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GOVERNMENT OF KHYBER PAKHTUNKHWA

Consultancy Services for Preparation of Geometric Design Standards Manual, KP

Interim Report 3

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1 INTRODUCTION

The Government of Khyber Pakhtunkhwa (KP) through Urban Policy Unit (UPU) & Planning and Development Department (P&DD) has intended to develop a Geometric Design Manual for various classes of road. In this regard, UPU has awarded a contract to Halcrow Pakistan (HPK) to undertake the task of developing the Geometric Design Manual for KP Province. Following are the objectives of this exercise:

1. To provide room for indigenous engineering practices.
2. To address the core issues not addressed in available manuals and standards.
3. To utilize roads as the dominant transport mode in KP
4. Efficient road transport to ensure:
   i. Commercial and industrial development
   ii. High quality social services delivery
   iii. Security and law enforcement
5. Good roads to facilitate low-cost road transport with lower transit times and greater safety, which:
   i. Reduce imports of vehicles, fuel, tires and spares
   ii. Improve marketability of goods produced in NWFP
   iii. Reduce incidence and severity of accidents
6. To ensure sustainable road maintenance to achieve the above objectives.

1.1 OVERVIEW OF THIS REPORT

Chapter 2 of this report addresses typical road cross-sectional elements that include cross-slopes, lane widths, shoulder widths and cross slopes, barriers, medians, side slopes, clear zones, lateral offsets and utilities. Similarly, geometric design elements in Chapter 3 discusses in detail sight distances namely (passing, stopping, intersection and decision), horizontal and vertical alignments, combination of horizontal and vertical alignments and the features affecting geometric design. Chapter 4 shows different types of at-grade intersections and roundabouts, their geometric design, priority intersections, signalized intersections and signalized roundabouts. Chapter 5 comprises Grade separations and interchanges in detail, which further elaborates types of grade separations like underpasses, flyovers, 3-legged, and 4-legged interchanges, ramps and ramp terminals. Roadside and highway facilities are encompassed in Chapter 6 with topics such as pedestrian facilities, kerbs and edging, fences, safety barriers, public transport facilities and bus turnouts, parking facilities, landscaping. Chapter 7 mainly narrates black spot mitigations and their methods of identification and improvement on roads, highways and in long tunnels. Road signage and markings has been covered in Chapter 8 describing various types of road signs (Regulatory, warning, directional and informatory signs) and types of road markings used as standards. Chapter 9 comprised mainly improvement of existing roads including their application of improvements, their geometric designs, roadway drainage, highway lightings and specific design drawing requirements.

Section-2 of this Report deals with task 7, section-3 deals with task 8, section-4 deals with task 9, section-5 deals with task 10, section-6 deals with task 11, section-7 deals with task 12, section-8 deals with task 13 and section-9 deals with task 14 respectively of the Terms of Reference (TOR).

2 CROSS SECTIONAL ELEMENTS

2.1 GENERAL

This chapter provides the principles to be followed while designing roads cross section for new roads or rehabilitation and improvement in an existing road. The cross section is made from a combination of distinct components that depends on the type of road and the facilities provided or required for various users.

To assure consistency in terminologies of this manual, the terms are based on "cross section," "roadway," and "carriageway" are defined as follows:

Cross section - A vertical section of the ground and roadway at right angles to the centreline of the roadway, including all elements of a highway or street from right-of-way line to right-of-way line.

Roadway - The portion of a highway, including shoulders, for vehicular use. A divided highway has two or more roadways.

Carriageway - The portion of the roadway for the movement of vehicles, exclusive of shoulders.

This standard defines and terms the components and provides guidance on details of their design. Cross section of a road includes certain or all of the following below elements:

- Carriageway and cross slopes
- Lane widths
- Shoulders and hard strips
- Medians
- Kerbs and traffic barriers
- Sidewalks
- Utility and landscape areas
- Drainage channels and side slopes
- Clear zones and lateral offset
- Frontage or service roads

Figure 2.1 and **Figure 2.2** illustrate the various components of the cross-section elements for a two-lane and dual carriageway road.

The above are different arrangements to be used depending on the functional classification of the road. The designer's need to adopt which of the components to include and the selection of the appropriate dimensions for the roadway cross section.

This standard provides details of the cross sections and horizontal clearance requirements to be used for all rural and urban roads. The information includes highways, motorways, expressways, arterial roads, collector roads, and local roads, including single road and dual carriageway, with related ramps and service roads. The width inside which these components exist is called the right of way or (ROW). The typical cross sections for different classification of roads are illustrated at the end of the chapter from **Figure 2.25** to **Figure 2.42**.

Wherever the standard roadway cross section needs modification and requires existing or proposed land, approval of the Overseeing Organization or other concerned authority must be taken before commencement of design. In such cases, amendments to the utilities layout may be required to suit the specific road cross section proposed. If required proposed revisions to the standardized utility locations must need the approval of the utility authorities.

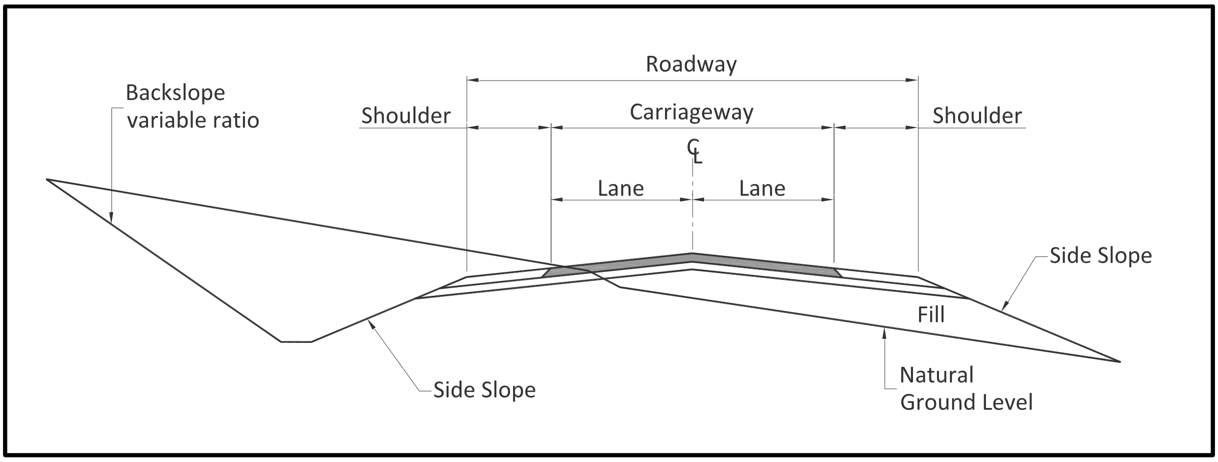


Figure 2.1: Cross Sectional Elements, Two Lane Road

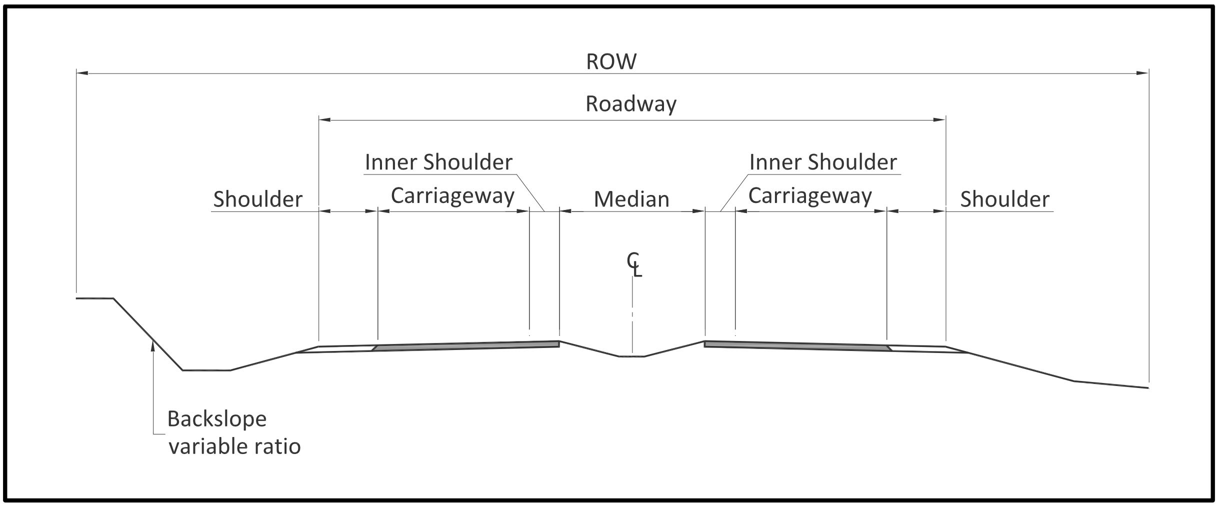


Figure 2.2: Cross Sectional Elements, Dual Carriageway

2.2 DESIGN REQUIREMENTS

2.2.1 Cross Slopes

Cross slope is defined as the transverse slope across the pavement from the centreline of an undivided roadway or the edge of the median of a divided roadway to the edge of the carriageway or the face of the kerb. Un-divided carriageways on tangents, or on flat curves, have a crown slope or high point in the middle and a cross slope downward toward both edges. Cross slopes on straight carriageways should be 2%, while unpaved carriageways shall have a cross slope of 4.0% to facilitate drainage.

2.2.2 Lane Width

Carriageway width is one of the most important safety factors in design. A wider lane provides higher capacity, higher driver comfort levels, consistent operation and lower accident rates. Lane width varies from 2.7 m to 3.65 m according to road classification. Details of lane width for different functional classification are provided in **Table 2.1** and **Table 2.2**.

The following parameters may restrict controls on the lane widths in urban areas, which include pedestrian crossings, right-of-way, or existing development; the use of 3.3 m lanes may be suitable. Similarly, for mountainous area, a lane width of 3.3 m may be used. Auxiliary lanes should be as wide as the through-traffic lanes but not less than 3.0 m.

Wherever possible, parking should be provided away, off-street from the roadway. In urban locations, parking may be provided adjoining the road in designated parking lanes. Parking lanes should be provided only on roads with posted speeds of 50 km/h or less. On-street parking is most appropriate on local roads and service roads.

2.2.3 Auxiliary Lanes

An auxiliary lane is an additional lane for used prior to the intersection left and right turning movements. Auxiliary lane is also provide to enable speed change, merging, diverging, weaving and separation slower vehicle from high speed traffic on steep upgrades . Auxiliary lanes should have a width equal to that of the through lane or should be at least 3m.

2.2.4 Shoulder Width and Cross Slope

Shoulder is a contiguous part of roadway used in case of emergency and law enforcement, and is used to accommodate stopped vehicles. Shoulders provide structural support for the pavement edges and side clearance between moving vehicles and stationary objects. Shoulders of sufficient width provide additional space for access by emergency service vehicles. They can also be used as temporary lanes to facilitate traffic movement during road maintenance operations. For these reasons, shoulders should be constructed to the same structural strength as the adjacent roadway. However, this may not be possible in most of the cases due to economic constraints. Shoulders are not used on urban street instead, footpaths are provided for pedestrian movement. Details of shoulder widths for different classes of roads are provided in **Table 2.1** and **Table 2.2**. Unpaved shoulder should have a minimum cross slope of 4%. However, cross slope of paved shoulder can range between 2% to 4%.

Where roadside barriers, walls, guardrails or other vertical elements are to be provided, it should be at a minimum offset of 0.6 m. Shoulders on structures should normally have the same width as usable shoulders on the approach roadways.

Table 2.1: Typical Roadway Cross Sectional Details- Rural Roads

Roadway Type		Design Speed (km/h)	Lanes in Each Direction	Lane Width (m)	Median Width (m)	Paved Shoulder Width		Earthen Shoulder (m)
						Outside (m)	Inside (m)	
Rural	Expressways / Freeway (Motorways/National Highways)	100 - 130	2	3.65	10.00	3.00	1.00	0.5
		100 - 130	3 or more	3.65	10.00	3.00	1.00	0.5
	Major Arterials (Provincial Highways)	80 - 110	2	3.65	3.60 - 9.0	2.50	1.00	1.00
	Minor Arterials (District and Tehsil Roads)	60 - 100	1 or 2	3.30 - 3.65	0 - 6.00	2.50	1.00	1.00
	Collectors (Tehsil and Farm-to-Market Roads)	30 - 100	1	3.00 - 3.65	-	1.50	-	1.00
	Locals (Farm-to-Market and Local Access Roads)	30 - 60	1	3.00	-	-	-	1.50

Table 2.2: Typical Roadway Cross Sectional Details- Urban Roads

Roadway Type		Design Speed (km/h)	Lanes in Each Direction	Lane Width (m)	Median Width (m)	Parking Width (m)
Urban	Expressway	100 - 130	2	3.65	3.00 (with barrier) 10.00 (without barrier)	-
		100 - 130	3 or more	3.65	6.60 (with barrier) 12.00 (without barrier)	-
	Major Arterial	60 - 100	2 or more	3.65	1.00 - 3.00	-
	Minor Arterial	60 - 100	2 or more	3.00 - 3.65	1.00 - 3.00	2.50
	Collector	50 - 60	2 or more	3.00 - 3.65	1.00 - 3.00	2.50
	Local and Service Road	30 - 50	1 (one-way)	4.00	-	2.50
		30 - 50	1	3.00	-	2.50

2.2.5 Median

Roadway median is designated based on the highway separating traffic moving in opposite directions. Medians can be open, depressed, flush or raised with kerbs or barrier with the travelled way surface. The width of a median ranges between 1.2 to 2.4 m but in some cases, even wider medians can be used. Typical layouts of medians are illustrated in **Figure 2.3**.

A depressed median may be used for freeways and for placement of utilities such as street lighting, drainage, and landscaped area. The appropriate width of a median depends on the functions served and the available ROW (right-of-way). The minimum median widths for different classification of roads are provided in **Table 2.1** and **Table 2.2**.

Medians on high-speed rural highways should be of sufficient width to prevent run-off road, high-speed, and head-on collisions. Kerbs should not be used in rural medians because they contribute to loss of control when struck at high speeds. Median widths of 20 m are generally sufficient to minimize the risk of head-on collisions.

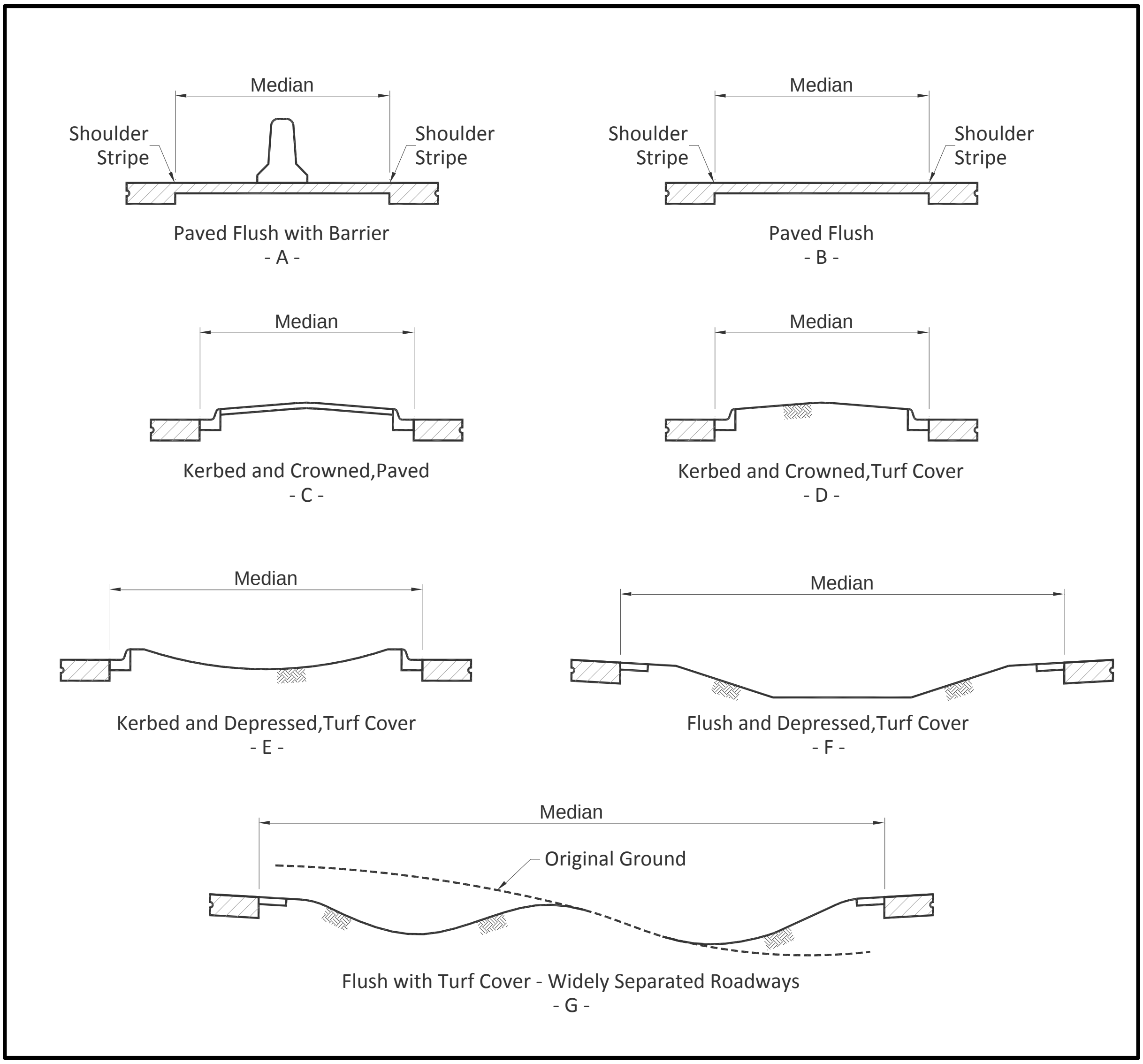


Figure 2.3: Typical Median Layouts

2.3 UTILITY

2.3.1 Introduction

2.3.1.1 General

Utility corridor means a space or provision of land pieces provided for running the utilities. Usually utility corridor is within the right of way but not all the time. Right of way is a typical land between building plot limit that is reserved as an infrastructure corridor for roads, public walkways, Railway, utility transmission, distribution etc.

In this section, basic requirements and philosophy shall be discussed to make a symmetry while planning the roadside utilities. For utility services, especially for urban environment, like telephone/IT lines, storm drains, sewers, gas lines, water lines, oil lines, power lines, road lighting, traffic signal electrical supply lines, irrigation lines and other services by any private or public utility company requires a respective service corridors.

2.3.1.2 Utilities Key Design Stages and Responsibilities

Usually the planning of roads and highway is the responsibility of municipality and planning division. Project starts with the data acquisition and its validation from statutory authorities. Following shall be the key stages of project.

Project Initiation

This is first stage, which includes collection of data from relevant authorities and its validation. Service authorities maintain records of their assets, and the designer may request to obtain these data through the RFI during Stage 1. At this stage, no approval is required from any authority.

Concept Design

Concept design options are to be prepared with the consultation of stakeholders. Generally, municipality and planning division are consulted. Written approval is required from all statutory authorities.

Preliminary Design

Development of recommended concept design with the consultation of stakeholders this stage does not require any approval from statutory authorities only liaison is enough.

Detail Design

Preparation of detail design and tender document is the last stage of design. At this stage, NOC is required in written form from statutory authorities prior to tender. Review and consultation with stakeholders to be done.

Construction

Before commencement of execution plan, shop drawings should be prepared by contractor, which has to be reviewed and approved by concern authorities. All key stakeholders to be on board prior to construction.

Consultation and NOC

The NOC is mandatory, which must be obtained from the relevant service provider or service authority. Throughout the design life of a project, it is necessary to coordinate with the relevant statutory authorities through the NOC process. In addition, an NOC allows follow-on design or construction work to be carried out. Consultants and contractors need to follow the guidelines and checklists provided in **Table 2.3**.

Table 2.3: Utility Impact Assessment Checklist for Designer

Item	Description		Yes	No	Comments
1	**Data Collection**				
1.1	Data collection of all the existing and proposed utilities from utility agencies. Information must be current (not older than 6 months.) The agencies are:				
	a. Urban Policy and Planning Unit (UPU), KP				
	b. Communication and Works Department (C & W)				
	c. Local Government / Peshawar Development Authority (PDA)				
	d. Pakhtunkhwa Highways Authority (PKHA)				
	e. National Highway Authority (NHA)				
	f. Transport and Mass Transit Department (TMTD)				
	g. Forest and Irrigation Department				
	h. Provincial Housing Authority (PHA)				
	i. Pakistan Railways				
	j. Others relevant agencies				
	Attributes of utilities should also be requested i.e. size, type, age, material, class and any other relevant information such as voltage, operating pressure etc.				
1.2	Coordination with adjacent as well as large projects, such as:				
	a. Mega Reservoirs				
	b. Infrastructure Development				
	c. Metro				
	d. Local Road & Drainage Program				
	e. Others				
1.3	Coordination with utility agencies regarding existing and future abandonments as well as possible upgrades within the vicinity of the project corridor				
1.4	Conduct topographic survey and geophysics survey (GPR, Radio detection & Trial Pits for better understanding of site.				
1.5	Assess all the discrepancies between the survey and the 'As Built' and forwarded to relevant agencies for confirmation.				
2	**Overview of the Roads and Surrounding Area**				
2.1	Identify the location of the project, such as:				
	a. Rural				
	b. Urban				
	c. Inner Urban				
2.2	Type of Roads:				
	a. Expressway				
	b. Arterial				
	c. Local Urban Collector				
	d. Local Urban Access				

Item	Description		Yes	No	Comments
	e. Others (please specify)				
2.3	Study the Land Use Plan, future adjacent developments (shopping malls, industries, town planning, hospitals, institutions etc…), and major utilities along the corridor e.g. substation, pump stations, water reservoirs/storage etc.				
3	**Utility Design and Layout**				
3.1	Complete the utilities layout design including all the wet and dry utilities (power, water, sewerage , storm drainage, Gas, telecom, etc.				
3.2	Propose Land acquisition required for utility design.				
4	**Analysis of each existing utility impact**				
4.1	Studied the proposed road configuration including a comparison of the existing profile vs. proposed profile				
4.2	Identify possible measures/changes that can be adopted to minimize the impact on existing utilities:				
	a. Geometric (profile and alignment)				
	b. Road Drainage & Kerb				
	c. Slope/Retaining Walls/Barriers				
	d. Structure/Bridges/Footing				
	e. Other design departures for maintaining the existing utilities				
4.3	Arrive at a decision hierarchy for utility adjustment with a most preferred option.				
	a. Keep As Is				
	b. Protect (including under the service and carriageway)				
	c. Relocate				
	d. Reroute				
4.4	Consider the following factors when adjusting the impacted existing utilities.				
	a. The size of the utility (distribution or transmission)				
	b. The age of the utility and assessment for potential upgrade				
	c. The size and frequencies of the chambers				
	d. The frequency of maintenance				
	e. The proposed location of the utility within the carriageway and the impact on traffic i.e. Lane Closures during maintenance or breakage.				
	f. The hierarchy cost of relocating utilities, i.e. cost of relocating a smaller cable/pipe vs. bigger cable/pipe.				
	g. Identification of Primary Substations, Pump Stations etc.				
	h. Potential risks in case of breakage				
5	**Conflict Analysis**				
5.1	Consider utility conflicts arising from;				
	a. Road configuration				
	b. Between different utilities				
	c. Major projects e.g. Metro				
	d. Adjacent projects				

Item	Description		Yes	No	Comments
6.1	The following documents should be provided for the full length of the Road project in readable scale (not less than 1:500 @ A1 size):				
	a. Presentation of change in the road levels i.e. existing vs. proposed in profile and cross-section.				
	b. Presentation of the road layout including other projects within the corridor e.g. Metro etc.				
	c. Presentation of existing and proposed utilities, where the utilities shown to be "protected", "abandoned", "removed", "kept as is", "relocated" and proposed new utilities.				
	d. Presentation of Utility Corridor in plan and cross-section				
	These corridors should be based on:				
	a. KP GDM typical cross-section				
	b. Existing utility to be maintained at their current location (i.e. "Keep As Is" or "Protect")				
	c. Planned future utilities (provided by utility provider)				
	d. Estimation of future utility needs considering the sensitivity of corridor.				
	e. Presentation of cross-sections showing KP-GDM allocated utility corridors, proposed utility corridors as well as the existing and proposed utilities.				
	f. Presentation of layout plan indicating the land to be acquired for utility purposes.				
6.2	It is encouraged to separate the "dry" utilities from "wet" utilities in the layout plans for ease of documentation.				

The designer is required to consult with, liaise, and get approval from a large number of service providers and government authorities. **Table 2.4** lists the authorities with whom the designer must consult regarding the highway project.

Table 2.4: List of Authorities Involved In Highways & Roads of KP Province

S.No	Name of Authority	Role & Responsibilities	Jurisdiction
1	Urban Policy and Planning Unit (UPU), KP	UPU is basically responsible for planning of roads and other infrastructure elements.	Entire KP province
2	Communication and Works Department (C&W)	1. Construction, maintenance and repairs of roads, bridges, ferries, tunnels, causeways 2. Roads Funds 3. Tolls 4. Laying standards and specifications for various types of roads and bridges for the province.	25 districts of KP namely Peshawar, Kohat, Hangu, Tank, Dera Ismail Khan, Lakki Marwat, Bannu, Karak, Mardan, Charsadda, Nowshera, Malakand, Swabi, Buner, Shangla, Swat, Lower Dir, Upper Dir, Chitral, Battagram, Mansehra, Abbottabad, Haripur, Kohistan and Torghar.
3	Local Government / Peshawar Development Authority (PDA)	1. Construction and rehabilitation of roads, drainage, bridges and parks. 2. Execution of services such as street lighting and fire brigade.	Majority areas of Peshawar

S.No	Name of Authority	Role & Responsibilities	Jurisdiction
4	Pakhtunkhwa Highways Authority (PKHA)	1. Prepare a master plan for the development, construction, operation and maintenance of provincial highways and roads in KP. 2. Procure plant, machinery, instruments and materials required for its use. 3. Levy, collect or cause to be collected tolls on provincial highways and other roads. 4. Conduct studies, surveys, experiments and technical researches. 5. Pre-Qualification of Consultants for planning, designing and supervision of projects.	Highways located in the boundaries of KP province
5	National Highway Authority (NHA)	Plan, promote, organize and implement programs for construction, development, operation, repairs and maintenance of National Highways/ Motorways and Strategic Roads and undertake work/incur expenditures on it.	Country wide
6	Transport and Mass Transit Department (TMTD)/Transport Planning Engineering Unit (TPU)	TMTD/TPU works together with other stakeholders in planning of roads.	Entire KP province
7	Forest and Irrigation Department	Forest and irrigation department works together with other stakeholders in planning of roads and infrastructure.	Entire KP province
8	Provincial Housing Authority (PHA)	PHA department works together with other stakeholders in planning of infrastructure.	Entire KP province

2.3.1.3 Environmental, Health and Safety Aspect

The environment, health, and safety (EHS) are important considerations to comply with the aims of this design manual. It is expected that these aspects will be considered at each stage of the design works.

All design stages are to comply with the EHS aspects of the Pakistan safety standards and international codes. The following are general considerations regarding utilities:

- To avoid potential damage to utilities, loss of supply to users, risks of effluent spills and water impacts on the environment, and risk to contractors' staff if high voltage power cables are damaged, all utilities need to be designed and installed in accordance with the recommendations and guidelines of the statutory authorities and manufacturers. This especially refers to depths and protection measures for those services.

- Only in exceptional circumstances, utilities may be designed or installed different from recommended guidelines. In these circumstances, special protection and warnings must be put in place to protect the public, future works and the environment.

2.3.1.4 Utilities Improvements

There are three types of Utility improvement projects:

1. New Utilities
2. Relocation of Utilities
3. Improvements/protection of existing utilities

New Utilities

While planning a highway/road within a new development, utilities should be planned for next 20 years & with approximation of future expansions. All procedures and sequence to be adapted for the design of utilities as mentioned in this report.

Relocation of Utilities

For improvement works like widening of road/highway imposes a challenge due to restricted right-of-way & placement of different utilities in close proximity. Similarly, enhancing the existing utility capacity shall also poses the same challenge. This type of work should be requested by the concerned utility & approved with the relevant authorities prior to commencement of design work.

Protection of Existing Utilities

Where the existing utilities are retained in any new or enhancement work of road/highway and in case of non-availability of as-built drawings of existing utilities, test holes shall be made at appropriate place and intervals during design development stage of enhancement work.

2.3.2 Provision and Planning of Utilities Corridors

2.3.2.1 General Principles

A system of utility corridors should be provided that allows service providers to locate their services in highway ROWs and designated utility corridors. The designated utility corridors help to facilitate the location of services for present and future needs.

For new roads and upgrades of existing roads, the designer has flexibility in providing the utility corridors required for each service provider. However, the designer needs to adhere to the service provider's requirements in the design. Where an existing corridor needs to be altered, the service provider and the concern authority need to agree to the change. The designer will need an NOC from the both the service provider and the statutory authority for the final utility corridor provision. The designer should, in his considerations minimize the impact on existing utilities.

2.3.2.2 Development of Preliminary Utility Corridors

Utility corridors are separate land areas, usually within the ROW of a roadway. The development of a utility corridor cross section depends on the following considerations:

- Functionality of the road and the road hierarchy
- Types of utilities and demand
- Geometric design standards for road elements, such as furniture, medians, and pavements

Routing of utilities must include the following considerations:

- Network optimization so that the shortest and most appropriate route is selected to serve existing and future developments
- Identification of major utilities, including access shaft locations and potential conflicts
- Analysis of the feasibility and constructability of the route in relation to current and future roadway and infrastructure projects
- Selection of a route that least impacts traffic, including necessary traffic diversions
- A sensitivity study, so that schools, hospitals, government buildings, and other sensitive sites are avoided where possible
- An overall master planning impact study of all future projects, including the Metro, local roads, drainage, expressway program, and agency proposals for future developments along the route to reduce the possibility of conflict
- Environmental aspects of utility routing and a high-level risk assessment for options considered
- Land acquisition; this is a last resort and is to be done only in special circumstances

2.3.2.3 Utility Requirement Planning for Present and Future Scenarios

A fundamental responsibility of the utility designer is to establish the present and future requirements of the individual service providers. The designer needs to establish a database of service authority utility data and request their individual requirements for the proposed roadway project. Information regarding future abandonments and possible upgrades near the project corridor is also required. Land use considerations are required so that greater use can be made of the utility corridor. The designer will need to arrange meetings and liaise with the service providers to formalize their requirements.

2.3.2.4 Utility Corridor Design Principles

The designer must follow the design road cross sections as guidance for the placement of utilities. The cross sections are not a prescriptive right of access for the service provider. In situations where the corridor space is severely restricted, service providers must share the available space. The object is to facilitate provision of designated utility corridors for all utilities for all roads types. The corridors must delineate the locations of existing and planned utilities, and if there is any spare capacity.

2.3.2.5 General Rule of Utility Corridor Considerations

The identification of final locations of utility corridors is considered to be an iterative process. For any road type, the first pass must consider the standard corridor locations, as defined in the typical cross sections outlined in the below sections i.e. from sections **Figure 2.6** to **Figure 2.18** of this report (N.B. these are typical & indicative arrangements only which shall be adjusted as per actual road width and type and as per requirement of utilities for the particular area land use). The main product in considering width requirements for utility corridors is the combined services layout drawings, also known as rainbow layout drawings.

These drawings indicate the space in the corridor for individual services. The drawings are prepared as a first draft in confirming whether there is sufficient space in any particular highway for the services. Drawings must be color-coded in accordance with **Figure 2.4. Figure 2.5** is an example of a color-coded combined services drawing for a straight section of road.

These standard sections have been developed to minimize the overall land take and indicate that, wherever possible, the corridors should be located outside of the main roadway. If it is necessary to locate corridors within the roadway, priority should be given to locate transmission corridors inside and distribution corridors closer to the plot boundaries.

Once the first pass has been generated, the next stage will be to compare the locations of existing utilities and how these positions comply or conflict with the standard locations.

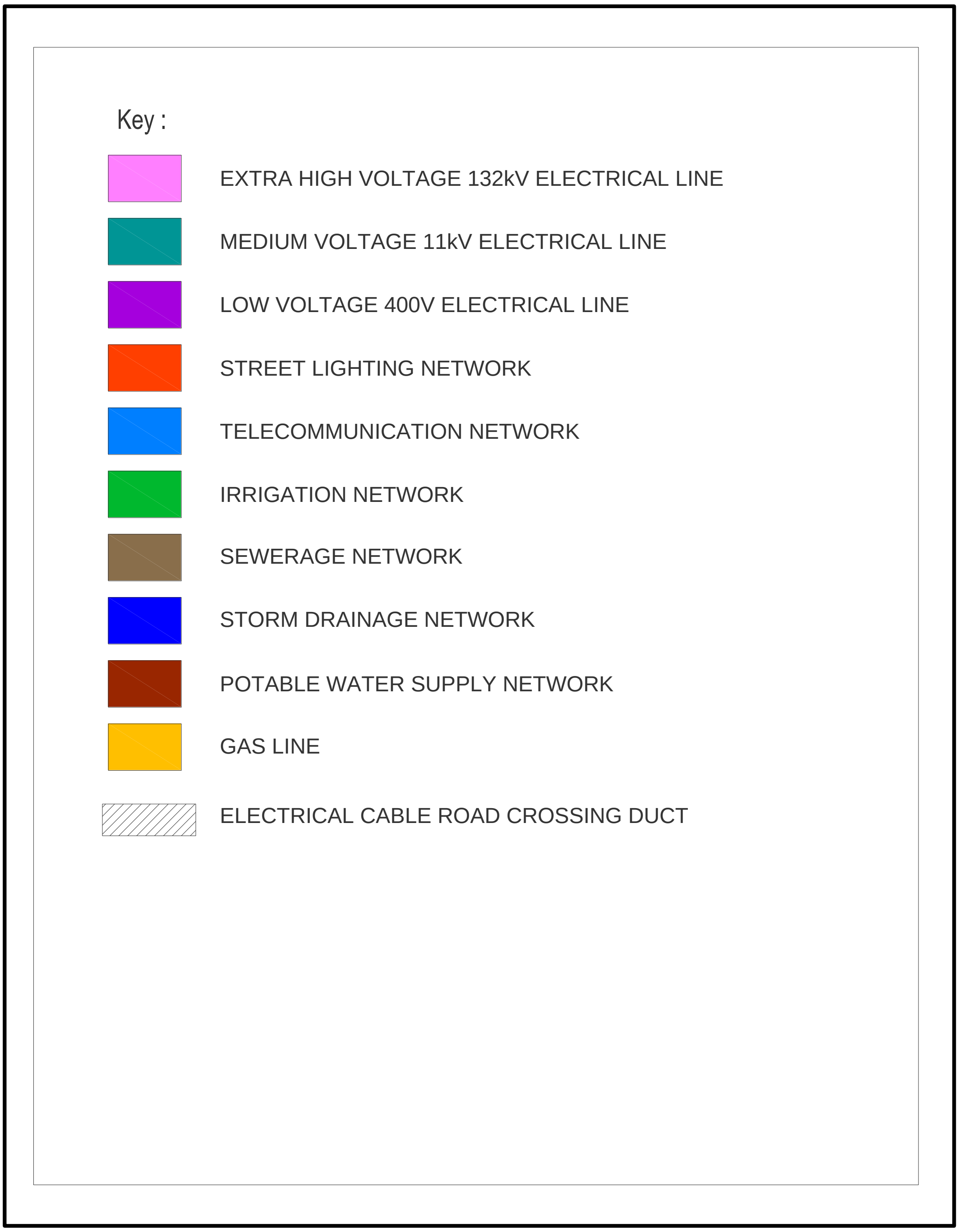


Figure 2.4: Color Coded Legends for Combined Service Drawings

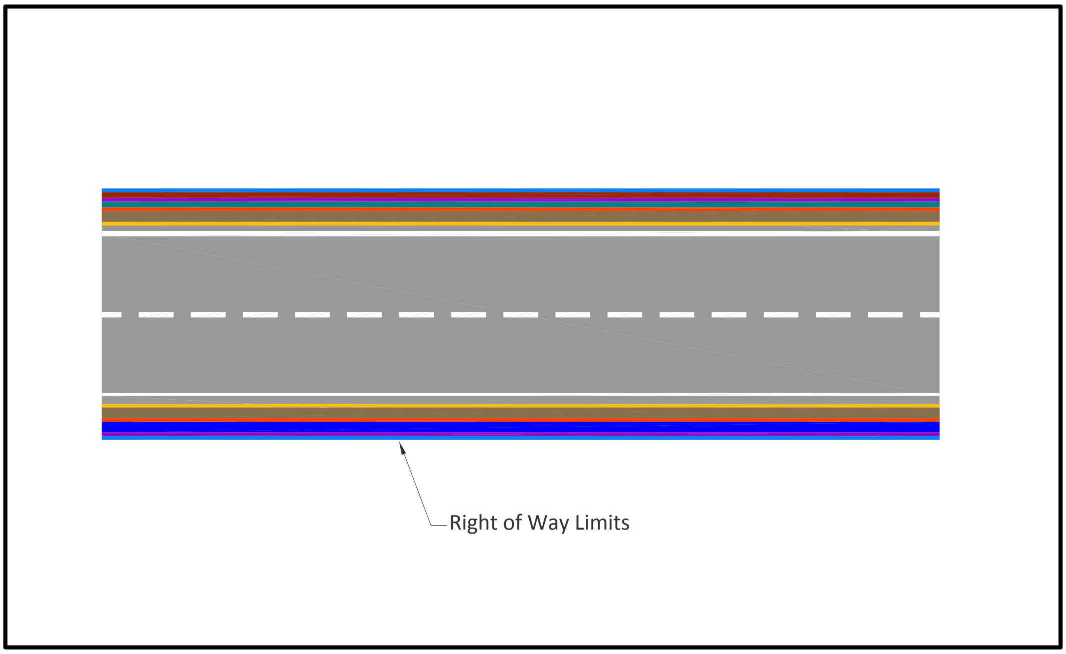


Figure 2.5: Color Coded Combined Services for a Straight Section of Collector Road

By defining the basic road functions, utility corridors can be determined by considering the following road elements:

- Median width: Lighting poles and pipes for treated sewage effluent (TSE) used for irrigation are normally placed in the median.
- Roadway width: Determined by the number of vehicle lanes that is required.
- Utility corridor width: Depends on the types of utilities and the depth and size of utilities to be placed. Each stakeholder utility provider has its own requirements.
- Service road width
- Parking and pavement area widths
- Rail corridor width

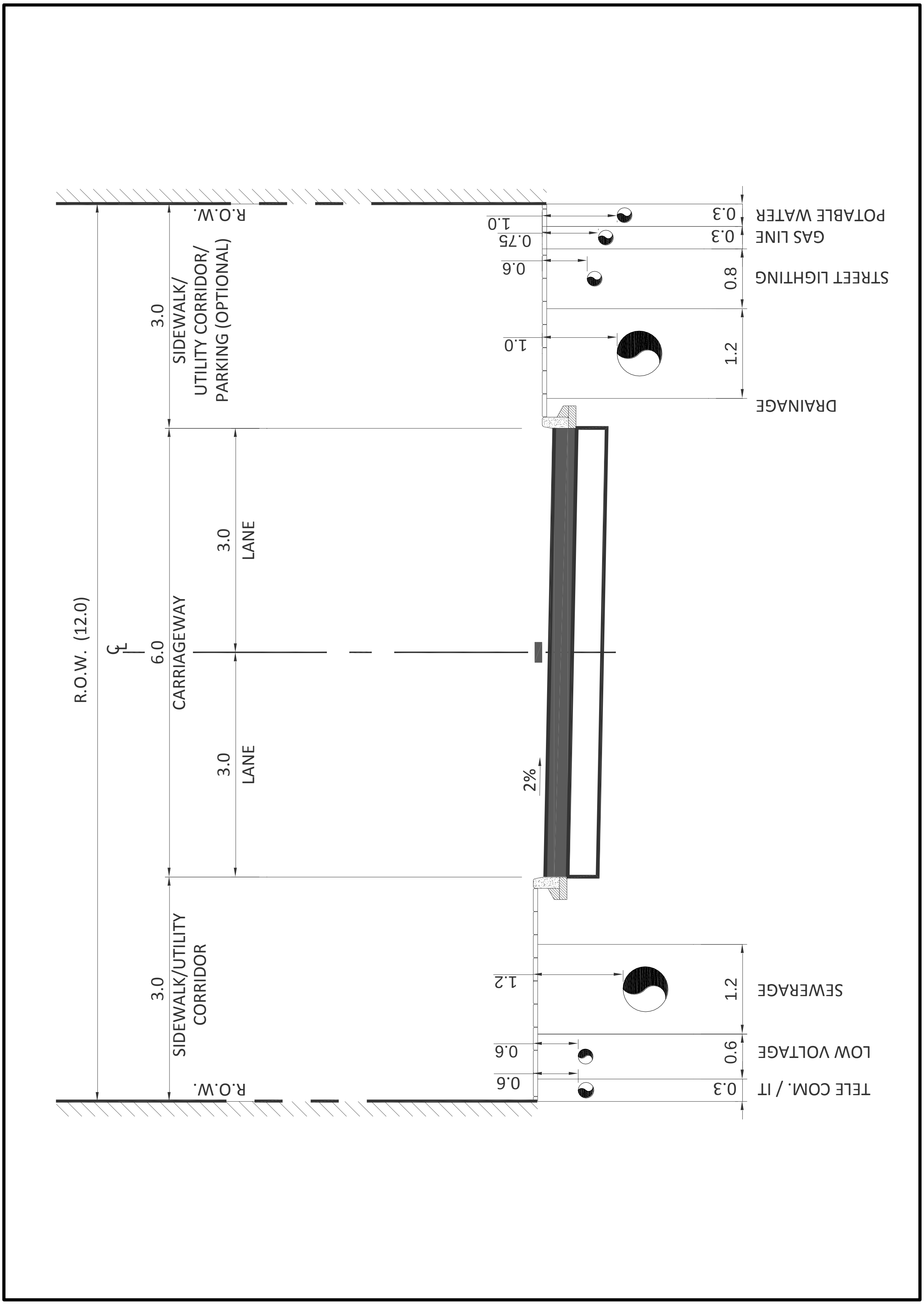


Figure 2.6: Cross Sections of Urban Local Road (12m ROW)

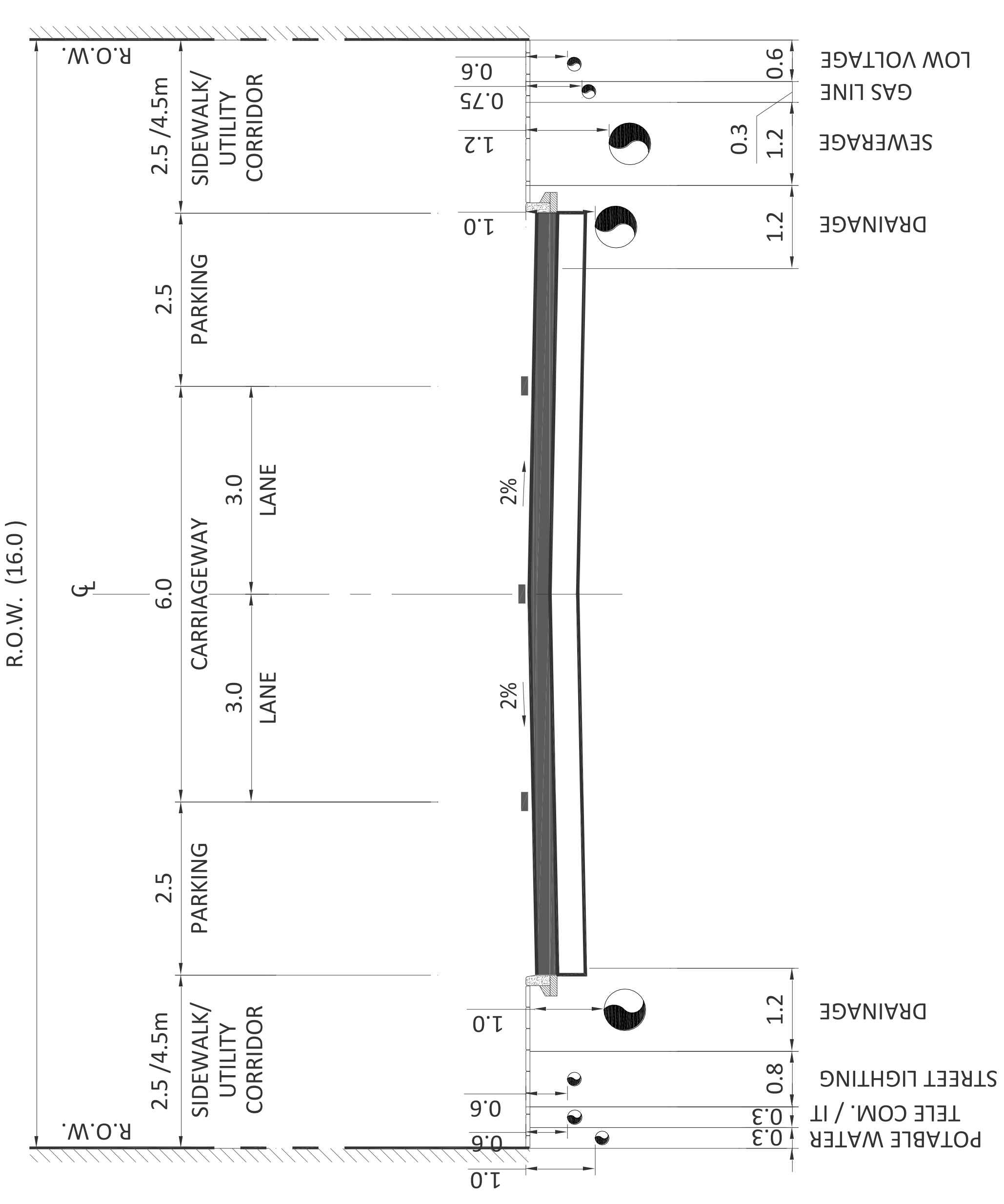


Figure 2.7: Cross Sections of Urban Local Road (16m ROW)

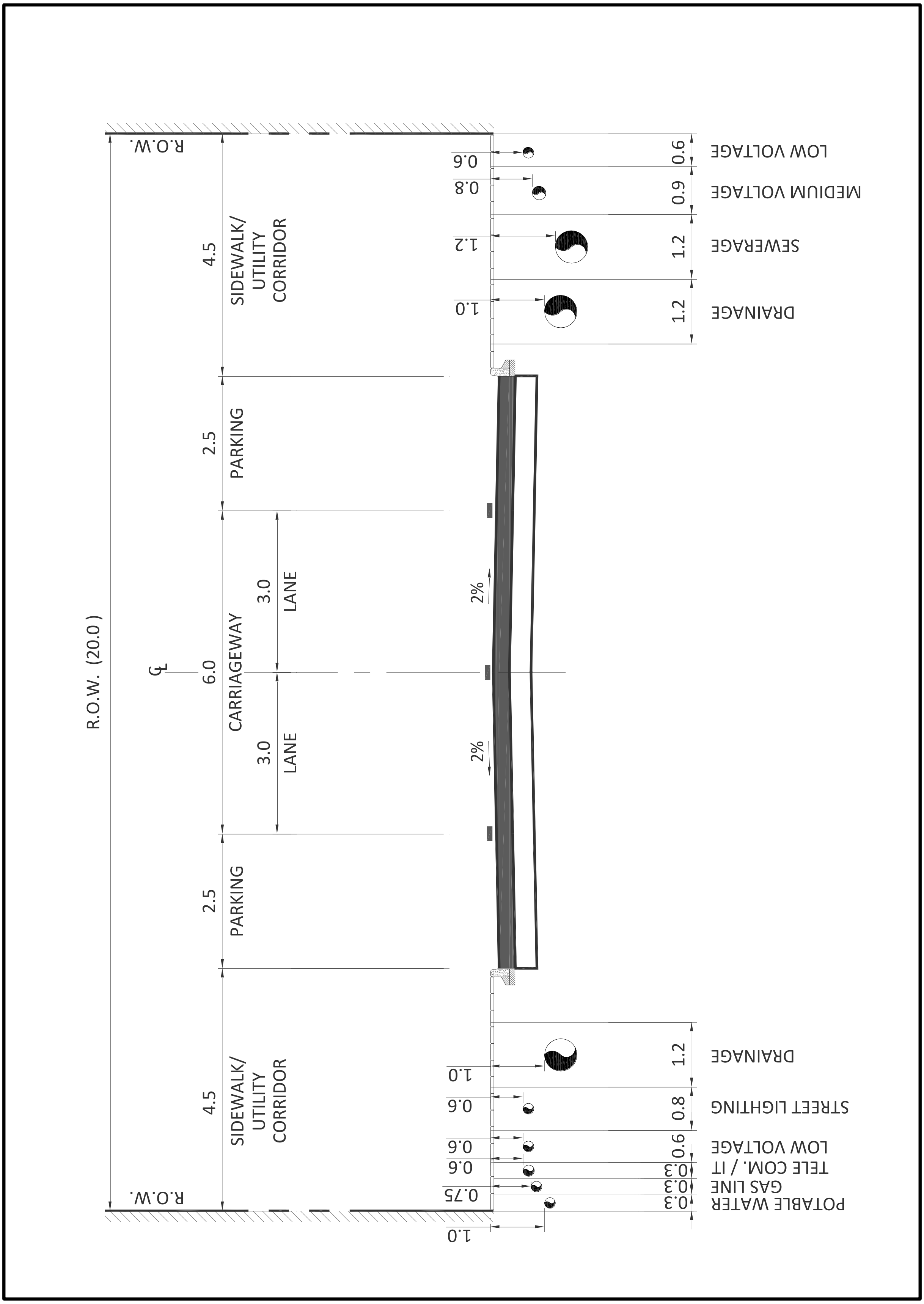


Figure 2.8: Cross Sections of Urban Local Road (20m ROW)

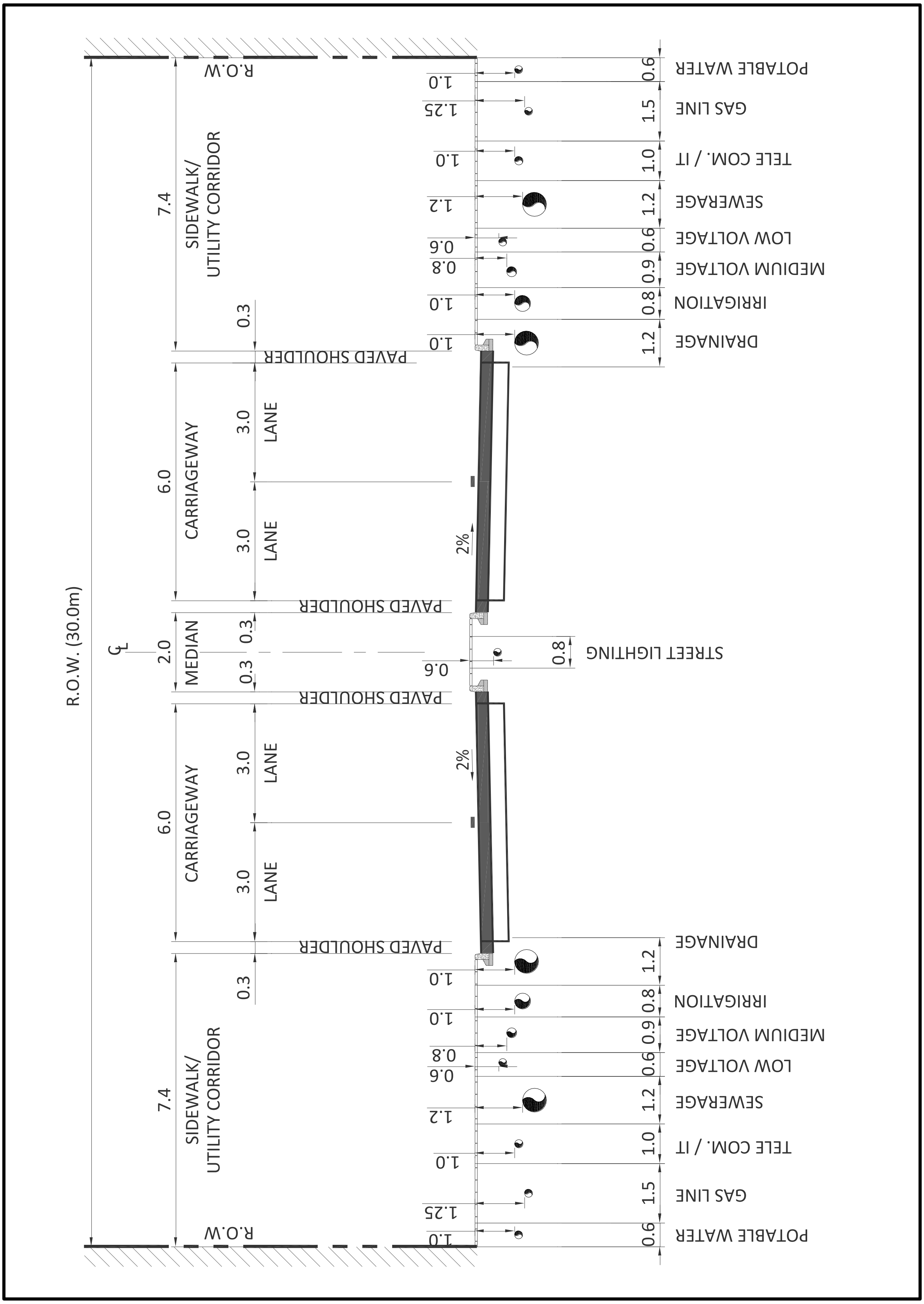


Figure 2.9: Cross Sections of Urban Collector Road (30m ROW)

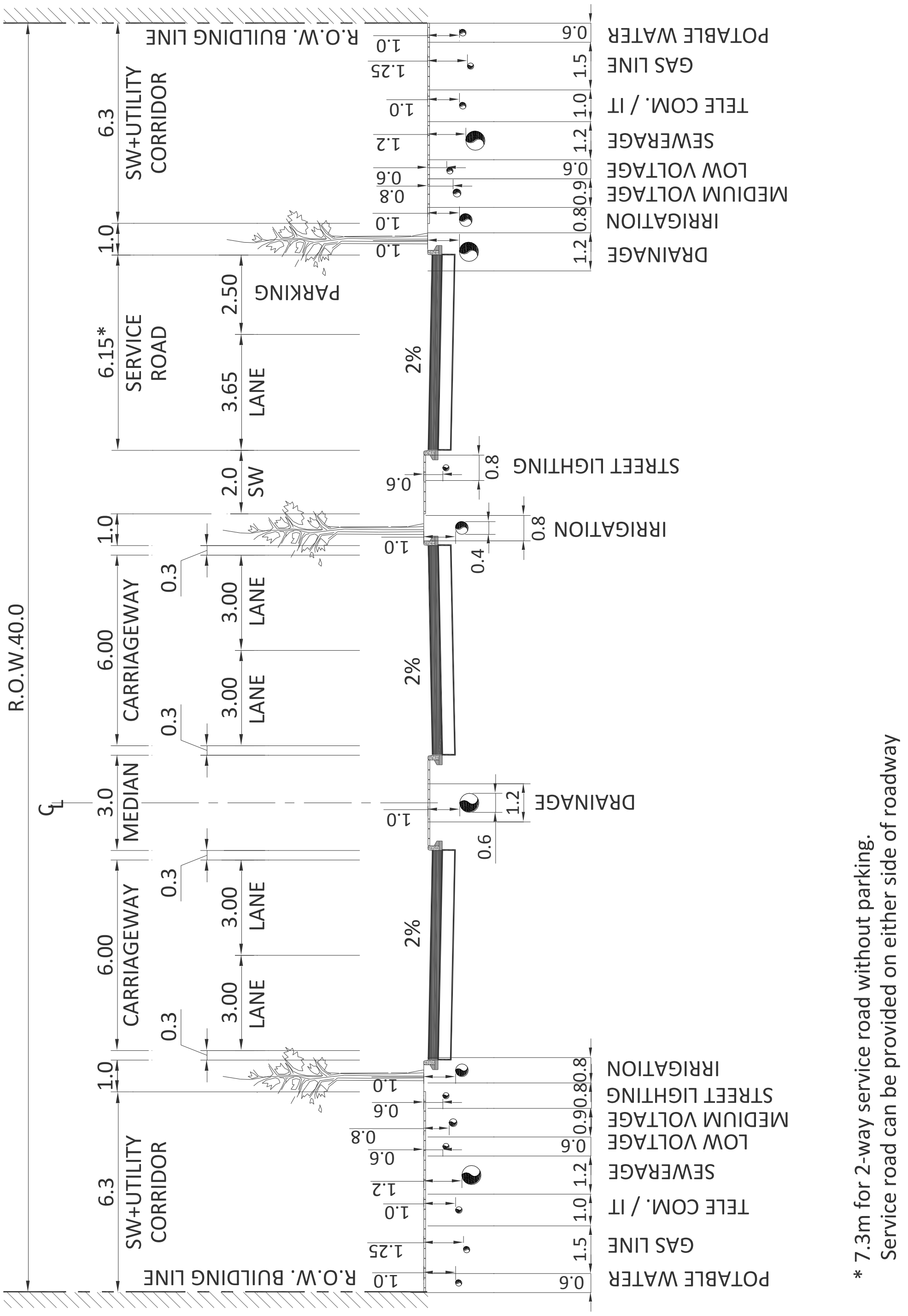


Figure 2.10: Cross Sections of Urban Collector Road (40m ROW)

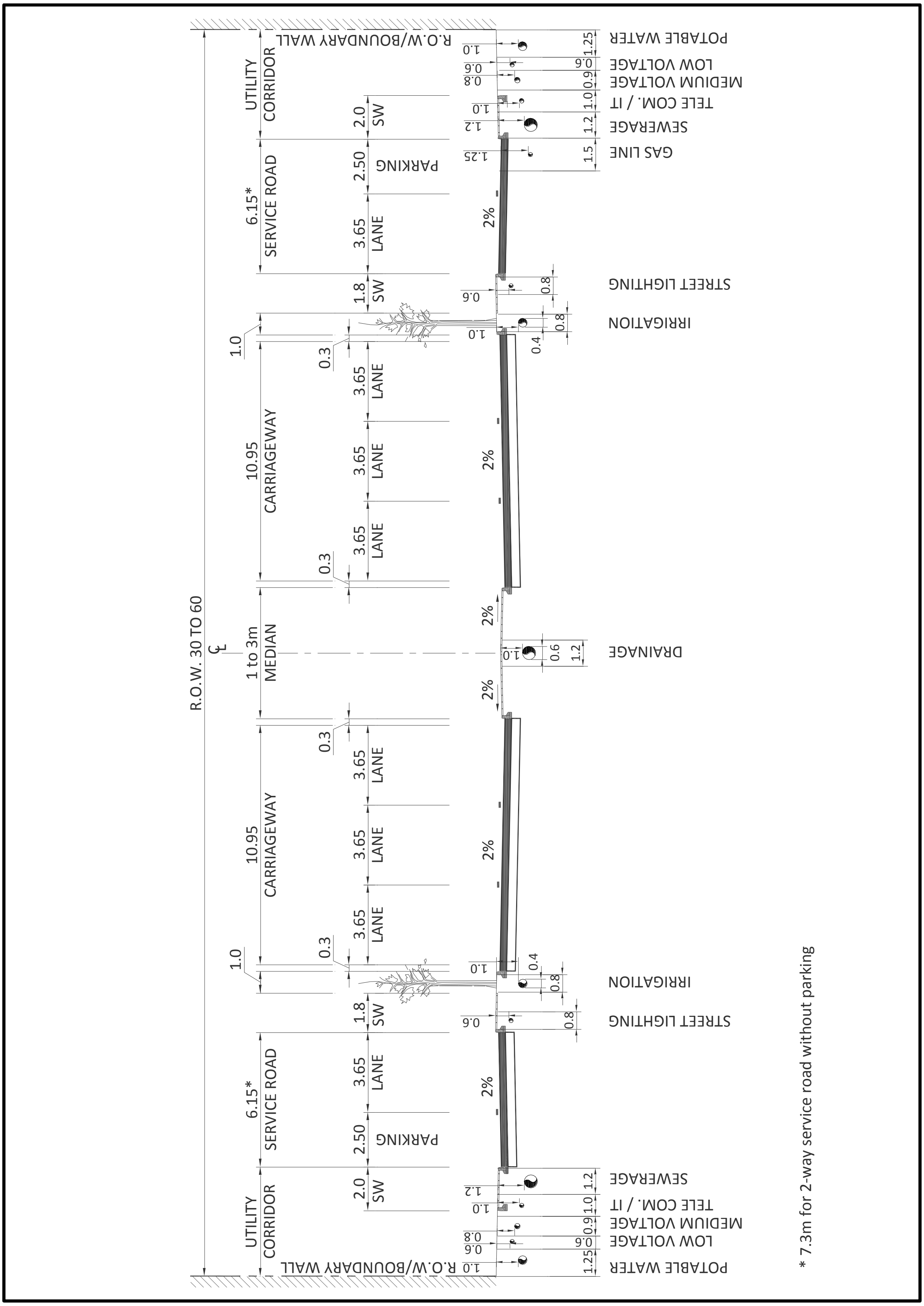


Figure 2.11: Cross Sections of Urban Major Arterial

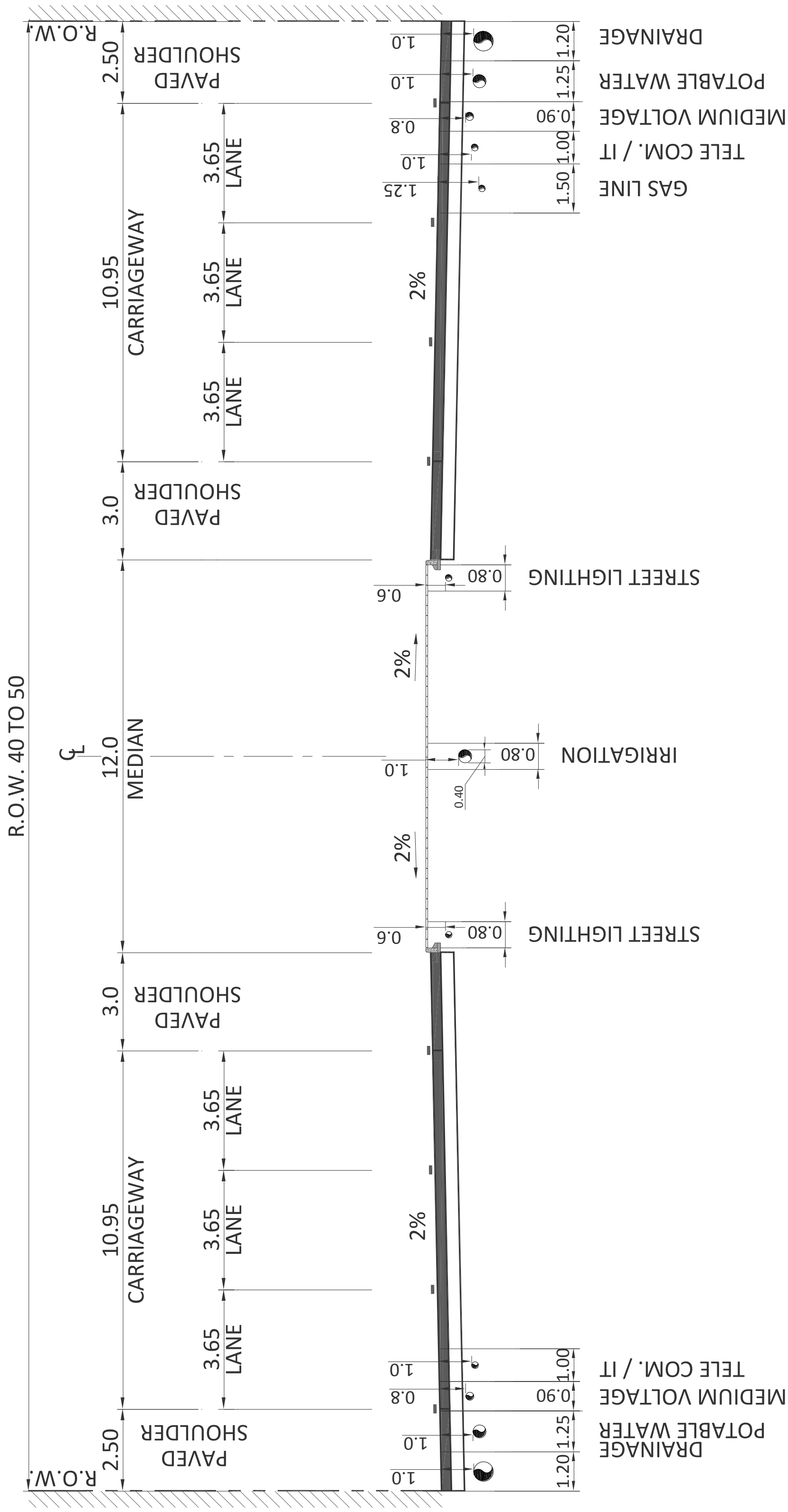


Figure 2.12: Cross Sections of Urban Expressway (3 lanes without barrier)

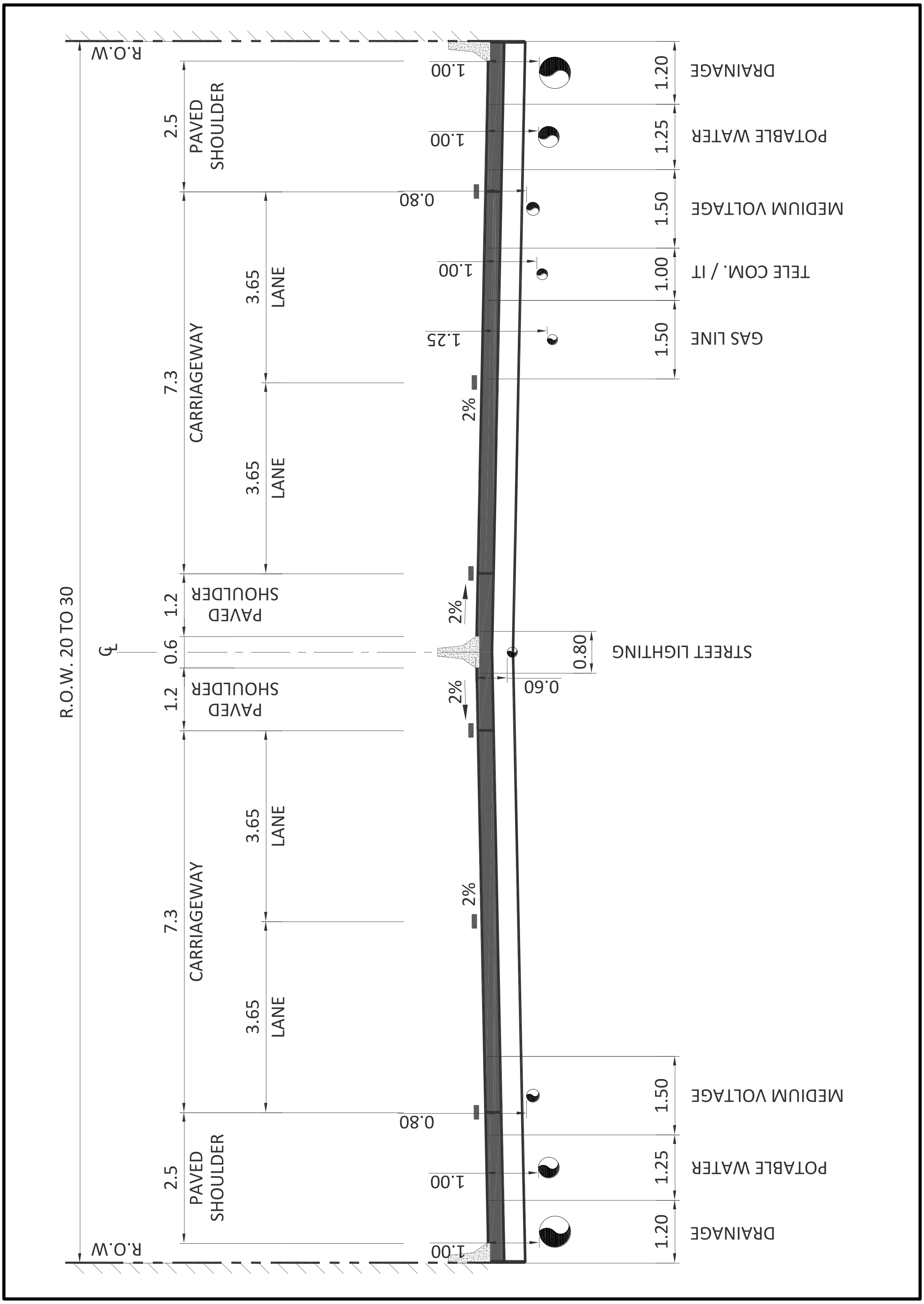


Figure 2.13: Cross Sections of Urban Major Arterial (2 lanes with barrier)

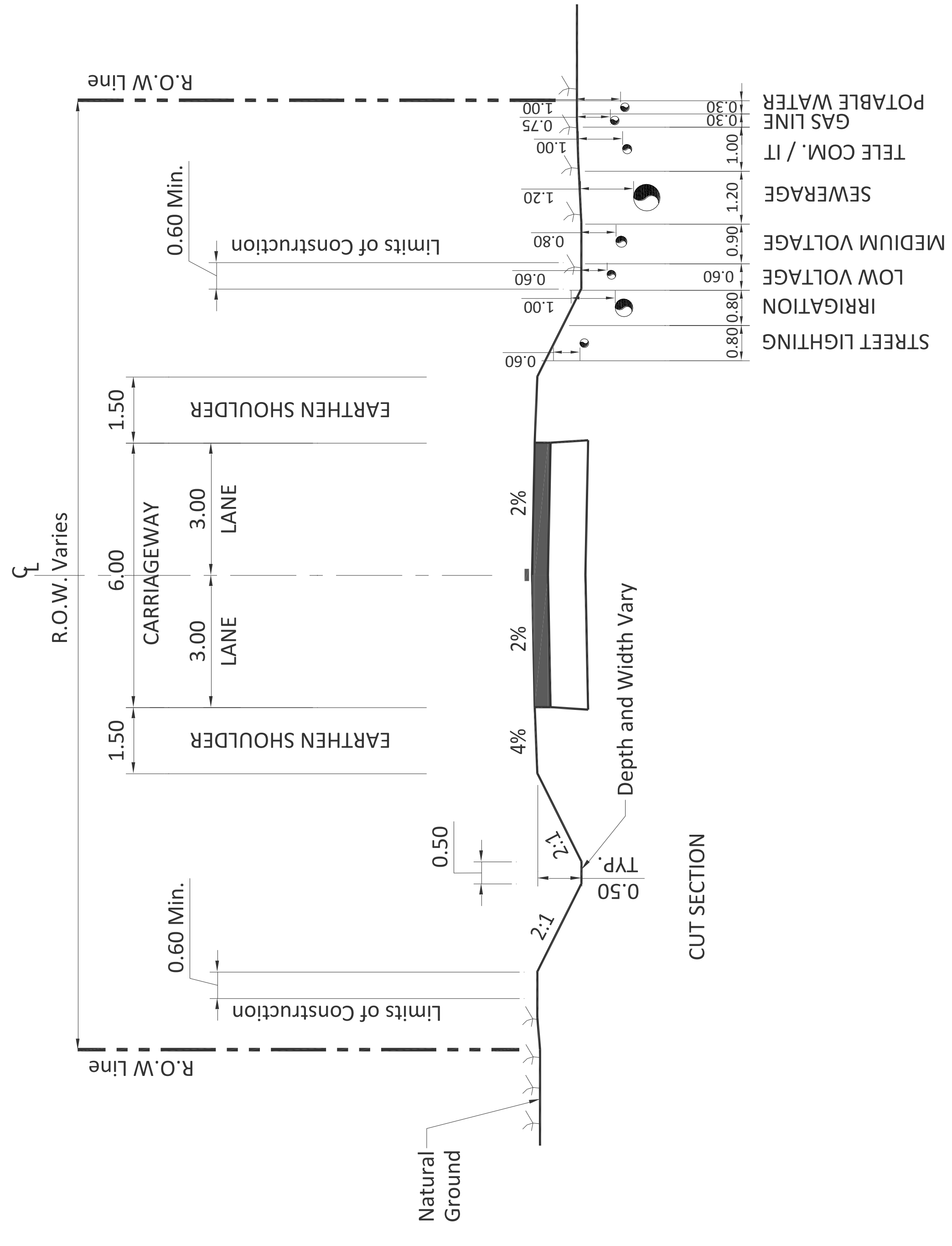


Figure 2.14: Cross Sections of Typical Rural Local Road

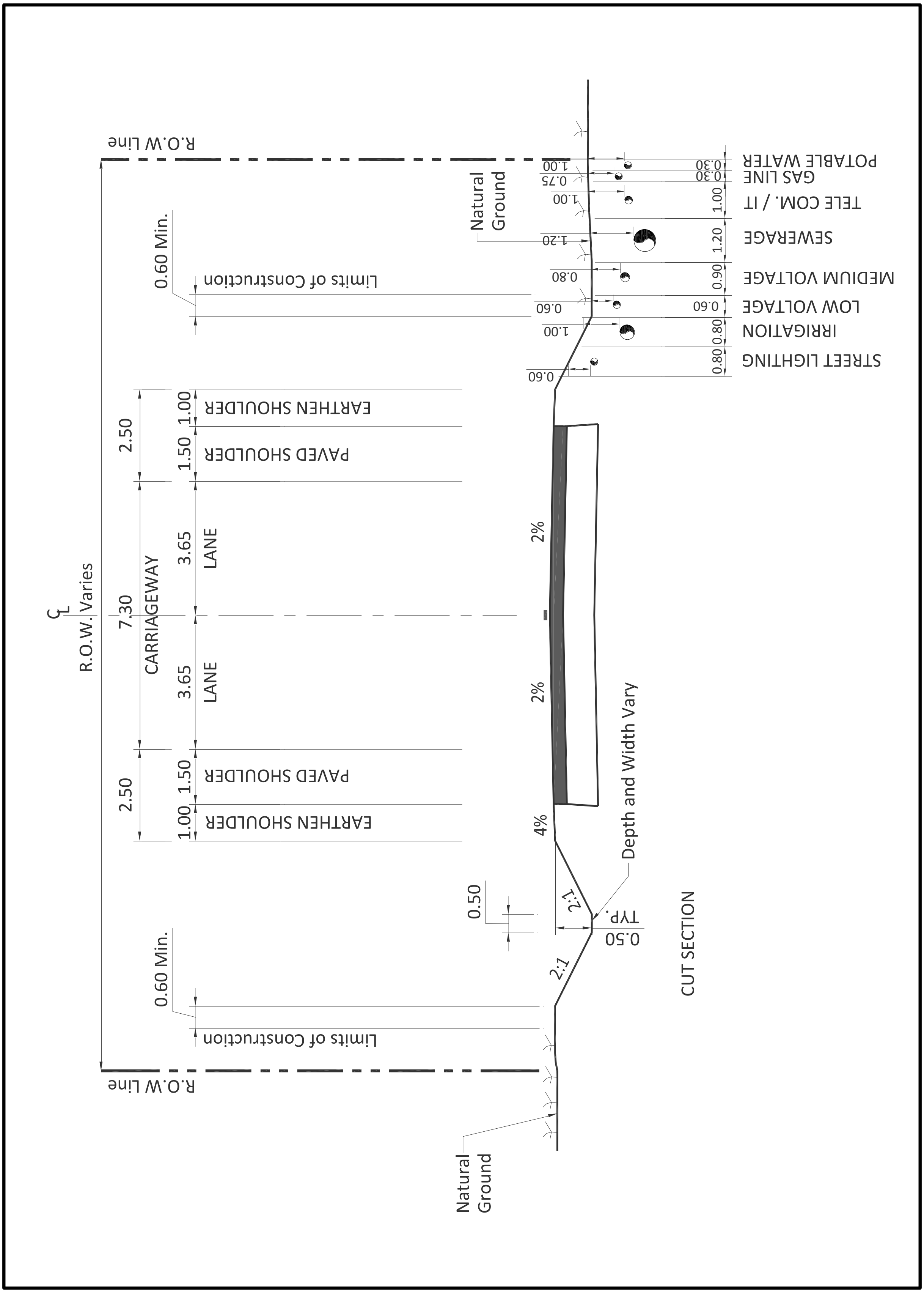


Figure 2.15: Cross Sections of Typical Rural Collector

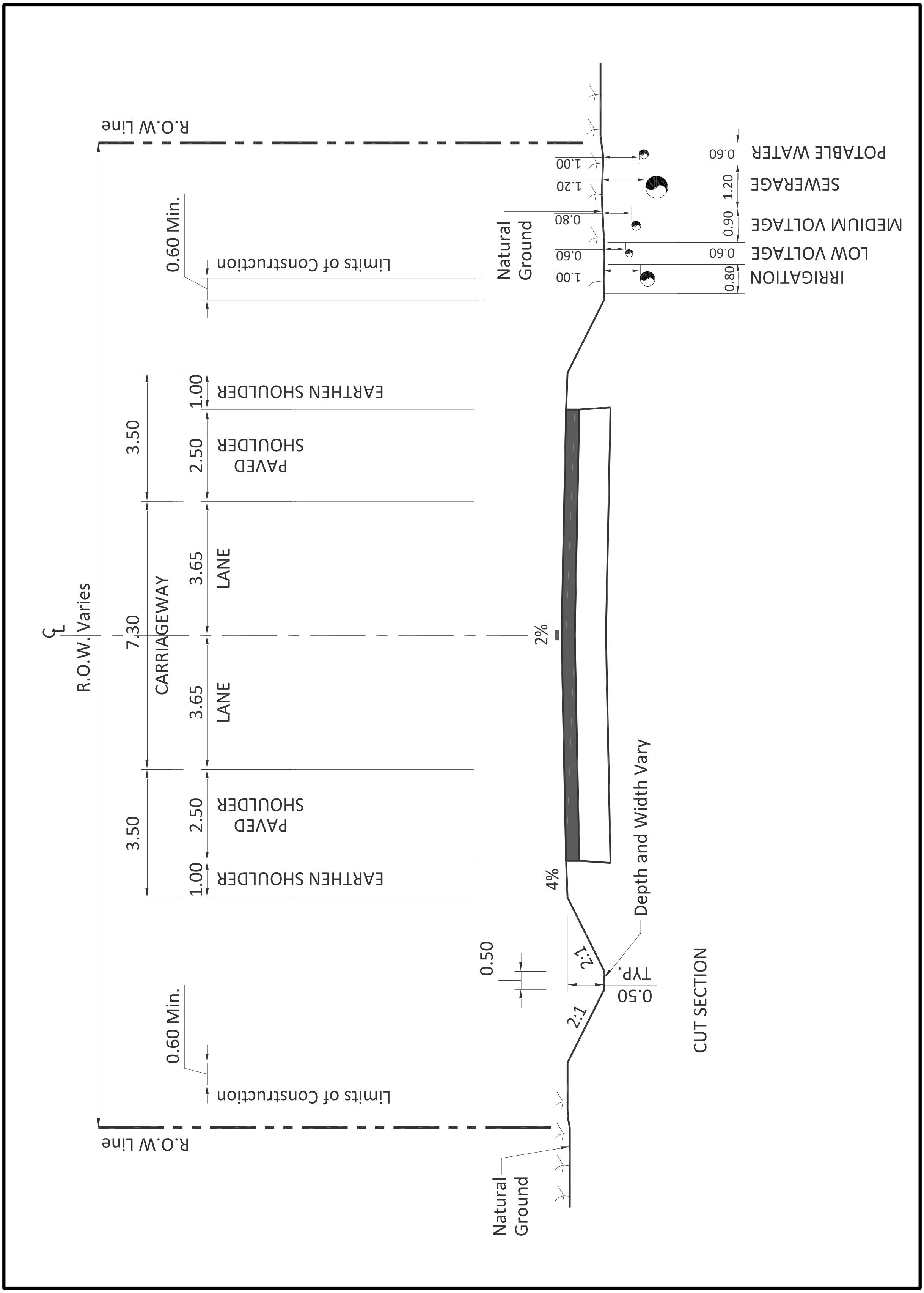


Figure 2.16: Cross Sections of Typical Rural Minor Arterial

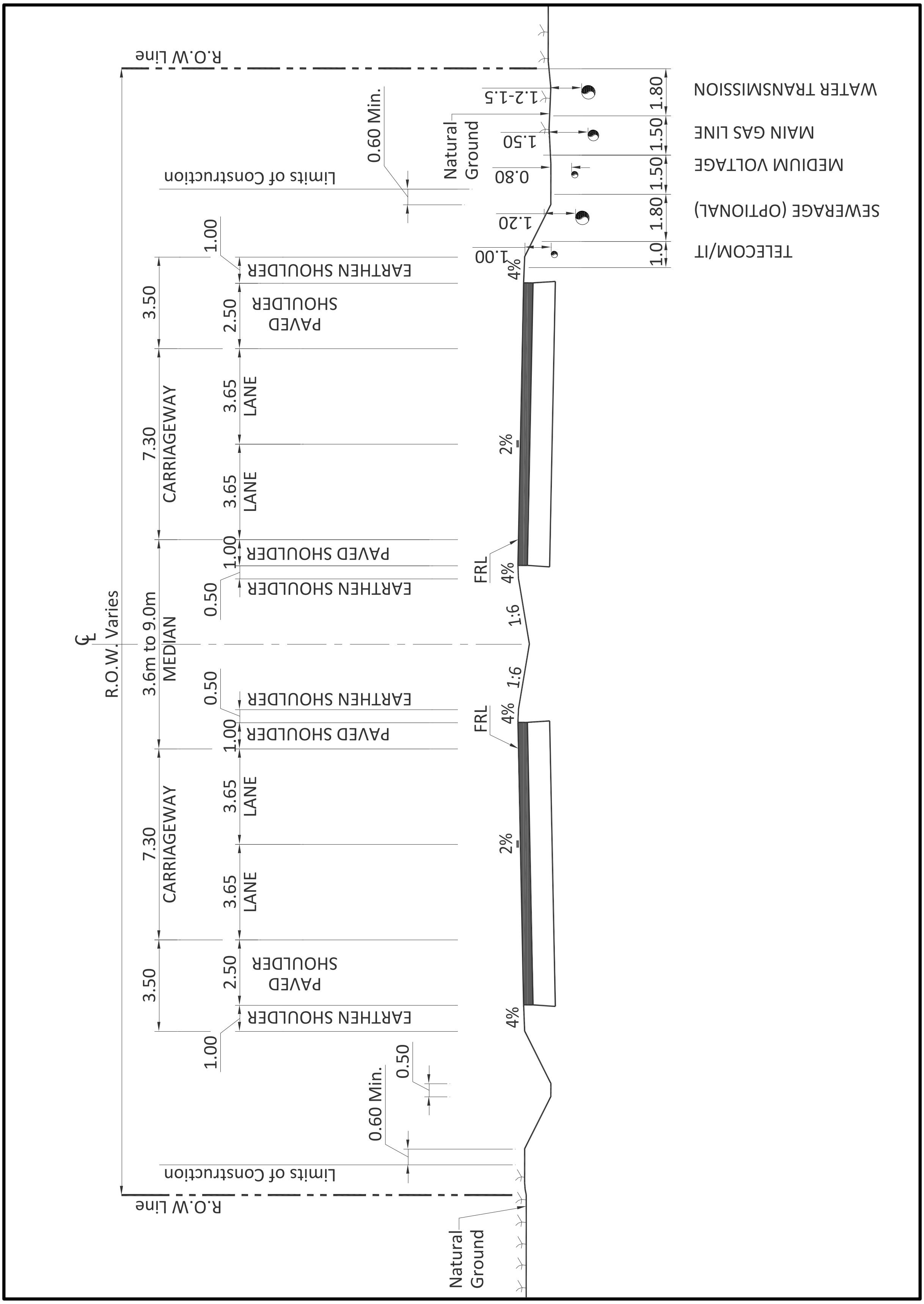


Figure 2.17: Cross Sections of Typical Rural Major Arterial

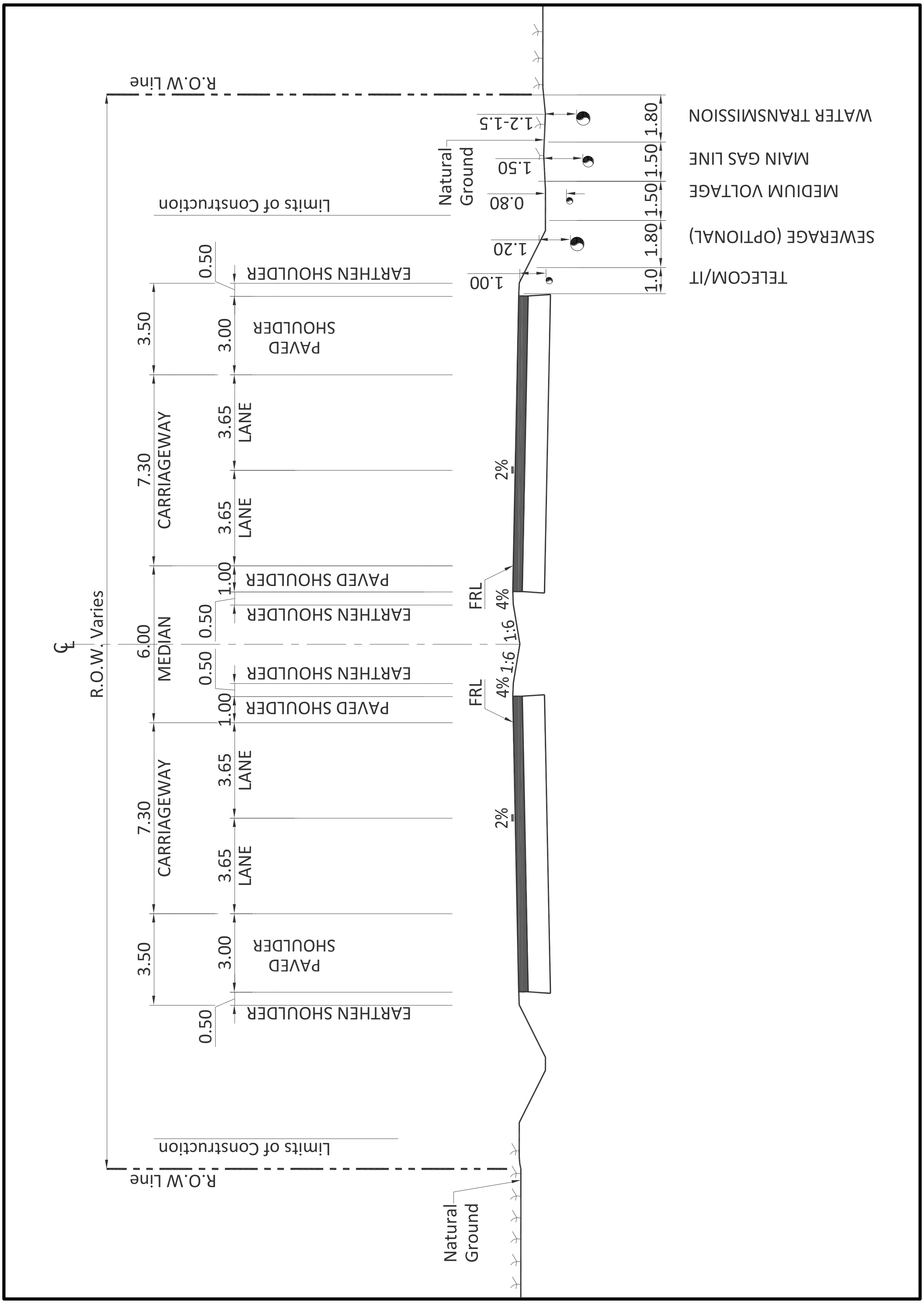


Figure 2.18: Cross Sections of Typical Rural Expressway/Freeway

Note:
The above cross sections are just for reference purpose. However, in Pakistan there is a practice of not using any separate irrigation pipe, hence its corridor may be omitted as per utility designer discrete.
Moreover, MV, LV & Street lighting cables may also run overhead but their corridor has to be allocated to include their pole foundations.

2.3.3 Utility Corridor Configuration

2.3.3.1 Typical Combined Services Utility Corridor for Simple T-Junction

Typically, road junctions include T-junctions, crossings, and roundabouts. For utilities that are pressurized, there is more opportunity to bend in the horizontal and vertical positions. However, a pressurized pipeline that has a bend will have a resultant thrust at the bend that could require a thrust block, depending on the construction material.

Parallel pressure pipelines may require a combined thrust block or the use of restraint joints. A concrete thrust block will form a restriction in the utility corridor, whereas a restraining joint will minimize the space requirement. Pressurized utilities and telecommunications and electrical cables need to be splayed across the intersecting corners of the various junction layouts. Gravity pipelines, such as foul water and storm water sewers, have less flexibility to maneuver around splayed corners. Where the junction layout is at-grade, it is likely that gravity pipelines could continue across the junction. Utilities are configured at simple T- junction is shown below. Details shown in the **Figure 2.19** is indicative and provides guidance however, each junction layout must be considered individually.

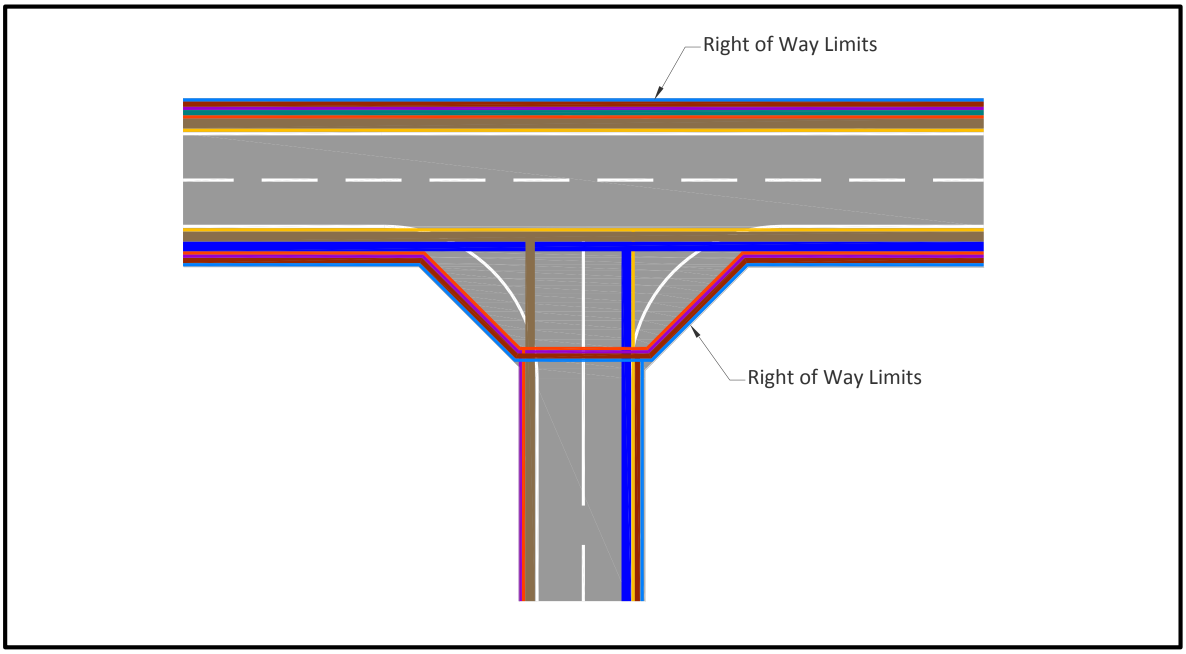


Figure 2.19: Color Coded Combined Service for a Simple T-Junction of Collector Road

2.3.3.2 Typical Combined Services Utility Corridor for Intersection

Proposed intersection utility layouts need to consider and include the following:

- Pressurized services and electrical cables can be diverted around the interchange
- Gravity-dependent services, such as foul water and storm water sewers, must be on straight alignments between manholes.

- If the intersection is at ground level, the ROWs for gravity pipelines can pass straight through the intersection.
- If there is an underpass at the intersection, it may not be possible to place the utility corridors or gravity pipelines through the intersection; a splayed utility corridor may have to be provided.

Details shown in the **Figure 2.20** is indicative and provides guidance however, each junction layout must be considered individually.

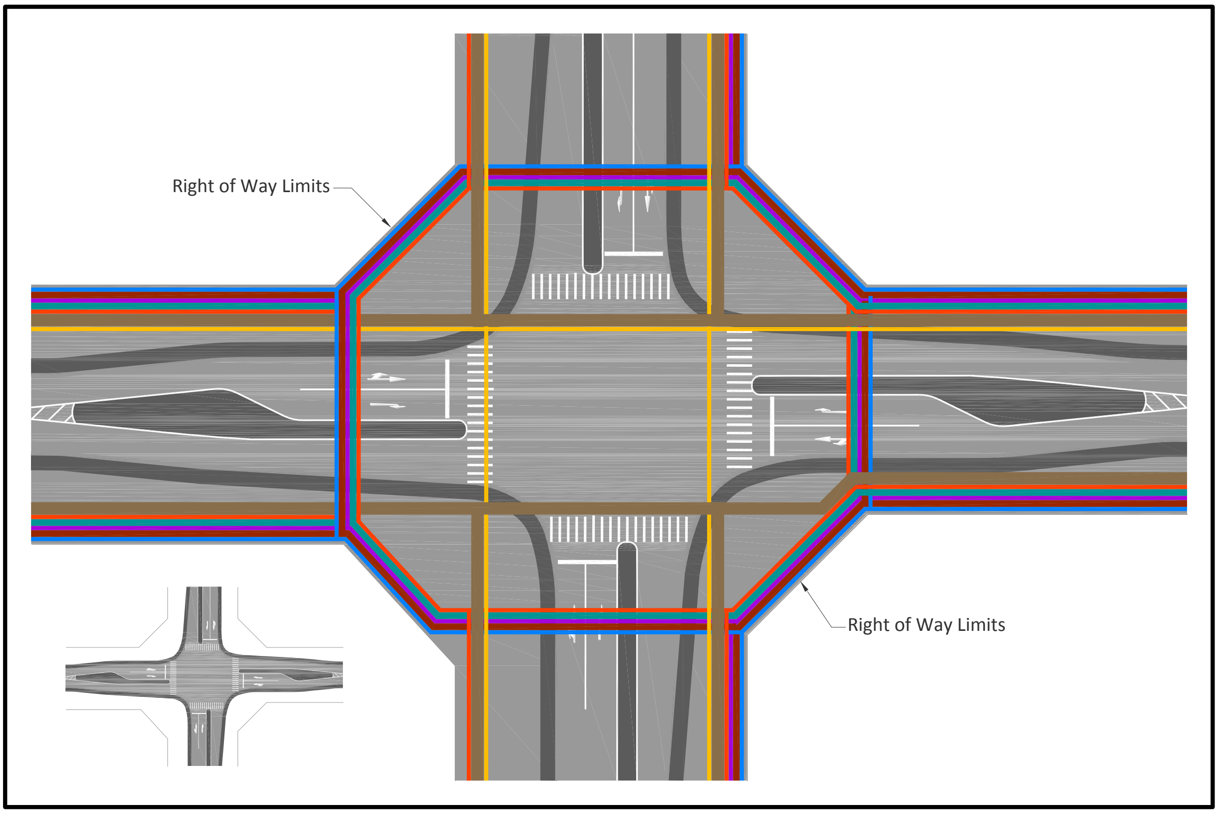


Figure 2.20: Color-Coded Combined Services Utility Corridor for a Major Collector Intersection

2.3.3.3 Typical Combined Services Utility Corridor for Roundabout

Figure 2.21 shows the color-coded combined services utility corridor on a major collector roundabout

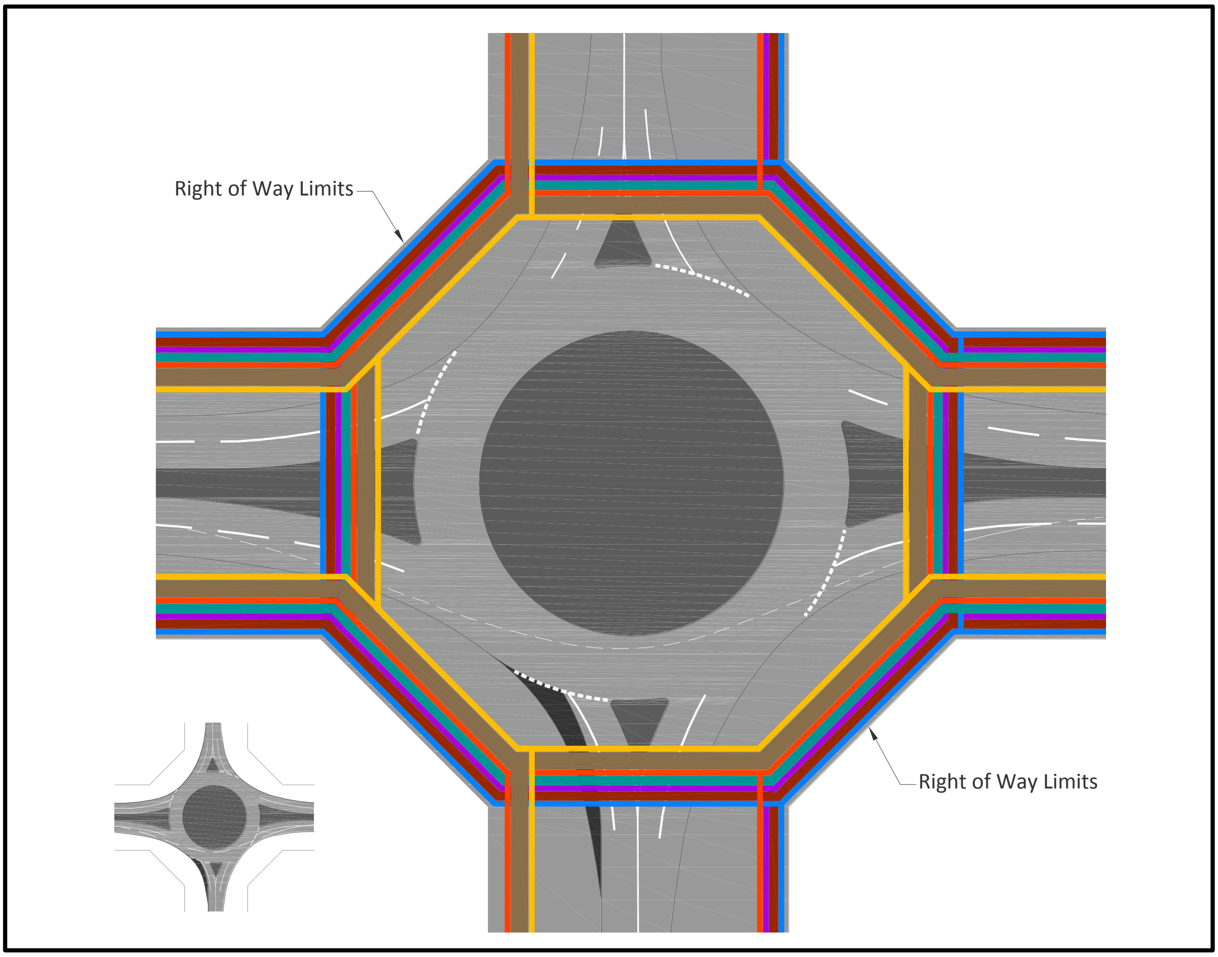


Figure 2.21: Color-Coded Combined Services Utility Corridor for A Major Collector Roundabout

2.3.3.4 Road Bridges

Road bridges are usually aboveground structures with an approach ramp at both ends. The support system allows the weight of the bridge to be dissipated to the ground; a mechanical joint allows the bridge to expand and contract due to changes in ambient temperature. Often a bridge will have some form of void within the deck structure. Telecoms, low- and medium-voltage power networks, and some service providers may be allowed to place their apparatus. Within the void (pressure services) or on top of the deck within the sidewalk. Pressure mains and cables typically can be accommodated through a bridge deck, but gravitational systems typically cannot.

Issues that will need to be resolved with the service authority and the bridge designers include:

- Access chambers at both ends of the bridge (in the approaches) must be carefully located to assist in providing access and to allow for the withdrawal for replacement of the utility.
- For pressure networks, high points require air valves to avoid accumulation of air within the pipe. These require quite a bit of space above the crown of the pipe, which can be difficult to accommodate, depending on the bridge design.
- Bridges have thermal expansion movement joints to accommodate the expansion and contraction that occurs within. The utilities have different rates of expansion and contraction and must be able to move independently of the bridge structure; therefore, support systems

must be carefully considered. The manner in which the utilities pass through the bridge abutments must be carefully designed to allow for differential movement.

- Dry services are often placed within the sidewalks of the bridges within protective steel ducts.

2.3.3.5 Road Overpasses
A road overpass is very similar to a road bridge except the end supports are usually embedded in a constructed embankment. The depth of the overpass allows routing of pressurized and non-gravity services across the overpass.

2.3.3.6 Road Underpasses
A road underpass is a belowground structure that allows traffic to pass beneath at- grade road. There is limited opportunity for routing utility services thorough an underpass.

It should be noted that underpasses require drainage, electricity, and water for lighting and life services.

2.3.4 Operational and Maintenance Consideration

2.3.4.1 Utilities' Services Reservations
Operational works involving a service provider's apparatus and corridors include the daily routine work required for that service to function. For example, irrigation system valves need to be operated regularly, and valve chambers must be sited so that operators are not at risk. Each service provider has a set of operational requirements to maintain their service.

The other type of operational work is planned or reactive maintenance when a system unexpectedly breaks down. Temporary diversions might need to be implemented and safe areas created to facilitate repair work. Larger pipeline systems require safe access to their apparatus, and the location of shaft access is very important.

2.3.5 Standard Widths for Utilities in Utilities Corridors

2.3.5.1 Utilities' Services Reservations
In Pakistan, following are the typical utilities that may form the part of utility corridors especially for Urban Roads:

1. Drainage / Sewer
2. Potable Water Supply
3. Telephone / IT
4. Natural Gas Piping
5. Electrical 11kV supply (overhead or underground)
6. Electrical 400/230 Volts supply (overhead or underground)
7. Street lighting system
8. Irrigation network
9. Storm water drainage
10. Empty space (for any future usage)
11. Any other special reserves of utility like 132 kV (EHV) underground/overhead transmission lines and oil pipelines etc.

2.3.5.2 Water Supply
The minimum recommended width in utility corridor for the water supply lines shall be as:

1. 0.3 m (1 ft) for Urban Local roads
2. 0.6 m (2 ft) for Urban Collector roads
3. 1.25 m (4.1 ft) for Urban Arterial roads

Similarly, the recommended depth from the road surface shall be between 1.0 (3.3 ft) to 2.0 m (6.6 ft) with 1.0 m (3.3 ft) being the minimum recommended depth for all the urban road classifications.

2.3.5.3 Sewerage Drain

The sewerage drainpipe sizes mainly depends on the requirement of flow & volume that is their provision in utilities' corridor varies significantly. Hence, the adequate provisions in the corridor shall be taken as per the type of area (residential, commercial or industrial) and respective road geometry.

2.3.5.4 Sewerage Manhole Spacing

Sewerage Manholes shall be provided at every intersection, change in grade, level, diameter or direction. In addition, for urban areas manholes should be spaced so that each plot can discharge its sewage in a manhole.

For Local and secondary roads, manholes minimum width should be 1.22 m (4 ft) and 1.85 m (6.1 ft).

