x 6.5 m long, approx.)
- urban signalised – 20 m^2 for a median island to accommodate a traffic signal (based on 2.0 m wide to accommodate traffic signal x 10 m long)
- rural – 65 m^2 for a median island in a side road (based on 1.6 m wide (1.0 m off-set to a 0.6 m radius nose) x 40 m long.

The widths and lengths of rural islands are generally greater than those used in urban situations because of the higher approach speeds that require greater offsets and the need for better conspicuity.

Table 6.3: Minimum median island length (*L*)

Design speed (km/h)	Length (m)
60	10
80	20
100	40

Table 6.4: Residual median island widths at urban intersections (*W*)

Median function	Desirable minimum width (*W*) (m)(1)
Separate traffic flows and a safety barrier	3.7(3)
Shelter a small sign	1.2
Shelter signal pedestals or lighting poles	2.0
Shelter pedestrians and traffic signals	2.5
Shelter pedestrians and TGSI(2) provision in median cut-through	2.5
Shelter turning vehicles and traffic signals	6.0(4)
Shelter crossing cars	7.0(5)

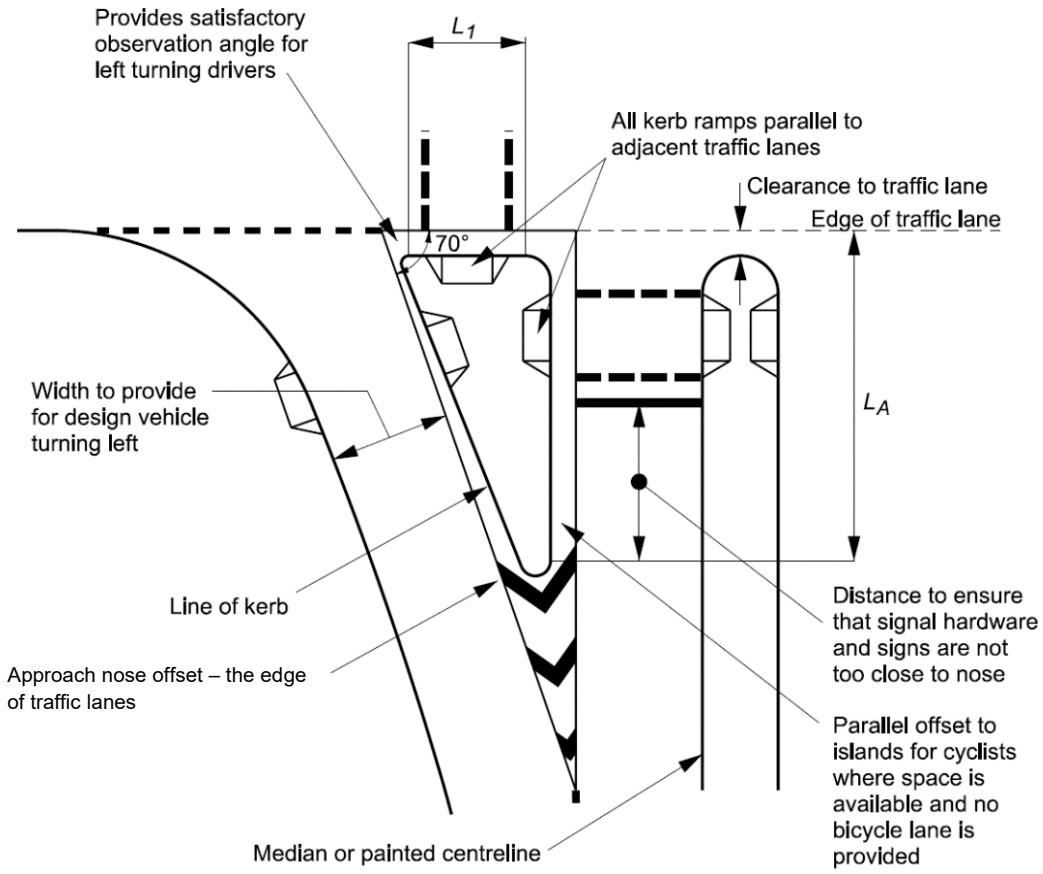
1 *Width measured to centre of edge line of traffic lane for barriers, as there is no kerb and channel provided in front of barriers. Assumes 1.5 m shoulder width and 100 mm wide edge line and concrete barrier width of 570 mm. Single slope concrete barrier and steel beam barrier (back-to-back) are marginally wider i.e. 620 mm. Refer to AS/NZS 3845.1-2015 for details.*
2 *TGSI is a tactile ground surface indicator to assist vision-impaired pedestrians, often constructed of tiles with raised dots to indicate a hazard or raised ribs to indicate a direction.*
3 *Based on a 3.5 m wide turning lane and 2.5 m residual median to accommodate pedestrians and traffic signals. It should be noted that a safety barrier is still important to consider for narrow medians, e.g. 1.5 m total width.*
4 *Based on length of car plus clearance of about 0.9 m – 1.0 m both front and back of car measured to line of kerb (length of 99.8th percentile car is 5.20 m and 85th percentile car is 4.91 m, refer to AS/NZS 2890.1-2004).*
5 *Widths for median functions other than barrier are measured to line of kerb.*

6.1.3 Raised High Entry Angle and Free-flow Left-turn Islands

High entry angle treatments and left-turn treatments with an acceleration lane (i.e. free-flow) can be used at rural and urban sites (Section 8.2 for rural treatments and Section 8.3 for urban treatments).

Figure 6.3 shows the key features of a raised high entry angle left-turn treatment for an urban intersection. The figure presents a signalised intersection approach but similar principles can be applied to an unsignalised approach.

Figure 6.3: Example of the features of a high entry angle left-turn treatment at an urban intersection (without bicycle lanes)



Note: For a description of L_A and L_1 see text below Figure 6.4.

Source: Based on Department of Main Roads (2006).

The key design features of an urban high entry angle treatment include the provision of:

- an adequate length of island (L_A) on the approach to accommodate adequate clearances, the pedestrian marked foot crossing, and all road furniture in a safe location (i.e. clearance to traffic signals; signs not in nose)
- a left-turn roadway aligned so that left-turning drivers position their vehicles at an angle that results in a safe and convenient observation angle (i.e. 70°–90°)
- adequate width to accommodate a left-turning design vehicle
- kerb ramps that are parallel to the traffic lane so that tactile ground surface indicators are also parallel to the kerb at crossing points
- a parallel offset on the approach where space is available and no bicycle lane is provided
  Where a bicycle lane is provided the width should equal the required width of the bicycle lane for the speed environment on the approach.
- a length of island (L_1) on the intersecting road that is adequate to accommodate the pedestrian marked foot crossing, corner radii and signal pedestals.

It is desirable, although not always possible because roads intersect at angles other than 90°, that pedestrian crosswalk lines should be straight for the entire crossing of the road and at right angles to the kerbline. This design is desirable to ensure that the kerb ramp and associated tactile ground surface indicators are parallel to the kerb to provide a clear direction to vision-impaired pedestrians and for ease of construction.

As referenced in the AGTM Part 6 free-flow left-turn lanes without a raised platform and/or any pedestrian priority can reduce perceived safety and level of service for pedestrians.

Dimensions of traffic islands are site specific. At signalised sites, the side of a high entry angle left-turn island on the intersection approach should be a minimum of 10.0 m long to accommodate the pedestrian crosswalk and to ensure that signal poles and signs are not located in the vulnerable area near the approach nose of the island. A minimum clearance of 0.5 m from the line of kerb to a signal lantern target board is required. The side of the island adjacent to the departure should be at least 4.0 m long to accommodate the pedestrian crosswalk.

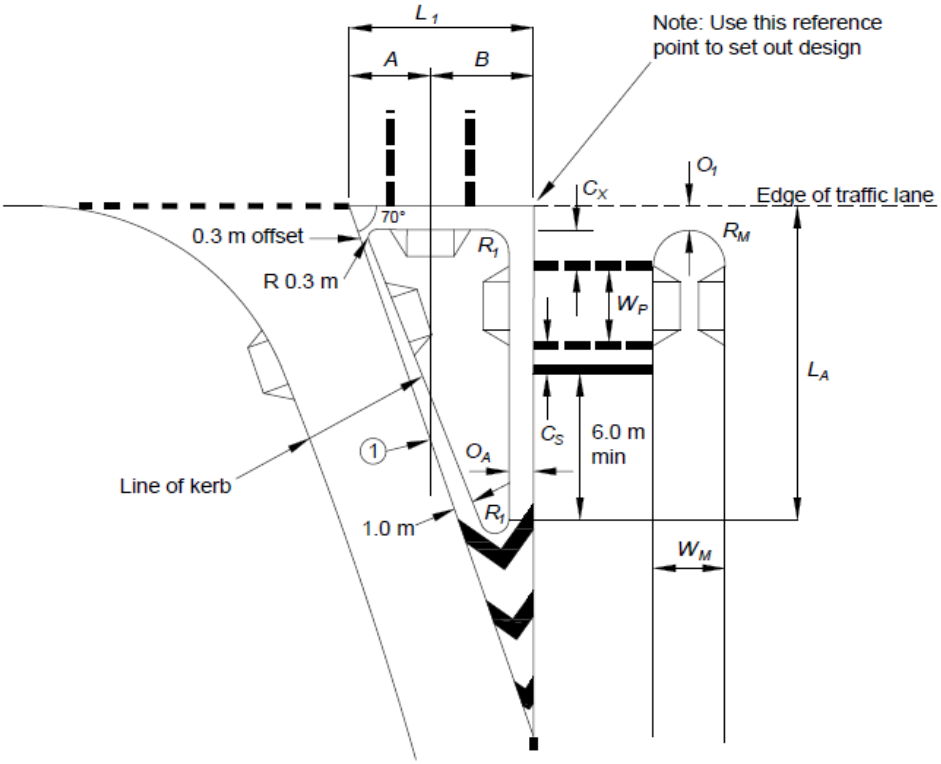
The desirable minimum area of an urban high entry angle left-turn treatment is approximately 40 m^2. Rural treatments may have a much larger area that will be a function of site conditions and the design vehicle.

Detailed information on the layout of high entry angle and free-flow left-turn island treatments is provided in Figure 6.4. Treatments with bicycle lanes are illustrated in the Austroads *Guide to Road Design Part 4: Intersections and Crossings – General* (AGRD Part 4) (Austroads 2023b).

6.1.4 Simple High Entry Angle Design Process

This section describes a process for deriving the minimum size required for a high entry angle left-turn treatment from first principles. A basic requirement is that the designer has a clear understanding of the facilities and users that have to be accommodated by the island. The example in Figure 6.4 is based on accommodating pedestrians and signal hardware and signs in positions where they are not likely to be damaged by vehicles. The parallel offset to the island on the approach to the stop line is to provide an informal space for cyclists.

Figure 6.4: Example of detailed island treatments showing offsets and crossing location



Source: Based on Department of Main Roads (2006).

The various dimensions shown in Figure 6.4 are used to determine the size and shape of the island which can be set out graphically or computed. The various dimensions are:

L_A	=	Length of the island on the intersection approach from the edge of the traffic lane on the intersecting road to the nose of the island.
L_1	=	Length of the island on the intersecting road departure from the edge of the traffic lane on the approach to the edge of the traffic lane in the left-turn slip lane.
O_1	=	Offset from the median nose to the edge of the traffic lane in the intersecting road or to the tangent point on R_1 where there is no median. Desirable offset is 0.5 m to 1.0 m but may have to be greater to accommodate a design vehicle swept path into the road.
C_X	=	Clearance between the nose of the median and the pedestrian crosswalk line.
C_S	=	Clearance between the pedestrian crosswalk line and the stop line.
W_P	=	Width of the pedestrian crosswalk, minimum 2.0 m (AS 1742.2-2009), generally 2.4 m – 3.0 m.
C_S	=	Clearance between the stop line and pedestrian crosswalk line, generally 1.2 m, minimum 0.8 m.
O_A	=	Offset from the median nose to the edge of the traffic lane on the approach road. Generally, 1.0 m for urban islands. However, some jurisdictions prefer to use 0.2 m per 10 km/h of approach speed (e.g. for approach speed of 60 km/h, O_A = 1.2 m).
R_1	=	Radius of leading nose on left-turn island. Desirable minimum 0.5 m to 1.0 m.
W_M	=	Width of median on approach.
R_M	=	Radius of median nose adjacent to intersecting road.
A	=	Distance that is a function of L_A and the angle of intersection of the left-turn slip lane.
B	=	Distance that is a function of approach nose offsets and the diameter of the approach nose (i.e. 2 x R_M).

The procedure for setting out the island is to:

- Use the intersection point of the edge lines on the approach road and intersecting road as a reference point as shown in Figure 6.4.
- Starting at the intersection point plot the distances $O_1 + C_X + W_P + C_S + 6.0$ along the edge of traffic lane on the approach. The 6.0 m is a nominal distance (one car storage length) from the stop line to the tangent point of the approach nose to ensure that signal hardware is set back from the nose which can be a vulnerable area for road furniture.
- Locate point 1 by marking the distances O_A, 2 x R_1, and the 1.0 m offset.
- Draw a line through point 1 so that the edge line of the left-turn slip lane intersects the intersecting road at 70°.
- Check to ensure that L_A is sufficient to accommodate the pedestrian crosswalk and results in an island large enough to accommodate any other necessary road furniture.

It can be seen that:

$L_A = (O_1 + C_X + W_P + C_S + 6.0)$

$L_1 = A + B = (L_A - R_1) \tan 20° + (O_A + 2 \times R_1 + 1.0)$ m.

Example:

Assuming that the approach speed is 60 km/h and that the median is 2.0 m wide to accommodate traffic signal hardware, in which case $C_X = R_M = 1.0$ m and $O_A = 0.2 \times 60/10 = 1.2$ m, and:

$$\begin{aligned} L_A &= (O_1 + C_X + W_P + C_S + 6.0) \\ &= (1.0 + 1.0 + 2.4 + 1.2 + 6.0) \\ &= 11.6 \text{ m} \end{aligned}$$

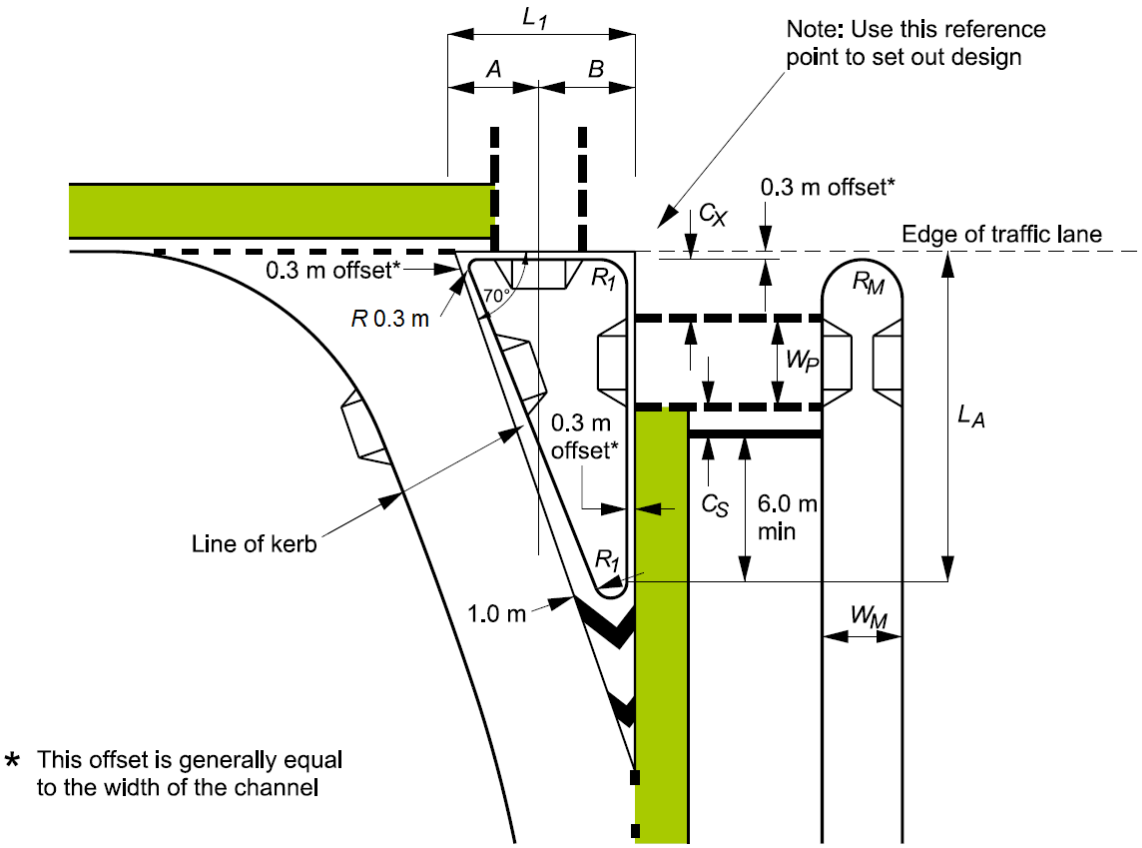
$$\begin{aligned} L_1 &= A + B = (L_A) \tan 20° + (O_A + 2 \times R_1 + 1.0) \text{ m} \\ &= 11.6 \times 0.364 + (1.2 + 2.0 + 1.0) \\ &= 4.2 + 4.2 \\ &= 8.4 \text{ m} \end{aligned}$$

Knowing the distances L_1 and L_A, and the various offsets, the raised island can be designed within the painted outline that has been defined through the process.

Figure 6.5 illustrates a high entry angle left-turn treatment with bicycle lanes on both intersecting roads at a signalised intersection. A similar procedure to that described previously can be used to set out the island and compute its size.

The bicycle lanes provided adjacent to high entry angle left-turn islands also perform the function of an offset to the leading noses of the island. This results in the sides of the island being parallel to the traffic lanes, with the advantage that kerb ramps are at right angles to the crosswalks.

Figure 6.5: Set-out details for a high entry angle CHL with a bicycle lane on the approach



Notes:
A head start treatment may be provided for cyclists as illustrated and where provided C_S should be 2.0 m.
Bicycle lane widths are provided in AGRD Part 3.
Refer to Section 6.1.4 and Figure 6.4 for definition of dimensions.

Alternative layout designs

This section presents alternative layout arrangements that are used by some road agencies. The designer should confirm with the relevant road agency prior to adopting one of these layouts.

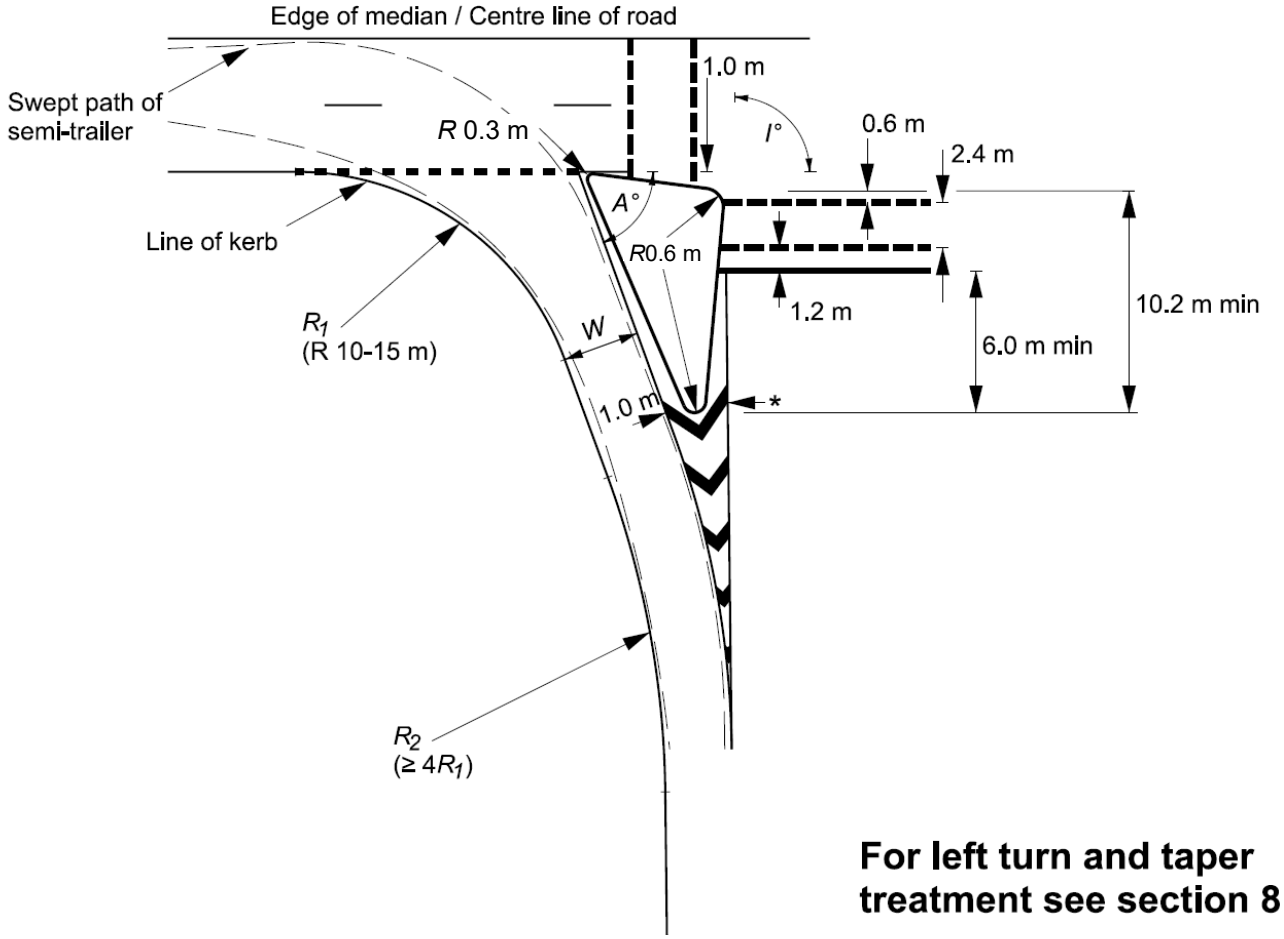
Alternative A

The method shown in Figure 6.4 and Figure 6.5 are preferred by designers in some road agencies.

Alternative A1 – High entry angle CHL turn treatment

This high entry angle treatment should be regarded as the minimum treatment, shown in Figure 6.6. Where approach speeds are higher (≥ 80 km/h) increased island length should be considered to improve island conspicuity.

Figure 6.6: High entry angle CHL turn treatment – alternative A1

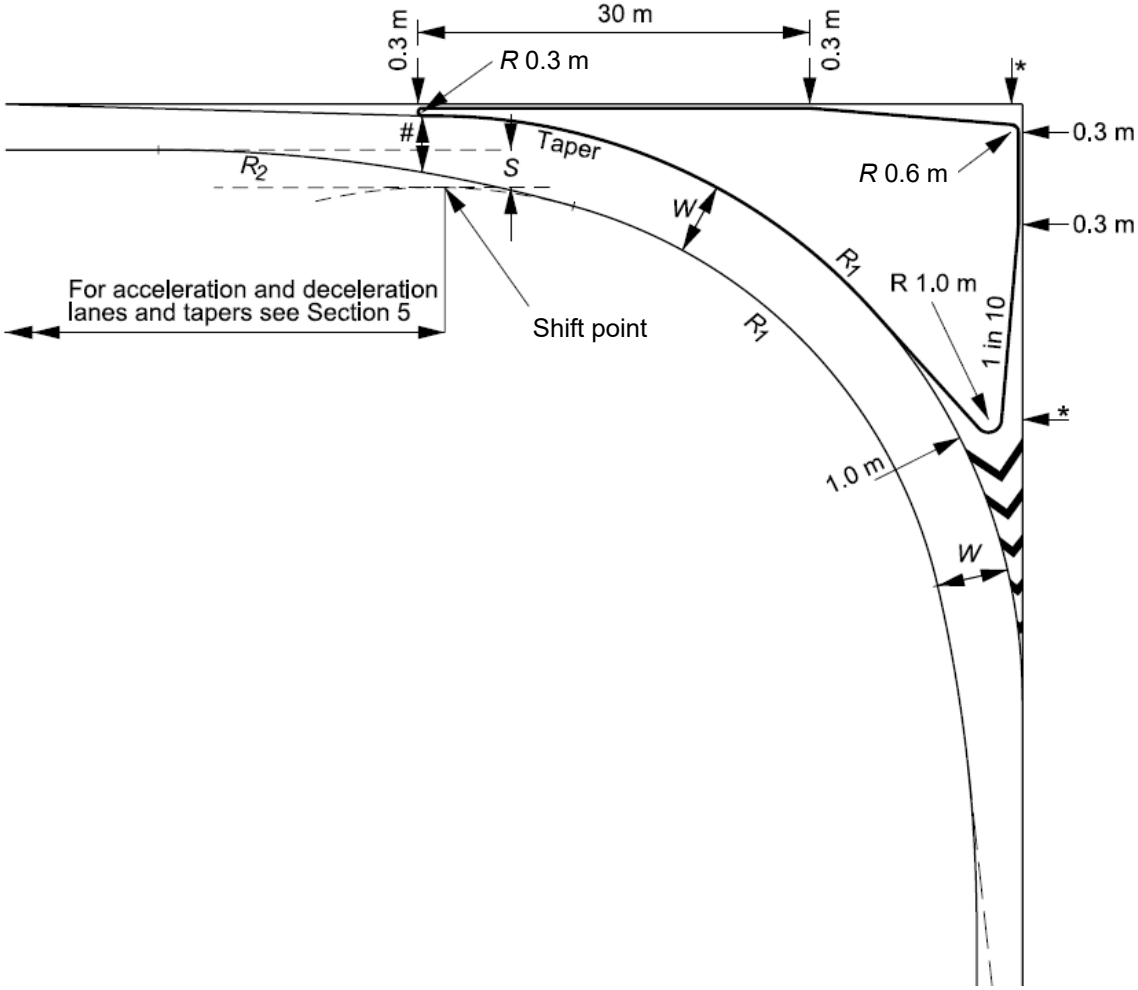


Notes:
A ° – 70º desirably to enhance stand-up angle for give way condition.
W – 4.6 m unless a greater width is required for turning trucks.
** – offset of 0.2 m per 10 km/h of approach speed.*
Kerbing to be semi-mountable. Barrier kerb should generally not be used.
The radius of the left-turn approach curve should be at least four times greater than that of the departure curve.
The only benefit in increasing the island dimensions above those shown would be in rural areas where island conspicuity would be a consideration.

Alternative A2 – CHL turn treatment with acceleration lane

The free-flow treatment provides an acceleration lane for traffic entering the intersecting road and may have an auxiliary lane on the approach to the treatment, shown in Figure 6.7.

Figure 6.7: CHL turn treatment with acceleration lane – alternative A2



Notes:

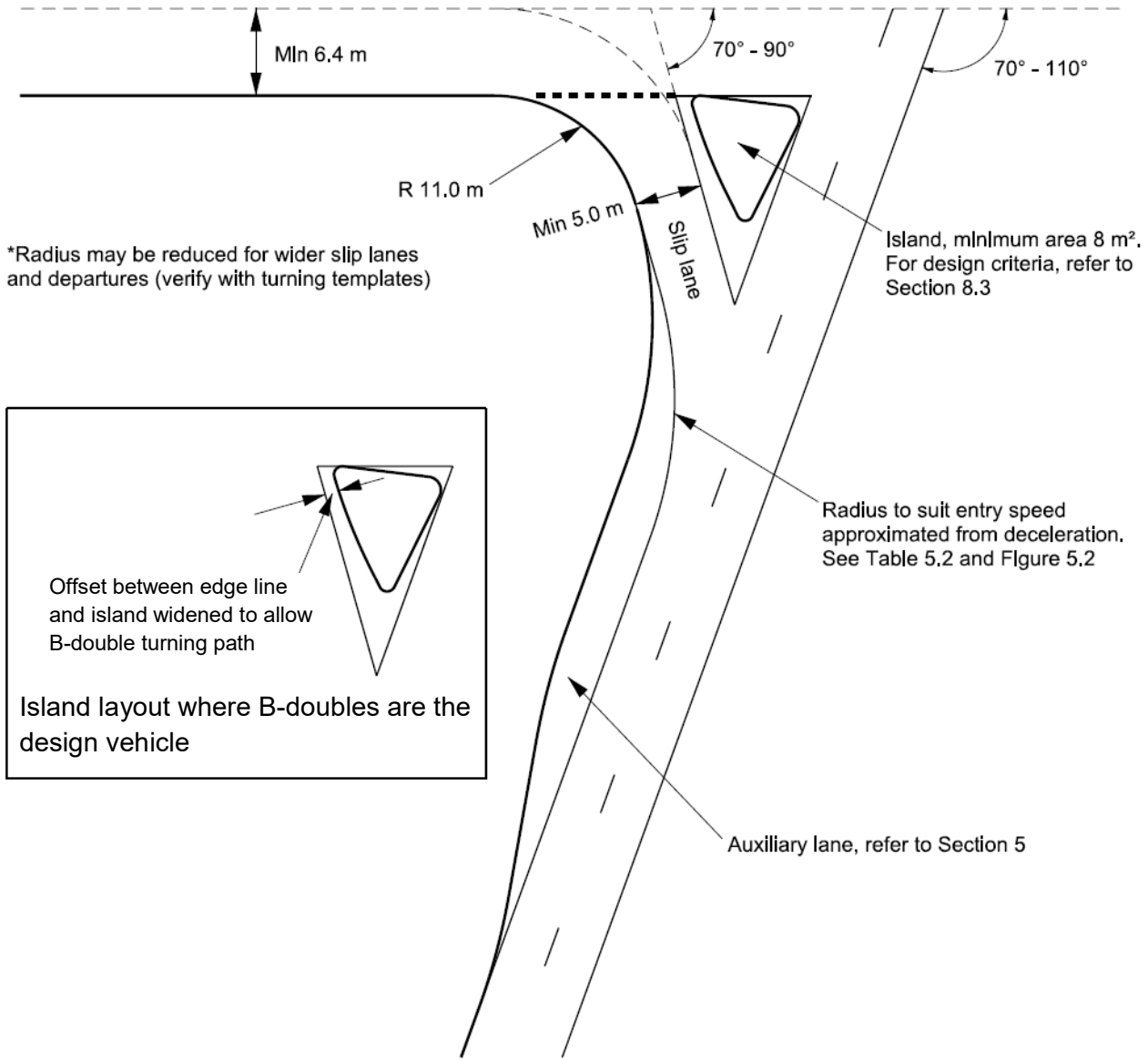
- *Radius of turn R_1 based on desirable speed of left turn on 80% (50% min.) of through road speed. A radius of 50 m should be regarded as an absolute minimum value in high-speed areas.*
- *The departure nose should always be located adjacent to the shift point.*
- *W – width of left-turn lane. Refer also to Section 6.4. Where the radius $R_1 \geq 100$ m, the left side kerb of the turning roadway may be deleted, in which case a width sufficient for a single lane flow should be provided.*
- *# – 4.6 m width of departure nose for single lane flow, or 7.0 m for two lane flow.*
- *R_1 – Radius of turning roadway.*
- *R_2 – a curve radius appropriate to achieve the width of departure nose. Typically $R_2 = 3 \times R_1$ where the road being entered has a straight alignment.*
- *$R_3 = R_1 + W$.*
- *S – shift distance between the curve of radius R_1 and the outer edge of the acceleration lane (S = 3.0 m for acceleration lane width of 3.5 m).*
- *– offset of 0.2 m per 10 km/h of approach speed.'*

Alternative B

Alternative B1 – High entry angle CHL turn treatment – urban

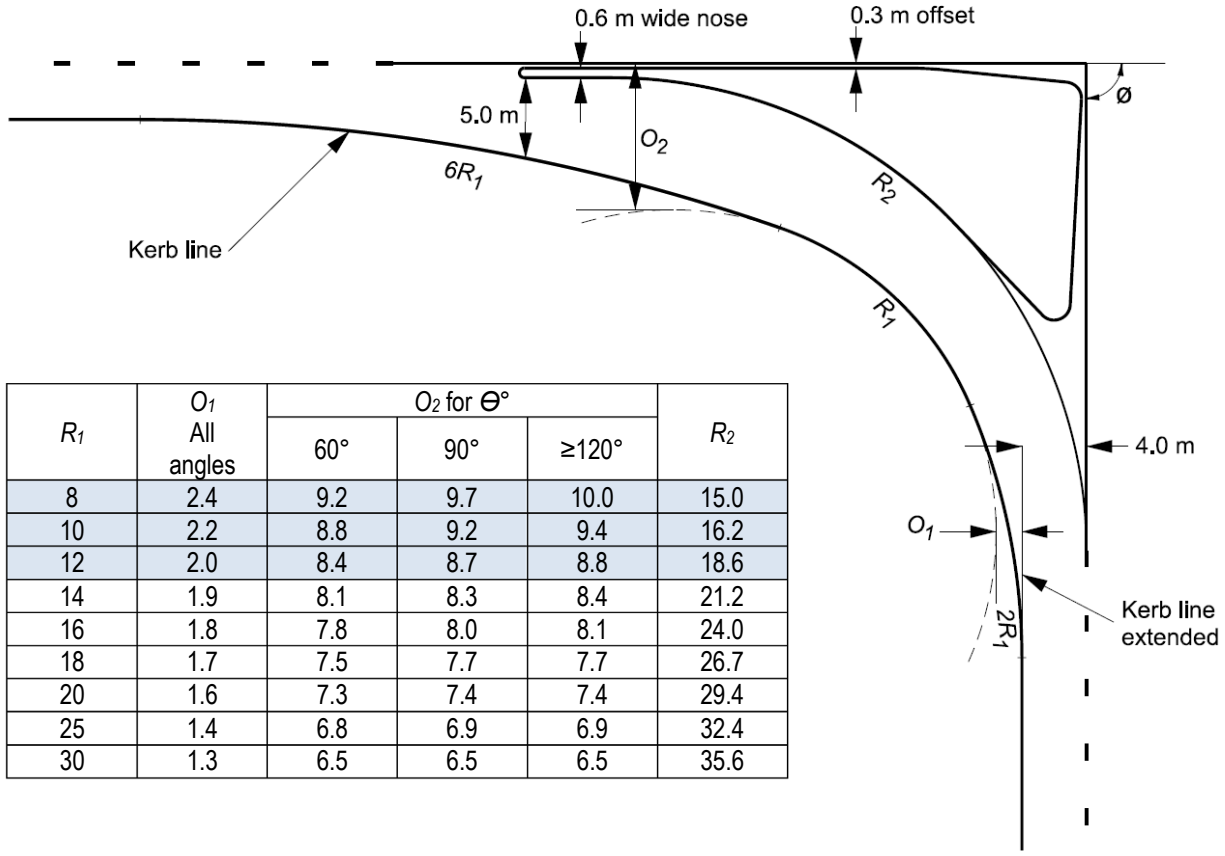
The methods in this section, shown in Figure 6.8 to Figure 6.11, may be preferred by some road agencies.

Figure 6.8: High entry angle CHL turn treatment for an urban site – alternative B1



Alternative B2 – CHL turn treatment with acceleration lane – urban

Figure 6.9: CHL turn treatment with acceleration lane for an urban site – alternative B2

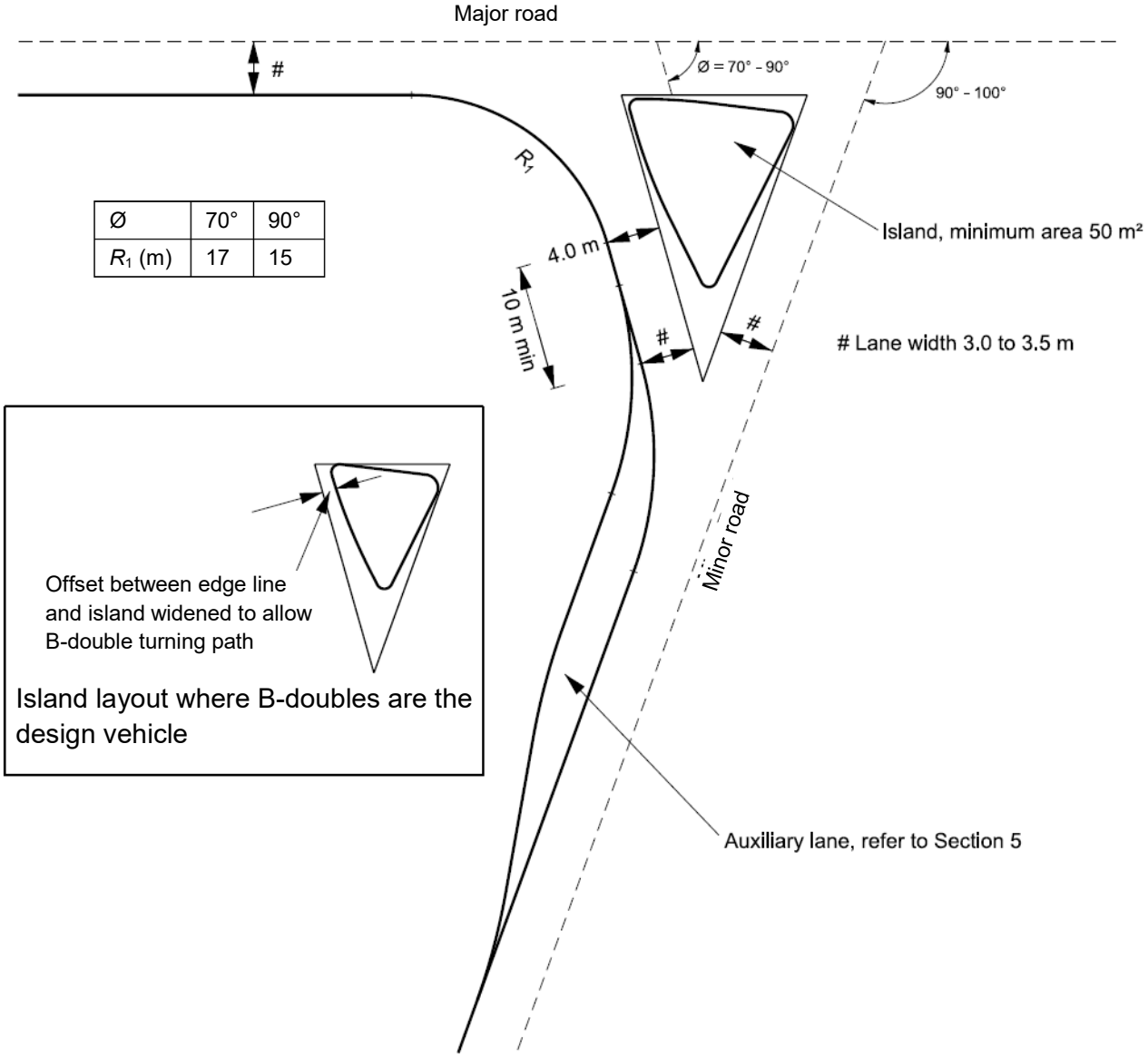


R_1	O_1 All angles	O_2 for Θ°			R_2
		60°	90°	≥120°	
8	2.4	9.2	9.7	10.0	15.0
10	2.2	8.8	9.2	9.4	16.2
12	2.0	8.4	8.7	8.8	18.6
14	1.9	8.1	8.3	8.4	21.2
16	1.8	7.8	8.0	8.1	24.0
18	1.7	7.5	7.7	7.7	26.7
20	1.6	7.3	7.4	7.4	29.4
25	1.4	6.8	6.9	6.9	32.4
30	1.3	6.5	6.5	6.5	35.6

R < 4 m – not suitable for B-double route

Alternative B3 – High entry angle CHL turn treatment – rural

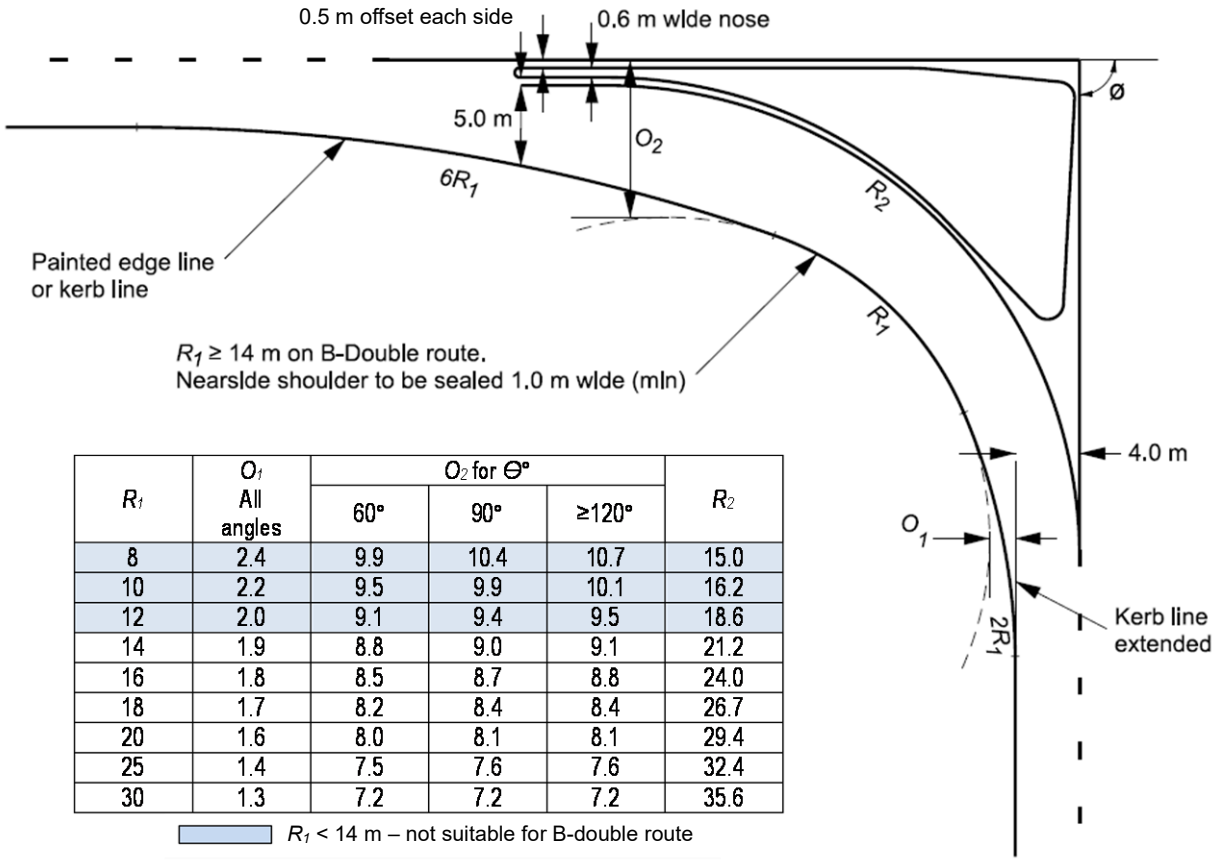
Figure 6.10: High entry angle CHL turn treatment for a rural site – alternative B3



Ø	70°	90°
R_1 (m)	17	15

Alternative B4 –CHL turn treatment with acceleration lane – rural

Figure 6.11: CHL turn treatment with acceleration lane for a rural site – alternative B4



R_1	O_1 All angles	O_2 for Θ°			R_2
		60°	90°	≥120°	
8	2.4	9.9	10.4	10.7	15.0
10	2.2	9.5	9.9	10.1	16.2
12	2.0	9.1	9.4	9.5	18.6
14	1.9	8.8	9.0	9.1	21.2
16	1.8	8.5	8.7	8.8	24.0
18	1.7	8.2	8.4	8.4	26.7
20	1.6	8.0	8.1	8.1	29.4
25	1.4	7.5	7.6	7.6	32.4
30	1.3	7.2	7.2	7.2	35.6

R_1 < 14 m – not suitable for B-double route

6.2 Painted Traffic Islands and Medians

6.2.1 General

Painted islands require less space as no clearances are required between kerbs and the edge of adjacent traffic lanes or between kerbs and the swept path of design vehicles. While pedestrians can seek refuge in a painted island it is not intended that they do so. In addition, islands that are outlined in a single line may be encroached upon by the design vehicle. Painted islands may have various sizes and shapes but are often used as splitter islands and left-turn islands.

6.2.2 Painted Medians

Painted medians and median islands can be used where space is limited, where the aim is to provide a lower cost treatment or where a raised island would have some other relative disadvantage.

Painted medians can be used:

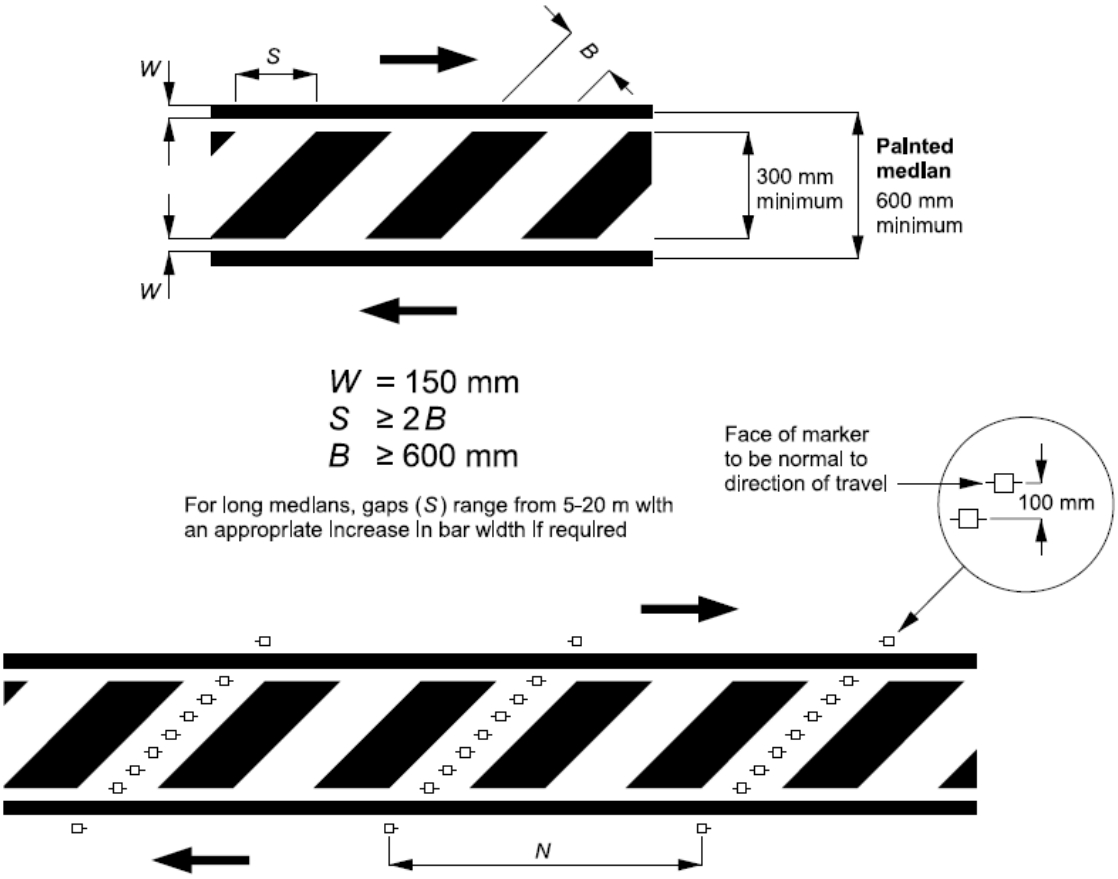
- at rural or urban sites
- on approaches to raised or depressed medians
- where an intersection is unlit
- where space is limited and the resultant width between kerbs would be too narrow for a raised median.

Figure 6.12 shows the basic width dimensions of painted median islands or painted median markings. Reference should be made to AS 1742.2-2009 *or MOTSAM: Part 1: Traffic Signs,* (NZ Transport Agency 2010a) and *Part 2: Markings* (NZ Transport Agency 2010b) for further details. The minimum width of diagonal marking that is practicable is 300 mm giving a minimum overall width of painted island of 600 mm.

The minimum length of a painted median island at an intersection should be in accordance with Table 6.3. This length excludes any transition between the splitter island width and centreline pavement marking.

The edge of a painted median island should be coincident with the edge of the adjacent traffic lane (i.e. the clearance is 0.0 m). Provided that drivers do not cross a double line they may drive over a painted traffic island in order to enter a right-turn lane which may be a consideration when designing for heavy vehicles or to improve storage for heavily trafficked right-turning movements in urban areas.

Figure 6.12: Examples of painted medians



Notes: Based on AS 1742.2-2009.
Pavement markers on the outside of an island are uni-directional raised reflective pavement markers (RRPMs).
Diagonal rows of RRPMs within the marked median should be considered as an alternative to RRPMs along the outline.
Two sets of RRPMs will not normally be required together.
N is generally 12 m for approaches to intersections.

Source: Department of Main Roads (2006).

6.2.3 Painted Left-turn Islands

Painted traffic islands should be delineated with raised reflective pavement markers (RRPMs). Further information is available in AS 1742.2-2009.

6.3 Desirable Clearances to Traffic Islands and Medians

The minimum recommended clearance from raised traffic islands to the edge of an adjacent traffic lane is provided in Table 6.5. This is the clearance from an island to a lane that is parallel to the island. Refer also to the Austroads *Guide to Road Design Part 3: Geometric Design* (AGRD Part 3) (Austroads 2016a) for further information.

