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6.1 INTRODUCTION

The design of an at-grade intersection requires understanding of the principles of both traffic and highway engineering. The operation of an intersection is influenced by its capacity, queue lengths and delays, accident potential, vehicle operating characteristics and traffic control. The physical layout of an intersection is defined by its horizontal and vertical alignment, roadway cross-sections, surface texture and drainage.

The successful integration of all these factors is required for good design, which must overcome

![Image of different intersection types]

Figure 6.1: Typical intersection types
the potential safety and operation conflicts that are inherent when traffic streams interact at intersections.

Although some guidance on capacity and traffic control is offered, the focus of this chapter is on application of the geometric principles that govern the physical layout and location of an intersection.

### 6.2 DESIGN PRINCIPLES

#### 6.2.1 General

The unique characteristic of intersections is that vehicles, pedestrians and bicycles travelling in many directions, must share a common area, often at the same time. The mitigation of the resulting conflicts is a major objective of intersection design. This conflict resolution is, in turn, influenced by construction and maintenance costs, environmental factors and the ease of implementation.

#### 6.2.2 Elements affecting design

**Human factors**

The driver's response, discussed in sub-section 3.2.1, is a major factor in intersection design. The recommended perception and reaction time of 2.5 seconds is normally used as an input to models determining intersection sight distance. However, because of heightened awareness at controlled intersections, circumstances may indicate a lower acceptable value of 2.0 seconds at busy urban intersections.

The concept of driver expectancy is crucial in the evaluation of drivers' response and tasks within intersections.

**Vehicle Characteristics**

The size and manoeuvrability of vehicles is a governing factor in intersection design, particu-

<table>
<thead>
<tr>
<th>Human factor</th>
<th>Design values</th>
<th>Design elements affected</th>
</tr>
</thead>
<tbody>
<tr>
<td>Perception/reaction time</td>
<td>2.0 – 4.0 seconds</td>
<td>Intersection sight distance</td>
</tr>
<tr>
<td>Gap acceptance</td>
<td>5.5 - 7.5 seconds</td>
<td>Intersection sight distance</td>
</tr>
<tr>
<td>Driver height of eye</td>
<td>1.05 metres</td>
<td>Sight distance</td>
</tr>
<tr>
<td>Pedestrian walking speeds</td>
<td>1-1.5 metres/second</td>
<td>Pedestrian facilities</td>
</tr>
</tbody>
</table>

Various vehicle characteristics and their influence on the design of channelised intersections are described in Table 6.2.

**Environmental Influences**

The type of highway and area, surrounding land...
use and the prevailing climate all have an influence on the type of design selected. Flexibility of approach is essential and the concepts of context-sensitive design as outlined in Chapter 2 should be applied.

Functional classification is a key to applying the appropriate design standards. Primary arterials carry high traffic volumes, operate at high speeds and are often used by drivers unfamiliar with them. Large trucks and buses are common and there is a driver expectancy for route continuity and a high level of service. Intersection design should reflect and make provision for the operating characteristics of drivers and their expectations in the various classes of roads. Channelisation should accommodate the expected vehicles in a simple and direct manner. Decision sight distance is an important element and traffic control devices and pavement markings should be placed with care.

The type of area and adjacent land use governs the selection of an appropriate intersection. In urban areas, pedestrian flows, on-street parking and bus and taxi activity are commonplace. In residential areas, bicycles and school crossings need to be considered. They are usually absent in rural areas, where utility and delivery vehicles are more common.

Local climate can influence design decisions. Where the presence of mist is a frequent occurrence, sight distance would be reduced. Heavy rainfalls can obscure signs and road markings and reduce pavement friction.

### 6.2.3 Traffic manoeuvres and conflicts

Typical manoeuvres that result in vehicle conflict at intersections are:
- Crossing;
- Merging;

| Table 6.2: Vehicle characteristics applicable to design of channelised intersections |
|---------------------------------|---------------------------------|
| **Vehicle Characteristics**     | **Intersection Design Elements Affected** |
| **Physical Characteristics:**   |                                 |
| Length                          | Lengths of storage lanes       |
| Width                           | Widths of lanes                |
| Height                          | Widths of turning roadways     |
| Wheelbase                       | Placement of overhead signals and signs |
| **Operational characteristics:**|                                 |
| Acceleration capability         | Acceleration tapers and lane lengths |
| Deceleration and braking capability | Lengths of deceleration lanes and tapers |
|                                 | Stopping sight distance        |
Diverging and merging may be either to the left or right, mutual or multiple. Crossings may be direct, if the angle of skew is between 75° and 120°, or oblique if the angle is in the range of 60° to 75°. Oblique skews should be avoided if at all possible. If the angle of skew is less than 60°, the possibility of replacing the skew by a staggered intersection should be considered. Angles of skew greater than 120° should be replaced by relocation of the intersection to an angle of skew closer to 90°.

Weaving is a combination of merging and diverging traffic moving in the same direction. It may be simple or complex.

The conflict at intersections created by the various manoeuvres leads to a unique set of operational characteristics. Understanding these is central to intersection designs and the most important characteristics are safety and capacity.

### Table 6.3: Features contributing to accidents at intersections and remedial measures

<table>
<thead>
<tr>
<th>Geometric features or conditions contributing to adverse accident experience at intersections</th>
<th>Traffic engineering actions that reduce accident experience or severity</th>
</tr>
</thead>
<tbody>
<tr>
<td>Poor approach sight distance</td>
<td>Addition/installation of exclusive turn-lanes</td>
</tr>
<tr>
<td>Poor corner sight distance</td>
<td>Upgrading of traffic control scheme</td>
</tr>
<tr>
<td>Steep grades at intersections</td>
<td>Improvement of sight distance</td>
</tr>
<tr>
<td>Inappropriate traffic control</td>
<td>Installation of lighting</td>
</tr>
<tr>
<td>Multiple approaches</td>
<td>Removal of fixed objects</td>
</tr>
<tr>
<td>Presence of curves within intersection</td>
<td>Increasing of corner radii</td>
</tr>
<tr>
<td>Number of adjacent driveways or access points</td>
<td>Application of channelisation</td>
</tr>
<tr>
<td>Inappropriate curb radii</td>
<td>Improve drainage paths and roughen surface</td>
</tr>
<tr>
<td>Narrow lanes</td>
<td></td>
</tr>
<tr>
<td>Poor drainage and skid resistance</td>
<td></td>
</tr>
</tbody>
</table>

6.2.4 Capacity

To operate successfully, an intersection must be able to handle peak traffic demands. The analysis of capacity is based on the operational characteristics of conflicting vehicles separated by the time constraints imposed by traffic control devices. The measuring and forecasting of traf-
fic flows and capacity analysis is a specialised subject and designers should refer to the manuals and references commonly used. The following is a brief summary of capacity as it relates to design.

**Signalised intersections**

The idealised flow rate through an intersection is known as the saturation flow rate per hour of green time. Initial driver reaction, vehicle acceleration and the behaviour of following vehicles all affect this flow rate. The capacity of an approach or leg of an intersection is proportional to the green time for that approach within the signal cycle in accordance with

\[
C = s \times \frac{g}{c}
\]

where
- \( C \) = capacity (veh/h)
- \( s \) = saturated flow rate (veh/h)
- \( g/c \) = the ratio of green time to signal cycle time.

The important factors affecting saturation flow are:
- Number of lanes;
- Widths of lanes;
- Proportion of heavy vehicles;
- Gradients in excess of 3%;
- On-street parking;
- Pedestrian activity; and
- Type and phasing of signals.

The critical factors are the total number of lanes and the need for exclusive turning lanes at each approach.

**Unsignalised intersections**

The capacity of the major road at Stop- and Yield-controlled intersections is not affected by the presence of the intersection. The capacity of the minor road is dependent on the distribution of gaps in the major road traffic and the gap acceptance of the minor road traffic. Gap acceptance is dependent on the reaction/response time, vehicle acceleration and vehicle length. It is not a function of approach speed on the major road. Gap acceptance times used in determining capacity are usually somewhat shorter than those used to compute intersection sight distance as described in Section 6.4.

Factors affecting capacity include:
- Operational speed of the major road;
- Intersection sight distance;
- Radii of turning roadways;
- Intersection layout and number of lanes;
- Type of area; and
- Proportion of heavy vehicles.

The critical factors are intersection sight distance and the number and arrangement of traffic lanes.

**6.2.5 Intersection types and selection**

There are four basic types or classes of intersection: three-legged T and Y intersections; four legged intersections with a defined crossing path; multi-legged intersections; and roundabouts, with the last-mentioned encompassing all three of the previous types. These are shown schematically in Figure 6.1. The design-
er’s selection of the basic intersections type is normally predicated on the design context, as intersections can vary greatly in scope, shape, degree of channelisation and traffic control measures.

Important factors to be considered in the selection of an intersection type are:
- Cost of construction;
- Type of area;
- Land use and land availability;
- Functional classes of the intersecting roads;
- Approach speeds;
- Proportion of traffic on each approach; and
- Volumes to be accommodated.

Careful consideration of these factors, together with the warrants for selecting appropriate traffic control devices, will lead to an appropriate choice or to a limited number of alternatives from which to make the final selection. The critical factors are cost and capacity.