For primary roads, its minimum width should be 1.85 m (6.1 ft) or 3.0m (10 ft) for small and large sizes of pipe respectively.

For sewerage drain, the depth from road top surface may vary from 1.2m (4 ft) to 7.62 m (25 ft).

Table 2.5: Sewerage Manhole Spacing

Sr.No	Diameter of Pipes (inch)	Maximum Allowable Spacing	
		Meter	Feet
1	9	45	150
2	12	60	200
3	15 ^{2} 24	75	250
4	27 ^{2} 39	90	300
5	42 ^{2} 60	120	400
6	Above 60	150	500

2.3.5.5 Telecommunication/IT

PTCL uses optical fibre cables for the transmission of telecommunication/IT services. The minimum recommended widths are as:

1. for Urban Local roads, 0.3 m (1 ft)
2. for primary and secondary roads, 0.75 m (2.5 ft)

The recommended depth from top surface for optical fiber / telephone utility should be 0.6 to 1.5 m (2 to 5 ft) for all the urban road classifications. Similarly, the minimum recommended distance of hand hole for this utility should be 250 m (820 ft).

2.3.5.6 Natural Gas

The minimum width in the utility corridor for the natural gas (also commonly known as Sui gas) shall be 0.3m (1 ft) for urban local road while, 1.5 m (5 ft) for Primary and Secondary roads. Similarly, the minimum recommended depth from road surface shall be 0.75 m (2.5 ft) for all the urban road classifications.

2.3.5.7 Electric Power Supply

The design of electrical utility corridors take into account issues associated with heat dissipation for the rating of the cables. In general, for buried cables, the higher the amperage or power rating, the greater the spacing requirement between cables. In those areas where ground surface temperatures are high, power cables can become de-rated if heat dissipation is not managed. However, spreading the cables horizontally has created wide corridors for electric service. Cables could be stacked to use the available space most efficiently.

In Pakistan, electric power would be supplied either through electric poles or by underground (buried) cables.

For supply through electric poles, the minimum width in the utility corridor shall be 0.6 m (2 ft) for local roads and 1.5 m (5 ft) for secondary and primary road classifications. These provisions are required for 11kV and 400V supply lines separately.

Table 2.6: Minimum Widths of Utility Corridor with Overhead Electric Supply

Sr.No	Road Classification	Width of Utility Corridor	
		Meter	Feet
1	Local Roads	4.54	14.9
2	Secondary Roads	5.63	18.5
3	Primary Roads	6.8	22.31

For underground MV and LV cables, the width reserved for electricity supply should be 0.85 m (2.28 ft) for Urban Local roads, while it should be 1.25 m (4.1 ft) for Primary and Secondary roads.

Similarly, the recommended minimum depth from road surface should be 2.3 m (7.6 ft) and hand hole distance for underground cables 45-60 m (150² 200 ft) respectively for all the urban road classifications.

Table 2.7: Minimum Widths of Utility Corridor with Underground Electric Supply

Sr.No	Road Classification	Width of Utility Corridor	
		Meter	Feet
1	Local Roads	4.79	15.72
2	Secondary Roads	6.1	20
3	Primary Roads	7.2	23.6

2.3.5.8 Empty Ducts

Empty ducts shall be provided in utility corridors for any future installation and/or to separate one utility from another. The empty duct width shall be 0.6 m (2 ft) and its depth from road top surface shall be 1.0 m (3.3 ft) for all urban road classification.

2.3.5.9 Indicative Corridor Widths for Some Common & Widely Used Road Types

Following tables may refer for planning of ‘urban local, primary & secondary roads’ types for underground/above ground utilities reservation.

Table 2.8: Utility Corridor Dimensions for Urban Local Roads

Utility	Width, m (ft)	Minimum Depth From Road Top Surface, m (ft)	Minimum Distance From Building Line, m (ft)	Hand Hole Details, m (ft)
Water Supply	0.3 (1)	1.0 (3.3)	1.0 (3.3)	N.A
Empty Duct	0.6 (2)	1.0 (3.3)	0.6 (2)	N.A
Sewerage	1.2 (4)	1.2 (5)	1.5 (5.0)	N.A
Optical Fiber / Ptcl	0.3 (1)	0.6 (2.0)	2. 0 (6.0)	0.3x0.3x0.75 (1.0x1.0x2.5)
Sui Gas	0.3 (1)	0.75 (2.5)	2.5 (8.25)	N.A
Street lighting Pole	0.8 (2.8)	0.6 (2.0)	3.0 (9.0)	0.6x0.6x0.9 (2x2x3)
Flood Water Drainage	1.2 (4)	1.0 (3.3)	4.0 (13.33)	N.A
Under Ground Electricity	1.5 (5.0)	0.6 – 0.8 (2.0 – 3.0)	3.0 (10.0)	N.A
Total Minimum Width For Utility Corridor				
With Under Ground Electric Facility				6.2 m

Table 2.9: Utility Corridor Dimensions for Urban Secondary Roads

Utility	Width, m (ft.)	Minimum Depth From Road Top Surface, m (ft.)	Minimum Distance From Building Line, m (ft.)	Hand Hole Details, m (ft.)
Water Supply	0.6 (2)	1.0 (3.3)	1.0 (3.3)	N.A
Empty Duct	0.6 (2)	1.0 (3.3)	0.9 (3)	NA
Sewerage	1.2 (4.0)	1.2 (4.0)	2.1 (7)	N.A
Optical Fiber / Ptcl	1.0 (3.3)	1.0 (3.3)	3.4 (11)	N.A
Sui Gas	1.5 (5.0	1.25 (4.1)	4.3 (14.0)	N.A
Street lighting Pole	0.8 (2.8)	0.6 (2.0)	5.0 (17.0)	0.6x0.6x0.9 (2x2x3)
Under Ground Electric Supply	1.5 (5.0)	0.6 – 0.8 (2.0 – 3.0)	5.5 (18)	As per design need
Total Minimum Width For Utility Corridor				
With Under Ground Electric Facility				7.2 m

Table 2.10: Utility Corridor Dimensions for Urban Primary Roads

Utility	Width, m (ft.)	Minimum Depth From Road Top Surface, m (ft.)	Minimum Distance From Building Line, m (ft.)	Hand Hole Details, m (ft.)
Water Supply	1.25 (4.1)	1.0 (3.3)	1.0 (3.3))	N.A
Empty Duct	0.6 (2)	1.0 (3.3)	1.5 (5.0)	N.A
Sewerage	1.2 (4.0)	1.2 (4.0)	3.0 (10.0)	N.A
Optical Fiber / Ptcl	1.0 (3.3)	1.0 (3.3)	4.0 (13.4)	250 (820)
Sui Gas	1.5 (5)	1.25 (4.1)	5.0 (16.66)	-
Street Lighting Pole	0.8 (2.8)	0.6 (2.0)	6.0 (20.0)	0.6x0.6x0.9 (2x2x3)
Under Ground Electricity	1.5 (5.0)	0.6 – 0.8 (2.0 – 3.0)	6.5 (21.66)	As per design need
Total Minimum Width For Utility Corridor				
With Under Ground Electric Facility				7.85 m

2.4 DRAINAGE CHANNELS

In roadway design, the drainage is an important element. The roadway drainage design is to provide the required services which allow the public suitable use of the roadway during times of important run-off and which minimize the possibility of adverse effects on adjacent property and existing drainage patterns. The requirement for satisfactory road drainage has a direct bearing on the following:

- The ability to use the road during and after rainfall
- The long-term serviceability of the road structure
- The provision of an acceptable urban environment
- Minimizing health risk caused by long-term surface ponds and stagnant waters

The construction of a highway may increase the risk of flooding to properties. The highway drainage system should provide four primary functions that, because of land use constraints, are usually dealt with differently in urban and rural situations. These functions are shown in **Figure 2.22.**

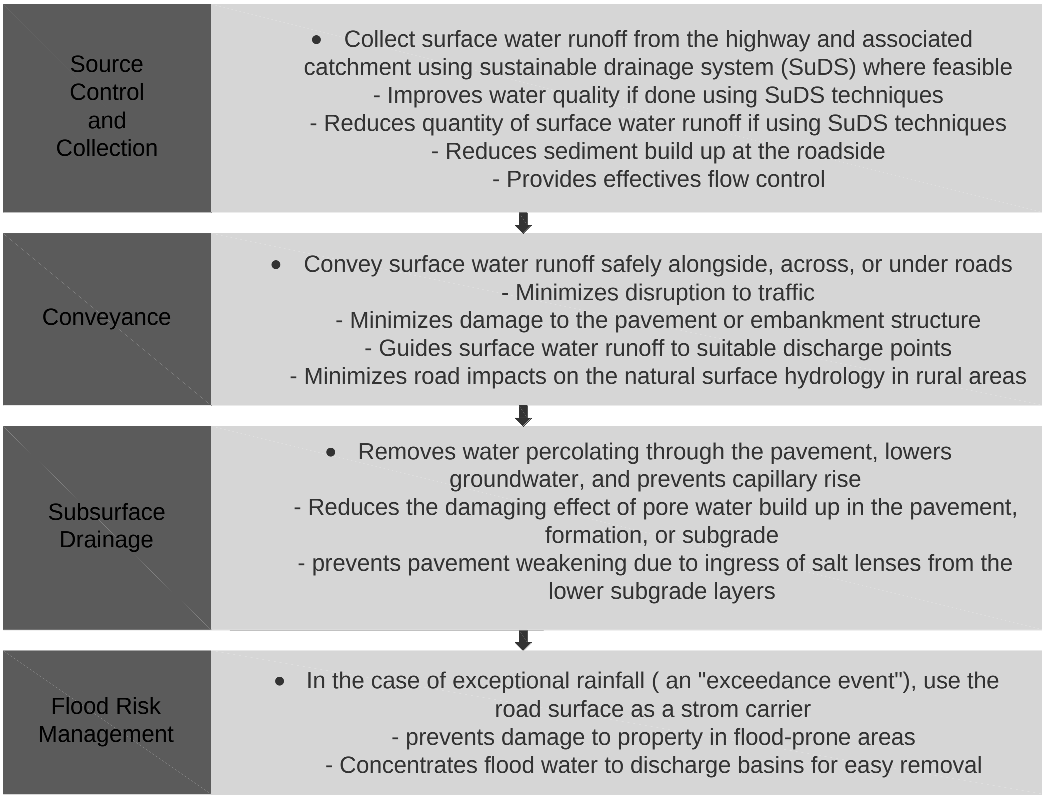
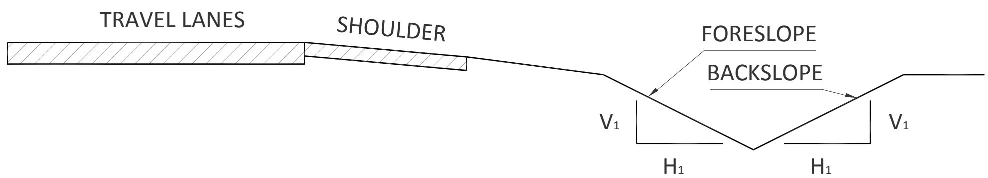


Figure 2.22: Functions of Highway Drainage

Drainage channels are provided along the road for quick runoff of the water during rains. In urban areas, usually covered roadside drains are provided whereas in rural areas, roads have open channels or ditches drainage. A typical setup is provided in **Figure 2.23** and **Figure 2.24**.



Foreslope: V_1/H_1 = 0.20 (1V:5H) desirable maximum
= 0.33 (1V:3H) absolute maximum
Backslope: V_2/H_2 = 0.33 (1V:3H) maximum
Change in slope: $V_1/H_1 + V_2/H_2$= 0.40 maximum

Figure 2.23: Typical V-Shape Drainage Channel

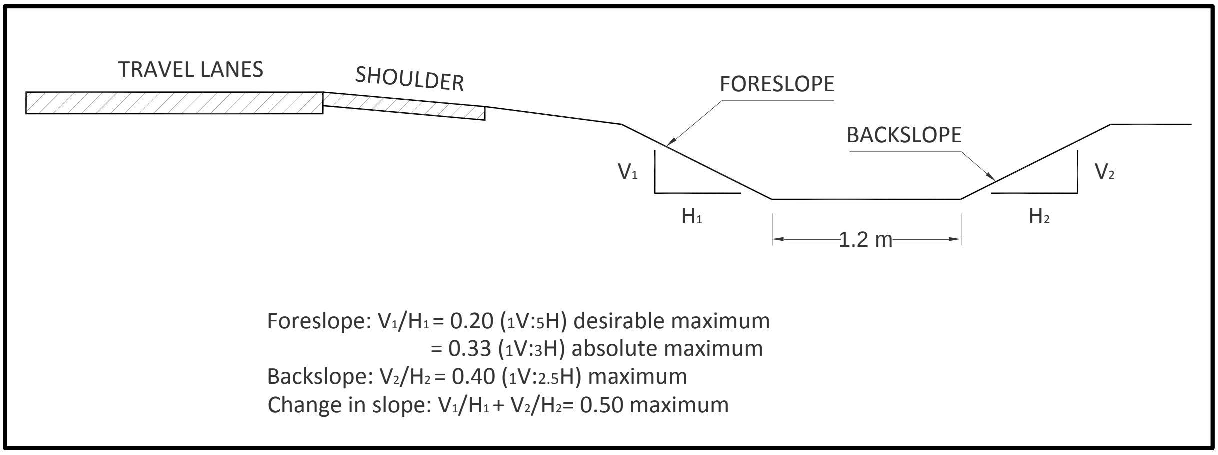


Figure 2.24: Typical Flat Bottom Drainage Channel

2.5 SIDE SLOPES

Side slopes should be designed to provide a reasonable opportunity for drivers who run off the road to recover or come to a stop without overturning. Earth cut-and-fill slopes should be flattened and liberally rounded as fitting with the topography and kept consistent with the overall type of highway. Side slope is expressed as a ratio of elevation change to lateral dimension from the beginning of the slope (Vertical: Horizontal).

Roadside slopes flatter than 1:4 can be successfully traversed by vehicles, and in most cases, the driver can maintain control and fully recover. Roads designed with such slopes may experience multiple roadside encroachments that are never recorded as crashes, because the driver is able to recover. Side slopes as steep as 1:3 are generally traversable, but not recoverable.

Effective erosion control, low-cost maintenance, and adequate drainage of the subgrade are dependent upon proper shaping of the side slopes. The rounding and flattening of slopes minimizes drifting and washout of loose material such as sand, thereby reducing maintenance costs. Detailed analysis of soil data determines the stability of the slopes and the erosion potential. The design should reflect not only initial construction cost but also the cost of maintenance, which is dependent on slope stability.

Slopes in earth cuts should not be steeper than 1:2 and preferably should be 1:3 to allow the use of mechanical maintenance equipment on the slope. If insufficient width requires slopes steeper than 1:2, then partial or full retaining walls or some method of slope stabilization should be used. Retaining walls should be set back from the roadway. Steep-sided cuts greater than 1:2 must be protected by a roadside barrier.

2.6 CLEAR ZONES AND LATERAL OFFSET

"The Clear Zone is the total width of a traversable land free of objects for the recovery of errant vehicles measured from the edge of the carriageway. It includes shoulders, bike lanes, and auxiliary lanes, unless the auxiliary lane functions as a through lane. The width of the Clear Zone is a function of the speed, traffic volume, and embankment slope. The Clear Zone should be clear of any unyielding fixed object including trees, utility poles, sign supports, and structures. The positioning of signs and other street furniture should be in accordance with *Manual of Uniform Traffic Control Devices*" [1].

In urban areas where right-of way is limited, it is not practical to provide the required Clear Zone. Urban areas are characterized by kerbs, sidewalks; lower operating speeds, frequent traffic stops, and turning movements. It is recommended that fixed objects must have a minimum clearance (lateral offset) from the face of the kerb. Fixed objects include but are not limited to structures, traffic signs, sign supports, light poles, utility poles, fire hydrants, and roadside furniture. Structures and fixed objects should not be placed within 1.2 m of the edge of shoulder or 0.6 m from the face of the kerb.

Same minimum criteria should be followed for the rural roads, however where space permits, the clear zone width may be increased to 6.5 m and 5 m for design speed of 80 km/h or less and 10 m and 7.5 m for design speed greater than 80 km/h for back slopes and fore slopes respectively.

2.7 FRONTAGE OR SERVICE ROADS

Frontage or service roads generally run parallel to and on one or both sides of arterials. They serve many different functions depending on the type of arterial they serve. Most prominent function of frontage roads is control of access to the arterials. Frontage roads provide access to and circulation of neighbourhood traffic on each side of the arterial. Continuous frontage roads provide an alternative route to arterials.

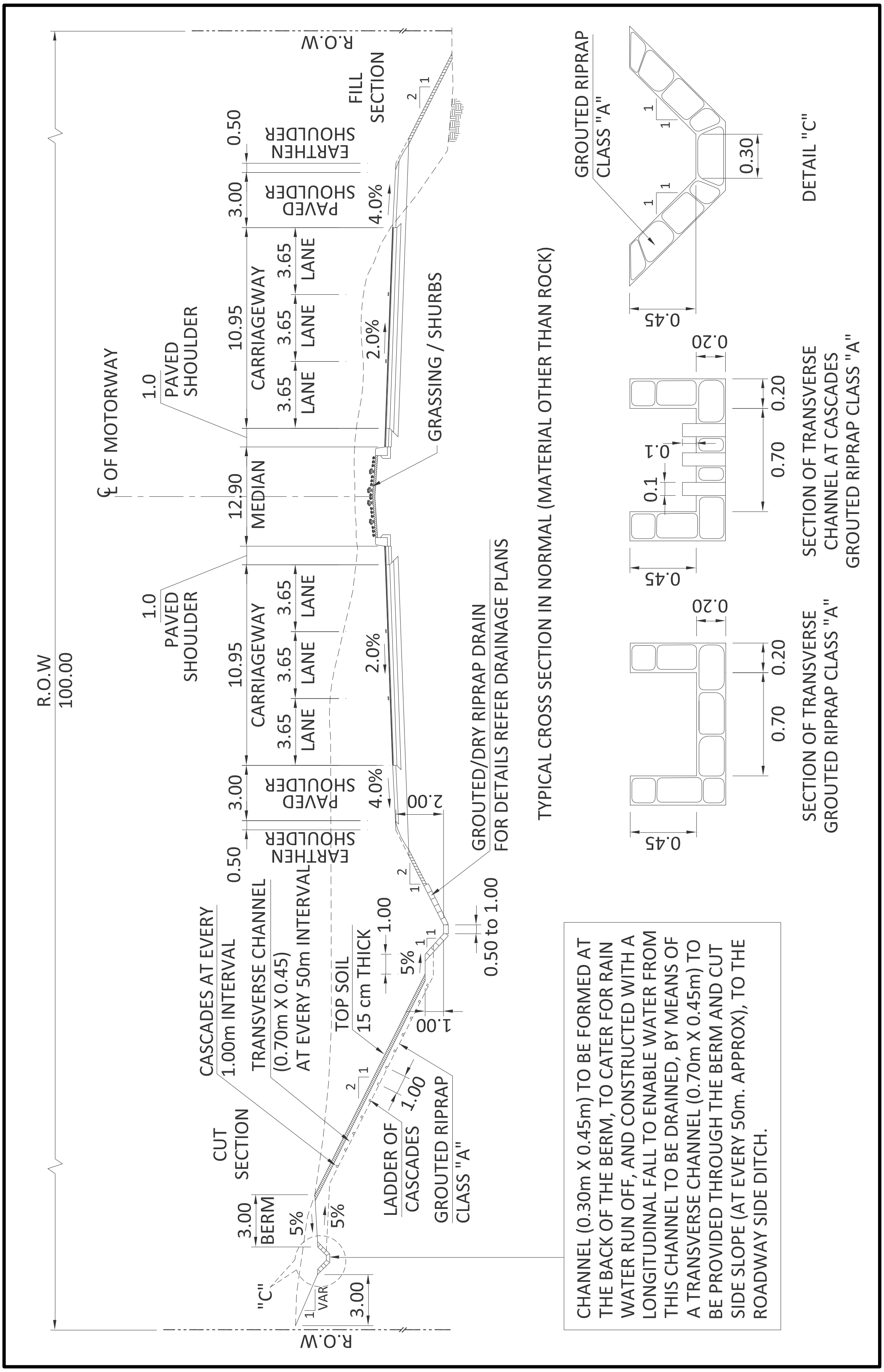


Figure 2.25: Typical Motorway Cross Section in Material Other Than Rock

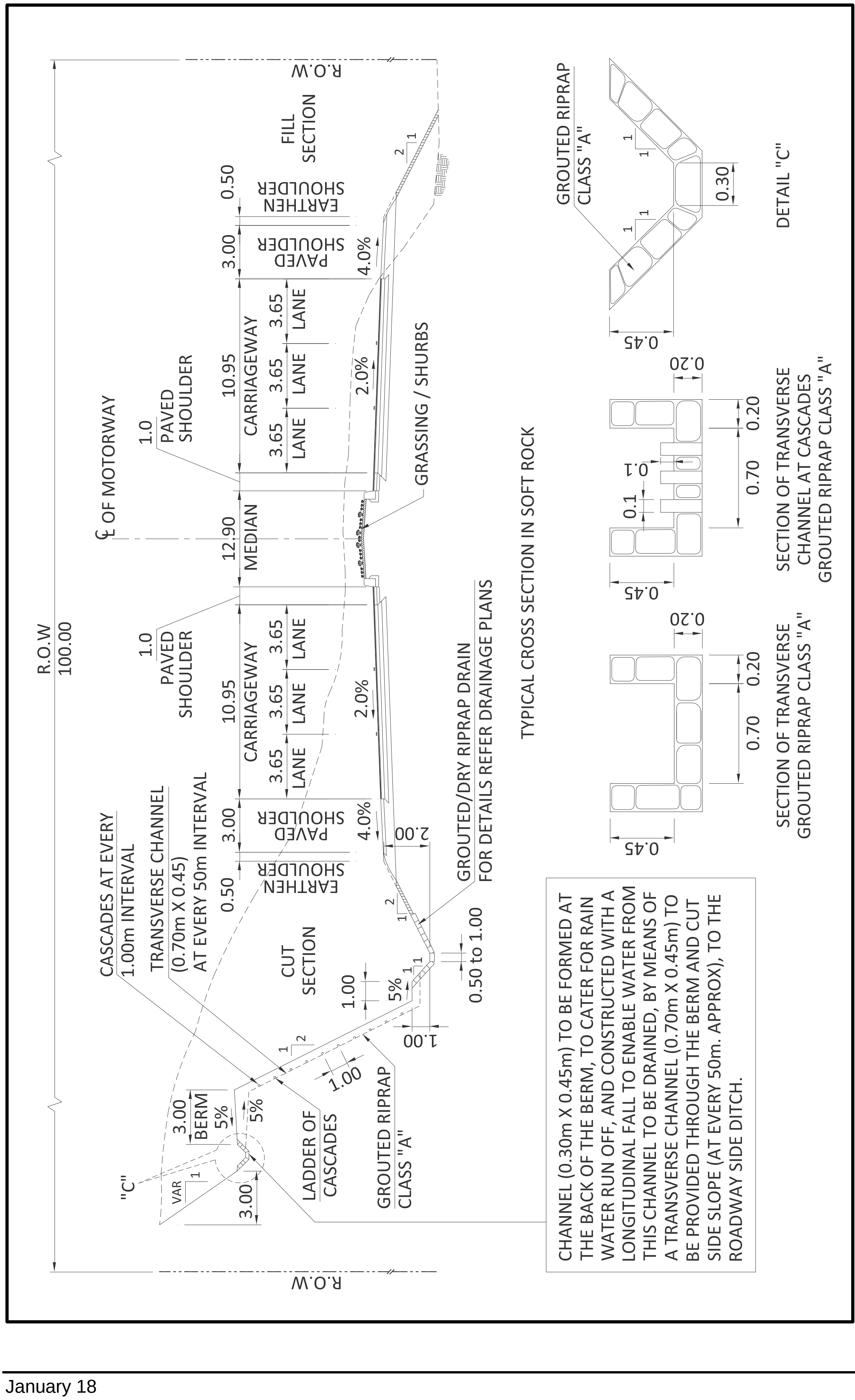


Figure 2.26: Typical Motorway Cross Section in Soft Rock

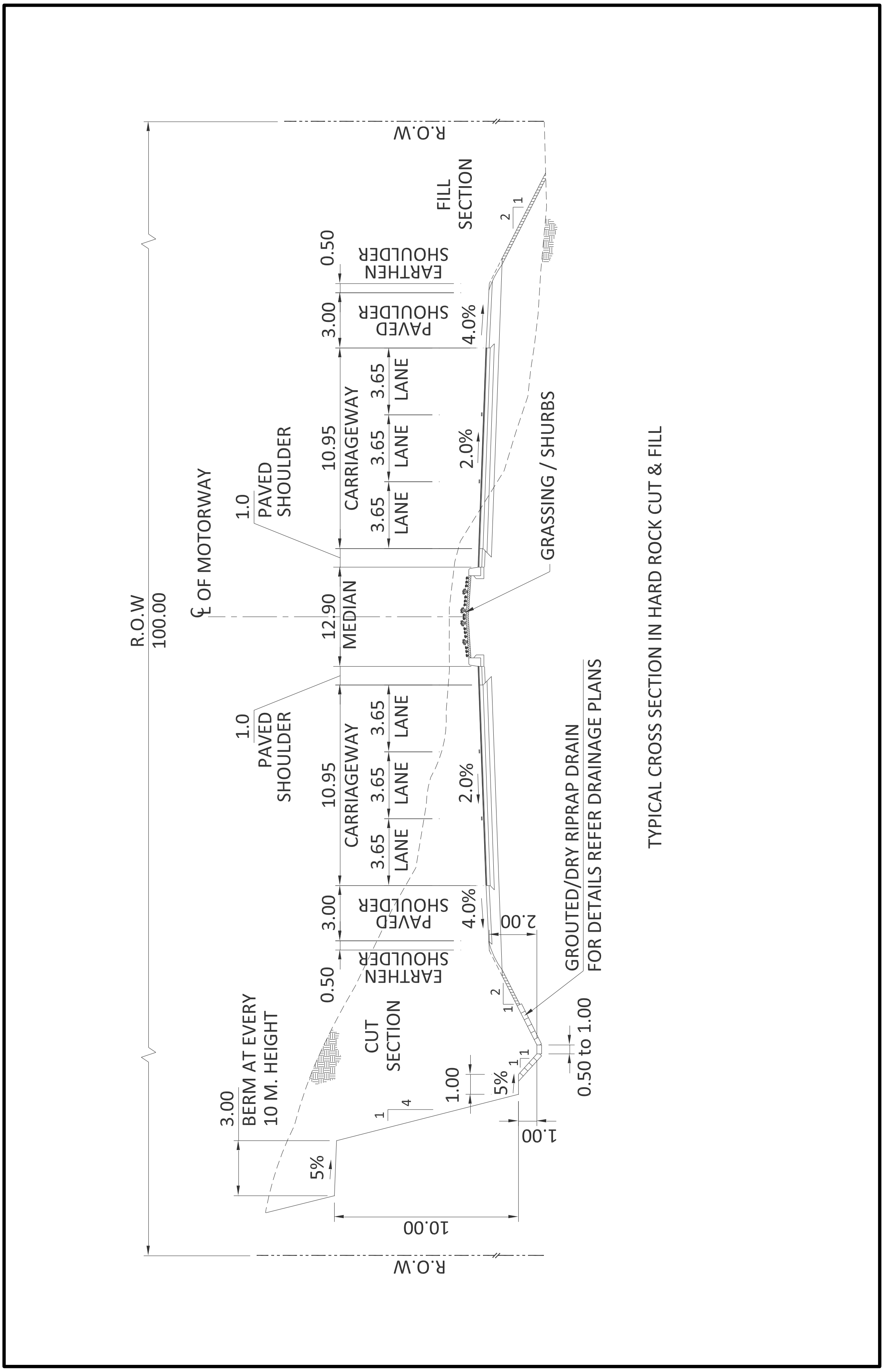


Figure 2.27: Typical Motorway Cross Section in Hard Rock

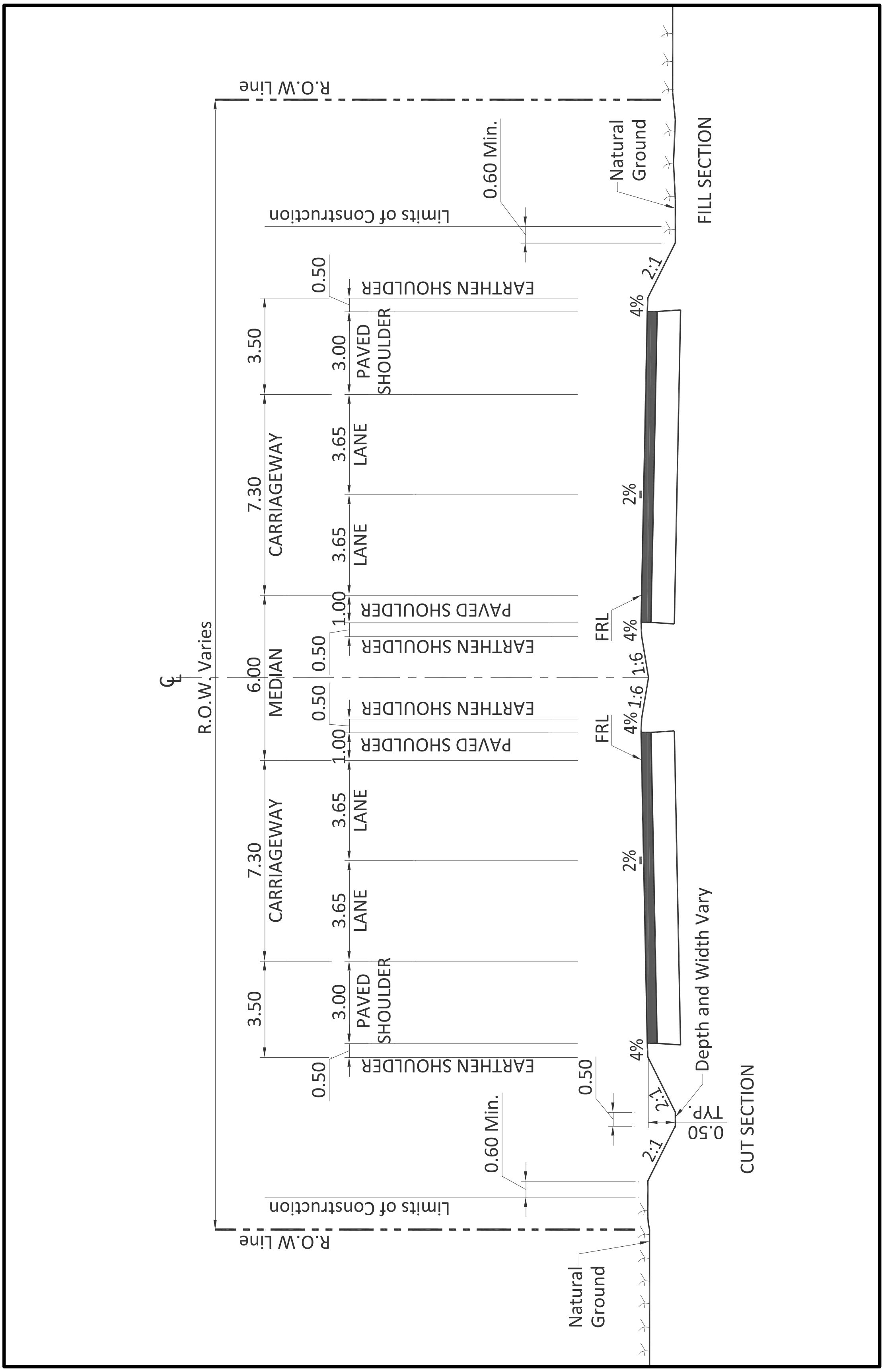


Figure 2.28: Typical Rural Expressway/Freeways (National Highway)

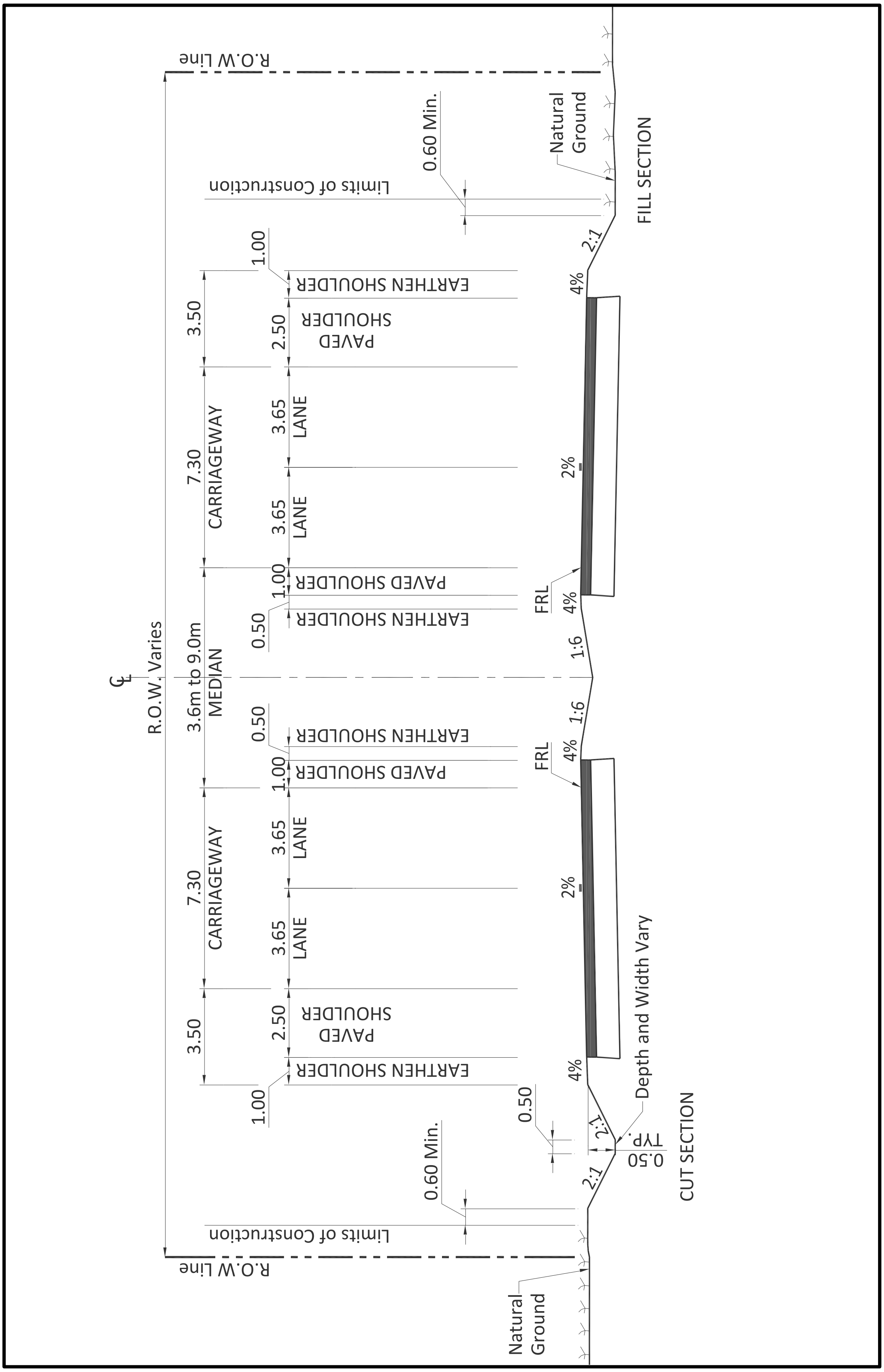


Figure 2.29: Typical Rural Major Arterial

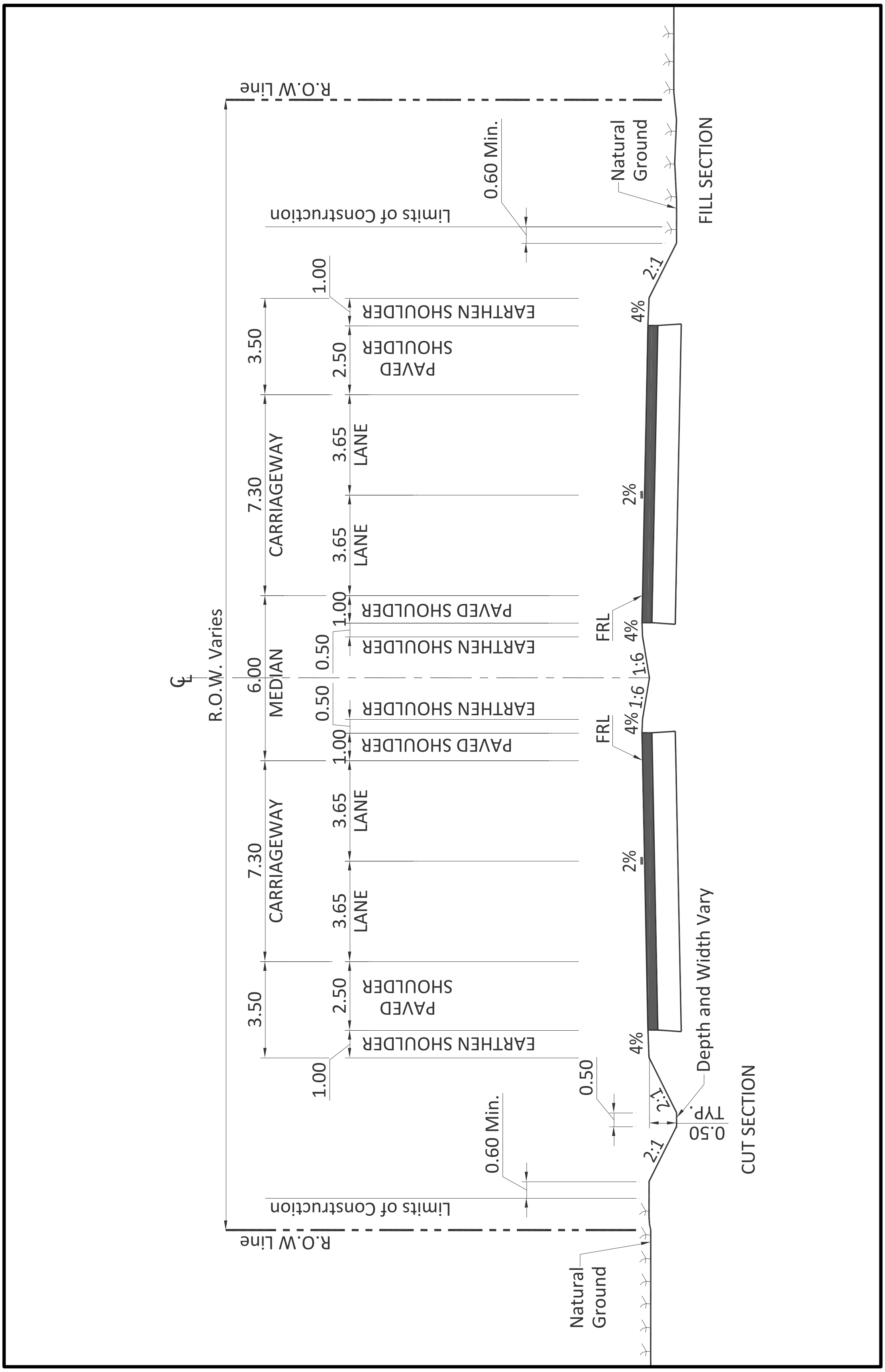


Figure 2.30: Typical Rural Minor Arterial-Divided

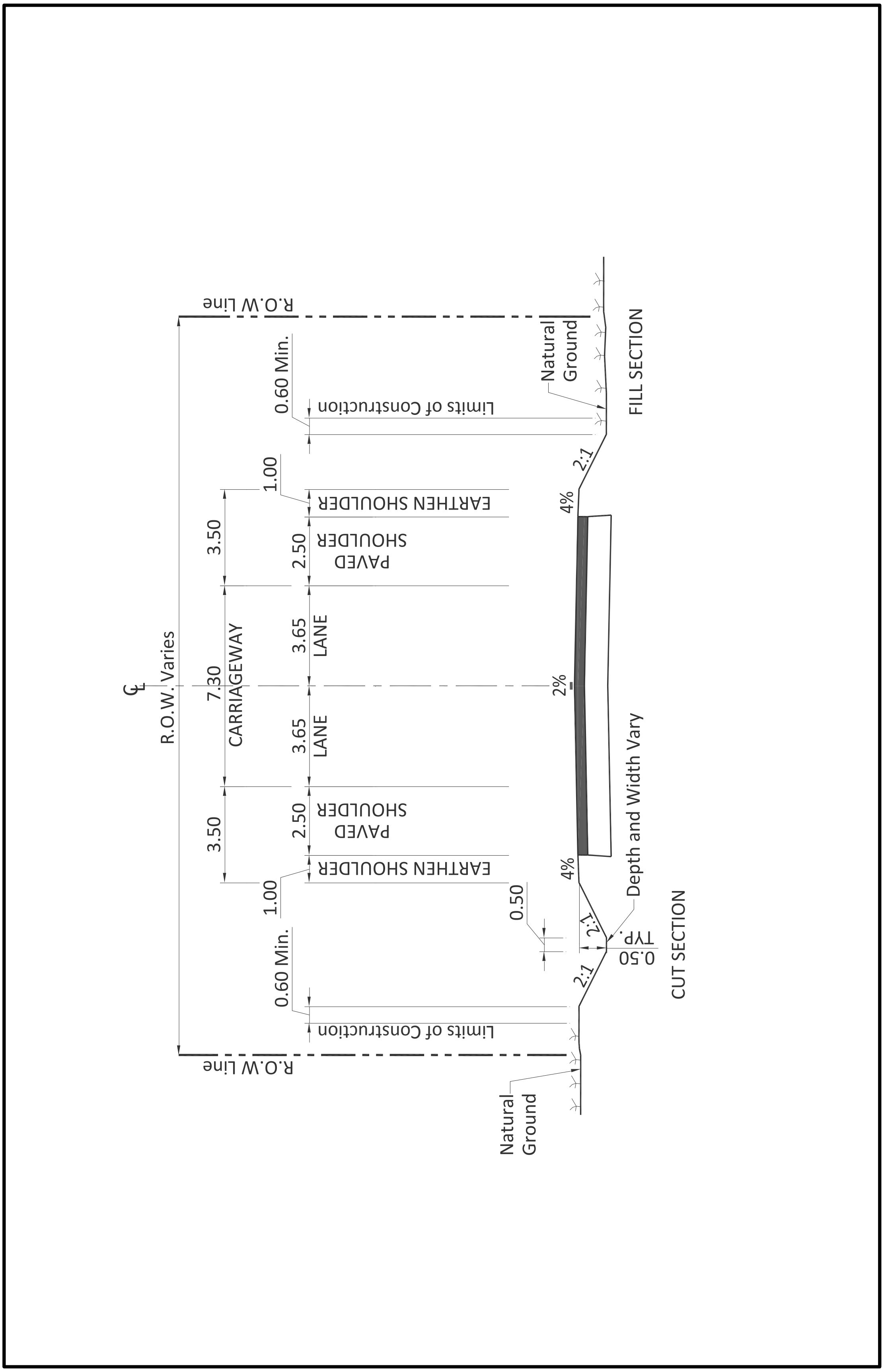


Figure 2.31: Typical Rural Minor Arterial-Undivided

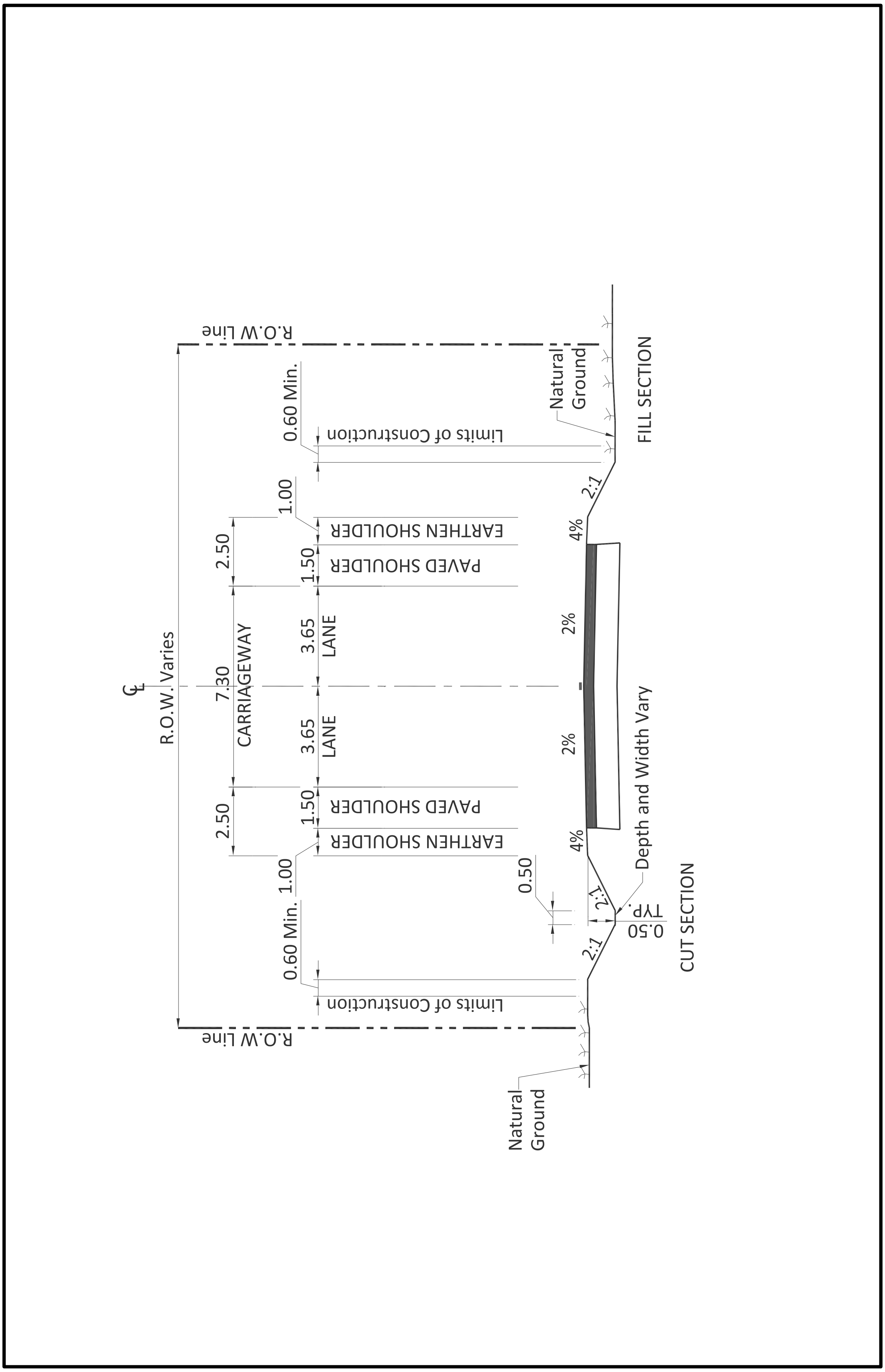


Figure 2.32: Typical Rural Collector

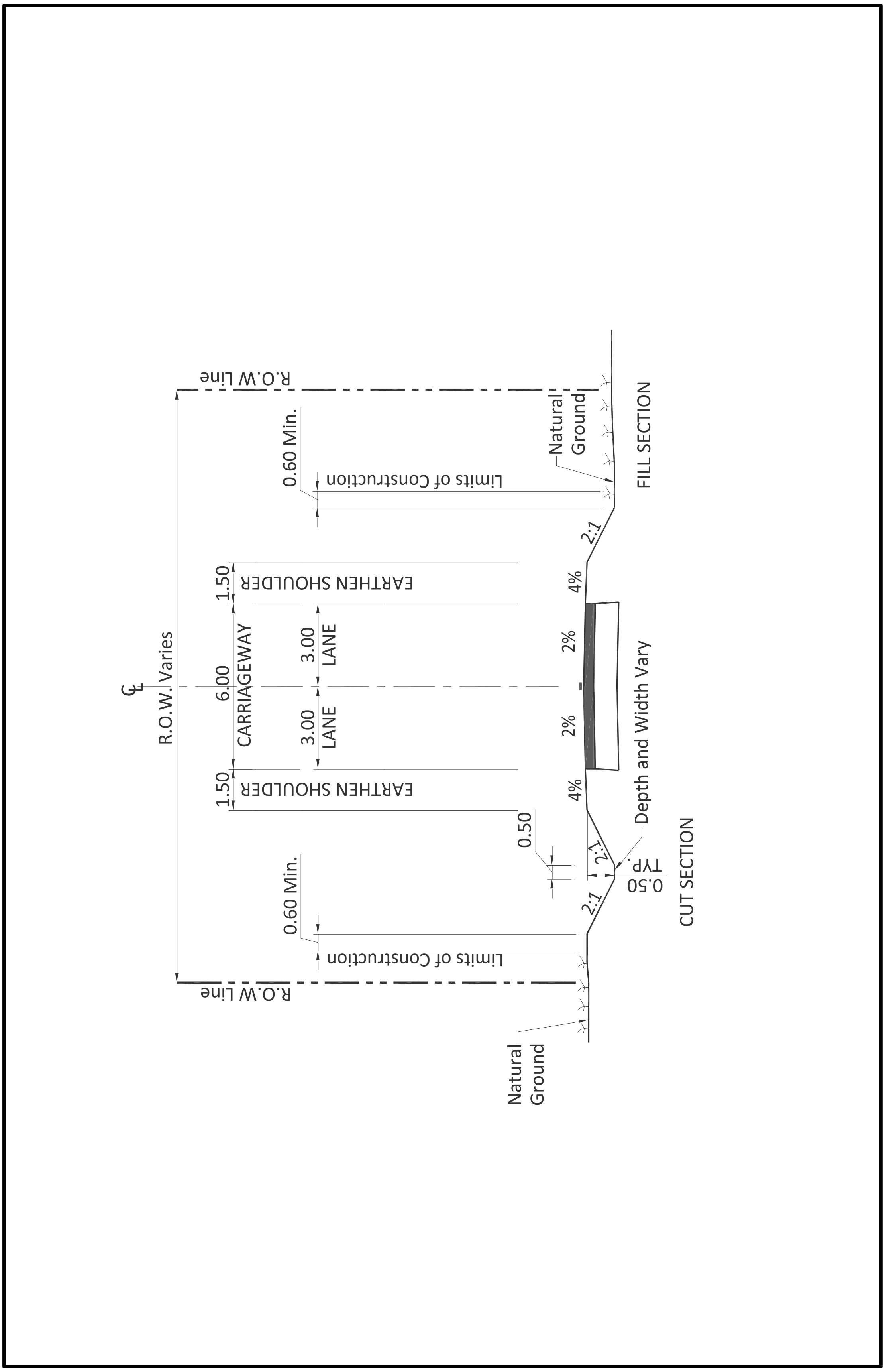


Figure 2.33: Typical Rural Local Road

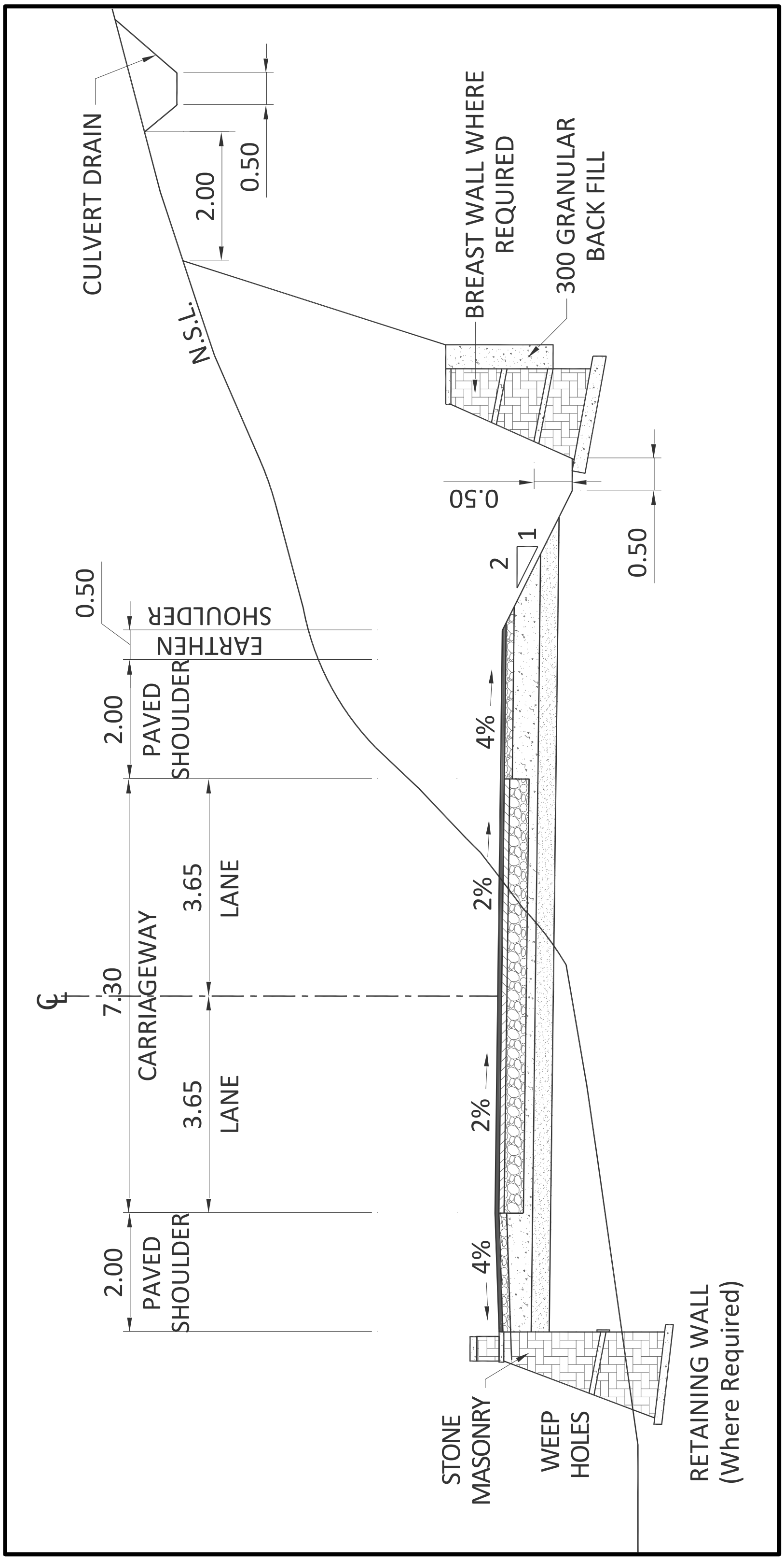


Figure 2.34: Typical Cut and Fill Section in Mountainous Terrain

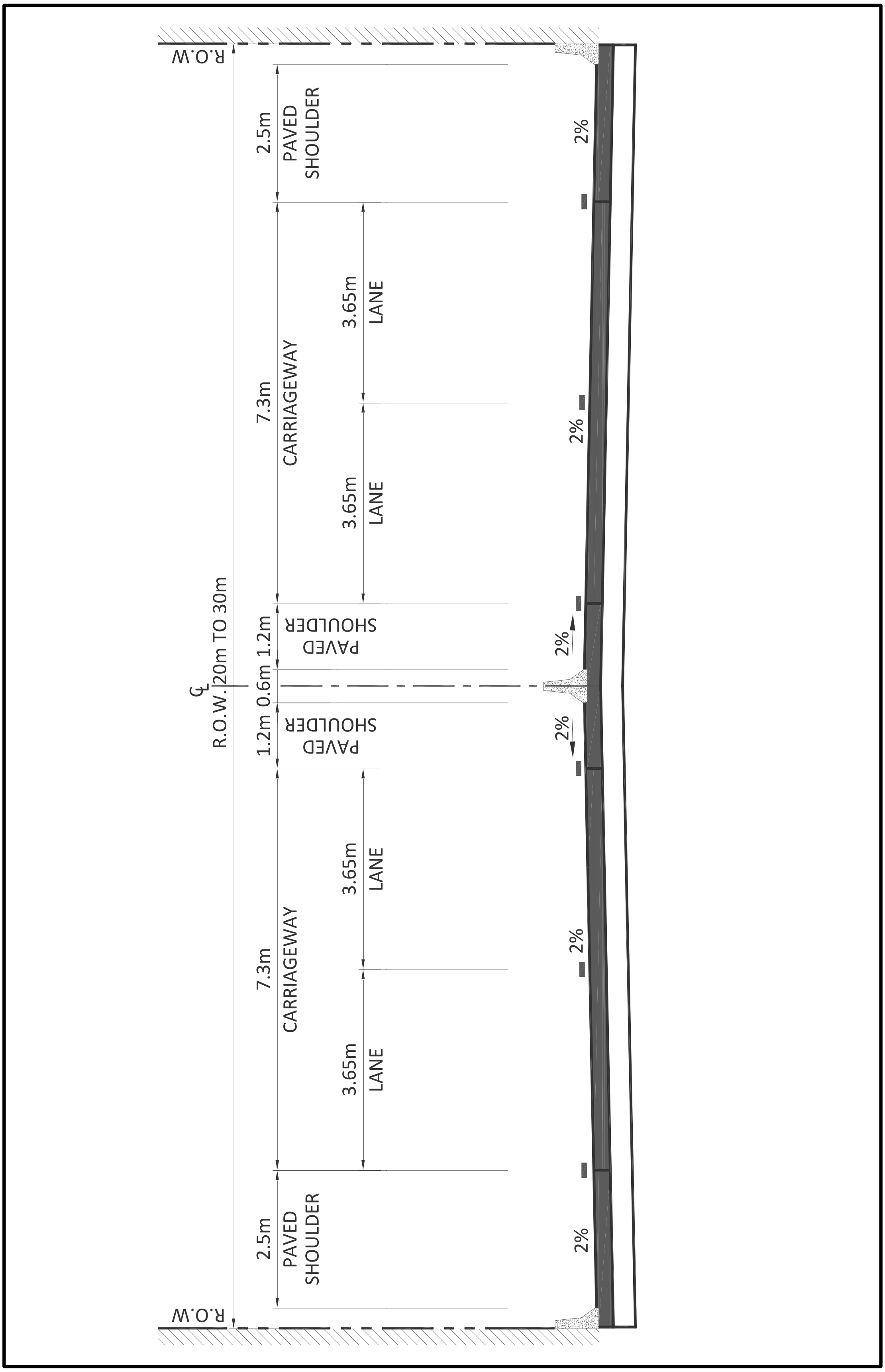


Figure 2.35: Typical Urban Expressway (2-Lanes with Barrier)

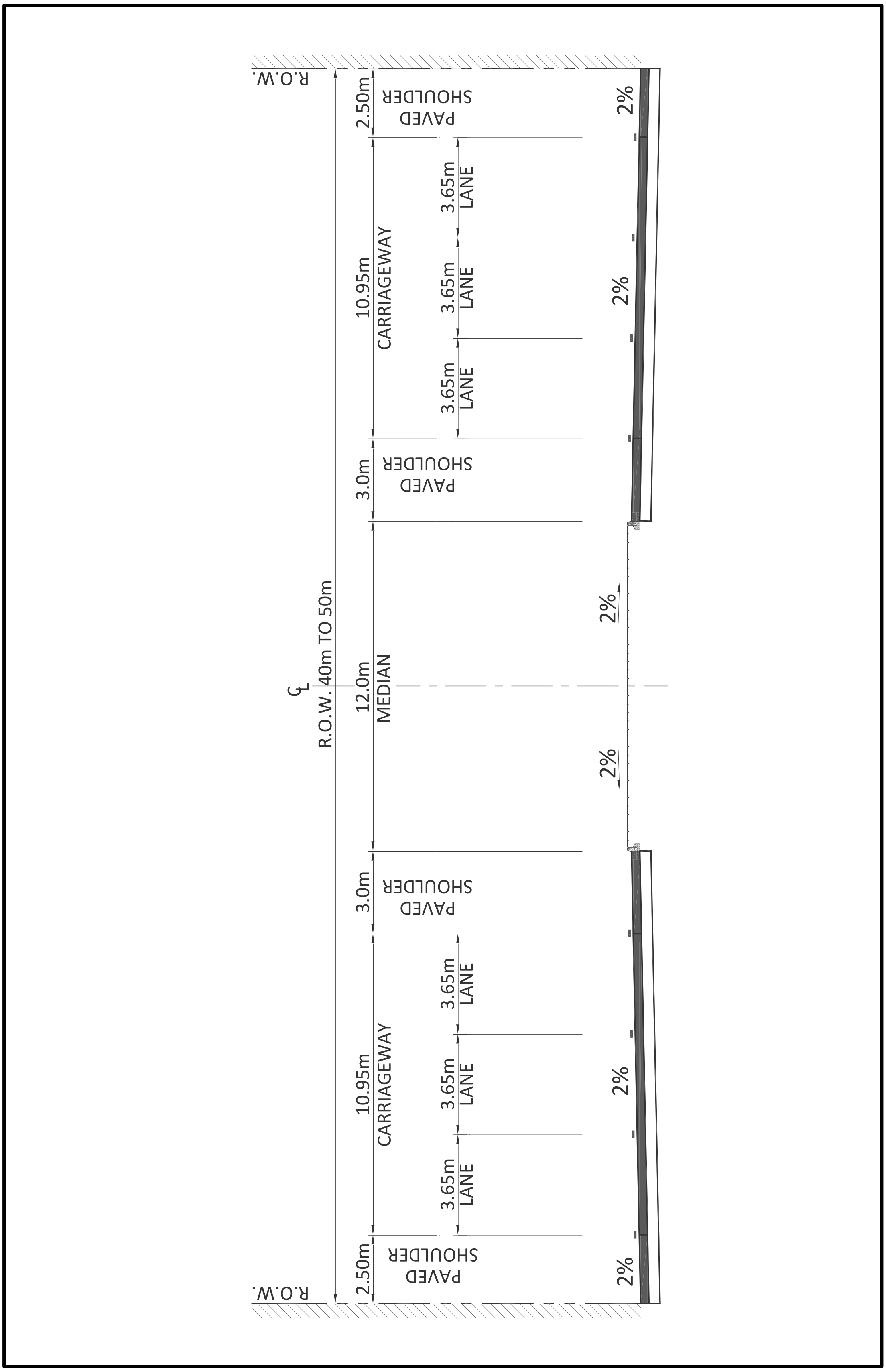


Figure 2.36: Typical Urban Expressway (3-Lanes without Barrier)

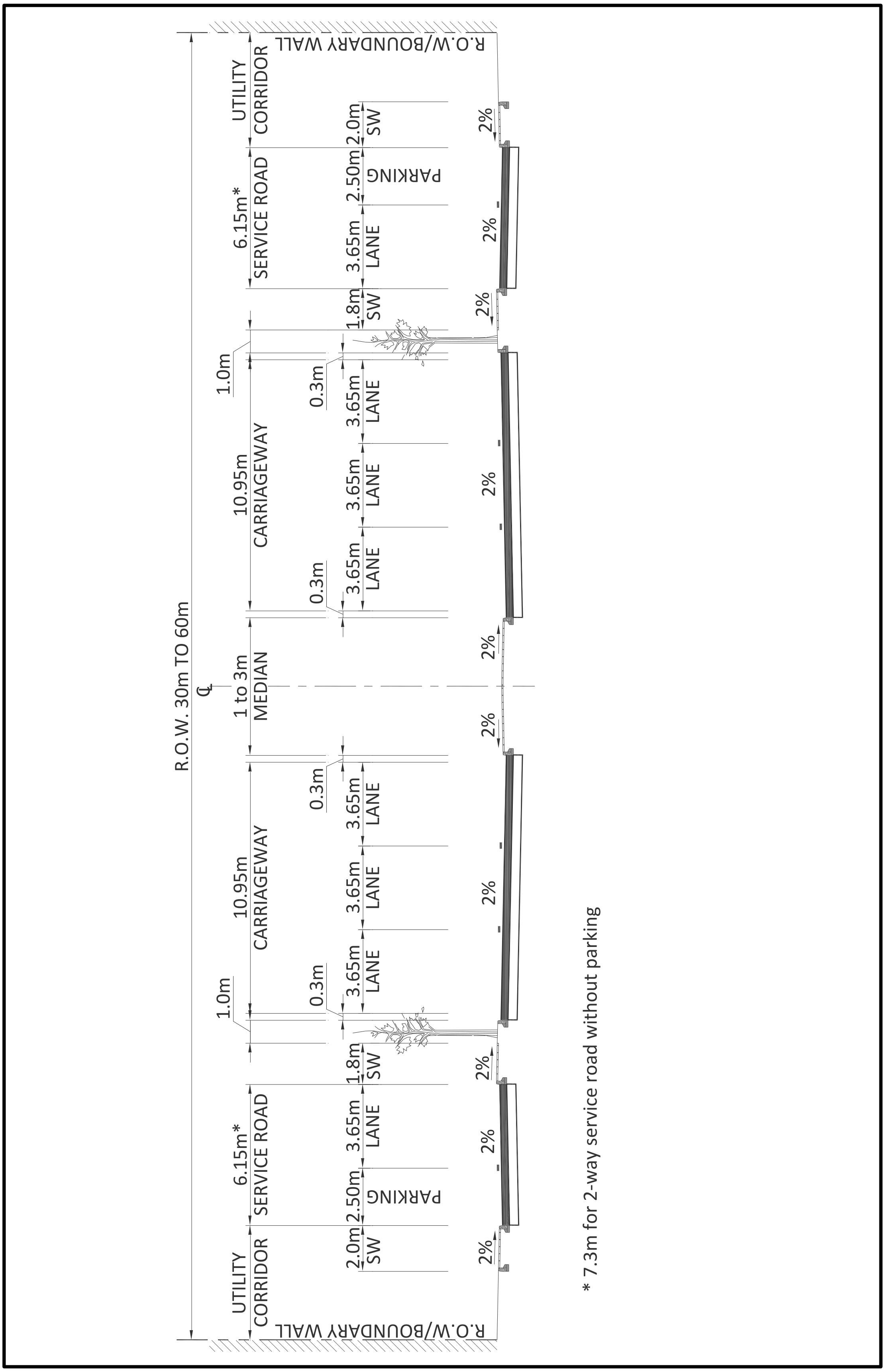


Figure 2.37: Typical Urban Major Arterial

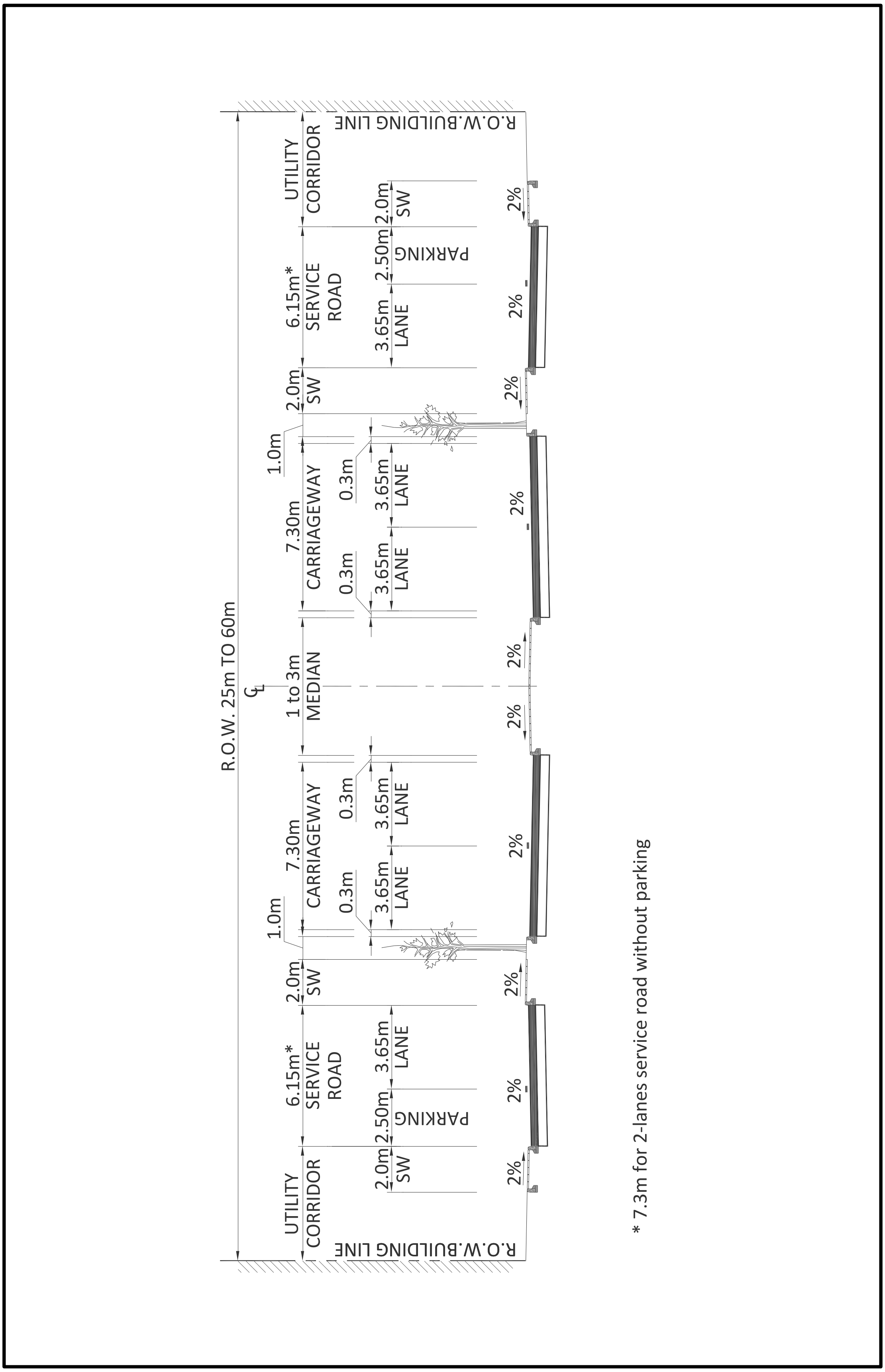
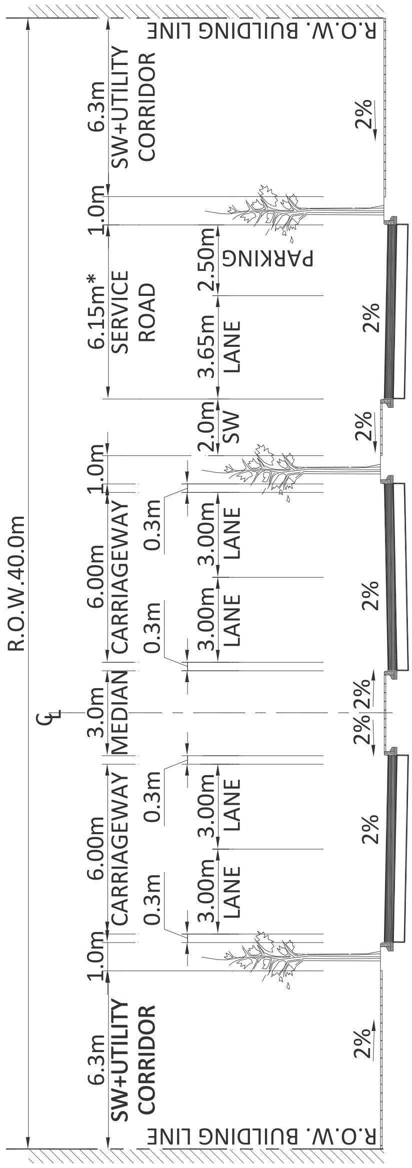


Figure 2.38: Typical Urban Minor Arterial



* 7.3m for 2-way service road without parking. Service road can be provided on either side of roadway

Figure 2.39: Typical Urban Collector (40 m ROW)

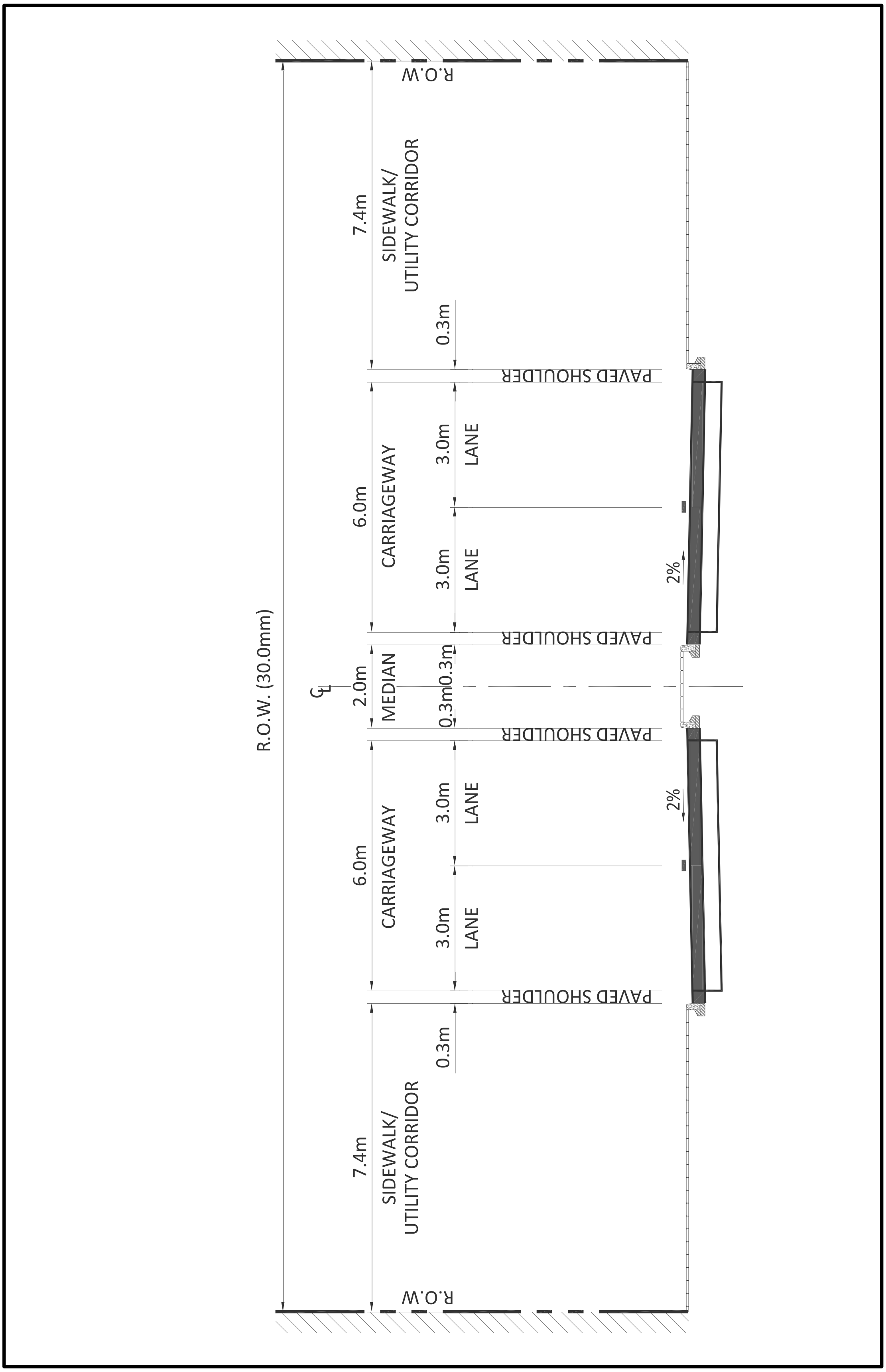
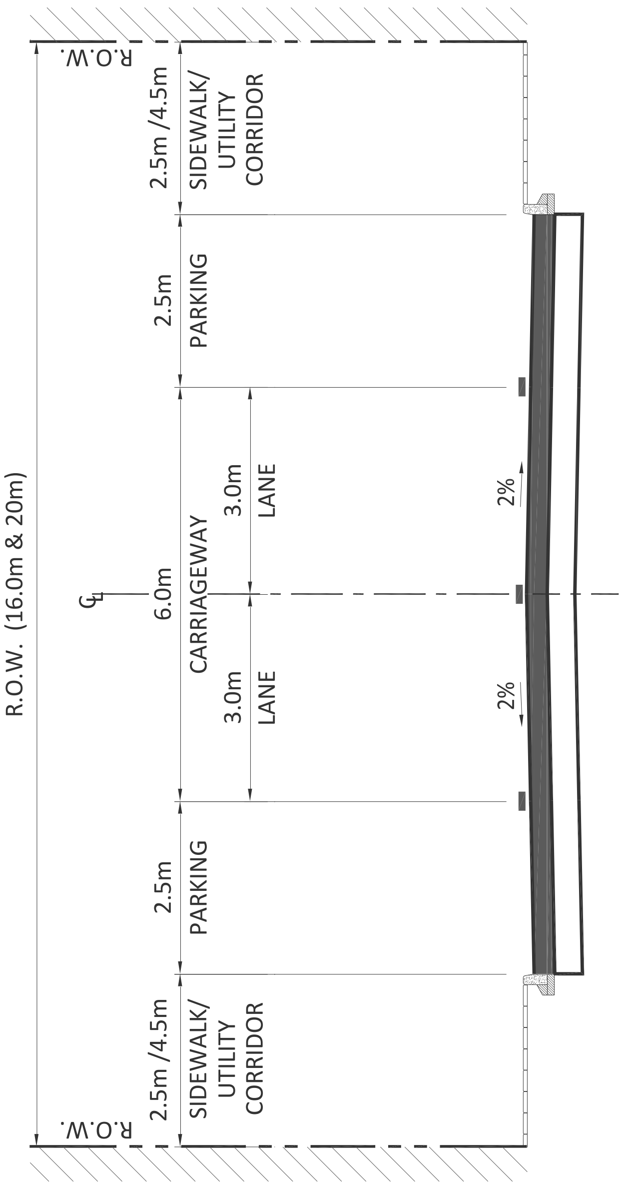


Figure 2.40: Typical Urban Collector (30 m ROW)



* For 20m ROW 1m landscaping may be provided
on both or either side of roadway

Figure 2.41: Typical Urban Local Road (16 m and 20 m ROW)

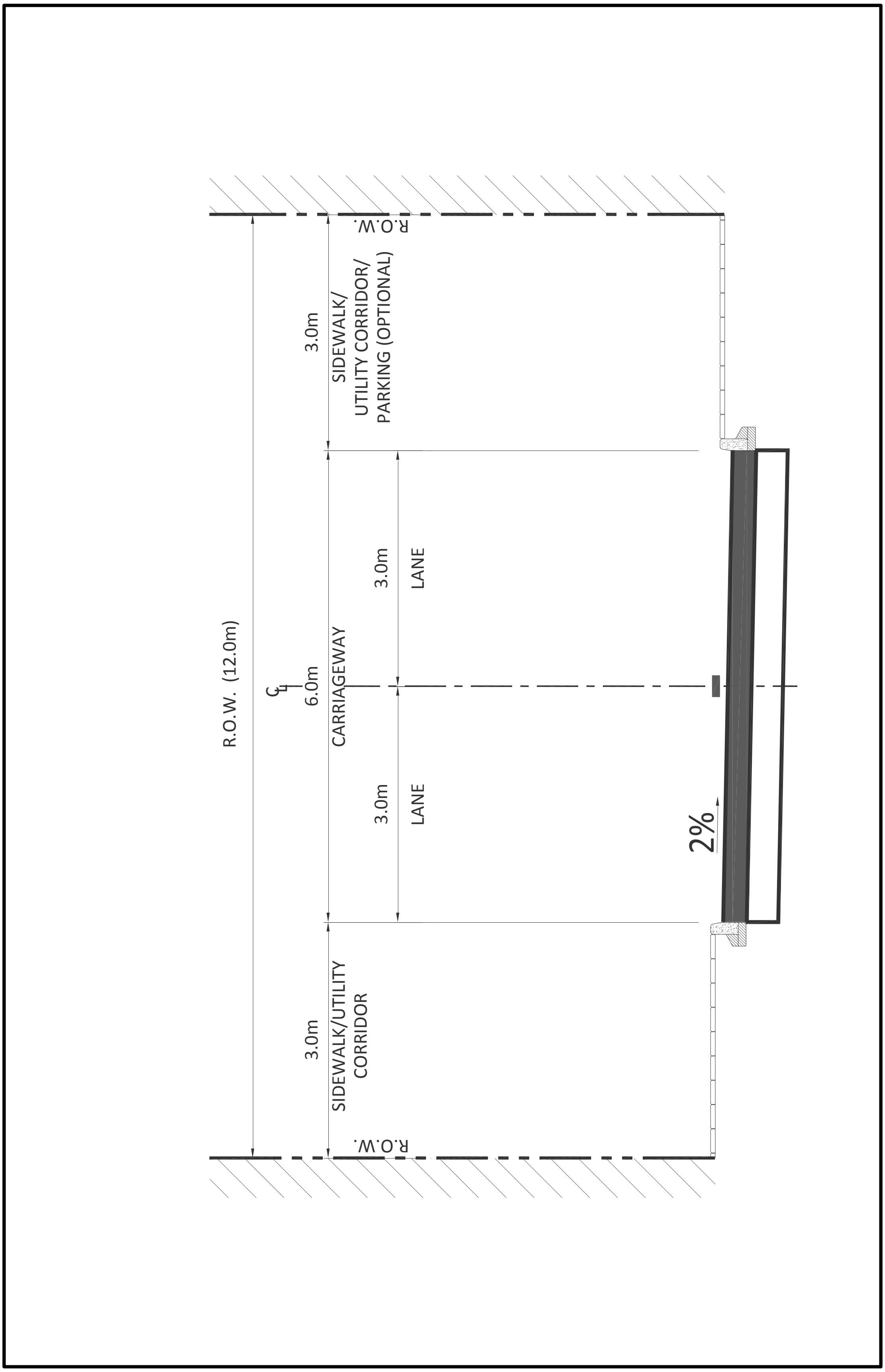


Figure 2.42: Typical Urban Local Road (12 m ROW)

3 GEOMETRIC DESIGN ELEMENTS

3.1 INTRODUCTION

Several principal elements of design are common to all classes of highways. These elements include sight distance, superelevation, carriageway widening, grades, horizontal and vertical alignments, and other elements of geometric design.

3.2 SIGHT DISTANCE

Sight distance is the continuous length of the roadway ahead that is visible to the driver. To avoid any conflict, such as striking an unexpected object on the road, drivers should be able to see far enough ahead in order to carry out safe and efficient legal manoeuvre. The designer must provide sight distances of sufficient length to ensure that drivers can control the operation of their vehicles when driving on the road.

Four types of sight distances should be considered while designing the roads and highways:

1. Stopping Sight Distance: applicable on all roadways.
2. Passing Sight Distance: applicable on two lane undivided roadways.
3. Decision Sight Distance: applicable on urban and rural roads where road users have to make complex decisions, for example exiting at interchanges.
4. Intersection Sight Distance: applicable at all intersections.

Eye height is 1.08 m for all sight distances. Object height is 0.60 m for stopping and decision sight distance and 1.08 m for intersection and passing sight distance. These heights are in reference to the passenger cars. For large trucks eye height varies from 1.8 m to 2.4 m with a recommended value of 2.3 m (Source: *AASHTO's A Policy on Geometric Design of Highways and Streets, 2011* [2]).

3.2.1 Stopping Sight Distance

Stopping sight distance is the distance required by the driver travelling at or near the design speed to stop once a stationary object becomes visible on the roadway. The roads should be designed such that the available sight distance on a roadway should be sufficiently long for a vehicle to make a stop. Although greater lengths are required.

The stopping sight distance is the sum of two components:

1. Distance traversed during break reaction time.
2. Distance to break the vehicle.

The design values for stopping sight distance at various speeds are shown in **Table 3.1**. These are calculated using the following equations from *AASHTO, 2011* [2]:

For roadways with grades less than 3%:

$$SSD = 0.278\,Vt + 0.039\,{V^2}/{a} \qquad \textit{Equation 3.1}$$

For roadway with grades greater than or equal to $\pm 3\%$:

$$SSD = 0.278\,Vt + \frac{V^2}{254\left[\left(a/_{9.81}\right) \pm G\right]} \qquad \textit{Equation 3.2}$$

Where,

SSD = stopping sight distance, (m)
V = design speed, (km/h)
t = brake perception and reaction time, (2.5 seconds) [2]
a = deceleration rate, (m/s^2) (3.4 m/s^2) [2]
G = percent grade, (+) for upgrade and (–) for downgrade [%/100]

Table 3.1: Stopping Sight Distance on Level Roadways

Design Speed (Km/h)	Stopping Sight Distance (m)						
	Level < 3%	Downgrades			Upgrades		
		3%	6%	9%	3%	6%	9%
20	20	20	20	20	19	18	18
30	35	32	35	35	31	30	29
40	50	50	50	53	45	44	43
50	65	66	70	74	61	59	58
60	85	87	92	97	80	77	75
70	105	110	116	124	100	97	93
80	130	136	144	154	123	118	114
90	160	164	174	187	148	141	136
100	185	227	243	262	203	194	186
110	220	227	243	262	203	194	186
120	250	263	281	304	234	223	214
130	285	302	323	350	267	254	243

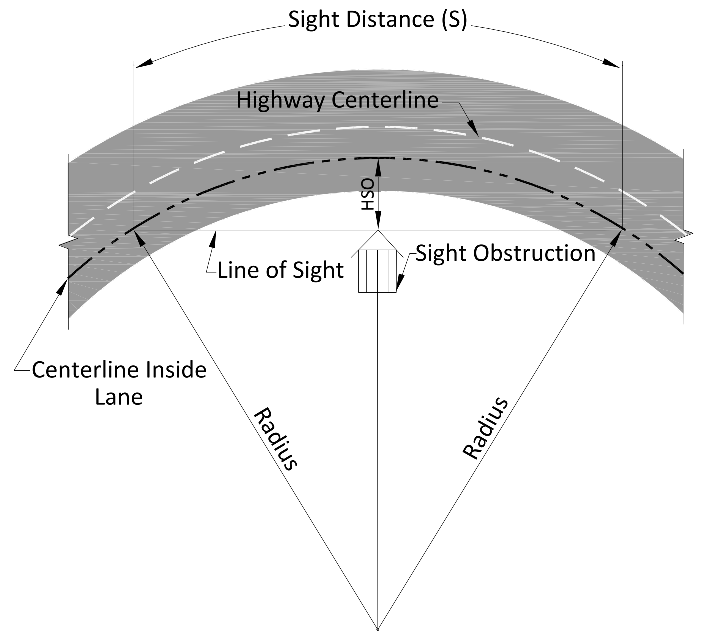
(Source: *AASHTO's A Policy on Geometric Design of Highways and Streets, 2011* [2])

The available sight distance is affected by both vertical and horizontal alignment and their combination with roadside obstructions. Thus, the available sight distance should be checked for both horizontal and vertical planes. On a tangent roadway, driver's line of sight may be limited by the vertical alignment of the roadway surface, specifically at crest vertical curves. On horizontal curves, the line of sight may be limited by obstructions outside the carriageway, such as bridge piers, retaining walls, fill slopes at bridge approaches, barriers, guardrails, buildings, back slopes in cut areas, etc. The sight distances should also be checked for both directions of travel along all roads and streets as the available sight distance on the highway is different in each direction.

The values shown in **Table 3.1** are based on passenger car operation. Separate stopping sight distances considerations for trucks are generally not required in highway design.

3.2.1.1 Horizontal Restriction to Stopping Sight Distance

Each individual horizontal curve should be checked and studied for available sight distance and sight obstructions. Where the removal of obstruction is impractical (such as walls, retaining walls, cut slopes, buildings, and longitudinal barriers), the design may needs to be adjusted to provide adequate sight distance.



(Source: *AASHTO's A Policy on Geometric Design of Highways and Streets, 2011* [2])

Figure 3.1: Components for Horizontal Stopping Sight Distance (HSS)

The horizontal stopping sight distance is measured along the centreline of the lane, as shown in **Figure 3.1**. To provide for the stopping sight distance, the sight line chord must not be obstructed by any feature outside the carriageway. The design process involved the calculation of horizontal sight line offset (HSO), which is the radial dimension from the centre of the lane to the limiting sight-obstructing feature (See **Figure 3.1**). The horizontal sight line offset to the obstruction is calculated using the following formula: (source AASHTO, 2011)

$$HSO = R\left[1 - \cos\left(\frac{28.65S}{R}\right)\right] \qquad \textit{Equation 3.3}$$

Alternatively, the SSD can be calculated for a given horizontal offset using the formula:

$$S = \frac{R}{28.65}\left[\cos^{-1}\left(\frac{R - HSO}{R}\right)\right] \qquad \textit{Equation 3.4}$$

Where,

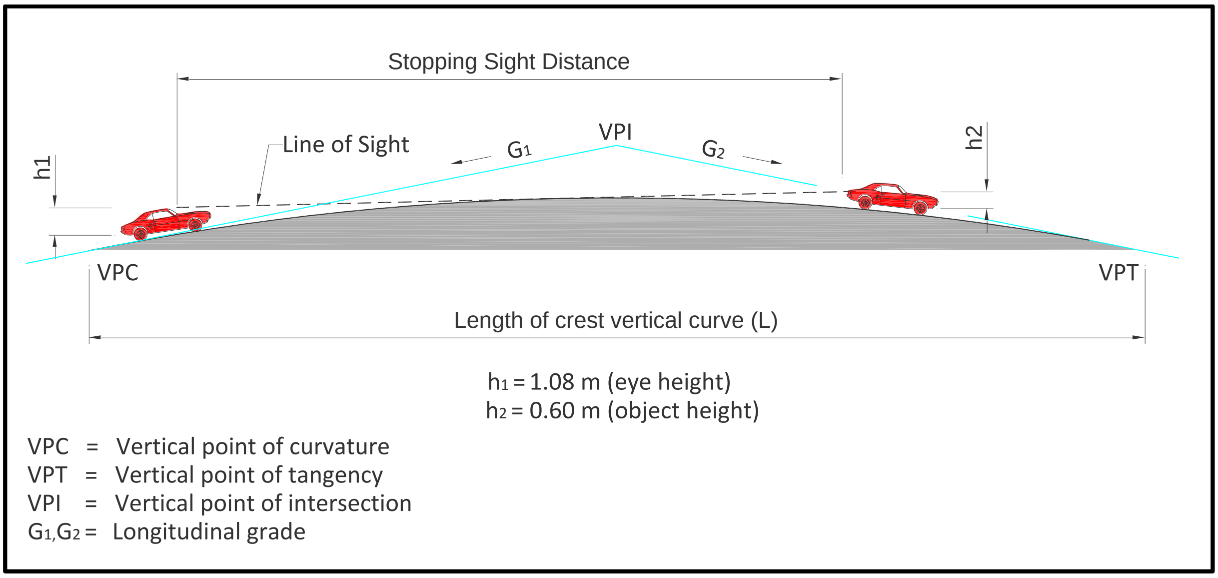
S = stopping sight distance along the curve, (m)
HSO = horizontal sight line offset measured from the centreline of inside lane, (m)
R = radius to centreline of inside lane, (m)

3.2.1.2 Vertical Restriction to Stopping Sight Distance

The most common sight restriction is a crest vertical curve as illustrated in below **Figure 3.2**. The SSD design parameters for eye height and object height noted above define the sight line that the crest curve should provide.

Vertical restriction, as illustrated in **Figure 3.3**, on sag vertical curves depends on the ability of the driver to see the roadway surface from the beams of headlights at nighttime with the following assumptions:

- Height of the head light: 0.6 m
- Height of the object: 0 m
- 1 degree upward divergence of headlight beam



(Source: *AASHTO's A Policy on Geometric Design of Highways and Streets, 2011* [2])

Figure 3.2: Stopping Sight Distance at Crest Vertical Curve

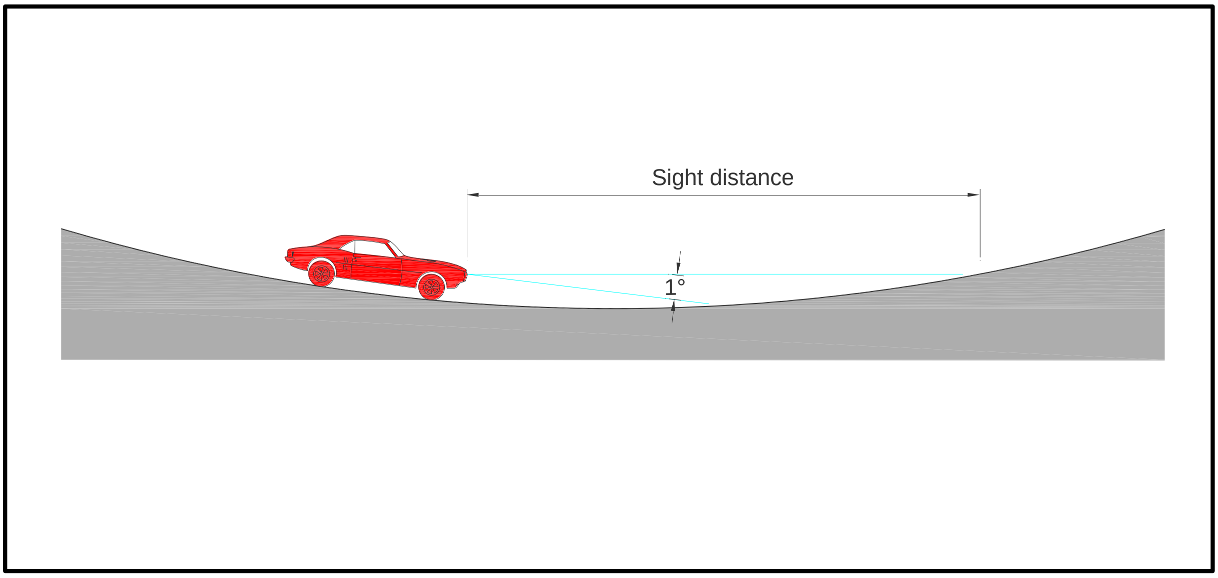
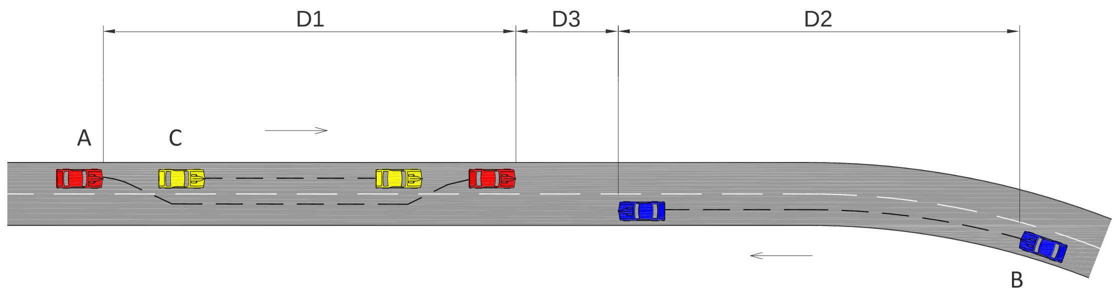


Figure 3.3: Stopping Sight Distance at Sag Vertical Curve

3.2.2 Passing Sight Distance

Passing sight distance is the distance required for a driver to observe the oncoming vehicle traveling in the opposing direction and to complete the passing manoeuvre safely without conflict with the opposing vehicle. It applies only to two-lane undivided roadways where the fast moving vehicles overtake slow moving vehicles.

Passing sight distance is measured between an eye height of 1.08 m and an object height of 1.08 m. A road with two or more traffic lanes in each direction does not require the need to check passing sight distance. **Table 3.2** lists the minimum values for passing sight distance. Where practical, consider using higher values as the basis of design.



A,B,C = Vehicles
D1 = Distance to complete the passing maneuver by vehicle A
D2 = Distance traveled by vehicle B
D3 = Distance between vehicle A and B
PSD is sum of the distances D1, D2 and D3.

(Source: *AASHTO's A Policy on Geometric Design of Highways and Streets, 2011* [2])

Figure 3.4: Passing Manoeuvre

PSD should be checked in both horizontal and vertical plane. Procedures provided in Section 3.2.1 of this manual can be used to check the available sight distance by substituting PSD for SSD and using object height of 1.08 m.

Passing Sight distance is considerably greater than stopping sight distance and it will be uneconomical to design complete road for passing. Therefore, on two lane roads, it is advisable to design sections of roads specifically for passing.

Table 3.2: Passing Sight Distance for Design of Two-Lane Highways

Design Speed (km/h)	Passing Sight Distance (m)
30	120
40	140
50	160
60	180
70	210
80	245
90	280
100	320
110	355
120	395
130	440

(Source: *AASHTO's A Policy on Geometric Design of Highways and Streets, 2011* [2])

3.2.3 Decision Sight Distance

Whenever driver needs to take complex and instantaneous decisions, greater distances may be needed for driver to perceive the information and perform necessary manoeuvres. The decision sight distance is the distance traversed by the driver in perceiving the necessary information, plotting a course, and making the necessary manoeuvres. DSD should be checked on all critical locations along a roadway. The values of decision sight distance are substantially greater than stopping sight distance.

Some examples of the critical conditions where DSD should be provided by the designer in advance of the condition are as follows:

- Exit and entrance ramps at the interchanges
- High-speed roadway diverge and merge areas
- Change in cross section of the roadway, as in lane drops or Toll plaza
- At-grade railroad crossings
- Signalized intersection on the downstream end of a crest vertical curve

Table 3.3 provides DSD values. Where practical, consider using higher values as the basis of design.

Table 3.3: Decision Sight Distance

Design Speed (km/h)	Decision Sight Distance (m)				
	Avoidance Manoeuvre				
	A	B	C	D	E
50	70	155	145	170	195
60	95	195	170	205	235
70	115	325	200	235	275
80	140	280	230	270	315
90	170	325	270	315	360
100	200	370	315	355	400
110	235	420	330	380	430
120	265	470	360	415	470
130	305	525	390	450	510

Notes:
Avoidance Manoeuvre A: Stop on rural road—t = 3.0 s
Avoidance Manoeuvre B: Stop on urban road—t = 9.1 s
Avoidance Manoeuvre C: Speed/path/direction change on rural road—“t” varies between 10.2 and 11.2 s
Avoidance Manoeuvre D: Speed/path/direction change on suburban road—“t” varies between 12.1 and 12.9 s
Avoidance Manoeuvre E: Speed/path/direction change on urban road—“t” varies between 14.0 and 14.5 s
(Source: *AASHTO’s A Policy on Geometric Design of Highways and Streets, 2011* [2])

The DSD values in **Table 3.3** are determined using the following equations. For avoidance manoeuvres “A” and “B”, the equation is:

$$DSD = 0.278Vt + 0.039\frac{V^2}{a}$$ ***Equation 3.5***

For avoidance manoeuvres C, D, and E the equation is:

$$DSD = 0.278Vt$$ ***Equation 3.6***

Where,

DSD = decision stopping sight distance, (m)

V = design speed, (km/h)

a = deceleration rate, (m/s^2) (3.4 m/s^2)

t = pre-manoeuvre time, seconds and varies with the avoidance manoeuvre (see notes in **Table 3.3**)

If provision of decision sight distance is not practical, then consideration should be given to move the critical decision point to a location where sufficient sight distance equal to or greater than decision sight distance is available.

3.2.4 Intersection Sight Distance

Intersection sight distance (ISD) is a critical design element of intersection design. ISD is the distance required for a driver approaching an intersection to see the traffic on the intersecting roadway in order to safely cross or make a left or right turn on to the intersecting roadway.

Sight lines for ISD involve varying driver positions along one road and another vehicle on the crossing road. These sight lines form a triangular wedge in each quadrant between the intersection roadways. This triangular wedge is called sight triangles. The sight triangles should be clear of any obstruction that may hinder driver's view of oncoming vehicles on the intersecting roadway.

The sight line defining ISD in both horizontal and vertical plane is based on an eye height and object height of 1.08 m.

The dimensions of the sight triangles depend on the design speed of the major roadway, type of intersection control (uncontrolled, yield control, stop control or signal control). This manual adopts the *AASHTO, 2011* [2] procedures to determine ISD for the following types of traffic control:

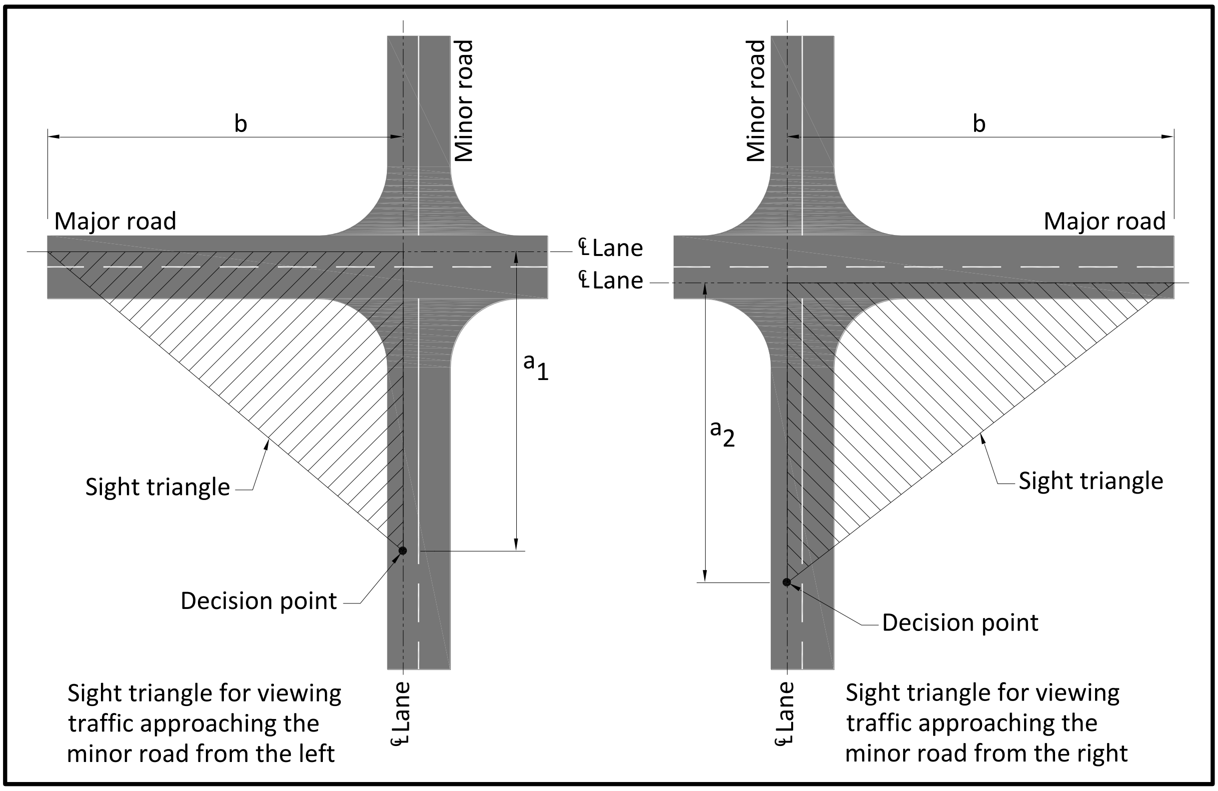
- **Case A** : Intersections with no control
- **Case B** : Intersections with stop control on the minor road
  - **B1** : Right turn from the minor road
  - **B2** : Left turn from the minor road
  - **B3** : Crossing manoeuvre from the minor road
- **Case C** : Intersection with yield control on the minor road
  - **C1** : Crossing manoeuvre from the minor road
  - **C2** : Right or left turn from the minor road
- **Case D** : Intersections with traffic signal control
- **Case E** : Intersection with all-way stop
- **Case F** : Right turns from the major road

3.2.4.1 Case A: Intersections with No Control

Case "A" shall be applied to roadways that are not controlled by yield signs, stop signs or traffic signals on low volume and low speed intersecting. It shall only be used only if authorized by the Overseeing Organization.