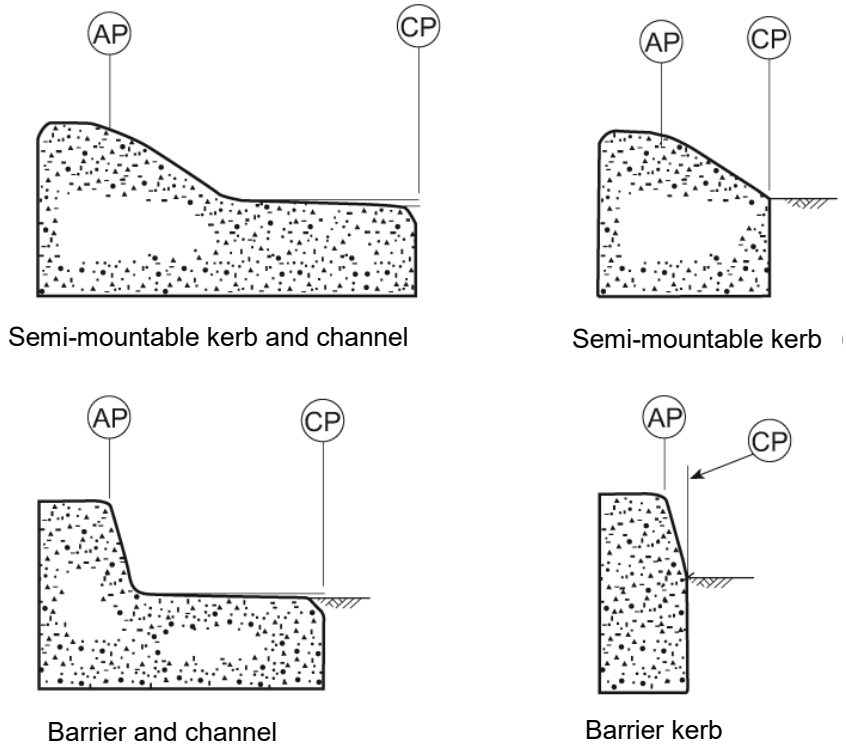
Table 6.5: Clearance between raised islands or medians and edge of traffic lane

Contest	Clearance from edge of traffic lane to CP on priority road
Lit intersection, speed zone ≤ 80 km/h	0.0 m
Lit intersection, speed zone ≥ 90 km/h (for semi-mountable kerbs)	0.5 m
Unlit intersection, speed zone ≤ 80 km/h	0.3 m
Unlit intersection, speed zone ≥ 80 km/h	0.5 m

Notes:
Offset to the edge of a traffic lane from CP on the minor road approach should be 0.0 m.
These clearances may need to be increased for small radius turns. Design vehicle turning path swept widths should be used as a check.
The clearances presume that a 3.5 m lane is provided adjacent to the island or median.

Figure 6.13 shows the definition of AP and CP. The point 'AP' is used to compute the area of the island or median.

Figure 6.13: Reference points for clearance to raised traffic islands



Notes:
The clearance point, marked CP, is used to apply clearance between the edge of the traffic lane and raised islands.
The area point marked AP is used to determine the areas of islands.

Source: Department of Main Roads (2006).

6.4 Road Width between Kerbs and between Kerb and Safety Barrier

6.4.1 General

The width provided between kerbs or between kerb and safety barrier must be sufficient to accommodate the design vehicle swept path and possibly the check vehicle (especially between barriers) plus a 0.5 m clearance from both sides of their swept path to the line of kerb or barrier.

It is desirable to provide a width no less than 5.0 m between kerbs and between kerbs and roadside barriers to allow for the passing of broken-down vehicles. It is important to apply this width where long lengths of parallel kerbing (or kerbing and barrier) apply. In some cases, the widths required to cater for the design vehicle swept paths are greater than 5.0 m.

The width of 5.0 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 require mountable or semi-mountable kerbing with sufficient offset to hardware (e.g. signs, light poles and traffic signal posts). In urban environments alternative routes to avoid 'blockages' are more likely to be available than in rural environments.

At locations where a raised traffic island is introduced it is important to provide satisfactory offsets and tapers to reduce the likelihood that drivers will shy away from the island nose (i.e. move laterally within the lane to increase clearance to it). Recommended offsets and tapers to urban and rural traffic islands and their noses are illustrated in Figure 6.1 and Figure 6.2, whilst Figure 6.4 provides details of offsets to left-turn islands.

A chevron or diagonal marking is provided in the space immediately in advance of the nose, details of which should be provided in accordance with AS1742.2 or *MOTSAM: Part 1: Traffic Signs*, (NZ Transport Agency 2010a) and *Part 2: Markings* (NZ Transport Agency 2010b).

6.5 Kerb and Channel

6.5.1 General

Kerbs may be used to separate areas used by vehicles from areas used by pedestrians or other modes of transport, or areas to be put to other uses.

Channels are used to collect and convey surface drainage and preferably should not be considered part of the traffic lanes.

The main uses of kerb and channel are to:

- collect surface drainage and to convey it to a point of discharge
- delineate the edges of carriageways
- separate carriageways from pedestrian areas
- control parking manoeuvres
- support the edge of the pavement
- reduce the width of the cut by substituting an underground drainage system in place of table drains.

It is important to ensure the kerb and/or channel aids in removing surface flows from the pavement and collection pits are located to minimise any effect on the road users. Refer also to the Austroads *Guide to Road Design Part 5A: Drainage: Road Surface, Networks, Basins and Subsurface (AGRD Part 5A)* (Austroads 2023f).

6.5.2 Kerb and Channel Types

There are four basic types of kerb and channel combinations (or kerbs):

- fully mountable
- semi-mountable
- barrier
- channels.

Examples of each type are shown in Commentary 5.

It should be noted that some jurisdictions may prefer to use kerbs that do not have a channel (or gutter). Where the pavement slopes away from a raised island or median there is no need to have an integrated channel.

7. Right-turn Treatments

Right-turn treatments are provided to improve road safety and capacity, and to minimise delay. In rural high-speed areas capacity is often not an issue whereas safety is paramount. However, intersection capacity is often an important consideration for urban intersections, in addition to safety.

The types and selection of right-turn treatments are discussed in the Austroads *Guide to Traffic Management Part 6: Intersections, Interchanges and Crossings Management* (AGTM Part 6) (Austroads 2020a).

Right-turn treatments:

- vary according to traffic requirements and site conditions
- can vary from a simple T-intersection treatment to major channelisation with traffic signal control
- can be minimal (i.e. BAR) if no superior treatment is justified by volume warrants and there are no safety problems (real or perceived) associated with right-turning vehicles
- should desirably have the right-turn lane protected by a median to guide through vehicles clear of stationary right-turn traffic
- should enable turning traffic to decelerate clear of the through traffic, where a channelised treatment is warranted
- can be defined by raised or painted median islands and pavement markings including arrows.

Lower order right-turn treatments (e.g. BAR and CHR(S)) generally should not be used with other geometric minima (e.g. steep downgrades). This is particularly true where there is reduced visibility to the turn treatment (e.g. they should not be located on smaller to moderate size crest curves). This is because turning drivers on the major road need to perceive the location of the side road and make the necessary speed reduction (or sometimes stop in the case of a BAR) in the through lane before the intersection. In these situations, a CHR turn treatment with a full-length deceleration lane should be used.

Right-turn treatments often involve the provision of auxiliary lanes, usually for deceleration and storage and occasionally for acceleration and merging (e.g. seagull treatment). Detailed information on the design of auxiliary lanes is provided in Section 5.

This section provides guidance and examples of layouts for common types of right-turn treatments. This information should enable designers to develop a shape and size for an intersection which can be used in conjunction with other information to develop a detailed layout that suits the particular site conditions. Where relevant to a site, the other information may include:

- topography
- motor traffic requirements (e.g. number of lanes)
- horizontal and vertical geometry
- design controls (e.g. watercourses, utilities)
- the need for public transport facilities
- other road user requirements (e.g. cyclists, pedestrians, trucks).

The design principles for right-turn treatments are similar for urban and rural situations, except that rural treatments generally have shoulders on the approaches and kerbing at the intersection, whereas urban treatments are likely to be (but not always) on roads that are fully kerbed.

Conversion of through lanes into turning lanes should only be used in existing extremely constrained locations because of the poor crash history associated with such treatments. They are not to be used in greenfield sites. Conversion of an approach through lane of a multilane road into an exclusive right-turn or left-turn lane should be avoided as it may cause some through traffic to change lanes at the last moment, creating a potential for crashes, 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 should be erected informing drivers of what to expect. The signs should be supplemented by pavement arrows.

7.1 Design Procedure

In applying swept path turning templates to design an intersection for opposed right turns that operate concurrently, the following procedure is suggested:

1. The plan view of each approach is plotted showing the road centreline or median kerbs, all traffic lanes and the left edge of the roads.
2. A trial location of the median noses in the side roads (i.e. roads which turning vehicles are entering) or the intersection point of the stop line/give way line and the side road centreline (where the side road has no median) is marked. The median noses in the side road should be located
   a. in rural situations in line with the back edge of the shoulder or the stop line/give way line, whichever is the greater setback from the major road edge line (or edge of pavement)
   b. in urban situations about 0.5–1.5 m from the left edge of the road to provide an offset between the nose and the traffic lane.
3. Trial turning templates are placed in the opposed right-turn lanes and adjusted until they clear the median nose or marked point (see point 2 above) by 0.5 m and comply with the required clearance between swept paths, refer to Appendix A16 of the Austroads *Guide to Road Design Part 4: Intersections and Crossings – General* (AGRD Part 4) (Austroads 2023b).
4. The location of median noses or stop lines in the major road (i.e. road from which the opposed vehicles are turning) is then plotted in relation to the design vehicle swept path.
5. The procedure is repeated for right turns from the side road.

It may be necessary to try various combinations of radii for the turning templates before the best layout is determined. During this iterative process it is necessary to ensure that the resulting pedestrian crosswalks can be accommodated close and preferably parallel to the roads.

7.2 Rural Right-turn Treatments – Undivided Roads

7.2.1 Rural Basic Right-turn Treatment (BAR)

The basic right-turn treatment (BAR) shown in Figure 7.1 is the minimum treatment for right-turn movements from a through road to side roads and local access points. 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.

Other aspects of the design are:

- On a terminating intersection leg no special provision is usually made for right turns when a BAR is used.
- 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.
- This layout should not be used where there is reduced visibility to the turn treatment. Right-turning drivers on the major road need to perceive the location of the side road and stop if necessary in the through lane before the intersection.

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. 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 that has no 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 crash rate.
- There are reasonably high right-turning volumes.
- The warrants dictate that a higher-level turn treatment is appropriate.

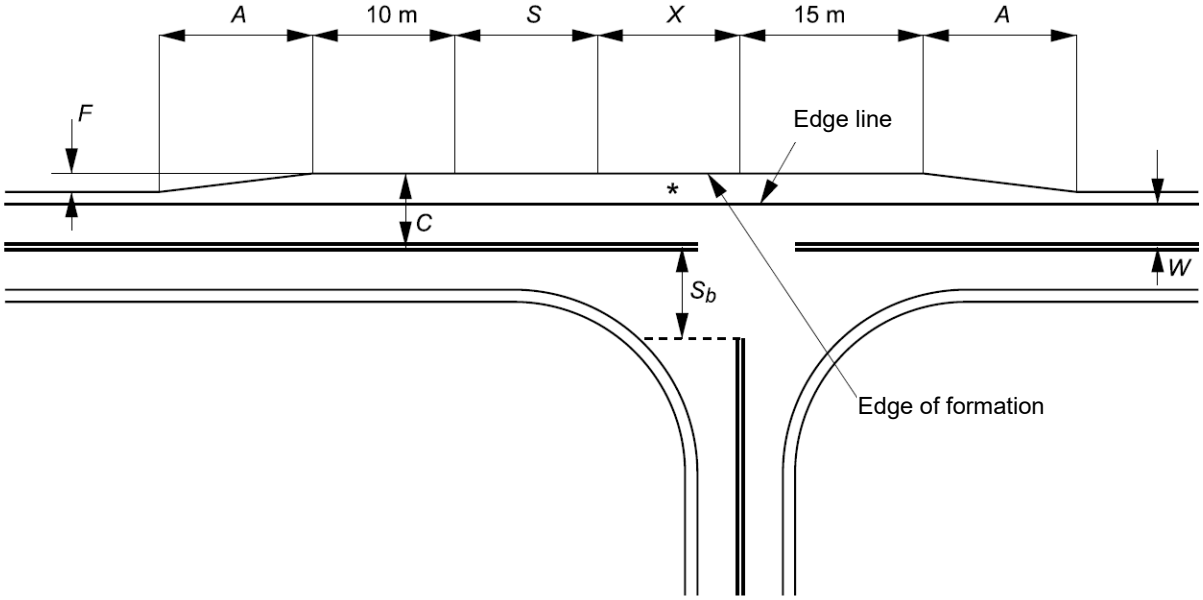
It is suggested that BAR treatments should generally have a barrier line on the major road approaches to reduce the likelihood of overtaking vehicles colliding with vehicles entering from the side road. Consideration should only be given to the use of a broken centreline in situations where overtaking opportunities are limited and the volume on the side road is very low.

The BAR turn treatment on a two-lane rural road as shown in Figure 7.1 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 crash rates, especially in high-speed areas. A more desirable treatment at such sites is a CHR(S) turn treatment discussed in Section 7.2.2.

Figure 7.1: Basic right (BAR) turn treatment on a two-lane rural road

* It is preferred that the widened shoulder is sealed, unless the shoulder can be maintained with a sound and even surface



Notes:
This treatment applies to the right turn from a major road to a minor road.
The dimensions of the treatment are:

- *W* = *Nominal through lane width (m) (including widening for curves). Width to be continuous through the intersection*
- *C* = *On straights – 6.5 m minimum*
  7.0 m 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* = $\frac{0.5VF}{3.6}$
  Increase length A on tighter curves (e.g. those with a side friction demand greater than the maximum desirable). Where the design through vehicle is larger than or equal to a 19 m semi-trailer the minimum speed used to calculate A is 80 km/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 12.5 m)*
- *X* = *Distance based on design vehicle turning path, typically 10–15 m*
- S_b = *Setback distance between the centre of the major road and the give way or stop line in the minor road. The holding line is typically placed in prolongation of the kerb line or edge line, however, it may be set back if there is a problem with the design vehicle over-running the holding line, or if it is desired to hold vehicles back some distance from the intersecting roadway (AS 1742.2 - 2009). The setback needs to be balanced such that sight distance is not negatively impacted to create a safety issue and the needs of pedestrians is met.*

Source: Department of Main Roads (2006).

7.2.2 Auxiliary Right-turn Treatment (AUR)

The auxiliary right-turn (AUR) treatment has an un-channelised right turn with an additional through lane for through traffic to bypass the vehicle waiting to turn right. As documented in AGTM Part 6 *sections 3.2.3 and 3.2.4* there are safety concerns regarding the use of the AUR treatment, particularly when compared to a channelised treatment (sections 7.2.3 and 7.2.4). As such, some jurisdictions have ceased the installation of new AUR treatments, including:

- Queensland Department of Transport and Main Roads,
- Transport for New South Wales,
- Department for Infrastructure and Transport South Australia,
- Department of Transport Victoria, and
- NZ Transport Agency.

AUR treatments shall not be used without approval from the relevant jurisdiction. Refer to Commentary 6 for further information on the design of AUR treatments.

7.2.3 Rural Channelised T-junction – Short Lane Type CHR(S)

The CHR(S) turn treatment shown in Figure 7.2 is a more desirable treatment than the BAR and AUR treatment because it provides greater protection for vehicles waiting to turn right from the centre of the road. This treatment is suitable where there are low to moderate through and turning volumes.

For higher volume sites, a full-length CHR turn treatment (Figure 7.3) is preferred. This type of intersection can only be used with linemarking. It is not to be used with raised or depressed islands as the turn lane is short and it is desirable that right-turning drivers travel over the painted chevron to exit the through traffic stream as soon as possible.

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 shoulder width of 1.0 m must be used on the through lane deviation.

The start of the right-turn taper occurs as a painted median width of 2.0 m, in lieu of the full turning lane width as per a full length CHR treatment.

The length of turn slot is based on a right-turning vehicle slowing to 80% of the design speed on the approach (i.e. a speed reduction of 20% in the through lane), prior to moving into the turn lane 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 occurs in the through lane, this treatment records far fewer rear-end crashes than do BAR turn treatments. The good safety performance occurs by removing stationary turning vehicles from the through traffic stream.

CHR(S) turn treatments should not be used where there is reduced visibility to the turn treatment.

Right-turning drivers on the major road need to perceive the location of the deceleration lane and the side road in time to make the necessary speed reduction in the through lane prior to diverging.

Table 7.1 provides the dimensions of the CHR(S) treatment for various design speeds.

Table 7.1: Dimensions of CHR(S) treatment for various design speeds

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

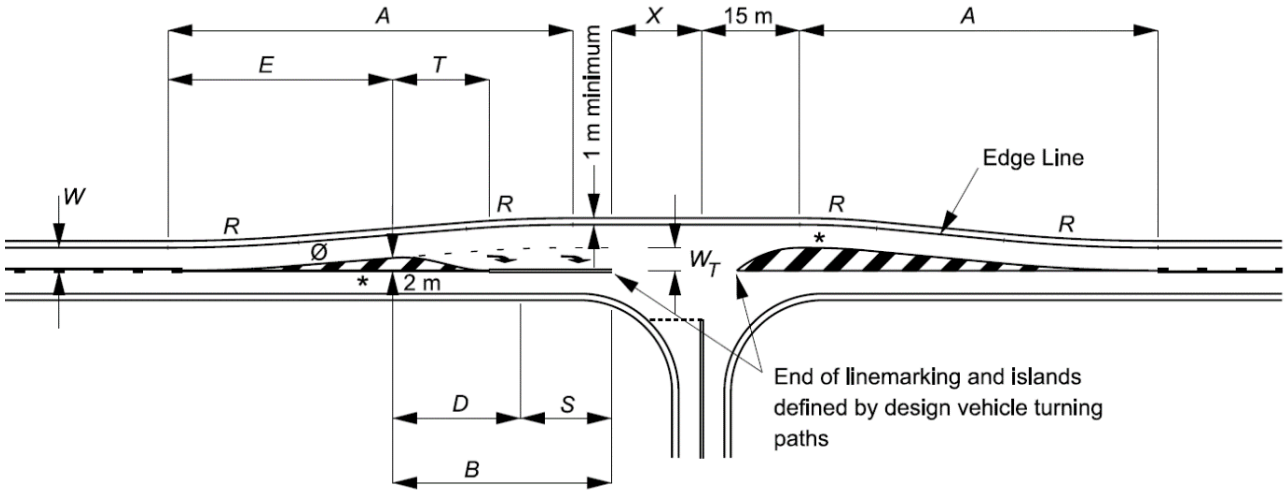
1 *Based on a diverge rate of 1 m/sec and a turn lane width of 3.0 m. Increase lateral movement length if the turn lane width > 3 m. If the through road is on a tight horizontal curve (e.g. one with a side friction demand greater than the maximum desirable), the lateral movement length should be increased 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 (Table 5.2). Adjust for grade using the 'correction to grade' factor in Table 5.3.*

3 *Based on a turn lane width of 3.0 m.*

4 *Where Type 2 road trains are required, minimum A = 60 m.*

Figure 7.2: Channelised right-turn treatment with a short turn slot [CHR(S)] two-lane rural road



Notes:
Ø – double barrier line not to be used this side of the island.
** – Islands are to comprise linemarking only, i.e. no raised or depressed medians. Diagonal rows of RRPMs within the painted islands should be used to improve the delineation of diagonal pavement marking.*
The holding line is typically placed in prolongation of the kerb line or edge line, however, it may be set back if there is a problem with the design vehicle over-running the holding line, or if it is desired to hold vehicles back some distance from the intersecting roadway (AS 1742.2 - 2009). The setback needs to be balanced such that sight distance is not negatively impacted to create a safety issue and the needs of pedestrians is met.
The dimensions of the treatment are defined below and values of A, D, R and T are shown in Table 7.1:

- W = *Nominal through lane width (m) (including 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), including widening for curves based on the design turning vehicle = 3.0 m minimum*
- A = *Length of lateral movement (Table 7.1)*
- B = *Total length of auxiliary lane including taper, diverge/deceleration and storage (m)*
- E = *Distance from start of taper to 2.0 m width (m) and is given by:*
  $$E = 2\left(\frac{A}{W_T}\right)$$
- T = *Taper length (m) and is given by:*
  $$T = \frac{0.33VW_T}{3.6}$$
- S = *Storage length to cater for one design turning vehicle (m)*
- V = *Design speed of major road approach (km/h)*
- X = *Distance based on design vehicle turning path, typically 10–15 m*

Source: Department of Main Roads (2006).

7.2.4 Rural Channelised T-junction – Full Length (CHR)

For this layout, all traffic is required to deviate and therefore the road alignment for the through movement must be designed to suit the operating speed. This deviation requires the pavement to be widened to provide a full-length right-turn lane as shown in Figure 7.3.

The minimum lengths of deceleration (*D*) for different design speeds are shown in Table 5.2 and should be based on the comfortable deceleration rate of 2.5 m/s^2. The storage length (*S*) is usually determined through the use of computer programs such as SIDRA.

Details of the departure end of the right-turn lane should be determined using turning path templates (minimum radius 15.0 m). This will depend on the width and the angle of intersection of the road that the turning vehicle is entering. There are no numerical warrants for the provision of raised medians in lieu of the painted medians, and some jurisdictions may require road lighting where raised medians are provided.

Pavement marking should be provided as shown in Figure 7.3. If the painted separation between opposing traffic flows is wider than a double white line, then the median should be delineated with diagonal markings and raised retroreflective pavement markers (Figure 6.12).

Table 7.2 provides the dimensions of the CHR treatment for various design speeds.

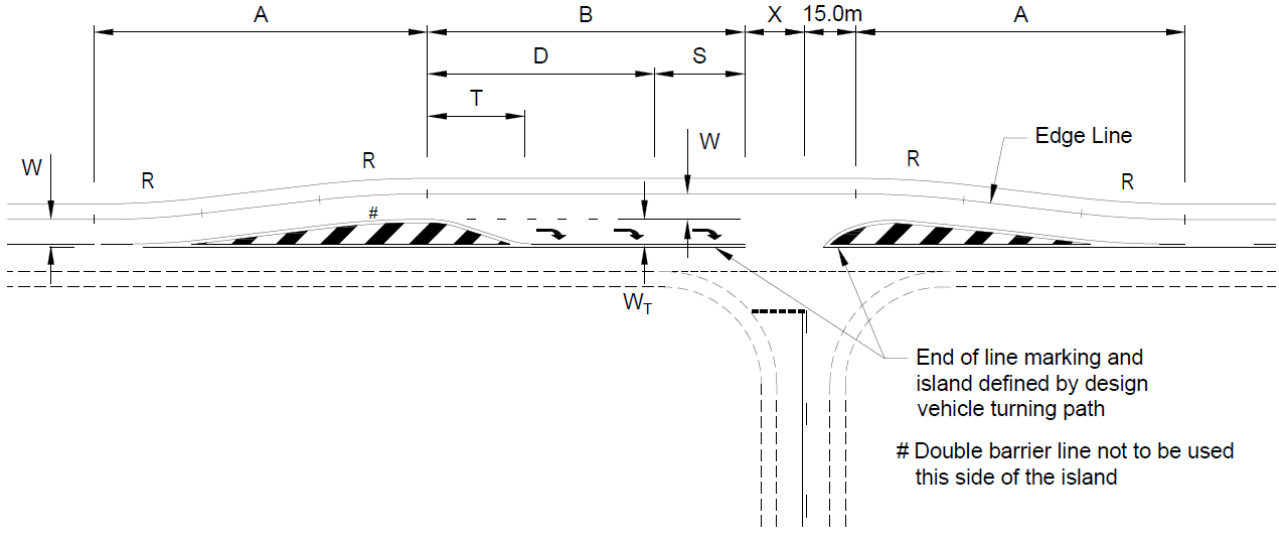
Table 7.2: Dimensions of CHR treatment for various design speeds

Design speed of major road approach (km/h)	Lateral movement length *A* (m)[1]		Desirable radius *R* (m)
	W_T = 3.5 m	W_T = 3.0 m	
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 1 m/sec. If the through road is on a tight horizontal curve (e.g. one with a side friction demand greater than the maximum desirable) increase the 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 = 60.0 m.*

Figure 7.3: Channelised right turn (CHR) on a two-lane rural road



Notes:

An alternative to the double white line on the offside edge of the right-turn slot is a 1.0 m painted median. The 1.0 m 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.

A raised concrete median on the minor road may be used with this treatment to minimise 'corner cutting', particularly for higher turning volumes.

The holding line is typically placed in prolongation of the kerb line or edge line, however, it may be set back if there is a problem with the design vehicle over-running the holding line, or if it is desired to hold vehicles back some distance from the intersecting roadway (AS 1742.2 - 2009). The setback needs to be balanced such that sight distance is not negatively impacted to create a safety issue and the needs of pedestrians is met.

The dimensions of the treatment are defined below and values of A and R are shown in Table 7.2:

- W = *Nominal through lane width (m) (including 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), including widening for curves based on the design turning vehicle. Desirable minimum = W, absolute minimum = 3.0 m*
- B = *Total length of auxiliary lane including taper, diverge/deceleration and storage (m)*
- D = *Diverge/deceleration length including taper. Adjust for grade using the 'correction to grade' factor (Section 5)*
- T = *Physical taper length (m) and is given by:*

$$T = \frac{0.33VW_T}{3.6}$$

- S = *Storage length (m) should be the greater of:*
  1. *the length of one design turning vehicle or*
  2. *(calculated car spaces – 1) x 8 m (AGTM Part 3 (Austroads 2017), or use computer program e.g. aaSIDRA)*
- V = *Design speed of major road approach (km/h)*
- X = *Distance based on design vehicle turning path, typically 10–15 m*

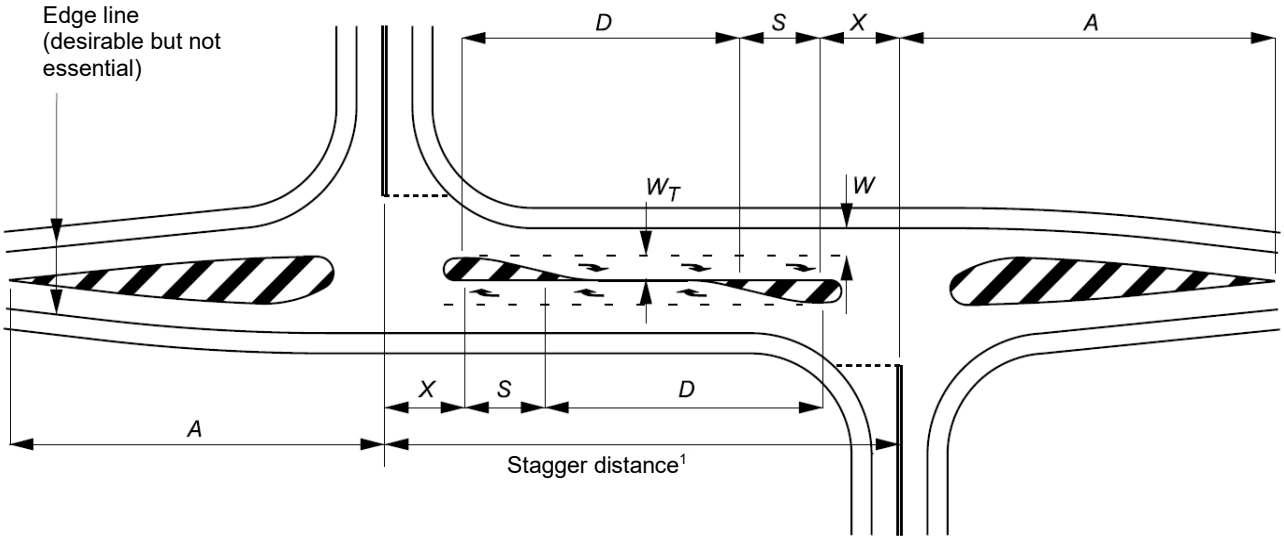
Source: Based on Department of Main Roads (2006).

7.2.5 Rural Left-right Staggered T-intersection

Overlapping right turns on a two-lane two-way road

Figure 7.4 shows a left-right staggered intersection where the right-turning lanes on the major road overlap. This layout may be suitable in situations where two existing minor roads are located relatively close to each other, or where constraints exist on both ends of the treatment which prevent it extending further along the major road. Refer to AGTM Part 6 for traffic management considerations regarding staggered T-intersections.

Figure 7.4: Rural left-right staggered T with overlapping turns on a two-lane road



1 *The stagger distance must be sufficient to ensure that deceleration and storage length can be accommodated. This should also 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 lane.*

Note: The dimensions of the treatment are:

- *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. Desirable minimum = W, absolute minimum = 3.0 m*
- *A = Design the through lane alignments in accordance with AGRD Part 3*
- *D = Diverge/deceleration length including taper (Table 5.2). Adjust for grade using the 'correction to grade' factor in Table 5.3*
- *S = Storage length (m) is the greater of:*
  1. the length of one design turning vehicle or
  2. (calculated car spaces –1) x 8 m (AGTM Part 3), or use computer program, e.g. aaSIDRA)
- *V = Design speed of major road approach (km/h)*
- *X = Distance based on design vehicle turning path, typically 10–15 m*

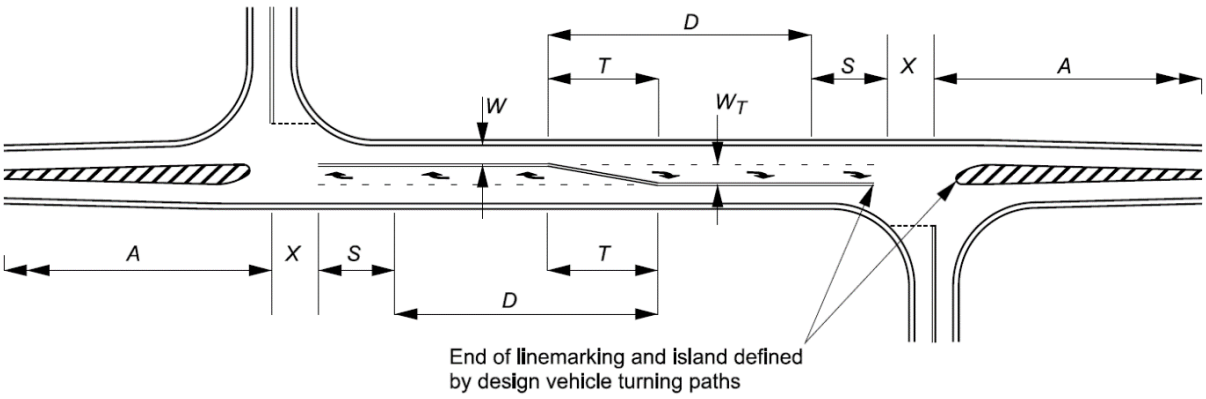
Source: Based on Department of Main Roads (2006).

The treatment requires a relatively wide pavement area and large islands at each end to shelter turning vehicles from both directions. This means that the transitions at each end required to guide through traffic may be relatively long. The islands may be raised or painted.

Back-to-back right turns on a two-lane two-way road

This treatment is shown in Figure 7.5. It results in a relatively narrow layout which requires shorter transitions than the overlapping layout. However, it requires a large stagger between intersections (e.g. about 300 m for a 100 km/h operating speed) which is often impracticable due to land acquisition and other constraints.

Figure 7.5: Rural left-right staggered T with back-to-back turns on a two-lane road



Note: The dimensions of the treatment are:

- *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. Desirable minimum = W, absolute minimum = 3.0 m*
- *A* = *Design the through lane alignments in accordance with AGRD Part 3*
- *D* = *Diverge/deceleration length including taper (Table 5.2). Adjust for grade using the 'correction to grade' factor in Table 5.3*
- *T* = *Physical taper length (m) is given by:*

$$T = \frac{0.33VW_T}{3.6}$$

- *S* = *Storage length (m) is the greater of:*
  1. *the length of one design turning vehicle or*
  2. *(calculated car spaces –1) x 8 m (AGTM Part 3), or use computer program, e.g. aaSIDRA)*
- *V* = *Design speed of major road approach (km/h)*
- *X* = *Distance based on design vehicle turning path, typically 10–15 m*

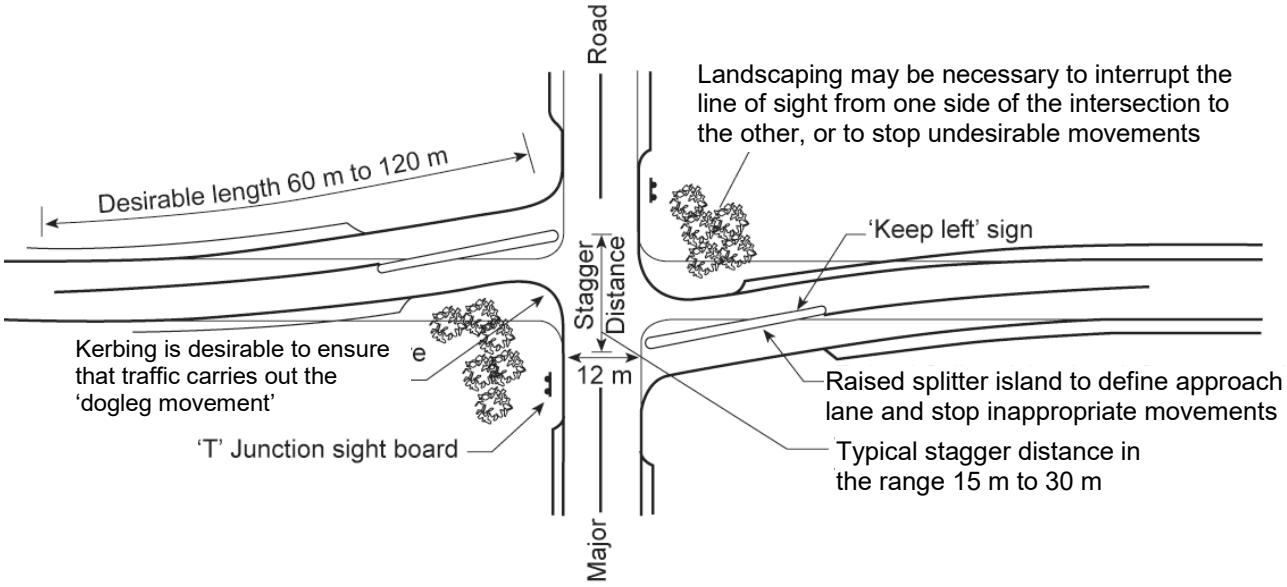
7.2.6 Rural Right-left Staggered T-intersection

Basic two-lane two-way road

This layout (Figure 7.6) should be designed to ensure that:

- the stagger distance between the minor legs is large enough to discourage drivers from 'taking a short-cut on the wrong side of the traffic islands (e.g. at least 15 m to 30 m depending on the site characteristics)
- the island treatments in the minor roads are long enough to also discourage wrong way movements
- sufficient width is provided on the major road within the intersection to enable through vehicles to pass slowly to the left of vehicles waiting to turn right (e.g. 12 m), a similar principle to the BAR treatment.

Figure 7.6: Right-left staggered T-intersection on a two-lane rural road (low turning volume)



Source: Based on Department of Main Roads (2006).

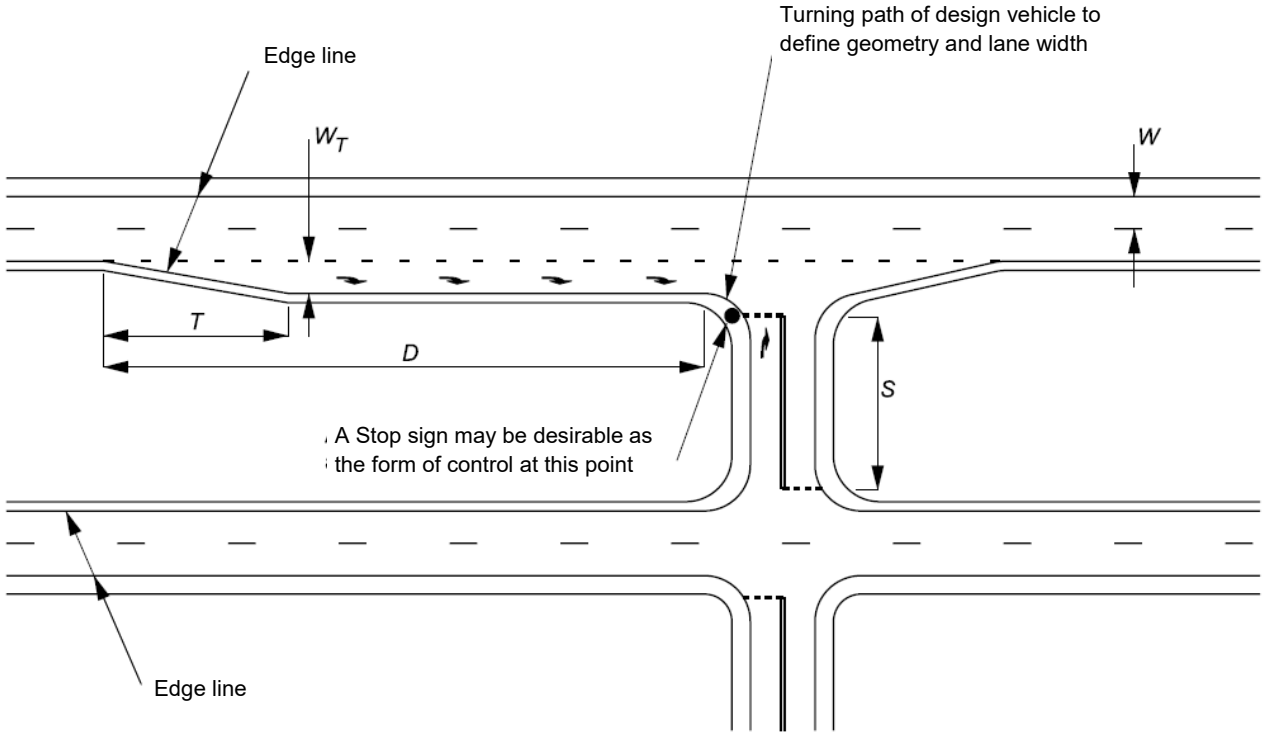
Refer to AGTM Part 6 for traffic management considerations regarding staggered T-intersections.

7.3 Rural Right-turn Treatments – Divided Roads

7.3.1 Two Stage Crossing on a Rural Road

The use of this treatment is discussed in AGTM Part 6 and illustrated in Figure 7.7. The width of the median should be sufficient to cater for the length of the turning design vehicle (denoted *S* in the figure). 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 with a clear view of approaching major road vehicles, although this may be difficult to achieve where a large heavy vehicle is used as the design vehicle. Turning paths are not to cross the centreline of the street being entered. The layout shown in Figure 7.7 may also be applicable in some urban situations.

Figure 7.7: Two stage crossing on a rural road



Notes:

- *An offset right-turn lane, refer to AGTM Part 6 is a more preferable solution for a two-stage crossing. The offset right-turn lane improves visibility for the right-turn vehicle from the side road, once stored in the median.*
- *The dimensions of the treatment are defined thus:*

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. Desirable minimum = W, absolute minimum = 3.0 m.*
D	=	*Diverge/deceleration length including taper – Table 5.2. Adjust for grade using the 'correction to grade' factor in Table 5.3.*
T	=	*Taper length (m) is given by Equation 5 being:* $T = \frac{0.33VW_T}{3.6}$
S	=	*Storage length (m) is the greater of:* *the length of one design turning vehicle or* *(calculated car spaces –1) x 8 m (Guide to Traffic Management Part 3: Traffic Studies and Analysis, (Austroads 2017)), or use computer program e.g. aaSIDRA.*
V	=	*Design speed of major road approach (km/h).*

Note: This layout is not used in New Zealand.

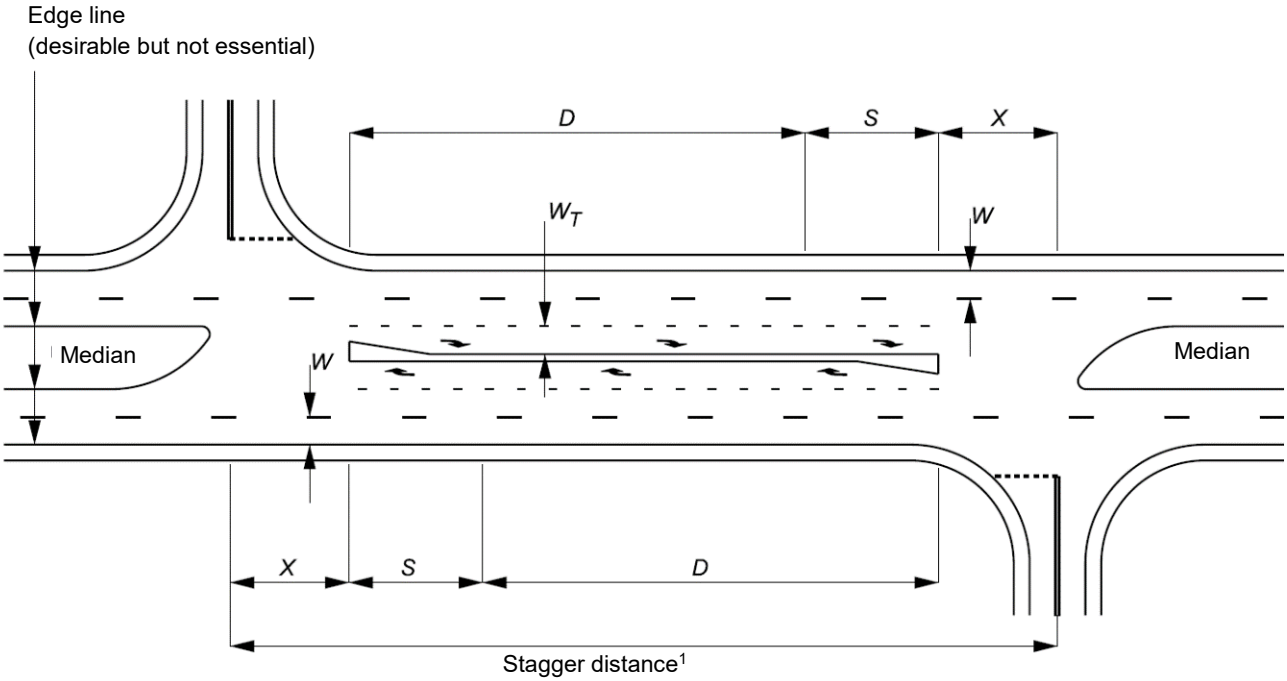
Source: Department of Main Roads (2006).

7.3.2 Left-right Staggered T – Divided Road

Overlapping right turns on a divided road

Figure 7.8 shows a diagram of a left-right staggered intersection where the right-turning lanes on the major road overlap. This layout may be suitable in situations where two minor roads are located relatively close to each other, or where constraints exist on both ends of the treatment which prevent it extending further along the major road.

Figure 7.8: Left-right staggered T-intersection on a divided rural road with overlapping right turns



1 *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.*

Note: The dimensions of the treatment are defined thus:

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. Desirable minimum = W, absolute minimum = 3 m.

D = Diverge/deceleration length including taper – Table 5.2 (adjust for grade using the 'correction to grade' factor in Table 5.3).

S = Storage length (m) is the greater of:

1. *length of one design turning vehicle or*
2. *(calculated car spaces –1) x 8 m or use computer program (e.g. aaSIDRA).*

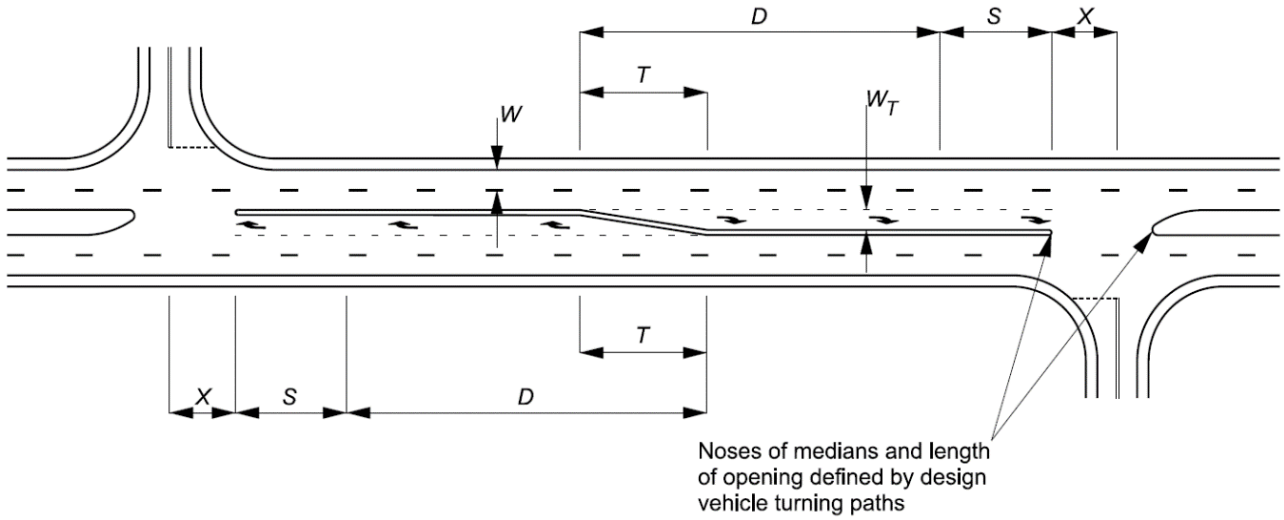
X = Distance based on design vehicle turning path, refer to Design Vehicles and Turning Path Templates Guide Austroads (2023a).

Source: Department of Main Roads (2006).

7.3.3 Back-to-back Right Turns on a Divided Road

This treatment is shown in Figure 7.9. It is suitable for use where the side roads are sufficiently staggered to enable the required deceleration and storage lengths to be accommodated, or where only a relatively narrow median can be achieved within the road reservation. However, the large stagger required between intersections (e.g. about 300 m for a 100 km/h operating speed) is often impracticable due to land acquisition and other constraints. If this treatment is impracticable and a wider median can be achieved overlapping right-turn lanes may be required.

Figure 7.9: Left-right staggered T-intersection on a divided rural road with back-to-back right-turns



Note: The dimensions of the treatment are defined thus:

- *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. Desirable minimum = W, absolute minimum = 3.0 m.*
- *D* = *Diverge/deceleration length including taper – Table 5.2. Adjust for grade using the 'correction to grade' factor in Table 5.3.*
- *T* = *Physical taper length (m) is given by Equation 5 being:* $T = \frac{0.33VW_T}{3.6}$
- *S* = *Storage length (m) is the greater of:*
  the length of one design turning vehicle or
  (calculated car spaces –1) x 8 m (Guide to Traffic Management Part 3: Traffic Studies and Analysis (Austroads 2017), or use computer program e.g. aaSIDRA.
- *V* = *Design speed of major road approach (km/h).*
- *X* = *Distance based on design vehicle turning path, refer to Design Vehicles and Turning Path Templates Guide (Austroads 2023a).*

Source: Based on Department of Main Roads (2006).

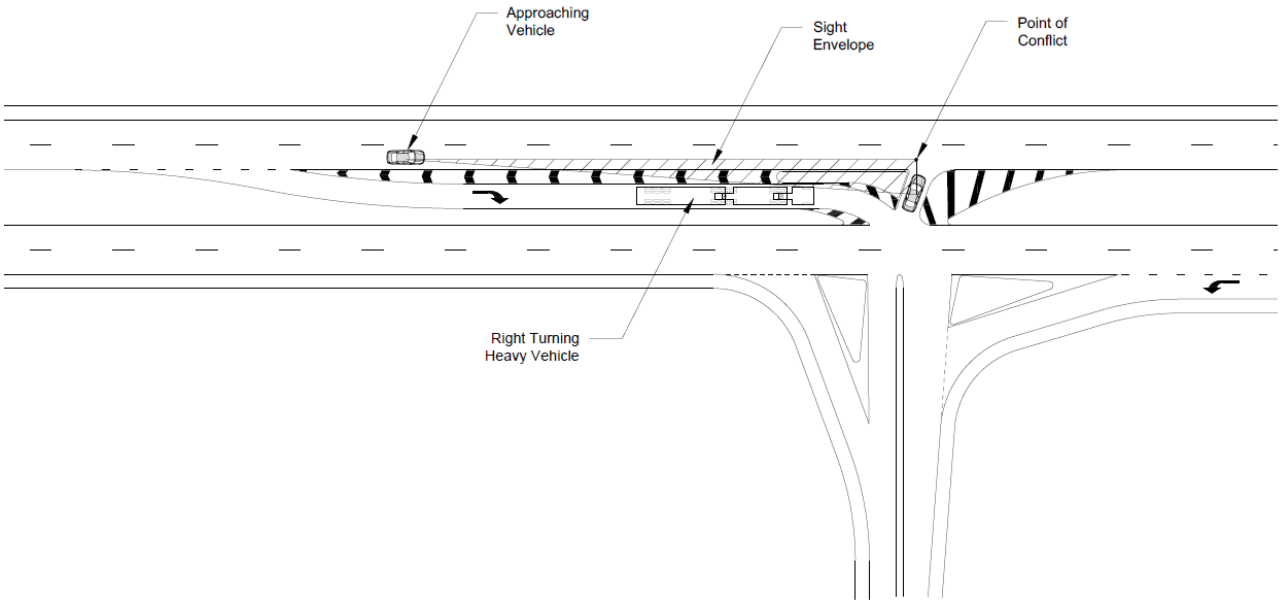
7.3.4 Offset Rural Channelised Right-turn Lane Treatment (CHR)

An offset right-turn lane maximises visibility for right turning drivers from the side road stored in the median (shown conceptually in Figure 7.10). This can be particularly beneficial if there is a significant volume of heavy vehicles turning right from the major road. Ideally, SISD should be provided between a vehicle stored in the median and approaching through traffic.

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 turning right into the side road. This requires a minimum median width for the treatment to be operationally effective.

This treatment can also be applied in urban areas.

Figure 7.10: Offset Right-turn Lane



7.3.5 Rural Seagull Treatments

Preferred rural seagull treatment

A 'seagull' is a particular form of channelised layout that is only suitable for T-intersections. The preferred seagull treatment is shown in Figure 7.11. It is used in situations where traffic analysis confirms that there is an operational advantage in right-turners from the minor road being able to accept a gap at the first carriageway and merge with major road traffic at the second carriageway.