A worldwide review of intersection design practice reported that, "typically the cheapest intersection type providing the required level of service is chosen". This cost is usually the sum of the design, construction and right-of-way costs. This view is consistent with South African experience, where the cost to the road authority is often the governing factor in the choice of intersection type.

At intersections carrying light crossing and turning volumes the capacity figures for uninterrupted flow generally apply. Table 6.4 below is a guide to the maximum traffic volumes that these intersections can accommodate.

When volumes exceed the above, the capacity of the intersection should be analysed in detail.

As safety at intersections is of key importance, the following summary of the interrelationship between intersections and accidents can act to guide the selection of an intersection type and layout:
- The U.S. National Safety Council estimated that 56 per cent of all urban accidents and 32 per cent of all rural accidents occur at intersections.
- The number of accidents is proportional to the volume and distribution of traffic on the major and minor roads.
- Roundabouts have considerable safety advantages over other types of at grade intersections.
- Poor sight distance leads to significantly higher injury and total accident rates. However, on roundabout approaches, accidents may actually increase with increasing sight distance.
- Medians should be as wide as practical at rural, unsignalised intersections but not wider than necessary at signalised intersections.
- Channelisation is usually beneficial but

<table>
<thead>
<tr>
<th>Table 6.4: Typical maximum traffic volumes for priority intersections</th>
</tr>
</thead>
<tbody>
<tr>
<td>Road Type</td>
</tr>
<tr>
<td>Major road</td>
</tr>
<tr>
<td>Minor road</td>
</tr>
</tbody>
</table>

Chapter 6: Intersection Design
kerbed islands in the major road may be hazardous in rural areas.

- The hazard of an intersection increases as the approach speed increases.

### 6.2.6 Location of intersections

Given the fact that intersections are the most dangerous part of any road network, it follows that their location deserves serious attention by the designer. It is necessary to minimise both the likelihood of crashes occurring and the consequences of the crashes that do occur. There are thus various restraints on the location of intersections that should be considered.

The need for drivers to discern and perform the manoeuvres necessary to pass safely through an intersection demands that decision sight distance be available on the major road approaches. The driver on the minor road requires adequate intersection sight distance, as well as the sight triangles described in Section 6.4, in order either to merge with traffic on the major road or to cross safely. It may be necessary to modify the alignment of either the major or the minor road, or both, to ensure that adequate sight distance is available. If this is not possible, the options available to the designer are to:

- relocate the intersection;
- provide all-way Stop control; or
- provide a Jug handle or Quarter link interchange, as described in Section 7.6.4.

Where heavy earthworks, possibly beyond the normal limits of the road reserve, are required to provide adequate sight distances, relocation may be an option.

The location of an intersection on a curve can create problems for the drivers on both legs of the minor road. Drivers on the minor road leg on the inside of the curve will have difficulty in seeing approaching traffic because this traffic will be partly behind them. This constitutes, in effect, an artificial angle of skew. Furthermore, there is a possibility that a portion of the sight triangle may fall outside the limits of the road reserve, which could hamper efforts to obtain a clear line of sight for the driver on the minor

<table>
<thead>
<tr>
<th>Design speed (km/h)</th>
<th>Radius (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>40</td>
<td>250</td>
</tr>
<tr>
<td>50</td>
<td>375</td>
</tr>
<tr>
<td>60</td>
<td>550</td>
</tr>
<tr>
<td>70</td>
<td>750</td>
</tr>
<tr>
<td>80</td>
<td>1,000</td>
</tr>
<tr>
<td>90</td>
<td>1,220</td>
</tr>
<tr>
<td>100</td>
<td>1,500</td>
</tr>
<tr>
<td>110</td>
<td>1,850</td>
</tr>
<tr>
<td>120</td>
<td>2,200</td>
</tr>
<tr>
<td>130</td>
<td>2,600</td>
</tr>
</tbody>
</table>
Drivers on the outside of a curve typically have little or no difficulty in seeing opposing vehicles on the major road. The opposing vehicles are partly in front of them and they have the additional height advantage caused by the super-elevation of the curve. However, they have to negotiate the turn onto the major road against an adverse super-elevation. The risks involved in sharp braking during an emergency should also be borne in mind when an intersection is located on a curve. In general, an intersection should not be located on a curve with a super-elevation greater than 6 per cent.

Stopping sight distances increase with steepening negative gradient. Stopping sight distance required on a gradient of -3 per cent is approximately 6 per cent longer than that on a level grade, whereas, on a -6 per cent gradient, it is approximately 16 per cent longer. It is suggested that, as a safety measure, intersections should not be located on gradients steeper than three per cent. The gradient is more critical on the minor road than on the major road because all vehicles on the minor road have to stop or yield.

One of the consequences of a collision between vehicles is that either or both may leave the road. It is therefore advisable to avoid locating intersections on high fills if at all possible. The obstruction of sight distance by bridge parapets should also be borne in mind. In the case of the crossing road ramp terminal at a narrow diamond interchange, both these problems may arise.

The location of an intersection may require modification to improve the angle of skew between the intersecting roads, as discussed in Section 6.3.1. If the angle of skew is less than 60°, the intersection can be replaced by two relatively closely spaced T-intersections. A vehicle on the minor road would thus follow the route comprising a right turn onto the major road, followed by a left turn off it. Any delay to the minor road vehicle would occur clear of the high-speed traffic on the major road. If the angle of skew is greater than 120°, relocation should be to a four-legged intersection in preference to the two T-intersections, because these would result in the minor road traffic following the route comprising a left turn onto the major road and a right turn off it with consequent delays to the major road traffic and increased risk to the vehicle waiting to complete the right turn off the major route. A right-left stagger or offset is, in short, to be preferred to a left-right stagger.

A further limitation on the location of intersections - being the spacing of successive intersections - is discussed in the following section.

### 6.2.7 Spacing of intersections

Designers seldom have influence on the spacing of roadways in a network as it is largely predicated by the original or developed land use. Nevertheless, the spacing of intersections impacts significantly on the operation, level of service and capacity of a roadway. It follows that intersection spacing should, inter alia, be based on road function and traffic volume. The principles described in the National Guidelines for Road Access Management in South Africa should therefore play a role in the determination...
of the location of individual intersections. This is of particular concern when the provision of a new intersection on an existing road is being considered.

Access management is aimed at maintaining an effective and efficient transportation system for the movement of people and goods, simultaneously supporting the development of the adjacent land use. Increasingly intensive land usage generally leads to demands for improved road infrastructure and the improved infrastructure makes access to it very attractive. Allowing access simply on the basis of its meeting some or other minimum geometric requirement results in increasing traffic conflicts and reduction in capacity so that the benefit of the original improvement is lost. This then leads to demands for further road improvements.

This cycle can only be broken by the development of a proper Access Management Plan by the local authority concerned. This plan specifies where intersections may be located. Furthermore, it defines the class of intersection that may be considered. Three classes of intersections are defined in the National Guidelines. These are:

- **Full access**, which allows for all possible movements at an intersection or access;
- **Partial access**, which allows left-in, left-out and right-in movements to and from a development or access road; and
- **Marginal access**, which allows only left-in and/or left-out movements to and from a development or access road.

These are illustrated in Figure 6.2.

Along signalised arterials, intersection spacing should be consistent with the running speed and signal cycle lengths, which are variables in themselves. If the spacing of the intersections is based on acceptable running speeds and cycle lengths, signal progression and an efficient use of the roadway can be achieved.

In Figure 6.2, these three variables are combined in a chart allowing the selection of a suitable intersection spacing. From this figure it transpires that the minimum spacing on arterial roads should be at least 400m. Where spacings closer than the minimum exist, a number of alternative actions can be considered, for example two-way flows can be converted to one-way operation or minor connecting roads can be closed or diverted, and channelisation can be used to restrict turning movements.

Where the crossing road of an interchange is an arterial, the suggested minimum distance along the arterial from the ramp terminal to the next intersection is 200m in the case of a collector road. If the next intersection is with an arterial the spacing between the ramp and the intersection should be increased to 600 metres for a Class 3 arterial and 800 metres for a Class 2 arterial.

The spacing between successive unsignalised intersections is measured by the separation between them with separation being defined as the distance between their reserve boundaries. Recommended access separations are provided in Table 6-5.

The left-in left-out class of access would typically only be applied under circumstances where
speeds or traffic volumes or both are high. This is in order to minimise the disruption that would be caused by right-turning vehicles. As such, it would normally be provided with acceleration and deceleration lanes using taper rates as listed in Table 6-12 and lengths listed in Table 7-5 or 7-6.

<table>
<thead>
<tr>
<th>Design speed (km/h)</th>
<th>Unsignalised marginal</th>
<th>All other access types</th>
</tr>
</thead>
<tbody>
<tr>
<td>40</td>
<td>20</td>
<td>80</td>
</tr>
<tr>
<td>50</td>
<td>35</td>
<td>110</td>
</tr>
<tr>
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<td>70</td>
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<td>80</td>
<td>100</td>
<td>200</td>
</tr>
<tr>
<td>100</td>
<td>170</td>
<td>300</td>
</tr>
<tr>
<td>120</td>
<td>250</td>
<td>350</td>
</tr>
</tbody>
</table>

Figure 6-2: Classes of intersections
6.2.8 Channelisation

The purpose of channelisation is to achieve safe and efficient operation by managing the conflicts that are inherent to intersections. NCHRP Report 279 reports that the objectives of good intersection design are:

- Reduction of the number of points of potential conflict to the minimum compatible with efficient operation;
- Reduction of the complexity of conflict areas whenever possible;
- Limitation of the frequency of actual conflicts; and
- Limitation of the severity of those conflicts that do occur.