Figure 3.5 illustrates the sight triangles on the major road and the minor road. Distances "a_1" and "a_2" are from the major road to the decision point i.e. location of driver's eye, along the minor road. Distance "b" is the required sight distance along the major road. **Table 3.4** shows values for "a_2" and "b" along the minor and major approaches. Distance "a_1" is equal to "a_2" plus the additional width as required. The values shown in **Table 3.4** are minimum values, higher values should be considered as the basis of design, wherever practical. **Table 3.5** provides the factors for the approach grade adjustments.

The area within the sight triangle should be clear of obstructions. This enables each driver approaching the intersection to see the each other, adjust speeds accordingly, and negotiate the intended manoeuvre without a conflict.



(Source: *AASHTO's A Policy on Geometric Design of Highways and Streets, 2011* [2])

Figure 3.5: Sight Triangles (Uncontrolled and Yield Controlled)

Table 3.4: Length of the Sight Triangle Legs — Case A - Intersections with No Control

Design Speed (km/h)	Length of Legs "a_2" and "b" (m)
20	20
30	25
40	35
50	45
60	55
70	65
80	75

Note: For approach grades greater than 3%, multiply the sight distance values in this table by the appropriate adjustment factor from **Table 3.5.**

(Source: *AASHTO's A Policy on Geometric Design of Highways and Streets, 2011* [2])

Table 3.5: Adjustment Factors for Intersection Sight Distance Based on Approach Grade

Approach Grade (%)	Design Speed (km/h)											
	20	30	40	50	60	70	80	90	100	110	120	130
6	1.1	1.1	1.1	1.1	1.1	1.1	1.2	1.2	1.2	1.2	1.2	1.2
5	1.0	1.0	1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.2	1.2	1.2
4	1.0	1.0	1.0	1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.1
-3 to +3	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0
+4	1.0	1.0	1.0	1.0	0.9	0.9	0.9	0.9	0.9	0.9	0.9	0.9
+5	1.0	1.0	1.0	0.9	0.9	0.9	0.9	0.9	0.9	0.9	0.9	0.9
+6	1.0	1.0	0.9	0.9	0.9	0.9	0.9	0.9	0.9	0.9	0.9	0.9

(Source: *AASHTO's A Policy on Geometric Design of Highways and Streets, 2011* [2])

3.2.4.2 Case B: Intersections with Stop Control on Minor Road

There are three scenarios for which sight triangles should be checked for intersections with stop control.

- **Case B1** : Right turn from minor road
- **Case B2** : Left turn from minor road
- **Case B3** : Crossing the major road from minor road

Figure 3.6 shows the sight triangles at stop controlled intersections. The decision point should be at least 4.4 m, preferably 5.4 m from the edge of the carriageway. Distances "a_1" and "a_2" are measured from the centre of the lanes of major road to the decision point along the minor road. Minimum length of "a_2" shall be equal to 5.4 m plus the width of pavement from the edge of the carriageway to the centreline of the lane. Distance "b" is the required sight distance along the major road. Distance "a_1" is equal to "a_2" plus the additional width as required.

ISD required on the major road is calculated using the following equation (source: *AASHTO, 2011):*

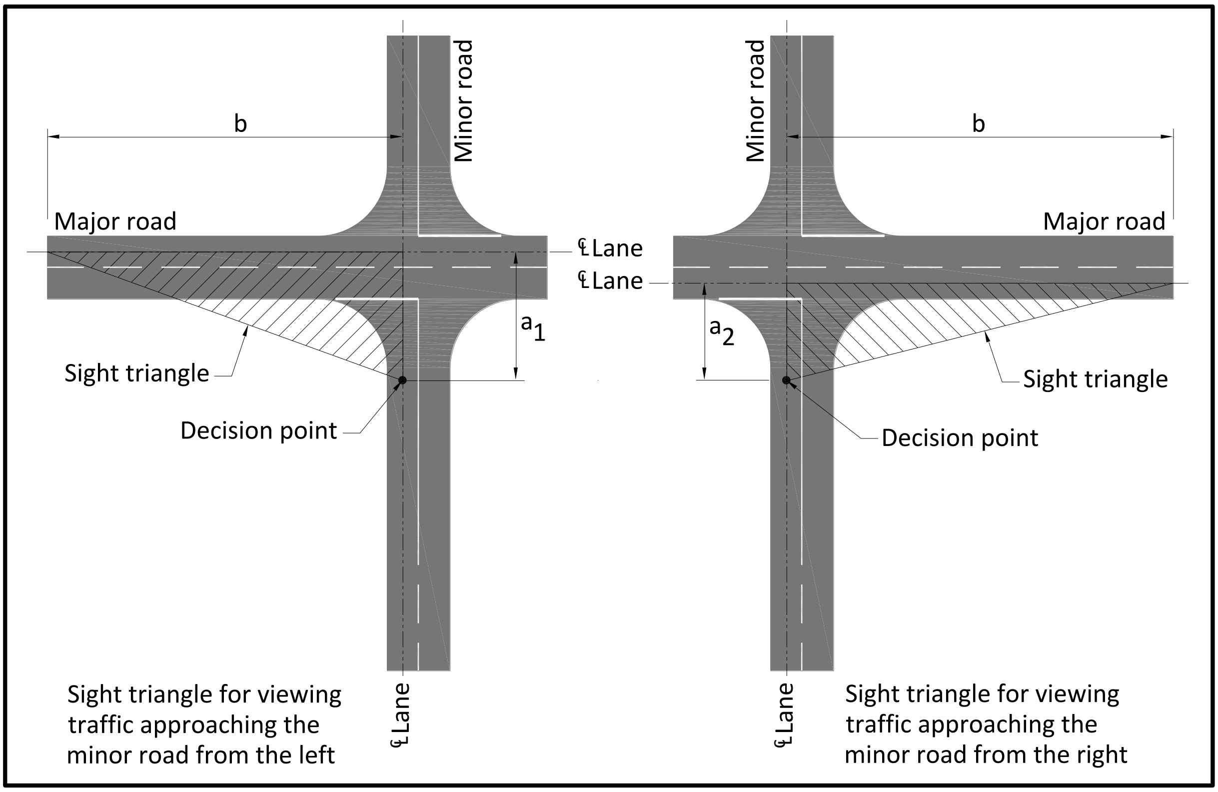
$$\boldsymbol{b = ISD = 0.278 V_{major} t_g} \qquad \textit{Equation 3.7}$$

Where,

ISD = intersection sight distance

V_{major} = design speed on the major road, km/h

t_g = time gap for minor road vehicle to enter the major road, seconds



(Source: *AASHTO's A Policy on Geometric Design of Highways and Streets, 2011* [2])

Figure 3.6: Sight Triangles (Stop Controlled)

Time gap values for all three cases are provided in **Table 3.6** and **Table 3.7** lists ISD values. These values are for stopped passenger cars to turn either left, right or cross a two-lane highway with no median and grades 3 % or less. Sight distance should be recalculated by using ISD equation provided above, inserting adjusted values of time gap for other conditions. The guidelines for these adjustments are provided in notes of **Table 3.6**. The values in **Table 3.7** are minimum values; consider using higher values as the basis of design, where practical.

Table 3.6: Time Gap — Case B1 - Right Turn from Stop, Case B2 - Left Turn from Stop and Case B3 - Crossing Manoeuvre

Design Vehicle	Time Gap, t_g, at Design Speed of Major Road (seconds)	
	Case B1	Case B2 and B3
Passenger car	7.5	6.5
Single unit truck	9.5	8.5
Intermediate Semitrailer	11.5	10.5

Note:
Time gaps are for stopped vehicle to turn left, right onto or to cross a two-lane highway with no median and with grades of 3 percent or less. The table values are adjusted as follows:
For multilane highways—for right turns and for crossing a major road with more than two lanes, add 0.5 second for passenger cars or 0.7 second for trucks for each additional lane, to be crossed and for narrow medians that cannot store the design vehicle.
For minor road approach grades—if the approach grade is an upgrade that exceeds 3 percent, add 0.2 second for right turns and 0.1 seconds for left and crossing for each percent grade.
(Source: AASHTO's A Policy on Geometric Design of Highways and Streets, 2011 [2])

Table 3.7: Intersection Sight Distance — Case B1 - Right Turn from Stop, Case B2 - Left Turn from Stop and Case B3 - Crossing Manoeuvre

Design Speed (km/h)	Intersection Sight Distance for Passenger Cars (m)	
	Case B1	Case B2 and B3
20	45	40
30	65	55
40	85	75
50	105	95
60	130	110
70	150	130
80	170	145
90	190	165
100	210	185
110	230	200
120	255	220
130	275	235

Intersection sight distance shown is for a stopped passenger car to turn right, left onto or to cross a two-lane highway with no median and grades 3 percent or less. For other conditions, the time gap should be adjusted and the sight distance recalculated.
(Source: *AASHTO's A Policy on Geometric Design of Highways and Streets, 2011* [2])

3.2.4.3 Case C: Intersections with Yield Control on Minor Road

Drivers approaching yield signs are permitted to enter or cross the major road without stopping, if there are no potentially conflicting vehicles on the major road. The sight distances needed by drivers on yield-controlled approaches exceed those for stop-controlled approaches. For intersections with yield control on minor roads, sight triangles should be checked for two scenarios:

- **C1**: Crossing manoeuvre from the minor road
- **C2**: Right or left turn from the minor road

ISD provided for the vehicle on the major road should allow the minor road vehicle to travel from the decision point to the intersection, cross or turn left or right, and clear the intersection safely. ISD on the major road is calculated using the following equation: (Source: *AASHTO, 2011* [2])

$$ISD = b = 0.278V_{major}t_g \qquad \textbf{\textit{Equation 3.8}}$$

Where,

B = ISD, length of leg of sight triangle along major road, (m)

V_{major} = design speed on the major road, (km/h)

tg = travel time for vehicle on the minor road to reach and turn left, right or cross the major road;

For Case C1 (Crossing manoeuvre from minor road), the length of the minor road approach leg, the time to travel from the decision point to the intersection, and the time gap, "t_g" are shown in **Table 3.8**. **Table 3.9** lists the length of the sight triangle values along the major road for different design speeds. The values of "t_g" for Case C1, as provided in **Table 3.8** are calculated using the following equation: (Source: *AASHTO, 2011* [2])

$$t_g = t_a + \left(\frac{w + L_a}{0.167V_{minor}}\right)$$ ***Equation 3.9***

Where,

t_a = travel time for vehicle on minor road to reach the major road from the decision point without stopping, (sec)

w = width of the intersection to be crossed, (m)

L_a = length of design vehicle, (m)

Passenger car (P) = 5.79 m

Single unit truck (SU-9) = 9.14 m

City bus = 12.19 m

Intermediate Semitrailer (WB-12) = 13.87 m

V_{minor} = design speed on the minor road, (km/h)

Table 3.8: Length of Minor Leg and Travel Time from the Decision Point — Case C1 - Crossing Manoeuvre from Yield Controlled Approaches

Design Speed (km/h)	Minor Road Approach		Travel Time, t_g [c] (seconds)
	Length of Leg [a] (m)	Travel Time, t_a [a, b] (seconds)	
20	20	3.2	7.1
30	30	3.6	6.5
40	40	4.0	6.5
50	55	4.4	6.5
60	65	4.8	6.5
70	80	5.1	6.5
80	100	5.5	6.5
90	115	5.9	6.8
100	135	6.3	7.1
110	155	6.7	7.4
120	180	7.0	7.7
130	205	7.4	8.0

Notes:

[a] For minor road approach grades that exceed 3 percent, multiply the distance to the time in this table by the appropriate factor from **Table 3.5**.

[b] Travel time applies to a vehicle that slows before crossing the intersection but does not stop.

[c] Values shown are for a passenger car crossing a two-lane highway with no median and with grades ≤ 3%.

(Source: *AASHTO's A Policy on Geometric Design of Highways and Streets, 2011* [2])

Table 3.9: Length of Sight Triangles along Major Road — Case C1 - Crossing Manoeuvre from Yield Controlled Intersections

Major Road Design Speed (km/h)	SSD (m)	Design Values Minor Road Design Speed (m)						
		20	30–80	90	100	110	120	130
20	20	40	40	40	40	45	45	45
30	35	60	55	60	60	65	65	70
40	50	80	75	80	80	85	90	90
50	65	100	95	95	100	105	110	115
60	85	120	110	115	120	125	130	135
70	105	140	130	135	140	145	150	160
80	130	160	145	155	160	165	175	180
90	160	180	165	175	180	190	195	205
100	185	200	185	190	200	210	215	225
110	220	220	200	210	220	230	240	245
120	250	240	220	230	240	250	260	270
130	285	260	235	250	260	270	280	290

Note:
Values in the table are for passenger cars and for grades 3 percent or less.
(Source: *AASHTO's A Policy on Geometric Design of Highways and Streets, 2011* [2])

For Case C2 the length of the sight triangle leg on the minor approach should be taken as 25 m for both left and right turns. The distance is established on the assumption that the right and left turns will slow down to 16 km/h without stopping to make the appropriate turn. Critical time gap values for Case C2 are listed in the **Table 3.10.** Length of the sight triangle values along the major road for different design speeds are shown in **Table 3.11.**

Table 3.10: Time Gap — Case C2- Right and Left Turn Manoeuvres from Yield-Controlled Intersections

Design Vehicle	Time Gap, t_g (seconds)
Passenger car	8.0
Single unit truck	10.0
Combination truck	12.0

Notes:
Time values shown are for a vehicle to turn left or right on to a two-lane highway with no median. The values should be adjusted for multilane highways as follows:
For left turns, no adjustment is required.
For right turns, add 0.5 seconds for Passenger car or 0.7 seconds for trucks for each additional lane.
(Source: *AASHTO's A Policy on Geometric Design of Highways and Streets, 2011* [2])

Table 3.11: Intersection Sight Distance along Major Road—Case C2- Right or Left Turn at Yield-Controlled Intersections

Design Speed (km/h)	SSD (m)	Length of Leg, b (m)
20	20	45
30	35	70
40	50	90
50	65	115
60	85	135
70	105	160
80	130	180
90	160	205
100	185	225
110	220	245
120	250	270
130	285	290

(Source: *AASHTO's A Policy on Geometric Design of Highways and Streets, 2011* [2])

3.2.4.4 Case D: Intersections with Traffic Signal Control

Traffic signal controlled intersections do not need the analysis for sight triangles. The first vehicle stopped at one approach should be able to see the other vehicles stopped on the other approaches.

If the signalized intersection is to operate with flashing yellow major road and flashing red on minor road at off-peak or at night-time, then intersection sight triangles for Case C are to be provided, both to the left and right on minor approach road. In addition, if left turns on a red signal are permitted, then appropriate sight distance should be provided for right turn according to Case B2.

3.2.4.5 Case E: Intersections with All-Way Stop Control

ISD at the intersections with all-way stop control is similar to the Intersections with traffic signal control. The first vehicle stopped at one approach should be able to see the other vehicles stopped on the other approaches.

3.2.4.6 Case F: Right Turns from Major Road

Regardless of the type of traffic control, adequate ISD should be provided for the right turning vehicles from the major road. The ISD provided should be based on a stopped vehicle selecting the appropriate gap in the opposing traffic and completing the manoeuvre, as demonstrated in **Figure 3.7**. **Table 3.12** shows the time gap for the right turns from the major road and **Table 3.13** shows the ISD values required for the vehicles to turn right from major road. The values shown in the **Table 3.13** are for passenger cars turning right from two-lane undivided major road. Adjust the time and the ISD accordingly, if an above average percentage of longer vehicles are anticipated.

If major road has been provided continuously for stopping sight distance and if Case B or Case C has been provided on minor approach road, the sight distance on major road for right turn will generally be sufficient and there will be no need to check the sight distance for Case F.

However, at three-leg intersections or driveways located on or near a horizontal curve or crest vertical curve on the major road, the availability of adequate sight distance for right turns from the major road should be checked. In addition, the availability of sight distance for right turns from divided highways should be checked because of the possibility of sight obstructions in the median.

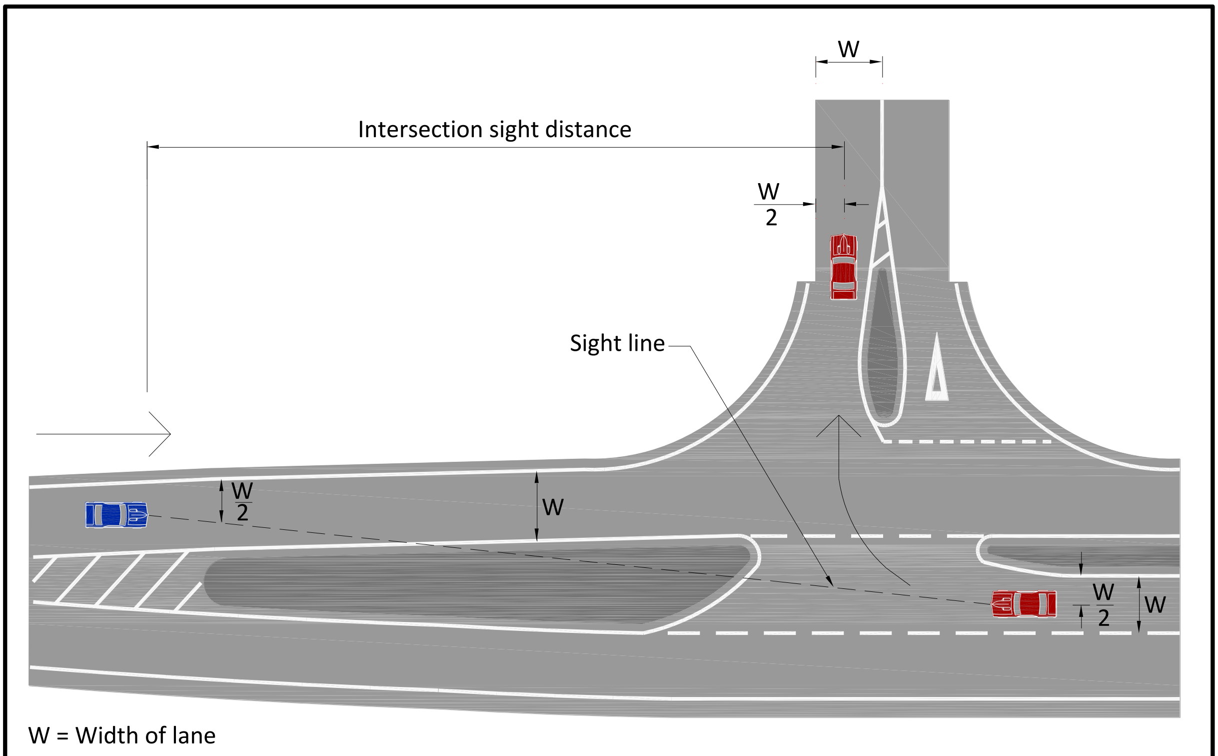


Figure 3.7: Right Turns from Major Roads

Table 3.12: Time Gap for Case "F", Right Turn from the Major Road

Design Vehicle	Time Gap, t_g (seconds)
Passenger Car	5.5
Single-Unit Truck	6.5
Combination Truck	7.5

Note:

For right-turning vehicles that cross more than one opposing lane, add 0.5 second for passenger cars and 0.7 second for trucks for each additional lane to be crossed.

(Source: *AASHTO's A Policy on Geometric Design of Highways and Streets, 2011* [2])

Table 3.13: Intersection Sight Distance — Case F, Right Turn from the Major Road

Design Speed (km/h)	SSD (m)	ISD (m)
20	20	35
30	35	50
40	50	65
50	65	80
60	85	95
70	105	110
80	130	125
90	160	140
100	185	155
110	220	170
120	250	185
130	285	200

(Source: *AASHTO's A Policy on Geometric Design of Highways and Streets, 2011* [2])

3.2.4.7 Special Considerations

If ISD cannot be provided because of environmental or right-of-way constraints, SSD should be provided at least on the major road along with additional safety measures, such as advance warning signs or reduced speed limit zones at the intersection area. The ISD criteria should also be applied to all private accesses and driveways along the highways.

3.3 Horizontal Alignment

3.3.1 General Considerations

Horizontal alignment consists of combination of straight lines called "tangents" and arcs called "curves." Horizontal curves are circular. These are introduced between the tangents to eliminate the abrupt deflection and to provide smooth transition between them.

In order to attain a consistent smooth flowing and aesthetically pleasing horizontal alignment, consideration should be given to the following practices:

- Alignment design should be consistent with topography and be as directional as possible while preserving community values.
- Length of the curves should be long enough to avoid the appearance of a kink.
- On high and long embankments, avoid using sharper curves.
- Flatter curves should be provided for certain design speed. Use of minimum design criteria should be avoided where practical and retained for critical conditions.
- Use of compound curves on high-speed roadways, design speed of 80 km/h or higher, should be avoided and if possible replace it with a simple curve. Where compound curves are used, the radius of flatter curve should not be more than twice the radius of sharper curve.
- Where reverse curves are provided on high-speed roadways, design speed of 80 km/h or higher, sufficient length of tangent between the reverse curves should be provided to adequate superelevation cross over between the curves.

- Brocken back curves, consisting of two curves in same direction separated by short tangent, should be avoided.
- Should be consistent with the design of the vertical alignment.
- Change in the median widths on tangent alignments should be avoided, where practical, so as not to introduce a distorted appearance.

3.3.2 Type of Curves and Curve Elements

3.3.2.1 Simple Circular Curve

The simple horizontal curve is an arc of a circle, as shown in **Figure 3.8**. Three geometric elements define the circular curve – the radius, central angle, and length of curve. Establishing any two of these elements, defines the third element.

3.3.2.2 Spiral Curve

The spiral is a transition curve mathematically defined as a curve with radius decreasing or increasing at a constant rate, as shown in **Figure 3.9**. Spirals promote uniform speeds by providing the natural turning path of a vehicle and minimizing the encroachment onto the adjacent lanes. In addition, it also provides the superelevation transition from the tangent to a simple curve, or between simple curves in a compound curve.

Spirals are encouraged to be used on all high-speed roadways, design speeds 80 km/h and higher, to facilitate the development of superelevation. *The AASHTO's Highway Safety Manual, 2010* [3] shows that the use of spirals on two-lane rural highways has a small but significant effect on reducing run-off-road crashes.

3.3.2.3 Compound Curve

A compound curve is two simple curves with different radii "back to back". When compound curves are used in open alignment, the larger radius should desirably be no more than 2 times the smaller radius. The use of compound curves for intersection design for larger vehicles minimizes the pavement area to enable off tracking. The **Figure 3.10** below shows the different elements encompassing the compound curve.

Compound curvature can be used to form the entire alignment of the turning roadway for design speed of 70 km/h or less. For higher design speeds, use of compound curves should be avoided as much as practical and replaced with simple circular curves. The minimum compound curve lengths are presented in **Table 3.14**.

If one of the curves has radius greater than twice the radius of other, then either a spiral or a circular curve of intermediate radius should be inserted between the two curves. If the calculated length of spiral is less than 30 m, then at least 30 m should be used. In this case, the length of spiral can be obtained from **Table 3.15** by using a radius that is the difference in the radii of the two arcs. For example, two curves to be connected by a spiral have radii of 250 m and 80 m. This difference of 170 m is very close to the minimum radius of 160 m in **Table 3.15** for which the suggested minimum length is about 60 m.

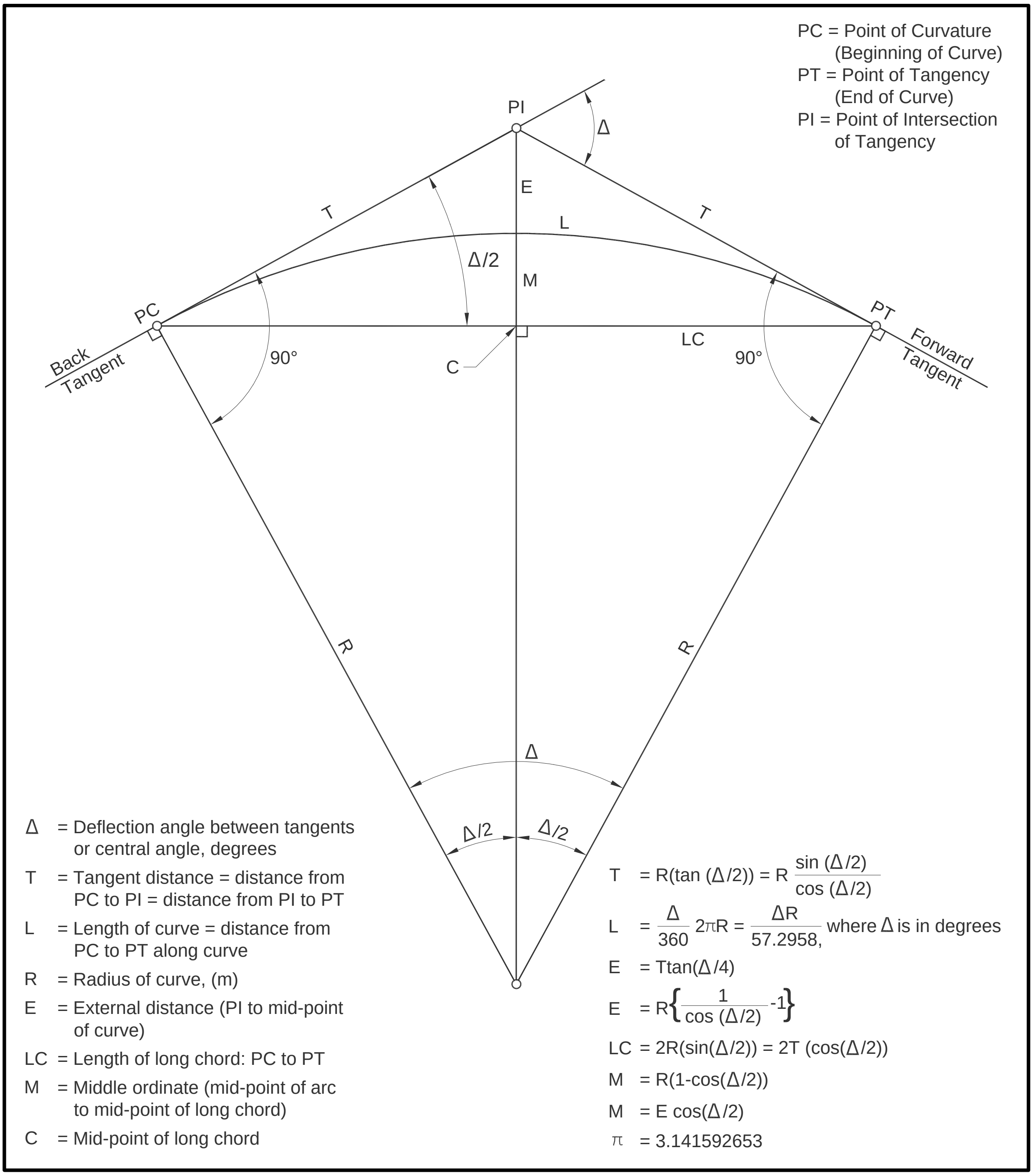
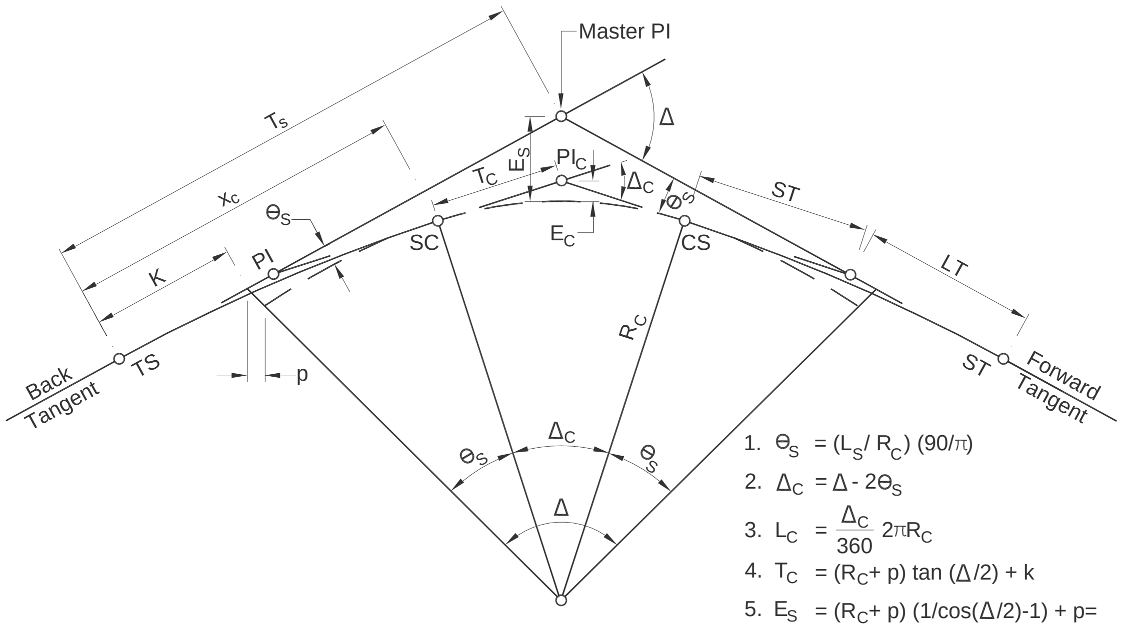


Figure 3.8: Simple Curve Elements



1. $\Theta_S = (L_S / R_C)(90/\pi)$
2. $\Delta_C = \Delta - 2\Theta_S$
3. $L_C = \dfrac{\Delta_C}{360} 2\pi R_C$
4. $T_C = (R_C + p)\tan(\Delta/2) + k$
5. $E_S = (R_C + p)(1/\cos(\Delta/2) - 1) + p = \left[\dfrac{(R_C + p)}{(1/\cos(\Delta/2)} - (R_C + p)\right] + p$

Master PI = Point of intersection of the main tangents.

PI_C = Point of intersection of circular curve tangents.

PI_s = Point of intersection of the main tangent and tangent of circular curve.

TS = Tangent to spiral, common point of tangent and spiral.

SC = Spiral to curve, common point of spiral and circular curve.

CS = Curve to spiral, common point of circularl curve and spiral.

ST = Spiral to tangent, common point of spiral and tangent.

R_c = Radius of circular curve.

L_s = Length of spiral.

L_c = Length of circular curve.

T_s = Tangent distance from Master PI to TS or ST, or tangent distance of completed combination of curves.

T_C = Tangent distance from SC or CS to PI_C.

E_S = External distance from Master PI to midpoint of circular curve portion.

LT = Long tangent of spiral only.

ST = Short tangent of spiral only.

LC = Long chord of spiral.

p = Offset distance from the main tangent to the PC or PT of the circular curve.

Δ = Intersection angle between main tangents of the entire curve.

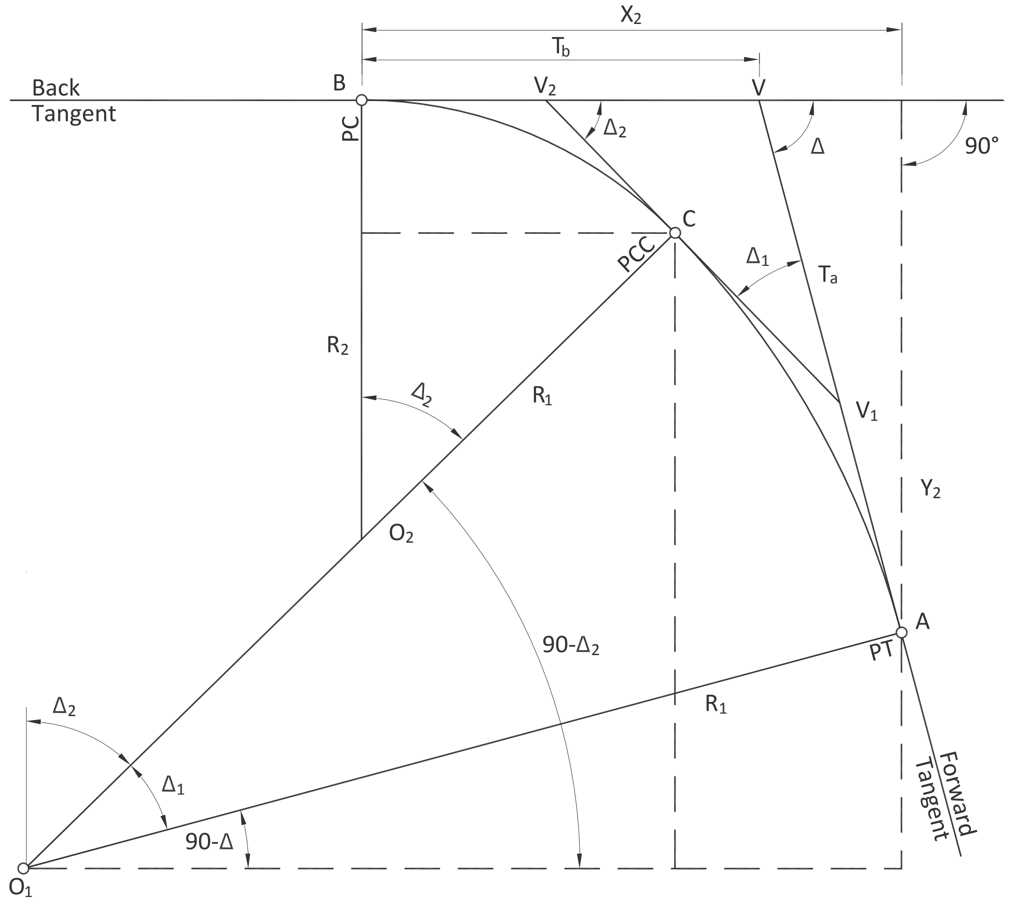
Δ_C = Intersection angle between tangents at the SC and the CS or the central angle of the circular curve.

Θ_S = Intersection angle between the tangent of the complete curve and the tangent at the SC, the spiral tangents intersection angle.

$x_C y_C$ = Coordinates of SC from the TS.

E_C = External distance from curve PI to midpoint of circular curve.

Figure 3.9: Simple Curve with Spirals



Given: R_1, R_2 Δ

Equations:

Δ = Total deflection angle = $\Delta_1 + \Delta_2$

$X_2 = R_1 \sin\Delta - (R_1 - R_2)\sin\Delta_2$

$Y_2 = R_2 - R_1 \cos\Delta + (R_1 - R_2)\cos\Delta_2$

$T_a = AV = Y_2 / \sin\Delta$

$T_b = BV = X_2 - T_a \cos\Delta$

$$\tan(\Delta_2 / 2) = \frac{R_1 (1-\cos\Delta) - T_a \sin\Delta}{R_1 (\sin\Delta) - T_a \cos\Delta - T_b}$$

PC = Point of Curvature (Beginning of Curve)

PT = Point of Tangency (End of Curve)

PCC = Point of Compound Curve

Figure 3.10: Compound Curve Elements

Table 3.14: Lengths of Circular Arcs for different Compound Curve Radii

Radius (m)	Length of Circular Arc (m)	
	Minimum	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's A Policy on Geometric Design of Highways and Streets, 2011* [2])

Table 3.15: Minimum Lengths of Spiral for Intersection Curves

Design Speed (km/h)	Minimum Radius (m)	Design Minimum Length of Spiral (m)
30	25	20
40	50	25
50	80	35
60	125	45
70	160	60

(Source: *AASHTO's A Policy on Geometric Design of Highways and Streets, 2011* [2])

3.3.2.4 Broken - Back Curve

Broken-back curves are those curves in which two curves of same direction are separated by a short tangent. **Figure 3.11** shows the elements of ***broken-back curve.*** This type of curve is unexpected by the driver and unpleasing in appearance. Attempt should be made to provide single circular curve or compound curves.

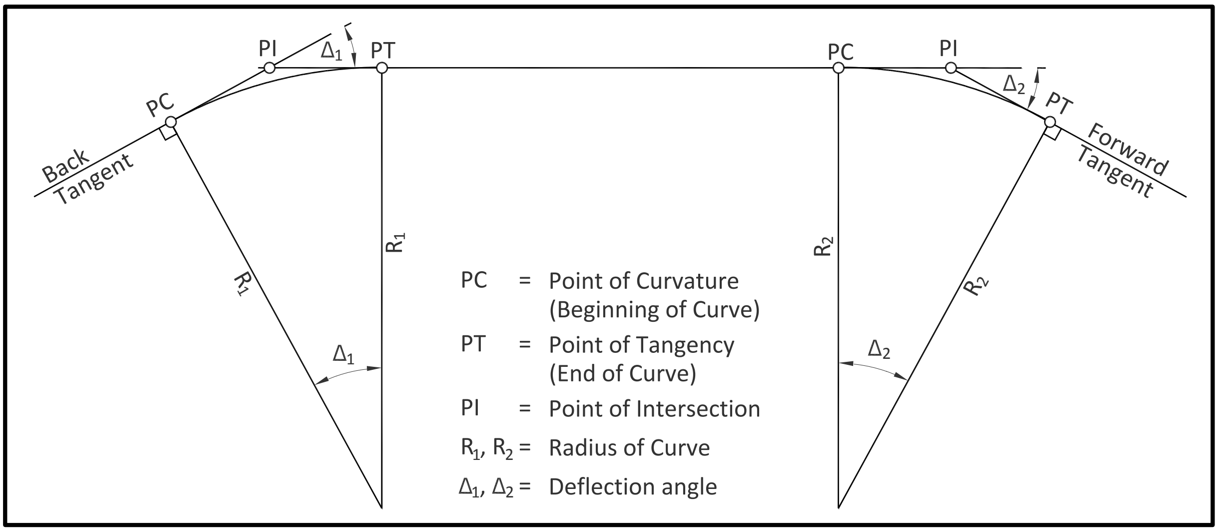


Figure 3.11: Broken Back Curve

3.3.2.5 *Reverse Curves*

Figure 3.12 shows ***reverse curves***, in which the alignment of the second curve is in the opposite direction to the first curve. Reverse curves without a tangent between them (top part of **Figure 3.12**) should be avoided by designing the road with a tangent between the curves as shown in the bottom half of the figure. Drivers operating at design speed cannot instantaneously change the direction and magnitude of their steering response, which is the action required in a reverse curve.

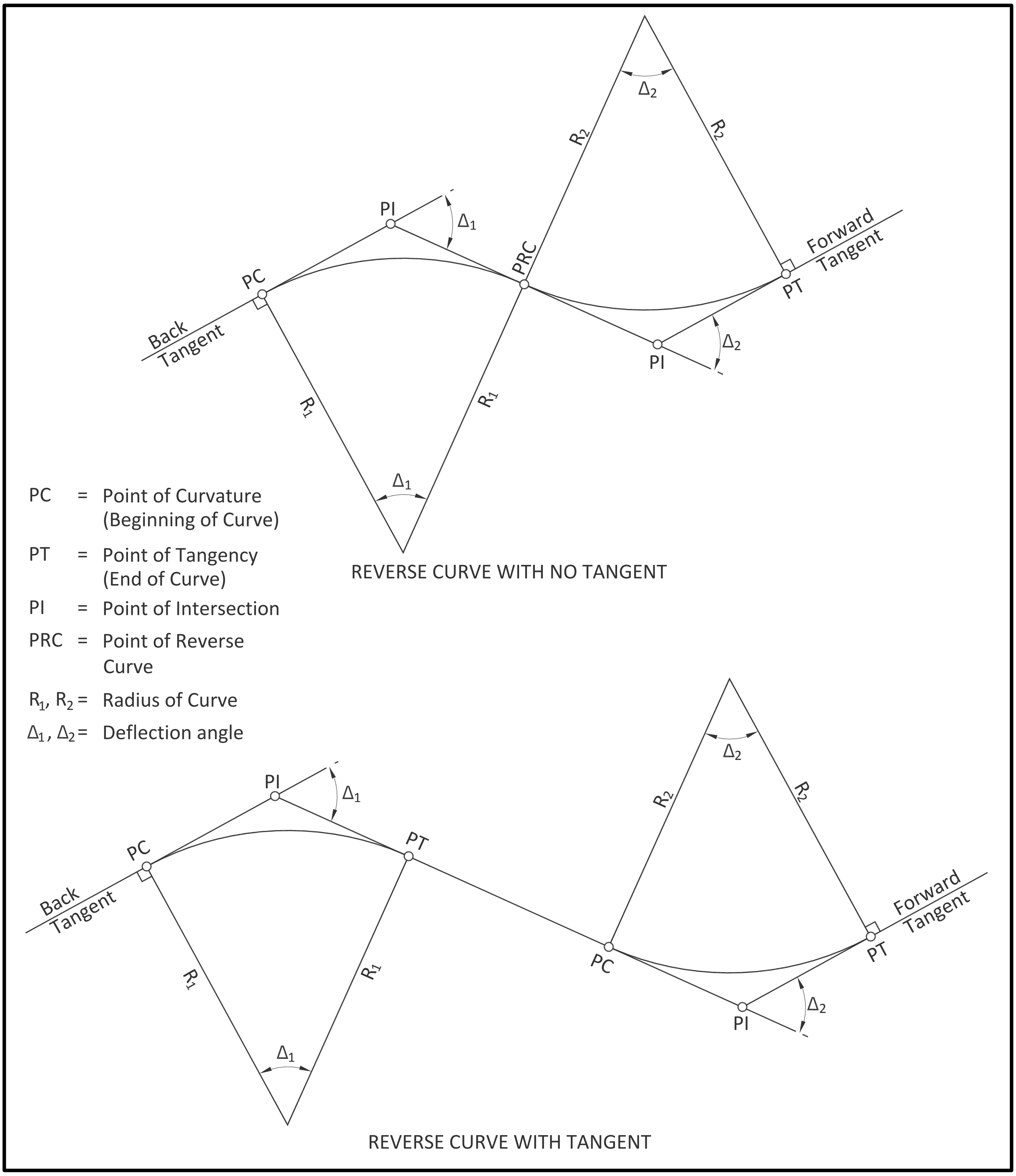


Figure 3.12: Reverse Curves

3.3.3 Design Considerations

This section provides guidance for the design of horizontal alignments and their components, such as horizontal curvature, superelevation and carriageway widening on horizontal curves.

3.3.3.1 Maximum Centreline Deflection without a Horizontal Curve

In order to eliminate abrupt deflection and to provide smooth transition, horizontal curves are introduced between tangents. For very low deflection angles (See **Table 3.16**), the designer may design the alignment without horizontal curve.

Table 3.16: Maximum Degree of Deflection without Horizontal Curve

Classification	Maximum degree of deflection without Horizontal Curve
High Speed Rural Roads	0.5 °
Low Speed Urban Roads	1.0 °

3.3.3.2 Minimum Length of Horizontal Curve

For small deflection angles, the horizontal curves should be long enough to avoid the appearance of a "kink." The minimum lengths of curves are shown in **Table 3.17**.

Table 3.17: Minimum Length of Horizontal Curve

Minimum Curve Length (m)	3V
Desirable Curve Length (m)	6V
Absolute Minimum Curve Length (m)	150 m for 5-degree deflection angle. Increased by 30 m for each 1-degree decrease in deflection angle.

Note: V= Design Speed in Km/h

3.3.3.3 Maximum Superelevation Rates for Roads and Highways

The maximum rates of superelevation used on highways are controlled by four factors:

1. Climate conditions (i.e., frequency and amount of snow and ice).
2. Terrain conditions (i.e., flat, rolling, or mountainous).
3. Type of area (i.e., rural or urban).
4. Frequency of very slow-moving vehicles whose operation might be affected by high superelevation rate.

These factors conclude that no single maximum superelevation can be recommended for a given condition. In order to promote design consistency, one maximum superelevation should be selected for similar climatic conditions and land use of the area.

Commonly, the highest superelevation rate for highways is 10 percent. Whereas, in areas with snow and ice, the maximum superelevation rate shall not exceed 8 percent. In certain conditions, like facilitating cross drainage on gravel road, higher superelevation rates may be applied. Generally, 8 percent is recognized as a reasonable maximum value for superelevation rate in rural areas. In Urban areas, maximum superelevation rate shall not exceed 4 percent.

Where traffic congestion or extensive marginal development restricts the speed of vehicles, a lower maximum rate of superelevation, usually 4 percent should be applied. Similarly, a low maximum rate or no superelevation should be provided on intersections or at locations where there are turning and crossing movements, warning devices, signals or similar conditions, which tends vehicles to drive slowly. In summary, it is recommended that:

- A rate of 8 percent should not be exceeded.
- A rate of 4 percent is applicable for urban design in areas with few constraints.
- Superelevation may be omitted on low-speed urban streets where severe constraints are present.

3.3.3.4 Minimum Radius without Superelevation or Adverse Crown

No superelevation is required on very flat curves. The minimum radius for a normal crown (NC) section for each design speed and maximum superelevation rate is shown **Table 3.18**.

Table 3.18: Minimum Radius without Superelevation

Design Speed (Km/h)	Minimum Radius Without Superelevation for each e_{max} (m)			
	4 %	6 %	8 %	10 %
20	163	194	184	197
30	371	421	443	454
40	679	738	784	790
50	951	1050	1090	1110
60	1310	1440	1490	1520
70	1740	1910	1970	2000
80	2170	2360	2440	2480
90	2640	2880	2970	3010
100	3250	3510	3630	3690
110	-	4060	4180	4250
120	-	4770	4900	4960
130	-	5240	5360	5410

(Source: *AASHTO's A Policy on Geometric Design of Highways and Streets, 2011* [2])

3.3.3.5 Minimum Radius with Superelevation

Based on the maximum allowable side friction factors, **Table 3.19** gives the minimum radius for each of the five maximum superelevation rates of 4, 6, 8 and 10 percent, calculated using the following equation:

$$R_{\min} = \frac{V^2}{127(0.01\, e_{max} + f_{max})} \qquad \textit{Equation 3.10}$$

3.3.3.6 Normal Cross Slope

Cross slope is defined as the transverse slope across the pavement from the centreline of an undivided roadway or the edge of the median of a divided roadway to the edge of the carriageway or the face of the kerb. The minimum rate of cross slope applicable to the carriageway is determined by drainage needs.

Consistent with the type of highway and amount of rainfall, snow, and ice, the usually accepted minimum values for cross slope range from 1.5 percent to 2.0 percent; however, the recommended cross slope is 2.0 percent for all types of roads.

Table 3.19: Minimum Radius Using Limiting Values of "e" and "f"

Design Speed (km/h)	Maximum Side Friction Factor, f_{max}	Minimum Radius with Superelevation for each e_{max}			
		4 %	6 %	8 %	10 %
15	0.40	4	4	4	4
20	0.35	8	8	7	7
30	0.28	22	21	20	19
40	0.23	47	43	41	38
50	0.19	86	79	73	68
60	0.17	135	123	113	105
70	0.15	203	184	168	154
80	0.14	280	252	229	210
90	0.13	375	336	304	277
100	0.12	492	437	394	358
110	0.11	-	560	501	454
120	0.09	-	756	667	597
130	0.08	-	951	832	739

Notes:
In recognition of safety considerations, use of e_{max} = 4.0% should be limited to urban conditions.
(Source: *AASHTO's A Policy on Geometric Design of Highways and Streets, 2011* [2])

3.3.4 Spiral Curve Transition

Spiral curves are defined in Section 3.3.2.2, and the curve elements are illustrated in **Figure 3.9**.

The principal advantages of providing spiral curves in horizontal alignment are as follows:

- A spiral transition curve follows the natural turning path of a vehicle. Spiral transitions provide a natural, easy-to-follow path for drivers.
- The transition curve length provides a suitable location for the superelevation runoff.
- It also provides a suitable location for attaining carriageway widening on circular curves.
- It avoids noticeable kinks in the alignment at the beginning and end of circular curve, thus enhances the design of the highway or road.

Maximum Radius of circular curves below which the use of a spiral curve transitions is recommended are provided in **Table 3.20**.

3.3.4.1 Length of Spiral

The spiral lengths listed in **Table 3.21** are recommended as desirable values for roads and highways design. Spiral curve lengths longer than those shown in **Table 3.21** may be needed at turning roadway terminals to develop the desired superelevation. In such cases, the length of runoff should be taken as the minimum length of spiral.

If the desirable spiral curve length shown in **Table 3.21** or the runoff length is less than the minimum spiral curve length determined from the **Equation 3.11**, the minimum spiral curve length should be used in design.

Table 3.20: Maximum Radius for Use of a Spiral Curve Transition

Design speed (km/h)	Maximum radius (m)
20	24
30	54
40	95
50	148
60	213
70	290
80	379
90	480
100	592
110	716
120	852
130	1000

(Source: *AASHTO's A Policy on Geometric Design of Highways and Streets, 2011* [2])

Table 3.21: Desirable Length of Spiral Curve Transition

Design Speed (km/h)	Spiral Length (m)
20	11
30	17
40	22
50	28
60	33
70	39
80	44
90	50
100	56
110	61
120	67
130	72

(Source: *AASHTO's A Policy on Geometric Design of Highways and Streets, 2011* [2])

The minimum length of spiral is based on consideration of driver comfort and shifts in the lateral position of vehicles. The minimum spiral length can be computed as:

$$L_{s,min} = \sqrt{24(P_{min})R}$$ ***Equation 3.11***

Where,

$L_{s,min}$ = minimum length of spiral, (m)

P_{min} = minimum lateral offset between the tangent and circular curve (0.20 m)

R = radius of circular curve, (m)

Spirals should not be so long (relative to the length of the circular curve) that drivers are misled about the sharpness of the approaching curve. A maximum length of spiral that should minimize the likelihood of such concerns can be computed as:

$$L_{s,max} = \sqrt{24(P_{max})R}$$ ***Equation 3.12***

Where,

$L_{s,max}$ = maximum length of spiral, (m)

P_{max} = maximum lateral offset between the tangent and circular curve (1.0 m)

R = radius of circular curve, (m)

3.3.5 Design Superelevation

Table 3.22 to **Table 3.25** show minimum values of "R" for various combinations of superelevation and design speeds for each of four values of maximum superelevation rate. Superelevation rate should be applied in the increments of 0.2 percent and no interpolation is required. The superelevation from the table shall be selected for a radius equal to or less than the required radius. For example, an 80 km/h curve with a maximum superelevation rate of 8 percent, and a radius of 570 m, should use the superelevation rate of 5.4 percent against the radius of 549 m.

Table 3.22: Minimum Radii for Design Superelevation Rates, Design Speeds, and e_{max} = 4%

e (%)	V_d = 20 km/h	V_d = 30 km/h	V_d = 40 km/h	V_d = 50 km/h	V_d = 60 km/h	V_d = 70 km/h	V_d = 80 km/h	V_d = 90 km/h	V_d = 100 km/h
	R (m)	*R* (m)	*R* (m)	*R* (m)	*R* (m)	*R* (m)	*R* (m)	*R* (m)	*R* (m)
NC	163	371	679	951	1310	1740	2170	2640	3250
RC	102	237	441	632	877	1180	1490	1830	2260
2.2	75	187	363	534	749	1020	1290	1590	1980
2.4	51	132	273	435	626	865	1110	1390	1730
2.6	38	99	209	345	508	720	944	1200	1510
2.8	30	79	167	283	422	605	802	1030	1320
3.0	24	64	137	236	356	516	690	893	1150
3.2	20	54	114	199	303	443	597	779	1010
3.4	17	45	96	170	260	382	518	680	879
3.6	14	38	81	144	222	329	448	591	767
3.8	12	31	67	121	187	278	381	505	658
4.0	8	22	47	86	135	203	280	375	492

Note: Use of e_{max} = 4% should be limited to urban conditions.

(Source: *AASHTO's A Policy on Geometric Design of Highways and Streets, 2011* [2])

Table 3.23: Minimum Radii for Design Superelevation Rates, Design Speeds, and emax = 6%

e (%)	V_d = 20 km/h	V_d = 30 km/h	V_d = 40 km/h	V_d = 50 km/h	V_d = 60 km/h	V_d = 70 km/h	V_d = 80 km/h	V_d = 90 km/h	V_d = 100 km/h	V_d = 110 km/h	V_d = 120 km/h	V_d = 130 km/h
	R (m)	*R (m)*	*R (m)*	*R (m)*	*R (m)*	*R (m)*	*R (m)*	*R (m)*	*R (m)*	*R (m)*	*R (m)*	*R (m)*
NC	194	421	738	1050	1440	1910	2360	2880	3510	4060	4770	5240
RC	138	299	525	750	1030	1380	1710	2090	2560	2970	3510	3880
2.2	122	265	465	668	919	1230	1530	1880	2300	2670	3160	3500
2.4	109	236	415	599	825	1110	1380	1700	2080	2420	2870	3190
2.6	97	212	372	540	746	1000	1260	1540	1890	2210	2630	2930
2.8	87	190	334	488	676	910	1150	1410	1730	2020	2420	2700
3.0	78	170	300	443	615	831	1050	1290	1590	1870	2240	2510
3.2	70	152	269	402	561	761	959	1190	1470	1730	2080	2330
3.4	61	133	239	364	511	697	882	1100	1360	1600	1940	2180
3.6	51	113	206	329	465	640	813	1020	1260	1490	1810	2050
3.8	42	96	177	294	422	586	749	939	1170	1390	1700	1930
4.0	36	82	155	261	380	535	690	870	1090	1300	1590	1820
4.2	31	72	136	234	343	488	635	806	1010	1220	1500	1720
4.4	27	63	121	210	311	446	584	746	938	1140	1410	1630
4.6	24	56	108	190	283	408	538	692	873	1070	1330	1540
4.8	21	50	97	172	258	374	496	641	812	997	1260	1470
5.0	19	45	88	156	235	343	457	594	755	933	1190	1400
5.2	17	40	79	142	214	315	421	549	701	871	1120	1330
5.4	15	36	71	128	195	287	386	506	648	810	1060	1260
5.6	13	32	63	115	176	260	351	463	594	747	980	1190
5.8	11	28	56	102	156	232	315	416	537	679	900	1110
6.0	8	21	43	79	123	184	252	336	437	560	756	951

(Source: *AASHTO's A Policy on Geometric Design of Highways and Streets, 2011* [2])

Table 3.24: Minimum Radii for Design Superelevation Rates, Design Speeds, and e_{max} = 8%

e (%)	V_d = 20 km/h	V_d = 30 km/h	V_d = 40 km/h	V_d = 50 km/h	V_d = 60 km/h	V_d = 70 km/h	V_d = 80 km/h	V_d = 90 km/h	V_d = 100 km/h	V_d = 110 km/h	V_d = 120 km/h	V_d = 130 km/h
	R (m)	*R (m)*	*R (m)*	*R (m)*	*R (m)*	*R (m)*	*R (m)*	*R (m)*	*R (m)*	*R (m)*	*R (m)*	*R (m)*
NC	184	443	784	1090	1490	1970	2440	2970	3630	4180	4900	5360
RC	133	322	571	791	1090	1450	1790	2190	2680	3090	3640	4000
2.2	119	288	512	711	976	1300	1620	1980	2420	2790	3290	3620
2.4	107	261	463	644	885	1190	1470	1800	2200	2550	3010	3310
2.6	97	237	421	587	808	1080	1350	1650	2020	2340	2760	3050
2.8	88	216	385	539	742	992	1240	1520	1860	2160	2550	2830
3.0	81	199	354	496	684	916	1150	1410	1730	2000	2370	2630
3.2	74	183	326	458	633	849	1060	1310	1610	1870	2220	2460
3.4	68	169	302	425	588	790	988	1220	1500	1740	2080	2310
3.6	62	156	279	395	548	738	924	1140	1410	1640	1950	2180
3.8	57	144	259	368	512	690	866	1070	1320	1540	1840	2060
4.0	52	134	241	344	479	648	813	1010	1240	1450	1740	1950
4.2	48	124	224	321	449	608	766	948	1180	1380	1650	1850
4.4	43	115	208	301	421	573	722	895	1110	1300	1570	1760
4.6	38	106	192	281	395	540	682	847	1050	1240	1490	1680
4.8	33	96	178	263	371	509	645	803	996	1180	1420	1610
5.0	30	87	163	246	349	480	611	762	947	1120	1360	1540
5.2	27	78	148	229	328	454	579	724	901	1070	1300	1480
5.4	24	71	136	213	307	429	549	689	859	1020	1250	1420
5.6	22	65	125	198	288	405	521	656	819	975	1200	1360
5.8	20	59	115	185	270	382	494	625	781	933	1150	1310
6.0	19	55	106	172	253	360	469	595	746	894	1100	1260
6.2	17	50	98	161	238	340	445	567	713	857	1060	1220
6.4	16	46	91	151	224	322	422	540	681	823	1020	1180
6.6	15	43	85	141	210	304	400	514	651	789	982	1140
6.8	14	40	79	132	198	287	379	489	620	757	948	1100
7.0	13	37	73	123	185	270	358	464	591	724	914	1070
7.2	12	34	68	115	174	254	338	440	561	691	879	1040
7.4	11	31	62	107	162	237	318	415	531	657	842	998
7.6	10	29	57	99	150	221	296	389	499	621	803	962
7.8	9	26	52	90	137	202	273	359	462	579	757	919
8.0	7	20	41	73	113	168	229	304	394	501	667	832

(Source: *AASHTO's A Policy on Geometric Design of Highways and Streets, 2011* [2])

Table 3.25: Minimum Radii for Design Superelevation Rates, Design Speeds, and e_{max} = 10%

e (%)	V_d = 20 km/h	V_d = 30 km/h	V_d = 40 km/h	V_d = 50 km/h	V_d = 60 km/h	V_d = 70 km/h	V_d = 80 km/h	V_d = 90 km/h	V_d = 100 km/h	V_d = 110 km/h	V_d = 120 km/h	V_d = 130 km/h
	R (m)	*R* (m)	*R* (m)	*R* (m)	*R* (m)	*R* (m)	*R* (m)	*R* (m)	*R* (m)	*R* (m)	*R* (m)	*R* (m)
NC	197	454	790	1110	1520	2000	2480	3010	3690	4250	4960	5410
RC	145	333	580	815	1120	1480	1840	2230	2740	3160	3700	4050
2.2	130	300	522	735	1020	1340	1660	2020	2480	2860	3360	3680
2.4	118	272	474	669	920	1220	1520	1840	2260	2620	3070	3370
2.6	108	249	434	612	844	1120	1390	1700	2080	2410	2830	3110
2.8	99	229	399	564	778	1030	1290	1570	1920	2230	2620	2880
3.0	91	211	368	522	720	952	1190	1460	1790	2070	2440	2690
3.2	85	196	342	485	670	887	1110	1360	1670	1940	2280	2520
3.4	79	182	318	453	626	829	1040	1270	1560	1820	2140	2370
3.6	73	170	297	424	586	777	974	1200	1470	1710	2020	2230
3.8	68	159	278	398	551	731	917	1130	1390	1610	1910	2120
4.0	64	149	261	374	519	690	866	1060	1310	1530	1810	2010
4.2	60	140	245	353	490	652	820	1010	1240	1450	1720	1910
4.4	56	132	231	333	464	617	777	953	1180	1380	1640	1820
4.6	53	124	218	315	439	586	738	907	1120	1310	1560	1740
4.8	50	117	206	299	417	557	703	864	1070	1250	1490	1670
5.0	47	111	194	283	396	530	670	824	1020	1200	1430	1600
5.2	44	104	184	269	377	505	640	788	975	1150	1370	1540
5.4	41	98	174	256	359	482	611	754	934	1100	1320	1480
5.6	39	93	164	243	343	461	585	723	896	1060	1270	1420
5.8	36	88	155	232	327	441	561	693	860	1020	1220	1370
6.0	33	82	146	221	312	422	538	666	827	976	1180	1330
6.2	31	77	138	210	298	404	516	640	795	941	1140	1280
6.4	28	72	130	200	285	387	496	616	766	907	1100	1240
6.6	26	67	121	191	273	372	476	593	738	876	1060	1200
6.8	24	62	114	181	261	357	458	571	712	846	1030	1170
7.0	22	58	107	172	249	342	441	551	688	819	993	1130
7.2	21	55	101	164	238	329	425	532	664	792	963	1100
7.4	20	51	95	156	228	315	409	513	642	767	934	1070
7.6	18	48	90	148	218	303	394	496	621	743	907	1040
7.8	17	45	85	141	208	291	380	479	601	721	882	1010
8.0	16	43	80	135	199	279	366	463	582	699	857	981
8.2	15	40	76	128	190	268	353	448	564	679	834	956
8.4	14	38	72	122	182	257	339	432	546	660	812	932
8.6	14	36	68	116	174	246	326	417	528	641	790	910
8.8	13	34	64	110	166	236	313	402	509	621	770	888
9.0	12	32	61	105	158	225	300	386	491	602	751	867
9.2	11	30	57	99	150	215	287	371	472	582	731	847
9.4	11	28	54	94	142	204	274	354	453	560	709	828
9.6	10	26	50	88	133	192	259	337	432	537	685	809
9.8	9	24	46	81	124	179	242	316	407	509	656	786
10.0	7	19	38	68	105	154	210	277	358	454	597	739

(Source: *AASHTO's A Policy on Geometric Design of Highways and Streets, 2011* [2])

3.3.6 Superelevation Transition

Superelevation transition is the length needed to change the cross slope of pavement from a normal cross slope section to a fully superelevated section. The superelevation transition section consists of two parts:

- Tangent Runout length **(*L_t*)**
- Superelevation runoff length **(*L_r*)**

The tangent runout (L_t) section consists of the length of roadway required to remove the adverse pavement cross slope from the normal cross slope rate to zero (flat) or vice versa, by rotating the outside edge of the carriageway.

The superelevation runoff (L_r) section consists of the length of roadway needed to accomplish a change cross slope from zero (flat) to full superelevation, or vice versa.

For circular curves without spiral curves 2/3 of the superelevation runoff shall be within the tangent while 1/3 shall be applied within the circular curve. The tangent runout section shall be introduced just in advance of the Runoff section on tangent. **Figure 3.13** illustrates application of superelevation on circular curves without transition curves. The ratio of the superelevation runoff length on the tangent and on the curve could be changed, depending on site conditions and geometrics, such as presence of a bridge approach or presence of the low point on a sag vertical curve within the limits of the transition. The maximum superelevation runoff distribution ratio allowed shall be 50 percent on the tangent and 50 percent on the curve with justification.

The length of superelevation run-off is calculated using the formula from *AASHTO, 2011* [2]:

$$L_r = \frac{(wn_1)e_d}{\Delta}(b_w)$$ ***Equation 3.13***

Where,

L_r = minimum length of superelevation runoff, (m)
w = width of one traffic lane, (m) (typically 3.65 m)
n_1 = number of lanes rotated
e_d = design superelevation rate, (percent)
b_w = adjustment factor for number of lanes rotated
Δ = maximum relative gradient, (percent)

Relative gradient (Δ) is the maximum allowable grade difference between the longitudinal grades at the axis of rotation and the outside edge of the pavement. **Table 3.26** lists the relative gradient values based on design speeds.

The adjustment factors (b_w) are listed in **Table 3.27**. They generally apply to undivided roadways with axis of rotation at the centreline of the roadway. For divided roadways, rotated about the median edge, and for interchange ramps, the adjustment factor should be one (1) regardless how many lanes are rotated.

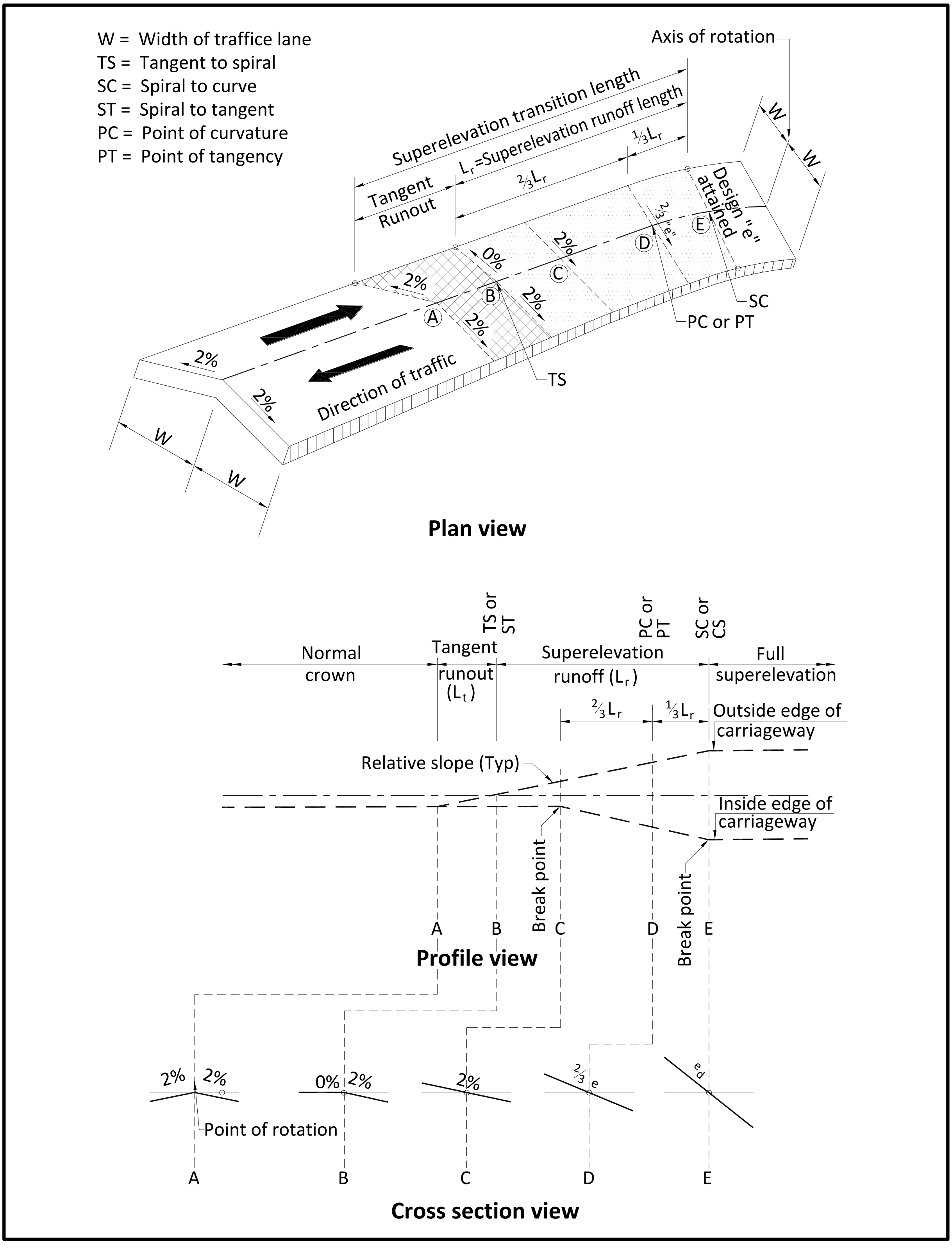


Figure 3.13: Superelevation Transition for Two-Lane Roadways

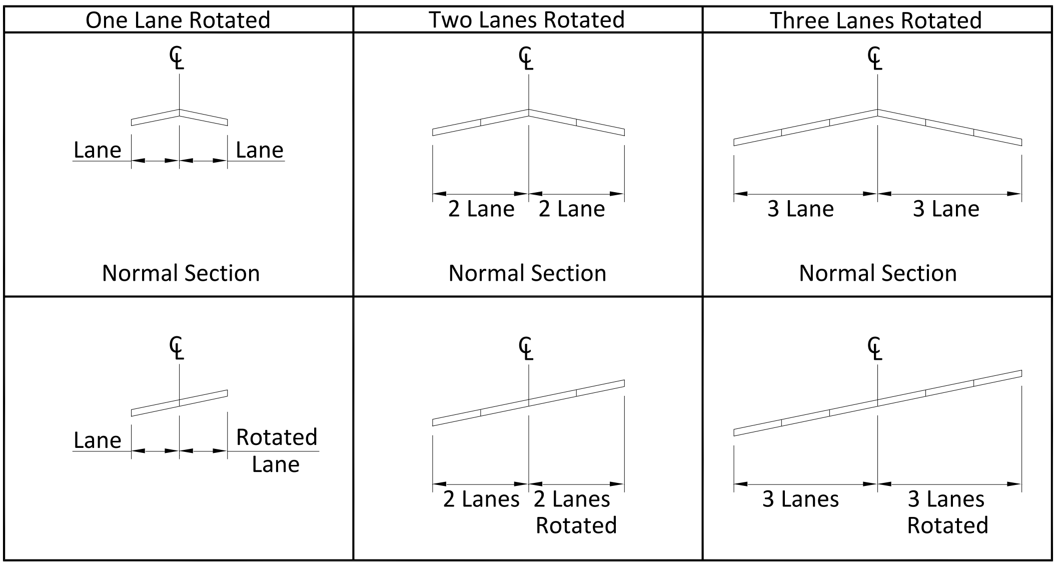
Table 3.26: Maximum Relative Gradients

Design Speed (km/h)	Maximum Relative Gradient (%)	Equivalent Maximum Relative Slope
20	0.80	1:125
30	0.75	1:133
40	0.70	1:143
50	0.65	1:154
60	0.60	1:167
70	0.55	1:182
80	0.50	1:200
90	0.47	1:213
100	0.44	1:227
110	0.41	1:244
120	0.38	1:263
130	0.35	1:286

(Source: *AASHTO's A Policy on Geometric Design of Highways and Streets, 2011* [2])

Table 3.27: Adjustment Factor for Number of Lanes Rotated

Number of Lanes Rotated, n_1	Adjustment Factor,* b_w	Length Increase Relative to One-Lane Rotated, (= n_1 x b_w)
1	1.00	1.0
1.5	0.83	1.25
2	0.75	1.5
2.5	0.70	1.75
3	0.67	2.0
3.5	0.64	2.25



* bw = [1 + 0.5 (n1 – 1)]/n1

(Source: *AASHTO's A Policy on Geometric Design of Highways and Streets, 2011* [2])

For curves with spiral curves, full superelevation shall be applied within the spiral curves. The length of runoff should be taken as the minimum length of spiral. The tangent runout section shall be introduced just in advance of the spiral curve on tangent. In this case, the complete circular curve shall be fully superelevated. **Figure 3.13** illustrates application of superelevation on transition curves.

The minimum length of tangent runout can be calculated by using the following equation from *AASHTO, 2011* [2]:

$$L_t = \frac{e_{NC}}{e_d} L_r$$ ***Equation 3.14***

When Spiral curve is provided the length of tangent runout is calculated by using the following formula (Source: *AASHTO, 2011* [2]):

$$L_t = \frac{e_{NC}}{e_d} L_s$$ ***Equation 3.15***

Where,
L_t = minimum length of tangent runout, (m)
e_{NC} = normal cross slope rate, (percent)
e_d = design superelevation rate, (percent)
L_r = minimum length of superelevation runoff, (m)
L_s = Length of spiral, (m)

Providing longer run-off lengths is desirable. However, considerations should be given to the combination of horizontal and vertical alignment. Longer runoff lengths may lead to flat areas of carriageway, and hence will not drain properly. In such cases, the designer may need to reduce the runoff length by applying lane adjustment factors.

3.3.6.1 Methods of Attaining Superelevation

Four methods can be used to transition the pavement to a superelevated cross section. These methods include:

- **Method 1** Revolving a carriageway with normal cross slopes about the centreline profile
- **Method 2** Revolving a carriageway with normal cross slopes about the inside-edge profile
- **Method 3** Revolving a carriageway with normal cross slopes about the outside-edge profile
- **Method 4** Revolving a straight cross slope carriageway about the outside-edge profile

Method 1 is the most commonly used method for undivided two-lane roadways and is mostly adopted. It could be used for divided roadways where the divided segments are not crowned, for divided roadways the inside median edge will act as the axis of rotation. Methods 2 and 3 are used on divided highways with a crown in the pavement. Method 4 is the method most commonly used for divided highways with straight cross slope and for interchange ramps where there is no crown. The methods for attaining superelevation are nearly the same and all four methods are acceptable. These four methods are illustrated in **Figure 3.14**.

Method 2 shown in **Figure 3.14** is preferable where the lower edge profile is a major control. As for drainage, where the overall appearance is a high priority, the methods 3 and 4 shown in **Figure 3.14** are desirable because the upper-edge profile—retains the smoothness of the control profile.

The angular breaks between the straight-line profiles must be rounded in the finished design. The minimum vertical curve length in meters can be equal to 0.2 times the design speed in km/h.

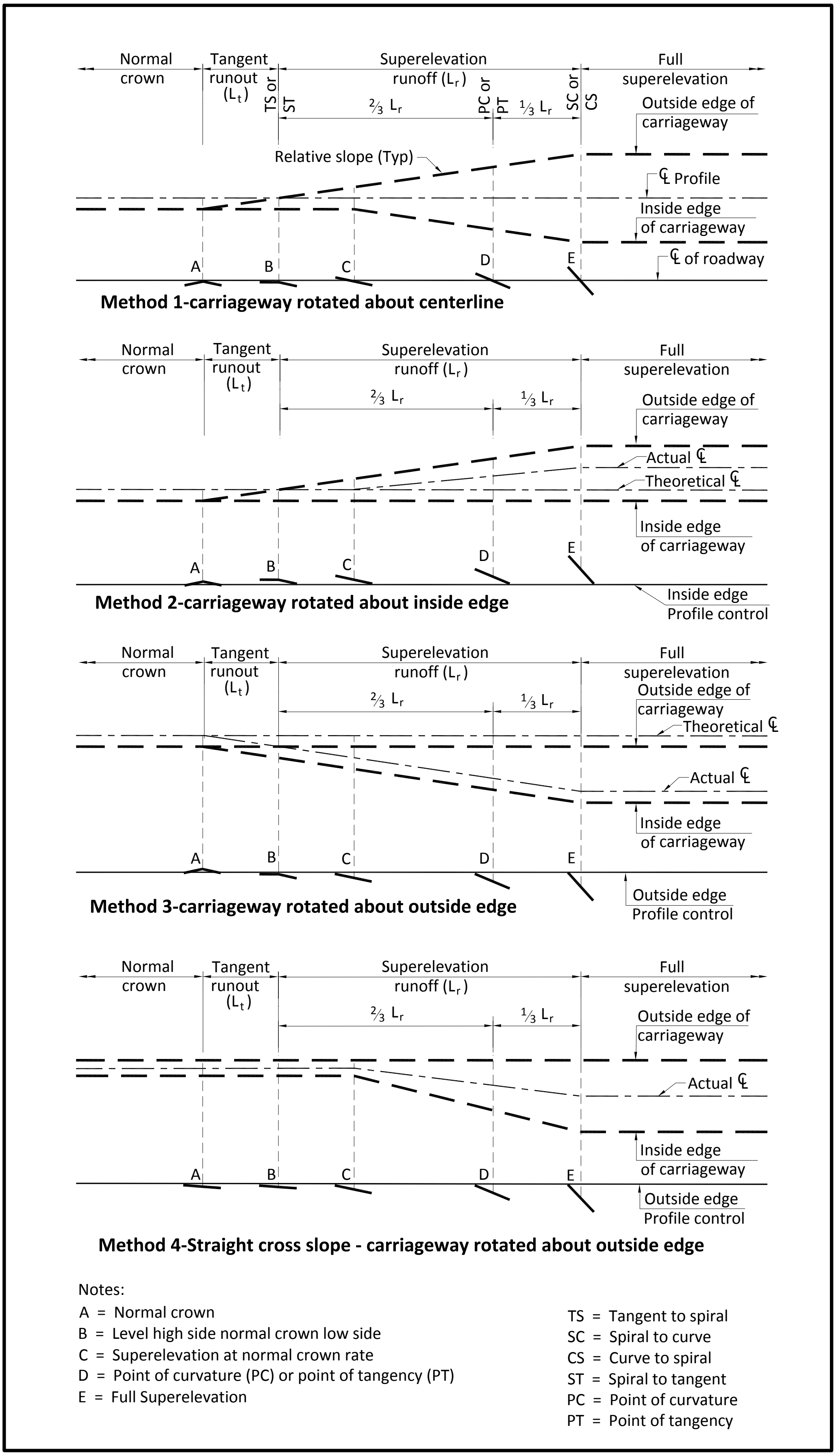


Figure 3.14: Methods of Attaining Superelevation for a Curve to the Right

3.3.6.2 Minimum Superelevation Transition Grades

Lack of adequate longitudinal grade and negligible cross slope during pavement rotation leads to potential surface drainage problems. In order to relieve such problems, two techniques can be used. One technique is to maintain 0.5 percent minimum profile grade in the transition section. Second technique involves maintaining the minimum edge of pavement grade of 0.2 percent (0.5 percent for kerbed sections) in transition area. Both methods should be applied to decide the minimum grade for transition areas.

The second grade criterion involves the following series of equations:

	Unkerbed	Kerbed
1.	$G \leq -\Delta^* - 0.2$	$G \leq -\Delta^* - 0.5$
2.	$G \geq -\Delta^* + 0.2$	$G \geq -\Delta^* + 0.5$
3.	$G \leq \Delta^* - 0.2$	$G \leq \Delta^* - 0.5$
4.	$G \geq \Delta^* + 0.2$	$G \geq \Delta^* + 0.5$

$$\Delta^* = \frac{(wn_1)e_d}{L_r}$$ ***Equation 3.16***

Where,

G = profile grade, (percent)

Δ^* = effective maximum relative gradient, (percent)

w = width of one traffic lane, (m) (typically 3.65 m)

n_1 = number of lanes rotated

e_d = design superelevation rate, (percent)

L_r = length of superelevation runoff, (m)

Example for Determining Minimum Transition Grade

Consider a roadway with no kerb having the maximum relative gradient of 0.65 percent. The first criteria excludes grades between -0.50 and +0.5 percent. The second criteria would involve the following calculations:

1. $G \leq -0.65 - 0.2$ => $G \leq -0.85$
2. $G \geq -0.65 + 0.2$ => $G \geq -0.45$
3. $G \leq +0.65 - 0.2$ => $G \leq +0.45$
4. $G \geq +0.65 + 0.2$ => $G \geq -0.85$

The second grade criterion excludes grades in the range of –0.85 to –0.45 percent and 0.45 to 0.85 percent. Thus, to satisfy both criteria, the profile grade should be outside the range of –0.85 to +0.85 percent in transition area for adequate pavement surface drainage.

3.3.7 Superelevation Transition on Compound

In general, compound curve transitions are most commonly considered for application to low-speed turning roadways at intersections.

Superelevation transition on compound curves depends on the distance between the point of curvature (PC) of the first curve and the point of compound curvature (PCC) of the second curve. Guidance is provided for two-lane roadways for two cases. The same guidance should be applied for multi-lane roadways.

- **Case-I**: Distance between the PC and PCC is less than or equal to 90 m
- **Case-II**: Distance between the PC and PCC is greater than 90 m

Case-I: Distance between the PC and PCC is less than or equal to 90 m

Figure 3.15 illustrates the development and positioning of superelevation transition on compound curves when the distance between PC and PCC is less than 90 m. In such case, the superelevation shall be applied such that two-third of the design superelevation rate of curve 1 is achieved at PC and full superelevation rate of curve 2 is attained at PCC. The equivalent maximum relative slope shall be constant for the entire transition length.

Case-II: Distance between the PC and PCC Greater than 90 m

Figure 3.16 illustrates the development and positioning of superelevation transition on compound curves when the distance between PC and PCC is greater than 90 m. In such case, the superelevation shall be applied such that two-third of the design superelevation rate of curve-1 is achieved at PC and full superelevation rate of curve-2 is attained at PCC. Full superelevation of curve-1 is maintained between the PC and PCC for some distance, before transitioning to curve-2 design superelevation rate.

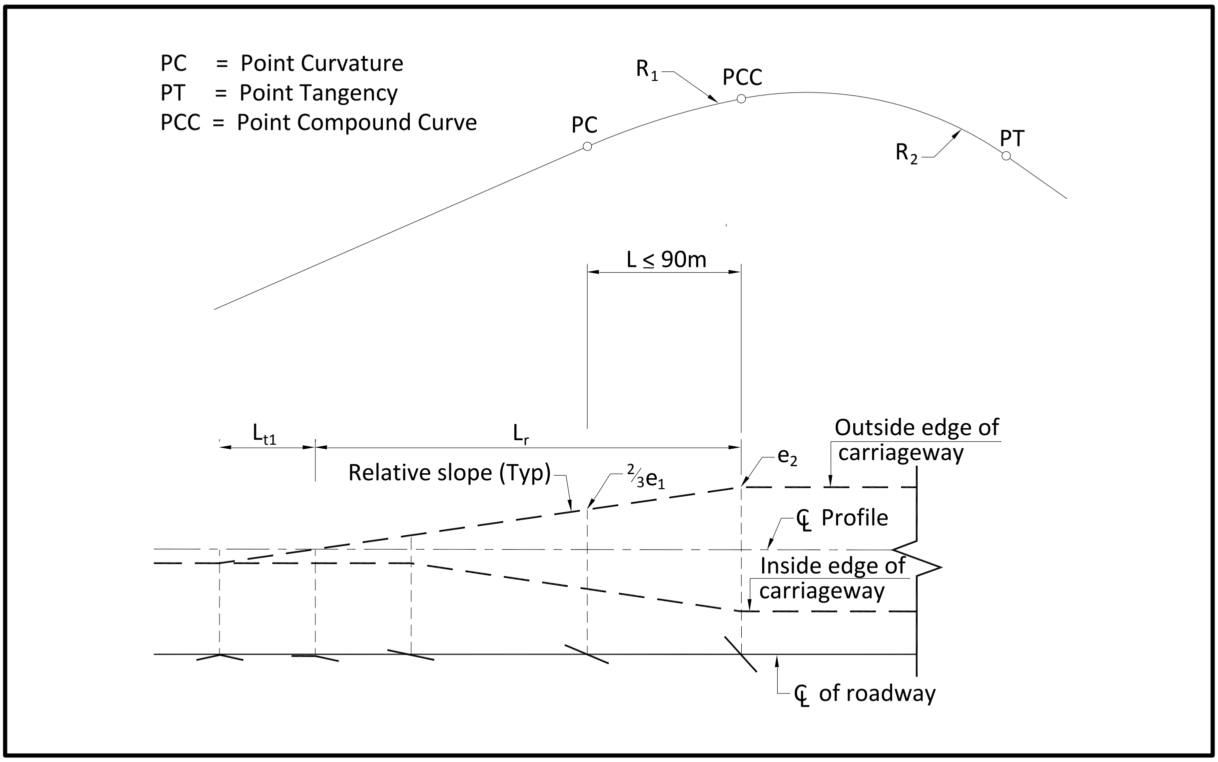


Figure 3.15: Superelevation Transition on Compound Curves (Distance between PC and PCC is less than or equal to 90 m)

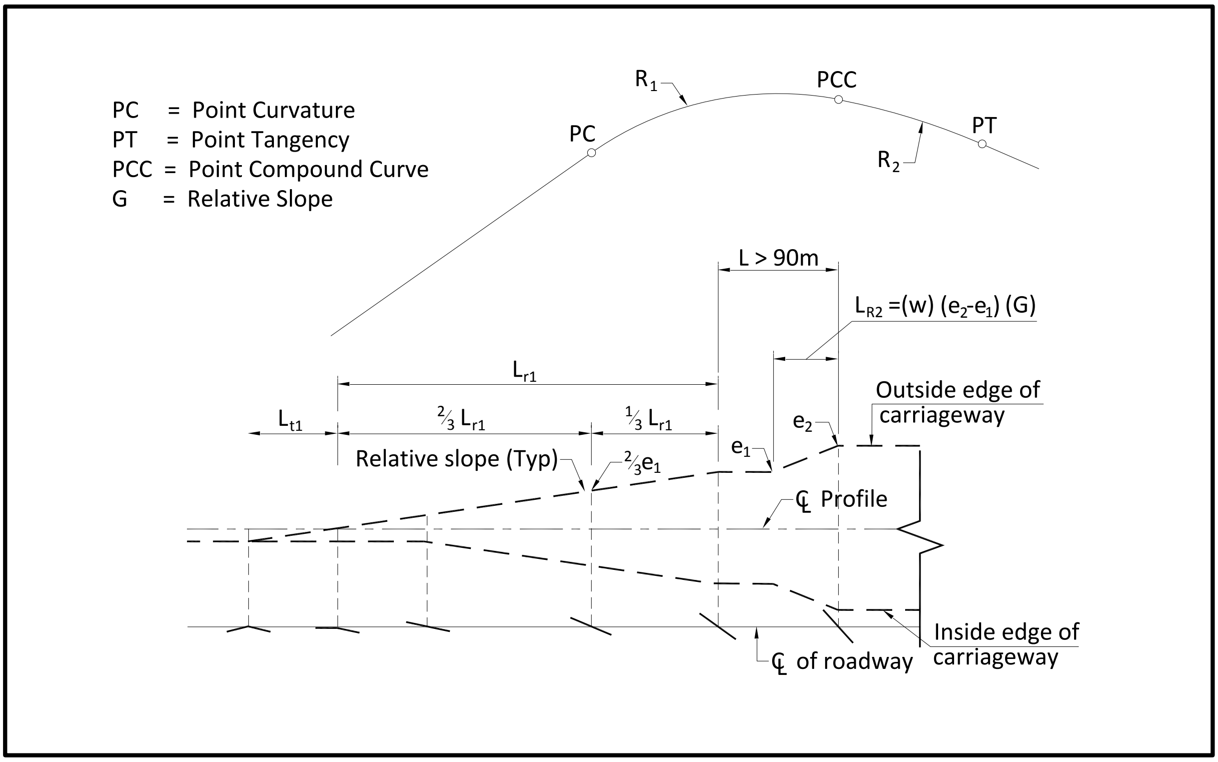


Figure 3.16: Superelevation Transition on Compound Curves (Distance between PC and PCC is greater than 90 m)

3.3.8 Superelevation on Reverse Curves

Figure 3.17 illustrates the method for development and positioning of superelevation transition length between reversing curves. The reverse curves should be separated by adequate tangent length to enable development of superelevation.

The preferred tangent length should be equal to:

$$\frac{2}{3}L_{r1} + \frac{2}{3}L_{r2} \qquad \textit{Equation 3.17}$$

At minimum tangent length should be equal to:

$$\frac{1}{2}L_{r1} + \frac{1}{2}L_{r2} \qquad \textit{Equation 3.18}$$

Where,

L_{r1} = superelevation runoff length for curve 1

L_{r2} = superelevation runoff length for curve 2

If there is no tangent present in the superelevation, runoff shall be positioned such that zero percent cross-slope is at the point of reverse curvature.

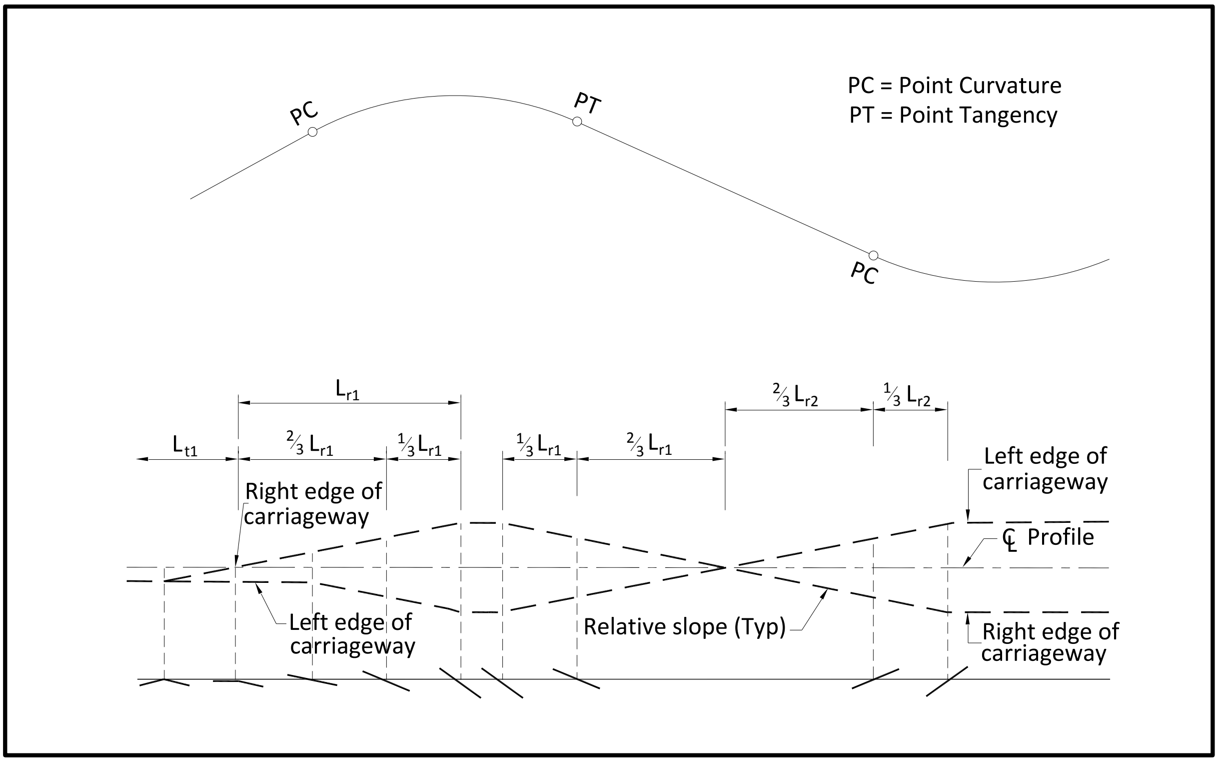


Figure 3.17: Superelevation between Reverse Curves

3.3.9 Shoulder Slopes on Superelevated Roadways

Normal paved shoulder slopes are 2 percent. A general guidance for rotation of shoulder in superelevated section is provided in **Figure 3.18** on divided roadways. On high side of the shoulder, the maximum allowed algebraic difference between the shoulder cross fall and carriageway cross fall shall not exceed 8 percent. For example if the superelevation is 7 percent, the shoulder slope should be 1 percent sloping away from the travel lanes to maintain the 8 percent break. On the low side of the shoulder, the slopes of the travel lane and shoulder are the same, 2 percent, and they should be rotated simultaneously to achieve the design superelevation.

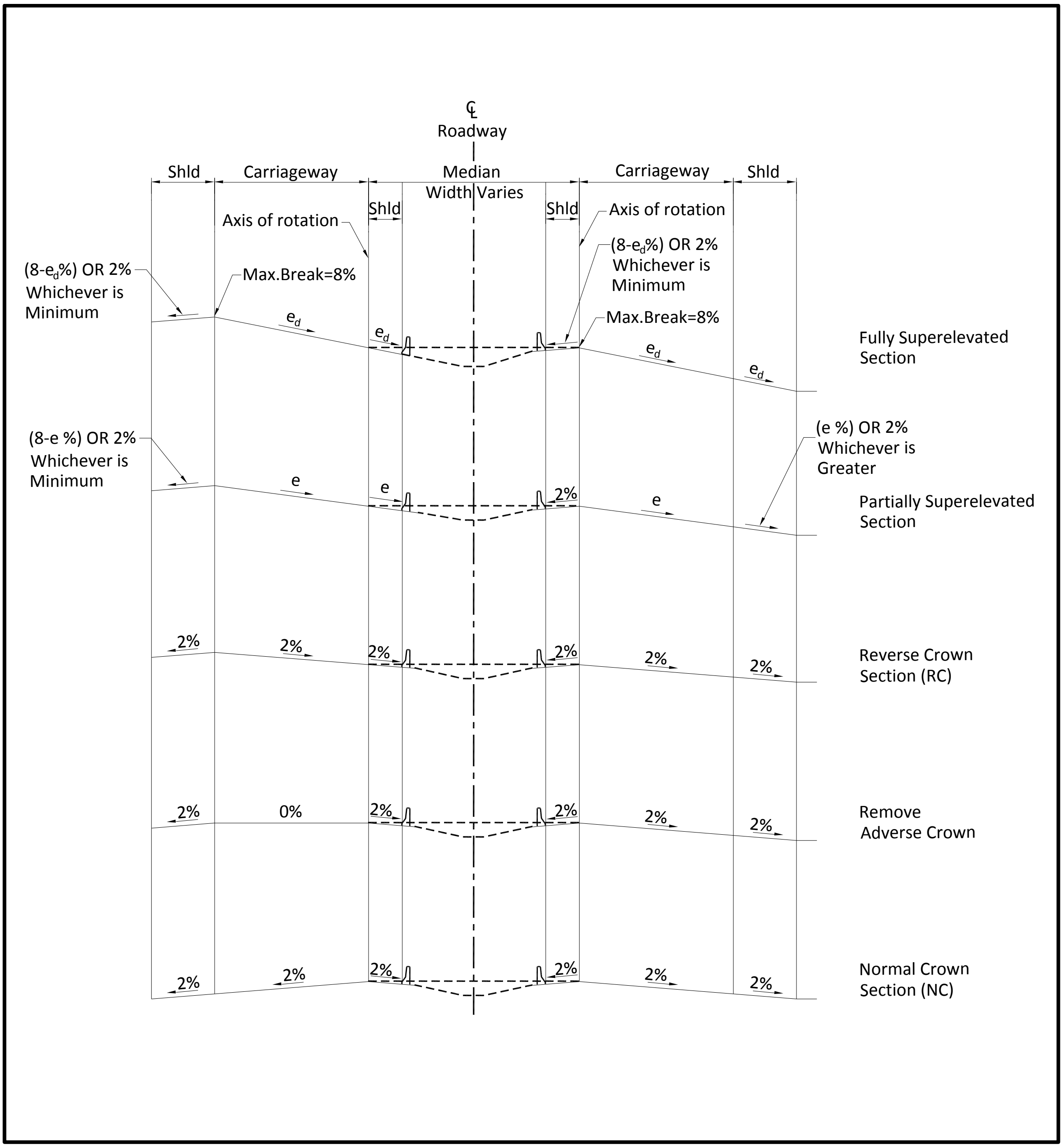


Figure 3.18: Shoulder Rotation during Superelevation Application

3.3.10 Superelevation for Low-Speed Urban Streets

On low-speed urban streets, the use of superelevation for horizontal curves can be minimized. The combined effect of various factors often makes the use of superelevation impractical on low-speed urban areas. These factors include:

- wide pavement areas
- the need to meet the grade of adjacent property
- surface drainage considerations
- the desire to maintain low-speed operation
- frequency of intersecting cross streets, alleys, and driveways

Therefore, horizontal curves on low-speed urban streets are frequently designed without superelevation, sustaining the lateral force solely with side friction.

3.3.10.1 Sharpest Curve without Superelevation Minimum Radius for Section with Normal Crown

The –2.0 percent row in **Table 3.28** provides the minimum curve radii for which a normal crown of 2.0 percent should be retained. Sharper curves should have no adverse cross slope and should be superelevated in accordance with **Table 3.28**.

Table 3.28: Minimum Radii and Superelevation for Low-Speed Urban Streets

e (%)	*Vd* = 20 km/h	*Vd* = 30 km/h	*Vd* = 40 km/h	*Vd* = 50 km/h	*Vd* = 60 km/h	*Vd* = 70 km/h
	R (m)	*R (m)*	*R (m)*	*R (m)*	*R (m)*	*R (m)*
–4.0	10	30	66	131	218	351
–3.0	10	28	63	123	202	322
–2.8	10	28	62	122	200	316
–2.6	10	28	62	120	197	311
–2.4	10	28	61	119	194	306
–2.2	10	27	61	117	192	301
–2.0	10	27	60	116	189	297
–1.5	9	27	59	113	183	286
0	9	25	55	104	167	257
1.5	9	24	51	96	153	234
2.0	9	24	50	94	149	227
2.2	8	23	50	93	148	224
2.4	8	23	50	92	146	222
2.6	8	23	49	91	145	219
2.8	8	23	49	90	143	217
3.0	8	23	48	89	142	214
3.2	8	23	48	89	140	212
3.4	8	23	48	88	139	210
3.6	8	22	47	87	138	207
3.8	8	22	47	86	136	205
4.0	8	22	47	86	135	203

Notes:
Superelevation may be optional on low-speed urban streets.
Negative superelevation values beyond –2.0 percent should be used for unpaved surfaces such as gravel, crushed stone, and earth. However, a normal cross slope of –2.5 percent may be used on paved surfaces in areas with intense rainfall. (Source: *AASHTO, 2011* [2])

3.3.11 Roadway Widening on Horizontal Curves

When a large vehicle traverses a horizontal curve or makes a turn, the rear wheels do not follow precisely the front wheel path due to offtracking. In order to offset this effect of offtracking, additional widening of carriageway may be necessary on horizontal curves to aid the manoeuvring of large vehicles. The widening depends on the design speed, horizontal curvature, width of carriageway and the design vehicle used. The additional width provided will allow the driver to manoeuvre comfortably through the curve without encroaching on the opposing lane or shoulder. Widening is expensive, thus small amounts of widening, less than 0.6 m, are not cost effective and may be disregarded.

The WB-19 design vehicle is considered representative for two-lane open-highway conditions. The carriageway widening values for a WB-19 vehicle on a two-lane highway are presented in **Table 3.29**. However, for other design vehicles the widening values of **Table 3.29** should be adjusted in accordance with **Table 3.30**. For open highways where radii are larger than 200 m, and design speed are over 50 km/h, the difference in track widths of different vehicles is not substantial, thus there is no need for adjustments. Widening values obtained from **Table 3.29** should be checked using commercially available truck turning template software and adjusted accordingly. After adjustment, if the widening value obtained is negative, then no widening is required and same carriageway width as on tangent shall be provided on curve.

3.3.11.1 Application of Widening on Curves

Widening should transition gradually on both the approach end and the departing end of the curve. It should produce smooth edge of the carriageway and should fit to the paths of vehicles entering or leaving the curve. On curves which have no spiral transition, the widening should be applied on the inside edge of the pavement only. Where spiral curves are provided, widening may be placed on the inside or divided equally between the outside and inside of the circular curve.

The widening distribution with respect to the curve should be consistent with the superelevation attainment methodology. For curves without spiral, widening should develop over the length of superelevation runoff; while for curves with spiral transition, the widening should be developed over the length of the spiral.

Regardless of how widening is provided, the final centreline markings after the application of widening should be placed midway between the edges of the widened pavement. **Figure 3.19** shows the widening application on two lane roadways.

Table 3.29: Design Values for Carriageway Widening on Open Highway Curves (Two- Lane Highways, One-Way or Two-Way)

Radius of Curve (m)	Roadway width = 7.2 m Design Speed (km/h)						Roadway width = 6.6 m Design Speed (km/h)						Roadway width = 6.0 m Design Speed (km/h)					
	50	60	70	80	90	100	50	60	70	80	90	100	50	60	70	80	90	100
3000														0.6	0.6	0.6	0.6	0.6
2500													0.6	0.6	0.6	0.6	0.6	0.7
2000													0.6	0.6	0.6	0.7	0.7	0.7
1500													0.6	0.7	0.7	0.7	0.7	0.8
1000											0.6	0.6	0.7	0.8	0.8	0.8	0.9	0.9
900										0.6	0.6	0.6	0.8	0.8	0.8	0.9	0.9	0.9
800									0.6	0.6	0.6	0.7	0.8	0.8	0.9	0.9	0.9	1.0
700							0.6	0.6	0.6	0.7	0.7	0.7	0.9	0.9	0.9	1.0	1.0	1.0
600							0.6	0.7	0.7	0.7	0.8	0.8	0.9	1.0	1.0	1.0	1.1	1.1
500					0.6	0.6	0.7	0.7	0.8	0.8	0.9	0.9	1.0	1.0	1.1	1.1	1.2	1.2
400		0.6	0.6	0.7	0.7	0.8	0.8	0.9	0.9	1.0	1.0	1.1	1.1	1.2	1.2	1.3	1.3	1.4
300	0.7	0.8	0.8	0.9	1.0	1.0	1.0	1.1	1.1	1.2	1.3	1.3	1.3	1.4	1.4	1.5	1.6	1.6
250	0.9	1.0	1.0	1.1	1.1		1.2	1.3	1.3	1.4	1.4		1.5	1.6	1.6	1.7	1.7	
200	1.1	1.2	1.3	1.3			1.4	1.5	1.6	1.6			1.7	1.8	1.9	1.9		
150	1.5	1.6	1.7	1.8			1.8	1.9	2.0	2.1			2.1	2.2	2.3	2.4		
140	1.6	1.7					1.9	2.0					2.2	2.3				
130	1.8	1.8					2.1	2.1					2.4	2.4				
120	1.9	2.0					2.2	2.3					2.5	2.6				
110	2.1	2.2					2.4	2.5					2.7	2.8				
100	2.3	2.4					2.6	2.7					2.9	3.0				
90	2.5						2.8						3.1					
80	2.8						3.1						3.4					
70	3.2						3.5						3.8					

Notes:

Values shown are for WB-19 design vehicle and represent widening in meters. For other design vehicles, use adjustments in **Table 3.30**.

For 3-lane roadways, multiply above values by 1.5.

For 4-lane roadways, multiply above values by 2.

