A Guide To The Design
of At-grade Intersections

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Jabatan Kerja Raya
Cawangan Jalan

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A GUIDE TO THE DESIGN OF AT- GRADE INTERSECTIONS

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FOREWORD

There has been tremendous progress in the road design methodology and process within JKR which are underlined in the numerous revised Technical Design Guides produced by JKR and REAM that updates the road design requirements in line with the current international standards and practices worldwide.

This Arahan Teknik (Jalan) ATJ 11/87 (Pindaan 2017), A Guide To The Design Of At-Grade Intersections, is the revision of the existing Arahan Teknik (Jalan) 11/87 which also takes input from, Malaysia Highway Capacity Manual, Austroads 2010, Guide to road design: part 4A Unsignalised and Signalised Intersections, Manual on Uniform Traffic Control Devices (MUTCD), AASHTO. A Policy of Geometric Design of Highways and Streets, (2001 & 2011), Transportation Research Board- Highway Capacity Manual 2000 and 2010, Design Manual for Roads and Bridges Volume 6 2006 and Queensland Department of Main Roads 2006, ‘Roundabouts’ in preparing the guideline.

The preparation of this guideline was carried out through many discussions held by the committee members. Feedbacks and comments received were carefully considered and incorporated into this guideline wherever appropriate.

This guideline will be reviewed and updated from time to time to cater for and incorporate the latest development in road geometric design. Any comments and feedback regarding this guideline should be forwarded to Unit Standard, Bahagian Pembangunan Inovasi & Standard, Cawangan Jalan, JKR.

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Last but not least, our compliment also goes to Dato'. Ir. Dr. Meor Aziz Osman, Deputy Director General of Works (Infra Sector), Ir. Zulakmal bin Haji Sufian, Senior Director of Road Branch, Dato' Ir. Hj. Che Noor Azeman bin Yusoff, Director of Road and Bridge Design, Ir. Dr. Lim Char Ching, Director of Forensics Engineering and Technical Support Division*, Cawangan Jalan, Jabatan Kerja Raya Malaysia* for their support and contribution towards the successful completion of this specification.

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1.0 INTRODUCTION

1.1 Definition and General Description

An intersection is defined as the general area where two or more roadways join or cross. It is an integral and important part of the highway system since much of the efficiency, safety, speed, cost of operation and maintenance, as well as capacity depend upon its design.

1.1.1 Design Objectives

The incidence of possible conflicts at intersections are very high, therefore, they are considered to be areas of high accident potential. The designer must strive to minimize these conflict points in his design, while providing adequately for the through, crossing and turning movements.

In the design of intersections careful consideration should be given to the appearance of the intersection as the driver will see it. Upon approaching an intersection, a reverse curve may appear compressed and confusing to the driver. To avoid abrupt changes in alignment, sufficient transitions or compound curves should be provided to allow the driver to comfortably negotiate them, and to ensure a pleasing appearance.

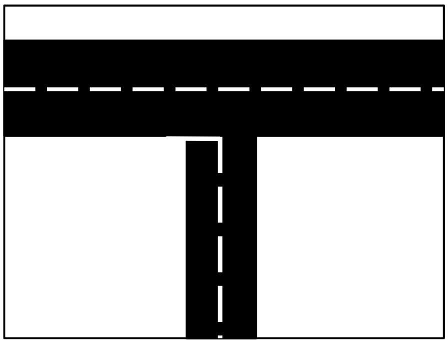
An understanding of driving habits and application of human factors is indispensable in the development of appropriate geometric design and the subsequent operational quality of the intersection. Operational safety and efficiency of an intersection depends highly on the design suitability. The driver's performance is improved when they use a highway facility designed to be within their capabilities and limitations.

When a design is incompatible with the attributes of drivers, the chances for driver error increases. Inefficient operation and accidents are often a result.

1.1.2 Types of At-Grade intersections.

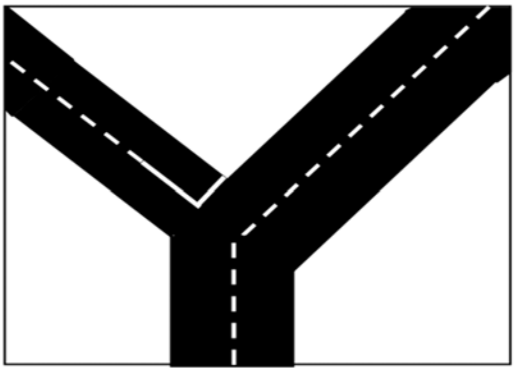
(a) T- intersections

- It is a three legged road intersection.
- At a T- intersection, one of the legs is generally a minor road connecting to a major road that does not cross.



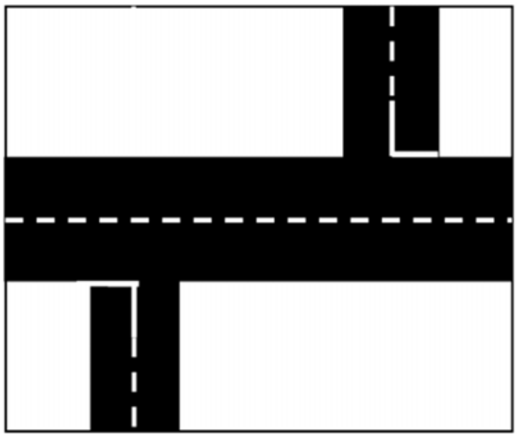
(b) Y- intersections

- Y- intersections generally has three (3) legged road of equal size, it consists of a minor road connecting to a major road which is not at right angle.



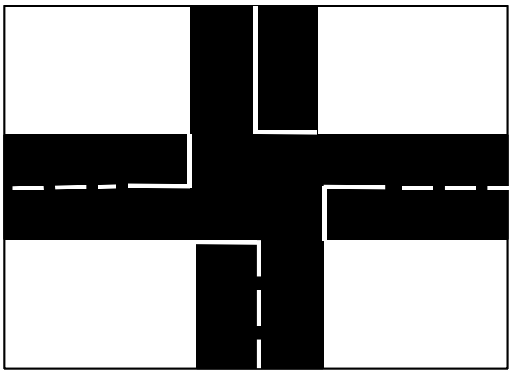
(c) Staggered intersections

- Where several roads meet a main road at a slight distance from each other thus not all coming together at the same point.



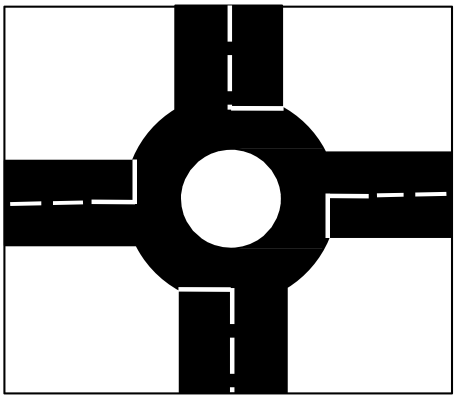
(d) Four- legged intersections

- This intersection is the most common and basic configuration for roads that cross each other.



(e) Roundabout

- A circular intersection in which road traffic must travel in one direction around a central island.



1.2 At-Grade Intersection Layouts

The following are the three basic types of intersection layouts at grade:

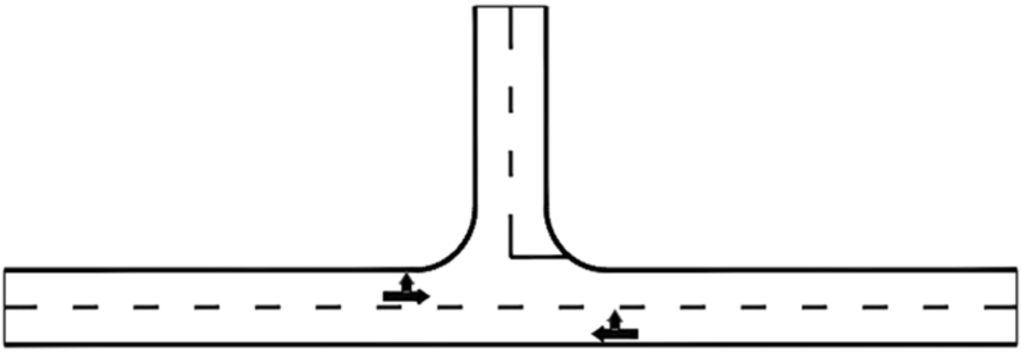
(a) Unchannelised and Unflared
(b) Unchannelised and Flared
(c) Channelised (Including Roundabouts).

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

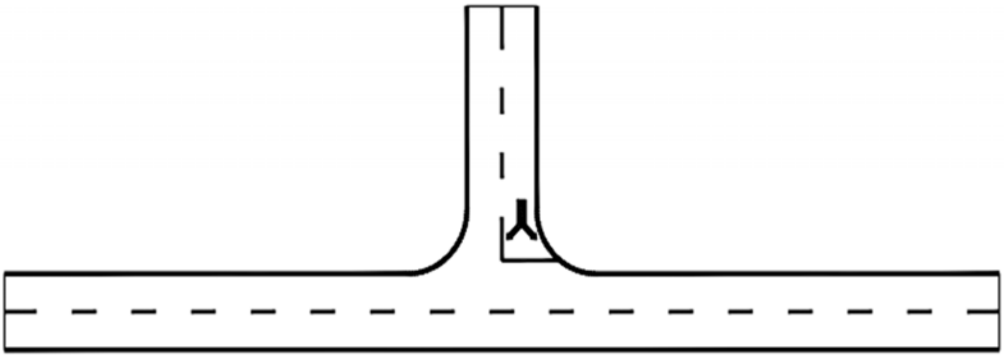
1.2.1 Unchannelised and Unflared Intersections

They are normally adequate where minor roads meet. In urban areas, many local street intersections remain unchannelised for economic reasons. In such cases, traffic can be controlled by signals or regulatory signs, such as STOP or GIVE WAY signs, on the minor roads.

However, regulatory signs are not a substitute for channelization.
(see **FIGURE 1.1**)



Basic left turn on major road



Basic left turn on minor road

FIGURE 1.1: UNCHANNELISED AND UNFLARED INTERSECTION LAYOUT
Source: Austroads 2010, Guide to Road Design: Part 4A Unsignalised and Signalised Intersections, Figure 4.2.

Note: All intersections should be designed based on Chapter 3.

1.2.2 Unchannelised and Flared Intersections

Simple unchannelised intersections may be flared to provide additional through lanes or auxiliary lanes (**FIGURE 1.2**), 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.

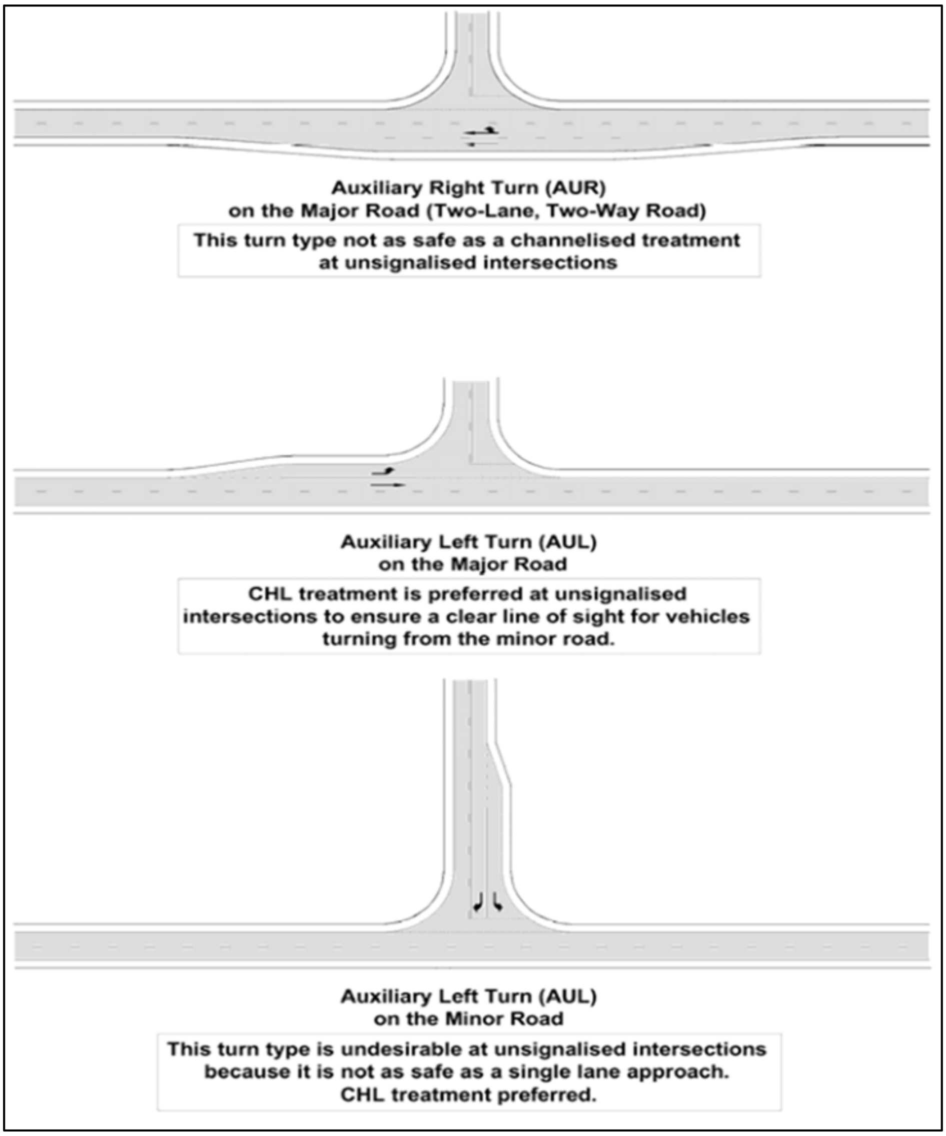


FIGURE 1.2: AUXILIARY LANE LAYOUT

Source: Austroads 2010, Guide to road design: Part 4A Unsignalised and Signalised Intersections, Figure 4.5.

Note: All intersections should be designed based on Chapter 3.

1.2.3 Channelised Intersections

A channelised intersection is one where the 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 (see **FIGURE 1.3**).

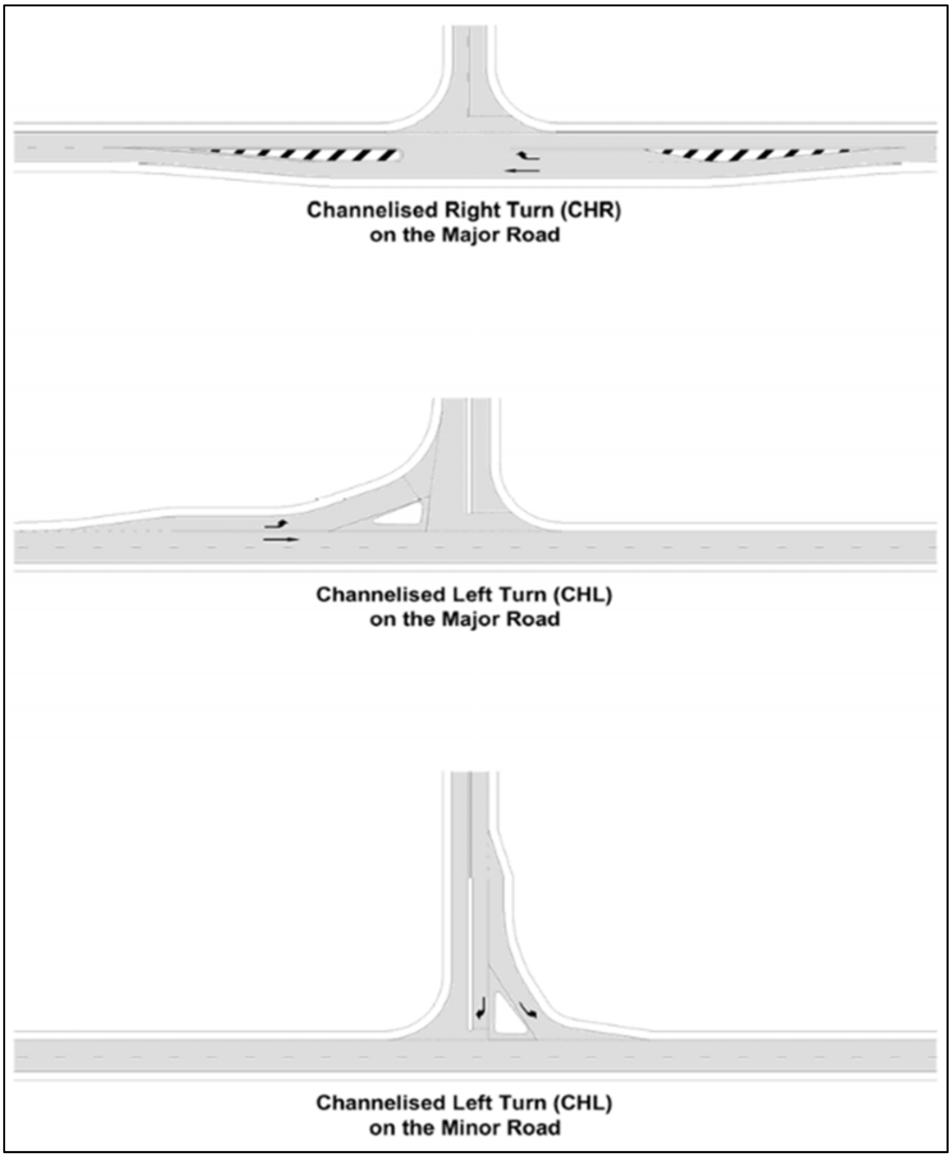


FIGURE 1.3: CHANNELISED INTERSECTION

Source: Austroads 2010, Guide to Road Design: Part 4A Unsignalised and Signalised Intersections, Figure 4.7

Note: All intersections should be designed based on Chapter 3

Speed change lanes allow left or right-turning vehicles to reduce or increase speed when leaving or entering the through road without adversely affecting the speed of the through traffic.

Right turn lanes permit through vehicles to pass on the left side of another vehicle waiting to complete a right turn at an intersection.

1.2.3.1 Channelisation for Intersection Treatments

It is not practicable or desirable to standardise the design of channelised layouts. Channelisation is the separation or regulation of conflicting traffic movements into definite paths of travel by traffic islands or pavement marking to facilitate the safe and orderly movements of both vehicles and pedestrians. It is not practicable or desirable to standardise the design of channelised layouts.

The layout for a particular site depends on the following:

- Traffic pattern,
- Traffic volume,
- The area which is economically available for improvement;
- Topography,
- Pedestrian movement,
- Parking arrangement,
- The planned ultimate development of the neighbourhood, and
- The layout of the existing roads.

As well as separating conflicting movements, channelisation is also used to:

(a) Reduce the area of conflict: The impact area is decreased when channelisation is provided and hence the probability of conflicts is also reduced. **FIGURE 1.4** shows the reduced conflict area in the same intersection after providing medians.

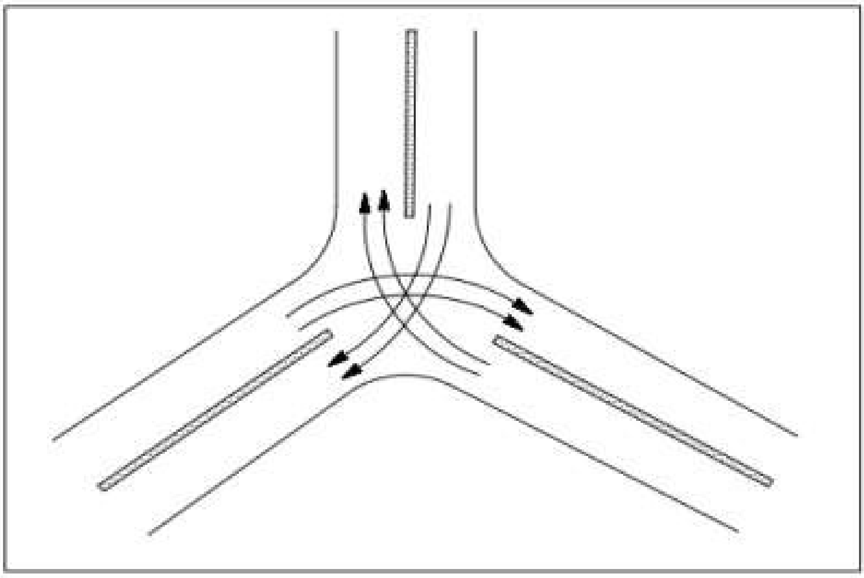


FIGURE 1.4: CONFLICT AREA IN CHANNELISED INTERSECTION

(b) Merge traffic stream at small angles, i.e. merging at small angles permits the flow of traffic stream with minimum speed differentials hence, the gap acceptance time is also small in such cases. The merging of roadways should be done as shown in **FIGURE 1.5**.

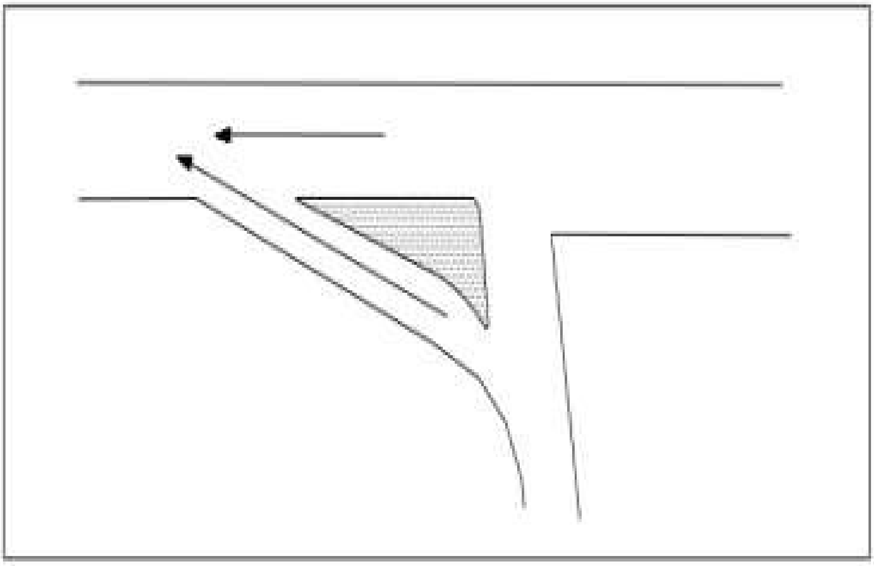


FIGURE 1.5: MERGING OF TRAFFIC STREAMS

Reduction of the speed of incoming traffic:

(a) By bending its path, i.e., the speed of vehicles entering into the intersection can be reduced by bending the path to the intersection approach. However as far as possible the path of the major traffic stream should not be bent. The above technique is shown in **FIGURE 1.6.**

(b) By funneling, i.e., the funneling technique can also be used for reducing the speeds of the incoming vehicles. Due to the decrease in the width of the lane at the approach, the drivers tend to reduce the speed of their vehicles near the intersection. **FIGURE 1.7** shows the funneling technique used for reduction of speed.

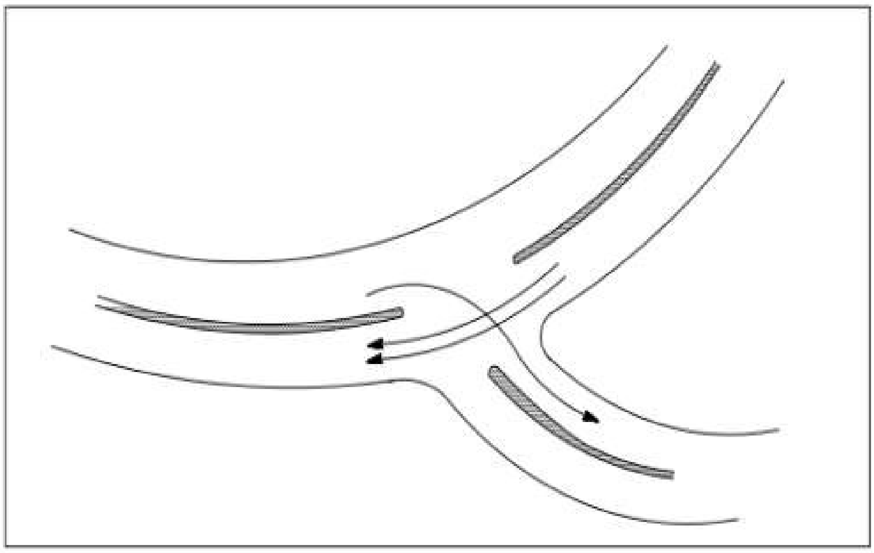


FIGURE 1.6: BENDING PATH OF INCOMING MINOR STREET

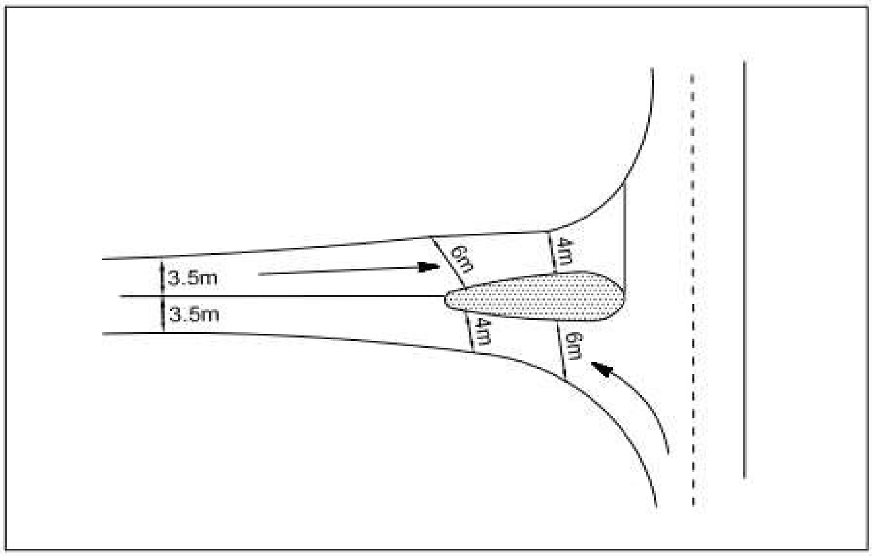


FIGURE 1.7: REDUCTION OF SPEED BY FUNNELING

(c) Protection for turning vehicles crossing/ conflicting traffic streams, i.e., the provision of a refuge between the two opposing stream allows the driver of a crossing vehicle to select a safe gap in one stream at a time and also provides a safer crossing maneuver. **FIGURE 1.8** further clarifies the above statement.

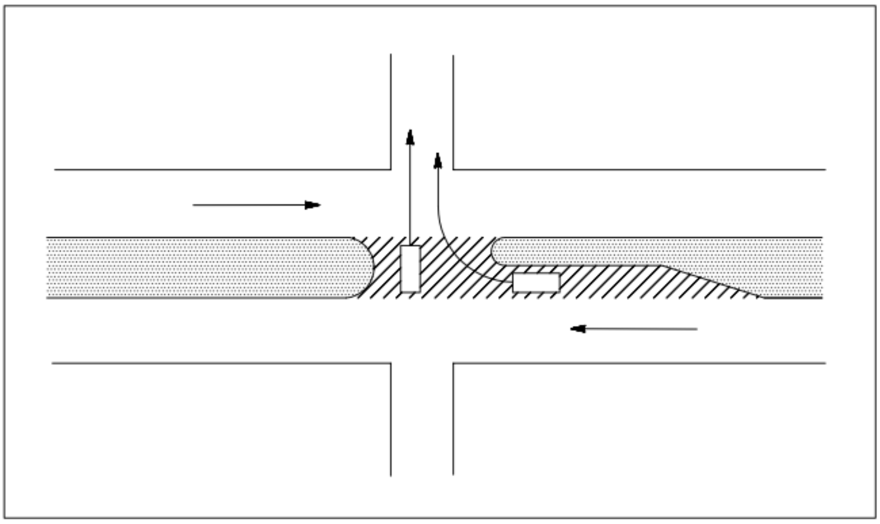


FIGURE 1.8: REFUGE AREA FOR PROTECTING CROSSING OR TURNING TRAFFIC

(d) Discourage prohibited turns by island placement and shape, i.e., undesirable and prohibited turns can be discouraged by the proper selection of shape and location of the islands. **FIGURE 1.9** shows how prohibited turns can be discouraged by proper shaping and placement of islands

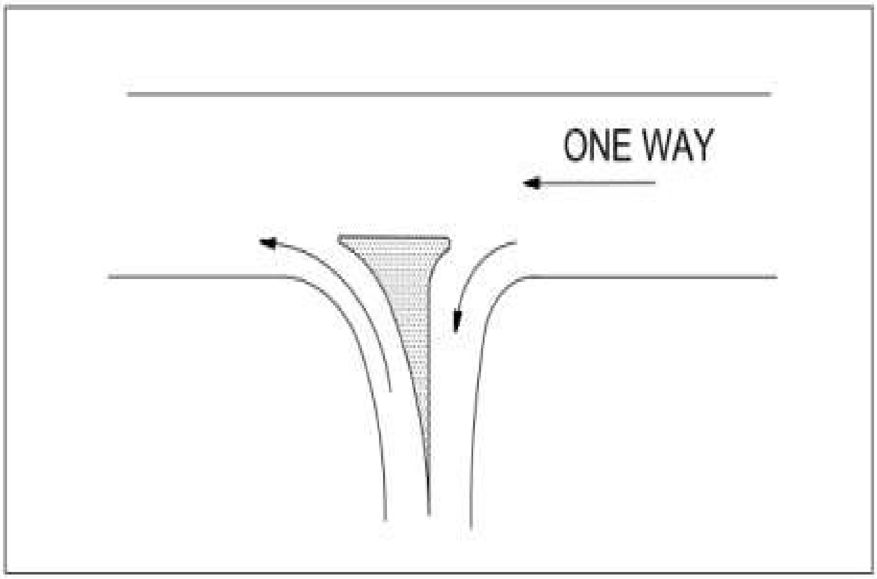


FIGURE 1.9: PROPERLY PLACED ISLANDS TO DISCOURAGE PROHIBITED MOVEMENTS

(e) Providing locations of traffic control devices: channelisation may provide locations for the installation of essential traffic control devices such as stop and directional sign, signals etc. **FIGURE 1.10** shows how channelizing devices can also be used for locating traffic control devices

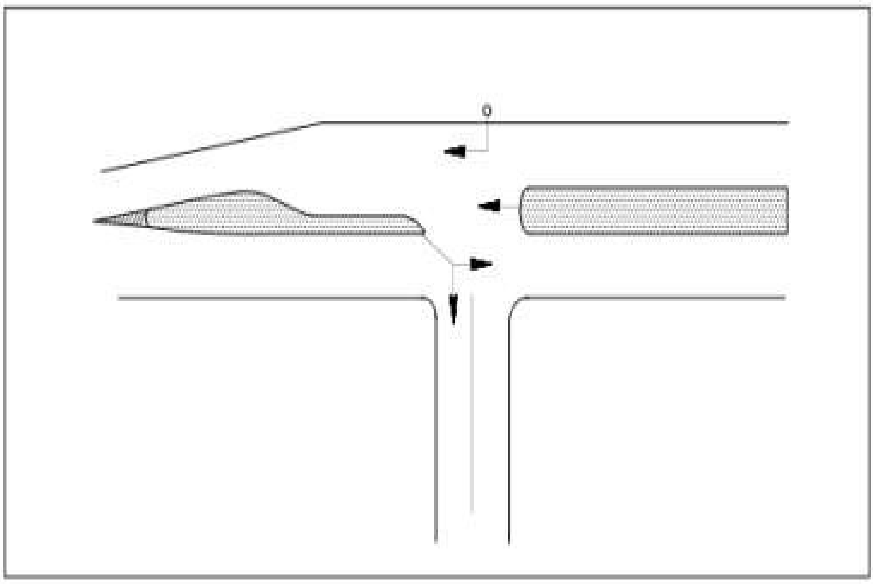


FIGURE 1.10: LOCATION OF SIGNAL POSTS ON MEDIANS AT INTERSECTION

(f) Channelisation may provide location for protection of pedestrian by means its should be non-traversable and wide median provide a refuge for pedestrian crossing the roads.

1.2.3.2 Excessive Channelisation

Care should be taken to install only the minimum number of island as excessive channelisation can:

(a) Result in unwarranted obstructions on the road pavement,
(b) Unnecessarily restricting parking and private access adjacent to the intersection,
(c) Cause problems of pavement maintenance and drainage, and
(d) Create confusion

1.3 Factors Influencing Design of Intersection

In analysing an intersection design, the following basic elements must be considered:

1.3.1 Traffic Factors

(a) Present and projected turning movements and turning volumes, including truck volumes,
(b) Capacity and service volumes, DHV'S, AM & PM peak hours,
(c) Physical and operating characteristics of vehicles,
(d) Vehicle operating speeds approaching the intersection, posted speeds and design speeds,
(e) Accident statistics,
(f) Warrants for traffic signals,
(g) Pedestrian movements,
(h) Parking controls,
(i) Public transit operations, and
(j) Regulatory, directional and destination signing.

1.3.2 Physical Factors

(a) Functional classification of the roadways involved,
(b) Basic lane requirements; present and future,
(c) Land use development adjacent to the intersection area,
(d) Site topography,
(e) Grades and sight distances,
(f) Angle of intersection,
(g) Environmental considerations, and
(h) Aesthetics.

1.3.3 Economic Factors

(a) Construction costs,
(b) Maintenance costs,
(c) Compensating costs to business adversely affected by the design, and
(d) Cost/benefit comparison of the above.

1.3.4 Human Factors

(a) Driving habits,
(b) Natural paths of movement,
(c) Physical comfort of the driver,
(d) Driver's expectations,
(e) Ability of drivers to make decisions and react, and
(f) Effect of surprise (sudden appearance of channelised islands and other obstructions).

Proper evaluation of these factors will enable the designer to evolve an intersection design which will assure an orderly movement of traffic, increases the capacity of the intersection, improves safety by minimizing the conflict points and provides maximum convenience to the travelling public.

1.4 Location of Intersection

The efficiency of major roads, in terms of capacity, speed and safety depends greatly upon the number, type and spacing of intersections and median openings. Intersections should not be located at sharp horizontal curves, steep grades or at the top of crest vertical curves, or at the bottom of sag vertical curves. The intersections shall be positioned:

- on straight arterial road alignment
- in sags or uniform grades < 2%
- away from bridges/structures to maximize sight distance, and
- minimize cost

The reference should be made to Table 1.1: desirable minimum spacings of intersections for the various categories of major roads

Location of intersections must be identified during the route location process so that suitable locations are identified.

1.5 Spacing of Intersections

The spacing of intersections depends on factors such as weaving distance and storage length required for queueing traffic at signalised intersections and the lengths of right turning lanes. Future co-ordination of traffic signals should also be carefully considered in determining intersection spacing.

TABLE 1.1 gives the desirable minimum spacing of intersections for the various categories of the major roads.

***TABLE 1.1: DESIRABLE MINIMUM SPACINGS OF INTERSECTIONS FOR THE VARIOUS CATEGORIES OF MAJOR ROADS**

Area	Category Of Major Road	Spacing, (m)
Rural	Expressway	3.000
	Highway	V x 20
	Primary	V x 10
	Secondary	V x 5
	Minor	V x 3
Urban	Expressway	1.500
	Arterial	V x 3 x n
	Collector	V x 3 x n
	Local Street	V x 3 x n

Note:
V = Design Speed in Km/h
n = Number of through lane in one direction
[* = Original figure and table from ATJ 11/87]
[= Malaysian Highway Capacity Manual 2006]**

1.6 Capacity

The design must provide adequate traffic handling capacity throughout the expected life of the intersection. This may involve the design of separate construction stages before the ultimate development of the intersection is reached.

1.7 Types of Conflicting Manoeuvres

There are four basic types of intersection manoeuvres such as diverging, merging, crossing and weaving.

The number of potential conflict points at an intersection depends on the: -
(a) Number of approaches to the intersection
(b) Number of lanes on each approach
(c) Type of signal control
(d) Extent of channelisation and
(e) Movements permitted
(f) The complexity of an intersection increases as the number of approach roads to the intersection increases. **FIGURE 1.11(A)** and **FIGURE 1.11(B)**, below, shows the number and type of conflicts that occur at intersections with three and four roads, respectively.

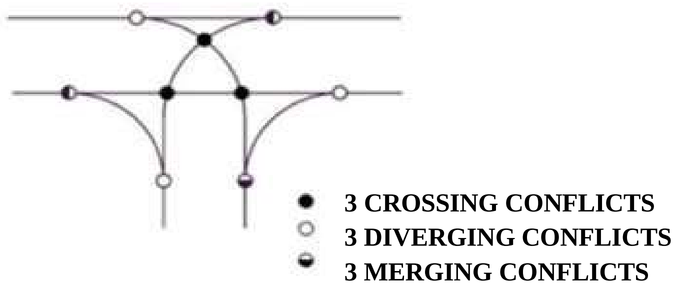


FIGURE 1.11(A): THREE-LEGGED INTERSECTION

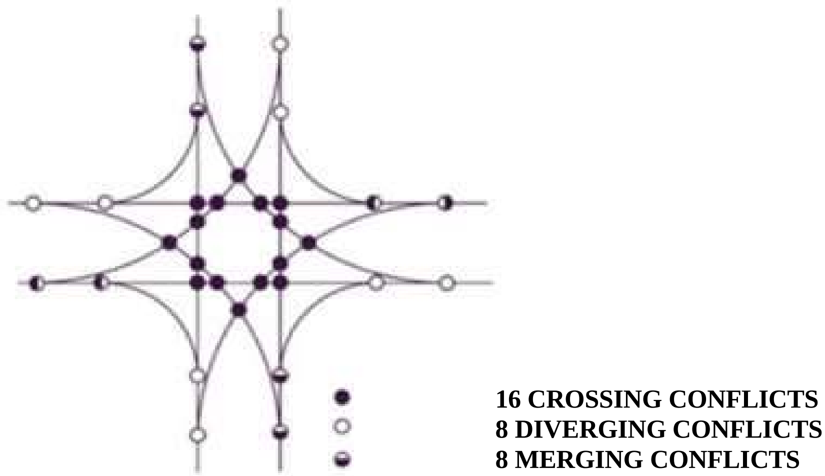


FIGURE 1.11(B): FOUR-LEGGED INTERSECTION

1.8 Points of Conflict

A conflict point is the point at which a road user crossing, merging with, or diverging from a road or driveway conflicts with another road user using the same road or driveway. It is any point where the paths of two through or turning vehicles diverge, merge, or cross.

The number of conflict points can be reduced by prohibiting certain traffic movements and by eliminating some roads from the intersection. Conflict points can be reduced by channelisation as shown in **FIGURE 1.12**.

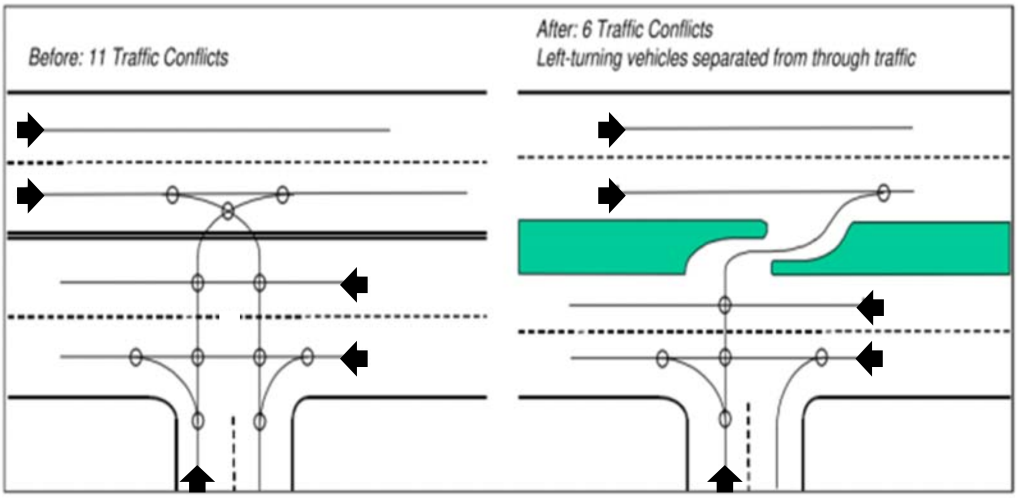


FIGURE 1.12: REDUCTION OF TRAFFIC CONFLICT BY CHANNELISATION

1.9 Safety

Safety is a prime consideration in any intersection design. Safe intersection design is based on the following principles: -

(a) Reduction of the number of points of conflict.
(b) Minimising the area of conflict.
(c) Separation of points of conflict.
(d) Giving preference to major movements.
(e) Control of speed.
(f) Provision of refuge areas, traffic control devices and adequate capacity.
(g) Definition of paths to be followed.

1.10 Area of Conflict

Where roads cross at an acute angle or the opposing legs of an intersection are off set, the excessive intersection area may result in a higher accident rate.

In general, large areas of uncontrolled pavement invite dangerous vehicle manoeuvres and should be eliminated. Channelisation and realignment can both reduce conflict area.

1.11 Major Movements

Preference should be given to the major traffic movements to allow them a direct free flowing alignment. Drivers who have travelled for long, uninterrupted distances at high speed will be slow to react to a sudden change in alignment or to the entry of a high speed vehicle from a minor road.

Minor movements should be subordinated to major or high speed movements. Adequate warning on minor approaches should be provided.

1.12 Control of Speed

The operating speed of traffic through an intersection depends on the:

(i) Alignment
(ii) Environment
(iii) Traffic volume and composition
(iv) Extent and type of traffic control devices, and to a lesser extent:
(v) The number of points of conflict
(vi) The number of possible manoeuvres
(vii) The relative speed of the manoeuvres

1.13 Traffic Control and Geometric Design

In intersection design, the possible use of control devices and other road furniture should be considered. Most of the criteria for geometric design are common to both signalised and unsignalised intersections. The design of an intersection to be controlled by signals can differ significantly from one requiring only channelisation and signs. For example, double right turn lanes which aim at shortening storage length are effective only at signalised intersections as at unsignalised intersections, the number of vehicles which can depart from the queue is dependent on the frequency of acceptable gaps in the major stream disregarding the number of storage lanes. Left turn lanes at a signalised intersection requires additional consideration, as queueing vehicles on the most left lane waiting for the green signal would block the entrance to the left turning channel. This is much less significant in unsignalised intersections.

2.0 DESIGN CONTROLS

2.1 Priority Control

All intersections shall be designed under the assumption that one of the intersecting roads has priority except where the intersection is signalised.

The priority road will normally be that which is of the higher design standard. If the two roads are of the same standard, then the priority road shall normally be that for which the highest traffic volume is predicted. If both roads predict high traffic volume, the intersections shall be upgraded to Grade Separated Interchange.

In T-intersections and staggered intersections (which may be considered as two T-intersections at some distance apart) the priority road shall be the through road. In a roundabout, priority shall be given to the circular vehicles movement while vehicles entering the roundabout have to give way to vehicles circulating it.

If the main traffic flow in a T- intersections is on the stem of the T, then a change of layout should be considered. For staggered intersections, a traffic/ intersections study will have to be carried out to ascertain if any modification to the intersection is necessary, and separation distance between them to comply to that stipulated in **TABLE 3.1**.

The two roads of the intersection are normally referred to as the major road (priority road) and the minor road.

2.2 Traffic

The capacities of minor intersections are in general sufficient to meet the expected traffic volumes and detailed traffic forecasts and capacity calculations are therefore normally not required. Intersections where the major road carries a large volume of through traffic or where the two roads carry nearly the same volume of traffic may on the other hand have insufficient capacity for crossing or turning traffic flows, for which particular types of capacity increasing measure may have to be taken. Detailed traffic forecasts for such intersections shall be carried out in order to provide the necessary data for capacity calculations.

A detailed traffic forecast shall provide hourly traffic flows in all directions in the design year. The design year shall be 10 years after construction for an isolated intersection or similar to the design year of the through roadway if the intersection is part of an overall road improvement project. A staged construction for a 5-year traffic requirement is acceptable for isolated intersections in urban areas.

The traffic forecast for any intersection design shall be at least the Level of Service ‘C’ maintained throughout the forecasted years. If such level of service cannot be sustained throughout the design life of the projected forecast, then the designer has to propose mitigation measures such as the provision of grade separated interchange.

Intersection levels of service are defined to represent reasonable ranges in control delay and intersection conditions as shown in **TABLE 2.1**.

For urban areas, the peak hour factor (PHF) should also be determined. In the absence of any data, a value of 0.85 for the PHF can be used.

TABLE 2.1: LEVEL OF SERVICE DEFINITIONS FOR SIGNALISED INTERSECTIONS

Level Of Service (LOS)	Intersection Conditions
A	Very short delay and most vehicles do not stop as result of favorable progressions, arrival of most vehicles during green phase, and short cycle length
B	Short delay and many vehicles do not stop or stop for short time as a result of short cycle lengths and good progression
C	Moderate delay, many vehicles have to stop, and occasional individual cycle failures as a result of some combination of long cycle lengths, high volume to capacity ratios, and unfavorable progression
D	Longer delays; many vehicles have to stop; and a noticeable number of individual cycle failures as a result of some combination of long cycle lengths, high volume to capacity ratios, and unfavorable progression
E	Long delays and frequent individual cycle failures result from one or both of the following: long cycle lengths or high volume to capacity ratios, which, in turn, result in poor progression
F	Delays considered unacceptable to most drivers occur when the vehicle arrival rate is greater than the capacity of the intersection for extended periods of time

Source: AASHTO 2011, Chapter 9, Table 9-1.

2.3 Design Speed

The design speed on the major road through the intersection should be similar to that on the open section. However, all at-grade intersection are not considered safe at design speeds exceeding 90km/hr. Hence, for design speeds exceeding 90km/hr, preference should be made to upgrade the at-grade intersection to an interchange or alternatively, speed limits at the intersection should be introduced.

Vehicles on the minor road can be assumed to approach the intersection at the design speed of the road and drivers should be able to perceive the intersection from a distance not less than the stopping sight distance as given in **TABLE 3.1**.

2.4 Design Vehicles

The design of the various intersection layouts should be made for the design vehicles P, SU or WB-15 as discussed in Section 3 of the latest Arahan Teknik(Jalan) 8/86 - "A Guide To Design Of Roads". **TABLE 2.2** shows a general scheme to select the design vehicle according to the category of road.