To achieve these objectives there are nine principles of channelisation design:

- Undesirable or wrong-way movements should be discouraged or prohibited;
- Vehicle paths should be clearly defined;
- The design should encourage safe vehicle speeds;
- Undesirable or wrong-way movements should be discouraged or prohibited;
- Vehicle paths should be clearly defined;
- The design should encourage safe vehicle speeds;

![Figure 6.3: Desirable signal spacings](image)

Figure 6.3: Desirable signal spacings

Report 279 reports that the objectives of good intersection design are:

- Channelisation should separate points of conflict whenever possible;
- Traffic streams should cross at close to right angles and merge at flat angles;
- High priority flows should have the greater degree of freedom;
- The design should be in the context of the traffic control scheme;
- Decelerating, slow-moving or stopped vehicles should be separated from higher-speed through lanes; and
- Refuge for pedestrians and the handicapped should be provided where appropriate.
The tools available to apply these principles are:

- Defining and arranging traffic lanes;
- Traffic islands of all sizes and types;
- Median islands;
- Corner radii;
- Horizontal and vertical approach geometry;
- Pavement tapers and transitions; and
- Traffic control devices.

The first six elements are a range of physical features while traffic control devices are an integral part of any intersection. These six elements are discussed in Section 6.5.

### 6.3 GEOMETRIC CONTROLS

#### 6.3.1 Angle of intersection

The angle of intersection of two roadways influences both the operation and safety of an intersection. Large skews increase the pavement area and thus the area of possible conflict. Operationally they are undesirable because:

- Crossing vehicles and pedestrians are exposed for longer periods;
- The driver’s sight angle is more constrained and gap perception becomes more difficult;
- Vehicular movements are more difficult and large trucks require more pavement area; and
- Defining vehicle paths by channelisation is more difficult.

For new intersections the crossing angle should preferably be in the range 75° to 120°. The absolute minimum angle of skew is 60° because drivers, particularly of trucks with closed cabs, have difficulty at this angle of skew in seeing vehicles approaching from their left. The designer should be able to specifically justify using an angle of skew less than 75°. In the remodelling of existing intersections, the accident rates and patterns will usually indicate whether a problem exists and provide evidence on any problems related to the angle of skews.

#### 6.3.2 Horizontal and vertical alignment

The horizontal and vertical alignments through and approaching an intersection are critical features. Simple alignment design allows for early recognition of the intersection and timely focus on the intersecting traffic and manoeuvres that must be prepared.

The following are specific operational requirements at intersections:

- The alignments should not restrict the required sight distance;
- The alignments should allow for the frequent braking and turning associated with intersections; and
- The alignments should not require undue direct attention to be detracted from the intersection manoeuvres and conflict avoidance.

As a general guide, horizontal curve radii at intersections should not be less than the desirable radius for the design speed on the approach roads.

For high-speed roads with design speeds in excess of 80km/h, approach gradients should not be greater than - 3 per cent. For low-speed roads in an urban environment this can be increased to - 6 per cent.
distance at the intersection to above-minimum requirements.

For new intersections, the gradient on the minor roadway is normally adjusted to form a smooth profile, as suggested in Figure 6.4.

Where major roadways intersect, the profiles of both roads are usually adjusted in approximately equal manner. When significant channelisation is introduced in association with complex gradients, intersections should be designed on an elevation plan to avoid discontinuities and ensure free drainage.

### 6.3.3 Lane widths and shoulders

Where intersecting roadways have shoulders or sidewalks, the main road shoulder should be continued through the intersection. Lane widths should be 3.7 m for through lanes and 3.6 metres for turning lanes. Where conditions are severely constrained, lane widths as low as 3.3 metres can be considered provided that approach speeds are below 80 km/h. In constricted urban conditions on low speed-roadways, lane widths of 3.0 metres should be the minimum adopted. Offsets from the edge of the turning roadway to kerb lines should be 0.6 to 1.0 metres.

### 6.4 SIGHT DISTANCE

#### 6.4.1 General Considerations

The provision of adequate sight distances and appropriate traffic controls is essential for safe intersection operation.

Stopping sight distance should be provided continuously on all roadways including at the approaches to intersections. However, in rural areas or when approach speeds are in excess of 80 km/h, the decision sight clearance set out in Section 3.5.8 should be provided on all approaches to intersections for safe operation, particularly where auxiliary lanes are added to the intersection layout to accommodate the turning movements.

In addition to these forms of sight distance, it is necessary to provide Intersection Sight Distance (ISD). This is the sight distance required by drivers entering the intersection to enable them to establish that it is safe to do so and then to carry out the manoeuvres necessary either to join or to cross the opposing traffic streams. Previously, values of ISD were derived from elaborate models based on assumptions of reaction times, speeds and acceleration rates of turning vehicles and the deceleration rates of the opposing vehicles, etc. The distances offered in sub-section 6.4.3 are derived from research into gap acceptance as reported in NCHRP Report 383 "Intersection Sight Distance".
6.4.2 Sight Triangles

Each quadrant of an intersection should contain a clear sight triangle free of obstructions that may block a driver's view of potentially conflicting vehicles on the opposing approaches. Two different forms of sight triangle are required. In the first instance, reference is to approach sight triangles. The approach triangle will have sides with sufficient lengths on both intersecting roadways such that drivers can see any potentially conflicting vehicle in sufficient distance.

Figure 6.4: Fitting minor road profiles to major road cross-sections
time to slow, or to stop if need be, before entering the intersection. For the departure sight triangle, the line of sight described by the hypotenuse of the sight triangle should be such that a vehicle just coming into view on the major road will, at the design speed of this road, have a travel time to the intersection corresponding to the gap acceptable to the driver of the vehicle on the minor road. Both forms of sight triangle are required in each quadrant of the intersection.

The line of sight assumes a driver eye height of 1.05 metres and an object height of 1.3 metres.

The approach and departure sight triangles are illustrated in Figure 6.5. The areas shown shaded should be kept clear of vegetation or any other obstacle to a clear line of sight. To this end, the road reserve is normally splayed to ensure that the entire extent of the sight triangle is under the control of the road authority. Furthermore, the profiles of the intersecting roads should be designed to provide the required sight distance. Where one or other of the approaches is in cut, the affected sight triangles may have to be "day lighted", i.e. the natural material occurring within the sight triangles may have to be excavated to ensure intervisibility between the relevant approaches.

Sight distance values are based on the ability of the driver of a passenger car to see an approaching passenger car. It is also necessary to check whether the sight distance is adequate for trucks. Because their rate of acceleration is lower than that of passenger cars and as the distance that the truck has to travel to clear the intersection is longer, the gap acceptable to a truck driver is considerably greater than that required by the driver of a passenger car. For design purposes, the eye height of truck drivers is taken as 1.8 metres for checking the availability of sight distance for trucks.

### 6.4.3 Intersection control

The recommended dimensions of the clear sight triangles vary with the type of traffic control used at an intersection because different types of control impose different legal constraints on drivers resulting in different driver behaviour. Sight distance policies for intersections with the following types of traffic control are presented below:

- Intersections with no control (Case A);
- Intersections with Stop control on the minor road (Case B);
  - Right turn from the minor road (Case B1);
  - Left turn from the minor road (Case B2);
  - Crossing manoeuvre from the minor road (Case B3);
- Intersections with Yield control on the minor road (Case C);
  - Crossing manoeuvre from the minor road (Case C1);
  - Left or right turn from the minor road (Case C2);
- Intersections with traffic signal control (Case D); and
- Intersections with all-way Stop control (Case E).

A sight-distance policy for stopped vehicles turning right from a major road (Case F) is also presented.
Intersections with no control (Case A)

Uncontrolled intersections are not used in conjunction with the main road network, but are common in rural networks and access roads to rural settlements. In these cases, drivers must be able to see potentially conflicting vehicles on intersecting approaches in sufficient time to stop safely before reaching the intersection. Ideally, sight triangles with legs equal to stopping sight distance should be provided on all the approaches to uncontrolled intersections.

If sight triangles of this size cannot be provided, the lengths of the legs on each approach can be determined from a model that is analogous to the stopping sight distance model, with slightly different assumptions.

Field observations indicate that vehicles approaching uncontrolled intersections typically slow down to approximately 50 per cent of their normal running speed. This occurs even when no potentially conflicting vehicles are present, typically at deceleration rates of up to 1.5m/s.

Figure 6.5: Sight triangles
Braking at greater deceleration rates, which can approach those assumed in the calculation of stopping sight distances, begins up to 2.5 seconds after a vehicle on the intersecting approach comes into view. Thus, approaching vehicles may be travelling at less than their normal running speed during all or part of the perception-reaction time and can brake to a stop from a speed less than the normal running speed.

