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

*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.


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This third edition contains minor editorial and technical changes throughout the Guide, and substantial parts of Sections 1.4, 3.2, 4.3-4.14, 5.1, 5.2, 6.1, 7.1-7.5, 8, 9 and 10 have been transferred to *AGRD Part 4* (Austroads 2017).

Updates have been made throughout this edition to include new and updated reference material and cross-references to other Guides. Most figures have been redrawn to provide a consistent appearance. The major updates include:

- **Section 1: Introduction**: Updates to road safety, including the Safe System principles.
- **Section 3: Sight Distance**: Incorporates additional information relating to safe intersection sight distance and additional description in tables to assist in interpretation.
- **Section 5: Auxiliary Lanes**: Section 5.2 (new number), has information included from the commentaries on turning radii and vehicle speeds, and Table 5.4 contains additional information on acceleration lane lengths.
- **Section 6: Traffic Islands and Medians**: Information from Appendix D has been included in Section 6.2.

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Mr Peter Aumann - ARRB Group Ltd (Project Leader).

It is also acknowledged that this third edition has been updated from the first and second editions prepared by Gary Veith.
Summary

The 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 4B: Roundabouts (Austroads 2015c)
- Part 4C: Interchanges (Austroads 2015d).

In addition, road designers should also refer to the Austroads Guide to Traffic Management Part 6: Intersections, Interchanges and Crossings (Austroads 2013a) which provides guidance on the traffic management aspects of intersection design and road users’ requirements.
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1. Introduction

1.1 Purpose

Austroads Guide to Road Design seeks to capture the contemporary road design practice of member organisations (Guide to Road Design Part 1: Introduction to Road Design (Austroads 2015a)). In doing so, it provides valuable guidance to designers in the production of safe, economical and efficient road designs.

The Guide to Road Design Part 4: Intersections and Crossings: General (AGRD Part 4) (Austroads 2017) 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 2015c)
- Part 4C: Interchanges (AGRD Part 4C) (Austroads 2015d).

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 4, under which Part 4A is listed, is one of eight parts that comprise the Austroads Guide to Road Design. 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.
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 <www.austroads.com.au>.

The Guide to Traffic Management Part 6: Intersections, Interchanges and Crossings (AGTM Part 6) (Austroads 2013a) should be regarded as a related document to this part of the Guide to Road Design, 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 Guide to Road Design Part 2: Design Considerations (Austroads 2015b) for objectives in the design of intersections, interchanges and crossings.

1.3 Design Criteria in Part 4A

AGRD Part 4 (Austroads 2017) 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. It 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.

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 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.

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.

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 (Austroads 2015a).

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 Guide to Traffic Management Part 3: Traffic Studies and Analysis (Austroads 2013e).

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

<table>
<thead>
<tr>
<th>Design element</th>
<th>Key considerations</th>
<th>Reference</th>
</tr>
</thead>
<tbody>
<tr>
<td>Alignment of the approaches</td>
<td>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</td>
<td>AGRD Part 3 (Austroads 2016b)</td>
</tr>
<tr>
<td>Cross-sections</td>
<td>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)</td>
<td>Guide to Traffic Management Part 5 (Austroads 2014) AGRD Part 3</td>
</tr>
<tr>
<td>Traffic lanes (approach and departure) required for satisfactory operation and safety</td>
<td>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</td>
<td>Guide to Traffic Management Part 3 (Austroads 2013e) AGRM Part 6 (Austroads 2013a)</td>
</tr>
<tr>
<td>Roadside areas</td>
<td>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</td>
<td>Guide to Road Design: Part 5 (Austroads 2013b) Part 5A (Austroads 2013c) Part 5B (Austroads 2013d) Part 6 (Austroads 2010)</td>
</tr>
<tr>
<td>Pavement area and the location and shape of median noses and kerb returns</td>
<td>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</td>
<td>AGRD Part 4 (Austroads 2017)</td>
</tr>
<tr>
<td>Left-turn and right-turn treatments</td>
<td>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</td>
<td>Section 8 Section 7</td>
</tr>
</tbody>
</table>
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

<table>
<thead>
<tr>
<th>Design element</th>
<th>Key considerations</th>
<th>Reference</th>
</tr>
</thead>
</table>
| 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  
• impaired user access  
• 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 | Guide to Traffic Management Part 9 (Austroads 2016a)                                                                                                      |
<table>
<thead>
<tr>
<th>Design element</th>
<th>Key considerations</th>
<th>Reference</th>
</tr>
</thead>
</table>
| 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 | Guide to Traffic Management Part 3 (Austroads 2013e)  
Australian Road Rules  
National Transport Commission (2012) |
| 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 best site condition for an intersection is where all approaches are able to have straight horizontal alignments and relatively flat vertical alignments. 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 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).

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 seconds 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.
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 AGRD Part 3 (Austroads 2016b) 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.

---

1 Department of Main Roads (2006) has been superseded and Figure 2.1 has not been carried forward into Queensland Department of Transport and Main Roads (2016).
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.

---

2 Department of Main Roads (2006) has been superseded and Figure 2.2 has not been reproduced in Queensland Department of Transport and Main Roads (2016).
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.

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)3.

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.

---

3 Department of Main Roads (2006) has been superseded and Figure 2.3 has not been carried forward into Queensland Department of Transport and Main Roads (2016).
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).^4^.

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.

---

^4^ Department of Main Roads (2006) has been superseded and Figure 2.4 has not been carried forward into Queensland Department of Transport and Main Roads (2016).
2.2.3 Combined Horizontal and Vertical Curves

The coordination of horizontal and vertical curves is discussed in AGRD Part 3 (Austroads 2016b). 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.

Source: Based on Department of Main Roads (2006)\(^5\).

---

\(^5\) Department of Main Roads (2006) has been superseded and Figure 2.5 has not been carried forward into Queensland Department of Transport and Main Roads (2016).
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%.

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. 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.
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 view point 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 AGRD Part 3 (Austroads 2016b) 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.

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

\[
ASD = \frac{R_T \times V}{3.6} + \frac{V^2}{254 \times (d + 0.01 \times a)}
\]

where

\[
\begin{align*}
ASD & = \text{approach sight distance (m)} \\
R_T & = \text{reaction time (sec), refer to AGRD Part 3 (Austroads 2016b) for guidance on values} \\
V & = \text{operating (85th percentile) speed (km/h)} \\
d & = \text{coefficient of deceleration, refer to Table 3.3 and AGRD Part 3 for values} \\
a & = \text{a longitudinal grade in \% (in direction of travel: positive for uphill grade, negative for downhill grade)}
\end{align*}
\]
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* (Austroads 2016b).
Table 3.1: Approach sight distance (ASD) and corresponding minimum crest vertical curve size for sealed roads (S < L)

<table>
<thead>
<tr>
<th>Design speed (km/h)</th>
<th>Based on approach sight distance for a car(^{(1)})</th>
<th>(R_T = 1.5) sec(^{(3)})</th>
<th>(R_T = 2.0) sec</th>
<th>(R_T = 2.5) sec</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>ASD (m)</td>
<td>K</td>
<td>ASD (m)</td>
<td>K</td>
</tr>
<tr>
<td>40</td>
<td>34</td>
<td>5.3</td>
<td>40</td>
<td>7.2</td>
</tr>
<tr>
<td>50</td>
<td>48</td>
<td>10.5</td>
<td>55</td>
<td>13.8</td>
</tr>
<tr>
<td>60</td>
<td>64</td>
<td>18.8</td>
<td>73</td>
<td>24.0</td>
</tr>
<tr>
<td>70</td>
<td>83</td>
<td>31.1</td>
<td>92</td>
<td>38.9</td>
</tr>
<tr>
<td>80</td>
<td>103</td>
<td>48.5</td>
<td>114</td>
<td>59.5</td>
</tr>
<tr>
<td>90</td>
<td>126</td>
<td>72.3</td>
<td>139</td>
<td>87.3</td>
</tr>
<tr>
<td>100</td>
<td>151</td>
<td>104</td>
<td>165</td>
<td>124</td>
</tr>
<tr>
<td>110</td>
<td>–</td>
<td>–</td>
<td>193</td>
<td>171</td>
</tr>
<tr>
<td>120</td>
<td>–</td>
<td>–</td>
<td>224</td>
<td>229</td>
</tr>
<tr>
<td>130</td>
<td>–</td>
<td>–</td>
<td>257</td>
<td>301</td>
</tr>
</tbody>
</table>

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 2016b).

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:

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

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 2016b) 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
- 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 ($S < L$)

<table>
<thead>
<tr>
<th>Design speed (km/h)</th>
<th>Based on safe intersection sight distance for cars$^{(1)}$</th>
<th>$R_T = 1.5$ sec$^{(3)}$</th>
<th>$R_T = 2.0$ sec</th>
<th>$R_T = 2.5$ sec</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$h_1 = 1.1; h_2 = 1.25, d = 0.36^{(2)};$ Observation time = 3 sec</td>
<td>SISD (m)</td>
<td>K</td>
<td>SISD (m)</td>
</tr>
<tr>
<td>40</td>
<td>67</td>
<td>4.9</td>
<td></td>
<td>73</td>
</tr>
<tr>
<td>50</td>
<td>90</td>
<td>8.6</td>
<td></td>
<td>97</td>
</tr>
<tr>
<td>60</td>
<td>114</td>
<td>14</td>
<td></td>
<td>123</td>
</tr>
<tr>
<td>70</td>
<td>141</td>
<td>22</td>
<td></td>
<td>151</td>
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<tr>
<td>80</td>
<td>170</td>
<td>31</td>
<td></td>
<td>181</td>
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<tr>
<td>90</td>
<td>201</td>
<td>43</td>
<td></td>
<td>214</td>
</tr>
<tr>
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<td>–</td>
<td>–</td>
<td></td>
<td>248</td>
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<tr>
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<td>–</td>
<td></td>
<td>285</td>
</tr>
<tr>
<td>120</td>
<td>–</td>
<td>–</td>
<td></td>
<td>324</td>
</tr>
<tr>
<td>130</td>
<td>–</td>
<td>–</td>
<td></td>
<td>365</td>
</tr>
</tbody>
</table>

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 (Austroads 2016b). 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 3º 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)

<table>
<thead>
<tr>
<th>Design speed (major road) (km/h)</th>
<th>Correction (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Upgrade</td>
</tr>
<tr>
<td></td>
<td>2%</td>
</tr>
<tr>
<td>40</td>
<td>–1</td>
</tr>
<tr>
<td>50</td>
<td>–1</td>
</tr>
<tr>
<td>60</td>
<td>–2</td>
</tr>
<tr>
<td>70</td>
<td>–3</td>
</tr>
<tr>
<td>80</td>
<td>–4</td>
</tr>
<tr>
<td>90</td>
<td>–5</td>
</tr>
<tr>
<td>100</td>
<td>–6</td>
</tr>
<tr>
<td>110</td>
<td>–7</td>
</tr>
<tr>
<td>120</td>
<td>–8</td>
</tr>
<tr>
<td>130</td>
<td>–10</td>
</tr>
</tbody>
</table>

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

Source: Department of Main Roads (2006)\(^7\).