***TABLE 2.2: DESIGN VEHICLES FOR INTERSECTION DESIGN**

Area	Category Of Road	Design Vehicle
Rural	Expressway	WB-15
	Highway	
	Primary	
	Secondary	SU
	Minor	SU / P
Urban	Expressway	WB-15
	Arterial	
	Collector	SU
	Local Street	SU / P

(a) For intersections formed by roads of different design vehicles, the higher design should primarily be chosen. However, if the frequency of turns made is small, the lower design vehicle may be used.

(b) Design vehicle P is normally applicable only to intersections of two local streets or minor roads carrying low volumes.

2.4.1 P Design

This design is used at intersections where in conjunction with parkways, minimum turns are appropriate such as at local street intersections, intersection of two minor roads carrying low volumes, or on major roads where turns are made only occasionally.

2.4.2 SU Design

This design is the recommended minimum for collector road in Urban and for secondary or minor road in Rural. For major Highways with important turning movements which involve a larger percentage of trucks or buses, the designer shall consider designing the roads with larger radii and speed change lanes.

2.4.3 WB 15 Design

This design should be used where truck combinations will make turning movements repeatedly. Where designs for such vehicle are warranted, the simpler symmetrical arrangements of three-centred compound curves, refer to the latest ATJ 8/86 - A Guide On Geometric Design Of Roads, figure 3.3 (WB-15 Design Curve) are preferred if smaller vehicles make up a sizable percentage of the turning volume. Because designs of WB, particularly when used in two or more quadrants of an intersection, produce large paved areas, it may be desirable to provide larger radii and use a corner triangular island

2.5 Selection of Intersection Type

The controlled priority of an at-grade intersection will normally provide adequate capacity for the traffic flags expected in most intersections. Where the predicted traffic flags exceed the capacity, other types of intersection have to be introduced. These are: -

(a) Roundabouts
(b) Signal Controlled Intersections
(c) Grade Separated Intersections or Interchanges

The fundamental factor which decides the type of intersection is traffic volume. **TABLE 2.3(A)** shows the general scheme to select the intersection type according to the traffic volume. Other factors such as class of road, lane configuration should also be taken into account, especially when the traffic volume falls near the boundary of the applicable range of an intersection type.

Factors other than traffic volume, such as heavy pedestrian volume, frequent accident occurence may demand signalisation. Coordinated traffic control along an arterial may also govern the selection of the intersection type in accordance with the type of neighbouring intersections. **TABLE 2.3(B)** shows the general scheme to select the intersection type according to the category of roads crossing.

2.5.1 Roundabout

Roundabouts may be applicable for total traffic volume (sum of all directions) of up to 6000 vehicles/hour and may, if the layout can be freely chosen, be designed to cater for any distribution of turning traffic.

However, roundabout design requires a larger land space in order to cater for the approaches of every intersection. But it can have more than four legged intersections since the approaches are free flow unsignalised. The roundabout can be signalised once it exceeds the capacity per leg (6,000veh/h) – if it is so required. A roundabout design can also accommodate for future upgrading if any of its approaches reaches its saturation capacity level, that is by converting it into an interchange such as flyover or ramp to relief the traffic congestion for the particular approach inside the roundabout.

2.5.2 Signal Controlled Intersections

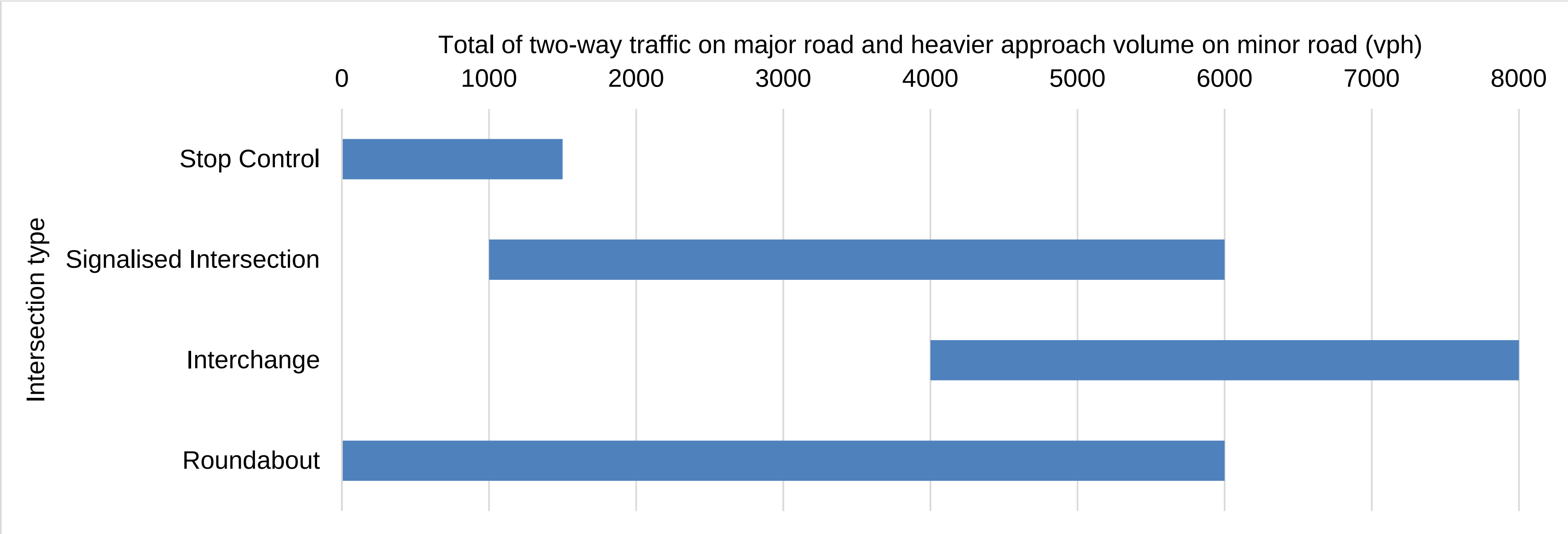
Signal controlled intersections are applicable to very high traffic volume of 1,000 veh/hour or more, provided that the necessary number of approach lanes are present and that there is no interference from, other nearby intersections

Appendix A gives the general warrants that are to be met before traffic control signals are installed.

Traffic signals require reliable electricity supply for their operation, hence limiting their use only to developed areas. The most economical solution may often be the selection of a priority controlled intersection initially, which is prepared for traffic control and to add in the traffic signals at a later stage.

Signalised intersections can handle heavy traffic with adequate number of approach lanes. This, however, requires longer clearance time for vehicles to cross the wide road, leading to less effectiveness in the handling of traffic.

***TABLE 2.3 (A): SELECTION OF INTERSECTION TYPE**



Note:

Roundabouts are usually ranged in types as follows:

1. Mini-roundabout
2. Single-lane roundabout
3. Multi-lane roundabout

*Refer chapter 4 for further explanation

*TABLE 2.3 (B): SELECTION OF INTERSECTION TYPE

According To Category Of Roads Crossing

Rural Area

Expressway	Highway	Primary	Secondary	Local	
I.C	I.C	I.C	-	-	**Expressway**
	I.C	I.C/S.I	S.I/SC	S.C	**Highway**
		S.I	S.I/SC	S.C	**Primary**
			S.C	S.C	**Secondary**
				S.C	**Local**

Urban Area

Expressway	Arterial	Collector	Local Street	
I.C	I.C	-	-	**Expressway**
	I.C/S.I	S.I	S.I/SC	**Arterial**
		S.I	S.C	**Collector**
			S.C	**Local Street**

LEGEND

I.C : Interchange
S.I : Signalised Intersection
S.C : Stop Control

2.5.3 Grade Separated Intersections (Interchanges)

Grade separated intersections serve very high traffic volumes with very little interference to the through traffic. They must be provided for all full access controlled roads and should be considered for road with design speeds exceeding 90 km/hr. Grade separation is also recommended if each of the road crossing has four through lanes or more. The design of interchanges is covered in the latest Arahan Teknik (Jalan) 12/87 – A Guide to the Design of Interchange.

2.6 General Control of Intersection

Minor roads at close proximity creates successive intersections on the major road. They should be treated as follows: -

(a) Local service roads should not be linked directly to the major road, but should be connected to collector roads or combined together into one and then linked to the major road at a proper location.

(b) Local streets should not be linked to the major road near major intersections. If this is unavoidable, only left-turning movements should be allowed. Right-turns from the major road and from the crossroad should be physically prevented with continuous kerbed median and remodeling the entrance to the minor road.

(c) When a new major road is being planned over an existing road network, coordination and adjustment on the layout and spacing of intersections which would be created along the road must be done. Relocation of existing roads and systematic traffic control may be required.

3.0 GEOMETRIC STANDARDS

3.1 General

The following geometric standards relate to the elements of at-grade intersection design which are required to provide for an acceptable level of traffic operations.

At-grade intersections generally handle a variety of potential conflicts among vehicles, motorcycles / bicycles and pedestrians. These recurring conflicts within a relatively small area, unique to each intersection, play a major role in the selection of design standards and guidelines. Arriving, departing, merging, turning and crossing paths of moving traffic have to be accommodated within the intersection.

A four-legged intersection has considerably more traffic conflict points than three-legged intersections and allows higher operating speeds on the minor road. Signalised four-legged intersections especially in rural areas should generally be avoided or eliminated.

Two staggered intersections can take the place of one four-legged intersections However, where large volumes of crossing traffic occur, a four-legged signalised intersection may be better than a pair of staggered T- intersections.

STOP or GIVE WAY signs should be provided on the minor road of unsignalised intersection.

3.2 Horizontal Alignment

The alignment and grade of the intersecting roads should permit users to discern and perform readily the manoeuvre necessary to pass through with minimum interference. Toward these ends, the alignment shall generally be as straight and the gradients as flat as practical. Site conditions generally dictate alignment and grade limitations on intersecting roads. However, it is sometimes possible to modify the alignment and grades thereby improve traffic operations.

A right angle intersection provides the most favorable conditions for intersecting and turning traffic movements. Specifically, it provides the shortest crossing distance and allows drivers to judge the relative position and speed of approach vehicles more easily. Minor deviations from right angles are generally acceptable provided the detrimental impact on visibility and turning movements for large trucks can be mitigated. The intersection angle should be designed as close to 90 degrees as practicably possible, but should not be less than 70 degrees. When a truck turns on an obtuse angle, the driver has blind areas on the left of the vehicle and increases the exposure time in crossing the main traffic flow.

When existing intersection angles are less than 70 degrees, the following retrofit improvement strategies are recommended:

(a) realign subordinate intersection legs;
(b) provide localized lane widening for tight turning radius or limited visibility;
(c) restrict or prohibit problematic turning movement;
(d) Installation of traffic control devices, particularly where the volume of traffic is high.

The practice of realigning roads intersecting at acute angles in the manner shown in **FIGURE 3.1(A** and **B)** has proved to be beneficial. The greatest benefit is obtained when the curves used to realign the roads allow operating speeds nearly equal to the major highway approach speeds.

Another method of realigning an intersection road is to make a staggered intersection as in **FIGURE 3.1(C** and **D).** Crossing vehicles must turn into major road and then re-enter the minor road. **FIGURE 3.1(D)** provides poor access continuity as the crossing vehicle must re-enter the minor road by making a right turn on the major road. The realignment in **FIGURE 3.1 (C)** gives better access continuity as right turning is avoided on the major road and thus interfering little with through traffic.

Where the major road is curving and the minor road constitutes an extension of one tangent, realigning the subordinate road as shown in **FIGURE 3.1(E)** is advantageous to guide traffic to major road and improve visibility at the point of intersection.

The minimum desirable distances between staggered T- intersections are given in **TABLE 3.1.**

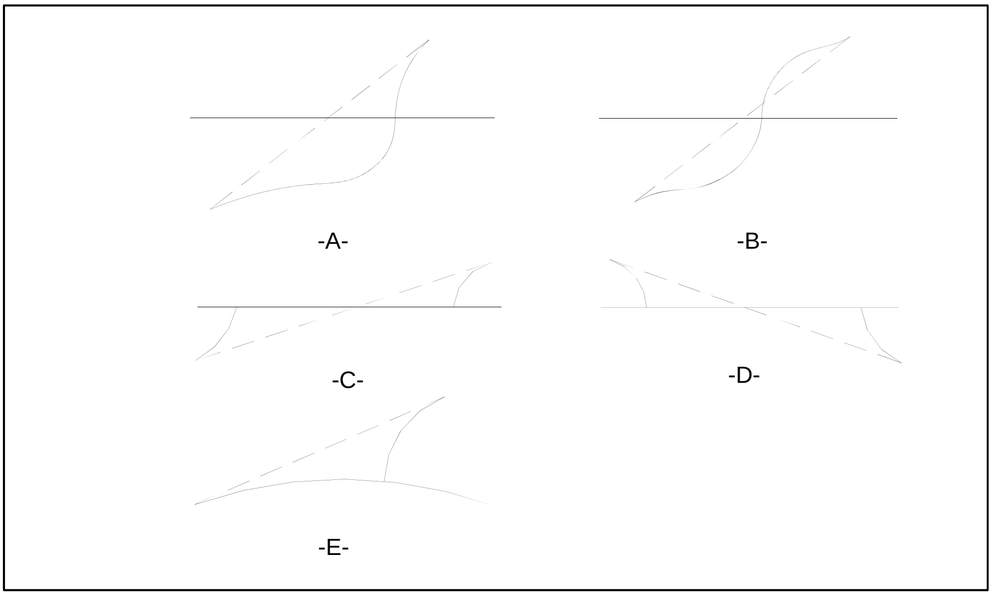


FIGURE 3.1 - REALIGNMENT VARIATIONS AT INTERSECTIONS
Source: AASHTO 2011, Chapter 9, Figure 9-14.

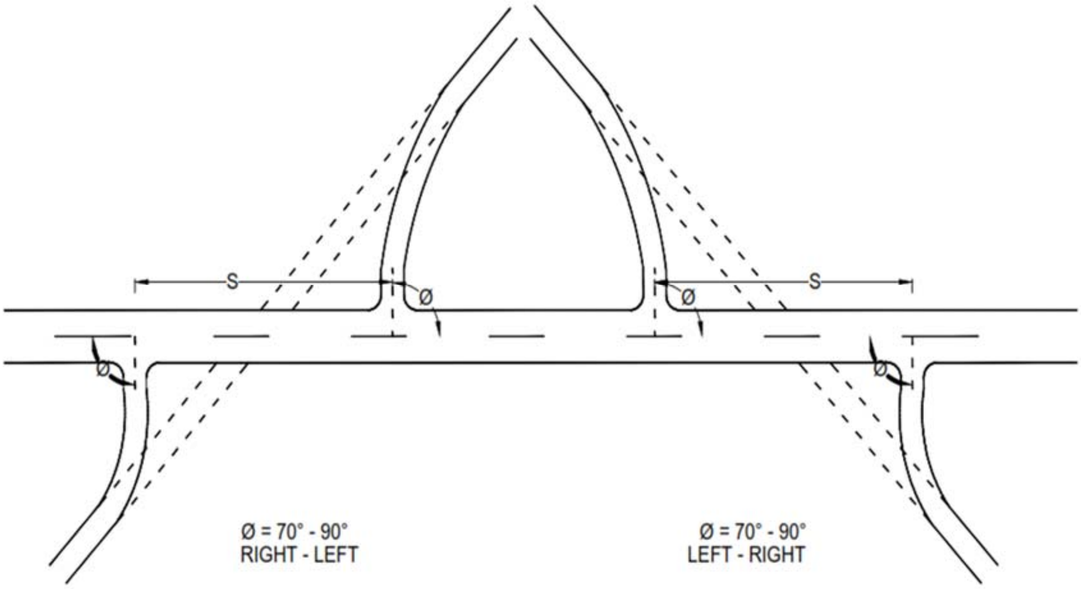


***TABLE 3.1: DESIRABLE SEPARATION OF STAGGERED T- INTERSECTIONS**

Design Speed Of Major Road (Km/h)	Separation (S) For Right/Left Stagger (m)	Separation (S) For Left/Right Stagger (m)
20	60	60
30	60	60
40	80	80
50	100	120
60	120	160
80	160	240
90	180	290
100	200	340

3.3 Vertical Alignment

It is desirable to avoid substantial grade changes at intersections. At all intersections the gradients of the intersecting roads should be as flat as practicable. Most drivers are unable to judge the increase or decrease in stopping or acceleration distance that is necessary on steep grades. Their normal reactions may be in error at a critical time. Grades in excess of 3% should be avoided on intersecting roads. Where conditions make such design unduly expensive, grades should not exceed 6% with a corresponding adjustment in stopping sight distances. These should be treated as special cases.

A general principle is that the horizontal and vertical alignment of the major road as well as its superelevation or crossfall is unchanged through the intersection and that the carriageway of the minor road and of the additional lanes are designed to match those of the major road.

The vertical profile of the minor road shall not have a gradient steeper than 2.5% over a section of 25m from the nearer edge of the major road. The grade shall also in general be connected tangentially (with or without a vertical curve) to the cross-section of the major road. If adverse topographic conditions make this unfeasible then the grade may be connected to the edge of the carriageway of the major road at an angle, provided that the difference in grade does not exceed 5%.

3.4 Sight Distance

3.4.1 General

The operator of a vehicle approaching an intersection at grade should have an unobstructed view of the whole intersection and a length of the intersecting road sufficient to permit control of the vehicles to avoid collisions. When traffic at the intersection is controlled by signals or signs, the unobstructed view may be confined to the area of control. It is advantageous on capacity grounds to increase where practicable the sight distances along the major road by up to 50% as this will allow several vehicles to emerge when large gaps in traffic stream on the main road occur.

As for the sight distance of the driver of a vehicle passing through at intersection, two aspects must be considered. There must be a sufficient unobstructed view to recognize potential conflicting traffic, the traffic signs or traffic signals at the intersection. And there must also be a sufficient sight distance to make a safe departure after the vehicle has stopped at the stop line. All intersections shall be either sign controlled as priority intersections or signalised control.

3.4.2 Approach Sight Triangle

The sight distance considered safe for approaching driver under various assumptions of physical conditions and driver behavior is directly related to vehicle speeds and to the resultant distances travelled during perception, reaction time and braking. The assumption of a driver's skill, i.e., perception and reaction time set the standards for sight distance and length of transitions.

The amount of time necessary to start deceleration is the driver's perception and reaction time which for intersection design can be taken as 2.5 seconds. In addition, the driver should begin actual braking some distance from the intersection to accomplish deceleration and avoid collision.

Sight distance should be measured from an eye level of 1050 mm (for passenger car) to the top of an object of height 1050 mm above the pavement. The use of an object height equal to the driver eye height makes intersection sight distances reciprocal (i.e., if one driver can see another vehicle, then the driver of that vehicle can also see the first vehicle).

There must be an unobstructed sight distance along both approaches of both roads at an intersection and across their included corners for a distance sufficient to allow the operator of vehicles, approaching simultaneously, to see each other in time to prevent collisions at the intersection. The sight triangle is shown on **FIGURE 3.2.**

Any object within the sight triangle high enough above the elevation of the adjacent roadways to constitute a sight obstruction should be removed or lowered. Such objects include cut slopes, trees, bushes and other erected objects. This also requires the elimination of parking within the sight triangle. Dangerous conditions may arise if, despite the provision of sufficient sight triangle, vehicles are allowed to park within the sight triangle thereby obstructing visibility.

When sight triangle is less than the desired, modifications in approach speeds by appropriate traffic control devices is required.

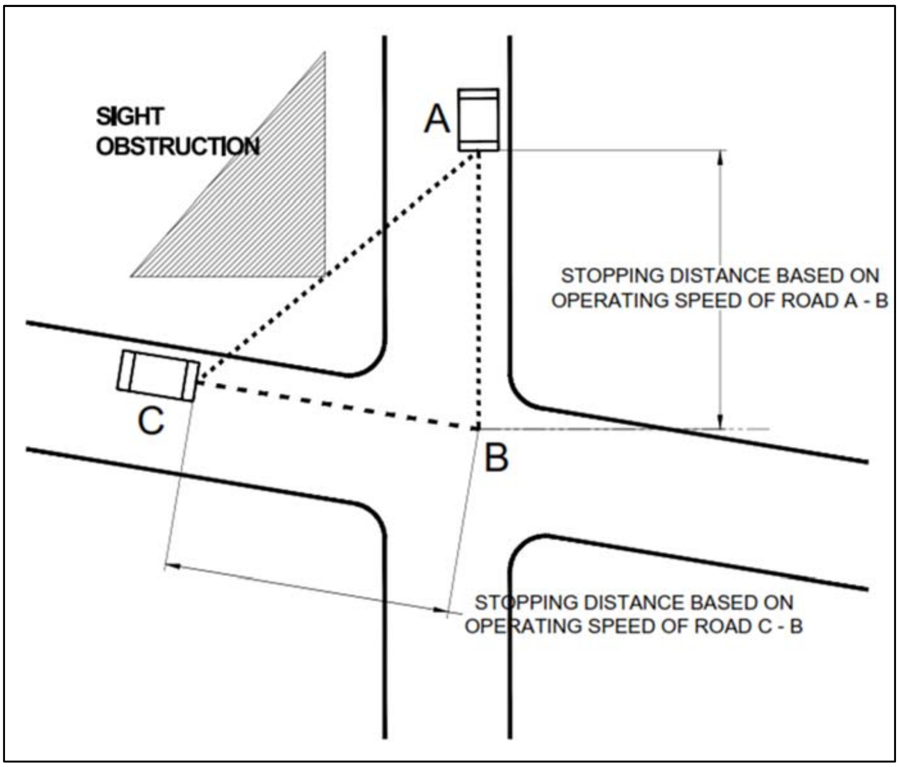


***FIGURE 3.2: INTERSECTION SIGHT TRIANGLE**

3.4.3 Approach Sight Distance

(a) Approach Sight Distance at No-Stop or Unsignalised Intersection

At no-stop or un-signalised intersections the operator of a vehicle on either road must be able to see the intersection in sufficient time to stop his vehicle if necessary before reaching the intersection. The safe stopping distances for intersection design are the same as those used for the design of any other section of the highway. These are as shown in the latest Arahan Teknik (Jalan) 8/86 "A Guide To Geometric Design Of Roads”. Where an obstruction which cannot be removed, except at prohibitive cost fixes the vertices of the sight triangle at points that are less than the safe stopping distances from the intersection, signs showing the safe speed should be so located that the driver can slow down to a speed appropriate to the available sight distance.

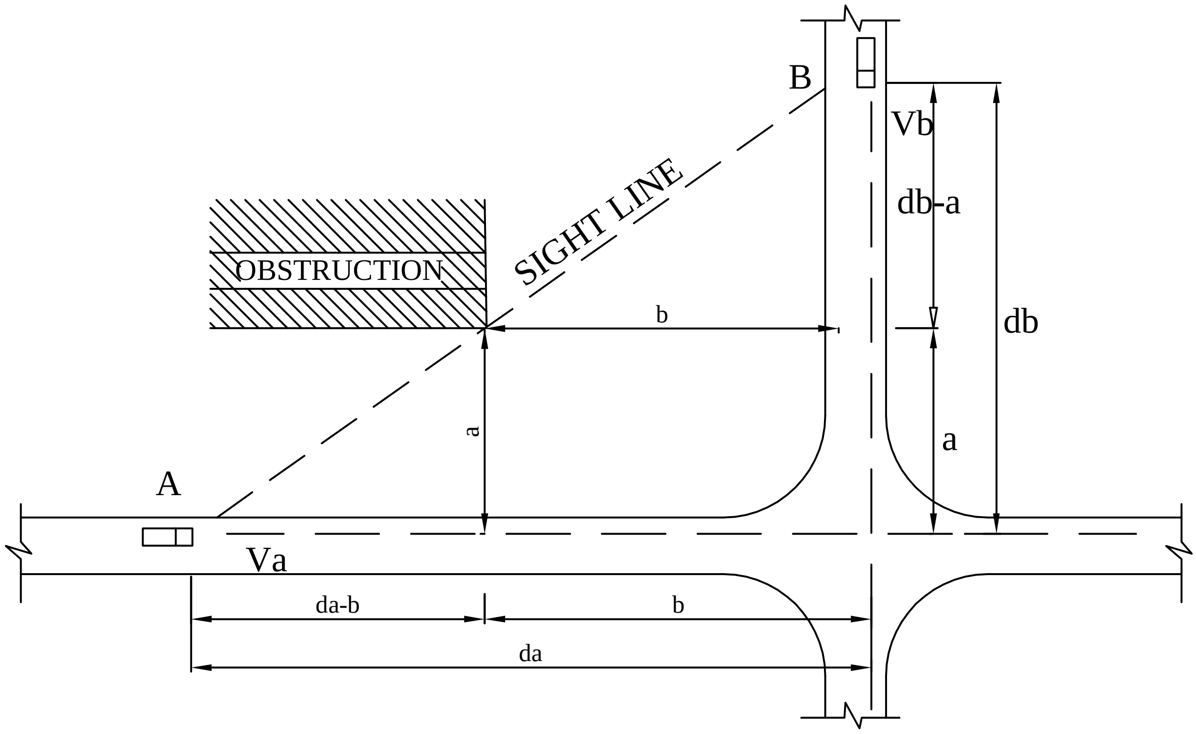
Referring to **FIGURE 3.3**, for a typical case, speed V_b is known and a and b are the known distances to the sight obstruction from the respective paths of vehicles A and B. The critical speed V1 of Vehicle B can then be evaluated in terms of these known factors. Distance d_a is the minimum stopping distance for Vehicle A. When vehicle A is at a distance d_a from the intersection and the drivers of Vehicles A and B first sight each other, Vehicle B is at a distance d_b from the intersection.

By similar triangle,

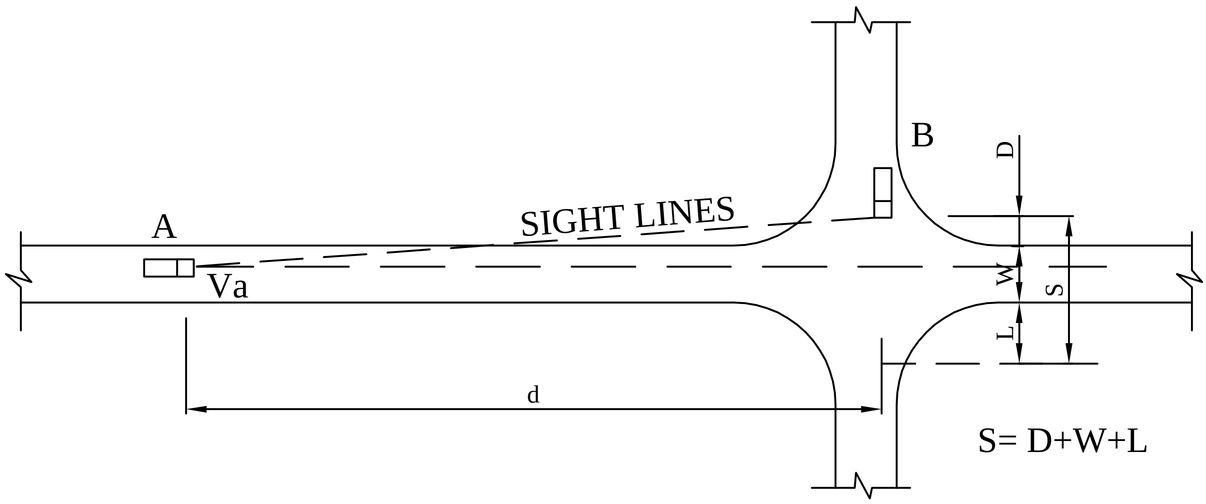
$$db = \frac{a \times d\alpha}{d\alpha - b}$$

and the critical speed V_b is that for which the stopping distance is d_b. The signs on road B showing the safe speed to approach the intersection should be so located that a driver can reduce his speed to V_b by the time he reaches the point that is distance d_b from the intersection. Similar calculations may be used to determine how far back an obstruction needs to be moved to provide sufficient sight distance for safe driving at desired vehicle speed on the respective roads.

For this case if the major road is one way a single sight triangle in the direction of approaching traffic will suffice. Similarly, if the major road has dual carriageways with no gap in the central reserve then a single sight triangle to the right will be need. If the minor road serves as a one-way exit from the major road, no sight triangle will be required provided forward visibility for turning vehicles is adequate.



NO STOP OR SIGNAL CONTROL AT INTERSECTION



STOP CONTROL ON MINOR ROAD

Figure 3.3: Sight Distance At Intersections
Minimum Sight Triangle

Scale : Not to scale

(b) Approach Sight Distance at Signalised Intersection

The sight distance is the sum of a distance travelled during the total reaction time which is the interval between the instant that the driver recognizes the traffic signals of the intersection ahead and the instant that the driver actually applies the brakes, and time taken for the vehicle to come to a stop.

The total reaction time can be further divided into the time required to make decision whether the brake should be applied or not, and the time for reaction after making the decision. Sufficient data is not available on the total reaction time. 10 seconds is adopted here. For urban areas, however, shorter total reaction time is used. This is because, with a lot of intersections in urban areas, drivers are always operating their vehicles with an anticipation of possible encounters of intersections. 6 seconds for urban areas is adopted here. A deceleration of 0.2g is taken as the allowable maximum without excessive discomfort. This is much lower than those used to obtain the stopping sight distance. This is because stops at intersections are quite routine, while stops to avoid possible collision on open road are much less frequent and more acute deceleration may be acceptable. From the discussion above, the sight distance for a signalised intersection is given as follows:

$$S = \frac{V \text{ x } t}{3.6} + \left\{\frac{1}{2\alpha} \times \left(\frac{V}{3.6}\right)^2\right\}$$

Where,

t = 10 sec. (rural), t = 6 sec (urban)

α = 0.2 x g = 0.2 x 9.8 = 1.96 m/sec^2

Sight distance should be measured from an eye level of 1050 mm (for passenger car) to the object height of 0.6m (tail light of passenger car)

(c) Approach Sight Distance at Stop Controlled Intersection

In this case, time for decision making as in signalised intersection is not necessary because every driver must stop. The reaction time of 2 seconds is taken. Accordingly, t = 2 seconds, alpha = 1.96m/sec^2 are substituted into the above formula.

From the discussion above, the criteria shown in Table 3-2 is obtained.

Sight distance should be measured from an eye level of 1050mm (for passenger car) to the pavement surface (object height at stop line of 0.0m)
On the major road, drivers can operate their vehicles without worrying about intersections. Stopping sight distance (SSD) and k values as per latest ATJ 8/86 defined for open road is sufficient.

On the major road, drivers can operate their vehicles without worrying about intersections. Stopping sight distance (SSD) and k values as per latest ATJ 8/86 deifned for open road is sufficient.

***TABLE 3.2: SIGHT DISTANCE FOR INTERSECTION APPROACH**

Design Speed Of Major Road	Signal Control		Stop Control On Minor Road
(Km/h)	Rural (m)	Urban (m)	
100	480	370	260
80	350	260	170
60	240	170	105
50	190	130	80
40	140	100	55
30	100	70	35
20	60	40	20

Note: On The Major Roads Of Stop Controlled Intersections, The Stopping Sight Distances Given In latest Arahan Teknik (Jalan) 8/86 “A Guide To Geometric Design Of Roads” Must Be Satisfied.

3.4.4 Departure Sight Triangle

At an intersection where traffic is controlled by STOP signs on the minor road it is necessary for the driver of a stopped vehicle to see enough of the major road to be able to cross before a vehicle on the major road reaches the intersection (See **FIGURE 3.3**). The required sight distance along the major highway can be expressed as: -

$$d = 0.28V\,(J + t_a)$$

Where,

d = minimum sight distance along the major road from the intersection, metres.

V = design speed of major road, km/hr.

J = sum of perception time and the time required to shift to first gear or actuate an automatic shift, seconds.

t_a = time required to accelerate and traverse the distance S to clear the major road, seconds.

The term J represents the time necessary for the vehicle operator to look in both directions and to shift gears, if necessary, preparatory to starting. A value of 2 seconds is assumed. In urban or suburban areas where drivers generally use many intersections with STOP sign control a lower value of 1.5 or even 1 second may apply. The time t required to cover a given distance during acceleration depends upon the vehicle acceleration. The acceleration of buses and trucks is substantially lower than that of passenger vehicles.

On flat grades, the acceleration time for SU (single unit) and semi-trailer is about 135 % and 160% respectively of that for passenger vehicles. The value of t can be read directly from **FIGURE 3.4** for nearby level conditions for a given distance S in m. Referring to **FIGURE 3.3** the distance S which the crossing vehicle must travel to cross the major road is given by

$$S = D + W + L$$

Where,

D = distance from near edge of pavement to front of stop vehicle

W = width of pavement along path of crossing vehicle

L = overall length of vehicle

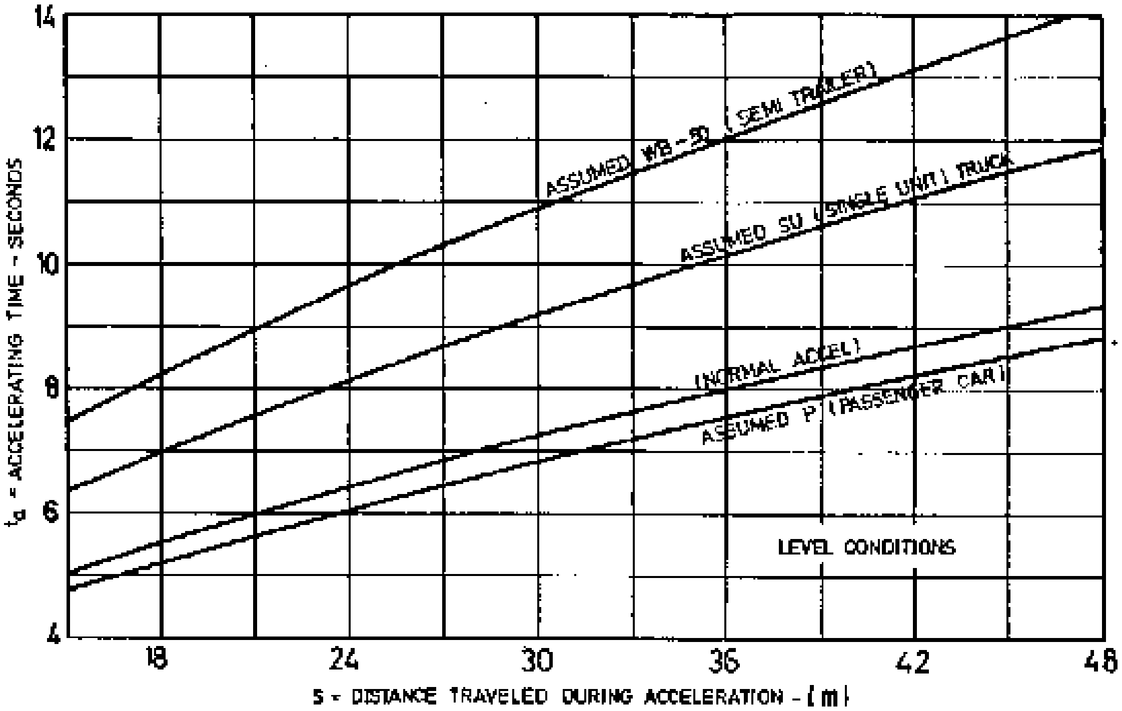


***FIGURE 3.4: SIGHT DISTANCE AT INTERSECTION DATA ON ACCELERATION FROM STOP**

For general design purposes a value of D = 3m is assumed. The value of L, the overall length of design vehicles can be assumed to be 5m, 10m and 15m for passenger cars, single unit trucks and semi-trailers respectively.

In testing whether the sight distance along a major road is adequate at an intersection the distance should be measured from an eye level of 1050mm to the top of an object of height 1330mm placed on the pavement.

In the case of divided roads, widths of median equal to or greater than the length of vehicle enable the crossing to be made in two steps. For divided highways/roads with medians less than L the median width should be included as part of W.

Along a major road, the longer distance of the two: the sight distance described here and the stopping sight distance must be satisfied. The former will exceed the latter at higher ranges of the design speeds.

Where the sight distance along a major road is less than that for departure at an intersection it is unsafe for vehicles on the major highways/ roads to proceed at the assumed design speed of the highways/ roads and signs indicating the safe approach speed should be provided.

The safe speed may be computed for a known sight distance and the width of pavement on the path of the crossing vehicle. On turning roadways and ramps, at least the minimum stopping sight distance should be provided continuously along such roadways. Where the major road has dual carriageways with a central median width enough to shelter turning vehicles (4.5m or more) the normal sight triangle to the left of the side road will not be needed but the central median should be clear of obstructions to driver visibility for at least d m.

3.4.5 Effect of Skew

When two roads intersect at an angle considerably less than a right angle and realignment to increase the angle of intersection is not justified, some of the factors for corner sight distance determination may need adjustment. The difficulty in looking for approaching traffic makes it undesirable to treat the intersection based on the assumptions of no control intersections even where traffic on both roads is light. Treatment by controlled intersection or safe departure whichever is the larger should be used at skew intersections. In case of departure the distance S is larger for oblique than for right angle intersections. The width of pavement on the path of the crossing vehicle, W, (See **FIGURE 3.5**) is the pavement width divided by the sine of the intersection angle.

The distance along the road can be computed by the formula:

$$d = 0.28V\,(2 + t_a)$$, reading ta directly from Figure 3.4

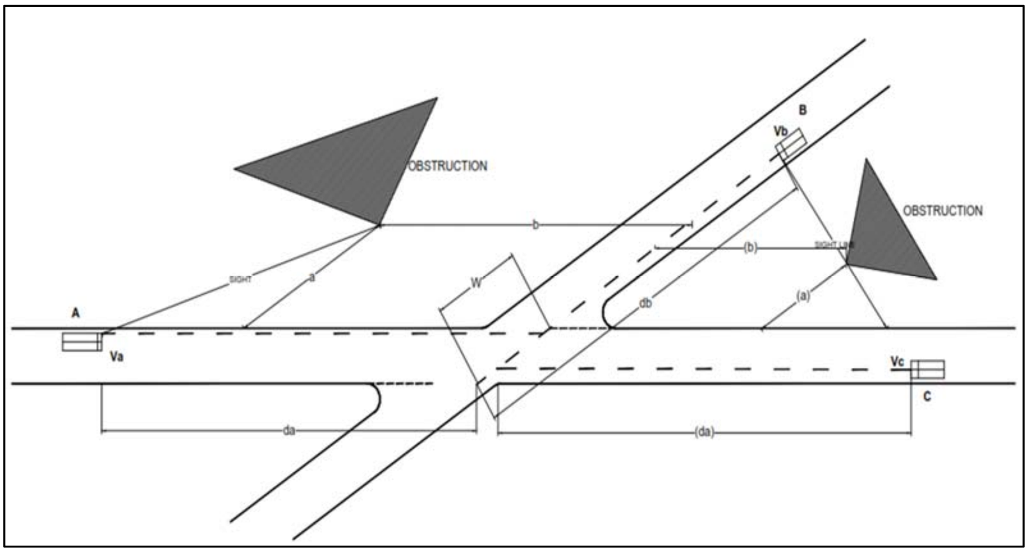


***FIGURE 3.5: SIGHT DISTANCE AT INTERSECTIONS EFFECT OF SKEW**

3.4.6 Effect of Grades

The differences in stopping distances on various grades at intersections are as given in **TABLE 3.3**. Grades on an intersection leg should be limited to 3 percent. In case of departure derivation of the time required to cross on the major road highway is affected by the grade of crossing on the minor road. Normally the grade across an intersection is so small that it need not be considered but when curvature on the major road requires the use of superelevation, the grade across it may be significant. The effect of grade on acceleration can be expressed as a multiple and to be used with the time, ta as determined for level conditions for a given distance as shown in **TABLE 3.4**.

The value of ta from **FIGURE 3.4** adjusted by the appropriate factors can be used in the formula d = 0.28V (2 + ta).

***TABLE 3.3: EFFECT OF GRADE ON STOPPING SIGHT DISTANCE WET CONDITIONS**

Design Speed. (Km/hr)	Correction In Stopping Distance – Metre					
	Decrease For Upgrades			Increase For Downgrades		
	3%	6%	9%	3%	6%	9%
30	-	-	3			3
40		3	3		3	6
50	-	3	6	3	6	9
60	3	6	9	3	9	15
80	6	9	-	6	15	-
100	9	15	-	9	24	-

***TABLE 3.4: CORRECTION FACTOR FOR THE EFFECT OF GRADE ON ACCELERATION TIME, TA**

Design Vehicle	Minor Road Grade (%)				
	-4	-2	0	+2	+4
Passenger Cars (P)	O.7	0.9	1.0	1.1	1.3
Single Unit Trucks (SU)	0.8	0. 9	1.0	1.1	1.3
Semi-Trailers (WB - 15)	0.8	0.9	1.0	1.1	1.7

3.5 Right Turn Lanes

3.5.1 General

The purpose of right-turn lane is to expedite the movement of through traffic, regulate the movement of turning traffic, increase in the capacity of the intersection and improve safety characteristics. Right turn lanes should be considered in the following cases:

(a) When the major road flow exceeds 600 vehicles/hr for both ways
(b) At all intersections on divided urban roads with a sufficiently wide median.
(c) At all intersections on undivided urban roads where right turning traffic is likely to cause unacceptable congestion and/or hazard.

Exclusive right turn lanes at signalised intersections should be installed as follows:

(i) Where exclusive right turn signal phasing is provided;
(ii) Where right turn volumes exceed 100 veh/h (right turn lanes may be provided for lower volumes as well base on highway agency's assessment of the need, the state of local practice, or both); and
(iii) Double right turn lanes should be considered where right turn volumes exceed 300 veh/h.

3.5.2 Warrants for BA, AU and CH Turn Treatments

These warrants apply to major road turn treatments for the basic, auxiliary lane and channelised layouts illustrated in Section 1.2.1, 1.2.2 and 1.2.3 respectively. The warrants are shown in Figure 3.6A and provide guidance on where a full-length deceleration lane must be used, may be acceptable based on traffic volume. Figure 3.6A contains two graphs for the selection of turn treatments on roads with a design speed:

(a) Greater than or equal to 100 km/h. Figure 3.6A (a) is appropriate for high speed rural roads
(b) Less than 100 km/h. Figure 3.6A (b) is appropriate for urban roads, including those on the urban fringe and lower speed rural roads.

If a particular turn from a major road is associated with some geometric minima (for example, limited sight distance, steep grade), consideration should be given to the adoption of a turn treatment of a higher order than that indicated by the warrants. For example, if the warrants indicate that a BAR turn treatment is acceptable for the relevant traffic volumes, but limited visibility to the right-turning vehicle is available, consideration should be given to the adoption of a AUL turn treatment instead. Another example is a major road on a short steep downgrade where numerous heavy vehicles travel quickly down the grade, in which case it would not be appropriate to adopt a BAL turn treatment. Instead, an CHR would be a preferred treatment.

In applying the warrants in Figure 3.6A designers should note that:

(a) Curve 1 represents the boundary between a BAR and a AUL turn treatment. Curve 2 represents the boundary between a AUL and a CHR turn treatment.
(b) The warrants apply to turning movements from the major road only (the road with priority). Figure 3.6B is to be used to calculate the value of the major road traffic volume parameter (Q_M).
(c) Traffic flows applicable to the warrants are peak hour flows, with each vehicle counted as one unit (i.e. do not use equivalent passenger car units [pcus]). Where peak hour volumes or peak hour percentages are not available, assume that the design peak hour volume equals 8% to 10% of the AADT for urban situations and that the design hour volume equals 11% to 16% of AADT for rural situations.
(d) If more than 50% of the traffic approaching on a major road leg turns left or right, consideration needs to be given to possible realignment of the intersection to suit the major traffic movement. However, route continuity issues must also be considered (for example, realigning a highway to suit the major traffic movement into and out of a side road would be unlikely to meet driver expectation).
(e) If a turn is associated with other geometric minima, consideration should be given to the adoption of a turn treatment of a higher order than that indicated by the warrants.
(f) Where the major road has four lanes (e.g. two in each direction) the value used for (Q_M) is the volume in the closest through lane to the turning movement.