Table 6.5 shows the distance travelled by an approaching vehicle during perception, reaction and braking time as a function of the design speed of the roadway on which the intersection approach is located. These distances should be

<table>
<thead>
<tr>
<th>Design speed (km/h)</th>
<th>Sight distance (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>20</td>
<td>20</td>
</tr>
<tr>
<td>30</td>
<td>25</td>
</tr>
<tr>
<td>40</td>
<td>30</td>
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<td>50</td>
<td>40</td>
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<td>60</td>
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<td>65</td>
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<td>80</td>
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<td>100</td>
<td>120</td>
</tr>
<tr>
<td>110</td>
<td>140</td>
</tr>
<tr>
<td>120</td>
<td>165</td>
</tr>
</tbody>
</table>

Table 6.6: Adjustment factors for approach sight distances based on approach gradient

<table>
<thead>
<tr>
<th>Approach gradient (%)</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>
<th>110</th>
<th>120</th>
</tr>
</thead>
<tbody>
<tr>
<td>-6</td>
<td>1.1</td>
<td>1.1</td>
<td>1.1</td>
<td>1.1</td>
<td>1.1</td>
<td>1.2</td>
<td>1.2</td>
<td>1.2</td>
<td>1.2</td>
<td>1.2</td>
</tr>
<tr>
<td>-5</td>
<td>1.0</td>
<td>1.1</td>
<td>1.1</td>
<td>1.1</td>
<td>1.1</td>
<td>1.1</td>
<td>1.1</td>
<td>1.1</td>
<td>1.2</td>
<td>1.2</td>
</tr>
<tr>
<td>-4</td>
<td>1.0</td>
<td>1.0</td>
<td>1.1</td>
<td>1.1</td>
<td>1.1</td>
<td>1.1</td>
<td>1.1</td>
<td>1.1</td>
<td>1.1</td>
<td>1.1</td>
</tr>
<tr>
<td>-3 to +3</td>
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<td>1.0</td>
<td>1.0</td>
<td>1.0</td>
<td>1.0</td>
<td>1.0</td>
<td>1.0</td>
<td>1.0</td>
<td>1.0</td>
<td>1.0</td>
</tr>
<tr>
<td>+4</td>
<td>1.0</td>
<td>1.0</td>
<td>1.0</td>
<td>0.9</td>
<td>0.9</td>
<td>0.9</td>
<td>0.9</td>
<td>0.9</td>
<td>0.9</td>
<td>0.9</td>
</tr>
<tr>
<td>+5</td>
<td>1.0</td>
<td>1.0</td>
<td>0.9</td>
<td>0.9</td>
<td>0.9</td>
<td>0.9</td>
<td>0.9</td>
<td>0.9</td>
<td>0.9</td>
<td>0.9</td>
</tr>
<tr>
<td>+6</td>
<td>1.0</td>
<td>0.9</td>
<td>0.9</td>
<td>0.9</td>
<td>0.9</td>
<td>0.9</td>
<td>0.9</td>
<td>0.9</td>
<td>0.9</td>
<td>0.9</td>
</tr>
</tbody>
</table>

Note: Based on ratio of stopping sight distance on specified approach grade to stopping sight distance on level terrain.
used as the legs of the sight triangles shown in Figure 6.4.

Where the gradient of an intersection approach exceeds three per cent, the leg of the clear sight triangle along that approach should be adjusted by multiplying the sight distance in Table 6.5 by an adjustment factor in Table 6.6.

If these sight distances cannot be provided, advisory speed signing to reduce speeds or installing Stop signs on one or more approaches should be investigated.

Uncontrolled intersections do not normally require departure sight triangles because they typically have very low traffic volumes. If a motorist finds it necessary to stop at an uncontrolled intersection because of the presence of a conflicting vehicle, it is unlikely that another potentially conflicting vehicle will be encountered as the first vehicle departs the intersection.

*Intersections with Stop control on the minor road (Case B)*

Departure sight triangles for intersections with Stop control on the minor road should be considered for three situations:
- Right turns from the minor road (Case B1);
- Left turns from the minor road (Case B2); and
- Crossing the major road from the minor-road approach (Case B3).

Approach sight triangles, as shown in Figure 6.4(A), need not be provided at Stop-controlled intersections because all minor-road vehicles should stop before entering or crossing the major road.

Vehicles turning right from the minor road have to cross the stream of traffic approaching from the right and then merge with the stream approaching from the left. Left-turning vehicles need only merge with the stream approaching from the right. As the merging manoeuvre requires that turning vehicles should be able to accelerate approximately to the speed of the stream with which they are merging, it necessitates a gap longer than that for the crossing manoeuvre.

*Right turn from the minor road (Case B1)*

A departure sight triangle for traffic approaching from the left, as shown in Figure 6.4(B), should be provided for right turns from the minor road onto the major road for all Stop-controlled approaches.

Field observations of the gaps accepted by the drivers of vehicles turning to the right onto the major road have shown that the values in Table 6.7 provide sufficient time for the minor-road vehicle to accelerate from a stop and merge with the opposing stream without undue interference. These observations also revealed that major-road drivers would reduce their speed to some extent to accommodate vehicles entering from the minor road. Where the gap acceptance values in Table 6.7 are used to determine the length of the leg of the departure sight triangle along the major road, most major-road drivers need not reduce speed to less than 70 percent of their initial speed.
Table 6.7 applies to passenger cars. However, for minor-road approaches from which substantial volumes of heavy vehicles enter the major road, the values for single-unit trucks or semi-trailers should be applied.

Table 6.7 includes adjustments to the acceptable gaps for the number of lanes on the major road and for the approach gradient of the minor road. The adjustment for the gradient of the minor-road approach need be made only if the rear wheels of the design vehicle would be on an upgrade steeper than 3 per cent when the vehicle is at the stop line of the minor-road approach.

The length of the sight triangle along the major road (distance "b" in Figure 6.4) is the product of the design speed of the major road in metres/second and the critical gap in seconds as listed in Table 6.7.

If the sight distances along the major road based on Table 6.7 (including the appropriate adjustments) cannot be provided, consideration should be given to the installation of advisory speed signs on the major-road approaches.

Dimension "a" in Figure 6.4 (b) depends on the context within which the intersection is being designed. In urban areas, drivers tend to stop their vehicles immediately behind the Stop line, which may be located virtually in line with the edge of the major road. A passenger car driver would, therefore, be located about 2.4 metres away from the Stop line. In rural areas, vehicles usually stop at the edge of the shoulder of the major road. In the case of a three metre wide shoulder the driver would thus be approximately 5.4 metres away from the edge of the travelled way.

<table>
<thead>
<tr>
<th>Design vehicle</th>
<th>Travel time (s) at design speed of major road</th>
</tr>
</thead>
<tbody>
<tr>
<td>Passenger car</td>
<td>7.5</td>
</tr>
<tr>
<td>Single-unit truck</td>
<td>9.5</td>
</tr>
<tr>
<td>Semi-trailer</td>
<td>11.5</td>
</tr>
</tbody>
</table>

Adjustment for multilane highways:
For right turns onto two-way highways with more than two lanes, add 0.5 seconds for passenger cars or 0.7 seconds for trucks for each additional lane, in excess of one, to be crossed by the turning vehicle.
For left turns, no adjustment is necessary.

Adjustment for approach gradients:
- If the approach gradient on the minor road exceeds +3 per cent:
  - Add 0.1 seconds per percent gradient for left turns
  - Add 0.2 seconds per percent gradient for right turns
Where the major road is a dual carriageway, two departure sight triangles have to be considered: a sight triangle to the right, as for the crossing movement (Case B3) and one using the acceptable gap as listed in Table 6.7 for vehicles approaching from the left. This presupposes that the width of the median is sufficient to provide a refuge for the vehicle turning from the minor road. If the median width is inadequate, the adjustment in Table 6.7 for multilane major roads should be applied with the median being counted as an additional lane.

The departure sight triangle should be checked for various possible design vehicles because the width of the median may be adequate for one vehicle type and not for another so that two different situations have to be evaluated.

**Left turn from the minor road (Case B2)**

A departure sight triangle for traffic approaching from the right, as shown in Figure 6.4, should be provided for left turns from the minor road. The lengths of the legs of the departure sight triangle for left turns should generally be the same as those for the right turns used in Case B1. Specifically, the length of the leg of the departure sight triangle (dimension "b") along the major road should be based on the travel times in Table 6.7, including appropriate adjustment factors.

Dimension "a" depends on the context of the design and can vary from 2.4 metres to 5.4 metres.

Where sight distances along the major road based on the travel times from Table 6.7 cannot be provided, it should be kept in mind that field observations indicate that, in making left turns, drivers generally accept gaps that are slightly shorter than those accepted in making right turns. The travel times in Table 6.7 can be decreased by 1.0 to 1.5 seconds for left turn manoeuvres, where necessary, without undue interference with major-road traffic. When the recommended sight distance for a left-turn manoeuvre cannot be provided, even with a reduction of 1.0 to 1.5 seconds, consideration should be given to the installation of advisory speed signs and warning devices on the major-road approaches.