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\(^7\) Department of Main Roads (2006) has been superseded and Figure 3.3 has not been carried forward into Queensland Department of Transport and Main Roads (2016).
3.2.3 Minimum Gap Sight Distance

**General**

Minimum gap sight distance (MGSD)\(^8\) 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 *Guide to Traffic Management Part 3: Traffic Studies and Analysis* (Austroads 2013e).

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 120° 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)*\(^9\).
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)\(^\text{10}\).

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.

\(^{10}\) Department of Main Roads (2006) has been superseded and Figure 3.5 has not been carried forward into Queensland Department of Transport and Main Roads (2016).
Table 3.5: Critical acceptance gaps and follow-up headways

<table>
<thead>
<tr>
<th>Movement</th>
<th>Diagram</th>
<th>Description</th>
<th>( t_a^{(1)} ) (sec)</th>
<th>( t_f^{(2)} ) (sec)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Left turn</td>
<td><img src="image1" alt="Diagram" /></td>
<td>Not interfering with A&lt;br&gt;Requiring A to slow</td>
<td>14–40&lt;br&gt;5</td>
<td>2–3&lt;br&gt;2–3</td>
</tr>
<tr>
<td>Crossing</td>
<td><img src="image2" alt="Diagram" /></td>
<td>Two lane/one way&lt;br&gt;Three lane/one way&lt;br&gt;Four lane/one way&lt;br&gt;Two lane/two way&lt;br&gt;Four lane/two way&lt;br&gt;Six lane/two way</td>
<td>4&lt;br&gt;6&lt;br&gt;8&lt;br&gt;5&lt;br&gt;8&lt;br&gt;8</td>
<td>2&lt;br&gt;3&lt;br&gt;4&lt;br&gt;3&lt;br&gt;5&lt;br&gt;5</td>
</tr>
<tr>
<td>Right turn from major road</td>
<td><img src="image3" alt="Diagram" /></td>
<td>Across one lane&lt;br&gt;Across two lanes&lt;br&gt;Across three lanes</td>
<td>4&lt;br&gt;5&lt;br&gt;6</td>
<td>2&lt;br&gt;3&lt;br&gt;4</td>
</tr>
<tr>
<td>Right turn from minor road</td>
<td><img src="image4" alt="Diagram" /></td>
<td>Not interfering with A&lt;br&gt;One way&lt;br&gt;Two lane/two way&lt;br&gt;Four lane/two way&lt;br&gt;Six lane/two way</td>
<td>14–40&lt;br&gt;3&lt;br&gt;5&lt;br&gt;8&lt;br&gt;8</td>
<td>3&lt;br&gt;3&lt;br&gt;3&lt;br&gt;5&lt;br&gt;5</td>
</tr>
<tr>
<td>Merge</td>
<td><img src="image5" alt="Diagram" /></td>
<td>Acceleration lane</td>
<td>3</td>
<td>2</td>
</tr>
</tbody>
</table>

1  \( t_a \) = critical acceptance gap (sec).<br>2  \( t_f \) = follow-up headway (sec).


Source: Department of Main Roads (2006)\(^\text{11}\).

Table 3.6: Table of minimum gap sight distances (‘D’ metres) for various speeds

<table>
<thead>
<tr>
<th>Critical gap acceptance time ((t_a)) (secs)</th>
<th>85(^{\text{th}}) percentile speed of approaching vehicle (km/h)</th>
<th>10</th>
<th>20</th>
<th>30</th>
<th>40</th>
<th>50</th>
<th>60</th>
<th>70</th>
<th>80</th>
<th>90</th>
<th>100</th>
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<tbody>
<tr>
<td>4</td>
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<td>122</td>
</tr>
<tr>
<td>5</td>
<td></td>
<td>14</td>
<td>28</td>
<td>42</td>
<td>55</td>
<td>69</td>
<td>83</td>
<td>97</td>
<td>111</td>
<td>125</td>
<td>139</td>
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<td>6</td>
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<td>75</td>
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<td>225</td>
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<td>139</td>
<td>167</td>
<td>194</td>
<td>222</td>
<td>250</td>
<td>278</td>
<td>305</td>
</tr>
</tbody>
</table>

\( \text{11 Department of Main roads (2006) has been superseded and Table 3.5 has not been carried forward into Queensland Department of Transport and Main Roads (2016).} \)
**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.

- **observation angle for new or reconstructed work maximum 120º.**
- **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 seconds at design speed (V km/h).

Minimum distance travelled by approaching vehicle in 5 seconds based on ta for left-turn from Table 3.5.

**Source:** Department of Main Roads (2006).12

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

[see Commentary 1]

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12 Department of Main Roads (2006) has been superseded and Figure 3.6 has not been carried forward into Queensland Department of Transport and Main Roads (2016).
3.3 Pedestrian Sight Distance Requirements

At intersections, pedestrian crossing facilities should be located where there is a clear view between approaching drivers and pedestrians on the crossing or waiting to cross a roadway (Figure 3.7).

This requires that:

- **ASD** should be provided between approaching vehicles (1.1 m eye height) and the surface of the roadway (0 m) at the crossing.

- **Crossing sight distance (CSD)** should be provided between approaching vehicles (1.1 m eye height) and a pedestrian waiting to cross the road. 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 is:

- necessary to ensure that the pedestrian can see approaching traffic in sufficient time to judge a safe gap and cross the roadway
- calculated from the critical safe gap (in the traffic stream) and the speed of approaching traffic
- given by Equation 3:

\[ CSD = t_c \times \frac{V}{3.6} \]

where:

- **CSD** = sight distance required for a pedestrian to safely cross the roadway
- **t_c** = critical safe gap (sec) = (crossing length/walking speed)
- **V** = 85th percentile approach speed (km/h).

Note: 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.

It is important that the line of sight is not obstructed. Provision of ASD (1.1 m to 0.0 m) 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. Provision of ASD should be used for crossings where the pedestrian has the priority.

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.

CSD should be provided at crossings where the pedestrian does not have the priority, to allow sufficient time to cross the road, clear of any approaching traffic.

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.
3.4 Sight Distance at Property Entrances

AGRD Part 4 (Austroads 2017) 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.

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13 Department of Main Roads (2006) has been superseded and Figure 3.7 has not been carried forward into Queensland Department of Transport and Main Roads (2016).
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 *AGTM Part 6* (Austroads 2013a).

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 need exists.

Where no bicycle facility is provided, the shoulders of major rural roads should be of an appropriate width to cater for cyclists (*AGRD Part 3* (Austroads 2016b)) 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 (Austroads 2013a).
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 *AGTM Part 6* (Austroads 2013a).

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.

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:

\[ B = \text{total length of auxiliary lane} \]
\[ D = \text{deceleration length (m)} \]
\[ L_d = \text{diverge length (m)} \]
\[ S = \text{storage length (m)} \]
\[ T = \text{physical lane taper length (m)} \]
\[ P = \text{length of parallel lane for deceleration (m)}. \]

Figure 5.1: Components of a deceleration turning lane

(a) Deceleration to a stop condition

(b) Deceleration to a turning speed

Source: Based on Department of Main Roads (2006)\(^{14}\).

\(^{14}\) Department of Main Roads (2006) has been superseded and Figure 5.1 has not been carried forward into Queensland Department for Transport and Main Roads (2016).
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:

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

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 (Ld). 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**

<table>
<thead>
<tr>
<th>Design speed of approach (km/h)</th>
<th>Taper length T (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>50</td>
<td>15</td>
</tr>
<tr>
<td>60</td>
<td>20</td>
</tr>
<tr>
<td>70</td>
<td>23</td>
</tr>
<tr>
<td>80</td>
<td>25</td>
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<tr>
<td>90</td>
<td>30</td>
</tr>
<tr>
<td>100</td>
<td>33</td>
</tr>
<tr>
<td>110</td>
<td>35</td>
</tr>
</tbody>
</table>

*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:

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

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 should be used.
- The column for a maximum design deceleration rate of 3.5 m/sec 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 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

<table>
<thead>
<tr>
<th>Design speed of approach road (km/h)</th>
<th>Length of deceleration D – including diverge taper $T$ (m)</th>
<th>Diverge length $L_d$ for lane widths (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Stop condition$^{(1)}$ (m)</td>
<td>Design speed of exit curve (km/h)$^{(2)}$</td>
</tr>
<tr>
<td></td>
<td>Comfortable 2.5 m/s$^2$</td>
<td>Maximum 3.5 m/s$^2$</td>
</tr>
<tr>
<td>50</td>
<td>40</td>
<td>30</td>
</tr>
<tr>
<td>60</td>
<td>55</td>
<td>40</td>
</tr>
<tr>
<td>70</td>
<td>75</td>
<td>55</td>
</tr>
<tr>
<td>80</td>
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<td>90</td>
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<tr>
<td>100</td>
<td>155</td>
<td>110</td>
</tr>
<tr>
<td>110</td>
<td>185</td>
<td>135</td>
</tr>
</tbody>
</table>

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)$^{15}$.

The deceleration distance determined from Table 5.2 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

<table>
<thead>
<tr>
<th>Grade</th>
<th>Ratio of ‘length on grade’ to ‘length on level’</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Upgrade</td>
</tr>
<tr>
<td>0–2%</td>
<td>1.0</td>
</tr>
<tr>
<td>3–4%</td>
<td>0.9</td>
</tr>
<tr>
<td>5–6%</td>
<td>0.8</td>
</tr>
</tbody>
</table>

Source: Department of Main Roads (2006)\textsuperscript{16}.

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 6).