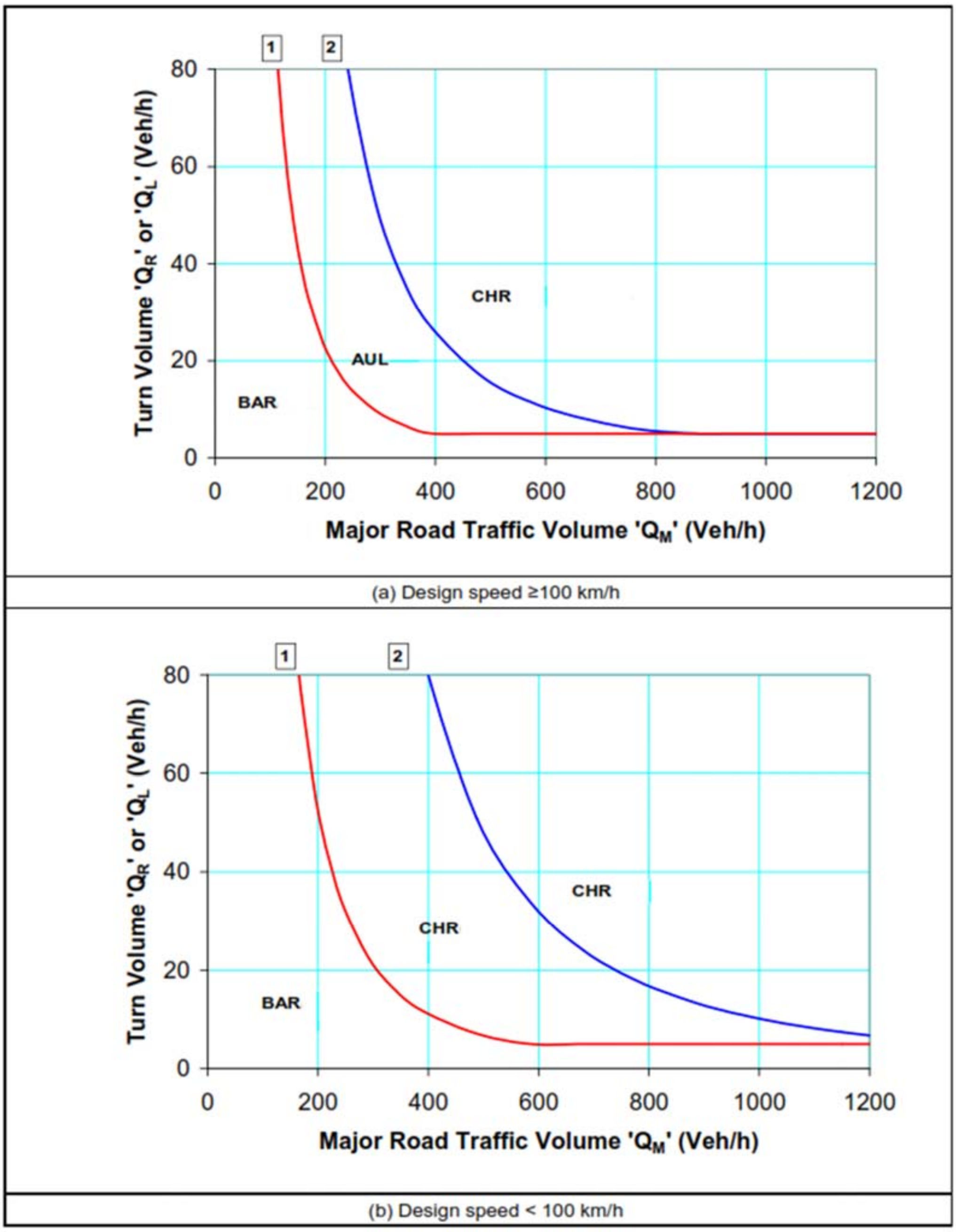
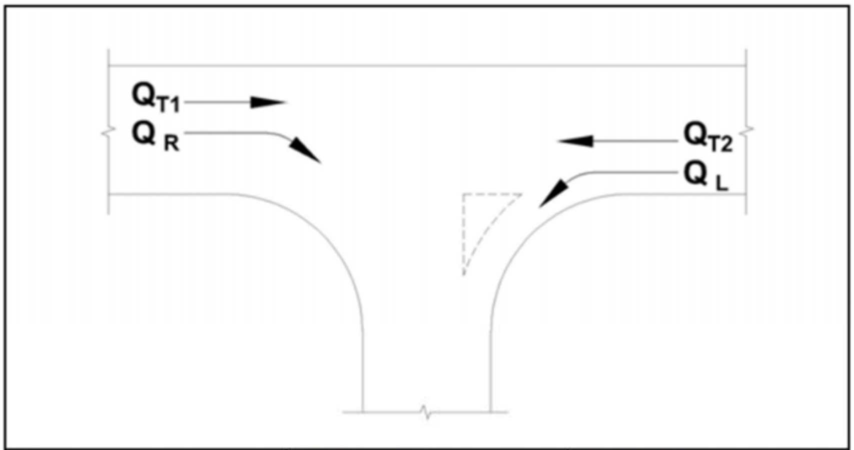


FIGURE 3.6A: WARRANTS FOR TURN TREATMENTS ON THE MAJOR ROAD AT UNSIGNALISED INTERSECTIONS

Source: Austroads 2010, Guide to road design: part 4A Unsignalised and Signalised Intersections, figure 4.9.



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

FIGURE 3.6B: CALCULATION OF THE MAJOR ROAD TRAFFIC VOLUME PARAMETER (Q_M)

Source: Austroads 2010, Guide to Road Design: Part 4A Unsignalised and Signalised Intersections, Figure 4.10

3.5.3 Design Considerations

The roadway widths of turning roadways at intersections are governed by the volumes of turning traffic and the types of vehicles to be accommodated. The turning radii and the widths are function of design speed and type of vehicles.

As a minimum, the turning path of a SU vehicle should be used for the design of right turns. In specific areas where larger vehicle type is common, the turning path for the larger vehicle type shall be adopted. It is necessary for the designer to analyse the likely paths and encroachments that result when a turn is made by vehicles larger than those for which design is made.

The vehicle executing the right-turn manoeuvre must not encroach on the shoulder with its front wheels opposite side of the road centre lines with its rear wheels. **FIGURE 3.7** illustrates the essential design features of right-turn lanes.

3.5.4 Length of Right Turn Lanes

The length of right turning lane consists of deceleration length and storage length. Total length should be the sum of these two components. Provision of deceleration clear of through-traffic is a desirable objective.

Where storage is required, the length should be increased according to the expected queue length. The storage length should be sufficiently long so that the through lane traffic is not blocked by vehicles standing in the through lanes while waiting for a signal change or gap in the opposing traffic flow. Storage length can be estimated as follows: -

(a) Signalised intersection

Storage length is calculated as L= 1.5 x N x S

Where

N = Average number of right turning vehicles in a cycle of signal phase (vehicle).
S = Average headway in distance (m)
S = 6m for a passenger car
S = 12m for other large commercial vehicles
If the commercial vehicle ratio is not known,
S = 7m may be used.

At signalised intersections, the required storage length depends on the signal cycle length, the signal phasing arrangement and the rate of arrivals and departures of right turning vehicles. The storage length is usually based on 1.5 to 2 times the average number of vehicles that would store per cycle.

(b) Unsignalised intersection

Effect of traffic fluctuation to the storage length is more significant in unsignalised intersections. The following formula can be applied:

L= 2 x M x S

Where,

M = Average number of right turning vehicles in a minute.
S = Average headway in distance (m)
S = 6m for a passenger car
S = 12m for other large commercial vehicles
If the commercial vehicle ratio is not known,
S = 7m may be used.

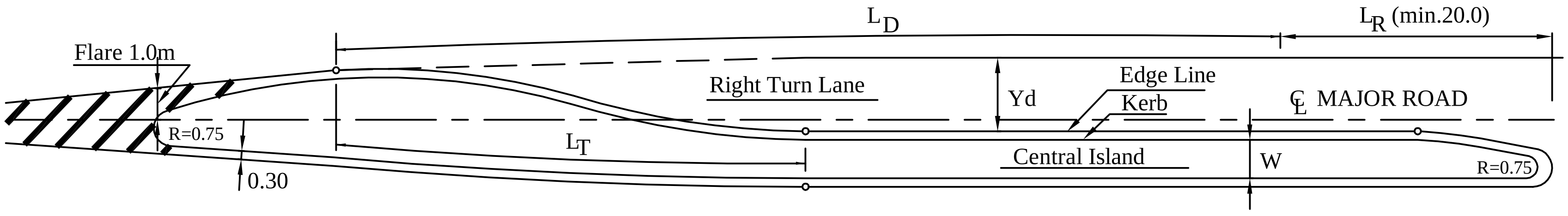
At both signalised and unsignalised intersections, a storage length of at least 20m should be provided

A right-turn lane shorter than required would cause the turning vehicles to follow up on the parallel lane and to obstruct through traffic. In urban areas, however, various constraints sometimes impose the reduction in the length of right-turn lanes. In this case, shortage in the length should be adjusted in the taper length with the storage length maintained as long as possible.

Where the right turn lane is obscured by a crest, it will be necessary to extend the length of the lane in order to give the driver adequate time to perceive the lane in time to start his deceleration.

If two or more lanes are provided to cope with heavy right turning traffic, storage length can correspondingly be reduced from that required for single lane operation.

A) LAYOUT



B) LENGTH OF TAPER

$$LT = \frac{1}{3} V\sqrt{Yd}$$

Where

Y = Design speed in km/h.

Yd = Width of right turn lane (m)

LT = May be rounded to the nearest multiple of 5m

For design of S-curve See Figure:3-22

C) WIDTH OF CENTRAL ISLAND

	Yd (m)	W (m)		
		Pedestrian Refuge	Signal pedestal No pedestrians	No Signal or pedestrians
Desirable	3.5	2.50	2.50	2.50
Minimum	3.0	2.50	1.80	1.20

D) DECELERATION LENGTH, LD (m)

Gradient in %		Design speed in km/h						
		20	30	40	50	60	80	100
Uphill	4	20	28	41	54	72	108	153
	2	20	30	45	60	80	120	170
Level	0	20	30	45	60	80	120	170
Downhill	2	20	30	45	60	80	120	170
	4	20	34	53	72	96	144	204

NOTES

1. The length of the reservoir space shall be rounded upwards to the nearest multiple at 5m
2. Deceleration lengths for other gradients may be found by interpolation or up to 6% by extrapolation.
3. All dimensions are in m.

JKR

Figure 3.7 : Right-Turn Lanes

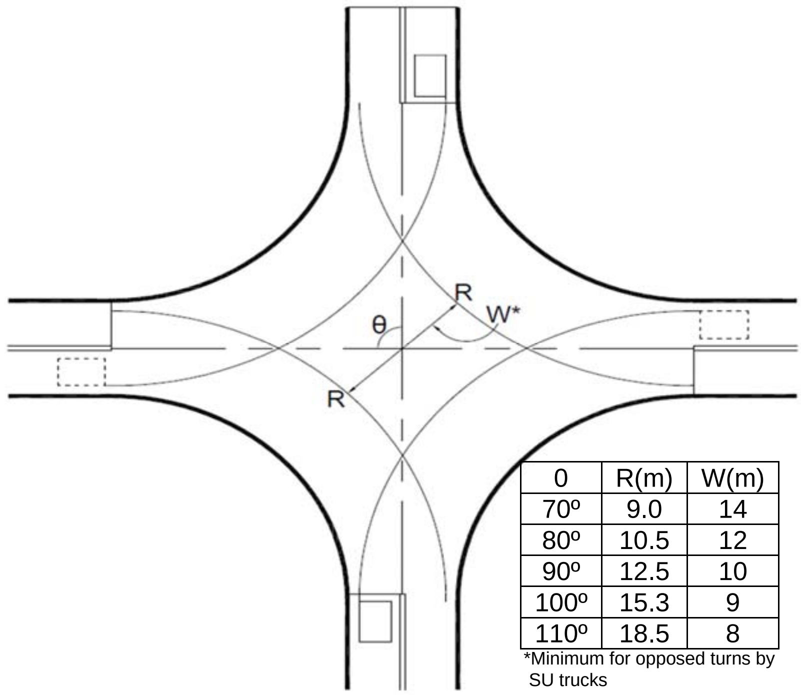
Scale : Not to scale

3.5.5 Width of Right Turn Lanes

Right turn lanes’ width shall desirably be equal to that of the through lane and shall not be less than 3.0m wide.

3.5.6 Opposed Right-Turns

When two opposing right turns are expected to run simultaneously, the turning radii and the tangent points should be such that there is a clear width (of at least 3.0m) between the outer wheel paths of opposing vehicles in accordance with **FIGURE 3.8**.



0	R(m)	W(m)
70º	9.0	14
80º	10.5	12
90º	12.5	10
100º	15.3	9
110º	18.5	8

*Minimum for opposed turns by SU trucks

***FIGURE 3.8: RIGHT TURN CLEARANCE**

3.5.7 Seagull Island

A seagull island is a triangular island used to separate right turning traffic from through traffic in the same carriageway as shown in **FIGURE 3.9**.

Adequate storage length is required in approach to the island and a merging taper appropriate to the speed of the through carriageway must be provided on the departure side.

It is essential that STOP lines, median noses and "seagull" islands be located to suit vehicle turning paths. **FIGURE 3.7** illustrates the essential design features of right-turn lanes.

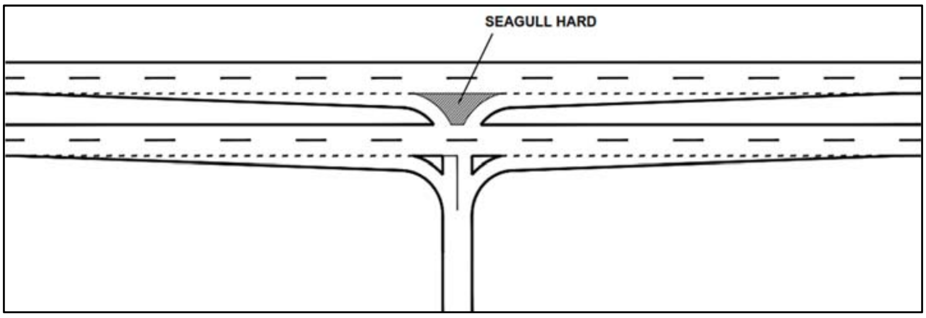


***FIGURE 3.9: SEAGULL ISLAND**

3.5.8 Central Island and Median Design

The minimum central island widths shall follow that as listed in **FIGURE 3.7 (C).**

Central islands may be made in one of the following ways: -

(a) Painted as cross hatched areas on the pavement (ghost islands).
(b) Raised island surrounded by kerbs.

Ghost island should be used where the island is of the width of or less than the turn lane. It should also be used in rural intersections where there is no street lighting.

Kerbed islands shall be used where the islands are wide. Medians should also be kerbed on both sides from the start of the taper of the right turning lane. The design considerations for kerbs should follow that laid down in the latest Arahan Teknik (Jalan) 8/86 - A Guide to Geometric Design of Roads.

3.6 Left Turn Lanes

3.6.1 General

The type of left turn lane and its treatment depends on: -

(a) Type and volume of traffic making the turn.
(b) Restrictions caused by the surrounding development.
(c) Speed at which the left-turn is to operate.

These factors determine the radius of the kerb and the width of the left-turn lane. There are two types of treatment for left-turns, Simple Left-Turns and Separate Left-Turn Lanes.

3.6.2 Simple Left-Turns

These are usually provided where traffic volumes are low and where land acquisition costs prevent more extensive treatment or the angle of turn prohibits the installation of an island.

At urban intersections the radius of the kerb for the left-turn should be a minimum of 6m. This allows most commercial vehicles to negotiate the turn at low speeds without encroaching either on the footway with the rear wheels or on the opposite side of the road's centre line with the front wheels.

While radii larger than 10m increase the speed of turning movements they reduce the safety of pedestrian crossings and create problems in locating signal pedestals and STOP lines. For simple left turns in urban areas, such radii should only be used after careful consideration of the above. At rural intersections where provision for pedestrian is not a consideration, larger radius curves may be used. Radii larger than 15m should not be used without left-turn island as they create large areas of uncontrolled pavement.

3.6.3 Separate Left-Turn Lanes

Where the volume of left-turning traffic is high or the skew favours such a layout, a corner island can be introduced to create a separate left-turn lane.

(a) Design Speed of Left-Turn Lane

Design speed of left-turn lane higher than that shown in **TABLE 3.5** should be chosen, considering the turning volume, availability of land and the design speed of the approach road.

***TABLE 3.5: MINIMUM DESIGN SPEEDS FOR LEFT-TURN CHANNEL**

Design Speed Of Approach Road (Km/h)	Minimum Design Speed Of Left-Turn Lanes (Km/h)
100	50
80	40
60	30
50	30
40	20
30	20
20	20

Principally, the minimum design speed of less than 20 km/h is not desirable. While it is desirable and often practical to design for turning vehicles operating at higher speed, it is often appropriate for safety and economy to use lower turning speeds at the intersections. Vehicles turning at intersections designed for minimum-radius turns have to operate at low speeds, perhaps less than 15 km/hr

(b) Radius for Separate Left-Turn Lanes

The minimum radii used for design should preferably be measured from the inner edge of the traveled way rather than the middle of the vehicle path or the centerline of the travelled way.

Where environmental and other constraints do not directly determine it, the radius R_1 of a separate left-turn lane depends on:

(i) The speed, V, at which vehicles operate,
(ii) The superelevation,e
(iii) The acceptable coefficient of friction, f between vehicle tyres and the pavement.

TABLE 3.6 gives the relationship between these factors.

The values of R_1 in the table are calculated from the formula

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

The use of simple curve should always be the priority unless there are constraints on its location, high cost of land acquisition involved, relocation of utilities, social issues and others.

The superelevation of curves on separate turning lanes at intersections usually has a low value mainly because of the difficulty of developing the superelevation on relatively short length of a separate turning lane. A desirable maximum value in rural areas is 0.08. In urban areas this should not exceed 0.04 to 0.06.

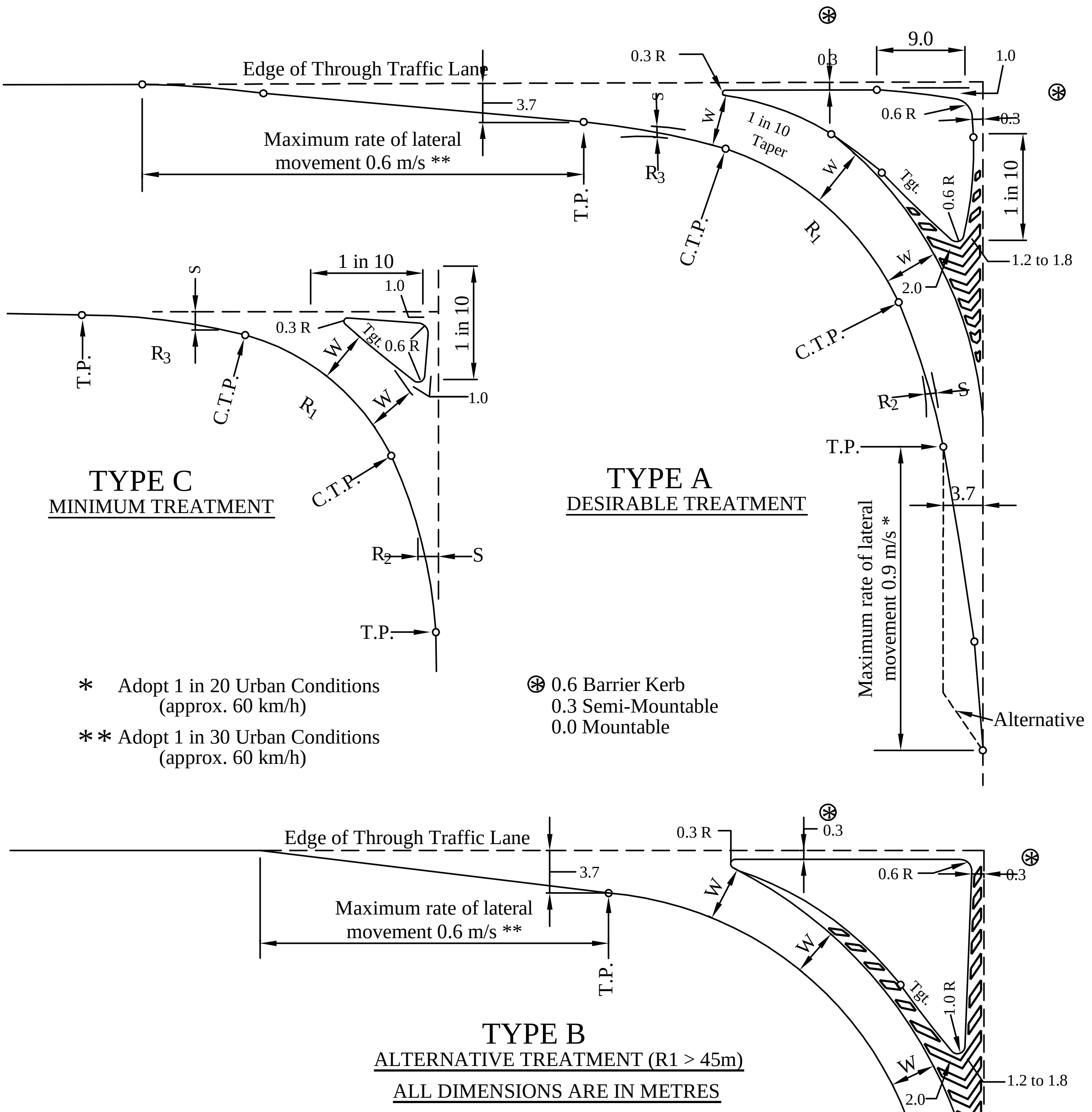
The values of f given in **TABLE 3.6** are greater than those used for open highway design as drivers turning on curves of small radii at intersections accept a lower level of comfort.

FIGURE 3.10 illustrates the combination of radii and widths required for the tracking of the design vehicle. For R_1 more than 45m the off-tracking is negligible and a single radius R_1 is acceptable. Method of attainment of superelevation runoff for open roads should basically be followed in the design of intersection.

For R_1 within the range of 12m to 30m the turn should be designed to provide for tracking of the design vehicle. A compound curve with successive radii 1.5R, R_1 and 3 R_1 satisfies this requirement. For radii R_1 between 30-45m the vehicle tracking can be accommodated by using a compound curve with successive radii 2 R_1, R_1 and 2 R_1.

***TABLE 3.6: TURNING RADII**

V, (km/H)	f	e (m/m)				
		0	0.02	0.04	0.06	0.08
		R_1, (m)				
20	0.34	10	9	9	8	8
30	0.28	25	23	22	20	19
40	0.23	55	50	46	43	40
50	0.19	104	93	85	78	72
60	0.17	167	149	135	123	112
80	0.16	315	280	252	229	210



R_1	W_1	W_2	W_3	S
12	6.5	7.5	10.3	1.5
14	6.2	7.2	10.1	1.4
16	6.0	7.1	9.9	1.3
18	5.9	6.9	9.7	1.3
20	5.7	6.8	9.6	1.2
22	5.6	6.7	9.5	1.2
24	5.5	6.6	9.4	1.1
26	5.4	6.5	9.3	1.1
28	5.4	6.5	9.2	1.1
30	5.3	6.4	9.1	1.0
45	5.0	6.1	8.8	0.9
60	4.8	5.9	8.6	0.9
90	4.6	5.8	8.4	0.8
120	4.5	5.7	8.3	0.7
150	4.5	5.6	8.2	0.7

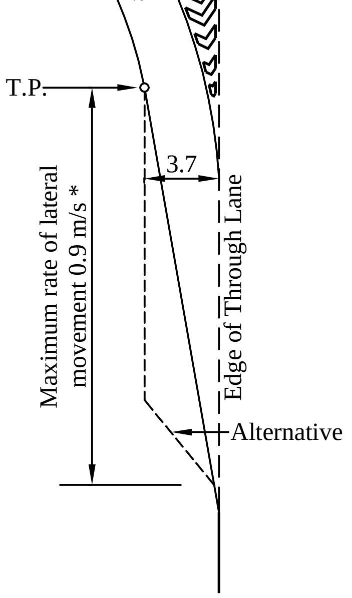
Where R_1 = 12m to 30m.
$R_2 = 1.5\ R_1$
$R_3 = 3\ R_1$

Where R_1 = 30m to 45m.
$R_2 = 2\ R_1$
$R_3 = 2\ R_1$

Where R_1 > 45m Single Rad. acceptable

LANE WIDTHS

W_1 = Single-lane flow.
W_2 = Single lane flow with provision for passing a stalled vehicle.
W_3 = Two-lane flow.



Traffic condition : Sufficient SU vehicles to govern design but some consideration for semitrailer vehicles.

Figure 3.10: Design of Separate Left-Turn Lanes

Scale : Not to scale

(c) Compound curves

Compound curve is a curve made up of two or more circular arcs of successively shorter or longer radii, joined tangentially without reversal of curvature, and used on some railroad tracks and highways as an easement curve to provide a less abrupt transition from tangent to full curve or vice versa. When the design speed of the turning roadway is 70 km/h or less, compound curvature can be used to form the entire alignment of the turning roadway. When the design speed exceeds 70 km/h, the exclusive use of compound curves is often impractical, as it tends to need a large amount of right-of-way. Thus, high-speed turning roadways follow the interchange ramp design guidelines and include a mix of tangents and curves. By this approach, the design can be more sensitive to right-of-way impacts as well as to driver comfort and safety. An important consideration is to avoid compound curve designs that mislead the motorist's expectation of how sharp the curve radius is. For compound curves on turning roadways, it is preferable that the ratio of the flatter radius to the sharper radius not exceed 2:1. This ratio results in a reduction of approximately 10km/h in average running speeds for the two curves.

Curves that are compounded should not be too short or their effect in enabling a change in speed from the tangent or flat curve to the sharp curve is lost. In a series of curves of decreasing radii, each curve should be long enough to enable the driver to decelerate at a reasonable rate. At intersections, a maximum deceleration rate of 5 km/h/s may be used (although 3 km/h/s is desirable). The desirable rate represents very light braking, because deceleration in gear alone generally results in overall rates between 1.5and 2.5 km/h/s Minimum compound curve lengths based on these criteria are presented in **TABLE 3.7.** The compound curve lengths in **TABLE 3.7** are developed on the premise that travel is in the direction of sharper curvature. For the acceleration condition, the 2:1 ratio is not as critical and may be exceeded.

TABLE 3.7: LENGTHS OF CIRCULAR ARCS FOR DIFFERENT COMPOUND CURVE RADII

Radius (m)	Minimum Length Of Circular Arc (M)	
	Acceptable	Desirable
30	12	20
50	15	20
60	20	30
75	25	35
100	30	45
125	35	55
150 Or More	45	60

Source: AASHTO 2011, Chapter 9, Table 3-14.

Compound curves are also unnecessary where there is a painted island or an island is either not required or cannot be provided. In these cases, the front wheels of the occasional semi-trailer can be steered wide enough to prevent the back wheels running over the kerb or running onto the shoulder.

When a corner island is to be introduced to create a separate left-turn lane and a three-centred curve is justified, the combination of radius and angle of turn should provide minimum island area as follows:

(i) In urban areas, $8m^2$

For adequate definition of the island, shelter for pedestrians as well as the posssible installation of traffic signals.

(ii) In rural areas, $50m^2$

For adequate definition of the island. **FIGURE 3.11** indicates the combination of radius and angle of turn which provides these minimum island areas.

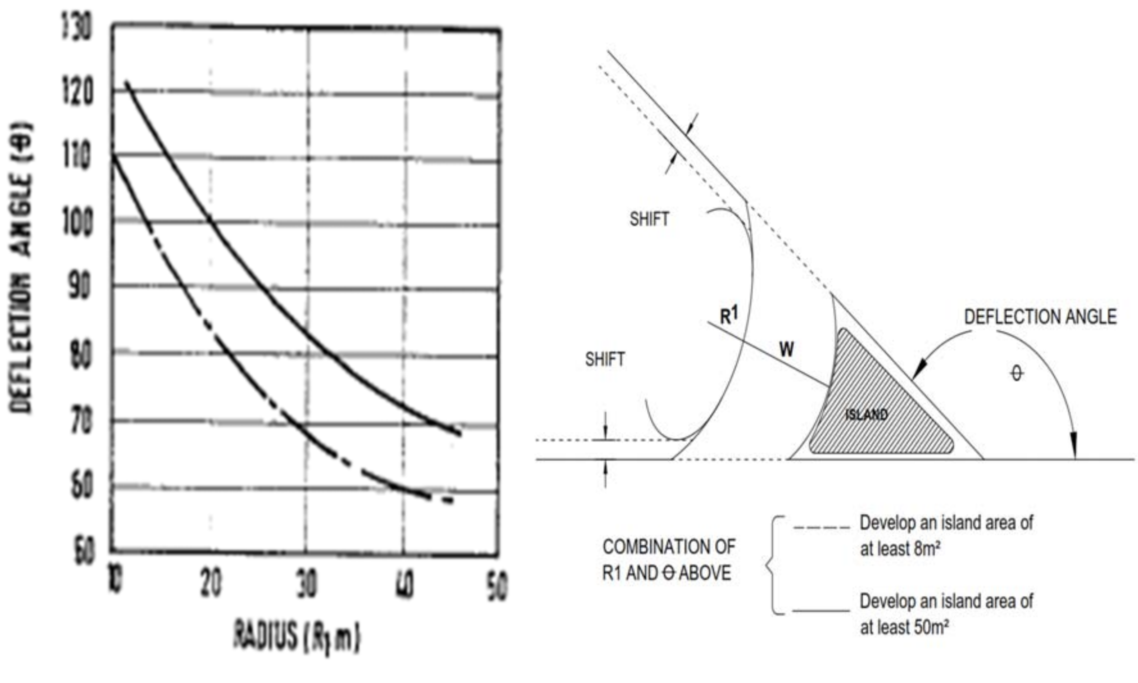


***FIGURE 3.11: ISLAND AREAS**

(d) Width of Left-Turn Lanes

The width of a left-turn lane depends on:

- Radius
- Volume and type of turning traffic
- Whether kerb side parking is permitted or prohibited.
- The length of the lane
- Whether both edges are kerbed

There are three design conditions:

(i) Single lane flow (width W1). This is the normal application and is used in rural or semi-urban locations where there is a shoulder on the inner edge of pavement. It may also be applied in urban areas where the inner edge of the lane is kerbed but the corner is small.

(ii) Single lane flow with provision for passing a stalled vehicle (Width W2). This width is desirable for urban locations where parking is prohibited and the corner island has an inner edge longer than approximately 20m.

(iii) Two lane flow (Width W3). This width is to be adopted where traffic volumes require two lanes and parking is prohibited. Width W is carried for the whole length of the left turn lane.

Design conditions which define the lane width of left-turn lane should be found in **TABLE 3.8** according to the class of road. The table in **FIGURE 3.10** gives the required widths for various radii and design conditions.

***TABLE 3.8: LANE WIDTHS FOR LEFT- TURN LANE**

Area	Category Of Road	Lane Width
Rural	Highway	W_3/W_2
	Primary	W_2
	Secondary	W_1
	Minor	W_1
Urban	Arterial	W_3/W_2
	Collector	W_2 /W_1
	Local Street	W_1

Note: The widths shown are determined for the design SU vehicle including some consideration for WB-15. A separate study is required if P-vehicle is employed for design. If two alternatives are given, one should be selected according to turning volume of traffic.

3.7 Pavement Tapers

3.7.1 General

Pavement tapers are used at the following places: -

(a) The ends of acceleration and deceleration lanes provided for left and right turn manoeuvres.

(b) The ends of widened carriageway or dual carriageways to assist the merging and diverging of through traffic manoeuvres.

3.7.2 Design Principles

The following are the general design principles on pavement tapers:

(a) Pavement tapers for diverging movements should provide for a rate of lateral movement of 0.9m per second.

(b) For merging movements, they should provide for a rate of lateral movement of 0.6m per second. However, where traffic volumes are high greater lengths may be provided.

(c) Care must be exercised in designing diverging tapers to ensure that through traffic is not led into an auxiliary lane in error.

(d) Care must be exercised with the location design of all merging tapers to ensure that there is sufficient sight distance for the approaching driver to realize the existence and geometry of the merge.

(e) Sufficient lengths of straight, horizontal and vertical alignment to allow four (4) seconds of travel at the prevailing speed should precede diverging tapers.

(f) Diverging and merging tapers should be designed to encourage low relative speed manoeuvres.

3.7.3 Taper Length

The minimum lengths of pavement taper for diverging and merging movements can be computed by the formula:

$$Td = \frac{V}{3.6} \times \frac{Yd}{0.9}$$

$$Tm = \frac{V}{3.6} \times \frac{Ym}{0.6}$$

Where,

Td - Min length of pavement taper for diverging movements (m)
Tm - Min length of pavement taper for merging movements (m)
Yd - Lateral deflection of diverging traffic (m)
Yd - Lateral deflection of merging traffic (m)

Various types of tapers which may be used are shown in **FIGURE 3.12.**

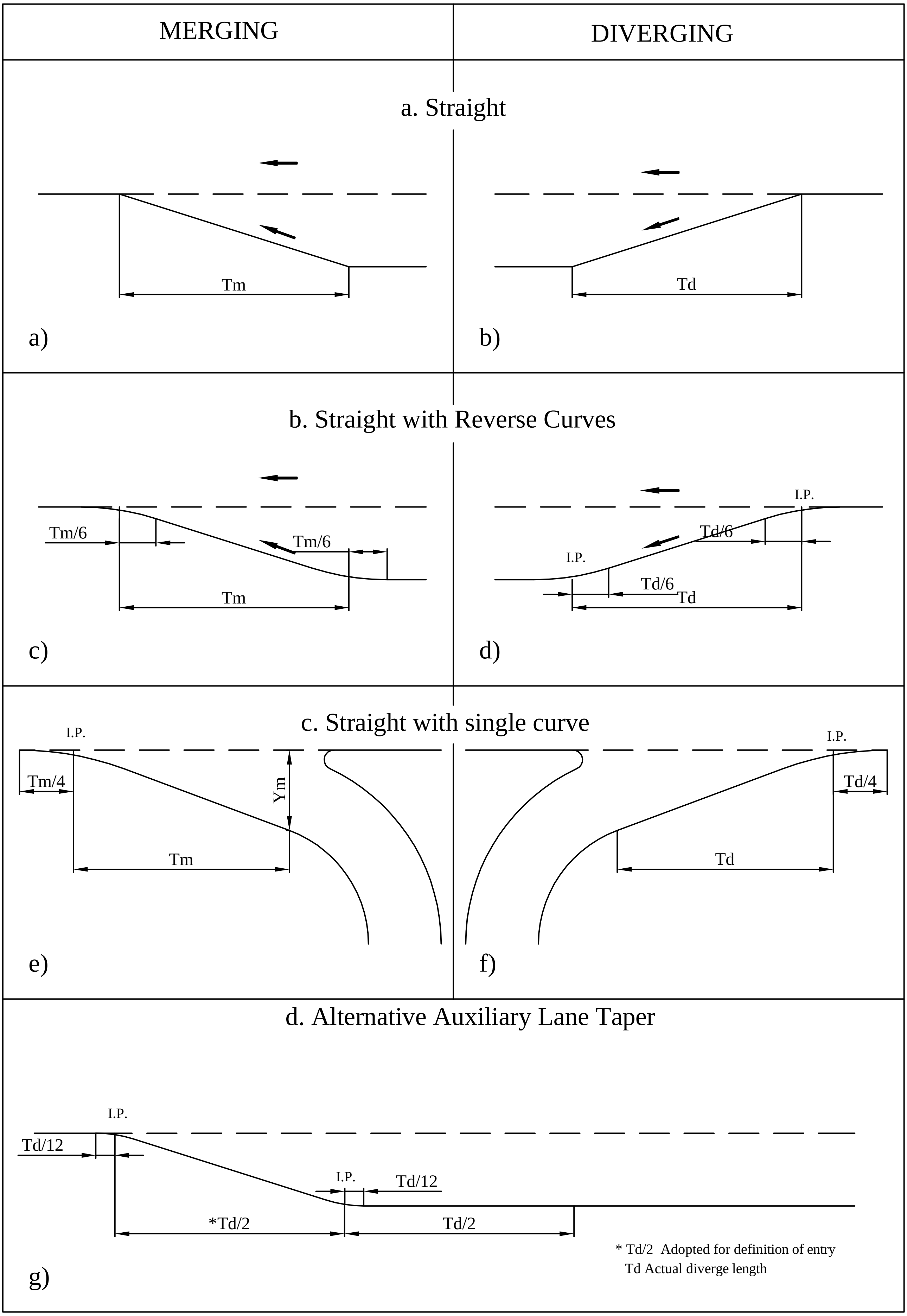


Figure 3.12: Types of Taper

Scale : Not to scale

3.8 Auxiliary Lanes

3.8.1 Deceleration Lanes

Left-turn deceleration movements should be separated from the through traffic stream. This may be done by providing in the left-turn approach a length of parallel by a diverge taper (Td). The combined length should be equal to the distance required to decelerate from the approach speed of the through road to the design speed of the left-turn. Lengths of deceleration lanes are as shown in **TABLE 3.9.**

The ratio from **TABLE 3.10** multiplied by the length from **TABLE 3.9** gives the length of deceleration lane on grade.

In urban areas, it is desirable that traffic using the left-turn should flow continuously. If calculation indicates that a queue would form at the STOP line, a length of parallel lane long enough for the left-turn vehicles to by-pass the end of the queue should be provided. **FIGURE 3.13** illustrates these principles.

Design Speed of Approach Road (km/h)	Length* of Deceleration Lane – (m) (including length of tapered approach) Where design speed of exit curve (km/h) is.						
	0**	20	30	40	50	60	80
40	45	40	32	–	–	–	–
50	60	54	46	32	–	–	–
60	80	74	64	50	28	–	–
80	120	112	104	94	82	64	–
100	170	162	154	144	132	118	80

*Length for level grade
(See Table 3.10 for Grade Correction)

**Length frequired when a vehicle decelerates to zero speed.

NOTE: Where the length of deceleration lane shown is less than the standard taper Td, Td should not be reduced.

TABLE 3.9 : LENGTH OF DECELERATION LANES

Grade	Ratio of Length on Grade to Length on Level	
	Upgrade	Downgrade
0 – 2%	1.0	1.0
3 – 4%	0.9	1.2
5 – 6%	0.8	1.35

TABLE 3.10 : CORRECTION FOR GRADE

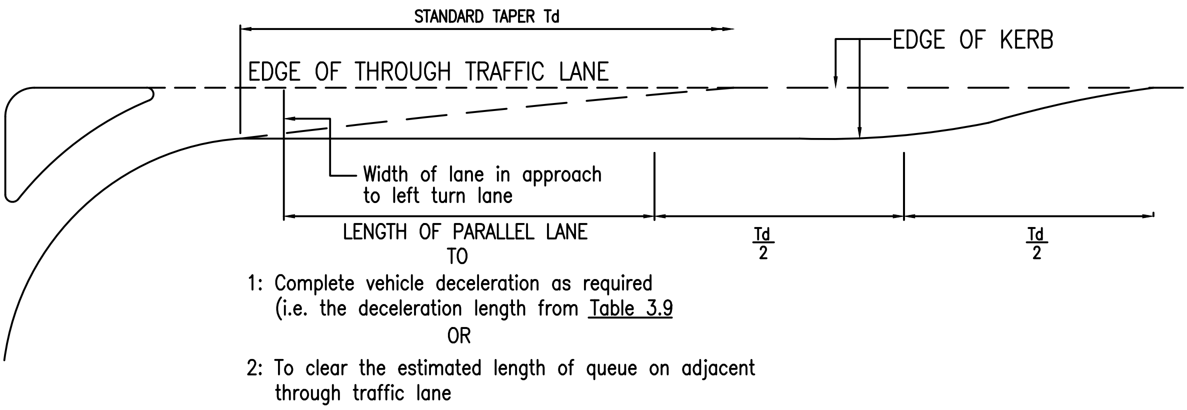


FIGURE 3.13 : TREATMENT IN APPROACH TO LEFT-TURNS

3.8.2 Acceleration Lanes

In urban areas where the through and left-turn movements are expected to flow concurrently, there should be an area which enables the two streams of traffic to merge at a small angle. When the volume of merging traffic is low, or where traffic signals are installed, this may be provided by a merging taper of length Tm at the exit of the left-turn.

The length of an acceleration lane is governed by the acceleration requirements from a stop or turning speed to the speed of the through road being entered. The acceleration should occur wholly within the lane. An acceleration lane has two basic design requirements;

(a) Acceleration length
(b) Merge length

The values for the minimum length of an acceleration lane are based on the greater of;

(a) The length required to accelerate from the turning speed to the design speed of road being entered
(b) The distance travelled in four seconds by the driver using an acceleration lane plus the required merge length

Where the volume of merging traffic is high and signals are not provided, a driver reaching the exit to the left-turn lane may not find any gap immediately available in the through traffic stream to permit merging. He should therefore be able to continue on a route parallel to the through traffic until a merging opportunity occurs or until he adjusts his speed to create an opportunity to merge. In such cases, a length of parallel acceleration lane together with the merging taper, Tm, should be considered. The combined length should be equal to the distance required for a vehicle to accelerate from the design speed of the left-turn to the design speed of the through road.

Lengths of acceleration lane are as shown in **TABLE 3.11**. If necessary, a correction for grade as shown in **TABLE 3.12** should be applied.

Design Speed Of Road Being Entered (km/h)	Length* Of Acceleration Lane – (m) (Including Length Of Pavement Taper) Where design speed of exit curve (km/h) is.						
	0**	20	30	40	50	60	80
40	65	45	35	–	–	–	–
50	95	75	60	40	–	–	–
60	135	120	100	75	40	–	–
80	230	215	200	180	145	100	–
100	330	315	295	275	250	205	100

*Length for level grade
(For grade correction see Table 3.12)

**Length required when a vehicle accelerates from zero speed.

NOTE: Where the length of acceleration lane shown is less than the standard taper Tm, Tm should not be reduced.

TABLE 3.11 : LENGTH OF ACCELERATION LANES

Design Speed Of Highway (km/h)	Ratio Of Length Of Grade To Length Of Level* For:					
	Design Speed Of Turning Roadway Curve (km/h)					
	Stop	20	40	60	80	All Speeds
	3 OR 4% Upgrade					3 TO 4% Downgrade
40	1.3	1.3	–	–	–	0.7
50	1.3	1.3	1.3	–	–	0.7
60	1.3	1.3	1.3	–	–	0.7
80	1.3	1.3	1.4	1.4	–	0.65
100	1.3	1.4	1.4	1.5	1.6	0.6
	5 OR 6% Upgrade					5 TO 6% Downgrade
40	1.4	1.4	–	–	–	0.6
50	1.4	1.5	1.5	–	–	0.6
60	1.5	1.5	1.5	–	–	0.6
80	1.5	1.5	1.6	1.9	–	0.55
100	1.6	1.7	1.8	2.2	2.5	0.5

* Ratio from this table multiplied by length in Table 3.11 gives length of speed–change lane on grade.
Figure 3.14 illustrates the application of an acceleration lane and or merging taper to a left–turn lane.

TABLE 3.12 : CORRECTION FOR GRADE

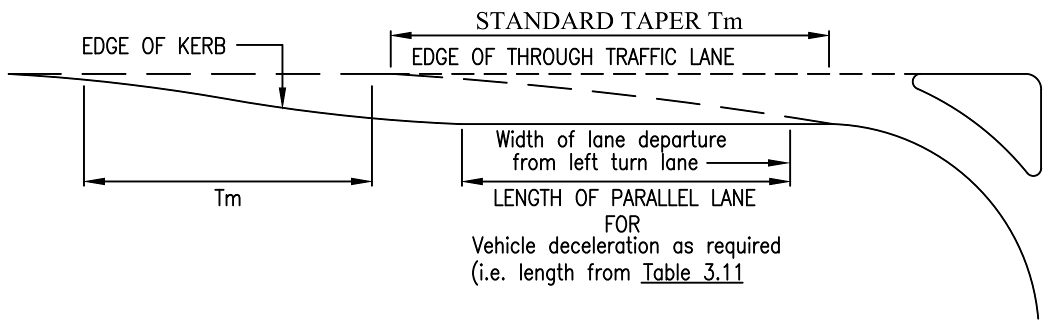


FIGURE 3.14 : TREATMENT FOR ACCELERATION LANE TAPER

3.8.3 Width of Auxiliary Lanes

Widths of auxiliary lanes shall desirably be 3.5m but shall not be less than 3.0m.

3.9 Island and Openings

3.9.1 General

(a) An island is a defined area between traffic lanes used for control of vehicle movements. Island also provides an area for pedestrian refuge and traffic control devices.

(b) Visibility to approaching traffic, both day and night, is an essential factor in any island location.

(c) There are two types of islands;

(i) Pedestrian Island

Pedestrian island provides refuge for people waiting for public transport or crossing wide streets.

ii) Traffic islands

Traffic islands are divisional or channelisation islands - refer to the latest NTJ18/97 for pedestrian island. Only traffic islands will be considered here.

3.9.2 Traffic Islands

(a) Traffic islands serve three (3) primary functions;

(i) Channelisation: to control and direct traffic movement, usually turning
(ii) Division: to divide opposing or same direction traffic streams
(iii) Refuge: to provide refuge for pedestrians

(b) Traffic islands are used to;

(i) Separate opposing streams of traffic;
(ii) Guide traffic away from and past fixed obstructions and other hazardous points;
(iii) Reduce the area of conflicts and control the angles at which conflicts occur;
(iv) Provide shelter for turning or crossing vehicles;
(v) Prohibit undesirable or unnecessary traffic movements;
(vi) Control speed;
(vii) Separate through and turning movements as well as define their respective alignments; and
(viii) Provide for and protect traffic control devices.

(c) Traffic islands may be defined by pavement markings, kerbs or a combination of both. Large islands in rural areas may be constructed without kerbs or with kerbs only at the points where separate roadways converge or diverge. The following design aspects should be considered for shape, location and size of islands:

(i) They should be located and designed so that the proper line of travel is obvious and any changes in direction are gradual and smooth.

(ii) The approach end of any island should be offset from the edge of the adjacent traffic lanes and preceded by appropriate pavement markings such as chevron markings. This approach offset should be a minimum of 1.0m. The sides of islands should also be offset from adjacent traffic lanes by 0.3m or 0.6m where semi-mountable or mountable kerbs are used. For roads with design speeds exceeding 80km/hr the offset should be increased to 0.6m and 1.2m.

(iii) Except for very large rural islands, islands should be delineated with semi-mountable type kerb. Where pedestrian refuge is being provided, barrier kerb should be used.

(iv) In urban areas, raised islands should be of an area not less than approximately 8m^2. A smaller area may be adopted where traffic signals need protection. Islands in rural areas should desirably have a minimum area of 50m^2. Smallest kerbed corner island area 5m^2 for urban and 7m^2 for rural. However, if possible, 9m^2 is preferable for both. In rural areas without any street lighting, raised islands should not be used. Instead, pavement markings should be used.

(v) Where an island has to provide for stop lines, traffic signals and pedestrian crossings, the side of the island should be a minimum of 6m long with a minimum width of 1.2m at the point where the signal pedestal is erected.

(vi) **FIGURE 3.15** shows desirable layouts for directional islands.