**Crossing manoeuvre from the minor road (Case B3)**

In most cases it can be assumed that the departure sight triangles for right and left turns onto the major road, as described for Cases B1 and B2, will also provide more than adequate sight distance for minor-road vehicles crossing the major road. However, it is advisable to check the availability of sight distance for crossing manoeuvres:

- Where right and/or left turns are not permitted from a particular approach and crossing is the only legal manoeuvre;
- Where the crossing vehicle has to cross four or more lanes; or
- Where substantial volumes of heavy vehicles cross the highway and where there are steep gradients on the departure roadway on the far side of the intersection that might slow the vehicle while its rear is still in the intersection.

Table 6.8 presents travel times and appropriate adjustment factors that can be used to deter-
mine the length of the leg of the sight triangle along the major road to accommodate crossing manoeuvres.

At divided highway intersections, depending on the width of the median and the length of the design vehicle, sight distance may be needed for crossing both roadways of the divided highway or for crossing the near lanes only and stopping in the median before proceeding.

For four-legged intersections with Yield control on the minor road, two separate sets of approach sight triangles as shown in Figure 6.5(A) should be provided: one set of approach

<table>
<thead>
<tr>
<th>Design vehicle</th>
<th>Travel time (s) at Design speed of major road</th>
</tr>
</thead>
<tbody>
<tr>
<td>Passenger car</td>
<td>6.5</td>
</tr>
<tr>
<td>Single-unit truck</td>
<td>8.5</td>
</tr>
<tr>
<td>Semi trailer</td>
<td>10.5</td>
</tr>
</tbody>
</table>

Adjustment for multilane highways:
For crossing a major road with more than two lanes, add 0.5 seconds for passenger cars and 0.7 seconds for trucks for each additional lane to be crossed. In the case of dual carriageways with inadequate width of median for refuge, count the median as another lane to be crossed.

Adjustment for approach grades:
If the approach gradient of the minor road exceeds +3 %, add 0.2 seconds per percent gradient in excess of 3 %.

a For minor-road approach gradients that exceed +3 per cent, multiply by the appropriate adjustment factor from Table 6.6.
b Travel time applies to a vehicle that slows before crossing the intersection but does not stop.

Intersections with Yield control on the minor road (Case C)

Vehicles entering a major road at a Yield-controlled intersection may, because of the presence of opposing vehicles on the major road, be required to stop. Departure sight triangles as described for Stop control must therefore be provided for the Yield condition. However, if no conflicting vehicles are present, drivers approaching Yield signs are permitted to enter sight triangles to accommodate right and left turns onto the major road and the other for crossing movements. Both sets of sight triangles should be checked for potential sight obstructions.

Crossing manoeuvres (Case C1)

The lengths of the leg of the approach sight triangle along the minor road to accommodate the crossing manoeuvre from a Yield-controlled
approach (distance "a" in Figure 6.5(A) are given in Table 6.9. The distances in Table 6.9 are based on the same assumptions as those for Case A control except that, based on field observations, minor-road vehicles that do not stop are assumed to decelerate to 60 per cent of the minor-road design speed rather than to 50 per cent. The distances and times in Table 6.9 should be adjusted for the gradient of the minor road approach, using the factors in Table 6.6.

The length of the leg of the approach sight triangle along the major road to accommodate the crossing manoeuvre (distance "b" in Figure 6.5(A)) should be calculated using the following equations:

\[ t_c = t_a + \frac{w + L_a}{0.167 V_{\text{minor}}} \]  

\[ b = 0.278 V_{\text{major}} t_c \]  

where:
- \( t_c \) = travel time to reach and clear the major road in a crossing manoeuvre (sec)
- \( b \) = length of leg of sight triangle along the major road (m)
- \( t_a \) = travel time to reach the major road from the decision point for a vehicle that does not stop (sec) (use appropriate value for the minor-road design speed from Table 6.9, adjusted for approach grade, where appropriate)
- \( w \) = width of intersection to be crossed (m)
- \( L_a \) = length of design vehicle (m)
- \( V_{\text{minor}} \) = design speed of minor road (km/h)
- \( V_{\text{major}} \) = design speed of major road (km/h)

These equations provide sufficient travel time for the major road vehicle, during which the minor-road vehicle can:
(1) Travel from the decision point to the intersection, while decelerating at the rate of 1.5m/s² to 60 per cent of the minor-road design speed; and then
(2) Cross and clear the intersection at the same speed.

Field observations did not provide a clear indication of the size of the gap acceptable to the driver of a vehicle located at the decision point on the minor road. If the required gap is longer than that indicated by the above equations, the driver would, in all probability, bring the vehicle to a stop and then select a gap on the basis of Case B. If the acceptable gap is shorter than that indicated by the above equations, the sight distance provided would, at least, provide a margin of safety.

If the major road is a divided highway with a median wide enough to store the design vehicle for the crossing manoeuvre, then only crossing of the near lanes need be considered and a departure sight triangle for accelerating from a stopped position in the median should be provided, based on Case B1.

**Left and right-turn manoeuvres (Case C2)**

To accommodate left and right turns without stopping (distance "a" in Figure 6.5(A)), the length of the leg of the approach sight triangle along the minor road should be 25 metres. This distance is based on the assumption that drivers making right or left turns without stopping will slow to a turning speed of 15 km/h. The length
Chapter 6: Intersection Design

The leg of the approach sight triangle along the major road (distance "b" in Figure 6.5(B)) is similar to that of the major-road leg of the departure sight triangle for Stop-controlled intersections in Cases B1 and B2.

For a Yield-controlled intersection, the travel times in Table 6.7 should be increased by 0.5 seconds. The minor-road vehicle requires 3.5 seconds to travel from the decision point to the intersection. These 3.5 seconds represent additional travel time that is needed at a Yield-controlled intersection (Case C). However, the acceleration time after entering the major road is 3.0 seconds less for a Yield sign than for a Stop sign because the turning vehicle accelerates from 15 km/h rather than from a stop. The net 0.5 seconds increase in travel time for a vehicle turning from a Yield-controlled approach is the difference between the 3.5 second increase in travel time on approach and the 3.0 second reduction in travel time on departure explained above.

Since approach sight triangles for turning manoeuvres at Yield-controlled are larger than the departure sight triangles used at Stop-controlled intersections, no specific check of departure sight triangles at Yield-controlled intersections should be necessary.

**Intersections with traffic signal control (Case D)**

In general, approach or departure sight triangles are not needed for signalised intersections. Indeed, signalisation may be an appropriate accident countermeasure for higher volume intersections with restricted sight distance and a history of sight-distance related accidents.

However, traffic signals may fail from time to time. Furthermore, traffic signals at an intersection are sometimes placed on two-way flashing operation under off-peak or night time conditions. To allow for either of these eventualities, the appropriate departure sight triangles for

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**Table 6.9: Leg of approach sight triangle along the minor road to accommodate crossing manoeuvres from yield-controlled approaches**

<table>
<thead>
<tr>
<th>Design speed (km/h)</th>
<th>Distance along minor road (m)</th>
<th>Travel time from decision point to major road ($t_d$)$^{a,b}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>30</td>
<td>30</td>
<td>3.4</td>
</tr>
<tr>
<td>40</td>
<td>40</td>
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<td>7.8</td>
</tr>
<tr>
<td>120</td>
<td>230</td>
<td>8.6</td>
</tr>
</tbody>
</table>

---

*a* For minor-road approach gradients that exceed +3 per cent, multiply by the appropriate adjustment factor from Table 6.6.

*b* Travel time applies to a vehicle that slows before crossing the intersection but does not stop.
Case B, both to the left and to the right, should be provided for the minor-road approaches.

**Intersections with all-way Stop control (Case E)**

At intersections with all-way Stop control, the first stopped vehicle on each approach would be visible to the drivers of the first stopped vehicles on each of the other approaches. It is thus not necessary to provide sight distance triangles at intersections with All-way Stop control. All-way Stop control may be an option to consider where the sight distance for other types of control cannot be achieved. This is particularly the case if signals are not warranted.

**Right turns from a major road (Case F)**

Right-turning drivers need sufficient sight distance to enable them to decide when it is safe to turn right across the lane(s) used by opposing traffic. At all locations, where right turns across opposing traffic are possible, there should be sufficient sight distance to accommodate these manoeuvres. Since a vehicle that turns right without stopping needs a gap shorter than that required by a stopped vehicle, the need for sight distance design should be based on a right turn by a stopped vehicle.

The sight distance along the major road to accommodate right turns is the distance that would be traversed at the design speed of the major road in the travel time for the appropriate design vehicle given in Table 6.10. This table also contains appropriate adjustment factors for the number of major-road lanes to be crossed by the turning vehicle.

<table>
<thead>
<tr>
<th>Design vehicle</th>
<th>Travel time (s) at design speed of major road</th>
</tr>
</thead>
<tbody>
<tr>
<td>Passenger car</td>
<td>5,5</td>
</tr>
<tr>
<td>Single-unit truck</td>
<td>6,5</td>
</tr>
<tr>
<td>Semi trailer</td>
<td>7,5</td>
</tr>
</tbody>
</table>

Adjustment for multilane highways:

For right turns that have to cross more than one opposing lane, add 0,5 s for passenger cars and 0,7 s for trucks for each additional lane to be crossed. In the case of dual carriageways where the median is not sufficiently wide to provide refuge for the turning vehicle, the median should be regarded as an additional lane to be crossed.