(Source: AASHTO, 2011) [2]

Table 3.30: Adjustments for Carriageway Widening Values on Open Highway Curves (Two-Lane Highways, One-Way or Two-Way)

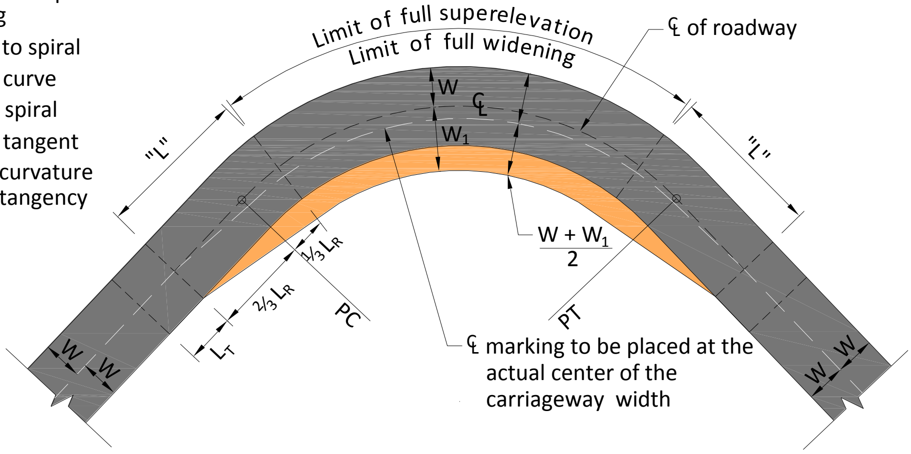
Radius of Curve (m)	Design Vehicle							
	SU-9	SU-12	WB-12	WB-20	WB-20D	WB-28D	WB-30T	WB-33D
3000	–0.4	–0.3	–0.3	0.0	0.0	0.0	0.0	0.0
2500	–0.4	–0.4	–0.3	0.0	0.0	0.0	0.0	0.1
2000	–0.4	–0.4	–0.4	0.0	0.0	0.0	0.0	0.1
1500	–0.4	–0.4	–0.4	0.0	–0.1	0.0	0.0	0.1
1000	–0.5	–0.4	–0.4	0.0	–0.1	0.0	0.0	0.1
900	–0.5	–0.4	–0.4	0.0	–0.1	0.0	0.0	0.1
800	–0.5	–0.5	–0.4	0.0	–0.1	0.0	0.0	0.2
700	–0.5	–0.5	–0.5	0.1	–0.1	0.1	0.0	0.2
600	–0.6	–0.5	–0.5	0.1	–0.1	0.1	–0.1	0.2
500	–0.6	–0.6	–0.5	0.1	–0.2	0.1	–0.1	0.3
400	–0.7	–0.6	–0.6	0.1	–0.2	0.1	–0.1	0.3
300	–0.8	–0.7	–0.7	0.1	–0.3	0.1	–0.1	0.4
250	–0.9	–0.8	–0.8	0.1	–0.3	0.2	–0.1	0.5
200	–1.1	–1.0	–0.9	0.2	–0.4	0.2	–0.2	0.6
150	–1.3	–1.2	–1.1	0.2	–0.6	0.3	–0.2	0.8
140	–1.4	–1.2	–1.2	0.3	–0.6	0.3	–0.2	0.9
130	–1.5	–1.3	–1.2	0.3	–0.6	0.3	–0.2	1.0
120	–1.6	–1.4	–1.3	0.3	–0.7	0.3	–0.3	1.1
110	–1.7	–1.5	–1.4	0.3	–0.8	0.4	–0.3	1.2
100	–1.8	–1.6	–1.5	0.4	–0.8	0.4	–0.3	1.3
90	–2.0	–1.8	–1.6	0.4	–0.9	0.4	–0.4	1.4
80	–2.2	–1.9	–1.8	0.5	–1.0	0.5	–0.4	1.6
70	–2.5	–2.2	–2.0	0.5	–1.2	0.6	–0.5	1.9

Notes:

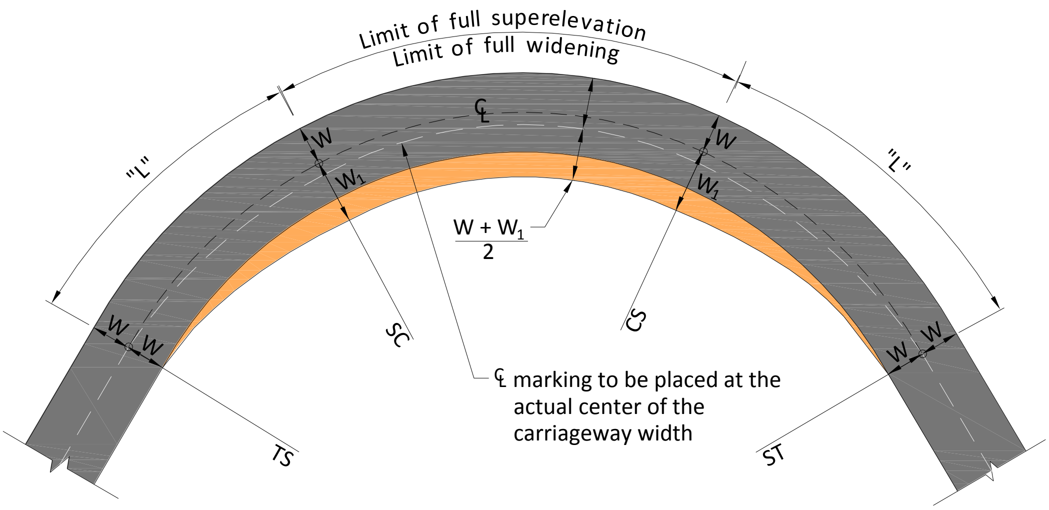
- Adjustments are applied by adding to or subtracting from the values in **Table 3.29**.
- Adjustments depend only on radius and design vehicle; they are independent of roadway width and design speed.
- For 3-lane roadways, multiply above values by 1.5.
- For 4-lane roadways, multiply above values by 2.

(Source: *AASHTO's A Policy on Geometric Design of Highways and Streets, 2011* [2])

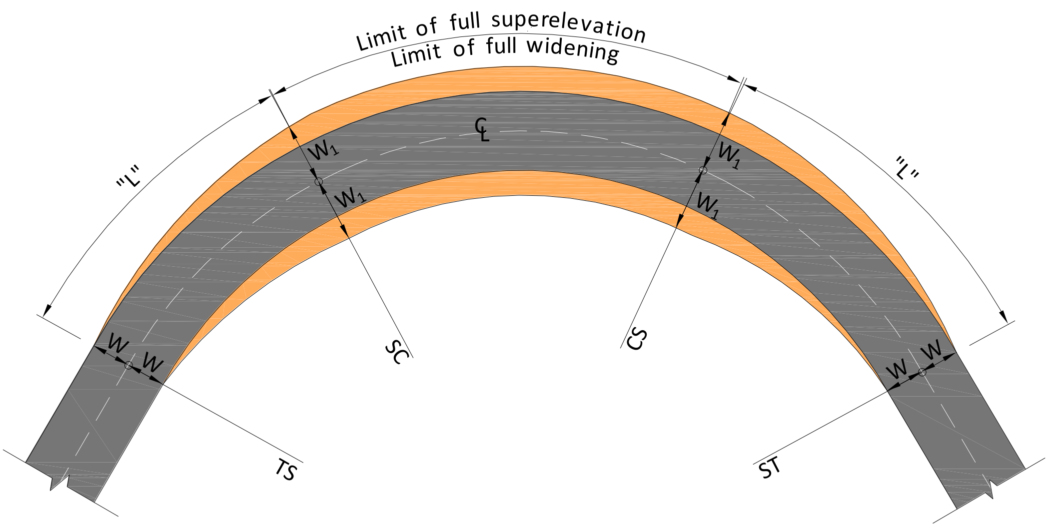
L = Superelevation transition
= $L_R + L_T$
W = Width of lane (m)
W_1 = Width of lane plus widening
TS = Tangent to spiral
SC = Spiral to curve
CS = Curve to spiral
ST = Spiral to tangent
PC = Point of curvature
PT = Point of tangency



Carriageway Widening for a Simple Curve - Widening on Inside of Curve



Carriageway Widening for Spiral Curve - Widening on Inside of Curve



Carriageway Widening for Spiral Curve - Equal Widening on Both Sides of Centerline

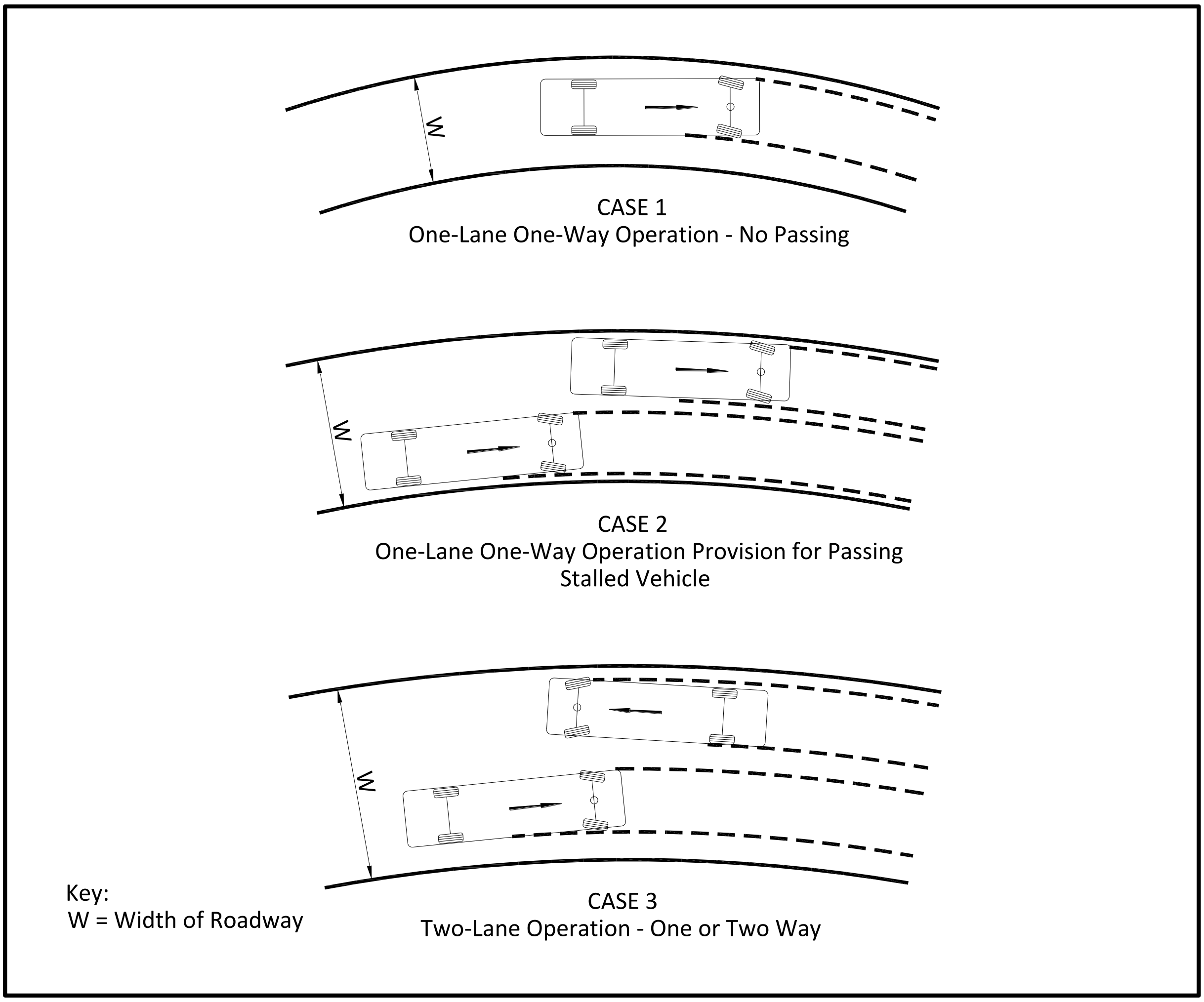
Figure 3.19: Carriageway Widening on Horizontal Curves

3.3.11.2 Widths for Turning Roadways at Intersections and Interchanges

The width of turning roadways at intersections and interchanges depends on the design vehicle, design speed, and the radius of the curvature of the inner edge of pavement. Selection of the design vehicle is based on the type, size, and frequency of vehicles expected to use the roadway. The width of the turning roadway is determined from the track width of the design vehicle combined with the radius of the curvature and the design speeds that typically range between 20 km/h to 30 km/h. The roadway width includes the width of the lane, width of the shoulders or clearances to the face of kerb.

Width of turning roadways also depends on the design traffic conditions and operational purposes of the roadway. There are three cases of operational purposes of the turning and are illustrated in **Figure 3.20**.

- **Case - I** : One way operation without provision for passing a stalled vehicle
- **Case - II** : One way operation with provision for passing a stalled vehicle
- **Case - III**: Two way operation (either one- or two-way)



(Source: *AASHTO's A Policy on Geometric Design of Highways and Streets, 2011* [2])

Figure 3.20: Turning Roadway Widths on Curves at Intersections

Design traffic conditions are described as follows:

- **Traffic Condition A:** Predominantly passenger cars with some consideration to single unit trucks (SU-9 and SU-12).

- **Traffic Condition B**: Includes sufficient number of single unit trucks (SU-9 and SU-12) that govern the design with some consideration to intermediate tractor-semitrailers combination vehicles (WB-12 and WB-15).
- **Traffic Condition C**: Includes sufficient number to intermediate tractor-semitrailers combination (WB-12 and WB-15) and interstate semitrailer (WB-20) that govern the design.

The radius of curvature of the inner edge of pavement depends on the design speed and the expected design vehicles using the roadway. **Table 3.31** shows pavement width values for various design traffic conditions, operational purposes and curvature of the inside edge of pavement. The values shown in this table should be used as a guide and check with a turning template or computer simulated turning template.

Table 3.31: Design Widths of Pavements for Turning Roadways

Radius on Inner Edge of Pavement, R (m)	Case-I			Case-II			Case-III		
	Design Traffic Condition								
	A	B	C	A	B	C	A	B	C
15	5.4	5.5	7.0	6.0	7.8	9.2	9.4	11.0	13.6
25	4.8	5.0	5.8	5.6	6.9	7.9	8.6	9.7	11.1
30	4.5	4.9	5.5	5.5	6.7	7.6	8.4	9.4	10.6
50	4.2	4.6	5.0	5.3	6.3	7.0	7.9	8.8	9.5
75	3.9	4.5	4.8	5.2	6.1	6.7	7.7	8.5	8.9
100	3.9	4.5	4.8	5.2	5.9	6.5	7.6	8.3	8.7
125	3.9	4.5	4.8	5.1	5.9	6.4	7.6	8.2	8.5
150	3.6	4.5	4.5	5.1	5.8	6.4	7.5	8.2	8.4
Tangent	3.6	4.2	4.2	5.0	5.5	6.1	7.3	7.9	7.9
Width Modification									
No Stabilized Shoulder	None			None			None		
Sloping Kerb	None			None			None		
Vertical Kerb									
One side	Add 0.3 m			None			Add 0.3 m		
Two sides	Add 0.6 m			Add 0.3 m			Add 0.6 m		
Stabilized shoulder, one or both sides	Lane width for conditions B & C on tangent may be reduced to 3.6 m where shoulder is 1.2 m or wider			Deduct shoulder width(s); minimum pavement width as under Case I			Deduct 0.6 m where Shoulder is 1.2 m or wider		

Notes:
A = predominantly Passenger vehicles, but some consideration for Single Unit trucks
B = sufficient SU-9 vehicles to govern design, but some consideration for semitrailer combination trucks
C = sufficient bus and combination-trucks to govern design

3.3.12 Horizontal Clearance or Lateral Offset

Road side elements, such as bridge piers, barriers, traffic signs and signals, power poles, trees and other landscaping items should be placed away from the carriageway edge in order to their conflicts with car doors, side mirrors of large trucks and other similar actions. Minimum lateral offset from any vertical obstruction, for roadways without kerbs shall be 1.2 m from edge of carriageway and for kerbed roadways, a minimum lateral offset shall be 0.6 m from the face of the kerb. At the kerbed intersections and kerbed driveway openings, a minimum lateral offset from the face of the kerb to the vertical obstruction shall be 0.9 m (see **Figure 3.21**).

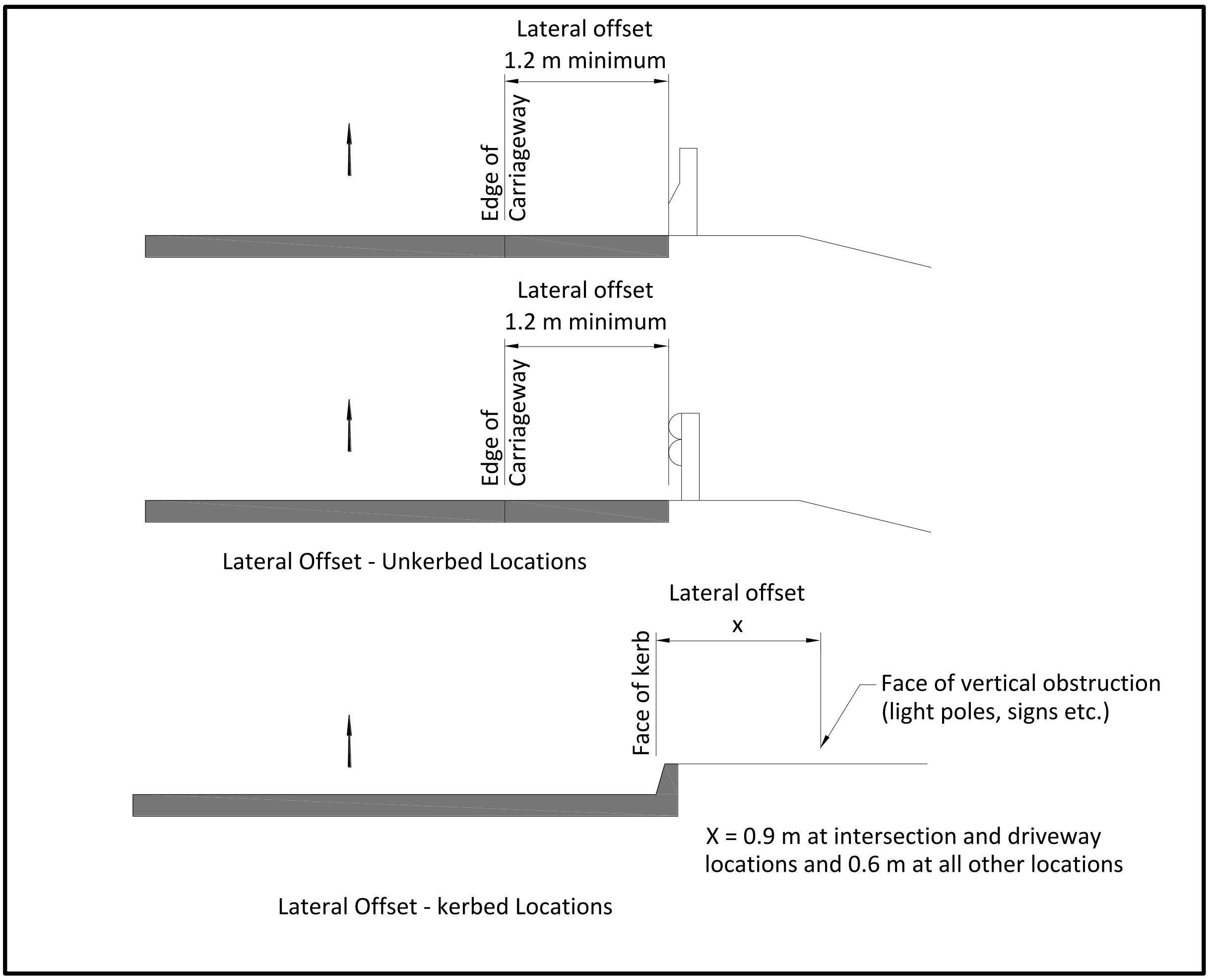


Figure 3.21: Horizontal Clearance or Lateral Offset

3.4 VERTICAL ALIGNMENT

3.4.1 General Considerations

Vertical alignment consists of a series of tangent longitudinal grades connected by vertical curves. Vertical curves are parabolic, with length symmetrical about the vertical point of intersection of the two tangents. Vertical alignment is also known as profile of road.

The design of the vertical curves depends on the design speed, and other factors like drainage, grades, and existing topography. To attain a consistent smooth flowing and aesthetically pleasing vertical alignment, consideration should be given to the following practices:

Design of vertical alignment is typically based on minimizing the total earthwork required for the roadway, providing drainage, and facilitating the operation of heavier vehicles

- Vertical alignment developed should be consistent and fit in to the topography.
- A “roller coaster” or “hidden dip” type of profile should be avoided.

- Broken-back vertical curves – two crest or two sag curves separated by a short tangent, less than 100 m, should be avoided.
- On long grades, it is desirable to place the steepest grades at the bottom and flatten the grades near the top of the ascent or to break the grade by short intervals of flatter grade instead of providing a uniform sustained grade that is only slightly below the recommended maximum.
- Where at-grade intersections occur on roadway sections with moderate to steep grades, the grade should be reduced through the intersection.
- Avoid sag or crest vertical curves at the superelevation transition areas of the horizontal curve.
- Avoid sag vertical curves in cut areas unless adequate drainage could be provided.
- In flat terrain, the elevation of the profile is often controlled by drainage. The profile should be developed such that adequate drainage can be provided.
- In areas where the surface water can be above the ground level or the groundwater table is immediately below the surface, the profile of the low edge of the finished shoulder should be at least 1.0 m above the water level.
- In areas of rock cut, if practical, the profile should be developed so that the low edge of the finished shoulder is at least 0.3 m above the rock level to avoid excess rock excavation.

3.4.2 Terrain

Variations in topography of the site affects both the horizontal and vertical alignments, but the impact of the topography on vertical alignments is more pronounced than on horizontal alignments. The variations in topography are classified into three terrain conditions: level, rolling and mountainous.

Vertical alignments can be designed with greater degree of flexibility (either long or short) without much impact on the construction cost.

The terrain in KP province comprises of level, rolling and mountainous and the guidance provided in this section is based on these three types.

In levelled terrain, highway sight distances are generally long (both horizontally and vertically) and can be achieved without major expenses or construction difficulty.

In rolling terrain, natural slopes consistently rise above and fall below road grade. The occasional steep slopes offer some sight restriction to horizontal and vertical alignment.

In mountainous terrain, there are abrupt changes in ground elevations with respect to the highway. To obtain acceptable horizontal and vertical alignment, benching and side hill excavation is normally required.

3.4.3 Longitudinal Grades

Vehicle operations, especially of trucks are greatly influenced by roadway longitudinal grades; therefore, it is important to ensure that the longitudinal grades provided comply with the maximum gradient for the design speed. The length of the gradient also should not exceed the critical lengths discussed in sections below. Relationships of grades and their lengths to design speed are discussed below.

3.4.3.1 Maximum Longitudinal Grades

Maximum longitudinal grades depend on the functional classification, terrain and design speed of the roadway. Wherever practical use flatter grades and avoid using maximum grades. In urban areas the grades should be as level as practical, consistent with the surrounding terrain. Where sidewalks are present, a maximum grade of 5 percent is recommended. The maximum gradients for different classification of roads are provided in **Table 3.32**.

Table 3.32: Maximum Grades in Percentage

Type of Road	Design Speed (km/h)	Freeway / Expressway		Arterial		Collector		Local		
		Rural	Urban	Rural	Urban	Rural	Urban	Rural	Urban	Industrial
LEVEL	130-110	3	3							
	100	3	3	3	5	5				
	90			4	5	6				
	80			4	6	6				
	70			5	6	7				
	60			5	7	7	9	7		
	50-40				8	7	9	7	15	8
	30					7		8	15	8
ROLLING	110-100	4	4	4	6					
	90			5	6					
	80			5	7	7				
	70			6	7	8				
	60			6	8	8	10			
	50				9	9	11	10	15	8
	40-30					10	12	11	15	8
MOUNTAINOUS	100-90	6	6		8					
	80-70	6	6	7	9					
	60			8	10	10	12			
	50				11	10	12		15	8
	40					11	13	15	15	8
	30					12		16	15	8

(Source: *AASHTO's A Policy on Geometric Design of Highways and Streets, 2011* [2])

3.4.3.2 Minimum Longitudinal Grades

To provide adequate longitudinal drainage, minimum longitudinal grade should be 0.5 percent for all classes of roads. Grades of 0.30 percent may be used where there is a paved surface accurately sloped and supported on firm subgrade. Flatter grades may be provided on unkerbed highways where cross slope is sufficient to drain the pavement surface. Particular attention should be given to the design of storm water inlets and their spacing to keep the spread of water on the carriageway within tolerable limits.

3.4.3.3 Minor Road Grades at Intersections

At intersections the cross slope of major road should be carried onto the minor road up to a distance depending on the grades and vertical design of minor road. The profile of the minor road will then follow the established design criteria of vertical design. Graphical representation of the profile on minor road is shown in **Figure 3.22** for guidance.

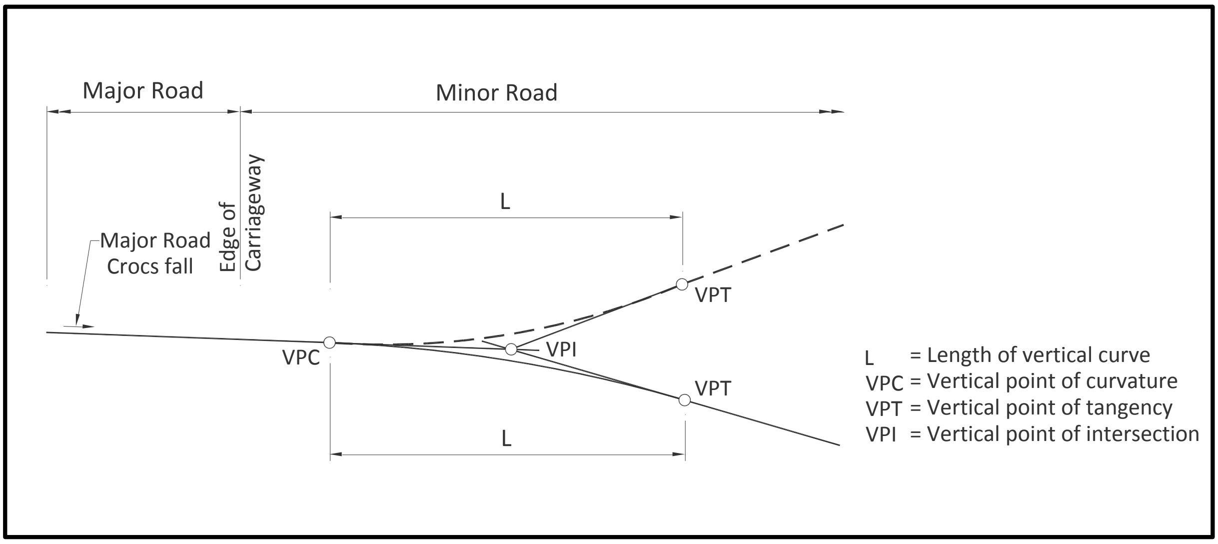


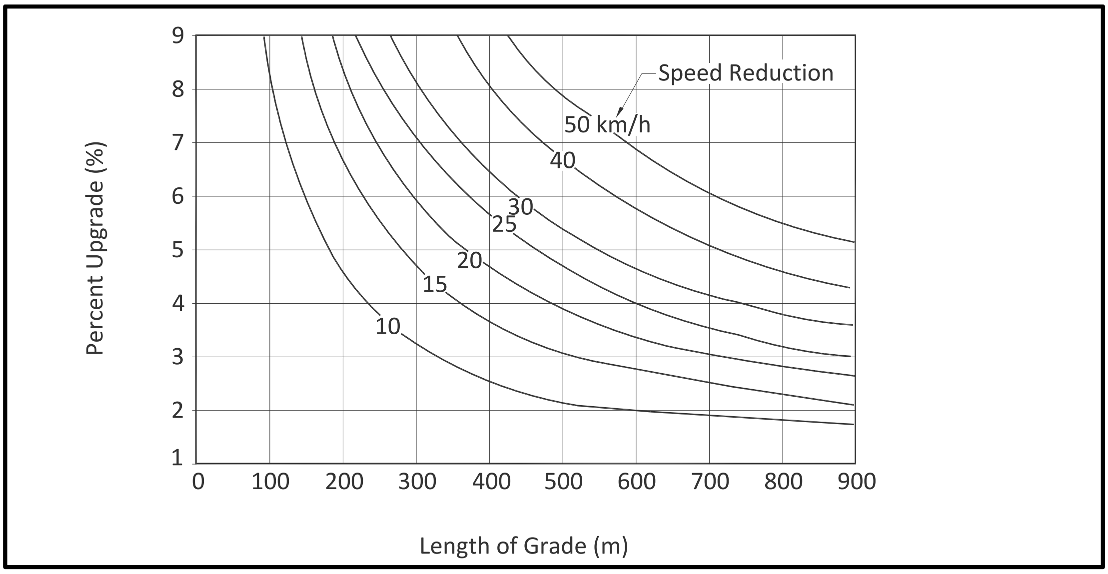
Figure 3.22: Minor Road Vertical Alignment Approach at Intersections

3.4.3.4 Critical Lengths of Grade for Design

In addition to maximum grade, the length of the grade in relation to required vehicle operations should also be considered. The maximum length of upgrade on which a loaded truck (120 kg/kW), entering speed of 110 km/h, can climb with the maximum 15 km/h reduction in speed from the average running speed is considered critical length of grade. The length of any grade that will cause the reduction in speed by various amounts of a representative truck is illustrated in **Figure 3.23**. The curve showing a 15-km/h speed reduction shall be used to determine the critical lengths of grade.

If the difference in grades is not too great, some of the length of vertical curve can be considered as part of the critical length. For vertical curve Types-II and IV (**Figure 3.26**), the critical length can be measured between vertical points of intersections (VPIs), whereas for Types-I and III (**Figure 3.26**), one quarter of the vertical curve length can be regarded as critical length.

Where reduction in grade length cannot be achieved below critical value, provision of climbing lane should be considered for slow-moving vehicles.



(Source: *AASHTO's A Policy on Geometric Design of Highways and Streets, 2011* [2])

Figure 3.23: Critical Lengths of Grade for Design, Assumed Typical Heavy Truck of 120 kg/kW, Entering Speed = 110 km/h

3.4.4 Climbing Lanes

Where reduction in grade length cannot be achieved below critical value, consideration should be given to the provision of climbing lanes. It is an auxiliary lane provided to remove slow moving vehicles, especially trucks, from mainstream of traffic. Climbing lanes are independently designed for each direction. They may or may not overlap depending on the profile. A typical illustration of climbing lanes is shown in **Figure 3.24**.

3.4.4.1 Warrant for Climbing Lanes

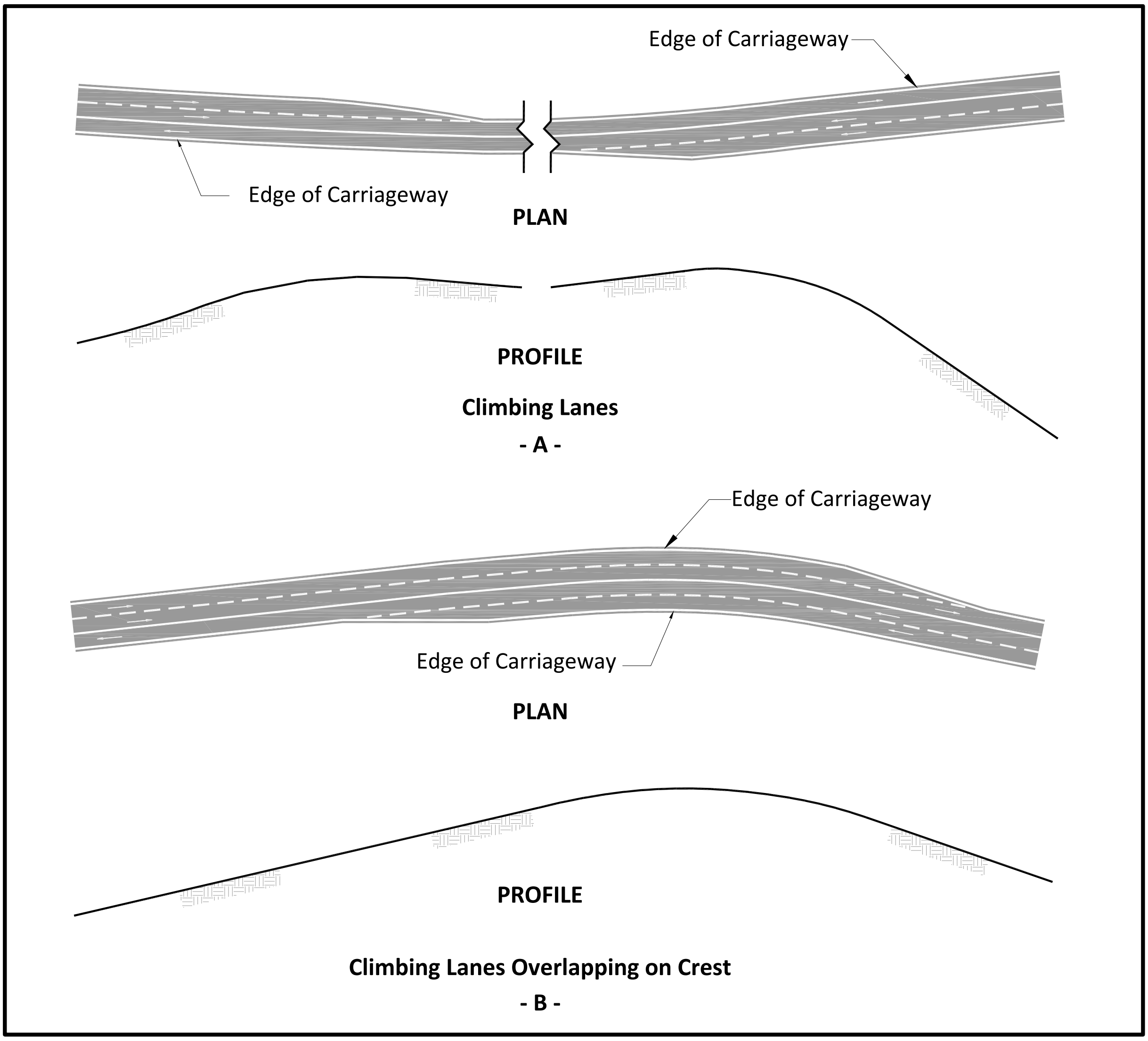
When the length of grade exceeds the critical length of grade, the following criteria should also be satisfied (due to economic considerations) before deciding on whether to provide climbing lane or not:

1. Traffic flow rate exceeds 200 vph.
2. Truck flow rate exceeds 20 vph.
3. The level of service is reduced by two or more levels on grade.

3.4.4.2 Climbing Lanes for Two-Lane Highways

The width of the climbing lanes should be the same as the width of the adjacent lane. Shoulders should be tapered to allow for escape of merging vehicle in case of hindrance to merging to the main lane due to presence of traffic in the lane. Climbing lanes should not merge in curves wherever possible.

The location of the beginning of an added lane depends on the approach speed of trucks on the grade and the sight distance restrictions on the approach. Where there are no sight distance restrictions, the added lane may be introduced on the upgrade beyond its beginning. The climbing lane can begin at a distance from the bottom of the grade to the point where truck speeds reduces by 15 km/h below the average running speed. This distance can be determined from **Figure 3.23**.



(Source: *AASHTO's A Policy on Geometric Design of Highways and Streets, 2011* [2])

Figure 3.24: Climbing Lanes on Two-Lane Highways

Where there are sight restrictions, upgrade approaches, or other conditions that reduces the speeds of approaching trucks, the added lane should be introduced near the foot of the grade. The added lane should be preceded by a tapered section of at least 90 m.

The climbing lane should extend to a point beyond crest vertical curve. The idea is that the truck could attain a speed that is within 15 km/h limit of the average running speed and merge into mainline without much interference with other traffic. The sight distance should also be sufficient to allow passing. The end of climbing lane should be at least 60 m beyond the point where passing sight distance is achieved. A taper length of at least 180 m should follow the lane to allow smooth merge to normal lane.

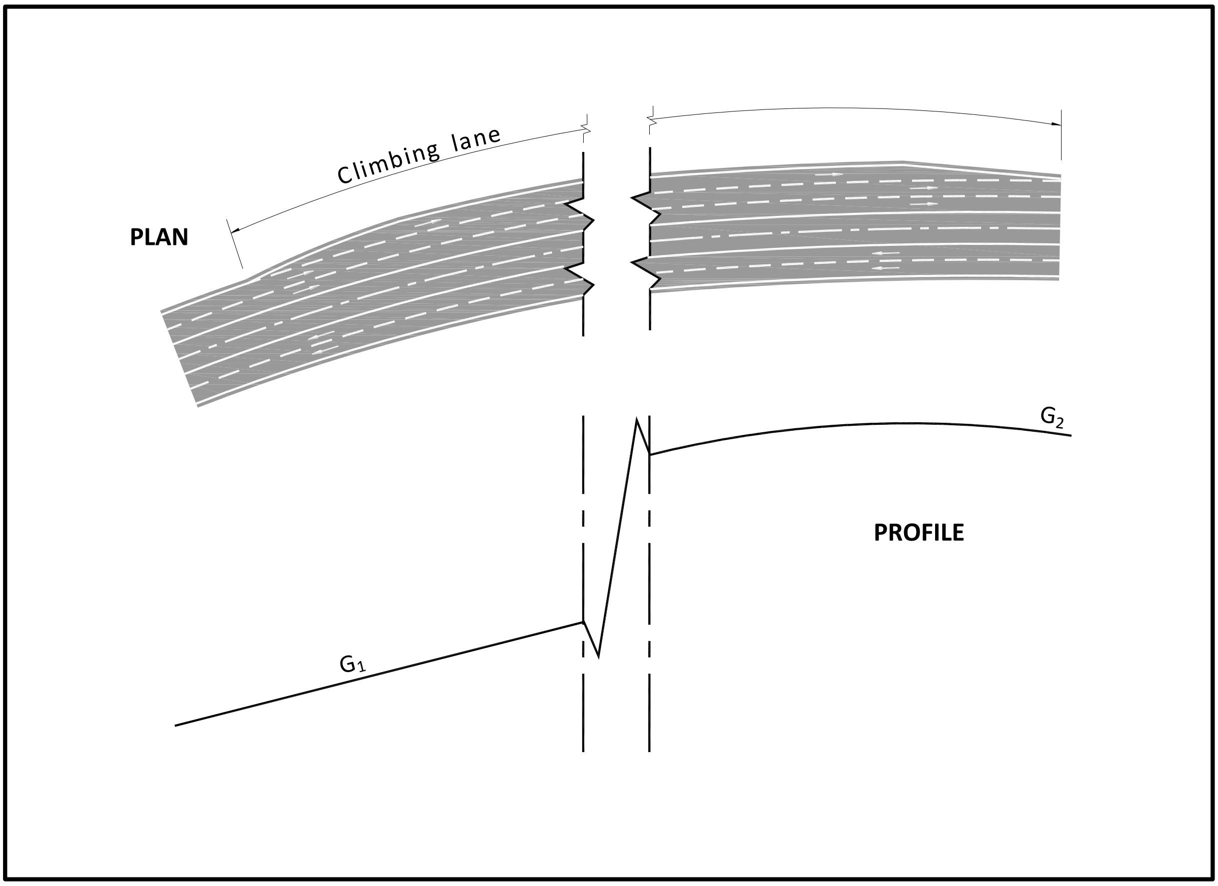
Proper signage and marking should be provided as per guidelines provided in *Manual of Uniform Traffic Control Devices*, *MUTCD* [1]. The cross slope of the climbing lane should be handled in the same manner as the mainline.

3.4.4.3 Climbing Lanes on Freeways and Multi-lane Highways

Climbing lanes are not as easily justified on multilane facilities as on two-lane highways, because on multilane facilities, there is no hindrance to passing. The main factors to determine the need for climbing lanes on multi-lane highways are critical lengths of grade (discussed previously), effects of trucks on grades, and service volumes for the desired level of service and the next reduced level of service. If the directional traffic volume is not equal to or greater than the service volume for level of service D, the climbing lanes should not be considered. As a guideline, if the service volume is greater than 1,700 vph per lane, the length of grade more than critical length and trucks percentage is sufficient then climbing lanes may be warranted. Economically, an increase in the number of lanes throughout the highway section is a better investment than the provision of climbing lanes on multilane highways.

Design of Climbing Lanes

Climbing lanes on multi-lane roads should be placed on the outer or left-hand side of the roadway as shown in **Figure 3.25.** Other design guidelines for multi-lane highways relating to cross slopes, location of terminal points and tapers for climbing lanes are similar to climbing lanes on two lane highways, and are already discussed. There is no need to consider passing sight distance on multi-lane highways.



(Source: *AASHTO's A Policy on Geometric Design of Highways and Streets, 2011* [2])

Figure 3.25: Climbing Lane on Freeways and Multi-lane Highways

3.4.5 Vertical Curves

Vertical curves shall be provided at all changes in gradients, except at junctions and on lower classes of roads where change in grade in less than 0.5 percent. The curvature of vertical curve should be designed to provide at least stopping sight distance shown in **Table 3.1** and comfort. Wherever practical, longer vertical curves should be used. Vertical curves in highways are parabolic. **Figure 3.26** illustrates types of vertical curves and their parameters.

There are two types of vertical curves used in highway design, crest and sag. The recommended minimum rate of curvature K for both types of vertical curves corresponding to the stopping sight distance and passing sight distance are provided in **Table 3.33** for each design speed. The vertical curves on dual carriageway roads shall be designed using the curvature value for stopping sight distance. For single carriageway, where passing is allowed on horizontal alignment, the crest curves should be designed using the k value for passing sight distance. If the horizontal alignment does not permit passing, due to roadside obstructions to visibility such as walls, retaining walls, cut slopes, buildings, and barriers, then there is no need to provide for passing sight distance. At locations where decisions are required, such as ramp exit gores, longer vertical curves should be provided to cater for decision sight distance. Refer Section 3.2.3 on Decision Sight Distance.

Table 3.33: Design Controls for Crest Vertical Curves Based on Stopping Sight Distance

Design Speed (km/h)	Stopping Sight Distance (m)	Rate of Vertical Curvature for Stopping Sight Distance, K [a]		Passing Sight Distance (m)	Rate of Vertical Curvature for Passing Sight Distance, K [a]
		Crest	Sag		Crest
20	20	1	3	-	-
30	35	2	6	120	17
40	50	4	9	140	23
50	65	7	13	160	30
60	85	11	18	180	38
70	105	17	23	210	51
80	130	26	30	245	69
90	160	39	38	280	91
100	185	52	45	320	119
110	220	74	55	355	146
120	250	95	63	395	181
130	285	124	73	440	224

[a] Rate of vertical curvature, K, is the length of curve per percent algebraic difference in intersecting grades (A), K = L/A.
(Source: *AASHTO's A Policy on Geometric Design of Highways and Streets, 2011* [2])

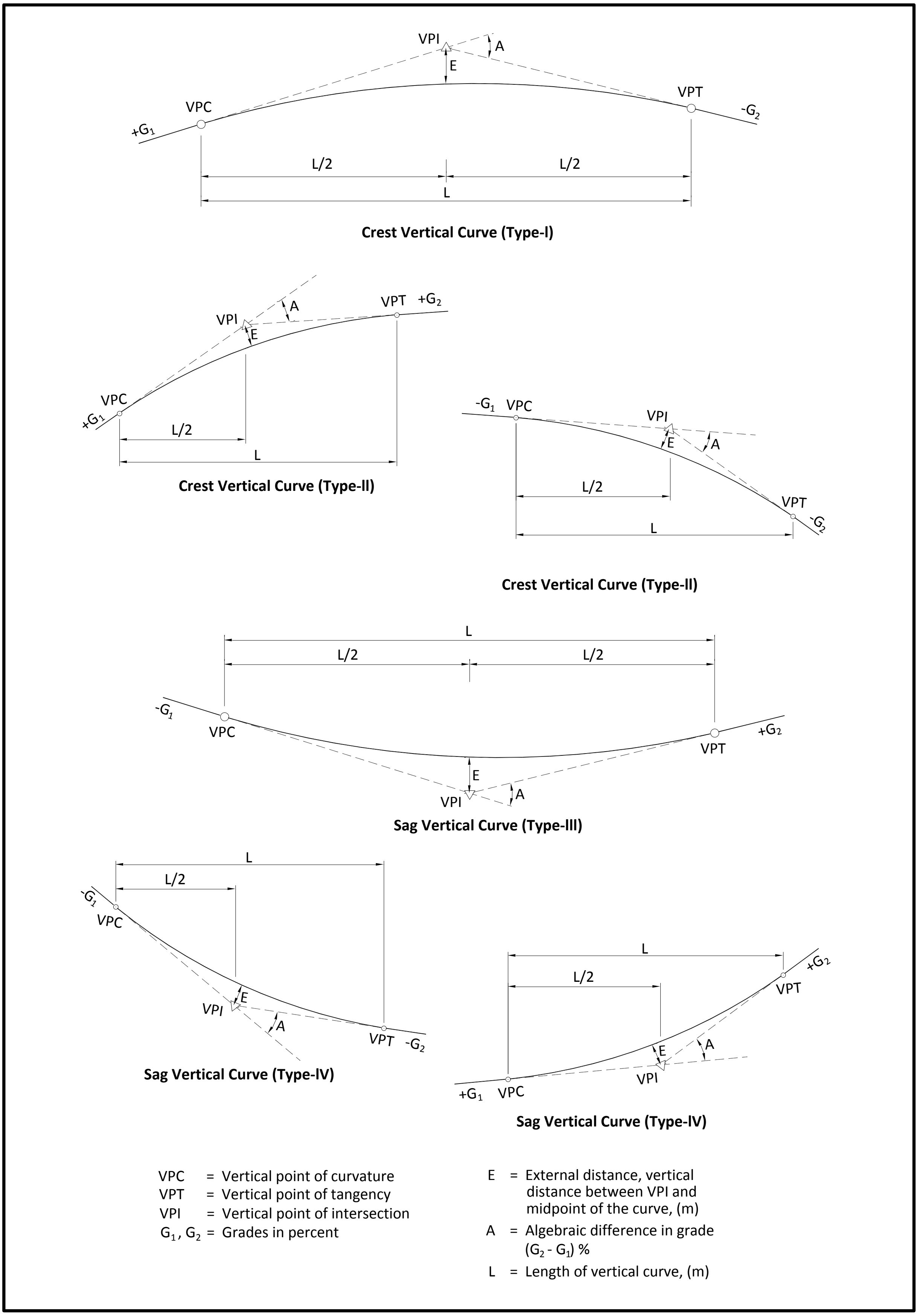


Figure 3.26: Vertical Curve Elements

Vertical curves lengths are related to rate of vertical curvature K and the algebraic difference between gradients A. The length can be calculated by the formula:

$$L = KA$$ ***Equation 3.19***

Where,

K = rate of vertical curvature

L = length of vertical curve, (m)

A = algebraic difference in longitudinal grades, ($G_2 - G_1$) (percent)

G_1, G_2 = longitudinal grades, (percent)

Where K-value for crest curves exceeds 51, drainage should be more carefully designed near high point of crest vertical curves. Kerbs may be removed near high point to provide adequate transverse drainage. Same criteria apply to sag vertical curves.

3.4.5.1 Minimum Length of Vertical Curve

Calculated lengths for both crest and sag vertical curves using previous section should be checked against the minimum curve lengths and the higher of the two values should be used. As per AASHTO's recommendations, the minimum curve lengths should be 0.6 times the design speed in km/h.

$$L_{min} = 0.6V$$ ***Equation 3.20***

Where,

L_{min} = Minimum length of vertical curve, (m)

V = Design speed, (Km/h)

For improved appearance of sag vertical curves, curve length of 30A or K = 30 may be used as minimum. On high-type highways, longer curves are appropriate to improve appearance.

3.4.5.2 Sight Distance at Undercrossing

Structural façade may restrict the sight line and limit the sight distance while passing under a structure on sag vertical curve as shown in **Figure 3.27**. Thus the sight distance undercrossing should be checked for stopping (passing for two lane roads). Preferably longer curves lengths than minimum should be provided.

Using an eye height of 2.4 m for a truck driver and an object height of 0.6 m for the taillights of a vehicle, the following equations for sag vertical curve length at undercrossings can be derived:

Case 1 - Sight distance greater than length of vertical curve (S > L):

$$L = 2S - \frac{800\,(C - 1.5)}{A}$$ ***Equation 3.21***

Case 2 - Sight distance less than length of vertical curve (S < L):

$$L = \frac{AS^2}{800\,(C - 1.5)}$$ ***Equation 3.22***

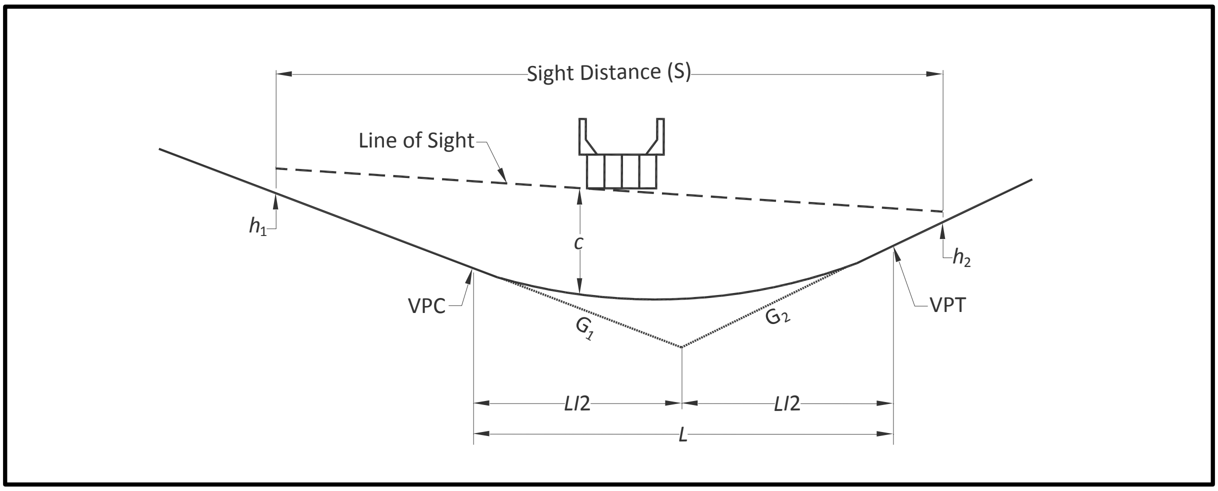
Where,

L = length of vertical curve, (m)

S = sight distance, (m)

C = vertical clearance, (m)

A = algebraic difference in grades, (percent)



(Source: *AASHTO's A Policy on Geometric Design of Highways and Streets, 2011* [2])

Figure 3.27: Sight Distance at Undercrossing

3.4.6 Vertical Clearances

Vertical clearance is typically determined for an entire route and may be governed by the established policies of the highway system. The vertical clearance of all structures above the carriageway and shoulders should be at least 0.3 m greater than the legal vehicle height, and allowance should be made for future resurfacing. Additional vertical clearance is desirable to compensate for snow or ice accumulation and for an occasional slightly over height load.

Guidance may be sought from the Overseeing Organization regarding the vertical clearances when planning work near electricity and water installations.

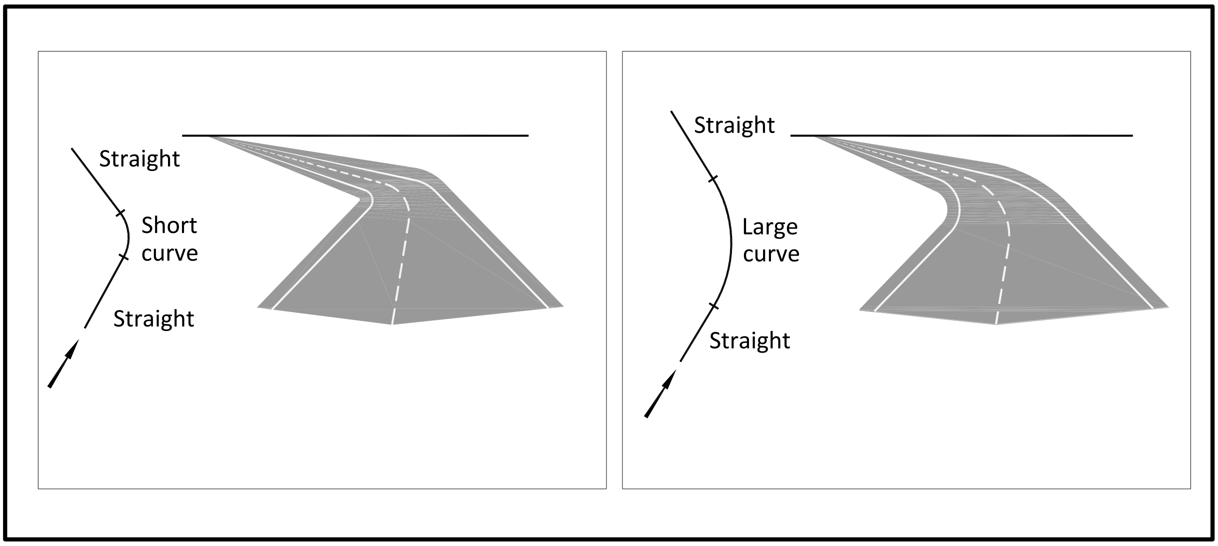
Road designers must work with bridge to provide the necessary vertical clearances across all traffic lanes including shoulders.

3.5 Combinations Of Horizontal And Vertical Alignment

Vertical and horizontal alignment should be in coordination so that the driver can see the road flowing smoothly and without any visual defects. The proper combination increases safety improves appearance and encourages uniform speed with no additional cost. Horizontal and vertical alignment integration should start at the early design stage.

3.5.1 Harmonizing the Horizontal Alignment

When two straight road segments are connected, the use of a short horizontal curve is likely to cause the appearance of a kink, as shown in **Figure 3.28**, which can be improved by employing a larger radius.



(Source: *AASHTO's A Policy on Geometric Design of Highways and Streets, 2011* [2])

Figure 3.28: Example of a Kink and Improvement with Larger Radius

Smooth-flowing alignments are required for sustaining posted speeds. The following are the principles to be followed in securing a satisfactory alignment:

- Care should be taken to ensure that embankments and cuttings do not make severe breaks in the natural skyline. This can be achieved by designing the road on a curve whenever possible to preserve an unbroken background.
- Short curves and tangents should not be used. Adjacent curves should be similar in length (See **Figure 3.30**, Sketch "L").
- Small changes of direction should not be made, as they give the perspective of the road ahead a disjointed appearance (See **Figure 3.28**).
- Curves of the same or opposite sense, which are visible from one another, should not be connected by short tangents. It is better to introduce a flat curve between curves of the same sense, or to extend the transition curves to a common point between curves of the opposite sense, see **Figure 3.29**, Sketch "D".
- Changes in horizontal and vertical alignment should be phased to coincide whenever possible, see **Figure 3.30**, Sketches "I" and "J".
- Flowing alignment can most readily be achieved by using large radius curves rather than tangents, see **Figure 3.30**, Sketch "G".
- The profile of the road over bridges should form part of the easy flowing alignment, see **Figure 3.29**, Sketch "B".
- At the start of horizontal curves, superelevation should not create large flat areas on which water would stand, see **Figure 3.29**, Sketch "D".
- Horizontal and vertical curves should be made as generous as possible at interchanges to enhance sight distance.

- Sharp horizontal curvature should not be introduced at or near the top of a pronounced crest. This is hazardous especially at night, because the driver cannot see the change in horizontal alignment; see **Figure 3.29**, Sketch "D".
- The view of the road ahead should not appear distorted by sharp horizontal curvature introduced near the low point of a sag curve, see **Figure 3.31**, Sketch "M".

Typical alignment and profile relationships are shown in Section-3.5.3 of this chapter.

3.5.2 Harmonizing the Vertical Alignment

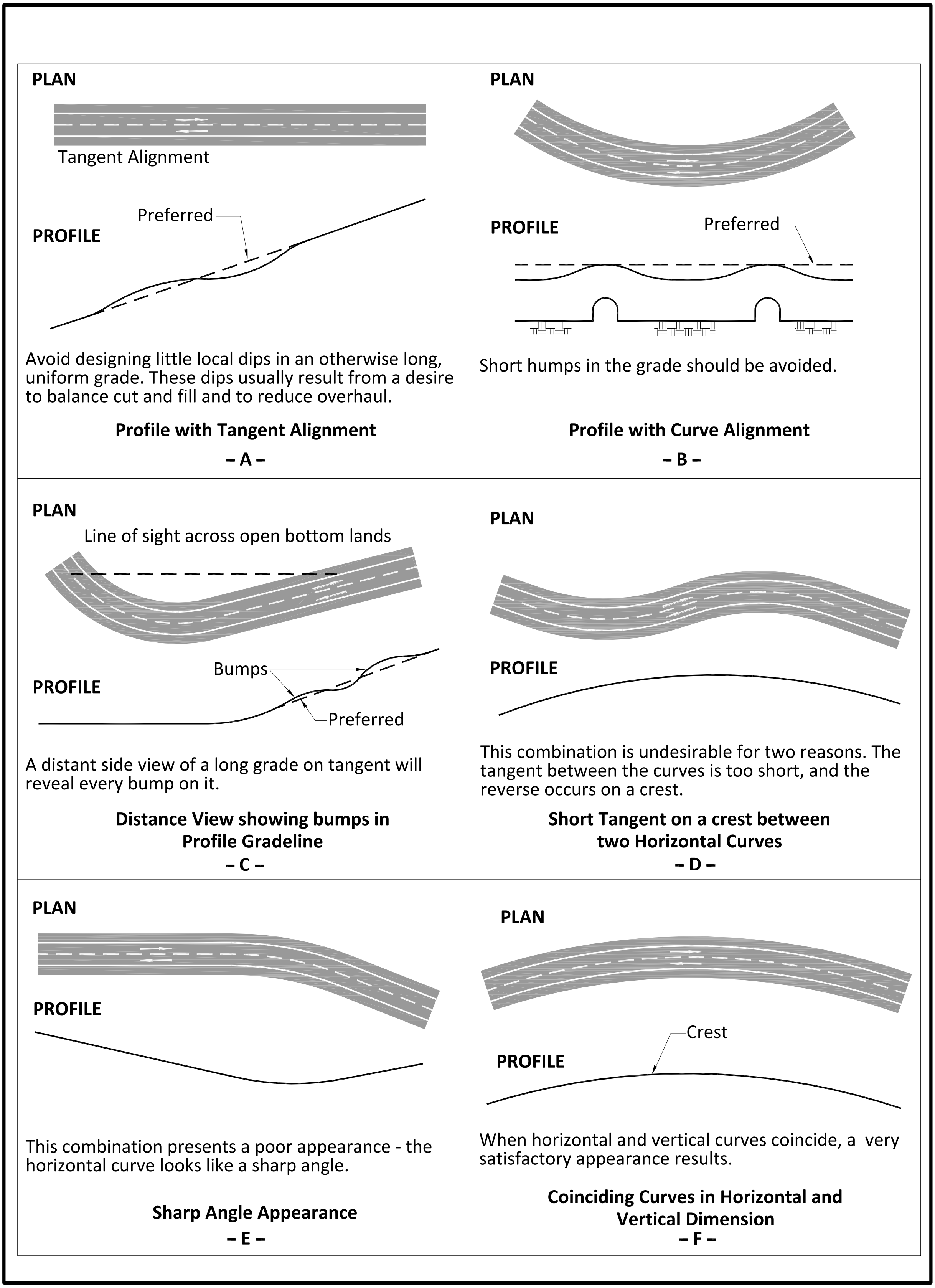
Vertical curvature plays an equally important part in achieving harmonious alignment as horizontal geometry. Inappropriate combinations of vertical curves and gradients can lead to a disjointed appearance that should be avoided. The following undesirable combinations of vertical elements should be avoided:

- A short crest or sag curve between two grades, see **Figure 3.32**, Sketch "R".
- A short grade between crest curves or sag curves, see **Figure 3.32**, Sketch "P".
- Reverse vertical curves causing small changes in height on level or near level sections of road, see **Figure 3.29**, Sketch "B".
- A consistent gradient containing either a shallow sag curve or a shallow crest curve, see **Figure 3.32**, Sketch "Q".
- Terracing on which two crest curves can be seen at one time, see **Figure 3.32**, Sketch "S".

As is the case with horizontal curves and tangents, vertical curve radii must be large enough to avoid the appearance of a kink and visual discontinuities.

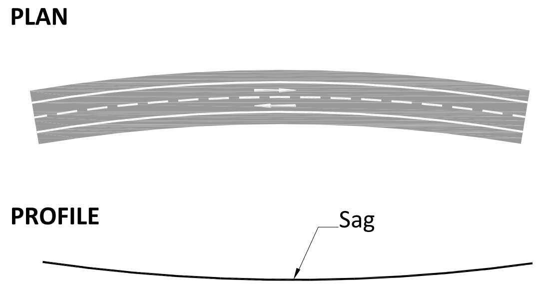
3.5.3 Phasing of Horizontal and Vertical Alignments

The corresponding elements of horizontal and vertical alignment should start and end at approximately the same points. The phasing of both alignment can be checked and analysed by printing them in the same roll of sheets. The visualization software can also be used to examine the appearance of design. The combined use of horizontal and vertical curvature of large curvature provides the best appearance. The following are some additional examples of horizontal and vertical combination, which results in awkward appearance and should be avoided. Some examples of poor and good practice are shown in sketches **Figure 3.29** to **Figure 3.32**.



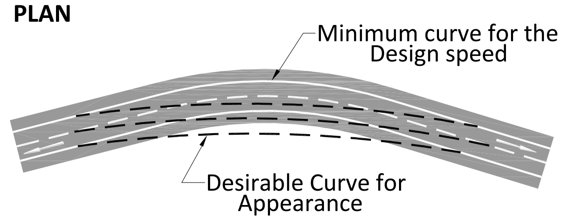
(Source: *AASHTO's A Policy on Geometric Design of Highways and Streets, 2011* [2])

Figure 3.29: Alignment Relationships in Roadway Design — 1 of 4



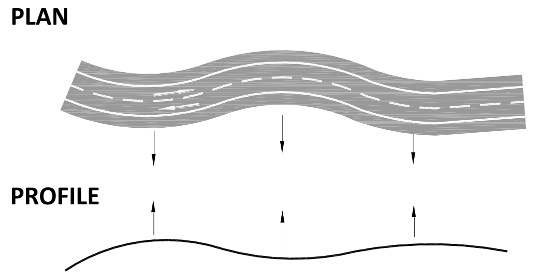
When horizontal and vertical curves oppose, a very satisfactory appearance results.

Opposing Curves in Horizontal and Vertical Dimensions
– G –



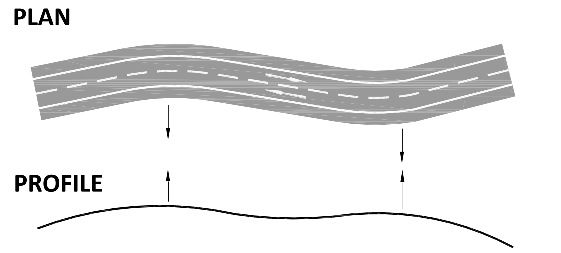
Very long flat curves, even where not required by a design speed and regardless of profile, also have a pleasing appearance when the central angle is very small.

Horizontal Alignment with Small Central Angles
– H –



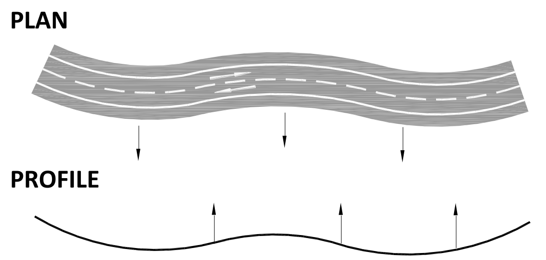
The classic case of coordination between horizontal and vertical alignment in which the vertices of horizontal and vertical curves coincide, creating a rich effect of three-dimensional S-curves composed of conves and concave helixes.

Coinciding Vertices in Horizontal and Vertical Dimensions
– I –

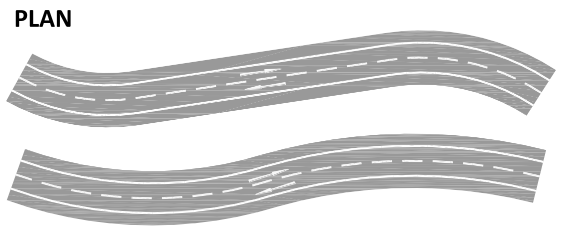


A legitimate case of coordination: one phase is skipped in the horizontal plane, but vertices still coincide. The long tangent in plan is softened by vertical curvature.

Coinciding Verticles with Single-Phase Skip
– J –



Weak Coordination of Horizontal and Vertical Alignment
– K –

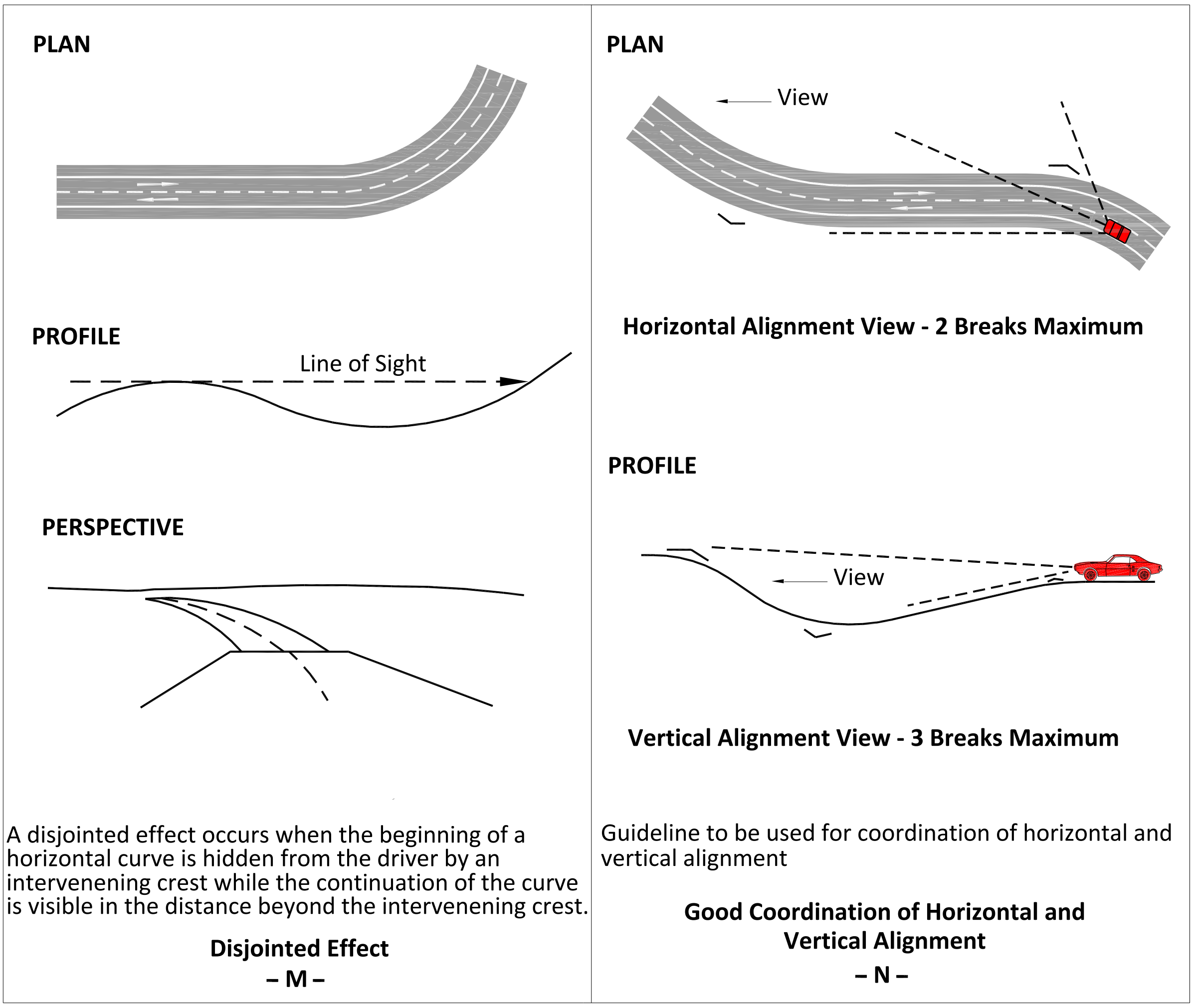


The upper line is an example of poor design because the alignment consists of a long tangent with short curves, whereas the balance between the curves and tangents in the lower alignment is the preferred design.

Horizontal Alignment should be balanced
– L –

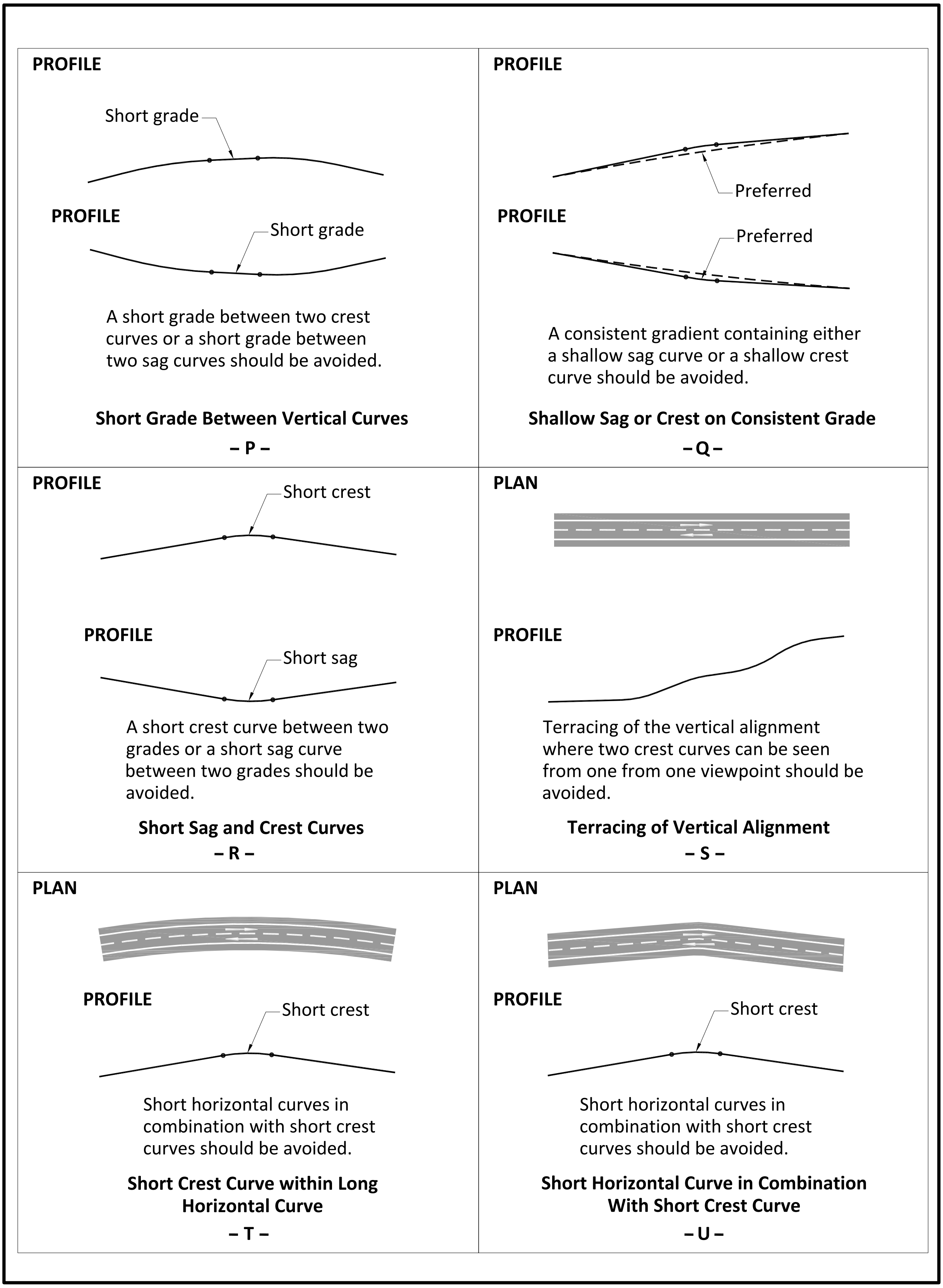
(Source: *AASHTO's A Policy on Geometric Design of Highways and Streets, 2011* [2])

Figure 3.30: Alignment Relationships in Roadway Design — 2 of 4



(Source: *AASHTO's A Policy on Geometric Design of Highways and Streets, 2011* [2])

Figure 3.31: Alignment Relationships in Roadway Design — 3 of 4



(Source: *AASHTO's A Policy on Geometric Design of Highways and Streets, 2011* [2])

Figure 3.32: Alignment Relationships in Roadway Design — 4 of 4

4 AT-GRADE INTERSECTIONS AND ROUNDABOUTS

4.1 INTRODUCTION

An area where two or more highways meet or cross each other including carriageway and roadside facilities is called an intersection or more commonly known as junction. Each road branching out of intersection is called an intersection leg. The most common type of intersection in Khyber Pakhtunkhwa is four-legged intersection. Intersections with more than four legs are not recommended.

The main objective of intersection design to provide safe passage for all transportation modes traversing the intersection, minimizing congestion and delays, minimizing fuel consumption, noise and air pollution. In addition, convenience, comfort, and efficient movement of vehicles and pedestrians travelling through the intersection are also the added objectives.

4.2 GENERAL CONSIDERATIONS

Following guidelines should be considered when locating and designing the intersection:

1. Most traffic accidents occur on at intersections with low traffic volume, therefore they require more attention while planning and designing as compared to intersections with heavier traffic volume.
2. Good intersection design allows transition through the intersection with minimum delay and provides maximum safety.
3. Intersections should not be located where it is difficult or expensive to provide adequate visibility or driving comfort.
4. The layout and operations of the intersection should provide clear visibility to all the conflicting movements through the intersection to the approaching driver.
5. The number of intersections should be kept as low as possible depending on the traffic demand.
6. The intersection on crests, high gradients and sharp horizontal curves should be avoided. The design of such intersections is complex due to introduction of superelevation. Minimum Sight distances are also difficult to attain.
7. If intersection on horizontal curves is inevitable, T-intersection on the outside of a curve provides visibility than those that are located on the inside of a curve.
8. Approach grades on intersection should not be greater than 2 percent.
9. Approach alignment should be as straight as possible If the intersecting angle is less than 60 degrees, attempt should be made to realign the intersecting roads to be as near 90 degrees as possible.

10. The profile of major road should be carried through the intersection, and the crossroad profile should be adjusted accordingly. If both intersecting roads are of same priority, then profiles of both roads should be carried through the intersection.

4.3 Types of Intersection

The basic types of intersections are:

- Three-leg (T or Y) Intersections
- Four-leg Intersections
- Multi-leg Intersections
- Roundabouts

Additional variants include staggered intersections in which two adjacent T-intersections function similar to a four-leg intersection, and indirect intersections that provide one or more of the intersection movements at a location away from the primary intersection.

Based on type of control use, at-grade intersections can be classified into the three categories, descriptions of which are provided **Table 4.1**.

Table 4.1: Basic Forms of Intersection Types

Intersection Type	Traffic Control		Description
	Major Road	Minor Road	
Priority Intersection	No Control	Stop or Give Way (yield) Control	Priority intersections may take various forms, depending on the number of links and their configuration.
Roundabout	All approaches operate on Give Way (yield) Control, Occasionally signal controlled		At-grade intersection incorporating a circulatory roadway around a central island. Roundabouts treat all legs equally, providing more capacity for minor road traffic, but impose delay and lower speeds on the major road.
Signalized Intersection	Signal Controlled		At-grade intersection where conflicting movements are separated over time by a signal control that allocates right-of-way in an alternating and regular pattern.

The intersection type is determined by the number of intersecting legs, functional classification of intersecting roads, topography, traffic volumes, speeds, and the desired operation of traffic. The type of intersections can vary greatly based on scope, shape, flaring and degree of channelization.

4.4 Selection of Intersection Type

The key purpose of intersection is to enable the safe and efficient transfer of vehicles through the intersection. The selection of the most appropriate type of intersection requires consideration of the following factors:

- Functional classification of the intersecting roads
- The design year traffic demand
- The terrain and topography
- Existing and planned land use

- The needs of non-motorized users, such as pedestrians and cyclists

Traffic Flows and Capacity

Consideration should be given to the expected traffic flows in current year and future traffic demand. The composition of turning movement of traffic influences the geometric layout of the intersection.

The maximum capacity of each intersection type differs. Appropriate intersection type changes as the traffic volume increases. **Figure 4.1** shows the decision making process of intersection selection for both rural and urban conditions.

- Priority intersections enable major road traffic to proceed through the intersection with no delay. As the volume increases, delays to turning traffic also increase. They maintain high quality of service on multi-lane roads by prohibiting right-turn movements from minor roads.
- Roundabouts have limited traffic capacity. They provide full turning and access capability to all approaches equally.
- Signalized intersections are the highest capacity, at-grade intersections. Their capacity depends on the number of through lanes, turning lanes and signal phasing and timing.

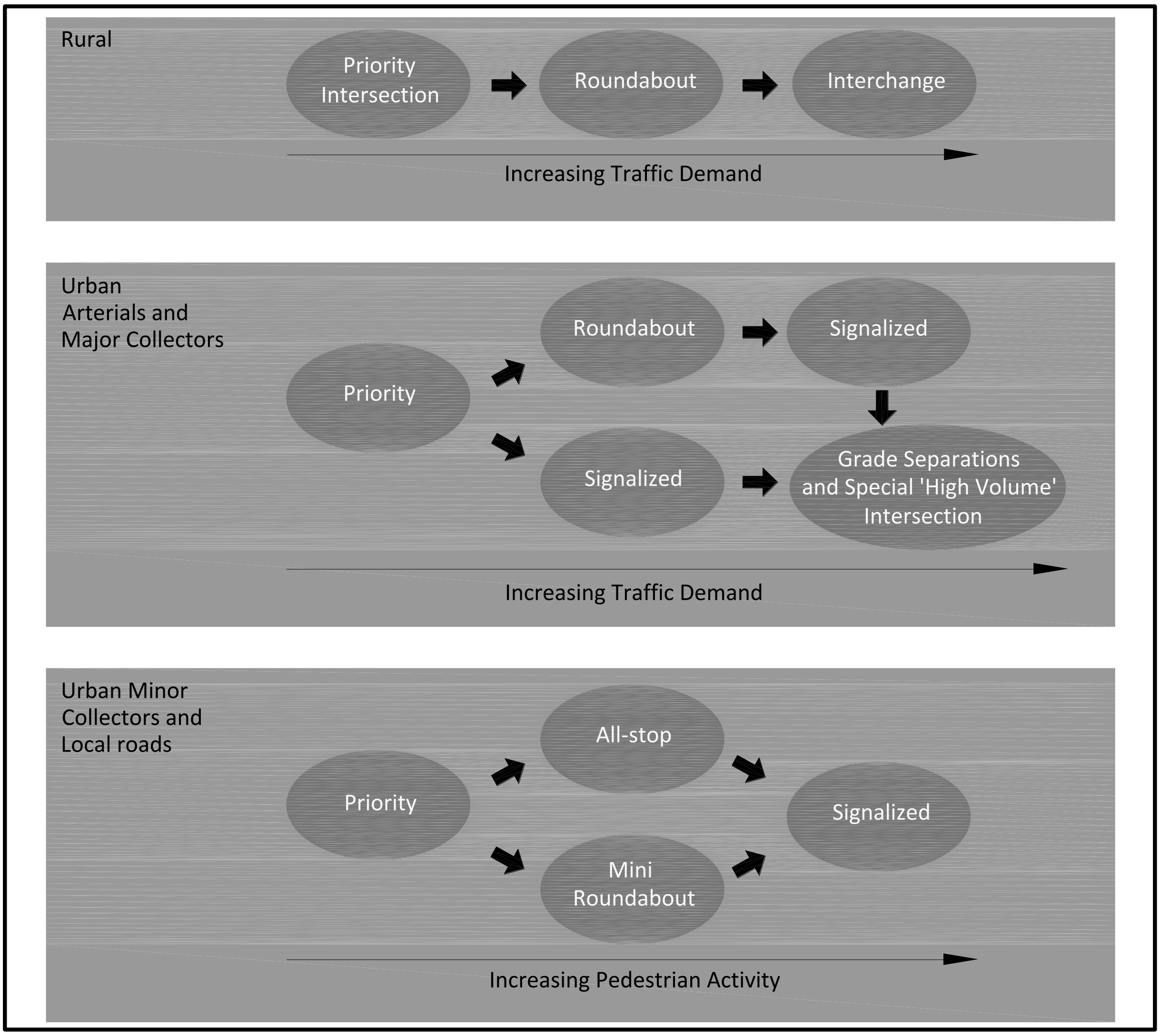


Figure 4.1: Progression of Decision Making in Intersection Type Selection

Route Designation

Based on the classification of intersecting roads, **Table 4.2** can be used as a guideline for the selection of type of intersection.

Table 4.2: Permitted Intersection Types on Urban and Rural Roads

Functional Classification		Priority Intersection	Roundabout	Signalized Intersection
URBAN	Expressway	x	x	x
	Major Arterial	✔	x	✔
	Minor Arterial	✔	x	✔
	Collector	✔	✔	✔
	Service Road	✔	✔	✔
	Local Road	✔	✔	✔
RURAL	Freeway	x	x	x
	Arterial	✔	✔	x
	Collector	✔	✔	x
	Local Road	✔	✔	x

Following are some additional factors that need consideration while selecting the type of intersection to be provided:

Operational Quality: Minimizing delay and providing high level of service is key to intersection type selection. The access and mobility of pedestrians and bicyclists is also an important factor regarding operational quality of intersection. Intersection design should reflect community needs and accommodate these users.

Safety Performance: Each type of intersection has different levels of risk (frequency, severity, and types of crashes) associated. While selecting an appropriate intersection type, the expected safety performance should also be considered.

Local Conditions: The land use, topography, natural and manmade constraints, proximity to adjacent intersections of different types, and need for access to adjoining properties also influences the selection of appropriate type of intersection.

Overview of Operational and Design Trade-offs: In most cases, the designer will have choices among the basic forms of permitted intersections. Selection of the best intersection type will involve a site-specific analysis of the trade-offs. **Table 4.3** is intended to assist the designer in review and consideration the most important objectives, integral attributes of each type, and typical costs and challenges associated with implementation.

Table 4.3: Summary of Basic Trade-offs Among Intersection Types

	Costs and Adverse Effects of Implementation								Traffic Operations and Safety			
	Reduce or Eliminate High-speed Crossing conflicts	Favour Higher Volume, higher class roadway	Provide for all movements in all directions	Minimize Total Delay	Accommodate non-motorized Users	Provide greatest total throughput or capacity	Minimize stopping, idling and resultant fuel consumption and emissions	Total Footprint	Operations and Maintenance Cost	Construction Cost	Effect on Access for properties adjacent to intersection	Visual Impacts
Priority Intersection	×	×	×	✔	O	×	✔	✔	✔	✔	✔	✔
Roundabout	✔	O	✔	O	×	O	✔	O	✔	O	O	✔
Signal Controlled Intersection	x	O	✔	×	✔	✔	×	O	O	O	O	O
Grade Separated Interchange	✔	✔	✔	✔	×	✔	✔	×	×	×	×	×

Notes:
✔ Performs best   O Performs moderately successfully
× Performs worst

4.5 PRIORITY INTERSECTION

4.5.1 Priority Intersection Types

At a priority intersection, the major road is priority road on which the traffic flow is continuous and uncontrolled by any traffic control. The traffic on minor road is controlled by either Give way (Yield) control or stop control.

All figures in this section are drawn with yield control, and can be replaced by stop control based on desired traffic operations. Stop control are only shown in figures of those intersections where it is specifically required.

At locations where there is continuous presence of pedestrians, well-marked pedestrian crossings with drop kerbs should be provided. Raised tables can also be provided on low speed urban locations to provide level crossings to pedestrians and ensure slow speed of traffic. The details of pedestrian crossings and related features are discussed in Chapter 6-ROADSIDE/HIGHWAY FACILITIES of this manual.

There are five basic types of priority intersections:

- Three-leg or T-type intersection
- Four-leg or crossing intersection

- Staggered intersection
- Skew or Y-type intersection
- Left-in, Left-out for divided highway

4.5.2 Priority Intersections for Two-Lane Undivided Major Roads

4.5.2.1 Three-Leg or T-Type Intersection

A T-intersection is the simplest form of intersection. The intersecting angle shall be between 70 to 110 degrees (i.e. the skew of no more than 20 degrees). This design is suitable for intersections on two-lane rural roads with small amount of turning traffic. It is also an appropriate for most local access roads in urban areas, where traffic flows are low even after full development of the area.

T-intersections may be un-channelized, partly channelized or channelized. Channelization can be done by introduction of either Ghost Islands or physical islands.

The **un-channelized** design is the simplest form of T-intersection that has no traffic islands. They are suitable at intersections where the turning traffic is very small. For moderate turning traffic volume, a traffic island can be introduced in minor road (**partly channelized**) to improve turning manoeuvres. In urban areas, the traffic island can be kerbed to provide refuge for pedestrians crossing the road. The **fully channelized** design has islands on both minor and major road. They are suitable when the intersection has high volume of turning traffic.

Roadway widths on two lane major roads, after the introduction of physical island, shall provide for passing a stalled vehicle. 4 m lane width with 1 m offset to median island shall be sufficient for a rural highway with paved shoulder. For urban kerbed roadway, 4 m lane width with 1 m offset to kerb on both side of the road shall be sufficient.

For high-speed roads, with design speed greater than 80 km/h, it is desirable to provide a deceleration lane for left turning traffic from major road. **Figure 4.2** to **Figure 4.5** illustrates different types of three-leg priority intersections.

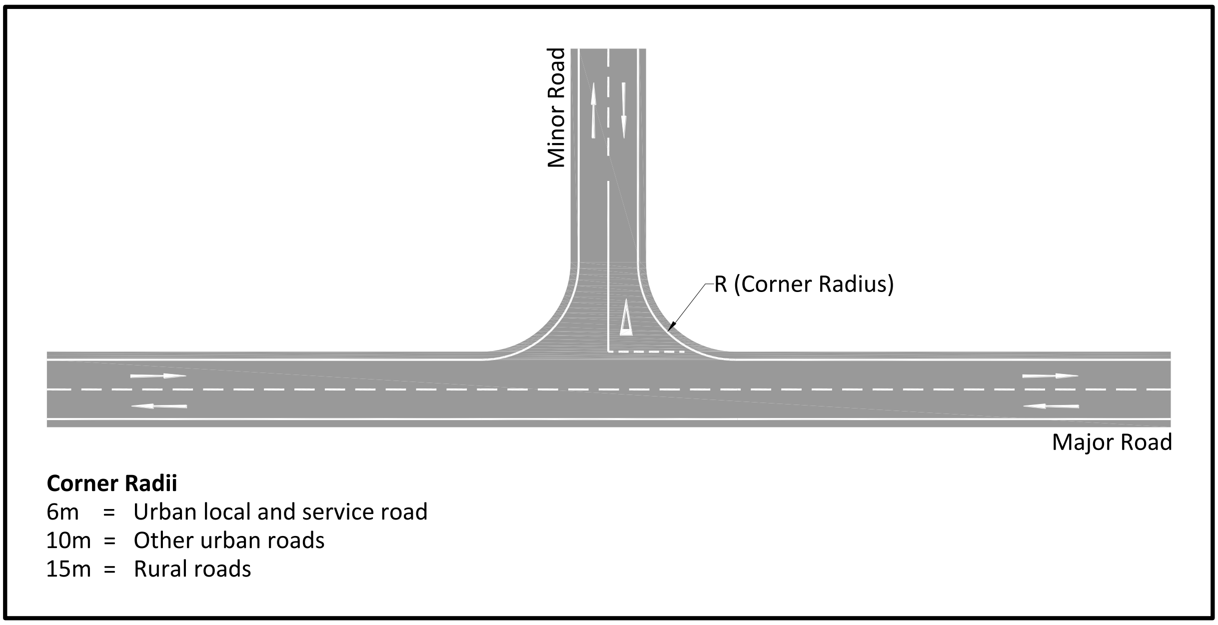


Figure 4.2: Simple T-Intersection

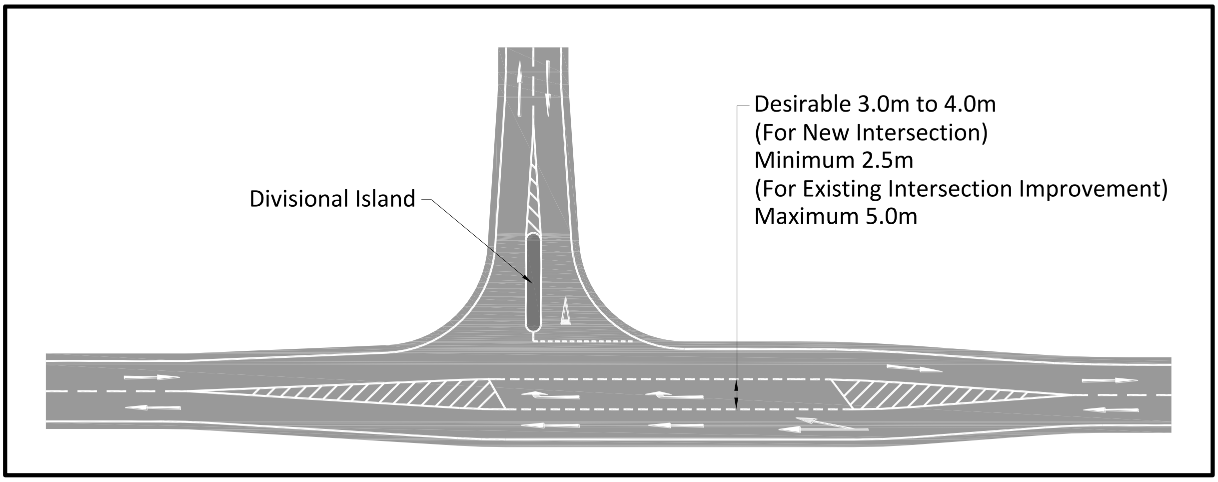


Figure 4.3: Ghost Island T-Intersection

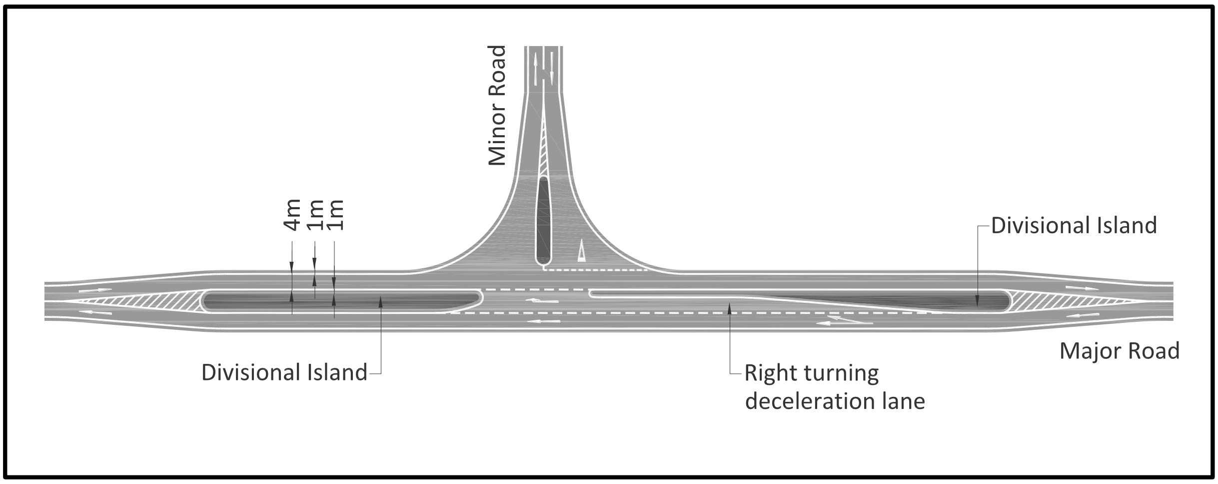


Figure 4.4: Priority T-Intersection on Two-Lane Highway with Major Road

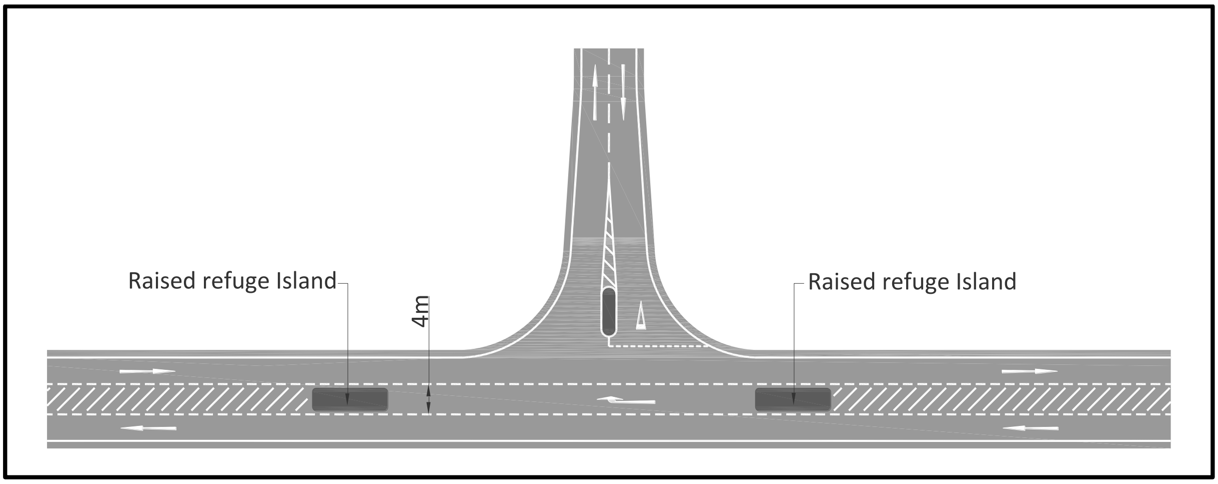


Figure 4.5: Channelized Right-Turn Urban Priority T-Intersection

4.5.2.2 Four-Leg or Crossroads Intersections

Four-Leg or crossroad priority intersections are formed when minor road crosses a major road, as shown on **Figure 4.6**. It has a very high number of conflict points, and has much higher risk of accidents than any other form of intersection. Use of this form of intersection is not recommended and should be avoided as much as possible. They should only be limited to roads with low volume and low speed. Staggered T- intersections can be provided as an alternative to crossroads to provide access to both sides of a major road. Existing crossroads should also be converted to Staggered T-intersections, roundabouts or signal controlled intersections, wherever possible.

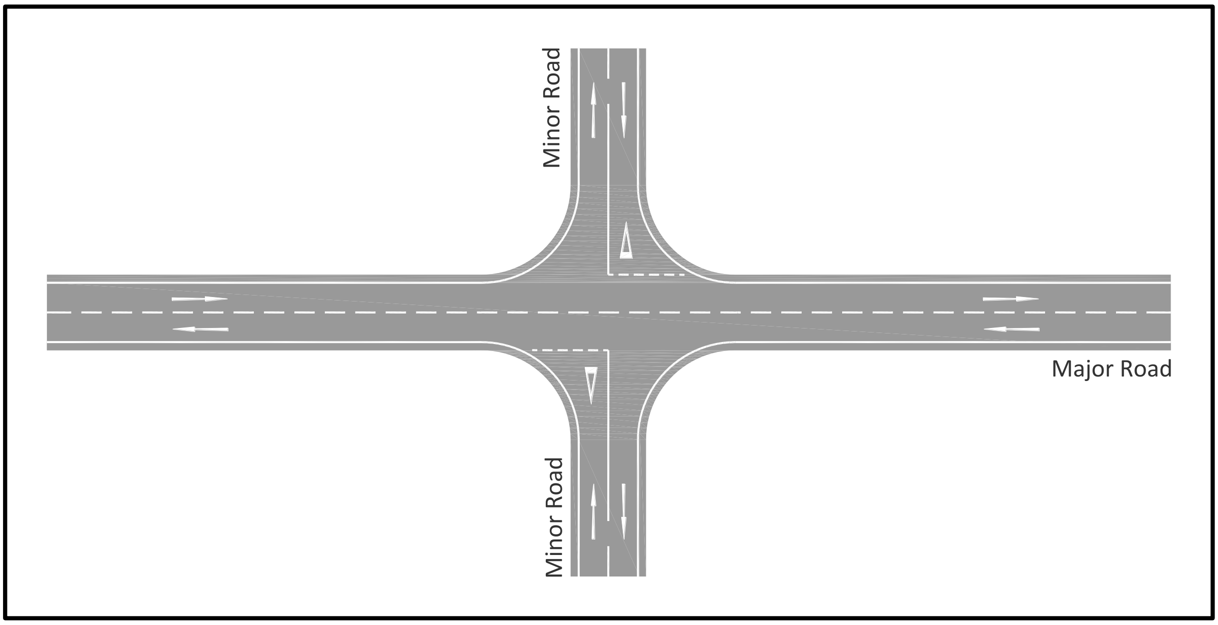


Figure 4.6: Crossroads

4.5.2.3 Staggered T-Intersections

The two priority T- intersections on opposite sides of the major road, placed at an offset to each other is referred to as Staggered T-Intersection. The design and operations of staggered T-intersection should be viewed as a single unit rather than two separate T-Intersections. This arrangement is preferable to crossroads since it reduces the number of potential conflicting points. It provides access to land-use on opposite sides of major priority road without the undesirable conflicts.

There are two alternative arrangements of staggered intersections, Left/Right Stagger and Right/Left Stagger. The Right/Left configuration is preferred as all queuing of right turning vehicles occur on approaches on major road rather the within the intersection. Un-channelized forms can be provided in residential areas with minimum 30 m distance between minor road centrelines. **Figure 4.7** to **Figure 4.9** illustrates different forms of un-channelized and channelized staggered T-intersections with a right-turn lane.

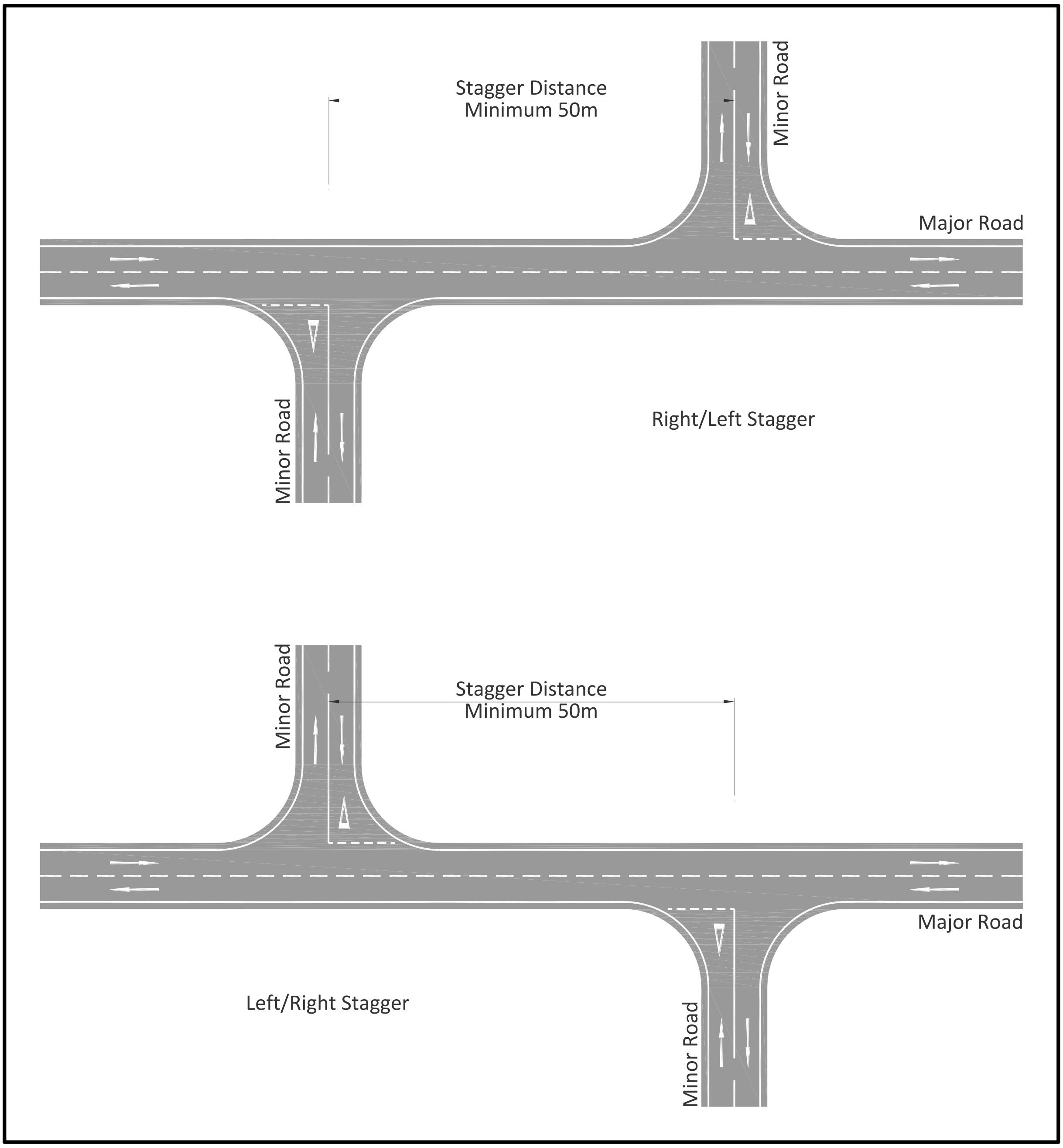


Figure 4.7: Simple Right/Left and Left/Right Staggered Intersections

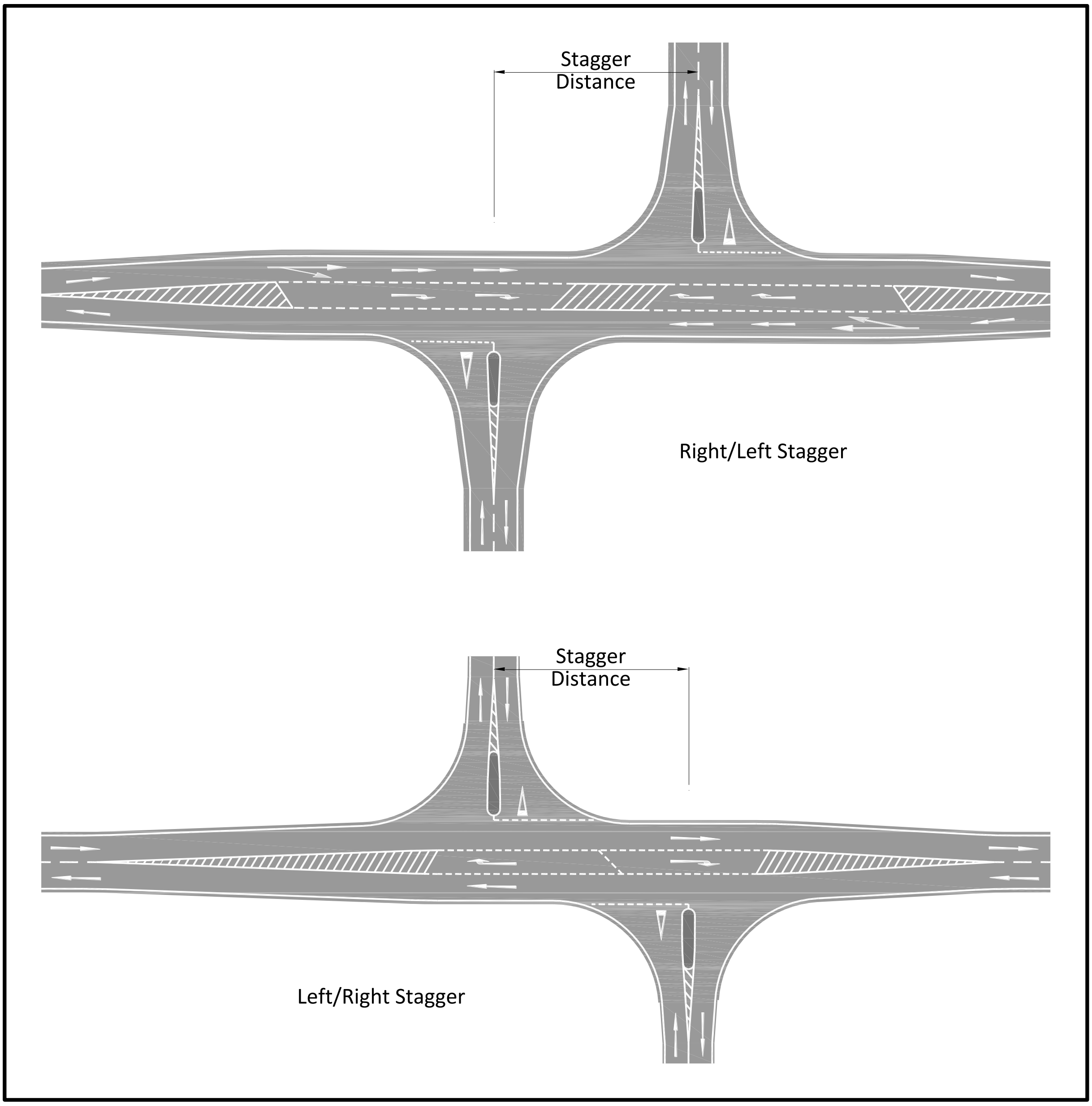


Figure 4.8: Right/Left and Left/Right Staggered Intersection with Ghost Island Right Turn Lane

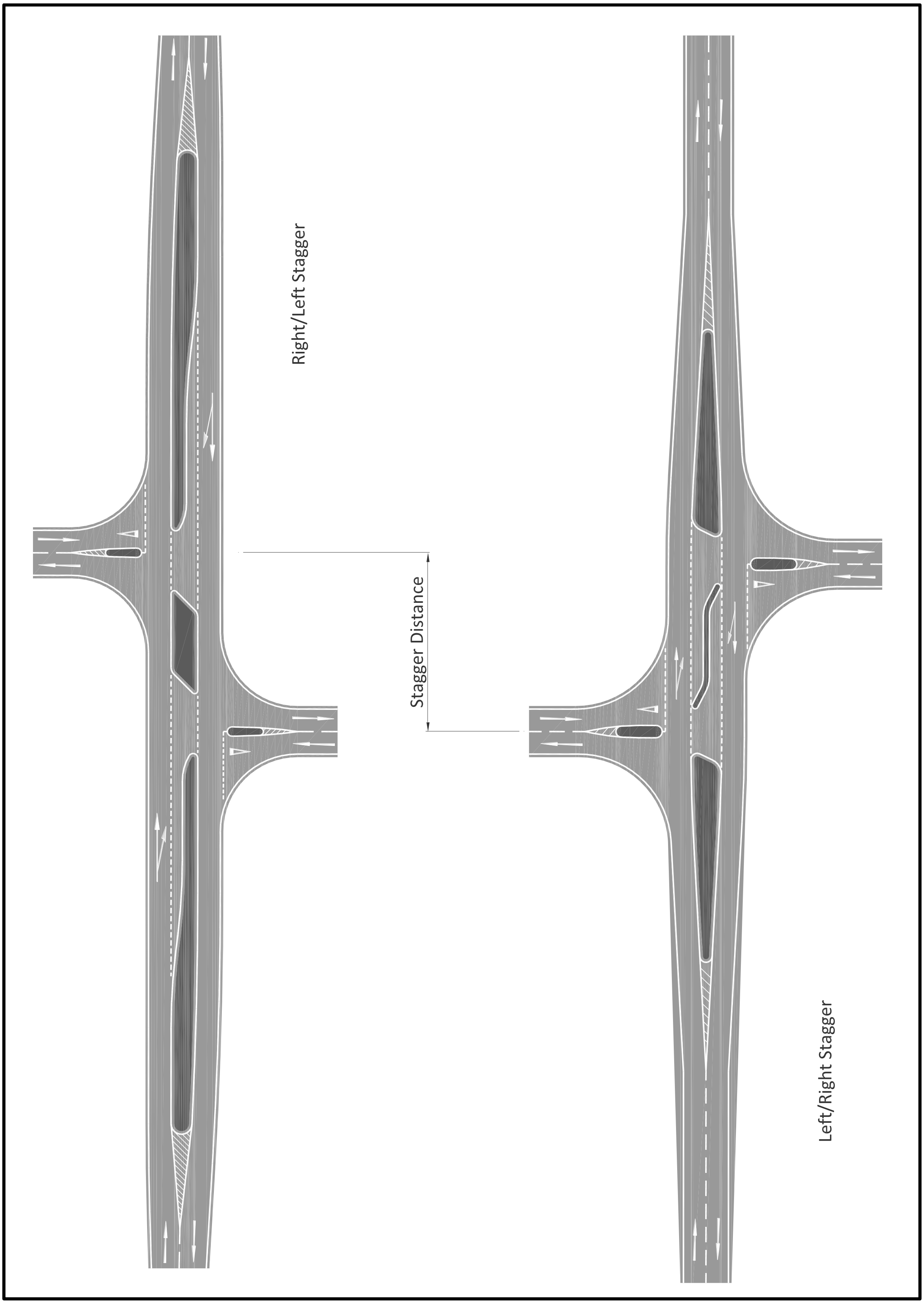


Figure 4.9: Right/Left and Left/Right Staggered Intersections with Right-Turn Lanes

4.5.2.4 Skew or Y-Intersections

Priority intersections are referred to as Skew intersections when minor road intersects the major road at an angle of 70 degree or less. They are also referred to as Y-Intersections. They can be left skewed or right skewed, with the combination including un-channelized or channelized with right-turn lane on major road, as shown in **Figure 4.10** and **Figure 4.11**. Such intersection creates difficulties for vehicles turning from minor road and reduces the driver's visibility. Large vehicles also require wide pavement area to complete the manoeuvre. The operations on such intersections can be improved by introducing channelization on minor and major road and increasing the intersection angle to 90 degrees.