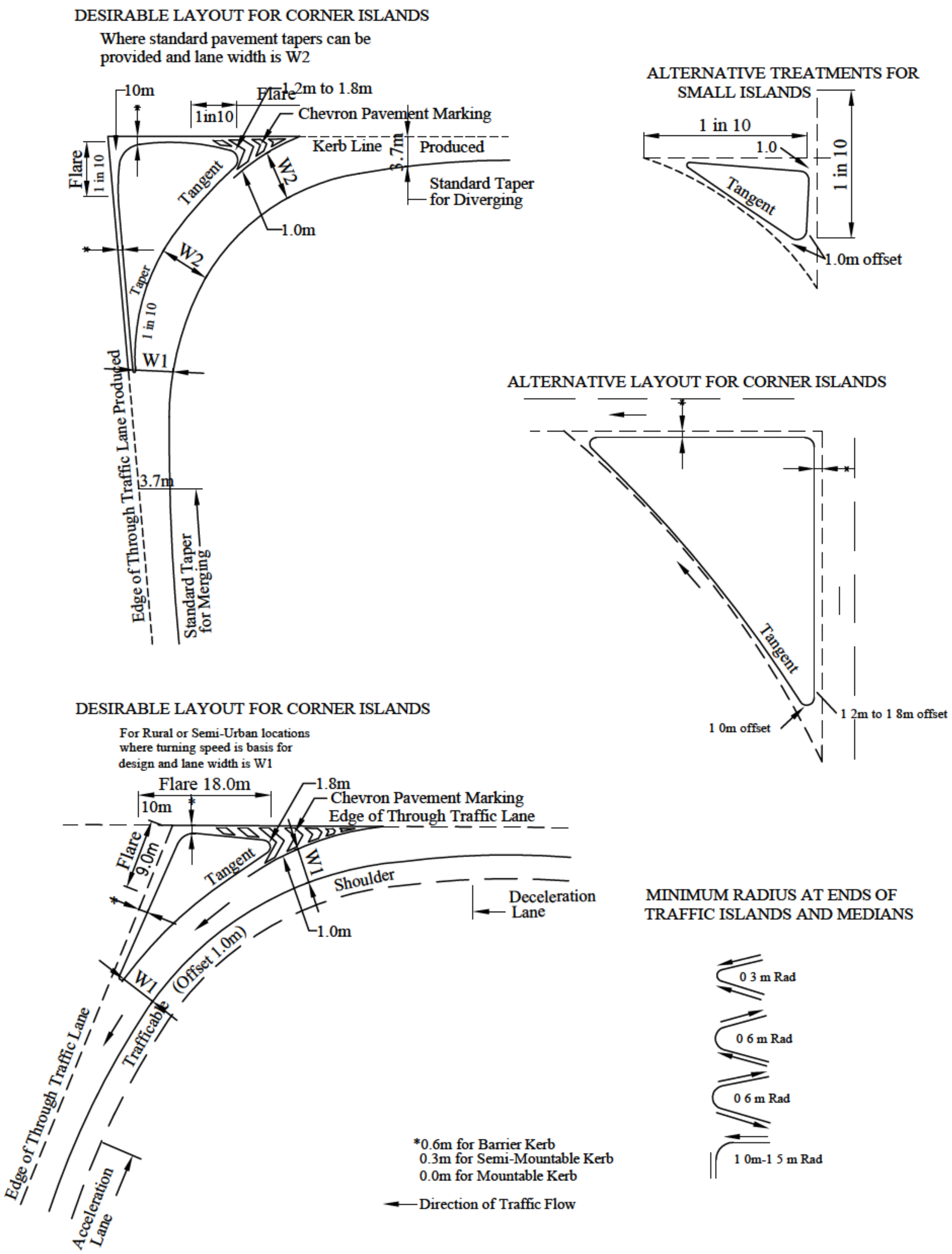


Figure 3.15: Directional Island

Scale : Not to scale

3.9.3 Median Islands

Medians are used to separate opposing traffic streams, provide refuge for pedestrians and reduce the number of points of crossing conflict along a road.

The following design aspects of medians should be considered:

(a) The approach end of each median island should be set back from the right hand edge of the adjacent traffic lane by at least 0.3m and preferably 0.5m to:
  (i) Reduce the probability of collision with the island and;
  (ii) Relieve the optical illusion of a construction in the lane at the start of the island.

(b) Unless stopping sight distance is available at its approach end, a median should not commence on or beyond a crest. Medians should not also begin on the arc or a horizontal curve but at or before the first tangent point or 30m or more beyond the second tangent point.

(c) A length of painted median should precede the approach end of the median so that the approaching driver will notice the obstruction ahead. On high speed roads, any short length of kerbed median should be offset from the delineated through traffic lane by approximately 0.5m (See **FIGURE 3.16**). If median is narrower than 2m, a length of barrier line may be used in the approach (See **FIGURE 3.17**) instead of the painted median.

(d) The first median end encountered by approaching drivers should display a reflectorised KEEP LEFT (Sign RM.4) sign. Where the island is less than 1.2m wide at the approach end, this sign should be placed up to 6m away from the end to protect it from approaching traffic.

(e) Where a median island is placed on a side road, the end adjacent to the through road should be as narrow as practicably possible and set back 0.6m behind the prolongation of the kerb line of the through road when:

  (i) No pedestrian crossing is provided or
  (ii) A minimum length of 2m of median can be provided between the pedestrian crossing and the through road.

  If (ii) is not possible, the end of the median should be terminated at the pedestrian crossings.

(f) Where a median would alter the number of lanes, the treatment to be adopted should follow that as shown in **FIGURE 3.18.**

(g) Semi-mountable kerbs should be used.

(h) Where kerbs cannot be used, painted medians should be used as shown in **FIGURE 3.19.**

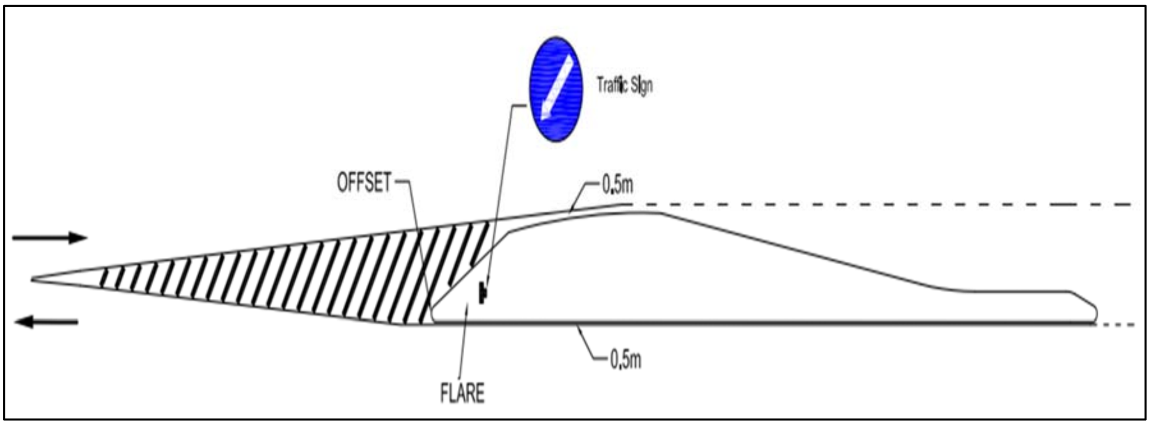


***FIGURE 3.16: OFFSET TO MEDIAN ISLAND**

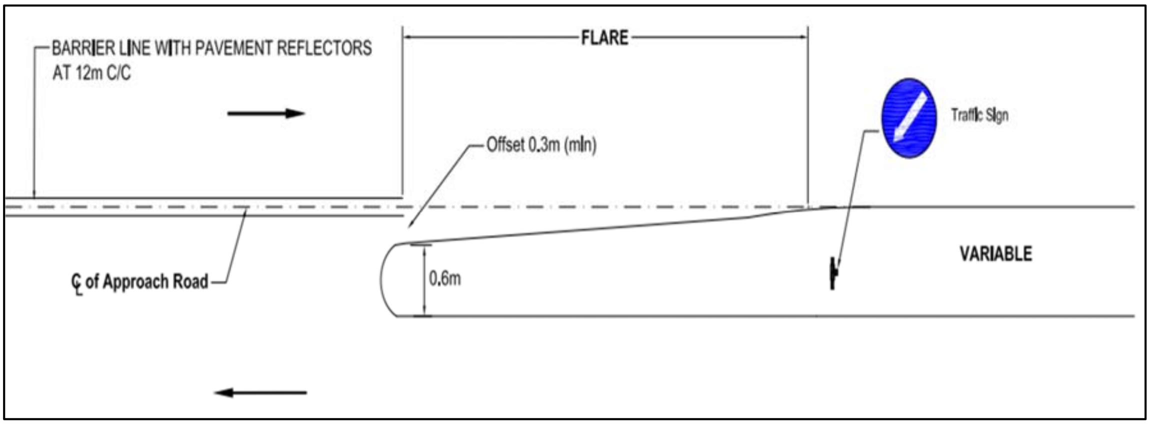
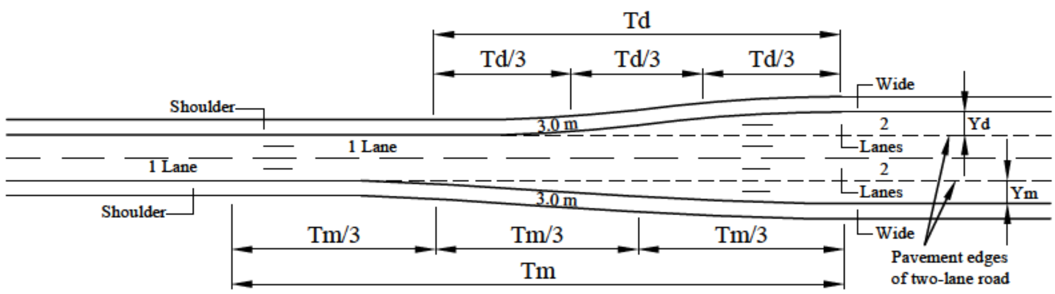
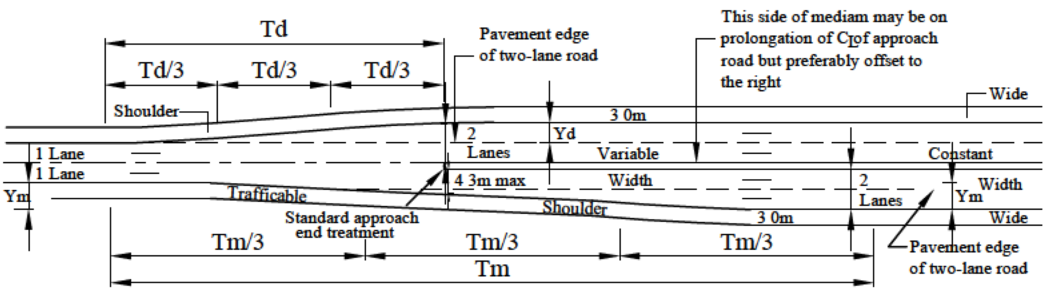


***FIGURE 3.17: END TREATMENT FOR NARROW MEDIAN**

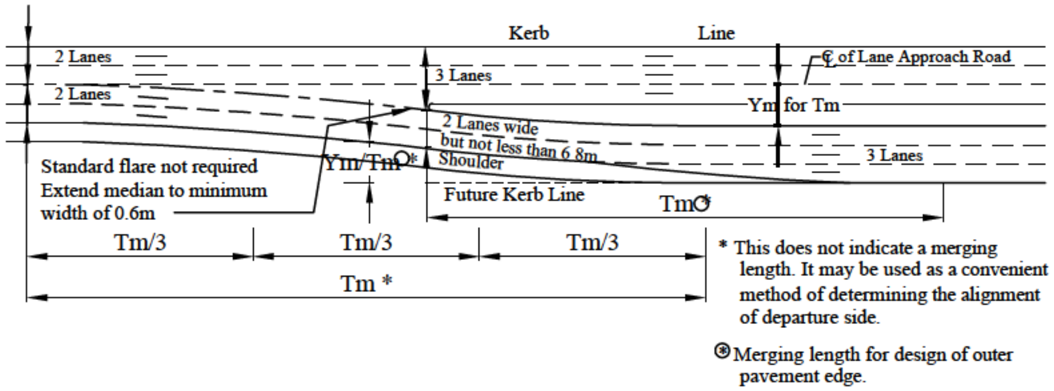
DEVELOPMENT OF A FOUR LANE ROAD FROM A TWO LANE ROAD WHEN WIDENING IS ON EACH SIDE



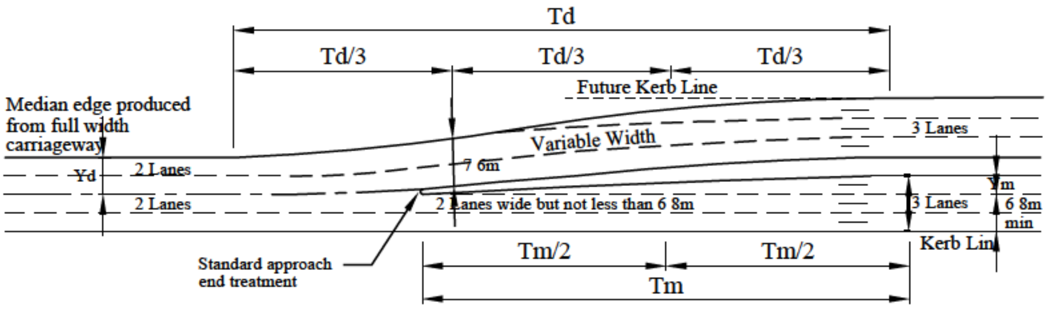
DEVELOPMENT OF A DIVIDED CARRIAGEWAY FROM A TWO LANE ROAD WHEN WIDENING IS ON EACH SIDE



DEVELOPMENT OF A DIVIDED CARRIAGEWAY FROM A FOUR LANE ROAD WHEN WIDENING IS ON DEPARTURE SIDE



DEVELOPMENT OF A DIVIDED CARRIAGEWAY FROM A FOUR LANE ROAD WHEN WIDENING IS ON APPROACH SIDE



APPROXIMATE RADIUS FOR PAVEMENT EDGES ARE

ARC LENGTH	Td/3	Tm/3	Td/3	Tm/3
RADIUS	0 068 Td V	0 102 Tm V	0 113 Td V	0 170 Tm V

Figure 3.18: Median Terminal Treatments

Scale : Not to scale

3.9.4 Median Openings

Where openings are provided in medians, the treatment of the median and ends should be in accordance with those shown in **FIGURE 3.20** depending on the width of the median.

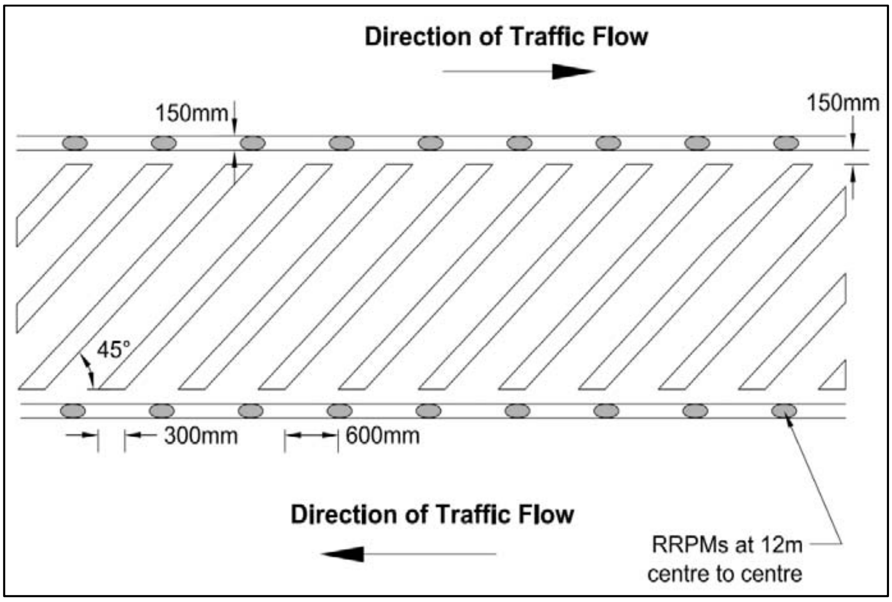


***FIGURE 3.19: PAINTED ISLAND**

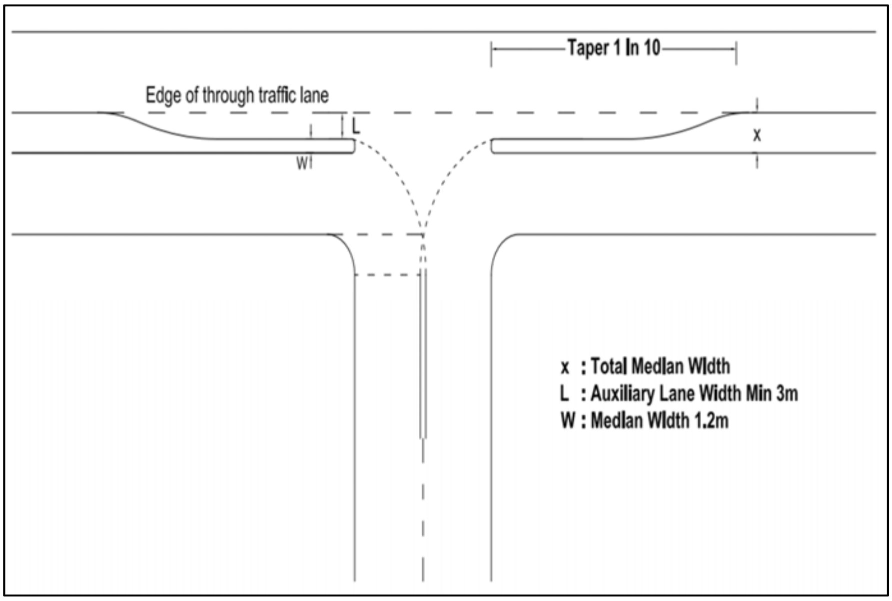


***FIGURE 3.20: MEDIAN OPENING**

3.9.5 Outer Separators

Outer separators are used to separate the through traffic lanes from service roads. They should be as wide as possible with a desirable width of 5.0m. Treatment for outer separator openings are as shown in **FIGURE 3.21**.

3.10 Widening of Major Road

Widening of the major road to provide space for the central island should be on a straight portion, done symmetrically around the centreline of the road and on a curve portion, done to the inside of the centreline. The same applies where widening of a median is required.

The length of the widening shall be determined by the formula:

$$L\omega = V\sqrt{\omega\ max}$$

Where

$L\omega$ = length of the widening in m

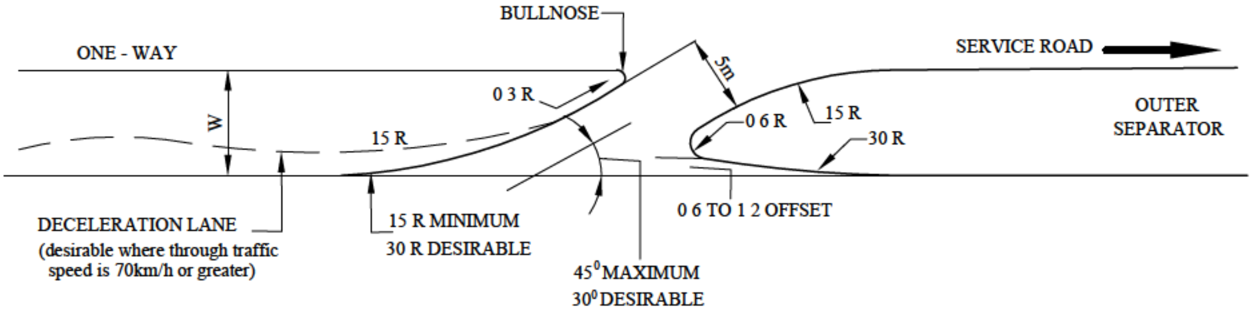
V = design speed of major road in km/hr.

$\omega\ max$ = larger of the two parts of the widening (m) on either side of the centerline,

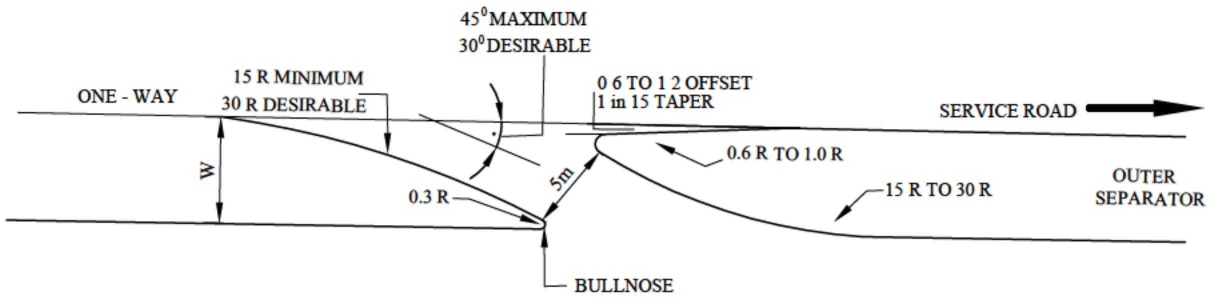
i.e., (W max = 1/2 of total widening $W\omega$ in the symmetrical case and $W\omega\ max$ = $W\omega$ in the case of one sided widening).

The outer edges of the carriageway shall be widened over the same length as the central widening even if the required widening is different from the central widening due to changes in lane width.

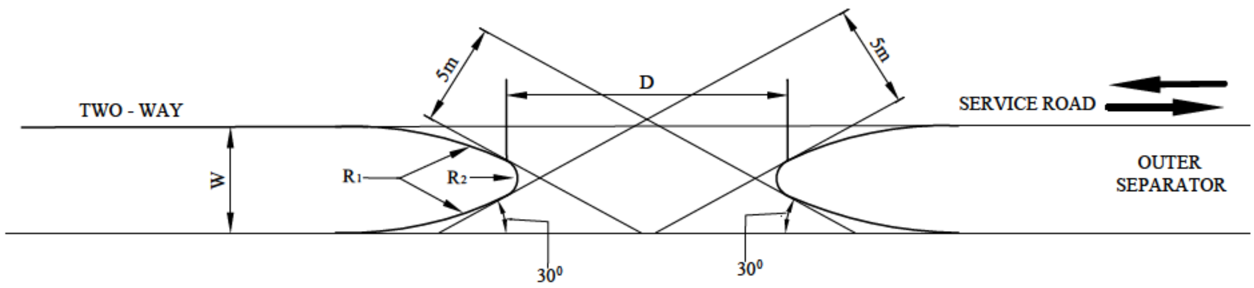
The widening of both inner and outer edges shall be carried out to a smooth continuous alignment composed of the usual alignment elements. S or Reverse curves composing of two circle arcs will in most cases provide a curvature which has acceptable dynamic and optical properties and is recommended. S-curve may when the road is on a curve produce adverse curvatures, in which case the length of widening should be increased or an alternative curvature selected. **FIGURE 3.22** shows aspects of widening of the major road by S-curves.



ONE-WAY ENTRANCE TO A SERVICE ROAD



ONE-WAY EXIT FROM A SERVICE ROAD



TWO-WAY OPENING BETWEEN THROUGH ROAD AND SERVICE ROADS

Radius		
Outer Separator Width W (m)	R_1 (m)	R_2 (m)
5-10	15	0.2W
11-15	23	0.2W
16-25	30	0.2W

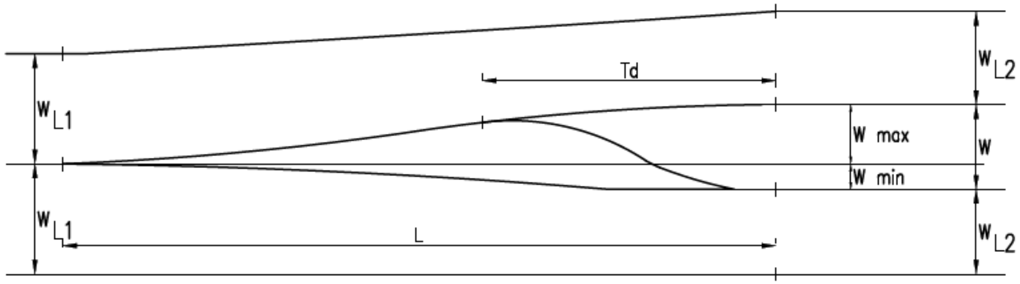
Note:-
Two way opening only applicable
for low speed ie 60km/h

Figure 3.21: Outer Separator Opening

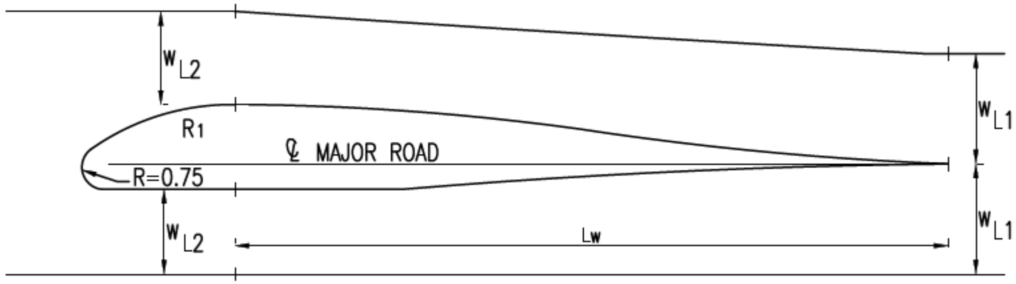
Scale : Not to scale

A) WIDENING OF MAJOR ROAD

1) With Right Turn Lane



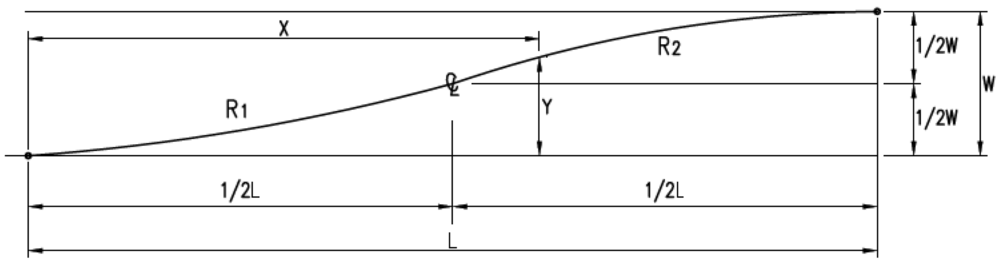
2) Without Right Turn Lane



L = Length of widening section = $V\sqrt{W\ max.}$ (m)
V = Design speed (km/h)
Wmax = Larger part of the total widening to either side of the centre

W_{L1} and W_{L2} = Lane width of through lane
Td = Length of taper
R_1 = Turn in radius (see figure. 3–23A and B)

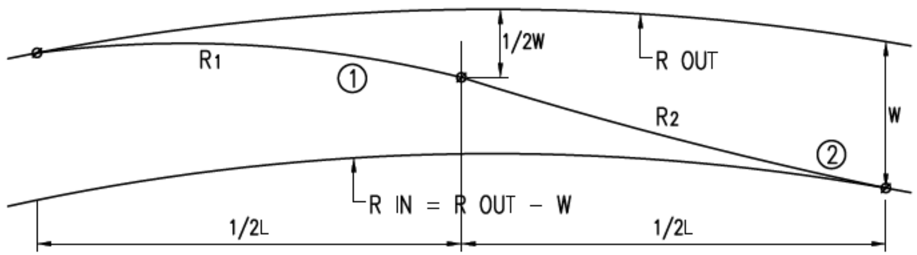
B) RADII OF S–CURVE TANGENT CENTRELINE



a) $R1 = R2 = \dfrac{L}{4W} + \dfrac{W}{4}$ b) $Y = \dfrac{Wx^2}{L^2}\left(3 - \dfrac{2x}{L}\right)$

where R1 and R2 = Radii of the S curve in m.
L = Length of widening section in m.
W = Widening in m.
x = Distance from straight of curve.
y = Lateral offset from parallel tangent.

C) RADII OF S–CURVE CENTRELINE ON CURVE



$$R1 = \frac{1}{\dfrac{1}{R\ OUT} + \dfrac{4W}{L^2}}$$

$$R2 = \frac{1}{\left(\dfrac{1}{R\ OUT-W}\right) - \left(\dfrac{4W}{L^2}\right)}$$

R2 is negetive when $L < 2\sqrt{WR_{IN}}$ in which case curve 2 is as shown

R2 is positive when $L < 2\sqrt{WR_{IN}}$ in which case curve 2 turn in the same direction as curve 1

R2 is infinite when $L < 2\sqrt{WR_{IN}}$ in which case curve 2 is thr straight tangent to curve 1 and the curve with radius R_{IN}

Figure 3.22 : Widening By S-Curves

Scale : Not to scale

3.11 Minor Road Treatment

3.11.1 Types of Treatments

Treatments on minor road for better traffic control benefits not only the minor road but also the major road. Quick departure of traffic from the major road and smooth merging into it helps to maintain a smooth and safe traffic flow on the major road. There are basically 3 degrees of treatments; description of which is given in Section 1.2.

The type of the minor road treatments should be selected according to the class of the road and that of the major road to which it is connected, as shown in **TABLE 3.13**.

Guide islands in the centre should be provided for flared or higher treatments.

3.11.2 Guide Islands

Guide islands are placed at the centre of the minor road at intersections to define the movements of turning traffic and to control the speed of turning and crossing vehicles. They also provide space for traffic control devices and refuge for pedestrians.

Guide island shall be designed to the following: -

(a) The shape and location of the island shall be such that it can be passed by the design vehicle both entering and leaving the major road.
(b) The front end of the island is shaped by the inner rear wheel paths of the design vehicle while the rear end is shaped to guide the approaching traffic.
(c) The largest width of the island shall be between 3.0 to 5.0m while the length shall be between 20 to 35m.
(d) The island shall be curved, preferably semi-mountable and off betted 0.3m.
(e) Mandatory keep left signs shall be placed at both ends of the island. Warning or information signs can be placed if they do not affect the visibility of the vehicles.

FIGURES 3.23A and **3.23B** gives the standard design of guide islands which are to be used.

*TABLE 3.13: MINOR ROAD TREATMENT

a) Rural Area

Minor Road					
Highway	Primary Road	Secondary Road	Minor Road		
C	C	F	F/N	Highway	Major Road
	C/F	F	N	Primary Road	
		F/N	N	Secondary Road	
			N	Minor Road	

* Normally at-grade intersection should not be adopted

b) Urban Area

Minor Road				
Arterial	Collector	Local Street		
C	C/F	F/N	Arterial	Major Road
	F	N	Collector	
		N	Local Street	

Where two alternatives are given, traffic volume should be taken into account for the selection

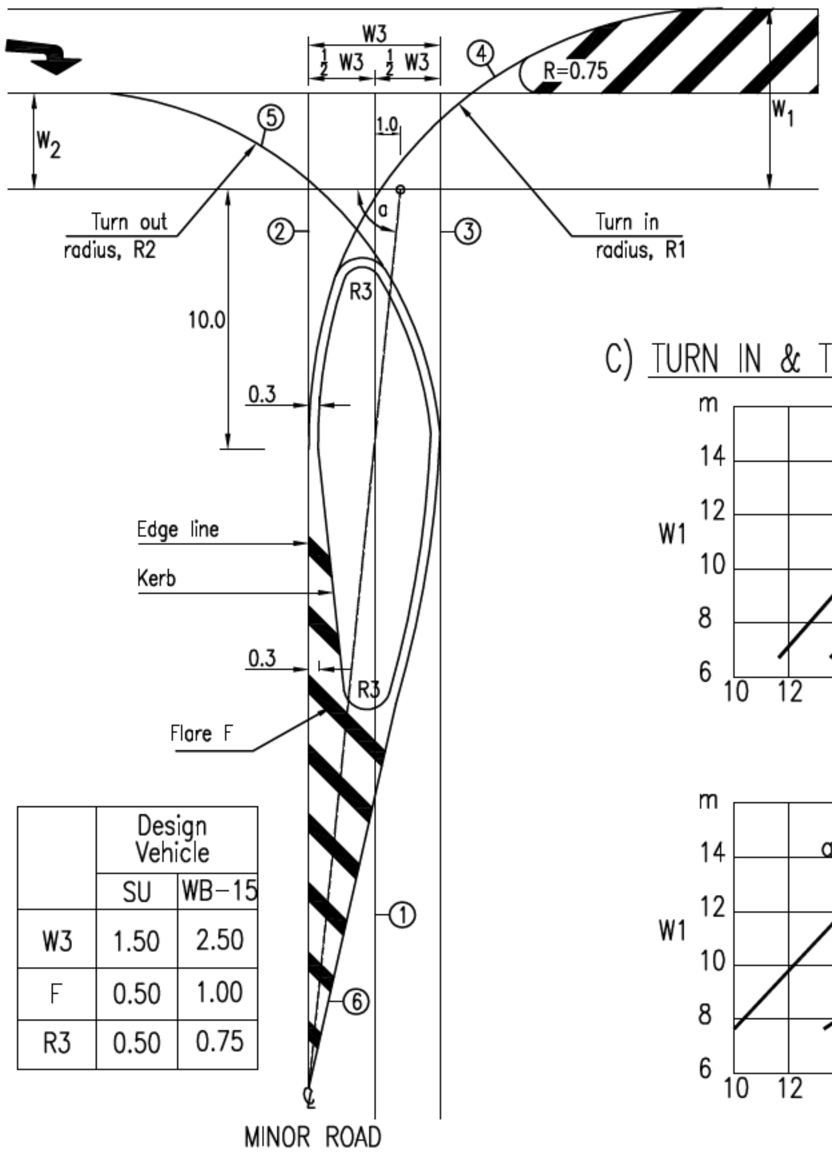
C = Channelised

F = Flared

N =NoTreatment

(INTERSECTING ANGLE 70° ≤ a ≤ 100°)

A) LAYOUT

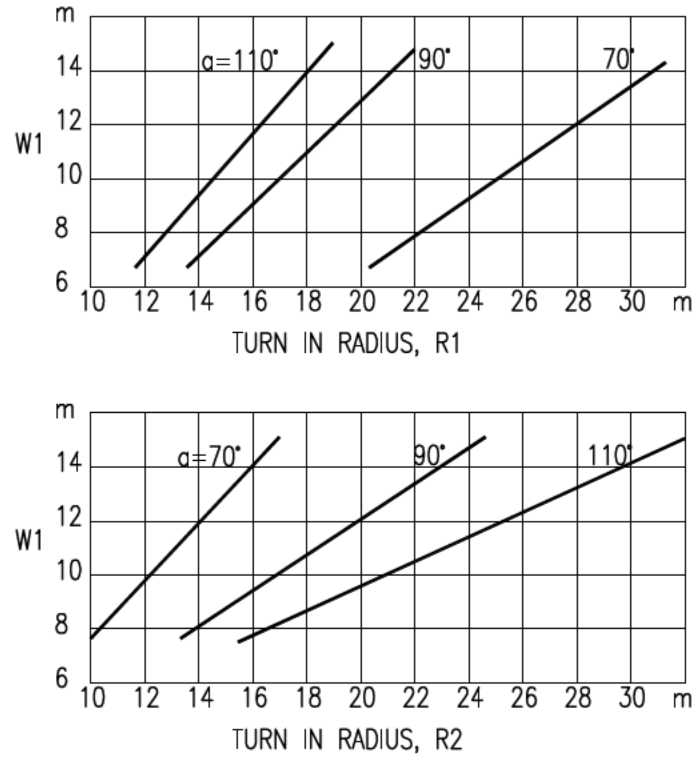


	Design Vehicle	
	SU	WB−15
W3	1.50	2.50
F	0.50	1.00
R3	0.50	0.75

B) TURN IN & TURN OUT RADII, SU–VEHICLES

a	70°	80°	90°	100°	110°
R1	16	13	10	10	10
a	12	13	14	17	21

C) TURN IN & TURN OUT RADII, WB–15–VEHICLES



PROCEDURE & NOTES

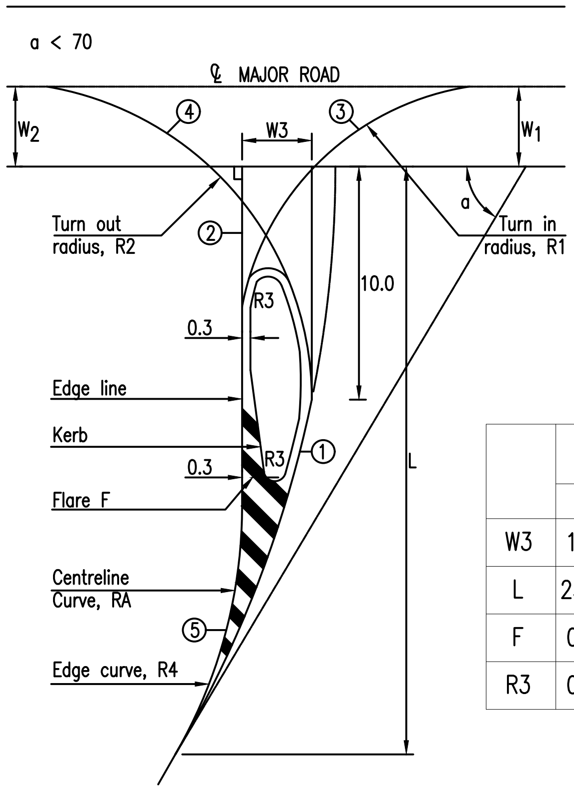
1. Establish line ① cutting the nearer edge of the through lane and the centreline of the minor road as indicated
2. Establish line ② and ③ parallel to line ① at the distance 1/2W3
3. Draw circle ④ with radius R1 (rounded to whole m) tangent to line ② and the nearer edge of the offside through lane.
4. Draw circle ⑤ with radius R2 (rounded to whole m) tangent to line ③ and the nearer edge of the right turn lane.
5. Extend line ② to intersect the centraline and draw line ⑥ through the Intersecting point tangent to circle ⑤
6. The location of the rear end of the kerbed part the island is determined and the kerb line is set 0.3m behind of circle ④ circle ⑤ and ⑥ and tangent between the and circle and the kerb line behind circle ④ the front and rear ends are rounded by circle with radius R3
7. All dimension are in metres

Figure 3.23A: Standard Design of Guide Island

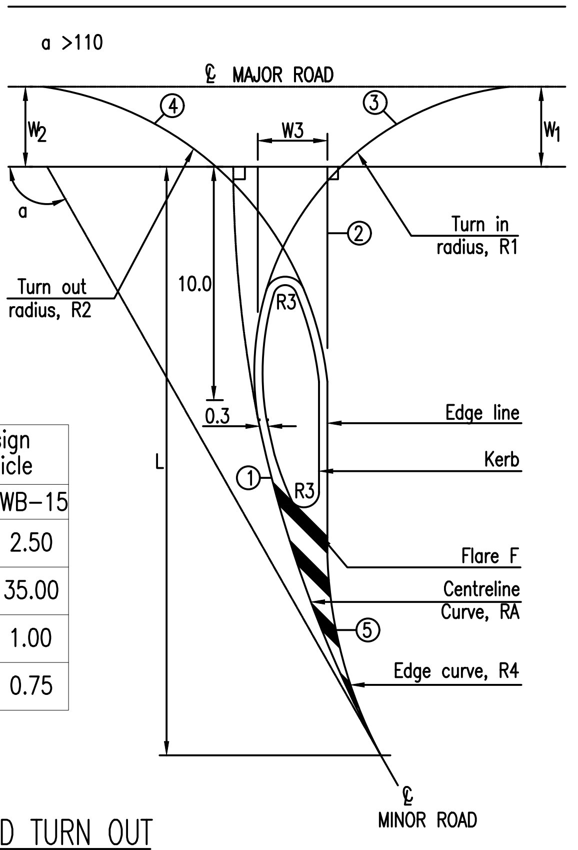
Scale : Not to scale

(INTERSECTING ANGLE a < 70° and a > 100°)

A) LAYOUT



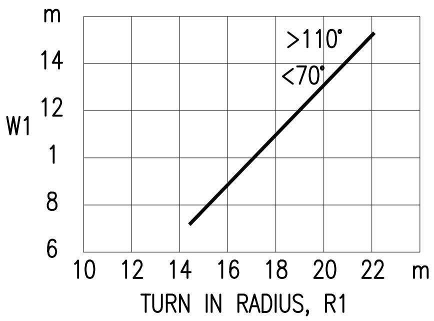
	Design Vehicle	
	SU	WB–15
W3	1.50	2.50
L	25.00	35.00
F	0.50	1.00
R3	0.50	0.75

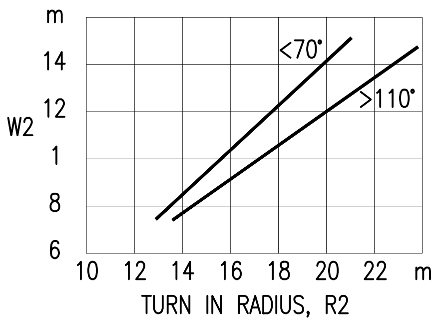


A) TURN IN AND TURN OUT RADII SU–VEHICLES

a	<70°	>110°
R1	11	12
R2	13	13–15

A) TURN IN AND TURN OUT RADII WB–15





PROCEDURE & NOTES

1. Change the centre line by use of curve 1 with radius RA > 50m. The curve should be perpendicular to the edge of the nearer through line.
2. Establish line 2 perpendicular to the edge of the through lane through a point at the distance W3 from the point on curve 1 which is 10m from the edge of through lane.
3. Draw circle 3 with radius R1 (rounded to whole m) tangent to line 2 or circle 1 and the nearer edge of the offside through lane
4. Draw circle 4 with radius R2 (rounded to whole m) tangent to circle 1 or line 2 and the nearer edge of the right turn lane.
5. Draw circle 5 tangent to the centreline within the distance L from the nearer edge the major road and tangent to line 2 Select the radius R4 as large as possible.
6. The kerbed part of the island is now established as described on Fig. 3–23A
7. All dimension Are in metres.

Figure 3.23B: Standard Design of Guide Island

Scale : Not to scale

Kerbed islands are sometimes difficult to see at night because of the glare from oncoming headlights or from distant luminaires or roadside businesses. Accordingly, where kerbed island are used, the intersection should have fixed-source lighting or appropriate delineation such as kerb-top reflectors.

Under certain conditions, painted, flush medians and island or traversable type medians may be preferable to the raised kerbed type islands. These conditions include the following: lightly developed areas that will not be considered for access management; intersections where approach speeds are relatively high; areas where there is little pedestrian traffic; areas where fixed-source lighting is not provided; median or corner islands where signals, signs, or luminaire supports are not needed; and areas where extensive development exists along a street and may demand left-turn lanes into many entrances.

Painted islands may be used at the traveled way edge. At some intersections, both kerbed and painted islands may be desirable. All pavement markings should be reflectorized. The use of thermoplastic stripping, raised dots, spaced and raised retro-reflective markers, and other form of long-life markings also may be desirable.

In general, introducing kerbed divisional islands at isolated intersections on high-speed highways is undesirable unless special attention is directed to providing high visibility for the islands. Kerbed divisional islands introduced at isolated intersections on high-speed highways should be 30 meters or more in length. When situated in the vicinity of a high point in the roadway profile or at or near the beginning of a horizontal curve, the approach end of the kerbed island should be extended so as to be clearly visible to approaching drivers.

3.11.3 Widening of the Minor Road

The width of the carriageway shall remain unchanged up to the corners of the intersection if no guide islands are present.

Where guide islands are present, the entry lane shall have a minimum width of 3.5m and the exit lane a minimum width of 4.5m past the island.

Provision of right-turn lanes and examination of the number needed are usually emphasized on the major road. However, increasing the number of right turning lane on minor crossroad especially at signalised intersections also profits the major road. Right turning vehicles departing from two lanes can clear the intersection in a shorter time.

3.11.4 Left Turn Lane on Minor Roads

An auxiliary lane reserved for left turning traffic may be added to the approach of the minor road if the left turning traffic exceeds 50% of the capacity for that movement, or where there are no space constraints.

The design of the left turn lanes shall follow the guidelines set out in Section 3.6.3.

3.12 Shoulders

The roadway width at intersections includes shoulders or equivalent lateral clearance outside the edges of pavement. A shoulder at intersections is provided for the same main reasons as that for the open highways. It is an area adjacent to the driving lane where a driver can make an emergency stop for a disabled vehicle. It is also used for travel of emergency vehicles. Due to improper vehicle operations, shoulders at intersections deteriorate at a faster rate than on open highways. Edges of pavement drop-off and gravel strewn onto the pavement are main concerns which require frequent inspection and maintenance. This section deals with shoulders treatment at intersections designed to minimize these concerns.

Shoulder widths shall in general remain unchanged in the intersection area but may be reduced to 2.0m along the deceleration and turning lanes.

At intersections the required shoulders width varies from a minimum of 0.5m to that of an open road cross-section. Where two roadways of different operational characteristics and functions intersect, the shoulder's width at intersection normally varies and serves as a transition from a wide shoulder at the main road to a narrow one at the side road.

Where the main road is designed with auxiliary lanes the shoulder width on the near side of the intersection of road to side road exit is transitioned within the length of the edge of pavement curve.

For the far side road to highway entrance the shoulder width of the side road is applied along the edge of pavement curve and transitioned to the shoulder width of the highway within the 30 m recovery taper length.

A uniform intersection shoulder width is designed where the intersecting roads are nearly equal in importance and have identical shoulder width.

In the vicinity of the intersection where kerbed islands are present, the shoulder structure should be of a stable gravel, hard or sealed shoulder type. In general, kerbs should not be used along the outer carriageway edges.

3.12.1 Shoulder Treatment at Open Throat Intersections

The shoulder treatment at open throat intersections is divided into three types:

(a) Gravel Shoulders
(b) Paved Shoulders
(c) Concrete Kerb and Gutter

Each type of shoulder treatment can be applied at intersections with or without tapers or deceleration lanes.

Delineators may be used in conjunction with either gravel or paved shoulder treatment at intersections. Generally, the application of delineators is discouraged as they are often damaged or destroyed by turning vehicles and their effectiveness is greatly reduced. However, delineators may provide a guidance to the drivers exiting and entering the highway in locations with restricted visibility and during poor weather conditions.

The shoulder treatment at intersections must be evaluated and designed for each location based on existing and anticipated future conditions.

(a) Gravel Shoulders

The shoulder treatment at intersections is usually achieved by surfacing the shoulder with gravel. However, unstabilized shoulders generally undergo consolidation with time and the elevation of the shoulder at the pavement edge tends to become somewhat lower resulting in pavement drop-off. Also turning manoeuvres contribute to gravel spillage onto the pavement area. Regular maintenance is necessary to reduce the accident potential.

(b) Paved Shoulders

At intersections, a high percentage of drivers cut across the shoulder when turning left from the through road to the side road. Also, decelerating vehicles on gravel roads, particularly abruptly stopping vehicles, drag gravel onto the paved intersection area. In order to reduce pavement edge drop-off and gravel strewing onto the pavement, the paved shoulder treatment at intersections is considered an effective design feature.

(c) Concrete Kerb And Gutter

Where shoulder gravel spillage is attributed to, or is anticipated to be caused by intersection turning manoeuvres, concrete kerb and gutter may be applied. A mountable or semi-mountable type is preferred. The paved shoulder with concrete kerb and gutter is deemed to be the most effective design. It discourages drivers to deviate from the appropriate turning path onto the shoulder.

3.13 Crossfall and Surface Drainage

Crossfalls in the intersection area should be designed with regard to drainage, driving comfort and visibility.

In general, the crossfall of the through lanes of the major road shall remain unchanged through the intersection.

Crossfalls on auxiliary lanes may follow the crossfall of the adjoining through lane or fall to the opposite side as is required by drainage or side friction criteria. The algebraic difference of crossfalls of two adjoining lanes should not exceed 5 percent.