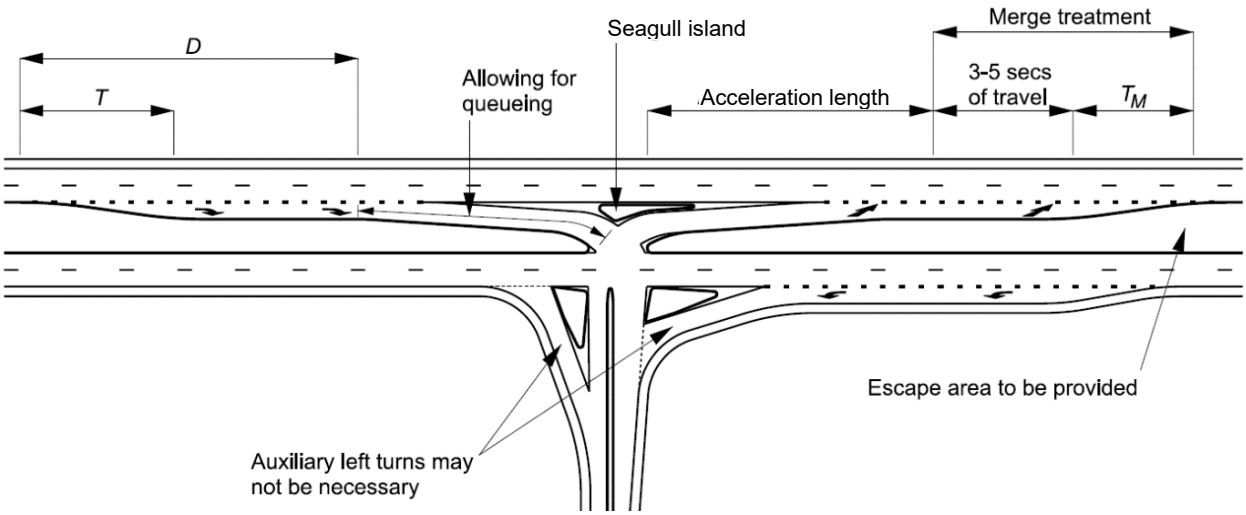
The paths for right turns into and from the minor road are channelised by a seagull island. The key features of the treatment are:

- Normal requirements for deceleration and storage apply to the turning lanes on the major road.
- The provision of an adequate acceleration lane for the merge into the second carriageway is critical to the successful operation of the treatment. The length should allow for
  - an adequate distance for acceleration
  - plus an observation time of 3 sec to 5 sec at the operating speed of the major road
  - plus a taper.

The safety of the treatment relies on the driver of the merging vehicle being able to observe vehicles in the median lane of the major road through the left-side rear-view mirror. Designers should ensure that road curvature and placement of road furniture in the seagull island do not impede the sight distance to the rear of the merging vehicle. 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. With seagull layouts, a minimum width between semi-mountable kerbs of 5.0 m is required to enable traffic to pass a disabled vehicle and thus prevent a blockage in the acceleration area. However, such widths between kerbs may encourage drivers to form two lanes and the provision of edge lines may be necessary to prevent this from happening.

Semi-mountable kerbs should be used throughout the treatment. Painted medians and islands should generally not be used.

Figure 7.11: Preferred rural seagull layout (right side merge)



Note: Due to higher speeds in the median lane and the inherently more difficult right side merge, the acceleration lane length should allow design speed to be attained plus an observation time of 3 sec – 5 sec prior to the start of the taper and where:

D = Diverge/deceleration length including taper (Table 5.2). Adjust for grade using the 'correction to grade' factor in Table 5.3

T = Physical taper length (m) is given by:

$$T = \frac{0.33VW_T}{3.6}$$

S = Storage length (m) is the greater of:

1. the length of one design turning vehicle or

2. (calculated car spaces – 1) x 8 m (AGTM Part 3 or use computer program e.g. aaSIDRA)

T_M = Merge taper length, refer to Section 5

Source: Department of Main Roads (2006).

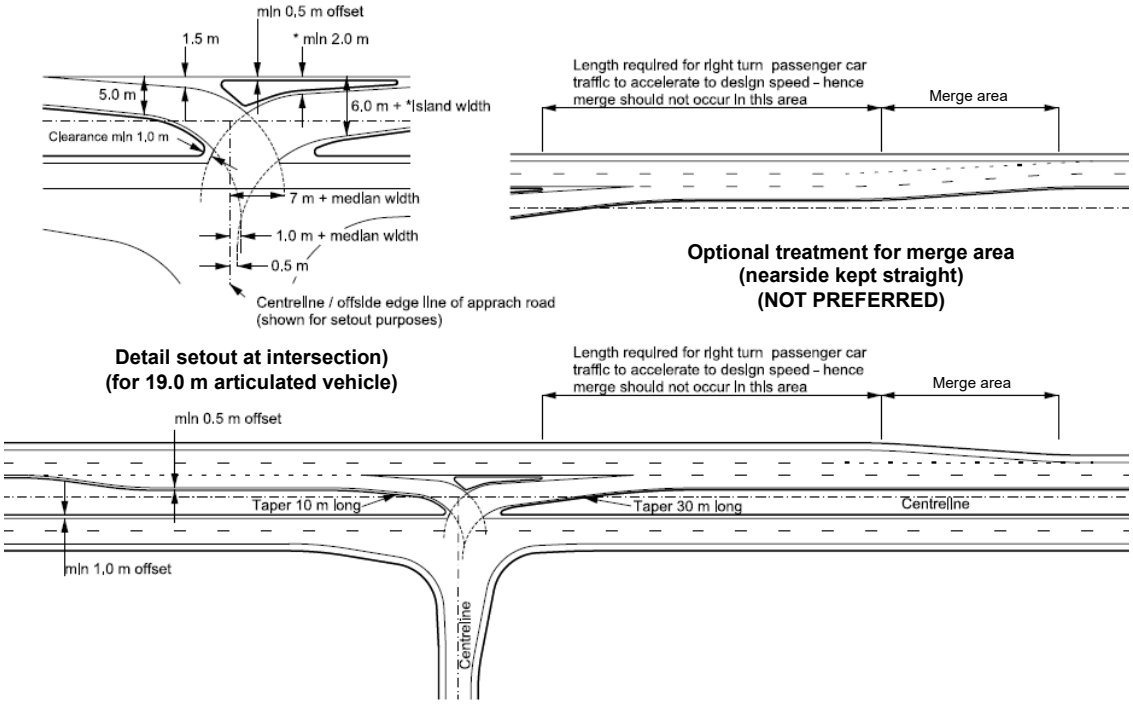
Alternative seagull layout

Where turning movements from the side road are high, or where the through traffic volumes and/or speed make gap acceptance in the merge area difficult, but a seagull treatment is still deemed to be the most appropriate treatment, then a dedicated exit lane should be provided. This alternative layout is shown in Figure 7.12.

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. 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.

It should be noted that acceleration of the joining vehicles can require a substantial length, particularly if it occurs on an upgrade (Table 5.5 and Table 5.6). Where the major road operating speed is high this may require the termination of the left lane to be 700 m or more from the intersection.

Figure 7.12: Alternative rural seagull layout (left side merge)



Source: Department of Transport and Main Roads (2006).

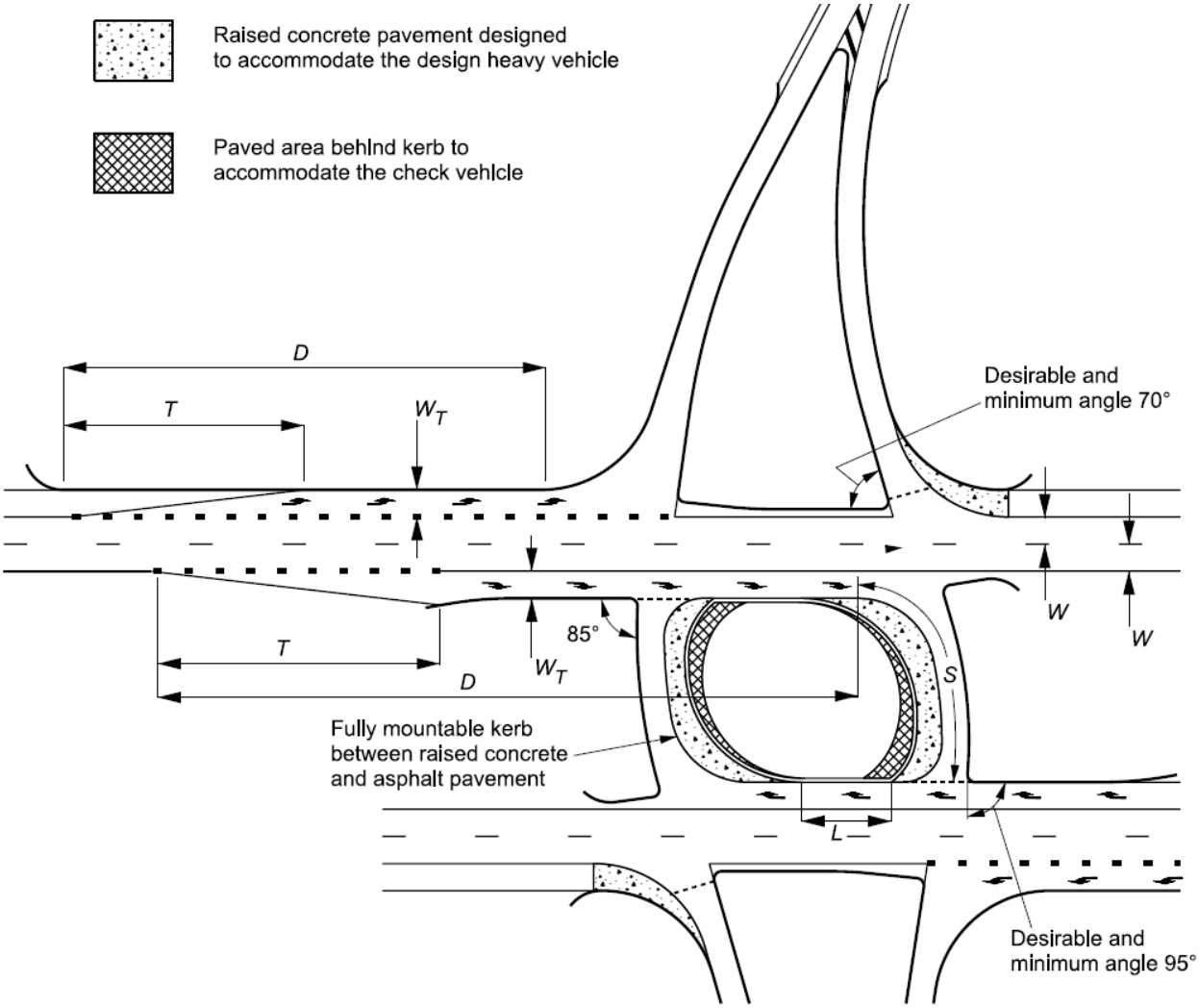
7.4 Rural Wide Median Treatment

Wide median treatments may be provided on rural divided highways to reduce the speed of traffic crossing or entering the highway. The key design characteristics of the treatment (Figure 7.13) are:

- The divided road has priority and the alignment of it is straight (or on a very large radius curve if a straight alignment is not possible).
- A large island is provided in the median together with large islands on the minor roads so that crossing traffic has to follow a deflected path which limits the approach and crossing speeds (similar to roundabouts).
- The islands in the minor roads are designed to encourage vehicles to stand-up at the holding line at 70° to comply with the required observation angle.
- The large median island is designed to accommodate the design vehicle (e.g. B-double), both turning and crossing, which requires a median width of about 30 m.
- Raised or painted (depending on jurisdictional practice) over-run areas for heavy vehicles should be provided in the central roadways to encourage crossing traffic (i.e. smaller vehicles) to adhere to the 'deflected' path.
- The sides of the median island adjacent to the major road should be straight over a substantial distance to diminish the possibility of drivers mistaking the treatment for a roundabout layout.
- Vehicles are aligned to stand-up at the 'second' carriageway (or cross it at low speed) at an angle of about 85° which is particularly important to enable drivers of trucks and vans to have clear sight lines to the left from their vehicles.
- The islands in the minor road should be set back at least 1.5 m from the edge of the major road traffic lane to allow the safe passage of cyclists (i.e. cyclists should not encroach on the traffic lane).

It is preferable that the major carriageways at a wide median treatment are at the same level as this assists drivers on the minor road approaches to comprehend the layout of the intersection. A difference in level may occur where an existing two-lane two-way road is duplicated and the new carriageway is constructed at a higher level (to comply with flood levels). In these cases, designers should ensure that the carriageways are designed to the same level through the intersection or that the design provides drivers intending to cross through the median with sufficient cues to enable them to follow the correct path and observe traffic control devices.

Figure 7.13: Rural wide median treatment



Note: The dimensions of the treatment are defined thus:

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. Desirable minimum = W, absolute minimum = 3.0 m.

D = Diverge/deceleration length including taper – Table 5.2. Adjust for grade using the 'correction to grade' factor in Table 5.3.

T = Physical taper length (m) (Equation 5) = $\frac{0.33VW_T}{3.6}$

S = Storage length (m) is the greater of:

1. the length of one design turning vehicle or

2. (calculated car spaces –1) x 8 m (AGTM Part 3), or use computer program e.g. aaSIDRA.

L = Nominal length so that drivers can perceive that the central island is not round – a measure to assist in minimising any confusion that the layout is a roundabout. The length should be at least the width of the through carriageway plus right-turning lane and preferably much longer. Suggested desirable minimum 25 m and absolute minimum 12 m.

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

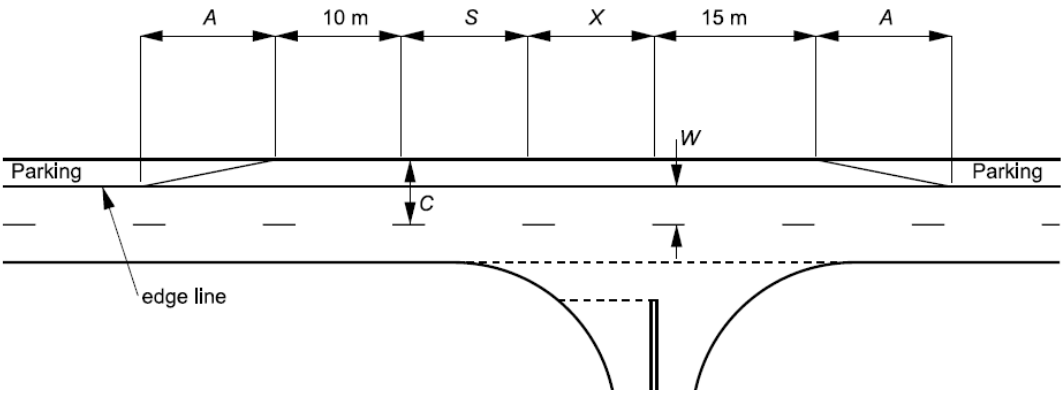
Source: Based on VicRoads (2011).

7.5 Urban Right-turn Treatments – Undivided Roads

7.5.1 Urban Basic Right-turn Treatment (BAR)

The BAR turn treatment shown in Figure 7.14 is applicable at intersections of two-lane urban roads and minor local roads where traffic volumes do not warrant a higher order treatment. It 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 6.0 m between the centreline and the edge of the pavement or kerb line while 6.5 m is the preferred minimum as it is adequate for heavy vehicles (excluding road trains) to pass right-turning vehicles.

Figure 7.14: Basic right-turn treatment (BAR) for a two-lane urban road



Notes: This diagram does not show any specific bicycle facilities. Where required bicycle facilities should be provided in accordance with this Part.
The holding line is typically placed in prolongation of the kerb line or edge line, however, it may be set back if there is a problem with the design vehicle over-running the holding line, or if it is desired to hold vehicles back some distance from the intersecting roadway (AS 1742.2 – 2009). The setback needs to be balanced such that sight distance is not negatively impacted to create a safety issue and the needs of pedestrians is met.
The dimensions of the treatment are defined thus:

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

C = On straights – 6.0 m minimum
– 6.5 m minimum for 19 m semi-trailers and B-doubles
– 7.0 m minimum for Type 1 and 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).

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

Increase length A on tighter curves (e.g. where side friction demand is greater than the maximum desirable). Where the design through vehicle is larger than or equal to a 19 m semi-trailer, the minimum speed used to calculate A is 80 km/h.

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

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

X = Distance based on design vehicle turning path, refer to Design Vehicles and Turning Path Templates Guide (Austroads 2023a).

Source: Department of Main Roads (2006).

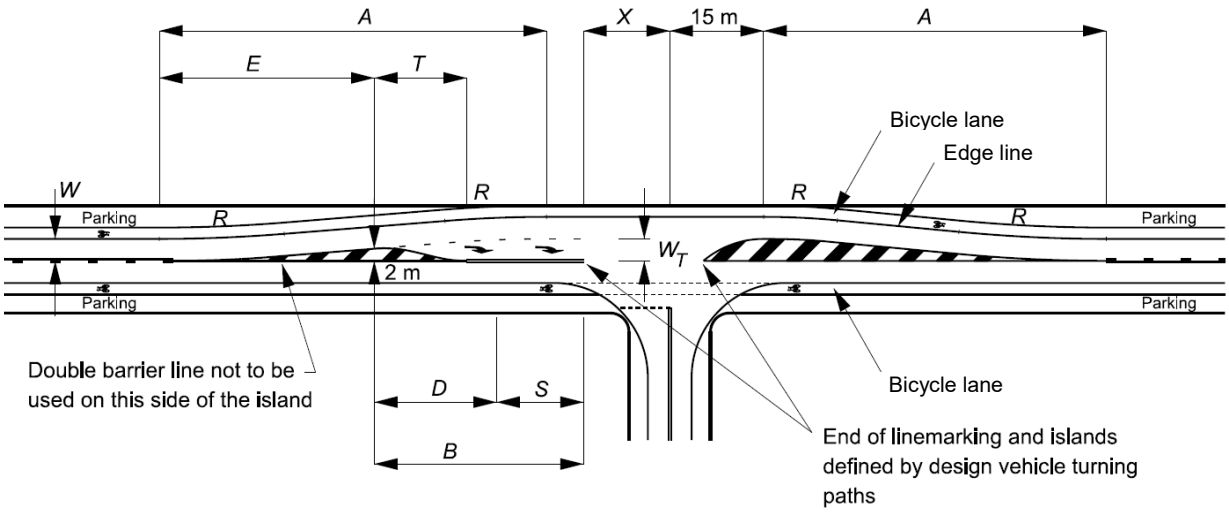
A turning radius in accordance with the Austroads *Design Vehicles and Turning Path Templates Guide* (AP-G34-23) (Austroads 2023a) should be used and the design turning vehicle's swept path should be used to determine the length of approach and departure widening for the site geometrics (i.e. angle of intersection, width of carriageways). No lane lines or right-turn arrows should be marked on the pavement for a BAR turn treatment. The provision of bicycle lanes should be considered, refer to AGTM Part 6 and AGRD Part 4 for further information.

This layout should not be used where there is reduced visibility to the turn treatment. Right-turning drivers on the major road need to perceive the location of the side road and stop if necessary in the through lane before the intersection.

7.5.2 Urban Channelised T-junction – Short Lane Type CHR(S)

A more desirable treatment than the BAR is a CHR(S) turn treatment as shown in Figure 7.15. CHR(S) turn treatments should not be used where there is reduced visibility to the turn treatment. Right-turning drivers on the major road need to perceive the location of the deceleration lane and the side road in time to make the necessary speed reduction in the through lane prior to diverging.

Figure 7.15: Urban CHR(S) treatment on a two-lane road



Notes:
This layout includes bicycle lanes. The layout may be used without providing bicycle lanes if insufficient space is available to accommodate them. If midblock bicycle lanes exist in the latter case, alternative treatments must be provided for cyclists to negotiate the intersection (e.g. a separate bicycle path on the nature strip).
Islands are to comprise linemarking only (i.e. no raised or depressed medians). Diagonal rows of raised reflective pavement markers within the painted island may be used to improve the delineation of the diagonal pavement markings.
The holding line is typically placed in prolongation of the kerb line or edge line, however, it may be set back if there is a problem with the design vehicle over-running the holding line, or if it is desired to hold vehicles back some distance from the intersecting roadway (AS 1742.2 - 2009). The setback needs to be balanced such that sight distance is not negatively impacted to create a safety issue and the needs of pedestrians is met.
The dimensions of the treatment are defined thus:

- 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) = 3.0 m minimum.*
- B = *Total length of auxiliary lane including taper, diverge/deceleration and storage (m).*
- E = *Distance from start of taper to 2.0 m width (m) = $(A/W_T) \times 2$.*
- T = *Physical taper length (m) given by Equation 5 being:* $T = \frac{0.33VW_T}{3.6}$
- R = *Radius (m).*
- S = *Storage length to cater for one design turning vehicle (m).*
- V = *Design speed of major road approach (km/h).*
- X = *Distance based on design vehicle turning path, refer to Design Vehicles and Turning Path Templates Guide (Austroads 2023a).*

Note: Values of A, D, R and T are shown in Table 7.3.

Source: Department of Main Roads (2006).

Table 7.3: Dimensions of urban CHR(S) treatment for various design speeds

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)
50	40(3)	15	110	15
60	50(3)	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 1 m/sec and a turn lane width of 3.0 m. Increase lateral movement length if turn lane width > 3 m. If the through road is on a tight curve (e.g. where side friction demand is greater than the maximum desirable), 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 (Table 5.2). Adjust for grade using the 'correction to grade' factor in Table 5.3. Based on a turn lane width of 3.0 m.*

3 *Where Type 2 road trains use the major road the minimum A = 60 m.*

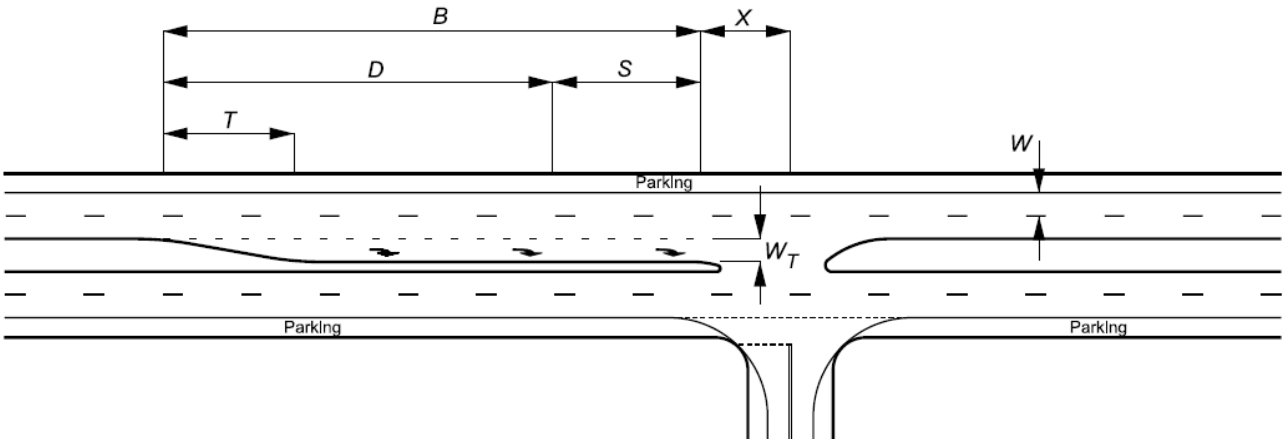
Source: Department of Main Roads (2006).

7.6 Urban Right-turn Treatments – Divided Roads

7.6.1 Channelised Right-turn (CHR) on Divided Urban Roads

Right-turn treatments on urban divided roads involve the provision of indented turn lanes as shown in Figure 7.16. The auxiliary lane should be of an appropriate length (Section 5 and Table 5.2). It is important to design the median noses to assist turning movements of the design vehicle and to encourage the drivers of vehicles turning right from the minor road to stand at the appropriate angle in the median (i.e. not at a low observation angle).

Figure 7.16: Urban channelised right-turn treatment (CHR)



Notes:

- *This diagram does not show any specific bicycle facilities. Where required bicycle facilities should be provided in accordance with this Part.*
- *A raised concrete median in the minor road may be used with this treatment.*
- *The holding line is typically placed in prolongation of the kerb line or edge line, however, it may be set back if there is a problem with the design vehicle over-running the holding line, or if it is desired to hold vehicles back some distance from the intersecting roadway (AS 1742.2 - 2009). The setback needs to be balanced such that sight distance is not negatively impacted to create a safety issue and the needs of pedestrians is met.*
- *The dimensions of the treatment are defined thus:*

  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. Desirable minimum = W, absolute minimum = 3.0 m.

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

  D = Diverge/deceleration length including taper – Table 5.2 (adjust for grade using the 'correction to grade' factor in Table 5.3).

  T = Physical taper length (m) given by Equation 5 being: $T = \frac{0.33VW_T}{3.6}$

  S = Storage length (m) is the greater of:

  1. length of one design turning vehicle

  2. (calculated car spaces –1) x 8 m or use computer program (e.g. aaSIDRA).

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

  X = Distance based on design vehicle turning path, refer to Design Vehicles and Turning Path Templates Guide (Austroads 2023a).

Source: Department of Main Roads (2006).

7.6.2 Two Stage Crossings on Divided Urban Roads

The use and design of basic median openings at minor road intersections on urban divided roads is discussed in AGTM Part 6. The basic treatment is a form of two stage crossing, the geometry of which is determined by the median width. Where very wide medians exist on urban roads, intersection treatments with minor roads may take the form of the rural two stage crossing discussed in AGTM Part 6.

7.6.3 Seagull Treatments on Divided Urban Roads

The use of urban seagull treatments and the geometric design principles are discussed in AGTM Part 6.

7.7 Right-turn Bans at Signalised Intersections

Consideration should be given to banning a right turn where:

- a right-turn lane cannot be provided, and the right-turning traffic would cause a safety and/or a capacity problem
- sight distance is poor and cannot be corrected, and other options such as erecting advance signs are not satisfactory.

If the right turn can be banned, several options may be considered as described in AGTM Part 6.

7.8 Right-turn Lanes for Cyclists

Right-turn lanes for cyclists are rarely used and should generally not be provided for cyclists at right-turn treatments on arterial roads or busy traffic routes because of the difficulty and crash risk for cyclists moving from the left of an intersection to the centre of the road in order to utilise such treatments. Conditions for the use of cyclist right-turn lanes and illustrations of their use at a signalised intersection are provided in Section 10.

8. Left-turn Treatments

This section contains guidance on the design of left-turn treatments. The types of left-turn treatments, and volume warrants and safety considerations for their selection are discussed in Austroads *Guide to Traffic Management Part 6: Intersections, Interchanges and Crossings Management* (AGTM Part 6) (Austroads 2020a).

The type of left-turn treatment used may depend on the:

- 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, property boundaries, service utilities and other structures
- provision for turning cyclists
- pedestrian movements.

These factors combine to determine the type of treatment to be adopted in any given situation. The types of treatments provided for left turns are similar for high- and low-speed environments and for different volume demands; however, the geometry is usually more generous for higher speeds or higher volumes and an auxiliary lane may be required AGTM Part 6.

Lower-order left-turn treatments (e.g. BAL and AUL(S)) generally should not be used with other geometric minima (e.g. steep downgrades). This is particularly true where there is reduced visibility to the turn treatment (e.g. they should not be located on smaller to moderate size crest curves). This is because drivers on the major road need to perceive the location of the side road and make the necessary speed reduction in the through lane before the intersection. In these situations, an AUL or CHL turn treatment with a full-length deceleration lane should be used.

8.1 General

8.1.1 Return Radius

A basic element of a left-turn treatment is the return radius or radii. The return is the circular arc or arcs joining the kerb or edge lines of intersecting roads. The return radius or radii are determined by:

- consideration of the factors listed above
- the design vehicle (Austroads *Guide to Road Design Part 4: Intersections and Crossings – General* (AGRD Part 4) (Austroads 2023b))
- the width and direction of the approach and departure to the turn
- whether a single radius return or a return with compound radii is appropriate.

A single radius return is commonly used, with compound radii returns being used to a greater extent in urban areas, and three centred curves being used in free-flow left-turn treatments to better represent the tracking of heavy vehicles. Compound radii returns are generally only used to avoid obstructions (Appendix A). The radius, or radii, of a return should be designed using the appropriate design vehicle turning path.

The types of left-turn treatments that may be provided are:

- single radius left turn without a left-turn island
- single radius left turn with left-turn island
- multiple radii left turn
- high entry angle left turn
- free-flow left turn.

Having selected a return radius, designers should ensure that the turning treatment:

- enables adequate sight lines and sight distance to approaching vehicles
- minimises areas of conflict
- has considered possible impact angles
- keeps crossing distances for pedestrians to a minimum.

As the return radius increases to accommodate larger design vehicles it becomes increasingly difficult to satisfy observation angle requirements for drivers to be able to see and safely give way to approaching vehicles (Section 3.2.3). At some sites this requirement may determine the type of treatment.

8.1.2 Intersection Angle

Where kerb lines intersect in the range 70° – 110° (e.g. at an existing intersection), and the design vehicle is a 19.0 m semi-trailer, a left-turn island of sufficient size cannot be provided in the residual area between intersecting kerb lines using a single, 11.0 m radius return. If the return radius is made larger, the observation angle requirement cannot be met. Accordingly, such a layout must be controlled by traffic signals. However, there are some exceptions as follows:

- legs entering on the outside of a horizontal curve
- entering traffic only needs to sight turning traffic (Figure 3.4).

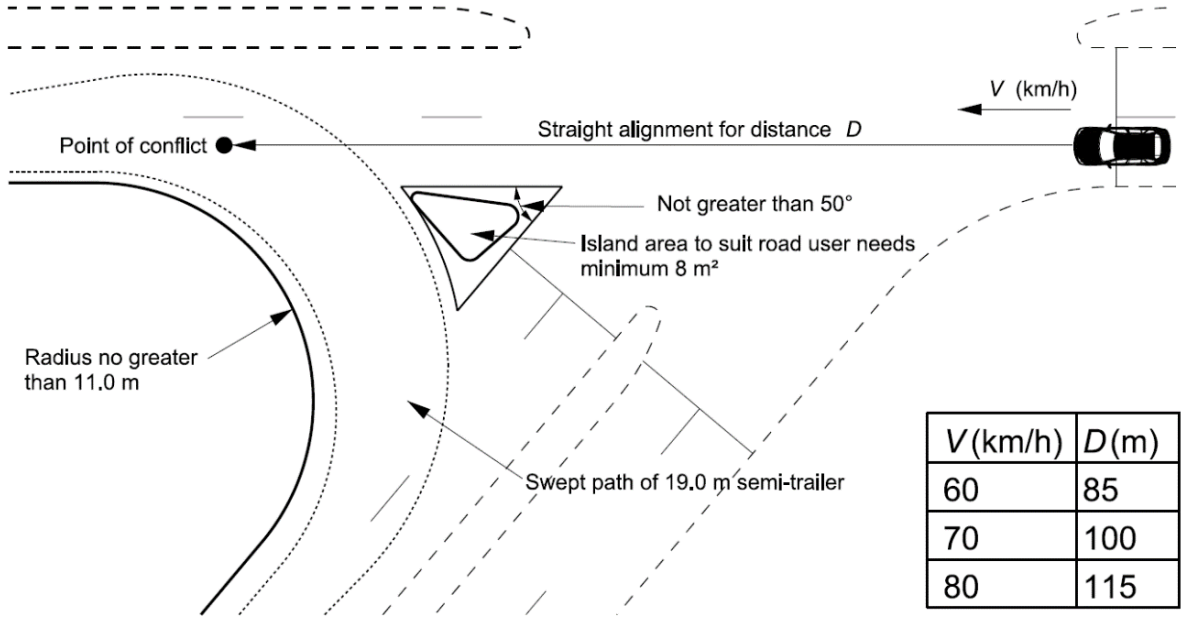
The return radius should be reduced when:

- entering on the inside of a horizontal curve
- the design vehicle is 12.5 m long (or less).

When the intersection angle is 130° (or more), a left-turn island can be provided for a 19.0 m semi-trailer as long as the return radius is not greater than 11.0 m. This is illustrated in Figure 8.1 and can result in a relatively small island. Observation angles for the above conditions should be checked with criteria shown in Figure 3.6.

The left-turn island will assist in reducing pedestrian crossing widths and areas of uncontrolled pavement. If a marked foot crossing is provided in the left-turn slip lane, approach sight distance (ASD) should be provided for the approach to the crossing and the pavement markings should be clearly visible over the entire length of ASD for drivers approaching the crossing.

Figure 8.1: Single radius turn-only combinations which meet island size and observation angle



V (km/h)	D (m)
60	85
70	100
80	115

Notes:
Values in tabulation are the lengths of straight alignment required for the corresponding 85th percentile approach speed, measured from the conflict point.
Refer to Figure 3.6 for observation angle requirements.

Source: Department of Main Roads (2006).

8.2 Rural Left-turn Treatments

8.2.1 Rural Basic Left-turn Treatment (BAL)

Figure 8.2 shows a minimum treatment for use in a rural situation (i.e. high-speed environment) which provides tapers leading into and out of the left-turn treatment in order to cater for the swept path of a large design vehicle. While the case illustrated in Figure 8.2 has a large articulated vehicle as the design vehicle, the size and detailed shape of the treatment will vary in accordance with the appropriate design vehicle for a particular site. Where the design vehicle is relatively small (e.g. car or service vehicle) a single radius turn may be adopted without tapers, provided that the design vehicle can perform the left turn without encroaching into an opposing traffic lane.

It should be appreciated that the:

- layout is the minimum form of treatment that should be applied to a rural left-turn
- layout has a single radius return, auxiliary lanes are not provided, and the layout is not channelised
- appropriate design vehicle should be used
- design vehicle should not cross the centreline of the side road
- angle of the intersection may be in the range 70° – 110°
- distance S_b is the setback distance between the centre of the major road and the give-way or stop line in the minor road
- layout should not be used where there is reduced visibility to the turn treatment. Left-turning drivers on the major road need to perceive the location of the side road in time to make the necessary speed reduction in the through lane prior to moving onto the widened shoulder.

New or reconstructed intersections must be designed to this requirement even if intersection legs have to be re-aligned. An exception is intersections that cater mainly for smaller vehicles (i.e. cars, vans and service vehicles), and only occasionally have to cater for a heavy vehicle. In these circumstances it may be considered appropriate to design a simple radius without tapers that is able to cater for the smaller design vehicles.

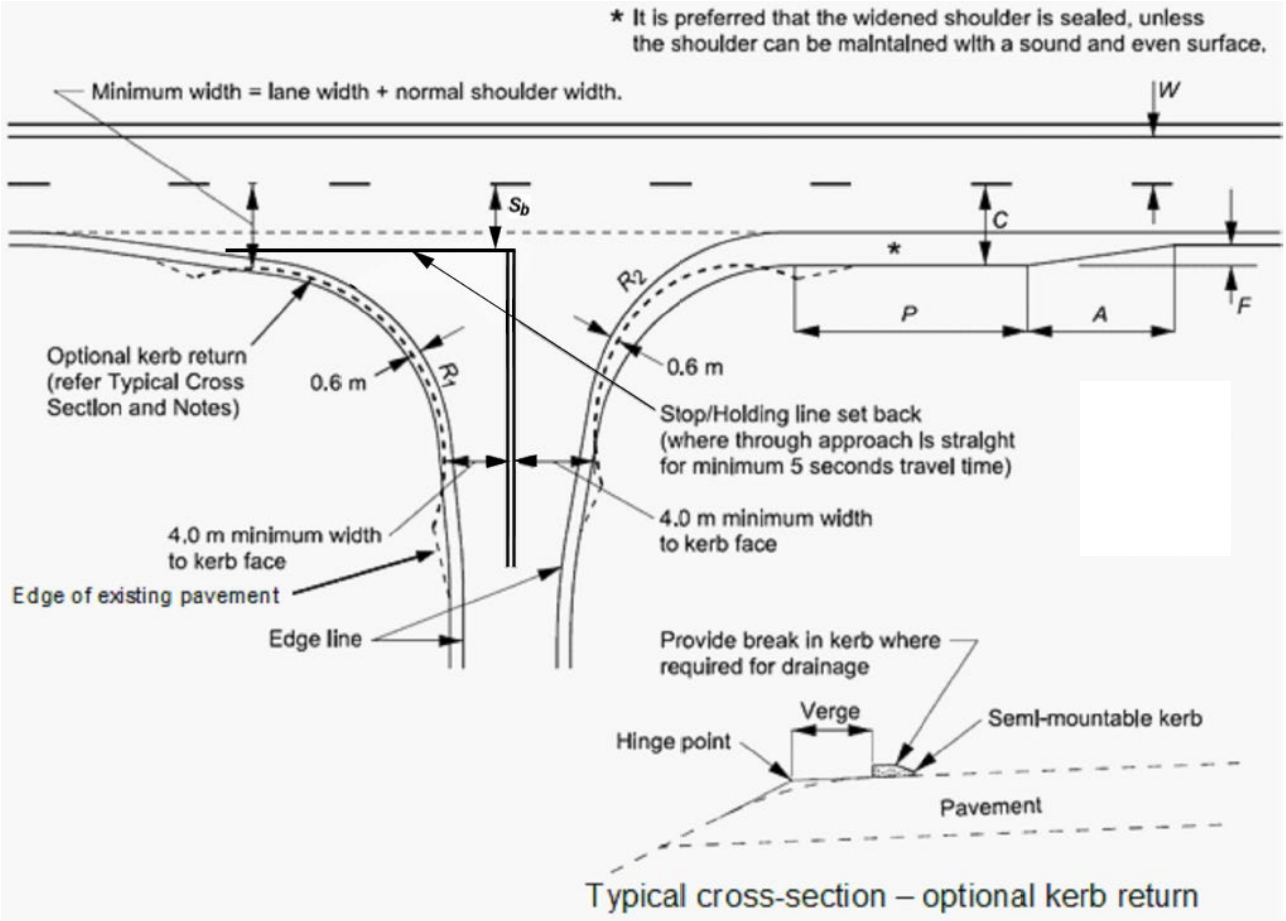
When the return radius (R) exceeds 11 m, the give way or stop line needs to be placed to allow the observation angle of 120° to be achieved.

Where the side road is located on:

- a straight, and its length is a minimum of 5 sec travel at the design speed, the holding line (particularly a stop line) should be located at distance S_b (note that 5 sec is the critical gap for drivers turning left; Table 3.5)
- the back of a curve, the holding line may be located closer to the through road
- the inside of a curve, the holding line may need to be located further back (limited to 8 m from the centreline of a two-lane rural road).

Where S_b exceeds 8 m other treatments (e.g. a high entry angle left-turn or a protected departure lane) should be considered in order to provide the 120° observation angle.

Figure 8.2: Rural basic left-turn treatment (BAL)



Notes:

- *R_1 and R_2 are determined by the swept path of the design vehicle.*
- *The dimensions of the treatment are defined thus:*

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

C = On straights – 6.0 m minimum.
On curves – 6.0 m plus curve widening (based on widening for the design turning vehicle plus widening for the design through vehicle).

$$A = \frac{0.5VF}{3.6}$$

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

F = Formation/carriageway widening (m).

P = Minimum length of parallel widened shoulder (Table 8.1).

S_b = Setback distance between the centre of the major road and the give way or stop line in the minor road. The holding line is typically placed in prolongation of the kerb line or edge line, however, it may be set back if there is a problem with the design vehicle over-running the holding line, or if it is desired to hold vehicles back some distance from the intersecting roadway (AS 1742.2 - 2009). The setback needs to be balanced such that sight distance is not negatively impacted to create a safety issue and the needs of pedestrians is met.

Source: Department of Main Roads (2006).

Table 8.1: Minimum length of widened parallel shoulder

Design speed of major road approach (km/h)	Minimum length of parallel widened shoulder *P* (m)
50	0
60	5
70	10
80	15
90	20
100	25
110	35
120	45

Note: Adjust the length for grade using the 'correction to grade' factor in Table 5.3.

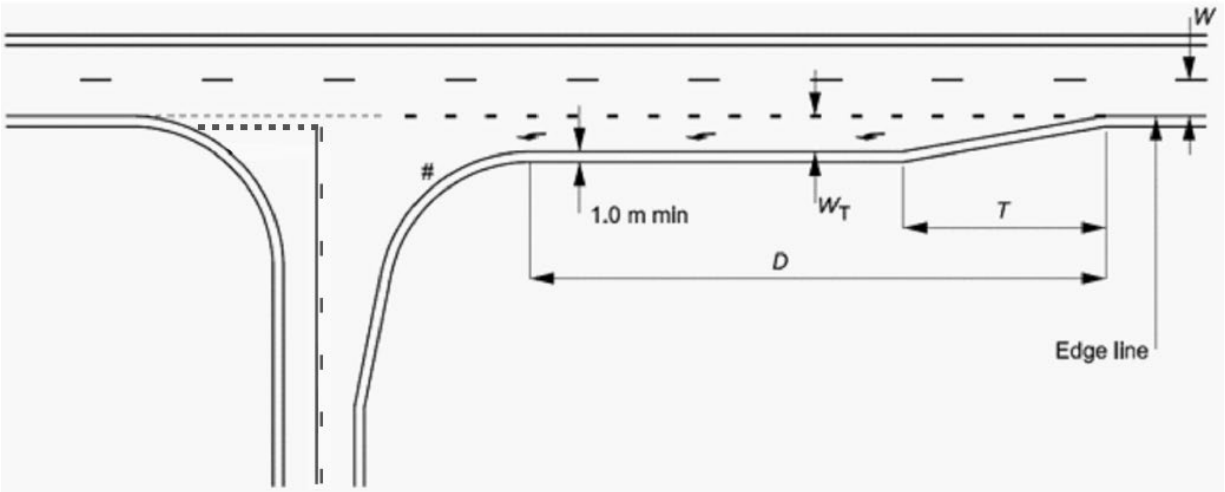
Source: Department of Main Roads (2006).

8.2.2 Rural Auxiliary Left-turn Treatment – Short Turn Lane [AUL(S)] on the Major Road

An AUL(S) turn treatment is shown in Figure 8.3. 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. The required length of treatment is shown in Table 8.2.

The AUL(S) layout should not be used where there is reduced visibility to the turn treatment. Left-turning drivers on the major road need to perceive the location of the deceleration lane and the side road in time to make the necessary speed reduction in the through lane prior to diverging.

Figure 8.3: Rural AUL(S) treatment with a short left-turn lane



Notes:

- *# For setting out details of the left-turn geometry, use vehicle turning path templates and/or Table 8.2.*
- *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 AGRD Part 4.*
- *The holding line is typically placed in prolongation of the kerb line or edge line, however, it may be set back if there is a problem with the design vehicle over-running the holding line, or if it is desired to hold vehicles back some distance from the intersecting roadway (AS 1742.2 - 2009). The setback needs to be balanced such that sight distance is not negatively impacted to create a safety issue and the needs of pedestrians is met.*
- *The dimensions of the treatment are defined as follows. Values of D and T are provided in Table 8.2.*

  W = Nominal through lane width (m) (including 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 the turn lane (m), including widening for curves based on the design turning vehicle = 3.0 m minimum.

  T = Physical taper length (m) given by Equation 5 being: $T = \frac{0.33VW_T}{3.6}$

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

Source: Department of Main Roads (2006).

Table 8.2: Dimensions for AUL(S) treatment on major leg

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 (Table 5.2). Adjust for grade using the 'correction to grade', (Table 5.3).*

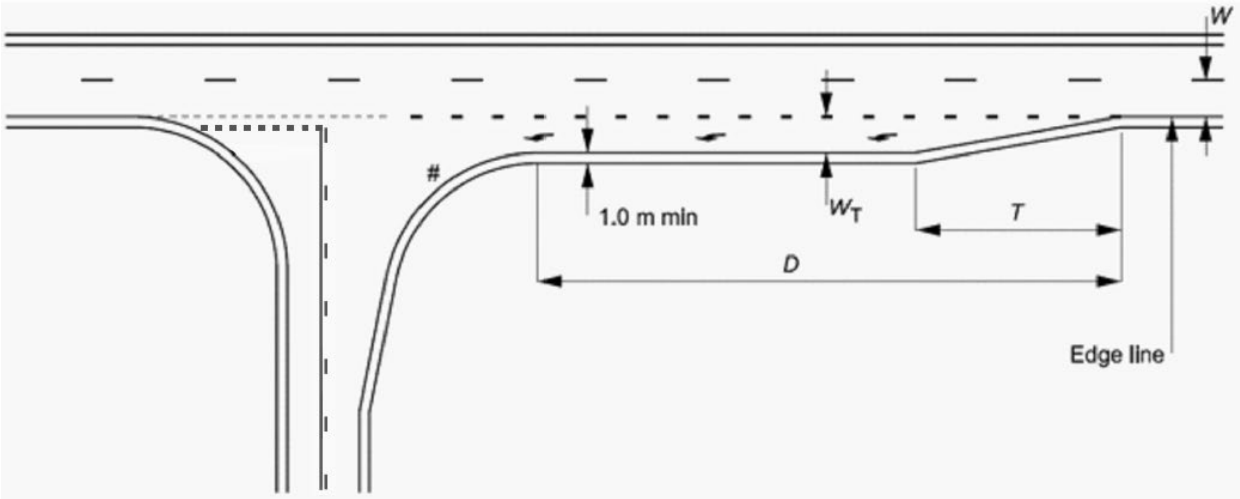
2 *Based on a turn lane width of 3.0 m.*

Source: Department of Main Roads (2006).

8.2.3 Rural Auxiliary Left-turn Lane Treatment (AUL)

A diagram of an AUL turn treatment on the major leg of a rural road is shown in Figure 8.4. 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 (refer to AGTM Part 6).

Figure 8.4: Auxiliary left-turn treatment (AUL) on a rural road



Notes:

- *# For setting out details of the left-turn geometry, use vehicle turning path software or templates.*
- *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 AGRD Part 4.*
- *The holding line is typically placed in prolongation of the kerb line or edge line, however, it may be set back if there is a problem with the design vehicle over-running the holding line, or if it is desired to hold vehicles back some distance from the intersecting roadway (AS 1742.2 - 2009). The setback needs to be balanced such that sight distance is not negatively impacted to create a safety issue and the needs of pedestrians is met.*
- *The dimensions of the treatment are defined thus:*

  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) = 3.0 m minimum.

  D = Diverge/deceleration length including taper – Table 5.2. (Adjust for grade using the 'correction to grade' in Table 5.3).

  T = Physical taper length (m) given by Equation 5 being: $T = \frac{0.33VW_T}{3.6}$

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

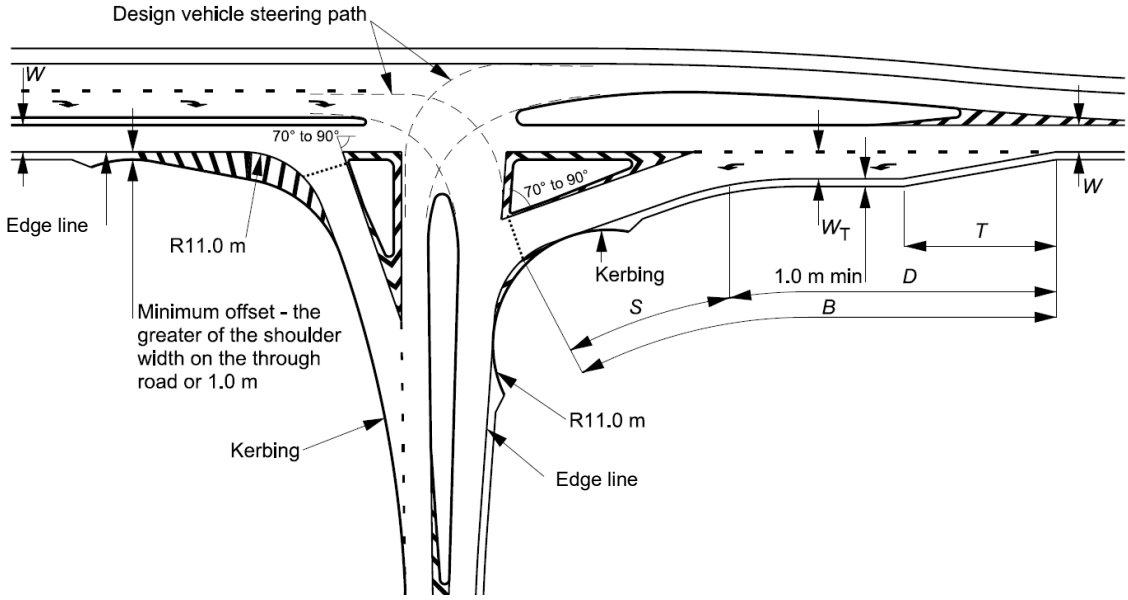
Source: Department of Main Roads (2006).

8.2.4 Rural Channelised Left-turn Treatment (CHL) with High Entry Angle

Provision of a left-turn island with a single radius return where edge lines intersect in the range 70° – 110° requires a high entry angle treatment to achieve an island of 50 m² (or more) in area and the observation sight requirements. Such a CHL left-turn treatment is shown in Figure 8.5 and is designed for use in high-speed environments (typically rural situations). The layout is similar to that shown for a low-speed environment with the exception that:

- the tracking of large vehicles is accommodated by a taper into the road being entered
- the left-turn island is considerably larger
- a left-turn auxiliary lane should be provided on major roads.

Figure 8.5: Rural (CHL) treatment with a high entry angle



Notes:
Approaches to left-turn slip lanes can create hazardous situations between cyclists and left-turning motor vehicles. One treatment to reduce the number of potential conflicts at left-turn slip lanes is given in AGRD Part 4.
Figure 6.4 details minimum offsets to islands.
Desirable minimum area of rural islands ≥ 50 m².
The dimensions of the treatment are defined as:

W = Nominal through lane width (m) (including 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), including widening for curves based on the design turning vehicle. Desirable minimum = W, absolute minimum = 3.0 m.