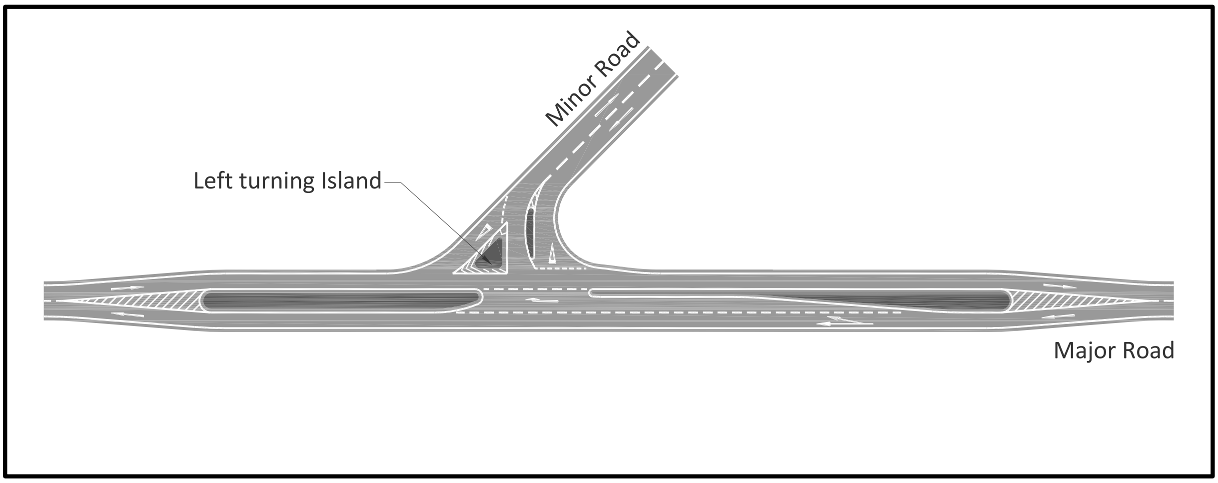


Figure 4.10: Priority Right-Skew T-Intersection with Major Road Right-Turn Lanes

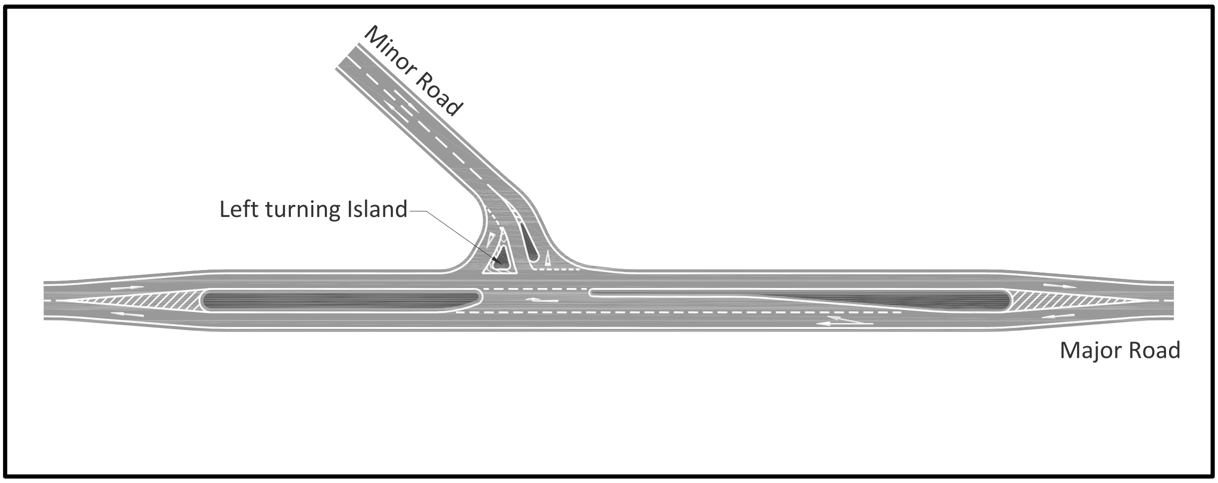


Figure 4.11: Priority Left-Skew T-Intersection with Major Road Right-Turn Lanes

4.5.3 Priority Intersections for Multi-Lane Divided Major Roads

Priority intersections on divided roads are considerably different from that on undivided roads with respect to design and operations. The right turn movement is prohibited on these intersections by continuation of a raised central median. The prohibited movement can be provided for by U-turns on nearby signalized intersection, roundabout or mid-block U-turns. In future, this type of layout can easily be converted to a signalized intersection by providing an opening in the median. The minor road can be a two-lane undivided road or multi-lane divided road.

To separate the decelerating and accelerating traffic from higher speed through traffic, auxiliary (acceleration and deceleration) lanes are generally provided on major road. The lengths of these auxiliary lanes are based on design speed of major road. **Figure 4.12** shows a typical multi-lane priority intersection with a two-lane undivided minor road.

In case of multi-lane divided minor road, the approaching minor road lanes shall be merged into single lane near the intersection. It is preferred to merge right lane into left lane. **Figure 4.13** shows the taper and geometry of the terminating two-lane section merging to one lane. The minor road approach can also be reduced by the introduction of U-Turn facility at termination of minor road, as illustrated in **Figure 4.14**. For left turning from major road, the turning roadway shall be designed for one lane and second lane can be developed at a convenient location.

A mid-block un-signalized U-turn can be provided on major road, 130 m to 200 m downstream of priority intersection, as shown in **Figure 4.15**, to cater for right turn movements, if downstream signalized intersection is too far or the increased traffic on signalized intersection significantly decreases its overall capacity or surrounding land-use require more direct access. The presence of U-Turn for right turn movement shall be properly signed on minor road approach.

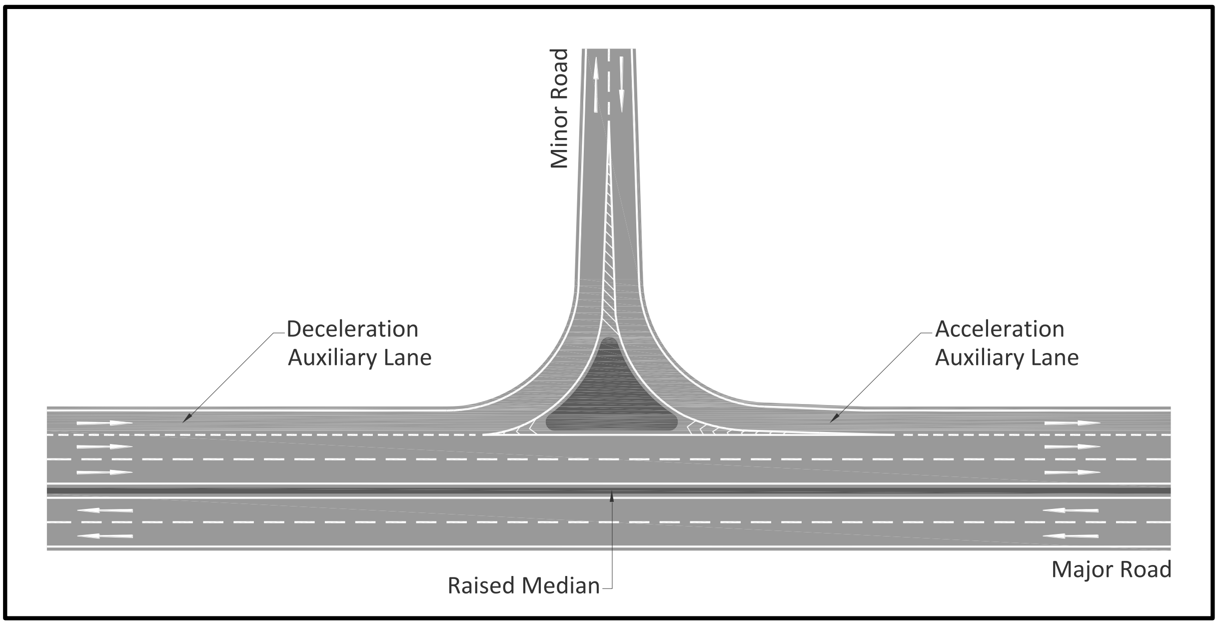


Figure 4.12: Priority T-Intersection on Multi-Lane Roadway with Median

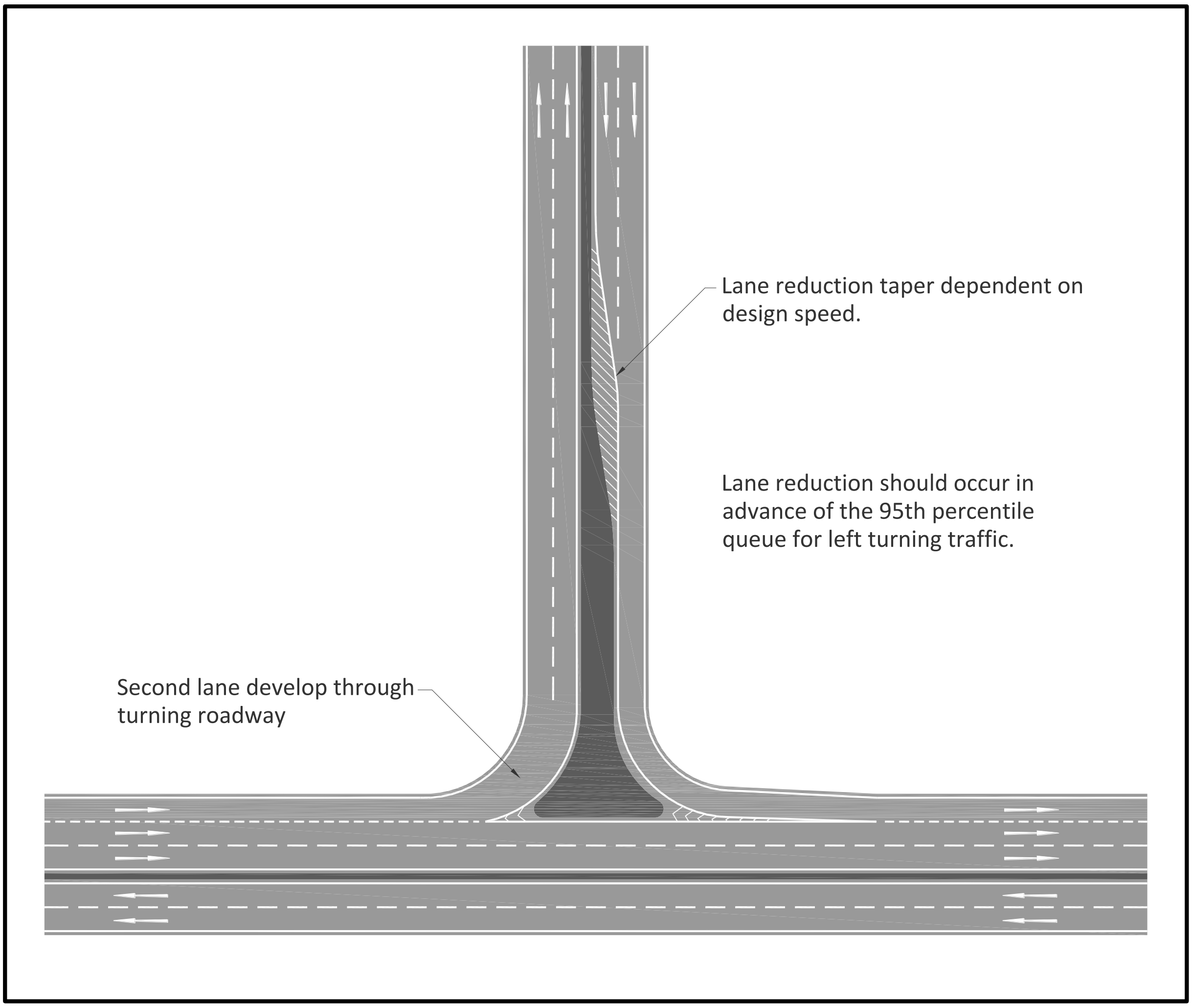


Figure 4.13: Priority Intersection between Two Multi-Lane Roads

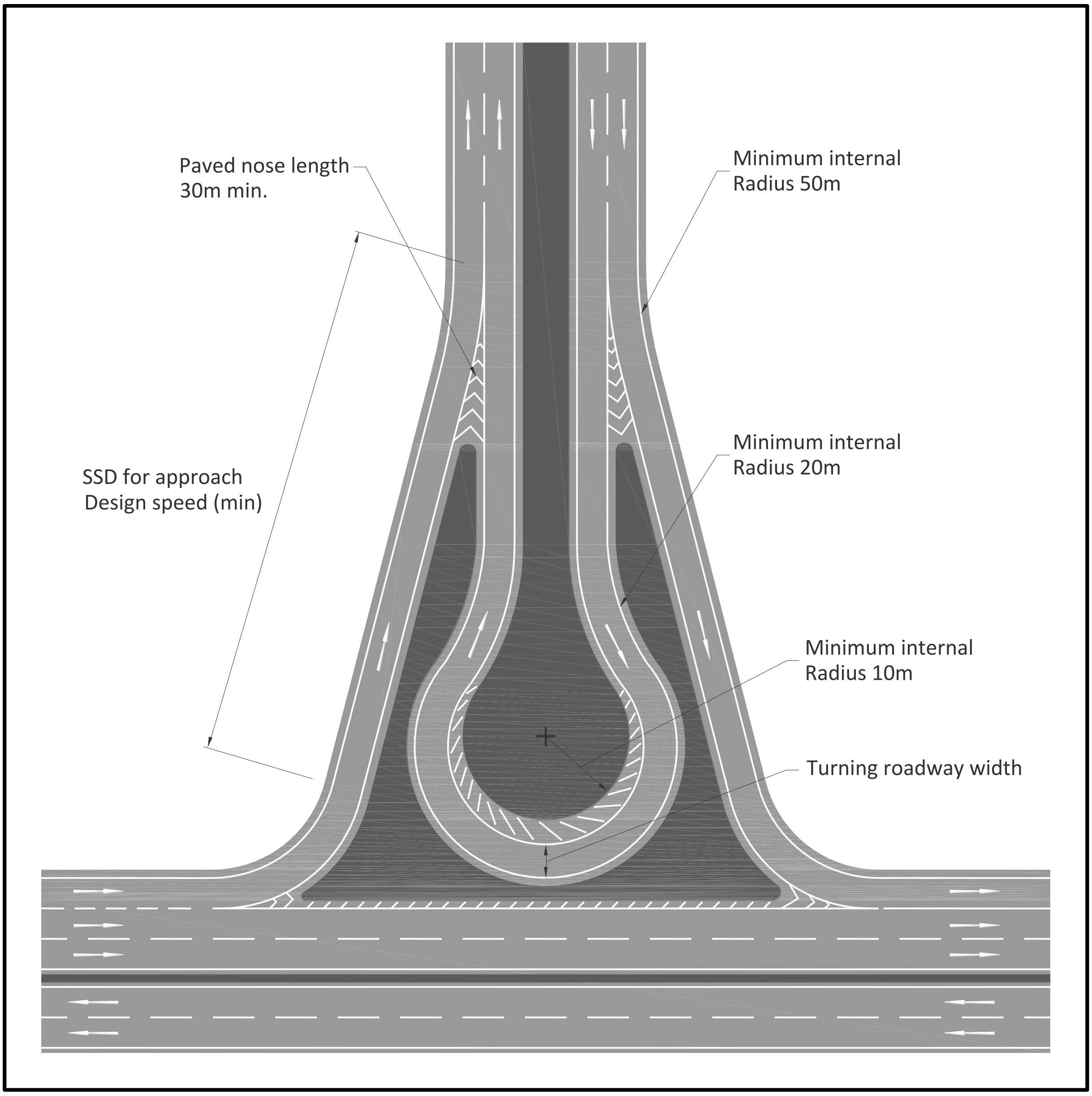


Figure 4.14: Minor Road Approach Reduced through U-Turn Facility (Urban Use Only)

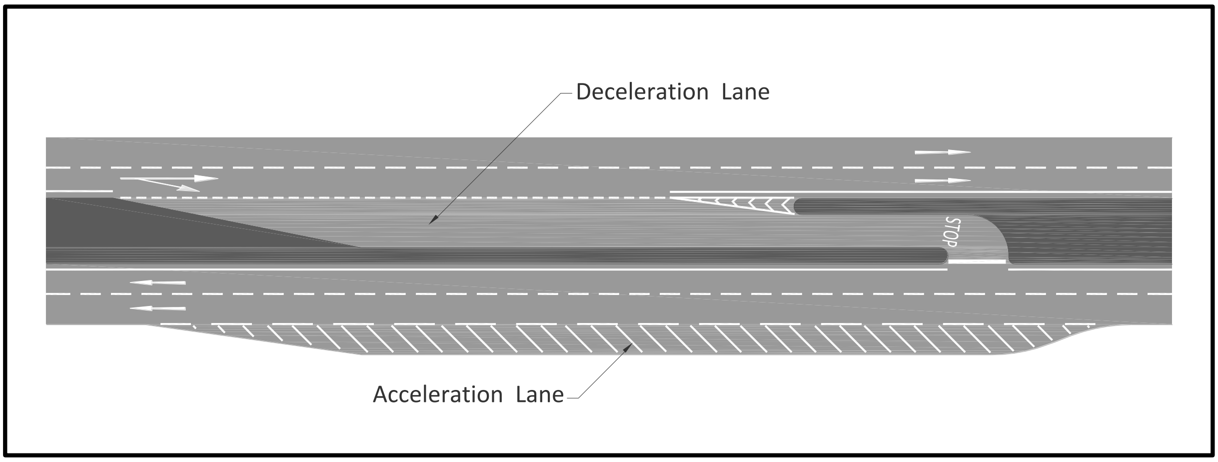


Figure 4.15: Un-Signalized Median U-Turn

4.5.4 Design Controls

4.5.4.1 Design Speed

Geometric standards for intersections depend on design speed of the major road.

4.5.4.2 Design Vehicles

The design vehicle for priority intersection shall be selected by the designer and shall be approved by the authority. The choice of design vehicle affects both the turning radius and the width of the turning roadway. **Table 4.4** provides guidance for selection of the appropriate design vehicle. Swept path templates need to be applied, using design software such as AutoTurn, to determine the optimum layout of intersection.

Table 4.4: Guidance for Selection of Design Vehicles at Intersections

Design Condition	Roadway Type	Design Vehicle [a]
Parking lot entry/exit	Local road, collector road	Passenger car
Local road intersection (un-signalized)	Urban local road, collector road	Single-unit truck or bus
Local road entry/exit	Rural arterial (priority)	Single-unit truck or bus
Collector road entry/exit	Rural arterial (priority)	Single-unit truck or bus
Signalized intersection	Minor or major arterials	WB-20, single-unit truck [b]
Two-lane roundabout	Minor or major arterials	WB-20, single-unit truck
Signalized intersection or roundabout	Freeway or expressway interchange ramp terminal	WB-20
U-turn movement	Signalized intersection	Passenger car or single-unit truck or bus

Notes:

[a] Emergency vehicles shall be able to access all conditions.

[b] More than one design vehicle may be used if patterns and types of traffic vary by approach leg.

WB = wheelbase

4.5.4.3 Design Level of Service

The capacity of the urban road network largely depends on the capacity of its intersections. In design of new road and intersection, the sizing of roadways and its elements are based on the selected Level of service. For priority intersections, the level of service is based on the ability of minor road traffic to find gaps and turn on major road.

4.5.4.4 Locating Priority Intersections

The most important consideration in locating a priority intersection on major road is that the alignment and profile provide clear line of sight to approaching drivers from minor road. Location of priority intersection is associated with the location of minor road. For new roads, the location of minor road itself is sometimes associated with the adjacent property lines. Thus, while locating the intersection, the designer shall balance between many physical controls and constraints with operational and design requirements.

4.5.4.5 Horizontal Alignment

Preferably, the priority intersections shall be sited on major road tangents, as the turning manoeuvre and acceleration is more difficult on curve. If unavoidable, it is preferred to provide T-intersections on the outside of a curve, where greater visibility is provided than those that are located on the inside of a curve. The intersection angle should be within 70 to 110 degrees. Passing should not be allowed on approaches to and within the intersection.

Skewed intersections, where roads intersect at angles less than 70 degrees creates special challenges to the designer and should be avoided to the extent practical. They require extensive pavement area on turning roadway, especially for large vehicles. In addition, the acute angle creates difficulty for the drivers to perceive the crossing traffic. Thus, the designer should attempt to bring the minor approach to as close to right angle as possible. If skew cannot be avoided, right turn channelization can be included to provide clarity to the arrangement of intersection, regardless of approach traffic volume.

Figure 4.16 illustrates a skewed priority T-intersection with major road right-turn channelization. Either a right- or left-hand skew may be designed. For high-speed roads, with design speed greater than 80 km/h, it is desirable to provide a deceleration lane for left turning traffic from major road as illustrated in **Figure 4.17**.

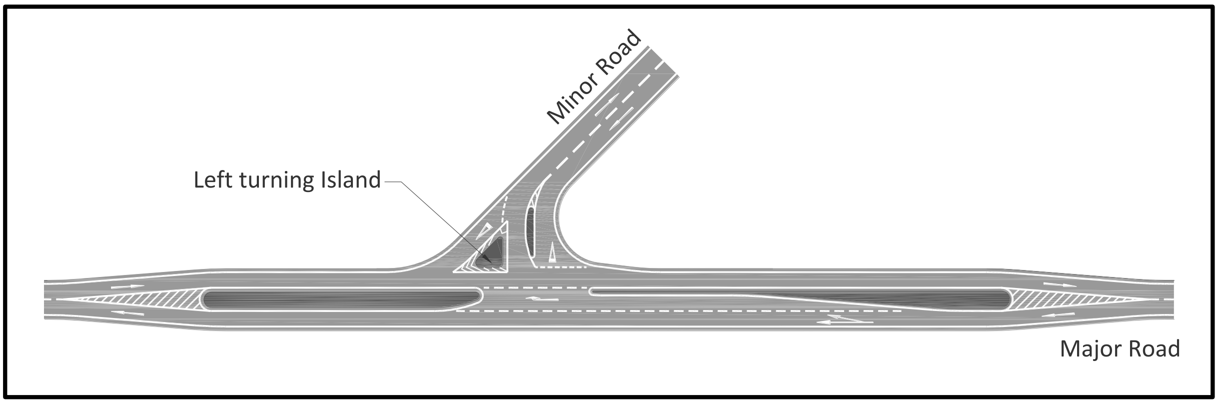


Figure 4.16: Priority Skewed T-Intersection with Major Road Right-Turn Lanes

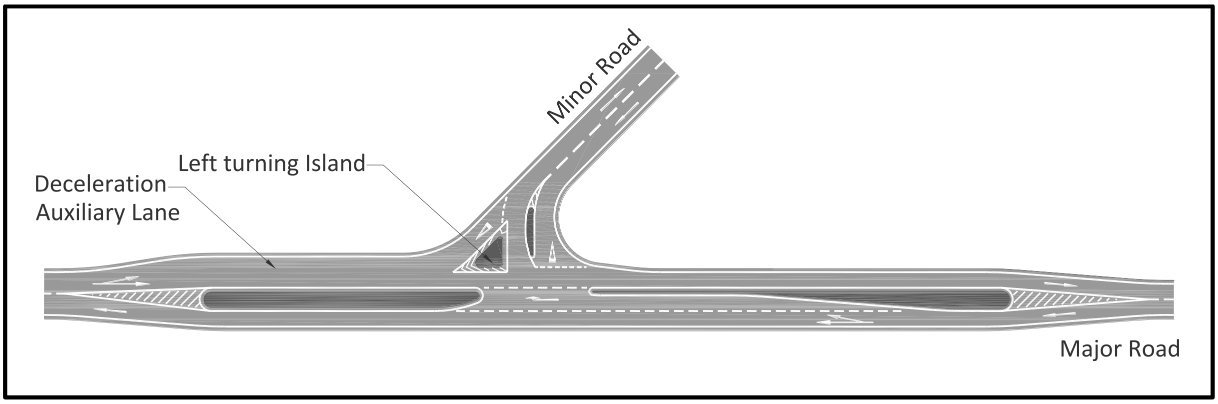


Figure 4.17: Priority Skewed T-Intersection with Major Road Right-Turn Lanes and Auxiliary Lane for Major Road Left Turns

Crossroad intersections with skew angle greater than 5 degrees should also be avoided in new roads. For reconstruction projects with exiting skewed intersections, **Figure 4.18** provides a range of possible solutions to address the skew. Designers should also avoid Y-intersection alignments, because it can create additional visibility problems when the intersection is hidden by crest vertical curve. Two design options for Y-intersections are shown in **Figure 4.19**.

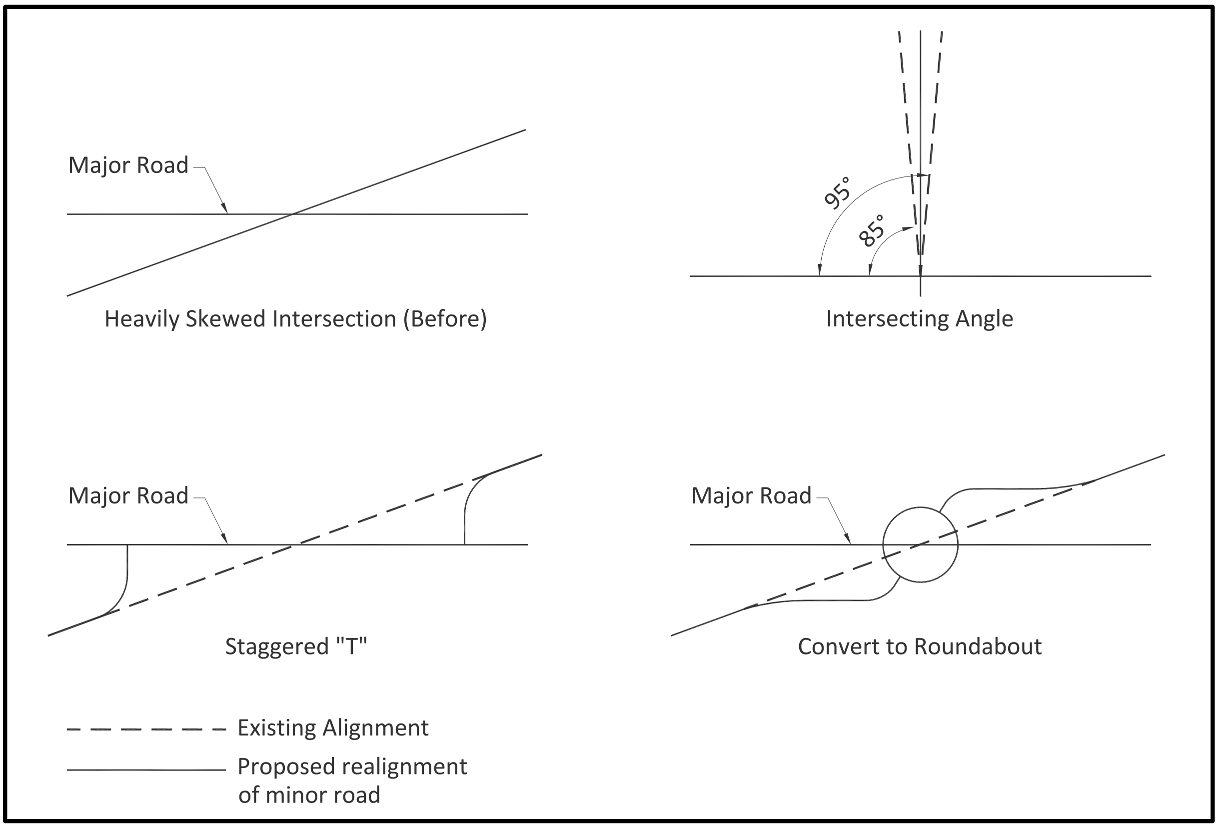


Figure 4.18: Design Solutions for Skewed Crossroads

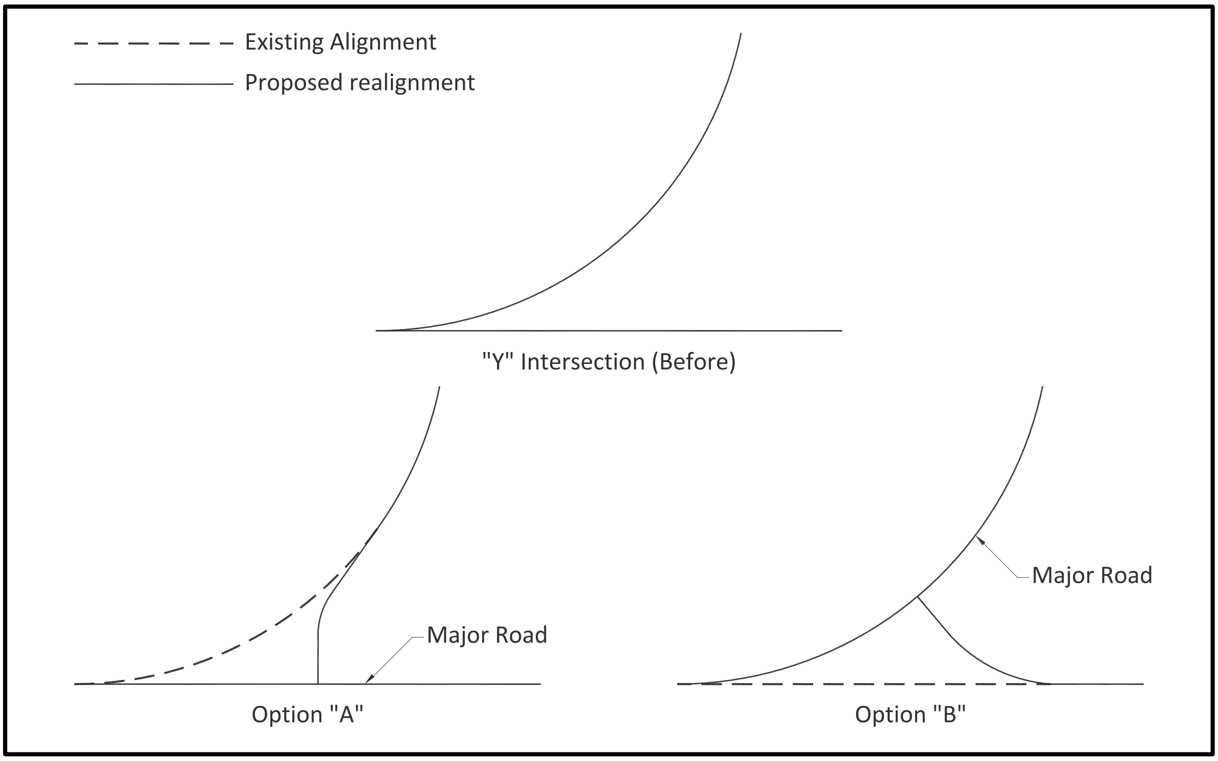


Figure 4.19: Design Solutions for Y-Intersections

4.5.4.6 Vertical Alignment

The intersections should be located on level ground, to the extent practical or where the major road approach is 2 percent or less. Steep uphill approach grades on minor road are also undesirable. Approach grades on minor road shall be minimized near intersection, to the extent practical, to provide a flat platform of 15 m to 20 m (10 m in residential area) or for at least one passenger car at yield line. The cross slope of major road should be carried onto the minor road up to a distance depending on the grades and vertical design of minor road. The profile of the minor road will then follow the established design criteria of vertical design. Graphical representation of the profile on minor road is shown in **Figure 4.20** for guidance.

The profile of major road should be carried through the intersection and edges of minor road should be adjusted accordingly by providing transition in cross slope of minor road to match with the edge profile of major road.

Full corner sight distance shall be provided on all approaches. The profile, cross slope and edge of pavement shall also be adjusted to provide proper drainage.

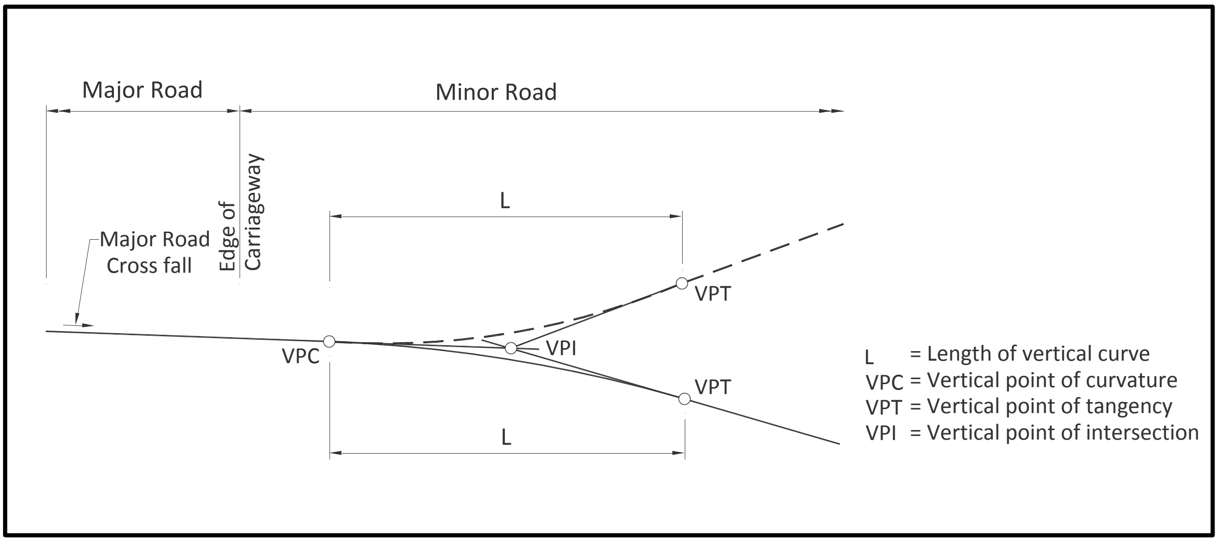


Figure 4.20: Minor Road Approach Gradient

4.5.4.7 Spacing of Intersections

The minimum spacing between adjacent intersections for the different road classifications are provided in **Table 4.5**.

Table 4.5: Minimum Intersection Spacing

Functional Classification	Minimum Intersection Spacing (m)	
	Urban	Rural
Major Arterial	600	1,000
Minor Arterial	150	1,000
Collector	100	500
Local Road	As required	As required

4.5.4.8 Visibility

Poor visibility not only involves safety implications, it also reduces the capacity of the intersection. Each intersection must be assessed on site-specific basis, based on factors such as, width of major road, traffic control provided on minor road, gradient of approaches, turning traffic volume and type of vehicles. At major/minor priority intersections, visibility splays should be provided as described below and attempt should be made to provide at existing intersections. Vehicle should not be parked within splay lines as they can obstruct visibility. Additionally, landscaping, traffic signs, and other road furniture should also be carefully placed within the visibility splay area in order to minimize the obstructive effect. For a one-way major road, a single visibility splay in the direction of approaching traffic will be sufficient.

On major road the driver should be able to see the minor road entry from a distance 1.5 times the stopping sight distance (SSD) corresponding to the design speed of major road.

The minor road traffic have to join or cross the major road when there are gaps in the major road traffic stream, therefore, it is essential for the minor road drivers to have sufficient visibility in each direction of the major road. The visibility splays for minor road are shown in **Figure 4.21** and **Figure 4.22**.

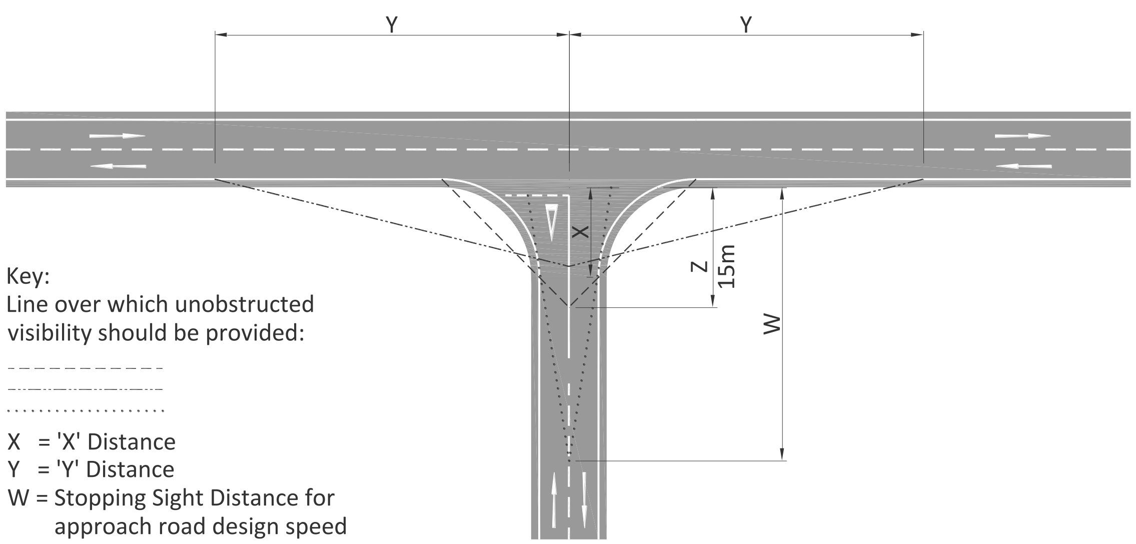


Figure 4.21: Visibility Criteria at Priority Intersections

Three visibility splays needs to be checked on intersection from minor approach road. They are:

1. The approaching drivers should be able to see the intersection from a distance equal to SSD (W in **Figure 4.21**) for the design speed of the minor road. In case, if yield or stop sign is provided, the visibility envelop should be widened to include the sign.
2. A driver approaching the intersection should be able to see the layout of intersection and its peripheral elements from a distance 15 m (Z in **Figure 4.21**) measured from the nearside edge of the major roadway.
3. The driver approaching the intersection should be able to see full intersection, both left and right, from a distance X along the minor road, up to a distance Y along the major road. The values of X and Y are given in **Table 4.6**. The X distance is measured along the centreline of minor road from the nearside edge of the major roadway, while Y is measured along the nearside edge of major road from its intersection with the centreline of minor road. The preferred X distance is 10 m. In some cases, when the X distance of 10 m cannot be achieved,

it can be reduce to 7.5 m for simple intersections with light traffic, 5.0 m in exceptionally difficult circumstances, or can be further reduced to 2.5 m in urban locations where only light vehicles are allowed.

On a curved road, if the line of sight is partially within the major road, it shall be made tangential to the nearer edge of the major road, as shown in **Figure 4.22**.

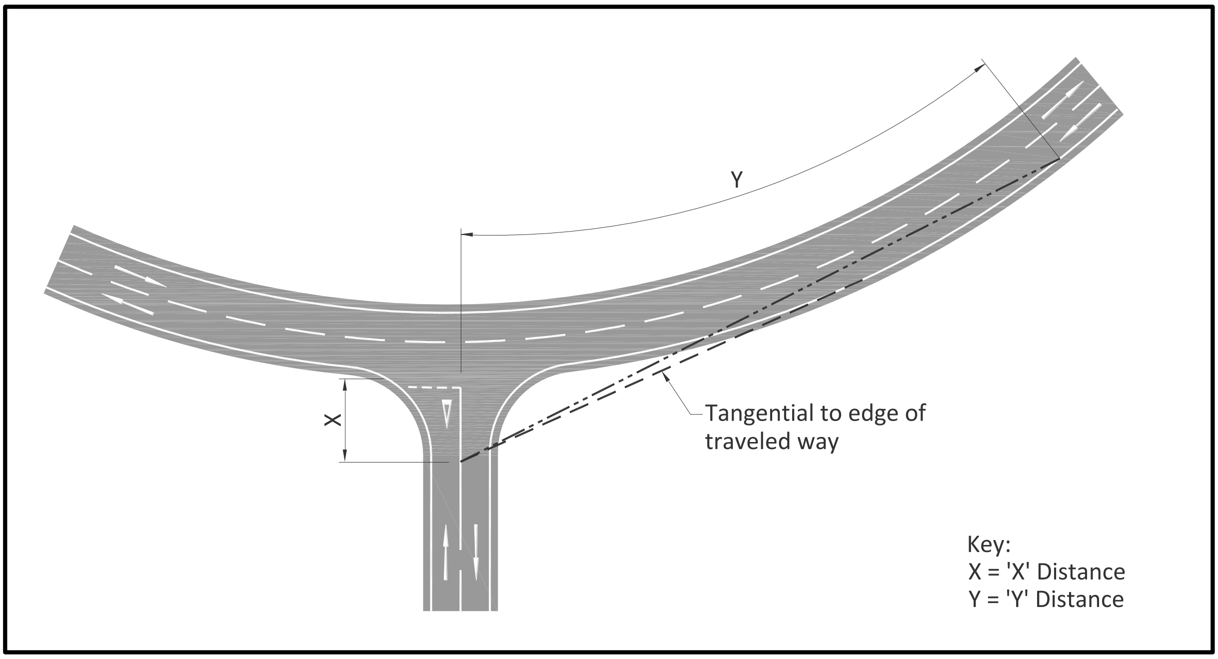


Figure 4.22: Visibility Criteria at a Curved Major Road

Table 4.6: Minimum X and Y Visibility Distances from the Minor Road

Design Speed of Major Road (km/h)	Y Distance (m)	Minimum X Distance (m)
120	295	10
100	215	10
80	160	10
70	120	7.5*
60	90	7.5*
50	70	5.0*
<50	50	2.5*

Note:
In all cases, the preferred "X" distance is 10 m. The minimum "X" distances given shall only be used in difficult circumstances.

4.5.5 Geometric Design Details

The geometric design parameters for priority intersections includes widths of roadways, corner radii, and auxiliary lane dimensions, taper, median openings, and traffic islands. The geometric parameters used mentioned below are sufficient for WB-15 Design Vehicles. Appropriate dimensions may vary based on functional needs of the intersection.

4.5.5.1 Corner Radii

The corner of the intersection should be designed such that it will accommodate the largest vehicle that will typically use the intersection. Corner may be designed with simple circular curve, compound curves, or simple curve at offset from through edge with tapers. The latter two options are best suitable for swept path of large design vehicles.

Corner Radii for Simple T-Intersection

For simple T-intersections without auxiliary lanes and with no provision for larger trucks or buses, the minimum circular corner radius shall be 10 m in rural areas and 6 m in urban local roads and service roads. Where buses or large trucks are to be accommodated the recommended corner radii, taper rate and length are given in **Table 4.7**. **Figure 4.23** shows circular corner radii incorporating tapers.

Table 4.7: Circular Corner Radii

Intersection Type	Taper Rate (T)	Length of Taper (L) (m)	Corner Radius (R) (m)
Urban Simple Intersection	1:5	30	10
Rural Simple Intersection	1:10	25	15
Staggered Intersection	1:8	32	15
All Other	Not applicable	Not applicable	20

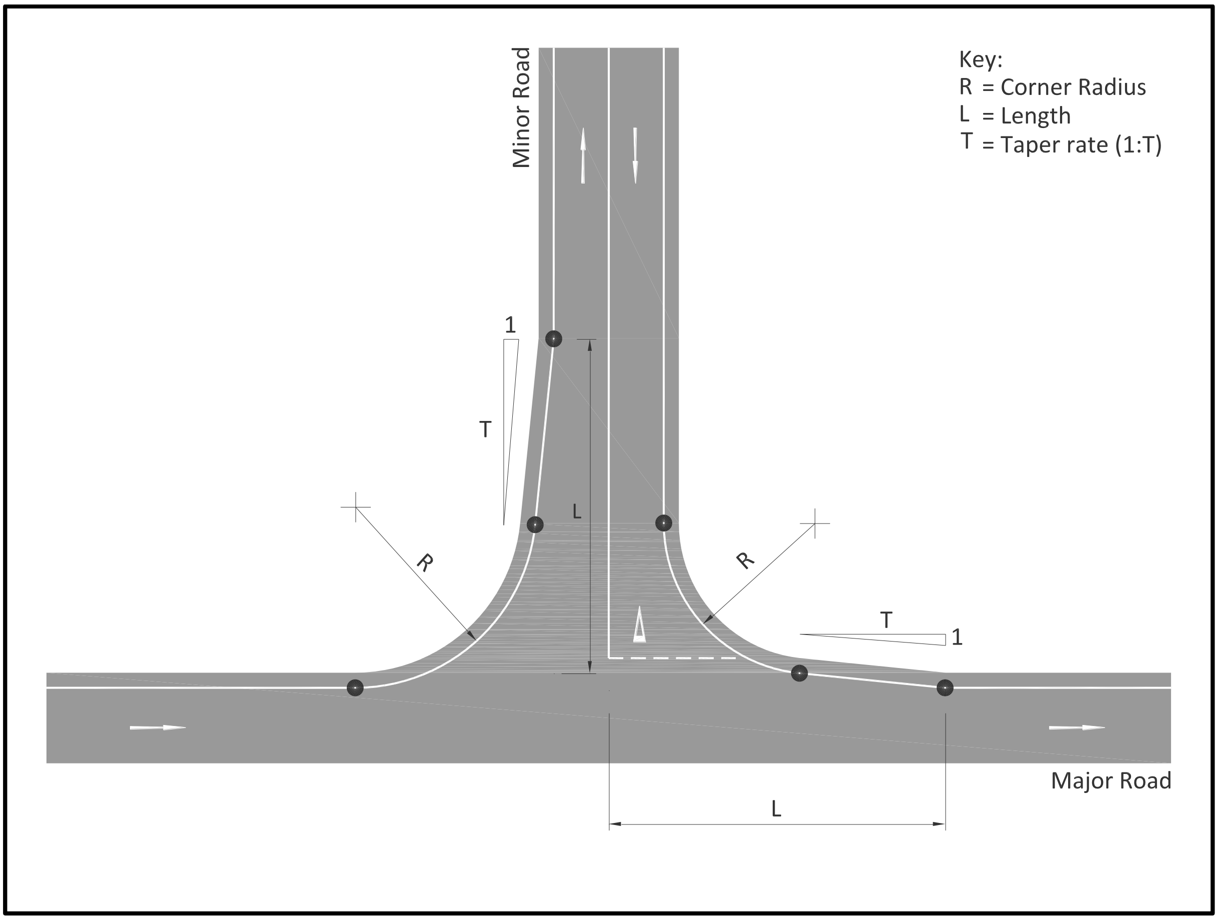


Figure 4.23: Circular Corner Radii Incorporating Tapers

Compound circular curves as shown in **Figure 4.24** shall be used, when turning traffic comprises a significant portion of large trucks.

The design of an intersection shall be checked, using design software such as AutoTurn, to ensure the design vehicle swept path remains within the kerb faces.

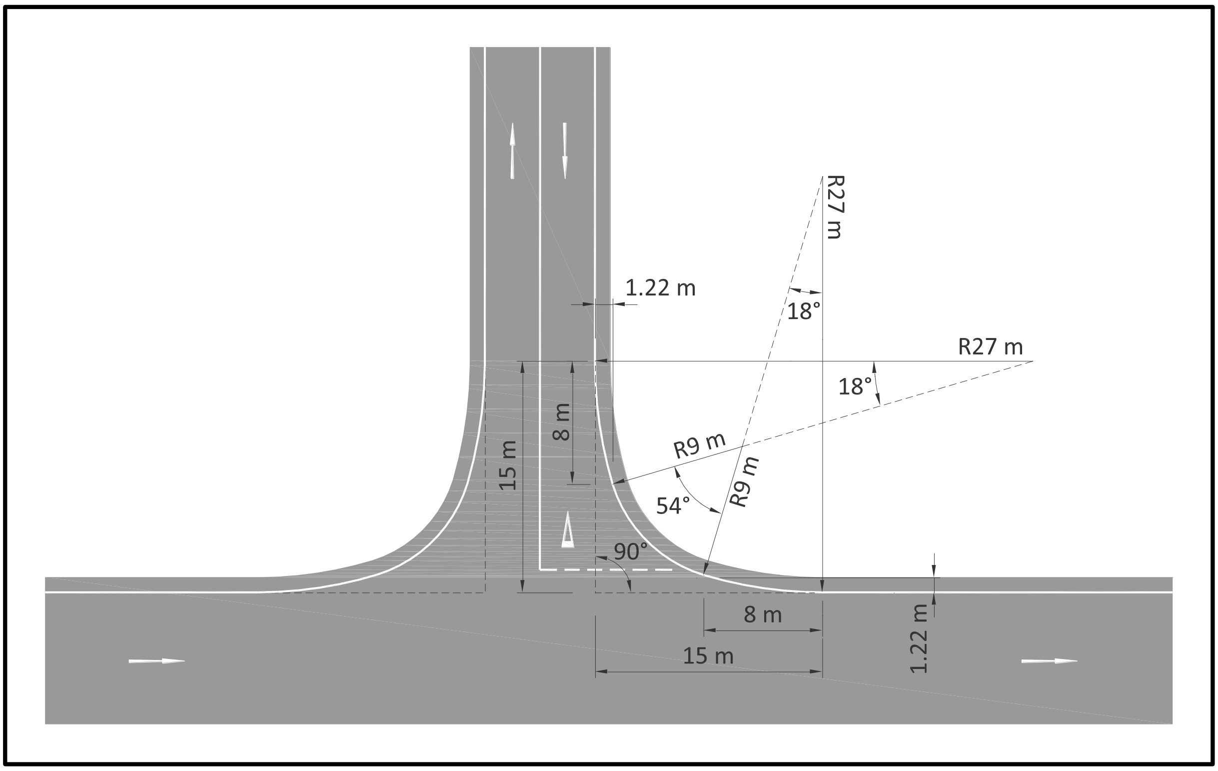


Figure 4.24: Circular Corner Radii Incorporating Compound Curve

Corner Radii for Channelized Left Turns

Figure 4.25 illustrates the layout of a channelized left-turn. The left-turn curve radius into the minor road at the edge of the turning roadway shall be 20 m for major road design speed of 80 km/h or less. For higher design speeds, the minimum radius shall be 40 m.

The minimum radius of the edge of turning roadway from minor road shall be 25 m for major road design speed of 80 km/h or less. For higher design speeds, the minimum radius shall be 30 m.

4.5.5.2 Roadway Widths

Through-Lane Widths

At simple intersections, ghost island intersections and intersections on multi-lane roads, the lane width through the intersection shall be the same as in advance of the intersection.

Roadway widths on two lane major roads, after the introduction of physical island, shall provide for passing a stalled vehicle. 4 m lane width with 1 m offset to median island shall be sufficient for a rural highway with paved shoulder. For urban kerbed roadway, 4 m lane width with 1 m offset to kerb on both side of the road shall be sufficient.

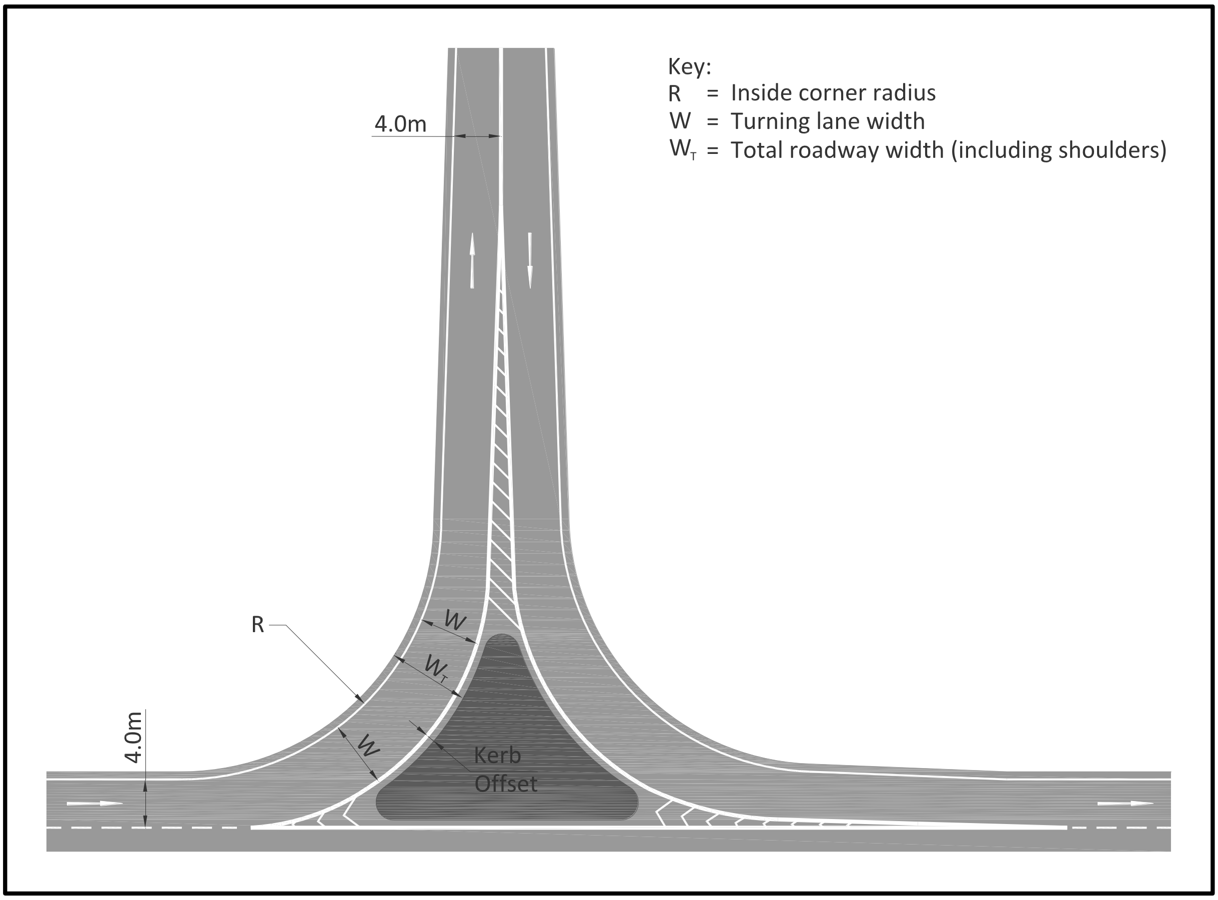


Figure 4.25: Typical Layout of Channelized Left Turn

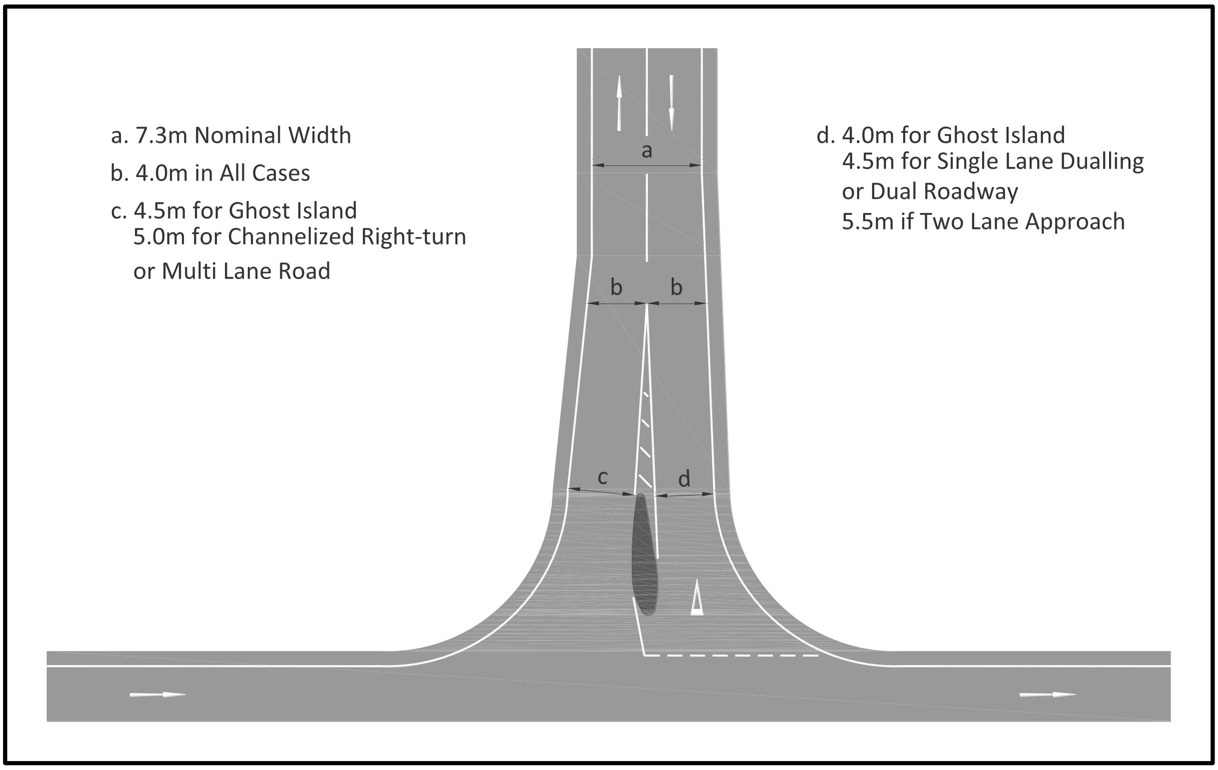


Figure 4.26: Minor Road Approaches

Widths of Minor Road Approaches

The width of the minor road approach is depends on whether a channelizing island is provided. When no channelizing island is provided on minor road approach, the nominal approach width of minor road shall continue up to the start of corner curve. When channelizing island is provided, both lanes shall be minimum 4.0 m wide at the start point of hatch marking surrounding the channelizing island. **Figure 4.26** illustrates the layout of such approaches.

Widths of Turning Roadway

In order to accommodate the swept path of large vehicles, the turning roadway should be widened (See **Figure 4.25**). The radius of curvature of the inner edge of pavement depends on the design speed and the expected design vehicles using the roadway. **Table 4.8** shows pavement width values for various design traffic conditions, operational purposes and curvature of the inside edge of pavement. The values shown in this table should be used as a guide and check with a turning template or computer simulated turning template.

Table 4.8: Design Widths of Pavements for Turning Roadways

Radius on Inner Edge of Pavement, R (m)	Case-I			Case-II			Case-III		
	Design Traffic Condition								
	A	B	C	A	B	C	A	B	C
15	5.4	5.5	7.0	6.0	7.8	9.2	9.4	11.0	13.6
25	4.8	5.0	5.8	5.6	6.9	7.9	8.6	9.7	11.1
30	4.5	4.9	5.5	5.5	6.7	7.6	8.4	9.4	10.6
50	4.2	4.6	5.0	5.3	6.3	7.0	7.9	8.8	9.5
75	3.9	4.5	4.8	5.2	6.1	6.7	7.7	8.5	8.9
100	3.9	4.5	4.8	5.2	5.9	6.5	7.6	8.3	8.7
125	3.9	4.5	4.8	5.1	5.9	6.4	7.6	8.2	8.5
150	3.6	4.5	4.5	5.1	5.8	6.4	7.5	8.2	8.4
Tangent	3.6	4.2	4.2	5.0	5.5	6.1	7.3	7.9	7.9
Width Modification									
No Stabilized Shoulder	None			None			None		
Sloping Kerb	None			None			None		
Vertical Kerb									
One side	Add 0.3 m			None			Add 0.3 m		
Two sides	Add 0.6 m			Add 0.3 m			Add 0.6 m		
Stabilized shoulder, one or both sides	Lane width for conditions B & C on tangent may be reduced to 3.6 m where shoulder is 1.2 m or wider			Deduct shoulder width(s); minimum pavement width as under Case I			Deduct 0.6 m where Shoulder is 1.2 m or wider		

Notes:
A = predominantly Passenger vehicles, but some consideration for Single Unit trucks
B = sufficient SU-9 vehicles to govern design, but some consideration for semitrailer combination trucks
C = sufficient bus and combination-trucks to govern design
(Source: *AASHTO's A Policy on Geometric Design of Highways and Streets, 2011* [2])

4.5.5.3 Auxiliary Lanes and Tapers

Auxiliary lanes at priority intersections are provided for acceleration or deceleration of turning traffic from major/minor road, separation of turning traffic from main traffic stream, and separation of queued traffic awaiting a gap in opposing through traffic. The width of the auxiliary lane shall be equal to the adjoining through lane. For reconstruction projects, in which the cross section is limited due to right-of-way or other constraints, lane widths may be reduced to 3.4 m.

Through-Lane Continuity and Left-Turn Lanes

The major road basic lanes should not be dropped at the left-turn. If the through lane is to be dropped due to substantial turning volume or transition in basic number of lanes, the reduction shall be done at least 200 m downstream of the intersection.

Left-Turn Diverge Auxiliary Lanes and Tapers

Diverge auxiliary lane at left-turn intersections allows the deceleration of vehicles exiting to occur off the main roadway. **Figure 4.27** shows a typical design of multi-lane priority intersection with left-turn diverge auxiliary lane.

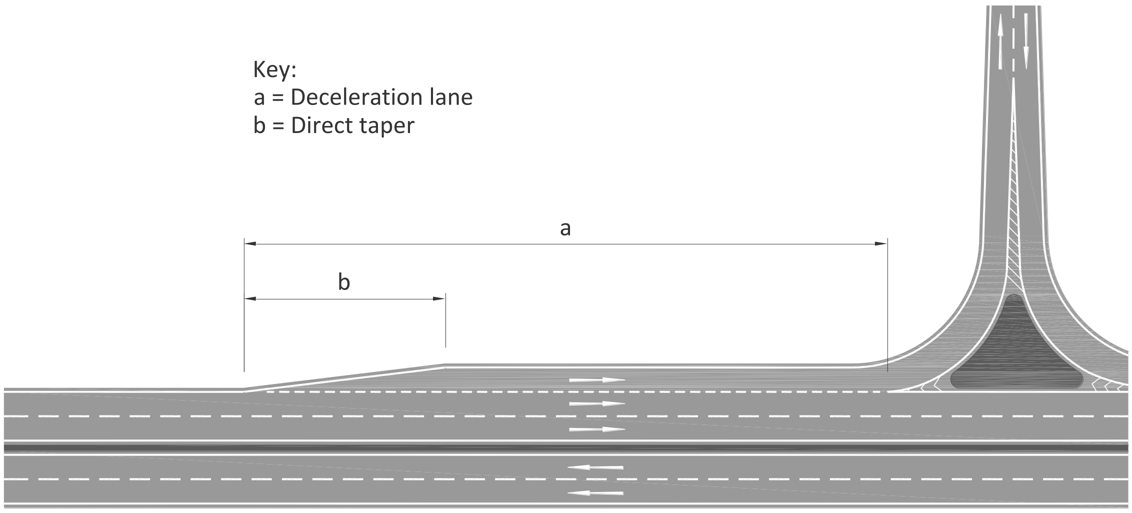


Figure 4.27: Left-Turn Auxiliary Lane on Diverge of Multi-Lane Priority Intersection

Diverging left-turn auxiliary lanes can be provided on rural multi-lane priority intersections, rural arterials and collector roads. In urban areas, they can be provided on multi-lane priority intersections with major road design speed of 100 km/h or greater. Based on traffic volume, right-of-way availability, and other factors, the diverging auxiliary lanes may be provided on other types of intersections. The length of Diverging auxiliary lane and taper lengths are provided in **Table 4.9** based on design speed of major road. The taper length as provided in **Table 4.9** is included in the auxiliary lane length (See **Figure 4.27**).

Shoulder width equal to that of the associated major road shall be provided along the auxiliary lane. Due to site and economic constraints, the designer may use some or all of the shoulder for the left turn lane at the approach to the intersection, with only minor additional widening of 1 m to 2 m. Where pedestrians crossing are predominant, an additional length of one or two queued vehicles should be provided in advance of pedestrian crossing.

Table 4.9: Minimum Diverge Auxiliary Lane Lengths on Priority Intersection

	Design Speed (km/h)	Minimum Auxiliary Lane Length-a (m)	Direct Taper Length-b (m)
Rural	50	80	5
	80	80	15
	100	160	25
	120	230	30
	140	Not applicable [a]	Not applicable
Urban	50	40	5
	60 [b]	50 (40) [c]	10
	70 [b]	65 (40) [c]	10
	80	80 (50) [c]	15
	100	120 (80)	25
	120	Not applicable [a]	Not applicable

Notes:

[a] For auxiliary lanes at interchanges, refer to Design for Signal Controlled Intersections.

[b] 60 km/h and 70 km/h design speeds may occur where a diverging auxiliary lane is required from a ramp.

[c] Figures in parentheses may be applied where intersection spacing is a constraint.

Diverging Tapers

In certain cases, due to right-of-way constraints, there is not enough space to provide auxiliary lane. In such cases, diverging taper can be provided, as illustrated in in **Figure 4.28**, although parallel auxiliary lane is the preferred option. The minimum length of diverging taper shall be same as the length of auxiliary lane provided in **Table 4.9**. Longer lengths are preferred.

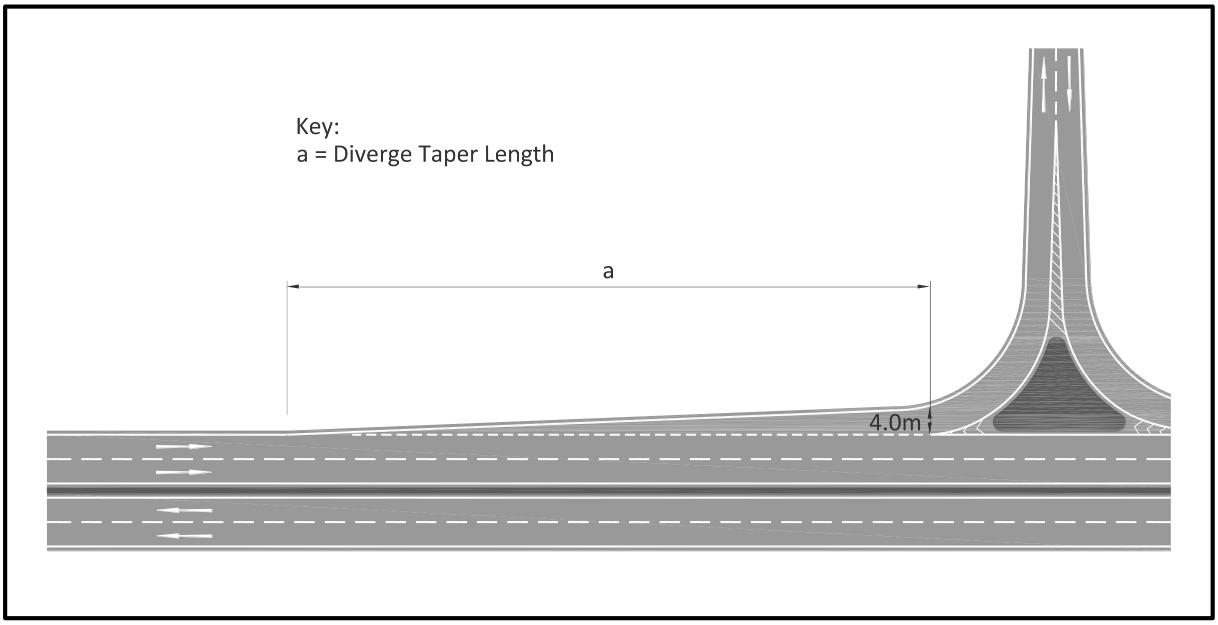


Figure 4.28: Alternative Taper Diverge

Left-Turn Merging Auxiliary Lanes and Tapers

Merging auxiliary lane at left-turn intersections allows the acceleration of minor road vehicles before merging with the faster traffic on major road. **Figure 4.29** shows a typical design of multi-lane priority intersection with left-turn merging auxiliary lane. Merging auxiliary lanes shall not be provided at priority intersection on two-lane roads.

Merging auxiliary lanes shall be provided at all rural priority intersections on multi-lane roads, for all functional classifications. In urban areas, they shall be provided on priority intersections of all multi-lane roads with a design speed of 100 km/h or greater, except where weaving problem is created due to their inclusion. The length of Merging auxiliary lane and taper lengths are provided in **Table 4.10** based on design speed of major road. The taper length of 35 m at the end of parallel section, as provided in **Table 4.10**, is included in the length of auxiliary lane (See **Figure 4.29**).

Merging auxiliary lanes shall be preceded by a short merge nose 40 m long between the auxiliary lane and the end of the approach curve. The minimum width at the back of the nose shall be 2 m, as illustrated in **Figure 4.30**.

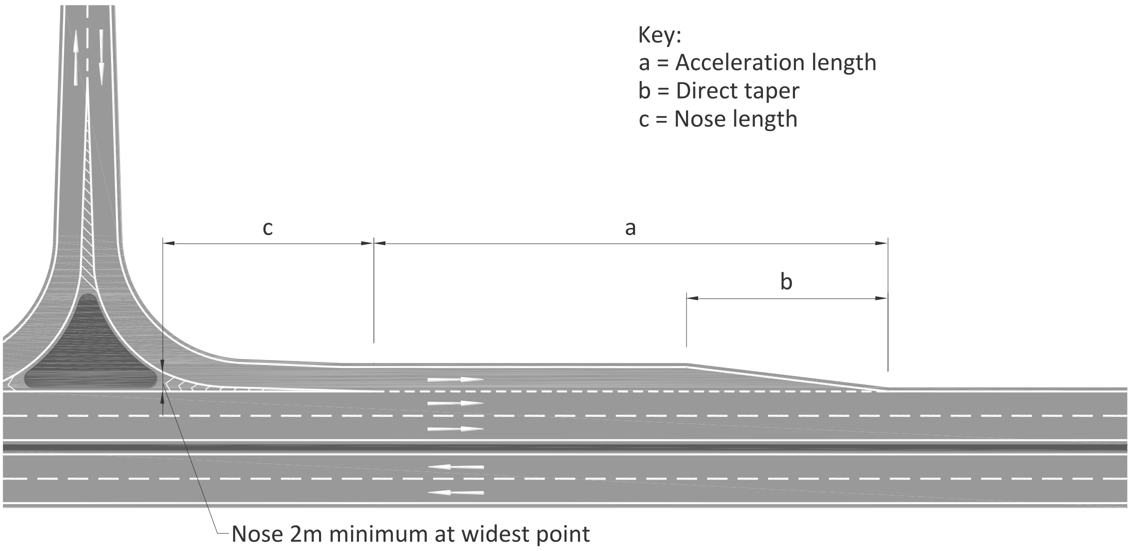


Figure 4.29: Auxiliary Lane on Merge

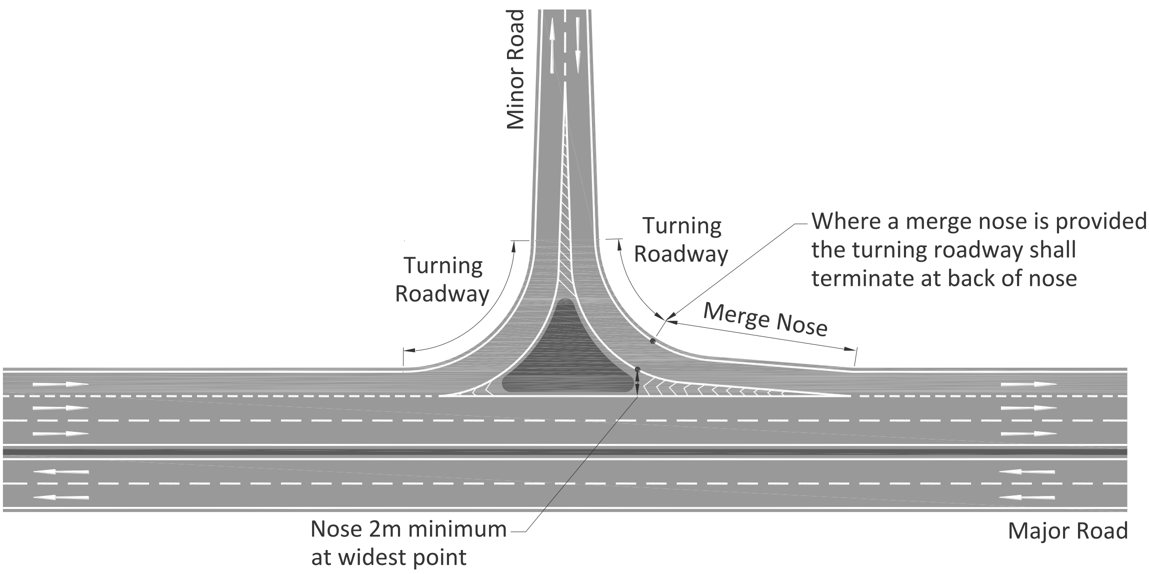


Figure 4.30: Turning Roadway Merge Nose at Priority Intersection

Table 4.10: Minimum Merge Auxiliary Lane Lengths on Priority Intersection

	Design Speed (km/h)	Minimum Auxiliary Lane Length - a (m)	Direct Taper Length - b (m)	Nose Length – c (m)
Rural	50	Not applicable [a]	Not applicable	Not applicable
	80	165	35	40
	100	285	35	40
	120	460	35	40
Urban	50	Not applicable [a]	Not applicable	Not applicable
	60 [b]	95	35	40
	70 [b]	95	35	40
	80	95	35	40
	100	150	35	40

Notes:

[a] At lower design speeds and where an auxiliary lane cannot be accommodated traffic, shall be required to yield.

[b] 60 km/h and 70 km/h design speeds may occur where merging auxiliary lane is required on to a ramp.

Merging Tapers

In certain cases, due to right-of-way constraints, there is not enough space to provide auxiliary lane. In such cases, merging taper can be provided, as illustrated in in **Figure 4.31**, although parallel auxiliary lane is the preferred option. Merging tapers shall not be provided at priority intersection on two-lane roads.

The initial width of the taper, which depends on the corner radius of turning lane, determined from **Table 4.8**, shall be decreased by a constant taper. The length of taper based on design speed of major road is given in **Table 4.11**. The minimum initial width of the merging taper shall be 4.0 m. Merging taper shall be precede by a short nose of 40 m length on dual carriageways with design speed of 120 km/h. The minimum width at the back of the nose should be 2 m as illustrated in **Figure 4.30.**

Table 4.11: Taper Length on Priority Intersection Merge

Design Speed (km/h)	Merging Length – a (m)
80	90
100	110
120	130

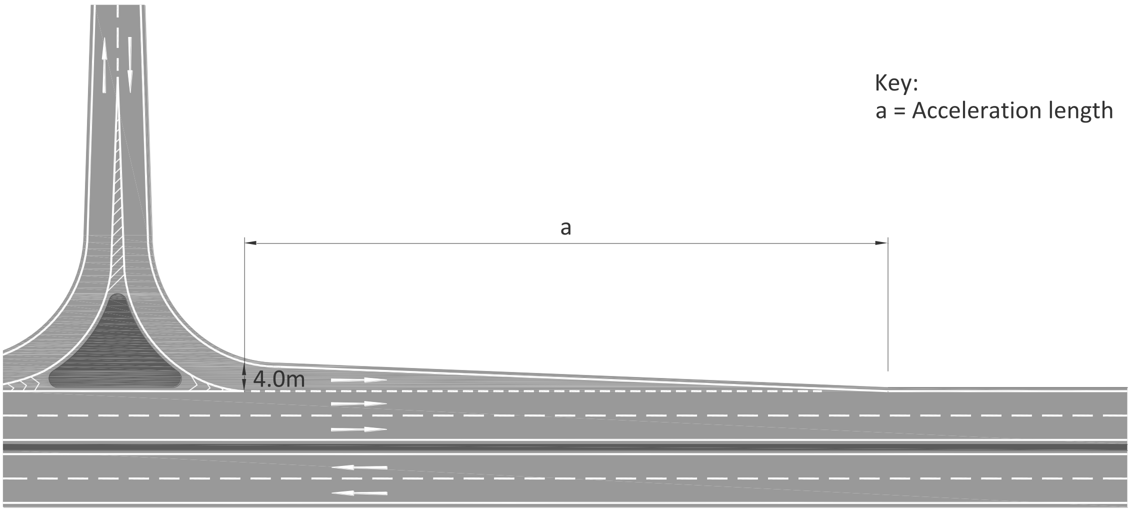


Figure 4.31: Alternative Taper Merge

Minor Road Left-Turn Approach where an Auxiliary Lane is not provided

On multi-lane roads with design speeds of 80 km/h and lower, the merging vehicles shall be required to yield at the intersection. The angle of incidence between the yielding traffic and the mainline roadway shall not be less than 70 degrees. **Figure 4.32** shows such a design. This design is also applicable to those roads where auxiliary lanes cannot be accommodated due to weaving or site constraints.

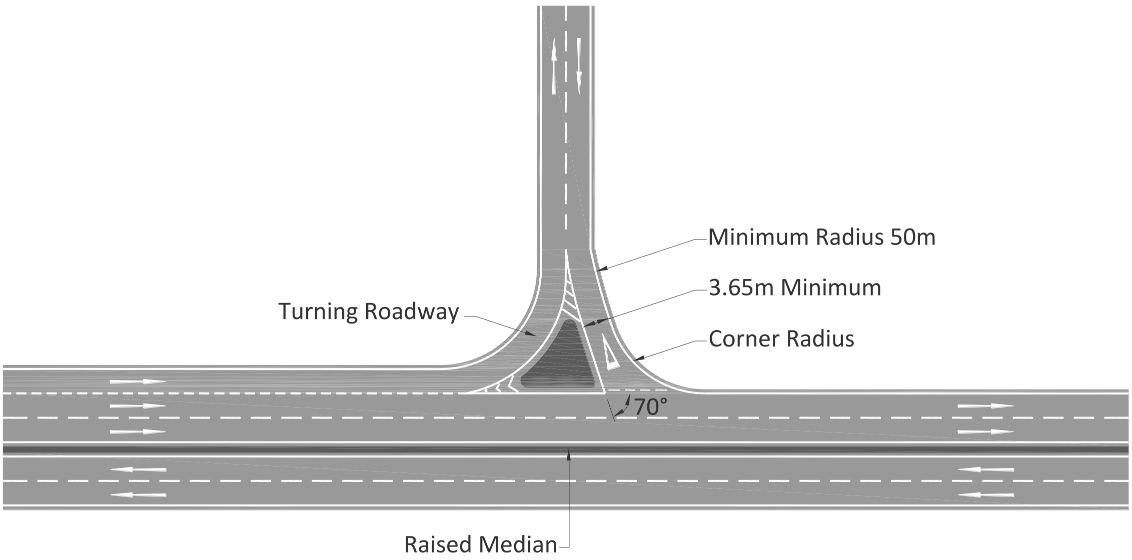


Figure 4.32: Left-Turn Approach Where an Auxiliary Lane Is Not Provided

4.5.5.4 Major Road Right-Turn lanes on Undivided Roads

In order to reduce the number of traffic conflicts on undivided roads, the right-turn movement from major road can be separated by including right-turn lanes at the intersection and channelized by introduction of divisional ghost islands or physical islands. Right-turn lanes provide a space for deceleration and queuing of turning vehicles leaving the major road. Channelization also aids the passage of through traffic, right-turning vehicles. Additionally, they provide space for warning and directional signage at an intersection.

Figure 4.33 and **Figure 4.34** show typical layouts of right-turn lanes with ghost islands and physical divisional islands.

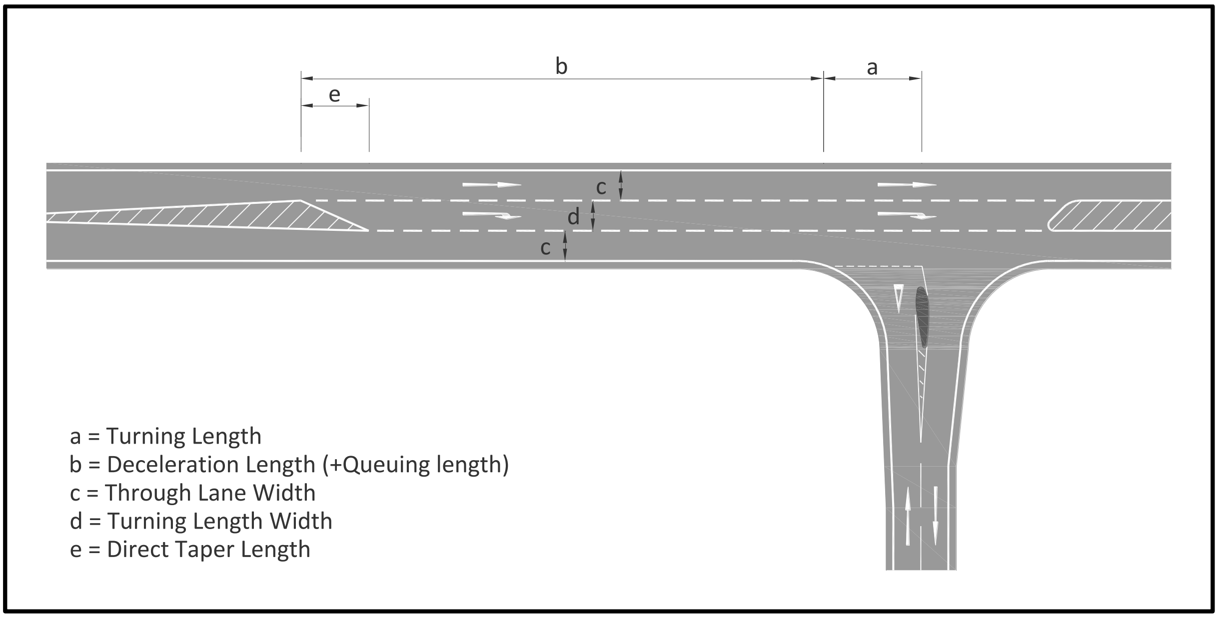


Figure 4.33: Priority Ghost Island T-Intersection with Right Turn Lane

Components Right-Turn Auxiliary Lane Length

The length of the right-turn lanes for turning vehicles consists of three components. These components are illustrated in **Figure 4.33** and **Figure 4.34**:

- Deceleration (or acceleration) length
- Storage or queuing length and turning length,
- Entering or exiting taper

Ideally, the total length of the auxiliary or left-turn lane shall be the sum of the length for these three components. Assuming a moderate amount of deceleration occurs within the through lanes is acceptable, as is assuming that deceleration occurs over the taper length.

Turning Length (T_l)

The turning length is the length provided for long vehicles, so that they can position themselves for the right turn. The preferred value of turning length is 10 m, irrespective of the design speed, gradients or type of intersection. It shall be measured form the centreline of the minor road.

Deceleration Length (L_{dl})

The deceleration lengths on channelized right-turns are provided in **Table 4.12**.

Queue Length (L_q)

The right-turn auxiliary lane shall be long enough to store the expected turning vehicles likely to accumulate during critical period. For most cases, it can be assumed that one to four vehicles will be queued during the design hour. 8 m per vehicle comprising length of vehicle plus gap shall be sufficient.

Storage requirements can also be estimated by using approved traffic analysis software. For low right turning volume, a minimum of 10 m queuing length for one vehicle shall be sufficient.

Direct Taper Lengths (L_{dt})

The direct taper lengths provided in **Table 4.9** for left turn diverge are also applicable for right turn diverge. Where an intersection is downstream from a crest vertical curve, the taper should be started farther upstream on the upgrade, so that it will be visible to the approaching drivers.

Right-Turn Auxiliary Lane Widths (d)

The width of the right-turn auxiliary lane shall be equal to the adjoining through lane. For reconstruction projects, in which the cross section is limited due to right-of-way or other constraints, lane widths may be reduced to 3.3 m.

Table 4.12: Minimum Channelized Right-Turn Deceleration Length (L_{dl})

Design Speed of Major Road (km/h)	Deceleration Length - L_{dl} (m)
50	55
60	75
70	100
80	130
100	200
120	Not Applicable

4.5.5.5 Channelizing Islands and U-Turn

The details of channelizing Islands and U-turns are provided in Section-4.6.4 to 0 in Signalized Intersections.

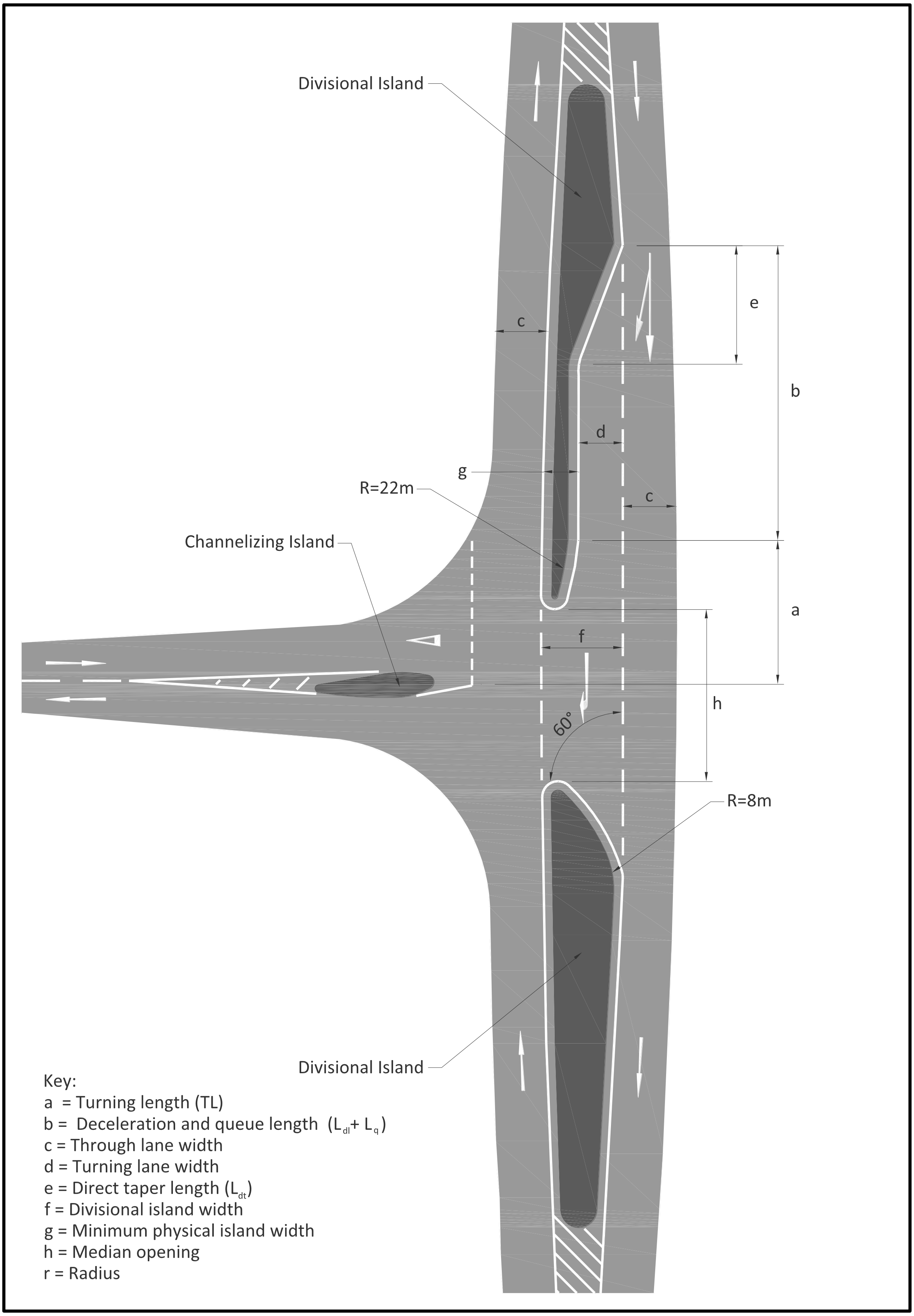


Figure 4.34: Priority T-Intersection with Right-Turn Lane

4.6 SIGNALIZED INTERSECTION

4.6.1 Types of Signalized Intersections

Figure 4.35 to **Figure 4.37** illustrates examples of different types of signalized intersections. No two intersections are alike. The examples provided are for guidance only. The additional features, such as channelizing islands, median openings and designated turning lanes may vary based on the requirements.

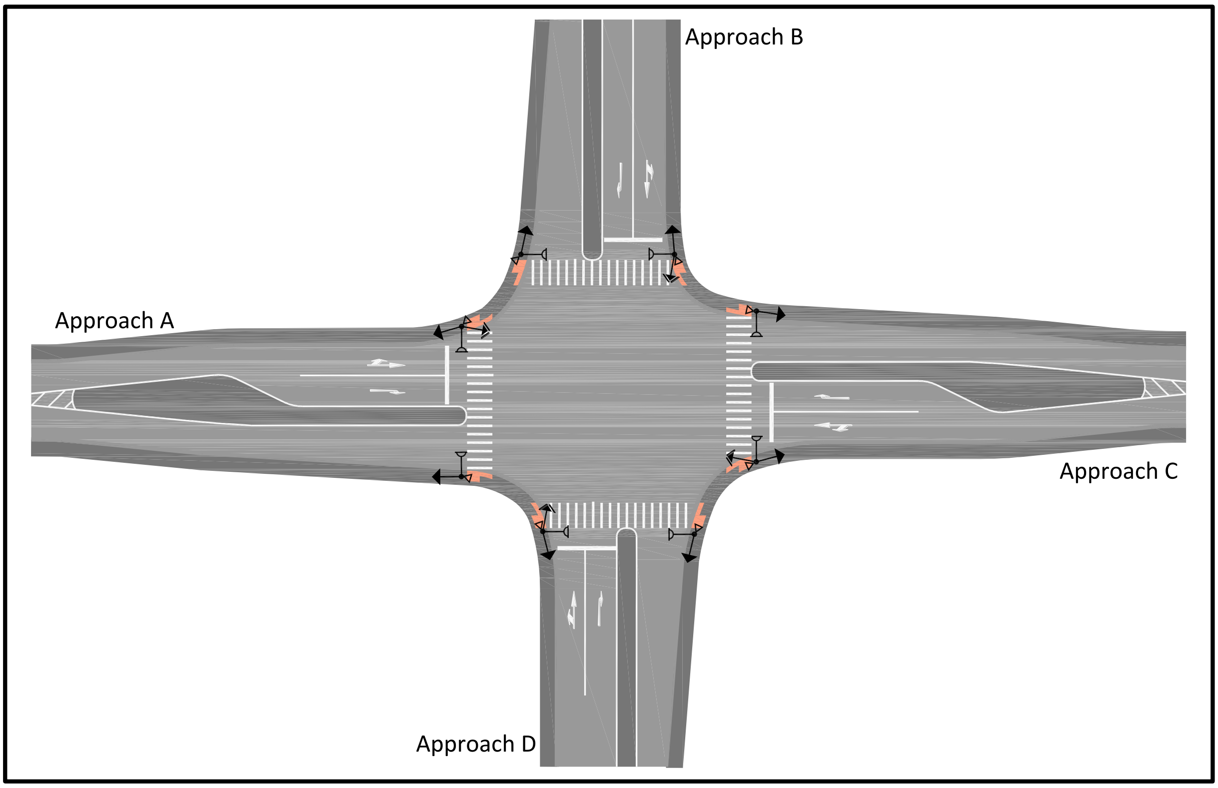


Figure 4.35: Example of Signalized Crossroads

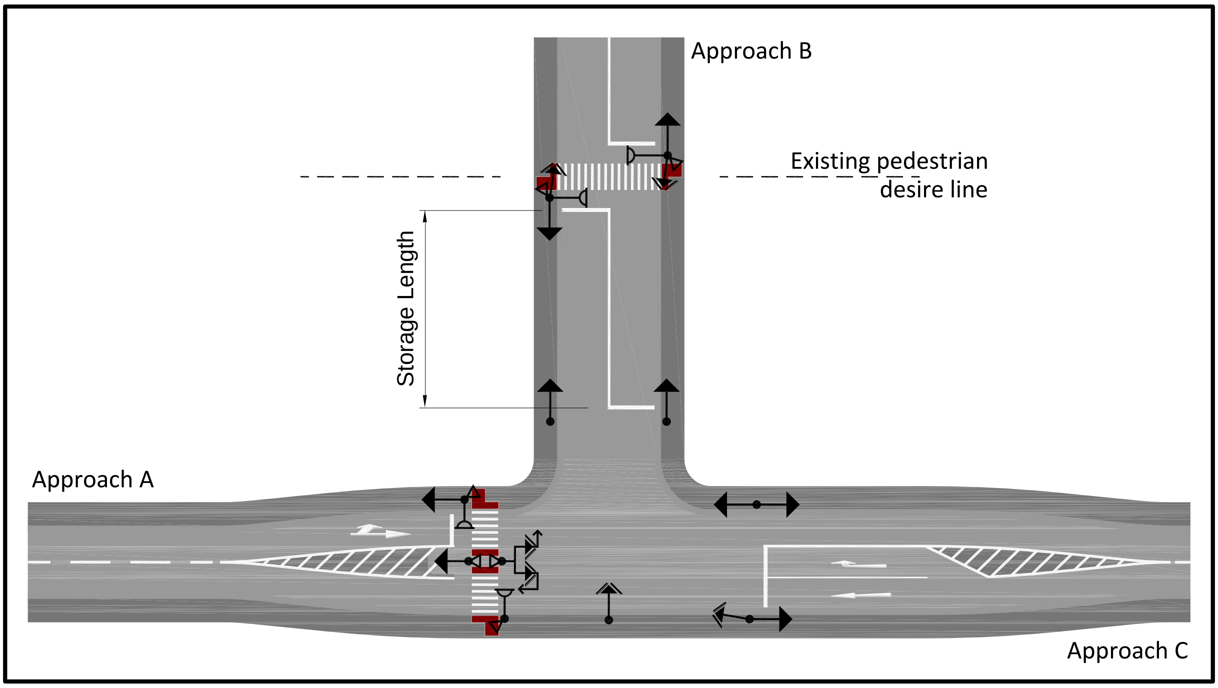


Figure 4.36: Example of a Small Signalized T-Intersection

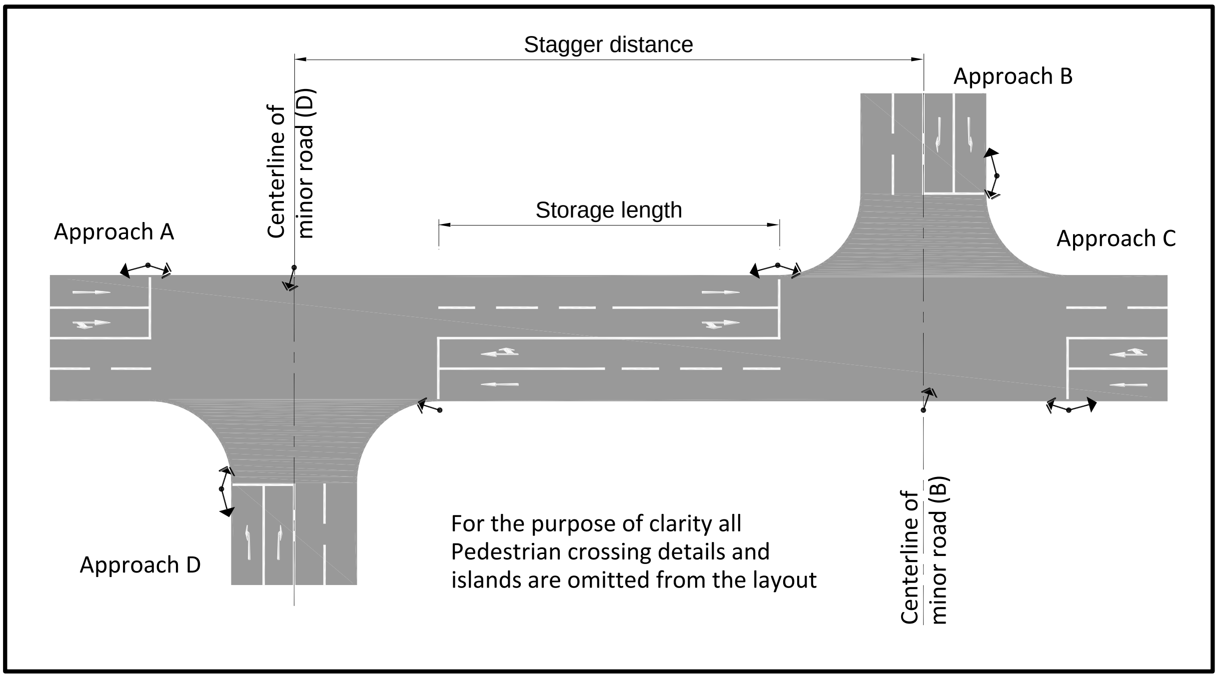


Figure 4.37: Example of Signalized Staggered Intersections

4.6.2 Design Controls

4.6.2.1 Design Speed

Several design criteria of intersection depends on the design speed of approach speed of vehicles. The design speed of each approach of the intersection can be determined by referring to Chapter 3-GEOMETRIC DESIGN ELEMENTS of this manual.

4.6.2.2 Visibility

All signalized intersection approaches shall have the minimum visibility equal to at least minimum stopping sight distance for the approach design speed provided in **Table 3.1** in Chapter 3-GEOMETRIC DESIGN ELEMENTS of this manual. The visibility envelope shall be increased to include the height of the signal head.

The area used for evaluating the visibility within the intersection, between drivers at each stop line or between drivers and pedestrians is termed inter-visibility zone, as shown in **Figure 4.38**. For each approach, the inter-visibility zone begins from 2.5 m behind the stop line across the full roadway width. To see the full extent of pedestrian crossing and its approach, the inter-visibility zone should also include the full hard standing area for pedestrians as shown in approaches A and B in **Figure 4.39**.

Major obstructions to inter-visibility within the zone, such as buildings, should be avoided. Minor obstructions, such as signal posts, sign supports, lighting columns, guardrails etc., are unavoidable, and the designer should consider minimizing the obstructive effect while placing these facilities.

The staggered intersection will include two independent inter-visibility zones, which will overlap to form a single zone, if the stagger distance is reduced.