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:

$$R = \frac{V^2}{127(e + f)} \quad \text{(6)}$$

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

<table>
<thead>
<tr>
<th>$V$ (km/h)</th>
<th>$f$ – maximum</th>
<th>$f$ – desirable</th>
</tr>
</thead>
<tbody>
<tr>
<td>25</td>
<td>0.36</td>
<td>0.30</td>
</tr>
<tr>
<td>30</td>
<td>0.36</td>
<td>0.30</td>
</tr>
<tr>
<td>40</td>
<td>0.36</td>
<td>0.30</td>
</tr>
<tr>
<td>50</td>
<td>0.35</td>
<td>0.30</td>
</tr>
<tr>
<td>60</td>
<td>0.33</td>
<td>0.24</td>
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<tr>
<td>70</td>
<td>0.31</td>
<td>0.19</td>
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<tr>
<td>80</td>
<td>0.26</td>
<td>0.16</td>
</tr>
<tr>
<td>90</td>
<td>0.20</td>
<td>0.13</td>
</tr>
<tr>
<td>100</td>
<td>0.16</td>
<td>0.12</td>
</tr>
<tr>
<td>110</td>
<td>0.12</td>
<td>0.12</td>
</tr>
<tr>
<td>120</td>
<td>0.11</td>
<td>0.11</td>
</tr>
<tr>
<td>130</td>
<td>0.11</td>
<td>0.11</td>
</tr>
</tbody>
</table>

Source: Derived from Department of Main Roads (2006)\textsuperscript{17} and AGRD Part 3 (Austroads 2016b).

\textsuperscript{16} Department of Main Roads (2006) has been superseded and Table 5.3 has not been carried forward into Queensland Department of Transport and Main Roads (2014).

\textsuperscript{17} Department of Main Roads (2006) has been superseded and Table 5.4 has not been carried forward into Queensland Department of Transport and Main Roads (2016).
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).\(^\text{18}\)

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.

\(^{18}\) Department of Main Roads (2006) has been superseded and Figure 5.2 has not been carried forward into Queensland Department of Transport and Main Roads (2016).
**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.
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.

---

19 Department of Main Roads (2006) has been superseded and Figure 5.3 has not been carried forward into Queensland Department of Transport and Main Roads (2016).
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

(a) Acceleration lane transition

(b) Added through lane

1 See Figure 5.2 for turning speeds.

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

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 seconds 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.

---

20 Department of Main Roads (2016) has been superseded and Figure 5.4 has not been carried forward into Queensland Department of Transport and Main Roads (2016).
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

<table>
<thead>
<tr>
<th>Design speed of road entered (km/h)</th>
<th>Length of acceleration lane $A$ (m) (including length of merge taper)</th>
<th>Design speed of entry curve (km/h)</th>
<th>4 sec travel (m)</th>
<th>Merge $T_m$ (m)</th>
<th>Min. desirable length 4 sec + $T_m$ (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0(2)</td>
<td>20</td>
<td>30</td>
<td>40</td>
<td>50</td>
<td>60</td>
</tr>
<tr>
<td>50</td>
<td>70</td>
<td>55</td>
<td>45</td>
<td>30</td>
<td>20</td>
</tr>
<tr>
<td>60</td>
<td>110</td>
<td>95</td>
<td>85</td>
<td>70</td>
<td>40</td>
</tr>
<tr>
<td>70</td>
<td>165</td>
<td>150</td>
<td>140</td>
<td>125</td>
<td>95</td>
</tr>
<tr>
<td>80</td>
<td>235</td>
<td>220</td>
<td>210</td>
<td>195</td>
<td>165</td>
</tr>
<tr>
<td>90</td>
<td>330</td>
<td>315</td>
<td>305</td>
<td>290</td>
<td>260</td>
</tr>
<tr>
<td>100</td>
<td>450</td>
<td>435</td>
<td>425</td>
<td>410</td>
<td>380</td>
</tr>
<tr>
<td>110</td>
<td>610</td>
<td>595</td>
<td>585</td>
<td>570</td>
<td>540</td>
</tr>
</tbody>
</table>

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.

Table 5.6: Correction of acceleration distances as a result of grade

<table>
<thead>
<tr>
<th>Design speed of road entered (km/h)</th>
<th>Ratio of length on grade to length on level(^1) for:</th>
<th>3 to 4% upgrade</th>
<th>5 to 6% upgrade</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Design speed of turning roadway curve (km/h)</td>
<td>Stop 30 50 60 80</td>
<td>Stop 30 50 60 80</td>
</tr>
<tr>
<td>50</td>
<td>Stop 30 50 60 80</td>
<td>1.4 1.5</td>
<td>1.5 1.5 1.5</td>
</tr>
<tr>
<td>60</td>
<td>1.3 1.3 1.3 1.3</td>
<td>1.5 1.5 1.5 1.5</td>
<td>1.5 1.5 1.7 1.9</td>
</tr>
<tr>
<td>80</td>
<td>1.3 1.3 1.4 1.4</td>
<td>1.5 1.5 1.7 1.9</td>
<td>1.5 1.5 1.7 1.9</td>
</tr>
<tr>
<td>100</td>
<td>1.3 1.5 1.6 1.6</td>
<td>1.6 1.7 1.9 2.2</td>
<td>1.5 1.5 1.7 2.5</td>
</tr>
<tr>
<td>110</td>
<td>1.4 1.5 1.6 1.8</td>
<td>1.5 1.5 1.7 2.5</td>
<td>1.5 1.5 1.7 2.5</td>
</tr>
<tr>
<td></td>
<td>3 to 4% downgrade All speeds</td>
<td>0.70 0.65 0.60 0.60</td>
<td>0.60 0.60 0.60 0.60</td>
</tr>
<tr>
<td></td>
<td>5 to 6% downgrade All speeds</td>
<td>0.60 0.60 0.60 0.60</td>
<td>0.55 0.55 0.55 0.55</td>
</tr>
</tbody>
</table>

\(^1\) Ratio from this Table multiplied by length in Table 5.5 gives length of speed change lane on grade.


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 \(\text{V km/h}\). 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.

[see 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 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, it may be relatively easy for through drivers to perceive the slow movement of these vehicles and to slow for them. Alternatively, where traffic volumes on the road and the number of trucks entering are relatively high, it may be preferable to provide an acceleration lane 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.

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>
<thead>
<tr>
<th>Downgrade (%)</th>
<th>100</th>
<th>90</th>
<th>80</th>
<th>70</th>
<th>60</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>2400</td>
<td>1500</td>
<td>910</td>
<td>550</td>
<td>320</td>
</tr>
<tr>
<td>1</td>
<td>1400</td>
<td>940</td>
<td>640</td>
<td>410</td>
<td>250</td>
</tr>
<tr>
<td>2</td>
<td>970</td>
<td>700</td>
<td>500</td>
<td>330</td>
<td>210</td>
</tr>
<tr>
<td>3</td>
<td>760</td>
<td>560</td>
<td>400</td>
<td>280</td>
<td>180</td>
</tr>
</tbody>
</table>

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>
<thead>
<tr>
<th>Upgrade (%)</th>
<th>100</th>
<th>90</th>
<th>80</th>
<th>70</th>
<th>60</th>
<th>50</th>
<th>40</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>–</td>
<td>–</td>
<td>2000</td>
<td>890</td>
<td>480</td>
<td>230</td>
<td>100</td>
</tr>
<tr>
<td>2</td>
<td>–</td>
<td>–</td>
<td>–</td>
<td>–</td>
<td>890</td>
<td>320</td>
<td>130</td>
</tr>
</tbody>
</table>

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 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. In practical terms this approximates the numerical value of the design speed, $V$ (based on a 3.5 m wide lane and $V$ is in km/h).

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

**Figure 5.5: Development of auxiliary through lane approach**

- $R^*$ = edge of pavement rounding
  - (100 m for deceleration lane)
  - (250 m for added through lane)

$T_m$ based on lateral shift

1.0 m/s for added through lane

$T_m$ = 50 m max – rural

30 m max - urban

} for deceleration lane

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**

6. Traffic Islands and Medians

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 6.3). 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 2013f).

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 m² 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.

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 6.1 for L.
- For widths of traffic islands or medians refer to Table 6.2.
- Source: Based on Department of Main Roads (2006)\(^\text{21}\).

\(^\text{21}\) Department of Main Roads (2006) has been superseded and Figure 6.1 has not been carried forward into Queensland Department of Transport and Main Roads (2016).
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 6.1 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 6.1. 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 6.1 and Table 6.2 can be used to determine a minimum area for median (splitter) islands, for example:

- urban unsignalised – 8.0 m² 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 m² 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 m² 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>
<thead>
<tr>
<th>Table 6.1: Minimum median island length (L)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Design speed (km/h)</td>
</tr>
<tr>
<td>---------------------</td>
</tr>
<tr>
<td>60</td>
</tr>
<tr>
<td>80</td>
</tr>
<tr>
<td>100</td>
</tr>
</tbody>
</table>
Table 6.2: Residual median island widths at urban intersections (W)

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

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.
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.
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.

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22 Department of Main Roads (2006) has been superseded and Figure 6.3 has not been carried forward into Queensland Department of Transport and Main Roads (2016).
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 kerb-line. 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.

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 m². 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 AGRD Part 4 (Austroads 2017).

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.
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.
- **\( O_A \)** = 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); Exception: for 80 km/h use 2.0 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. \( 2 \times R_M \)).

Source: Based on Department of Main Roads (2006)\(^{23} \).

\(^{23}\) Department of Main Roads (2006) has been superseded and Figure 6.4 has not been carried forward into Queensland Department of Transport and Main Roads (2016).
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 \( O_A \), \( 2 \times R_1 \), 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:

\[
L_A = (O_1 + C_X + W_P + C_S + 6.0)
\]

\[
L_1 = A + B = (L_A - R_1) \tan 20^\circ + (O_A + 2 \times R_1 + 1.0) \text{ m.}
\]

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 \( C_X = R_M = 1.0 \text{ m} \) and \( O_A = 0.2 \times 60/10 = 1.2 \text{ m} \), and:

\[
L_A = (O_1 + C_X + W_P + C_S + 6.0)
\]

\[
= (1.0 + 1.0 + 2.4 + 1.2 + 6.0)
\]

\[
= 11.6 \text{ m}
\]

\[
L_1 = A + B = (L_A - R_1) \tan 20^\circ + (O_A + 2 \times R_1 + 1.0) \text{ m}
\]

\[
= 11.6 \times 0.364 + (1.2 + 2.0 + 1.0)
\]

\[
= 4.2 + 4.2
\]

\[
= 8.4 \text{ m}
\]

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 Cs should be 2.0 m. Bicycle lane widths are provided in AGRD Part 3 (Austroads 2016b).