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

D = Diverge/deceleration length including taper. Adjust for grade using the 'correction to grade' factor (Section 5).

T = Physical taper length (m) and is given by Equation 5 being: $T = \frac{0.33VW_T}{3.6}$

S = Storage length (m) should be the greater of:
1. the length of one design turning vehicle or
2. (calculated car spaces –1) x 8 m (Guide to Traffic Management Part 3: Traffic Studies and Analysis (Austroads 2017)), or use computer program e.g. aaSIDRA.

Source: Department of Main Roads (2006).

8.2.5 Offset Rural Channelised Left-turn Lane Treatment (CHL)

In situations where there are significant numbers of vehicles, particularly heavy vehicles making a left turn from the major road at an intersection with an auxiliary left-turn treatment, restricting sight distance for vehicles turning out of the minor road, particularly right-turning vehicles, offsetting the left-turn lane from the adjacent through lane on the minor road improves the sight distance for vehicles turning out of the minor road. In particular, the sight distance to vehicles following a left-turning vehicle can be substantially improved. An offset left-turn lane should therefore be considered at an intersection where sight distance past left-turning vehicles may improve the intersection safety.

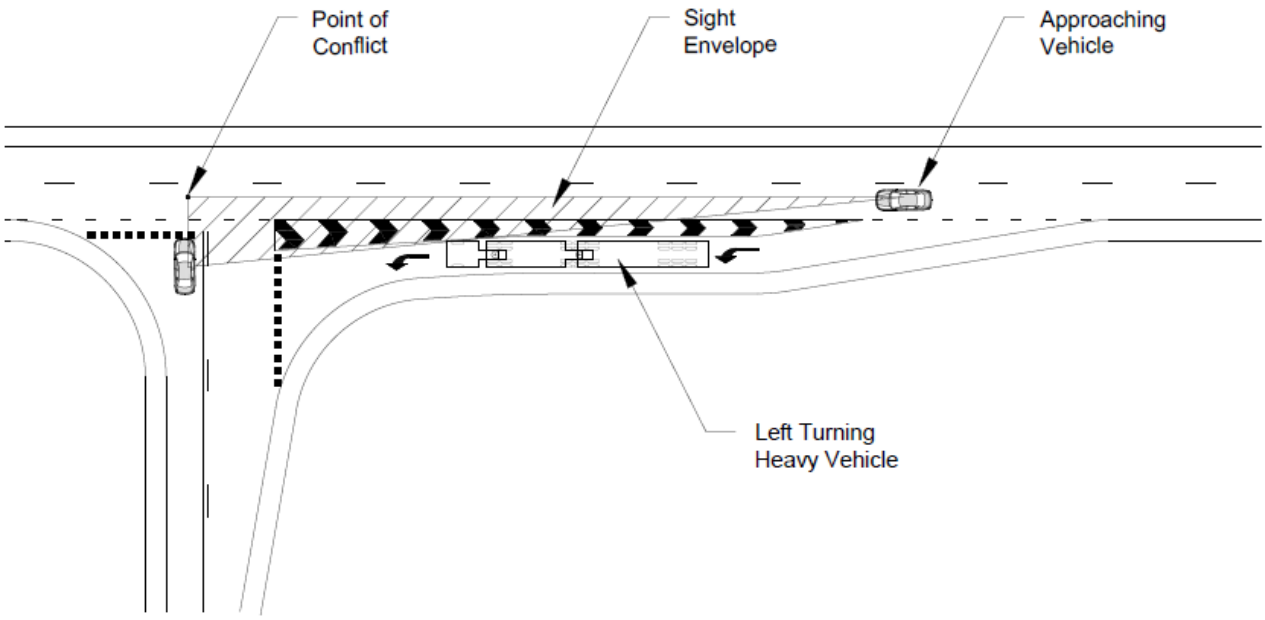
The factors that may warrant the use of an offset rural channelised left-turn lane include:

- high through traffic volumes on the major road
- high proportion / number of vehicles (particularly heavy vehicles) turning left from the major road
- the capacity of the turning movements from the minor road and resultant delays to vehicles
- intersection geometry and sight lines.

An acceleration lane can be incorporated on the minor leg of the intersection. Refer to AGTM Part 6 which provides discussion on when to implement an acceleration lane and Section 5 for acceleration lane design guidance.

A diagrammatic layout for the intersection is shown at Figure 8.6. The offset required for the left-turn lane is defined by the sight envelope required to achieve minimum gap sight distance (MGSD) as defined in Section 3.2.3. Designers should also consider lateral shift requirements in developing the offset left-turn lane.

Figure 8.6: Offset rural CHL treatment



Notes:

- *The left-turn channelisation should comply with the layouts for CHL treatments at Section 8.2.4.*
- *The offset from the adjacent through lane is determined based on the provision of MGSD as per Section 3.2.3.*
- *Provision of cycle lanes through the intersection should be as per Section 8.2.7. The cycle lane width can be included within the calculated offset and may negate the need for any further offset.*

Source: Queensland Department of Transport and Main Roads (2020b).

8.2.6 Rural Channelised Left-turn Treatment (CHL) with an Acceleration Lane

A channelised left-turn treatment with an acceleration lane comprises multiple radii returns i.e. 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 CHL turn treatment is shown in Figure 8.7.

A CHL with an acceleration lane (protected left-turn lane) can be useful where:

- the observation angle falls below guideline requirements (e.g. the intersection is located on the inside of a curve)
- insufficient gaps are available in the major road traffic stream for the left-turning movement
- 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.

Figure 8.7 shows the geometric details of a free-flow left-turn treatment with a three centred curve, suitable for use in a high-speed environment. Guidance on the required length of deceleration lane and acceleration lane is provided in Sections 5.2 and 5.3, respectively.

A free-flow treatment enables drivers turning left from the major road to decelerate at a comfortable rate clear of following traffic, turn left at a designated speed and join the intersecting road at its operating speed. It is provided where:

- intersection capacity can be improved by using an exclusive free-flow left-turn lane
- a protected departure lane is required for safety reasons (e.g. observation sight distance is less than required).

Free-flow left-turn treatments are generally not suited to locations where traffic turns at a moderate to high speed (e.g. ≥ 30 km/h) and cyclists and pedestrians need to cross the roadway, because of the risk to these vulnerable road users. They may not be suited to sites where a substantial proportion of the traffic turning left has to subsequently turn right at an adjacent intersection as operational problems may result in relation to weaving traffic streams.

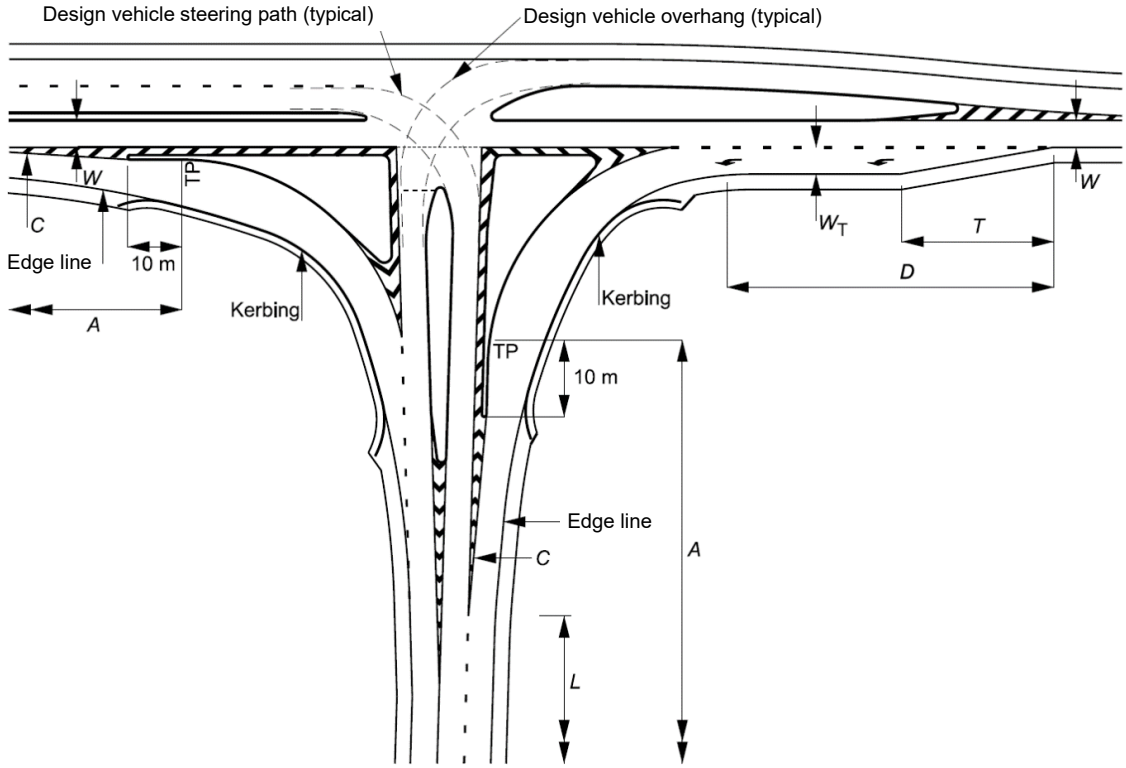
A key feature of the treatment is the three-centred curve that provides for the tracking of the design vehicle through the left-turn roadway (e.g. a 19 m semi-trailer).

A three-centred curve must not be used for unsignalised left-turn treatments that do not have a left-turn island. An appropriately designed island is necessary to:

- protect the departure lane
- control the path of exiting vehicles
- minimise crossing widths for pedestrians
- minimise the area of pavement that is not utilised by traffic.

Details of set-out parameters are provided in Figure 8.7. Commentary 7 illustrates the influence of incorrect and correct design on potential driver behaviour and the protection of the acceleration lane on the departure from the treatment.

Figure 8.7: Rural CHL treatment with an acceleration lane



Notes:

Key distances:

A = See Table 5.5 for length of the acceleration lane.

L = Minimum distance between end of chevron and start of merge taper to be based on 2 sec of travel time.

C = Maximum length of chevron taper based on 1:50.

Figure 6.4 details minimum offsets to islands.

Refer to Figure 7.3 (CHR turn treatment) for details of the dimensions T, D, W, and W_T.

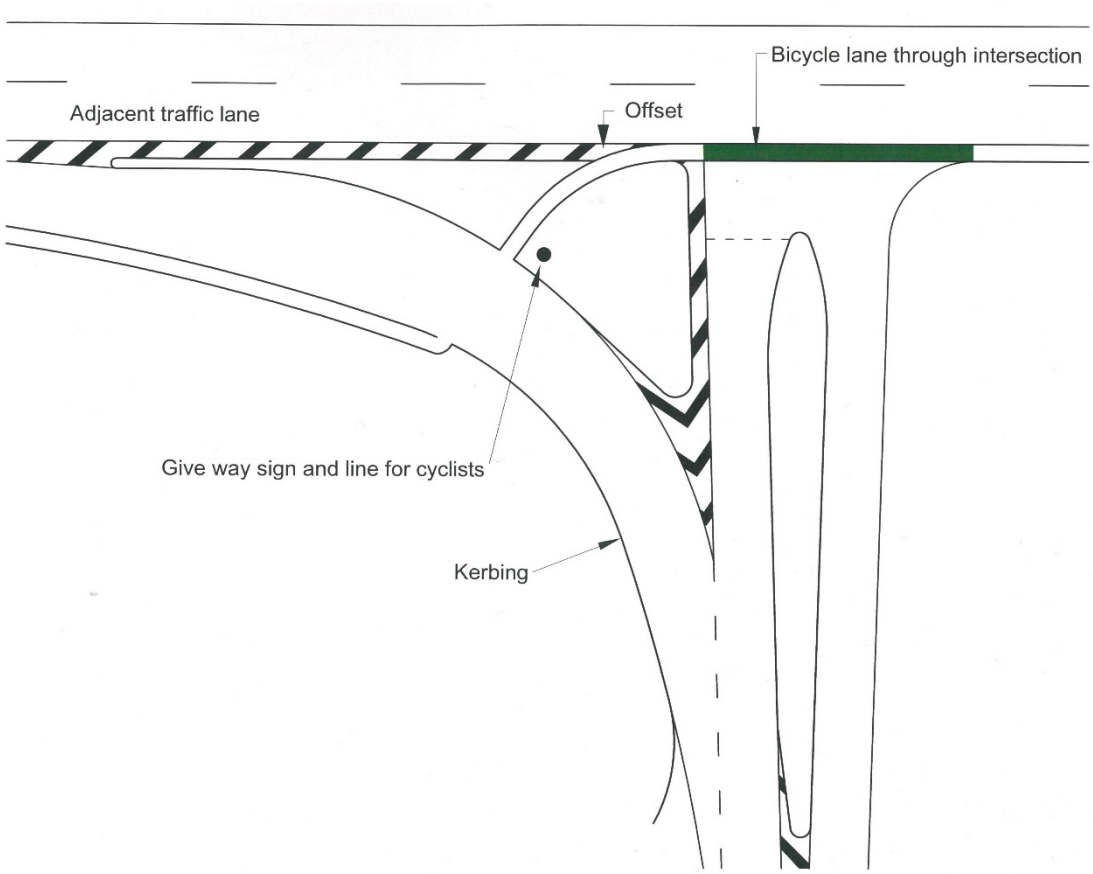
Desirable minimum area of islands ≥ 50 m².

Source: Department of Main Roads (2006).

8.2.7 Provision for Cyclists at Rural Free-flow Left-turn Lanes on Bicycle Routes

Figure 8.8 illustrates how a bicycle lane may be designed to provide a safer treatment for cyclists at a rural free-flow left-turn island. The treatment discourages cyclists from travelling in a path between the auxiliary lane and the adjacent through lane and being caught between through traffic and merging traffic. The width of the lane should be in accordance with the Austroads *Guide to Road Design Part 3: Geometric Design* (AGRD Part 3) (Austroads 2016a). The shape of the island treatment should be based on Figure 8.7 for a rural island and Figure 8.14 for an urban island, amended where necessary to accommodate a bicycle lane as shown in Figure 8.8.

Figure 8.8: Provision for cyclists at rural free-flow left-turn treatments



8.3 Urban Left-turn Treatments

8.3.1 Urban Basic Left-turn Treatment (BAL)

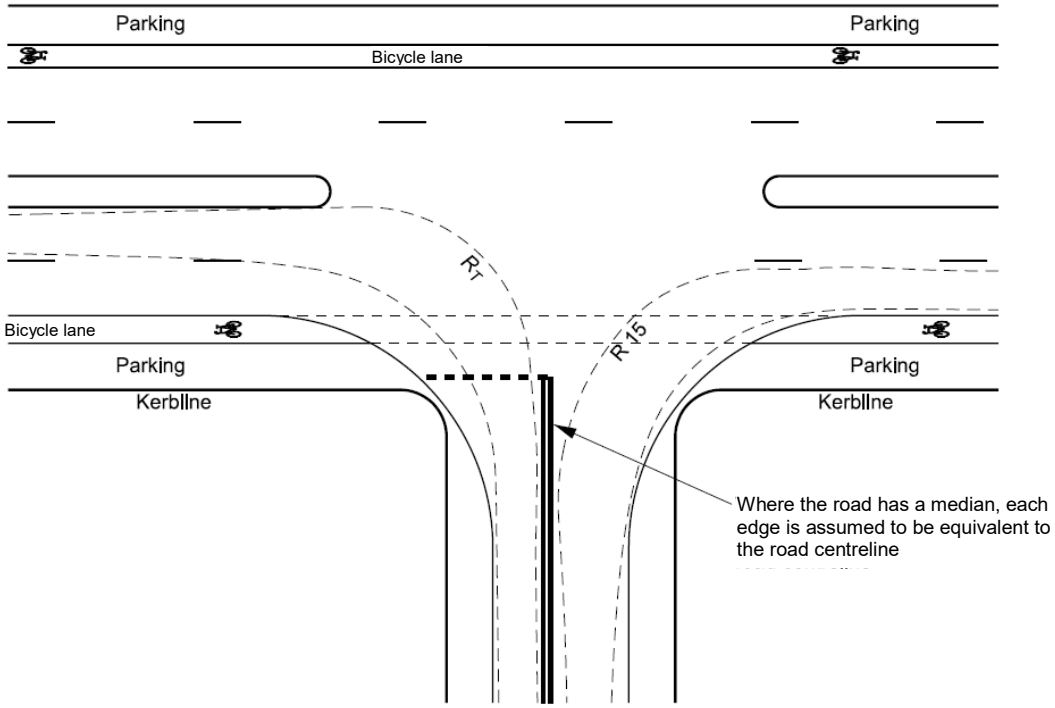
Single radius left-turn and tapers – low-speed environment

This is a simple treatment where the kerb lines of the intersecting roads are joined by a single radius circular arc and tapers to accommodate the swept path of turning vehicles (Figure 8.9). At sites in both rural and urban situations the left turn should be designed to enable the design vehicle to turn from the left lane into the minor road without crossing the centreline of the minor road.

While Figure 8.9 shows a large design vehicle (e.g. a B-double) which may apply where the minor road serves an industrial area, the design vehicle may be some other vehicle such as a single unit truck or bus. Designers should refer to Section 5 of AGRD Part 4 for guidance on the choice of an appropriate design vehicle.

The BAL layout should not be used where there is reduced visibility to the turn treatment. Left-turning drivers on the major road need to perceive the location of the side road in time to make the necessary speed reduction in the through lane prior to turning.

Figure 8.9: Basic left-turn treatment (BAL) on an urban road



Notes:
Where the approach is two lanes or more in width, heavy vehicles (12.5 m long or more) must turn from the kerbside or adjacent lane, unless otherwise controlled by signs and pavement arrows.
Where a side street approach and/or departure is not used by vehicles over 12.5 m long, a turning path for a bus/truck may be used.
This diagram does not show any specific bicycle facilities. Where specific bicycle facilities are required (e.g. exclusive bicycle lanes), designers should refer to AGRD Part 4 Section 9.
The holding line is typically placed in prolongation of the kerb line or edge line, however, it may be set back if there is a problem with the design vehicle over-running the holding line, or if it is desired to hold vehicles back some distance from the intersecting roadway (AS 1742.2 - 2009). The setback needs to be balanced such that sight distance is not negatively impacted to create a safety issue and the needs of pedestrians is met.

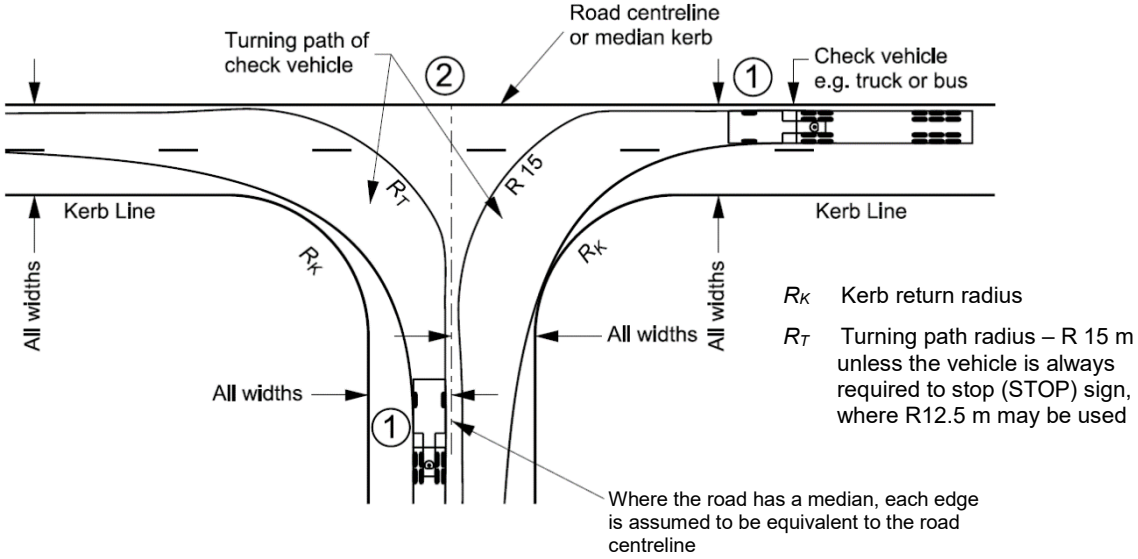
Source: Department of Transport and Main Roads (2006).

Single radius left-turn without tapers – low-speed environment

At low-speed urban intersections where a minor side road is predominantly used by cars, light trucks and service vehicles, and only used occasionally by larger vehicles, it may be appropriate to design for the occasional large vehicle to turn from the second lane from the left. Figure 8.10 shows the minimum treatment that may be used in a constrained low-speed, low-volume urban environment (e.g. built-up urban situation) where the design vehicle would normally be a service vehicle and a 19.0 m semi-trailer is the checking vehicle. The checking vehicle, while not crossing the centreline, may find it necessary to cross adjacent lanes to enter the minor road. This treatment is suitable only where very infrequent access would be required by this class of checking vehicle.

A return radius for an initial design for the side road can be selected from Table 8.3, which provides for a semi-trailer or truck/bus (12.5 m long) as the design vehicle. However, the radius of the turn is a function of the design vehicle, and the design should always be checked using swept path turning templates.

Figure 8.10: Application of a check vehicle swept path to a single radius treatment (BAL) for an urban intersection



Note: The design vehicle and the check vehicle will vary depending on the traffic characteristics at each site.

The observation angle of 120° to approaching traffic will be exceeded when the kerb return radius exceeds 11 m and the approach on the through road is straight for a distance equal to or greater than that travelled in 5 sec at the design speed of the through road (Figure 3.6). Hence, the kerb return radius of 11 m should only be exceeded when:

- entering on the outside of a horizontal curve
- leaving a through road without a slip lane
- entering traffic only needs to sight turning traffic (Figure 3.4b).

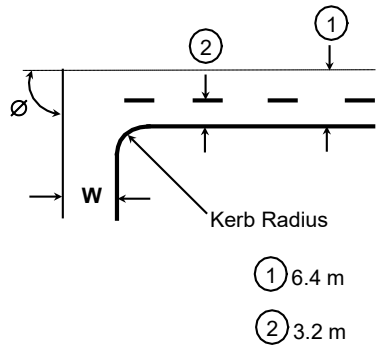
The kerb return radius should be reduced when entering on the inside of a curve.

Observation angles for the above conditions should be checked with criteria shown in Figure 3.4, Figure 3.5 and Figure 3.6.

Pedestrian crossing widths are generally not a problem if minimum kerb return radii are used. It should 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.

Table 8.3: Minimum kerb radii for low speed environment

Minimum left-turn kerb radii							
Vehicle type		(1) Turning from lane adjacent to kerb side lane – approach width 6.4 m			(2) Turning from kerb side lane – approach width 3.2 m		
	ϕ W (m)	70°	90°	110°	70°	90°	110°
Semi-trailer 19.0 m long	6.4	10	11	12	16	16	15
	5.5	12	14	14	18	18	16
Bus/truck 12.5 m long	6.4	3	6	8	12	12	12
	5.5	6	8	10	13	13	13



Notes:
Where approach and/or departure are curved, or widths vary from above, use turning templates to determine kerb radius and check observation sight distance.
Green shaded area represents situations where the kerb radius exceeds 11.0 m and the observation angle will be exceeded.
A left-turn movement from the lane adjacent to the kerb side lane is permissible in some circumstances, (Australian Road Rules)). For new intersections, application of such movements should only be considered when applying swept paths of the check vehicle.

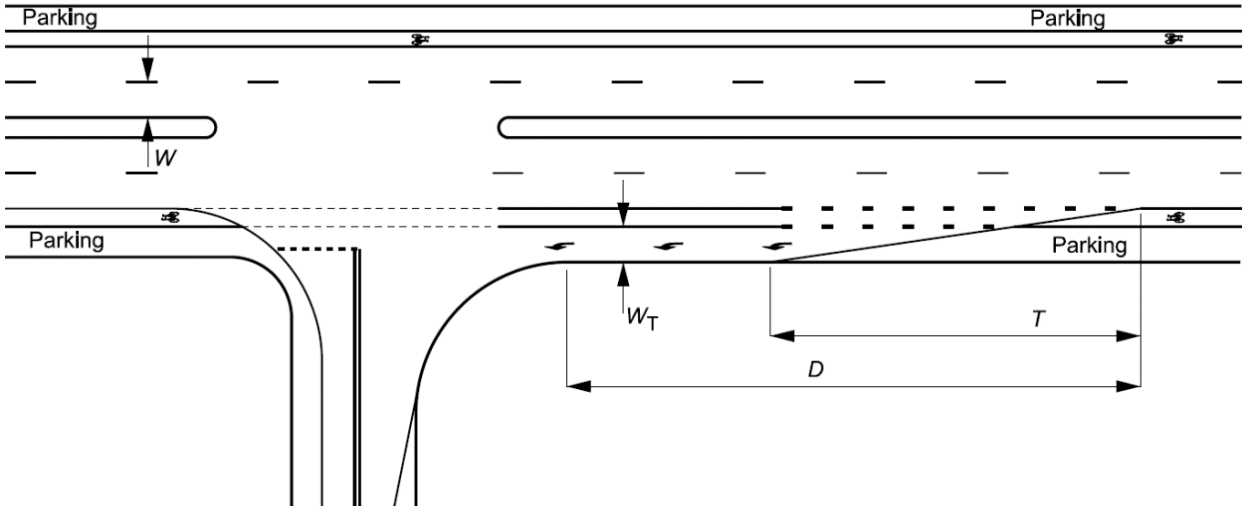
Source: Department of Main Roads (2006).

8.3.2 Urban Auxiliary Left-turn Treatment – Short Turn Lane [AUL(S)] on the Major Road

The BAL turn treatment from the major to minor road in Figure 8.9 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 8.11. Although some deceleration of the left-turning vehicles occurs in the through lane, this treatment records very few rear-end vehicle crashes on the major road (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 (AGTM Part 6). The AUL(S) layout should not be used where there is reduced visibility to the turn treatment. Left-turning drivers on the major road need to perceive the location of the deceleration lane and the side road in time to make the necessary speed reduction in the through lane prior to diverging.

Figure 8.11: Auxiliary left-turn treatment [AUL(S)] on the major leg of an intersection



Notes:

For setting out details of the left-turn geometry, use vehicle turning path templates and/or the details in Table 8.4.

Approaches to left-turn lanes can create hazardous situations between cyclists and left-turning vehicles. Treatments to reduce the number of potential conflicts at left-turn slip lanes are given in AGRD Part 4 Section 9.

The holding line is typically placed in prolongation of the kerb line or edge line, however, it may be set back if there is a problem with the design vehicle over-running the holding line, or if it is desired to hold vehicles back some distance from the intersecting roadway (AS 1742.2 - 2009). The setback needs to be balanced such that sight distance is not negatively impacted to create a safety issue and the needs of pedestrians is met.

Values of D and T are provided in Table 8.4 and the dimensions of the treatment are defined as follows:

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) = 3.0 m minimum.

T = Physical taper length (m) given by:

$$T = \frac{0.33V\,W_T}{3.6}$$

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

Source: Department of Transport and Main Roads (2006).

Table 8.4: Dimensions for D and T in AUL(S) treatment

Design speed of major road approach (km/h)	Diverge/deceleration length *D* (m)(1)	Taper length *T* (m)(2)
50	20	20
60	25	20
70	35	30
80	45	30
90	55	40

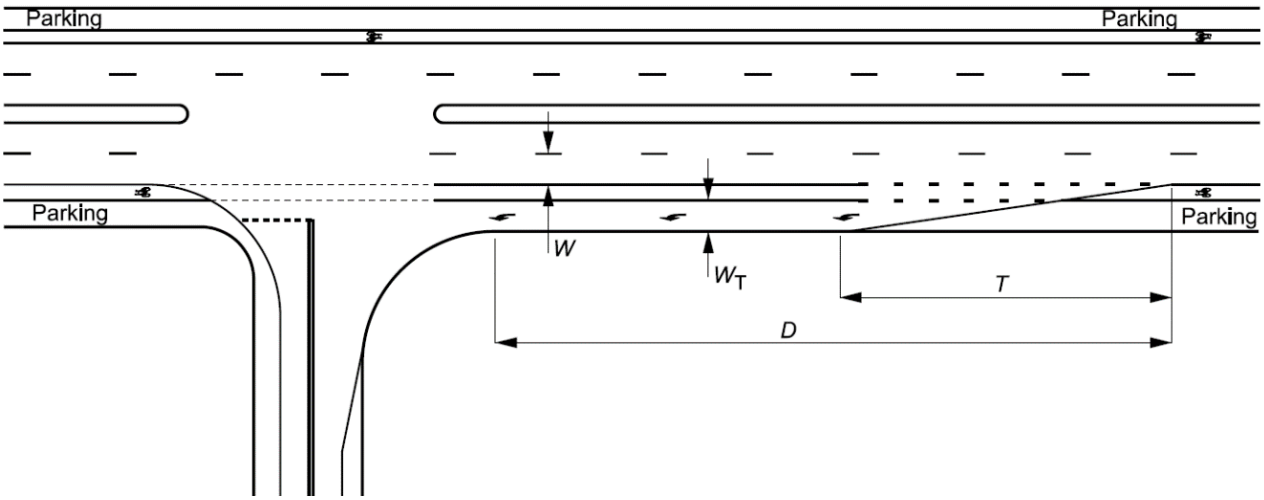
1 *Based on a 20% reduction in through road speed at the start of the taper and a value of deceleration of 3.5 m/s^2 (Table 5.2). Adjust for grade using the 'correction to grade' factor in Table 5.3.*

2 *Based on a turn lane width of 3.0 m and a bicycle lane width of 1.5 m.*

8.3.3 Urban Auxiliary Left-turn Treatment (AUL) on the Major Road

A diagram of an AUL turn treatment on the major leg of a divided urban road is shown in Figure 8.12. 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 traffic demand warrants the treatment.

Figure 8.12: Auxiliary left-turn treatment (AUL) on the major leg of an urban road



Notes:
For setting out details of the left-turn geometry, use vehicle turning path templates.
The holding line is typically placed in prolongation of the kerb line or edge line, however, it may be set back if there is a problem with the design vehicle over-running the holding line, or if it is desired to hold vehicles back some distance from the intersecting roadway (AS 1742.2 - 2009). The setback needs to be balanced such that sight distance is not negatively impacted to create a safety issue and the needs of pedestrians is met.
The dimensions of the treatment are defined as:

- 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) = 3.0 m minimum.*
- D = *Diverge/deceleration length including taper – Table 5.2. (adjust for grade by applying the 'correction to grade' factor in Table 5.3).*
- T = *Physical taper length (m) given by:* $T = \frac{0.33VW_T}{3.6}$
- V = *Design speed of major road approach (km/h).*

Source: Department of Main Roads (2006).

8.3.4 Urban Channelised Left-turn Treatment (CHL) with High Entry Angle

Figure 8.13 illustrates a high entry angle treatment for an intersection in a low-speed environment. The treatment is suitable for unsignalised or signalised intersections when appropriate road markings are provided and the medians are wide enough to accommodate signal hardware.

Where bicycle lanes pass through the intersection the design should be amended to accommodate them as indicated in Figure 8.15 and Figure 8.16, which relate to unsignalised and signalised intersections, respectively. The lengths of the sides of a high entry angle left-turn island for a low-speed environment can be derived from first principles. Detailed information for setting out the islands is provided in Section 6 for cases with and without a bicycle lane.

Where kerb lines intersect in the range 70° to 110°, and 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 a left-turn island that is adequate for lower-speed environments, as well as an observation angle of 120° (or less) to traffic approaching on a straight road of length not less than 5 sec of travel at the design speed.

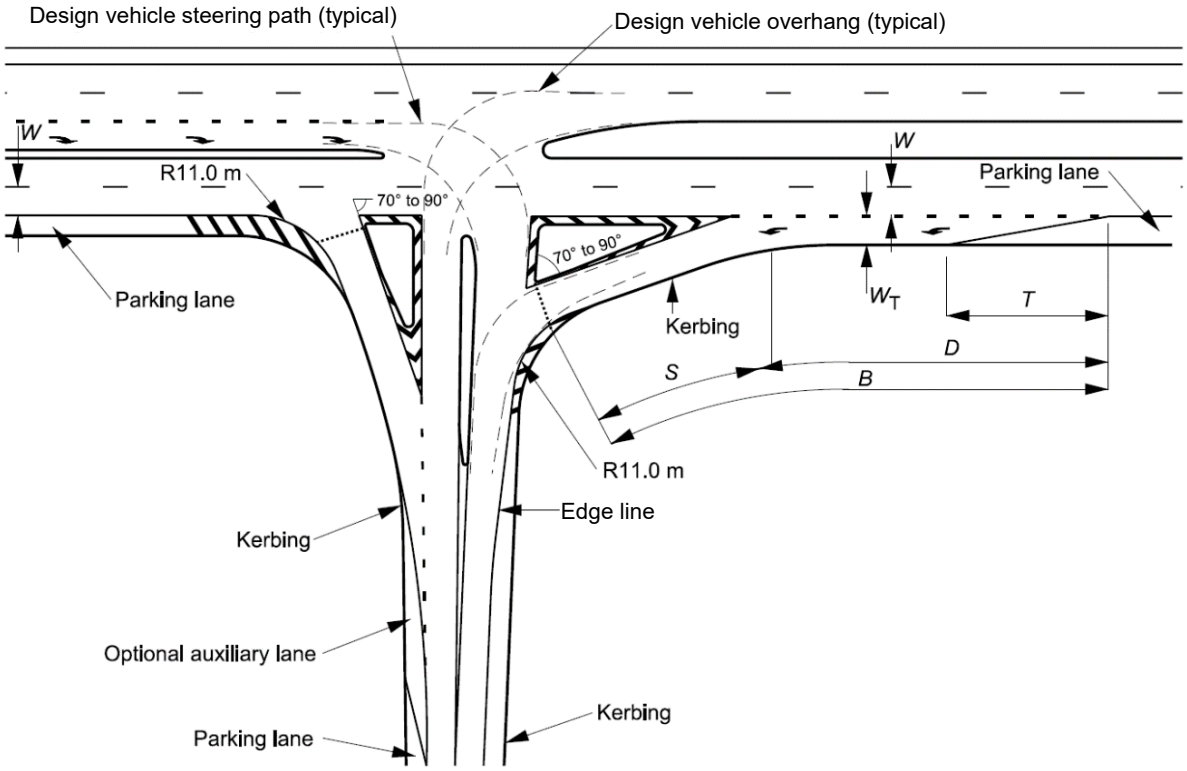
In some situations, it may be necessary to adopt a multiple radii return to avoid an expensive design control (e.g. a telecommunications pit). This is an acceptable treatment under EDD principles (Appendix A).

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, ASD should be provided on the approach to the crossing so that the pavement markings should be clearly visible over the entire length of ASD on approach to the crossing.

Appropriate bicycle treatments may be required adjacent to the left-turn island. Such treatments include linemarking and logos for bicycle lanes and may be supported by warning signs for drivers using the slip lane to watch for cyclists.

Alternative CHL layouts with high entry angles, which may be preferred by some road agencies, are discussed in Section 6.

Figure 8.13: Urban CHL treatment with high entry angle left-turn island



Notes:

Approaches to left-turn slip lanes can create hazardous situations between cyclists and left-turning vehicles. One treatment to reduce the number of potential conflicts at left-turn slip lanes is given in Figure 8.15 and Figure 8.16.

Figure 6.4 details minimum offsets to islands.

Refer to Figure 7.3 for details of the dimensions T, D, S, B, W, and W_T.

Desirable minimum area of islands – 25 m² for an unsignalised intersection and 40 m² for a signalised intersection.

Source: Department of Transport and Main Roads (2006).

8.3.5 Urban Channelised Left-turn Treatment (CHL) with Acceleration Lane

Figure 8.14 shows the geometric details of a free-flow left-turn treatment with a three-centred curve, suitable for use in a low-speed environment (urban situation with road lighting). This CHL treatment with an acceleration lane comprises multiple radii returns (i.e. it consists of compound circular arcs having two or three radii in order to best match the swept paths of turning trucks). Consideration should also be given to providing an acceleration lane on high-speed roads as a means of reducing speed differentials between turning slow-moving heavy vehicles and the faster moving through traffic, as a means of reducing crash risk.

CHL treatments with acceleration lanes are useful where:

- the observation angle falls below guideline requirements (e.g. the intersection is located on the inside of a curve)
- insufficient gaps are available in the major road traffic stream for the left-turning movement
- left-turning heavy vehicles will cause excessive slowing of the major road traffic stream.

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
- minimises the area of uncontrolled pavement.

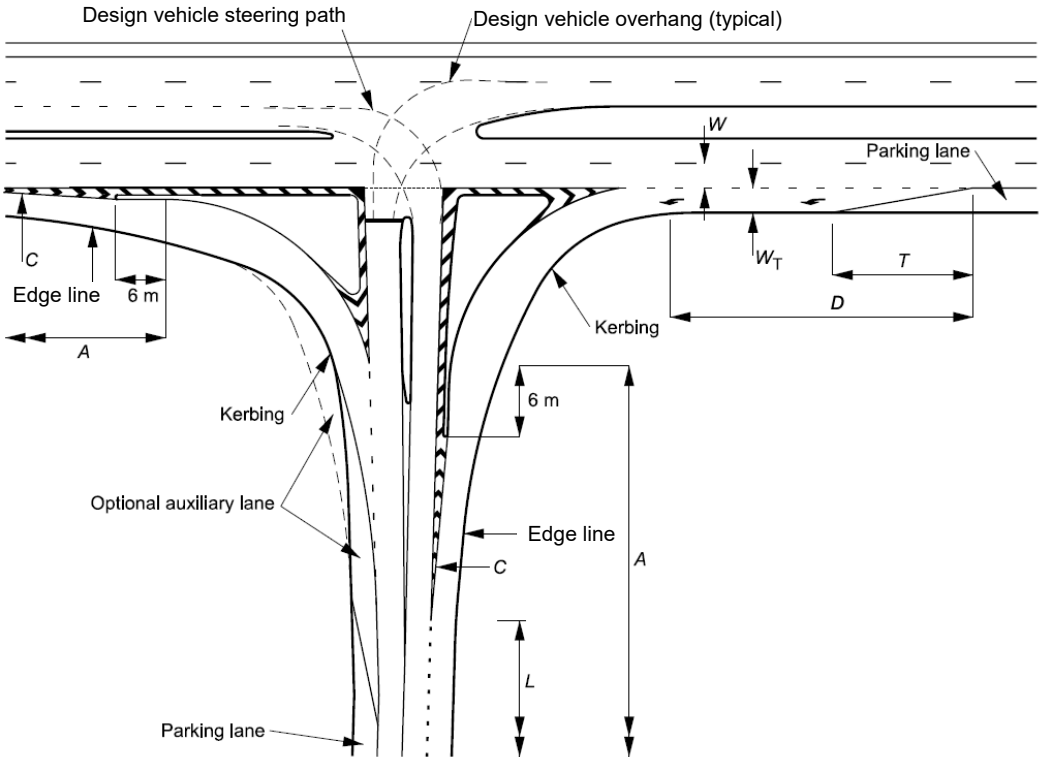
If the path of exiting vehicles is not controlled by the island nose the following will occur:

- the observation angle to approaching through traffic will be exceeded where the through approach is straight for a distance less than five sec of travel at the design speed
- an inadequate acceleration taper will result.

Where the intersection is used by pedestrians, an island can reduce the width of pavement to be crossed. If a pedestrian crossing is provided in the left-turn slip lane, then ASD must be available on the approach to the crossing, and pavement markings should be clearly visible over the entire length of ASD on the approach to the crossing.

Alternative CHL layouts with acceleration lanes, which may be preferred by some road agencies, are given in Section 6.

Figure 8.14: Urban CHL with acceleration lane



Notes: Key distances are:

A = See Table 5.5 for length of acceleration lane.

L = Minimum distance between end of chevron and start of merge taper to be based on 2 sec of travel time.

C = Maximum length of chevron taper based on 1:50.

Approaches to left-turn slip lanes can create hazardous situations between cyclists and left-turning vehicles. One treatment to reduce the number of potential conflicts at left-turn slip lanes is given in Figure 8.8.

Figure 6.4 details minimum offsets to islands.

Refer to Figure 7.16 (CHR turn treatment) for details of the dimensions T, D, W and W_T.

Desirable minimum area of islands ≥ 50 m².

The holding line is typically placed in prolongation of the kerb line or edge line, however, it may be set back if there is a problem with the design vehicle over-running the holding line, or if it is desired to hold vehicles back some distance from the intersecting roadway (AS 1742.2 - 2009). The setback needs to be balanced such that sight distance is not negatively impacted to create a safety issue and the needs of pedestrians is met.

Source: Department of Transport and Main Roads (2006).

8.3.6 Provision for Cyclists at Urban Channelised Treatments

General

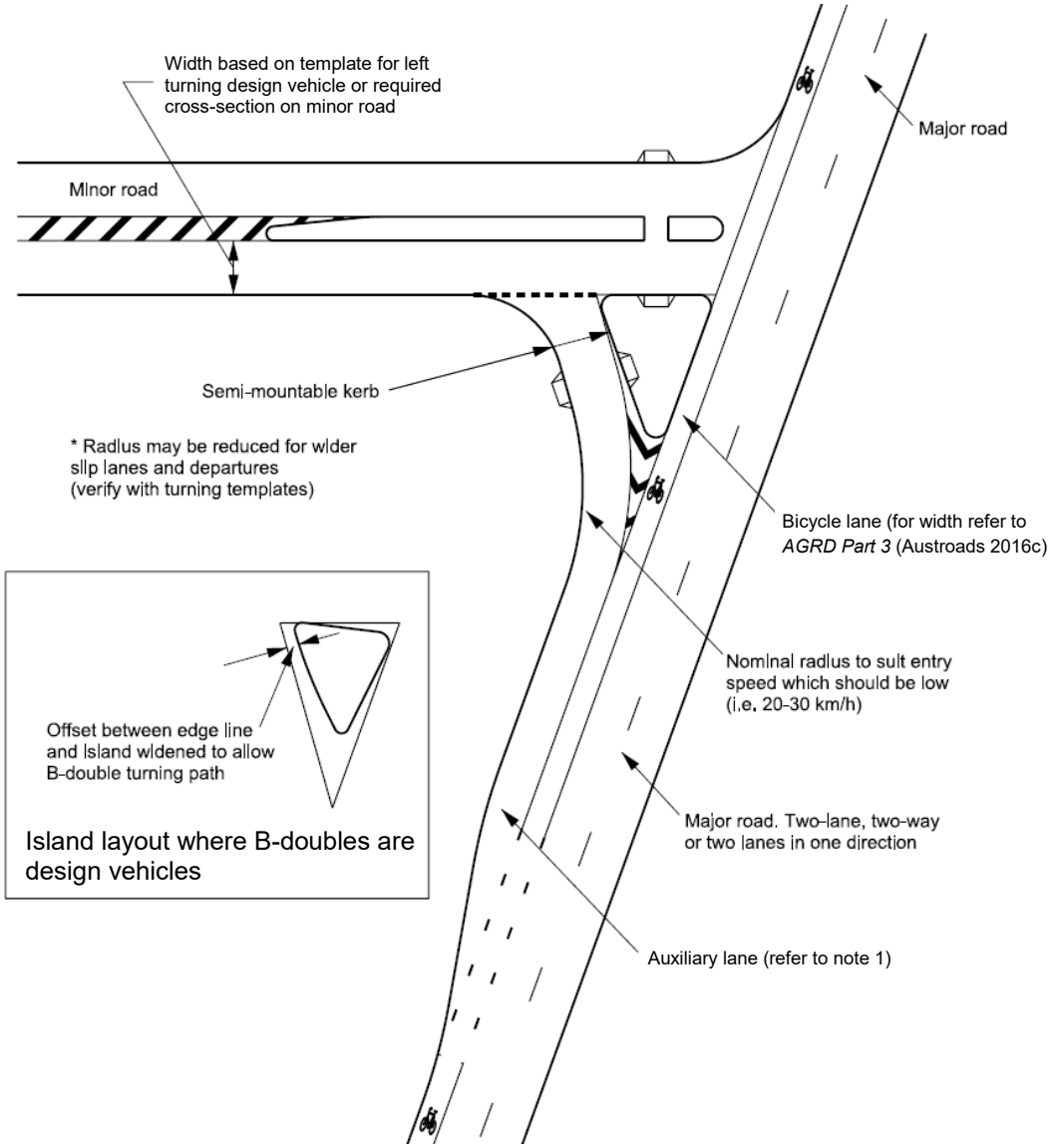
Approaches to exclusive left-turn treatments may create serious conflict points between cyclists and left-turning motor vehicles. The bicycle treatments presented in this section show how bicycle lanes should continue through the intersection, between general traffic lanes if necessary, to provide cyclists with a safer treatment and equity.

On priority cycling routes where there are long deceleration or acceleration tapers, large-radius curves and high speeds, it is particularly desirable that a bicycle lane be marked through the diverge areas and merge areas. These treatments provide space for cyclists and also warn drivers of the possible 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 bicycle 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 or in a wide kerbside lane would normally deliver adequate safety.

Figure 8.15 and Figure 8.16 respectively show bicycle lanes on the approaches to unsignalised and signalised intersections that have a high entry angle treatment.

Figure 8.15: Provision for cyclists at an unsignalised CHL treatment in a low-speed environment



1 *For details for auxiliary lane see Section 5.*

Notes:

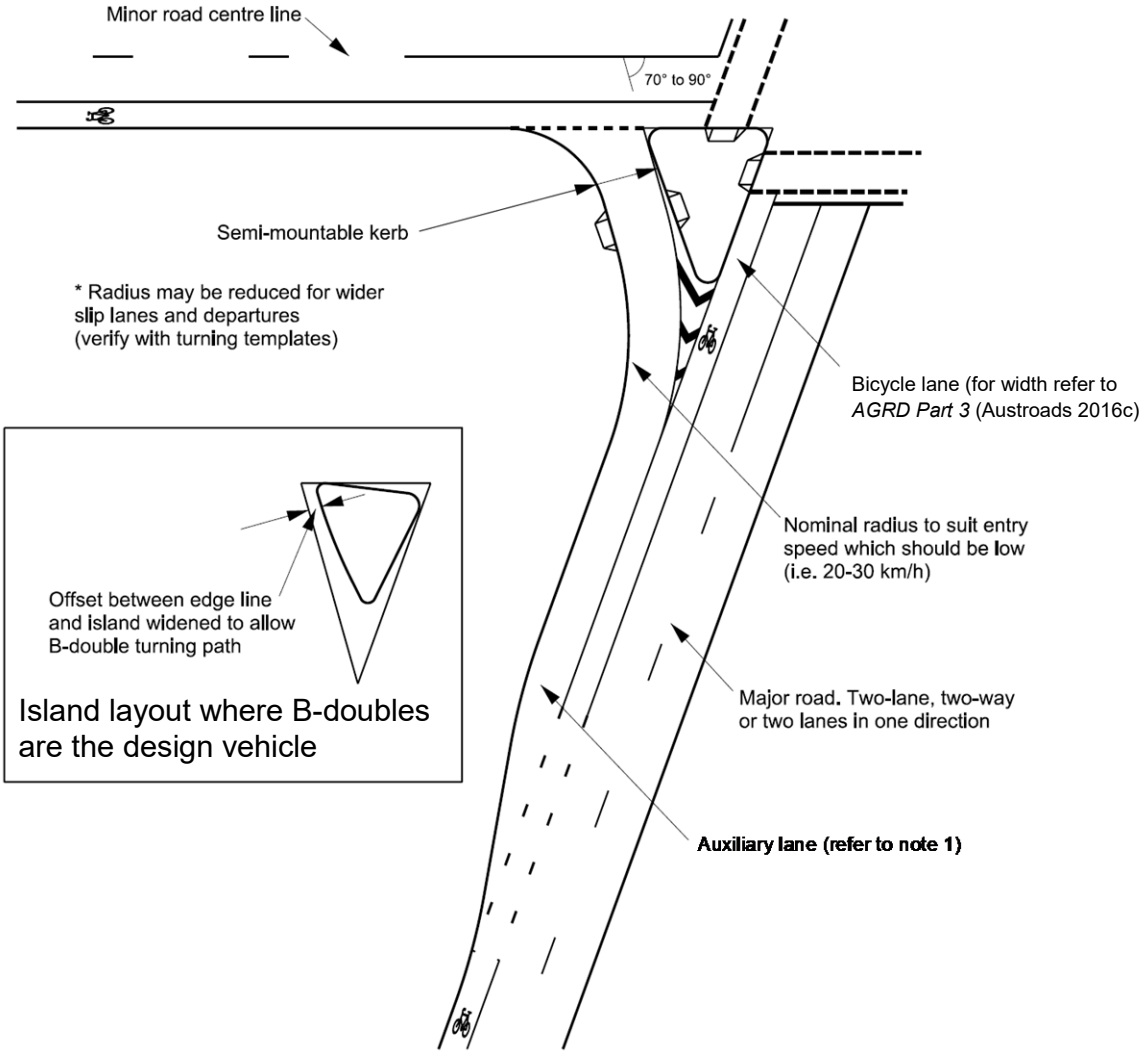
Bicycle lanes on the priority road must be continuous through unsignalised intersections and connect to bicycle facilities on the approach and departure.

In cases where a bicycle route turns the corner bicycle lanes may be accommodated within the minor road and within the left-turn roadway.

Green pavement surfacing may be used where high numbers of cyclists and motor vehicles interact. Where this is not the case normal surfacing, road markings and bicycle logos may be used to delineate the bicycle lane.

A minimum width of 5.0 m is required in the left-turn slip lane to enable vehicles to pass a disabled vehicle by mounting the semi-mountable kerb. It is therefore necessary to have a solid surface immediately behind the kerbs.

Figure 8.16: Provision for cyclists at signalised CHL treatment in a low-speed environment



1 *For details for auxiliary lane see Section 5.*

Free flow left-turn island with acceleration lane

The treatment illustrated in Figure 8.8 for rural sites can also be applied to urban sites. The treatment at urban sites will vary only in the detail of the dimensions of the treatment (Figure 8.14). At urban locations the intersection is likely to have lighting that will enhance the conspicuity of cyclists and their safety in dark conditions.