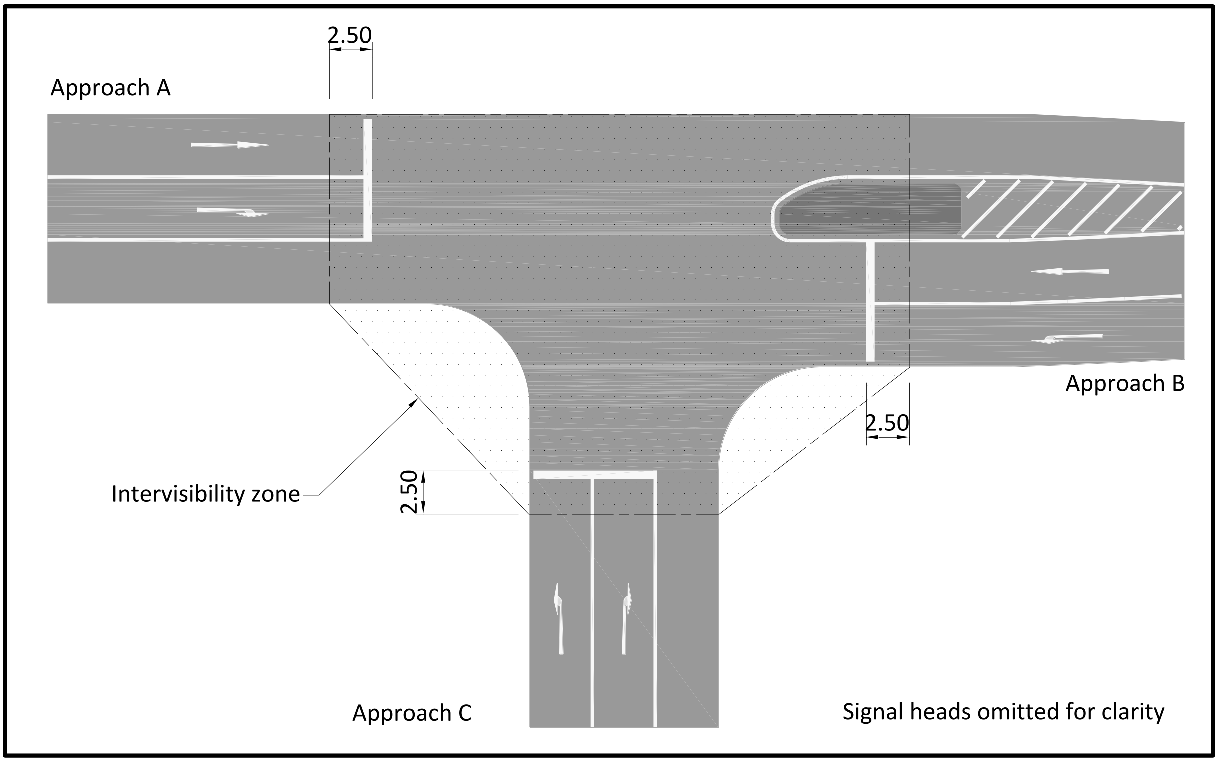


Figure 4.38: Intersection Inter-visibility Zone without Pedestrian Crossings

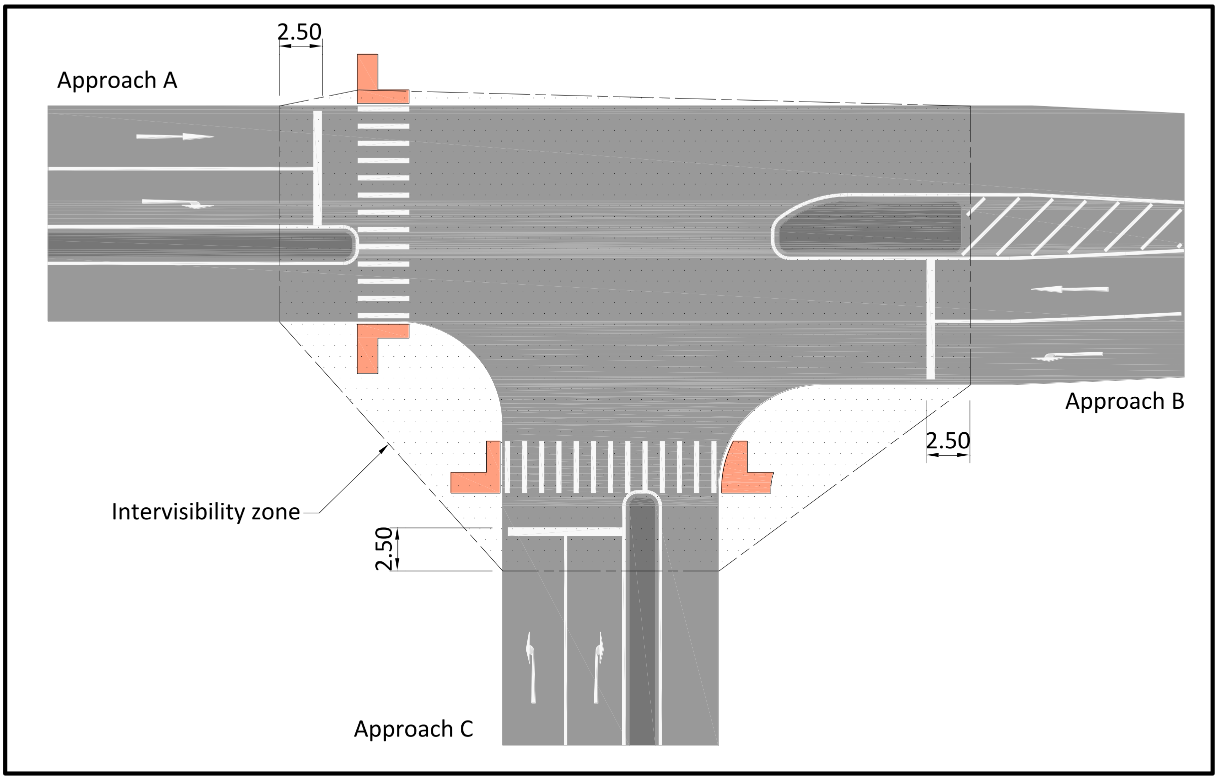


Figure 4.39: Intersection Inter-visibility Zone with Pedestrian Crossings

4.6.2.3 Swept Paths Requirements

Swept path should be checked for all permitted turning movements on a signalized intersection. The swept path of the design vehicle can be formed by any available simulation software of vehicle movement, such as AutoTurn or Vehicle Tracking, over computer generated intersection design model. It helps to identify the excess pavement area and minimize the required widening. It also aids the designer to provide allowance for swept path of large vehicles while preparing layout of the intersection. A typical example of swept path analysis on an intersection is shown in **Figure 4.40**.

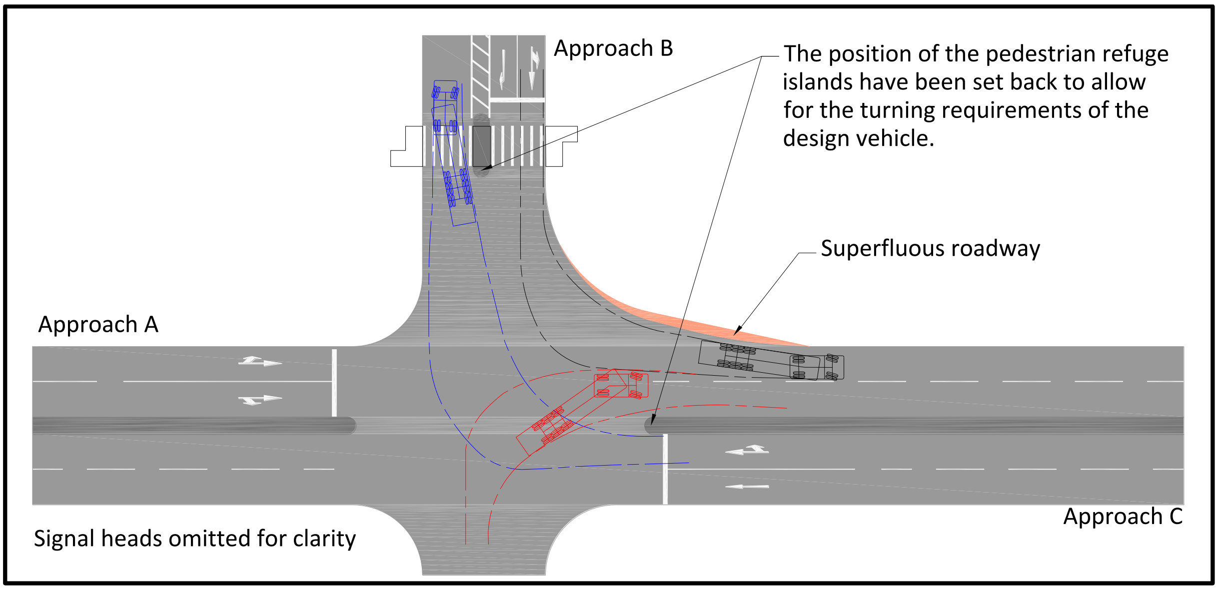


Figure 4.40: Examples of Swept Paths

For multiple channelized left or right turning lanes, the swept path should be checked for simultaneous turn of the design vehicle and a passenger car for two lanes, or cars for more than two lanes. If the turning volume of large vehicle is high, a larger swept path may need to be considered. The simultaneous turn of opposing right turn should also be checked. The location of stop lines and median island nose on both approaches should allow these movements to occur without overlapping.

In certain situations, the large turning vehicle may encroach upon the opposing lane. In such case, the stop line and pedestrian crossing can be set back from the intersection as shown in **Figure 4.41**. At small T-intersections, additional width for the swept path of large vehicles may be provided by setting back the sidewalk opposite the minor road, as shown on **Figure 4.42**.

Simultaneous right turn should also be checked. At least 2 m clearance should be provided between right turn opposing vehicles as shown in **Figure 4.43**. If the turning volumes and speeds are high (for example, at single-point urban diamond interchanges), the clearance should be increase to 3 m. Median can be introduced to separate right turning traffic from the through traffic. Opposing right-turning approach lanes may be aligned to assist simultaneous right turns.

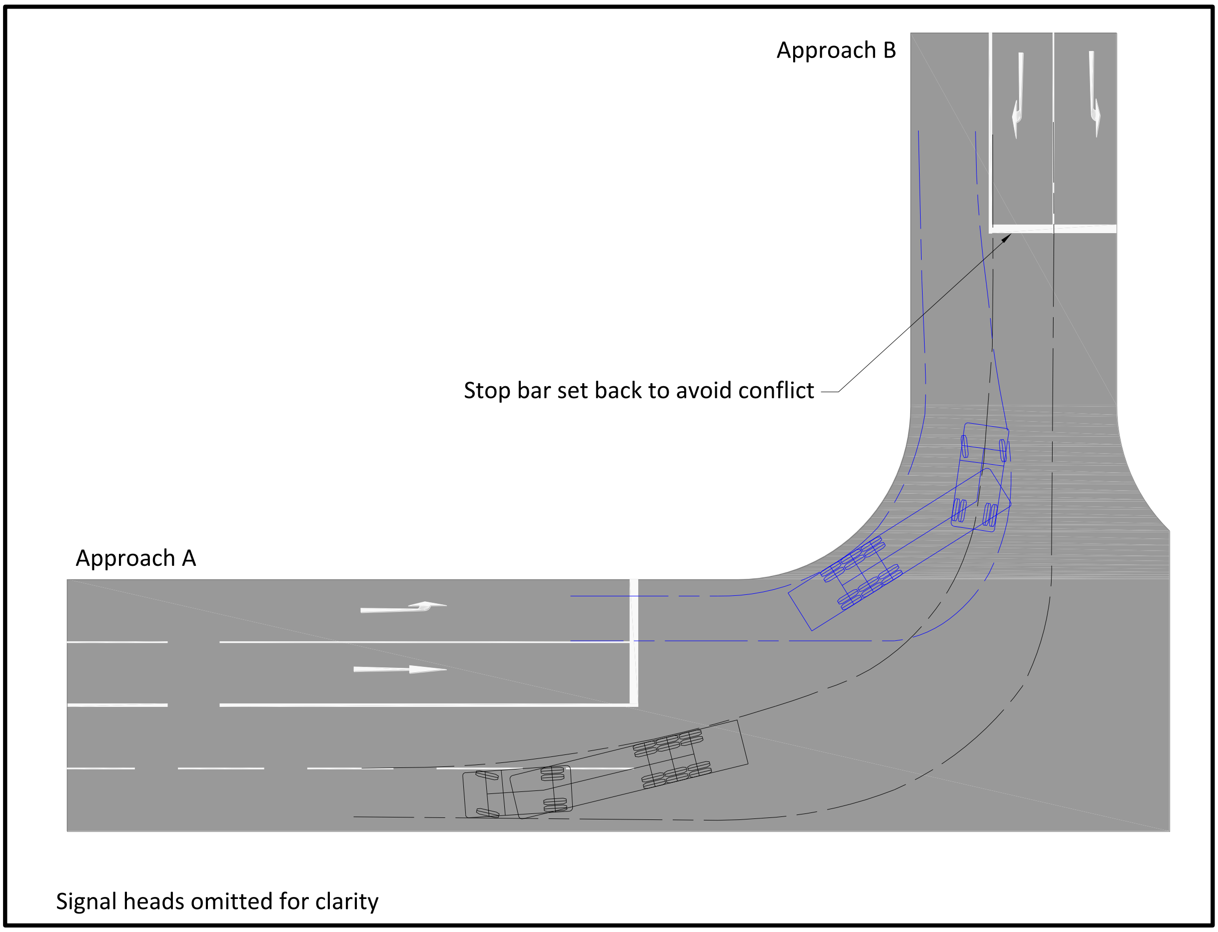


Figure 4.41: Stop Lines That Are Set Back to Accommodate Swept Path of Large Vehicles

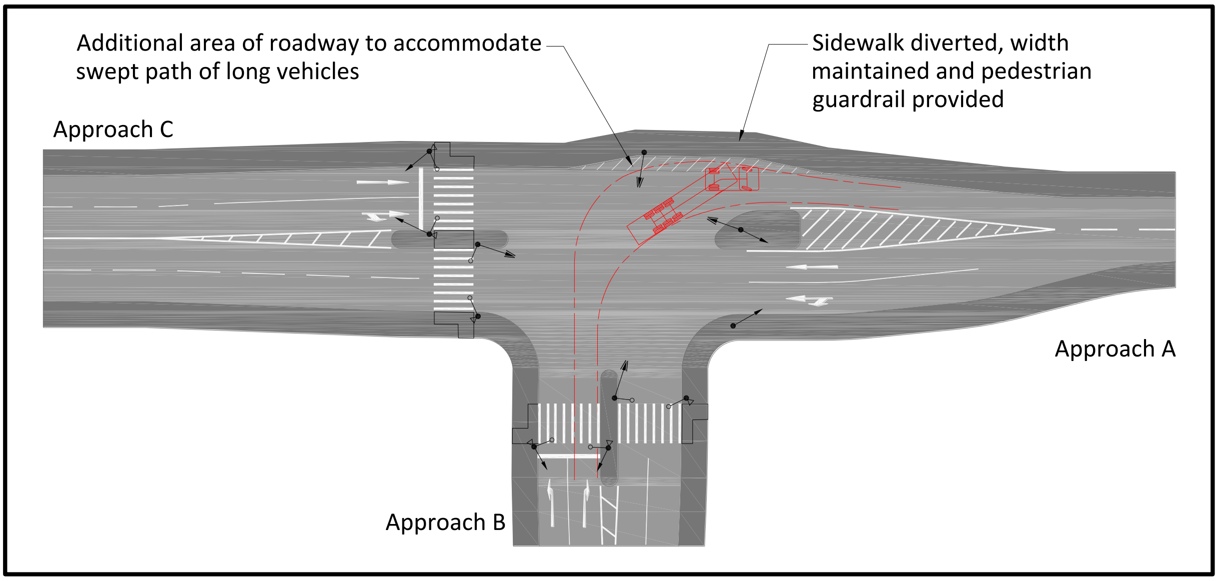


Figure 4.42: Localized Widening at T-Intersection to Accommodate Swept Path of Large Vehicles

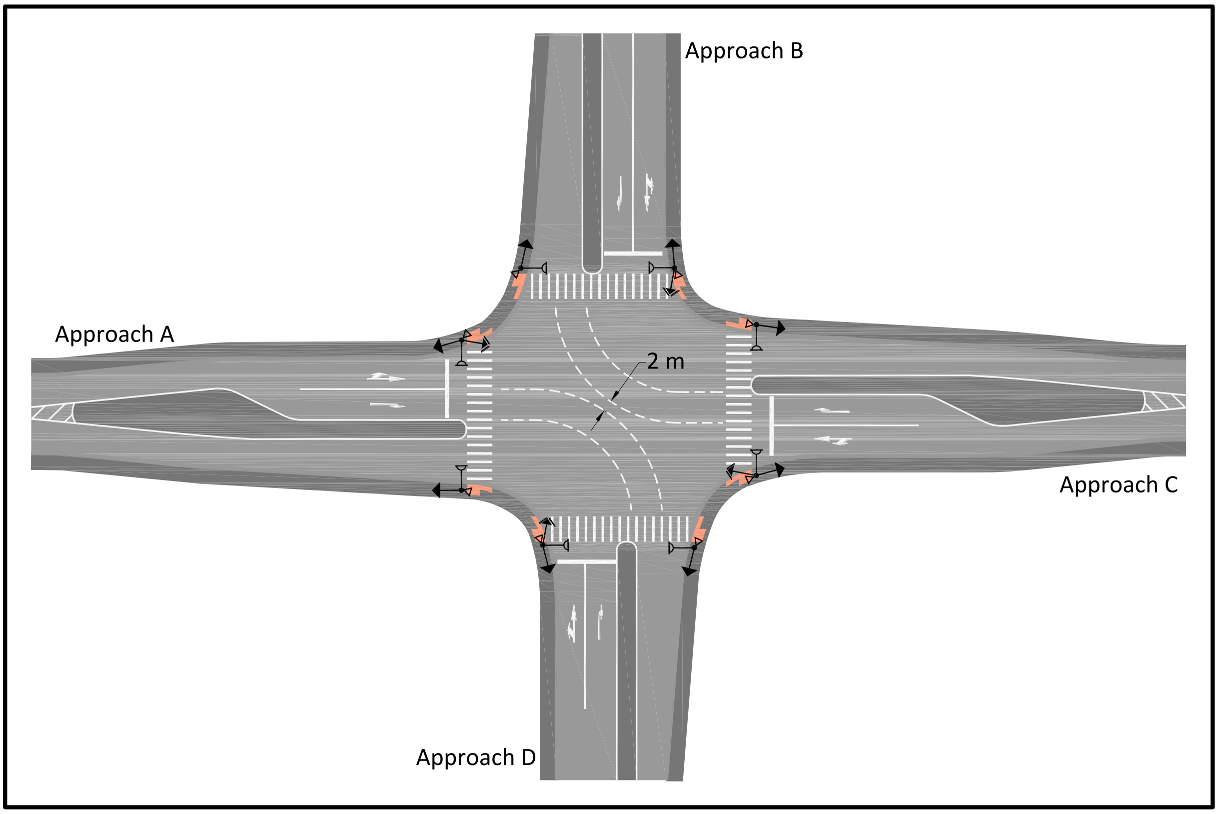


Figure 4.43: Simultaneous Right Turns

4.6.2.4 Lane Continuity

The number of through lanes at the stop line shall be maintained across the intersection. If it is necessary to reduce the number of through lanes on the departure, the lane reduction should be beyond the intersection inter-visibility zone over a distance of at least 100 m for a single-lane measured from the limit of the intersection inter-visibility zone, as shown on **Figure 4.44**. The lane shall be dropped beyond the intersection at the taper rates provided in **Table 4.13.**

Table 4.13: Roadway Widening and Lane Drop Taper Rates at Intersections

Design Speed (km/h)	Taper Rate*
20	1:10
30	1:15
40	1:20
50	1:25
60	1:30
70	1:35
80	1:40
90	1:45
100	1:50
110	1:55
120	1:60
130	1:65

* Taper rates provided also are applicable to calculate the required length to drop lanes at the intersections.

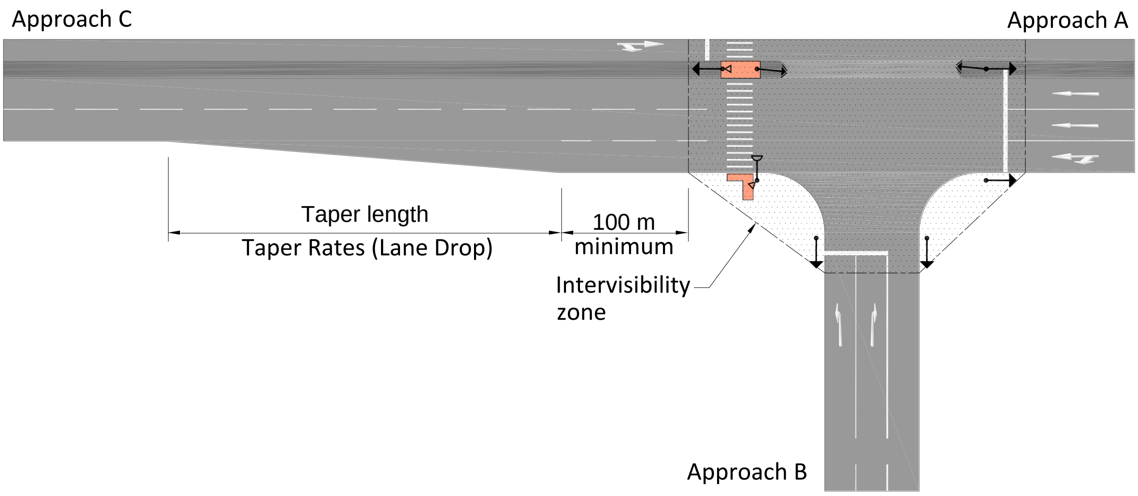


Figure 4.44: Lane Continuity through Intersection Inter-visibility Zone

4.6.3 Geometric Design Details

4.6.3.1 Corner Radii

Corner radii of signalized intersections shall be in accordance with Section 4.5.5.1, Corner Radii for Priority Intersections.

4.6.3.2 Approach Roadway Widths

The approach lane widths of signalized intersections shall be equal to through lane widths. For reconstruction or improvement projects, if the space is limited, the approach lane width of through lane can be reduces to 3 m for design speed equal to or less than 50 km/h.

4.6.3.3 Deceleration Lanes

At signalized intersections, dedicated left- or right-turn deceleration lanes can be provided to increase the capacity of intersection. Physical segregation of turning lanes can be done by channelizing islands discussed later in this chapter. Dedicated right-turn lanes or U-turn lanes shall be signalized. Dedicated left-turn lanes may be signalized or controlled by yield road markings and signs. The components of deceleration lanes are illustrated in **Figure 4.45** for undivided two-lane road, and **Figure 4.46** and **Figure 4.47** illustrates those for divided multi-lane roads for left and right turning lanes. Widening of roadway to include divisional island prior to introducing left turn lane at intersection, as shown in **Figure 4.45**, shall be done according to taper rates provided in **Table 4.13**. The deceleration lane widths at signalized intersections shall be equal to through lane widths. For reconstruction or improvement projects, if the space is limited, the lane width can be reduced to 3.3 m for design speed equal to or less than 50 km/h and 3.5 m for 80 km/h or less.

Deceleration Length

The deceleration lane lengths are provided in **Table 4.14**. The taper length is included deceleration lane length as shown in **Figure 4.46**.

Taper Length

Straight-line taper are preferred. A short curve is desirable at each end of long tapers. Where curves are used at the ends, the tangent section should be about one-third to one-half of the total length. The taper rate shall be 8:1 [L:T] for design speeds up to 50 km/h and 15:1 [L:T] for design speeds of 80 km/h and greater.

Table 4.14: Deceleration Length at Segregated Left- and Right-Turn Approach Lanes

Design Speed (km/h)	Deceleration Length (m)
50	55
60	75
70	100
80	130
100	200

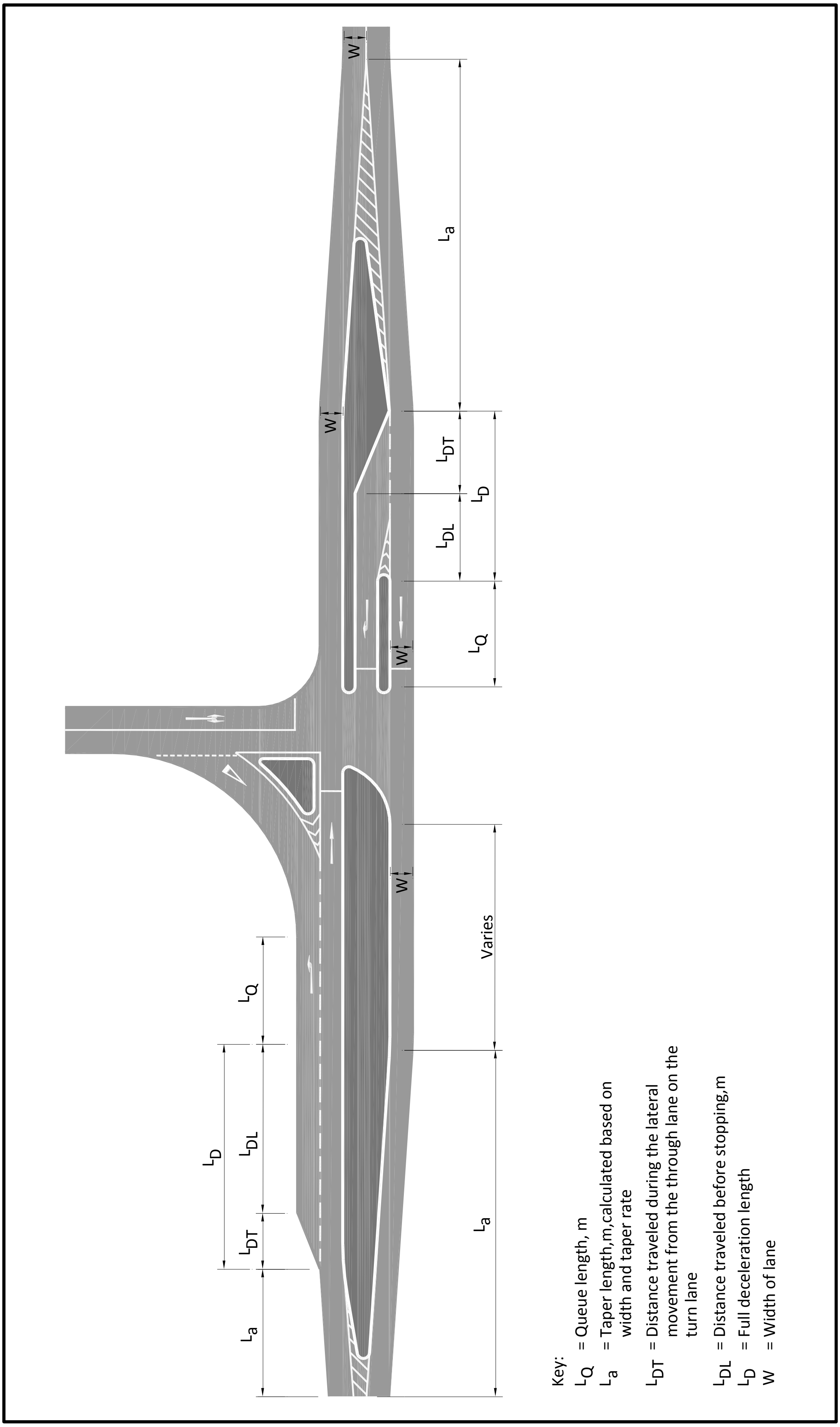


Figure 4.45: Deceleration Lanes at Intersection Approaches

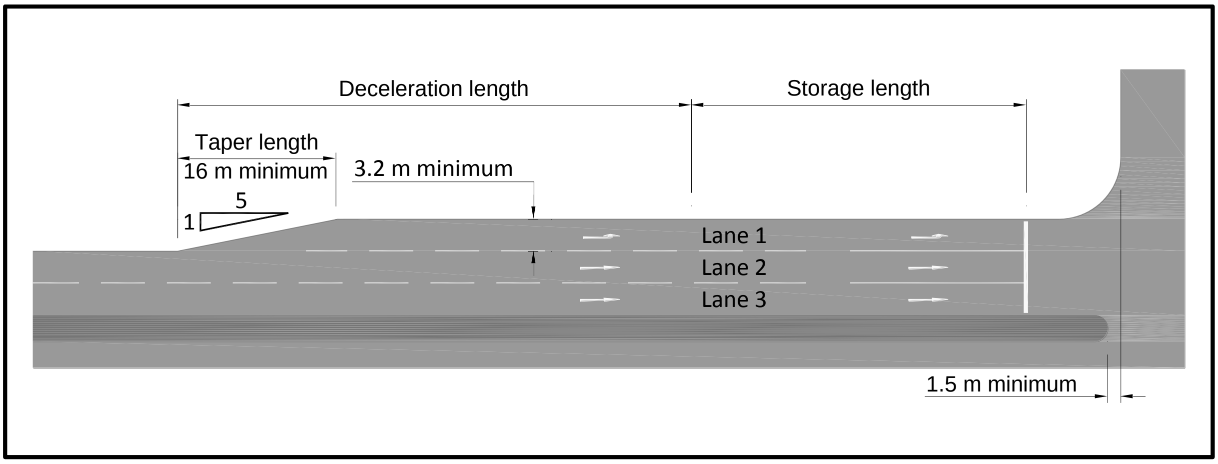


Figure 4.46: Left-Turn Approach Lane

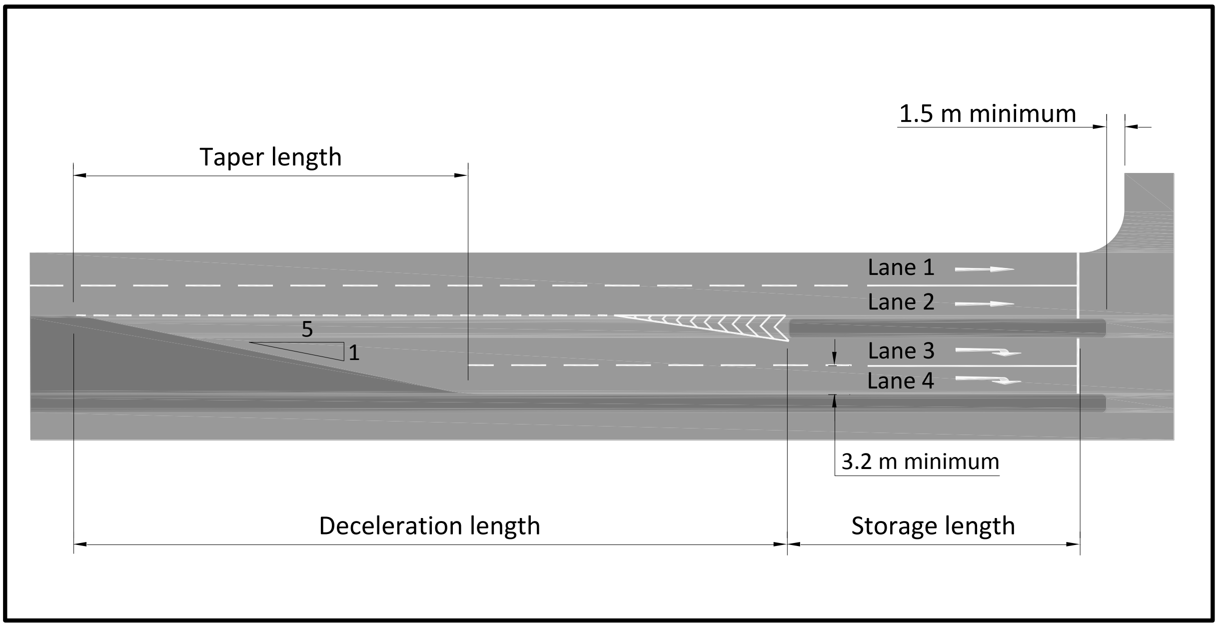


Figure 4.47: Right-Turn Approach Lane

Storage Length

The auxiliary lane shall be long enough to store the expected turning vehicles likely to accumulate during critical period. The storage length measures from the stop line should be sufficient, over which vehicles can queue without obstructing or being obstructed by vehicles in the adjacent lane. The storage length of left and right-turn approach lanes should be designed to meet the capacity requirements of the intersection based on a traffic analysis of the intersection operation. The following equation can be used as a general guide in initial plan development, (Source: *UK, DMRB 2004*):

Length of 95th percentile queue in a lane (m) = design hour volume for the turning movement (vph)/3

Provision of storage lanes for left-turning traffic is optional; their provision and length should be justified based on traffic demands and operational conditions.

Right-turn storage lanes should be separated by a channelizing island. The width of the island should be a minimum of 1.5 m, with adequate kerb offset from traffic lanes provided. The minimum island length upstream from the signal stop line should be 10 m. For a single lane right-turn or U-turn facility, the minimum pavement width between the island and median should be 6 m.

Where midblock or displaced pedestrian crossings are installed as shown in **Figure 4.48**, designers should provide adequate storage length to avoid traffic queuing in the crossing.

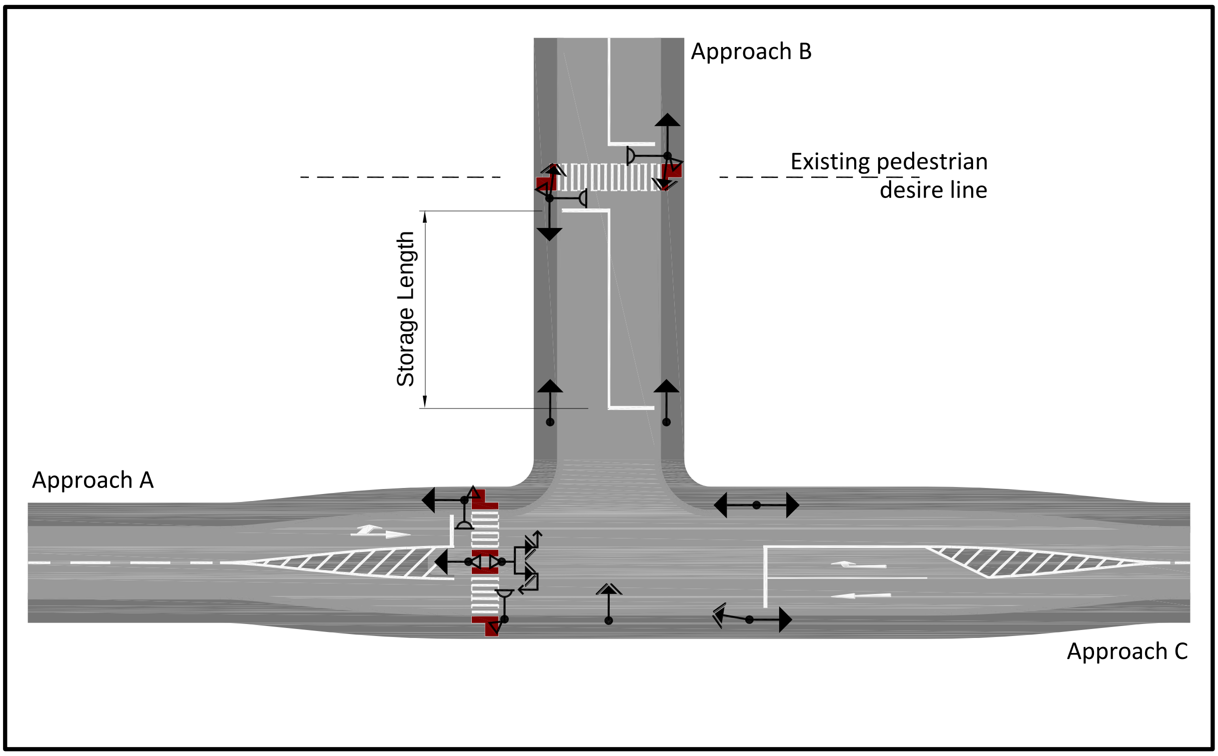


Figure 4.48: Displaced Pedestrian Crossing

4.6.3.4 Acceleration Merging Lanes at Left-Turn Lanes

Acceleration lanes are not desirable at signalized intersections. A yield control or stop control shall be sufficient for left turning vehicles when entering a road with design speed of 50 km/h or less. Acceleration lanes are advantageous on high volume roads where openings between vehicles in peak hour traffic are short and infrequent. A merging acceleration lane should only be provided when entering a multi-lane road with a design speed of 80 km/h or more.

Figure 4.49 shows a left-turn design with acceleration lanes. Design values for acceleration length are given in **Table 4.15** and adjustment factors for grades are given in **Table 4.16**. For other geometric factors, including merge nose and taper lengths, Section-4.5.5.3 on “Left-Turn Merging Auxiliary Lanes and Tapers” for Priority Intersections can be referred.

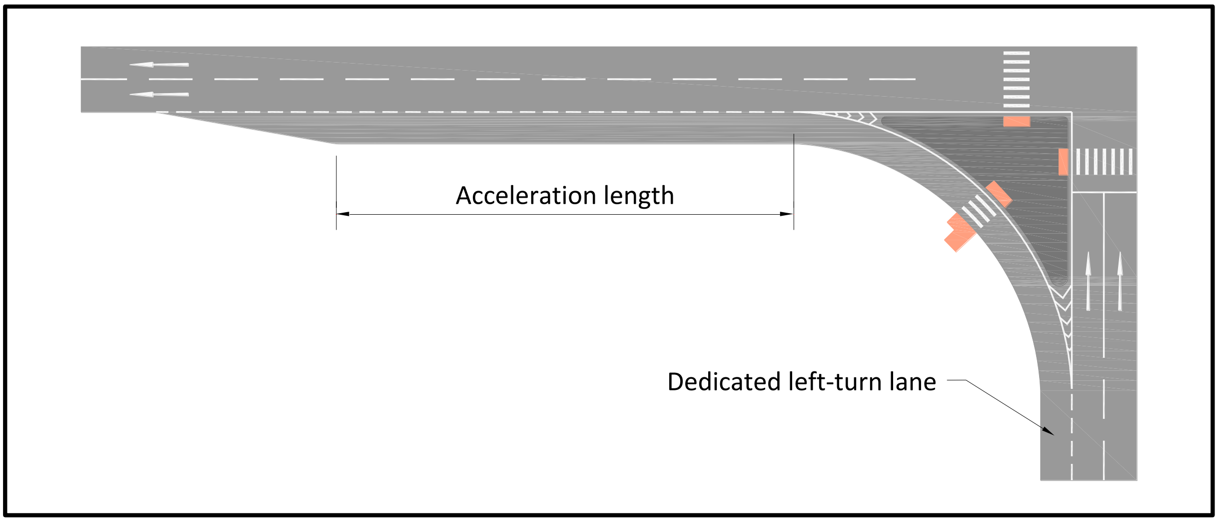
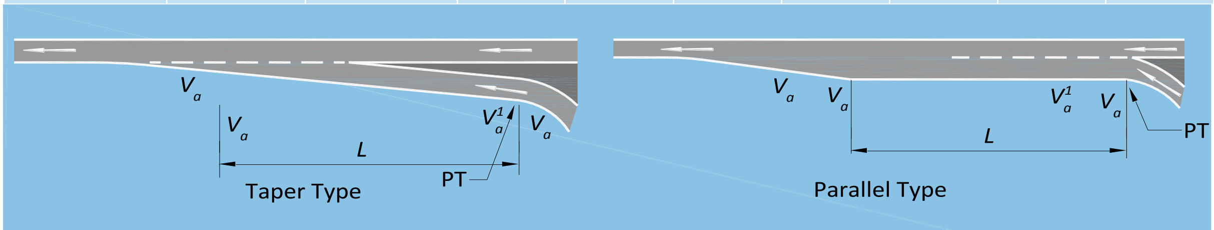


Figure 4.49: Left-Turn Lane with Acceleration Lanes

Table 4.15: Minimum Acceleration Lengths for Entrance Terminals with Flat Grades of Two Percent or Less

Highway Design Speed, V (Km/h)	Speed Reached, V_a (Km/h)	Acceleration Length, L (m) for Design Speed of Entrance Curve, V' (km/h)							
		Stop Condition	20	30	40	50	60	70	80
		Initial Speed V'_a (km/h)							
		0	20	28	35	42	51	63	70
50	37	60	50	30	-	-	-	-	-
60	45	95	80	65	45	-	-	-	-
70	53	150	130	110	90	65	-	-	-
80	60	200	180	165	145	115	65	-	-
90	67	260	245	225	205	175	125	35	-
100	74	345	325	305	285	255	205	110	40
110	81	430	410	390	370	340	290	200	125
120	88	545	530	515	490	460	410	325	245



Note: Uniform 50:1 to 70:1 tapers are recommended where lengths of acceleration lanes exceed 400m.

(Source: *AASHTO's A Policy on Geometric Design of Highways and Streets, 2011* [2])

Table 4.16: Acceleration Lane Adjustment Factors as a Function of Grade

Design Speed of Highway (km/h)	Ratio of Length on Grade to Length of Level for Design Speed of Turning Curve (km/h) [a]					
	40	50	60	70	80	All Speeds
	3 to 4% Upgrade					3 to 4% Downgrade
60	1.3	1.4	1.4	-	-	0.7
70	1.3	1.4	1.4	1.5	-	0.65
80	1.4	1.5	1.5	1.5	1.6	0.65
90	1.4	1.5	1.5	1.5	1.6	0.6
100	1.5	1.6	1.7	1.7	1.8	0.6
110	1.5	1.6	1.7	1.7	1.8	0.6
120	1.5	1.6	1.7	1.7	1.8	0.6
	5 to 6% Upgrade					5 to 6% Downgrade
60	1.5	1.5	-	-	-	0.6
70	1.5	1.6	1.7	-	-	0.6
80	1.5	1.7	1.9	1.8	-	0.55
90	1.6	1.8	2.0	2.1	2.2	0.55
100	1.7	1.9	2.2	2.4	2.5	0.5
110	2.0	2.2	2.6	2.8	3.0	0.5
120	2.3	2.5	3.0	3.2	3.5	0.5

Notes:
[a] Ratio from this table multiplied by the length in **Table 4.15** gives length of speed change lane on grade.
(Source: *AASHTO's A Policy on Geometric Design of Highways and Streets, 2011* [2])

4.6.3.5 Staggered Signalized Intersections

Stagger Distances

The stagger distance at an intersection is the distance along the major road between the centrelines of the two minor roads, as shown on **Figure 4.50**. Where the stagger distance is greater than 250 m, the intersections shall be considered as two independent, signalized T-intersections.

With a stagger distance between 75 and 250 m, the intersections should be treated as two separate signalized T-intersections with the coordination of the traffic signals favouring the following major flows of traffic through the intersection (refer to **Figure 4.50**):

- Straight ahead on the major road (approach A to C and approach C to A)
- Flow to and from the two staggered approaches (approach B to D and approach D to B)
- A combination of these two traffic flows

Where the stagger distance is less than 75 m, the intersections should be considered a single signalized intersection.

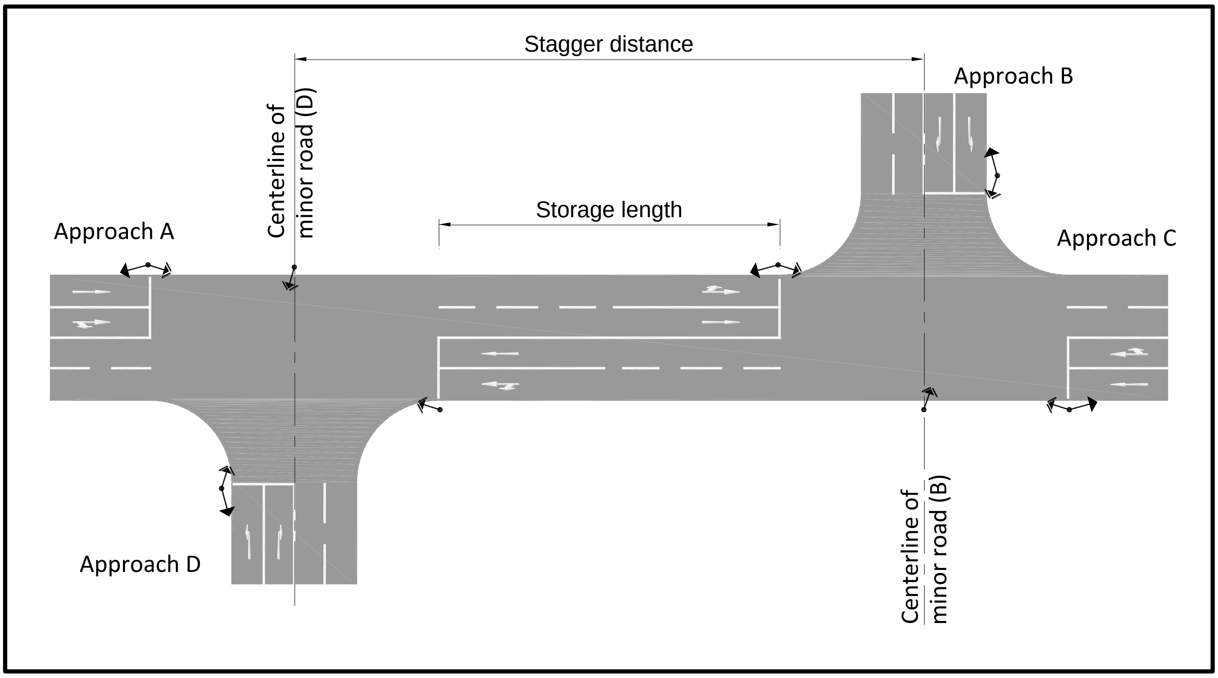


Figure 4.50: Stagger Distance and Storage Length

Storage Length

The storage length is the distance between the inner stop lines, as shown in **Figure 4.50**. The available storage length depends on the stagger distance and whether pedestrian crossing facilities are provided within the stagger distance. When the stagger distance is less than 75 m, it becomes more difficult to provide for inner stop lines, pedestrian crossing facilities, and associated signals. The shortest effective storage length is 15 m. With a storage length less than 15 m, the intersection should be a signalized crossroad with special consideration for the longer clearance distances. In such cases, the storage of turning vehicles shall be provided on outside approaches of the intersection. Suitable traffic modelling software can be used to verify the satisfactory operation of the staggered intersections.

4.6.3.6 Turning Roadways with Corner Islands

Where left-turn traffic movement is high, or significant delays are faced by left-turning vehicles then segregated left turn lane with a corner island should be considered. The minimum turning lane width and approach and departure radii shall be in accordance with **Table 4.17** based on the angle of intersection. The layout of turning lane shall be checked by swept path analysis of the design vehicle. **Figure 4.51** shows a typical example of exclusive left-turn lane with channelizing island without acceleration lane on rural intersection. These segregated left-turn lanes may either be yield controlled or signalized. The details of corner islands are discussed in Section-4.6.4.

Associated pedestrian crossing facilities should also be considered while making layout of the left-turn lane.

Uncontrolled left-turn lanes make pedestrians crossings more difficult because of increased vehicle speeds; therefore, it is important to consider the whole effect of the geometric layout on pedestrian crossing facilities.

Table 4.17: Typical Designs for Turning Roadways

Angle of Turn	Design Classification	Three Centred Compound Curve		Minimum Lane Width (m)
		Radii (m)	Offset (m)	
75	A	45-23-45	1.0	4.2
	B	45-23-45	1.5	5.4
	C	67-41-67	1.5	6.7
90	A	45-15-45	1.0	4.2
	B	45-15-45	3.4	6.4
	C	61-21-61	3.4	7.6
105	A	36-12-36	0.6	4.5
	B	46-11-46	3.5	8.8
	C	55-18-55	2.9	9.8
120	A	30-9-30	0.8	4.8
	B	46-9-49	3.2	10.0
	C	43-17-43	2.1	13.7
135	A	30-9-30	0.8	4.8
	B	46-9-46	3.0	11.6
	C	43-14-43	2.1	15.8
150	A	30-9-30	0.8	4.8
	B	46-9-46	2.7	12.8
	C	49-12-49	1.8	16.1

Notes:
Design classification:
A - Primarily passenger vehicles; permits occasional design single-unit trucks to turn with restricted clearances
B - Provides adequately for the SU-9 and SU-12 design vehicles; permits occasional WB-19 design vehicles to turn with slight encroachment on adjacent traffic lanes
C - Provides fully for the WB-19 design vehicle
(Source: *AASHTO's A Policy on Geometric Design of Highways and Streets, 2011* [2])

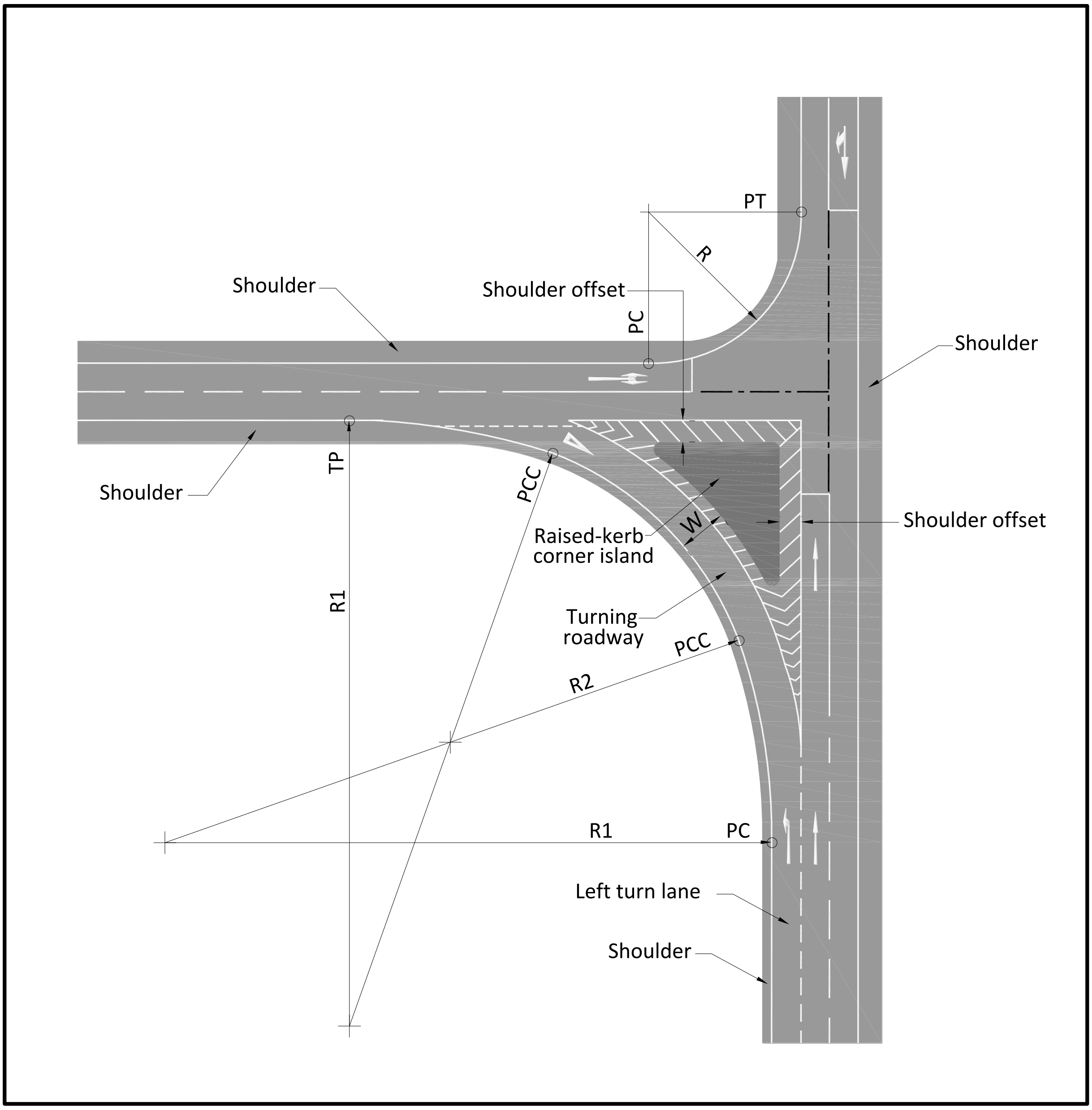


Figure 4.51: Left-Turn Lane without Acceleration Lanes on Rural Intersection

4.6.4 Channelizing Islands and Pedestrian Refuge

Channelizing island is an area between traffic lanes, provided for regulating conflicting traffic movement, providing areas for pedestrians refuge and traffic control devices. Unused areas can also be converted to islands to eliminate confusing traffic movements on spacious intersections. They also eliminate the unused areas on spacious intersections. Islands may be formed by raised kerbs or pavement markings using thermoplastic reflectorized striping. The shape and size of the island depends on the layout and dimensions of the intersection.

While designing channelizing islands, the horizontal and vertical geometry of the road should also be considered. Special care should be taken where the islands is beyond the crest vertical curve or on substantial horizontal curvature.

4.6.4.1 Divisional Islands

At intersections of undivided highways, divisional islands are often introduced to alert the driver of the crossroad ahead and to regulate the traffic through the intersection. Widening of roadway to include divisional island shall be done such that the follow path is clear to the driver.

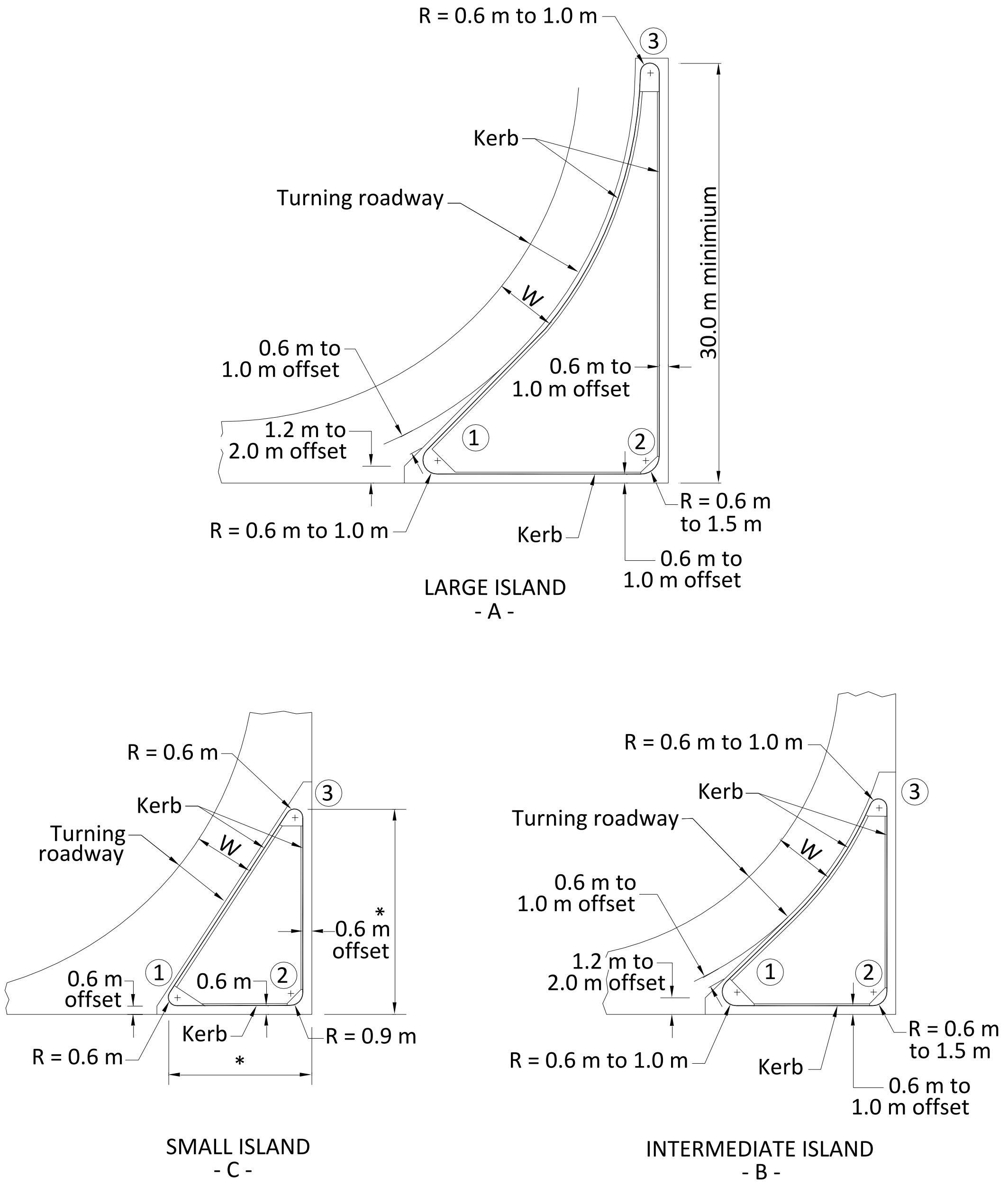
Reverse curve alignment shall be provided when the island is introduced in on tangent. The taper rate shall be in accordance with **Table 4.13** base on design speed. In high-speed rural roads equal to or greater than 80 km/h, the reverse curve radii shall be equal to or greater than 1,165 m. For design speed up to 70 km/h, the radii can be reduced to 620 m. When the alignment at intersection is on curve, the advantage should be taken to introduce the island without using reverse curve**.** On horizontal and crest vertical curves, the approach end of the kerbed island should be extended to be clearly visible to the driver.

The divisional island shall not be less than 1.2 m wide (absolute minimum 0.5 m in restricted space) and 6 m long. Kerbed divisional islands on high-speed roads shall not be less than 30 m.

4.6.4.2 Corner Islands

Where the left turning radius of inner edge of roadway is designed to accommodate semitrailer combination, the pavement area within the intersection becomes excessively large resulting in reduced proper traffic control. A corner island can be provided in such case, to reduce pavement area and to separate through and left turning traffic.

The island may be small, intermediate, or large depending on size. The area of a small kerbed corner island shall not be less than 5 m^2 for urban and 7 m^2 for rural roads. However, the desirable area is 9 m^2 is desirable for both. The length of any side of the corner island shall not be less than 3.5 m (4.5 m preferable) after rounding of corners. A large-kerbed corner islands are those one with side dimensions of at least 30 m. See **Figure 4.52** and **Figure 4.53** for details regarding kerbed corner islands with and without shoulders.



1. 2, 3 - designates a specific corner of island.
2. Ramp the 1 and 2 noses of Kerbed corner islands unless the Kerb function is for the protection of pedestrians, signals, light standards, or sign truss supports.
3. All corner radii are to the face fo Kerb at flowline.
4. Dimensions marked with an asterisk are controlled by the minimum area requirements of the island, whereas "W" is a minimum dimension.
5. All dimension in meters unless otherwise noted.

Figure 4.52: Details of Corner Islands

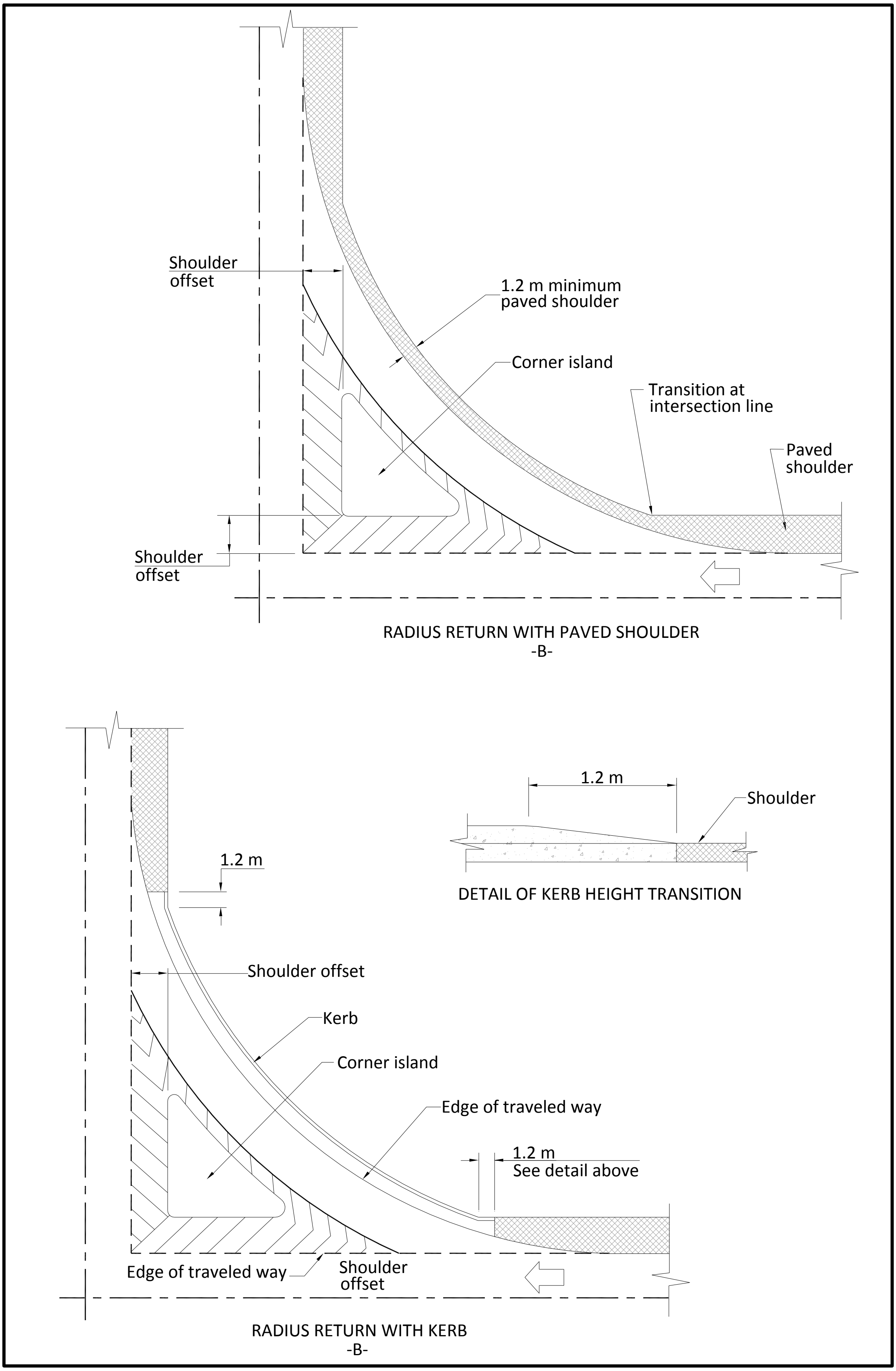
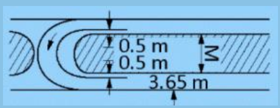
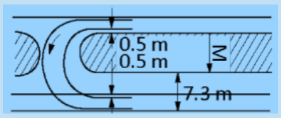
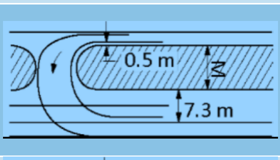
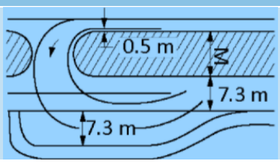


Figure 4.53: Shoulder and Kerb Radius Return Transition

4.6.5 U-Turns at Intersections

At intersections where U-turns are allowed, the roadway including shoulders and median should be wide enough to allow the design vehicle to turn from the right turn lane around the median into the roadway of the opposing traffic lanes. U-turns are not recommended from the through lanes. The minimum width of medians to accommodate U-turns by different design vehicles turning from the lane adjacent to the median is given in **Table 4.18**.

Table 4.18: Minimum Median Widths Needed for U-Turns

Type of Manoeuvre		M – Minimum Width of Median for Design Vehicle					
		P	WB-12	SU-9	BUS	SU-12	WB-19
		Length of Design Vehicle					
		5.7 m	15.0 m	9.0 m	12.0 m	12.0 m	21.0 m
Inner lane to inner lane		9 m	18 m	19 m	19 m	23 m	21 m
Inner lane to outer lane		5 m	15 m	15 m	16 m	19 m	17 m
Inner lane to shoulder		2 m	12 m	12 m	12 m	16 m	14 m
Inner lane to jug handle		2 m	10 m	10 m	10 m	19 m	12 m

(Source: *AASHTO's A Policy on Geometric Design of Highways and Streets, 2011* [2])

4.6.6 Median U-Turns

A mid-block U-turn can be provided on major road between signalized intersections, if downstream-signalized intersection is too far, the increased U-turn traffic on downstream-signalized intersection significantly decreases its overall capacity or surrounding land-use require access that is more direct. See **Figure 4.54**. The U-turn shall not be provided on roads with design speed greater than 80 km/h.

The location of U-turn should not interfere with the operations of the next downstream-signalized intersection or roundabout. As a guideline, there should be at least a distance of 300 m between the U-turn to the centre of the signalized intersection. Decision sight distance shall also be checked while deciding the location of U-turn.

If median U-turn is designed for larger vehicles, then additional pavement opposite to U-turn shall be provided to accommodate the swept path of large design vehicles.

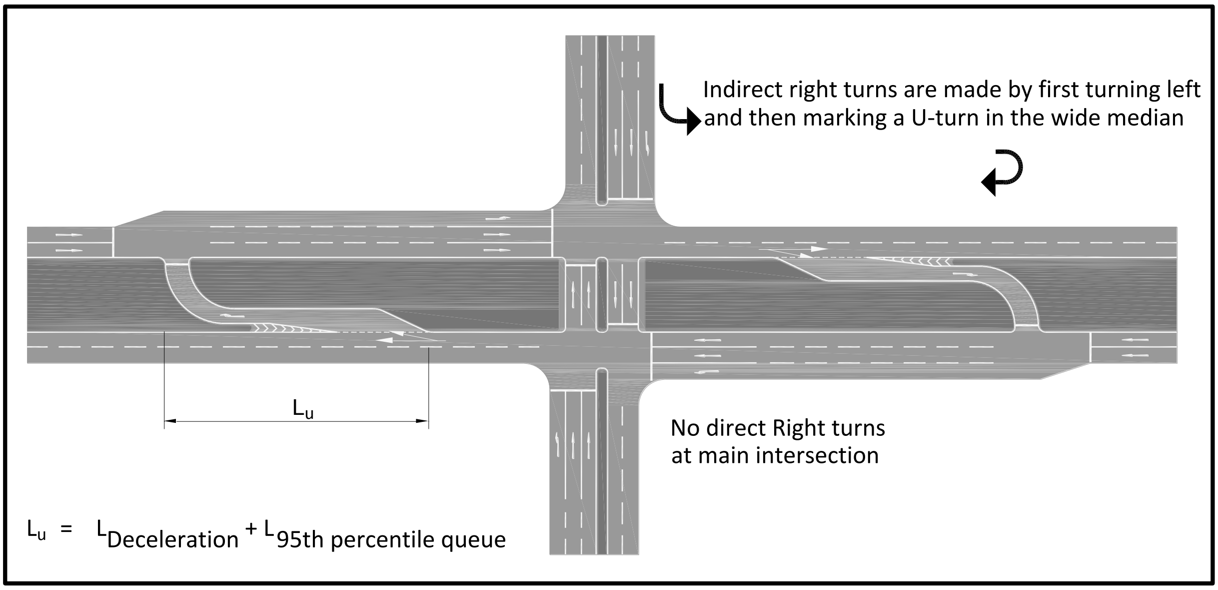


Figure 4.54: Example of a Median U-turn Intersection

4.6.7 Signals

4.6.7.1 Visibility and Positioning

Signals on the intersections shall not be located where they would obstruct the pedestrians crossing path of obstruct the visibility of drivers. The requirements of foundation, maintenance, and all technical approvals shall be determined at the early stage of layout preparation. Additional signal heads on tall posts above the primary signal head or on mast arms above the roadway may be provided to improve the signal visibility. Vertical clearances should be checked if mast arm signal heads are provided.

4.7 ROUNDABOUTS

Intersection on which the traffic circulates the clockwise around a central island is called a roundabout. The traffic entering the roundabout must yield to the vehicles on the circulatory carriageway. Roundabouts in addition to function as an intersection, also provide safe U-Turn facility, ease the change in cross section of road and aid the heavy right turn flow.

4.7.1 Types of Roundabouts

The types of roundabouts and their key features are given in **Table 4.19** and are described below.

4.7.1.1 Mini-Roundabouts

Mini-roundabouts are appropriate on residential or recreational local roads where volume of large vehicle is low. The inscribed circle diameter (ICD) of mini-roundabouts range between 18 m to 22 m, and have only single-lane entries and exits on each leg, as shown in **Figure 4.55**. Low-level mountable kerbs or flush painted islands should be used to allow overrun by large vehicles.

Table 4.19: Roundabout Categories and Key Features

Roundabout Type	Lanes at Entry	Urban Classifications	Rural Classifications	Key Features
Mini-Roundabout	One lane on all legs	Local road only	Not applicable	Fully mountable central island featuring low-level mountable kerb
Single-lane Roundabout	One lane on all legs	Minor collector, Service, local	Collector, local	Raised island with truck apron
Two-lane Roundabout	Two lanes on at least one leg	Boulevard, major and minor collector, service, local	Arterial, collector, local	Raised island with truck apron on smaller layouts
Grade-separated Roundabout	Two lanes on at least one leg	Expressway, major arterial, collector distributor	Freeway, arterial	Incorporates at least two bridges in layout

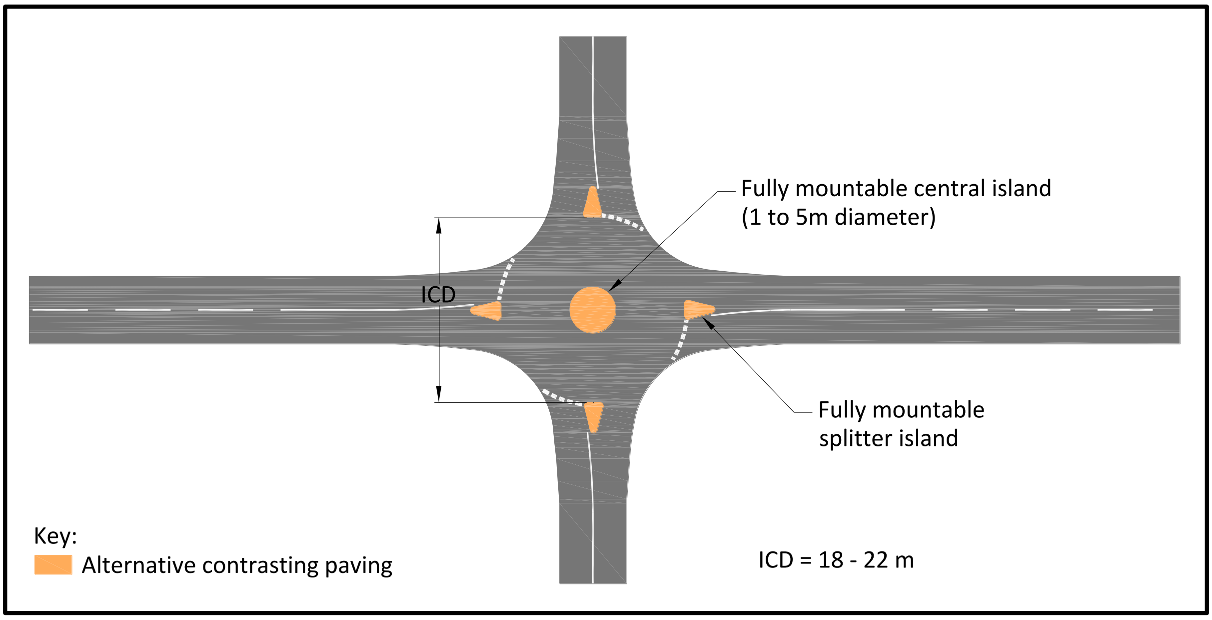


Figure 4.55: Mini-Roundabout

4.7.1.2 Single-Lane Roundabouts

Figure 4.56 illustrates a single-lane roundabout. As the name suggests, it incorporates single-lane entries and exits. They are useful on low classification urban and rural routes. The central island diameter shall be minimum 4 m, while width of the circulatory carriageway shall not exceed 6 m, so as not to allow the passing of vehicles. Truck apron should be provided to accommodate turning movement of large vehicles. This type of roundabout should only be provided where all approaches are undivided roads having 50 km/h or less design speed.

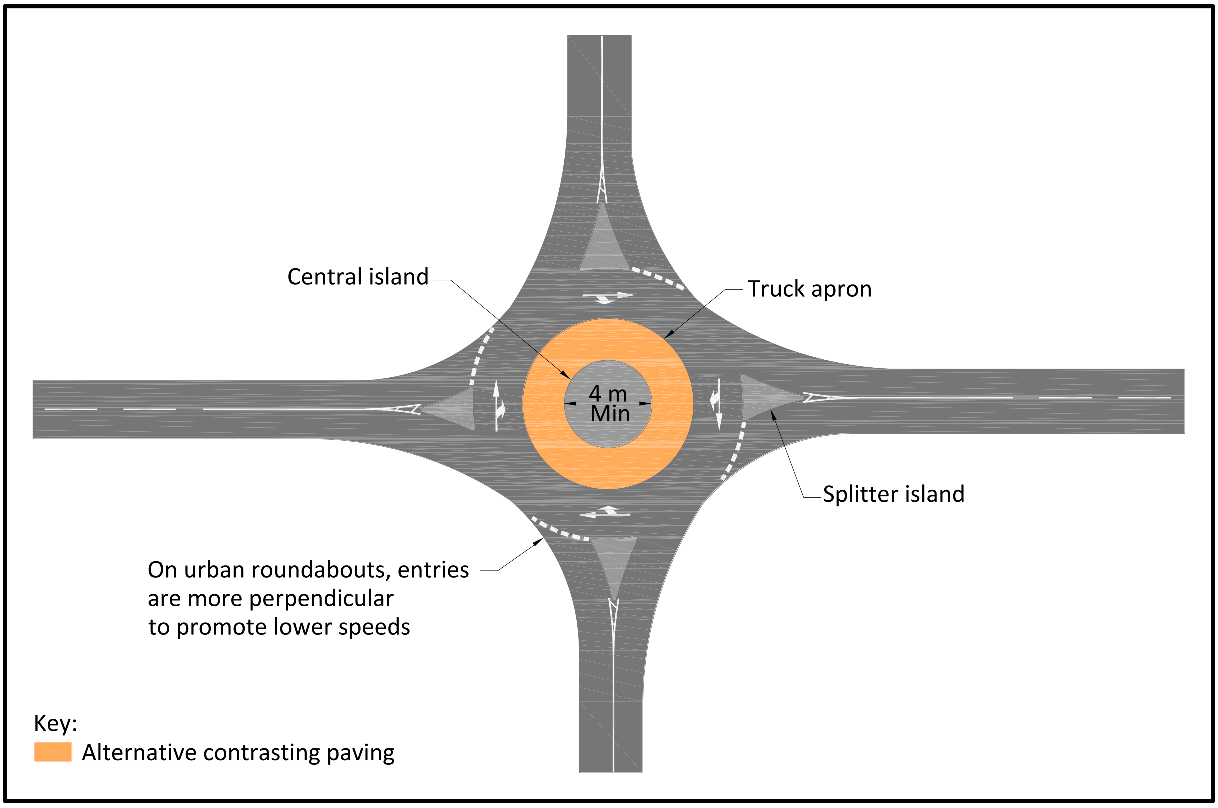


Figure 4.56: Single-lane Roundabout

4.7.1.3 Multi-Lane Roundabouts

Figure 4.57 illustrates a typical multi-lane roundabout. They have kerbed central islands with minimum diameter of 4 m. To allow more than one vehicles to enter or leave the roundabout simultaneously, these roundabouts normally have flared entries and exit. The circulatory carriageway should also be wide enough to allow more than one vehicles to turn simultaneously. Where one of the approaches is a divided road, two-lane roundabouts should be provided.

4.7.1.4 Grade Separated Roundabouts

Figure 4.58 illustrates an example of typical grade separated roundabout. These roundabouts have at least one of the approach at different level, roundabout being at higher level than major road or at-grade with major road at higher level. Grade separated roundabouts should be designed with at least two lanes with circulatory carriageway wide enough to accommodate simultaneous turn of two vehicles.

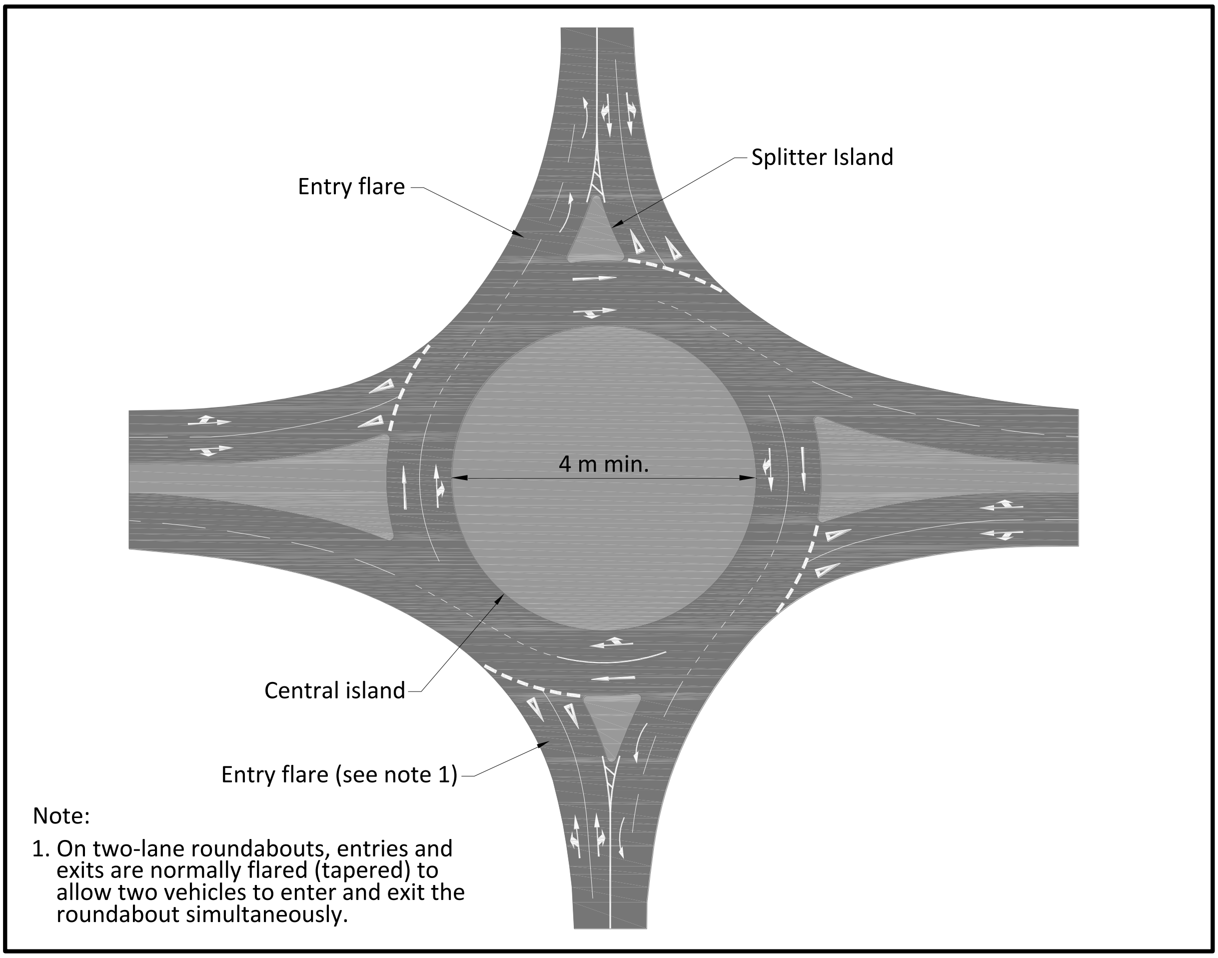


Figure 4.57: Two-lane Roundabout

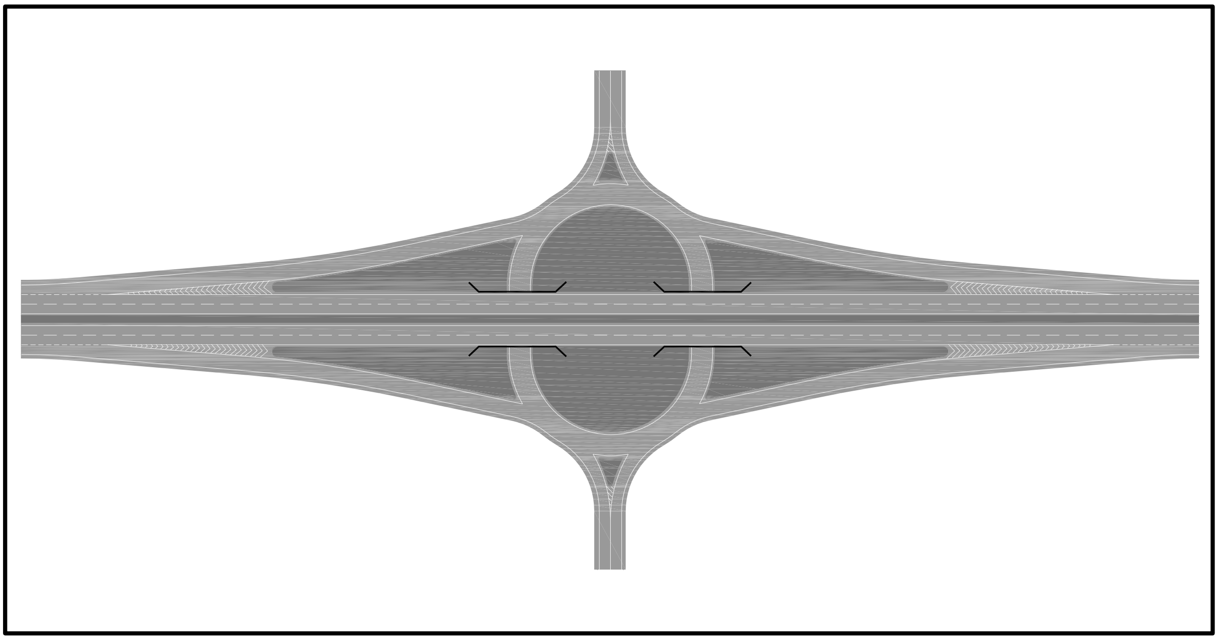


Figure 4.58: Grade Separated Roundabout

4.7.1.5 Signalized Roundabouts

A signalized roundabout has traffic signals installed on at least one the approaches and at the corresponding point on the circulatory roadway. The traffic signal may be installed only on exiting roundabouts is excessive queuing occur on approaches, or there is unbalanced traffic flow, or due to safety concerns. New roundabouts shall not be designed as signalized roundabouts.

4.7.2 Space Allocation for Public Transport

Where roundabouts are located on transit routes, and dedicated bus lanes are provided on approach roads, then bus lanes should terminate in advance of roundabouts and resumes beyond the roundabout, as shown in **Figure 4.59**. Bus stops should not be located near roundabouts and never within circulatory carriageway.

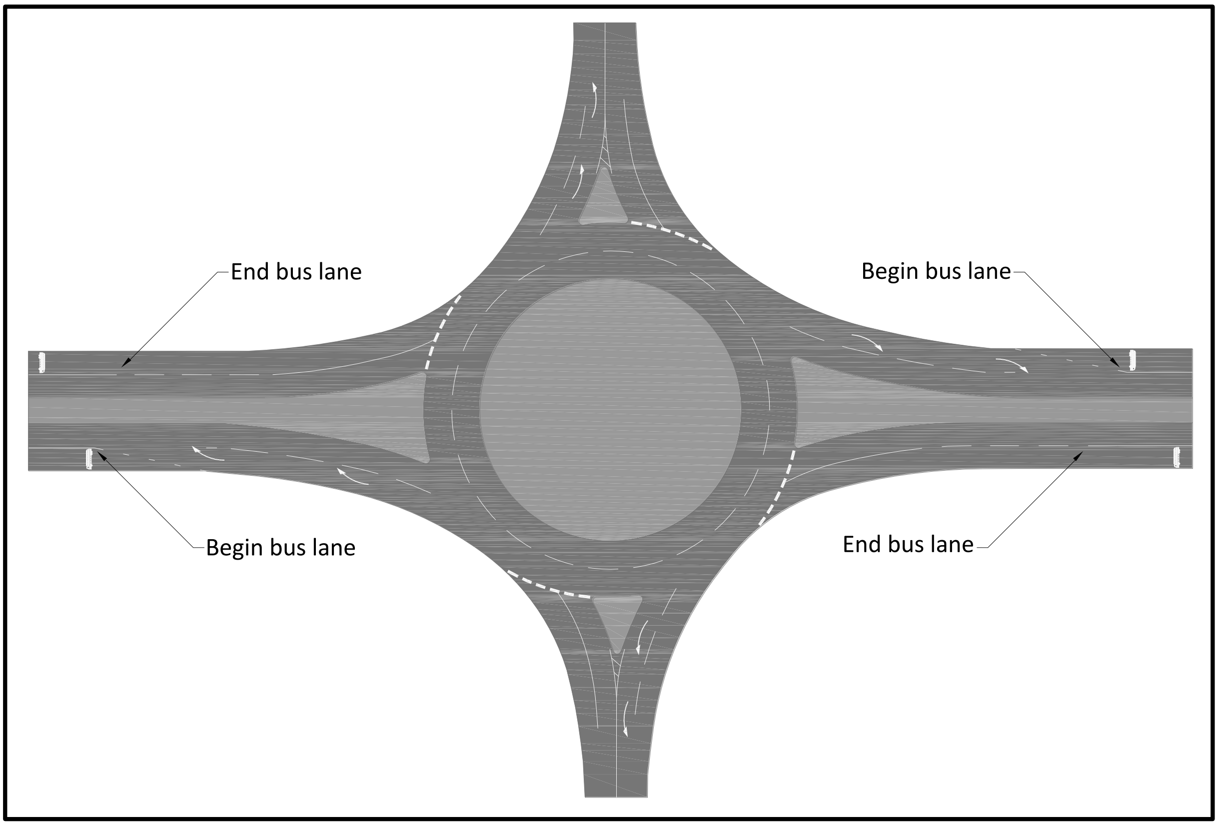


Figure 4.59: Bus Lane Treatment at Roundabouts

4.7.3 Geometric Design

4.7.3.1 General Considerations

The roundabouts operates most safely at low speeds, however the geometric features ensuring low speed entries and circulatory movement, such as horizontal curvature and narrow carriageway width reduces the overall capacity of roundabout. The geometric parameters of roundabouts should be selected such that it provides balance between safety and capacity. The overall layout of roundabout should be checked, using design software such as AutoTurn, to ensure the design vehicle swept path remains within the carriageway.

Location of Roundabouts

The roundabouts should preferably be located on level ground or in sag or the approaches should be flattened to a maximum of 2 percent before entry wherever possible. The location of roundabout at or near crest vertical curve is not recommended. Steep downhill gradients should also be avoided. Roundabouts should be avoided between signalled controlled intersections.

The roundabouts on a single route should be similar in design to the extent practical. For closely spaced roundabouts, the queue length and available storage length should be checked.

Design Vehicle
The longest vehicle that is expected to use the roundabout should be selected as the design vehicle, however, some allowance for swept path widths of longer vehicles may be considered.

4.7.3.2 Roundabout Key Dimensions
Figure 4.60 illustrates the basic features and dimensions of a roundabout, which are defined as follows:

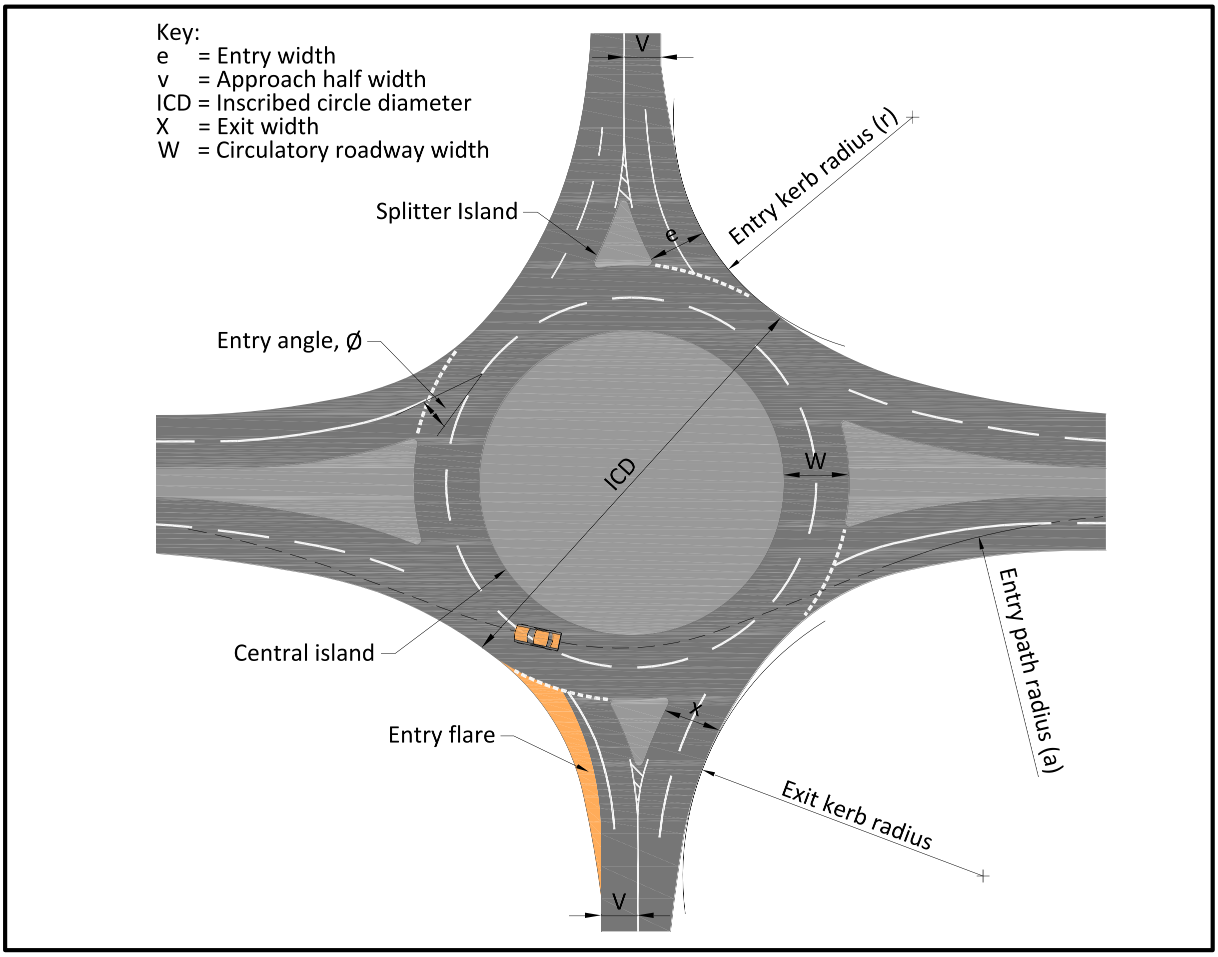


Figure 4.60: Roundabout Key Dimensions

Inscribed Circle Diameter (ICD)
The diameter of largest circle that can be fitted into the roundabout is called inscribed circle diameter (ICD). The minimum ICD values for each type of roundabout are provided in **Table 4.20**. The inscribed circle diameter should be clear of splitter islands.

Table 4.20: Minimum Inscribed Circle Diameters for Roundabouts

Roundabout Type	Minimum ICD (m)	Recommended Design Vehicle	Central Island Treatment
Mini-roundabout	18	WB-15 tractor-semitrailer	Fully mountable central island featuring low level mountable kerb
Single-lane Roundabout	28	WB-15 tractor-semitrailer	Raised central island with truck apron
Two-lane Roundabout	28	WB-15 tractor-semitrailer	Raised central island, with truck apron on smaller layouts

Circulatory Carriageway

The one-way clockwise road surrounding the central island or truck apron is called the circulatory carriageway. The carriageway should preferably be circular. Tight bends should be avoided.

The width of the circulatory carriageway must be between 1.0 and 1.2 times the maximum entry width excluding any overrun area. At multi-lane and grade separated roundabouts, the width of circulatory carriageway should not be greater than 15 m, while at mini-roundabouts and single roundabouts, it shall not exceed 6 m. Truck apron can be provided in addition to circulatory carriageway width to accommodate large vehicles.

Short lengths of reverse curve between entry and adjacent exit should be avoided by linking the adjacent entry and exits curves with a straight segment.

Central Island

The central island is the raised or marked island at the centre of the roundabout. The central island may also include truck apron on small roundabouts. Central island diameter of shall be minimum 4 m, and should be circular in shape. The ICD, circulatory carriageway width and central island diameter are mutually dependent; therefore, once any two of these are determined the third is determined automatically.

Swept path width for a WB-15 tractor-semitrailer, on a roundabout with inscribed circle diameters in the range of 28 m to 36 m is shown in **Figure 4.61**. **Table 4.21** shows turning width dimensions for WB-15 tractor-semitrailer at smaller roundabouts.

Table 4.21: Turning Dimensions at Small Roundabouts

Central Island Diameter (m)	Radius R1 (m)	Radius R2 (m)	Minimum ICD (m)
4.0	3.0	13.0	28.0
6.0	4.0	13.4	28.8
8.0	5.0	13.9	29.8
10.0	6.0	14.4	30.8
12.0	7.0	15.0	32.0
14.0	8.0	15.6	33.2
16.0	9.0	16.3	34.6
18.0	10.0	17.0	36.0

(Source: *Geometric Design of Roundabouts, Department for Transport, 2007* [4])

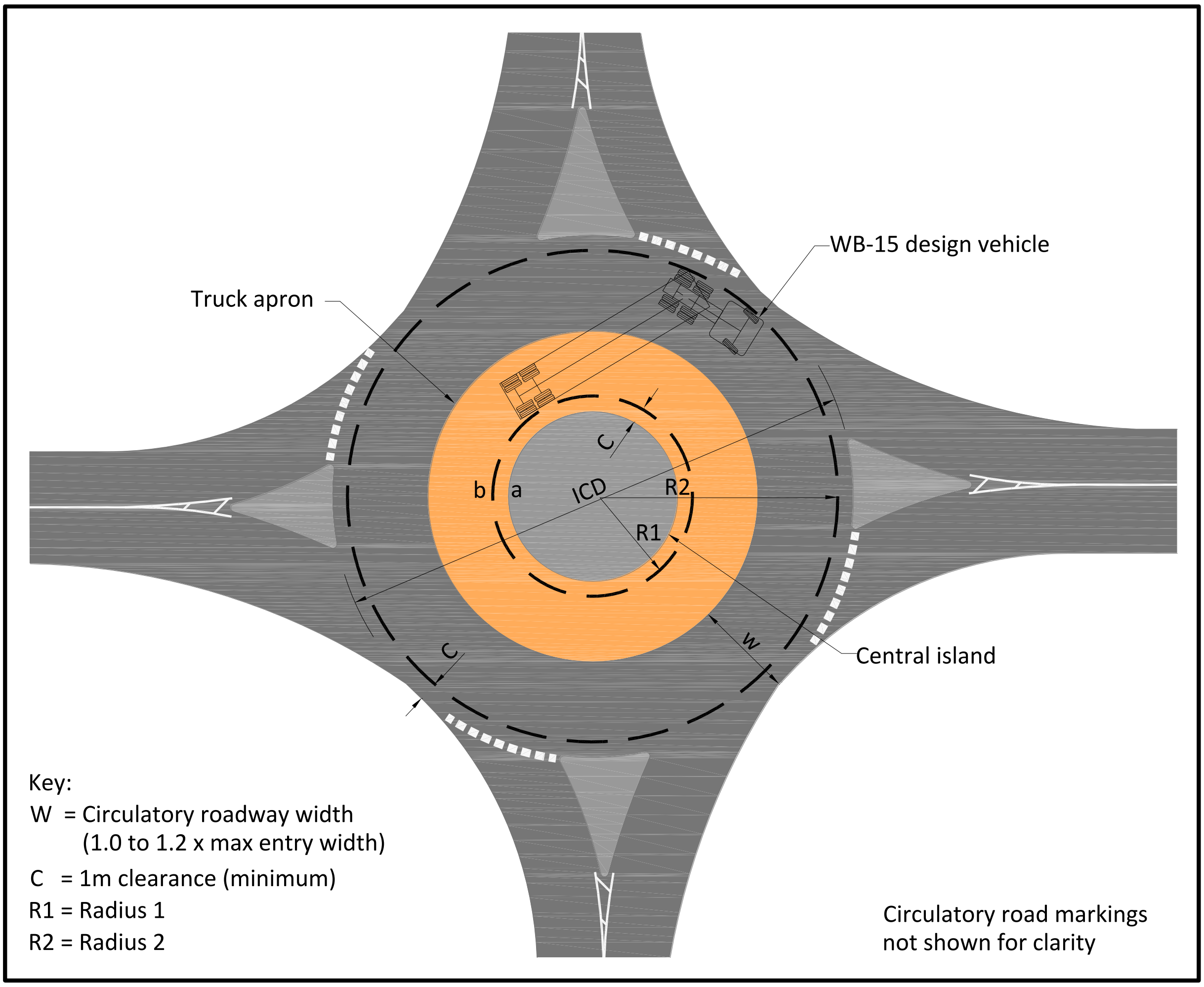


Figure 4.61: Turning Layout of Small Roundabout

Truck apron (a raised low profile area) can be provided to ensure sufficient entry deflection for light vehicles on a single lane roundabout. Truck apron is capable of being mounted by large vehicles, but deflects the passenger cars due to low-level mountable kerbs. Truck aprons should have the same cross slope as the circulatory carriageway.

Splitter Islands

Splitter islands are kerbed raised islands on each approach of roundabout. They are shaped and located to separate and direct the traffic into and out of the roundabout. The splitter islands should be kerbed preferably, but if sufficient space is not available to accommodate kerbed islands, as in the case of a smaller roundabout, the islands may be made entirely of road markings. The raised islands may be enhanced by road markings to further guide vehicles into the roundabout.

Kerbed splitter islands can be used as pedestrian refuges if they are large enough to provide safe standing space for pedestrians with pushchairs and wheelchairs. Splitter islands can also be used to place signs and other street furniture.

Alignment of Entry Lanes

On high-speed rural roads, in order to reduce the overlapping of vehicle paths, the kerb line of splitter island (or median in case of divided road), should lie on and arc, which when projected forward will meet the central island tangentially, as shown in **Figure 4.62**. In urban areas, it is not necessary, but attempt should be made to achieve such alignment of entry lane wherever possible.

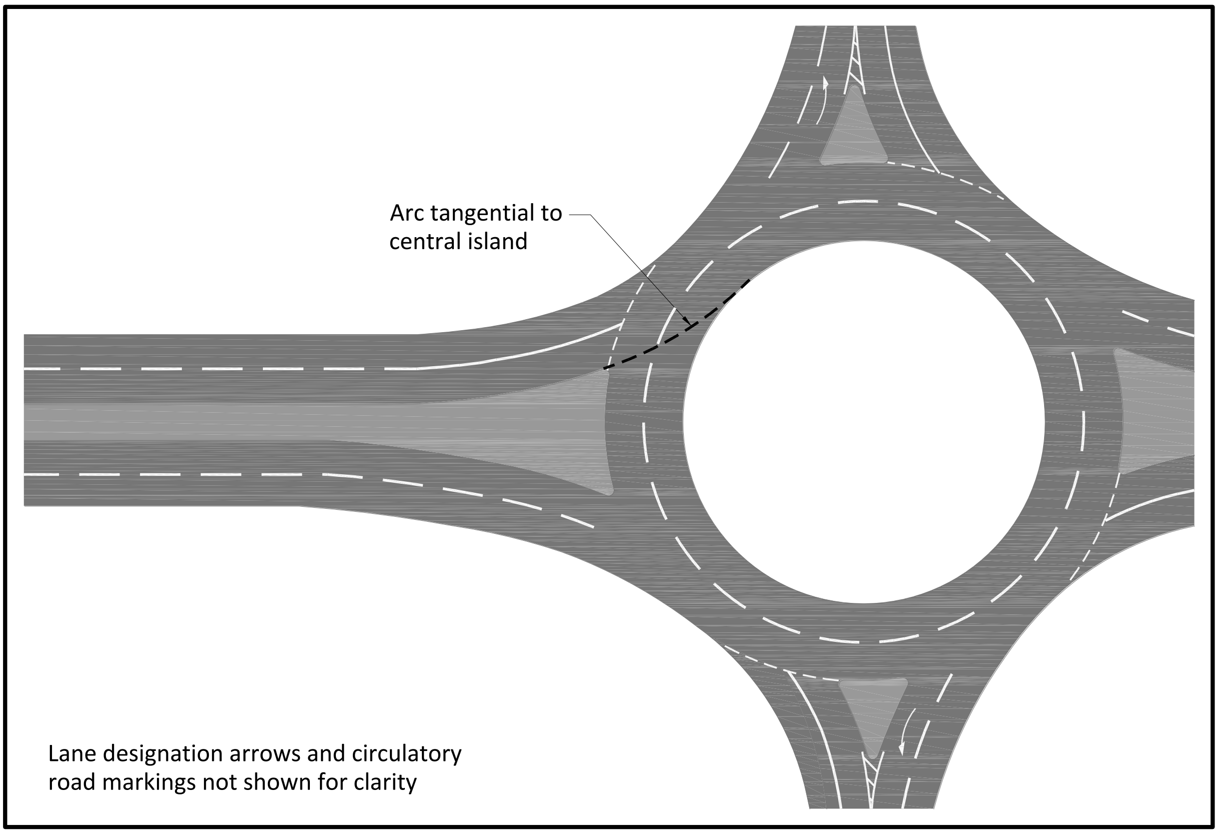


Figure 4.62: Arc Projected from the Splitter Island to the Central Island at Entry

Entry Kerb Radius (r)

Entry kerb radius, r, is the minimum radius of the left kerb line over a distance of 25 m in advance of give way line to 10 m downstream of it as shown in **Figure 4.63**. **Table 4.22** provides the minimum entry kerb radius for the various roundabout types.

The increase in entry radius increases the capacity of the roundabout, but there is no significant improvement in the capacity if the radius is increased above 20 m.

Table 4.22: Entry Kerb Radius

Roundabout Type	Minimum Entry Kerb Radius, r (m)
Mini-Roundabout	6
Single-lane Roundabout	10
Two-lane Roundabout	10
Two-lane Roundabout (particularly for large vehicles)	20

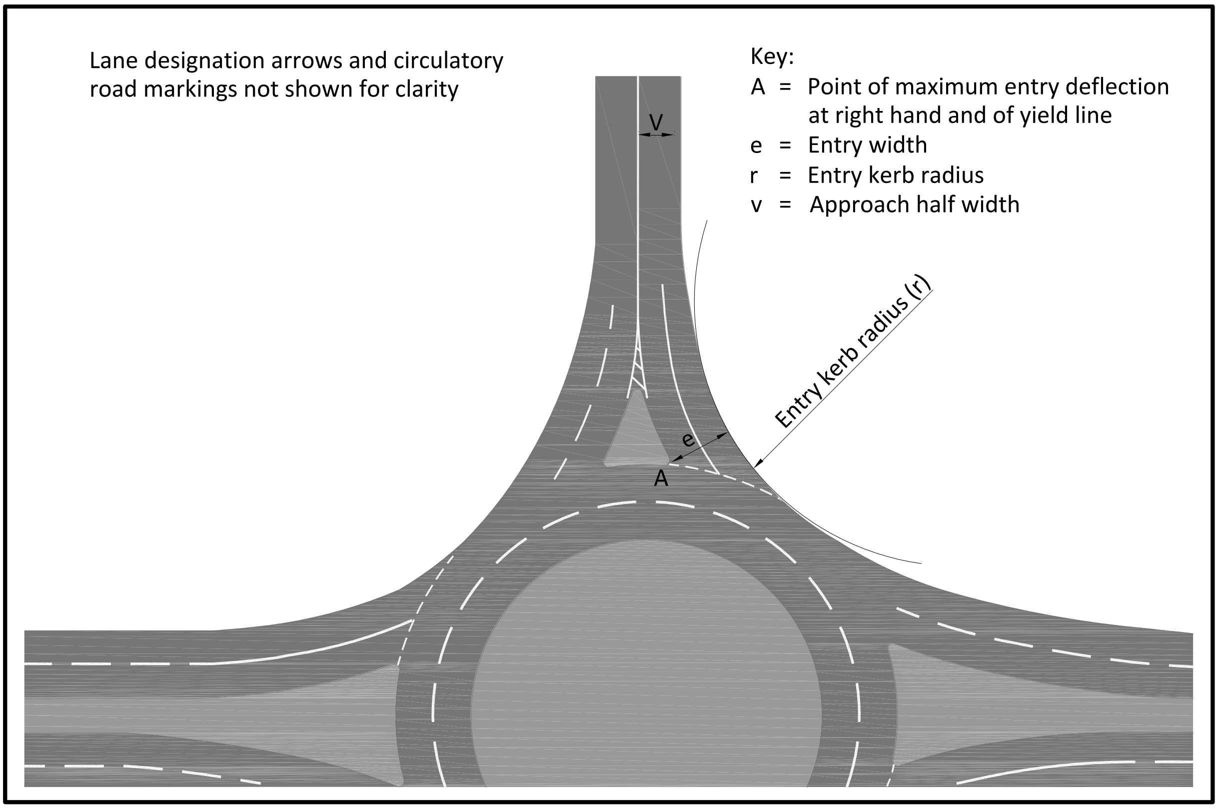


Figure 4.63: Approach Half Width, Entry Width, and Entry Radius

Approach Half Width (v)

The approach half width, v, is the width of approaching carriageway to the roundabout, in advance of the entry flare, as shown in **Figure 4.63**. It is the shortest distance between the left edge of the carriageway and centreline of two-lane road or right edge of the carriageway in case of divided road. Where edge marking is provided, the approach half width should be measured between markings rather than kerb-to-kerb. The approach half width is used by some capacity models to estimate the capacity of the roundabout.