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 ° – 70° 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.
- $R_2 = R_1 + W$.
- $S$ – 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**

*Radius may be reduced for wider slip lanes and departures (verify with turning templates)*

- Offset between edge line and island widened to allow B-double turning path
- Island layout where B-doubles are the design vehicle

- Island, minimum area 8 m². For design criteria, refer to Section 8.3
- Radius to suit entry speed approximated from deceleration. See Table 5.2 and Figure 5.2
- Auxiliary lane, refer to Section 5
Alternative B2 – CHL turn treatment with acceleration lane – urban

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

<table>
<thead>
<tr>
<th>$R_1$</th>
<th>$O_1$ for all angles</th>
<th>$O_2$ for $\Theta^\circ$</th>
<th>$R_2$</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>$60^\circ$</td>
<td>$90^\circ$</td>
</tr>
<tr>
<td>8</td>
<td>2.4</td>
<td>9.2</td>
<td>9.7</td>
</tr>
<tr>
<td>10</td>
<td>2.2</td>
<td>8.8</td>
<td>9.2</td>
</tr>
<tr>
<td>12</td>
<td>2.0</td>
<td>8.4</td>
<td>8.7</td>
</tr>
<tr>
<td>14</td>
<td>1.9</td>
<td>8.1</td>
<td>8.3</td>
</tr>
<tr>
<td>16</td>
<td>1.8</td>
<td>7.8</td>
<td>8.0</td>
</tr>
<tr>
<td>18</td>
<td>1.7</td>
<td>7.5</td>
<td>7.7</td>
</tr>
<tr>
<td>20</td>
<td>1.6</td>
<td>7.3</td>
<td>7.4</td>
</tr>
<tr>
<td>25</td>
<td>1.4</td>
<td>6.8</td>
<td>6.9</td>
</tr>
<tr>
<td>30</td>
<td>1.3</td>
<td>6.5</td>
<td>6.5</td>
</tr>
</tbody>
</table>

$R < 4$ 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

<table>
<thead>
<tr>
<th>Ø</th>
<th>70°</th>
<th>90°</th>
</tr>
</thead>
<tbody>
<tr>
<td>R₁ (m)</td>
<td>17</td>
<td>15</td>
</tr>
</tbody>
</table>

Offset between edge line and island widened to allow B-double turning path

Island layout where B-doubles are the design vehicle
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 6.1. 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.

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.

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.


24 Department of Main Roads (2006) has been superseded and Figure 6.12 has not been carried forward into Queensland Department of Transport and Main Roads (2016).
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 6.3. This is the clearance from an island to a lane that is parallel to the island. Refer also to AGRD Part 3 (Austroads 2016b) for further information.

Table 6.3: Clearance between raised islands or medians and edge of traffic lane

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

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)25.

25 Department of Main Roads (2006) has been superseded and Figure 6.13 has not been carried forward into Queensland Department of Transport and Main Roads (2016).
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.

Where a safety barrier is placed close to the edge of the road it is important that the clearance available from the edge of the traffic lane to the barrier meets the ‘shy line’ requirements in the Guide to Road Design Part 6: Roadside Design, Safety and Barriers (Austroads 2010).

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 Guide to Road Design Part 5A: Drainage: Road Surface, Networks, Basins and Subsurface (Austroads 2013c).
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

This section contains guidance on the design of right-turn treatments. For information on the types and selection of right-turn treatments refer to AGTM Part 6 (Austroads 2013a) and AGRD Part 4 (Austroads 2017).

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 AGRD Part 4.

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-left Staggered T-intersection

**Basic two-lane two-way road**

This layout (Figure 7.1) 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 7.1: Right-left staggered T-intersection on a two-lane rural road (low turning volume)

Source: Based on Department of Main Roads (2006)26.

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 (Austroads 2013a) and illustrated in Figure 7.2. 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 7.2 may also be applicable in some urban situations.

26 Department of Main Roads (2006) has been superseded and Figure 7.1 has not been carried forward into Queensland Department of Transport and Main Roads (2016).
Figure 7.2: Two stage crossing on a rural road

Notes:
- An offset right-turn lane, refer to AGTM Part 6 (Austroads 2013a) 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:
  \[ W = \text{Nominal through lane width (m) (including widening for curves).} \]
  \[ W_T = \text{Nominal width of turn lane (m), including widening for curves based on the design turning vehicle. Desirable minimum } = W, \text{ absolute minimum } = 3.0 \text{ m.} \]
  \[ D = \text{Diverge/deceleration length including taper – Table 5.2. Adjust for grade using the ‘correction to grade’ factor in Table 5.3.} \]
  \[ T = \text{Taper length (m) is given by Equation 5 being: } T = \frac{0.33VW_T}{3.6} \]
  \[ S = \text{Storage length (m) is the greater of: the length of one design turning vehicle or (calculated car spaces –1) x 8 m (Guide to Traffic Management Part 3: Traffic Studies and Analysis, (Austroads 2013e)), or use computer program e.g. aaSIDRA.} \]
  \[ V = \text{Design speed of major road approach (km/h).} \]

Note: This layout is not used in New Zealand.

Source: Department of Main Roads (2006)\textsuperscript{27}.

\textsuperscript{27} Department of Main Roads (2006) has been superseded and Figure 7.2 has not been carried forward into Queensland Department of Transport and Main Roads (2016).
7.3.2 Left-right Staggered T – Divided Road

Overlapping right turns on a divided road

Figure 7.3 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 7.3: Left-right staggered T-intersection on a divided rural road with overlapping right turns

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. \((\text{calculated car spaces} - 1) \times 8 \text{ m or use computer program (e.g. aaSIDRA)}\).
- \( X \) = Distance based on design vehicle turning path, refer to Design Vehicles and Turning Path Templates Austroads (2013f).

Source: Department of Main Roads (2006)\(^{28}\).

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\(^{28}\) Department of Main Roads (2006) has been superseded and Figure 7.3 has not been carried forward into Queensland Department of Transport and Main Roads (2016).
7.3.3 Back-to-back Right Turns on a Divided Road

This treatment is shown in Figure 7.4. 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 7.4: 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
  - \((\text{calculated car spaces} - 1) \times 8\) m (Guide to Traffic Management Part 3: Traffic Studies and Analysis, (Austroads 2013e), 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 (Austroads 2013f).

Source: Based on Department of Main Roads (2006)\(^{29}\).

\(^{29}\) Department of Main Roads (2006) has been superseded and Figure 7.4 has not been carried forward into Queensland Department of Transport and Main Roads (2016).
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 7.5) 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.
Note: The dimensions of the treatment are defined thus:

- **W** = Nominal through lane width (m) (including widening for curves).
- **WT** = 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) = \( \frac{0.33V_{WT}}{3.6} \)
- **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 (Guide to Traffic Management Part 3: Traffic Studies and Analysis, (Austroads 2013e)), 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 7.6 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 7.6: 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 dimensions of the treatment are defined thus:

\[ W = \text{Nominal through lane width (m) (including widening for curves). Width to be continuous through the intersection.} \]

\[ C = \begin{cases} 6.0 \text{ m minimum} \\ 6.5 \text{ m minimum for 19 m semi-trailers and B-doubles} \\ 7.0 \text{ m minimum for Type 1 and Type 2 road trains} \end{cases} \]

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.

\[ V = \text{Design speed of major road approach (km/h).} \]

\[ S = \text{Storage length to cater for one design turning vehicle (m) (minimum length 12.5 m).} \]

\[ X = \text{Distance based on design vehicle turning path, refer to Design Vehicles and Turning Path Templates (Austroads 2013f).} \]

Source: Department of Main Roads (2006)30.

A turning radius in accordance with Design Vehicles and Turning Path Templates (Austroads 2013f) 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 (Austroads 2013a) and AGRD Part 4 (Austroads 2017) for further information.

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30 Department of Main Roads (2006) has been superseded and Figure 7.6 has not been carried forward into Queensland Department of Transport and Main Roads (2016).
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 7.7. 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 7.7: 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 dimensions of the treatment are defined thus:

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

\[ W_T = \text{Nominal width of turn lane (m) (incl. widening for curves based on the design turning vehicle) = 3.0 m minimum.} \]

\[ B = \text{Total length of auxiliary lane including taper, diverge/deceleration and storage (m).} \]

\[ E = \text{Distance from start of taper to 2.0 m width (m) = } (A/W_T) \times 2. \]

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

\[ R = \text{Radius (m).} \]

\[ S = \text{Storage length to cater for one design turning vehicle (m).} \]

\[ V = \text{Design speed of major road approach (km/h).} \]

\[ X = \text{Distance based on design vehicle turning path, refer to Design Vehicles and Turning Path Templates (Austroads 2013f).} \]

Note: Values of A, D, R and T are shown in Table 7.1.

Source: Department of Main Roads (2006)\textsuperscript{31}.

\textsuperscript{31} Department of Main Roads (2006) has been superseded and Figure 7.7 has not been carried forward into Queensland Department of Transport and Main Roads (2016).
Table 7.1: Dimensions of urban CHR(S) treatment for various design speeds

<table>
<thead>
<tr>
<th>Design speed of major road approach (km/h)</th>
<th>Lateral movement length $A$ (m)$^{(1)}$</th>
<th>Diverge/deceleration length $D$ (m)$^{(2)}$</th>
<th>Desirable radius $R$ (m)</th>
<th>Taper length $T$ (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>50</td>
<td>40$^{(3)}$</td>
<td>15</td>
<td>110</td>
<td>15</td>
</tr>
<tr>
<td>60</td>
<td>50$^{(3)}$</td>
<td>25</td>
<td>175</td>
<td>15</td>
</tr>
<tr>
<td>70</td>
<td>60</td>
<td>35</td>
<td>240</td>
<td>20</td>
</tr>
<tr>
<td>80</td>
<td>65</td>
<td>45</td>
<td>280</td>
<td>20</td>
</tr>
<tr>
<td>90</td>
<td>75</td>
<td>55</td>
<td>350</td>
<td>25</td>
</tr>
</tbody>
</table>

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)$^{32}$.

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 7.8. 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).

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32 Department of Main Roads (2006) has been superseded and Table 7.1 has not been carried forward into Queensland Department of Transport and Main Roads (2016).
Figure 7.8: 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 dimensions of the treatment are defined thus:
  \[ W = \text{Nominal through lane width (m) (including widening for curves).} \]
  \[ W_T = \text{Nominal width of turn lane (m), including widening for curves based on the design turning vehicle.} \]
  Desirable minimum = \( W \), absolute minimum = \( 3.0 \) m.
  \[ B = \text{Total length of auxiliary lane including taper, diverge/deceleration and storage (m).} \]
  \[ D = \text{Diverge/deceleration length including taper – Table 5.2 (adjust for grade using the ‘correction to grade’ factor in Table 5.3).} \]
  \[ T = \text{Physical taper length (m) given by Equation 5 being:} \]
  \[ T = \frac{0.33VW_T}{3.6} \]
  \[ S = \text{Storage length (m) is the greater of:} \]
  1. length of one design turning vehicle
  2. (calculated car spaces –1) x 8 m or use computer program (e.g. aaSIDRA).
  \[ V = \text{Design speed of major road approach (km/h).} \]
  \[ X = \text{Distance based on design vehicle turning path, refer to Design Vehicles and Turning Path Templates (Austroads 2013f).} \]

Source: Department of Main Roads (2006)\(^{33}\).