8.3.7 Left-turn Treatments for Large Vehicles

The extent of roadway required to accommodate large vehicles such as road trains at BAL turn treatments can become large, creating an undesirable situation for smaller vehicles (e.g. large undefined areas of pavement, accumulation of debris in areas of unused pavement). 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 vehicle (e.g. service truck or semi-trailer, as relevant) and provide an additional area for the larger vehicles, such as road trains, in a different material separated by a white line and diagonal markings. Some jurisdictions may prefer to have this additional area slightly raised above the turning roadway for the design vehicle (e.g. fully mountable kerb 50 mm high) to further discourage smaller vehicles from encroaching into the additional area.

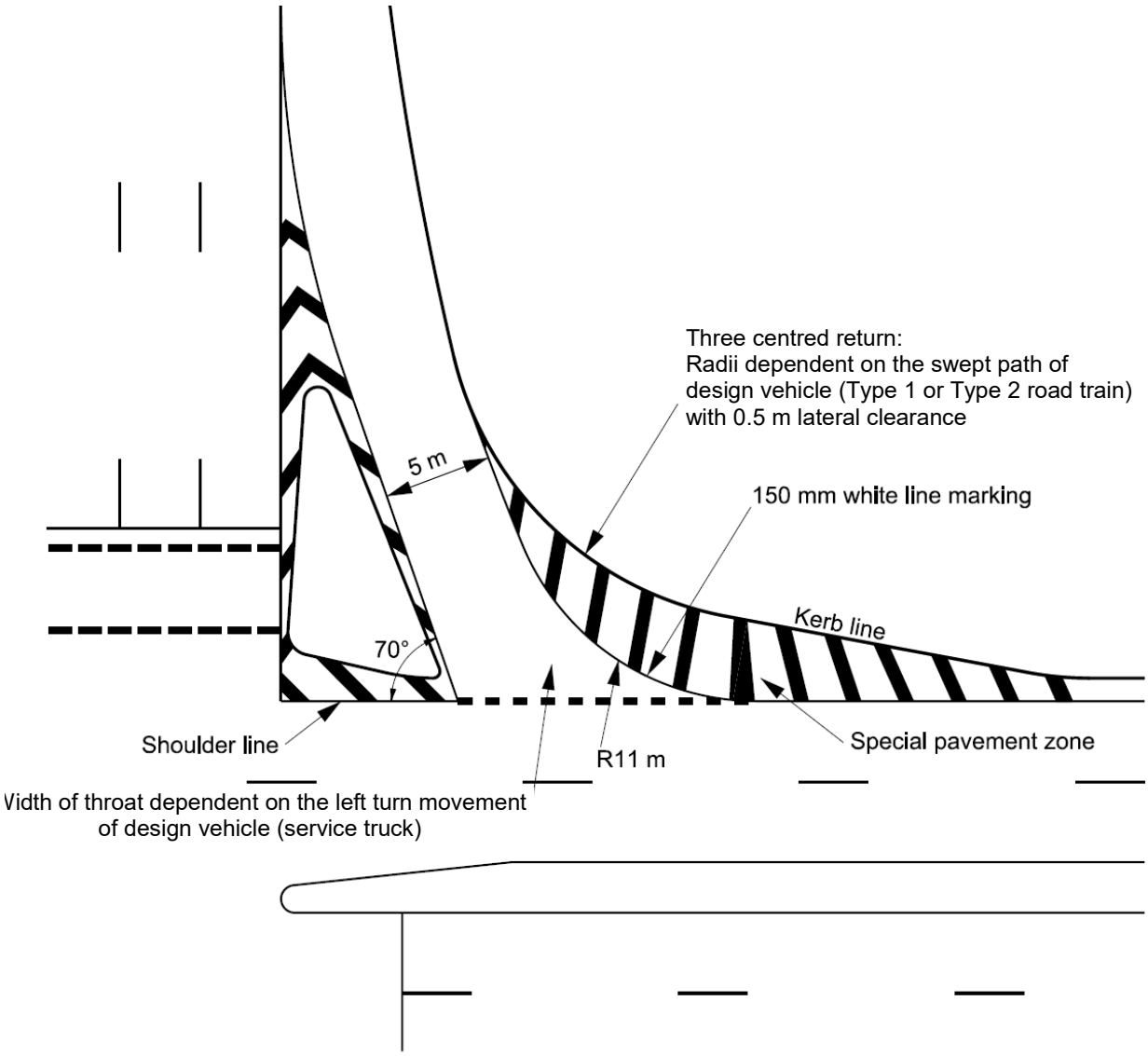
Figure 8.17 and Figure 8.18 respectively show a normal and alternative treatment developed to cater for road trains using a CHL high entry angle turn treatment. Although these figures show urban intersections similar layouts can be developed for rural sites. It should be noted that the kerb line and widths of both treatments are identical as they are both designed for a left-turning type 1 or type 2 road train. The difference is in the marking which is designed for the swept path of a design service truck in Figure 8.17 and for a design prime mover and semi-trailer in Figure 8.18. The shape of the marking in the former treatment covers more pavement area and would be more effective in encouraging drivers of cars to stand at the required angle of 70°.

Because the road trains have to travel over the marked areas in Figure 8.17 and Figure 8.18 and other vehicles may also traverse the area, these painted treatments at unsignalised urban left-turn roadways may pose a safety issue for pedestrians who may not be able to understand where to wait for a gap in the traffic. It is therefore suggested that in urban areas a signalised pedestrian crossing should be provided across the left-turn roadway and, if provided, the slightly raised kerb (say 50 mm) and contrasting pavement be flush with the left-turn roadway within the crossing.

In deciding to use a raised area designers and jurisdictions should be mindful that the height of 50 mm may be hazardous to some pedestrians, cyclists and motorcyclists. This height is between a flat surface and a step height and may constitute a trip hazard for pedestrians. A possible solution is to define a pedestrian path through the diagonal markings and ensure that the raised area slopes to meet the road pavement where the pedestrians cross. The issue for cyclists and motorcyclists using the left-turn roadway is that they are highly likely to become unstable should their wheels strike the low kerb. For this reason, it is desirable to provide road lighting and/or a high standard of delineation where these treatments are used.

Detailed further examples of this type of treatment and the swept path provisions for the treatments illustrated in Figure 8.17 and Figure 8.18 are provided in Appendix C.

Figure 8.17: CHL for road trains – normal treatment



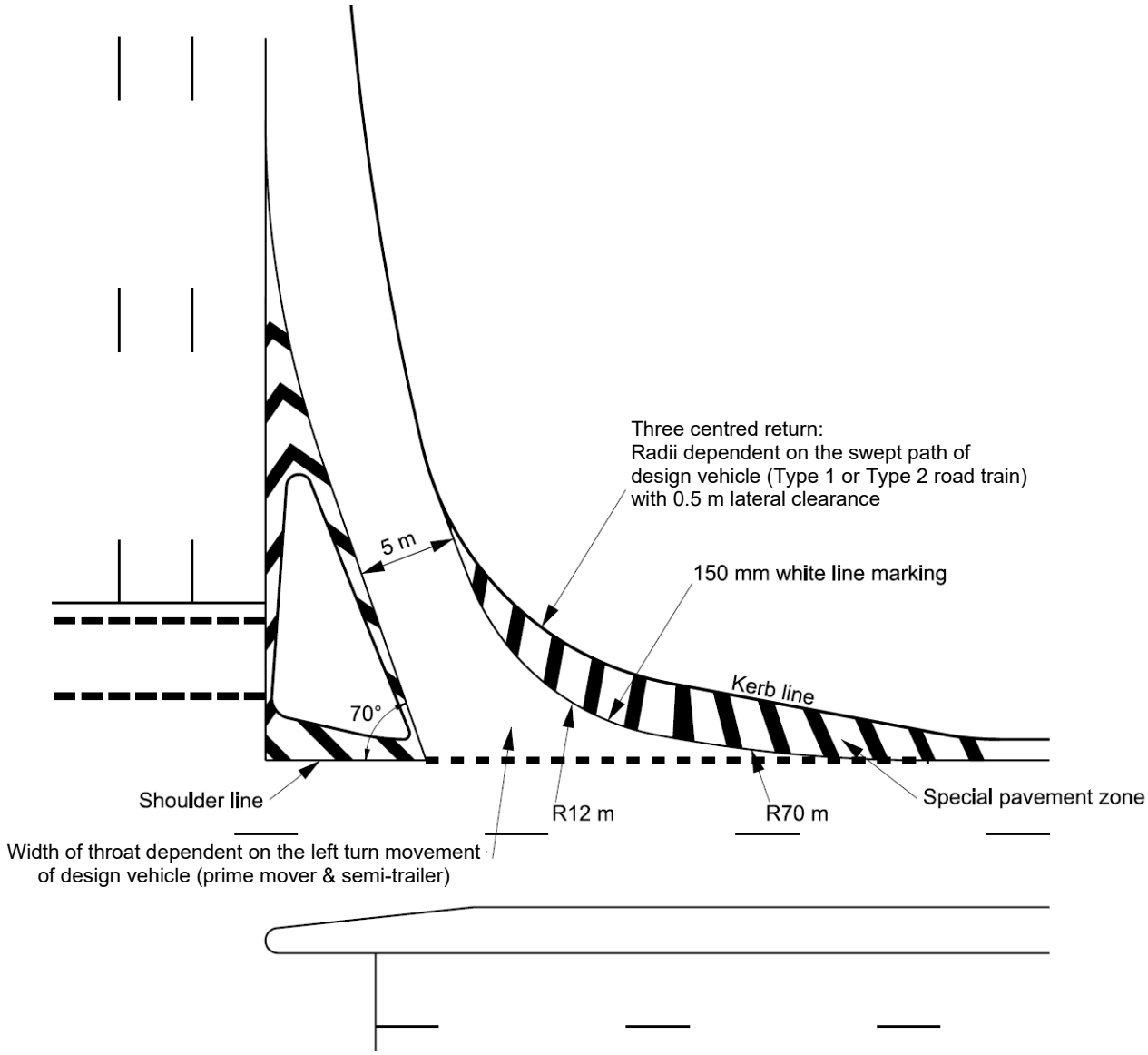
Notes:
This treatment:

- *is shown for an urban site but a similar layout is applicable to rural sites*
- *provides a special pavement area for the passage of large single unit trucks, prime movers and semi-trailer combinations and B-doubles*
- *promotes a desirable observation angle for all vehicle types if drivers of smaller vehicles minimise any encroachment onto the special pavement zone*
- *assumes that road train operation has been allowed because there is sufficient sight distance to avoid the use of stop signs.*

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.

Source: Department of Main Roads (2006).

Figure 8.18: CHL for road trains – alternative treatment



Notes:
This treatment:

- *is for areas where there is a high volume of large single unit trucks and prime mover and semi-trailer combinations (basic setting out details)*
- *is shown for an urban site. A similar layout is also applicable to rural sites*
- *may be used where the volume of large SU trucks and prime movers and semi-trailer combinations will cause unacceptable maintenance problems for the linemarking on the special pavement zone if the normal treatment in Figure 8.17 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*
- *assumes that road train operation has been allowed because there is sufficient sight distance to avoid the use of stop signs.*

Source: Department of Main Roads (2006).

8.3.8 Offset Urban Channelised Left-turn Treatment (CHL)

The use of offset channelised left-turn treatments can be considered for urban applications as defined in Section 8.2.5 for rural treatments. Similar to rural environments, factors that may warrant the use of an offset urban channelised left-turn lane include:

- high through traffic volumes on the major road
- high proportion/number of vehicles (particularly heavy vehicles) turning left from the major road
- the capacity of the turning movements from the minor road and resultant delays to vehicles
- intersection geometry and sight lines.

However, as space within an urban road corridor is often limited, introducing an offset urban channelised left-turn lane is often not possible without acquisition of additional land at a significant cost and disruption. An alternative to address the potential factors on low-speed urban roads could be to signalise the intersection to provide prioritised movement for motorists exiting the side street.

9. U-Turn Treatments

9.1 General

The provision of U-turn treatments relates to divided roads. On divided urban roads the need for vehicles to perform U-turns at median openings may require special provision to be made. This is particularly so where traffic regulations in some states prohibit U-turns at signalised intersections.

Drivers are permitted to U-turn at unsignalised intersections on divided roads unless a sign is erected to prohibit the turn. Unsignalised intersections are often spaced frequently enough on urban roads to remove the need for any additional special treatments for U-turning vehicles.

Roundabouts provide a convenient means for drivers to U-turn on both divided roads and undivided roads. Where jurisdictional traffic regulations prohibit drivers from U-turning at signalised intersections, it is usually desirable to provide a U-turn lane before the intersection and completely separate it from the right-turn lanes so that delay is minimised for all turning vehicles.

Although cars, small trucks and vans may be able to U-turn at intersections the manoeuvre is problematic for larger heavy vehicles because it is generally impracticable to provide enough space for these vehicles. However, where industrial or commercial properties abut an arterial road, it is often desirable and sometimes practicable to provide a suitably located and designed U-turn treatment for heavy vehicles to access the properties. U-turn lanes should have appropriate deceleration and storage lengths (Section 5).

9.2 Rural Roads

Rural divided roads that have at-grade intersections usually require very wide medians to provide sufficient storage in the median to safely accommodate heavy vehicles that are crossing the major road or turning into the major road from a side road (e.g. a B-double is 25 m to 27 m long and requires a 30 m wide median). These vehicles can generally U-turn in the space provided by such a wide median and shoulders. On freeways U-turns should only be permitted at interchanges.

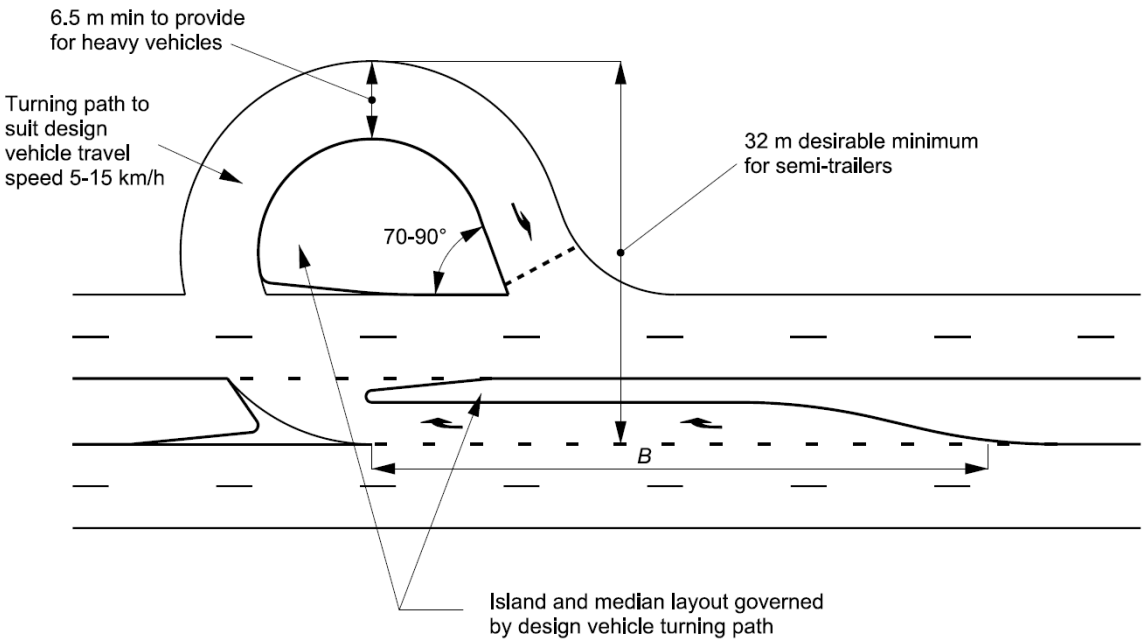
9.3 Urban Roads

On urban divided roads where a demand exists for heavy vehicles to U-turn and space is available, the treatments illustrated in Figure 9.1 may be appropriate. These treatments comprise a jug handle layout suitable for semi-trailers and a treatment where auxiliary lanes enable a single unit truck/bus to U-turn.

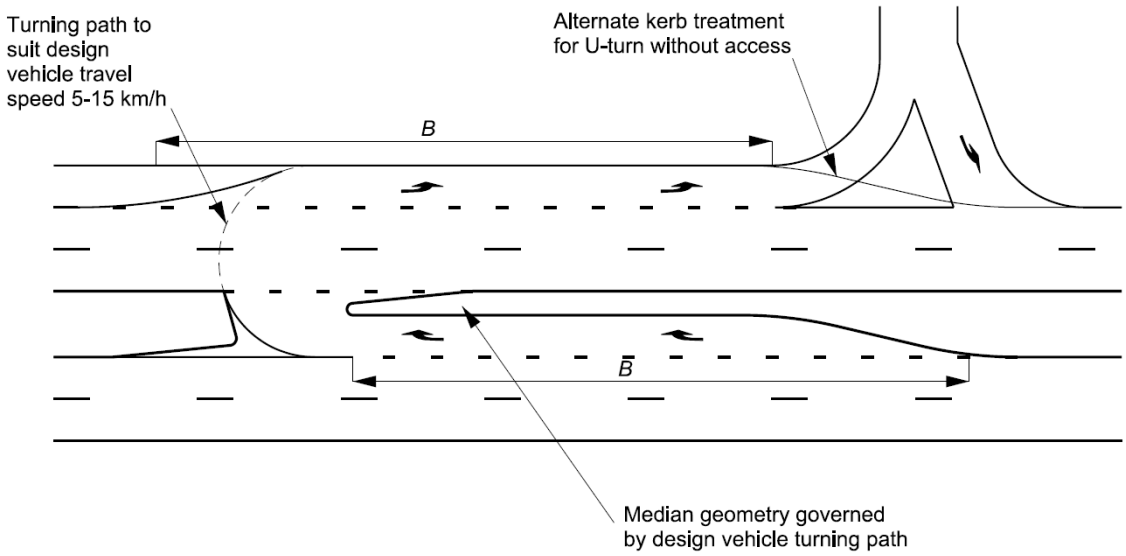
On roads with relatively narrow medians, unsignalised intersections may be designed to allow cars to U-turn. Where intersections are a substantial distance apart and demand exists, the mid-block U-turn treatment shown in Figure 9.2 may be provided. Often the treatment can only cater for cars; however, they may also be strategically placed to enable trucks to U-turn into service roads.

It is desirable that U-turn treatments are located away from intersections; however, a signalised U-turn treatment may be provided on the approach to a signalised intersection adjacent to right-turn lanes as shown in Figure 9.3.

Figure 9.1: Desirable U-turn treatment (mid-block or in advance of an intersection)



(a) Jug-handle U-turn facility (suitable for semi-trailers)



(b) U-turn facility - urban areas - restricted to single unit vehicles

Figure 9.2: Desirable U-turn treatment (mid-block or in advance of an intersection

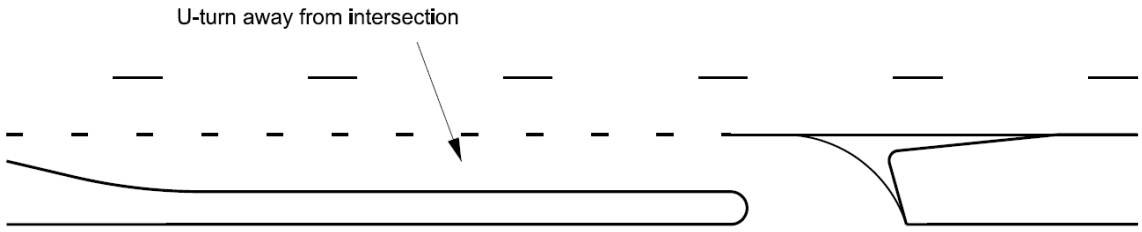
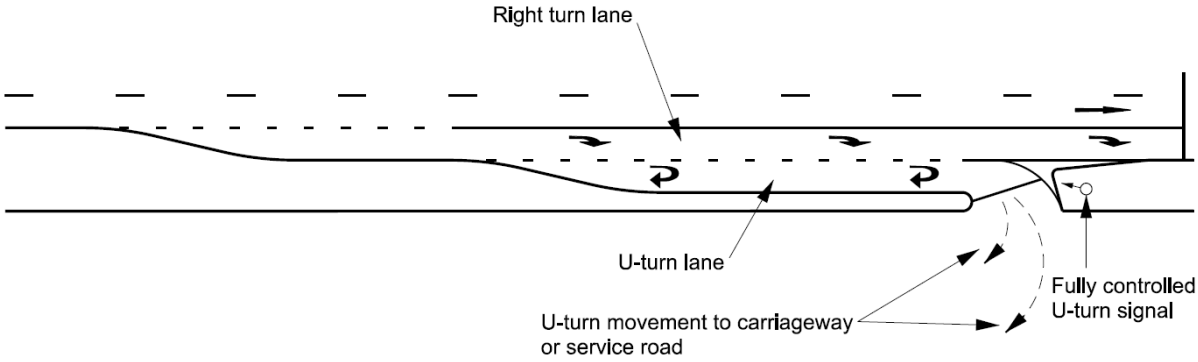


Figure 9.3: U-turn lane on approach to signalised intersection



10. Signalised Intersections

Intersections are signalised to address either a road safety or a traffic/transportation operational issue. Signalisation changes intersection priority control from gap-acceptance to signal compliance. This means a change in control from one which is very prone to road user error to one which is less so. The driver workload and skills needed to negotiate a signalised intersection are much lower than for a comparable non-signalised intersection.

Signalised intersections on major arterial roads often require a number of through lanes and turn lanes to ensure that the intersecting routes have adequate capacity as a result of the various intersecting traffic movements that must share time. Austroads *Guide to Traffic Management Part 6: Intersections, Interchanges and Crossings Management* (AGTM Part 6) (Austroads 2020a) provides information on the traffic management aspects of signalised intersections, for example:

- Section 3 includes traffic management considerations in the selection of traffic signals as a method of control and provides general guidance on the signalisation of intersections.
- Section 5 provides discussion on road space allocation, lane management, signal phasing and timings, signal coordination and traffic detection.

Consideration of traffic management and road safety issues and aspects is an essential part of developing conceptual and functional layouts for signalised intersections, and the subsequent development of a signal layout plan. Whilst signalising an intersection provides a reduction in the likelihood of a crash, the crashes which occur are still likely to be severe. Speed, turning controls, site size, traffic volumes, approach geometry, and signal visibility all affect the likelihood of these crashes.

Practitioners must ensure that, where practicable, the needs of all road users are identified (i.e. through developed plans/strategies or specific site investigations) and incorporated into the intersection design.

It is particularly important that the design of signalised intersections provides adequately for public transport (refer to Section 6 of the Austroads *Guide to Road Design Part 4: Intersections and Crossings – General* (AGRD Part 4*)* (Austroads 2023b)), pedestrians including people who have mobility or vision impairments (refer to Section 8 of AGRD Part 4) and cyclists (refer to Section 9 of AGRD Part 4). These requirements may include:

- public transport lanes, stops and priority measures
- pedestrian facilities (including treatments to assist people who have impaired vision or mobility)
- bicycle lanes, paths and crossings.

Moderating both impact speeds and angles is an effective method to reduce crash severity. Innovative solutions and treatments for signalised intersections can be found in the Austroads *Guide to Road Design Part 7: New and Emerging Treatments* (AGRD Part 7) (Austroads 2021b).

Further information on these requirements can be found in AGTM Part 6.

10.1 Design Process

The process of designing a signalised intersection is described in Section 2. It involves operational and geometric requirements that are inter-related and determine the information that is presented on functional design and signal layout plans.

10.2 Sight Distance

The sight distance provided at signalised intersections should be in accordance with the general requirements for intersections as described in Section 3 of this Part. While the existence of illuminated signal aspects provides an additional cue to drivers that they are approaching an intersection, approach sight distance (ASD) is desirable so that these drivers can see the layout of the intersection approach, the pavement and traffic paths within the intersection.

It is desirable to provide ASD, minimum gap sight distance (MGSD) and safe intersection sight distance (SISD) at both signalised and unsignalised movements at signalised intersections, as signals may not always function because of power outages or crash damage to the controller. Stopping sight distance (SSD) should be available at all points on each roadway and to the signal displays as a minimum.

Overhead signal faces may be used where the stopping sight distance to a post-mounted display is inadequate (e.g. because of vertical or horizontal alignment, awnings, poles, trees or similar sight obstructions) or where the roadway is too wide for kerb-mounted lanterns to fall within the driver's line of sight. However, overhead signals may be omitted if there is a likelihood that they could appear to apply to an adjacent upstream signalised intersection. In constrained situations (e.g. some urban intersections) an EDD value for SSD and SISD may have to be considered (Appendix A).

Provision of SSD to the rear end of a stationary queue of vehicles waiting at a stop signal reduces the probability of a rear-end collision. Refer to the Austroads *Guide to Traffic Management Part 3: Transport Studies and Analysis Methods* (AGTM Part 3) (Austroads 2020d) for further guidance on the assessment of queue lengths.

For further information on sight distance requirements at signalised intersections, refer to AS 1742.14 and the Austroads *Guide to Traffic Management Part 10: Transport Control – Types of Devices* (AGTM Part 10) (Austroads 2020c).

10.3 Signal Operation Considerations

10.3.1 Traffic Operation at an Intersection

Traffic signal operation considerations are discussed in detail in the Austroads *Guide to Traffic Management Part 9: Transport Control Systems – Strategies and Operations* (AGTM Part 9) (Austroads 2020e). However, it is important that designers recognise that the geometric layout and the operation of a signalised intersection are inter-related. For example:

- Adequate space must be available within the intersection for concurrent right turns to occur from opposite directions (i.e. diamond turn).
- In situations where roads intersect at oblique angles and space is constrained the geometry may lead to less efficient phasing. An example of this is illustrated and briefly discussed in Commentary 8.

10.3.2 Proximity to Other Intersections

Desirably, intersections should be separated by at least 5 sec of travel time at the design speed to provide time for drivers to process information relating to traffic, the road layout and traffic signs. However, it is preferable that signalised intersections be separated by a considerable distance so that:

- traffic queues from one intersection do not back up through an adjacent intersection and adversely affect traffic safety or operation
- inefficiencies do not result from the signal phasings at the intersections having to be interdependent
- safety issues do not arise because of the 'see through effect', whereby a driver approaching along the major road focuses on green lights at the second intersection rather than red lights at the first intersection.

Commentary 9 provides an example of how a traffic queue resulting from a signalised intersection can adversely affect sight distance and hence safety at an upstream intersection.

10.4 Intersection Layouts

10.4.1 General

Signalised intersection layouts take many forms depending on the:

- reservation available
- classifications of the intersecting roads
- traffic volumes
- constraints such as public utilities, buildings and other road use (e.g. parking, pedestrian malls on one leg)
- the angles of the intersecting roads.

The typical signalised intersections are usually at crossroads or T-intersections where the roads intersect at a right angle.

Figure 10.1 is an example of a signalised intersection and shows the type of detail provided on a signal layout plan that would be prepared, to assist in the specification and installation of traffic signals. Lane arrangements vary depending on the space available on the intersection approaches, the movements to be accommodated or permitted and the volume of traffic (e.g. double right-turn and double left-turn lanes).

A further example of a signalised intersection that has complementary double right-turn and left-turn lanes is shown in Figure 10.2.

Figure 10.1: Example of a signalised intersection on a divided arterial road

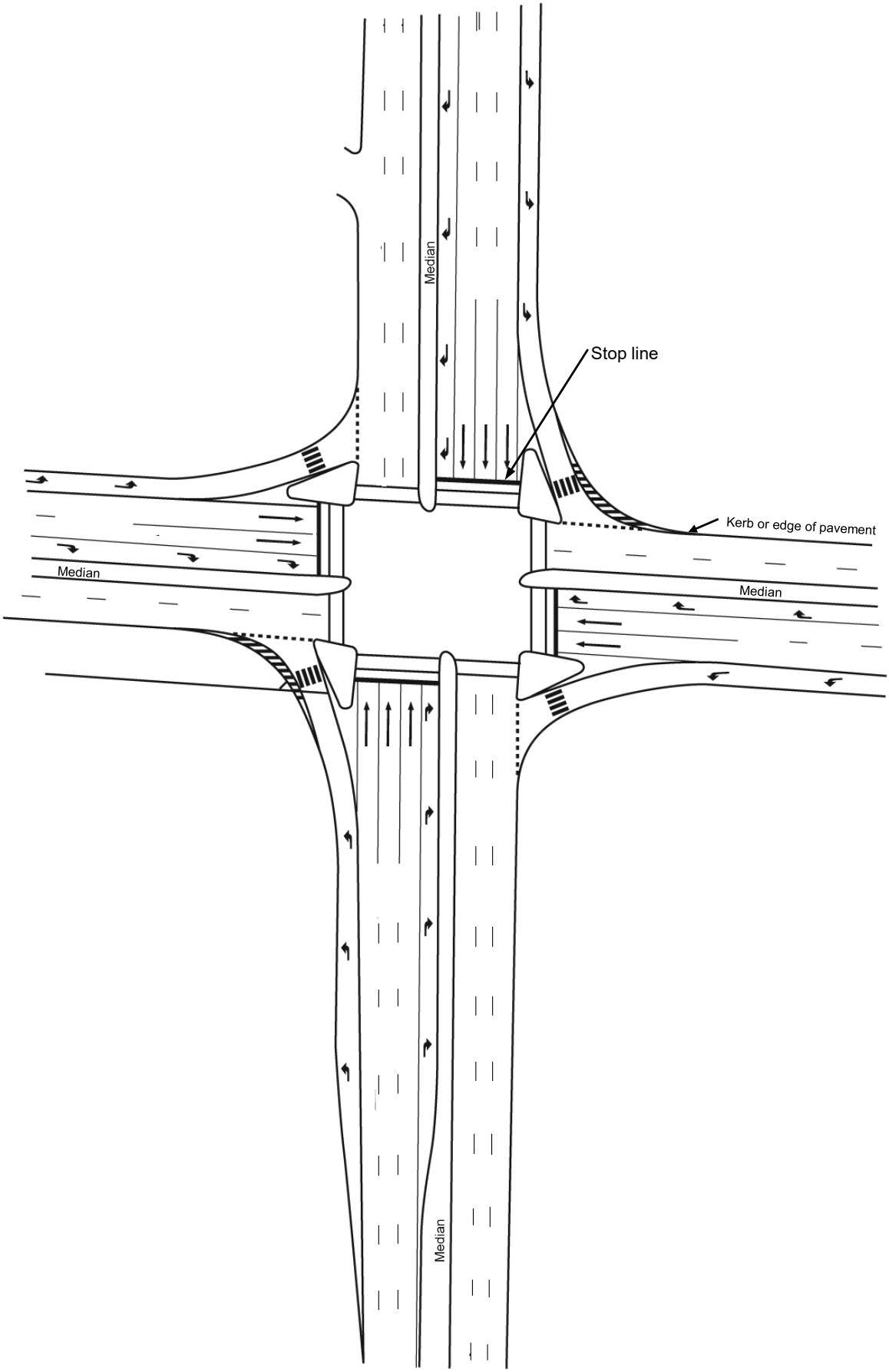
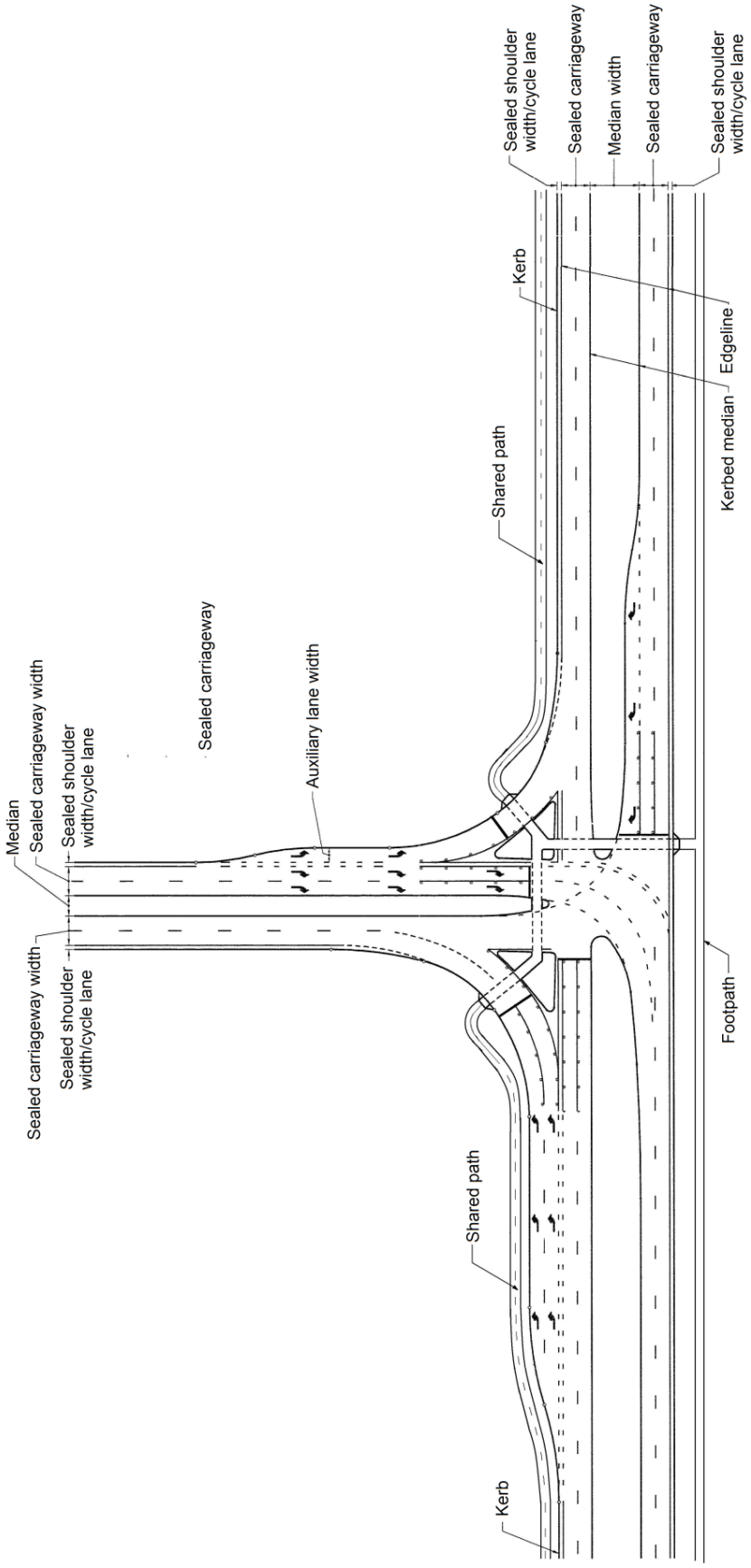


Figure 10.2: An example of a signalised intersection with double left and right-turn lanes, dual carriageway and bicycle facility in both roads

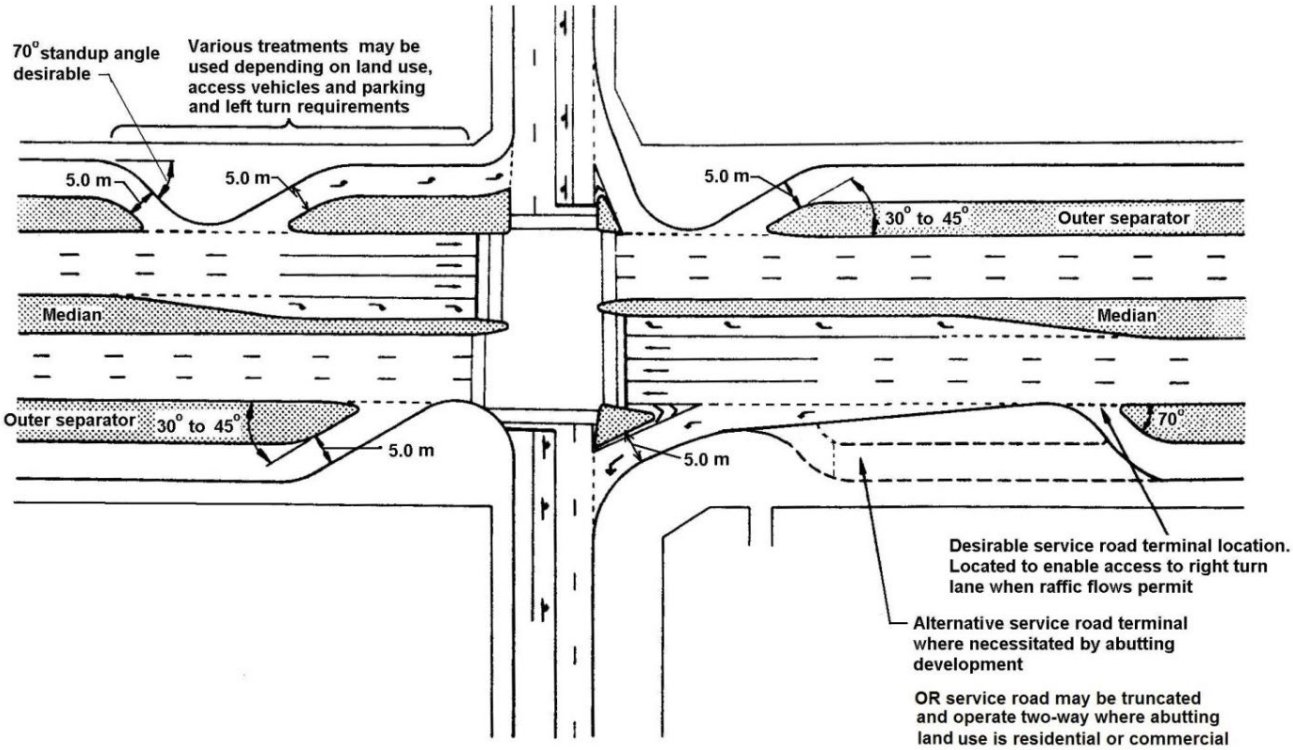


Source: Main Roads Western Australia (2019).

10.4.2 Service Road Treatments

AGTM Part 6 discusses service roads at unsignalised intersections, and the same principles apply to signalised intersections. Figure 10.3 shows examples of service road terminations at a major signalised intersection. It is desirable that the access from the service road is located so that drivers can conveniently move across to the right-turn lane when possible during lower traffic flows on the major road. During peak periods drivers may have to turn left from the service road and seek an alternative route.

Figure 10.3: Typical service road treatments at a signalised intersection



10.5 Traffic Lanes

10.5.1 General

Traffic lanes at signalised intersections generally comprise either through lanes, right-turn lanes or left-turn lanes as discussed in AGTM Part 6, Section 7 and Section 8. In addition U-turn lanes may be accommodated in medians on the approaches to signalised intersections. The U-turn movement can be unsignalised or incorporated into the signal phasing with U-turn signal aspects and lanterns.

10.5.2 U-Turning Lanes

U-turns may be permitted at traffic signals in some jurisdictions subject to local guidelines which may require a U-turn lane and signal phase to be provided.

At sites where a U-turn facility is not to be provided at the signals, a separate U-turn lane could be located a sufficient distance in advance of the signalised intersection to ensure that:

- it is clear to right-turning drivers that it is a U-turn lane and not a right-turn lane
- U-turning drivers have time to perform the manoeuvre without conflicting with left-turning vehicles from the intersecting road.

Where a jurisdiction allows U-turning from right-turn lanes (i.e. on divided roads) it should be recognised that the U-turners do cause delay for the right-turners. This may be an issue, especially if they have to give way to pedestrians or left-turning traffic.

There is often a need to provide for traffic to U-turn from the major road into a service road and this can sometimes be facilitated by a signalised U-turn lane indented into the residual median part way along the right-turn lane/s. Such a treatment is illustrated in Figure 9.3 and an example is shown in Figure 10.4.

Figure 10.4: Example of a U-turn lane combined with a double right-turn lane

10.6 Cyclist Facilities

10.6.1 On-road Bicycle Lanes

General

Signalised intersections are often associated with traffic routes and are therefore utilised by commuter cyclists. Wherever practicable, traffic routes and signalised intersections should provide the space and operational conditions to support cycling as a viable alternative mode of transport.

The needs of cyclists should be considered in relation to detection, signal phasing and timing and road space. Off-road paths are often provided for non-commuter cyclists (e.g. the young and inexperienced cyclists) and these paths often have to be incorporated into the functional layouts of signalised intersections. Traffic management considerations for cyclists at intersections are discussed in the AGTM Part 6.

Illustrations of exclusive right-turn lanes for cyclists are presented in this section. However, these right-turn lanes are rarely used and should generally not be provided for cyclists at right-turn treatments on arterial roads or busy traffic routes because of the difficulty and crash risk for cyclists moving from the left of the road on an intersection approach to the centre of the road in order to utilise such treatments. Exclusive right-turn lanes for cyclists would only be provided where:

- it can be demonstrated that the volume of traffic on an arterial road/traffic route is low enough for cyclists to be able to safely access the cyclist right-turn lane, and there is sufficient cyclist demand to justify the facility
- the speed environment is very low (e.g. 50 km/h) and cyclist demand is significant.

These conditions may exist within the business centres of cities or activity centres and may be associated with particular precincts (e.g. universities or sporting recreational areas).

Bicycle lanes on signalised intersection approaches

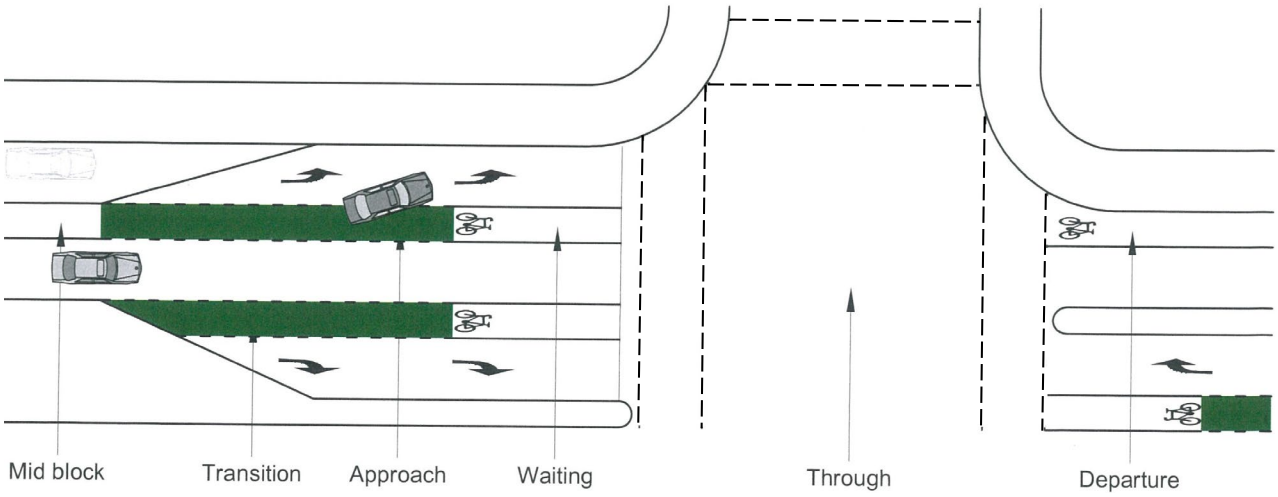
The types of lanes that may have to be incorporated into traffic routes, and therefore signalised intersections, include:

- wide kerbside lanes
- bicycle lanes adjacent to car parking lanes
- exclusive bicycle lanes.

Wide kerbside lanes provide additional width for cyclists to share the left lane with motor vehicles but without any designated space being marked on the road surface. It is desirable that wide kerbside lanes be carried through intersections as a reduction in the width of the left lane at intersections to allocate space for other uses (e.g. provide a painted right-turn lane) creates a squeeze point for cyclists.

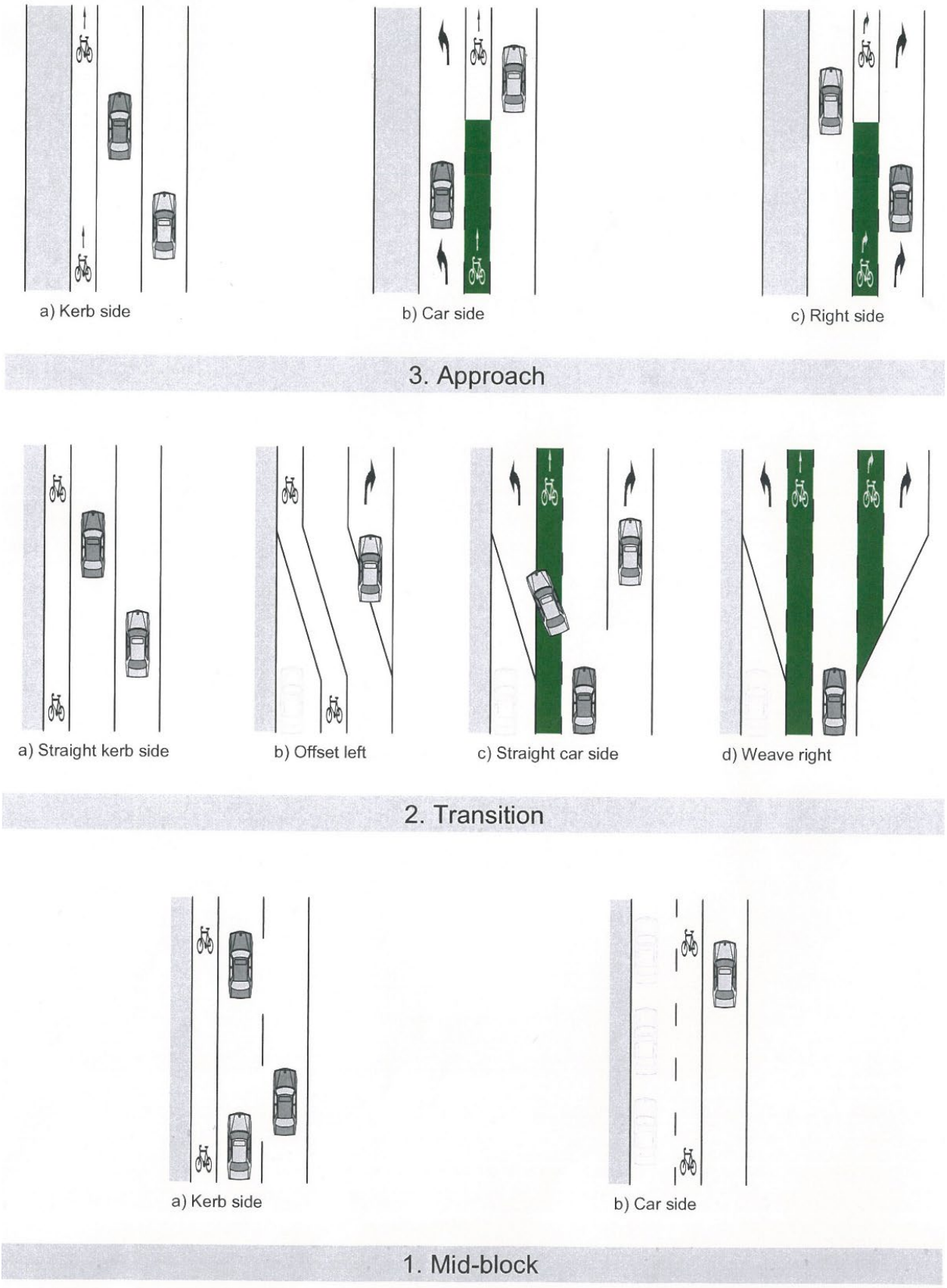
Where a bicycle lane exists or is planned on roads leading up to a signalised intersection the design should assist the safe passage of cyclists through the intersection. Ideally, a visually separated operating space should be provided within each of the six elements illustrated in Figure 10.5. However, where space is constrained and all elements cannot be satisfactorily addressed designers should meet as many of the requirements as possible. Design options for the six elements are shown in Figure 10.6 and Figure 10.7.

Figure 10.5: Provision of a bicycle operating space at intersections – the six elements



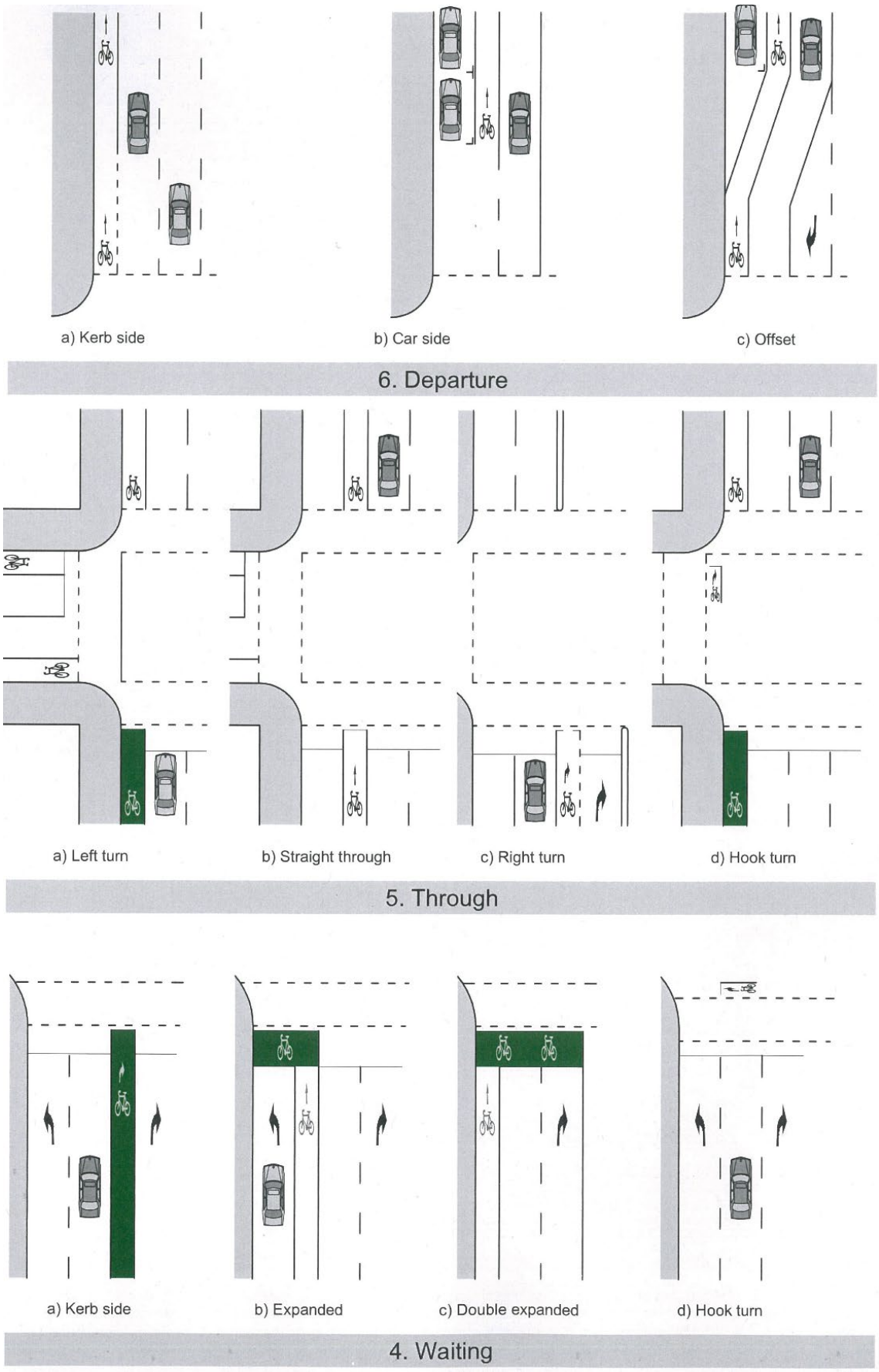
Source: Based on Roads and Traffic Authority (2005).