If stopping sight distance has been provided continuously along the major road and, if sight distance for Case B (Stop control) or Case C (Yield control) has been provided for each minor-road approach, sight distance should generally be adequate for right turns from the major road. However, at intersections or driveways located on or near horizontal or vertical curves on the major road, the availability of adequate sight distance for right turns from the major road should be checked. In the case of dual carriageways, the presence of sight obstructions in the median should also be checked.
At four-legged intersections, opposing vehicles turning right can block a driver’s view of oncoming traffic. If right-turn lanes are provided, offsetting them to the right, to be directly opposite one other, will provide right-turning drivers with a better view of oncoming traffic.

Effect of skew on sight distance

When two highways intersect at an angle outside the range of 75° to 120° and where realignment to increase the angle of intersection is not justified, some of the factors for determination of intersection sight distance will need adjustment.

Each of the clear sight triangles described above is applicable to oblique-angle intersections. As shown in Figure 6.6, the legs of the sight triangle will lie along the intersection approaches and each sight triangle will be larger or smaller than the corresponding sight triangle would be at a right-angle intersection. The area within each sight triangle should be clear of sight obstructions, as described above.

At skew intersections, the length of the travel paths for crossing manoeuvres will be increased. The actual path length for a crossing manoeuvre can be calculated by dividing the total width of the lanes (plus the median width, where appropriate) to be crossed by the sine of the intersection angle and adding the length of the design vehicle. The actual path length divided by the lane width applied to the major road cross-section gives the equivalent number of lanes to be crossed. This is an indication of the number of additional lanes to be applied to the adjustment factor shown in Table 6.8 for Case B3.

The sight distances offered for Case B can, regardless of the form of control, also accommodate turning movements from the minor road to the major road at skew intersections. In the obtuse angle, drivers can easily see the full

Figure 6.6: Effect of skew on sight distance at intersections
sight triangle and, in addition, often accelerate from the minor road at a higher rate than when they have to negotiate a ninety-degree change of direction. In the acute-angle quadrant, drivers are often required to turn their heads considerably to see across the entire clear sight triangle. For this reason, it is suggested that Case A should not be applied to oblique-angle intersections. Stop or Yield control should be applied and the sight distances appropriate to either Case B or Case C provided. Even in a skew intersection it is usually possible for drivers to position their vehicles at approximately 90° to the major road at the Stop line, offering added support for the application of Case B for skew intersections.

When driving through a deflection angle greater than 120°, the right turn to the minor road may be undertaken at crawl speeds. Allowance could be made for this by adding the time, equivalent to that required for crossing an additional lane, to the acceptable gap.

6.5 CHANNELISATION ELEMENTS

At-grade intersections with large paved areas, such as those with large corner radii or with angles of skew differing greatly from 90°, permit unpredictable vehicle movements, require long pedestrian crossings and have unused pavement areas. Even at a simple intersection there may be large areas on which vehicles can wander from natural or expected paths. Under these circumstances it is usual to resort to channelisation.

As stated in sub-section 6.2.8, the fundamental function of channelisation is to manage the conflicts that are inherent in any intersection. In that sub-section, nine principles of channelisation are listed. In essence, channelisation is the process whereby a vehicle can be guided safely through the intersection area from an approach leg to the selected departure leg. Guidance is offered by lane markings that clearly define the required vehicle path and also indicate auxiliary lanes for turning movements. A variety of symbols is also used as road markings to indicate inter alia that turns, either to the left or to the right, from selected lanes are mandatory. At intersections that are complex or have high volumes of turning traffic, it is usually necessary to reinforce the guidance offered by road markings by the application of:

- Channelising islands;
- Medians and median end treatments;
- Corner radii;
- Approach and departure geometry;
- Pavement tapers and transitions;
- Traffic control devices including signs and signals; and
- Arrangement and position of lanes

6.5.1 Channelising islands

Islands are included in intersections for one or more of the following purposes:

- Separation of conflicts;
- Control of angle of conflict;
- Reduction of excessive pavement areas;
- Regulation of traffic and indication of the proper use of the intersection;
- Arrangement to favour a predominant turning movement;
- Protection of pedestrians;
- Protection and storage of turning vehicles; and
- Location of traffic control devices.
The three main functions of channelising islands are thus:

- **Directional** - to control and direct movements, usually turning;
- **Division** - which can be of opposing or same direction, usually through, movements; and
- **Refuge** - either of turning vehicles or of pedestrians.

Typical island shapes are illustrated in Figure 6.7.

The designer should bear in mind that islands are hazards and should be less hazardous than whatever they are replacing.

Islands may be kerbed, painted or simply non-paved. Kerbed islands provided the most positive traffic delineation and are normally used in urban areas to provide some degree of protection to pedestrians and traffic control devices. Painted islands are usually used in suburban areas where speeds are low, e.g. in the range of 50 km/h to 70 km/h and space limited. In rural areas, kerbs are not common and, at the speeds prevailing in these areas, typically 120 km/h or more, they are a potential hazard. If it is necessary to employ kerbing at a rural intersection, the use of mountable kerbing should be considered. As an additional safety measure, a kerbed island could be preceded by a painted
island. Non-paved islands are defined by the pavement edges and are usually used for large islands at rural intersections. These islands may have delineators on posts and may be landscaped.

Islands are generally either long or triangular in shape, with the circular shape being limited to application in roundabouts. They are situated in areas not intended for use in vehicle paths.

Drivers tend to find an archipelago of small islands confusing and are liable to select an incorrect path through the intersection area. As a general design principle, a few large islands are thus to be preferred to several small islands. Islands should not be less than about 5 square metres in area to ensure that they are easily visible to approaching drivers.

Directional islands are typically triangular with their dimensions and exact shape being dictated by:

- The corner radii and associated tapers;
- The angle of skew of the intersection;
- And
- The turning path of the design vehicle.

A typical triangular island is illustrated in Figure 6.8. The approach ends of the island usually have a radius of about 0.6 metres and the offset between the island and the edge of the travelled way is typically 0.6 to 1.0 metres to allow for the effect of kerbing on the lateral placement of moving vehicles. Where the major road has

---

**Figure 6.8: Typical directional island**
shoulders, the nose of the island is offset about one metre from the edge of the usable shoulder, the side adjacent to the through lane being tapered back to terminate at the edge of the usable shoulder, thus offering some guidance and redirection. A kerbed cross-section on the major road suggests that the nose of the island should be offset by about 1.6 metres from the edge of the travelled way, with the side adjacent to the through lane being tapered back to terminate 0.6 metres from the edge of the through lane.

Dividing, or splitter, islands usually have a teardrop shape. They are often employed on the minor legs of an intersection where these legs have a two-lane, two-way or four-lane undivided cross-section. The principle function of a dividing island is to warn the driver of the presence of the intersection. This can be achieved by the left edge of the island being, at the widest point of the island, in line with the left edge of the approach leg. To the approaching driver, it would thus appear as though the entire lane had been blocked off by the island. If space does not permit this width of island, a lesser blocking width would have to be applied but it is doubted

Note:

1. Width of island controlled by nose placement as determined by control radii and angle of intersection or added right turn lane.
2. Vertical and horizontal alignment on approach to provide minimum stopping sight distance.
3. Control dimension, W, should be from edge to edge of pavement and should not include channel.

\[
\begin{align*}
W_1 &= \text{Undivided approach width} \\
W_2 &= \text{Divided approach width} \\
W_3 &= \frac{W_1}{2} \text{ or } 4.3 \text{ m whichever is larger} \\
W_4 &= \frac{W_3 + W_2}{2} \text{ desirable} \\
W_5 &= W_2 + 0.3 \text{ m}
\end{align*}
\]

**Figure 6.9: Typical divisional (splitter) island**

...
whether anything less than half of the approach lane width would be effective. The taper that can be employed to achieve this effect safely is discussed in Section 6.5.3. A typical dividing island is illustrated in Figure 6.9.

The shape of the splitter island discussed in Section 6.6.5 is derived from the need to redirect vehicles entering a roundabout through an angle of not more than $30^\circ$. Although it serves to create the illusion of about a half of the approach lane being blocked off, its true function is to achieve the desired extent of deflection. Furthermore, this form of splitter island does not accommodate vehicles turning from the left. In effect, it has a teardrop shape, albeit distorted by its abutting a curving roadway with a relatively short radius rather than a straight road.

The kerb height should ensure that the island would be visible within normal stopping sight distance. However, it may be advisable to draw the driver’s attention to the island by highlighting the kerbs with paint or reflective markings.

As in the case of the triangular island, the nose of the dividing island should be offset by about one metre but, in this case, to the right of the centreline of the minor road. Dividing islands are usually kerbed to enhance their visibility and the offset between the kerbing and the edge of the travelled way should thus be 0.6 metres as discussed above. For the sake of consistency, the radius of the nose should be of the order of 0.6 metres.

The balance of the shape of the island is defined by the turning paths of vehicles turning to the right, both from the minor road to the major road and from the major road to the minor.