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 (Austroads 2013a). 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.

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\(^{33}\) Department of Main Roads (2006) has been superseded and Figure 7.8 has not been carried forward into Queensland Department of Transport and Main Roads (2016).
8. **Left-turn Treatments**

This section contains guidance on the design of left-turn treatments. For information on the types and selection of left-turn treatments refer to *AGTM Part 6* (Austroads 2013a) and *AGRD Part 4* (Austroads 2017).

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 (*AGRD Part 4* (Austroads 2017))
- 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

<table>
<thead>
<tr>
<th>V (km/h)</th>
<th>D (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>60</td>
<td>85</td>
</tr>
<tr>
<td>70</td>
<td>100</td>
</tr>
<tr>
<td>80</td>
<td>115</td>
</tr>
</tbody>
</table>

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)34.

34 Department of Main Roads (2006) has been superseded and Figure 8.1 has not been carried forward into Queensland Department of Transport and Main Roads (2016).
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:
    
    \[
    W = \text{Nominal through lane width (m) (including widening for curves)}.
    \]
    
    \[
    C = \text{On straights – 6.0 m minimum.}
    \]
    
    \[
    \text{On curves – 6.0 m plus curve widening (based on widening for the design turning vehicle plus widening for the design through vehicle).}
    \]
    
    \[
    A = \frac{0.5VF}{3.6}
    \]
    
    \[
    V = \text{Design speed of major road approach (km/h)}.
    \]
    
    \[
    F = \text{Formation/carriageway widening (m)}.
    \]
    
    \[
    P = \text{Minimum length of parallel widened shoulder (Table 8.1)}.
    \]
    
    \[
    S_b = \text{Setback distance between the centre of the major road and the give way or stop line in the minor road.}
    \]
    
    **Source:** Department of Main Roads (2006)\(^{35}\).

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\(^{35}\) Department of Main Roads (2006) has been superseded and Figure 8.2 has not been carried forward into Queensland Department of Transport and Main Roads (2016).
### Table 8.1:  Minimum length of widened parallel shoulder

<table>
<thead>
<tr>
<th>Design speed of major road approach (km/h)</th>
<th>Minimum length of parallel widened shoulder P (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>50</td>
<td>0</td>
</tr>
<tr>
<td>60</td>
<td>5</td>
</tr>
<tr>
<td>70</td>
<td>10</td>
</tr>
<tr>
<td>80</td>
<td>15</td>
</tr>
<tr>
<td>90</td>
<td>20</td>
</tr>
<tr>
<td>100</td>
<td>25</td>
</tr>
<tr>
<td>110</td>
<td>35</td>
</tr>
<tr>
<td>120</td>
<td>45</td>
</tr>
</tbody>
</table>

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

*Source: Department of Main Roads (2006)*[^36]

#### 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.

[^36]: Department of Main Roads (2006) has been superseded and Table 8.1 has not been carried forward into Queensland Department of Transport and Main Roads (2016).
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 (Austroads 2017).
• The dimensions of the treatment are defined as follows. Values of D and T are provided in Table 8.2.

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

\[
WT = \text{Nominal width of the turn lane (m), including widening for curves based on the design turning vehicle = 3.0 m minimum.}
\]

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

\[
V = \text{Design speed of major road approach (km/h).}
\]

Source: Department of Main Roads (2006)\textsuperscript{37}.

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

<table>
<thead>
<tr>
<th>Design speed of major road approach (km/h)</th>
<th>Diverge/deceleration length (D) (m)\textsuperscript{1}</th>
<th>Taper length (T) (m)\textsuperscript{2}</th>
</tr>
</thead>
<tbody>
<tr>
<td>50</td>
<td>15</td>
<td>15</td>
</tr>
<tr>
<td>60</td>
<td>25</td>
<td>15</td>
</tr>
<tr>
<td>70</td>
<td>35</td>
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</tr>
<tr>
<td>80</td>
<td>45</td>
<td>20</td>
</tr>
<tr>
<td>90</td>
<td>55</td>
<td>25</td>
</tr>
<tr>
<td>100</td>
<td>70</td>
<td>30</td>
</tr>
<tr>
<td>110</td>
<td>85</td>
<td>30</td>
</tr>
<tr>
<td>120</td>
<td>100</td>
<td>35</td>
</tr>
</tbody>
</table>

\textsuperscript{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\textsuperscript{2} (Table 5.2). Adjust for grade using the ‘correction to grade’, (Table 5.3).

\textsuperscript{2} Based on a turn lane width of 3.0 m.

Source: Department of Main Roads (2006)\textsuperscript{38}.

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\textsuperscript{37} Department of Main Roads (2006) has been superseded and Figure 8.3 has not been carried forward into Queensland Department of Transport and Main Roads (2016).

\textsuperscript{38} Department of Main Roads (2006) has been superseded and Table 8.2 has not been carried forward into Queensland Department of Transport and Main Roads (2016).
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 (Austroads 2013a)).

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 (Austroads 2017).
- The dimensions of the treatment are defined thus:
  \[ W = \text{Nominal through lane width (m) (incl. widening for curves). For a new intersection on an existing road, the width is to be in accordance with the current link strategy.} \]
  \[ W_T = \text{Nominal width of turn lane (m) (incl. widening for curves based on the design turning vehicle) } = 3.0 \text{ m minimum.} \]
  \[ D = \text{Diverge/deceleration length including taper – Table 5.2. (Adjust for grade using the ‘correction to grade’ in Table 5.3).} \]
  \[ T = \text{Physical taper length (m) given by Equation 5 being: } T = \frac{0.33VW_T}{3.6} \]
  \[ V = \text{Design speed of major road approach (km/h).} \]


39 Department of Main Roads (2006) has been superseded and Figure 8.4 has not been carried forward into Queensland Department of Transport and Main Roads (2016).
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 (Austroads 2017). Figure 6.4 details minimum offsets to islands.
Desirable minimum area of rural islands ≥ 50 m².
The dimensions of the treatment are defined as:

\[ W = \text{Nominal through lane width (m) (including widening for curves). For a new intersection on an existing road, the width is to be in accordance with the current link strategy.} \]
\[ W_T = \text{Nominal width of turn lane (m), including widening for curves based on the design turning vehicle. Desirable minimum} = W, \text{absolute minimum} = 3.0 \text{ m}. \]
\[ B = \text{Total length of auxiliary lane including taper, diverge/deceleration and storage (m).} \]
\[ D = \text{Diverge/deceleration length including taper. Adjust for grade using the ‘correction to grade’ factor (Section 5).} \]
\[ T = \text{Physical taper length (m) and is given by Equation 5 being:} \quad T = \frac{0.33W_T}{3.6}. \]
\[ S = \text{Storage length (m) should be the greater of:} \]
\[ 1. \text{the length of one design turning vehicle or} \]
\[ 2. \text{(calculated car spaces −1) x 8 m (Guide to Traffic Management Part 3: Traffic Studies and Analysis (Austroads 2013e)), or use computer program e.g. aaSIDRA.} \]

Source: Department of Main Roads (2006)40.

40 Department of Main Roads (2006) has been superseded and Figure 8.5 has not been carried forward into Queensland Department of Transport and Main Roads (2016).
8.3 Urban Left-turn Treatments

8.3.1 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 8.6. 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 8.6: 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 dimensions of the treatment are defined as:

\[ W = \text{Nominal through lane width (m) (incl. widening for curves).} \]

\[ W_T = \text{Nominal width of turn lane (m) (incl. widening for curves based on the design turning vehicle) = 3.0 m minimum.} \]

\[ D = \text{Diverge/deceleration length including taper – Table 5.2. (adjust for grade by applying the 'correction to grade' factor in Table 5.3).} \]

\[ T = \text{Physical taper length (m) given by: } T = \frac{0.33VW_T}{3.6} \]

\[ V = \text{Design speed of major road approach (km/h).} \]

Source: Department of Main Roads (2006)\textsuperscript{41}.

\textsuperscript{41} Department of Main Roads (2006) has been superseded and Figure 8.6 has not been carried forward into Queensland Department of Transport and Main Roads (2016).
8.3.2 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 8.7 and Figure 8.8 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 8.7 and for a design prime mover and semi-trailer in Figure 8.8. 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 70º.

Because the road trains have to travel over the marked areas in Figure 8.7 and Figure 8.8 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 8.7 and Figure 8.8 are provided in Appendix C.
Figure 8.7: 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)42.

42 Department of Main Roads (2006) has been superseded and Figure 8.7 has not been carried forward into Queensland Department of Transport and Main Roads (2016).
Figure 8.8: 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 8.7 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)\(^{43}\).

\(^{43}\) Department of Main Roads (2006) has been superseded and Figure 8.8 has not been carried forward into Queensland Department of Transport and Main Roads (2016).
9. **Signalised Intersections**

9.1 **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.

9.2 **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. In constrained situations (e.g. some urban intersections) an EDD value for SSD and SISD may have to be considered (Appendix A).

For further information on signalised intersections, refer to *AGRD Part 4* (Austroads 2017) and *AGTM Part 6* (Austroads 2013a).
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**Australian and New Zealand Standards**


AS/NZS 3845.1-2015, *Road safety barrier systems and devices - road safety barrier systems*

**Australian Standards**

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 Guide to Road Design Part 2: Design Considerations (Austroads 2015b) 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.1.3 ‘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.1.5 ‘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 *AGRD Part 3* (Austroads 2016b).
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 approach sight distance.

Section 3.2.2 provides the formula for the calculation of safe intersection sight distance.