Figure 10.6: Design options for signalised intersections (mid-block, transition and approach)



Source: Adapted from Roads and Traffic Authority (2005).

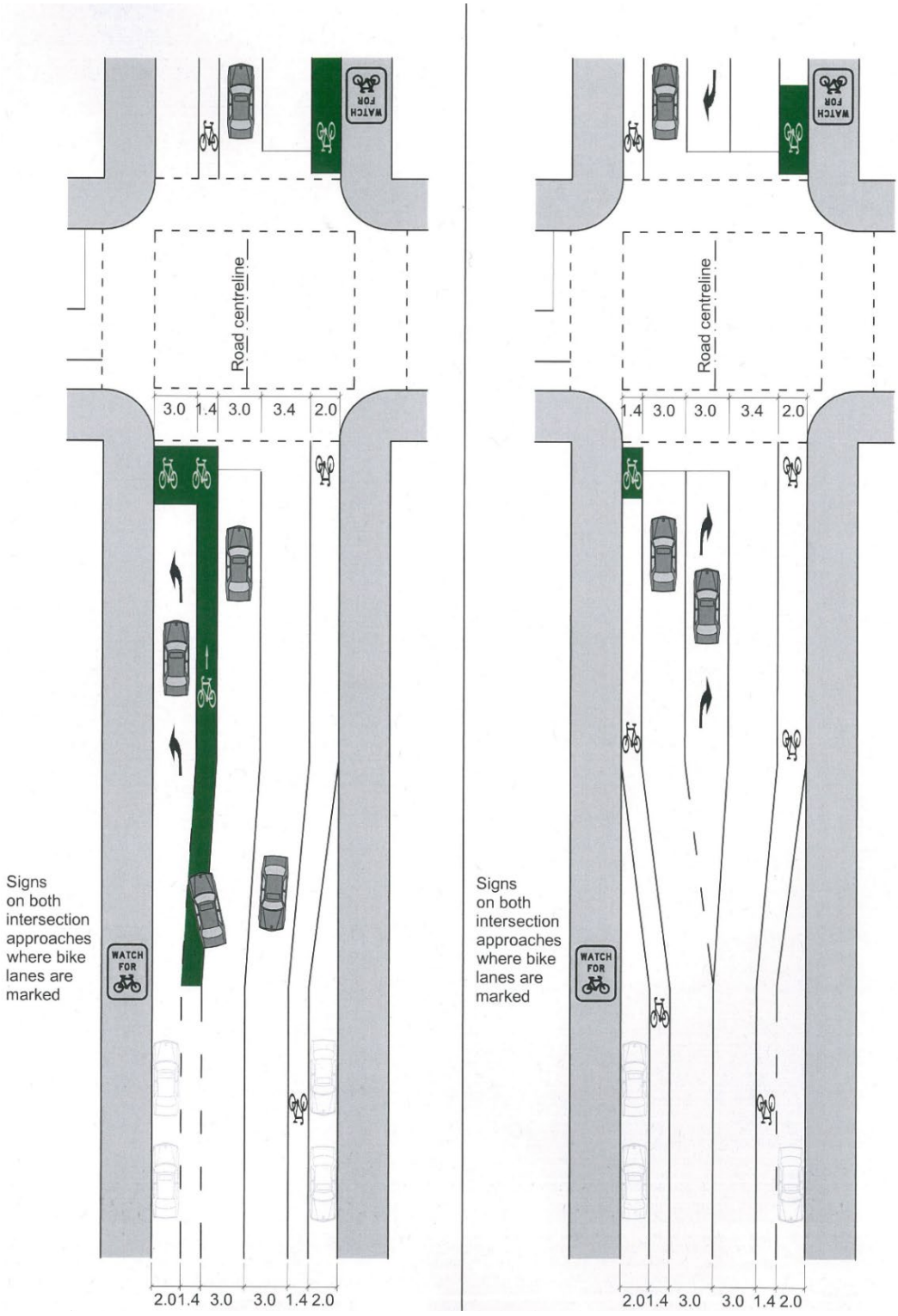
Figure 10.7: Design options for signalised intersections (waiting, through and departure)



Source: Adapted from Roads and Traffic Authority (2005).

Figure 10.8 shows two common examples of bicycle lanes marked through signalised intersections where the width between kerbs is approximately 13 m and parking is provided. In the car side option no separate left-turn lane is provided for cyclists, resulting in them having to make the left-turn from the vehicle lane and the expanded storage box. Cyclists using the kerbside option can wither turn left or proceed straight through the intersection from the bicycle lane.

Figure 10.8: Examples of bicycle lanes through signalised intersection – car side and kerbside



Note: All dimensions are in metres.

Source: Adapted from Roads and Traffic Authority (2005).

Head start and expanded storage areas

These storage areas are provided to position cyclists in a highly visible location while they are waiting to proceed through the intersection thereby improving safety. Figure 10.9 shows four combinations of head start and expanded storage areas at signalised intersections. The treatments in each of the four examples can be used in isolation or in any combination. A summary of the various treatments is given below.

Example (a)

The purpose of this treatment is to store a cyclist in advance of a motor vehicle driver in the adjacent left-turn lane or through lane so that the cyclist can be easily seen by a stationary driver at the stop line. This is particularly important where the vehicle is a van or truck, in which case the cyclist would otherwise be hidden from view below the left side window. This treatment:

- reduces the potential for conflict between cyclists and traffic using the left lane
- is suitable where cyclist numbers are relatively low
- allows cyclists to pass on the left side of a queue of vehicles waiting to turn left
- has an area that is only as wide as the bicycle lane on the approach
- has a bicycle stop line that is located 0.2 m in advance of the pedestrian crosswalk line and 2.0 m (i.e. storage length for one bicycle) beyond the motor vehicle stop line
- may be placed to the left of a left-turn lane, a through lane, or a combined through and left-turn lane.

Example (b)

This treatment locates the bicycle lane between the left-turn lane and through lane and provides additional storage width and length. Cyclists travelling straight ahead travel to the right of queued or moving left-turning vehicles. However, left-turning vehicles are required to change lanes across the bicycle lane at the start of the left-turn lane. Cyclists intending to turn left should desirably share the left-turn lane with motor vehicles. However, it is likely that some left-turning cyclists will use the bicycle lane to pass the queue and access the storage box.

Example (c)

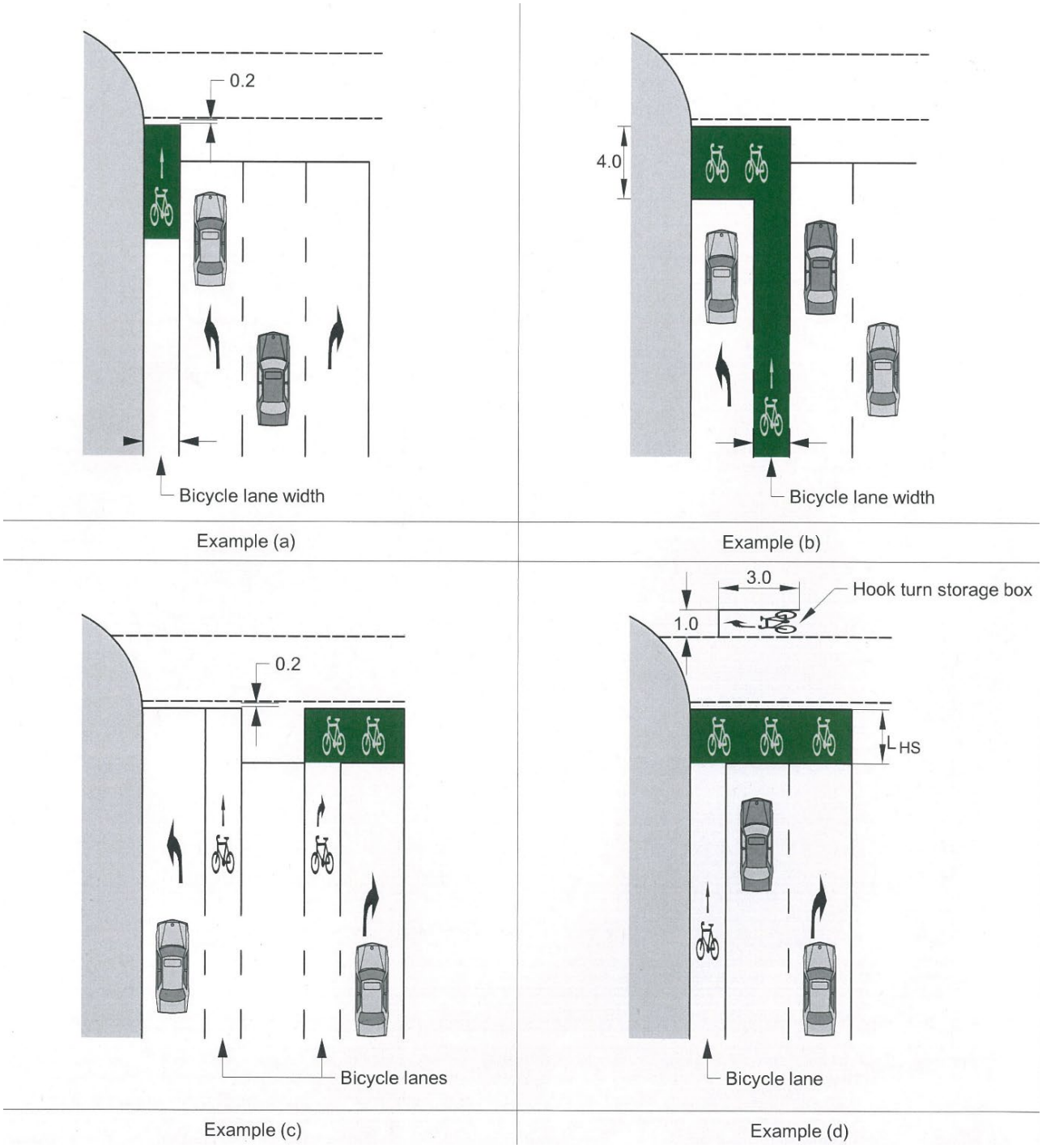
This illustration includes two treatments, the first being a bicycle lane for cyclists travelling straight through the intersection. In this case, left-turning cyclists are expected to share the left-turn lane with motor vehicles. The second treatment is a right-turn expanded storage area for high volumes of bicycle turning traffic. These are rarely used and are not intended for use in higher speed zones (> 60 km/h) because of the difficulty and conflict associated with cyclists crossing traffic lanes to access the right-turn bicycle lane. However, these treatments may be appropriate in lower speed zones (≤ 60 km/h) where bicycle volumes are high and the turn is made into a single lane mixed-function road that does not have marked bicycle lanes (e.g. inner city areas).

Where bicycle lanes are provided in the intersecting road and bicycle turning volumes are not high, it is more acceptable to install a head start storage area only in the right-turn bicycle lane. In this instance it is also necessary to include additional turning lines within the intersection to guide right-turning cyclists and delineate the cyclists' path for drivers.

Example (d)

This example also shows two treatments. The hook turn storage area, provided to accommodate cyclists in a safe position while they are waiting for a green traffic signal phase for the intersecting road, can be used generally throughout the road system. The second treatment, an expanded storage area shared by left-turning, through and right-turning cyclists is suitable only for lower speed areas (e.g. 50 km/h).

Figure 10.9: Head start and expanded storage areas



Note: LHS denotes length of head start area.

Source: Adapted from Roads and Traffic Authority (2005).

In high traffic locations or where the number of bicycle turning movements is significant, or compliance by motor vehicle drivers is poor (i.e. encroachment into the bicycle storage area) it may be necessary to improve the delineation of the storage area by paving it with a green surface. It should be noted that:

- not all jurisdictions use head start areas across multiple lanes, particularly through lanes
- a head start area may be used where there is no bicycle lane on the intersection approach.

The treatment in Figure 10.9 (a) is not suitable for use where a green left-turn arrow is provided on the approach as the treatment encourages cyclists to store at the stop line. Even without the treatment, left-turn phases are problematic for through cyclists waiting in the vicinity of the stop line.

The bicycle lane for the through cyclist movement depicted in Figure 10.9 (c) can remove this conflict and should be used where a left-turn phase is provided.

In practice, many cyclists intending to turn right ride to the left of motor vehicles which are turning or intending to turn right in order to avoid conflict with this traffic stream. This means that they may be exposed to conflict with through traffic. The right-turn bicycle lane shown in Figure 10.9 (c) creates space for cyclists, providing protection from moving motor vehicles and enabling cyclists to easily advance to the head of the right-turning queue.

If the volume of cyclists is high then consideration may be given to providing a larger storage area as shown in Figure 10.9 (b) and (c).

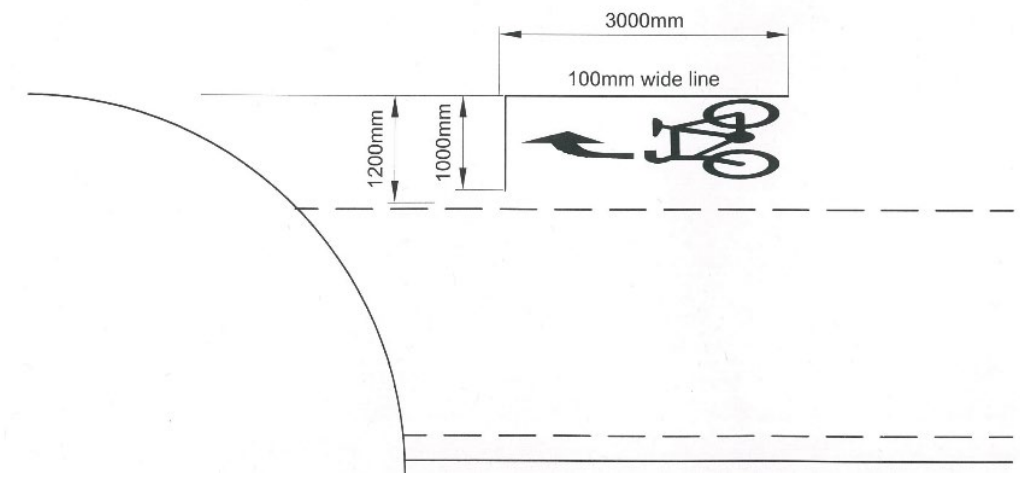
Hook turn storage boxes and hook turn restrictions

Under the *Australian Road Rules* cyclists on an approach at a signalised intersection can use a hook turn as an alternative to a conventional right-turn from the centre of the road. Cyclists undertake a hooked turn by travelling straight at the intersection and waiting at the far corner of the intersecting road. Where the aim is to encourage the use of hooked turns, or to ban a conventional right turn that may be hazardous to cyclists, a hook turn storage box can be provided as illustrated in Figure 10.9 (d) and Figure 10.10.

The hook turn box should not be located as illustrated if the left-turn lane has a signalised left-turn arrow. In this case the box may be placed in front of the adjacent lane if the signal phasing permits. Additional in-ground signal detection in the hook turn box should be considered where the box is placed at a side street approach to a major road to ensure that cyclists can call a phase.

It should be noted, however, that the box turn may be illegal in some states and the traffic signal phasing at some intersections may not suit a hook turn. For instance, waiting cyclists who have performed the first stage of a hook turn manoeuvre could be in conflict with an exclusive left-turn phase for the intersecting road or a diagonal pedestrian crossing phase.

Figure 10.10: Bicycle hook turn box detail



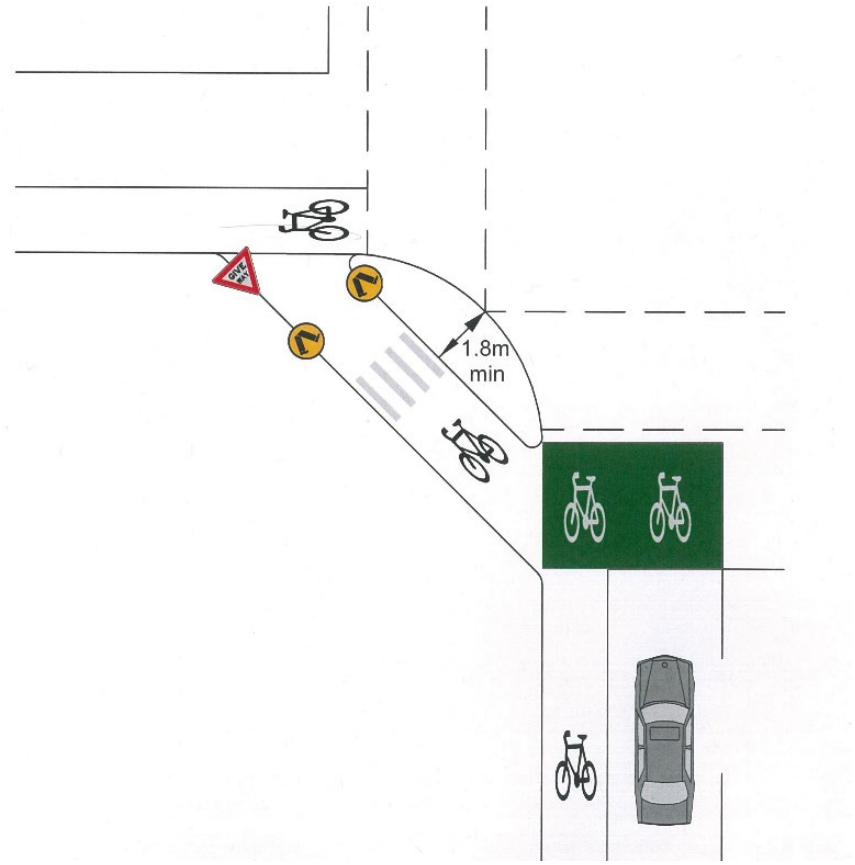
Notes:
In this case the hook turn box is located in the area between the crossing line or vehicle stop line and the crossroad kerb line. The hook turn box should not be located in this position if the left lane has a signalised turn phase. In this case the bay may be located in front of the adjacent traffic lane if signal phasing permits.
Additional in-ground signal detection should be installed in hook turn boxes where the box is located at a side street entrance to a major arterial road with signal priority.

Source: Adapted from Roads and Traffic Authority (2005).

Left-turn bypass treatment

Left-turn access through signals may be provided for cyclists where a major bicycle route turns left through a signalised intersection as shown in Figure 10.11. This treatment has a bicycle lane in the intersecting road. Where there is no bicycle lane in the intersecting road the bypass should be designed as a free-flow arrangement where the bicycle lane is directed into an off-road path parallel to the intersecting road to join it with a protected transition (kerb extension).

Figure 10.11: Bicycle lane left-turn bypass at a signalised intersection



Source: Adapted from Roads and Traffic Authority (2005).

Bypass of T-intersection

In order to limit the delay to cyclists at T-intersections, circumstances may permit the construction of a bypass of the intersection for cyclists opposite the discontinuing leg of an intersection, as shown in Figure 10.12. The bypass may be separated by channelisation as shown or a separated path treatment can be used where the bicycle path is raised and adjacent to the footpath. A disadvantage of the channelised treatment may be the accumulation of debris and the consequent maintenance.

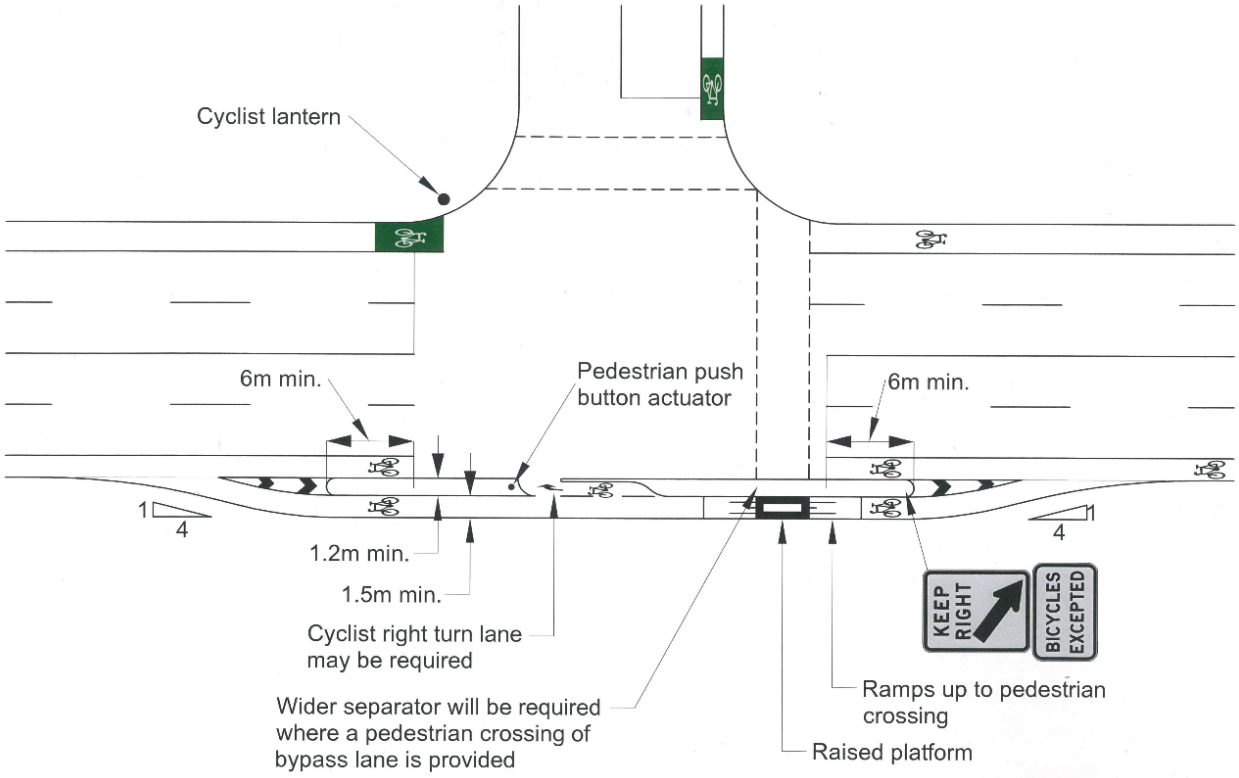
This treatment may be appropriate where:

- property access does not exist opposite the discontinuing leg of the intersection
- pedestrian activity in the vicinity of the intersection is limited and the number of pedestrian crossing movements of the bicycle path is low
- the proportion of mobility and vision-impaired pedestrians is low.

Where the bypass lane/path passes over a pedestrian crossing, the crossing should be designated in a manner that is consistent with local practice and should incorporate one or a combination of:

- warning signs
- zebra crossing (AS 1742.10-2009)
- raised platform
- give-way controls.

Figure 10.12: Cyclist bypass lane at a signalised T-intersection



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Appendix A Extended Design Domain (EDD) for Intersections

A.1 General

This section provides extended design domain (EDD) values for intersection design criteria. These are values outside of the normal design domain (NDD) that through research and/or operating experience, particular road agencies have found to provide a suitable solution in constrained situations (typically at brownfield sites).

EDD may be considered when:

- reviewing the geometry of existing intersections
- new intersections are being retrofitted on existing roads in constrained locations
- improving the standard of existing intersections in constrained locations
- building temporary intersections.

Application of EDD involves identification and documentation of driver 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 driver can reasonably be expected to have when they are taking reasonable care for their own safety). The decision to use EDD should not be taken lightly.

In applying this Part:

- NDD values given in the body of this Part should be used wherever practical.
- Design values outside of NDD are only to be used if approved in writing by the delegated representative from the relevant road agency. The relevant road agency may be a state road agency, municipal council or private road owner.
- If using EDD values, the reduction in standard associated with their use should be appropriate for the prevailing local conditions. Generally, EDD should be used for only one parameter in any application and not be used in combination with any other minimum or EDD value for any related or associated parameters.

Through collective experience it has been accepted for a very long time that the use of minimum values for several parameters at the same location does not constitute good practice and generally leads to an inferior or unsafe design.

Designers should refer to the Austroads *Guide to Road Design Part 1: Objectives of Road Design* (AGRD Part 1) (Austroads 2021a) and Cox and Arndt (2005) for further information on EDD. The following sections deal with EDD for specific road design parameters and situations.

A.2 EDD for Sight Distance at Intersections

A.2.1 Application of EDD for Sight Distance at Intersections

EDD for sight distance at intersections is calculated by the same process as that used for the NDD intersection sight distance. The main difference with EDD is that less conservative values are used for some of the terms (e.g. coefficient of deceleration, observation time), where justified based on an acceptable level of driver capability being provided. The development of EDD for sight distance at intersections has included checks for safety involving the relationship between sight distance and crash rate that was established in Arndt (2004).

EDD 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
- where a new intersection must be installed on an existing road and it is impractical to achieve the normal design domain criteria.

Compliance with EDD for sight distance at intersections

Sight distance at an intersection is deemed to comply with EDD if all of the following conditions are met:

1. EDD approach sight distance (ASD) capability is provided for the base cases given in Table A 1. Appendix A.2.3 'EDD approach sight distance' provides the criteria for determining appropriate EDD approach sight distance capability for the base cases. Application of ASD is always applied on the minor road at unsignalised intersections. It is normally only required on the major road at unsignalised intersections if
   a. sufficient cues are not provided through other means
   b. drivers may be distracted by other features at the intersection
   c. the intersection is complex or non-standard.
2. EDD minimum gap sight distance (MGSD) is applied in accordance with the normal design domain MGSD criteria, except that an object height of 1.25 m is used (top of a passenger car) instead of 0.65 m (indicator height).
3. EDD safe intersection sight distance (SISD) capability is provided for the base cases and any relevant check case given in Table A 1. Section A.2.5 'EDD Safe Intersection Sight Distance' provides the criteria for determining appropriate EDD safe intersection sight distance capability for the base cases and provides guidance for assessing the check cases.
4. EDD stopping sight distance is provided at all locations throughout the intersection. Refer to the criteria in the Austroads *Guide to Road Design Part 3: Geometric Design* (AGRD Part 3) (Austroads 2016a).
5. The following are general considerations for sight distance at intersections
   a. Application of EDD sight distance at intersections is only appropriate when crash data indicates that there are no sight distance related crashes.
   b. Because EDD uses less conservative values, there is less margin for error (although some margin is still provided in the EDD values). Design issues such as choosing the correct operating speed and allowing for the effect of grade become more critical.
   c. Generally, an EDD value should not be combined with any other lower order geometric value for the same element.
   d. 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. Future arrangements/planning must be satisfied (e.g. allow for future fencing, safety barriers).
   f. Geometric and other features of the road should not be misleading and should not distract drivers.
   g. Horizontal curves and vertical curves should not be considered in isolation. Sight distances/lines should be checked in both the vertical and horizontal planes taking into account both the horizontal and vertical curvature.
   h. Particular attention must be given to checking truck requirements on routes with high numbers of heavy vehicles. Some capability for trucks should be provided on all roads.

Formulae

Section 3.2.1 provides the formula for the calculation of ASD.

Section 3.2.2 provides the formula for the calculation of SISD.

Section 5 of AGRD *Part 3* provides the formula 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.

Section 8 of AGRD *Part 3* provides formulae for the calculation of vertical curve radii required to obtain stopping sight distance for a crest or sag curve.

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).

A.2.2 Base and Check Cases

Base cases

For EDD approach sight distance and EDD safe intersection sight distance, it is mandatory to provide sufficient driver capability for cars and trucks during daylight hours (norm-day and truck-day base cases respectively) as shown in Table A 1. These are the same conditions used for the normal design domain.

Table A 1: Case types used for EDD sight distance

Case type	Case code	Case description
Base case (mandatory application)	Norm-day	Normal car driver travelling at the 85th percentile speed in daylight hours
	Truck-day	Truck in daylight hours
Check case (ensure that adequate capability exists under these conditions, as relevant)	Norm-night	Normal car driver travelling at the 85th percentile speed on an unlit road at night
	Truck-night	Truck travelling on an unlit roadway at night
	Mean-day	Car driver travelling at the mean free speed in daylight hours (about 0.85 times the 85th percentile speed)
	Mean-night	Car driver travelling at the mean free speed on an unlit roadway at night (about 0.85 times the 85th percentile speed)
Optional check case to confirm borderline cases	Skill-day	Skilled car driver travelling at the 85th percentile speed in daylight hours
	Skill-night	Skilled car driver travelling at the 85th percentile speed on an unlit roadway at night

Check cases

Because the EDD base cases may well use less conservative values for some of the terms, it becomes important to provide suitable capability for other combinations of driver and lighting conditions. For example, that suitable capability is provided for car and truck drivers at night (norm-night and truck-night check cases in Table A 1) and that suitable capability is provided for drivers travelling at the mean free speed (mean-day and mean-night). The check cases are particularly important for EDD SISD.

Sufficient capability for the night-time check cases will not be needed if the road has continuous route lighting.

Generally, the check cases in Table A 1 are not applied to EDD ASD because of the large sight distances, and associated costs that would result. In addition, Arndt (2004) showed that provision of ASD was not a significant predictor of any crashes at unsignalised intersections. The reason given for this result was that good perception of intersections can often be provided even when ASD fell below the NDD values.

Optional check cases

The optional checks in Table A 1 are used to ascertain whether skilled drivers have sufficient safe intersection sight distance capability. This may be helpful when analysing borderline cases. For example, if the skill-day and skill-night check case capabilities are not available, it would not be a suitable solution under EDD. These optional check cases may even be used when determining if any capability exists under a design exception.

A.2.3 EDD Approach Sight Distance (ASD)

EDD approach sight distance values and corresponding crest curve sizes for the base cases are calculated using the following:

- Eye height (h_1 = 1.1 m for cars and 2.4 m for trucks) and object height (h_2 = 0 m) as per Table A 4 of Appendix A of AGRD Part 3.
- Reaction times (R_T = 1.5 sec / 2.0 sec / 2.5 sec) as per Table A 5 of Appendix A of AGRD Part 3.
- Longitudinal deceleration (d = 0.61 / 0.46 for cars and 0.29 for trucks) as per Table A 6 of Appendix A of AGRD Part 3.

EDD approach sight distance values and corresponding crest curve sizes are given in the following tables:

- Table A 2 for the norm-day base case, which uses a coefficient of deceleration of 0.61, is suitable for sealed roads in predominantly dry areas with an AADT < 4000 veh/d. In order to be classified as a predominantly dry area, the average number of days per year with rainfall greater than 5 mm should be less than 40. Refer to the Bureau of Meteorology website for the amount of rainfall at any particular site.
- Table A 3 for the norm-day base case, which uses a coefficient of deceleration of 0.46, is suitable for sealed roads with normal road conditions i.e. wet roads. This table should be used for the norm-day base case for all roads other than those in the previous dot point.
- Table A 6 for the truck-day base case.

Grade corrections for d = 0.61, d = 0.46 and d = 0.29 are provided in Table A 4, Table A 5 and Table A 7, respectively.

The approach sight distances given in Table A 2, Table A 3 and Table A 6 are based on the sight distance being measured on a horizontal straight or a horizontal curve with a side friction factor less than or equal to the desirable maximum value. If this does not apply, individual calculations are required to determine the approach sight distance.

The vertical crest curve sizes given in Table A 2, Table A 3 and Table A 6 are based on the:

- sight distance being less than the length of the crest curve
- average grade over the braking length being zero
- horizontal alignment at the particular location results in passenger car drivers using a side friction factor less than or equal to the desirable maximum value.

If any of the above does not apply, individual calculations are required to determine the vertical crest curve size.

Generally, the check cases in Table A 1 are not applied to EDD approach sight distance because of the large sight distances, and associated costs that would result.

Table A 2: Minimum EDD approach sight distance and corresponding crest vertical curve size for the norm-day base case for sealed roads with level grades in predominantly dry areas (m)

Design speed (km/h)	Based on the norm-day base case[1] h_1 = 1.1 h_2 = 0 Roads in predominantly dry areas with AADT < 4000 veh/d[2] (d = 0.61)[3]					
	R_T = 1.5 sec		R_T = 2.0 sec		R_T = 2.5 sec	
	ASD (m)	K	ASD (m)	K	ASD (m)	K
40	27	3.3	33	4.8	–	–
50	37	6.2	44	8.8	–	–
60	48	10.6	57	14.5	–	–
70	61	16.8	71	22.6	–	–
80	75	25.3	86	33.4	–	–
90	90	36.6	102	47.5	115	59.9
100	106	51.3	120	65.6	134	81.6
110	124	69.8	139	88.1	154	109
120	–	–	160	116	176	141
130	–	–	181	149	199	181

1 *If the average grade over the braking length is not zero, calculate the approach sight distance values using the correction factors in Table A 4 (or use Equation 1 in Section 3.2.1) by applying the average grade over the braking length.*

2 *In order to be classified as a predominantly dry area, the average number of days per year with rainfall greater than 5 mm should be less than 40. Refer to the Bureau of Meteorology website for the amount of rainfall at any particular site.*

3 *On any horizontal curve with a side friction factor greater than the desirable maximum value for cars, calculate the stopping sight distance with the coefficient of deceleration reduced by 0.05.*

Notes:

Generally, check case capability is not required under EDD approach sight distance.

Combinations of design speed and reaction times not shown in this table are generally not used.

The crest vertical curve sizes are based on the sight distance being less than the length of the crest curve.

Table A 3: Minimum EDD approach sight distance and corresponding crest vertical curve size for the norm-day base case for sealed roads with level grades for normal road conditions (m)

Design speed (km/h)	Based on the norm-day base case[1] h_1 = 1.1 h_2 = 0 Normal road conditions (d = 0.46)[2]					
	R_T = 1.5 sec		R_T = 2.0 sec		R_T = 2.5 sec	
	ASD (m)	K	ASD (m)	K	ASD (m)	K
40	30	4.2	36	5.9	–	–
50	42	8.1	49	11.0	–	–
60	56	14.2	64	18.7	–	–
70	71	23.0	81	29.7	–	–
80	88	35.3	99	44.7	–	–
90	107	51.9	119	64.7	132	79.0
100	127	73.6	141	90.6	155	109
110	149	101	165	123	180	147
120	–	–	190	164	207	194
130	–	–	217	214	235	251

1 *If the average grade over the braking length is not zero, calculate the approach sight distance values using the correction factors in Table A 5 (or use Equation 1 in Section 3.2.1) by applying the average grade over the braking length.*

2 *On any horizontal curve with a side friction factor greater than the desirable maximum value for cars, calculate the stopping sight distance with the coefficient of deceleration reduced by 0.05.*

Notes:

Generally, check case capability is not required under EDD approach sight distance.

Combinations of design speed and reaction times not shown in this table are generally not used.

The crest vertical curve sizes are based on the sight distance being less than the length of the crest curve.

Table A 4: Grade corrections to stopping sight distance for d = 0.61

Design speed (km/h)	Correction (m)							
	Upgrade				Downgrade			
	2	4	6	8	2	4	6	8
40	0	–1	–1	–1	0	1	1	2
50	–1	–1	–1	–2	1	1	2	2
60	–1	–1	–2	–3	1	2	3	4
70	–1	–2	–3	–4	1	2	3	5
80	–1	–3	–4	–5	1	3	5	6
90	–2	–3	–5	–6	2	4	6	8
100	–2	–4	–6	–7	2	5	7	10
110	–2	–5	–7	–9	3	5	9	12
120	–3	–6	–8	–11	3	7	10	14
130	–3	–7	–10	–13	4	8	12	16

Table A 5: Grade corrections to stopping sight distance for *d* = 0.46

Design speed (km/h)	Correction (m)							
	Upgrade				Downgrade			
	2	4	6	8	2	4	6	8
40	–1	–1	–2	–2	1	1	2	3
60	–1	–2	–4	–5	1	3	5	6
70	–2	–3	–5	–6	2	4	6	9
80	–2	–4	–6	–8	2	5	8	12
90	–3	–6	–8	–10	3	7	10	15
100	–4	–7	–10	–13	4	8	13	18
110	–4	–8	–12	–15	5	10	16	22
120	–5	–10	–14	–18	6	12	18	26
130	–6	–12	–17	–21	7	14	22	30

Table A 6: Minimum EDD approach sight distance and corresponding crest vertical curve size for the truck-day base case for sealed roads with level grades (m)

Design speed (km/h)	Based on the truck-day base case (1) h_1 = 2.4 h_2 = 0 d = 0.29(2)					
	R_T = 1.5 sec		R_T = 2.0 sec		R_T = 2.5 sec	
	ASD (m)	K	ASD (m)	K	ASD (m)	K
40	38	3.1	44	4.0	–	–
50	55	6.3	62	7.9	–	–
60	74	11.4	82	14.1	–	–
70	96	19.1	105	23.1	–	–
80	120	30.1	131	35.9	–	–
90	147	45.3	160	53.3	172	62.0
100	177	65.6	191	76.3	205	87.7
110	210	92.0	225	106	241	121

1 *If the average grade over the braking length is not zero, calculate the approach sight distance values using the correction factors in Table A 7 (or use Equation 1 in Section 3.2.1) by applying the average grade over the braking length.*

2 *On any horizontal curve with a side friction factor greater than the desirable maximum value for trucks, calculate the stopping sight distance with the coefficient of deceleration reduced by 0.05.*

Notes:

Generally, check case capability is not required under EDD approach sight distance.

Combinations of design speed and reaction times not shown in this table are generally not used.

The crest vertical curve sizes are based on the sight distance being less than the length of the crest curve.

Table A 7: Grade corrections to stopping sight distance for d = 0.29

Design speed (km/h)	Correction (m)							
	Upgrade				Downgrade			
	2	4	6	8	2	4	6	8
40	–1	–3	–4	–5	2	3	6	8
50	–2	–4	–6	–7	3	5	9	13
60	–3	–6	–8	–11	4	8	13	19
70	–4	–8	–11	–14	5	11	17	25
80	–6	–11	–15	–19	6	14	23	33
90	–7	–13	–19	–24	8	18	29	42
100	–9	–16	–23	–29	10	22	35	52
110	–11	–20	–28	–36	12	26	43	63

A.2.4 Observation Times for EDD Safe Intersection Sight Distance

The observation times used for the base cases under EDD safe intersection sight distance are given in Table A 8. A range of values are given depending on factors such as the complexity of the intersection and traffic volume. The observation times used for the check cases are given in the note to Table A 8.

Table A 8: Driver observation time for safe intersection sight distance under EDD

Observation time *OT* (sec)	Typical use
1.5	T-intersections on single carriageway roads (two-lane, two-way roads and one-way roads) that have a traffic volume < 4000 veh/d Cross intersections on single carriageway roads (two-lane, two-way roads and one-way roads) that have a traffic volume < 400 veh/d Simple intersection arrangements e.g. left in, left out on all roads
2.0	T-intersections on single carriageway roads (two-lane, two-way roads and one-way roads) that have a traffic volume ≥ 4000 veh/d Cross intersections on single carriageway roads (two-lane, two-way roads and one-way roads) that have a traffic volume ≥ 400 veh/d
2.5	T-intersections and cross intersections on multi-lane roads Intersections in overtaking lanes Complex intersection layouts Situations in which drivers may be distracted by other features

Note: The observation times in this table are applicable to the norm-day and truck-day base cases. The minimum observation times for the check cases are given below:

- *mean-day and skill-day as per this table*
- *norm-night, truck-night, mean-night and skill-night use 1.0 sec less than the values given in this table. Use of the lower observation times is associated with the additional cues drivers are given by observing the glow of the oncoming vehicle headlights.*

A.2.5 EDD Safe Intersection Sight Distance (SISD)

EDD safe intersection sight distance values and corresponding crest curve sizes for the base and check cases are calculated using the following:

- Eye height and object height as per Table A 4 of Appendix A of AGRD Part 3 – refer to Note 3 for the check cases
  - h_1 = 1.1 m and h_2 = 1.25 m for norm-day and mean-day
  - h_1 = 2.4 m and h_2 = 1.25 m for truck-day
  - h_1 = 0.65 m and h_2 = 1.25 m for norm-night and mean-night
  - h_1 = 2.4 m and h_2 = 0.8 m for truck-night (minimum acceptable).
- Reaction times as per Table A 5 of Appendix A of AGRD Part 3 – refer to Note 5 for the check cases
  - R_T = 1.5 sec/2.0 sec/2.5 sec for norm-day, truck-day, norm-night and truck-night
  - R_T = 2.0 sec/2.5 sec for mean-day and mean-night.
- Longitudinal deceleration as per Table A 6 of Appendix A of AGRD Part 3 – refer to Note 3 for the check cases. Dry weather only stopping is not used under SISD because the primary hazard at intersections is other vehicles, which are prevalent in wet as well as dry conditions
  - d = 0.46 for norm-day and norm-night
  - d = 0.29 for truck-day and truck-night
  - d = 0.41 for mean-day and mean-night.
- Observation times as per Table A 8 – refer to the note for the check cases.

Application of the base cases

EDD safe intersection sight distance values and corresponding crest curve sizes for the base cases are given in the following tables:

- Table A 9, Table A 10 and Table A 11 for the norm-day base case, which use observation times of 1.5 sec, 2.0 sec and 2.5 sec, respectively.
- Table A 12, Table A 13 and Table A 14 for the truck-day base case, which use observation times of 1.5 sec, 2.0 sec and 2.5 sec, respectively.

Grade corrections for d = 0.46 and d = 0.29 are provided in Table A 5 and Table A 7, respectively.

The safe intersection sight distances given in Table A 9 to Table A 14 are based on the sight distance being measured on a horizontal straight or a horizontal curve with a side friction factor less than or equal to the desirable maximum value. If this does not apply, individual calculations are required to determine the safe intersection sight distance.

The vertical crest curve sizes given in Table A 9 to Table A 14 are based on the:

- sight distance being less than the length of the crest curve
- average grade over the braking length being zero
- horizontal alignment at the particular location results in car drivers using a side friction factor less than or equal to the desirable maximum value.

If any of the above does not apply, individual calculations are required to determine the vertical crest curve size.

Application of the check cases

Appropriate check case capability is obtained when the minimum observation times listed in the note to Table A 8 are met.

When evaluating sight distance in the horizontal plane only (i.e. where there are no restrictions to visibility in the vertical plane), the provision of:

- Norm-day base case SISD values given in Table A 9 to Table A 11 will usually provide satisfactory sight distance capability for the norm-night, mean-day and mean-night check cases.
- Truck-day base case SISD values given in Table A 12 to Table A 14 will provide satisfactory sight distance capability for the truck-night check case.

Table A 9 to Table A 14 show that suitable check case capability has been provided for the crest curve sizes listed. Particular combinations of speeds and reaction times in the tables will produce greater check case capability than that shown.

Individual calculations are required to determine the crest curve sizes for the check cases where the:

- sight distance is greater than the length of the crest curve
- average grade over the braking length is not zero
- horizontal alignment at the particular location results in car drivers using a side friction factor greater than or equal to the desirable maximum value.

Table A 9: Minimum EDD safe intersection sight distance and corresponding crest vertical curve size for sealed roads with level grades for the norm-day base case using an observation time of 1.5 sec

Design speed (km/h)		Based on norm-day safe intersection sight distance[1] h_1 = 1.1 h_2 = 1.25 d = 0.46[2] O_T = 1.5 sec					
		R_T = 1.5 sec		R_T = 2.0 sec		R_T = 2.5 sec	
		SISD (m)	K	SISD (m)	K	SISD (m)	K
40		47	2.4	53	2.9	–	–
50		63	4.2	70	5.2	–	–
60		81	7.0	89	8.5	–	–
70		100	10.7	110	12.9	–	–
80		121	15.7	133	18.7	–	–
90		144	22.2	157	26.2	169	30.5
100		169	30.4	183	35.6	197	41.2
110		195	40.6	211	47.2	226	54.3
120		–	–	240	61.3	257	70.1
130		–	–	271	78.2	289	89.0
Do all of the crest curve sizes listed provide acceptable car check case capability[3]	Norm-night[4]	Yes (d = 0.46, h_1 = 0.65 m, h_2 = 1.25 m, O_T = 0.6 sec)					
	Mean-day	Yes (d = 0.41, h_1 = 1.1 m, h_2 = 1.25 m, O_T = 1.7 sec)					
	Mean-night[4]	Yes (d = 0.41, h_1 = 0.65 m, h_2 = 1.25 m, O_T = 1.2 sec)					

1 *If the average grade over the braking length is not zero, calculate the safe intersection sight distance values using the correction factors in Table A 5 (or use Equation 2 in Section 3.2.2) by applying the average grade over the braking length.*

2 *On any horizontal curve with a side friction factor greater than the desirable maximum value for cars, calculate the stopping sight distance with the coefficient of deceleration reduced by 0.05.*

3 *This part of the table identifies whether the crest curve sizes listed provide acceptable check case capability in accordance with Section A.2.5, subsection 'Application of the check cases'. The minimum capabilities listed for the check cases assume the same combination of design speeds and reaction times as those listed in the table, except:*

- *where particular check cases use a different speed according to Table A 1*
- *where particular check cases use a different reaction time according to Note 5 of Table A 5 of Appendix A of* AGRD Part 3.

4 *Drivers will usually be alerted by the glow from the other vehicle's headlights before seeing the vehicle.*

Notes:

Combinations of design speed and reaction times not shown in this table are generally not used.

The crest vertical curve sizes are based on the sight distance being less than the length of the crest curve.

Table A 10: Minimum EDD safe intersection sight distance and corresponding crest vertical curve size for sealed roads with level grades for the norm-day base case using an observation time of 2.0 sec

Design speed (km/h)		Based on norm-day safe intersection sight distance[1] h_1 = 1.1 h_2 = 1.25 d = 0.46[2] O_T = 2.0 sec					
		R_T = 1.5 sec		R_T = 2.0 sec		R_T = 2.5 sec	
		SISD (m)	K	SISD (m)	K	SISD (m)	K
40		53	2.9	58	3.6	–	–
50		70	5.2	77	6.3	–	–
60		89	8.5	97	10.1	–	–
70		110	12.9	120	15.3	–	–
80		133	18.7	144	22.0	–	–
90		157	26.2	169	30.5	182	35.2
100		183	35.6	197	41.2	211	47.2
110		111	47.2	226	54.3	241	61.9
120		–	–	257	70.1	273	79.5
130		–	–	289	89.0	307	101
Do all of the crest curve sizes listed provide acceptable car check case capability[3]	Norm-night[4]	Yes (d = 0.46, h_1 = 0.65 m, h_2 = 1.25 m, O_T = 1.1 sec)					
	Mean-day	Yes (d = 0.41, h_1 = 1.1 m, h_2 = 1.25 m, O_T = 2.8 sec)					
	Mean-night[4]	Yes (d = 0.41, h_1 = 0.65 m, h_2 = 1.25 m, O_T = 1.8 sec)					

Notes:
1, 2, 3, 4 – refer to notes 1–4, respectively, in Table A 9.
Also refer to the general notes for Table A 9.

Table A 11: Minimum EDD safe intersection sight distance and corresponding crest vertical curve size for sealed roads with level grades for the norm-day base case using an observation time of 2.5 sec

Design speed (km/h)		Based on norm-day safe intersection sight distance[1] h_1 = 1.1 h_2 = 1.25 d = 0.46[2] O_T = 2.5 sec					
		R_T = 1.5 sec		R_T = 2.0 sec		R_T = 2.5 sec	
		SISD (m)	K	SISD (m)	K	SISD (m)	K
40		58	3.6	64	4.3	-	-
50		77	6.3	84	7.5	-	-
60		97	10.1	106	11.9	-	-
70		120	15.3	129	17.8	–	–
80		144	22.0	155	25.5	–	–
90		169	30.5	182	35.2	194	40.2
100		197	41.2	211	47.2	224	53.7
110		226	54.3	241	61.9	256	70.0
120		–	–	273	79.5	290	89.5
130		–	–	307	101	325	113
Do all of the crest curve sizes listed provide acceptable car check case capability [3]	Norm-night[4]	Yes (d = 0.46, h_1 = 0.65 m, h_2 = 1.25 m, O_T = 1.5 sec).					
	Mean-day	Yes (d = 0.41, h_1 = 1.1 m, h_2 = 1.25 m, O_T = 2.9 sec).					
	Mean-night[4]	Yes (d = 0.41, h_1 = 0.65 m, h_2 = 1.25 m, O_T = 2.3 sec)					

Notes:
1, 2, 3, 4 – refer to notes 1–4, respectively, in Table A 9.
Also refer to the general notes for Table A 9.