The crossfall of the minor road shall, towards the edge of the through lane, be the same as the gradient of the through lane. Where the major road is on a steep grade, this may create an adverse camber for turning vehicles. In such a situation, diverging lanes should be considered.

Superelevation of corner lanes in connection with triangular islands on the minor road shall in general not exceed 6 percent.

3.14 Weaving

3.14.1 Weaving for At-Grade Intersections

Weaving is defined as the crossing of streams of traffic moving in the same direction accomplished by merging and diverging. Weaving occurs in a number of situations like interchanges, intersection freeway sections and on at-grade all-purpose roads. A knowledge of the capacity of weaving section is, therefore, of considerable importance.

A weaving section is a road segment where the pattern of traffic entering and leaving at contiguous points of access results in vehicle paths crossing each other. Traffic through a weaving section consists of weaving traffic and non-weaving traffic. These two groups tend to separate themselves and this enables each class to be examined and analysed separately. Non-weaving traffic is treated in the usual way as a lane of road.

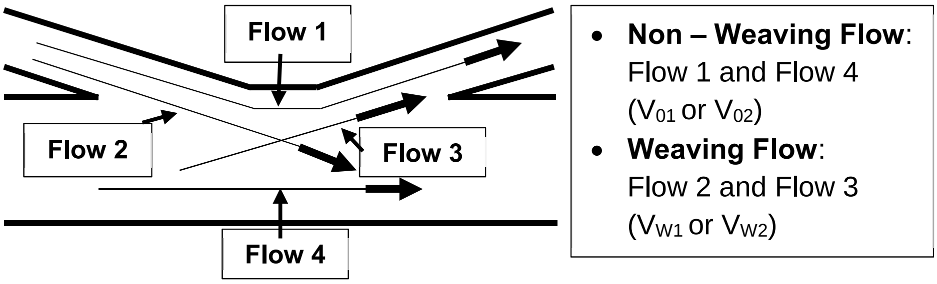


FIGURE 3.24: TERM USED WEAVING
Source: Design Manual for Roads and Bridges Volume 6 Section 2 Part 1 TD 22/06, Figure 2/9.

Since every vehicle in the weaving stream of traffic must cross an imaginary line connecting the noses of the entrance and exit forks, the total number of vehicles passing through the weaving section cannot exceed the capacity of a single lane. In order to accommodate such weaving movements, additional roadway width beyond that on the approaches is usually required. Also, as the weaving volume increases, longer lengths of weaving needed. Thus the weaving operation is independent on the length and width of weaving section.

The first step in the solution of a weaving section is to determine the weaving length required from the known weaving volume and the desired quality of flow from the **FIGURE 3.24** and **FIGURE 3.25**.

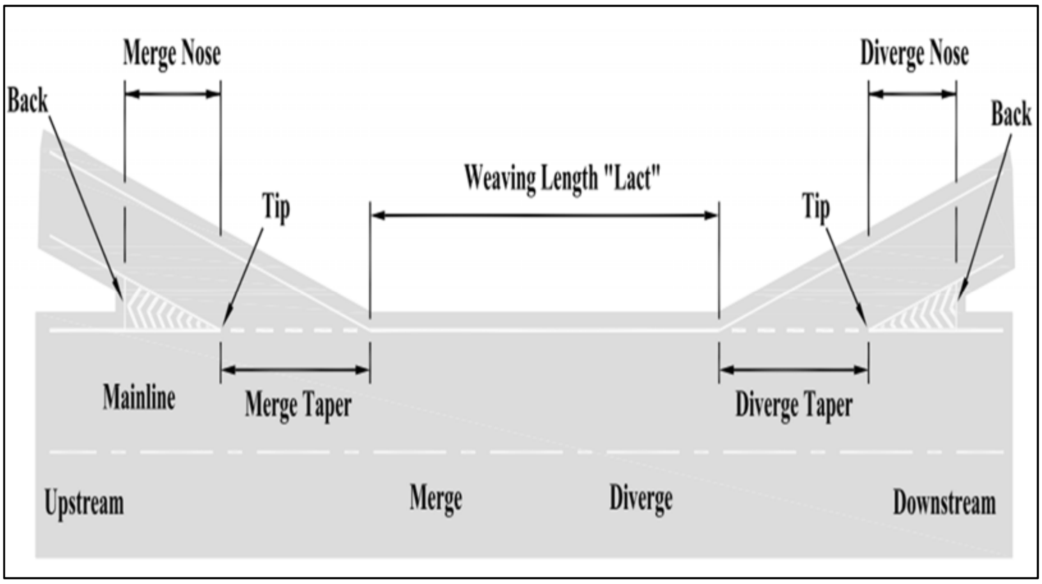


FIGURE 3.25: DEFINITION OF TERMS USED IN WEAVING AND MEASUREMENT OF WEAVING LENGTH FOR TAPER AND AUXILIARY LANE LAYOUTS
Source: Design Manual for Roads and Bridges Volume 6 Section 2, Part 1 TD 22/06, Figure 4/9.

3.14.2 Weave Distance

Weave distance is the distance from a driveway to a nearby intersection or interchange ramp, where a driver exits a driveway, merges into the nearest lane and then “weaves” across lanes, one at a time, in order to turn at an intersection on the opposite side of the roadway. Weave movement for left-turn and right-turn shown in **FIGURE 3.26.**

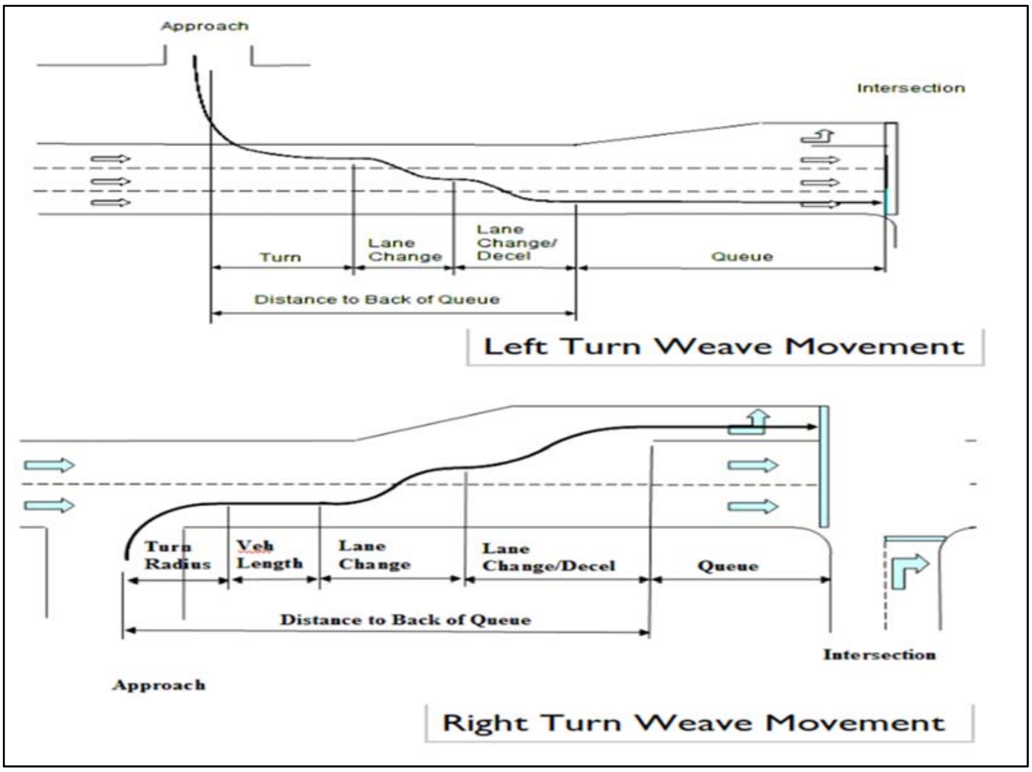


FIGURE 3.26: WEAVE MOVEMENT

3.14.3 Weaving Configuration

The most critical aspect of operations within a weaving segment is lane changing. Weaving vehicles, which must cross a roadway to enter on the right and leave on the left, or vice versa, accomplish these maneuvers by making the appropriate lane changes. The configuration of the weaving segment (i.e., the relative placement of entry and exit lanes) has a major effect on the number of lane changes required of weaving vehicles to successfully complete their maneuver. There is also a distinction between lane changes that must be made to weave successfully and additional lane changes that are discretionary (i.e., are not necessary to complete the weaving maneuver). The former must take place within the confined length of the weaving segment, whereas the latter are not restricted to the weaving segment itself. **TABLE 3.14** may be used to establish configuration type.

TABLE 3.14: DETERMINING CONFIGURATION TYPE

Number of Lane Changes Required by Movement v_{w1}	Number of Lane Changes Required by Movement v_{w2}		
	0	1	≥ 2
0	Type B	Type B	Type C
1	Type B	Type A	N/A
≥ 2	Type C	N/A	N/A

Note:
N/A = not applicable; configuration is not feasible
V_{W1} = larger of the two weaving flow rates in the weaving segment (pc/hr)
V_{W2} = smaller of the two weaving flow rates in the weaving segment (pc/hr)

Source: TRB HCM 2000, Chapter 24 Freeway Weaving, Exhibit 24-5.

The three types of geometric configurations are defined as follows:

(a) Type A

Weaving vehicles in both directions must make one lane change to successfully complete a weaving maneuver.

(b) Type B

Weaving vehicles in one direction may complete a weaving maneuver without making a lane change, whereas other vehicles in the weaving segment must make one lane change to successfully complete a weaving maneuver.

(c) Type C

Weaving vehicles in one direction may complete a weaving maneuver without making a lane change, whereas other vehicles in the weaving segment must make two or more lane changes to successfully complete a weaving maneuver.

3.14.4 Designing for Weaving Sections

The step by step computational procedure for the evaluation of the level of service in an existing or projected simple weaving area is as below. For further discussion on the calculation of LOS should be refer to HCM 2000.

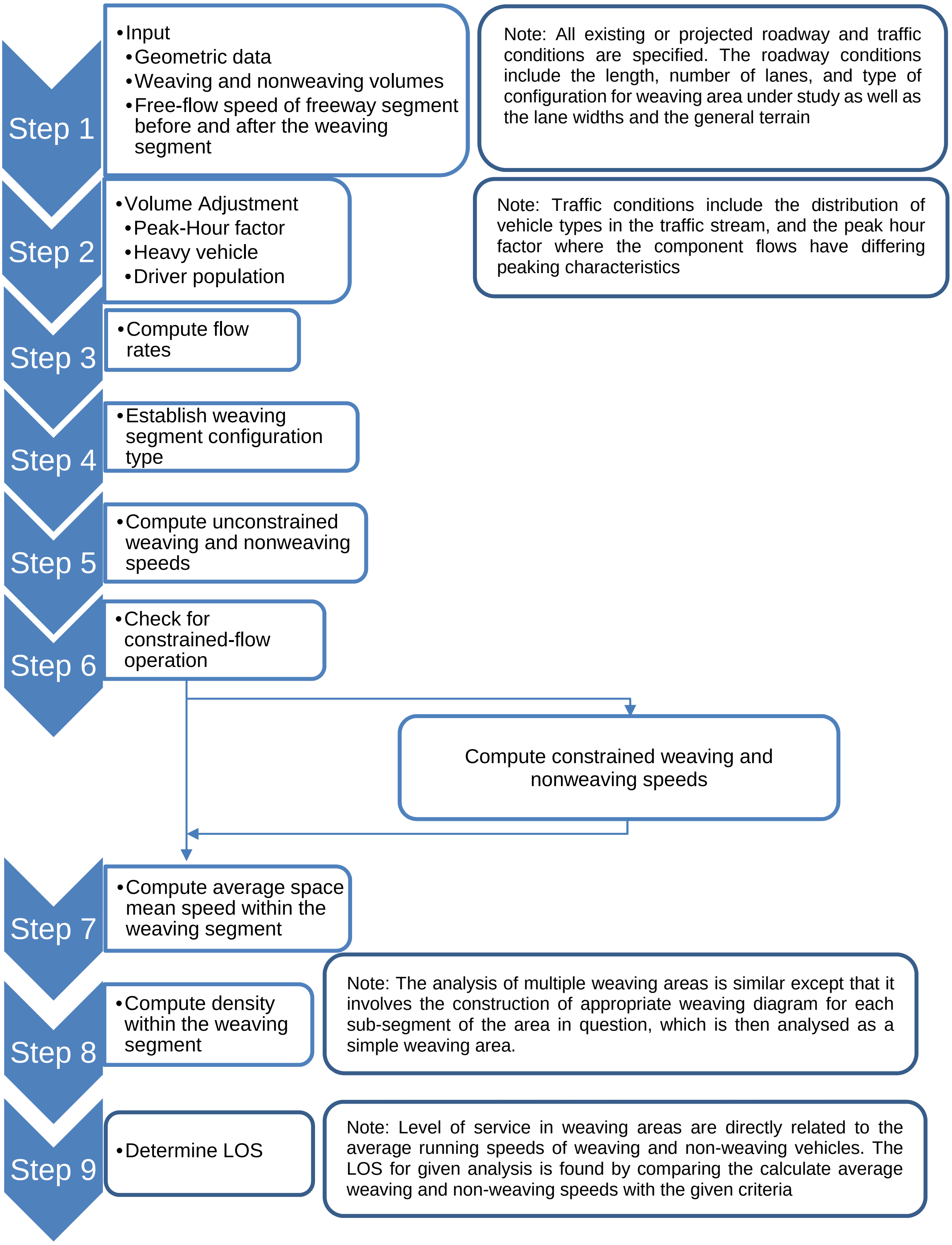


FIGURE 3.27: WEAVING METHODOLOGY

Source: TRB HCM 2000, Chapter 24 Freeway Weaving, Exhibit 24-1.

4.0 CAPACITY OF INTERSECTIONS

4.1 General

The capacity of a roadway is determined primarily by constraints that are present at intersections. Vehicles turning to and from the primary roadway at unsignalised intersections cause through vehicles to stop or slow, thereby interrupting traffic flow and reducing level of service. The available green time at signalised intersections is substantially less than the total time available for free flow. For this reason, capacity and level of service analysis is one of the most important considerations in the design of intersections.

Intersection capacity is the maximum hourly rate at which vehicles can reasonably be expected to pass through the intersection under prevailing traffic, roadway and signalization condition.

The capacity of signalised intersection, unsignalised intersection and suburban arterial indicates the ability of the facilities to accommodate of moving stream of people or vehicles. It is a measure of the supply side of any transportation facilities.

4.2 Level of Service

Level of Service is a qualitative measure of the effect of a number of factors, which include speed and travel time, traffic interruptions, freedom to manoeuver, safety, driving comfort and convenience and operating costs. The level of service concept is used in the capacity analysis of intersections. The required level of service to be used for intersections along the various categories of roads are as shown in **TABLE 4.1**.

***TABLE 4.1: LEVEL OF SERVICE**

Areas	Category Of Road	Level Of Service
Rural	Expresssway	C
	Highway	C
	Primary	D
	Secondary	D
	Minor	E
Urban	Expressway	C
	Arterial	D
	Collector	D
	Local Street	E

4.3 Factors Affecting Capacity and Level of Service

4.3.1 Ideal Conditions

An ideal condition is one for which further improvement will not achieve any increase in capacity. Ideal condition is the condition representative of good weather, good pavement conditions, users familiar with the facility, and no incidents to hold back traffic flow. Specific ideal conditions for each facility are identified.

Predictive adjustments must be included to reflect the absence of ideal conditions in order to evaluate the conditions that are not ideal in most capacity analysis. The prevailing conditions are generally divided into roadway, traffic and control conditions. Variation in vehicle control and technology represent conditions that change in the long term.

4.3.2 Roadway Conditions

Roadway factors include geometric conditions and design elements. These factors may affect the capacity of a road, and can change the result in the measurement of effectiveness, such as speed. Roadway factors include the following:

(a) The type of facility and its development environment
(b) Lane width
(c) Lateral clearances and shoulder widths
(d) Design speed
(e) Alignments
(f) Availability of queuing space at intersections

The type of facility is critical, because the existence of other major facility type factors will significantly affect flow characteristics and capacity. In several cases, the development environment has also been found to affect the performance.

Lane and shoulder widths can have significant impact on traffic flow. The vehicles will travel closer if the lanes are narrow. Motorist compensate by slowing down or observing larger longitudinal spacing for given speed, which effectively reduces capacity and service flow rates.

Narrow shoulders and lateral obstructions also have effect on capacity and level of service. Motorist will try to move away from roadside or median objects to be safe and not to cause a hazard. Consequently, it will bring them laterally closer to vehicles in adjacent lanes and create the same response as those present in narrow lanes.

According to TRB 2000 (Transportation Research Board), restricted design speeds affect the operations and level of service; drivers are forced to drive their vehicle at somewhat reduced speeds and to be more vigilant in reacting to changes in horizontal and vertical alignments resulting from a reduced design speed.

4.3.3 Traffic Conditions

Vehicle type and lane or directional distributions are the traffic conditions that influence the capacities and level of services. The procedures assume that drivers are familiar with the facility. Less efficient use of roadway facilities during weekends or roadways leading to recreation areas is generally attributed mainly to the lack of specific local knowledge.

The capacity can be affected due to differences in vehicle composition. With respect to the Malaysian traffic characteristics, there are several classes of vehicle types, which can categorized as in **TABLE 4.2**:

****TABLE 4.2: VEHICLE CLASSIFICATIONS IN MALAYSIA**

Class	Type Of Vehicle
1	Passenger car, taxi, pickup, small van
2	Lorry, large van, heavy vehicle with 2 axles
3	Large lorry, trailer, heavy vehicle with 3 axles and
4	Bus
5	Motorcycle, scooter

The capacity in the traffic stream can be affected if there is the existence of heavy vehicles. Heavy vehicles are defined as having more than four tyres touching the pavement.

Heavy vehicles in Malaysia contain:

(a) Large van, lorry with two-axles
(b) Lorry with three and more axles
(c) Bus include school bus, factory bus and express bus

Heavy vehicles can have an impact as follows:

(i) They are larger than passenger cars and therefore occupy more roadway space than passenger car
(ii) They have poor acceleration, deceleration and the ability to maintain speed on upgrades

Large gaps formed in the traffic stream because of these large vehicles cannot keep pace with passenger cars, and it is difficult for the passenger car to reduce the gap by undertaken maneuvers. These gaps create inefficiencies in the use of roadway space.

Heavy vehicles may also affect downgrade operations, particularly where downgrades are steep enough to require operation of such vehicles in a low gear. This maneuver will also create large gaps in the traffic stream, due to heavy vehicles must operate at lower speed.

There is considerable variation in the characteristics and performance capabilities of vehicles within each class of heavy vehicle.

In Malaysia, high percentage of motorcycles traverses through traffic stream and currently there is no proper consideration of traffic engineering aspects of motorcycles in the design. Improper consideration of motorcycles in the design of intersections results in inaccurate design and thus can possibly can cause significant amount of traffic congestion.

The type of motorcycles prevalent on Malaysian roads is that of small size motorcycles where the length of its wheel bases is about 0.5 meter. The size is small as compared to that of passenger cars where the length of its wheel bases is about 1.68 meter. Due to its small size, motorcycles can weave in and out of traffic stream especially when approaching signalised intersection. This enables the motorcyclist to get closer to the stop line, therefore the concept of First In First Out is violated.

Motorcyclist’s unique characteristics are that they can travel alongside other vehicles within a lane. As a result, the flow is not in a structured discipline.

4.3.4 Control Conditions

For signalised intersection, control in terms of the time available for movement of specific traffic flow is a critical element affecting capacity, service flow rates and level of service. The type of control, signal phasing, green time allocation, cycle length and the relationship with the adjacent control measures affects operations at traffic light intersections.

Stop signs at unsignalised intersection also affect capacity. Motorist travelling on the minor street must find gaps in the major traffic flow. Thus, the capacity of such approaches depends on traffic conditions on the major street.

Restriction of kerb parking can increase the number of lanes available on the street or highway. Turn restrictions can eliminate conflicts at intersections and subsequently increase the capacity. Lane use controls can positively allocate available roadway space especially during peak congestion period. Lane use control can be used at intersections and to create reversible lanes on critical arterials.

4.3.5 Technology

Emerging transportation technologies such as Intelligent Transportation System (ITS) are being developed to enhance the safety and efficiency of roads in Malaysia. This technology will allow real-time information to be gathered and used by drivers and traffic control system operators to provide better vehicle navigation, roadway system control, or both.

However, many of roadway improvement related to ITS are system level improvements such as incident response and driver information system, so they are not expected to have much impact on capacity for individual roadway facility.

4.4 Unsignalised Intersections

4.4.1 General

Capacity analysis are seldom required for rural intersections since their volumes are rarely sufficient to make capacity a design consideration. Safety is normally the major consideration in rural situations, which may necessitate the provision of separate lanes for left or right turning vehicles.

The method of capacity analysis as detailed below is therefore more pertinent for urban intersections. It is based on the Malaysian Highway Capacity Manual 2006, Highway Planning Unit, and Ministry of Works Malaysia. The designer is advised to refer to the above publication for a better understanding of the subject.

4.4.2 Methodology

There are several parameters affecting the capacity of the unsignalised intersection, i.e., intersections geometric, critical gap, follow up time and vehicle characteristics. Nevertheless, two of the most important parameters affecting the capacity and performance of unsignalised intersection are the critical gap and the follow up time.

The overall methodology for analyzing unsignalised intersection is shown in **FIGURE 4.1**. The methodology takes into consideration the Malaysian traffic condition such as the prevalence of motorcycles in the traffic stream. The presence of motorcycles is taken into consideration in the estimation of critical gap and follow up times, and also in the computation of the potential capacity. The procedure for estimating control delays and threshold for level of service are adapted from Transportation Research Board, 2000.

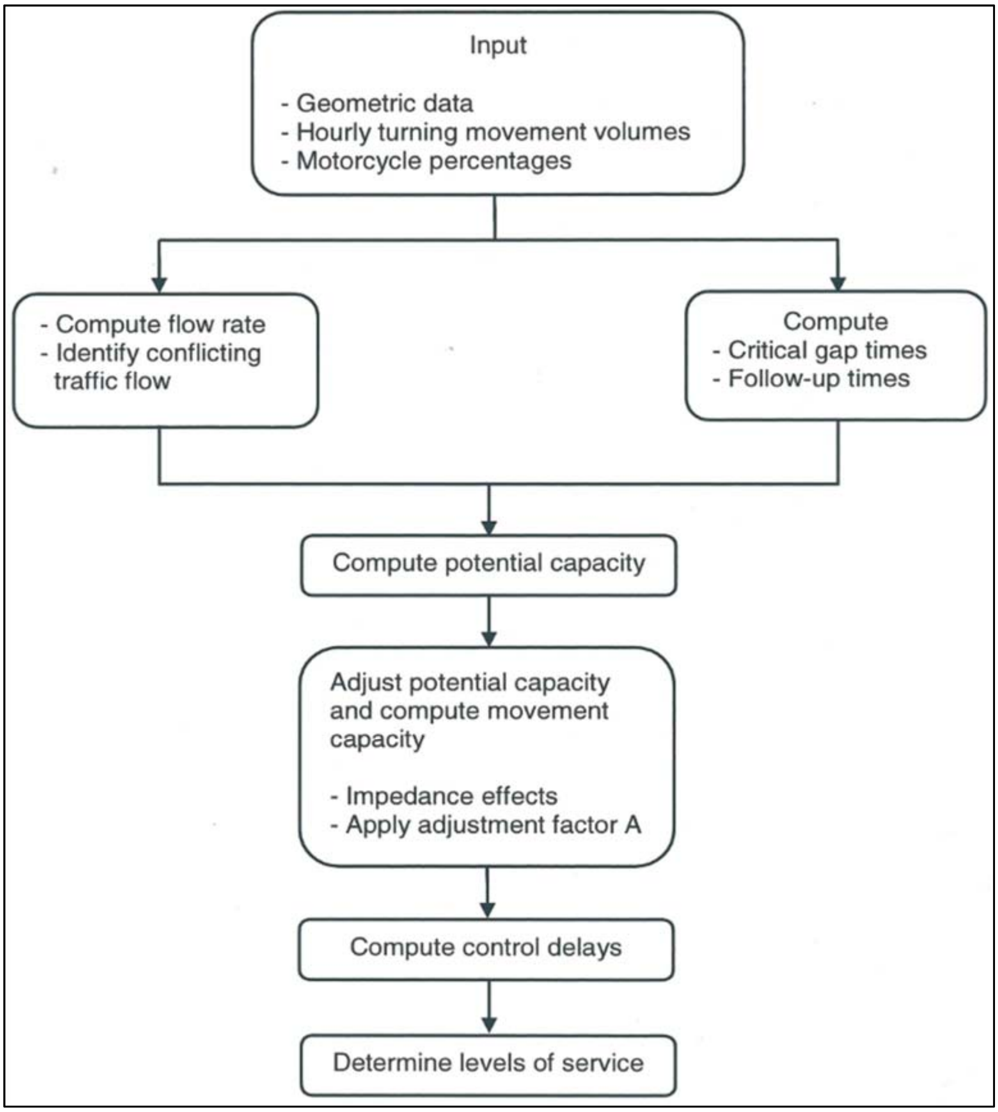


****FIGURE 4.1: OPERATIONAL ANALYSIS PROCEDURES**

Capacity analysis at two-way stop-controlled (TWSC) intersections depends on a clear description and understanding of the interaction of drivers on the minor or stop-controlled approach with the drivers on the major street. Procedures described in this chapter rely on gap acceptance model developed by Troutbeck (1992) where the data were collected at several TWSC intersections in Malaysia.

4.4.3 Priority Streams

The priority of traffic stream at unsignalised intersection must be correctly identified. Some of the streams have absolute priority, while others have to give way or yield to higher rank stream. **FIGURE 4.2** shows the comparative priority in the stream for two-ways stop control intersection.

In the four-leg intersection, the priority of movement is described in four level ranks. Movement of rank 1 includes through traffic on the major street and left turning from the major street. For movement rank 2, it includes right-turning from the major street and left-turning from the minor street. Rank 3 movements include through traffic on the minor street, as for the case of T-intersections, right turning from the minor road will be included. There are no rank 4 in T-intersections, but in four-leg intersection, rank 4 includes right-turning from the minor street.

To demonstrate the priority stream concept and application, it is best to see how a four-leg intersection operates in such manner. However, in Malaysia, the concept of two-way stopped control at a four-leg intersection is rarely found, especially in urban areas, where most of the two-way stopped control are upgraded to signalised intersection.

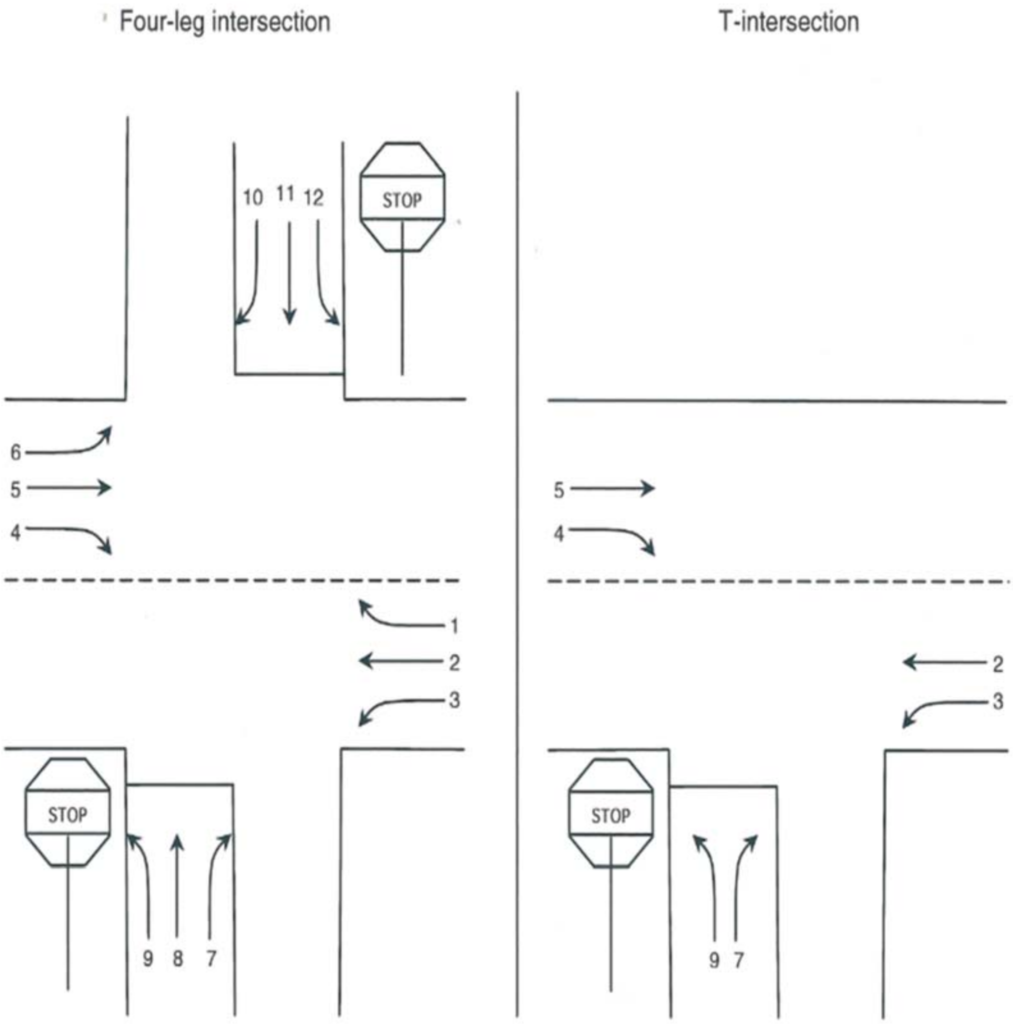
4.4.4 Conflicting Traffic

Each movement at a TWSC intersection faces a different set of conflicts that are related to the nature of movement computation of conflict flows is as shown in **FIGURE 4.3**, which illustrates the computation of parameter $V_{c,x,}$ the conflicting flow rate for movement x, that is, the total flow rate that conflicts with movement x (veh/h).

A typical unsignalised T-intersection is as illustrated in **FIGURE 4.4**. There are six different types of traffic movements at the T- intersections. The hierarchy of the unsignalised T-intersections has three levels of conflicting streams that should be considered, i.e., movement Z, Y and X. The conflicting streams mean that the movements cannot cross the intersections except the driver gives priority to other movement which has simple potential conflict and high saturation flow.

Right turn movement from major road, i.e. movement Z, is the first conflicting stream because this movement is from the major stream where major stream is a priority stream that the minor stream should be aware of it and must be given priority.

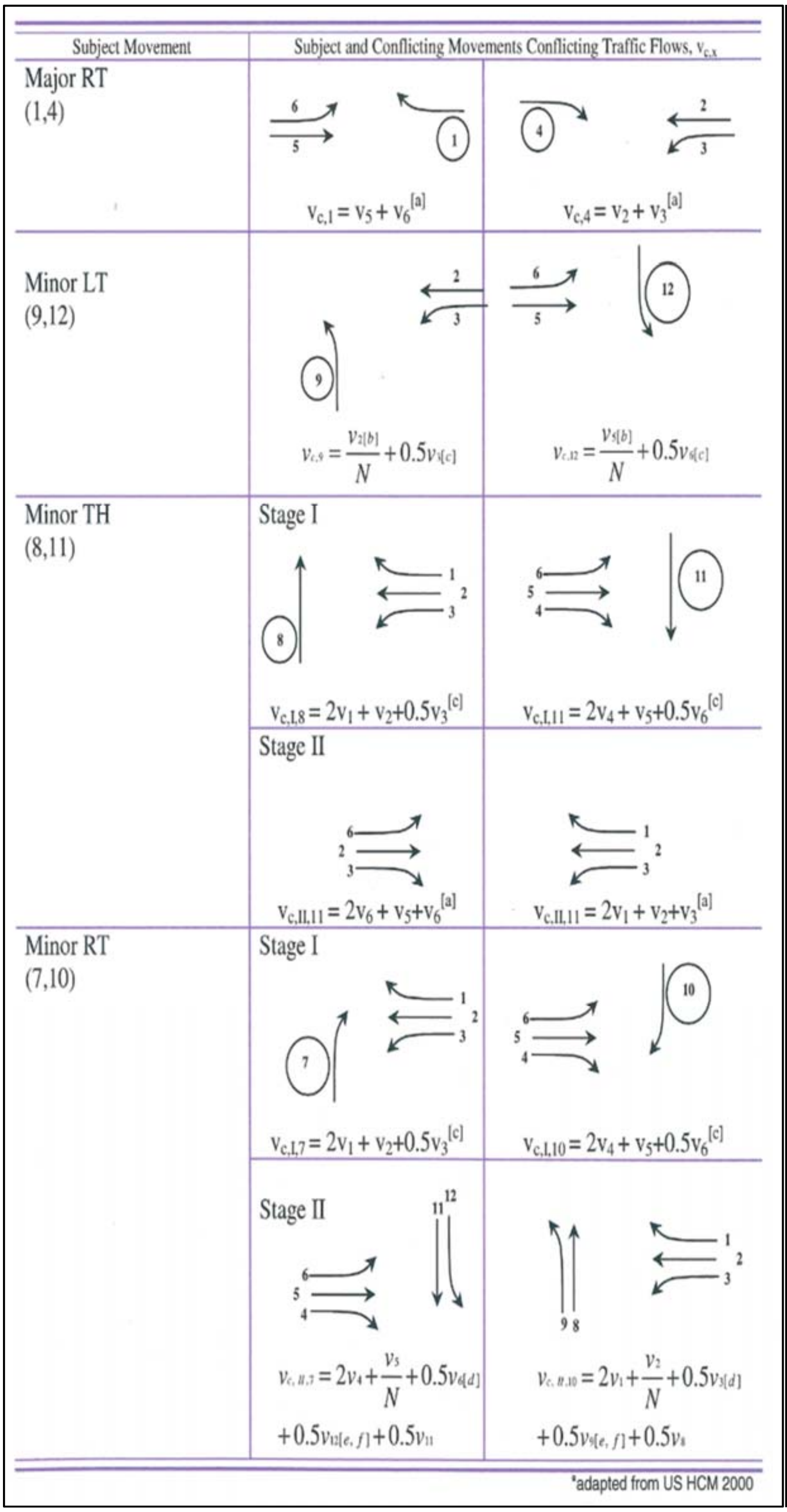
Second conflicting stream is left turn movement from minor stream. The last conflicting stream is right turn movement from minor stream. For left turn movement from minor stream, the driver has to give way only to movement ‘A’, but for right turn movement from minor stream, the driver has to give way to movements ‘A', 'B' and 'Z' which are more complicated.



Rank	Traffic stream
1	2, 3, 5, 6
2	1, 4, 9, 12
3	8, 11
4	7,10

Rank	Traffic stream
1	2, 3, 5
2	4, 9
3	7

****FIGURE 4.2: TRAFFIC STREAMS AT A TWO-WAY STOP CONTROL (TWSC) INTERSECTION**



Subject Movement	Subject and Conflicting Movements Conflicting Traffic Flows, $v_{c,x}$	
Major RT (1,4)	$v_{c,1} = v_5 + v_6$ [a]	$v_{c,4} = v_2 + v_3$ [a]
Minor LT (9,12)	$v_{c,9} = \frac{v_{2[b]}}{N} + 0.5v_{3[c]}$	$v_{c,12} = \frac{v_{5[b]}}{N} + 0.5v_{6[c]}$
Minor TH (8,11)	Stage I $v_{c,I,8} = 2v_1 + v_2 + 0.5v_3$ [c]	$v_{c,I,11} = 2v_4 + v_5 + 0.5v_6$ [c]
	Stage II $v_{c,II,11} = 2v_6 + v_5 + v_6$ [a]	$v_{c,II,11} = 2v_1 + v_2 + v_3$ [a]
Minor RT (7,10)	Stage I $v_{c,I,7} = 2v_1 + v_2 + 0.5v_3$ [c]	$v_{c,I,10} = 2v_4 + v_5 + 0.5v_6$ [c]
	Stage II $v_{c,II,7} = 2v_4 + \frac{v_5}{N} + 0.5v_{6[d]} + 0.5v_{12[e,f]} + 0.5v_{11}$	$v_{c,II,10} = 2v_1 + \frac{v_2}{N} + 0.5v_{3[d]} + 0.5v_{9[e,f]} + 0.5v_8$

*adapted from US HCM 2000

Note:
[a] If left-turning traffic from major stress is separated by a triangular island and has to comply with a yield or stop sign, v_6 and v_3 need not be considered.
[b] If there is more than one lane in the major street, the flow rates in the right lane are assumed to be v_2/N or v_5/N where N is the number of through lanes. Can specify different lane distribution if field data is available.
[c] If there is a left-turn lane on the major street, v_3 or v_6 should not be considered.
[d] Omit the farthest left-turn v_3 for subject movement 10 or v_6 for movement 7 if the major street is multilane.
[e] If left-turning traffic from the minor street is separated by a triangular island and has to comply with a yield or stop sign, v_9 an v_{12} need not be considered.
[f] Omit v_9 and v_{12} for multilane sites, or use one-half their values if the minor approach is flared

Note:
if it not a two-stage gap acceptance, then conflicting flow rates labelled Stage I and Stage II should be added together and considered as one conflicting flow rate for the movement in question

****FIGURE 4.3: DEFINITION AND COMPUTATION OF CONFLICTING FLOWS**

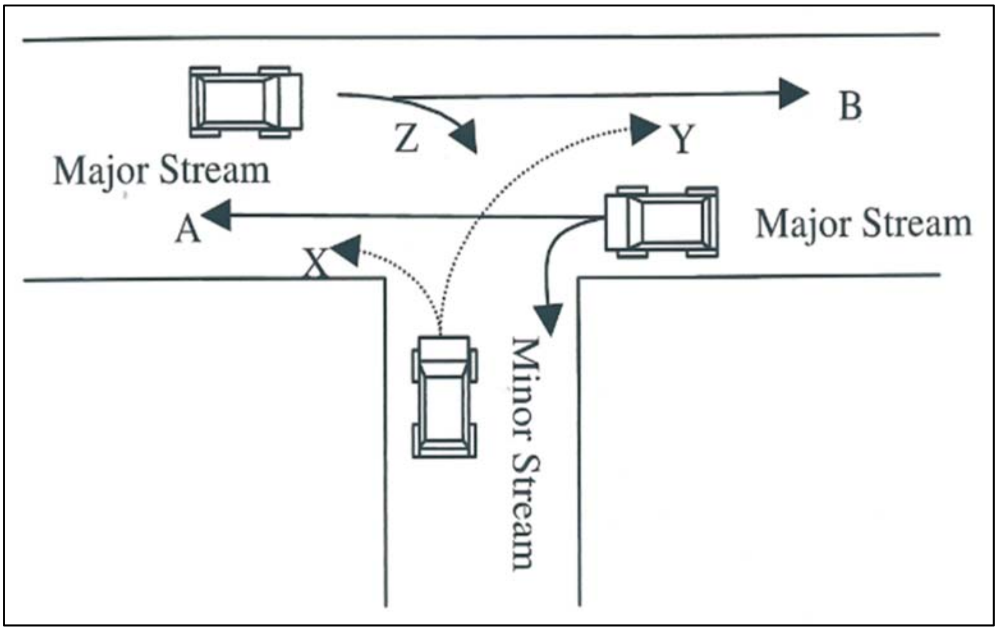


**FIGURE 4.4: VEHICLES MOVEMENT AT A T- INTERSECTIONS (TWSC)

4.4.5 Critical Gap and Follow-Up Time

The critical gap, t_c, is the minimum time interval between the vehicles of the major stream that is necessary for the vehicles in the minor stream to enter the conflict area.

In addition, several vehicles of the minor stream can only follow one behind the other within a certain time space, which is defined as follow-up time. The follow-up time is the average time gap between two vehicles of the minor stream being queued and entering the same major stream gap one behind the other. Some of the well-known estimation procedures for critical gaps have been checked for consistency in Miller (1972) and Troutbeck (1992).

The estimation procedures should be consistent. The procedures should be able to reproduce the average critical gap reliably, without being dependent on other parameters such as traffic volumes on the major or the minor street, delay experienced by the drivers, and other external influences. The generalised control gap formula taking into consideration the influence of motorcycles is as shown by Equation 4.1.

$$t_{c,x} = t_{c,base} - t_{c,M}\, P_{M,x} \qquad (4.1)$$

Where,

$t_{c,x}$ = critical gap for movement x (sec)
$t_{c,base}$ = base critical gap from Table 4.3
$t_{c,M}$ = adjustment factor for motorcycle ref. to **TABLE 4.4**
$P_{M,x}$ = proportion of motorcycles for movement x.

The follow-up time is computed for each minor movement using Equation 4.2. As for the critical gap, adjustments are made for the presence of motorcycle.

$$t_{f,x} = t_f, base \; - \; t_{f,M}, P_M \qquad (4.2)$$

Where,

$t_{f,x}$ = follow-up time for movement x (sec)
$t_{f,base}$ = base follow-up time from Table 4.3
$t_{f,M}$ = adjustment factor for motorcycle referring to **TABLE 4.4**
P_M = proportion of motorcycles for movement x

The value of base critical gap and follow-up time are as shown in **TABLE 4.3**. The value of the critical gap and follow up time will influence the capacity at unsignalised intersection. However, more accurate capacity estimates will be produced if field measurement can be acquired.

****TABLE 4.3: BASE CRITICAL GAP AND FOLLOW-UP TIME FOR TWSC INTERSECTIONS**

Vehicle Movement	Base Critical Gap, $T_{c,\,base}$ (second)		Base Follow-up Time, $T_{f,\,base}$ (second)	
	Single Lane	Multi Lane	Single Lane	Multi Lane
Right turn from major	3.5	3.7	2.0	2.1
Left turn from minor	3.2	3.3	1.9	2.1
Right turn from minor	4.0	4.2	2.2	2.4

****TABLE 4.4: ADJUSTMENT FACTOR FOR MOTORCYCLE**

$t_{c,m}$		$t_{f,m}$	
Single-lane	Multi-lane	Single-lane	Multi-lane
0.424	0.252	0.738	0.815

TABLE 4.4 shows the value of $t_{c,m}$ and $t_{f,m}$ to be included in Equations 4.1 and 4.2, in order to get the critical gap and follow-up time.

4.4.6 Potential Capacity

The potential capacity is defined as the "ideal" capacity for a specific subject movement, assuming the following conditions:

(a) Traffic on the major roadway does not block the minor road.
(b) Traffic from nearby intersections does not back up into the intersection under consideration.
(c) A separate lane is provided for the exclusive use of each minor street movement under consideration.
(d) No other movements impede the subject movement.

When traffic becomes congested in a high-priority movement, it can impede the potential capacity. These impedance effects can be derived by multiplying the potential capacity to the series of impedance factor for every impeded movement. The impedance effect will be discussed later in this chapter.

The gap acceptance method employed in the procedure used· in determining the capacity of these intersections computes the potential capacity of each minor traffic stream in accordance with Equation 4.3. The use of small critical gap and follow-up time based on **TABLE 4.2** results in the increase in potential capacity of the unsignalised intersection.

In order to obtain a realistic value, the potential capacity needs to be calibrated in the field and the potential capacity adopted from the US HCM 2000 is adjusted by using an adjustment factor, A_x. The adjustment factor will ensure that the estimated potential capacity will be according to the Malaysian traffic condition. The value for adjustment factor, A_x is shown in **TABLE 4.5.**

$$C_{mp}, x = A_x \left(V_c, x \; \frac{e^{-V_{c,x}, \; t_{c,x}/3600}}{1 - e^{-V_{c,x}, \; t_{f,x}/3600}} \right) \tag{4.3}$$

$C_{mp,x}$ = potential capacity for movement x (veh/h)
$V_{c,x}$ = conflicting flow rate for movement x (veh/h)
$t_{c,x}$ = critical gap for movement x (sec)
$t_{f,x}$ = follow-up time for movement x (sec)
A_x = adjustment factor for movement x (refer table 4.5)

****TABLE 4.5: ADJUSTMENT FACTOR FOR CAPACITY, A**

Vehicle maneuver	A value	
	Single lane	Multi lane
Right-turn from major street	1.000	1.000
Left turn from minor street	0.4846	0.5181
Right turn from minor street	0.4375	0.4864

4.4.7 Right Turn From Major

Based on Equation 4.1 and Equation 4.2, the critical gaps and follow-up times for right turning into minor road from major road have been tabulated according to the proportion of motorcycles in that particular movement. **TABLE 4.6** and **TABLE 4.7** are the critical gaps and follow-up times for single lane and multilane facility, respectively.

****TABLE 4.6: CRITICAL GAPS AND FOLLOW UP TIMES FOR RIGHT TURN FROM MAJOR MOVEMENT (SINGLE LANE APPROACH)**

Proportion of motorcycles for movement x, $P_{M,x}$	Critical gap for movement x (sec), $t_{c,x}$	Follow up times for movement x (sec), $t_{f,x}$
0.0	3.500	2.000
0.1	3.458	1.926
0.2	3.415	1.852
0.3	3.373	1.779
0.4	3.330	1.705
0.5	3.288	1.631
0.6	3.246	1.557
0.7	3.203	1.483
0.8	3.161	1.410
0.9	3.118	1.336
1.0	3.076	1.262

****TABLE 4.7: CRITICAL GAPS AND FOLLOW UP TIMES FOR RIGHT TURN FROM MAJOR MOVEMENT (MULTI-LANE APPROACH)**

Proportion of motorcycles for minor movement x, P_M	Critical gap for minor movement x (sec), $t_{c,x}$	Follow up times for minor movement x (sec), $t_{f,x}$
0.0	3.700	2.100
0.1	3.658	2.019
0.2	3.615	1.937
0.3	3.573	1.856
0.4	3.530	1.774
0.5	3.488	1.693
0.6	3.446	1.611
0.7	3.403	1.530
0.8	3.361	1.448
0.9	3.318	1.367
1.0	3.276	1.285

4.4.8 Left Turn From Minor

Based on Equation 4.1 and Equation 4.2, the critical gaps and follow up times for left turning from major road have been tabulated according to the proportion of motorcycles in that particular movement. **TABLE 4.8** and **TABLE 4.9** are critical gaps and follow-up times for single lane and multilane, respectively.