Section 5 of AGRD Part 3 (Austroads 2016b) 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

<table>
<thead>
<tr>
<th>Case type</th>
<th>Case code</th>
<th>Case description</th>
</tr>
</thead>
<tbody>
<tr>
<td>Base case (mandatory application)</td>
<td>Norm-day</td>
<td>Normal car driver travelling at the 85th percentile speed in daylight hours</td>
</tr>
<tr>
<td></td>
<td>Truck-day</td>
<td>Truck in daylight hours</td>
</tr>
<tr>
<td>Check case (ensure that adequate capability exists under these conditions, as relevant)</td>
<td>Norm-night</td>
<td>Normal car driver travelling at the 85th percentile speed on an unlit road at night</td>
</tr>
<tr>
<td></td>
<td>Truck-night</td>
<td>Truck travelling on an unlit roadway at night</td>
</tr>
<tr>
<td></td>
<td>Mean-day</td>
<td>Car driver travelling at the mean free speed in daylight hours (about 0.85 times the 85th percentile speed)</td>
</tr>
<tr>
<td></td>
<td>Mean-night</td>
<td>Car driver travelling at the mean free speed on an unlit roadway at night (about 0.85 times the 85th percentile speed)</td>
</tr>
<tr>
<td>Optional check case to confirm borderline cases</td>
<td>Skill-day</td>
<td>Skilled car driver travelling at the 85th percentile speed in daylight hours</td>
</tr>
<tr>
<td></td>
<td>Skill-night</td>
<td>Skilled car driver travelling at the 85th percentile speed on an unlit roadway at night</td>
</tr>
</tbody>
</table>

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 ($h_1 = 1.1 \text{ m for cars and 2.4 m for trucks}$) and object height ($h_2 = 0 \text{ m}$) as per Table A 4 of Appendix A of AGRD Part 3 (Austroads 2016b).
- Reaction times ($R_T = 1.5 \text{ sec / 2.0 sec / 2.5 sec}$) 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)

<table>
<thead>
<tr>
<th>Design speed (km/h)</th>
<th>Based on the norm-day base case(^{(1)})</th>
<th>Roads in predominantly dry areas with AADT&lt;4000 veh/d(^{(2)}) ((d = 0.61)^{(3)})</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>(h_1 = 1.1) (h_2 = 0)</td>
<td>(R_T = 1.5) sec (R_T = 2.0) sec (R_T = 2.5) sec</td>
</tr>
<tr>
<td>ASD (m)</td>
<td>(K)</td>
<td>ASD (m)</td>
</tr>
<tr>
<td>40</td>
<td>27</td>
<td>3.3</td>
</tr>
<tr>
<td>50</td>
<td>37</td>
<td>6.2</td>
</tr>
<tr>
<td>60</td>
<td>48</td>
<td>10.6</td>
</tr>
<tr>
<td>70</td>
<td>61</td>
<td>16.8</td>
</tr>
<tr>
<td>80</td>
<td>75</td>
<td>25.3</td>
</tr>
<tr>
<td>90</td>
<td>90</td>
<td>36.6</td>
</tr>
<tr>
<td>100</td>
<td>106</td>
<td>51.3</td>
</tr>
<tr>
<td>110</td>
<td>124</td>
<td>69.8</td>
</tr>
<tr>
<td>120</td>
<td>–</td>
<td>–</td>
</tr>
<tr>
<td>130</td>
<td>–</td>
<td>–</td>
</tr>
</tbody>
</table>

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)

<table>
<thead>
<tr>
<th>Design speed (km/h)</th>
<th>Based on the norm-day base case&lt;sup&gt;(1)&lt;/sup&gt;</th>
<th>Normal road conditions (&lt;i&gt;d&lt;/i&gt; = 0.46)&lt;sup&gt;(2)&lt;/sup&gt;</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>&lt;i&gt;R&lt;/i&gt; = 1.5 sec</td>
<td>&lt;i&gt;R&lt;/i&gt; = 2.0 sec</td>
</tr>
<tr>
<td></td>
<td>ASD (m)</td>
<td>&lt;i&gt;K&lt;/i&gt;</td>
</tr>
<tr>
<td>40</td>
<td>30</td>
<td>4.2</td>
</tr>
<tr>
<td>50</td>
<td>42</td>
<td>8.1</td>
</tr>
<tr>
<td>60</td>
<td>56</td>
<td>14.2</td>
</tr>
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<td>70</td>
<td>71</td>
<td>23.0</td>
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<td>88</td>
<td>35.3</td>
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<td>90</td>
<td>107</td>
<td>51.9</td>
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<td>127</td>
<td>73.6</td>
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<td>149</td>
<td>101</td>
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<tr>
<td>120</td>
<td>–</td>
<td>–</td>
</tr>
<tr>
<td>130</td>
<td>–</td>
<td>–</td>
</tr>
</tbody>
</table>

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 $d = 0.61$

<table>
<thead>
<tr>
<th>Design speed (km/h)</th>
<th>Correction (m)</th>
<th>Upgrade</th>
<th>Downgrade</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>2</td>
<td>4</td>
<td>6</td>
</tr>
<tr>
<td>40</td>
<td>0</td>
<td>−1</td>
<td>−1</td>
</tr>
<tr>
<td>50</td>
<td>−1</td>
<td>−1</td>
<td>−1</td>
</tr>
<tr>
<td>60</td>
<td>−1</td>
<td>−1</td>
<td>−2</td>
</tr>
<tr>
<td>70</td>
<td>−1</td>
<td>−2</td>
<td>−3</td>
</tr>
<tr>
<td>80</td>
<td>−1</td>
<td>−3</td>
<td>−4</td>
</tr>
<tr>
<td>90</td>
<td>−2</td>
<td>−3</td>
<td>−5</td>
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<td>−8</td>
</tr>
<tr>
<td>130</td>
<td>−3</td>
<td>−7</td>
<td>−10</td>
</tr>
</tbody>
</table>

Table A 5: Grade corrections to stopping sight distance for $d = 0.46$

<table>
<thead>
<tr>
<th>Design speed (km/h)</th>
<th>Correction (m)</th>
<th>Upgrade</th>
<th>Downgrade</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>2</td>
<td>4</td>
<td>6</td>
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<td>100</td>
<td>−4</td>
<td>−7</td>
<td>−10</td>
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<td>−8</td>
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<td>−14</td>
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<tr>
<td>130</td>
<td>−6</td>
<td>−12</td>
<td>−17</td>
</tr>
</tbody>
</table>
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)

<table>
<thead>
<tr>
<th>Design speed (km/h)</th>
<th>Based on the truck-day base case (^{(1)})</th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>(h_1 = 2.4) (h_2 = 0)</td>
<td></td>
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<tr>
<td></td>
<td>(d = 0.29)</td>
<td></td>
<td></td>
<td></td>
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<tr>
<td></td>
<td>(R_T = 1.5) sec</td>
<td>(R_T = 2.0) sec</td>
<td>(R_T = 2.5) sec</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>ASD (m)</td>
<td>(K)</td>
<td>ASD (m)</td>
<td>(K)</td>
<td>ASD (m)</td>
</tr>
<tr>
<td>40</td>
<td>38</td>
<td>3.1</td>
<td>44</td>
<td>4.0</td>
<td>–</td>
</tr>
<tr>
<td>50</td>
<td>55</td>
<td>6.3</td>
<td>62</td>
<td>7.9</td>
<td>–</td>
</tr>
<tr>
<td>60</td>
<td>74</td>
<td>11.4</td>
<td>82</td>
<td>14.1</td>
<td>–</td>
</tr>
<tr>
<td>70</td>
<td>96</td>
<td>19.1</td>
<td>105</td>
<td>23.1</td>
<td>–</td>
</tr>
<tr>
<td>80</td>
<td>120</td>
<td>30.1</td>
<td>131</td>
<td>35.9</td>
<td>–</td>
</tr>
<tr>
<td>90</td>
<td>147</td>
<td>45.3</td>
<td>160</td>
<td>53.3</td>
<td>172</td>
</tr>
<tr>
<td>100</td>
<td>177</td>
<td>65.6</td>
<td>191</td>
<td>76.3</td>
<td>205</td>
</tr>
<tr>
<td>110</td>
<td>210</td>
<td>92.0</td>
<td>225</td>
<td>106</td>
<td>241</td>
</tr>
</tbody>
</table>

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 \(d = 0.29\)

<table>
<thead>
<tr>
<th>Design speed (km/h)</th>
<th>Correction (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Upgrade</td>
</tr>
<tr>
<td></td>
<td>2</td>
</tr>
<tr>
<td>40</td>
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<td>–3</td>
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<td>70</td>
<td>–4</td>
</tr>
<tr>
<td>80</td>
<td>–6</td>
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<td>–7</td>
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<tr>
<td>100</td>
<td>–9</td>
</tr>
<tr>
<td>110</td>
<td>–11</td>
</tr>
</tbody>
</table>
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

<table>
<thead>
<tr>
<th>Observation time OT (sec)</th>
<th>Typical use</th>
</tr>
</thead>
</table>
| 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 (Austroads 2016b) – refer to Note 3 for the check cases
  - \( h_1 = 1.1 \text{ m} \) and \( h_2 = 1.25 \text{ m} \) for norm-day and mean-day
  - \( h_1 = 2.4 \text{ m} \) and \( h_2 = 1.25 \text{ m} \) for truck-day
  - \( h_1 = 0.65 \text{ m} \) and \( h_2 = 1.25 \text{ m} \) for norm-night and mean-night
  - \( h_1 = 2.4 \text{ m} \) and \( h_2 = 0.8 \text{ m} \) for truck-night (minimum acceptable).
- Reaction times as per Table A 5 of Appendix A of AGRD Part 3 (Austroads 2016b) – refer to Note 5 for the check cases
  - \( R_T = 1.5 \text{ sec}/2.0 \text{ sec}/2.5 \text{ sec} \) for norm-day, truck-day, norm-night and truck-night
  - \( R_T = 2.0 \text{ sec}/2.5 \text{ sec} \) 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 seconds

<table>
<thead>
<tr>
<th>Design speed (km/h)</th>
<th>Based on norm-day safe intersection sight distance(1)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$h_1 = 1.1$ $h_2 = 1.25$ $d = 0.46$ $O_T = 1.5$ sec</td>
</tr>
<tr>
<td></td>
<td>$R_T = 1.5$ sec</td>
</tr>
<tr>
<td></td>
<td>SISD (m) $K$</td>
</tr>
<tr>
<td>40</td>
<td>47  2.4</td>
</tr>
<tr>
<td>50</td>
<td>63  4.2</td>
</tr>
<tr>
<td>60</td>
<td>81  7.0</td>
</tr>
<tr>
<td>70</td>
<td>100 10.7</td>
</tr>
<tr>
<td>80</td>
<td>121 15.7</td>
</tr>
<tr>
<td>90</td>
<td>144 22.2</td>
</tr>
<tr>
<td>100</td>
<td>169 30.4</td>
</tr>
<tr>
<td>110</td>
<td>195 40.6</td>
</tr>
<tr>
<td>120</td>
<td>–  –</td>
</tr>
<tr>
<td>130</td>
<td>–  –</td>
</tr>
</tbody>
</table>

**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.1.5, 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 (Austroads 2016b).