Table A 12: Minimum EDD safe intersection sight distance and corresponding crest vertical curve size for sealed roads with level grades for the truck-day base case using an observation time of 1.5 sec

Design speed (km/h)	Based on truck-day safe intersection sight distance[1] h_1 = 2.4 h_2 = 1.25 d = 0.29[2] O_T = 1.5 sec					
	R_T = 1.5 sec		R_T = 2.0 sec		R_T = 2.5 sec	
	SISD (m)	K	SISD (m)	K	SISD (m)	K
40	55	2.1	61	2.6	–	–
50	76	4.0	83	4.8	–	–
60	99	6.9	107	8.1	–	–
70	125	11.0	135	12.7	–	–
80	154	16.6	165	19.1	–	–
90	185	24.0	197	27.4	210	31.0
100	219	33.7	233	38.1	247	42.8
110	256	46.0	271	51.7	286	57.7
Do all of the crest curve sizes listed provide acceptable truck-night check case capability[3][4]	Yes (d = 0.29, h_1 = 2.4 m, h_2 = 0.8 m, O_T = 0.7 sec)					

1 *If the average grade over the braking length is not zero, calculate the safe intersection sight distance values using the correction factors Table A 7 (or use Equation 2 in Section 3.2.2) by applying the average grade over the braking length.*

2 *On any horizontal curve with a side friction factor greater than the desirable maximum value for trucks, calculate the stopping sight distance with the coefficient of deceleration reduced by 0.05.*

3 *This part of the table identifies whether the crest curve sizes listed provide acceptable check case capability in accordance with Section A.2.5, subsection 'Application of the check cases'. The minimum capabilities listed for the check case assumes the same combination of design speeds and reaction times as those listed in the table.*

4 *Drivers will usually be alerted by the glow from the other vehicle's headlights before seeing the vehicle.*

Notes:

Combinations of design speed and reaction times not shown in this table are generally not used.

The crest vertical curve sizes are based on the sight distance being less than the length of the crest curve.

Table A 13: Minimum EDD safe intersection sight distance and corresponding crest vertical curve size for sealed roads with level grades for the truck-day base case using an observation time of 2.0 sec

Design speed (km/h)	Based on truck-day safe intersection sight distance[1] h_1 = 2.4 h_2 = 1.25 d = 0.29[2] O_T = 2.0 sec					
	R_T = 1.5 sec		R_T = 2.0 sec		R_T = 2.5 sec	
	SISD (m)	K	SISD (m)	K	SISD (m)	K
40	61	2.6	66	3.1	–	–
50	83	4.8	89	5.6	–	–
60	107	8.1	116	9.4	–	–
70	135	12.7	144	14.6	–	–
80	165	19.1	176	21.7	–	–
90	197	27.4	210	31.0	222	34.8
100	233	38.1	247	42.8	261	47.8
110	271	51.7	286	57.7	302	64.0
Do all of the crest curve sizes listed provide acceptable truck-night check case capability[3] [4]	Yes (d = 0.29, h_1 = 2.4 m, h_2 = 0.8 m, O_T = 1.1 sec)					

Notes:
1, 2, 3, 4 – refer to notes 1–4, respectively, in Table A 12.
Also refer to the general notes for Table A 12.

Table A 14: Minimum EDD safe intersection sight distance and corresponding crest vertical curve size for sealed roads with level grades for the truck-day base case using an observation time of 2.5 sec

Design speed (km/h)	Based on truck-day safe intersection sight distance[1] h_1 = 2.4 h_2 = 1.25 d = 0.29[2] O_T = 2.5 sec					
	R_T = 1.5 sec		R_T = 2.0 sec		R_T = 2.5 sec	
	SISD (m)	K	SISD (m)	K	SISD (m)	K
40	66	3.1	72	3.6	–	–
50	89	5.6	96	6.5	–	–
60	116	9.4	124	10.8	–	–
70	144	14.6	154	16.7	–	–
80	176	21.7	187	24.5	–	–
90	210	31.0	222	34.8	235	38.8
100	247	42.8	261	47.8	275	53.0
110	286	57.7	302	64.0	317	70.6
Do all of the crest curve sizes listed provide acceptable truck-night check case capability[3] [4]	Yes (d = 0.29, h_1 = 2.4 m, h_2 = 0.8 m, O_T = 1.6 sec)					

Notes:
1, 2, 3, 4 – refer to notes 1–4, respectively, in Table A 12.
Also refer to the general notes for Table A 12.

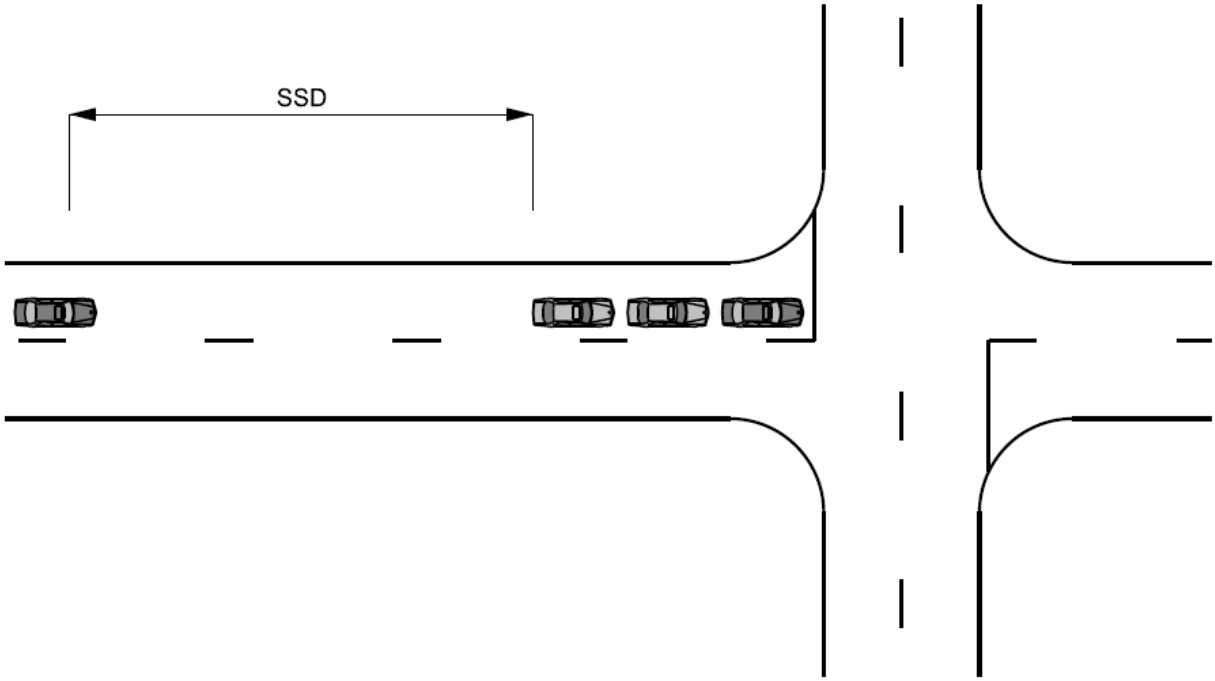
A.2.6 Sight Distance at Constrained Urban Intersections

The desirable criteria for signalised and unsignalised intersections are ASD and SISD (refer to Section 3.2.1 and Section 3.2.2, respectively). However, at some urban intersections where provision of ASD is not possible due to severe restrictions on sight lines it is necessary to ensure that stopping sight distance (SSD) can be provided to the tail-lights of cars that may queue on the approach.

The stopping sight distance should be available to any point in the queue up to the back of the 95th percentile peak hour queue (Figure A 1) for:

- an approaching car, SSD measured from car driver eye height to tail-light (1.1 m to 0.8 m)
- an approaching truck measured from truck driver eye height to tail-light (2.4 m to 0.8 m).

Figure A 1: Sight distance to back of the queue



A.3 EDD for Sight Distance at Domestic Accesses

A.3.1 Application of EDD for Sight Distance at Domestic Accesses

EDD for sight distance at domestic accesses is calculated by the same process as that used for the NDD access sight distance. The main difference with EDD is that less conservative values are used for some of the terms where justified based on an acceptable level of driver capability being provided for entering and exiting drivers.

A domestic access is one that services three or fewer domestic units.

EDD 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
- where a new access must be installed on an existing road and it is impractical to achieve the normal design domain criteria.

Under EDD for sight distance at domestic accesses, it is necessary to provide sufficient driver capability for cars and trucks stopping during daylight hours (norm-day and truck-day base cases, respectively) as shown in Table A 1 (Section A.2.1 of this Appendix). These are the same conditions used for the NDD. In addition to these base cases, designers should ensure that adequate capability exists for any check case listed in Table A 1 that is deemed relevant. For example, sufficient capability for the night-time check cases will not be needed if the road has continuous route lighting. For borderline cases, the optional checks in Table A 1 may be used.

EDD sight distance at domestic accesses requires mandatory application of the following:

- EDD MGSD – apply in accordance with the NDD MGSD criteria, except use an object height of 1.25 m (top of a passenger car) instead of 0.65 m (indicator height).
- EDD SISD – use an observation time (O_T) of 0.5 sec less than the values given in Table A 8.
- EDD SSD – at all locations on a roadway, stopping sight distance to hazards is required. Refer to the criteria in Section A.3 of AGRD Part 3.

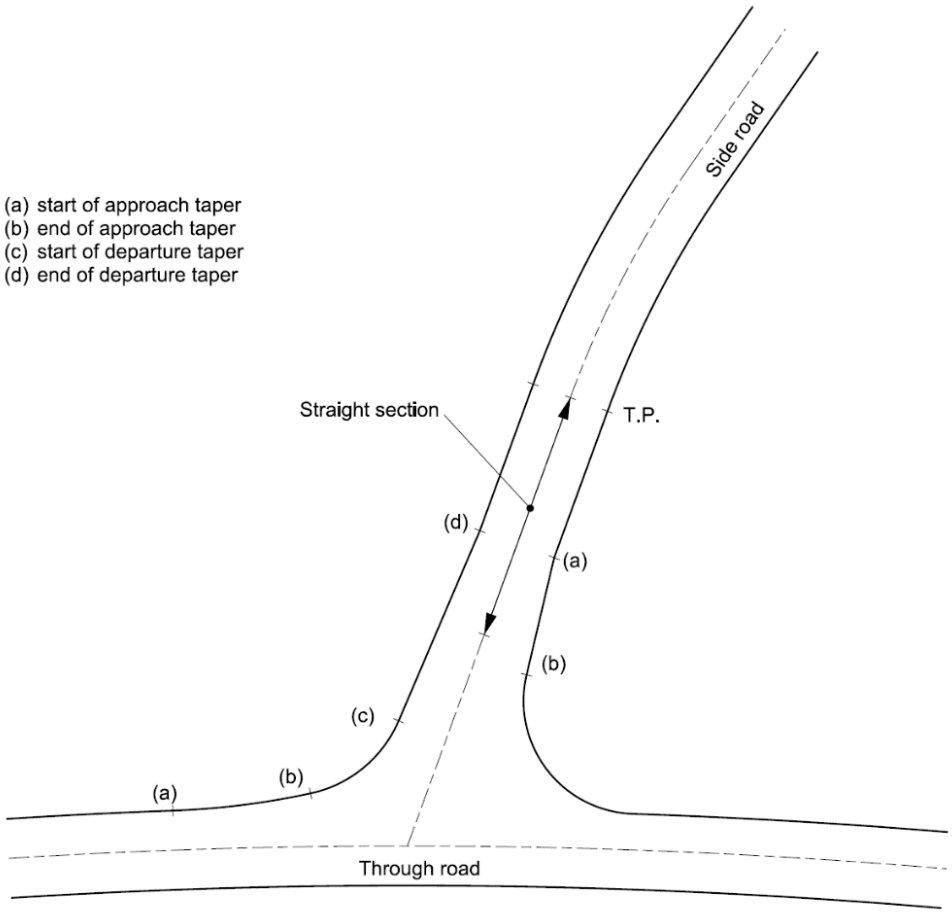
Normally, the provision of ASD at domestic accesses is not necessary due to the familiarity of their location by users.

Application of EDD sight distance at domestic accesses is only applicable when crash data indicates that there are no sight distance-related crashes.

A.4 EDD for Minor Road Approaches

The minimum alignment treatment is shown in Figure A 2.

Figure A 2: Minimum minor road approach alignment at rural sites



A.5 EDD for Intersection Turn Treatments

A.5.1 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 basic right-turn (BAR), auxiliary right-turn (AUR) and basic left-turn (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.

A.5.2 Use of EDD Turn Treatments

This section presents EDD dimensions for CHR and AUL turn treatments that are smaller than the minimums used for the NDD (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 auxiliary right-turn (AUR) treatment to form a CHR turn treatment). The EDD 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 section 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 turnslot with dimensions as per the NDD.

A.5.3 General Considerations

The use of the EDD turn treatments can only be justified provided they meet the following conditions:

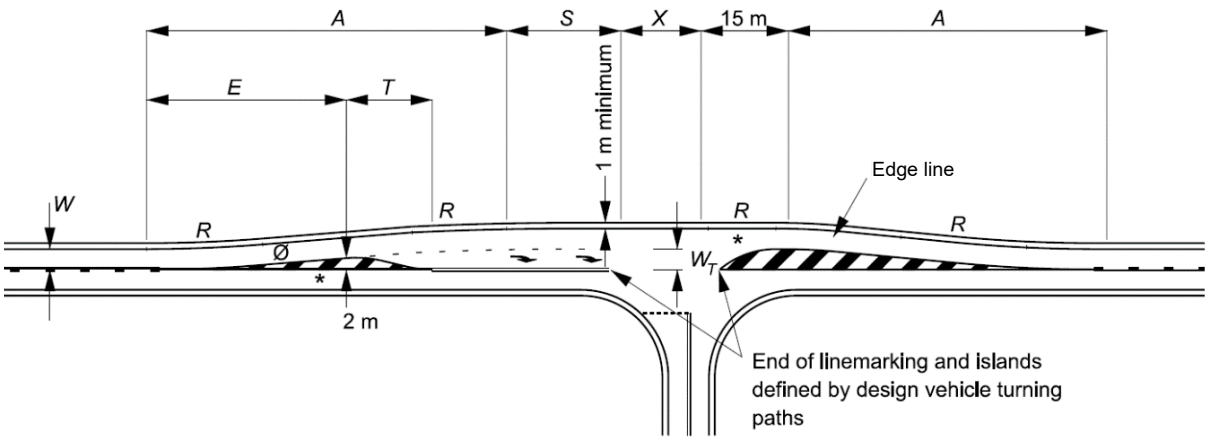
- They are not combined with other minima such as
  - very tight horizontal curves (e.g. horizontal curves with a side friction demand near or greater than the absolute maximum)
  - reduced visibility to the treatment (e.g. smaller to moderate size crest curves)
  - a major road on a steep downgrade.
- 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 crash data indicates that there is not a high crash rate related to the use of the shorter dimensions e.g. not a high rear-end crash rate at the start of the turn lanes.
- 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.

A.5.4 Minimum EDD Channelised Right-turn Treatment for Two-lane Two-way Roadways without Medians

Figure A 3 shows a minimum EDD 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 A 3 so that the resulting alignment of the through lane means that a minimal decrease in speed is required for through drivers. To determine whether a minimal decrease in speed is achieved, draw vehicle paths along the through road for the proposed layout and use the operating speed model to calculate the operating speed on each segment. Table A 15 provides dimensions for various terms listed in Figure A 3.

Figure A 3: Minimum extended design domain channelised right-turn treatment for two-lane, two-way roadways without medians



Notes:

- *Ø – Double barrier line should not be used this side of the island.*
- **– Diagonal rows of RRPMs within the painted islands should be used to delineate chevrons.*
- *Diagram shown for a rural intersection layout. The dimensions shown are also suitable for an urban intersection layout which may include bicycle lanes and parking.*
- *The dimensions of the treatment are defined as follows. Values of A, R and T are provided in Table A 15.*

  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 = 2.8 m minimum.*

  E = *Distance from start of taper to 2.0 m width (m)* = $2\left(\frac{A}{W_T}\right)$

  S = *Storage length (m) is the greater of:*
  1. *the length of one design turning vehicle*
  2. *(calculated car spaces –1) x 8 m (Guide to Traffic Management Part 3: Traffic Studies and Analysis (Austroads 2017)), or use computer program e.g. aaSIDRA.*

  T = *Physical taper length given by:* $T = \frac{0.2VW_T}{3.6}$

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

  X = *Distance based on design vehicle turning path, refer to Design Vehicles and Turning Path Templates Guides (Austroads 2023a).*

Source: Department of Main Roads (2006).

Table A 15: Dimensions relating to Figure A 3

Design speed of major road approach (km/h)	Minimum lateral movement length *A* (m)(1)	Desirable radius *R* (m)	Taper length, *T*(2) (m)
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 1.25 m/sec and a turn lane width of 3.0 m. Increase lateral movement length if turn lane width > 3 m. If the through road is on a tight horizontal curve (e.g. one with a side friction demand greater than the maximum desirable) increase lateral movement length so that a minimal decrease in speed is required for the through movement.*
2 *Based on-turn lane width of 3 m.*
3 *Where Type 2 road trains are required, minimum A = 60 m.*

Source: Department of Main Roads (2006).

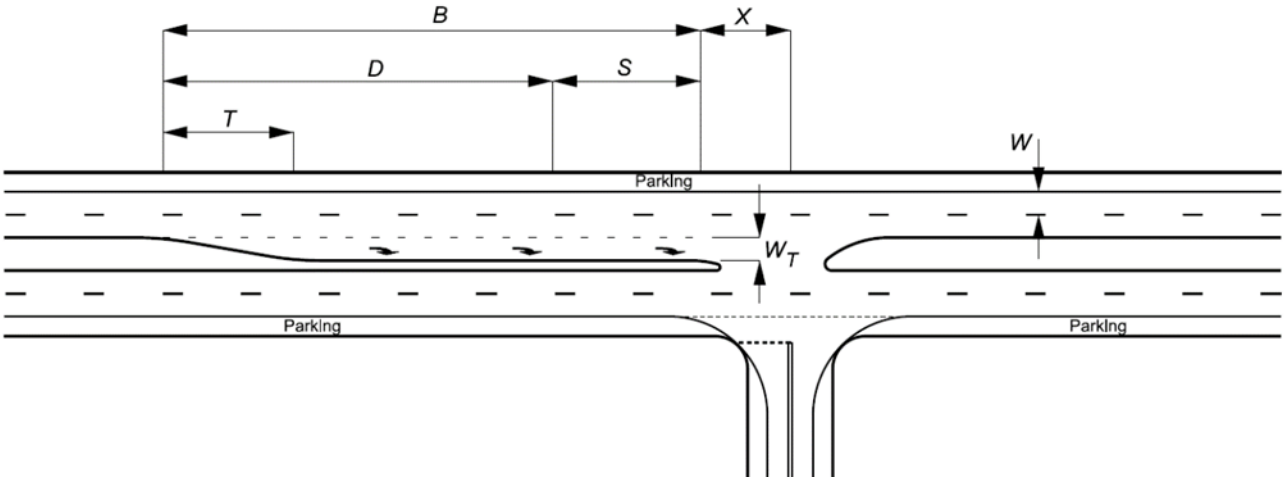
A.5.5 Minimum EDD Channelised Right-turn Treatment for Roadways with Medians

Figure A 4 shows a minimum EDD 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 NDD.

Table A 16 provides dimensions for various terms listed in Figure A 4.

Figure A 4: Minimum extended design domain channelised right-turn treatment for roadways with medians



Notes:

- *Variables are defined thus:*
  - *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 = 2.8 m minimum.*
  - *B = Total length of auxiliary lane (m), including taper, diverge/acceleration and storage.*
  - *S = Storage length (m), greater of:*
    1. *the length of one design turning vehicle or*
    2. *(calculated car spaces –1) x 8 m (refer to Austroads Guide to Traffic Management Part 3: Traffic Studies and Analysis (Austroads 2017)), or use computer program e.g. aaSIDRA.*
  - *T = Physical taper length =* $\frac{0.2VW_T}{3.6}$
  - *V = Design speed of major road approach (km/h).*
  - *X = Distance based on design vehicle turning path, refer to Austroads (2023a).*
- *Values of D and T are shown in Table A 16.*
- *Diagram shown for an urban intersection layout. The dimensions shown are also suitable for a rural intersection layout.*

Source: Department of Main Roads (2006).

Table A 16: Dimensions relating to Figure A 4

Design speed of major road approach (km/h)	Minimum diverge/deceleration length *D*(1) (m)	Taper length *T*(2) (m)
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 by:*
- *decreasing length D by 10% for upgrades of 3 to 4% and by 20% for upgrades of 5 to 6%*
- *increasing length D by 20% for downgrades of 3 to 4% and by 35% for downgrades of 5 to 6%.*

2 *Based on a turn lane width of 3.0 m.*

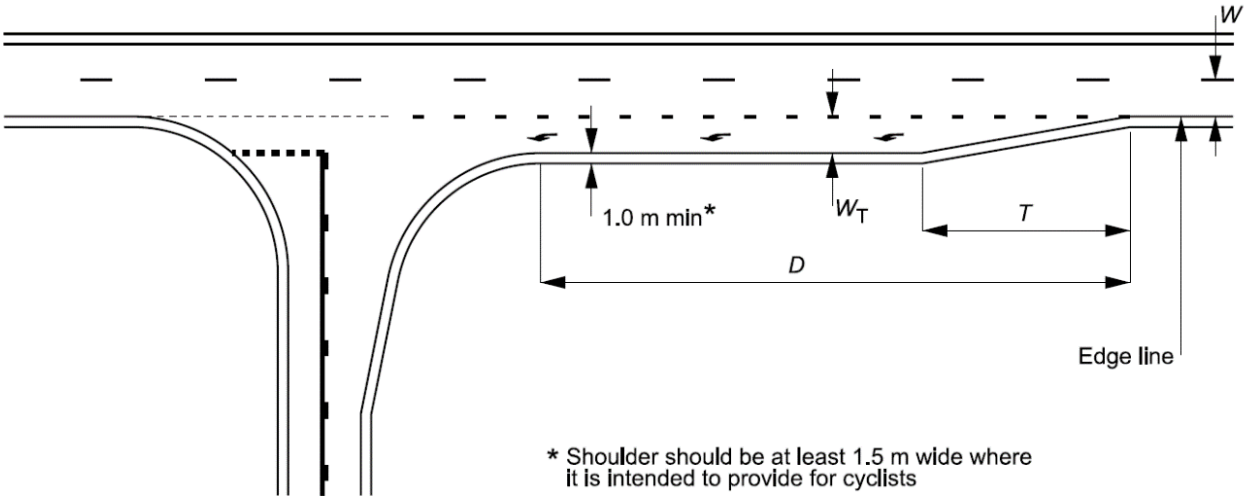
Source: Department of Main Roads (2006).

A.5.6 Minimum EDD Auxiliary Left-turn Treatment

Figure A 5 shows a minimum EDD auxiliary left-turn treatment. This treatment can be used at intersections on existing roads where sufficient area of pavement already exists to introduce an indented left-turn lane. Alternatively, the treatment may be applied as new construction at existing intersections where insufficient length is available to introduce a left-turn lane with dimensions as per the NDD.

Table A 17 provides dimensions for various terms listed in Figure A 5.

Figure A 5: Minimum extended design domain auxiliary left-turn treatment



Notes:

- *Variables are defined thus:*
  - *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 = 2.8 m minimum.*
  - *T = Physical taper length (m) =* $\frac{0.2VW_T}{3.6}$
  - *V = Design speed of major road approach (km/h).*
- *Values of D and T are shown in Table A 17.*
- *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 and kerbs are provided.*
- *Approaches to left-turn slip lanes can create hazardous situations between cyclists and left-turning motor vehicles. Possible treatment that may be provided to reduce the number of potential conflicts at left-turn slip lanes are shown in AGRD Part 4 (Austroads 2023b).*

Source: Department of Main Roads (2006).

Table A 17: Dimensions relating to Figure A 5

Design speed of major road approach (km/h)	Minimum diverge/deceleration length $D^{(1)}$ (m)	Taper length $T^{(2)}$ (m)
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.*

2 *Adjust for grade by:*
- *decreasing length D by 10% for upgrades of 3 to 4% and by 20% for upgrades of 5 to 6%*
- *increasing length D by 20% for downgrades of 3 to 4% and by 35% for downgrades of 5 to 6%.*

Note: Based on a turn lane width of 3.0 m.

Source: Department of Main Roads (2006).

A.5.7 EDD for Right-turn Treatments on Existing Urban Roads (S Lanes)

An 'S' lane treatment at an intersection converts three through lanes into two through lanes with a right-turn bay. This is shown in Figure A 6.

'S' lanes should only be considered as a retrofit on an existing three-lane carriageway in constrained situations where it is not possible to widen the carriageway for the right-turn bay. As with any proposed intersection treatment, a 'S' lane should be evaluated by examining capacity, safety, economic and environmental issues for comparison of existing conditions with any proposal.

Before and after studies on major arterial roads in Sydney (Parramatta Road, Victoria Road and the Pacific Highway) showed that 'S' lanes may provide the following advantages and disadvantages:

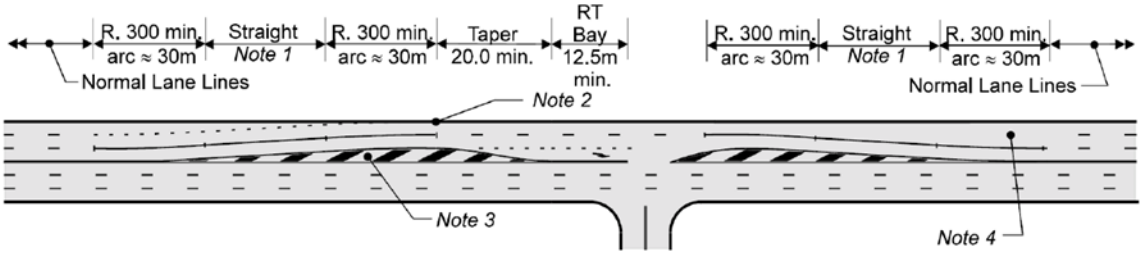
Advantages

- Lane changing by through vehicles reduced incidence of rear-end collisions involving right-turning vehicles reduced.
- Free flowing conditions provided for vehicles in the offside through lane (adjacent to centreline or median).
- May reduce travel times.

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.
- Problems for cyclists where three through lanes reduce to two.
- May require relocation of bus stops, taxi ranks, mail collection points.
- Rigid kerbside objects (poles, trees, signposts, etc) may have to be moved where three through lanes merge into two.

Figure A 6: S lane treatment



Notes:

1. *Length of straight equal to 1–2 sec of travel at through speed for reversal steering. 60 km/h – 17 to 33 m; 80 km/h – 22 to 44 m.*
2. *Provision for cyclists is to be incorporated into the design; particularly at this squeeze point.*
3. *Refer to Roads and Maritime Services (2017) Figure A.10 for other methods of treatment of the medians.*
4. *Continuity lines to be used where nearside lane is used for parking (including public transport stops) or dedicated left-turn lane.*

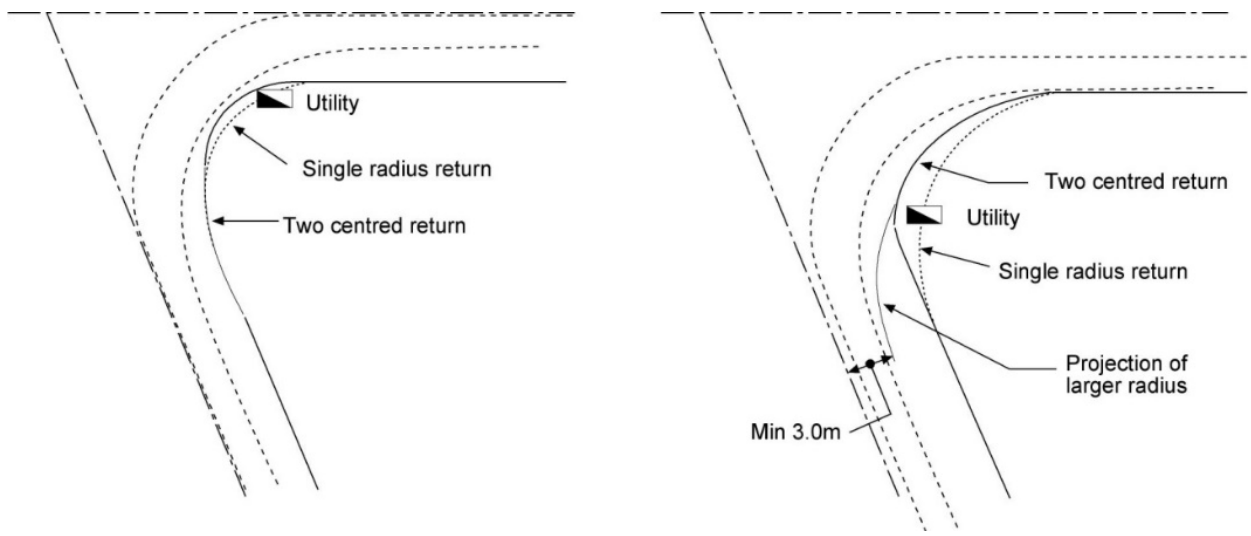
Source: Roads and Maritime Services (2017).

A.6 EDD Treatment of a Constrained Left-turn Radius

In some situations it may be necessary to adopt a multiple radii return to avoid an expensive design control (e.g. telecommunications pit). In such cases in a low-speed environment it may be acceptable to adopt a multiple radius curve which consists of compound circular arcs having two or three radii. Figure A 7 illustrates how a two centred curve may be advantageous in avoiding physical restrictions, such as utilities in the footway. This treatment is most effective for acute angle turns.

The width of the traffic lane being entered should be large enough to enable a vehicle following the projection of the larger radius to remain on the correct side of the centre line.

Figure A 7: Use of compound curves to avoid expensive relocation of an obstruction



Source: Department of Main Roads (2006).

A.7 EDD for Existing Channelised Four-way Intersection – Right-turn CHR

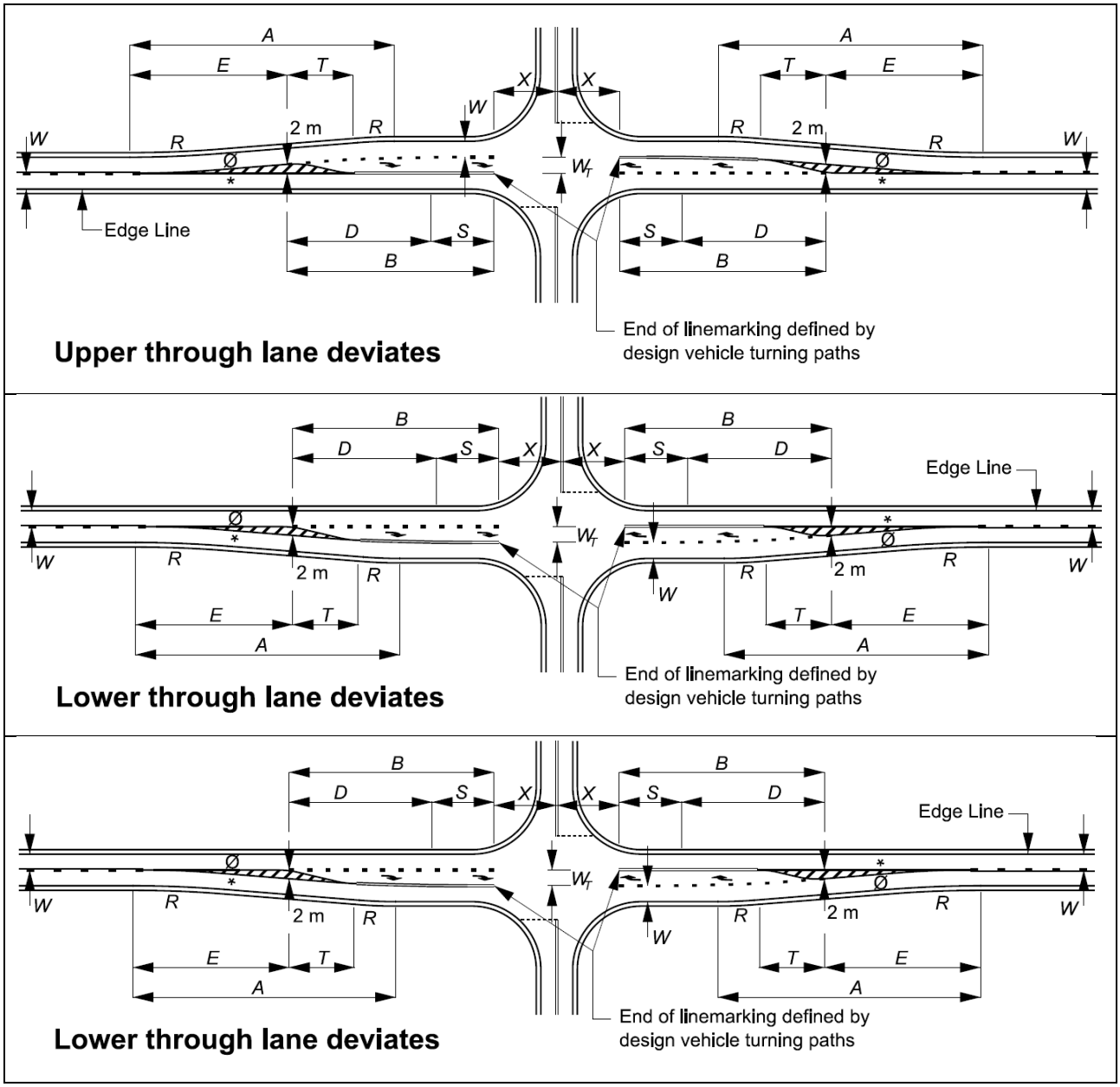
It is undesirable to build new four-way unsignalised intersections in rural situations. However, many four-way rural unsignalised intersections exist. At some of these intersections, traffic volumes and other considerations may dictate the need to retrofit CHR(S) or CHR turn treatments.

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 A 8 for a CHR(S) turn treatment) and Figure A 9 for a CHR turn treatment. The deviation can be fully on one side of the intersection, as shown in the two upper diagrams in these figures, or partly on each side, as shown in the lower diagram of these figures. Site details will generally dictate which option is the best.

The layouts shown in Figure A 8 and Figure A 9 may also be applicable to existing four-way urban intersections, except that the intersection will usually be kerbed and parking lanes (rather than shoulders) may exist on the intersection approaches and departures.

The definition of the notated dimensions and the key features of Figure A 8 and Figure A 9 are the same as those that relate to the T-intersections shown in Figures A 29 and A 30 in the Austroads *Guide to Road Design Part 4: Intersections and Crossings – General* (AGRD Part 4) (Austroads 2023b), respectively.

Figure A 8: Retrofitting CHR(S) treatments to a rural four-way intersection



Notes: Refer to Figure A 30 in AGRD Part 4 (Austroads 2023b) for the dimensions labelled in these diagrams.

- *Ø – Double barrier line is not to be used this side of the island.*
- ** – Islands are to comprise linemarking only, i.e. no raised or depressed medians. Diagonal rows of RRPMs within the painted islands should be used to improve the delineation of diagonal pavement marking.*
- *Variables are defined thus:*

  W = Nominal through lane width (m) (including 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), including widening for curves based on the design turning vehicle. Desirable minimum = W, absolute minimum = 3.0 m.

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

  D = Diverge/deceleration length including taper. Adjust for grade using the 'correction to grade' factor (Section 5).

  E = Distance from start of taper to 2.0 m width and is given by: $E = 2\left(\frac{A}{W_T}\right)$

  T = Physical taper length (m) and is given by: $T = \frac{0.33\ V\ W_T}{3.6}$

S = Storage length (m) should be the greater of:

1. the length of one design turning vehicle or

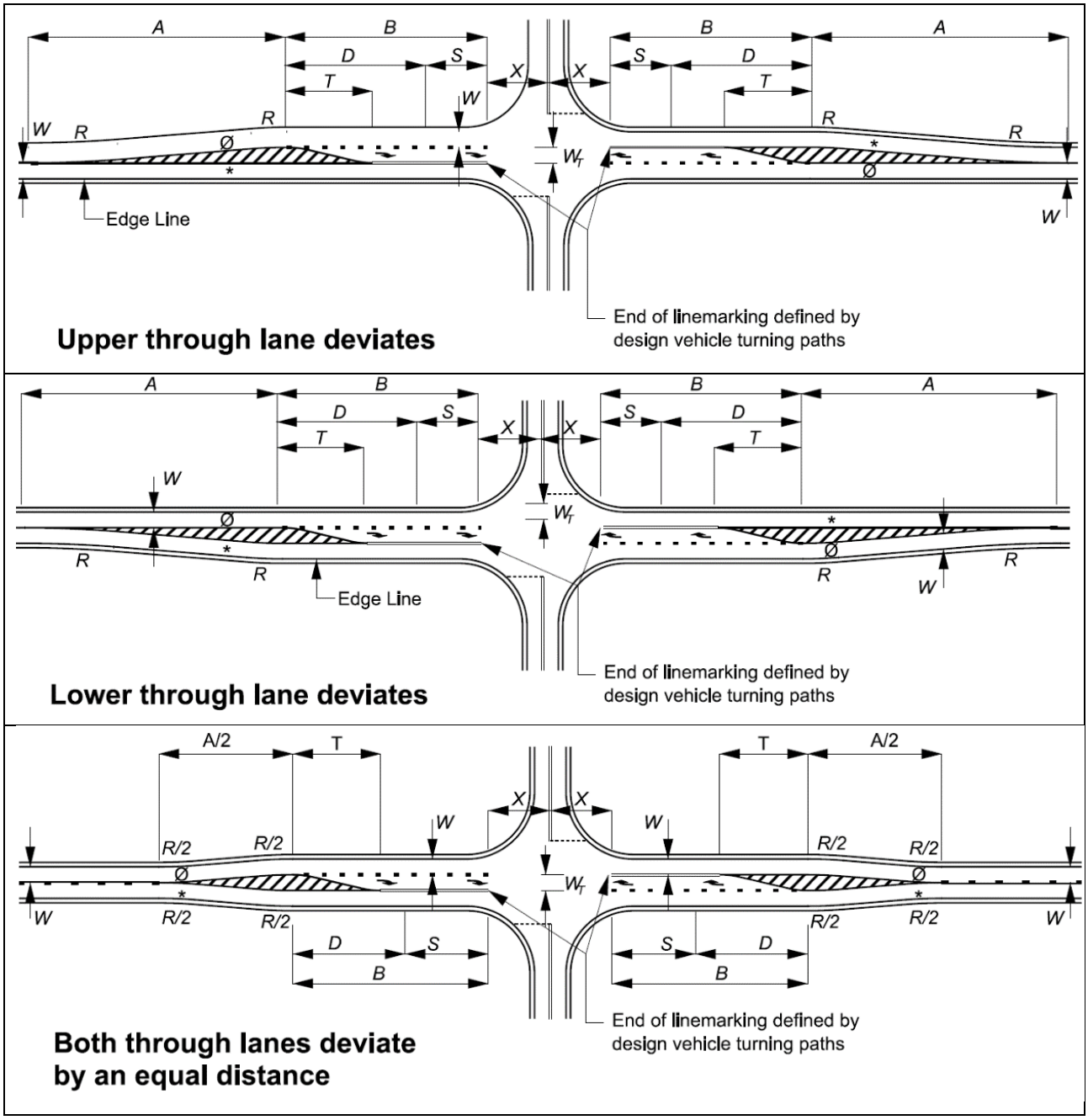
2. (calculated car spaces –1) x 8 m (Guide to Traffic Management Part 3: Traffic Studies and Analysis (Austroads 2017), or use computer program, e.g. aaSIDRA).

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

X = Distance based on design vehicle turning path, refer to Design Vehicles and Turning Path Templates Guide (Austroads 2023a).

Source: Department of Main Roads (2006).

Figure A 9: Retrofitting CHR treatments to a rural four-way intersection



Note: Refer to Figure A 30 in AGRD Part 4 (Austroads 2023b) for the dimensions labelled in these diagrams.

- *Ø – Double barrier line is not to be used this side of a line-marked island.*
- ** – Islands may be line-marked or raised. If line-marked, it is preferable that diagonal rows of RRPMs within the painted islands be used to improve the delineation of diagonal marking.*
- *Variables are defined thus:*

W	=	*Nominal through lane width (m) (including 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), including widening for curves based on the design turning vehicle. Desirable minimum = W, absolute minimum = 3.0 m.*
B	=	*Total length of auxiliary lane including taper, diverge/deceleration and storage (m).*
D	=	*Diverge/deceleration length including taper. Adjust for grade using the 'correction to grade' factor (Section 5).*
E	=	*Distance from start of taper to 2.0 m width and is given by:* $E = 2\left(\frac{A}{W_T}\right)$

T = *Physical taper length (m) and is given by:* $T = \frac{0.33\ V\ W_T}{3.6}$

S = *Storage length (m) should be the greater of:*

1. the length of one design turning vehicle or

2. (calculated car spaces –1) x 8 m (Guide to Traffic Management Part 3: Traffic Studies and Analysis (Austroads 2017), or use computer program e.g. aaSIDRA).

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

X = *Distance based on design vehicle turning path, refer to Design Vehicles and Turning Path Templates Guide (Austroads 2023a).*

Source: Department of Main Roads (2006).

A.8 EDD for Median Widths

Table A 18 provides EDD median widths for intersections for various median functions.

Table A 18: EDD median widths at intersections

Median function	Absolute minimum width (m)
Separate traffic flows and a safety barrier(1)	1.9
Shelter a small sign	1.0
Shelter signal pedestals or lighting poles	1.4
Shelter pedestrians and traffic signals	2.0
Shelter turning vehicles and traffic signals	5.0
Shelter crossing vehicles	6.0

1 *Widths measured to edge of traffic lane for concrete barriers, as there is no kerb and channel associated with concrete barriers. Other widths are measured to line of kerb.*

A.9 Adverse Crossfall for Turning Movements

Crossfall, particularly in conjunction with a longitudinal grade can be problematic for heavy vehicles turning at an intersection. Where speeds on the approach to an intersection are low, turning speeds are very low (i.e. ≤ 10 km/h), and it is not possible to achieve a crossfall within desirable limits (Section 2.2.4) an absolute maximum adverse crossfall up to –7% may be considered. In this situation traffic management devices such as appropriate warning signs may be required on the approach.

A.10 EDD for Acceleration Lanes for Trucks

If the speed of heavy vehicles at the merge is much slower than the speed of the through traffic (30 km/h to 40 km/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 either:

- a basic left-turn treatment comprising a give way or stop situation (i.e. a BAL) or a high entry angle (CHL) treatment
- an acceleration lane length that is based on Table 5.7 or Table 5.8 (where practicable) or alternatively on the length required for cars to accelerate to the design speed of the through lane.

Although the BAL and CHL treatments result in slow-moving heavy vehicles on the through road, where the traffic volume on the road and the number of trucks entering is not high, and good sight distance is available on the through road, it may be relatively easy for through drivers to perceive the slow movement of these vehicles and to slow for them. Alternatively, where SISD is insufficient and/or traffic volumes on the road and the number of trucks entering are relatively high, it may be preferable to provide an acceleration lane, even if it is shorter than the desirable length, in order to establish the presence of the entering truck on the major road, even though a higher than desirable speed differential between trucks and cars may occur near the merge area. However, one of the major disadvantages of a shorter than desirable acceleration lane is the ambiguity for approaching through traffic to judge the truck merge movement, particularly if sight distance to the merge area is obscured. This should be optimised to ensure the lane ends at a location where sight distance to the merge is adequate.

Another aspect to consider is the grade of the road, which affects a vehicles ability to accelerate. For situations where vehicles will be travelling uphill after turning on to the through road, consideration may need to be given to provide an acceleration lane, even if it is shorter than the desirable length, to allow heavy vehicles some length of parallel lane to build speed. Where steep grades are maintained over long distances a climbing lane may be warranted (refer to AGRD Part 3).

A risk assessment should be undertaken when considering an acceleration lane that is shorter than desirable. It should take into account:

- exposure (e.g. traffic volumes and percentage of heavy vehicles)
- likelihood (e.g. sight distances such as SISD and MSD)
- severity (e.g. potential speed differential).

Appendix B Truck Stability at Intersections

B.1 Introduction

Truck stability is an important consideration in design because the rolling over of trucks or the loss of loads can have serious safety issues for road users in addition to the delays and costs to the community.

As discussed in Section 2.2.4 adverse crossfall within an intersection can lead to instability for heavy vehicles, particularly those with high loads. This Appendix discusses the parameters associated with truck stability in design and provides a table of critical truck turning speeds as a function of radius and superelevation.

B.2 Lateral Friction Force on Vehicles

Depending on the circumstances at a site a truck turning at a particular speed can slide on the road surface or tend to overturn. On wet surfaces, trucks tend to lose stability by sliding. On dry, low radius curves, high trucks are more likely to roll which is sometimes an issue at intersections.

When a truck travels around a curve that has no superelevation (i.e. a flat surface) sufficient lateral friction usually develops at the tyre/road interface to force the truck to turn in a circular path. Without this force, the truck would travel in a straight line and Equation A1 in Appendix B.3 describes this condition.

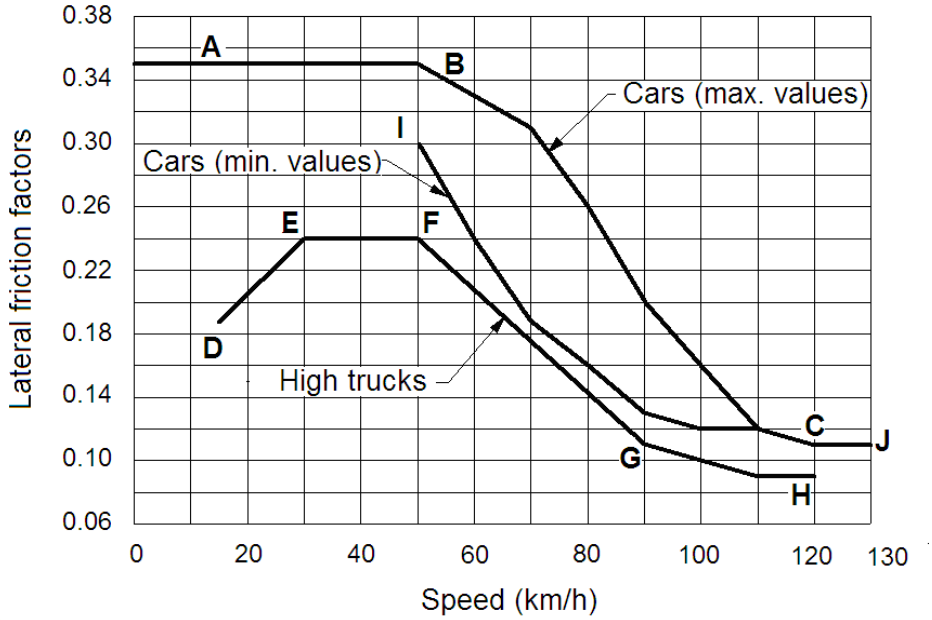
For design purposes, friction factors have to be defined which are less than the maximum at which vehicles lose control. The side friction factors (i.e. lateral) that should be used in design are discussed and provided in Austroads *Guide to Road Design Part 3: Geometric Design* (AGRD Part 3) (Austroads 2016a).

Some friction factors for cars and trucks on sealed roads are shown in Figure B 1. It is emphasised that these lateral friction factors were calculated from field measurements of 85th percentile speeds and superelevation on dry sealed rural roads. These friction factors are therefore more a measure of the 85th percentile driver comfort level than of the friction developed at the tyre/road interface. This information is therefore provided only to assist discussion on heavy vehicle stability and should not be used for design purposes.

Figure B 1 specifies the lateral friction which is being utilised when truck instability is imminent. In most cases it is not necessary to know whether this instability is caused by sliding or by rolling.

If, for analysis purposes, the moving truck is deemed to be a static point mass located at the centre of mass and the horizontal component of the force at the tyre/road interface is $F = mv^2/R$, there is a moment at which the truck tends to overturn as illustrated in Figure B 2.

Figure B 1: Variation of friction factors with speed



Notes:

Car friction factors:

- *Car friction factors apply also to small single unit trucks.*
- *Friction factors from A to B are based on measured speeds on roundabouts.*
- *The figures from B to C are based on measurements reported in McLean (1988).*
- *Friction factors I to J are contained in AGRD Part 3 (Austroads 2016a).*

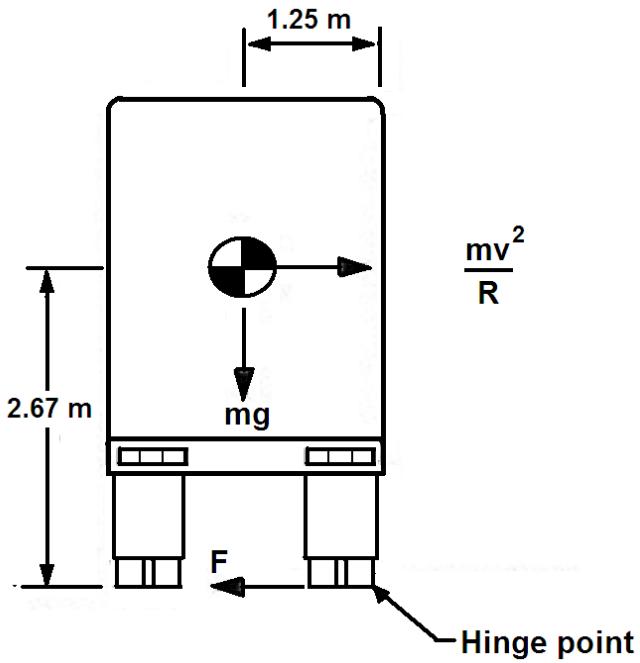
High truck friction factors:

- *Point D was fixed to permit a maximum speed of 15 km/h on a 15 m radius curve (E = –0.05 assumed).*
- *Points E and F are based on the rollover threshold for a large truck with homogeneous load. (Table B 1).*
- *Points G and H were fixed to provide radii consistent with car requirements at 100 km/h taking into account the speed differential between car speeds and truck speeds.*

Source: Based on VicRoads (1994)[2].