****TABLE 4.8: CRITICAL GAPS AND FOLLOW UP TIMES FOR LEFT TURN FROM MINOR MOVEMENT (SINGLE LANE APPROACH)**

Proportion of motorcycles for movement x, P_M	Critical gap for movement x (sec), $t_{c,x}$	Follow up times for movement x (sec), $t_{f,x}$
0.0	3.200	1.900
0.1	3.158	1.826
0.2	3.115	1.752
0.3	3.073	1.679
0.4	3.030	1.605
0.5	2.988	1.531
0.6	2.946	1.457
0.7	2.903	1.383
0.8	2.861	1.310
0.9	2.818	1.236
1.0	2.776	1.162

****TABLE 4.9: CRITICAL GAPS AND FOLLOW UP TIMES FOR LEFT TURN FROM MINOR MOVEMENT (MULTI-LANE APPROACH)**

Proportion of motorcycles for movement x, P_M	Critical gap for movement x (sec), $t_{c,x}$	Follow up times for movement x (sec), $t_{f,x}$
0.0	3.300	2.100
0.1	3.258	2.019
0.2	3.215	1.937
0.3	3.173	1.856
0.4	3.130	1.774
0.5	3.088	1.693
0.6	3.046	1.611
0.7	3.003	1.530
0.8	2.961	1.448
0.9	2.918	1.367
1.0	2.876	1.285

4.4.9 Right Turn from Minor

Based on Equation 4.1 and Equation 4.2, the critical gaps and follow-up times for right turning from minor road have been tabulated according to the proportion of motorcycles in that particular movement. **TABLE 4.10** and **TABLE 4.11** are the critical gaps and follow-up times for single lane and multilane facilities.

****TABLE 4.10: CRITICAL GAPS AND FOLLOW UP TIME FOR RIGHT TURN FROM MINOR MOVEMENT (SINGLE-LANE APPROACH)**

Proportion of motorcycles for minor movement x, P_M	Critical gap for minor movement x (sec), $t_{c,x}$	Follow up times for minor movement x (sec), $t_{f,x}$
0.0	4.000	2.200
0.1	3.958	2.126
0.2	3.915	2.052
0.3	3.873	1.979
0.4	3.830	1.905
0.5	3.788	1.831
0.6	3.746	1.757
0.7	3.703	1.683
0.8	3.661	1.610
0.9	3.618	1.536
1.0	3.576	1.462

****TABLE 4.11: CRITICAL GAP AND FOLLOW UP TIME FOR RIGHT TURN FROM MINOR MOVEMENT (MULTI-LANE APPROACH)**

Proportion of motorcycles for minor movement x, P_M	Critical gap for minor movement x (sec), $t_{c,x}$	Follow up times for minor movement x (sec), $t_{f,x}$
0.0	4.200	2.400
0.1	4.158	2.319
0.2	4.115	2.237
0.3	4.073	2.156
0.4	4.030	2.074
0.5	3.988	1.993
0.6	3.946	1.911
0.7	3.903	1.830
0.8	3.861	1.748
0.9	3.818	1.667
1.0	3.776	1.585

4.4.10 Impedance Effect

Vehicle Impedance

Vehicle use gaps at a two-way stopped control (TWSC) intersection in a prioritized manner. When traffic becomes congested in a high-priority movement, it can be impeded by lower-priority movements from using gaps in the traffic stream, thus reducing the capacity of these movements.

Minor traffic streams of Rank 2 (including right turns from the major street and left turns from the minor street) must yield only to the major-street through and left-turning traffic streams of Rank 1. There are no additional impedances from other minor traffic streams, and so the movement capacity of each Rank 2 traffic stream is equal to its potential capacity as indicated by Equation 4.4.

$$C_m, j = C_p, j \quad (4.4)$$

where j denotes movements of Rank 2 priority.

For TWSC T-intersection, the probability that the major-street right-turning traffic will operate in a queue-free state is computed using Equation 4.5 as adapted from the HCM 2000.

$$P_0, j = 1 - \frac{V_j}{C_m, j} \quad (4.5)$$

Where,

j = 3 (major-street right-turn movements of Rank 2).

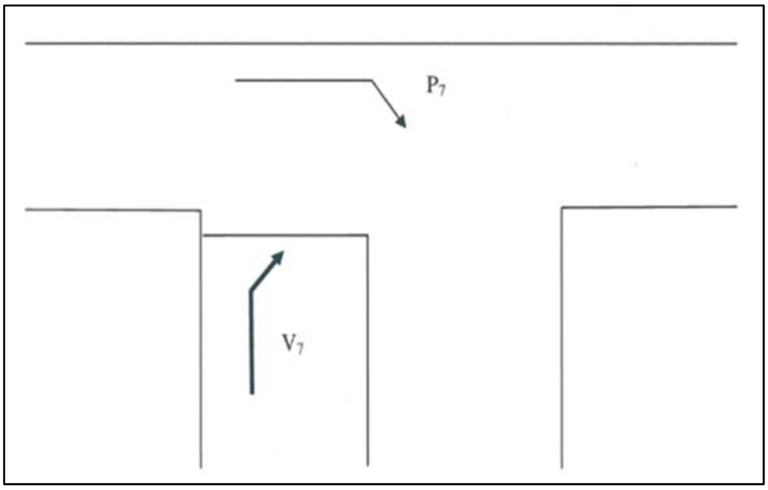


FIGURE 4.5: EXAMPLE OF IMPEDANCE EFFECT

Referring to **FIGURE 4.5**, right turn movement is impeded by queue of the major-street right turning.

The movement capacity for right turn from minor is computed using Equation 4.6.

$$C_m, 7 = C_p, 7 \times P_O, 7 \quad (4.6)$$

4.4.11 Movement Capacity

The potential capacity, C_p, x , of minor street movements is given in **FIGURE 4.6** for a single lane street and in **FIGURE 4.7** for a multilane street. These graphs show the application of the values in **TABLE 4.3**, **TABLE 4.5** and the Equation 4.3. The potential capacity is presented as vehicles per hour (veh/h). The figure indicates that the potential capacity is a function of the conflicting flow rate, v_c, x expressed as an hourly rate, as well as the minor-street movement.

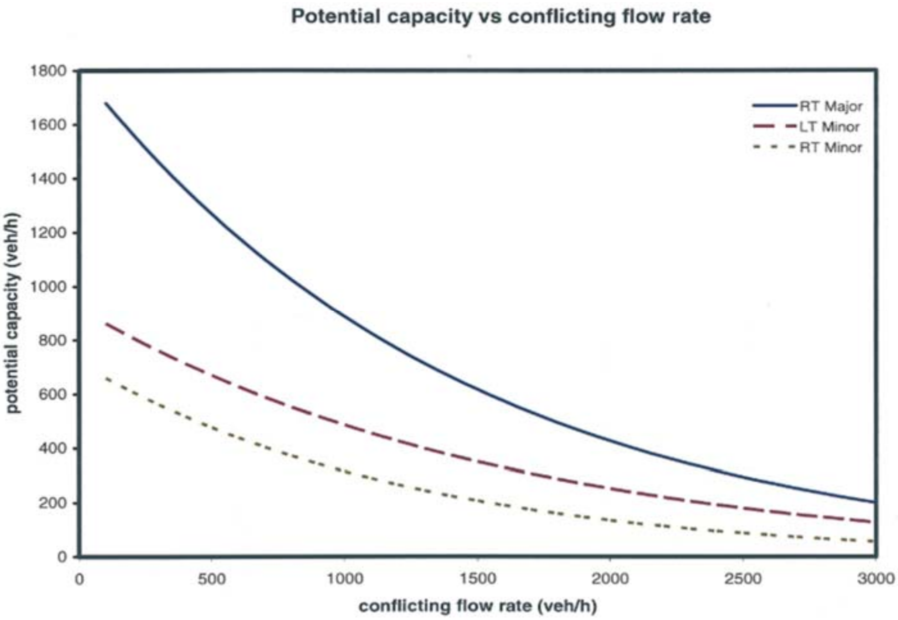


**FIGURE 4.6: POTENTIAL CAPACITY FOR SINGLE LANE

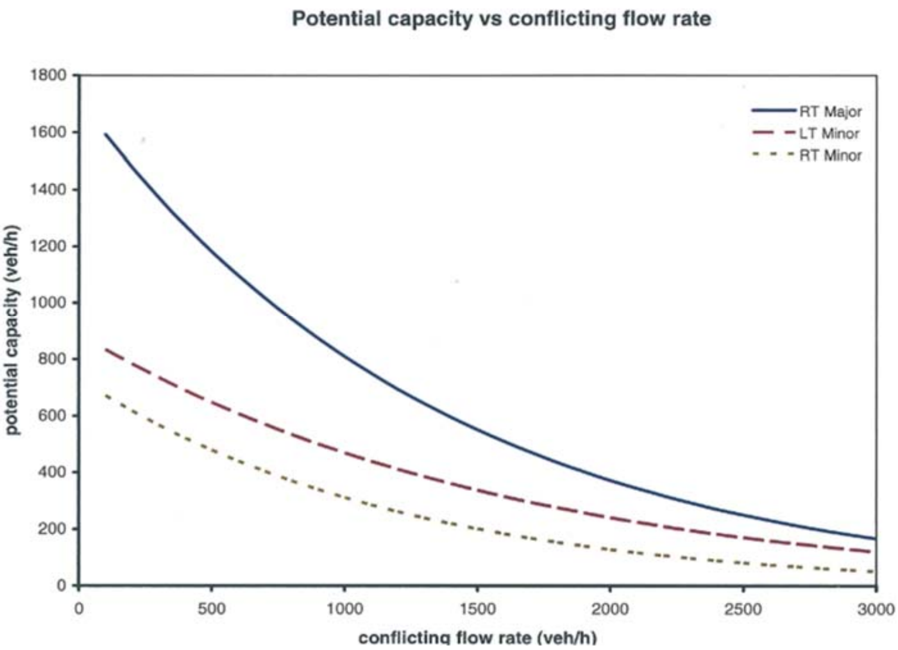


**FIGURE 4.7: POTENTIAL CAPACITY FOR MULTILANES

4.4.12 Capacity for Right Turn from Major Road

Capacity calculation for right turn from major road is based on Equation 4.3. It is important to know the conflicting flow rate in order to calculate the potential capacity for each movement. **TABLE 4.12** and **4.13** simplify the potential capacity for each single lane and multilane for right turn from major road, respectively.

**TABLE 4.12: POTENTIAL CAPACITY FOR RIGHT TURN FROM MAJOR FOR SINGLE LANE (MOVEMENT X =4)

P_M	Conflicting Flow Rate, $v_{c,4}$														
	100	200	300	400	500	600	700	800	900	1000	1100	1200	1300	1400	1500
	Potential Capacity, $c_{mp,4}$														
0.0	1679	1566	1460	1361	1268	1181	1100	1024	954	887	826	768	714	664	617
0.1	1744	1626	1517	1414	1318	1228	1144	1065	992	924	860	800	744	692	644
0.2	1813	1692	1578	1471	1372	1278	1191	1110	1034	963	896	834	777	723	672
0.3	1889	1762	1644	1533	1430	1333	1243	1158	1079	1005	936	872	812	755	703
0.4	1971	1839	1716	1601	1493	1393	1298	1210	1128	1051	979	912	850	791	736
0.5	2061	1923	1795	1675	1562	1457	1359	1267	1181	1101	1026	956	891	830	773
0.6	2159	2015	1881	1755	1638	1528	1426	1330	1240	1156	1078	1005	936	872	813
0.7	2266	2116	1976	1844	1721	1606	1499	1398	1304	1216	1134	1058	986	919	857
0.8	2385	2228	2080	1942	1813	1692	1579	1474	1375	1283	1197	1116	1041	970	905
0.9	2518	2351	2196	2051	1915	1788	1669	1558	1454	1357	1266	1181	1102	1028	958
1.0	2665	2490	2326	2172	2029	1894	1769	1651	1542	1439	1343	1253	1170	1091	1018

	Conflicting Flow Rate, $v_{c,4}$														
	1600	1700	1800	1900	2000	2100	2200	2300	2400	2500	2600	2700	2800	2900	3000
	Potential Capacity, $c_{mp,4}$														
0.0	573	533	495	459	427	396	367	341	316	293	272	252	233	216	200
0.1	598	556	517	480	446	414	384	357	331	307	285	264	245	227	210
0.2	625	581	540	502	467	434	403	374	347	322	299	278	258	239	222
0.3	654	608	566	526	489	455	423	393	365	339	315	292	271	252	234
0.4	685	638	594	552	514	478	444	413	384	357	331	308	286	266	247
0.5	720	670	624	580	540	503	468	435	404	376	350	325	302	281	261
0.6	757	705	657	611	569	530	493	459	427	397	369	344	319	297	276
0.7	798	744	693	645	601	560	521	485	452	420	391	364	339	315	293
0.8	843	786	733	683	636	593	552	514	479	446	415	387	360	335	312
0.9	894	833	777	724	675	629	586	546	509	474	442	411	383	357	332
1.0	950	886	826	771	719	670	625	582	543	506	471	439	409	381	355

TABLE 4.13: POTENTIAL CAPACITY FOR RIGHT TURN FROM MAJOR FOR MULTILANE (MOVEMENT X = 4)

P_M	Conflicting Flow Rate, $v_{c,4}$														
	100	**200**	**300**	**400**	**500**	**600**	**700**	**800**	**900**	**1000**	**1100**	**1200**	**1300**	**1400**	**1500**
	Potential Capacity, $c_{mp,4}$														
0.0	1592	1479	1373	1274	1182	1097	1017	943	874	810	750	694	643	595	551
0.1	1656	1537	1427	1324	1228	1138	1055	978	906	840	777	720	666	617	570
0.2	1725	1601	1485	1377	1277	1184	1097	1016	942	872	807	747	692	640	592
0.3	1800	1669	1548	1435	1330	1233	1142	1058	980	907	840	777	719	665	615
0.4	1882	1745	1617	1499	1389	1287	1192	1104	1022	946	875	810	749	693	641
0.5	1972	1827	1693	1569	1453	1346	1246	1153	1068	988	914	846	782	723	669
0.6	2071	1918	1777	1645	1523	1410	1305	1208	1118	1034	957	885	818	756	699
0.7	2180	2019	1869	1730	1601	1482	1371	1269	1174	1086	1004	928	858	793	733
0.8	2302	2130	1972	1825	1688	1562	1445	1336	1236	1143	1056	977	903	834	771
0.9	2438	2256	2087	1930	1785	1651	1527	1412	1305	1206	1115	1031	952	880	813
1.0	2591	2397	2216	2050	1895	1752	1619	1497	1383	1278	1181	1091	1008	931	860

	Conflicting Flow Rate, $v_{c,4}$														
	1600	**1700**	**1800**	**1900**	**2000**	**2100**	**2200**	**2300**	**2400**	**2500**	**2600**	**2700**	**2800**	**2900**	**3000**
	Potential Capacity, $c_{mp,4}$														
0.0	509	471	435	402	372	343	317	293	270	249	230	212	196	180	166
0.1	528	488	451	417	385	356	329	303	280	258	238	220	203	187	172
0.2	547	506	468	432	400	369	341	315	291	268	247	228	210	194	179
0.3	569	526	486	449	415	383	354	327	302	279	257	237	219	202	186
0.4	592	548	506	468	432	399	369	340	314	290	268	247	228	210	194
0.5	618	571	528	488	451	416	384	355	327	302	279	257	237	219	202
0.6	646	597	552	510	471	435	402	371	342	316	291	269	248	229	211
0.7	678	626	578	534	493	456	421	388	358	331	305	282	260	240	221
0.8	712	658	608	561	518	478	442	408	376	347	320	296	273	252	232
0.9	751	693	640	591	546	504	465	429	396	366	337	311	287	265	244
1.0	794	733	677	625	577	533	492	454	419	386	356	329	303	280	258

4.4.13 Capacity for Left Turn from Minor

TABLE 4.14: POTENTIAL CAPACITY FOR LEFT TURN FROM MINOR FOR SINGLE LANE (MOVEMENT X = 9)

P_M	Conflicting Flow Rate, $v_{c,9}$														
	100	200	300	400	500	600	700	800	900	1000	1100	1200	1300	1400	1500
	Potential Capacity, $c_{mp,9}$														
0.0	862	810	760	714	670	628	589	553	518	486	455	427	400	374	350
0.1	897	843	792	743	698	655	614	576	540	507	475	445	417	391	366
0.2	935	879	825	775	728	683	641	601	564	529	496	465	436	409	383
0.3	977	918	862	810	760	714	670	629	590	554	520	487	457	428	402
0.4	1022	960	902	848	796	748	702	659	619	581	545	511	480	450	422
0.5	1071	1007	946	889	835	785	737	692	650	610	573	537	504	473	444
0.6	1126	1058	995	935	878	825	775	728	684	642	603	566	531	499	468
0.7	1186	1115	1048	985	926	870	818	768	722	678	637	598	562	527	495
0.8	1253	1178	1108	1042	979	920	865	813	764	718	674	634	595	559	525
0.9	1328	1249	1175	1105	1039	977	918	863	811	762	716	673	632	594	558
1.0	1412	1329	1250	1176	1106	1040	978	919	864	812	764	718	675	634	596

	Conflicting Flow Rate, $v_{c,9}$														
	1600	1700	1800	1900	2000	2100	2200	2300	2400	2500	2600	2700	2800	2900	3000
	Potential Capacity, $c_{mp,9}$														
0.0	328	307	287	269	251	235	220	205	192	179	167	156	146	136	127
0.1	343	321	300	281	263	246	230	215	201	188	176	164	153	143	134
0.2	359	336	315	295	276	258	242	226	212	198	185	173	162	151	141
0.3	376	353	330	310	290	271	254	238	223	208	195	182	171	159	149
0.4	395	371	347	326	305	286	268	251	235	220	206	193	180	169	158
0.5	416	390	366	343	322	302	283	265	248	232	218	204	191	179	167
0.6	439	412	387	363	340	319	299	280	263	246	231	216	202	190	178
0.7	465	436	409	384	360	338	317	297	279	261	245	230	215	202	189
0.8	493	463	434	408	383	359	337	316	297	278	261	245	229	215	202
0.9	524	492	462	434	408	383	359	337	317	297	279	262	245	230	216
1.0	560	526	494	464	436	409	384	361	339	318	299	280	263	247	232

****TABLE 4.15: POTENTIAL CAPACITY FOR LEFT TURN FROM MINOR FOR MULTILANE (MOVEMENT X = 9)**

P_M	Conflicting Flow Rate, $v_{c,9}$														
	100	**200**	**300**	**400**	**500**	**600**	**700**	**800**	**900**	**1000**	**1100**	**1200**	**1300**	**1400**	**1500**
	Potential Capacity, $c_{mp,9}$														
0.0	834	783	735	690	648	607	569	534	500	469	439	411	385	360	337
0.1	868	814	764	717	672	630	591	554	519	486	455	426	399	373	349
0.2	904	848	795	746	699	656	614	576	539	505	473	442	414	387	362
0.3	943	884	829	777	729	683	640	599	561	525	492	460	431	403	377
0.4	986	924	866	812	761	713	667	625	585	548	512	479	448	419	392
0.5	1033	968	907	850	796	745	698	653	611	572	535	501	468	438	409
0.6	1085	1016	952	891	834	781	731	684	640	599	560	524	490	458	428
0.7	1142	1069	1001	937	877	821	768	718	672	629	588	550	514	480	449
0.8	1206	1129	1056	988	925	865	809	757	708	662	618	578	540	505	472
0.9	1277	1195	1118	1046	978	914	855	799	747	699	653	610	570	533	497
1.0	1358	1270	1187	1110	1038	970	907	848	792	740	692	646	603	564	526

P_M	Conflicting Flow Rate, $v_{c,9}$														
	1600	**1700**	**1800**	**1900**	**2000**	**2100**	**2200**	**2300**	**2400**	**2500**	**2600**	**2700**	**2800**	**2900**	**3000**
	Potential Capacity, $c_{mp,9}$														
0.0	315	295	276	257	241	225	210	196	183	171	159	148	138	129	120
0.1	327	305	285	267	249	233	217	203	189	177	165	154	143	134	125
0.2	339	317	296	277	258	241	225	211	197	183	171	160	149	139	129
0.3	352	329	308	287	269	251	234	219	204	191	178	166	155	144	134
0.4	367	343	320	299	280	261	244	228	212	198	185	173	161	150	140
0.5	383	358	334	312	292	272	254	237	222	207	193	180	168	157	146
0.6	400	374	349	326	305	285	266	248	231	216	202	188	175	164	153
0.7	419	392	366	342	319	298	278	260	242	226	211	197	184	171	160
0.8	441	412	385	359	335	313	292	273	255	238	222	207	193	180	168
0.9	465	434	405	378	353	330	308	287	268	250	233	218	203	189	177
1.0	492	459	429	400	373	349	325	304	283	264	247	230	214	200	186

4.4.14 Capacity for Right Turning from Minor

****TABLE 4.16: POTENTIAL CAPACITY FOR RIGHT TURN FROM MINOR FOR SINGLE LANE (MOVEMENT X= 7)**

P_M	Conflicting Flow Rate, $v_{c,7}$														
	100	**200**	**300**	**400**	**500**	**600**	**700**	**800**	**900**	**1000**	**1100**	**1200**	**1300**	**1400**	**1500**
	Potential Capacity, $c_{mp,7}$														
0.0	660	609	561	517	477	439	404	372	342	315	290	266	245	225	207
0.1	683	630	581	536	494	455	419	386	355	327	301	276	254	234	187
0.2	708	653	602	555	512	472	435	400	369	339	312	287	264	243	195
0.3	735	678	625	577	532	490	452	416	383	353	325	299	275	253	203
0.4	763	704	650	599	553	510	470	433	399	367	338	312	287	264	212
0.5	794	733	676	624	576	531	489	451	416	383	353	325	299	276	222
0.6	828	764	705	651	600	554	511	471	434	400	369	340	313	288	232
0.7	864	798	737	680	627	579	534	492	454	419	386	356	328	302	243
0.8	904	835	771	711	657	606	559	516	476	439	405	373	344	317	255
0.9	947	875	808	746	689	636	587	542	500	461	425	392	362	333	268
1.0	995	920	849	784	724	669	617	570	526	485	448	413	381	352	283

	Conflicting Flow Rate, $v_{c,7}$														
	1600	**1700**	**1800**	**1900**	**2000**	**2100**	**2200**	**2300**	**2400**	**2500**	**2600**	**2700**	**2800**	**2900**	**3000**
	Potential Capacity, $c_{mp,7}$														
0.0	190	174	160	147	134	123	113	104	95	87	80	73	67	61	56
0.1	197	181	166	153	140	128	118	108	99	91	83	76	70	64	51
0.2	205	189	173	159	146	134	123	113	104	95	87	80	73	67	54
0.3	214	197	181	166	153	140	129	118	108	99	91	84	77	70	56
0.4	223	205	189	174	160	147	135	124	114	104	96	88	81	74	59
0.5	233	215	198	182	167	154	141	130	119	109	101	92	85	78	62
0.6	244	225	207	190	175	161	148	136	125	115	106	97	89	82	66
0.7	256	236	217	200	184	169	156	143	132	121	111	102	94	87	69
0.8	269	248	228	210	194	178	164	151	139	128	118	108	100	91	73
0.9	283	261	241	222	204	188	173	159	147	135	124	114	105	97	78
1.0	299	276	254	234	216	199	183	169	155	143	132	121	112	103	83

**TABLE 4.17: POTENTIAL CAPACITY FOR RIGHT TURN FROM MINOR FOR MULTILANE (MOVEMENT X = 7)

P_M	Conflicting Flow Rate, $v_{c,7}$														
	100	***200***	***300***	***400***	***500***	***600***	***700***	***800***	***900***	***1000***	***1100***	***1200***	***1300***	***1400***	***1500***
	Potential Capacity, $c_{mp,7}$														
0.0	671	617	567	521	479	440	403	370	340	311	285	261	239	219	201
0.1	694	638	587	539	495	454	417	382	350	321	294	270	247	226	207
0.2	719	661	607	558	512	470	431	395	362	332	304	278	255	233	214
0.3	746	685	629	578	530	486	446	409	375	343	315	288	264	241	221
0.4	775	712	653	599	550	504	462	424	388	356	326	298	273	250	229
0.5	807	740	679	623	571	524	480	440	403	369	338	309	283	259	237
0.6	841	771	707	649	595	545	499	457	419	384	351	321	294	269	246
0.7	878	805	738	676	620	568	520	476	436	399	365	334	306	280	256
0.8	918	842	771	707	647	593	543	497	455	416	381	349	319	292	267
0.9	963	882	808	740	678	621	568	520	476	435	398	364	333	305	278
1.0	1012	927	849	777	711	651	596	545	499	456	417	381	349	319	291

	Conflicting Flow Rate, $v_{c,7}$														
	1600	***1700***	***1800***	***1900***	***2000***	***2100***	***2200***	***2300***	***2400***	***2500***	***2600***	***2700***	***2800***	***2900***	***3000***
	Potential Capacity, $c_{mp,7}$														
0.0	183	168	153	140	128	117	107	97	89	81	74	67	61	56	51
0.1	189	173	158	145	132	121	110	101	92	84	76	70	63	58	53
0.2	195	179	163	149	136	125	114	104	95	86	79	72	66	60	54
0.3	202	185	169	154	141	129	118	107	98	89	81	74	68	62	56
0.4	209	191	175	160	146	133	122	111	101	92	84	77	70	64	58
0.5	217	198	181	165	151	138	126	115	105	96	87	80	73	66	60
0.6	225	206	188	172	157	143	131	119	109	99	91	83	75	69	63
0.7	234	214	195	179	163	149	136	124	113	103	94	86	78	72	65
0.8	244	223	203	186	170	155	142	129	118	108	98	90	82	75	68
0.9	254	232	212	194	177	162	148	135	123	112	102	94	85	78	71
1.0	266	243	222	203	185	169	155	141	129	117	107	98	89	81	74

4.4.15 Shared Lane Capacity

Minor street Approaches:

Equation 4. 7 is used to compute shared lane capacity.

$$C_{SH} = \frac{\sum_y V_y}{\sum_y \left(\frac{V_y}{C_{m,y}}\right)} \qquad (4.7)$$

C_{SH} - capacity of the shared lane (veh/hr)
V_y - flow rate for movement y in a shared lane (veh/hr)
$C_{m,y}$ - movement capacity for movement y in a shared lane (veh/hr)

Major Street Approaches:

Equation 4.8 is a derived equation from Equation 4.6 for potential capacity for queues on a major street with shared right-turn lanes may be taken into consideration.

$$\dot{P}_{0,j} = 1 - \frac{1 - P_{0,j}}{1 - \left\{\frac{v_{i1}}{s_{i1}} + \frac{v_{i2}}{s_{i2}}\right\}} \qquad (4.8)$$

Where,
$P_{0,j}$ = probability of queue-free state for movement j assuming an exclusive right-turn lane on major street
j = 1,4 (major street right turning traffic streams)
i1 = 2,5 (major street through traffic streams)
i2 = 3,6 (major street left traffic streams
s_{i1} = saturation flow rate for the major street through traffic stream (veh/hr) – this parameter can be measured in the field
s_{i2} = saturation flow rate for the major street left turning traffic stream (veh/hr) – this parameter can be measured in the field
v_{i1} = major street through flow rate (veh/hr)
v_{i2} = major street left turning flow rate (0 if an exclusive left turn lane is provided (veh/hr))

4.4.16 Estimating Queue Lengths

Queue length estimation is an important calculation for unsignalised intersections. It is stated in TRB, 2000, theoretical studies and empirical observations have demonstrated that the probability distribution of queue lengths for any minor movement at an unsignalised intersection is a function of the capacity of the movement and the volume of traffic being served during the analysis period.

Equation 4.9 and **FIGURE 4.8** can be used to calculate and to predict the 95th percentile queue length for any minor movement at an unsignalised intersection during the 15 minutes peak hour period.

$$Q_{95} \approx 900T\left[\frac{V_x}{C_{m.x}} - 1 + \sqrt{\left(\frac{V_x}{C_{m,x}} - 1\right)^2 + \frac{\left(\frac{3600}{C_{m,x}}\right)\left(\frac{V_x}{C_{m,x}}\right)}{150T}}\right]\left(\frac{C_{m,x}}{3600}\right) \tag{4.9}$$

Where,

Q_{95} = 95th percentile queue (veh)
V_x = flow rate for movement x (veh/hr)
$C_{m.x}$ = capacity of movement x (veh/hr)
T = analysis time period (h) (T=0.25 for a 15 minutes period)

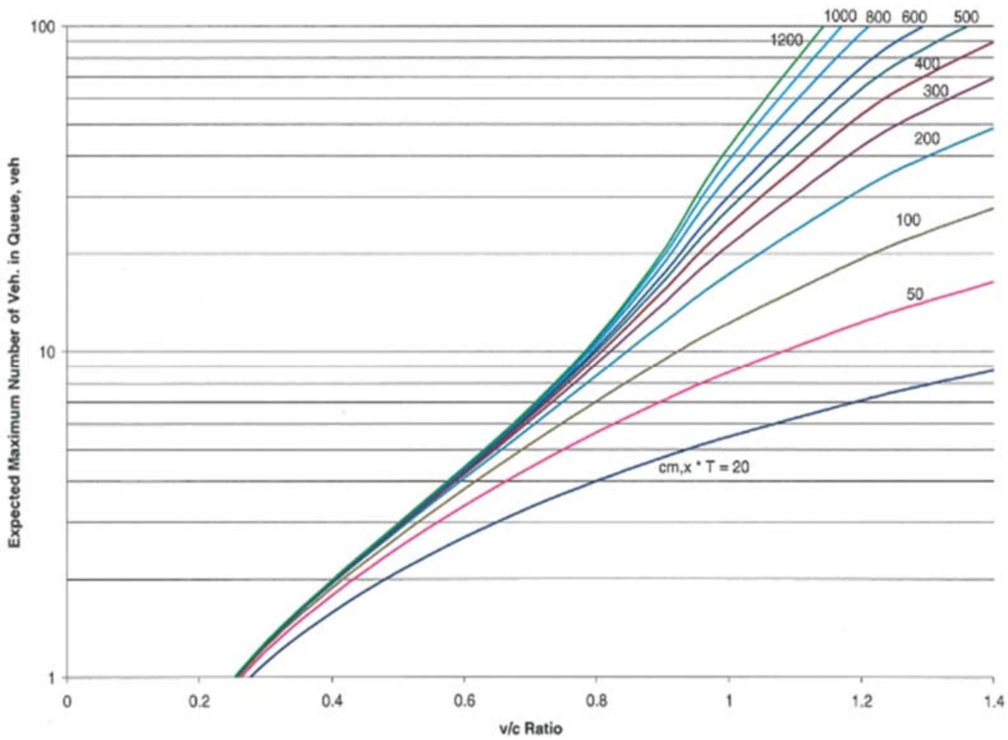


**FIGURE 4.8: 95th PERCENTILE QUEUE LENGTH

4.4.17 Controlled Delay

Total delay is the difference between the travel time actually experienced and the reference travel time that would result during base conditions, in the absence of incident, controlled traffic, or geometric delay. It includes initial deceleration delay, queue move-up time, stopped delay, and final acceleration delay. Controlled delay in field measurements is defined as the total elapsed time from the time a vehicle stops at the end of queue to the time the vehicle departs from the stop line. This total elapsed time includes the time required for the vehicle to travel from the last-in-queue position to the first-in-queue position, including deceleration of vehicles from free-flow speed to the speed of vehicles in queue.

Average controlled delay for any particular minor movement is a function of the capacity of the approach and the degree of saturation. Equation 4.10 is the analytical model used to estimate controlled delay. It assumes that the demand is less than capacity for the period of analysis. If the degree of saturation is greater than 0.9, average controlled delay is significantly affected by the length of the analysis period. Mostly, a 15-minute period is recommended. During the 15-minute period, if demand exceeds capacity, the delay calculated may not be accurate. In this case, the period of analysis should be extended to include the period of oversaturation.

$$D = \frac{3600}{C_{m,x}} + 900T\left[\frac{V_x}{C_{m,x}} - 1 + \sqrt{\left(\frac{V_x}{C_{m,x}} - 1\right)^2 + \frac{\left(\frac{V_x}{C_{m,x}}\right)\left(\frac{3600}{C_{m,x}}\right)}{450T}}\right] + 5 \qquad (4.10)$$

Where

D = controlled delay (sec/veh)

V_x =movement volume (veh/hr)

$C_{m.x}$ =capacity of movement x (veh/hr)

T = 0.25 (analysis time period in hours)

The constant value of 5 s/veh is added to the Equation 4.10 to take into consideration the acceleration and deceleration of vehicles. This equation is graphically shown in **FIGURE 4.9**, based on a 15-minute analysis and for a discrete range of capacities.

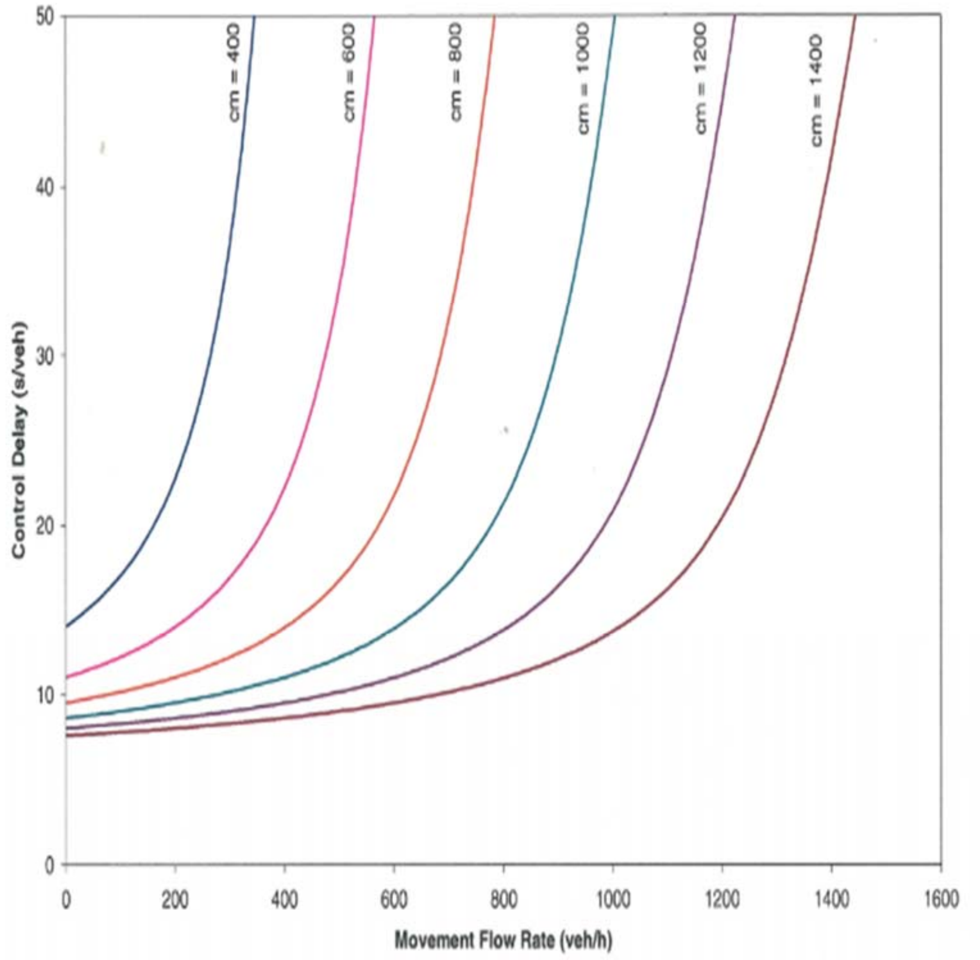


****FIGURE 4.9: CONTROLLED DELAY AND FLOW RATE**

4.4.18 Level of Service

Level of Service (LOS) for an unsignalised intersection is determined by the estimation of controlled delay for each movement. LOS is not defined for the intersection as a whole. LOS criteria are given in **TABLE 4.18**.

****TABLE 4.18: LEVEL OF SERVICE FOR UNSIGNALISED INTERSECTION**

Level of Service	Average Controlled Delay (sec/veh)
A	0 - 10
B	> 10 - 15
C	> 15 - 25
D	> 25 - 35
E	> 35 - 50
F	> 50

4.4.19 Application

The analysis of TWSC intersections is usually being used for existing intersection either to evaluate operational conditions or to estimate the effects under current traffic demands.

The procedure for analyzing unsignalised intersection is divided into three modules, based on the following calculations;

(a) Initial Calculations

Worksheet 1 through 5 are used to record input conditions, compute the critical gap and follow-up time, and determine the flow patterns that result from any upstream signalised intersections that may affect the capacity of the subject intersection.

(b) Capacity Calculations

Worksheet 6 through 9 are used to compute the capacity of each movement and make adjustments for the effects of two stage gap acceptance, shared lanes, or flared minor street approaches.

(c) Delay and LOS Calculations

In this module, worksheets are used to calculate the delay, queue length and LOS for each approach.

(i) Sequence of Capacity Calculation

Priority of gap use and movements are important to the computation of capacity. The computational sequence is as follows:

- Left turns from the minor street
- Right turns from the major street
- Through movements
- Right turns from the minor street

(ii) Computational Steps

The following steps describe each computation and summarized using worksheets. According to TRB, 2000 there are several worksheets to be filled for the computational stage.

- Geometries and Movements (Worksheet 1)
  The sketch shows designated movement numbers, v_1 through v_2 denoting major-street movements and v_7 through v_{12} denoting minor-street movements. Lane arrangement, grade, street particular and geometric data are entered in appropriate fields and columns.

- Volume Adjustment (Worksheet 2)
  Measured or forecast volumes (veh/hr) for each movement are used to compute hourly flow rates by dividing volume by PHF. Proportion of heavy vehicles (HV) is the percentage of heavy vehicles divided by 100 and is used to compute the critical gap and follow up time.

- Site Characteristics (Worksheet 3)
  Information about lanes and traffic movements is entered in this worksheet.

- Critical Gap and Follow-up Time (Worksheet 4)
  TABLE 4.3, Equation 4.1 and Equation 4.2 are used to calculate the critical gap and follow-up time, which is used in Equation 4.3 to determine potential capacities. The presence of motorcycle will take into consideration with some adjustment according to **TABLE 4.4**.

- Impedance and Capacity Calculation (Worksheet 6)
  The capacity for each movement is calculated using this worksheet. There will be some equations as shown in the worksheet for calculation convenience. Flow rates are entered to worksheets 1 and 2. Calculation must be made in order according to the turning priority, considering first the left turns from minor street, followed by right turns from the major street, etc.

- Shared Lane Capacity (Worksheet 8)
  Equation 4.7 is used to compute shared lane capacity in this worksheet.

- Controlled Delay, Queue Length, Level of Service (Worksheet 10)
  Worksheet 10 is used to compute controlled delay, average queue length, and level of service. Controlled delay for each movement can be estimated from **FIGURE 4.11** and Equation 4.10. The 95th percentile queue can be estimated from Equation 4.9 and LOS can be determined from **TABLE 4.18**

(iii) Planning and Design Applications

In order to plan, the analyst requires geometric and traffic flow data, and base values of critical gap and follow-up time are used (refer **Table 4.3**). The effect of upstream signals, two-stage gap acceptance and flared left-turn approaches are normally not taken into consideration in the planning analysis. Malaysian HCM excludes the methodology of these parts.

Planning stage uses the similar worksheets with some exceptions as noted below:

- Worksheet 1 is used to describe basic conditions.
- Worksheet 2 is used to summarize the vehicles volumes.
- Worksheet 3 is used to note the lane designation for each movement.
- Worksheet 4 in planning analysis usually unused, as the base values are used without adjustment.
- Worksheet 5 is for analyzing upstream signals, therefore it is not applicable for planning analysis.
- Worksheet 6 is used to compute the movement capacities.
- Worksheet 7 is used to include the effects of two-stage gap acceptance when there is a divided roadway.
- Worksheet 8 is used to compute shared-lane capacities.
- Worksheet 9 is unused, since the effect of compute flared minor-street approaches is not applicable.
- Worksheet 10 is not used because the impedance and delay for major through movements are not accounted for in planning analysis.
- Worksheet 11 is used to compute capacity, delay and LOS for rank 1 vehicles.

Note**:**

Malaysian Highway Capacity Manual only provides the essential worksheets, and which are suitable to the various conditions. These worksheets are originally taken from US HCM 2000, with some adjustments to take into consideration of Malaysian road condition.

4.4.20 Potential Improvements

It should be noted that the above methodology is not a formal warrant for the consideration of signalisation. Where unacceptable levels of service are found, improvements such as channelisation, lane use controls, sight distance improvements, multiway stop control etc. should be considered first. Only when such improvements do not improve the level of service should signalisation be considered.

4.5 Roundabouts

4.5.1 Introduction

A roundabout is an intersection with a central island around which traffic must travel clockwise and into which the entering traffic must yield to circulating traffic. Not all circular intersections can be classified as roundabouts.

4.5.2 Types of Roundabouts

Roundabouts can be classified into three (3) basic categories according to size and number of lanes:

(a) Mini-roundabouts (**FIGURE 4.10 (A)**)

(i) Used in low-speed urban environments, with average operating speeds of 50 km/h or less.
(ii) Right-of-way constraint
(iii) Inexpensive because only need minimal additional pavement at the intersecting roads
(iv) Pedestrian-friendly with short crossing distance and very low vehicular speed on approaches and exits.

(b) Single-lane roundabouts (**FIGURE 4.10 (B)**)

(i) Having single entry lane at all legs and one circulatory lane.
(ii) Allows slightly higher speeds at the entry, on the circulatory roadway, and at the exit.

(c) Multi-lane roundabouts (**FIGURE 4.10 (C)**)

(i) All roundabouts that have at least one entry with two or more lanes.
(ii) May have a different number of lanes on one or more approaches.
(iii) Also include roundabouts with entries on one or more approaches that flare from one to two or more lanes.
(iv) Speeds at the entry, on the circulatory roadway, and at the exit are similar or may be slightly higher than those for the single-lane roundabouts.

Any of the 3 categories may be appropriate for application in rural or urban areas. Roundabouts in urban areas may need smaller inscribed circle diameters due to smaller design vehicles and constraints of existing right-of-way. They may also include more extensive pedestrian and bicycle features. Roundabouts in rural areas typically have higher approach speeds and thus may need special attention to visibility, approach alignment, and across-sectional details.

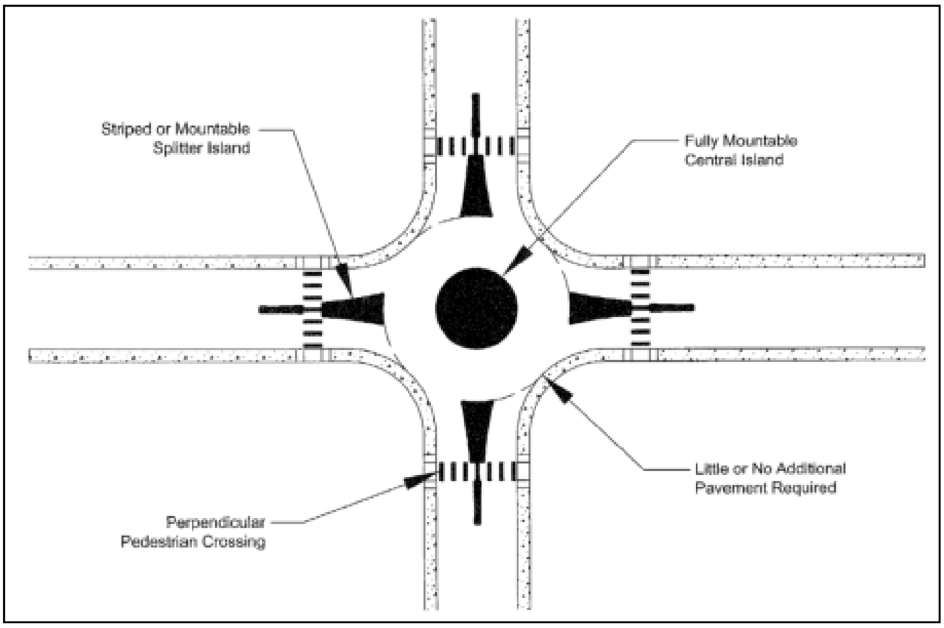


FIGURE 4.10 (A): TYPICAL MINI-ROUNDABOUT

Source: AASHTO 2011, Chapter 9, Figure 9-11.

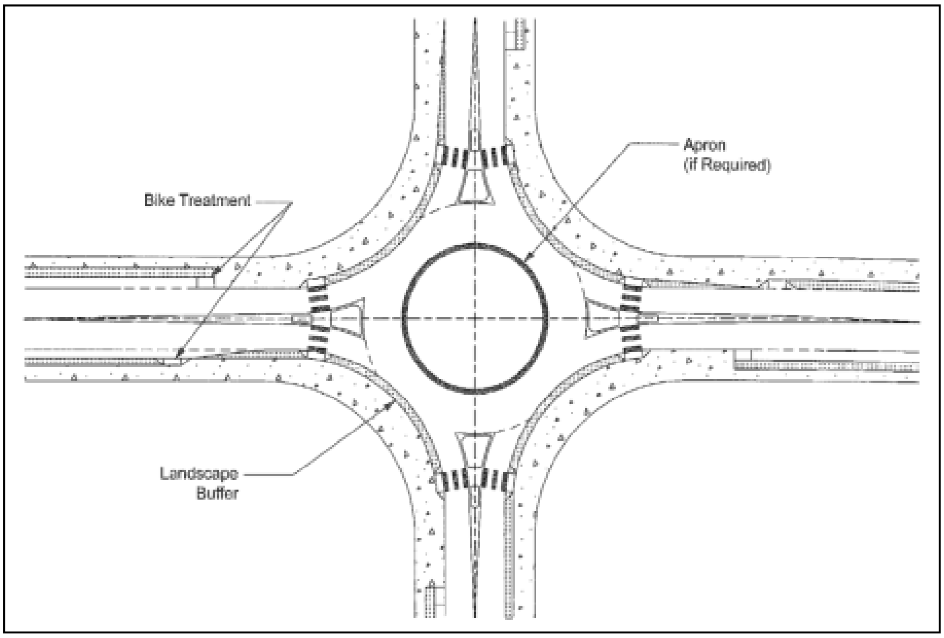


FIGURE 4.10 (B): TYPICAL SINGLE-LANE ROUNDABOUT

Source: AASHTO 2011, Chapter 9, Figure 9-12.