Median islands are discussed in Section 4.4.6 and outer separator islands in Section 4.4.7. At intersections, the end treatment of median islands is important. The width of the opening between two median ends should match the width of the minor road, including its shoulders, or, where the minor road is kerbed, the opening should not be narrower than the surfaced width of the minor road plus an offset of 0.6 to 1.0 metres.

The median end treatment is determined by the width of the median. Where the median is three metres wide or less, a simple semicircle is adequate. For wider medians, a bullet nose end treatment is recommended. The bullet nose is formed by arcs dictated by the wheel paths of turning vehicles and an assumed nose radius of 0.6 to 1.0 metres. It results in less intersection pavement area and a shorter length of opening than the semicircular end. Above a median width of five metres, the width of the minor road controls the length of the opening. A flattened bullet nose, using the arcs as for the conventional bullet nose but with a flat end as dictated by the width of the crossing road and parallel to the centreline of the minor road, is recommended. These end treatments are illustrated in Figure 6.10.

The bullet nose and the flattened bullet nose have the advantage over the semicircular end treatment that the driver of a right turning vehicle has a better guide for the manoeuvre for most of the turning path. Furthermore, these end treatments result in an elongated median,
which is better placed to serve as a refuge for pedestrians crossing the dual carriageway road. An additional disadvantage of the use of the semicircular end treatment for wide medians is that, whereas the bullet nose and the flattened bullet nose both guide the vehicle towards the left of the centreline of the minor road, the semicircular end treatment tends to direct the vehicle into the opposing traffic lane of the minor road.

6.5.2 Turning roadway widths

Directional islands are bounded by the major and minor roads and by a short length of one-way, typically one-lane, turning roadway. The width of the turning roadway is defined by the swept area of the design vehicle for the selected radius of curvature and the type of operation envisaged. Reference is typically to three types of operation, being:

<table>
<thead>
<tr>
<th>Case</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>One-lane one-way travel with no provision for passing stopped vehicles;</td>
</tr>
<tr>
<td>2</td>
<td>One-lane one-way travel with provision for passing a stopped vehicles; and</td>
</tr>
<tr>
<td>3</td>
<td>Two-lane one-way operation</td>
</tr>
</tbody>
</table>

Three traffic conditions should also be considered, being:

<table>
<thead>
<tr>
<th>Condition</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>Insufficient SU vehicles in the turning traffic stream to influence design;</td>
</tr>
<tr>
<td>B</td>
<td>Sufficient SU vehicles to influence design; and</td>
</tr>
<tr>
<td>C</td>
<td>Sufficient semi-trailers to</td>
</tr>
</tbody>
</table>

Figure 6.10: Typical median end treatments
Influence design

Turning roadways are short so that design for Case 1 is usually adequate. It is reasonable to assume, even in the absence of traffic data, that there will be enough trucks in the traffic stream to warrant the application of Condition B to the design. Turning roadway widths are listed in Table 6.11.

Three-centred curves are an effective alternative to the single radius curves listed in Table 6.11. These curves typically have a ratio of 3:1:3 between the successive radii. However, asymmetric combinations, e.g. 2:1:4, have also proved very useful in the past. These curves closely follow the wheel path of a vehicle negotiating the turn thus enabling the use of narrower lanes than with a single radius curve. In addition, three-centre curves allow the use of smaller central radii than do the equivalent single curves. Under Case C conditions, a 55:20:55 metre radius three-centred curve is the equivalent of a thirty metre single radius curve and permits the use of a 6.0 metre wide turning roadway compared to the 6.4 metre lane width of the single curve. The three-centred curve is particularly useful for Case C conditions because semi-trailers require an inordinate width of turning roadway. For example, the required width of turning roadway for Case 1, Condition C and a design speed of 20 km/h is 7.9 metres whereas, under the same circumstances, passenger cars require only 4.0 metres. Drivers of passenger cars could thus quite easily perceive the turning roadway as being intended for two-lane operation.

It is not possible to list all the possible alternative three-centred curve combinations. When three-centred curves are considered, the designer should determine the required roadway width by the use of templates.

### 6.5.3 Tapers

There are two types of taper, each with different geometric requirements. These are:
- The active taper, which forces a lateral transition of traffic; and

<table>
<thead>
<tr>
<th>Design speed (km/h)</th>
<th>Inner radius (m)</th>
<th>Case 1 Condition</th>
<th>Case 2 Condition</th>
<th>Case 3 Condition</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>A</td>
<td>B</td>
<td>C</td>
<td>A</td>
</tr>
<tr>
<td>20</td>
<td>15</td>
<td>4.0</td>
<td>5.5</td>
<td>7.9</td>
</tr>
<tr>
<td>25</td>
<td>20</td>
<td>4.0</td>
<td>5.2</td>
<td>6.7</td>
</tr>
<tr>
<td>30</td>
<td>30</td>
<td>4.0</td>
<td>4.9</td>
<td>6.4</td>
</tr>
<tr>
<td>35</td>
<td>40</td>
<td>3.7</td>
<td>4.9</td>
<td>6.4</td>
</tr>
<tr>
<td>40</td>
<td>60</td>
<td>3.7</td>
<td>4.9</td>
<td>5.2</td>
</tr>
<tr>
<td>50</td>
<td>80</td>
<td>3.7</td>
<td>4.6</td>
<td>5.2</td>
</tr>
<tr>
<td>60</td>
<td>100</td>
<td>3.7</td>
<td>4.6</td>
<td>4.9</td>
</tr>
<tr>
<td>70</td>
<td>150</td>
<td>3.7</td>
<td>4.3</td>
<td>4.6</td>
</tr>
<tr>
<td>Tangent</td>
<td></td>
<td>3.7</td>
<td>4.0</td>
<td>4.3</td>
</tr>
</tbody>
</table>
The passive taper, which allows a lateral transition of traffic.

The active taper is used to narrow a roadway or lane or as a lane drop, i.e., when two lanes merge into one. The passive taper either widens or adds a lane. Active tapers constitute a hazard insofar as that a driver that fails to perceive the change in circumstances, may either drive off the travelled way or hit the adjacent kerbing. Passive tapers, on the other hand, create space on the travelled way and, thus, are not hazards. Consequently, active tapers should be long and passive tapers may be short.

If a turning roadway is preceded by an auxiliary lane to allow for deceleration or followed by an auxiliary lane allowing for acceleration, these lanes will be added or dropped by means of passive and active tapers respectively. In the absence of the auxiliary lanes, the turning roadway can, in the case of a left turn, be created by a passive taper from the left edge of the through lane to the left edge of the turning roadway.

The turning roadway may be terminated by an active taper connecting its left edge to the left edge of the through lane. Apropos the suggestion that the angle of skew of the intersection should not fall outside the range of $75^\circ$ to $120^\circ$, it is important to note that a very short active taper may result in the creation of a local angle of skew of less than $75^\circ$. This would make it very difficult for the driver on the turning roadway to observe opposing traffic on the through lane. At an angle of skew of less than $5^\circ$ the driver should, using the rear view mirror, be able to observe opposing traffic comfortably. In short, tapers in the range between $1:10$ and $1:0.3$ should be avoided (the latter corresponding to an angle of skew of $75^\circ$).

Acceptable tapers rates are suggested in Table 6.12.

In entering a deceleration lane, a vehicle follows a reverse or S-curve alignment, which is effectively a passive taper immediately followed by an active taper. Four different combinations of taper can be employed. These are:

- A straight-line taper;
- A partial tangent taper;
- A symmetrical reverse curve; or
- An asymmetrical reverse curve.

In urban areas, short straight-line tapers appear to offer better targets for the approaching driver. Urban intersections operate at slow speeds during peak periods and, particularly for right-turn-
Ining traffic, the need for storage may be more important than the ability to enter the deceleration lane at relatively high speeds. Tapers could therefore be as sharp as 1:2, which is about the limit of manoeuvrability of a passenger car at crawl speeds.

As speeds are higher in rural than in urban areas, the other forms of taper listed above may warrant consideration. The partial tangent taper is a straight line taper preceded by a short curved section with a radius such that the desired taper rate is achieved at a point about one third of the way across the width of the auxiliary lane. The symmetrical reverse curve taper has curved sections of equal radius at either end. The guideline suggested for the partial tangent, i.e. the curve traversing one third of the lane width, applies to the entry and the exit curves of the taper. The asymmetrical reverse curve usually has an entry curve radius about twice that of the exit curve.

6.6 ROUNDABOUTS

6.6.1 Introduction

Traffic circles constructed in the 1930s and 1940s were intended to operate in a weaving mode. As such, the diameters of the circles were large. No clear guidance was offered regarding priority of one vehicle over another and, in consequence, accident rates at traffic circles tended to be higher than those at conventional intersections. In an effort to rectify this situation, it was decided (in the situation of driving on the left) that vehicles should yield to those on their left. In effect, this meant that vehicles already in the circle had to give way to those wishing to enter it. Not surprisingly, gridlock resulted at heavy flow rates. Ultimately, traffic circles fell into disfavour and were replaced by conventional three- and four-legged intersections.

Modern roundabouts differ from traffic circles in their uniform characteristics and operation. Internationally, roundabouts operate on the "Yield on entry" rule so that, where vehicles drive on the left, vehicles yield to the right and vice versa. South Africa applies the same rule except that, in the case of the mini-roundabout, the rule is slightly modified by the use of the R2.2 sign which "...indicates to the driver of a vehicle approaching a traffic circle that he shall yield right of way to any vehicle which will cross any yield line at such junction before him and which, in the normal course of events, will cross the path of such driver's vehicle."