Entry Width (e)

Entry width, e, is the width of the carriageway at entry point, as shown in **Figure 4.63**, measured perpendicular to the approaching lanes, between left kerb and splitter island or median. For capacity assessment, the width should be measured between edge marking or hatching at the entry point. One or two extra lanes should be added to the approach of single-lane or multi-lane roundabouts, but generally, no entry should be more than four lanes wide.

The entry width shall not exceed 10.5 m for single lane roundabout and 15 m for multi-lane roundabout. Lane width at the entry point must not be less than 3 m and not greater than 4.5 m. The lane width of 4.5 m is suitable for single lane entries, whereas 3 to 3.5 m lane width is appropriate for multi-lane entries. If flaring is provided, the width of the additional lane shall be minimum 2.5 m.

Average Effective Flare Length (l')

Localized widening at the point of entry is called entry flare. Single lane roundabouts should be slightly flared at the entry to accommodate large vehicles. Two lane roundabouts are usually flared with addition of one or two lanes at the entry point to increase the capacity. The average effective flare length, l', is the average length over which the entry is widened, as shown as curve CF' in **Figure 4.64**.

For urban areas, the desirable minimum flare length is 5 m is whereas 25 m is adequate in rural areas. Flare lengths greater than 25 m have little effect in increasing capacity. Flare lengths should not be greater than 100 m. Entry widening should be developed gradually on high-speed roads.

The measurement of average effective flare length is shown in **Figure 4.64** and is constructed as follows:

- AB = entry width, e
- GH = approach half width, v
- Construct a curve GD parallel to AH at a distance v from it.
- Construct CF' parallel to BG at a distance one-half of BD from it.
- Average effective flare length, l’ = CF’

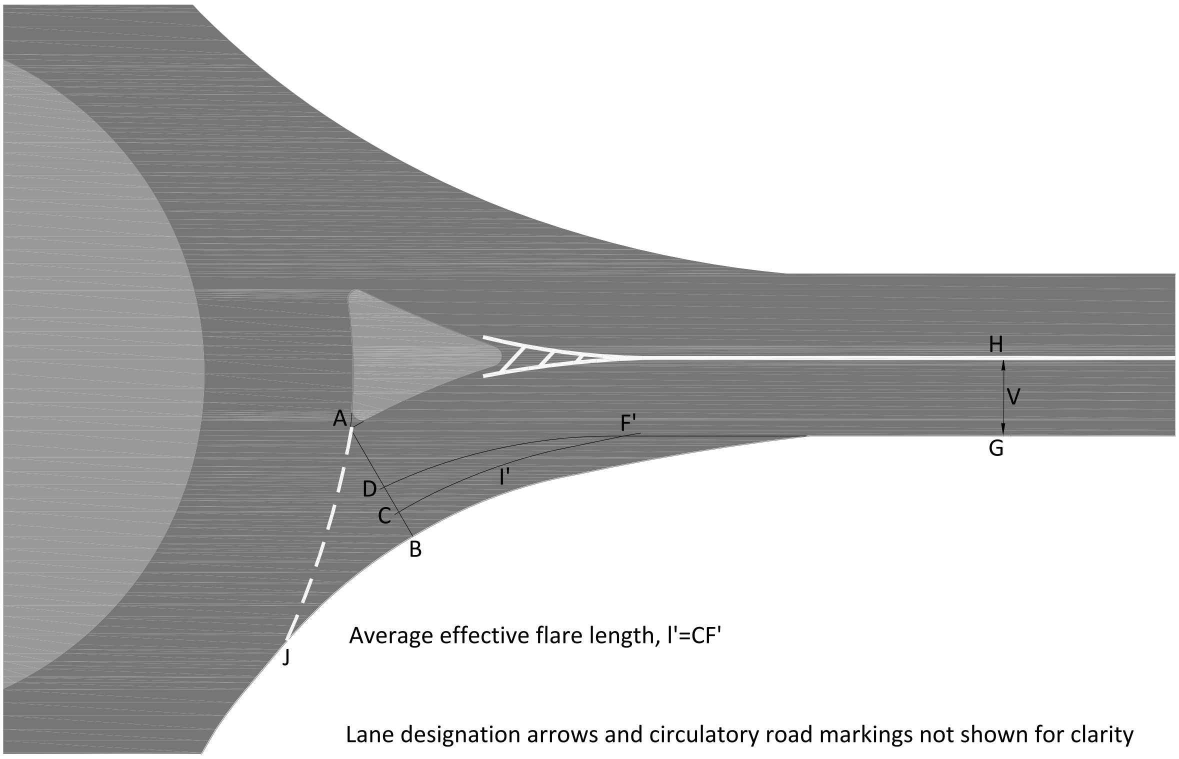


Figure 4.64: Average Effective Flare Length

Entry Angle (φ)

The entry angle, φ, serves as a geometric proxy for the conflict angle between entering and circulating traffic streams. Depending on the size of the roundabout, the following methods should be used to determine the entry angle. For large roundabouts where links are widely spaced the entry angle is measured as shown in **Figure 4.65**.

Line BC is a tangent to line EF, which is midway between the left entry kerb line and the splitter or median island, where this intersects the circulatory roadway. Curve AD is constructed as the locus of the midpoint of the used section of the circulatory roadway, which is a proxy for the average direction of travel for traffic circulating past the entry.

The entry angle is the acute angle between BC and the tangent to AD at the point of intersection between BC and AD, as shown in **Figure 4.65**.

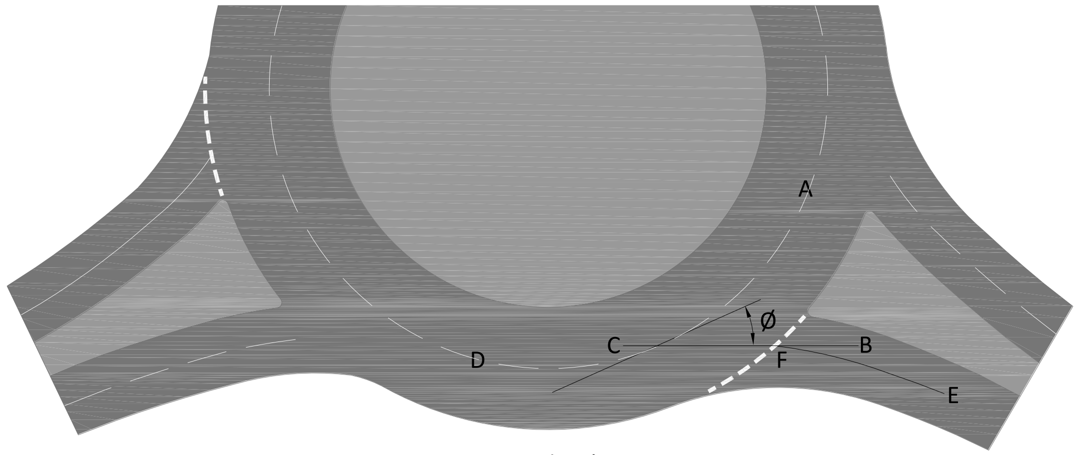


Figure 4.65: Entry Angle Measurement on Large Roundabouts

Figure 4.66 shows the layout for smaller roundabouts. This construction is used when there is insufficient separation between entry and adjacent exit to be able to define the path of the circulating vehicle clearly. The angle between the projected entry and exit paths is measured and then halved to find the entry angle, φ.

Line BC is the same as in **Figure 4.65**. Line GH is the tangent to line JK, which is in the following exit, midway between the left kerb and the splitter or median island, where this intersects the outer edge of the circulatory roadway.

BC and GH intersect at L. The entry angle, φ is then defined by:

$$\varphi = \frac{BLH}{2}$$ ***Equation 4.1***

The entry angle, φ, should lie between 20 and 60 degrees with the optimum angle being between 30 and 45 degrees. High entry angles tend to lower capacity and produce excessive entry deflection, which can lead to sharp braking at entries accompanied by rear end crashes, especially on high-speed approaches. Low entry angles force drivers into merging situations where they will be forced to look over their shoulder or use side mirrors to merge with circulating traffic.

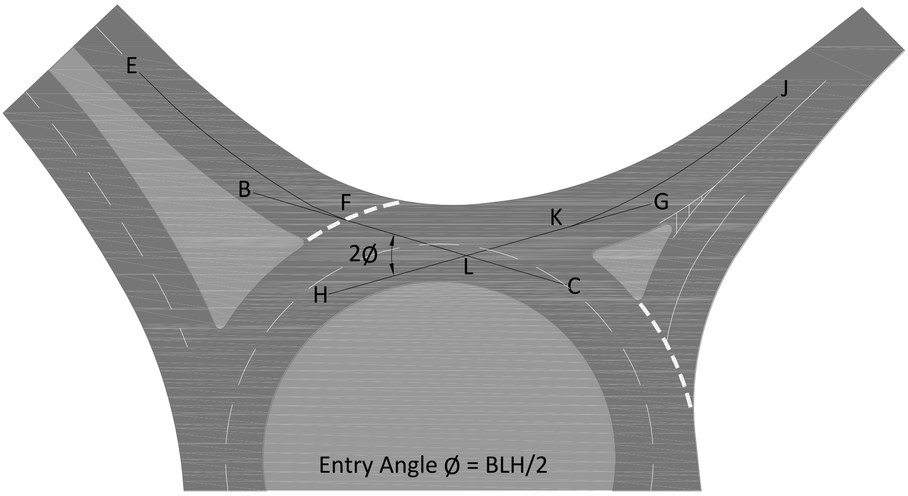


Figure 4.66: Entry Angle Measurement at Smaller Roundabouts

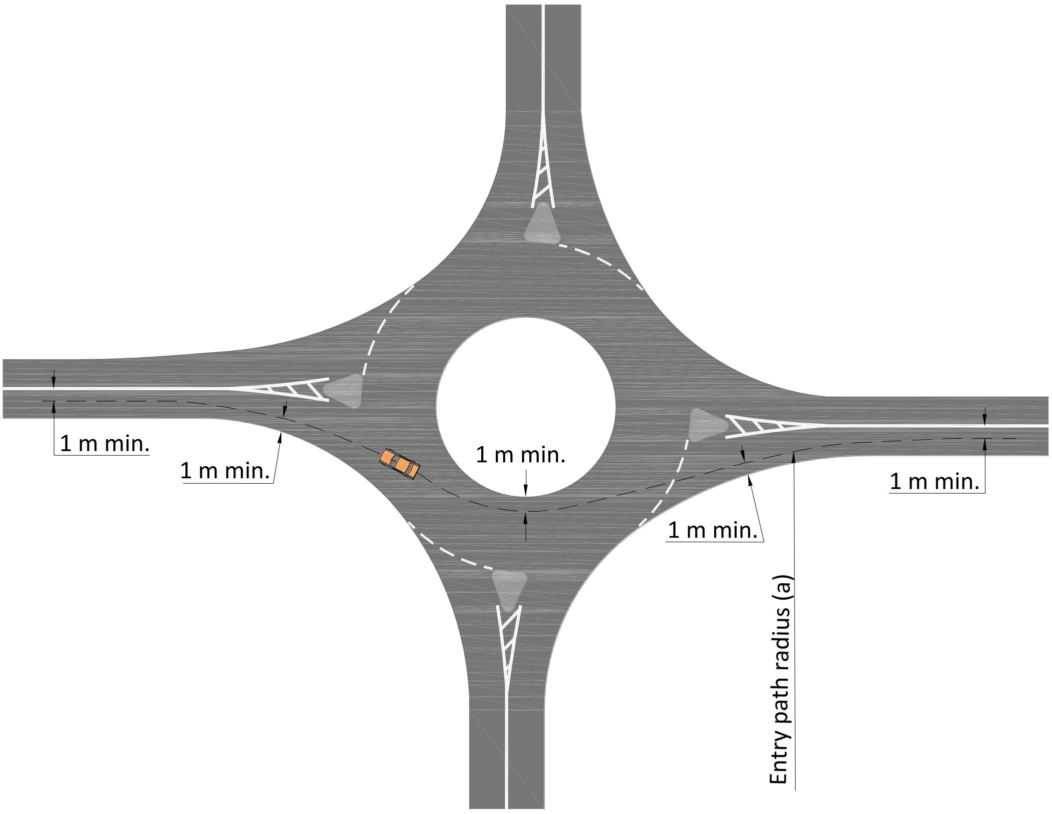
Entry Path Radius

Entry path radius is one of the most important safety factors at roundabouts. It is a measure of the amount of entry deflection to the left imposed on vehicles at entry to the roundabout. This governs the speed of vehicles through the roundabout and ensures that drivers yield to circulating vehicles.

The entry path radius should be checked for all turning movements. It should not exceed 70 m at single-lane roundabouts. At other roundabouts, except mini-roundabouts, the entry path radius should not exceed 100 m *(Geometric Design of Roundabouts, Department for Transport, 2007* [4]*)*. At mini-roundabouts, there is no maximum value for entry path radius.

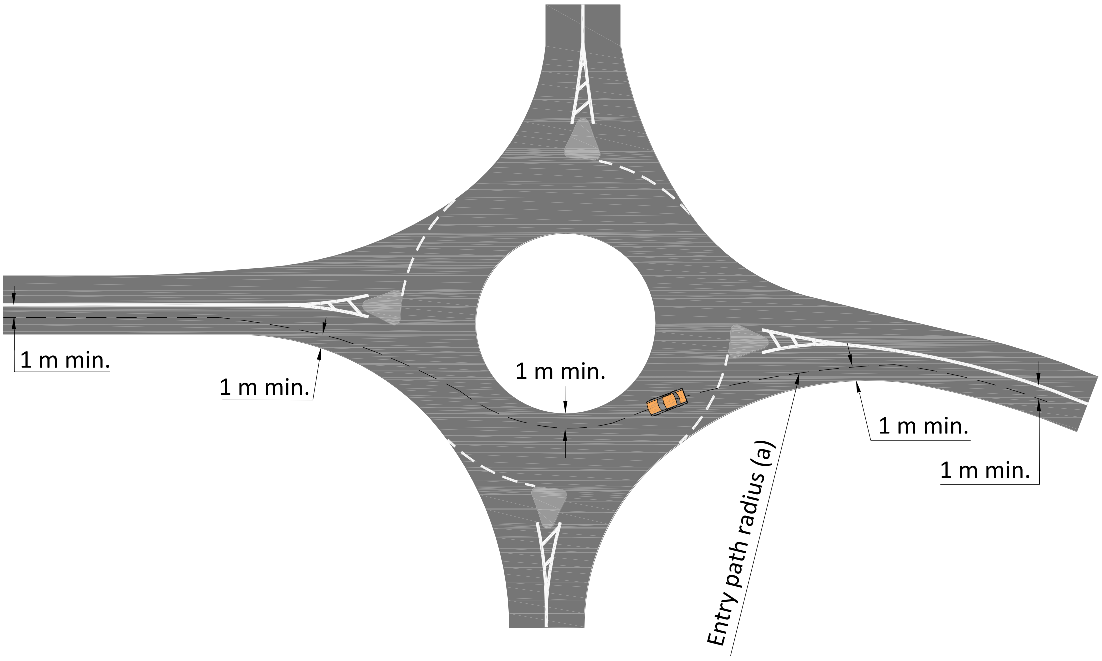
Determining the entry path radius is based on the following assumptions and shown in **Figure 4.67** through **Figure 4.70**.

- Assuming entering vehicle is 2 m wide so that it maintains a distance of at least 1 m between its centreline and any kerb or edge marking, and that it is continuing straight ahead at a four-leg roundabout and across the head of the tee at a three leg roundabout.
- There is no other traffic on the approach and on the circulatory roadway.
- The driver negotiates the site constraints with minimum deflections and that lane markings by the yield line are ignored.
- The commencement point of the vehicle path is located 50 m in advance of the yield line and at least 1 m from the edge of carriageway or centreline.
- The vehicle proceeds as follows:
    - First, toward the yield line
    - Then, toward the central island of the roundabout, passing through a point not less than 1 m from the right hand kerb, the position of which relative to the starting point depends on the amount of approach flare to the right
    - Then continuing on a smooth path with its centreline, never passing closer than 1 m from the central island, more in some configurations



Note:
1. the fastest path is the smoothest, flattest path that a vehicle can take through the roundabout.
2. the commencement point of the fastest path is located 50 m in advance of the yield line and at least 1 m from the edge of traveled way or centerline.
3. the entry path radius, a is measured over the smallest best fit circular curve over a distance of 25 m occurring along the vehicle path in the vicinity of the yield line, but not more than 50 m in advance of it.

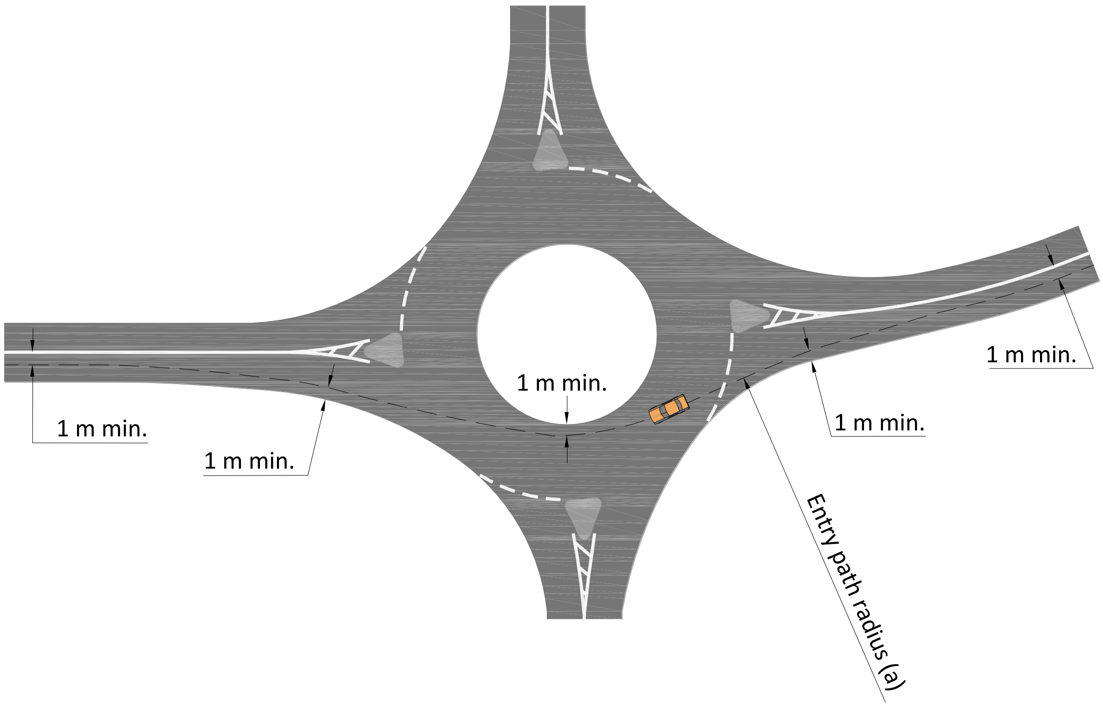
Figure 4.67: Entry Path Radius Determination



Note:

1. the fastest path is the smoothest, flattest path that a vehicle can take through the roundabout.
2. the commencement point of the fastest path is located 50 m in advance of the yield line and at least 1 m from the edge of traveled way or centerline.
3. the entry path radius, a is measured over the smallest best fit circular curve over a distance of 25 m occurring along the vehicle path in the vicinity of the yield line, but not more than 50 m in advance of it.

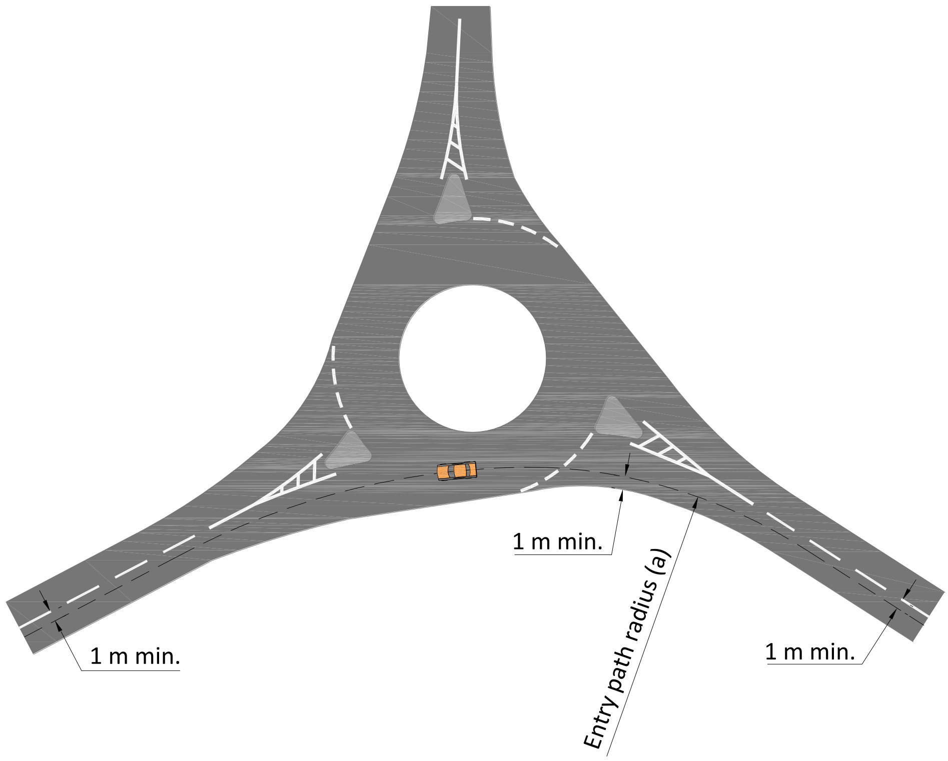
Figure 4.68: Entry Path Radius Determination for a Left-Curving Approach



Note:

1. the fastest path is the smoothest, flattest path that a vehicle can take through the roundabout.
2. the commencement point of the fastest path is located 50 m in advance of the yield line and at least 1 m from the edge of traveled way or centerline.
3. the entry path radius, a is measured over the smallest best fit circular curve over a distance of 25 m occurring along the vehicle path in the vicinity of the yield line, but not more than 50 m in advance of it.

Figure 4.69: Entry Path Radius Determination for a Right-Curving Approach



Note:

1. the fastest path is the smoothest, flattest path that a vehicle can take through the roundabout.
2. the commencement point of the fastest path is located 50 m in advance of the yield line and at least 1 m from the edge of traveled way or centerline.
3. the entry path radius, a is measured over the smallest best fit circular curve over a distance of 25 m occurring along the vehicle path in the vicinity of the yield line, but not more than 50 m in advance of it.

Figure 4.70: Entry Path Radius Determination for a Typical Three-leg Roundabout

On a layout of the roundabout, to a scale not less than 1:500, draw the centreline of the most realistic path that a vehicle would take in its complete passage through the roundabout on a smooth alignment without sharp transitions.

The exact path drawn is a matter of personal judgment. The results should be examined for compliance and consistency with the appropriate clauses in this section. Any reverse of curvature in the vehicle path around the central island should be drawn so that there is no sharp deviation between that curve and the entry curve. Particular care in checking entry path radius is needed when considering small central island designs.

This tightest radius can be measured by means of suitable templates. The entry path radius is measured on the curved length of path near the yield line, but not more than 50 m in advance of it as shown in **Figure 4.67** through **Figure 4.70**. The entry path radius is the radius of the best-fit circular curve over a length of 25 m.

At single-lane and smaller two-lane roundabouts incorporating a truck apron, the entry path radius is measured relative to the outer perimeter of the truck apron rather than that of the central island.

One method for increasing entry deflection at roundabouts is to stagger the legs, such that the centreline of each leg intersects with the roundabout slightly left of centre, as shown in **Figure 4.71**. This method also results in a reduction in the overall size of the roundabout, minimizing land acquisition, and in addition, helps to provide a clear exit route of sufficient width to avoid conflicts.

Another method for increasing entry deflection at roundabouts, especially on high-speed divided highways, is to use a combination of reverse curves on the approach as shown in **Figure 4.72**. This method also has the added benefit of helping to reduce vehicle speeds on the immediate approach to the roundabout.

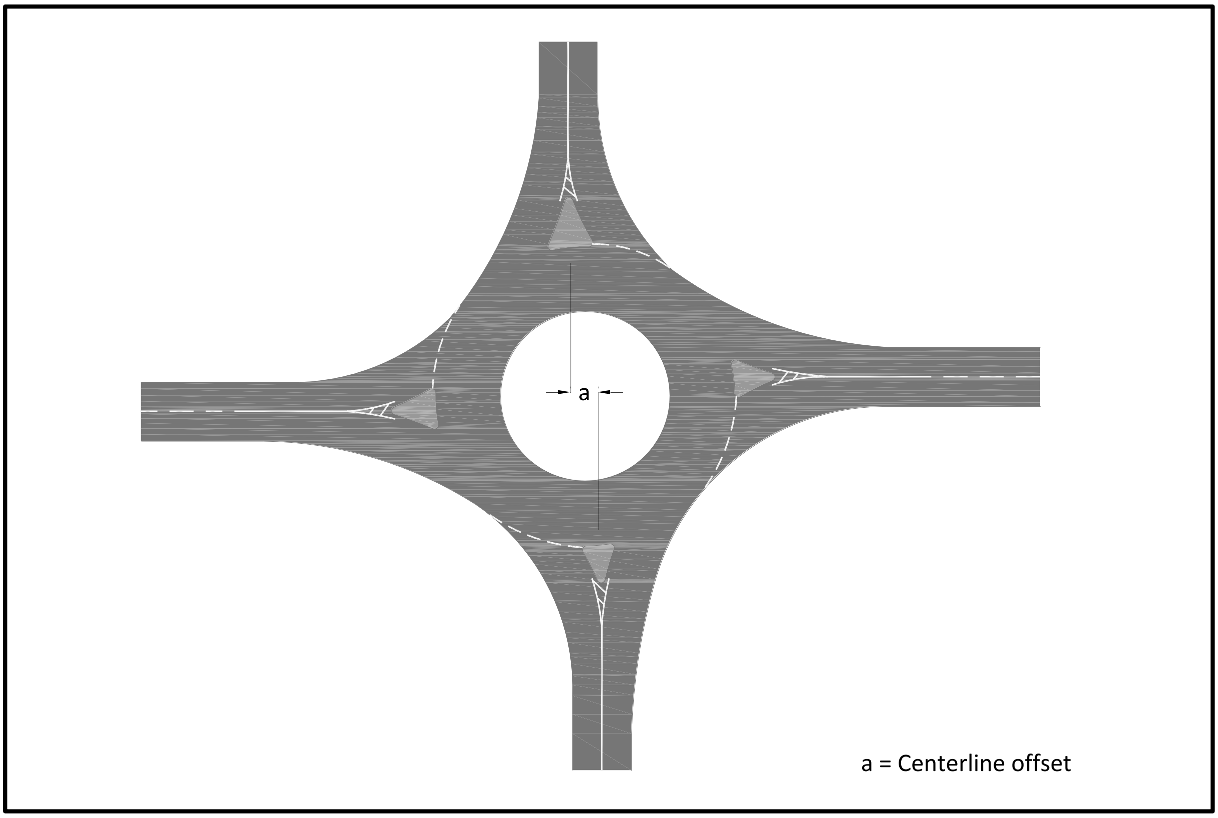


Figure 4.71: Staggering of Roundabout Legs to Increase Entry Path Radius

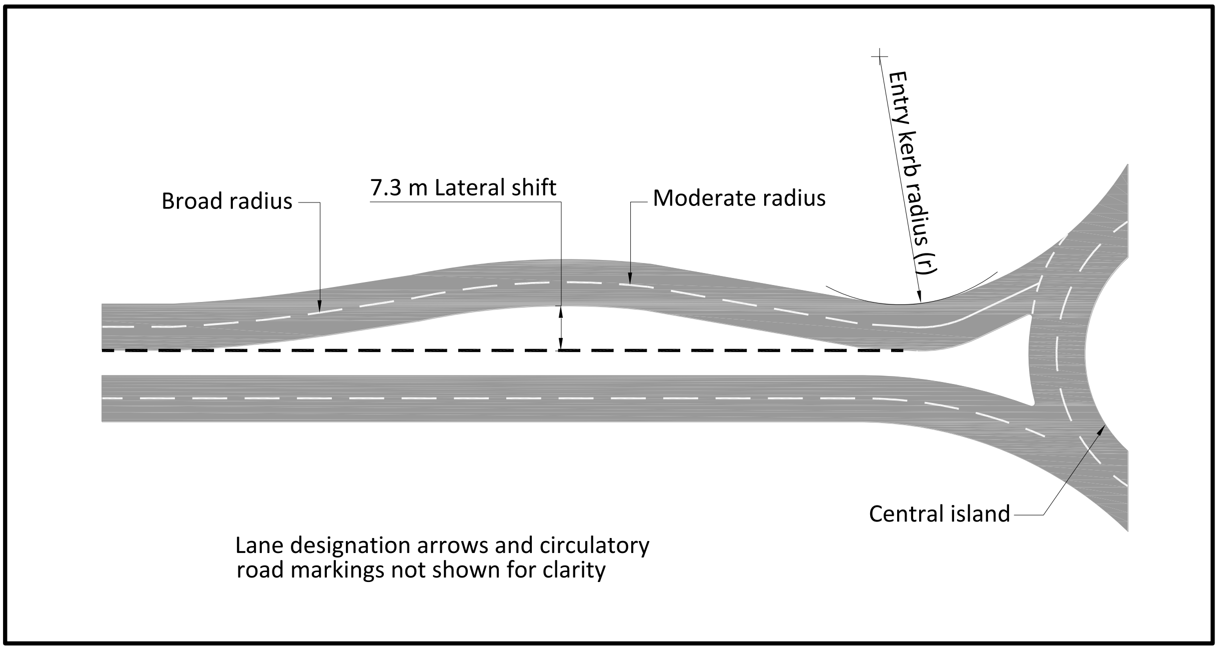


Figure 4.72: Reverse Curves on Approach to Roundabouts

Exit Width

The exit width is measured similarly to entry width. It is the distance between the left kerb line and the splitter island or, in the case of a divided highway, the median where it intersects with the outer edge of the circulatory roadway. As with the entry width, it is measured perpendicular to the left hand kerb line. Values typically are similar to or slightly less than entry widths; that is, exits have less flaring.

The number of lanes at an exit should be equal to the number of lanes at the corresponding entry for the straight though-traffic movement. At single-lane roundabouts, only one lane should be provided. There should be no more than two lanes on an exit.

On undivided highway exits where the length of the splitter island is 20 m or greater, a minimum width of 6 m, measured perpendicular to the left kerb, should be provided adjacent to splitter island to allow traffic to pass a broken down vehicle. **Figure 4.73** shows a typical two-lane roundabout exit using some of the principles described here.

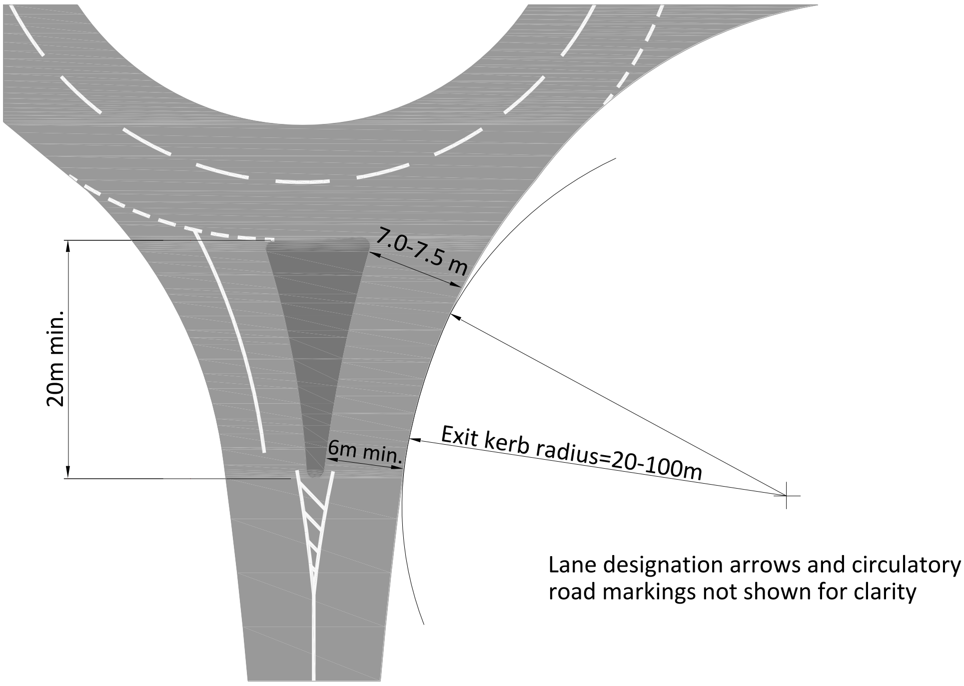


Figure 4.73: Typical Two-lane Roundabout Exit Where Island Length is ≥ 20 m

Exit Kerb Radius

Table 4.23 lists acceptable ranges of exit kerb radius. For a two-lane roundabout, a value of 40 m is desirable, but for larger roundabouts on high-speed roads, a higher value can better fit the overall intersection geometry. A compound curve starting with a radius of 40 m, developing to a larger radius, of up to 100 m, usually offers the best solution. Larger values of exit radii can lead to high exit speeds, which may not be appropriate where pedestrian crossing facilities are located immediately downstream.

Table 4.23: Exit Kerb Radius

Roundabout Type	Minimum Exit Kerb Radius (m)	Maximum Exit Kerb Radius (m)
Mini-roundabout	6	20
Single-lane Roundabout	15	20
Two-lane Roundabout	20	100

(Source: Based on guidance from *Geometric Design of Roundabouts, Department for Transport, 2007* [4])

The spacing of an exit and the preceding entry should not be less than the combination of the minimum entry kerb radius and the minimum exit kerb radius. If a roundabout is to be modified to include an additional entry, care should be taken so that this does not affect safety at the preceding entry and the following exit. It may be necessary to redesign the whole roundabout if adequate spacing between entries and exits cannot be achieved.

Free Left-turn Lanes

Free left-turn lanes are a useful method for providing improved service to vehicles intending to leave a roundabout at the first exit after entry. Through a free left-turn lane, vehicles are able to proceed directly to the first exit without having to interact with vehicles on the circulatory roadway. This is particularly appropriate at two-lane roundabouts.

There is a simple procedure to provide guidance to determine if a free left-turn lane would be beneficial to a roundabout design and merit further investigation. It is based on total traffic in-flows at entry, vehicle composition, left-turning traffic, and the number of entry lanes. The inclusion of a free left-turn lane should be considered if the following is true for the individual approach:

$$\boldsymbol{R} = \frac{\boldsymbol{F}}{\boldsymbol{E}} \qquad \textit{Equation 4.2}$$

Where,
R = flow of left-turning vehicles per hour
F = total entry flow in vehicles per hour
E = total number of entry lanes including the free left-turn lane

In cases where R and F/E are very close, consideration of other factors, such as safety, should be included in the appraisal.

The removal of flow from the circulatory roadway can improve the overall performance of the roundabout, but vehicle composition should be examined when considering the use of these lanes. If the left-turning vehicles are predominantly light and there are high proportions of large vehicles leaving the roundabout, there could be problems with different speeds at the merge, particularly if this is on an uphill gradient. If dedicated lanes are to be used in such situations, they should finish with a yield line at the exit from the lane.

The use of free left-turn lanes in urban areas where pedestrians are expected is not recommended. Pedestrians should be channelled using a guardrail to a suitable crossing point. If this is not possible, the channelizing island should be of sufficient width to accommodate the anticipated peak number of pedestrians, and the location of pedestrian crossing points should be carefully considered.

Free left-turn lanes should include a fully kerbed channelizing island. Vehicles are channelled into the left-hand lane by lane arrows and road markings supplemented by advance direction signs. The operation of the free left-turn lane should not be impaired by traffic queuing to use the roundabout itself.

Free left-turn bypass lanes should not be designed to encourage high speeds. The curve radius used for the free left-turn lane will depend on both the design speed of the approach road and site constraints. The driver’s perception of the approach and free left-turn lane radii will be a determining factor in their approach speed. Therefore, the designer should consider the need for speed reduction measures on the approach depending on the minimum curve radii used. For divided highways, a minimum inside curve radius of 30 m is recommended, and in all instances, the inside curve radii should not be less than 10 m. The radius at exit should not be less than the radius at entry.

Examples of free left-turn lanes at roundabouts are shown in **Figure 4.74** to **Figure 4.76**.

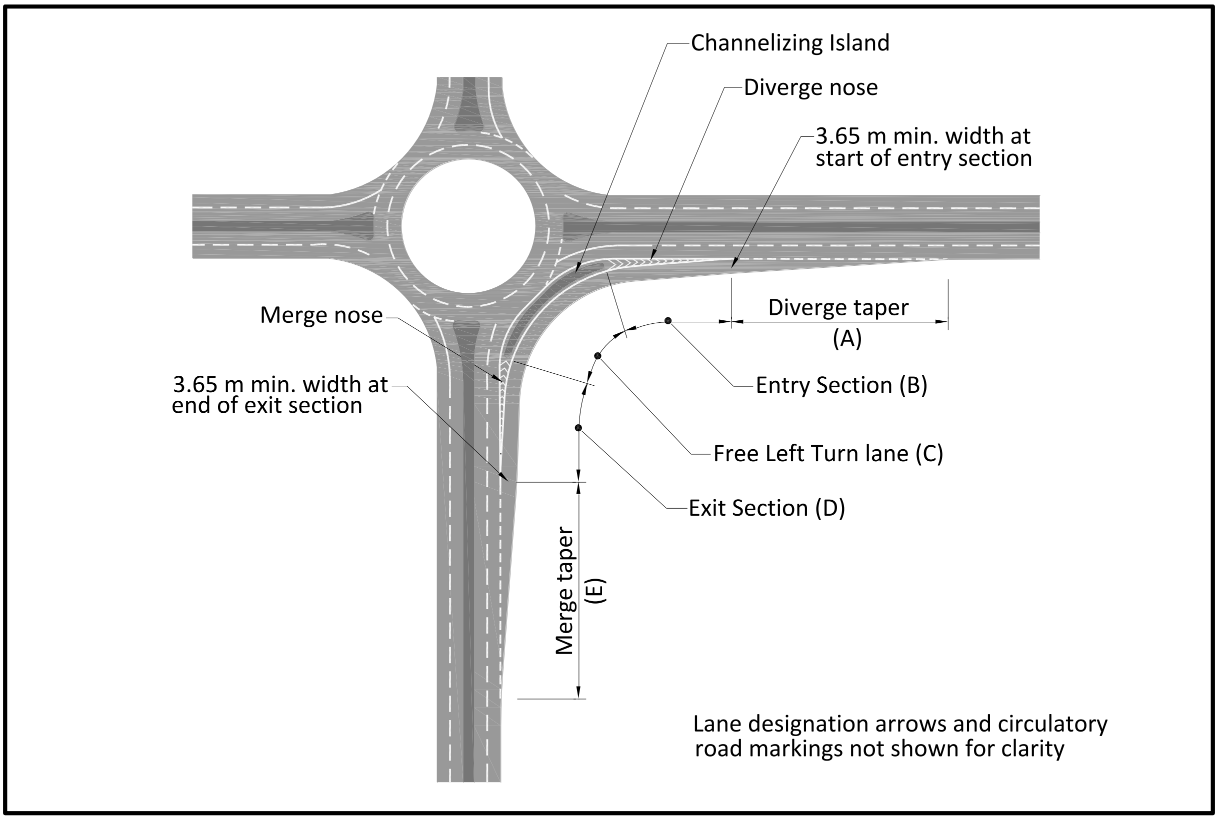


Figure 4.74: Free Left-turn Lane with Direct Taper-Diverge and Merge

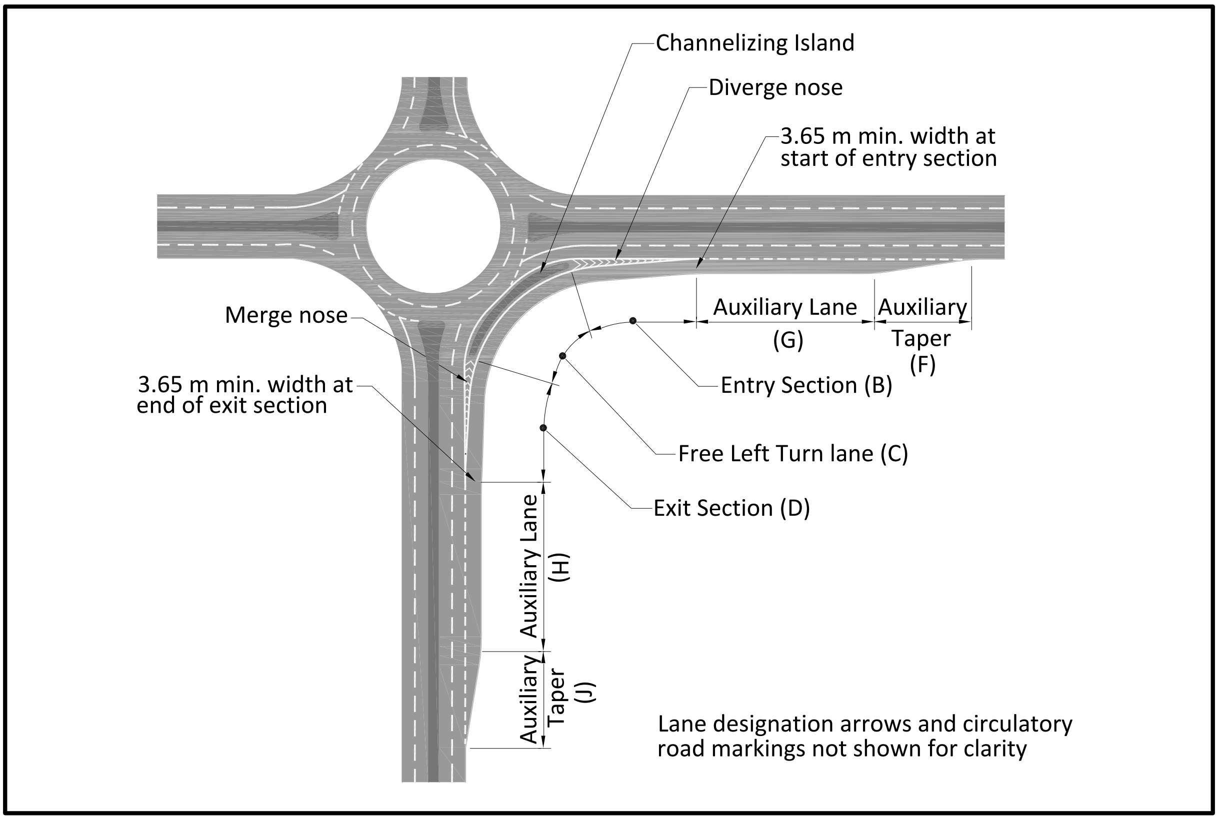


Figure 4.75: Free Left-turn Lane with Auxiliary Lane-Diverge and Merge

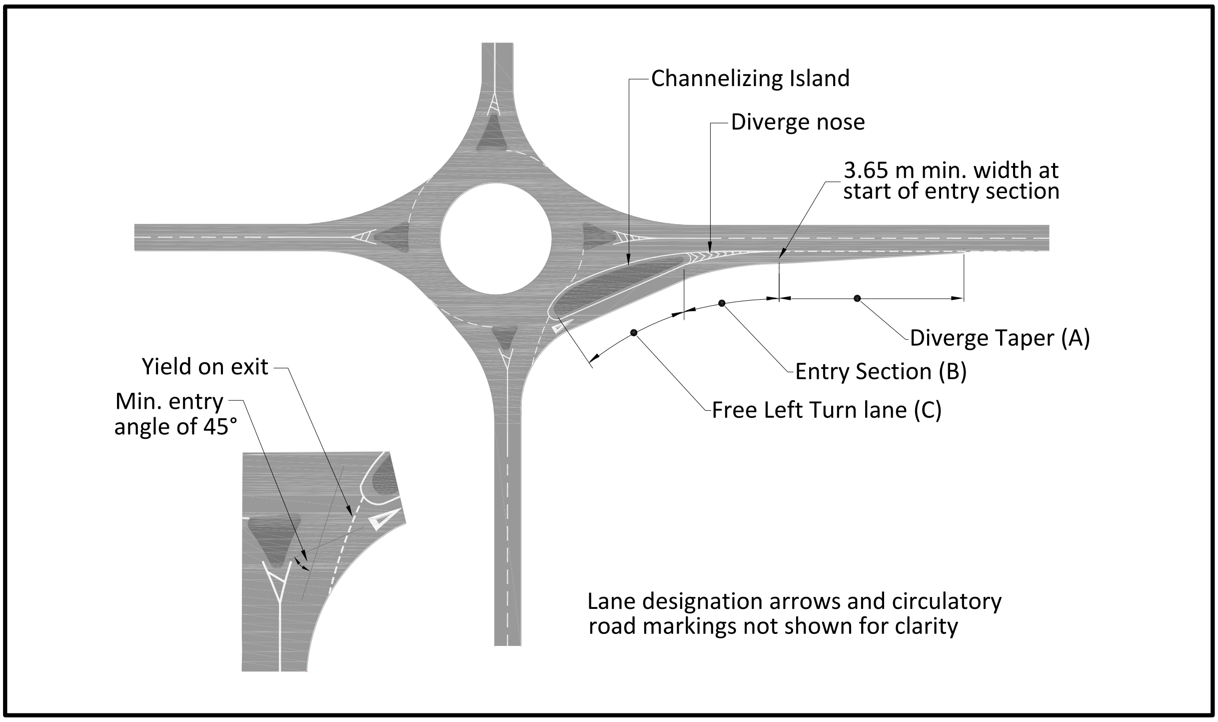


Figure 4.76: Free Left-turn Lane with Direct Taper Diverge and Yield Control on Exit

4.8 SIGNALIZED ROUNDABOUTS

4.8.1 General

4.8.1.1 Application of Design

Roundabouts are designed to operate freely, without traffic signals. One solution for reconstruction projects with a roundabout operating with a v/c ratio greater than 1.0 and an LOS F may be to signalize the roundabout. The guidance in this section regarding the provision of traffic signals at roundabouts is applicable only to existing roundabouts that may require such an improvement. By policy, signalized roundabouts shall not be a design solution for new intersections.

These issues include specific traffic concerns associated with certain roundabouts that may be resolved by installing traffic signals with or without geometric changes. For a roundabout that may require or benefit from signals on most or all approaches, traffic micro-simulation software is generally necessary to characterize the level of improvement and enable development of design details.

When traffic flows exceed the level at which a roundabout can no longer self-regulate, that is when the v/c ratio is greater than 1.0, the throughput of particular entries can be restricted. The provision of traffic signals at one or more approaches can provide capacity for the affected roundabout approach movements in the following manner:

- Where delays are excessive because of imbalanced flows, signals can alter the natural priority to better address the imbalance.
- Where throughput is inadequate because of high circulating speeds rather than high flow, signals can achieve an overall improvement in throughput.

- Where significant pedestrian traffic crosses one or more approaches, providing dedicated pedestrian crossing time through signalization can provide for pedestrian's safe crossing.
- Where it is possible to coordinate the roundabout as part of an overall urban traffic control network, usually fixed-time urban traffic control, traffic signals can reduce overall delays by eliminating the random element of yield control.

A combination of physical improvements, pavement markings, and signalling can increase capacity, provide better balance and stability of queues, and reduce traffic speed. Lane control at signalized roundabouts may be improved by the effective use of route indications on the external and internal approaches to direct traffic to the appropriate departure lane. Where used in this section, external approach pertains to approaches into the roundabout and internal approach pertains to approaches within the circulatory roadway of the roundabout.

Roundabouts are self-regulating under normal priority control at each approach. However, signalized roundabouts have signal control on one or more of the external approaches and on one or more of the internal approaches. Signalized roundabouts may operate in a self-regulating manner, depending on a combination of the extent, method, and duration of the signal control conditions and the roundabout size.

4.8.1.2 Pedestrian Crossing Facilities

In urban areas, care should be taken to provide mobility to pedestrians. Where pedestrian volumes are high, crossing facilities should be designed in accordance with the guidance.

Signals for traffic on the external approaches give pedestrians a natural point of focus. Ideally, the pedestrian route across the rest of the circulatory roadway should be related to pedestrian desire lines and will affect the signal system that is adopted. Pedestrians may be directed onto the central island of the roundabout or to crossing facilities provided on the departure lanes of the roundabout. Pedestrian crossings on departure lanes should be sufficiently far from the circulatory roadway to minimize the likelihood of queuing vehicles interfering with circulating traffic. This will depend on traffic flow, overall roadway circulation, and the length of traffic stop time at pedestrian signals.

Traffic signals at approaches to the roundabout help reduce conflicts between vehicular and pedestrian movements.

4.8.1.3 Geometric Design Standards

This section provides guidance on the application of geometric design standards for signalized roundabouts.

The geometric design of the key elements described in Design for Roundabouts, of this manual should be applied to external and internal approaches to a roundabout that always operate in a self-regulating manner; that is, approaches without signal control or with part-time signal control. The guidance in should also be applied to approaches with indirect signal control.

The geometric design standards in Geometric Modifications, in this part should be applied to external approaches to a roundabout that operate with direct full-time signal control. These standards should also be considered for external approaches that operate with direct part-time signal control; however, under these circumstances, the standards in Design for Roundabouts, of this manual take priority.

For signalized roundabouts, the intersection inter-visibility zone on the circulatory roadway should be measured to a point 2.5 m beyond the secondary signal, as shown on **Figure 4.77**.

The requirements for intersection inter-visibility apply to internal and external approaches to a roundabout that operate under signal control at any time.

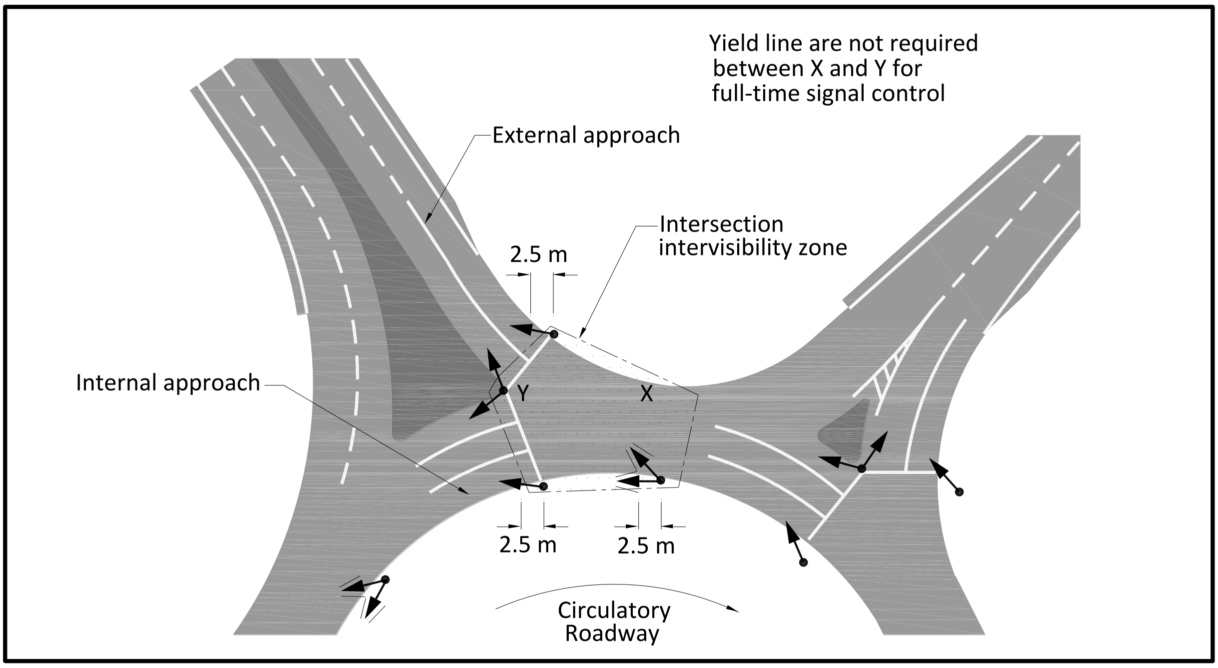


Figure 4.77: Intersection Control Area

4.8.1.4 Size

The engineering challenges in designing a signalized roundabout are usually greater for a small roundabout than a large one. The physical constraints on the internal link lengths between each signalized approach affect the interaction between the individual approach signals. Signal control can often not be appropriate on smaller roundabouts.

4.8.2 Signal Control

4.8.2.1 Extent of Signal Control

A roundabout is fully controlled when signal control is provided on all internal and external approaches.

A roundabout is partially controlled when one or more of the approaches remain under priority control. Partial control can be used where traffic congestion does not occur on all external approaches. It can be easier to coordinate the signals of a four-approach roundabout if one external approach is priority controlled.

4.8.2.2 Methods of Signal Control

Direct signal control consists of traffic signals situated on one or more of the external approaches and the internal approach immediately to the right of each signalized external approach. **Figure 4.78** shows a large, signalized roundabout with partial, direct signal control where traffic flow on the minor roads is light and continues to operate in a self-regulating manner under normal priority control.

Indirect signal control consists of signals situated at such a distance away from the roundabout approach that the external approach continues to operate in a self-regulating manner under normal priority control. **Figure 4.79** shows an example of a signalized roundabout with indirect signal control on approaches A, B, and C.

Indirect signal control may be used where an external approach with a very heavy traffic flow is preceded to the left by an external approach with a very light traffic flow. Heavy traffic flow into and circulating the roundabout can proceed virtually uninterrupted, possibly with high circulatory speeds. This situation causes an unbalanced traffic flow that adversely affects the capacity of the roundabout and can lead to excessive queue lengths and substantial delays to other external approaches, which in turn can result in congestion at preceding intersections or on-ramps.

Indirect signal control can be introduced to control approach flows on one or more external approaches and can provide a gap in the circulatory traffic to favour external approaches where previously there were excessive delays and queues, as shown on **Figure 4.79**. The capacity of the external approaches can then be balanced, but increases in vehicle gap distances can be detrimental to cyclists and pedestrians crossing the approaches.

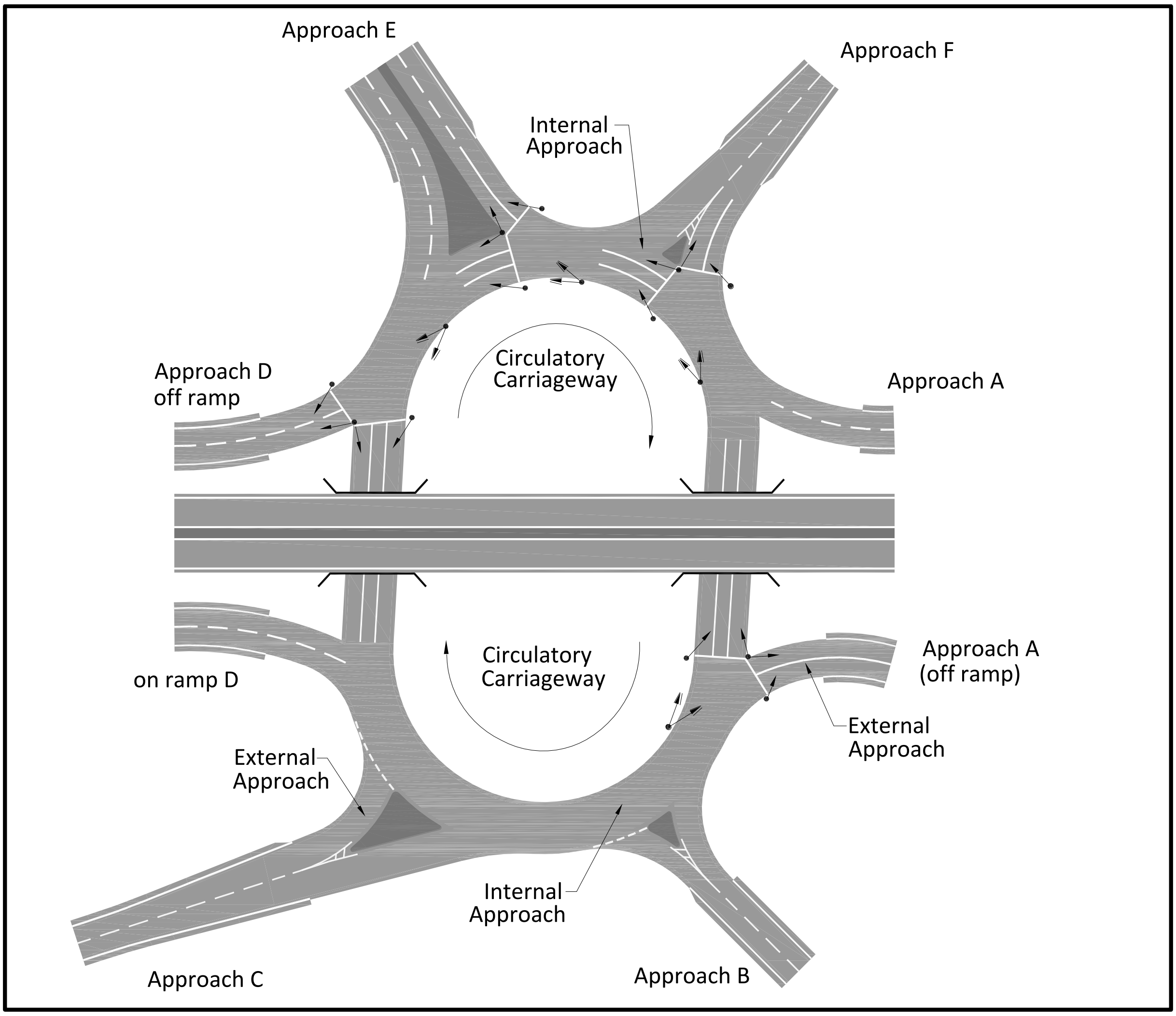


Figure 4.78: Large Signalized Roundabout with Partial Direct Signal Control

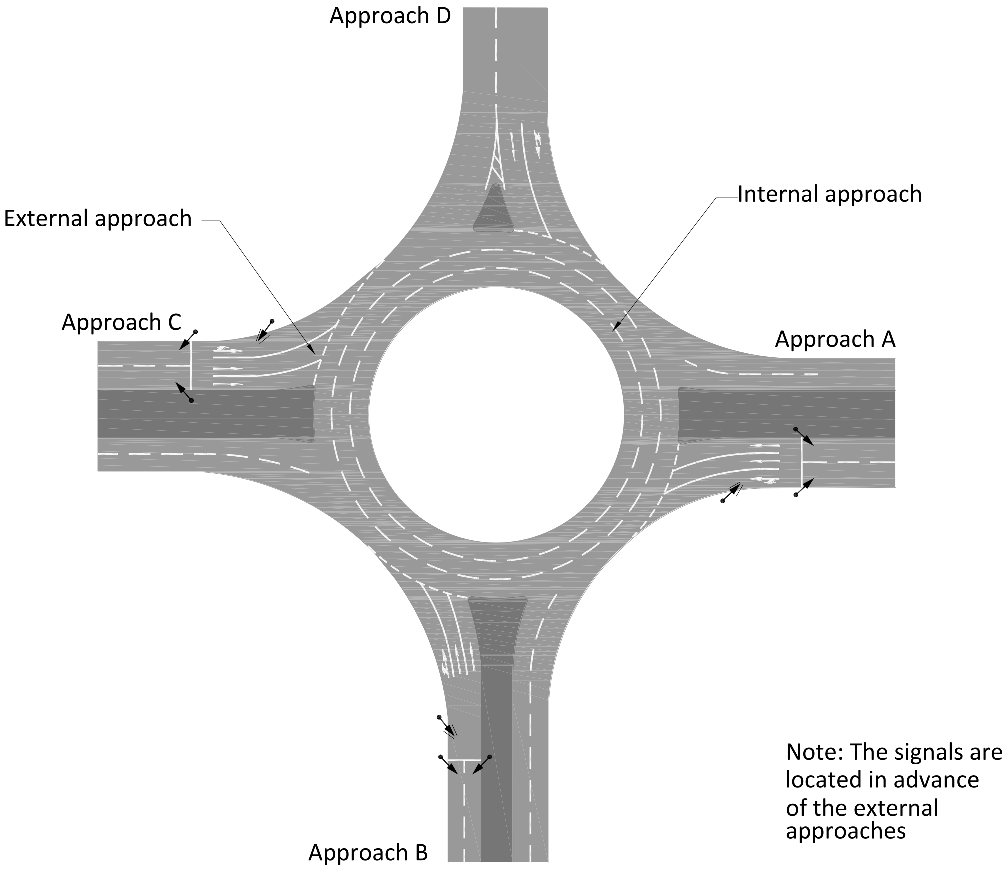


Figure 4.79: Signalized Roundabout with Indirect Signal Control

4.8.2.3 Duration of Signal Control

Full-time signal control is provided by permanently operating traffic signals. Under full-time signal control, the yield pavement marking across signalized entries to the roundabout should not be provided. Installing traffic signals for full-time signal control often involves making geometric modifications as an integral part of improving throughput in ways that might not necessarily be effective or improve safety on a roundabout with priority control.

Part-time signal control is provided by traffic signals that are switched on only at set times, generally peak flow periods, or by queue detectors under certain traffic conditions. When traffic flow is light, the signals are turned off and the roundabout operates under normal priority control. When traffic flow is directional, signal control can be used on different external approaches at different times of day.

Part-time signal control often causes lower off-peak delays. In addition, part-time signal control allows limited ability for the designer to make geometric changes as part of the signal installation because the physical layout suitable for priority control must be retained. Part-time control can cause confusion because of the need for associated permanent signing and striping, which can increase crashes when the traffic signals are not in use. For these reasons, full-time signal control is preferred to part-time control.

Part-time signal control is not suitable where any of the following apply:

- Requirement for pedestrian crossing facilities on the circulatory roadway

- Potential benefit from incorporation in a linked system
- Significant queues form throughout the day, not just during peak periods

The roundabouts shown on **Figure 4.78** and **Figure 4.79** could operate under either full or part-time signal control.

4.8.3 Geometric Modifications

4.8.3.1 General

The following geometric modifications may be required for signalized roundabouts to:

- Increase capacity on external approaches
- Increase capacity on the circulatory roadway
- Accommodate queues at internal stop lines on the circulatory roadway
- Improve forward visibility and inter-visibility in the intersection inter-visibility zone
- Provide alignment improvements and lane control measures
- Provide specific measures for pedestrians, cyclists, and buses

These modifications may take the form of the following:

- Additional external approach lanes
- Additional internal approach lanes within the circulatory roadway
- Increased size of splitter islands to achieve longer internal approaches
- Segregated facilities or signalled crossing facilities

4.8.3.2 External Approach Lane on the Outside

The provision of an additional external approach lane on the right, as shown on **Figure 4.80**, is appropriate where:

- Sufficient space is available within the central median.
- An approach lane to the left side would result in inadequate internal queuing capacity on the internal approach.
- The additional external approach lane does not significantly reduce queuing capacity on the internal approach to the right.

4.8.3.3 External Approach Lane on the Inside

The provision of an additional external approach lane on the inside (left), as shown on **Figure 4.81**, is appropriate where:

- Sufficient space is available on the inside.
- An external approach lane on the inside would result in inadequate queuing capacity on the internal approach on the right side.
- The additional external approach lane does not significantly reduce queuing capacity on the internal approach to the left.

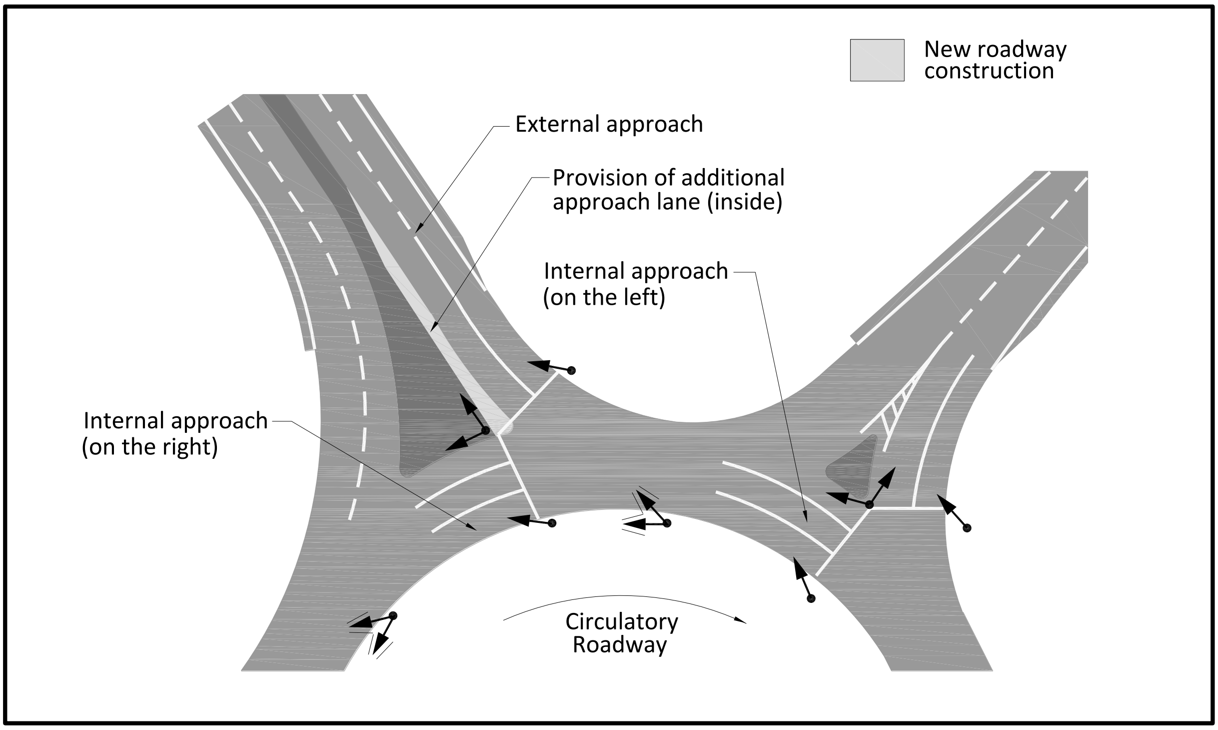


Figure 4.80: Provision of Additional External Approach Lane on the Outside

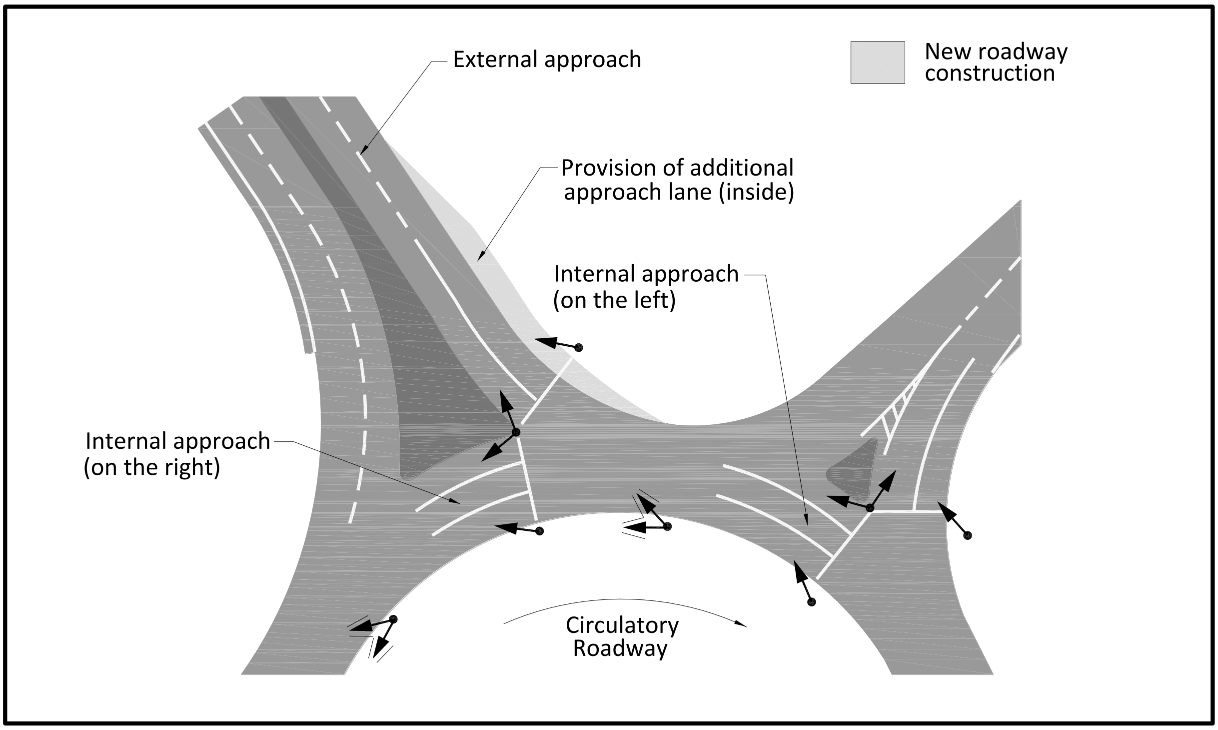


Figure 4.81: Provision of Additional External Approach Lane on the Inside

4.8.3.4 Internal Approach Lanes

As shown on **Figure 4.78**, the internal queuing capacity on the circulatory roadway to the left of approach E is limited by the width of the median on the multi-lane approach. Similarly, internal queuing capacity is severely limited to the left of approach F by the size of the deflection island. Internal approach lanes less than 15 m in length may result in the blockage of the departure lane, particularly where large vehicles are present. The effect of internal queues on circulation should be considered when assessing the roundabout performance.

Under full-time signal control, the capacity on the internal approaches to approaches E and F shown on **Figure 4.78** could be improved by geometric modifications such as shown for approach F on **Figure 4.82** by significantly reducing the approach radii.

Under part-time signal control, the scope for geometric modifications is reduced because the design is required to comply with Design for Roundabouts, of this manual.

Figure 4.82 shows how the queuing capacity on the internal approach to the right of approach F may be improved by increasing the size of the splitter island and realigning the external approach.

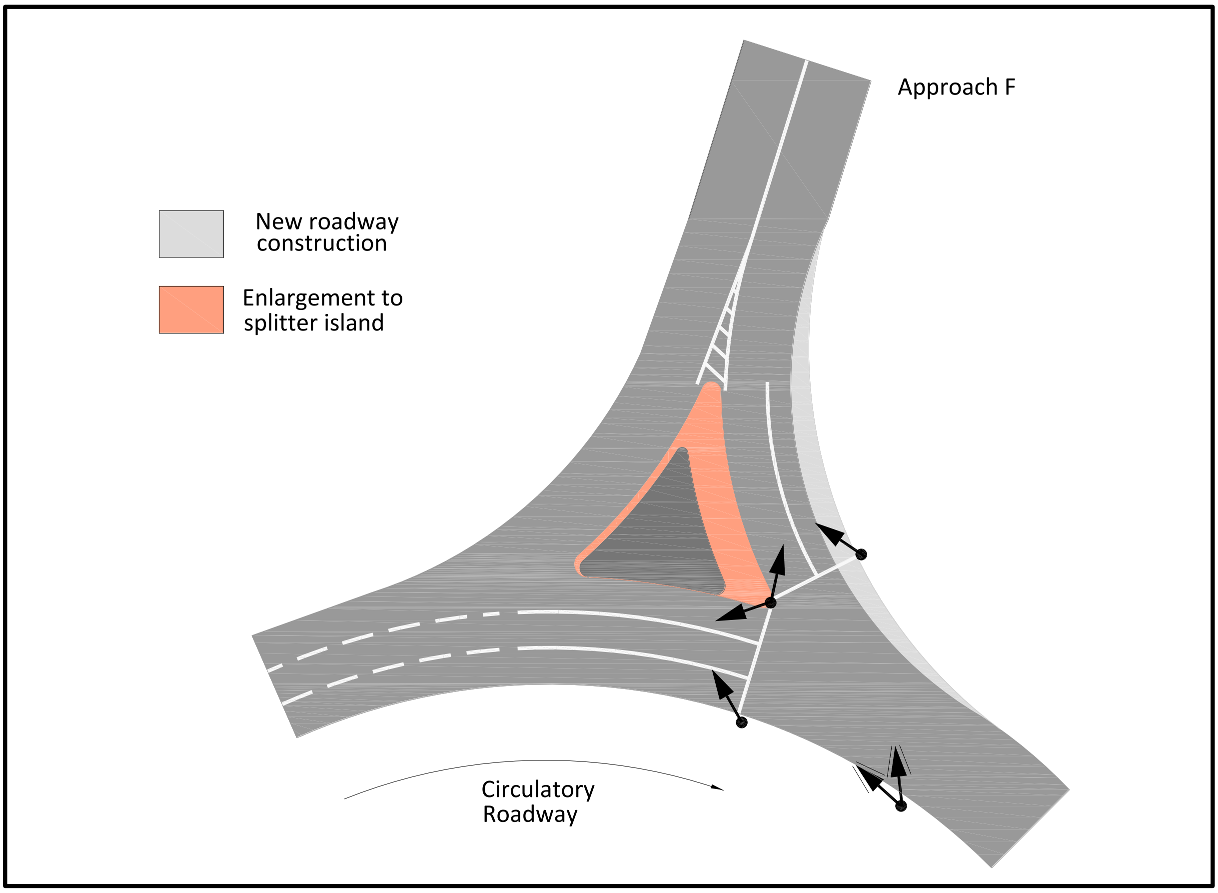


Figure 4.82: Improvement to Internal Queuing Lanes

5 GRADE SEPARATIONS AND INTERCHANGES

5.1 INTRODUCTION

The capability to accommodate high traffic volume safely and proficiently through intersections depends mainly on how intersecting traffic is controlled. The maximum proficiency, safety and capability are accomplished when intersecting through traffic lanes are physically divided.

A grade-separated junction separates conflicting traffic streams in vertical space with a combination of ramps and grade separations at the junction of two or more roadways. This reduces or eliminates traffic conflicts, improves safety and increases traffic capacity. Crossing conflicts are eliminated by grade separations and turning conflicts are eliminated or minimized depending on the interchange configuration.

In general, the less junction interference is experienced, the junction is better able to handle complex high traffic flows but it is more costly to construct.

Terrain, right-of-way, traffic capacity, traffic configuration, roadway classification, signing requirements, economics, design speed, access control, capacity and safety all influences the selection and design of grade separations and interchanges. In order to determine the appropriate layout of an interchange, alternative concepts should be prepared and each site should be studied accordingly since interchange types vary widely. Essential interchange elements include the freeway, cross road, median, ramps, and auxiliary lanes.

5.2 WARRANTS FOR GRADE SEPARATION AND INTERCHANGES

Interchanges are very costly and should be used only where necessary. High cost of constructing an interchange limits its use to those cases where the additional expenditure can be justified. The following six conditions should be considered when determining if an interchange is justified at a particular site:

1. **Design Designation**
   The warrant for providing grade separations or interchanges for all intersecting roadways crossing the highway is to develop a highway with full access control. For example, intersections on an expressway or major arterial becoming part of an expressway alignment or the alignment of another planned or existing limited access facility falling in the path of a new expressway warrants grade separation.

2. **Reduction of bottlenecks or spot congestion**
   Interchanges is required where the intersection capacity is not sufficient, resulting in intolerable congestion on heavily travelled routes.

3. **Reduction of crash frequency and severity**
   A grade separation or interchange may be warranted where there is a high rate of serious accidents at some of the at-grade intersections and as such low-cost corrective measures fails to lessen or eradicate the prevailing risks. In addition to the reduction in crash frequency and severity, the operational efficiency for all traffic movements is also improved at the interchange.

4. **Site topography**
   At some locations, the type of intersection that can be constructed economically is grade-separated design. To satisfy appropriate design criteria, the topography at the site may be such that any other type of intersection is physically impossible or not cost effective.

5. **Traffic volume warrant**
   A specific volume of traffic at an intersection cannot be completely rationalized as the warrant for an interchange. An at-grade interchange would certainly be a warrant if the traffic volumes were in excess of the capacity of intersection. The elimination of traffic conflicts due to high crossing volume greatly improves the movement of traffic; hence, interchanges are desirable at crossroads with heavy traffic volumes.

6. **Road-user benefits**
   the road-user costs (such as wear on tyres, fuel usage, travel time, repairs, crashes that result from speed changes, stops, and waiting) associated with stopped delays at congested at-grade intersections are large. The added cost of the extra travel distance on interchanges is counterbalanced by the cost savings resulting from the reduction in stopping and delays. The relation of road-user benefits to the cost of improvement indicates an economic warrant for that improvement.

5.3 GRADE SEPARATIONS

Grade separation is the separation of different flows of traffic using physical means. Various types of structures are employed to separate the grades of two intersecting roadways or a highway and a railroad to cross one another at different elevations or levels typically by providing a bridge structure.

A grade separation structure should conform to the natural lines of the highway approaches in alignment, profile, and cross section.

A detailed study should be made at each proposed highway grade separation to determine whether the major roadway should be carried over or under the crossroad. Often this decision is based on features such as topography and highway classification. The first consideration is to select the design that best fits the existing topography, as they are most pleasing and economical to construct and maintain. It may be appropriate to study secondary factors where topography does not govern, and the following general guidelines should be examined:

1. An underpass highway has a general advantage that an approaching interchange may be easily seen by drivers and providing advance warning of the likely presence of interchange ramps.

2. A flyover offers best possibility for stage construction, both in the highway and structure, with minimum damage of the original investment and the ultimate development is reached without loss of the initial facility.

3. Drainage challenges may be reduced by carrying the major highway over the crossroad without altering the crossroad grade.

4. A cost analysis that takes into account the bridge type, span length, and roadway cross section, angle of skew, soil condition and cost of approaches will determine which of the two should be adopted.

5. The overcrossing structure has no limitation as to vertical clearance, which can be a significant advantage in the case of oversized loads.

6. It may be appropriate to have the higher volume facility depressed and crossing under the lower volume facility to reduce noise impact.
7. Where there is no prominent advantage to the selection of either an underpass or a flyover, the type that provides better sight distance on the major road (desirably safe passing distance if the road is two lane) should be preferred.

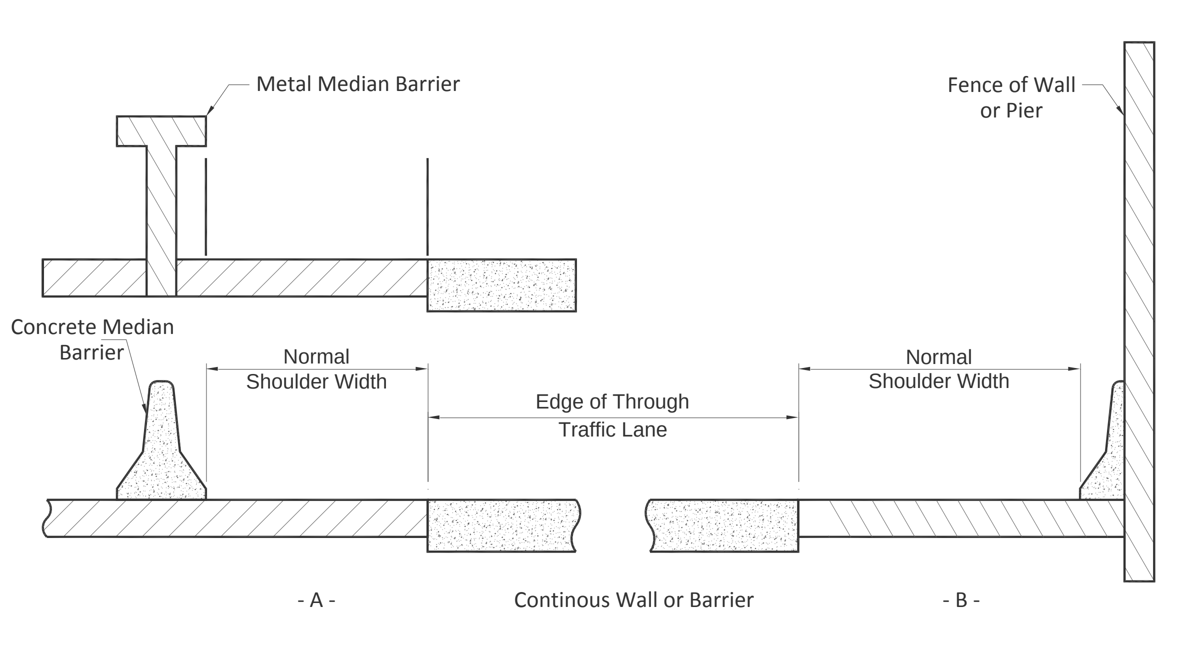
5.3.1 Underpass

The underpass should be consistent with the design standards of the rest of the facility to the extent practical. It is desirable that the entire roadway cross section, including the median, carriageway, shoulders, and clear roadside areas, be carried through the structure without change. However, some reduction in the basic roadway cross section may be needed due to structural design limitations; vertical clearance limitations; controls on grades and vertical clearance; limitations due to skewed crossings; and cost factors.

5.3.1.1 Lateral Offsets

The minimum lateral offsets at underpasses are illustrated **Figure 5.1**. For further details, see the *AASHTO's Roadside Design Guide, 2011* [5].

The change in lateral width should be accomplished through gradual modifications in the cross section of the approach roadway rather than abruptly at the structure, where the decrease in horizontal lateral offset through an underpass cannot be avoided. Such transitions in width should have a gradual rate of 50:1 or more.



(Source: *AASHTO's A Policy on Geometric Design of Highways and Streets, 2011* [2])

Figure 5.1: Lateral Offsets for Major Roadway Underpasses

5.3.1.2 Vertical Clearance

With the aid of the established policies of the highway system, vertical clearance is typically determined for an entire route. However, the vertical clearance of all structures above the carriageway and shoulders should be kept at least 0.3 m greater than the legal vehicle height. In addition to this, allowance for future resurfacing should be provided. Additional vertical clearance is desirable to cater for accumulation of snow or ice and occasional passage of over height vehicles

5.3.2 Flyover

The roadway dimensions over a flyover or other bridge should be the same as that of the approach roadway, unless the cost becomes excessive. Overpass structures should have liberal lateral offset on the roadways at each level. All piers and abutment walls should be suitably offset from the carriageway.

5.3.2.1 Bridge Railing

The typical bridge railing has some form of concrete base or parapet on which metal or concrete rail or rails are mounted on structurally adequate posts. The bridge railing should be designed in a manner to accommodate the impact of design vehicle on the structure. That is, without penetrating or vaulting over the railing, the design vehicle should be effectively redirected. Similarly, the railing should not squeezed or hurdle the design vehicle, causing sudden slowing down or twist, and it should not cause the design vehicle to overturn.

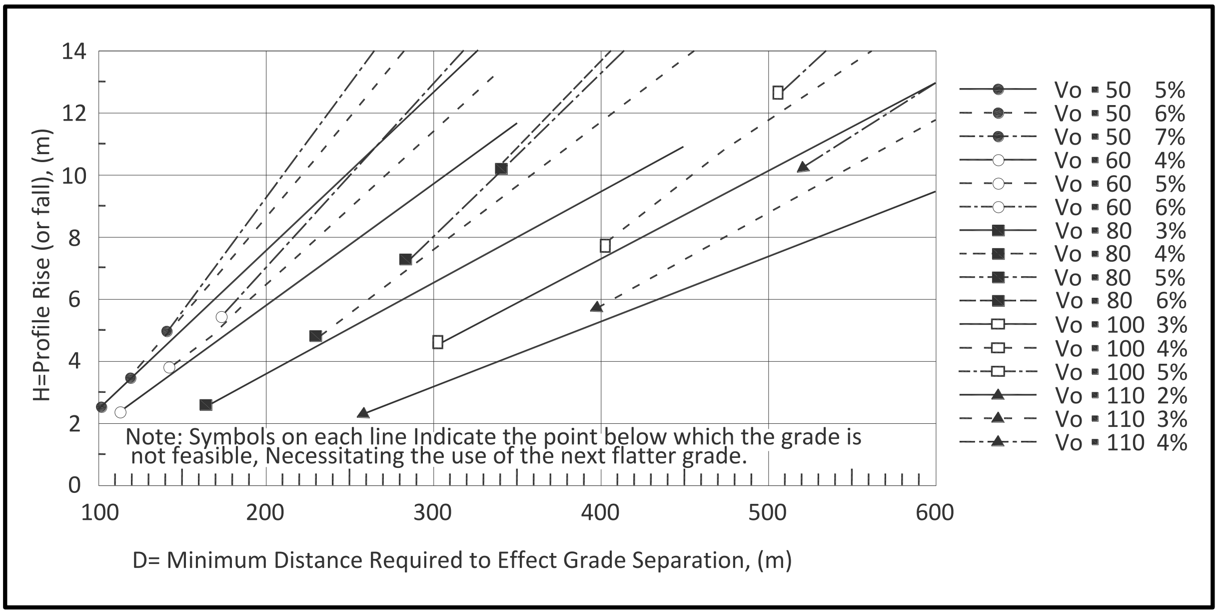
Stopping sight distance might be restricted if bridge railings are located on the inside of horizontal curves. To provide adequate stopping sight distance, alteration of the horizontal alignment or the offset to the bridge railing may be needed.

5.3.2.2 Lateral Offset

On overpass structures, it is appropriate to carry the full width of the approach roadway. Exceptions may be made on major structures with a high unit cost for facilities. No exceptions can be made for freeways. The selection of cross-section dimensions that are different from those on the approach roadway should be subject to individual economic studies. The minimum structure width should match the kerbed approach roadway in the case of a kerbed roadway. Both left and right bridge railing on the structure should align with the guardrail on the approach roadway. Additional width for speed-change lanes or weaving sections is needed across overpass structures at certain interchanges. The minimum lateral offset to the bridge rail should be at least equal to the width of shoulder on the approach ramp, where the auxiliary lane is a continuation of a ramp. Where auxiliary lane is a weaving lane connecting entrance and exit ramps or is a parallel-type speed-change lane across the entire structure, the offset to the parapet should be of uniform width and be at least equal to the shoulder width on the ramp,.

5.3.3 Longitudinal Distance to Achieve Grade Separation

The roadway gradient, the design speed, and the amount of rise or fall needed to achieve the separation all are obligatory for the longitudinal distance needed for adequate design of a grade separation. **Figure 5.2** shows the horizontal distances needed in flat terrain. whether or not a grade separation is practical for given conditions, what gradients may be involved, and what profile adjustments, if any, may be needed on the cross road, the **Figure 5.2** may be used as a guide to quickly determine the preliminary design.



(Source: *AASHTO's A Policy on Geometric Design of Highways and Streets, 2011* [2])

Figure 5.2: Distance Needed to Achieve Grade Separation

5.4 INTERCHANGES

With respect to function, interchange configuration are covered in two categories:

1. System Interchange
2. Service Interchange

The term "system interchanges" is used to identify interchanges that connect two or more freeways whereas the term "service interchange" applies to interchanges that connect a freeway to lesser facilities.

This section includes various commonly used interchange configurations. For further details and additional information, refer Chapter 10 of *AASHTO's A Policy on Geometric Design of Highways and Streets, 2011* [2].

5.4.1 Three-Leg Interchanges

Three intersecting legs comprised three-legged interchanges. They usually consist of one or more roadway grade separations and one-way roadways for all traffic movements. Three-legged interchanges should only be used when one of the three legs is permanently terminated because of difficulty in future expansion.

Some of the examples of three-leg interchanges are shown in **Figure 5.3**, most common type of which is the trumpet interchange.

5.4.1.1 Trumpet Interchange

Trumpet interchange is a common junction pattern for three intersecting legs. **Figure 5.4** illustrates the widely used trumpet pattern. The heavier Traffic volume should be favoured on direct alignment, while, the lesser volume can be looped. Skewed crossings are more desirable than right-angle crossings because the skewed crossing has a rather shorter travel distance and flatter turning radius for the heavier right-turning volume, and there is less angle of turn for both right turns. A disadvantage of this interchange is that it is difficult to convert to a four-legged interchange. **Figure 5.5** shows the actual example of a trumpet interchange.

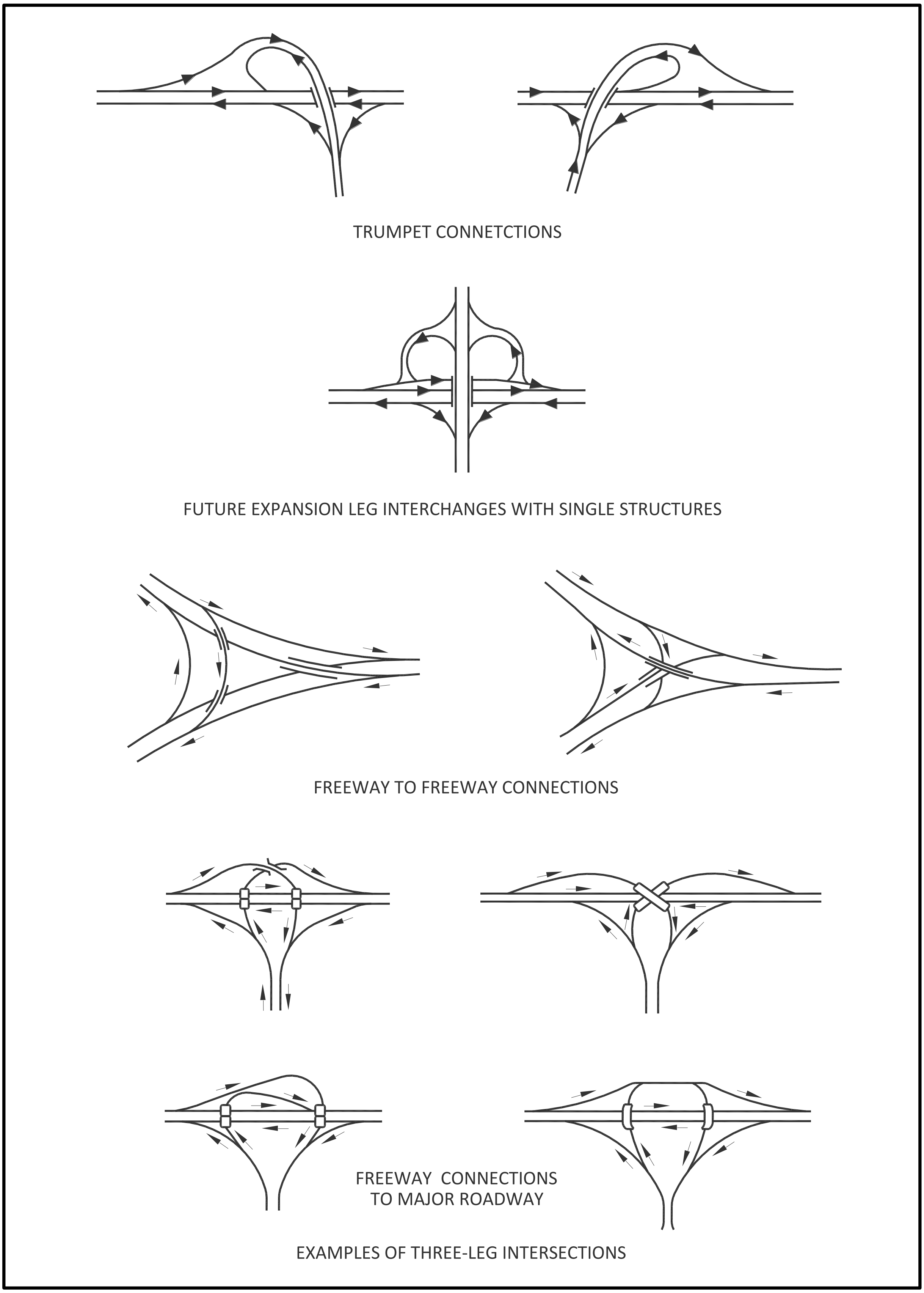


Figure 5.3: Examples of Three-Legged Interchanges

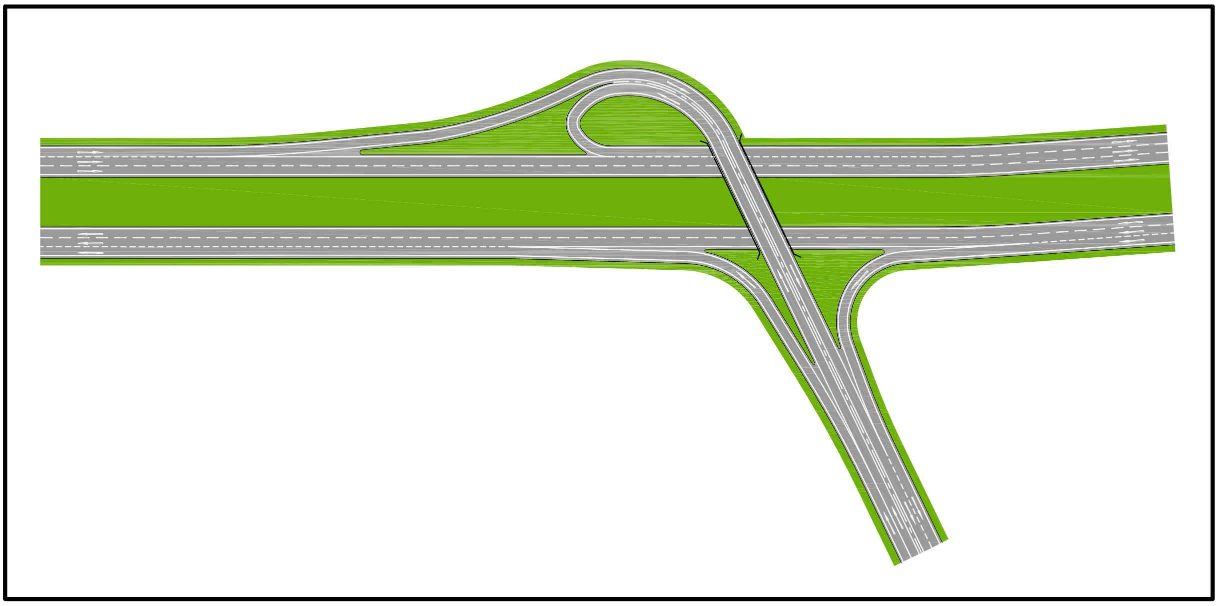


Figure 5.4: Typical Trumpet Interchange

Figure 5.5: Example of Trumpet Interchange

5.4.2 Four-Legged Interchanges

Interchanges with four intersection legs may be grouped under six general configurations:

1. Diamond Interchanges
2. Partial Cloverleaf
3. Full Cloverleaf
4. Double Roundabout Interchange
5. Directional Interchanges

5.4.2.1 Diamond Interchange

Diamond interchange is the simplest form of interchange giving access from through route to the existing ground level road system. This type of interchange requires single bridge structure, provides a clear through route for mainline traffic and ramp connection in all directions with a single cross route by means of at-grade signal controlled junction. **Figure 5.6** shows a typical arrangement of Tight Diamond Interchange. Some basic forms of diamond interchanges are shown in **Figure 5.7.**

Advantages

1. High design standard single exits in advance of the structure.
2. High design standard single entrances beyond the structure.
3. Little right-of-way comparatively needed.
4. Relatively low construction cost.
5. Direct crossroad turning manoeuvres.
6. Primary roadway signing is being simplified by single exit feature.
7. On or under the structure, no need for speed change lanes.
8. No weaving on the primary roadway.

Disadvantages

1. Ramp intersection capacity limits the Overall capacity.
2. Due to right turning movements, capacity is lowered on the minor road.
3. Without signalization, there is a likelihood of increased accidents.
4. Wrong-way movements' probability.
5. At the minor road, turning traffic from the primary roadway is accommodated to stop. Storage lane treatment may be required.
6. Future expansion not likely to be a prospect.

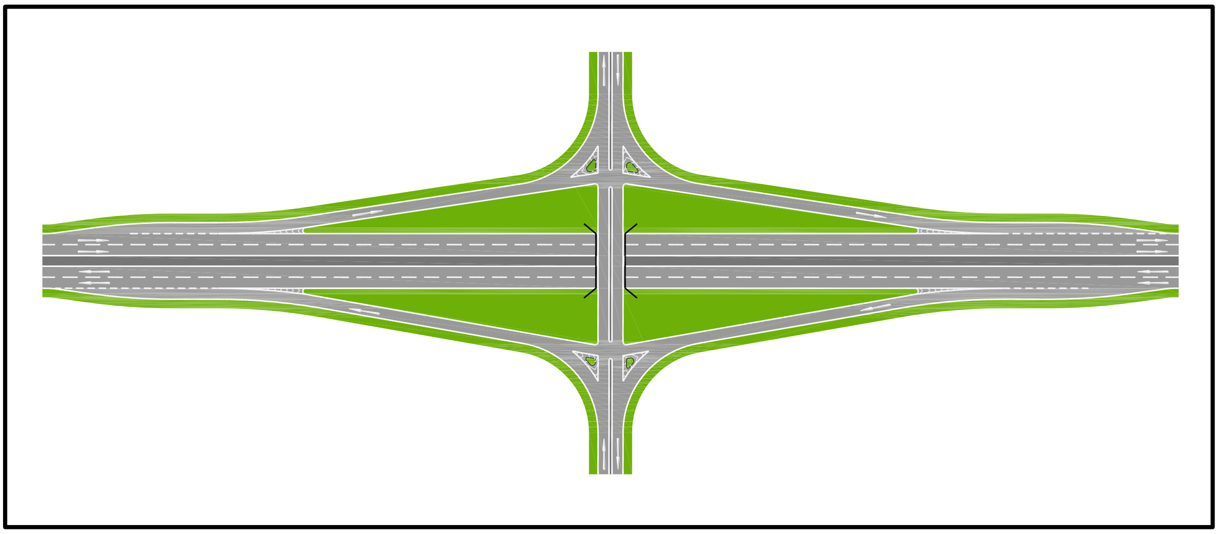


Figure 5.6: Typical Diamond Interchange

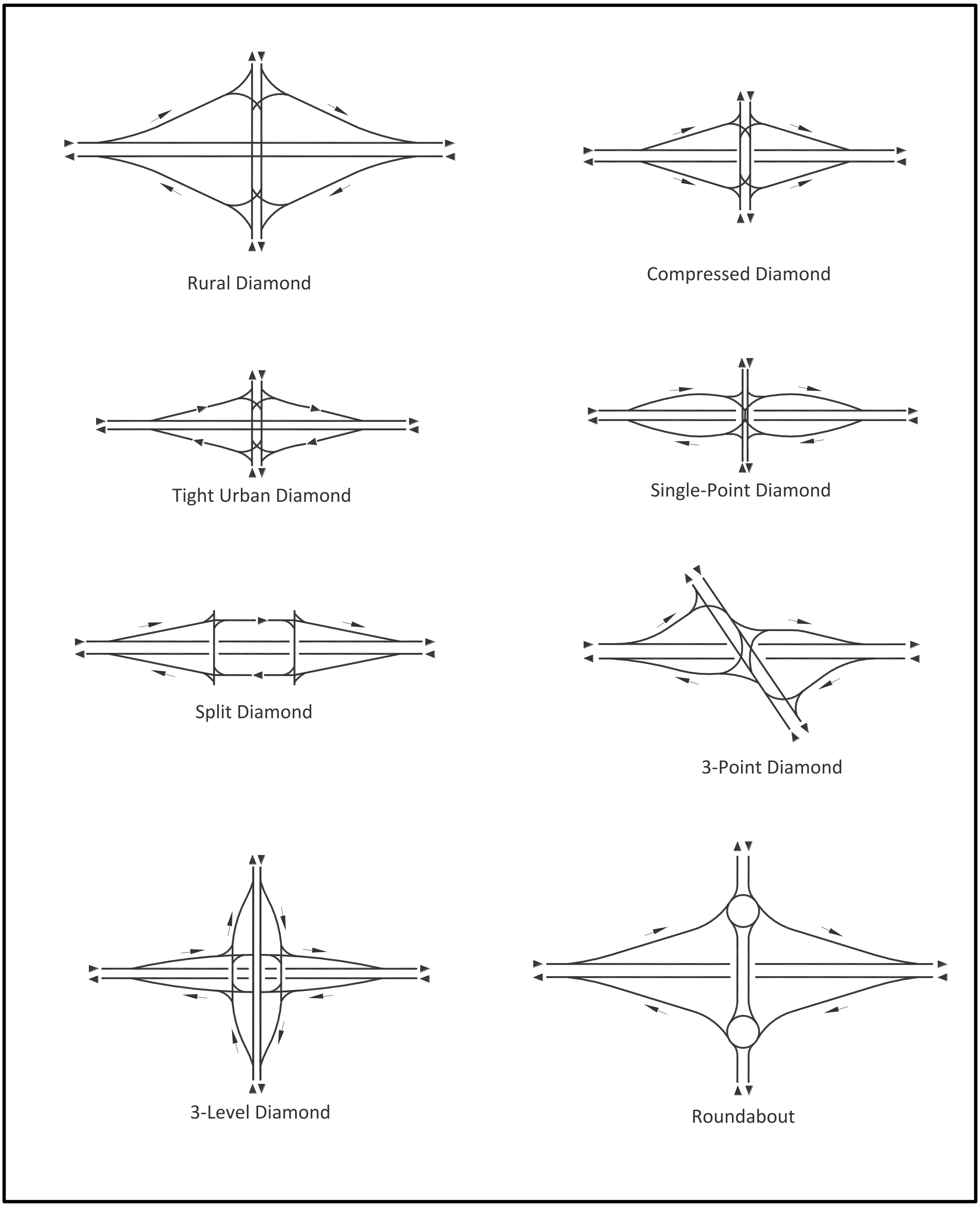


Figure 5.7: Basic Forms of Diamond Interchange

5.4.2.2 Full Cloverleaf Interchange

Cloverleaf interchanges are four-legged interchanges that comprise of loop ramps to accommodate right-turning movements. In this form, all traffic is unstopped. The loop ramps form weaving sections that generally are the controlling traffic operational feature of the interchange. Where traffic volumes are even moderate, the full cloverleaf creates the potential for substantial speed differentials along the high speed mainline. In such cases, the use of Collector-Distributor (CD) roads is considered a best practice. Operating experience in many places demonstrates that full cloverleaf interchanges produce greater frequencies of multi-vehicle crashes along the mainline weaving sections than other forms that do not have weaving. Where space is available, they are better adjusted to suburban or rural areas, because cloverleaf interchanges are significantly more expansive than diamond interchanges and consequently are less common in urban areas.

The use of CD roads for the freeway provides for both single-exit design and removal of lower speed weaving traffic from the higher speed freeway mainline. However, with these design requirements, the interchange will take considerably more right-of-way and have a much greater construction cost than other forms of interchanges suitable for low to moderate volumes in rural conditions. **Figure 5.8** illustrates a typical full cloverleaf interchange. **Figure 5.9** shows an example of existing cloverleaf interchange.