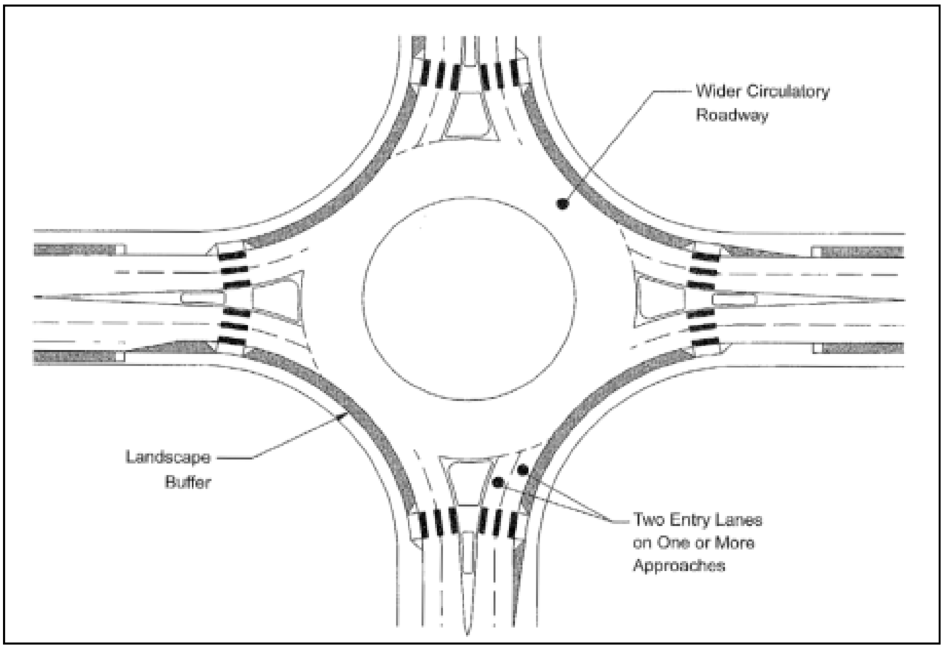


FIGURE 4.10 (C): TYPICAL MULTI-LANE ROUNDABOUT
Source: AASHTO 2011, Chapter 9, Figure 9-13.

4.5.3 General Safety Performance of Roundabouts

In general, a well-designed roundabout is the safest type of intersection control. 'Before and after' type studies have shown that in general, fewer vehicle accidents occur at roundabouts than at intersections containing traffic signals, stop or give way signs. The primary reason for this is that the potential relative speeds of vehicles are considerably lower for a well-designed roundabout than for other types of at-grade intersections.

FIGURE 4.11 shows two intersection treatments for roadways that cross at a 90° angle. The desired speed on each of the cross roads is 60 km/h. The upper diagram in **FIGURE 4.11** shows a typical at-grade intersection treatment. The potential relative speed of vehicles on adjacent roadways at this intersection is 85 km/h.

The lower diagram in **FIGURE 4.11** shows a roundabout at the intersection of these cross roads. The potential relative speed of entering and circulating vehicles at this roundabout is 46 km/h. This value is much lower than the 85 km/h for the at-grade intersection.

Higher potential relative speeds of vehicles will result in higher multiple vehicle accident rates and greater accident severity. The at-grade intersection will generally record significantly higher multiple vehicle accident rates than the roundabout.

Well-designed roundabouts achieve a lower potential relative speed of vehicles on the cross roads primarily because of the presence of entry curvature. Entry curvature limits the speed at which drivers can enter the circulating carriageway.

Conversely, a poorly designed roundabout with little entry curvature or deflection results in high speeds through the roundabout creating high potential relative speeds between vehicles. Multiple vehicle accident rates at these roundabouts can actually be higher than for an equivalent at-grade intersection. Therefore, it is important that designers give special attention to geometric design of roundabouts.

Special consideration must be given to pedestrian movement(s) at roundabouts. While not necessarily less safe than other intersection types, children and elderly pedestrians feel less safe at roundabouts, particularly at exits. This is because, unlike traffic signals, roundabouts do not give priority to pedestrians over through traffic. It is also important to note that several studies have shown that roundabouts increase the risk of accidents to cyclists. Roundabouts designed with good entry curvature (as discussed above) minimize cyclist accidents.

Note that at locations where there are high levels of cycle and pedestrian traffic, roundabouts may not be the most appropriate intersection treatment and alternative treatments should be considered.

For roundabouts to perform effectively they must be easily identified in the road system, the layout must be apparent to approaching drivers and the approaches must encourage drivers to enter the intersection slowly. Adequate sight distance should be provided to enable drivers to observe the movements of other vehicles, cyclists, and pedestrians.

Roundabouts operate as a series of separate T-intersections. The approaching drivers are required to give way to the circulating vehicles on the roundabout and to look for an acceptable gap in the circulating traffic so that they can enter in a safe manner. The behavior of the driver is related to the geometry of the roundabout and prevailing traffic conditions.

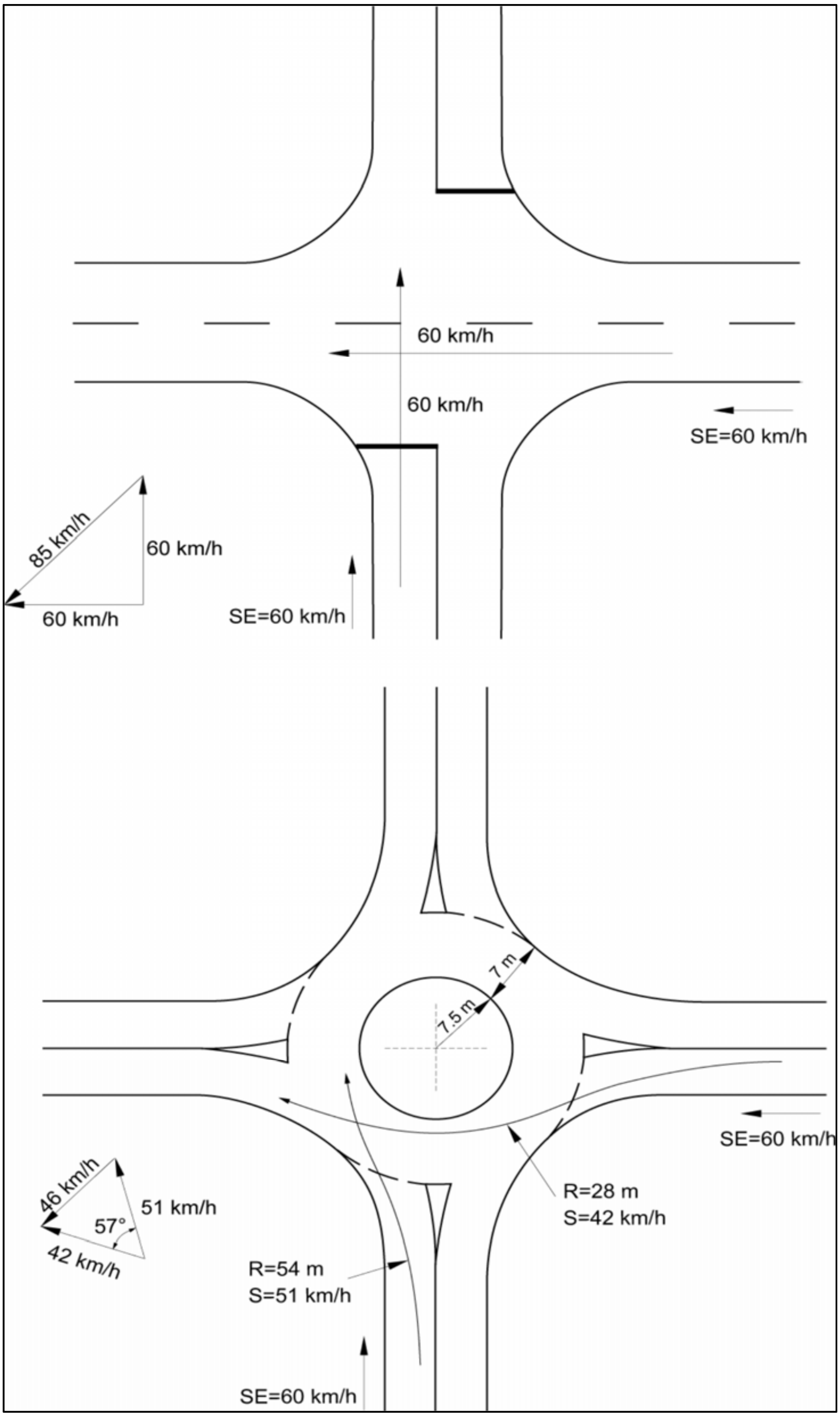


FIGURE 4.11: TWO INTERSECTION TREATMENTS FOR ROADWAYS THAT CROSS AT A 90° ANGLE

Source: Queensland Department Of Main Roads 2006, 'Roundabouts', In Road Planning And Design Manual, Chapter 14, Figure 14.2.

4.5.4 Traffic Capacity of Roundabouts

4.5.4.1 Level of Service Criteria

The Level of Service (LOS) criteria for automobiles in roundabouts are given in Table 4.19. As the table notes, LOS F is assigned if the volume-to-capacity ratio of a lane exceeds 1.0 regardless of the control delay. For assessment of LOS at the approach and intersection levels, LOS is based solely on control delay. The thresholds in Table 4.19 are based on the considered judgment of the Transportation Research Board Committee on Highway Capacity and Quality of Service. As discussed later in this chapter, roundabouts share the same basic control delay formulation with two-way and all-way STOP-controlled intersections, adjusting for the effect of YIELD control. However, at the time of publication of this edition of the Highway Capacity Manual (HCM), no research was available on traveler perception of quality of service at roundabouts. In the absence of such research, the service measure and thresholds have been made consistent with those for other unsignalised intersections, primarily on the basis of this similar control delay formulation.

TABLE 4.19: LOS CRITERIA

Control Delay (s/veh)	LOS By Volume-To-Capacity Ratio[a]	
	v/c ≤ 1.0	v/c > 1.0
0-10	A	F
> 10-15	B	F
> 15-25	C	F
> 25-35	D	F
> 35-50	E	F
> 50	F	F

Source: TRB HCM 2010, Volume 3 Interrupted Flow, Exhibit 21-1.

Note**:** for approaches and intersection wide assessment, LOS is defined solely by control delay.

4.5.4.2 Capacity Concept

The capacity of a roundabout approach is directly influenced by flow patterns. The three flows of interest, the entering flow (V_e), the circulating flow (V_c), and the exiting flow (V_{ex}), are shown in **FIGURE 4.12.**

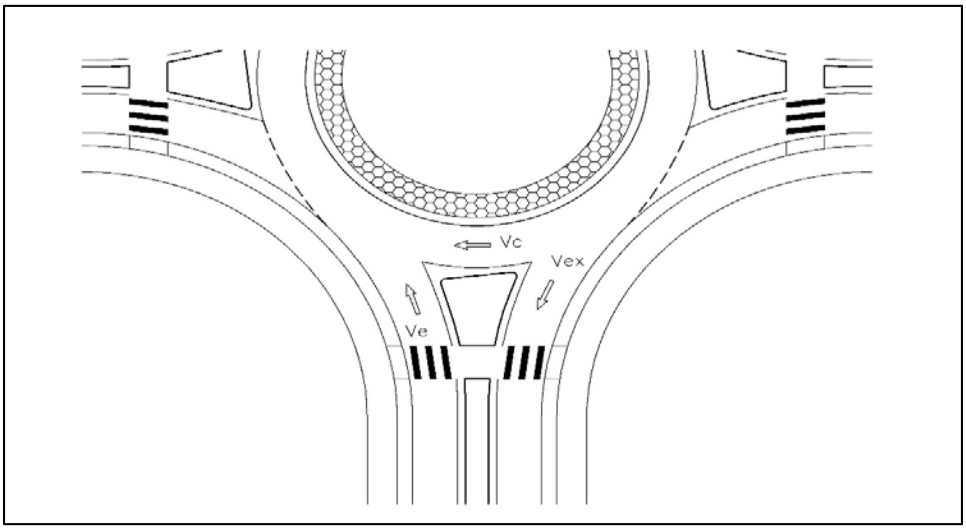


FIGURE 4.12: ANALYSIS ON ONE ROUNDABOUT LEG

Source: TRB HCM 2010, Volume 3 Interrupted Flow, Exhibit 21-2.

4.5.4.3 Input Data Needed

(a) Number and configuration of lanes on each approach
(b) Either of the following: Demand volume for each entering vehicle movement and pedestrian crossing movement during the peak 15 min, or
(c) Demand volume for each entering vehicle movement and each pedestrian crossing movement during the peak hour, and a peak hour factor for the hour
(d) Percentage of heavy vehicles
(e) Volume distribution across lanes for multilane entries
(f) Length of analysis period (e.g., peak 15-min period within the peak hour).

4.5.4.4 Single Lane Roundabouts

The capacity of a single entry lane conflicted by one circulating lane, e.g., a single-lane roundabout, Figure 4.13 (A) is based on the conflicting flow. The equation for estimating the capacity is given by as Equation 4.11:

$$c_{e,pce} = 1{,}130\ e^{(-1.0 x 10^{-3}) v_{c,pce}} \qquad (4.11)$$

$c_{e,pce}$ = lane capacity, adjusted for heavy vehicles (pc/h)

$v_{c,pce}$ = conflicting flow rate (pc/h)

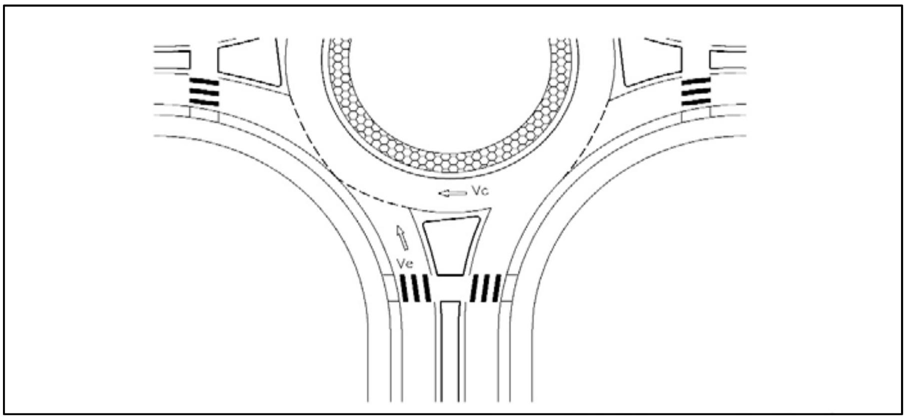


FIGURE 4.13 (A): EXAMPLE OF ONE-LANE ENTRY CONFLICTED BY ONE CIRCULATING LANE

Source: TRB HCM 2010, Volume 3 Interrupted Flow, Exhibit 21-3.

4.5.4.5 Multilane Roundabouts

Multilane roundabouts have more than one lane on at least one entry and at least part of the circulatory roadway. The number of entry, circulating, and exiting lanes may vary throughout the roundabout.

For roundabouts with up to two circulating lanes, the entries and exits can be either one or two lanes wide, plus a possible left-turn bypass lane.

Capacity for Two-Lane Entries Conflicted by One Circulating Lane

Equation 4.12 gives the capacity of each entry lane conflicted by one circulating lane **FIGURE 4.13 (B)** as follows:

$$c_{e,pce} = 1{,}130\ e^{(-1.0 x 10^{-3}) v_{c,pce}} \qquad (4.12)$$

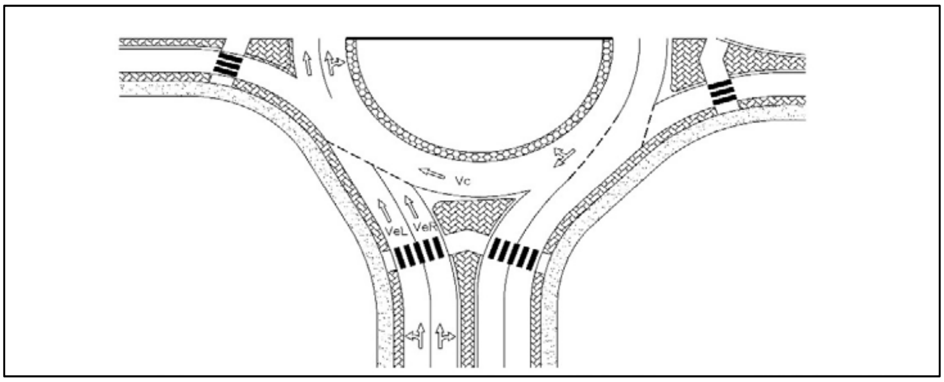


FIGURE 4.13 (B): EXAMPLE OF TWO-LANE ENTRY CONFLICTED BY ONE CIRCULATING LANE

Source: TRB HCM 2010, Volume 3 Interrupted Flow, Exhibit 21-4.

Capacity for One-Lane Entries Conflicted by Two Circulating Lanes

Equation 4.13 gives the capacity of each entry lane conflicted by two circulating lanes **FIGURE 4.13 (C)** as follows:

$$c_{e,pce} = 1{,}130\ e^{(-0.7 x 10^{-3}) v_{c,pce}} \tag{4.13}$$

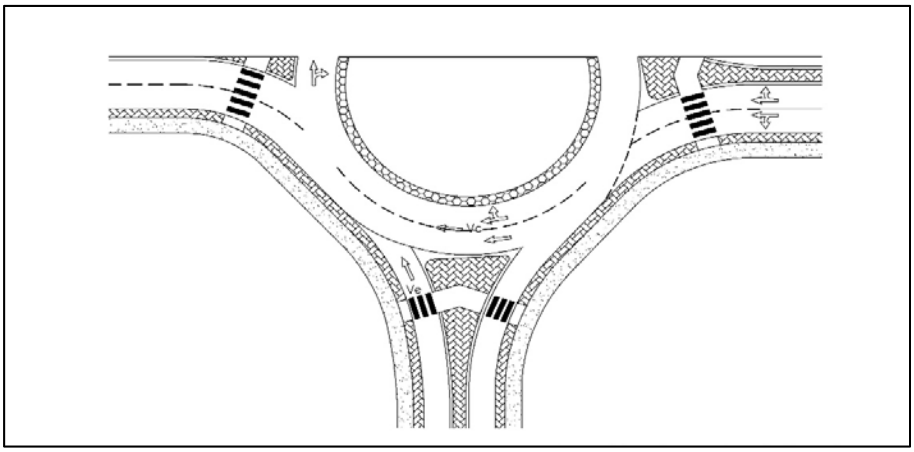


FIGURE 4.13 (C): EXAMPLE OF ONE-LANE ENTRY CONFLICTED BY TWO CIRCULATING LANES

Source: TRB HCM 2010, Volume 3 Interrupted Flow, Exhibit 21-5.

Capacity for Two-Lane Entries Conflicted by Two Circulating Lanes

Equation 4.14 and Equation 4.15 gives the capacity of the left and right lanes, respectively, of a two-lane roundabout entry conflicted by two circulating lanes (**FIGURE 4.13 (D)**).

$$c_{e,R,pce} = 1{,}130\ e^{(-0.7 x 10^{-3}) v_{c,pce}} \tag{4.14}$$

$$c_{e,L,pce} = 1{,}130\ e^{(-0.75 x 10^{-3}) v_{c,pce}} \tag{4.15}$$

Where,

$c_{e,R,pce}$ = capacity of the left entry lane, adjusted for heavy vehicles (pc/h)

$c_{e,L,pce}$ = capacity of the right entry lane, adjusted for heavy vehicles (pc/h)

$v_{c,pce}$ = conflicting flow rate (total of both lanes) (pc/h)

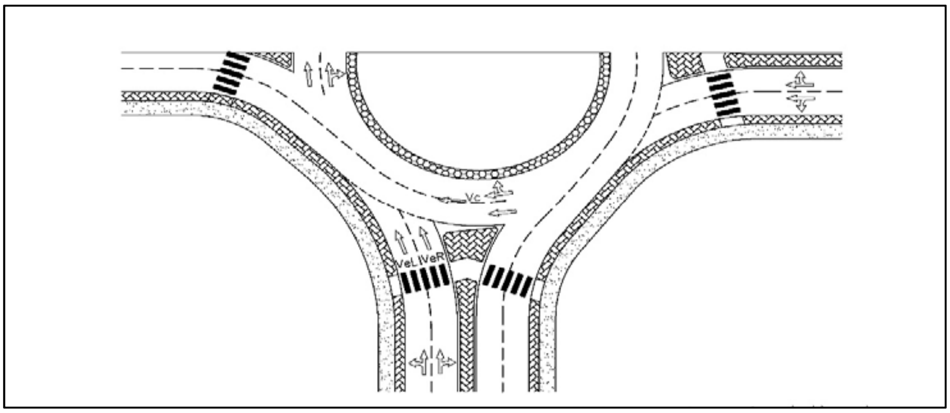


FIGURE 4.13 (D): EXAMPLE OF TWO-LANE ENTRY CONFLICTED BY TWO CIRCULATING LANES

Source: TRB HCM 2010, Volume 3 Interrupted Flow, Exhibit 21-6.

FIGURE 4.13 (E) presents a plot showing Equation 4.11, Equation 4.14 and Equation 4.15. The dashed lines represent portions of the curves that lie outside the range of observed field data.

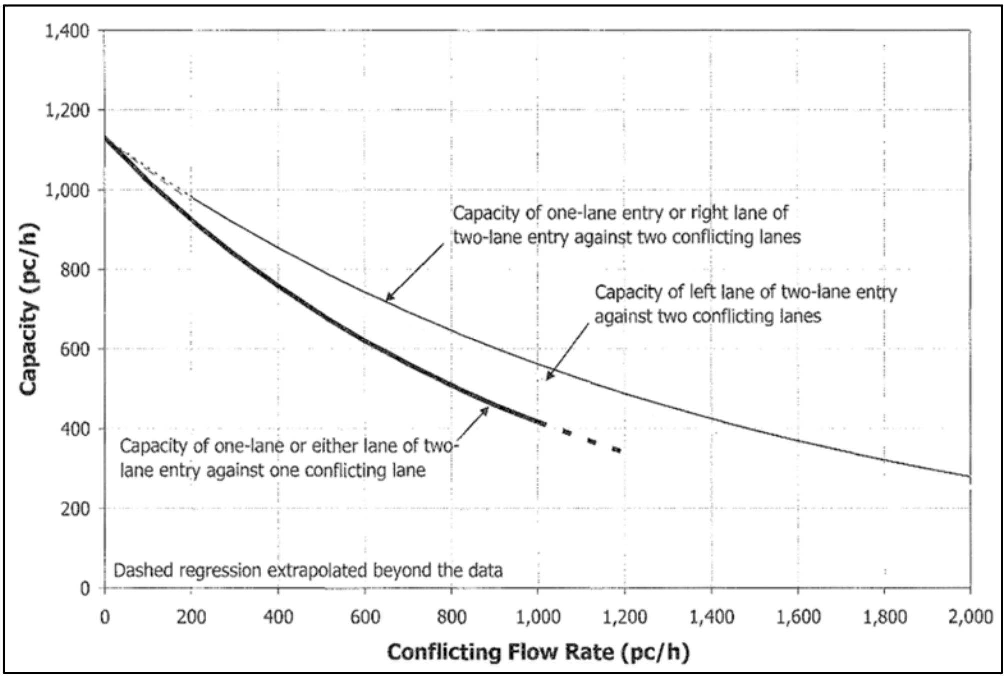


FIGURE 4.13 (E): CAPACITY OF SINGLE-LANE AND MULTILANE ENTRIES

Source: TRB HCM 2010, Volume 3 Interrupted Flow, Exhibit 21-7.

4.5.4.6 Left Turn Bypass Lanes

Two common types of left-turn bypass lanes are used at both single-lane and multilane roundabouts (**FIGURE 4.14**).

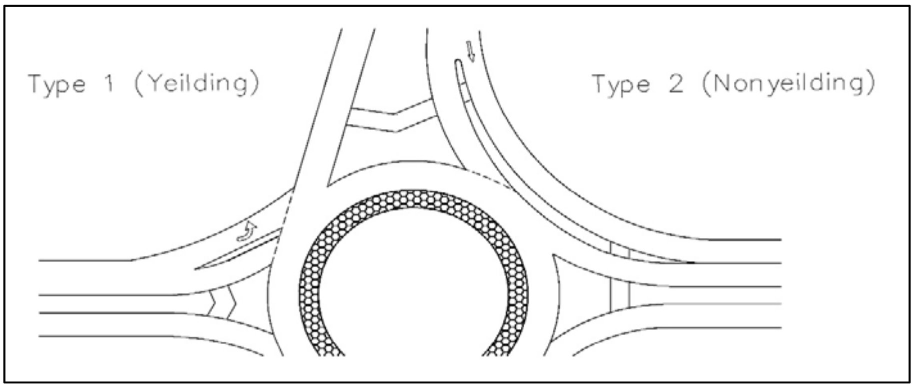


FIGURE 4.14: LEFT -TURN BYPASS LANES

Source: TRB HCM 2010, Volume 3 Interrupted Flow, Exhibit 21-8.

Type 1 (Yielding Bypass Lane)

A Type 1 bypass lane terminates at a high angle, with left-turning traffic yielding to exiting traffic. The capacity for a bypass lane opposed by one exiting lane can be approximated by using Equation 4.16:

$$c_{bypass,pce} = 1{,}130\, e^{(-1.0 x 10^{-3}) v_{c,pce}} \tag{4.16}$$

The capacity for bypass lane opposed by two exiting lanes can be approximated by using Equation 4.17:

$$c_{bypass,pce} = 1{,}130\, e^{(-0.7 x 10^{-3}) v_{c,pce}} \tag{4.17}$$

Where,

$c_{bypass,pce}$ = capacity of the bypass lane, adjusted for heavy vehicles (pc/h); and

$v_{ex,pce}$ = conflicting exiting flow rate (pc/h)

Type 2 (Non-yielding Bypass Lane)

A Type 2 bypass lane merges at a low angle with exiting traffic or forms a new lane adjacent to exiting traffic.

4.5.4.7 Exit Capacity

German research has suggested that the capacity of an exit lane, accounting for pedestrian and bicycle traffic in a typical urban area, is in the range of a 1,200 to 1,300 vehicles per hour (veh/h).

4.5.4.8 Roundabout Analysis Methodology

The capacity of a given approach is computed by using the process shows in **FIGURE 4.15.**

- Step 1 — •Convert movement demand volumes to flow rates
- Step 2 — •Determine circulating and exiting flow rates
- Step 3 — •Determine entry flow rates by lane
- Step 4 — •Determine the capacity of each entry lane and bypass lane in passenger car equivalents (pce)
- Step 5 — •Determine pedestrian impedance to vehicles
- Step 6 — •Convert lane flow rates and capacities into vehicles per hour
- Step 7 — •Compute average control delay for each lane
- Step 8 — •Determine LOS for each lane on each approach
- Step 9 — •Compute the average control delay and LOS for each approach and entire roundabout
- Step 10 — •Compute 95th percentile queues for each lane

FIGURE 4.15: ROUNDABOUT ANALYSIS METHODOLOGY

Source: TRB HCM 2010, Volume 3 Interrupted Flow, Exhibit 21-9.

4.5.5 Sites for Roundabouts

Roundabouts can be used satisfactorily at a wide range of sites, such as:

(a) Intersections on arterial roads in urban areas;
(b) Intersections on rural roads; and
(c) Intersections at motorway terminals (and at terminals of roads performing a motorway function).

Roundabouts perform better at the intersection of roads with roughly similar traffic flows and a high proportion of right turning traffic. Roundabouts can improve safety by simplifying conflicts, reducing vehicle speeds and providing a clearer indication of the driver's right of way compared to other forms of channelisation.

4.5.5.1 Appropriate Sites for Roundabouts

Since so many factors need to be considered, it is not possible to specify whether roundabouts should or should not be installed in various general situations. However, **TABLE 4.20** may be used as a guide to the general applicability of a roundabout treatment to the intersection of various functional road classifications. **TABLE 4.20** should not be used as the only assessment as it is more appropriate to consider each case on its merits, evaluating the advantages and disadvantages of alternative treatments.

TABLE 4.20: PLANNING GUIDE FOR THE USE OF ROUNDABOUTS AT INTERSECTIONS OF VARIOUS ROAD TYPES

	Arterial Road/Rural Highway	Sub-Arterial Road/Rural Road	Collector Road	Local Street
Motorway	D	E	E	E
Arterial Road/Rural Highway	B	B	C	C
Sub-Arterial Road/Rural Road	-	B	B	C
Collector Road	-	-	A	B
Local Street	-	-	-	A

Notation:
A. Likely to be an appropriate treatment
B. Maybe an appropriate treatment
C. Not likely to be an appropriate treatment
D. Not appropriate on motorway itself (may be appropriate as part of an interchange (e.g. at the intersection of ramps and the minor road)
E. Not likely to have an interchange (or roundabout) between a motorway and this road type

Source: Queensland Department of Main Roads 2006, 'Roundabouts', In Road Planning And Design Manual, Chapter 14, Table 14.1.

Roundabouts may be appropriate in the following situations:

(a) At intersections where traffic volumes on the intersecting roads are such that:

(i) STOP or GIVE WAY signs or the T-intersections rule results in unacceptable delays for the minor road traffic. In these situations, roundabouts would decrease delays to minor road traffic, but increase delays to the major road traffic.

(ii) Traffic signals would result in greater delays than a roundabout. It should be noted that in many situations roundabouts provide a similar capacity to signals, but may operate with lower delays and better safety, particularly in off-peak periods.

(b) At intersections where there are high proportions of right-turning traffic:

Unlike most other intersection treatments, roundabouts can operate efficiently with high volumes of right-turning vehicles. Indeed, these right-turning vehicles contribute to good roundabout operation as is illustrated in **Figure 4.16**. In this example the right turner from Leg A to Leg D would stop the through movement from Leg C to leg A thus allowing traffic from Leg D to enter the roundabout. Traffic from Leg D would then stop the through movement from Leg A thus allowing traffic from Leg B to enter the roundabout. Right turners from Leg A in this example would initiate traffic flow on adjacent entries and Leg D which would otherwise experience longer delay.

(c) At rural cross intersections (including those in areas with high desired speeds) at which there is an accident problem involving crossing or right turn (versus opposing) traffic. However, if the traffic flow on the lower volume road is less than about 200 vehicles per day, consideration could be given to using a staggered “T” treatment.

(d) At intersections of arterial roads in outer urban areas where traffic speeds are high and right turning traffic flows are high. A well-designed roundabout could have an advantage over traffic signals in reducing right turn opposed type accidents and overall delays.

(e) At “T” or cross intersections where the major traffic route turns through a right angle. This often occurs on highways in country towns. In these situations, the major movements within the intersection are turning movements.

(f) Where major roads intersect at “Y” or “T” intersections, where a high proportion of right turning traffic exists.

(g) At locations where traffic growth is expected to be high and where future traffic patterns are uncertain or changeable.

(h) At intersections of local roads where it is desirable not to give priority to either road.

(i) At intersections with more than four legs, if one or more legs cannot be closed or relocated or some turns prohibited, roundabouts may provide a convenient and effective treatment whereas:

  (i) With STOP or Give Way signs, it is often not practical to define priorities adequately;
  (ii) Signals may be less efficient due to the large number of phases required (resulting in a high proportion of lost time).

  Two lane roundabouts with more than four legs, however, often cause operational problems and generally should be avoided.

  Care should be taken in assessing the future traffic volumes and their patterns. It is possible that a site considered appropriate for a roundabout now, may become inappropriate in the future, requiring extensive modification to the intersection. Designers should consider the potential to build in flexibility in the design to accommodate possible future changes, particularly when land use changes alter traffic patterns considerably.

4.5.5.2 Inappropriate Sites for Roundabouts

Roundabouts may be inappropriate in the following situations:

(a) Where a satisfactory geometric design cannot be provided due to insufficient space or unfavourable topography, or there is an unacceptably high cost of construction (which includes the cost of property acquisition, service relocations, etc).

(b) Where traffic flows are unbalanced with high volumes on one or more approaches, and some vehicles would experience long delays. This is especially true for roundabouts on high desired speed, high volume rural roads which intersect with a very low volume road. In these cases, the number of single vehicle accidents generated by the roundabout can substantially exceed the number of multiple vehicle accidents generated by an at-grade intersection.

(c) Where a major road intersects a minor road and a roundabout would result in unacceptable delay to the major road traffic. A roundabout causes delay and deflection to all traffic, whereas control by STOP or GIVE WAY signs or the T-intersections rule would result in delays to only the minor road traffic.

(d) Where there is considerable pedestrian activity and due to high traffic volumes it would be difficult for pedestrians to cross any leg.

(e) At an isolated intersection in a network of linked traffic signals. In this situation a signalised intersection linked to the others or simply an at-grade intersection would generally provide a better level of service.

(f) At an isolated intersection where the treatment is inconsistent with the network/link and the expectations of the driver, i.e., design consistency is not maintained.

(g) Where peak period reversible, i.e., tidal flow lanes may be required.

(h) Where large multi-combination or over dimensional vehicles frequently use the intersection and insufficient space is available to provide for their swept turning paths.

(i) Where traffic flows leaving the roundabout would be interrupted by downstream traffic control which could result in queuing back into the roundabout. An example of this is a nearby signalised pedestrian crossing. The use of roundabouts at these sites need not be completely discounted, but they are generally found to be less effective than adopting a signalised intersection treatment.

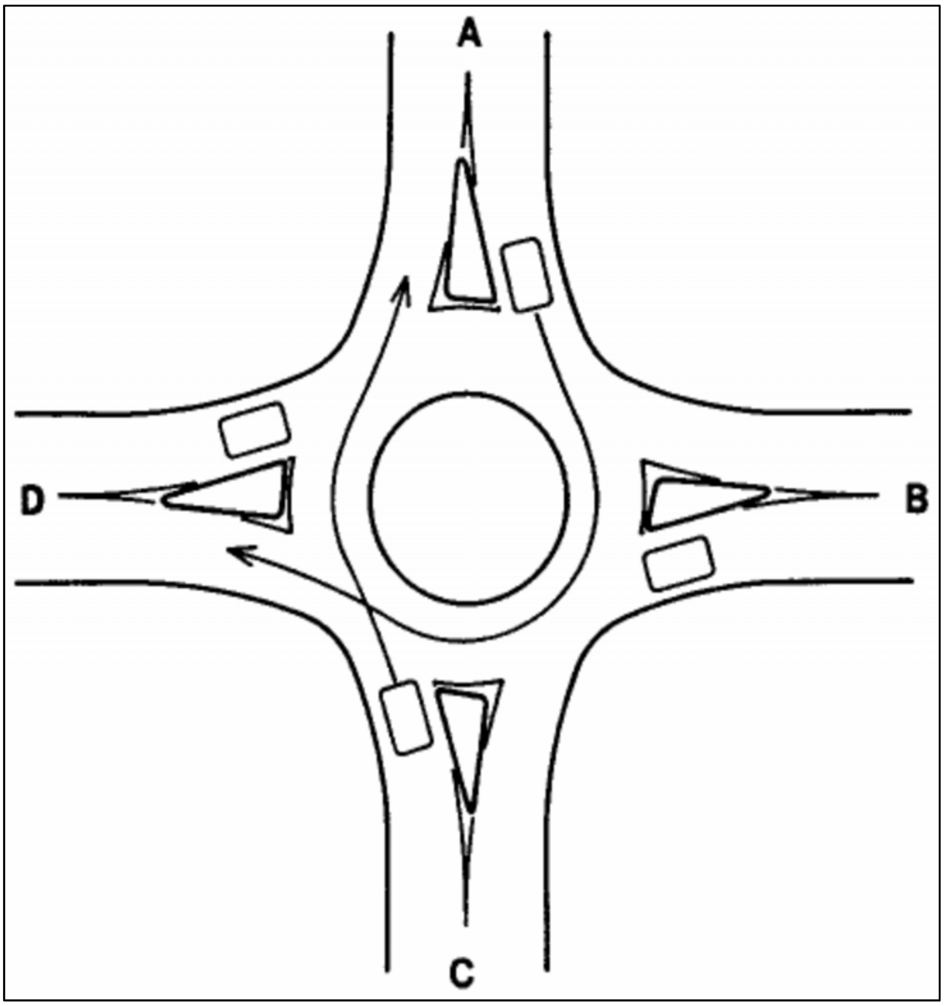


FIGURE 4.16: EFFECT OF TURNING VEHICLES ON ROUNDABOUT OPERATION

Source: Queensland Department Of Main Roads 2006, 'Roundabouts', In Road Planning And Design Manual, Chapter 14, Figure 14.3.

5.0 OTHER RELATED ELEMENTS

5.1 Pedestrian Facilities

5.1.1 General

Pedestrian facilities such as crossings, refuge islands and pedestrian actuated traffic signals are an integral part of intersection design and should be provided where required.

5.1.2 Pedestrian Crossing

(a) Pedestrian crossings should be placed to match the flow line of pedestrian traffic. Pedestrian crossing located against the natural flow of traffic demand would invite jaywalkers outside of it.

(b) Pedestrian crossings should be placed perpendicular to the road. This makes the distance to cross and green time to be allotted to pedestrians the shortest. This is desirable to maintain high capacity.

(c) Pedestrian crossings should be placed closer to the centre of the intersection. Pedestrian crossings placed closer to the centre make the intersection smaller and require less time to pass through it. Smaller intersection has a larger capacity with shorter clearance time in signal phasing.

(d) Pedestrian crossings should be placed where drivers approaching have a fine view of it.

(e) Pedestrian crossings shorter than 15m is recommended. If the road to be crossed is longer than 15m, refuge islands should be provided to enable pedestrians to cross it in two green signals.

(f) The width of pedestrian crossing should be determined for the number of pedestrians and the length of green time allotted to the pedestrian phase. However, every pedestrian crossing having various widths is not desirable. The minimum width should be fixed at 4m and 2m for major roads and minor roads respectively. The width should be increased by whole meters.

(g) Where the pedestrian crossing is used by disabled people such as for the visually impaired, an adequate warning sound system and usage of special materials such as tactile interlocking and guiding blocks, and suitable kerbs where necessary, including handrails on the up-and-down ramps should be considered. To overcome the problem for the hard of hearing and those with speech impediment, adequate highly visible appropriate sign boards should also be considered. (Cross reference with latest ATJ 18/97: Basic Guidelines on Pedestrian Facilities) and in compliance with the guidelines as stipulated in the “Garis Panduan Perancangan Rekabentuk Sejagat”, (JPBD Semenanjung Malaysia, 2011).

PEDESTRIAN FACILITIES

FIGURE 5.1: INTERLOCKING BLOCK LEADING TO PEDESTRIAN CROSSING

FIGURE 5.2: THERMOPLASTIC COLORED ZEBRA CROSSING AT STOP INTERSECTIONS

FIGURE 5.3: TACTILE INTERLOCKING AND GUIDING BLOCK FOR THE VISUALLY IMPAIRED

Although it may theoretically be desirable to place pedestrian crossing at the extension of the side walk, it is usually located several meters (3m to 4m, or at least 1m) behind the extension of boundary line between carriageway and sidewalk. Barriers should be provided along the rounded corner between the two thresholds to pedestrian crossing. This is practiced for the following reasons:

(a) Pedestrian crossings placed on the elongation of intersecting sidewalks may cross in the intersection. In the cases like this, pedestrians tend to wait in the intersection or move to the next intersecting pedestrian crossing without making a complete crossing of the first one.

(b) Left-turning vehicles must often wait for the pedestrians who are crossing the street in the same phase. The pedestrian crossings should be set back to make some space for these vehicles so that they will not be an obstacle to the straight through vehicles.

(c) Rounded corners of sidewalk created by setting back two pedestrian crossings give space for traffic signs and lighting.

(d) Rounded corners protected by barriers provide pedestrians with a psychological effect of a relaxed and safe feeling and invites composed behavior. When space permits, flower pots placed along the corners are further preferable to enhance aesthetics.

(e) Pedestrian crossing should be placed 1m to 2m back from the top of the central median.

5.2 Lighting

Lighting affects the safety of users at intersections and the ease and comfort of traffic operations, especially at night. Intersections which are channelised should include lighting even though it is not warranted. If lighting is not available, the island should be equipped with reflectorized road pavement markers (RRPM). Delineators and chevron sign/ blinking chevron on curve corners approaching non-lighted intersections is encouraged to enhance night visibility.

5.3 Public Utilities

The location and size of underground and overhead public utility services which are close enough to the intersection to be affected together with any planned extension or amplification should be determined in the preliminary stages of the design.

Service tunnels/culverts or ducts should also be provided for future services to avoid the digging of the carriageway pavement. Any open trenching proposal for services that will cross or which runs parallel on the carriageway pavement should not be allowed.

The determination on the relocation of utility services should make use of the current or latest technology such as Ground Mapping by using Ground Penetration Radar (GPR), Horizontal Direct Drilling (HDD), Pipe Jacking and sharing of common box culvert for utilities so as to cause the least inconvenience to road users and damage to the environment (cross reference with the latest ATJ 4/85: Second Schedule – Application for The Installation of Public Utility Service Within Road Reserve).

5.4 Vehicle Parking Restriction

Vehicles parked near intersections can obstruct the flow of turning traffic. Parking should be prohibited within the minimum distance specified in the Road Traffic Ordinance.

Apart from any statutory requirements, on the approach side of a signalised intersection, parking should be prohibited for a distance large enough to store as many vehicles as can cross the STOP line in one phase from the kerbed lane. Parking should also be prohibited between vehicle detectors and the intersection.

There should also be adequate parking restrictions on the exit side of the intersection to enable the kerbed lane vehicles to disperse smoothly.

5.5 Traffic Signs and Lane Markings

All traffic signs and traffic control devices must follow that as laid down in the latest Arahan Teknik (Jalan) 2A/85, 2B/85 and 2D/85. **FIGURE 5.1** indicates the typical lane and pavement markings for a cross urban intersection.

For the latest specification on road marking, All Weather Thermoplastic (AWT) should be considered to enhance the users' visibility in all conditions.

5.6 Drainage

The provision of proper drainage facilities is important for any road and more so at intersections. Careful attention to the requirements for an adequate drainage system and protection of the intersection from flooding should be considered as an integral part of the intersection design.

The drainage system should also take into consideration of any possible hazard to the public by ensuring proper installation works and provision of suitable drainage covers for sumps and manholes.

5.7 Landscaping

Landscaping has been a neglected feature in the design of roads and in particular at intersections. In the consideration of landscaping for intersections, the element of road safety must not be neglected, nor compromised. Within the intersection area, only low shrubs/trees should be considered to ensure that sight distances are not affected.

At any intersection with median landscape, the height of shrubs/trees shall not be more than 0.5m from the road surface and its visibility should not, in any way, be covered or blocked from sight for at least a minimum distance of 50m from the intersection.

In matters of safety where landscaping works are to be carried out, the recommendations and guideline of the Road Authority where the intersections are sited needs to be referred to. In the case of JKR roads, those of the Road Safety Auditor of JKR or other appointed party's proposals and requirements need to be strictly adhered to (cross reference with the latest ATJ 19/97: Intermediate the Guidelines to Road Reserve Landscaping).

5.8 Environmental Consideration

An environmental approach is used throughout the planning, design and construction stages. Energy saving elements which also help maintain the environment and with low maintenance cost such as LED lighting system is recommended. Also, any latest technology in energy saving should be considered in order to incorporate environmental elements in roads/ highways construction.

5.9 Special Pavement Types at Selected Locations

For improvement and to protect the life span of road pavement area and ensure low maintenance cost for at-grade intersections, application of heavy duty interlocking-type pavement or any suitable material of latest technology is recommended (cross reference with the latest ATJ 5/85: Manual on Pavement Design).