Drivers are thus inclined to adopt the approach that the rule of first-come-first-served applies at mini-roundabouts except that, in the case of simultaneous arrivals at the mini-roundabout, drivers will yield to the vehicle on the right.

A number of geometric elements are incorporated in the design of roundabouts and all elements appear in all roundabouts. These elements are illustrated in Figure 6.11.

6.6.2 Operation of roundabouts

Roundabouts operate by deflecting the vehicle path so as to slow traffic and promote yielding. The roadway entry is usually flared to increase capacity.
Delays at roundabouts are usually less than at conventional intersections and, in consequence, capacity is higher. The proviso is that the combined intersection flow should be less than 3 500 veh/h. Reduced delays improve vehicle operating costs.

Roundabouts have less potential conflict points than conventional intersections. In the case of the four-legged intersection, 32 conflict points are replaced by 8, as illustrated in Figure 6.12. In both roundabouts and conventional intersections, the diverge is also counted as being a conflict point. The safety performance of roundabouts is often superior to that of most conventional intersections and the reduced number of conflict points at roundabouts result in an observable reduction in accident rates.

In spite of their undoubted advantages, roundabouts are not appropriate to every situation. They may be inappropriate:
- Where spatial restraints (including cost of land), unfavourable topography or high construction costs make it impossible to provide an acceptable geometric design;
- Where traffic flows are unbalanced, with high flows on one or more approaches;
- At intersections of major roads with minor roads, where roundabouts would cause serious delays to traffic on the major roads;
- Where there are substantial pedestrian flows;
- As an isolated intersection in a network of linked signalised intersections;

**Figure 6.11: Elements of roundabouts**

In both roundabouts and conventional intersections, the diverge is also counted as being a conflict point.
• In the presence of reversible lanes;
• Where semi-trailers and/or abnormal vehicles are a significant proportion of the total traffic passing through the intersection and where there is insufficient space to provide the required layout; and
• Where signalised traffic control down stream could cause a queue to back-up through the roundabout.

Roundabouts can be considered when:
• Intersection volumes do not exceed 3 000 veh/h at three-legged or 4 000 veh/h at four-legged intersections;
• The proportional split between the volumes on the major and minor road does not exceed 70/30;
  o where, on three-legged intersections, the intersection flow is less than 1 500 veh/h or,
  o on four-legged intersections, is less than 2 000 veh/h;
• One major flow has a predominant through movement that is:
  o Between 50 and 80 per cent of the approach volume; or
  o Between 25 and 40 per cent of the intersection volume; and
  o High volumes of right-turning movements, i.e. more than 25

![Figure 6.12: Intersection conflict points](image)

6.6.3 Design speed

The design speed within the roundabout should ideally range between 40 to 50 km/h. Unfortunately, this suggests a radius of between 60 and 80 metres hence requiring an overall
diameter of the roundabout of the order of 150 metres. Very often, the space for this size of intersection will simply not be available and some lesser design speed will have to be accepted.

Where the design speeds on the approaches are high, e.g. more than 15 km/h faster than the design speed within the facility, it may be necessary to consider forcing a reduction in vehicle speed. This could be by means of horizontal reverse curvature. The ratio between the radii of successive curves should be of the order of 1.5:1.

Speed humps should not be employed as speed-reducing devices on major roads or on bus routes. Where design speeds are of the order of 100 km/h or more, the speed hump would have to be long and the height low to ensure that the vertical acceleration caused by the speed hump does not cause the driver to lose control. It is suggested that, in practice, a suitable profile would be difficult to construct. Bus passengers, particularly those sitting in the rear overhang of the vehicle, would find traversing a speed hump distinctly uncomfortable. Speed humps should thus only be used at roundabouts in residential areas or where the intention is to apply traffic calming.

### 6.6.4 Sight distance

Site visibility is important in the design of roundabouts. Specifically, approaching drivers should have a clear view of the nose of the splitter or separator island. At the yield line and while traversing the roundabout, they should have an uninterrupted view of the opposing legs of the intersection at all times. This requirement suggests that the elaborate landscaping schemes sometimes placed on the central islands of roundabouts are totally inappropriate to the intended function of the layout.

Decision sight distance for intersections as described in Table 3.5 should be provided on each approach to the roundabout to ensure that drivers can see the nose of the splitter island. It follows that roundabouts should not be located on crest curves.

### 6.6.5 Components

The various components of a roundabout are illustrated in Figure 6.11.

**Deflection**

A very important component is the deflection forced on vehicles on the approach to the roundabout. The intention is to reduce the speed of vehicles so that, within limits, the greater the deflection the better. The limit is that the minimum acceptable angle of skew at an intersection is 60°, as discussed previously in this chapter. This corresponds to a deflection on entry of 30°. The approach radius should not exceed 100 metres, which corresponds to the recommended design speed of 40 to 50 km/h.

**Entries and exits**

The widths of single lane approaches to roundabouts are typically of the order of 3.4 to 3.7 metres. The entry width is one of the most important factors in increasing the capacity of the roundabout and can be increased above the
width of the approach by flaring, i.e. by providing a passive taper with a taper rate of 1:12 to 1:15. The recommended minimum width for a single-lane entry is 5 metres.

If demanded by high approach volumes, the flaring could add a full lane to the entry to increase capacity. The width of a two-lane entry should be of the order of 8 metres. A variation on the two-lane entry is to have, in effect, a single-lane circulatory roadway with an auxiliary lane provided for the benefit of vehicles turning left at the roundabout. The auxiliary lane is shielded from traffic approaching from the right by moving the end of the splitter island forward to provide a circulatory road width adequate only for single-lane travel. This approach could be adopted with advantage when left-turning traffic represents 50 per cent or more of the entry flow or more than 300 veh/h during peak hours.

**Circulatory roadway**

The circulatory roadway width is a function of the swept path of the design vehicle and of the layout of the exits and entries and generally should be either equal to or 1.2 times the width of the entries. The width should be constant throughout the circle.

In the construction of the swept path of the design vehicle, it should be noted that drivers tend to position their vehicles close to the outside kerbs on entry to and exit from the roundabout and close to the central island between these two points. The vehicle path, being the path of a point at the centre of the vehicle, should thus have an adequate offset to the outside and inside kerbs. For a vehicle with an overall width of 2.6 metres, the offset should thus be not less than 1.6 metres with 2.0 metres being preferred. To ensure that vehicles do not travel faster than the design speed, the maximum radius on the vehicle path should be kept to 100 metres or less.

As a general guideline, the circulatory roadway should be sufficiently wide to allow a stalled vehicle to be passed but without sufficient trucks in the traffic stream to influence design (usually described as Case 2, Condition A in the case of turning roadways). The minimum roadway width for single-lane operation under these circumstances would be of the order of 6.5 metres between kerbs. Two-lane operation would require a roadway width of about 8 metres. If trucks are present in the traffic stream in sufficient numbers to influence design, the circulatory road width should be increased by 3 metres both in the single-lane and in the two-lane situation. A significant proportion of semi-trailers would require the width of the circulatory road width to be increased to 13 metres and 16 metres in the single-lane and the two-lane situation respectively.

A circulatory road width of 13 metres would make it possible for passenger cars to traverse the roundabout on relatively large radius curves and at correspondingly high speeds. To avoid this possibility, the central island should be modified as discussed below.

The cross-slope on the roadway should be away from the central island and equal to the camber on the approaches to the intersection.
Central island

The central island consists of a raised non-traversable area, except in the case of mini-roundabouts where the central island may simply be a painted dot. The island is often landscaped but it should be ensured that the landscaping does not obscure the sight lines across the island. Historically, central islands were often square or, if they had more than four entries, polygonal. Negotiating the right angle bends was only possible at crawl speeds and this led to substantial delays and congestion. It is now customary to provide circular islands.

While, for semi-trailers, the width of the circulatory road between kerbs would have to be 13 metres in a single-lane configuration, all other vehicles could be served by a road width of 9,5 metres. A mountable area or apron could thus be added to the central island to accommodate this difference. The apron should have crossfall steeper than that of the circulatory road, principally to discourage passenger vehicles from driving on it and a crossfall of 4 to 5 per cent is recommended.

Splitter islands

Splitter islands should be provided on the approaches to roundabouts to:

- Allow drivers to perceive the upcoming roundabout and to reduce entry speed;
- Provide space for a comfortable deceleration distance;
- Physically separate entering and exiting traffic;
- Prevent deliberate and highly dangerous wrong-way driving;
- Control entry and exit deflections; and
- Provide a refuge for pedestrians and cyclists and a place to mount traffic signs.

The sizes of splitter islands are dictated by the dimensions of the central island and inscribed circle. As a general guideline, they should have an area of at least 10 square metres so as to ensure their visibility to the oncoming driver. The length of splitter islands should be equal to the comfortable deceleration distance from the design speed of the approach to that of the roundabout.

Ideally, the nose of the splitter island should be offset to the right of the approach road centre-line by about 0,6 to 1 metre.