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 seconds

<table>
<thead>
<tr>
<th>Design speed (km/h)</th>
<th>Based on norm-day safe intersection sight distance$^{(1)}$</th>
<th>( h_1 = 1.1 )</th>
<th>( h_2 = 1.25 )</th>
<th>( d = 0.46 )</th>
<th>( \text{Norm-day} )</th>
<th>( OT = 2.0 )</th>
<th>( \text{Norm-night} )</th>
<th>( OT = 1.1 )</th>
<th>( \text{Mean-day} )</th>
<th>( OT = 2.8 )</th>
<th>( \text{Mean-night} )</th>
<th>( OT = 1.8 )</th>
</tr>
</thead>
<tbody>
<tr>
<td>( R_T = 1.5 ) sec</td>
<td>( R_T = 2.0 ) sec</td>
<td>( R_T = 2.5 ) sec</td>
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<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>SISD (m)</td>
<td>( K )</td>
<td>SISD (m)</td>
<td>( K )</td>
<td>SISD (m)</td>
<td>( K )</td>
<td>SISD (m)</td>
<td>( K )</td>
<td>SISD (m)</td>
<td>( K )</td>
<td>SISD (m)</td>
<td>( K )</td>
<td>SISD (m)</td>
</tr>
<tr>
<td>40</td>
<td>53</td>
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<td>58</td>
<td>3.6</td>
<td>–</td>
<td>–</td>
<td></td>
<td></td>
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<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>50</td>
<td>70</td>
<td>5.2</td>
<td>77</td>
<td>6.3</td>
<td>–</td>
<td>–</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>60</td>
<td>89</td>
<td>8.5</td>
<td>97</td>
<td>10.1</td>
<td>–</td>
<td>–</td>
<td></td>
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<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>70</td>
<td>110</td>
<td>12.9</td>
<td>120</td>
<td>15.3</td>
<td>–</td>
<td>–</td>
<td></td>
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<td></td>
<td></td>
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<td></td>
</tr>
<tr>
<td>80</td>
<td>133</td>
<td>18.7</td>
<td>144</td>
<td>22.0</td>
<td>–</td>
<td>–</td>
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<td>47.2</td>
<td>226</td>
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</tr>
<tr>
<td>120</td>
<td>–</td>
<td>–</td>
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<td>130</td>
<td>–</td>
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<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

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, \( OT = 1.1 \) sec)
- Mean-day: Yes (\( d = 0.41, h_1 = 1.1 \) m, \( h_2 = 1.25 \) m, \( OT = 2.8 \) sec)
- Mean-night$^{(4)}$: Yes (\( d = 0.41, h_1 = 0.65 \) m, \( h_2 = 1.25 \) m, \( OT = 1.8 \) 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 seconds

| Design speed (km/h) | Based on norm-day safe intersection sight distance(1) |  |  |  |
|:-------------------|:---------------------------------|:|:|:|
|                   | $h_1 = 1.1$ $h_2 = 1.25$ $d = 0.46$ | $R_T = 1.5$ sec | $R_T = 2.0$ sec | $R_T = 2.5$ sec |
|                   | $OT = 2.5$ 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, $OT = 1.5$ sec). |
| Mean-day | Yes ($d = 0.41$, $h_1 = 1.1$ m, $h_2 = 1.25$ m, $OT = 2.9$ sec). |
| Mean-night(4) | Yes ($d = 0.41$, $h_1 = 0.65$ m, $h_2 = 1.25$ m, $OT = 2.3$ 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 seconds

<table>
<thead>
<tr>
<th>Design Speed (km/h)</th>
<th>Based on truck-day Safe Intersection Sight Distance&lt;sup&gt;(1)&lt;/sup&gt;</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$h_1 = 2.4$ $h_2 = 1.25$ $d = 0.29$&lt;sup&gt;(2)&lt;/sup&gt; $O_T = 1.5$ sec</td>
</tr>
<tr>
<td></td>
<td>$R_T = 1.5$ sec</td>
</tr>
<tr>
<td></td>
<td>K</td>
</tr>
<tr>
<td>SISD (m)</td>
<td>SISD (m)</td>
</tr>
<tr>
<td>40</td>
<td>55</td>
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<tr>
<td>50</td>
<td>76</td>
</tr>
<tr>
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<td>99</td>
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<tr>
<td>70</td>
<td>125</td>
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<td>80</td>
<td>154</td>
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<tr>
<td>100</td>
<td>219</td>
</tr>
<tr>
<td>110</td>
<td>256</td>
</tr>
</tbody>
</table>

Do all of the crest curve sizes listed provide acceptable truck-night check case capability?<sup>(3)(4)</sup> 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.1.5, 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 seconds

<table>
<thead>
<tr>
<th>Design speed (km/h)</th>
<th>Based on truck-day safe intersection sight distance(^{(1)}) (h_1 = 2.4 \ h_2 = 1.25 \ d = 0.29^{(2)} \ OT = 2.0) sec</th>
<th>(R_T = 1.5) sec</th>
<th>(R_T = 2.0) sec</th>
<th>(R_T = 2.5) sec</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>SISD (m)</td>
<td>K</td>
<td>SISD (m)</td>
<td>K</td>
</tr>
<tr>
<td>40</td>
<td>61</td>
<td>2.6</td>
<td>66</td>
<td>3.1</td>
</tr>
<tr>
<td>50</td>
<td>83</td>
<td>4.8</td>
<td>89</td>
<td>5.6</td>
</tr>
<tr>
<td>60</td>
<td>107</td>
<td>8.1</td>
<td>116</td>
<td>9.4</td>
</tr>
<tr>
<td>70</td>
<td>135</td>
<td>12.7</td>
<td>144</td>
<td>14.6</td>
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<tr>
<td>80</td>
<td>165</td>
<td>19.1</td>
<td>176</td>
<td>21.7</td>
</tr>
<tr>
<td>90</td>
<td>197</td>
<td>27.4</td>
<td>210</td>
<td>31.0</td>
</tr>
<tr>
<td>100</td>
<td>233</td>
<td>38.1</td>
<td>247</td>
<td>42.8</td>
</tr>
<tr>
<td>110</td>
<td>271</td>
<td>51.7</td>
<td>286</td>
<td>57.7</td>
</tr>
</tbody>
</table>

Do all of the crest curve sizes listed provide acceptable truck-night check case capability\(^{(3)(4)}\) Yes \((d = 0.29, \ h_1 = 2.4 \ m, \ h_2 = 0.8 \ m, \ OT = 1.1\) 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 seconds

<table>
<thead>
<tr>
<th>Design speed (km/h)</th>
<th>Based on truck-day safe intersection sight distance(^{(1)}) (h_1 = 2.4 \ h_2 = 1.25 \ d = 0.29^{(2)} \ OT = 2.5) sec</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>(R_T = 1.5) sec</td>
</tr>
<tr>
<td></td>
<td>SISD (m)</td>
</tr>
<tr>
<td>40</td>
<td>66</td>
</tr>
<tr>
<td>50</td>
<td>89</td>
</tr>
<tr>
<td>60</td>
<td>116</td>
</tr>
<tr>
<td>70</td>
<td>144</td>
</tr>
<tr>
<td>80</td>
<td>176</td>
</tr>
<tr>
<td>90</td>
<td>210</td>
</tr>
<tr>
<td>100</td>
<td>247</td>
</tr>
<tr>
<td>110</td>
<td>286</td>
</tr>
</tbody>
</table>

Do all of the crest curve sizes listed provide acceptable truck-night check case capability\(^{(3)(4)}\) Yes \((d = 0.29, \ h_1 = 2.4 \ m, \ h_2 = 0.8 \ m, \ OT = 1.6\) 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 less 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.1.1 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 (Austroads 2016b).

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 turn-slot 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.

\[
W = \text{Nominal through lane width (m), including widening for curves.}
\]

\[
W_{tr} = \text{Nominal width of turn lane (m), including widening for curves based on the design turning vehicle = 2.8 m minimum.}
\]

\[
E = \text{Distance from start of taper to 2.0 m width (m) = } 2 \left( \frac{A}{W_{tr}} \right)
\]

\[
S = \text{Storage length (m) is the greater of:}
\]

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 2013e)), or use computer program e.g. aaSIDRA.

\[
T = \text{Physical taper length given by: } T = \frac{0.2W_{tr}}{3.6}
\]

\[
V = \text{Design speed of major road approach (km/h).}
\]

\[
X = \text{Distance based on design vehicle turning path, refer to Design Vehicles and Turning Path Templates (Austroads 2013f).}
\]

Source: Department of Main Roads (2006)\(^\text{44}\).

\(^{44}\) Department of Main Roads (2006) has been superseded and Figure A 3 has not been carried forward into Queensland Department of Transport and Main Roads (2016).
Table A 15: Dimensions relating to Figure A 3

<table>
<thead>
<tr>
<th>Design speed of major road approach (km/h)</th>
<th>Minimum lateral movement length $A$ (m)$^{(1)}$</th>
<th>Desirable radius $R$ (m)</th>
<th>Taper length, $T^{(2)}$ (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>60</td>
<td>40$^{(3)}$</td>
<td>175</td>
<td>10</td>
</tr>
<tr>
<td>70</td>
<td>50$^{(3)}$</td>
<td>240</td>
<td>15</td>
</tr>
<tr>
<td>80</td>
<td>55$^{(3)}$</td>
<td>280</td>
<td>15</td>
</tr>
<tr>
<td>90</td>
<td>60</td>
<td>350</td>
<td>15</td>
</tr>
<tr>
<td>100</td>
<td>70</td>
<td>425</td>
<td>20</td>
</tr>
<tr>
<td>110</td>
<td>75</td>
<td>500</td>
<td>20</td>
</tr>
<tr>
<td>120</td>
<td>80</td>
<td>600</td>
<td>20</td>
</tr>
</tbody>
</table>

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)$^{45}$

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.