APPENDIX

APPENDIX A

GENERAL WARRANTS FOR TRAFFIC CONTROLLED SIGNALS

PLEASE REFER TO LATEST ATJ 13/87-A Guide to the Design Of Traffic Signal (please refer to S.O/S.O.R for latest version of document)

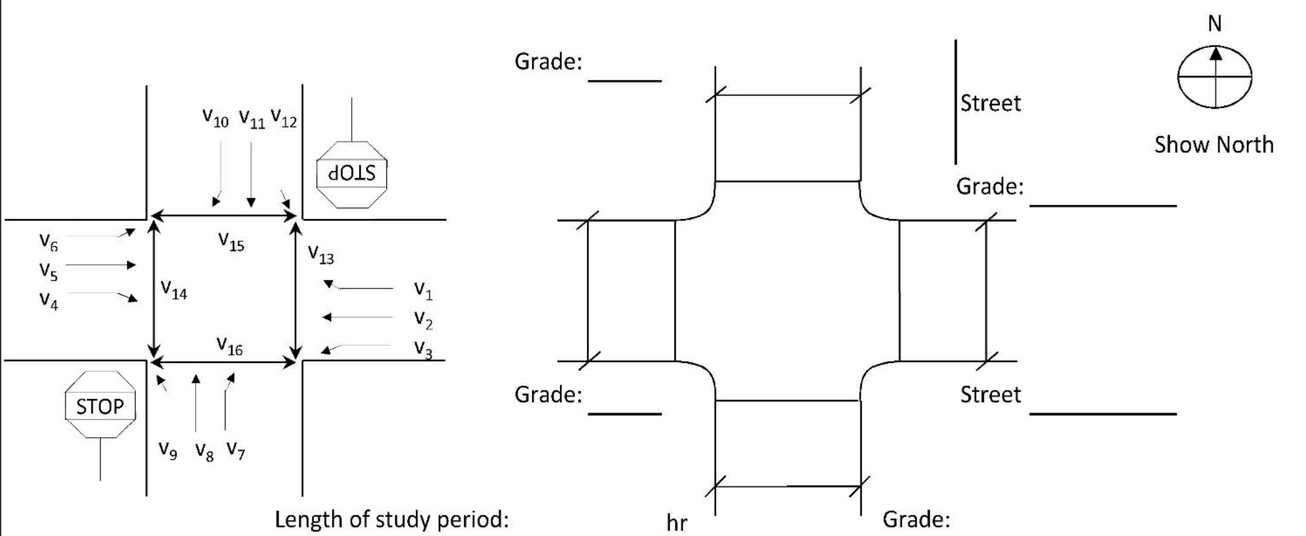
APPENDIX B

WORKSHEET FOR CAPACITY CALCULATIONS FOR UNSIGNALISED INTERSECTIONS

Worksheet 1

General Information		Site Information	
Analyst	________	Intersection	________
Agency or Company	________	Jurisdiction	________
Date Performed	________	Analysis Year	________
Analysis Time Period	________		

Geometry and Movements



Worksheet 2

Vehicle Volumes and Adjustments

	Vehicle Volumes and Adjustments											
Movement	1	2	3	4	5	6	7	8	9	10	11	12
Volume (veh/h)												
Peak-hour factor, PHF												
Hourly flow rate (veh/hr)												
Proportion of motorcycles, P_M												

Pedestrian Volumes and Adjustments

Movement	13	14	15	16
Flow, V_x (ped/hr)				
Lane width, w (m)				
Walking speed, [1] S_p (m/s)				
Percent blockage, fp				

1. Default walking speed = 1.2m/s

Worksheet 3

General Information

Project Description: ______________________________

Lane Designation

Movements	Lane 1	Lane 2	Lane 3	Grade, G	Left Turn Channelized?
1, 2, 3					
4, 5, 6					
7, 8, 9					
10, 11, 12					

Flared Minor-Street Approach

Movement 9 ☐ Yes ☐ No Storage space, n ____________ (number of vehicles)

Movement 12 ☐ Yes ☐ No Storage space, n ____________ (number of vehicles)

Median Storage*

* Includes raised or striped median (RM), or two-way left-turn lane (TWLTL)

Movement 9 ☐ Yes Type ______ ☐ No Storage space, n ____________ (number of vehicles)

Movement 12 ☐ Yes ______ ☐ No Storage space, n ____________ (number of vehicles)

Upstream Signals

	Movements	Distance to Signal, D (m)	Prog Speed S_{prog} (km/h)	Cycle Length, C (s)	Green Time, g_{eff} (s)	Arrival Type	Saturation Flow Rate, s (veh/h)	Progressed Flow, V_{prog} (veh/h)
S_2	protected LT					3		
	TH							
S_5	protected LT					3		
	TH							

Computing Delay to Major-Street Vehicles

Data for Computing Effect of Delay to Major-Street Vehicles	S_2 Approach	S_5 Approach
Shared-lane volume, major-street through vehicles, v_{i1}, blocked by RT		
Shared-lane volume, major-street right-turn vehicles, v_{i2}, blocked by RT		
Saturation flow rate, major-street through vehicles, s_{i1}		
Saturation flow rate, major-street right-turn vehicles, s_{i2}		
Number of major-street through lanes		
Length of study period, T (h)		

TWSC - UNSIGNALISED INTERSECTIONS WORKSHEET

Worksheet 4

General Information

Project Description ______________________________

Critical gap and Follow-Up Time

$$t_{c,x} = t_{c,base} - t_{c,M}P_{M,x}$$

	Major RT		Minor LT		Minor TH		Minor RT	
Movement	1	4	9	12	8	11	7	10
$t_{c,base}$ (Table 4.2)								
$t_{c,m}$ (Table 4.3)								
P_M (from worksheet 2)								
t_c								

$$t_{f,x} = t_{f,base} - t_{f,M}P_M$$

	Major RT		Minor LT		Minor TH		Minor RT	
Movement	1	4	9	12	8	11	7	10
$t_{f,base}$ (Table 4.2)								
$t_{f,m}$ (Table 4.3)								
P_M (from worksheet 2)								
t_f								

TWSC - UNSIGNALISED INTERSECTIONS WORKSHEET

Worksheet 6

General Information

Project Description ______________________________

Impedence and Capacity Calculation

Step 1 : LT from Minor Street	v_9	v_{12}
Conflicting flows (Figure 4.3)	$v_{c,9}$ =	$v_{c,12}$ =
Adjustment factor for capacity (Table 4.4)	A_9 =	A_{12} =
Potential capacity (Equation 4.3)	$c_{mp,9}$ =	$c_{mp,12}$ =
Movement capacity	$c_{mm,9} = c_{mp,9}$ =	$c_{mm,12} = c_{mp,12}$ =
Prob of queue-free state (Equation 4.5)	$P_{0,9}$ =	$P_{0,12}$ =
Step 2 : RT from Major Street	v_4	v_1
Conflicting flows (Figure 4.3)	$v_{c,4}$ =	$v_{c,1}$ =
Adjustment factor for capacity (Table 4.4)	A_4 =	A_1 =
Potential capacity (Equation 4.3)	$c_{mp,4}$ =	$c_{mp,1}$ =
Movement capacity	$c_{mm,4} = c_{mp,4}$ =	$c_{m,1} = c_{p1}\ P_{p,1}$ =
Prob of queue-free state (Equation 4.5)	$P_{0,4}$ =	$P_{0,1}$ =
Step 3 : TH from Minor Street (4-leg intersections only)	v_8	v_{11}
Conflicting flows	$v_{c,8}$	$v_{c,11}$
Potential capacity	$c_{mp,8}$ =	$c_{mp,11}$ =
Ped impedence factor	$p_{p,8}$ =	$p_{p,11}$ =
Capacity adjustment factor due to impeding movement	$f_8 = P_{0,4}\ P_{0,1}\ P_{p,8}$=	$f_{11} = P_{0,11}\ P_{0,1}\ P_{p,11}$=
Movement capacity	$c_{mm,8} = c_{mp,8}$ =	$c_{mm,11} = c_{mp,11}$ =
Prob of queue-free state	$P_{0,8}$ =	$P_{0,11}$ =
Step 4 : RT from Minor Street (4-leg intersections only)	v_7	v_{10}
Conflicting flows	$v_{c,7}$	$v_{c,10}$
Potential capacity	$c_{mp,7}$ =	$c_{mp,10}$ =
Ped impedence factor	$P_{p,7}$ =	$P_{p,10}$ =
Major right, minor through impedence factor	$P_7 = P_{0,11}\ f_{11}$ =	$P_{10} = P_{0,8}\ f_8$ =
Major right, minor through adjusted impedence factor	P_7=	P_{10}=
Capacity adjustment factor due to impeding movements	$f_7 = P_7\ P_{0,12}\ P_{p,7}$=	$f_{10} = P_{10}\ P_{0,9}\ P_{p,10}$=
Movement capacity	$c_{m,7} = f_7\ c_{p,7}$=	$c_{m,10} = f_{10}\ c_{p,10}$=
Step 5 : RT from Minor Street (T- intersections only)	v_7	v_{10}
Conflicting flows (Figure 4.3)	$v_{c,7}$	$v_{c,10}$
Adjustment factor for capacity (Table 4.4)	A_7 =	A_{10} =
Potential capacity (Equation 4.3)	$c_{mp,7}$ =	$c_{mp,10}$ =
Capacity adjustment factor due to impeding movement	$f_7 = P_{0,4}\ P_{0,1}$ =	$f_{10} = P_{0,4}\ P_{0,1}$ =
Movement capacity (Equation 4.6)	$c_{mm,7} = c_{mp,7}\ f_7$=	$c_{mm,10} = c_{mp,10}\ f_{10}$=

Notes

1. For 4-leg intersections use Steps 1,2,3, and 4.
2. For T-intersections use Steps 1,2, and 5.

Worksheet 8

General Information

Project Description ________________________________

Shared-Lane Capacity

$$c_{SH} = \frac{\sum_y v_y}{\sum_y \left(\frac{v_y}{c_{m,y}}\right)} \quad \text{(Equation 4.7)}$$

Movement	v (veh/h)	c_m (veh/h)	c_{SH} (veh/h)
7			
8			
9			
10			
11			
12			

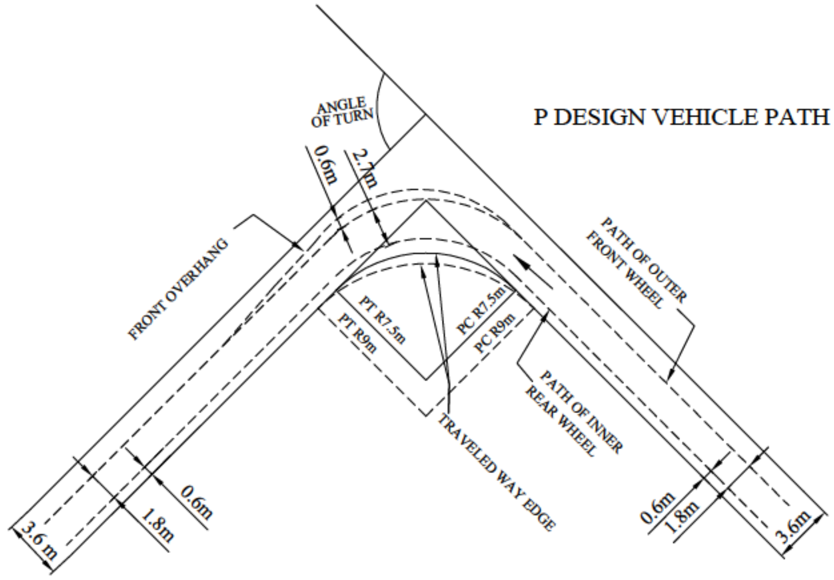
Worksheet 10

Control Delay, Queue Length, Level of Service

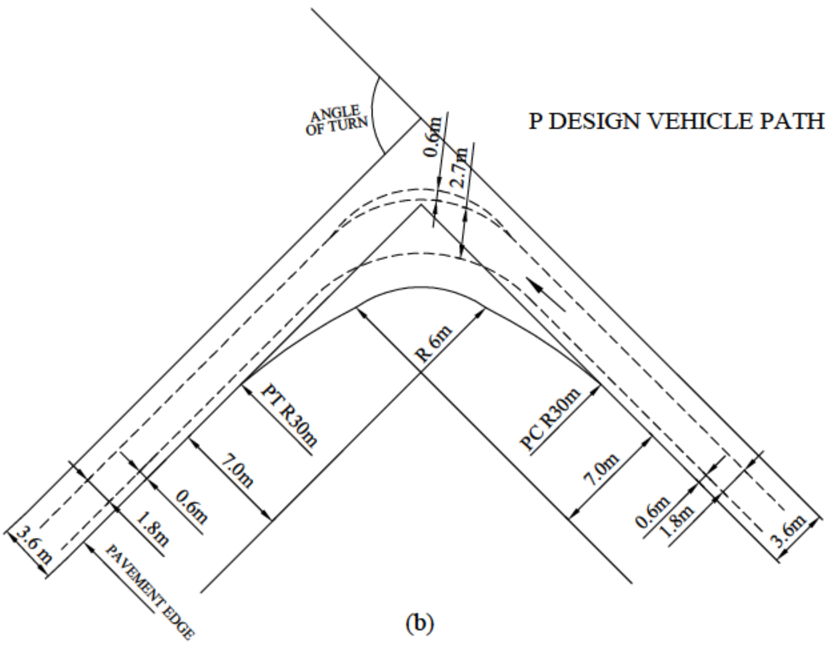
Lane	v(veh/h)	c_m(veh/h)	v/c	Queue Length (Equation 4.9)	Control Delay (Equation 4.10)	LOS (Table 4.1)	Delay and LOS
1 7 8 9							
2 7 8 9							
3 7 8 9							
1 10 11 12							
2 10 11 12							
3 10 11 12							

Movement	v(veh/h)	c_m(veh/h)	v/c	Queue Length (Equation 4.9)	Control Delay (Equation 4.10)	LOS (Table 4.1)
1						
4						

Source: Adapted from MHCM 2006 and TRB HCM 2000, Unsignalised Intersection.

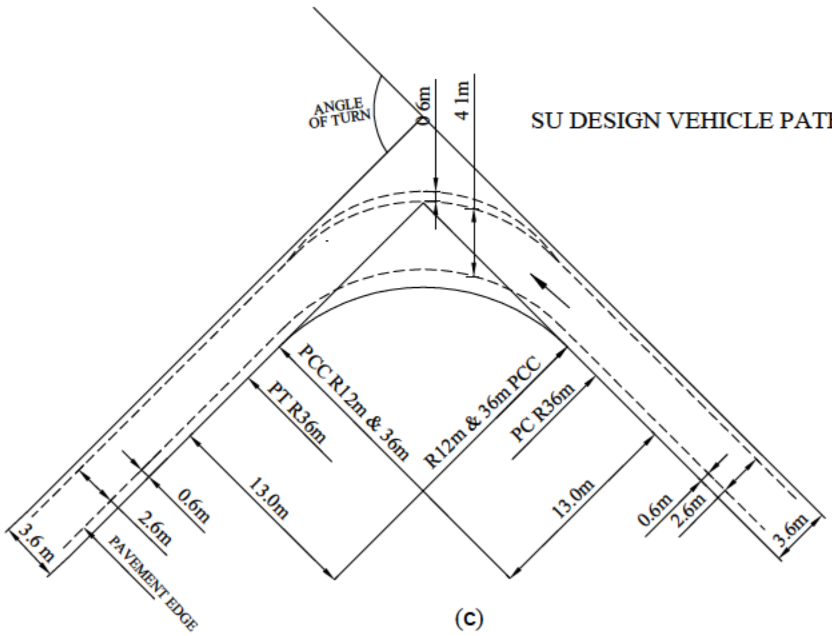


(a)
MINIMUM SIMPLE CURVE 7.5M OR 9M RADIUS

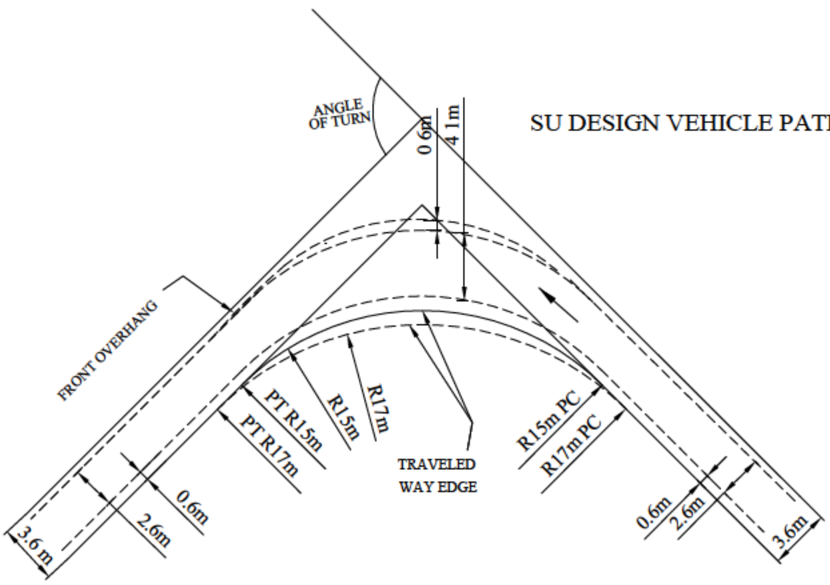


(b)
THREE-CENTERED COMPOUND CURVE
30m - 6m - 30m RADIUS, OFFSET 1m

JKR	*APPENDIX C
Scale : Not to scale	C1



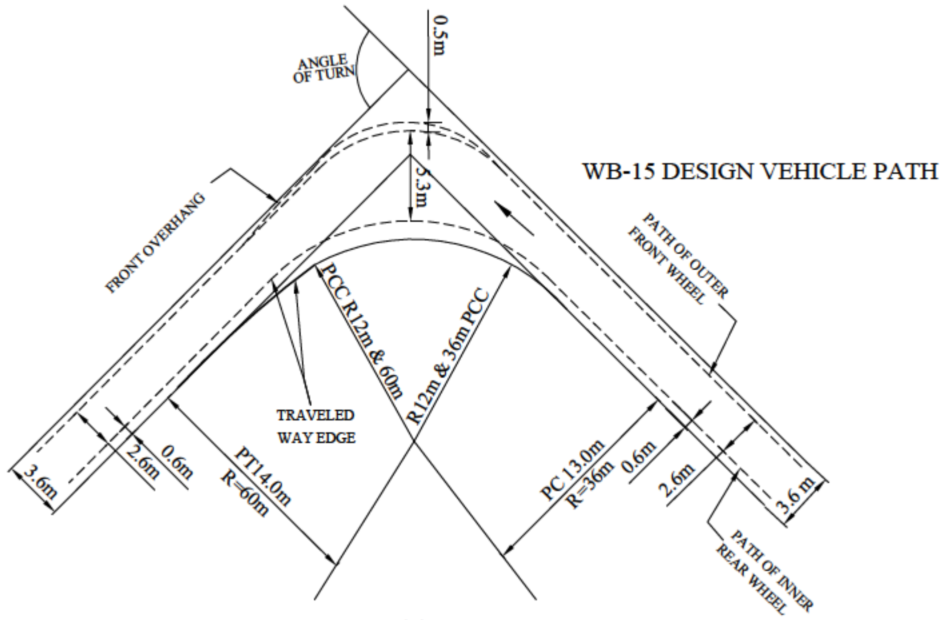
(c)
THREE-CENTERED COMPOUND CURVE
36m - 12m - 36m RADIUS, OFFSET 1m



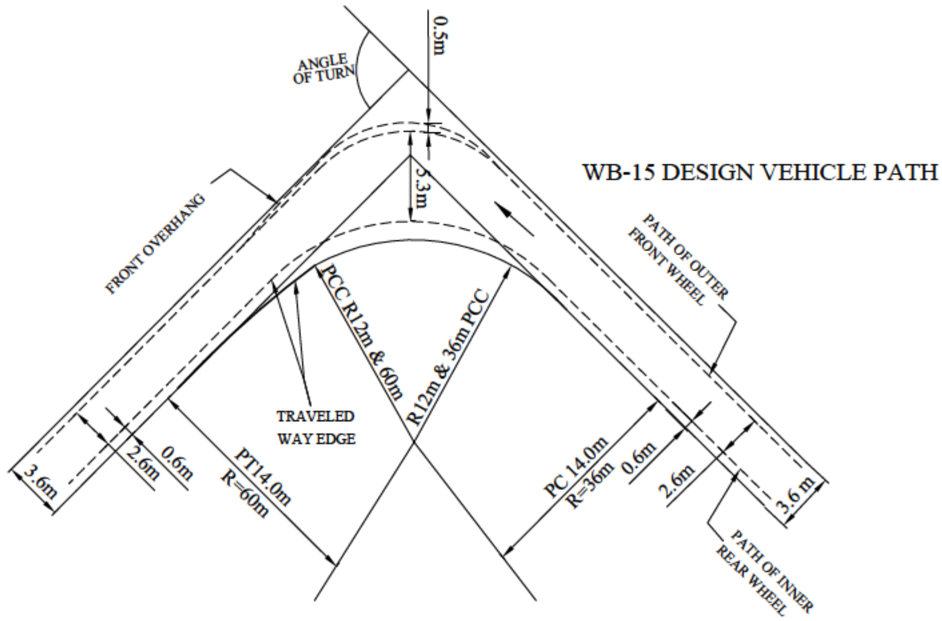
(d)
MINIMUM SIMPLE CURVE
15m OR 17m RADIUS

*APPENDIX C

Scale : Not to scale

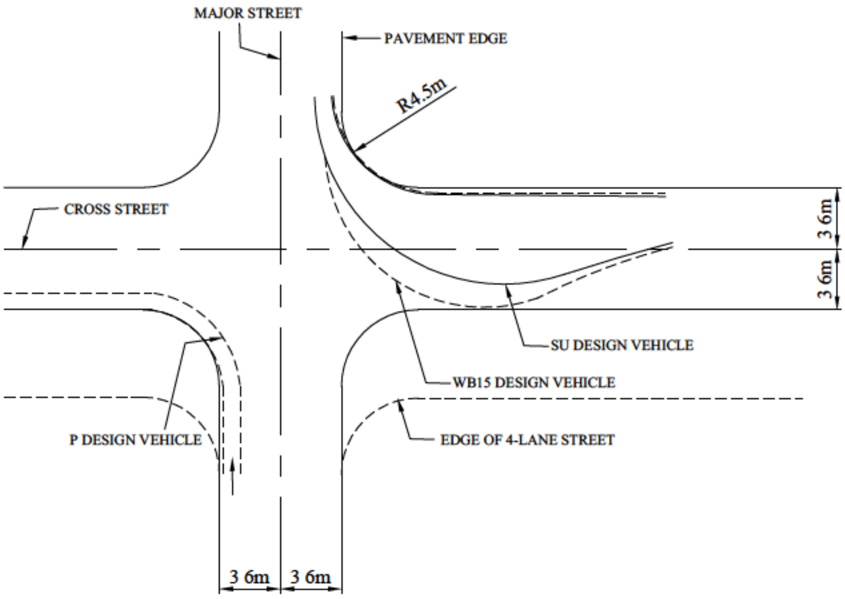


(e)
THREE-CENTERED COMPOUND CURVE
36m - 12m - 60m RADIUS, OFFSET 1m & 2m

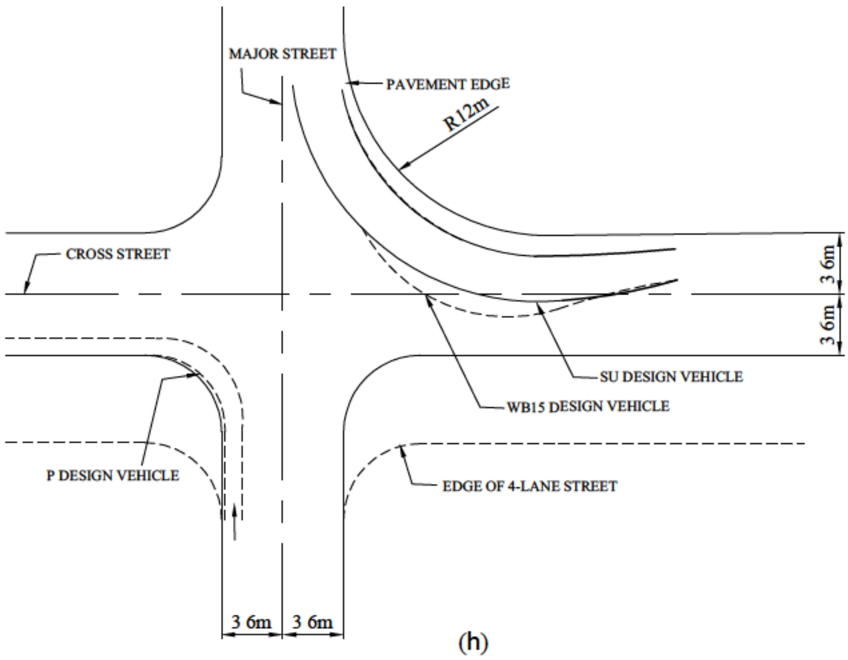


(f)
THREE-CENTERED COMPOUND CURVE
36m - 12m - 60m RADIUS, OFFSET 2m

JKR	*APPENDIX C	
	Scale : Not to scale	C3

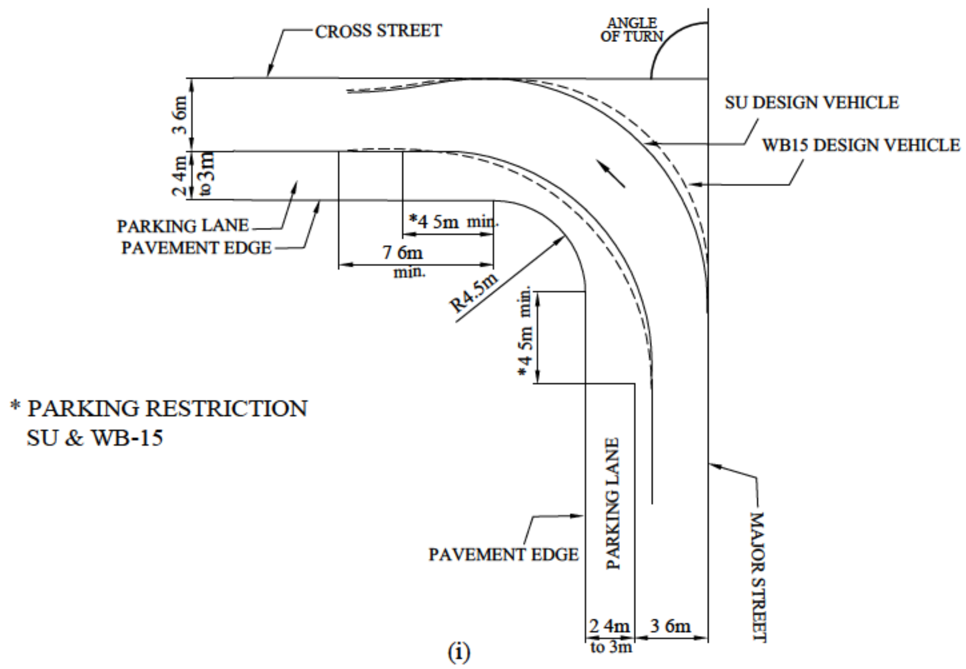


(g)
EFFECT OF CURB RADIUS ON TURNING PATH
OF VARIOUS DESIGN VEHICLE

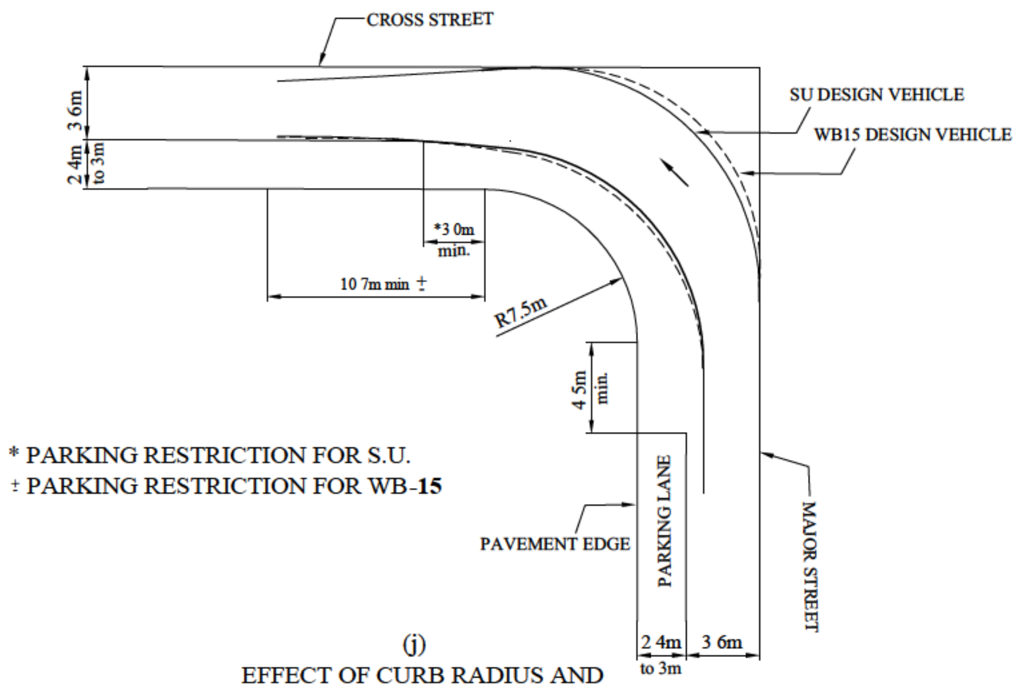


(h)
EFFECT OF CURB RADIUS ON TURNING PATH
OF VARIOUS DESIGN VEHICLE

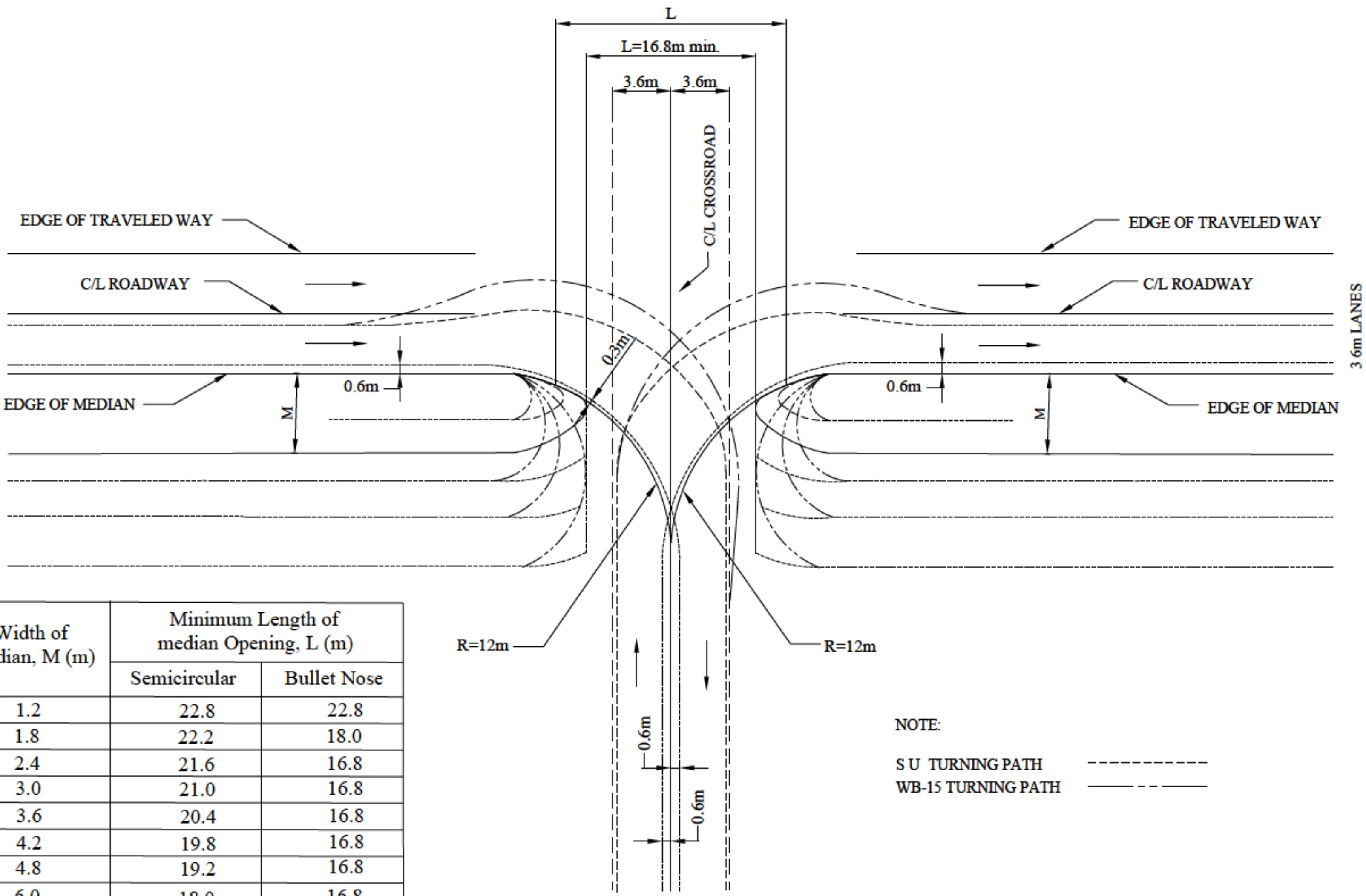
JKR	*APPENDIX C
Scale : Not to scale	C4



(i)
EFFECT OF CURB RADIUS AND
PARKING ON TURNING PATH

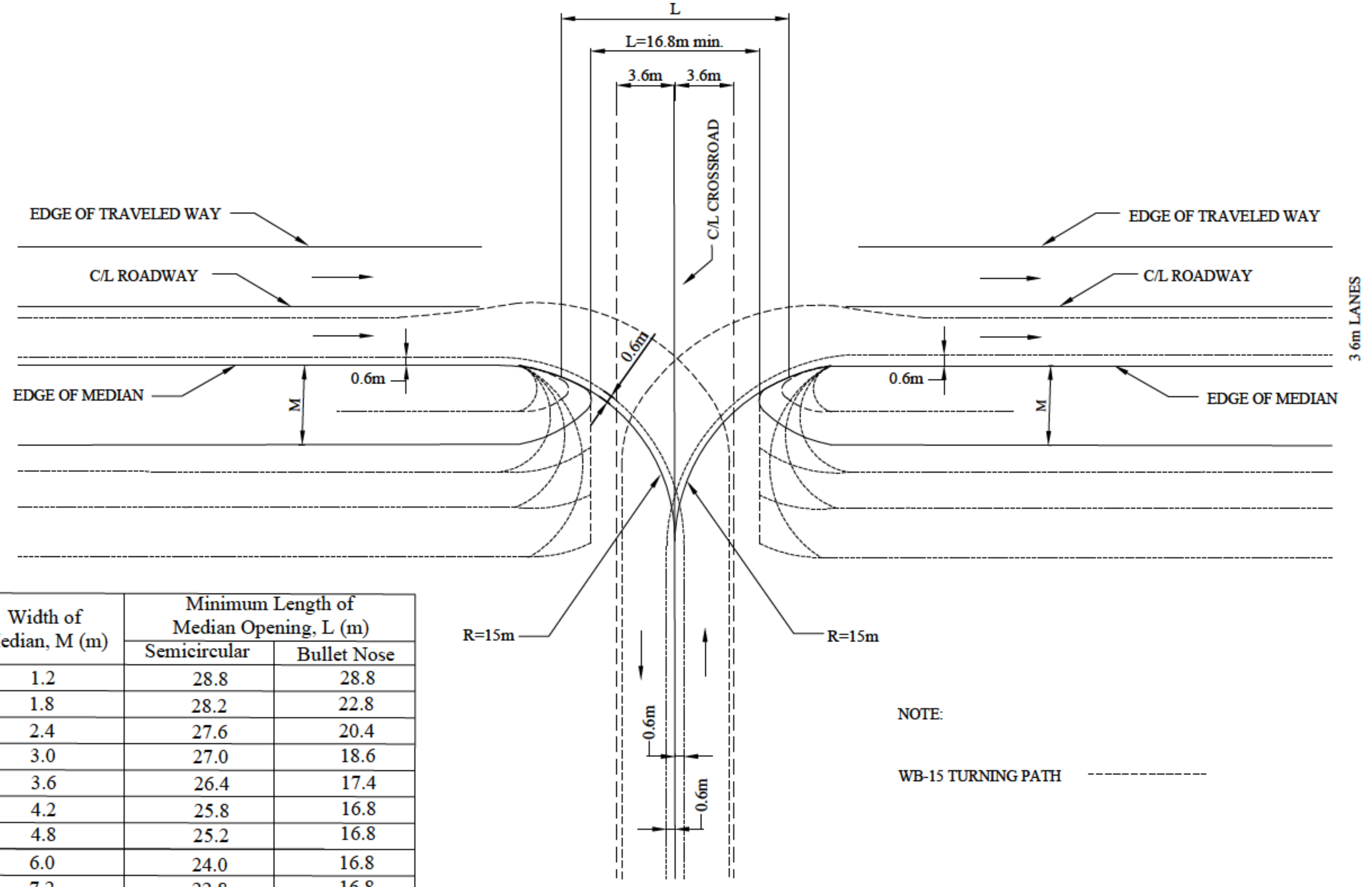


(j)
EFFECT OF CURB RADIUS AND
PARKING ON TURNING PATH



Width of Median, M (m)	Minimum Length of median Opening, L (m)	
	Semicircular	Bullet Nose
1.2	22.8	22.8
1.8	22.2	18.0
2.4	21.6	16.8
3.0	21.0	16.8
3.6	20.4	16.8
4.2	19.8	16.8
4.8	19.2	16.8
6.0	18.0	16.8
7.2	16.8	16.8

(k)
MINIMUM DESIGN OF MEDIAN OPENINGS
P DESIGN VEHICLE: CONTROL RADIUS OF 12 METRES



Width of Median, M (m)	Minimum Length of Median Opening, L (m)	
	Semicircular	Bullet Nose
1.2	28.8	28.8
1.8	28.2	22.8
2.4	27.6	20.4
3.0	27.0	18.6
3.6	26.4	17.4
4.2	25.8	16.8
4.8	25.2	16.8
6.0	24.0	16.8
7.2	22.8	16.8
8.4	21.6	16.8
9.6	20.4	16.8
10.8	19.2	16.8
12.0	18.0	16.8

(I)

MINIMUM DESIGN OF MEDIAN OPENINGS

P DESIGN VEHICLE: CONTROL RADIUS OF 15 METRES

EXAMPLE CALCULATIONS FOR CAPACITY OF UNSIGNALISED INTERSECTIONS
EXAMPLE 1: T INTERSECTION

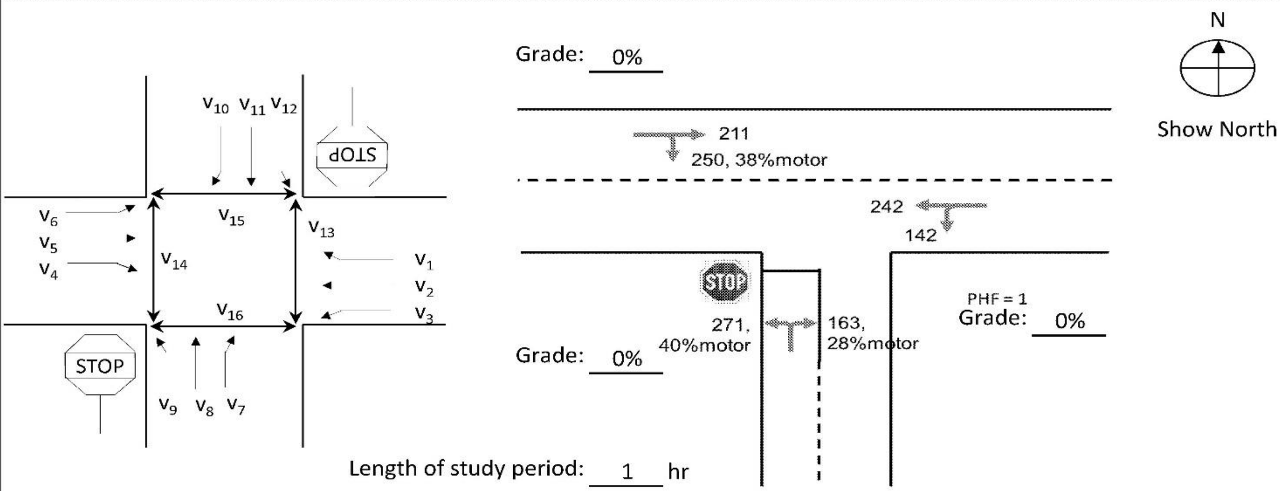
4.4 WORKSHEET

UNSIGNALISED INTERSECTIONS WORKSHEET

Worksheet 1

General Information		Site Information	
Analyst		Intersection	
Agency or Company		Jurisdiction	
Date Performed		Analysis Year	
Analysis Time Period			

Geometry and Movements



Worksheet 2

Vehicle Volumes and Adjustments

	Vehicle Volumes and Adjustments											
Movement	1	2	3	4	5	6	7	8	9	10	11	12
Volume (veh/h)		242	142	250	211		163		271			
Peak-hour factor, PHF		1.0	1.0	1.0	1.0		1.0		1.0			
Hourly flow rate (veh/hr)		242	142	250	211		163		271			
Proportion of motorcycles, P_M				0.38			0.28		0.40			

Pedestrian Volumes and Adjustments

Movement	13	14	15	16
Flow, V_x (ped/hr)	0	0	0	0
Lane width, w (m)				
Walking speed, [1] S_p (m/s)				
Percent blockage, fp				

1. Default walking speed = 1.2m/s

Worksheet 3

General Information

Project Description: ______________________________

Lane Designation

Movements	Lane 1	Lane 2	Lane 3	Grade, G	Left Turn Channelized?
1, 2, 3	2,3			0	N
4, 5, 6	4,5			0	N
7, 8, 9	7,9			0	N
10, 11, 12					

Flared Minor-Street Approach

Movement 9 ☐ Yes ☐ No Storage space, n ________ (number of vehicles)

Movement 12 ☐ Yes ☐ No Storage space, n ________ (number of vehicles)

Median Storage*

* Includes raised or striped median (RM), or two-way left-turn lane (TWLTL)

Movement 9 ☐ Yes Type ______ ☒ No Storage space, n ________ (number of vehicles)

Movement 12 ☐ Yes Type ______ ☐ No Storage space, n (number of vehicles)

Upstream Signals

	Movements	Distance to Signal, D (m)	Prog Speed S_{prog} (km/h)	Cycle Length, C (s)	Green Time, g_{eff} (s)	Arrival Type	Saturation Flow Rate, s (veh/h)	Progressed Flow, V_{prog} (veh/h)
S_2	protected LT					3		
	TH							
S_5	protected LT					3		
	TH							

Computing Delay to Major-Street Vehicles

Data for Computing Effect of Delay to Major-Street Vehicles	S_2 Approach	S_5 Approach
Shared-lane volume, major-street through vehicles, v_{i1}, blocked by RT		
Shared-lane volume, major-street right-turn vehicles, v_{i2}, blocked by RT		
Saturation flow rate, major-street through vehicles, s_{i1}		
Saturation flow rate, major-street right-turn vehicles, s_{i2}		
Number of major-street through lanes		
Length of study period, T (h)		

TWSC - UNSIGNALISED INTERSECTIONS WORKSHEET

Worksheet 4

General Information

Project Description ____

Critical gap and Follow-Up Time

$$t_{c,x} = t_{c,base} - t_{c,M}P_{M,x}$$

	Major RT		Minor LT		Minor TH		Minor RT	
Movement	1	4	9	12	8	11	7	10
$t_{c,base}$ (Table 4.2)		3.5	3.2				4.0	
$t_{c,m}$ (Table 4.3)		0.424	0.424				0.424	
P_M (from worksheet 2)		0.38	0.40				0.28	
t_c		3.34	3.03				3.88	

$$t_{f,x} = t_{f,base} - t_{f,M}P_M$$

	Major RT		Minor LT		Minor TH		Minor RT	
Movement	1	4	9	12	8	11	7	10
$t_{f,base}$ (Table 4.2)		2.0	1.9				2.2	
$t_{f,m}$ (Table 4.3)		0.738	0.738				0.738	
P_M (from worksheet 2)		0.38	0.40				0.28	
t_f		1.72	1.60				1.99	

TWSC - UNSIGNALISED INTERSECTIONS WORKSHEET

Worksheet 6

General Information

Project Description ____________________

Impedence and Capacity Calculation

Step 1 : LT from Minor Street	v_9		v_{12}
Conflicting flows (Figure 4.3)	$v_{c,9}$ =	313	$v_{c,12}$ =
Adjustment factor for capacity (Table 4.4)	A_9 =	0.4846	A_{12} =
Potential capacity (Equation 4.3)	$c_{mp,9}$ =	895	$c_{mp,12}$ =
Movement capacity	$c_{mm,9} = c_{mp,9}$ =	895	$c_{mm,12} = c_{mp,12}$ =
Prob of queue-free state (Equation 4.5)	$P_{0,9}$ =	0.697	$P_{0,12}$ =
Step 2 : RT from Major Street	v_4		v_1
Conflicting flows (Figure 4.3)	$v_{c,4}$ =	384	$v_{c,1}$ =
Adjustment factor for capacity (Table 4.4)	A_4 =	1.000	A_1 =
Potential capacity (Equation 4.3)	$c_{mp,4}$ =	1605	$c_{mp,1}$ =
Movement capacity	$c_{mm,4} = c_{mp,4}$ =	1605	$c_{m,1} = c_{p1}\ P_{p,1}$ =
Prob of queue-free state (Equation 4.5)	$P_{0,4}$ =	0.844	$P_{0,1}$ =
Step 3 : TH from Minor Street (4-leg intersections only)	v_8		v_{11}
Conflicting flows	$v_{c,8}$		$v_{c,11}$
Potential capacity	$c_{mp,8}$ =		$c_{mp,11}$ =
Ped impedence factor	$p_{p,8}$ =		$p_{p,11}$ =
Capacity adjustment factor due to impeding movement	$f_8 = P_{0,4}\ P_{0,1}\ P_{p,8}$=		$f_{11} = P_{0,11}\ P_{0,1}\ P_{p,11}$=
Movement capacity	$c_{mm,8} = c_{mp,8}$ =		$c_{mm,11} = c_{mp,11}$ =
Prob of queue-free state	$P_{0,8}$ =		$P_{0,11}$ =
Step 4 : RT from Minor Street (4-leg intersections only)	v_7		v_{10}
Conflicting flows	$v_{c,7}$		$v_{c,10}$
Potential capacity	$c_{mp,7}$ =		$c_{mp,10}$ =
Ped impedence factor	$P_{p,7}$ =		$P_{p,10}$ =
Major right, minor through impedence factor	$P_7 = P_{0,11}\ f_{11}$ =		$P_{10} = P_{0,8}\ f_8$ =
Major right, minor through adjusted impedence factor	P_7=		P_{10}=
Capacity adjustment factor due to impeding movements	$f_7 = P_7\ P_{0,12}\ P_{p,7}$=		$f_{10} = P_{10}\ P_{0,9}\ P_{p,10}$=
Movement capacity	$c_{m,7} = f_7\ c_{p,7}$=		$c_{m,10} = f_{10}\ c_{p,10}$=
Step 5 : RT from Minor Street (T- intersections only)	v_7		v_{10}
Conflicting flows (Figure 4.3)	$v_{c,7}$	1024	$v_{c,10}$
Adjustment factor for capacity (Table 4.4)	A_7 =	0.4375	A_{10} =
Potential capacity (Equation 4.3)	$c_{mp,7}$ =	343	$c_{mp,10}$ =
Capacity adjustment factor due to impeding movement	$f_7 = P_{0,4}\ P_{0,1}$ =	0.844	$f_{10} = P_{0,4}\ P_{0,1}$ =
Movement capacity (Equation 4.6)	$c_{mm,7} = c_{mp,7}\ f_7$=	290	$c_{mm,10} = c_{mp,10}\ f_{10}$=

Notes

1. For 4-leg intersections use Steps 1,2,3, and 4.
2. For T-intersections use Steps 1,2, and 5.

Worksheet 8

General Information

Project Description ______________________________

Shared-Lane Capacity

$$c_{SH} = \frac{\sum_y v_y}{\sum_y \left(\frac{v_y}{c_{m,y}}\right)}$$ (Equation 4.7)

Movement	v (veh/h)	c_m (veh/h)	c_{SH} (veh/h)
7	163	290	502
8			
9	271	895	
10			
11			
12			

Worksheet 10

Control Delay, Queue Length, Level of Service

Lane	v(veh/h)	c_m(veh/h)	v/c	Queue Length (Equation 4.9)	Control Delay (Equation 4.10)	LOS (Table 4.1)	Delay and LOS
1 ~~7~~ 8 ~~9~~	434	502	0.87	13.715	51.84	F	51.84 F
2 7 8 9							
3 7 8 9							
1 10 11 12							
2 10 11 12							
3 10 11 12							

Movement	v(veh/h)	c_m(veh/h)	v/c	Queue Length (Equation 4.9)	Control Delay (Equation 4.10)	LOS (Table 4.1)
1						
4	250	1605	0.16	0.553	7.66	A

Source: Adapted from MHCM 2006 and TRB HCM 2000, Unsignalised Intersection.

APPENDIX D

LIST OF REFERENCES

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