2 VicRoads (1994) has been superseded and Figure B 1 and Figure B 2 have not been carried forward into VicRoads (2011).

Figure B 2: Overturning moment on a turning truck



Source: VicRoads (1994)[3].

B.3 Other Factors that Affect Truck Stability

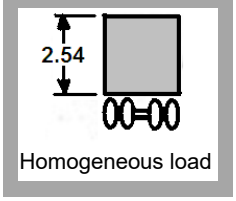
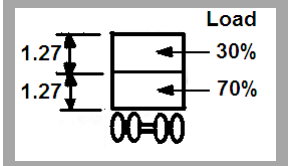
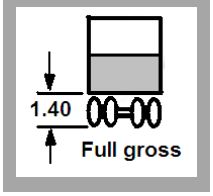
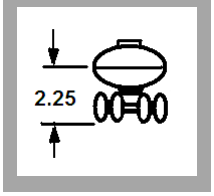
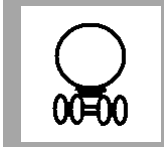
Other effects which tend to reduce the stability of trucks include:

- adverse superelevation which reduces the horizontal distance between the centre of gravity and the hinge point
- the dynamic effects associated with wheel bounce tend to reduce the stability of trucks on curves
- the rigidity of the fifth wheel linkage between the prime mover and the trailer on articulated vehicles directly affects the stability of the rig
- the changes in geometry which occur on low radius curves.

When these factors are taken into account, the critical lateral acceleration (also known as the rollover threshold) at which instability occurs for a large vehicle with a homogeneous load is approximately 0.24 g. Critical lateral accelerations for other vehicle body types on level surfaces are shown on Table B 1.

3 VicRoads (1994) has been superseded and Figure B 2 has not been carried forward into VicRoads (2011).

Table B 1: Stability parameters for trucks

Loading					
Rollover Threshold	0.24 g	0.28 g	0.34 g	0.32 g	0.26 g
Centre of gravity height	2.67 m	2.41 m	2.12 m	2.25 m	2.54 m

Source: Ervin et al. (1985).

Equation A1 shows that the lateral friction developed at the tyre/road interface as vehicles turn is directly related to the square of the vehicle speed. A speed can be reached at which the force required to maintain a circular path exceeds the force which can be developed by friction and superelevation. At this point, the vehicle starts to slide tangentially to the alignment of the road:

$$E + F = \frac{v^2}{gR} \qquad \text{A1}$$

$$E + F = \frac{a}{g}$$

on flat surfaces, E = 0 and therefore $F = \frac{a}{g}$ (special case)

But as the critical lateral acceleration a = 0.24 g from Figure B 2 and F = 0.24.

This shows that the numeral before g in the rollover thresholds in Table B 1 can be construed as a friction factor at which the vehicle is likely to overturn. This value of F fixes the upper limit to friction factors for trucks on Figure B 1. Variations from this limit occur at speeds above 50 km/h and speeds below 30 km/h.

The reduction in friction factors above 50 km/h are consistent with observed operating speeds in the field, that is, truck speeds should be approximately 10 km/h below car speeds. This speed range above 50 km/h is also the range where instability is generally initiated by sliding.

The deviation from the friction factor of 0.24 value below 30 km/h is necessary to match limiting speeds shown on vehicle turning templates (Austroads 2023a). In the speed range involved, curve radii are sufficiently low to influence the geometry of articulated vehicles and this effect could explain the low stability of trucks at low speeds.

Use of the truck friction factors in Figure B 1 provides for the majority of trucks. There have been some trucks which have rolled at lower friction factors. Rollovers at low speed can be initiated by a range of factors including:

- ***Tripping***. Vehicles sliding sideways can overturn at speeds below 10 km/h when tripped by a kerb or pothole. For this reason road surfaces must be kept in good condition where critical turning movements occur.
- ***Loading.*** Small lateral offsets of the centre of gravity of the load significantly reduce the lateral stability of the truck. Uneven longitudinal loading also reduces the vehicle's stability.

- ***Load shift.*** As for example liquid in tankers or cattle on high trucks.
- ***Dynamic forces.*** Associated with tyre and suspension bounce. These forces are related to the speed of the vehicle and the condition of the pavement.
- ***Aquaplaning.*** Leading to loss of control and rollover.
- ***Braking.*** As the brakes are applied, the friction available in the radial direction decreases. If the wheels lock, lateral stability and steering is lost.
- ***Rearward amplification.*** A 'whiplash' effect; specifically, it is the ratio of the maximum lateral acceleration at the rear axle over the lateral acceleration on the prime mover.
- ***Speed.*** The indications are that critical lateral accelerations (or friction forces) are speed-dependent as shown by Figure B 1.

B.4 Critical Turning Speeds for Trucks in Intersections

Turning paths within intersections are designed using turning templates for the appropriate design vehicle. Once the turning paths have been established, the minimum crossfall (or maximum adverse crossfall) may be calculated for each turning path. Critical speeds for high trucks (i.e. speeds at which the least stable are at the point of overturning) can then be obtained from Table B 2. The least stable truck was used as a basis for design friction factors for the critical turning radii.

Table B 2: Critical truck turning speeds

Radius (m)	Critical speeds for high trucks within intersections (km/h)														
	Positive superelevation (m/m)							0	Negative superelevation (i.e. adverse)						
	0.07	0.06	0.05	0.04	0.03	0.02	0.01	0	–0.01	–0.02	–0.03	–0.04	–0.05	–0.06	–0.07
10	17	17	17	16	16	15	15	14	13	12	11	10	9	7	4
11	19	18	18	17	17	16	16	15	14	13	12	11	10	8	5
12	20	19	19	18	18	17	16	16	15	14	13	12	10	8	5
13	21	20	20	19	19	18	17	17	15	15	14	12	11	9	6
14	22	21	20	20	19	19	18	18	17	16	14	13	12	10	7
15	22	22	21	21	20	20	19	18	17	16	15	14	12	10	7
16	23	23	22	22	21	21	20	19	18	17	16	14	13	11	8
17	24	24	23	23	22	21	21	20	19	18	17	15	13	11	8
18	25	25	24	24	23	22	22	21	20	19	17	16	14	12	9
19	26	26	25	24	24	23	22	22	21	19	18	16	15	13	10
20	27	26	26	25	25	24	23	22	21	20	19	17	15	13	10
21	28	27	27	26	25	25	24	23	22	21	19	18	16	14	11
22	29	28	28	27	26	25	25	24	23	21	20	18	17	14	11
23	30	29	28	28	27	26	25	25	23	22	21	19	17	15	12
24	30	30	29	28	28	27	26	25	24	23	21	20	18	16	13
25	31	30	30	29	29	28	27	26	25	24	22	20	18	16	13
26	31	31	30	30	29	29	28	27	26	24	23	21	19	17	14
27	32	32	31	30	30	29	29	28	26	25	23	22	20	17	14
28	33	32	32	31	30	30	29	28	27	26	24	22	20	18	15
29	33	33	32	32	31	30	30	29	28	26	25	23	21	19	16
30	34	33	33	32	32	31	30	30	28	27	25	24	22	19	16
31	34	34	33	33	32	31	31	30	29	28	26	24	22	20	17

Radius (m)	Critical speeds for high trucks within intersections (km/h)														
	Positive superelevation (m/m)							0	Negative superelevation (i.e. adverse)						
	0.07	0.06	0.05	0.04	0.03	0.02	0.01	0	–0.01	–0.02	–0.03	–0.04	–0.05	–0.06	–0.07
32	35	34	34	33	33	32	31	31	30	28	27	25	23	20	17
33	36	35	34	34	33	33	32	31	30	29	27	26	23	21	18
34	36	35	35	34	34	33	32	32	31	30	28	26	24	22	19
35	37	36	35	35	34	33	33	32	31	30	29	27	25	22	19
36	37	37	36	35	35	34	33	33	32	30	29	27	25	23	20
37	38	37	36	36	35	34	34	33	32	31	30	28	26	23	20
38	38	38	37	36	36	35	34	34	32	31	30	29	26	24	21
39	39	38	37	37	36	35	35	34	33	32	31	29	27	25	21
40	39	39	38	37	37	36	35	34	33	32	31	30	28	25	22
41	40	39	38	38	37	36	36	35	34	33	31	30	28	26	23
42	40	40	39	38	37	37	36	35	34	33	32	30	29	26	23
43	41	40	39	39	38	37	36	36	35	33	32	31	30	27	24
44	41	40	40	39	38	38	37	35	35	34	33	31	30	28	24
45	42	41	40	40	39	38	37	37	35	34	33	32	30	28	25
46	42	41	41	40	39	38	38	37	36	35	33	32	31	29	26
47	43	42	41	40	40	39	38	37	36	35	34	32	31	29	26
48	43	42	42	41	40	39	39	38	37	35	34	33	31	30	27
49	43	43	42	41	40	40	39	38	37	36	34	33	32	30	27
50	44	43	42	42	41	40	39	39	37	36	35	33	32	30	28

Notes:
Radii shown represent the outside radius of the curve.
The desirable maximum effective adverse crossfall within intersections should be kept as low as possible and not more than 5%.
Where truck operating speeds are likely to exceed the safe speed, warning signs should be provided.
Factors not taken into account in Table B 2 include:

- *Surface condition – truck speeds in the table are appropriate for use on surfaces which are in good condition i.e. surfaces which are free from potholes, corrugations, oil slicks or loose gravel. On rough or uneven surfaces the critical speed reduces by approximately 33%.*
- *Grade – steering manoeuvres on grades increase the instantaneous crossfall. There are also other effects such as load shift which adversely affect the steering, but the precise effects cannot be calculated. It is suggested that the estimated critical speed should be reduced by approximately 2% for each 1% of downhill grade on the curves.*
- *Driven radius – although the driven radius differs from the radius of the road, it is not necessary to correct for this because the radii used to determine friction factors were road radii not driven radii. However, if a better estimate of the driven radius is required, the following points should be taken into account*
  - *for large deflection angles, the driven radius is approximately 85% of the road radius*
  - *for small deflection angles, drivers cut the curve and drive a larger radius than the road radius.*

Source: VicRoads (1994)[4].

4 VicRoads (1994) has been superseded and Table B 2 has not been carried forward into VicRoads (2011).

Appendix C Swept Paths for Road Trains at High Entry Angle Left-Turn Treatments

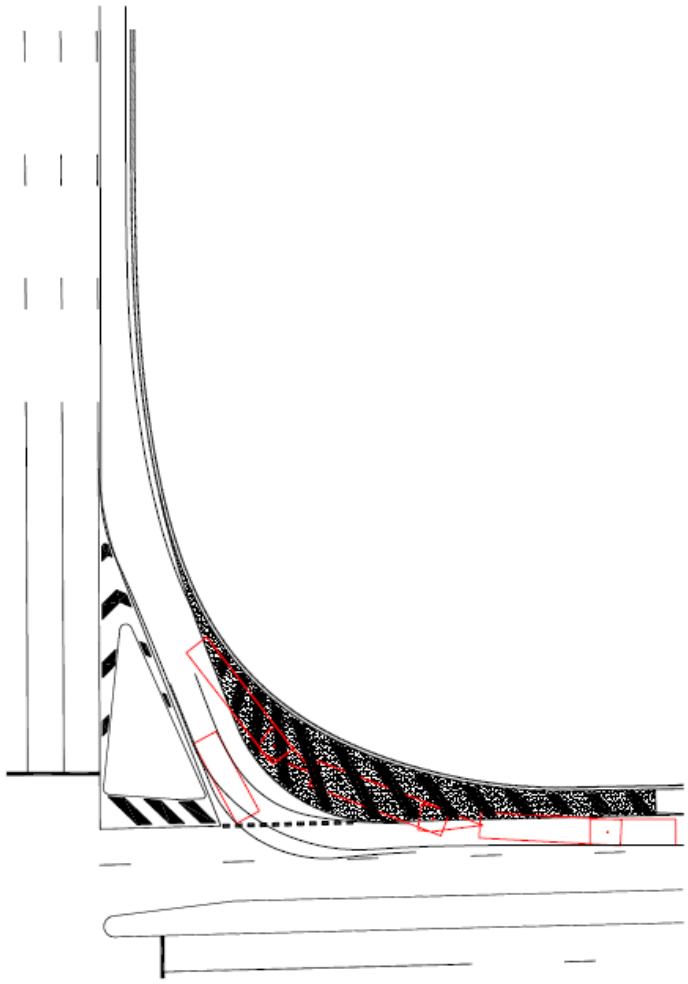
C.1 Introduction

This Appendix contains illustrations of the left-turning paths necessary for road trains at 'normal' and 'alternative' high entry angle left-turn treatments illustrated in Figure 8.17 and Figure 8.18 of this Part.

C.2 Examples of Vehicle Swept Paths

C.2.1 Normal Treatment

Figure C 1: Swept path provisions for road trains at channelised left-turns – normal treatment

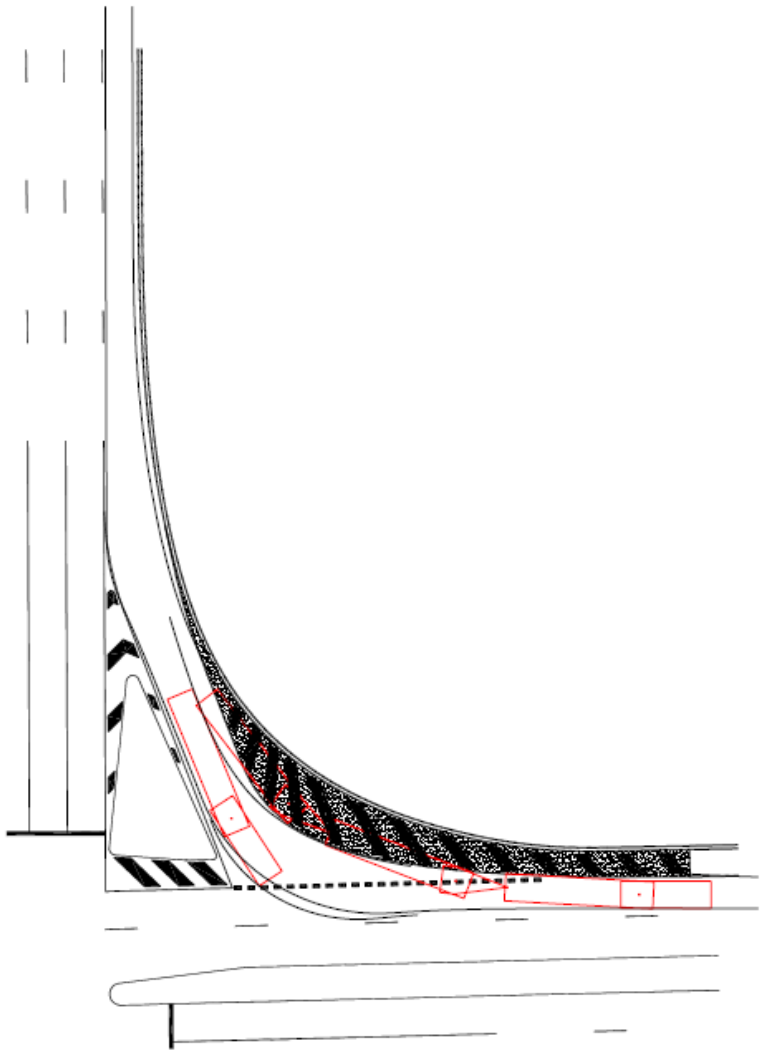


Note: This treatment is shown for an urban site. A similar layout is also applicable to rural sites.

Source: Department of Main Roads (2006).

C.2.2 Alternative Treatment

Figure C 2: Swept path provisions for road trains at channelised left-turns – alternative treatment



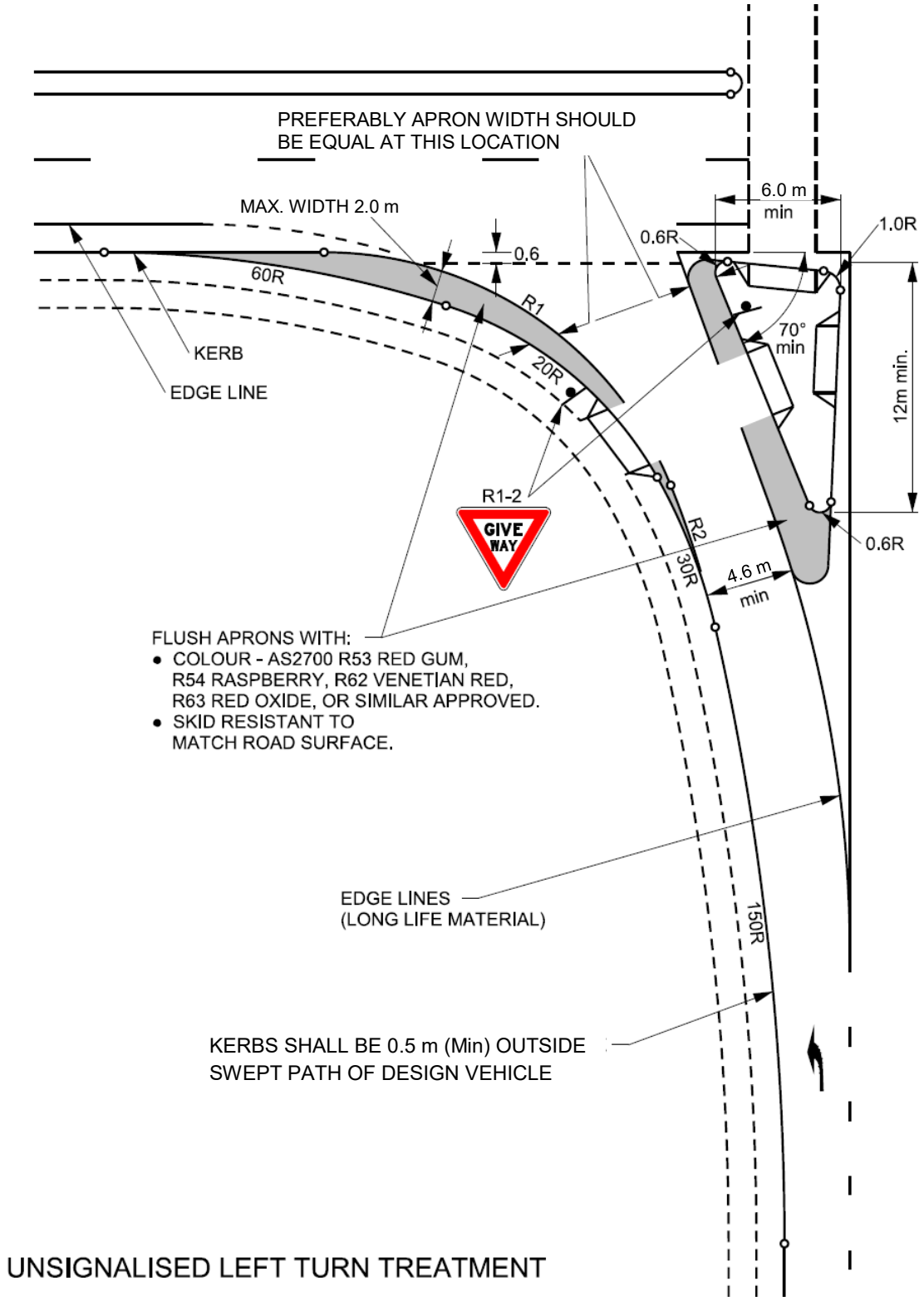
Notes:
This treatment is shown for an urban site. A similar layout is also applicable to rural sites.
To be used for sites where there is a high volume of large single unit trucks and prime mover and semi-trailer combinations.

Source: Department of Main Roads (2006).

C.3 Detailed Examples from Main Roads Western Australia

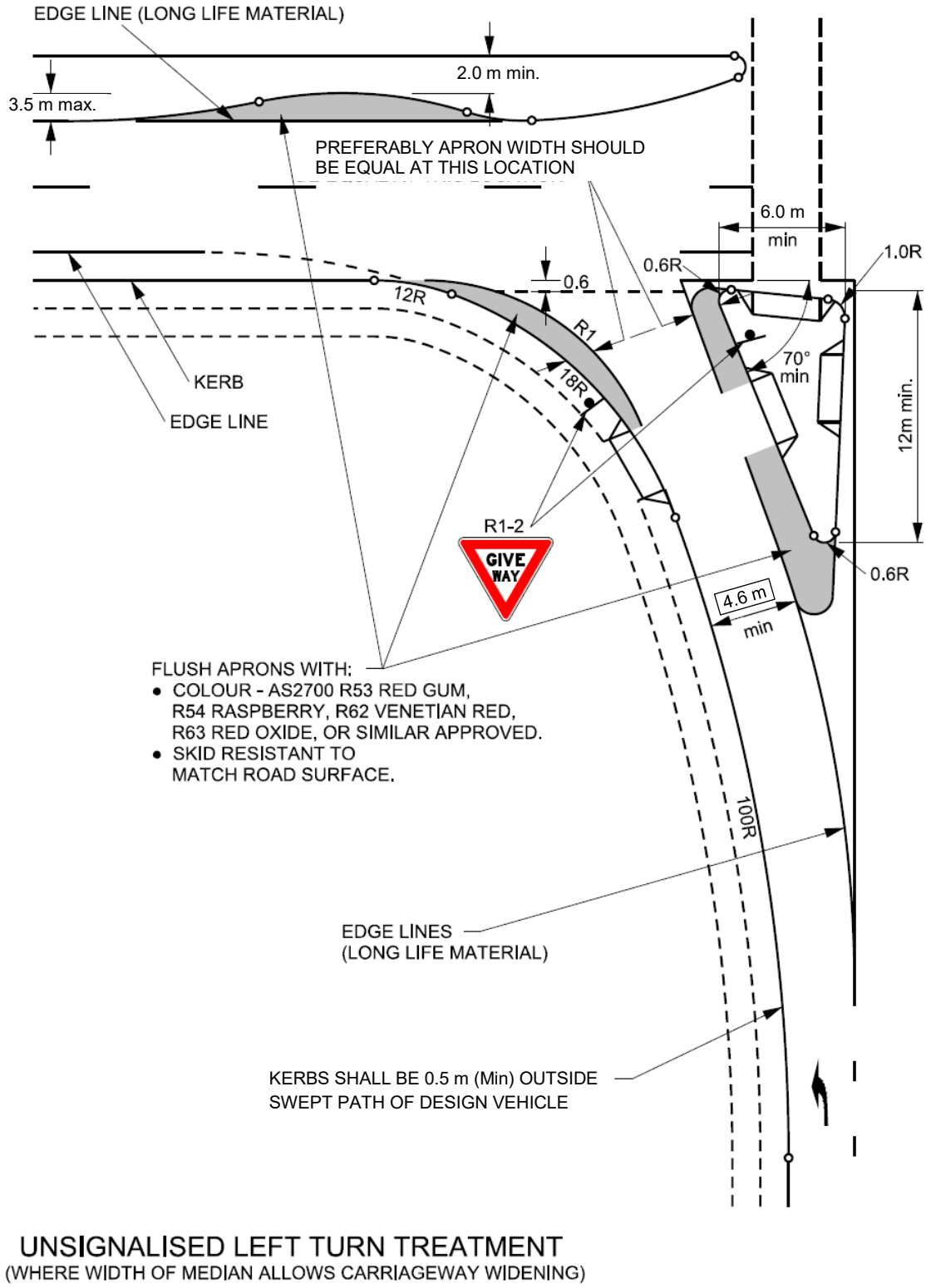
The following examples (Figure C 3 and Figure C 4) are from Main Roads Western Australia Guideline Drawing number 200031-0015-6. They are provided for general guidance only as the details within figures may not be applicable in other jurisdictions.

Figure C 3: Example of corner treatment on heavy combination vehicle route



Source: Adapted from Main Roads Western Australia (2017).

Figure C 4: Example of corner treatment on heavy combination vehicle route median allows carriageway widening



UNSIGNALISED LEFT TURN TREATMENT
(WHERE WIDTH OF MEDIAN ALLOWS CARRIAGEWAY WIDENING)

Source: Adapted from Main Roads Western Australia (2017).

Commentary 1

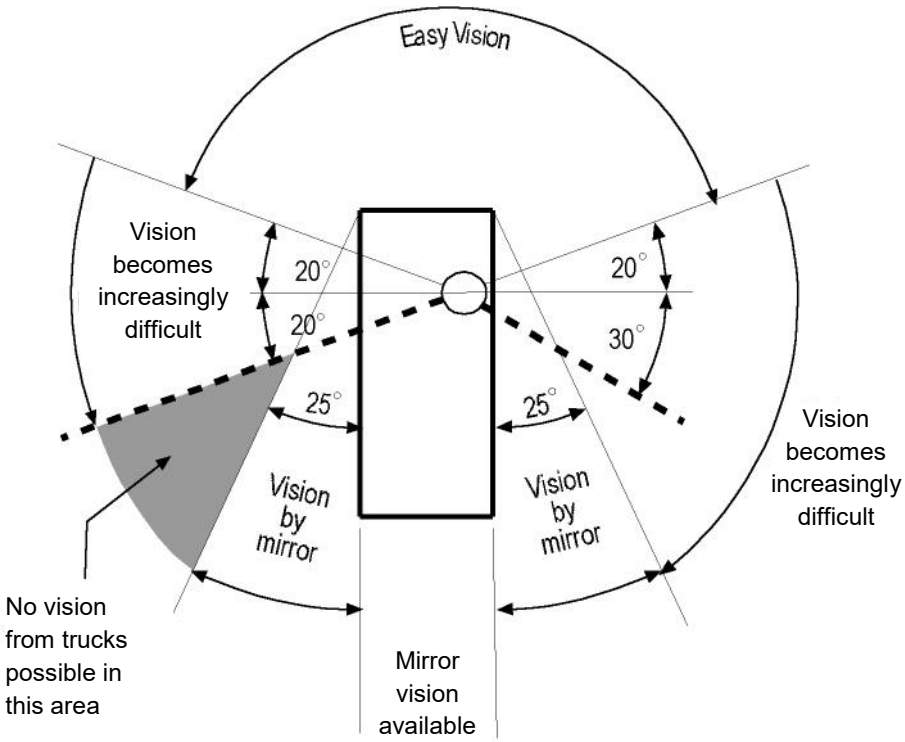
There are no design rules dealing with visibility from vehicles. Ackerman (1989) provides the visibility angles shown in Figure C1 1. At each point where a vehicle has to give way (e.g. give-way or stop lines) or is about to enter a traffic stream (e.g. merge situation), the vehicle paths, and orientation should be developed with these visibility angles in mind. The maximum desirable angles are shown by the dotted lines.

Road centrelines should be designed to intersect at between 70° and 110° in both urban and rural situations. For a curved alignment the angle should be measured to an approaching vehicle at a distance from the intersection equal to the design intersection sight distance. The orientation of vehicles prior to all points of conflict, including movements such as left and right merges, should comply with the visibility requirements of Figure C1 1.

The acceptable maximum observation angle for a left-turning driver is 120°. This means that a driver would not be required to significantly change driving position to sight approaching traffic. An angle greater than 120° 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.

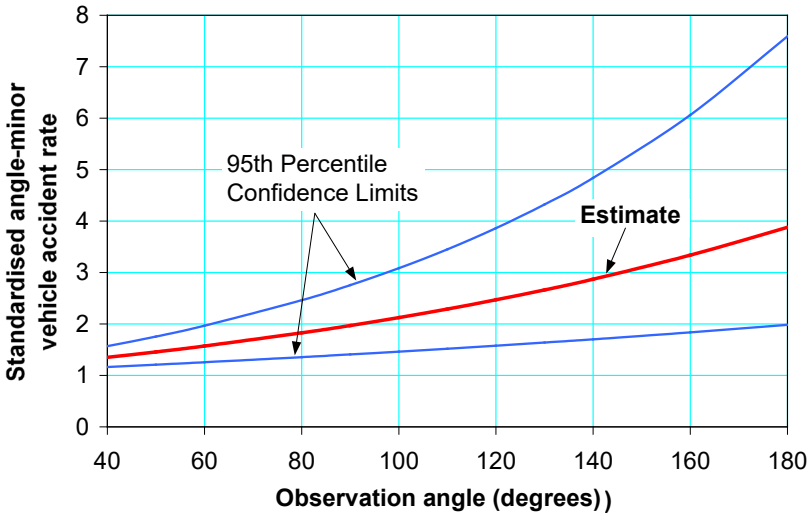
Arndt (2004) found that larger observation angles increased angle-minor vehicle accident rates (accidents resulting from minor road drivers failing to give way and colliding with drivers on the major road). The observation angle was measured between a line representing the instantaneous direction of travel of minor road drivers 4 m behind the holding line and a line tangential to the major road. This relationship is shown in Figure C1 2 and confirms the need to limit the observation angle, and therefore, the skew of the intersection.

Figure C1 1: Visibility angles and sight restrictions due to vehicle design



Source: Adapted from Ackerman (1989).

Figure C1 2: Effect of observation angle on angle-minor vehicle accident rates



Source: Arndt (2004).

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Commentary 2

Table C2 1 provides acceleration rates for a typical passenger car (Roads and Traffic Authority 1999) that may be of interest to road designers.

Table C2 1: Acceleration rates of a typical passenger car

Travel speed		Acceleration rate	
(km/h)	(m/s)	(km/h/s)	(m/s^2)
40	11.11	4.7	1.3
50	13.88	4.3	1.2
60	16.66	3.6	1.0
70	19.44	3.2	0.9
80	22.22	2.9	0.8
90	25.00	2.5	0.7
100	27.77	2.1	0.6
110	30.55	1.8	0.5

Source: Roads and Traffic Authority (1999).

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Commentary 3

The merge length for an acceleration lane or an auxiliary through lane can be calculated from Equation C1:

$$T_m = \frac{VY}{3.6S} \qquad \text{C1}$$

where

T_m = merge length (m)

V = design speed (km/h)

S = rate of lateral movement
acceleration lane merge – 1.0 m/s
through lane merge – 0.6 m/s

Y = width of lateral movement

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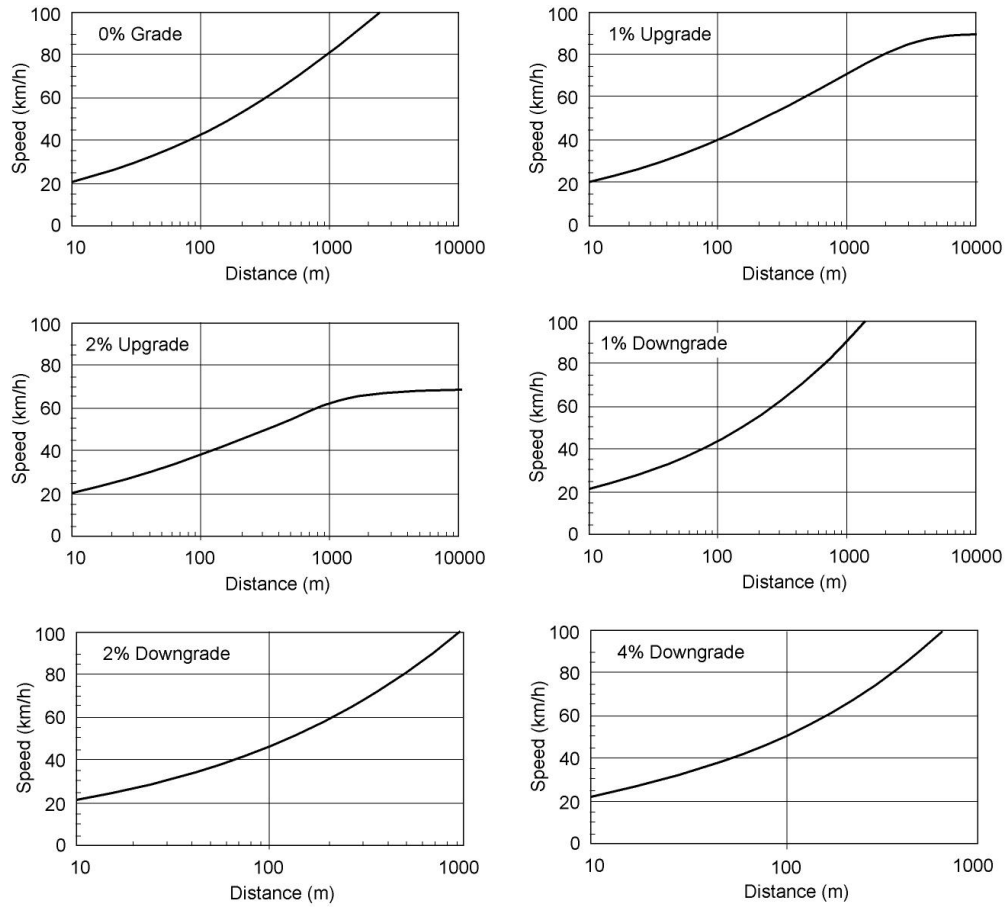
Commentary 4

The following graphs (Figure C4 1) show the speed profile of a semi-trailer on nominal upgrades and downgrades. They provide an indication of the speed decrement that could be expected for heavy vehicles on gradients merging with through traffic, and also indicate the length of acceleration lane that would be required to achieve that decrement.

Figure C4 1: Profiles for a semi-trailer starting from rest on constant grades



Source: Austroads (2002).

The computer software package VEHSIM (Queensland Department of Transport and Main Roads) 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.

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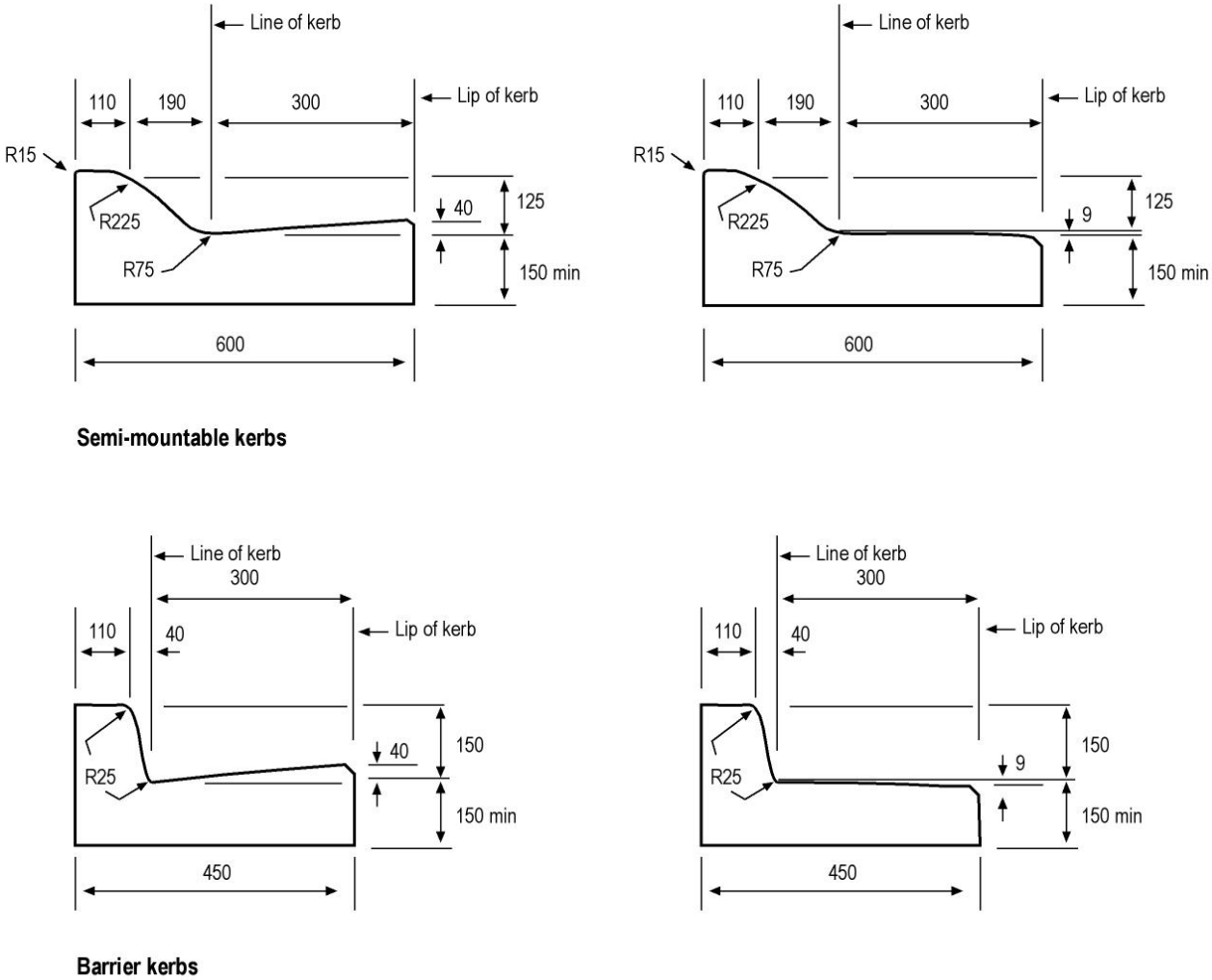
Commentary 5

Examples of kerb and channel are shown in Figure C5 1, the purpose of which is to illustrate the most common forms of kerb and channel. The dimensions are provided only to indicate typical sizes. Individual road agencies often have a broader range of kerb and channel types for specific applications and their guidelines should be referred to during design.

Figure C5 1: Examples of semi-mountable and barrier kerbs used on medians and traffic islands



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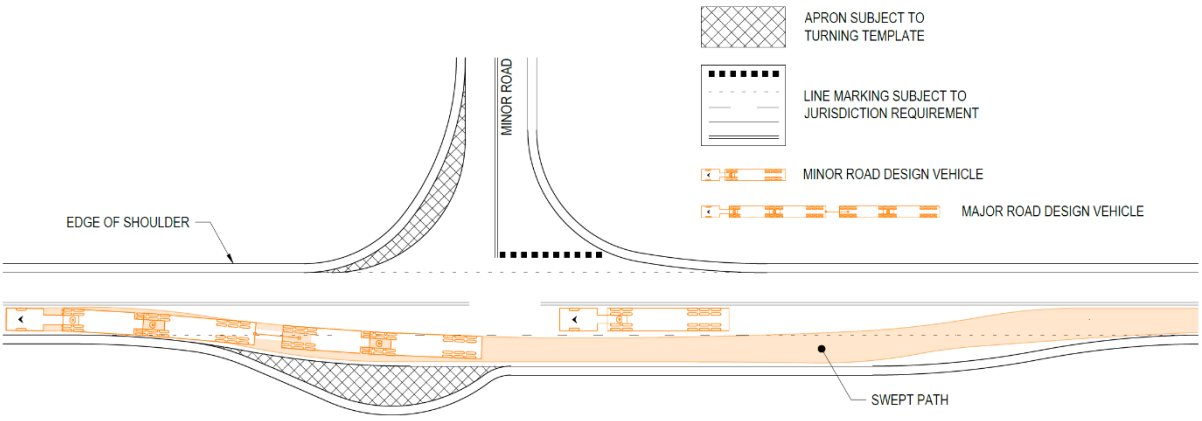
Commentary 6

Figure C6 1 shows an example layout for an AUR treatment. As stated in Section 7.2.2, **AUR treatments shall not be used without approval from the relevant jurisdiction.** Some jurisdictions, such as Main Roads Western Australia and Department of Infrastructure, Planning and Logistics Northern Territory, may approve the use of an AUR based on case-by-case basis if supported with comprehensive traffic and geometric assessments in place to ensure the use of an AUR will not compromise road safety. The relevant jurisdictional design criteria should be met. This criteria should include, but is not limited to, the following:

- Design speeds
- Design and check vehicles
- Sight distances
- Taper requirements (approach and departure)
- Signs and line marking
- Vehicle swept paths
- Vehicle storage requirements
- Auxiliary lane (Section 5)
- Shoulder widths
- Seal requirements
- Drainage requirements
- Crash history
- Consistent link intersection layouts
- Design life

Figure C6 1: Example AUR layout



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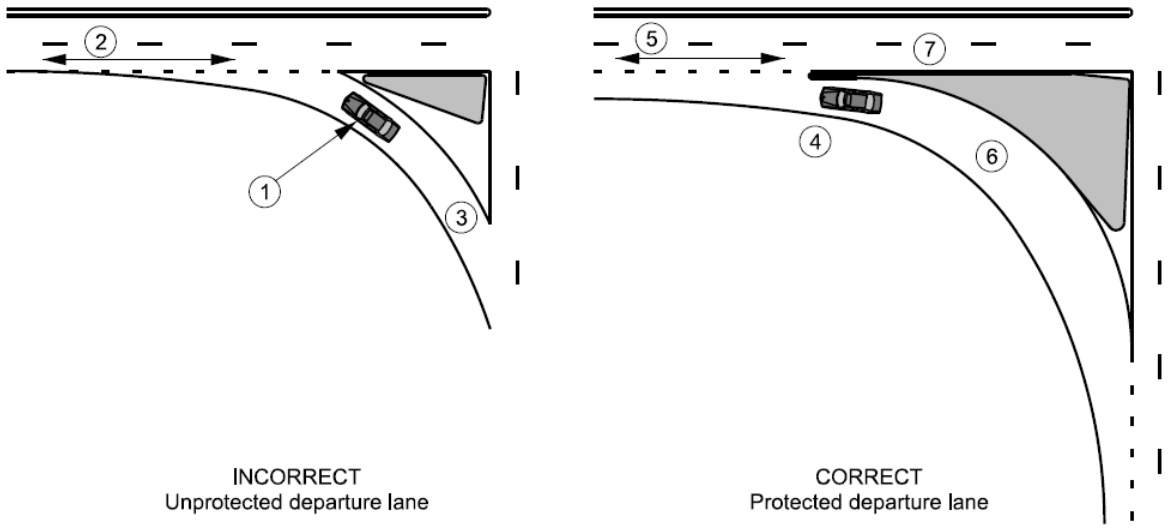
Commentary 7

If the island nose does not control the path of exiting vehicles the following will occur:

- The observation angle to approaching through traffic will be exceeded where the through approach is straight for a distance less than 5 sec of travel at the design speed.
- An inadequate acceleration taper will result.

These points are illustrated in Figure C7 1.

Figure C7 1: Incorrect and correct treatment of an unsignalised three centred kerb return using an island



Notes:

1. *Observation angle exceeded.*
2. *No acceleration lane.*
3. *High relative speed at point of conflict.*
4. *Observation angle requirements.*
5. *Adequate and protected acceleration lane.*
6. *Low relative speed at point of contact.*
7. *Details of nose treatment are given in Section 8.*

Source: Adapted from Department of Main Roads (2006) .

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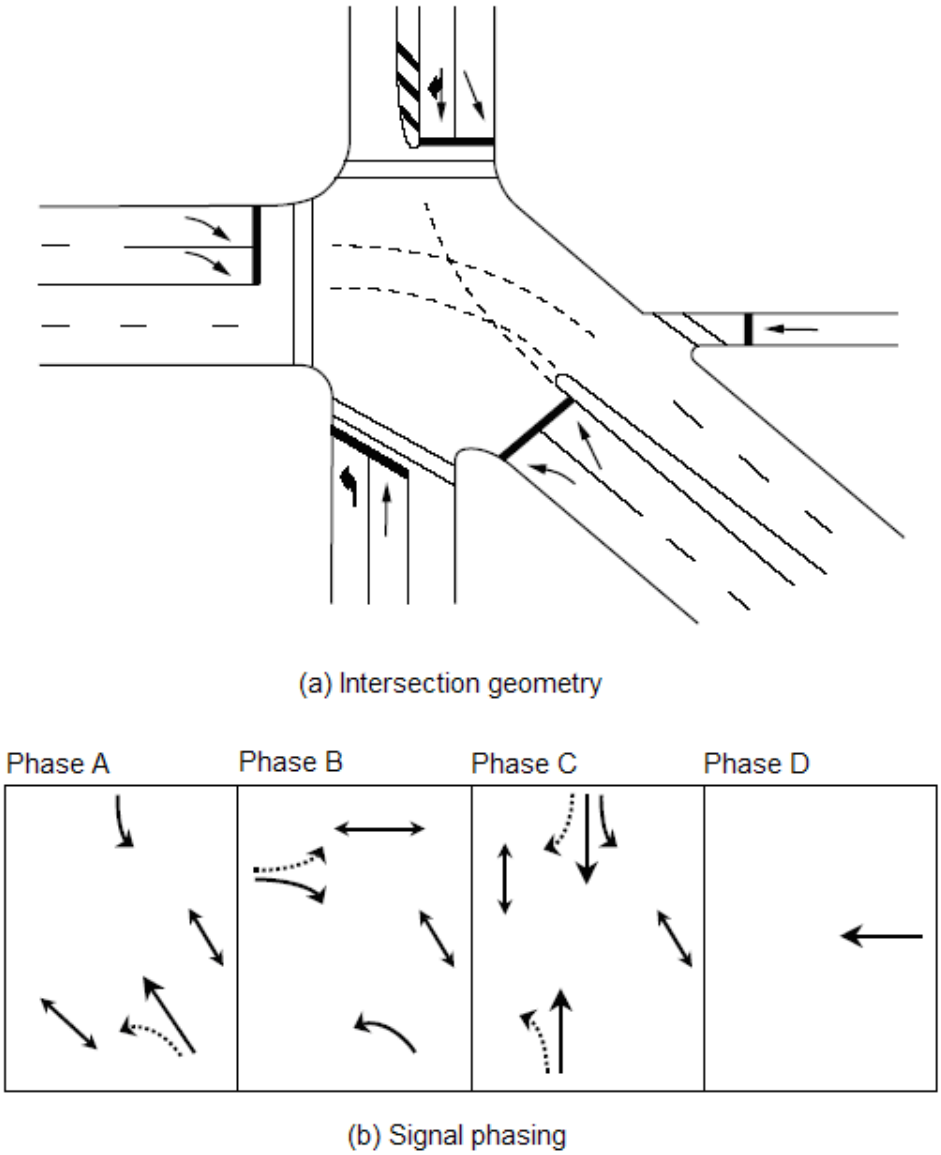
Commentary 8

Figure C8 1 shows an intersection where one approach intersects at an oblique angle, space is limited and a minor fifth leg has to be accommodated. The oblique leg complicates the design in that the through movement is not obvious from the layout and a shared right-turn/through lane is created. This geometric layout requires the use of the four signal phases as illustrated in Figure C8 1 and the use of appropriate signal displays (e.g. arrows inclined in the direction of the movements).

Figure C8 1: Intersection where the geometry influences signal phasing



Source: Austroads (2020c).

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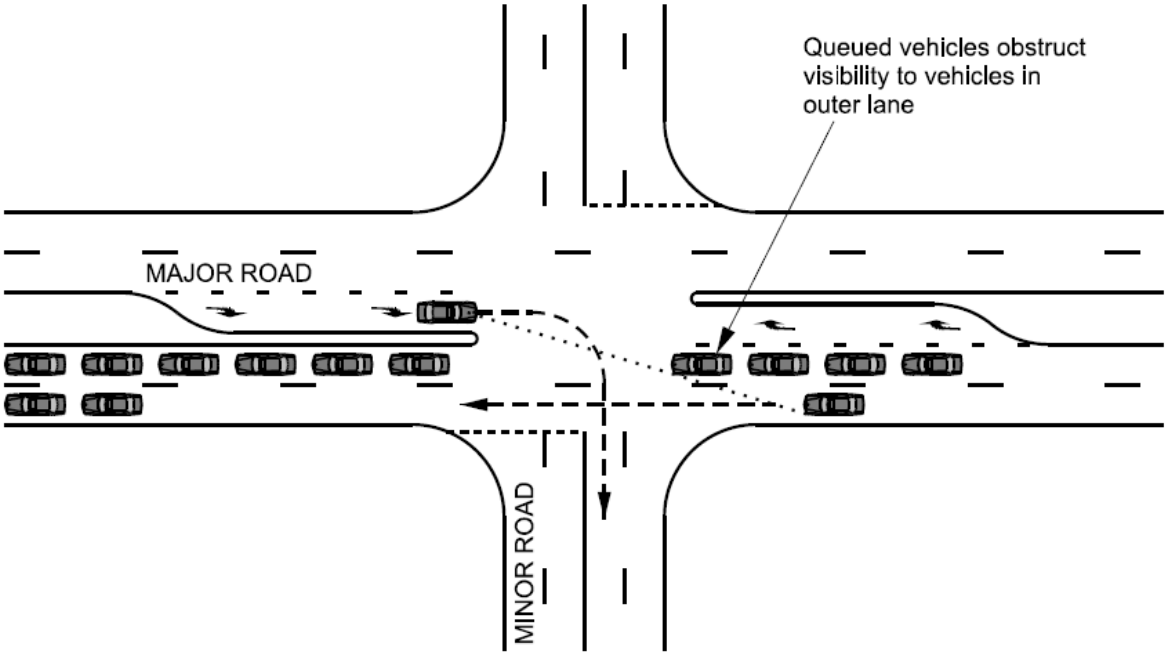
Commentary 9

Arndt (2004) found that angle-major vehicle crash rates (crashes resulting from a right-turning major road driver failing to give way and colliding with an oncoming vehicle – refer to Appendix D of the Austroads *Guide to Road Design Part 4: Intersections and Crossings – General* (AGRD Part 4) (Austroads 2023b)) 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 (e.g. an intersection with traffic signals). A diagram of this scenario is shown in Figure C9 1.

Figure C9 1: Queuing through an unsignalised intersection due to a downstream blockage



Source: Adapted from Department of Main Roads (2006).

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Guide to Road Design Part 4A: Unsignalised and Signalised Intersections provides road designers and other practitioners with guidance on the detailed geometric design of all at grade intersections (excluding roundabouts). It includes information on the types of unsignalised and signalised intersections and their use; an intersection layout design process and factors to be considered; and detailed geometric design requirements for various types of intersection.

Guide to Road Design Part 4A

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