45 Department of Main Roads (2006) has been superseded and Table A 15 has not been carried forward into Queensland Department of Transport and Main Roads (2016).
Figure A 4: Minimum extended design domain channelised right-turn treatment for roadways with medians

Notes:
- Variables are defined thus:
  \[ W = \text{Nominal through lane width (m), including widening for curves.} \]
  \[ W_T = \text{Nominal width of turn lane (m), including widening for curves based on the design turning vehicle \geq 2.8 \text{ m minimum.}} \]
  \[ B = \text{Total length of auxiliary lane (m), including taper, diverge/acceleration and storage.} \]
  \[ S = \text{Storage length (m), greater of:} \]
  1. the length of one design turning vehicle or
  2. \((\text{calculated car spaces} - 1) \times 8 \text{ m (refer to Austroads Guide to Traffic Management Part 3: Traffic Studies and Analysis (Austroads 2013e)), or use computer program e.g. aaSIDRA.}\)
  \[ T = \text{Physical taper length} = \frac{0.2W_T}{3.6} \]
  \[ V = \text{Design speed of major road approach (km/h).} \]
  \[ X = \text{Distance based on design vehicle turning path, refer to Austroads (2013f).} \]
- 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)\textsuperscript{46}.

Table A 16: Dimensions relating to Figure A 4

<table>
<thead>
<tr>
<th>Design speed of major road approach (km/h)</th>
<th>Minimum diverge/deceleration length ( D^{(1)} ) (m)</th>
<th>Taper length ( T^{(2)} ) (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>50</td>
<td>15</td>
<td>10</td>
</tr>
<tr>
<td>60</td>
<td>20</td>
<td>10</td>
</tr>
<tr>
<td>70</td>
<td>25</td>
<td>15</td>
</tr>
<tr>
<td>80</td>
<td>35</td>
<td>15</td>
</tr>
<tr>
<td>90</td>
<td>45</td>
<td>15</td>
</tr>
<tr>
<td>100</td>
<td>55</td>
<td>20</td>
</tr>
<tr>
<td>110</td>
<td>65</td>
<td>20</td>
</tr>
<tr>
<td>120</td>
<td>80</td>
<td>20</td>
</tr>
</tbody>
</table>

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)\textsuperscript{47}.

\textsuperscript{46} Department of Main Roads (2006) has been superseded and Figure A 4 has not been carried forward into Queensland Department of Transport and Main Roads (2016).

\textsuperscript{47} Department of Main Roads (2006) has been superseded and Table A 16 has not been carried forward into Queensland Department of Transport and Main Roads (2016).
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:
  \[ W = \text{Nominal through lane width (m), including widening for curves.} \]
  \[ W_T = \text{Nominal width of turn lane (m), including widening for curves based on the design turning vehicle = 2.8 m minimum.} \]
  \[ T = \text{Physical taper length (m) =} \frac{0.2VW_T}{3.6} \]
  \[ V = \text{Design speed of major road approach (km/h).} \]
- 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 2017).

Source: Department of Main Roads (2006)\(^{48}\).

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\(^{48}\) Department of Main Roads (2006) has been superseded and Figure A 5 has not been carried forward into Queensland Department of Transport and Main Roads (2016).
Table A 17: Dimensions relating to Figure A 5

<table>
<thead>
<tr>
<th>Design speed of major road approach (km/h)</th>
<th>Minimum diverge/deceleration length $D^{(1)}$ (m)</th>
<th>Taper length $T^{(2)}$ (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>50</td>
<td>15</td>
<td>10</td>
</tr>
<tr>
<td>60</td>
<td>20</td>
<td>10</td>
</tr>
<tr>
<td>70</td>
<td>25</td>
<td>15</td>
</tr>
<tr>
<td>80</td>
<td>35</td>
<td>15</td>
</tr>
<tr>
<td>90</td>
<td>45</td>
<td>15</td>
</tr>
<tr>
<td>100</td>
<td>55</td>
<td>20</td>
</tr>
<tr>
<td>110</td>
<td>65</td>
<td>20</td>
</tr>
<tr>
<td>120</td>
<td>80</td>
<td>20</td>
</tr>
</tbody>
</table>

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/s².

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)\(^{49}\).

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 6 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 6: Use of compound curves to avoid expensive relocation of an obstruction

Source: Department of Main Roads (2006)\(^{50}\).

\(49\) Department of Main Roads (2006) has been superseded and Table A 17 has not been carried forward into Queensland Department of Transport and Main Roads (2016).

\(50\) Department of Main Roads (2006) has been superseded and Figure A 6 has not been carried forward into Queensland Department of Transport and Main Roads (2016).
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 7 for a CHR(S) turn treatment) and Figure A 8 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 7 and Figure A 8 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 7 and Figure A 8 are the same as those that relate to the T-intersections shown in Figures A 29 and A 30 in AGRD Part 4 (Austroads 2017) respectively.

Figure A 7: Retrofitting CHR(S) treatments to a rural four-way intersection
Guide to Road Design Part 4A: Unsignalised and Signalised Intersections

Notes: Refer to Figure A 30 in AGRD Part 4 (Austroads 2017) 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.
- **WT** = 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 2013e), 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 (Austroads 2013f).

Source: Department of Main Roads (2006)\(^5^1\).

\(^5^1\) Department of Main Roads (2006) has been superseded and Figure A 7 has not been carried forward into Queensland Department of Transport and Main Roads (2016).
Figure A 8: Retrofitting CHR treatments to a rural four-way intersection

Note: Refer to Figure A 30 in AGRD Part 4 (Austroads 2017) for the dimensions labelled in these diagrams.

- Ø – Double barrier line is not to be used this side of a linemarked island.
- Islands may be linemarked or raised. If linemarked, 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:
  
  \[
  \begin{align*}
  W &= \text{Nominal through lane width (m) (including widening for curves). For a new intersection on an existing road, the width is to be in accordance with the current link strategy.} \\
  W_T &= \text{Nominal width of turn lane (m), including widening for curves based on the design turning vehicle. Desirable minimum} = W, \text{absolute minimum} = 3.0 \text{ m.} \\
  B &= \text{Total length of auxiliary lane including taper, diverge/deceleration and storage (m).}
  \end{align*}
  \]
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 2013e), 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 (Austroads 2013f).

Source: Department of Main Roads (2006)52.

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

<table>
<thead>
<tr>
<th>Median function</th>
<th>Absolute minimum width (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Separate traffic flows and a safety barrier¹</td>
<td>1.9</td>
</tr>
<tr>
<td>Shelter a small sign</td>
<td>1.0</td>
</tr>
<tr>
<td>Shelter signal pedestals or lighting poles</td>
<td>1.4</td>
</tr>
<tr>
<td>Shelter pedestrians and traffic signals</td>
<td>2.0</td>
</tr>
<tr>
<td>Shelter turning vehicles and traffic signals</td>
<td>5.0</td>
</tr>
<tr>
<td>Shelter crossing vehicles</td>
<td>6.0</td>
</tr>
</tbody>
</table>

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. \( \leq 10 \text{ km/h} \)), 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.

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52 Department of Main Roads (2006) has been superseded and Figure A 8 has not been carried forward into Queensland Department of Transport and Main Roads (2016).
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 AGRD Part 3 (Austroads 2016b).

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 2016b).

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)\textsuperscript{53}.

\textsuperscript{53} VicRoads (1994) has been superseded and Figure B 1 and Figure B 2 have not been carried forward into VicRoads (2011).
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.

<table>
<thead>
<tr>
<th>Loading</th>
<th>Rollover Threshold</th>
<th>Centre of gravity height</th>
</tr>
</thead>
<tbody>
<tr>
<td>Homogeneous load</td>
<td>0.24 g</td>
<td>2.67 m</td>
</tr>
<tr>
<td>30% Load</td>
<td>0.28 g</td>
<td>2.41 m</td>
</tr>
<tr>
<td>70% Load</td>
<td>0.34 g</td>
<td>2.12 m</td>
</tr>
<tr>
<td>Full gross</td>
<td>0.32 g</td>
<td>2.25 m</td>
</tr>
<tr>
<td></td>
<td>0.26 g</td>
<td>2.54 m</td>
</tr>
</tbody>
</table>

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:

\[ E + F = \frac{v^2}{gR} \]

\[ E + F = \frac{a}{g} \]

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 2013f). 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

<table>
<thead>
<tr>
<th>Radius (m)</th>
<th>Critical speeds for high trucks within intersections (km/h)</th>
<th>Positive superelevation (m/m)</th>
<th>0</th>
<th>Negative superelevation (i.e. adverse)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
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<td></td>
</tr>
<tr>
<td>0.07</td>
<td></td>
<td></td>
<td>-0.01</td>
<td>-0.02</td>
</tr>
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<td>11</td>
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<td>Radius (m)</td>
<td>Critical speeds for high trucks within intersections (km/h)</td>
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<td>Positive superelevation (m/m)</td>
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</tbody>
</table>

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.


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54 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 8.7 and Figure 8.8 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)\textsuperscript{55}.

\textsuperscript{55}  Department of Main Roads (2006) has been superseded and Figure C 1 has not been carried forward into Queensland Department of Transport and Main Roads (2016).
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)\textsuperscript{56}.

\textsuperscript{56} Department of Main Roads (2006) has been superseded and Figure C 2 has not been carried forward into Queensland Department of Transport and Main Roads (2016).
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 (2016).
Figure C 4: Example of corner treatment on heavy combination vehicle route median allows carriageway widening

KERBS SHALL BE 0.5 m (Min) OUTSIDE SWEPT PATH OF DESIGN VEHICLE

FLUSH APRONS WITH:
- COLOUR - AS2700 R53 RED GUM, R54 RASPBERRY, R62 VENETIAN RED, R63 RED OXIDE, OR SIMILAR APPROVED.
- SKID RESISTANT TO MATCH ROAD SURFACE.

EDGE LINES (LONG LIFE MATERIAL)

KERBS SHALL BE 0.5 m (Min) OUTSIDE SWEPT PATH OF DESIGN VEHICLE

UN SIGNALISED LEFT TURN TREATMENT
(WHERE WIDTH OF MEDIAN ALLOWS CARRIAGEWAY WIDENING)

Source: Adapted from Main Roads Western Australia (2016).
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


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

<table>
<thead>
<tr>
<th>Travel speed (km/h)</th>
<th>Acceleration rate (m/s)</th>
<th>Acceleration rate (km/h/s)</th>
<th>Acceleration rate (m/s²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>40</td>
<td>11.11</td>
<td>4.7</td>
<td>1.3</td>
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<tr>
<td>50</td>
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<td>3.6</td>
<td>1.0</td>
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<td>70</td>
<td>19.44</td>
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<tr>
<td>110</td>
<td>30.55</td>
<td>1.8</td>
<td>0.5</td>
</tr>
</tbody>
</table>

**Commentary 3**

The merge length for an acceleration lane or an auxiliary through lane can be calculated from Equation C1:

\[ T_m = \frac{VY}{3.6S} \]

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

**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 (Department of 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.

**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**
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.