Advantages

1. Elimination of Right-turn conflicts.
2. Single structure design.
3. Unnecessary traffic signals needed.
4. Phased construction being offered by itself.

Disadvantages

1. Involves large right-of-way areas.
2. Capacity can strictly be restricted by generated weaving manoeuvres.
3. For speed control of inner loop, insufficient deceleration length and weaving length available.
4. For collector distributor roads, high weave volumes are needed.
5. Traffic signings Obscured due to double exit on the primary roadway.
6. Reduced safety features.
7. Right turners required extra travel distance.
8. Due to tight curves, large trucks may experience complications.

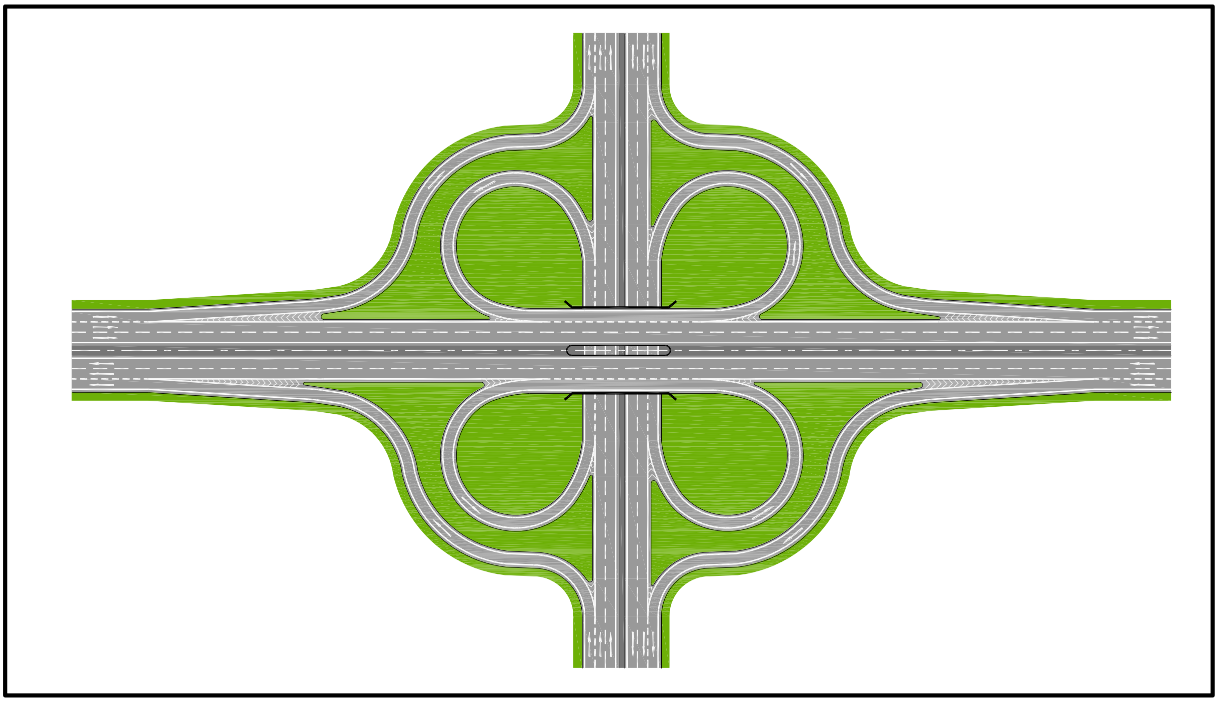


Figure 5.8: Typical Full Cloverleaf Interchange

Figure 5.9: Example of Cloverleaf Interchange

5.4.2.3 Partial Cloverleaf Interchange

A partial cloverleaf interchange is one in which there are at least one and no more than three loop ramps. **Figure 5.11** shows the various forms of Partial cloverleaf interchanges. This interchange is suitable for locations where by removing two right-turn movements from the intersections, the remaining right-turn conflicts can be tolerated. A typical form of Partial cloverleaf is shown in **Figure 5.10**. An example of existing cloverleaf interchange is shown in **Figure 5.12.**

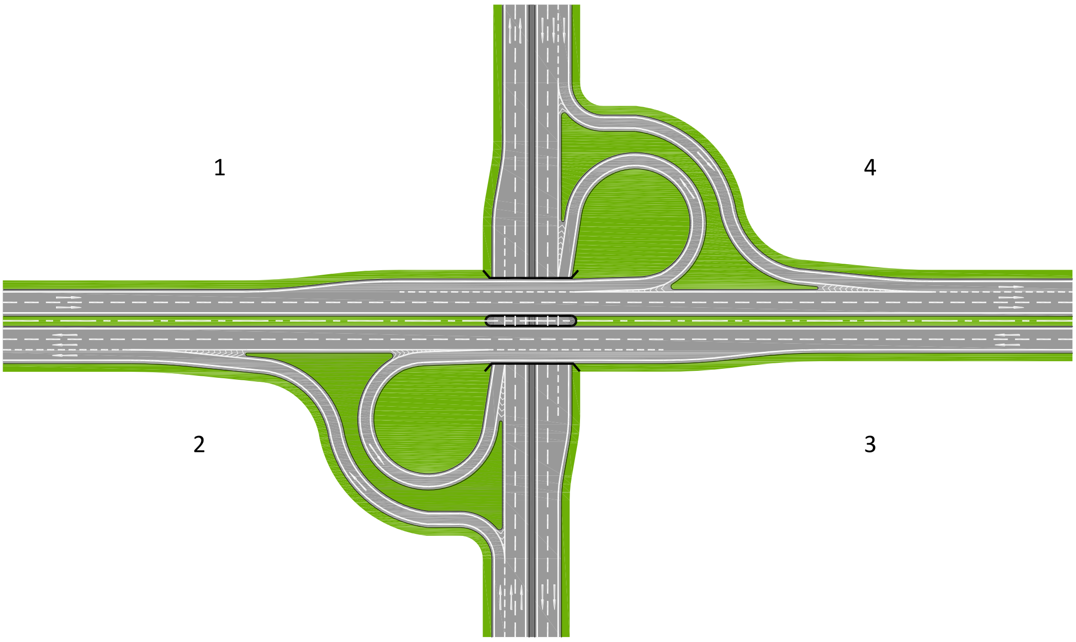


Figure 5.10: Typical Partial Cloverleaf Interchange

Advantages

1. Construction phasing made appropriate.
2. In advance of structure, exit terminals are provided.
3. Elimination of weaving.
4. Traffic signing is being simplified by single exit.
5. Optimization of traffic volume/capacity can be organized.
6. If structure opening is sufficiently wide, future expansion is possible.

Disadvantages

1. For right-turns, minor road has stop condition.
2. Right-turn storage is being required by minor road.
3. The ramp terminals capacity and safety is limited by points of conflict on the minor roadway.
4. At the minor roadway, left-turn primary roadway traffic stops.

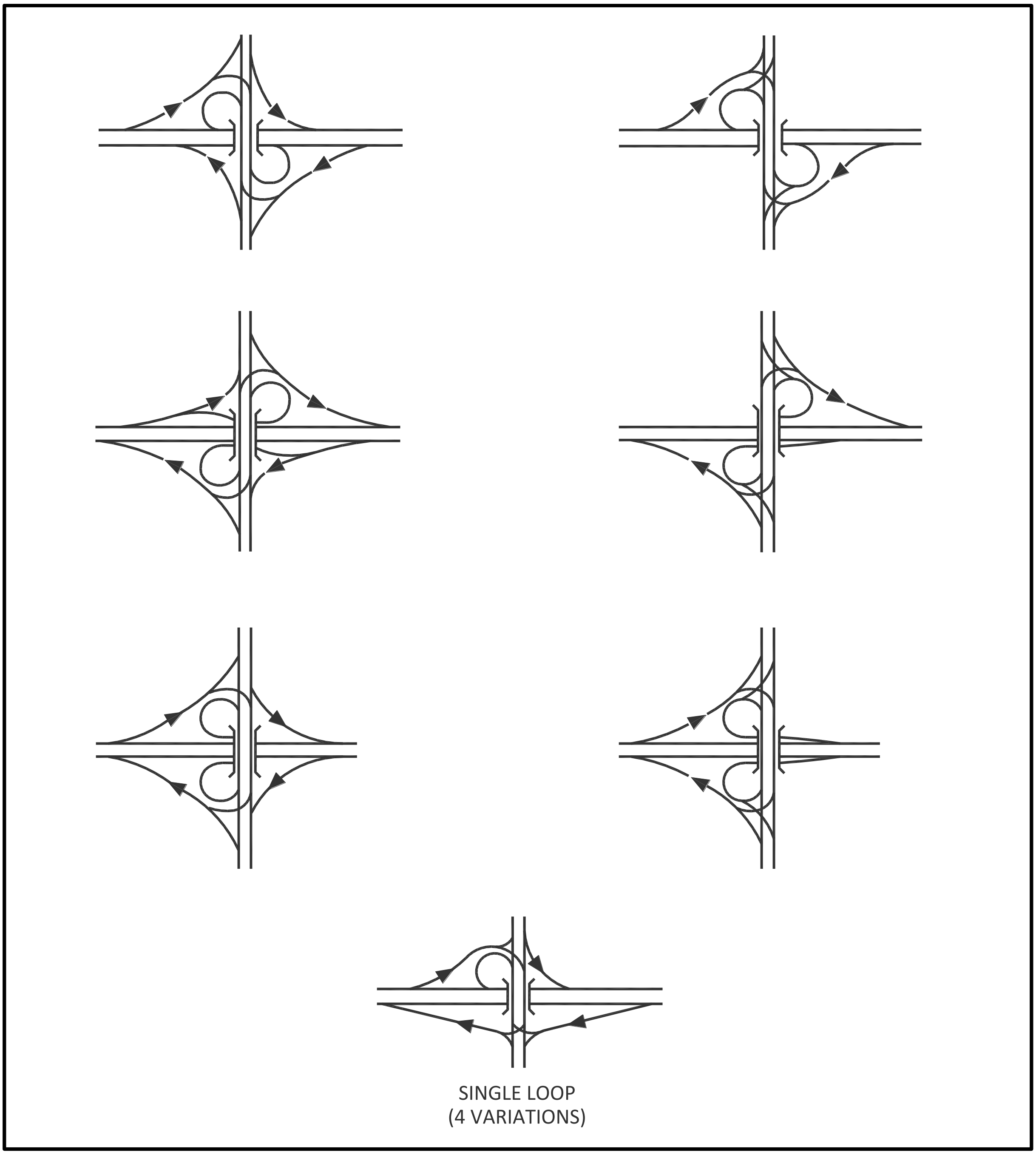


Figure 5.11: Basic Forms of Partial Cloverleaf

Figure 5.12: Example of Partial Cloverleaf Interchange

5.4.2.4 Directional Interchange

A direct connection is defined as a one-way roadway that does not deviate greatly from the intended direction of travel. Interchanges that use direct connections for the major right-turn movements are termed directional interchanges. See **Figure 5.14** for basic forms of Directional Interchange. Direct connections for one or all right-turn movements would qualify an interchange to be termed directional, even if the minor right turn movements are accommodated on loops.

A direct connection is defined as a one-way roadway that does not diverge greatly from the planned direction of travel. The term directional interchange can be described as those interchanges that use direct connections for the major right-turn movements.

Figure 5.14 depicts basic forms of Directional Interchange. An interchange to be termed eligible directional interchange if direct connections for one or all right-turn movements would be possible, even if the minor right turn movements are accommodated on loops. Higher levels of service can be realized on direct connections and, in some instances, on semi-direct ramps because of relatively high speeds and better terminal design.

For one or more right turning movements, directional interchanges may have one or more grade separations with direct or semi-direct ramps.

Semi-direct or direct connections for one or more right-turning movements are often required at major interchanges in urban areas. Interchanges involving two primary roadways nearly always call for directional layouts. In such cases, turning movements in one or two quadrants are often comparable in volume to the through movements.

At major interchanges in urban areas, semi-direct or direct connections for one or more right-turning movements are often essential. For directional layouts, interchanges must be involving two primary roadways. In such cases, turning movements in one or two quadrants are often similar in volume to the through movements.

Figure 5.13 illustrates the typical form of directional interchange with two semi-direct connections. An example of existing directional interchange can be seen in **Figure 5.15**.

Advantages

1. Travel distance reduced.
2. Volume and speed Increased.
3. Elimination of weaving.
4. While driving on a loop evades the indirection.
5. Levels of service higher.
6. Little right-of-way necessary.

Disadvantages

1. Costs of construction substantial.
2. Time-consuming and detailed study required.

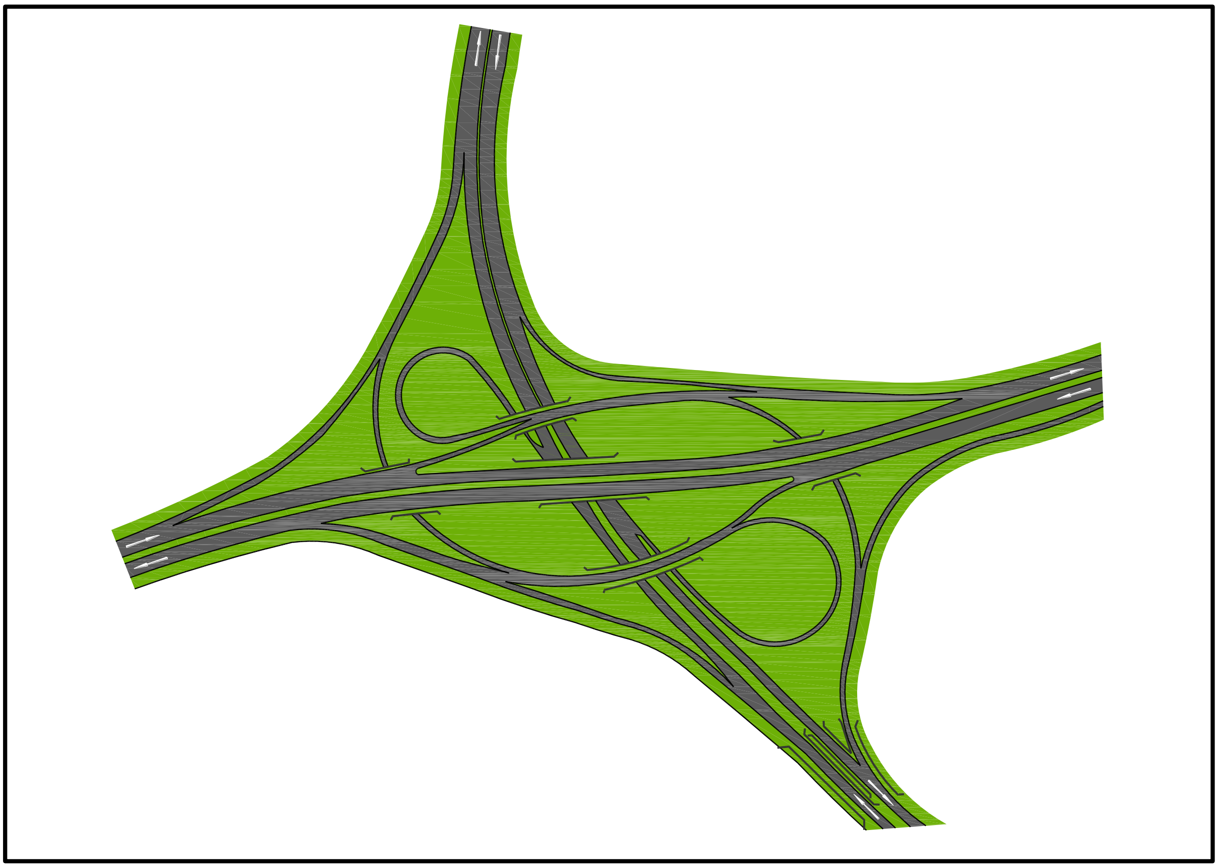


Figure 5.13: Typical Directional Interchange-Two Semi-direct Connections

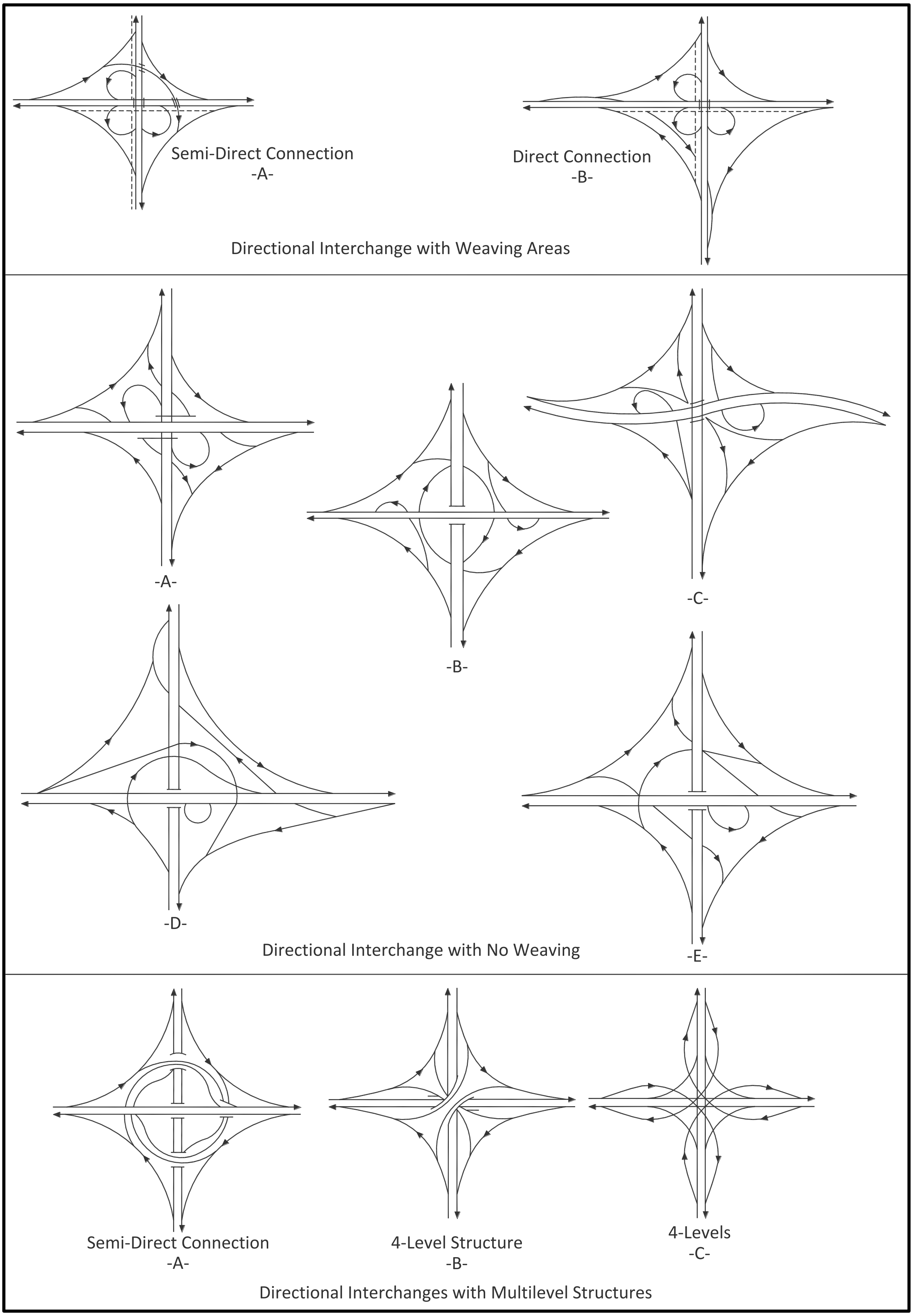


Figure 5.14: Basic Forms of Directional Interchange

Figure 5.15: Example of Existing Directional Interchange

5.4.2.5 Grade Separated Roundabouts

Grade Separated Roundabouts are probably the most common type of interchanges in many parts of the world. The layout is suitable for a situation in which the mainline crosses a traffic stream that can be routed through a roundabout. In general, the roundabout and cross route are at existing grade while the mainline is either elevated or depressed. **Figure 5.16** illustrates the typical layout of grade-separated roundabout. **Figure 5.17** shows an example of existing grade separated roundabout.

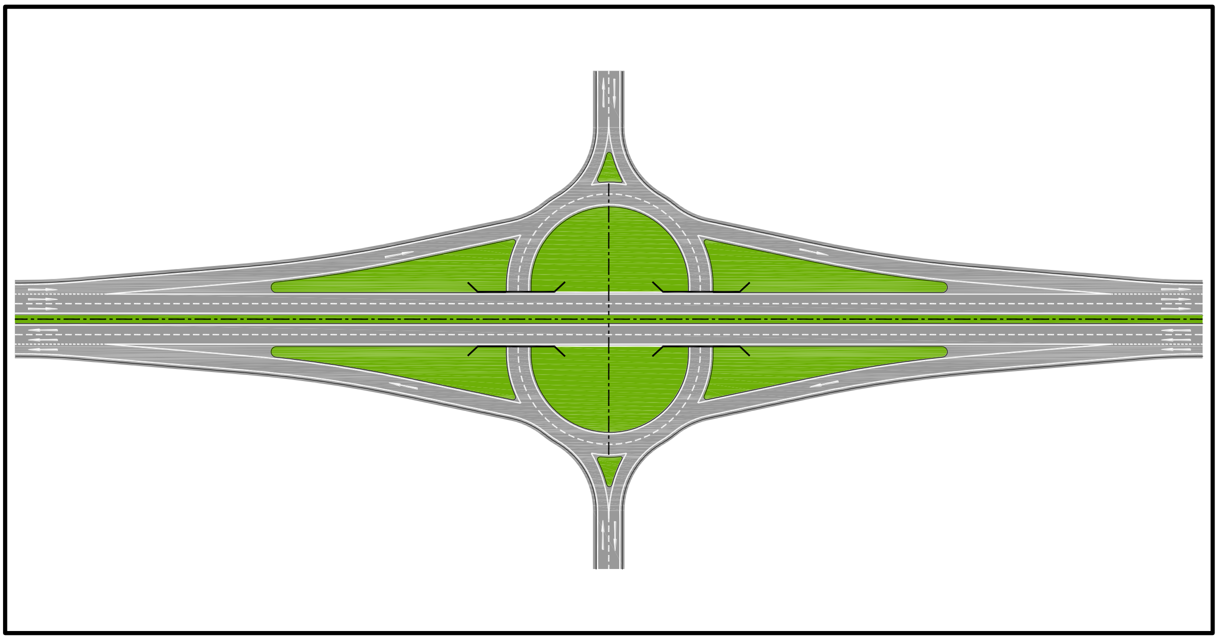


Figure 5.16: Typical Grade Separated Roundabout

Figure 5.17: Example of Existing Grade Separated Roundabout

Advantages

1. It can be adjusted to deal with more than one cross routes.
2. Allows full connectivity.
3. Avoids mainline merging.

Disadvantages

1. Requires two or more bridge structures.

5.4.2.6 Combination Interchange

When one or two turning movements have very high volumes with respect to the other turning movements, analysis may indicate the need for a combination of two or more of the previously discussed interchanges.

5.4.3 Design Considerations

5.4.3.1 Selection of Type of Interchange

An important principle of expressway and freeway planning and design is the selection of the most appropriate interchange form for the context as defined by functional classification of road and interchange (System or Service Interchange) and location (urban or rural).

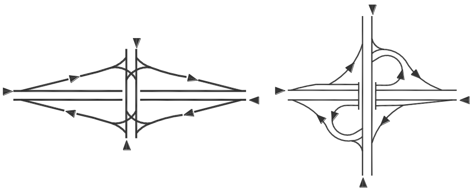
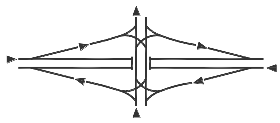
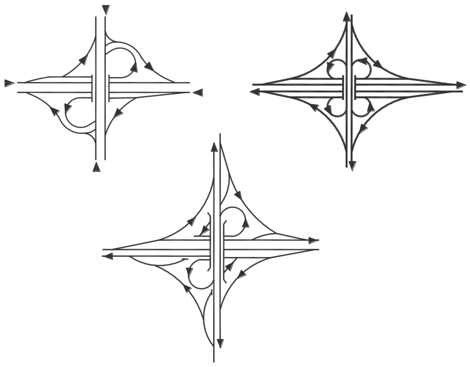
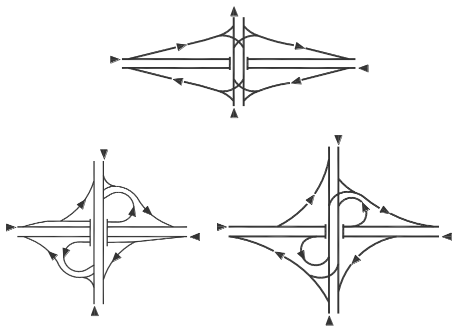
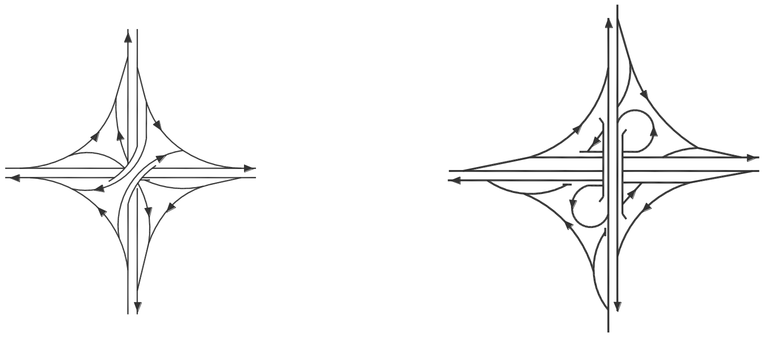
Type of Intersecting Facility	Rural	Urban
Lower Volume Arterial		
Higher Volume Arterial		
Freeway		

Figure 5.18: Appropriate Interchange Type Related To Type of Intersecting Facility

In rural areas, the spacing of interchanges is likely to be so great that each can be considered entirely on its own merits. Topographical and traffic flow considerations predominate, and consistency of exit patterns and minimizing of weaving on the mainline have a considerable influence on the choice. If a new route is being designed, it is good practice to consider it in its entirety. This requires that the interchanges are planned in to the location studies so that the final alignment is compatible (in three-dimensional terms) with the interchange sites.

In urban areas, interchanges are closer, and each interchange is likely to be influenced by the next one upstream and downstream. Consideration therefore, needs to be given to issues of capacity, weaving and lane balance on the mainline, which in turn may limit the choice of interchange type.

On a continuous urban route, all the interchanges should be considered together as a system, rather than being considered individually. Arrangements for the entire corridor can be sketched, and alternative interchange strategies can be developed, analysed and compared. It is important not to forget the intersecting minor roads, and confirm that they are suitable for the additional traffic, which the presence of an interchange will channel onto them.

In general, cloverleaf interchanges are less well suited to urban areas because of the amount of land they occupy. **Figure 5.18** summarizes the designation of appropriate interchange types based on location and type of road.

5.4.3.2 Approach to Structure

Alignment, Profile and Cross Section

1. The design speed, alignment, profile, and cross section in the intersection area, should be consistent with those on the approaching highways.
2. Preferably, the geometric design at the highway grade separation should be better than that for the approaching highways to counterbalance any possible sense of restriction caused by abutments, piers, kerbs, and rails.
3. Any relatively sharp horizontal or vertical curves should be avoided.
4. The general controls for horizontal and vertical alignment and their combination, should be adhered to closely.
5. The alignment and profile of the through highways at an interchange should be relatively flat with high visibility.
6. Sometimes it is not practical to design both intersecting roadways on a tangent with flat grades. In such case, major highway should be given preference.
7. Gradients that may slow down commercial vehicles or that may be difficult to negotiate under icy conditions should be avoided.

Sight Distance

1. Sight distance on the highways through a grade separation should be at least as long as that needed for stopping and preferably longer.
2. Where exits are involved, decision sight distance is preferred, wherever practical.
3. Design of the vertical alignment should be same as that of the approaching highway.
4. The horizontal sight distance limitations of piers and abutments at curves usually present a more difficult design challenge than that of vertical limitations. Thus, above-minimum radii should be used for curvature on highways through interchanges. If sufficiently flat curvature cannot be used, the offset to abutments, piers, or rails should be increased to obtain the proper sight distance, even though this involves increasing structure spans or widths.

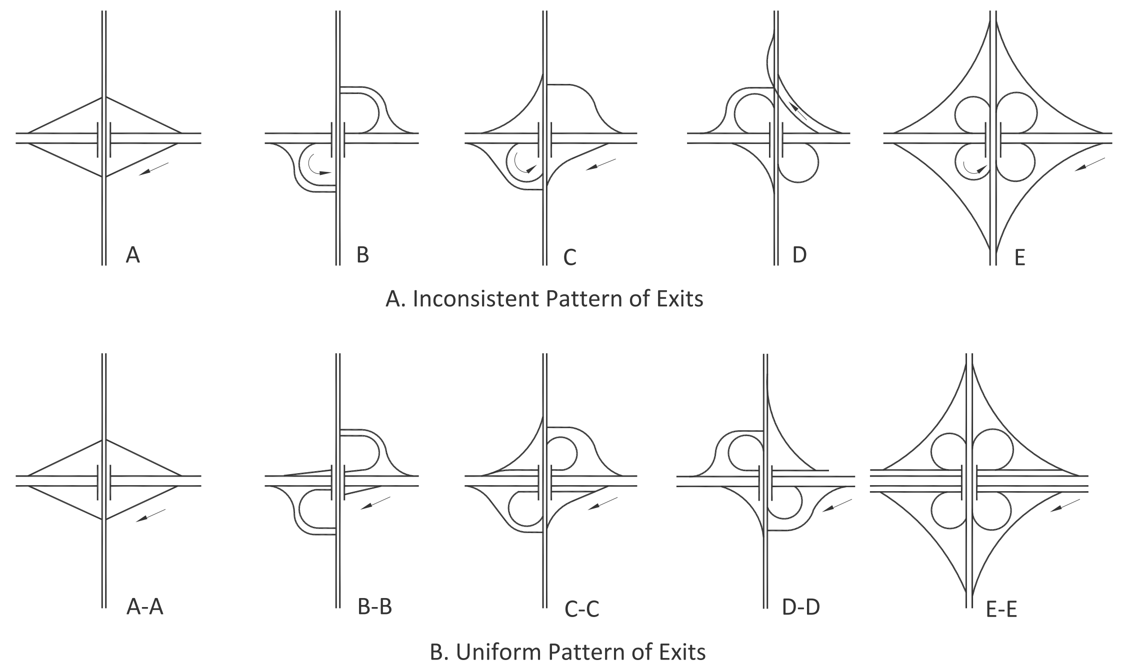
5.4.3.3 Provide Appropriate Interchange Spacing

Minimum interchange spacing is determined by weaving volumes, ability to provide signs, signal progression and required lengths of speed change lanes. Interchange spacing has a pronounced effect on primary roadway operation. In general, the minimum interchange spacing of an urban primary road and rural road should be 1.5 km and 3 km respectively.

However, in areas of concentrated urban development, minimum spacing standards are difficult to achieve due to the need for direct access to various facilities, such as shopping areas, residential built-up areas, leisure and entertainment hubs, etc. Thus, spacing of less than 1.5 km may be developed by grade-separated ramps or by adding collector-distributor roads.

5.4.3.4 Provide Uniform Interchange Patterns

As far as practical, all interchanges along a primary roadway should be uniform in terms of geometric layout and general appearance as shown in **Figure 5.19**. Because interchanges are closely spaced in urban areas, shorter distances are available to inform drivers of the course to be followed in leaving an expressway. An inconsistent arrangement of exit ramp locations with respect to the structure and some exits on right side of the carriageway causes driver confusion, resulting in drivers slowing down on high-speed lanes and making unexpected manoeuvres. Examples of inconsistent exit arrangements are illustrated in **Figure 5.19**.



(Source: *AASHTO's A Policy on Geometric Design of Highways and Streets, 2011* [2])

Figure 5.19: Arrangement of Exit between Successive Interchanges

5.4.3.5 Route Continuity

This concept is defined as providing a route on which changing lanes is not necessary to continue on the through route. The principle of route continuity simplifies the driving task in that it reduces lane changes, simplifies signing, delineates the through route, and reduces the driver's search for directional signing.

5.4.3.6 Develop Design that can be Simply Signed

The location of ramps and minimum distances between ramp junctions both depend largely on whether effective signing can be provided to inform, warn, and control drivers. Location and design of interchanges, individually and as a group, should be evaluated for proper signing.

5.4.3.7 Maintain Basic Number of Lanes

The basic number of lanes is defined as a minimum number of lanes designated and maintained over a significant length of a route, irrespective of changes in traffic volume and lane balance needs. Stating it another way, the basic number of lanes is a constant number of lanes assigned to a route, exclusive of auxiliary lanes. The basic number of lanes is predicated on the general volume level of traffic over a substantial length of the facility.

A change in the basic number of lanes is only needed when traffic volume significantly increase or decrease over a substantial length of the facility to justify an addition or reduction of lane. The key assessment of the basic number of lanes shall adhere to the Design Year LOS and guidelines presented in *Highway Capacity Manual HCM (HCM)* [6] on capacity analysis.

5.4.3.8 Principle of Lane Balance

The term “lane balance” refers to the arrangement of lanes at the points of exit and entrance.

Once the basic number of lanes is determined for each roadway, the balance in the number of lanes should be confirmed based on the following principles, also illustrated in **Figure 5.20**:

1. At entrances, the number of lanes beyond the merging point should not be less than the sum of all the merging lanes roadways minus one, but may be equal to the sum of merging lanes.
2. At exits, the number of approach lanes on the highway should be equal to the number of lanes on the highway beyond the exit, plus the number of lanes on the exit, minus one. Exceptions to this principle occur at cloverleaf loop-ramp exits that follow a loop-ramp entrance and at exits between closely spaced interchanges*. In such case, the auxiliary lane may be dropped in a single-lane exit such that the number of approaching lanes is equal to the number of all diverging lanes.
3. Only one traffic lane should be dropped at a time.

*Closely spaced interchanges are those where the distance between the end of the taper of the entrance terminal and the beginning of the taper of the exit terminal is less than 450 m, and a continuous auxiliary lane between the terminals is being used.

The principle of lane balancing sometimes conflict with the concept of continuity in basic number of lanes, as illustrated in **Figure 5.21**. In arrangement “A”, lane balance is maintained, but there is no compliance with the basic number of lanes. In arrangement “B”, continuity in the basic number of lanes is provided but the pattern does not conform to the principles of lane balance. Arrangement “C” is a pattern where the concepts of lane balance and basic number of lanes are brought into coordination by means of adding auxiliary lanes or removing them from carriageway. Auxiliary lanes may be added to satisfy capacity and weaving needs between interchanges, to accommodate traffic pattern variations at interchanges, and for simplification of operations. The principles of lane balance should be applied in the use of auxiliary lanes. In this manner, the appropriate balance between traffic load and capacity is provided, and lane balance and operational flexibility are realized.

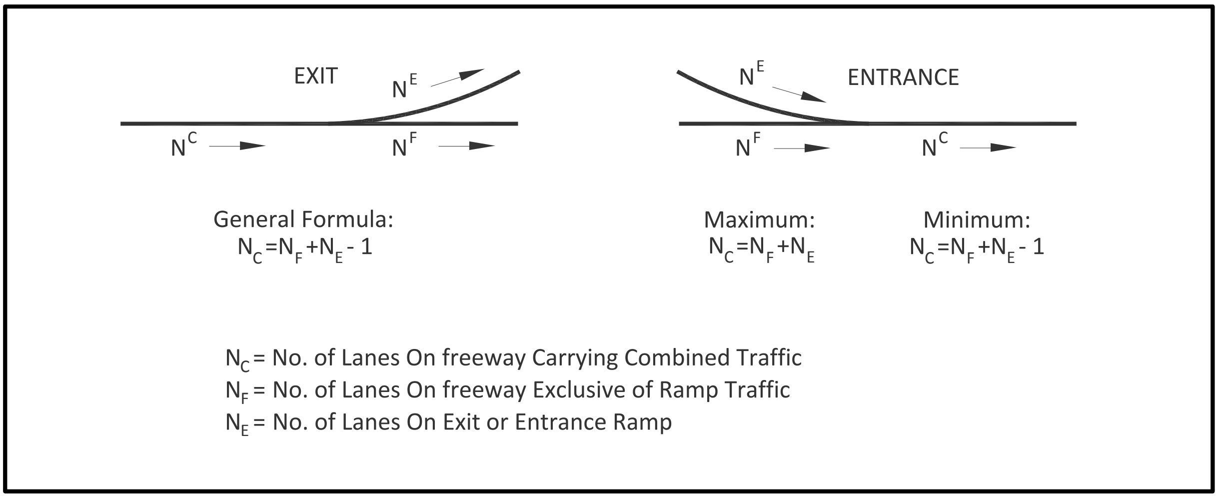
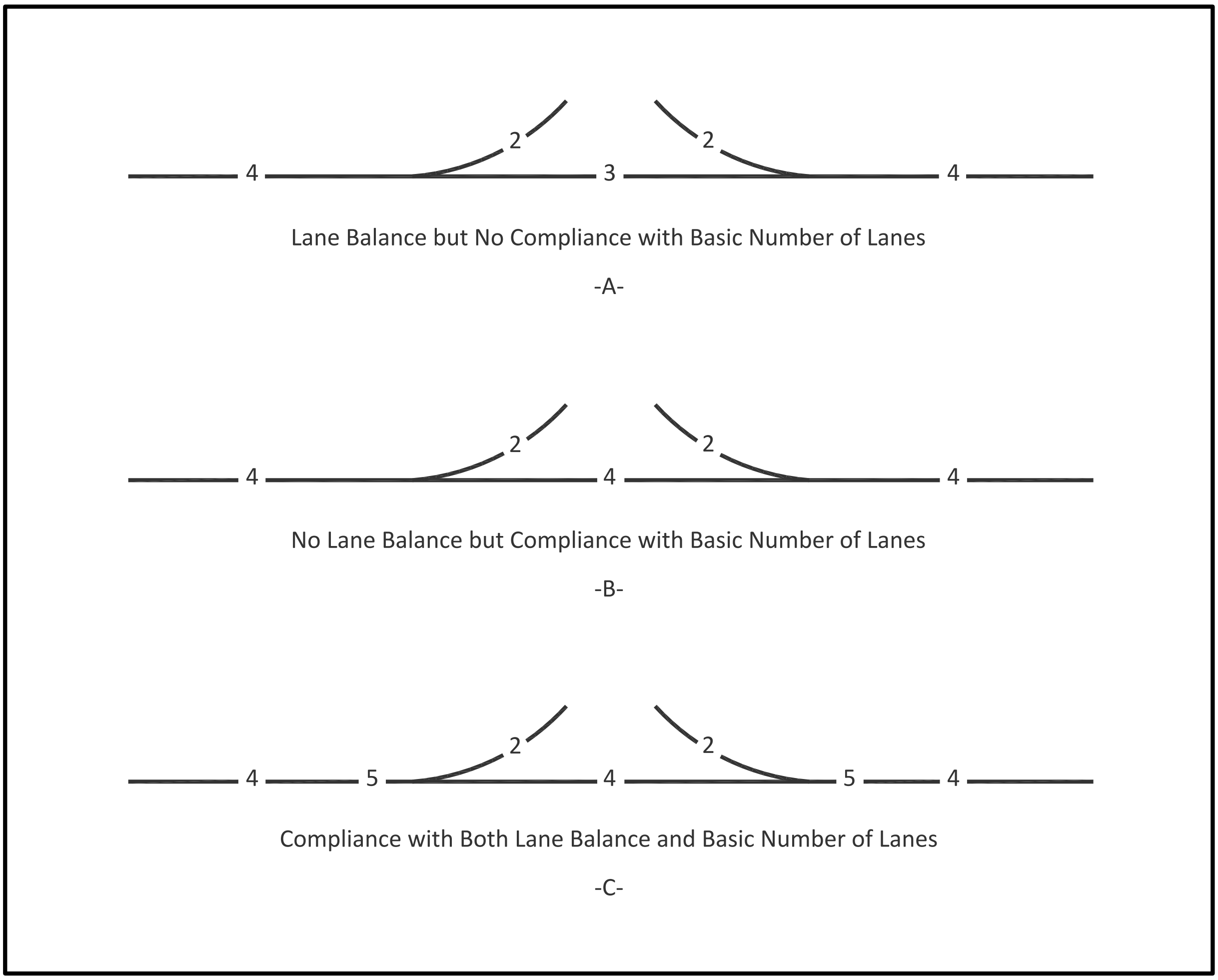


Figure 5.20: Principle of Lane Balancing



(Source: *AASHTO's A Policy on Geometric Design of Highways and Streets, 2011* [2])

Figure 5.21: Coordination of Lane Balancing with Basic Number of Lanes

5.4.3.9 Auxiliary Lanes

An auxiliary lane is defined as the portion of the roadway adjoining the carriageway for emergency stopping, speed change, turning, turning storage, weaving, truck climbing, and other purposes supplementary to through-traffic movement.

An auxiliary lane may be provided:

1. To comply with the concept of lane balance.
2. To comply with capacity needs.
3. To accommodate speed changing.
4. When interchanges are closely spaced.
5. To accommodate weaving.
6. To provide for manoeuvring of entering.
7. When the distance between the end of taper on the entrance terminal and the beginning of the taper on the exit terminal is short.

An auxiliary lane may be introduced as a single exclusive lane or in conjunction with a two-lane entrance. Auxiliary lanes may be dropped in a single or two-lane exit or carried to the physical gore nose before tapering into the through roadway. The width of auxiliary lane should be equal to through lane.

5.4.3.10 Lane Reduction

Basic number of lanes should not be reduced between and within interchanges simply to accommodate variations in traffic volumes. Instead, auxiliary lanes, as needed, should be added or removed from the basic number of lanes, as described in previous section on “Auxiliary Lanes.”

Basic number of lanes, if needed, should only be reduced beyond a principal interchange involving a major fork or at a point downstream from an interchange with another primary road, if the exit volume is large enough to justify lane reduction. If a basic lane is to be dropped between interchanges, it should be accomplished at a distance of 600 to 900 m from the previous interchange to allow for adequate signing.

The lane-drop transition should be located on tangent horizontal alignment and sag vertical curve or on the approach side of crest vertical curve to provide good visibility.

The lane reduction should be made on the left side of the road, using a desirable taper rate of 70:1 (50:1 minimum).

5.4.3.11 Avoid Designing Weaving Section

Weaving is the crossing of traffic flows. A weaving section is formed by an entrance ramp followed by a downstream exit ramp. Weaving sections can occur within or between interchanges, as shown in **Figure 5.22**. Weaving traffic produces the single most adverse influence on traffic operational quality and safety performance on high volume roads. Weaving sections reduce interchange capacity, thus, where feasible, should be eliminated from the main facility. Where cloverleaf interchanges are used, consideration should be given to the inclusion of collector-distributor roads on the main facility.

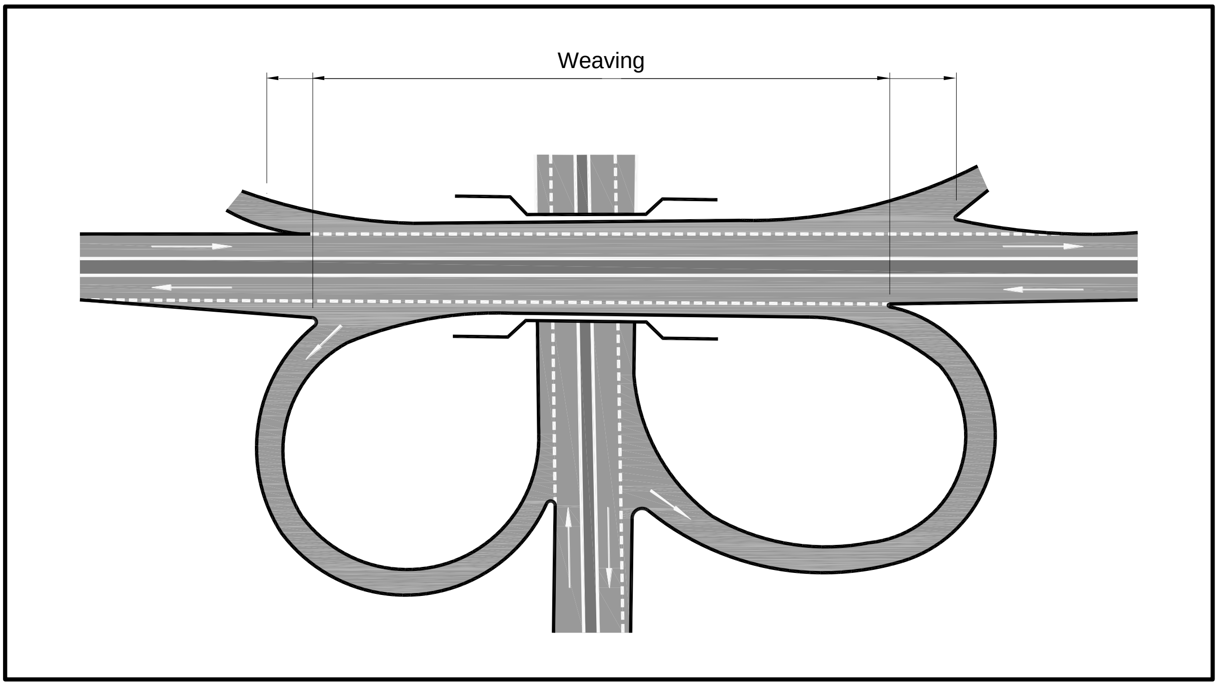


Figure 5.22: Weaving Section

The capacity of weaving sections may be seriously restricted unless the weaving section has adequate length, adequate width, and lane balance. Refer to the *Highway Capacity Manual (HCM)* [6] for capacity analysis of weaving sections.

5.4.3.12 Collector Distributor Roads

An example of where this occurs can be found in a full cloverleaf in an urban or suburban area. They should be analysed for suitability, but at the minimum, they may be one or two lanes width depending on capacity needs and lane balance should be maintained at entry/exits to the mainline. The design speed usually ranges from 60-80 km/h, but it should not be 20 km/h below the mainline design speed. Correct signage benefits the arrangement as far as traffic conflicts are concerned. Outer separations between mainline and Collector-Distributor roads should be as wide as practical, but minimum widths are tolerable. Widths should allow for shoulder widths comparable to the mainline to be used to enable barrier erection to discourage indiscriminate crossovers. Advantages include that weaving is transferred from the main roadway, single entrances/exits are developed, along with a uniform pattern of exits can be maintained.

5.4.3.13 Prefer Single Exit Designs

In general, single exit interchanges are superior to those with two exits, especially if one of the exits has long ramps. Whether used in conjunction with a full or partial cloverleaf, the single exit design may improve operational efficiency of the entire facility. Single exit design reduces the number of exit ramps along the mainline, facilitating appropriate ramp spacing. It also facilitates simpler and more readily understood signing by unfamiliar drivers. For special cases, typically associated with high volume system interchanges, the combined exiting traffic demand volumes for both directions of travel may require a three-lane exit, which can be difficult to design or prove to be impractical. In such cases, two exits may be the only reasonable solution. This condition should be understood and fully investigated during the interchange design process.

The full cloverleaf interchange, where a weaving section exceeds 1,000 vehicles per hour (vph), is an example where operational efficiency may be improved by the development of single exits and entrances. This can be achieved by inclusion of collector-distributor road.

The preference for single-exit design does not extend to single entrances. Interchanges may be designed with one or two entrances for each direction of travel.

The reasons for developing single exits where applicable are:

- Transfer the weaving from the main road to the slower road.
- Provide a high-speed exit from the main roadway for all exiting traffic.
- Simplify signing and the decision process.
- Satisfy driver expectancy, by having the exit in advance of the separation structure.
- Supplying uniformity of exit patterns.
- Provide decision sight distances for all traffic exiting from the main roadway.

5.4.3.14 Discourage Wrong way Entry

Wrong-way entry onto mainline should be given special consideration at all stages of interchange design in order to discourage wrong-way movements. Factors contributing are mainly interchange configuration and the exit ramp terminal features in relation to the main road.

These factors are outlined as follows:

- Partial interchanges are most commonly vulnerable to wrong way entry. Lack of provision of any one or more of the movements at an interchange can result in wrong way entry.
- Exit ramps that connect to two way frontage roads also encourage wrong way.
- One-way ramps that connect as an un-channelized T-junction may lead to incorrect entry.
- Unusual or odd arrangements of exit ramps are confusing and lead to wrong way entry.

5.5 RAMPS

A ramp is typically a one-way roadway connecting interchange legs and consists of a terminal at each leg and a connecting road. The geometry of the connecting road usually involves curvature and a grade.

5.5.1 Design Considerations

5.5.1.1 Design Speed of Ramps

On the intersecting roadways, ramp design speeds should be carefully chosen to match the low-volume running speed. However, in some cases, this is not practical and lower design speeds may be selected, but not less than the lower range presented in **Table 5.1**. Only design speeds of 80 km/h or higher apply to freeway and expressway exits.

Values in **Table 5.1** apply to the sharpest, or governing, ramp curve, usually on the ramp proper. These speeds do not apply to the ramp terminals, which should be properly transitioned and provided with speed-change facilities suitable for the highway speed. For corresponding minimum radius, refer to Chapter 3-GEOMETRIC DESIGN ELEMENTS of this manual.

Table 5.1: Guide Values for Ramp Design Speed as Related to Highway Design Speed

Highway Design Speed (Km/h)	50	60	70	80	90	100	110	120
Ramp Design Speed (Km/h)								
Upper Range (85%)	40	50	60	70	80	90	100	110
Middle Range (70%)	30	40	50	60	60	70	80	90
Lower Range (50%)	20	30	40	40	50	50	60	70

(Source: *AASHTO's A Policy on Geometric Design of Highways and Streets, 2011* [2])

5.5.1.2 Sight Distances

The minimum sight distance at any point along a ramp should be at least as great as the Design Stopping Sight Distance as shown in **Table 5.2**. Decision sight distance is desirable along the primary roadway prior to an exit nose. Sight distance for passing is not needed. The entire exit terminal, including the exit nose and a section of the ramp beyond the gore should be in clear view. The sight distance on a freeway preceding the approach nose of an exit ramp should exceed the minimum stopping sight distance, desirably by 25 percent or more, for the through traffic design speed.

5.5.1.3 Grades and Profile

The profile of the ramp consists of a central portion of appreciable grade, coupled with terminal vertical curves and connection to the profiles of intersecting legs. The profile at the terminal largely depends on the through road profile and should be followed accordingly. Central grades of Ramp should be as flat as practical. On one-way ramps, distinction should be made between upgrades and downgrades. As a general criterion, the gradient on upgrade ramp should not exceed the values shown in **Table 5.3**. Downgrades should be limited to 3% or 4% on ramps with sharp horizontal curvature and significant heavy truck or bus traffic.

The cases in which grades are the controlling factor in the length of the ramp are as follows:

1. For intersection angles of 70 degrees or less, it may be necessary to locate the ramp farther from the structure than necessary to provide sufficient length of ramp with reasonable grade;
2. Where the intersection legs are on considerable grade, with the upper road ascending and the lower road descending from the structure, the ramp will have to attain a large difference in elevation that increases with the distance from the structure;

Where a ramp leaves the lower road on a downgrade and meets the higher road on a downgrade, longer-than-usual vertical curves at the terminals may need a long ramp to meet grade limitations.

Table 5.2: Stopping Sight Distance for Ramps

Design Speed (Km/h)	Stopping Sight Distance (m)						
	Level	Downgrades			Upgrades		
		3%	6%	9%	3%	6%	9%
20	20	20	20	20	19	18	18
30	35	32	35	35	31	30	29
40	50	50	50	53	45	44	43
50	65	66	70	74	61	59	58
60	85	87	92	97	80	77	75
70	105	110	116	124	100	97	93
80	130	136	144	154	123	118	114
90	160	164	174	187	148	141	136
100	185	227	243	262	203	194	186
110	220	227	243	262	203	194	186

(Source: *AASHTO's A Policy on Geometric Design of Highways and Streets, 2011* [2])

Table 5.3: Allowable Maximum Gradient on Ascending Ramps

Ramp Design Speed (Km/h)	Maximum Ascending Gradient (%)
70 - 80	3 - 5
60 - 70	4 - 6
40 - 50	5 - 7
20 - 30	6 - 8

(Source: *AASHTO's A Policy on Geometric Design of Highways and Streets, 2011* [2])

5.5.1.4 Superelevation and Cross Slopes

Refer Chapter 3-GEOMETRIC DESIGN ELEMENTS of this manual, for superelevation rates and cross slopes of ramps.

The cross slope on portions of ramps on tangent should normally be sloped one way at a practical rate of 2 percent for high-type pavements.

Where feasible, the curve radius should be increased to reduce the required standard superelevation rate. The superelevation development is started or ended along the auxiliary lane of the ramp terminal. Both the edge of carriageway and the edge of shoulder should be examined at ramp junctions to assure a smooth transition.

Another important control in developing superelevation along the ramp terminal is that of the crossover crown line at the edge of the through-traffic lane. The maximum algebraic difference in cross slope between the auxiliary lane and the adjacent through lane is shown in **Table 5.4.**

The exit terminal, the ramp proper, and the entrance terminal should be studied in combination to ascertain the appropriate design speed and superelevation rates.

The methods of developing superelevation at free-flow ramp terminals are illustrated in **Figure 5.23**.

Table 5.4: Maximum Algebraic Difference in Cross Slope at Turning Roadway Terminals

Design Speed of Exit or Entrance Curve (Km/h)	Maximum Algebraic Difference in Cross Slope at Crossover Crown Line (%)
30 and under	5.0 to 8.0
40 and 50	5.0 to 6.0
60 and over	4.0 to 5.0

(Source: *AASHTO's A Policy on Geometric Design of Highways and Streets, 2011* [2])

5.5.2 Pavement Width

Ramp pavement widths are governed by the type of operation, curvature, and volume and type of traffic. Chapter 3-GEOMETRIC DESIGN ELEMENTS of this manual and Section 3.3.11 on "Widths for Turning Roadways at Intersections" of *AASHTO, 2011* [2] may be referenced for additional discussion on the treatments at the edge of the carriageway. Design widths of ramp pavements for various design traffic conditions are given in **Table 5.5**. The three general design traffic conditions are:

- **Traffic Condition A** - Predominantly Passenger vehicles, but some consideration for Single-Unit trucks, small volume of trucks or occasional large trucks.
- **Traffic Condition B** - Sufficient Single-Unit vehicles to govern design, but some consideration for semi-trailer vehicles, 5 to 10 percent of the total traffic.
- **Traffic Condition C -** Sufficient bus and semi-trailer combination of vehicles to govern design, mainly large semi-trailer trucks.

Traffic condition for the design of an interchange ramp should be taken for the class of road where the interchange is located. If the roads connected by the interchange are of different classes, the higher class should be used. For one-lane ramps of interchange, Case-II - One-Lane, One-Way Operation with provision for passing stalled vehicle should be selected as design case. Case-I values are recommended minimums and further reduction is not in order, even with a usable shoulder.

5.5.2.1 Shoulder Width and Lateral Clearance

Design values for shoulders and lateral offsets on the ramps are as follows:

1. For one-way operation, a paved shoulder width of 0.6 to 1.2 m (minimum 0.6 m) is desirable on the right and 2.4 to 3.0 m on the left.
2. For freeway ramp terminals where the ramp shoulder is narrower than the freeway shoulder, the paved shoulder width of the through lane should be carried into the entrance/exit terminal. The transition to the narrower ramp shoulder shall be accomplished gradually on the ramp end of the terminal with no abrupt changes.
3. Lateral clearance from the edge of the carriageway should be at least 1.8 m (preferably 3.0 m) on the left side, and at least 1.2 m on right side.
4. Where ramps pass under structures, the total roadway width should be carried through the structure. Structural supports should be located beyond the clear zone). The Chapter 2-CROSS SECTIONAL ELEMENTS of this manual and *AASHTO's Roadside Design Guide* [5] provides guidance on clear zone and the use of roadside barriers.
5. Full approach roadway width of ramp should be carried over the structure on overpasses.

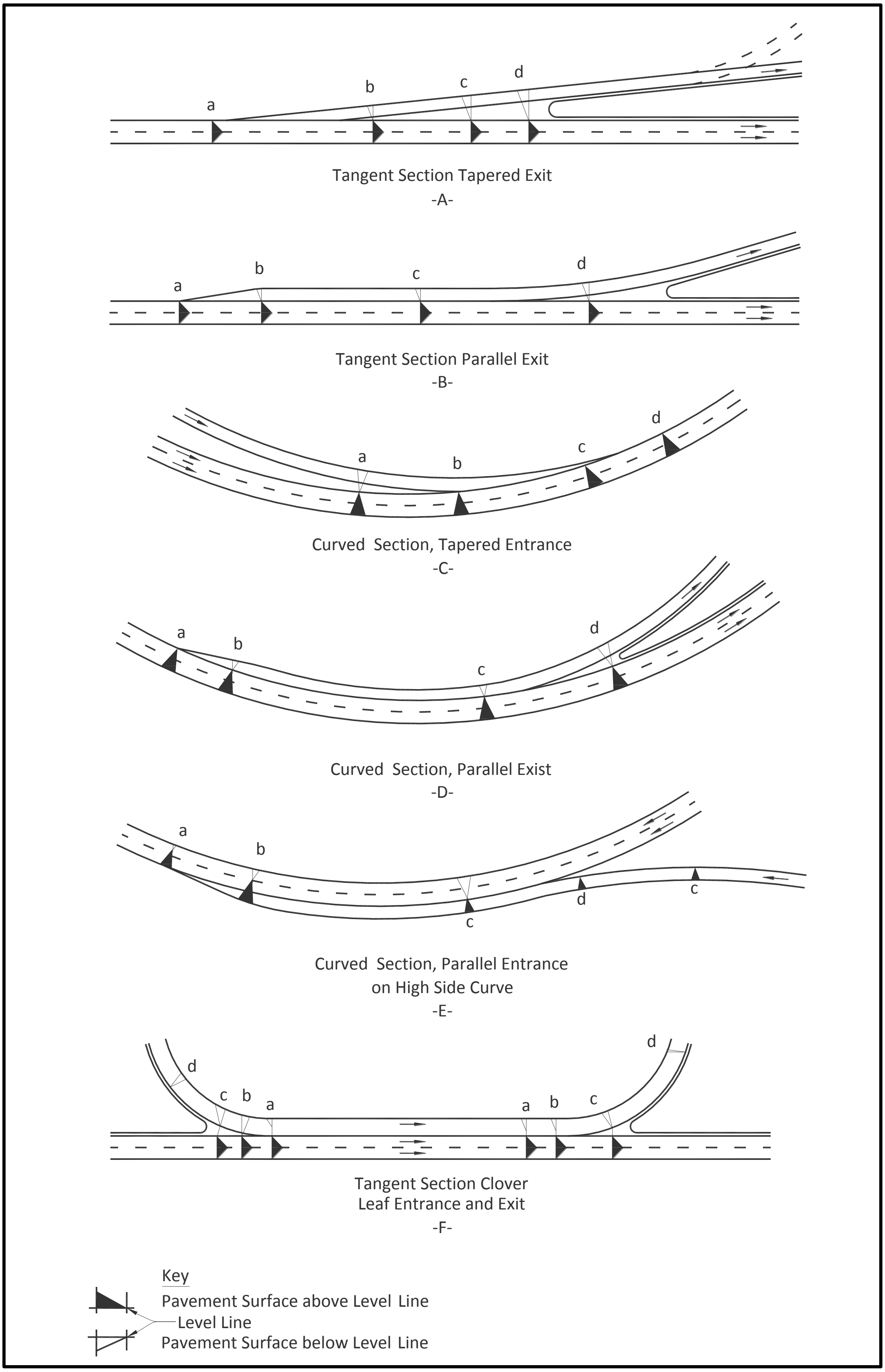


Figure 5.23: Development of Superelevation at Free-Flow Ramp Terminals

5.5.2.2 Kerbs and Barriers

Kerbs should not be provided on interchange ramps, except only to facilitate difficult drainage situations (such as in urban areas) where enclosed drainage is required due to restrictive right-of-way. Where kerbs are not used, full-depth paving should be provided on shoulders because of the frequent use of shoulders for turning movements. On low-speed facilities, kerbs may be placed at the edge of roadway. Vertical kerbs are seldom used in conjunction with shoulders, except where pedestrian protection is needed. The use of kerbs on intermediate or higher design speed facilities is not recommended except in special cases. If needed, mountable kerbs should be placed at the outer edge of shoulder on high-speed facilities. The provision of kerbs on interchanges in rural areas is strongly discouraged.

Table 5.5: Design Widths of Pavements for Turning Roadways

Radius on Inner Edge of Pavement, R (m)	Case-I			Case-II			Case-III		
	Design Traffic Condition								
	A	B	C	A	B	C	A	B	C
15	5.4	5.5	7.0	6.0	7.8	9.2	9.4	11.0	13.6
25	4.8	5.0	5.8	5.6	6.9	7.9	8.6	9.7	11.1
30	4.5	4.9	5.5	5.5	6.7	7.6	8.4	9.4	10.6
50	4.2	4.6	5.0	5.3	6.3	7.0	7.9	8.8	9.5
75	3.9	4.5	4.8	5.2	6.1	6.7	7.7	8.5	8.9
100	3.9	4.5	4.8	5.2	5.9	6.5	7.6	8.3	8.7
125	3.9	4.5	4.8	5.1	5.9	6.4	7.6	8.2	8.5
150	3.6	4.5	4.5	5.1	5.8	6.4	7.5	8.2	8.4
Tangent	3.6	4.2	4.2	5.0	5.5	6.1	7.3	7.9	7.9
Width Modification									
No Stabilized Shoulder	None			None			None		
Sloping Kerb	None			None			None		
Vertical Kerb									
One side	Add 0.3 m			None			Add 0.3 m		
Two sides	Add 0.6 m			Add 0.3 m			Add 0.6 m		
Stabilized shoulder, one or both sides	Lane width for conditions B & C on tangent may be reduced to 3.6 m where shoulder is 1.2 m or wider			Deduct shoulder width(s); minimum pavement width as under Case I			Deduct 0.6 m where Shoulder is 1.2 m or wider		

Notes:
Case-I - One-Lane, One-Way Operation with no provision for passing stalled vehicle
Case-II - One-Lane, One-Way Operation with provision for passing stalled vehicle
Case-III - Two-Lane Operation - either one-way or two-way
(Source: *AASHTO's A Policy on Geometric Design of Highways and Streets, 2011* [2])

5.6 RAMP TERMINALS

A portion adjacent to the through carriageway, including tapers, speed-change lanes, and islands, can be designated as a terminal of a ramp. There are three basic elements of ramp; the exit or point of diverge called Exit Terminal, the ramp proper, and the merge or ramp terminus called Entrance Terminal.

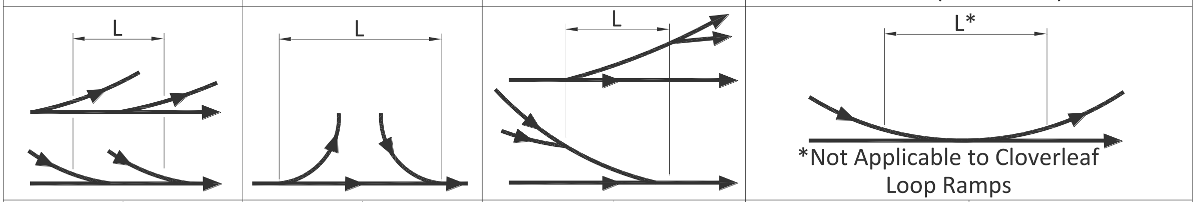
Further arrangement of terminals can be made either as a taper or parallel type, according to the configuration of the speed-change lane and single or multi-lane, according to the number of lanes on the ramp at the terminal.

5.6.1 Distance between Successive Ramp Terminals

Sufficient spacing between terminals is required for all drivers to change lanes readily and in a measured manner. It also produces even distribution of traffic density across the lanes, which in turn results in lesser speed differentials across the lanes. Spacing between successive outer ramp terminals is dependent on the classification of the interchanges involved, the combination of the ramp pairs (entrance or exit), and weaving potential.

The five possible combinations of ramp-pairs are:

1. An entrance followed by an entrance (EN-EN)
2. An exit followed by an exit (EX-EX)
3. An exit followed by an entrance (EX-EN)
4. An entrance followed by an exit (EN-EX) (weaving)
5. Turning roadways



EN - EN or EX - EX		EX - EN		TURNING ROADWAYS		EN - EX (WEAVING)			
						System to Service Interchange		Service to Service Interchange	
Full Freeway	CDR or FDR	Full Freeway	CDR or FDR	System Interchange	Service Interchange	Full Freeway	CDR or FDR	Full Freeway	CDR or FDR
Minimum Lengths Measured between Successive Ramp Terminals									
300 m	240 m	150 m	120 m	240 m	180 m	600 m	480 m	480 m	300 m

Notes: FDR-Freeway distributor road
CDR-Collector distributor road
EN-Entrance
EX-Exit

Figure 5.24: Recommended Minimum Ramp Terminal Spacing

5.6.2 Exit Terminal

The exit terminal includes the taper, deceleration lane and distance, gore, and physical nose. The taper and deceleration lane provide for physical separation of traffic leaving the through road, and enable the driver to decelerate in a manner away from the higher speed through traffic. The gore and physical nose are specific locations denoting where the geometry of the exit or diverge taper is controlled. **Figure 5.26** to **Figure 5.29** contains details for typical exit ramps.

5.6.2.1 Location of Exit Terminals

Exit terminal may be located at any point along a mainline alignment. In locating an exit ramp, care should be taken to avoid situations that may surprise a driver or create difficulties in developing ramp superelevation or profiles. For diverge geometry following a horizontal curve, the designer should locate the exit nose such that appropriate superelevation and transitions can be designed. In advance of the nose, the right edge of pavement will continue the mainline superelevation rate.

Another consideration in locating an exit is its visibility to the driver. Provision of decision sight distance is preferred. The exit terminal should not be near grade-separated structure. If it is not practical to place the exit terminal in advance of the structure, it should be placed on far side of the structure at a distance sufficient for the driver to see the exit and begin exit manoeuvre. For cases in which the crossroad is under the freeway and the freeway's vertical alignment is on an upgrade, it is desirable for the exit nose and diverge geometry to be located on the upgrade so that it is clearly visible to the driver.

5.6.2.2 Exit Gore Detail

The term "gore" indicates an area downstream from the shoulder intersection points as illustrated in **Figure 5.25**. The geometric layout of exit gore is an important part of exit terminal design. It is the decision point that should be clearly seen and understood by approaching drivers. In a series of interchanges along a freeway, the gores should be uniform and have the same appearance to drivers.

The minimum width at the physical nose, as measured between the carriageway of the mainline and that of the ramp, should equal the full left shoulder of the mainline, a nose width of 1.2 m, and the full right shoulder of the ramp.

Ahead of the nose, the alignment of the ramp, both horizontally and vertically, is tied to the mainline geometry. Once the exit ramp alignment reaches the physical nose, it becomes independent of the mainline. The ramp and gore should be designed to prevent ponding and to avoid abrupt changes in cross slope between the mainline, gore area, and ramp. The difference in cross slopes from one element to the next should not exceed the values mentioned in **Table 5.4**. Special care is when mainline alignment is in sag vertical curvature at or near the physical nose, or when the mainline geometry is on a horizontal curve to the right.

In setting the final design, it may be necessary to shift the location of the physical nose to enable reasonable ramp cross slopes and edge profiles. It also may be necessary to adjust the alignment of the ramp proper to enable its superelevation.

The entire triangular area, or neutral area, should be striped to delineate the proper paths on each side and to assist the driver in identifying the gore area. The *Manual of Uniform Traffic Control Devices (MUTCD)* [1]may be further referred for guidance on channelization.

The unpaved area beyond the gore nose should be graded nearly level with the roadways as practical so that vehicles accidentally entering will not be overturned or abruptly stopped by steep slopes. The graded gore area should be kept clear of any physical obstruction like heavy sign supports, luminaire supports, and roadway structure supports.

The details of recovery area at exit terminal are provided from **Figure 5.26** to **Figure 5.29.**

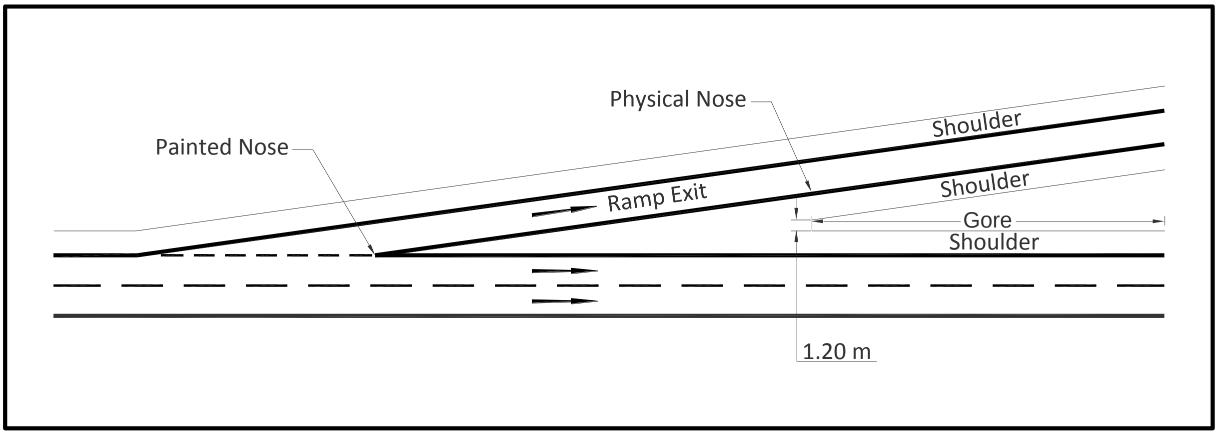


Figure 5.25: Typical Exit Gore Detail

5.6.2.3 Design for Deceleration

Exit ramps can be designed as either taper-type designs or parallel exit designs. Taper-type exits are preferred, because as drivers decelerate, they also move laterally away from the higher speed mainline traffic. Taper type exits should be used on rural freeway interchanges and where right-of-way is limited. Parallel-type exits may be used where right-of-way is available and their use facilitates ramp proper alignment. However, either type of design is acceptable.

A parallel-type exit terminal usually begins with a taper, followed by an added lane that is parallel to the carriageway. The taper portion of a parallel-type deceleration lane should have a taper of 75 m. In parallel-type exit, the drivers exit the through lane sufficiently in advance of the exit nose and decelerate on the added lane (deceleration lane). Typical parallel-type exit terminals are shown in **Figure 5.26** and **Figure 5.27** for single-lane and multi-lane exits, respectively.

Table 5.6: Minimum Length of Taper beyond an Offset Nose

Design Speed of Approach Highway (Km/h)	Taper Rate, Z:1 (m)
50	15:1
60	20:1
70	22.5:1
80	25.0:1
90	27.5:1
100	30.0:1
110	35.0:1
120	40.0:1

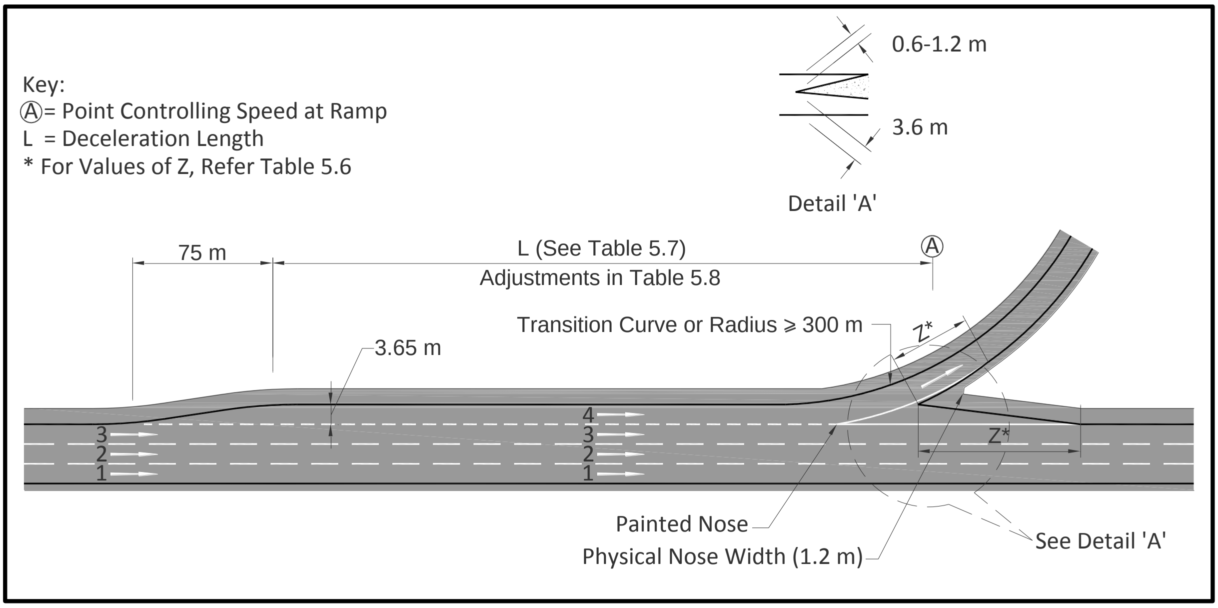


Figure 5.26: One Lane Exit Ramp-Parallel Design

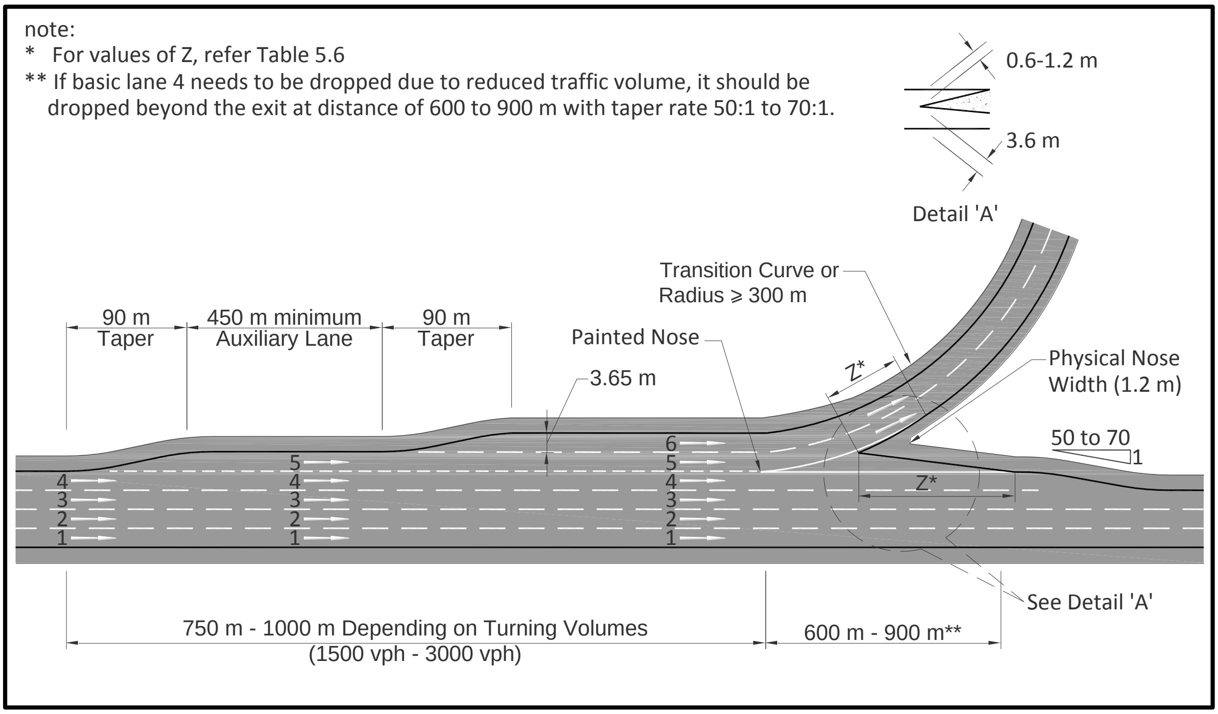


Figure 5.27: Two Lane Exit Ramp-Parallel Design

The taper-type exit fits the direct path preferred by most drivers, permitting them to follow an easy path within the diverging area. The divergence angle of 2 to 5 degrees should be used. Vehicles decelerate after clearing the through-traffic lane and before reaching the point limiting design speed for the ramp proper. **Figure 5.28** and **Figure 5.29** show typical designs for single-lane and multi-lane taper-type exits, respectively.

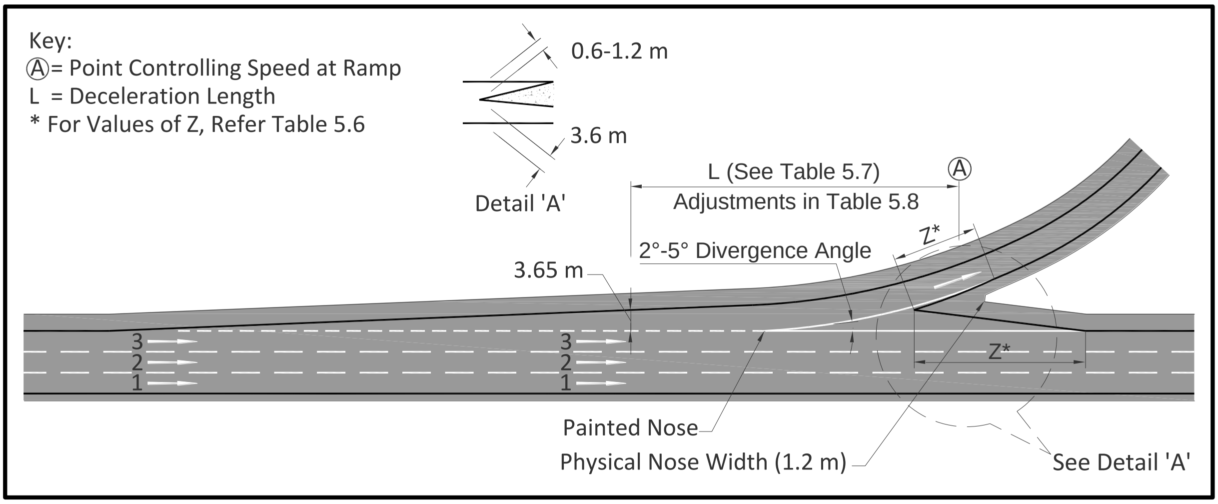


Figure 5.28: One Lane Exit Ramp-Taper Design

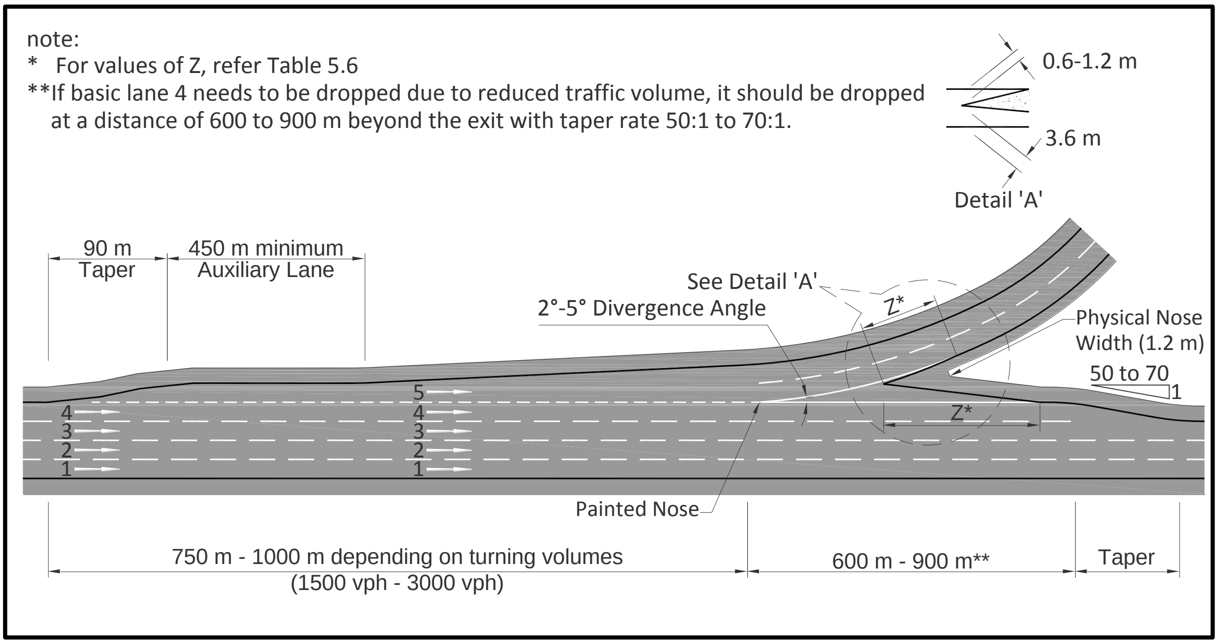


Figure 5.29: Two Lane Exit Ramp-Taper Design

In cases where the basic number of lanes is to be reduced beyond a two-lane exit, it should be accomplished at a distance of 600 to 900 m from the previous interchange to allow for adequate signing. The minimum taper rate for lane drop should be 50:1, and the desirable taper rate is 70:1.

The critical design feature of the exit regardless of type is the length of deceleration lane provided to drivers. Minimum deceleration lengths for various combinations of design speeds for the highway and for the ramp roadway are given in **Table 5.7**. Grade adjustments are given in **Table 5.8**.

Research confirms that crash frequencies on exit ramps are related to the length of deceleration provided. Therefore, designers are encouraged to lengthen the deceleration dimension in cases where exiting design volumes are high or the costs of doing so are minimal.

Table 5.7: Minimum Deceleration Lengths for Exit Terminals with Flat Grades of Two Percent or Less

Deceleration Length, L (m) for Design Speed of Exit Curve, V' (km/h)									
Highway Design Speed, V (Km/h)	Speed Reached, V_a (Km/h)	Stop Condition	20	30	40	50	60	70	80
		For Average Running Speed on Exit Curve V'_a (km/h)							
		0	20	28	35	42	51	63	70
50	47	75	70	60	45	-	-	-	-
60	55	95	90	80	65	55	-	-	-
70	63	110	105	95	85	70	55	-	-
80	70	130	125	115	100	90	80	55	-
90	77	145	140	135	120	110	100	75	60
100	85	170	165	155	145	135	120	100	85
110	91	180	180	170	160	150	140	120	105
120	98	200	195	185	175	170	155	140	120

(Source: *AASHTO's A Policy on Geometric Design of Highways and Streets, 2011* [2])

Table 5.8: Deceleration Lane Adjustment Factors as a Function of Grade

Design Speed of Highway (km/h)	Ratio of Length on Grade to Length on Level for Design Speed of Turning Curve (km/h) [a]	
All Speeds	3 to 4% Upgrade 0.9	3 to 4% Downgrade 1.2
All Speeds	5 to 6% Upgrade 0.8	5 to 6% Downgrade 1.35

Notes:
1. Ratio from this table multiplied by the length in **Table 5.7** gives length of speed change lane on grade.
(Source: *AASHTO's A Policy on Geometric Design of Highways and Streets, 2011* [2])

5.6.2.4 Exit Terminals on Curve

Where curves on a freeway are relatively sharp and there are exits located on these curves, some adjustments in design may be desirable to avoid operational difficulties.

On freeways having design speeds of 100 km/h or more, the curves are sufficiently gentle so that either the parallel type or the taper type of speed-change lane can be provided. With the parallel type, the added lane should be on the same curvature as the mainline. With the taper type, the dimensions applicable to terminals located on tangent alignment are also suitable for use on curves. On curved sections, the ramp at exit terminal is tapered at the same rate relative to the through-traffic lanes as on tangent sections as illustrated in **Figure 5.30**.

On freeways having design speed of 80 km/h, where the curves are relatively sharp, the parallel type of speed-change lane has an advantage over the taper type. At exits, the parallel type is less likely to confuse through traffic. Parallel-type speed-change lanes at exit terminals on curves are illustrated in **Figure 5.31**.

Exits on right-turning curves (**Figure 5.31C**) should be avoided, if practical. The design should provide a definite break in the left edge of the carriageway to provide a visual cue to the through driver to avoid being unintentionally led off the through roadway. To make the deceleration lane more apparent to approaching motorists, the taper should preferably be no more than 30 m in length. The deceleration lane should begin either upstream or downstream from the PC. It should not begin right at the PC, as the deceleration lane appears to be an extension of the tangent, and motorists are more likely to be confused. The ramp proper should begin with a section of tangent or a long-radius curve to permit a long and gradual reversing of the superelevation.

An alternate design, which will usually avoid operational concerns, is to locate the exit terminal a considerable distance upstream from the PC. In this design, a separate and parallel ramp roadway is provided to connect with the ramp proper.

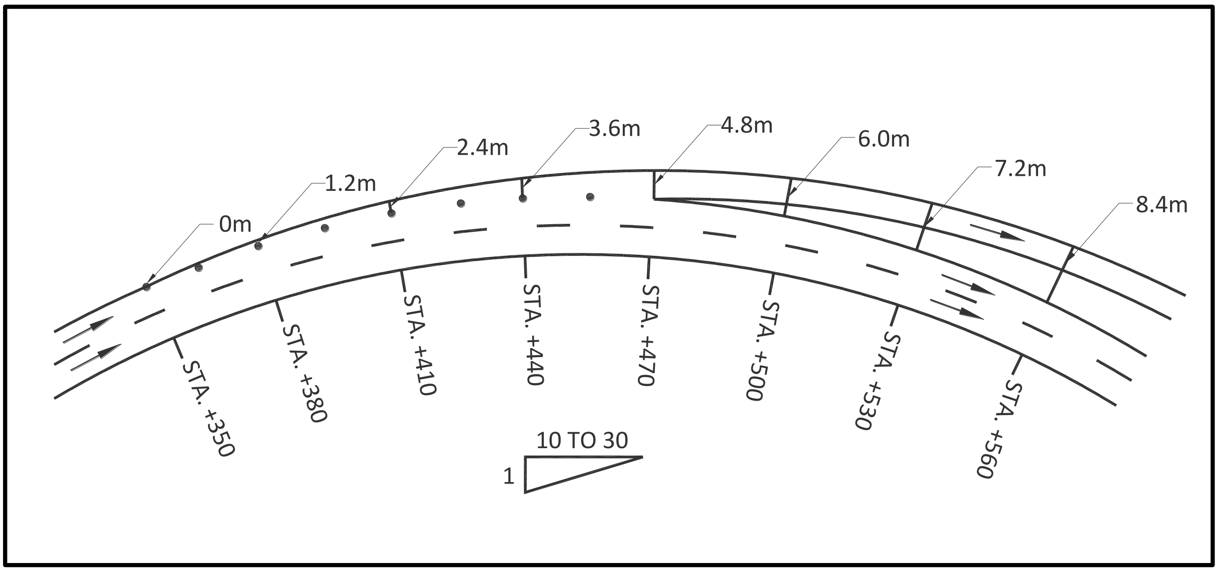


Figure 5.30: Layout of Taper Type Exit Terminal on Curve

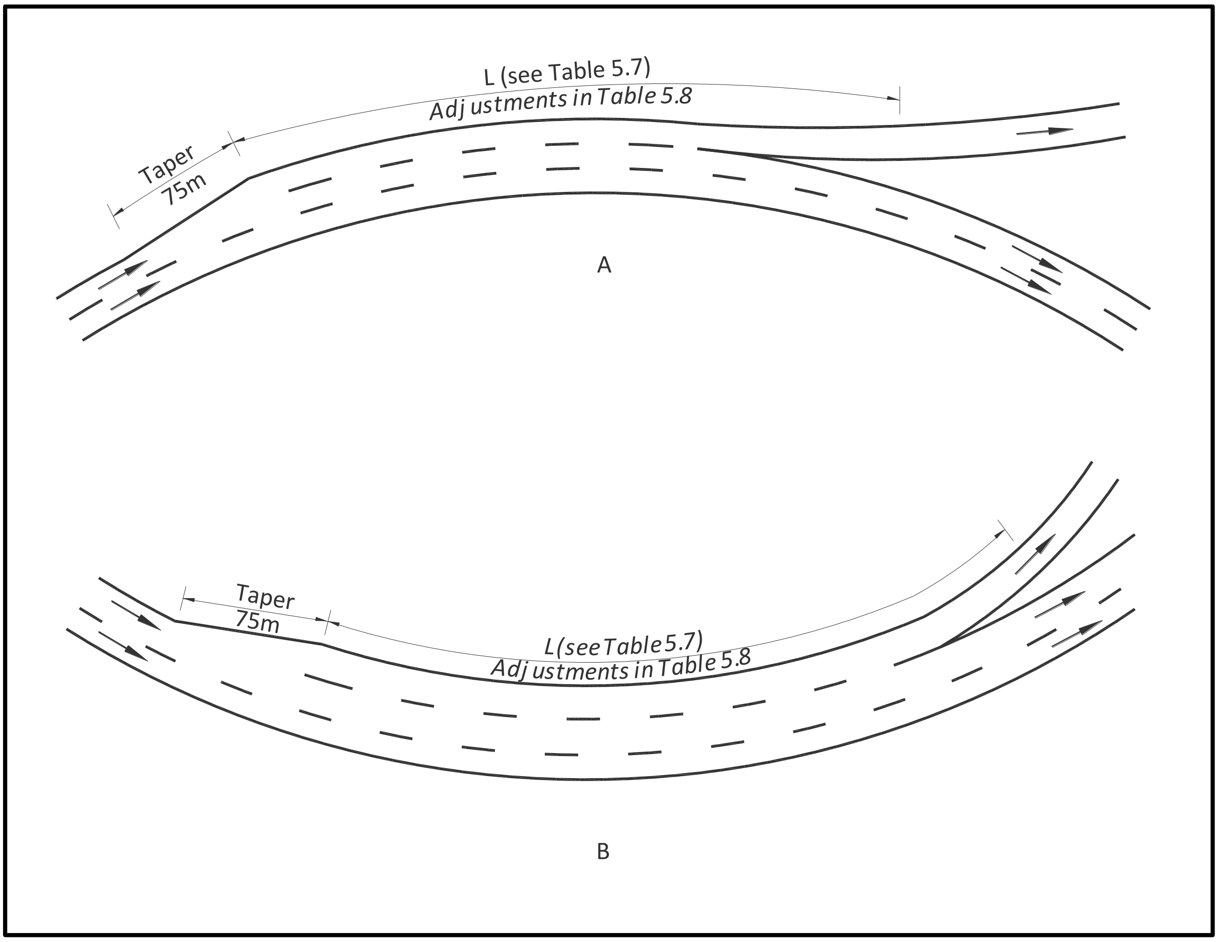


Figure 5.31: Layout of Parallel Type Exit Terminal on Curve

5.6.3 Entrance Terminal

The entrance terminal includes the merging end, known also as "physical nose", entrance gore, acceleration distance and lane, and taper. The acceleration lane and taper provide for physical separation of traffic entering the through road, and enable the driver to accelerate safely away from the higher speed through traffic. The gore and merging end are the areas where the geometry of the ramp proper is not independent, but rather controlled in three dimensions by the geometry of the mainline.

Figure 5.33 to **Figure 5.36** contains greater design details for entrance ramps.

5.6.3.1 Location of Entrance Terminals

Entrance terminal may be located at any point along a mainline alignment. Separate entrances in many cases are preferred operationally as they distribute the traffic volume entering the mainline. However, the location of the second entrance will be influenced by both the upstream entrance and the next downstream exit, the combination of which creates a potential weaving section. In locating multiple separate entrance ramps, designers should study and test the local system of ramps using the terminal spacing criteria in **Figure 5.24**, which addresses both successive entrances and weaving dimensions.

In locating an entrance ramp, care should be taken to avoid situations that may create difficulties in development of ramp superelevation or profiles. Entrance ramps profile should be controlled by the vertical alignment of the mainline to the extent possible. Downgrade alignment facilitates the acceleration of merging vehicles to the mainline speed. For cases in which the merging end must be designed along an upgrade because of ramp spacing, bridge, or right-of-way concerns, the acceleration lane should be lengthened to counteract the adverse effects of the upgrade on acceleration.

5.6.3.2 Entrance Gore Detail

Although the term "gore" generally refers to the area between a through roadway and an exit ramp, the term may also be used to refer to the similar area between a through roadway and an entrance ramp. At an entrance terminal, the point of convergence (beginning of all paved area) is defined as the "merging end." At this point, the alignment of ramp proper is no longer independent and is tied directly to the mainline geometry. The entrance terminal is less of a decision area because it points downstream and separates traffic streams already in lanes.

The separation between the ramp and mainline at the ramp nose should include the left shoulder of the ramp, physical nose (width of 0.6 m to 1.2 m), and right shoulder of the mainline.

Appropriate vertical geometry and edge elevations of ramp are critical aspects of the design of the ramp as it merges to the mainline. The ramp and gore should be designed to prevent ponding and to avoid abrupt changes in slope between the mainline, gore, and ramp. The difference in cross slopes from one element to the next should not exceed the values mentioned in **Table 5.4.** Special care is needed for cases in which the mainline alignment is in sag vertical curvature at the merging end, or when the mainline geometry is on a horizontal curve to the right.

In setting the final design, it may be necessary to shift the location of the merging end to enable reasonable cross slopes and edge profiles of ramp. It also may be necessary to adjust the alignment of the ramp proper to enable its superelevation.

Figure 5.32 shows an entrance ramp, where a reduction in the ramp lane width is required to maintain a single-lane entrance.

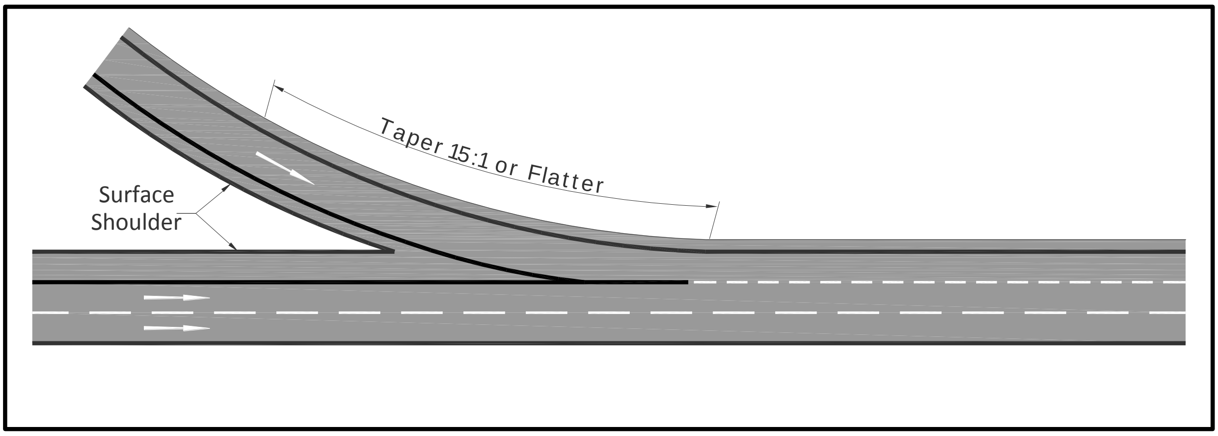


Figure 5.32: Carriageway Narrowing On Entrance Ramp

5.6.3.3 Design for Acceleration

Entrance ramps can be designed as taper-type or as parallel type. The design basis for merging is the assumption that each merging vehicle operates independently and has sufficient gaps in lane one traffic to enable merging. Both taper-type and parallel-type entrances are considered acceptable for any freeway or expressway interchange.

The parallel-type entrance provides an added lane of sufficient length to enable a vehicle to accelerate to near-main speed prior to merging. A 90 m taper is provided at the end of the added lane. Parallel-type entrances are encouraged on entrance ramps of higher volume expressway. Typical designs of parallel-type entrance are shown in **Figure 5.33** and **Figure 5.34** for single-lane and multi-lane entrances, respectively.

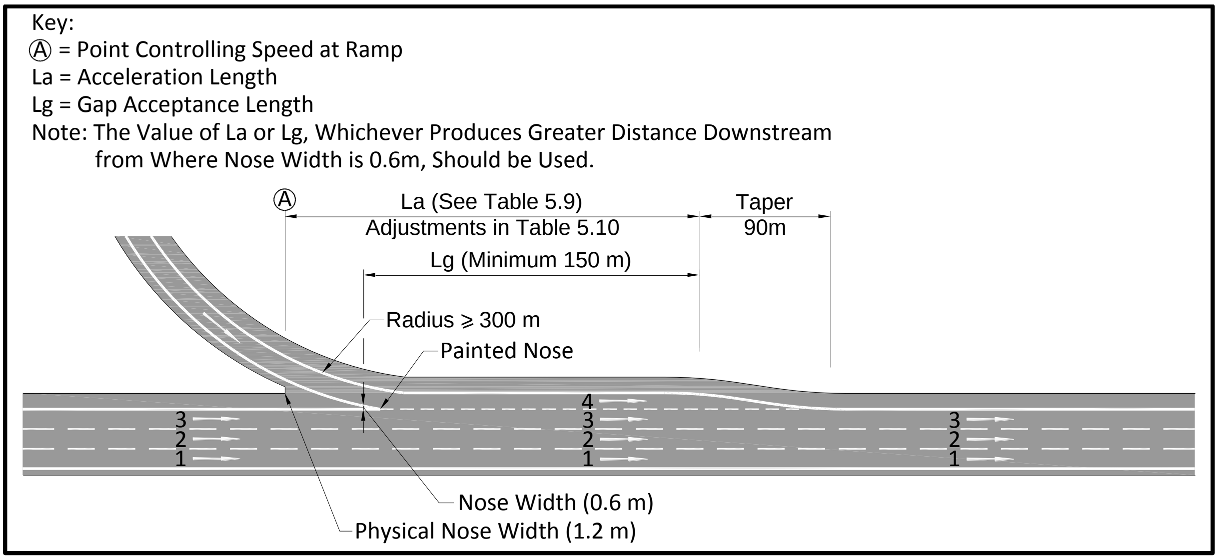


Figure 5.33: One Lane Entrance Ramp-Parallel Design

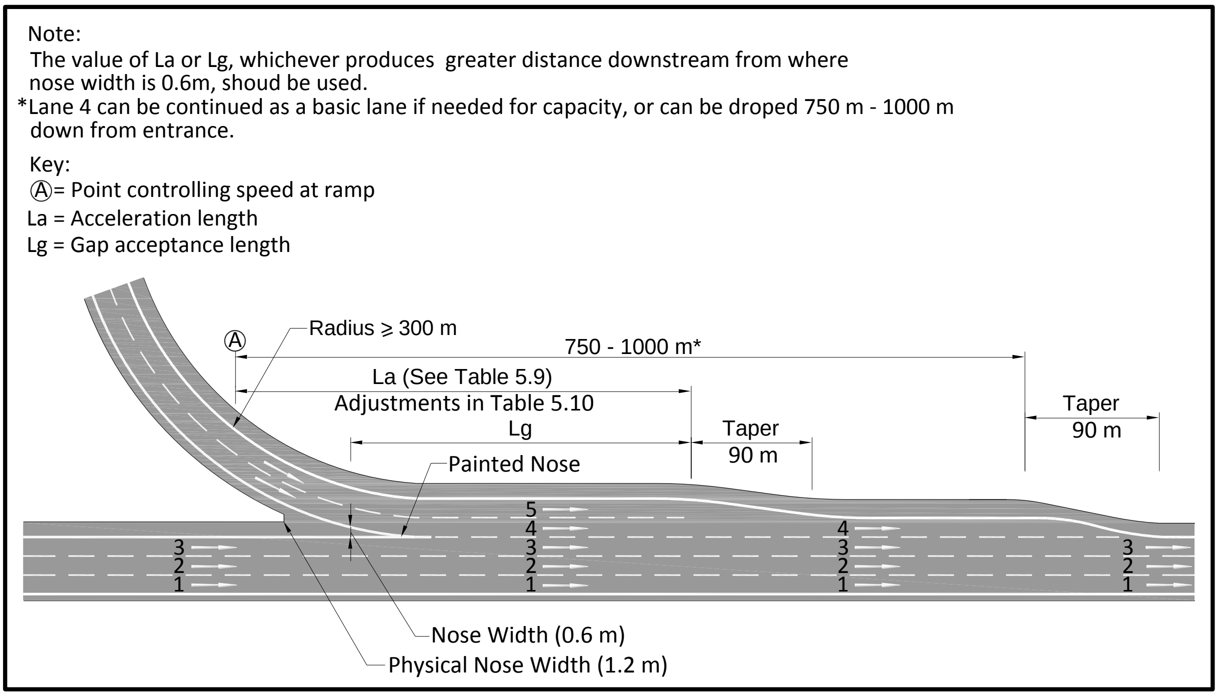


Figure 5.34: Two Lane Entrance Ramp-Parallel Design

When properly designed, the taper-type entrance operates smoothly at all volumes up to and including the design capacity of merging areas. The entrance is merged into the mainline with a long, uniform taper. Desirable rate of taper should be approximately 50:1 to 70:1 (longitudinal to lateral) between the outer edge of the acceleration lane and the edge of the through-traffic lane. The gap acceptance length (L_g) is also a consideration in the design of taper-type entrances. **Figure 5.35** and **Figure 5.36** show typical designs for single-lane and multi-lane taper-type entrances.

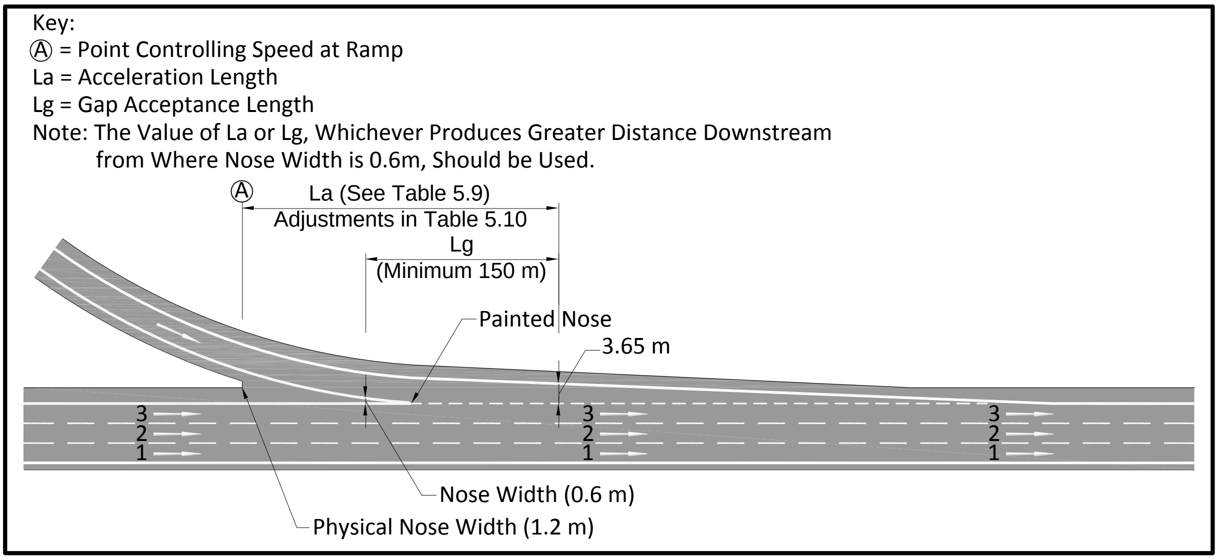


Figure 5.35: One Lane Entrance Ramp-Taper Design

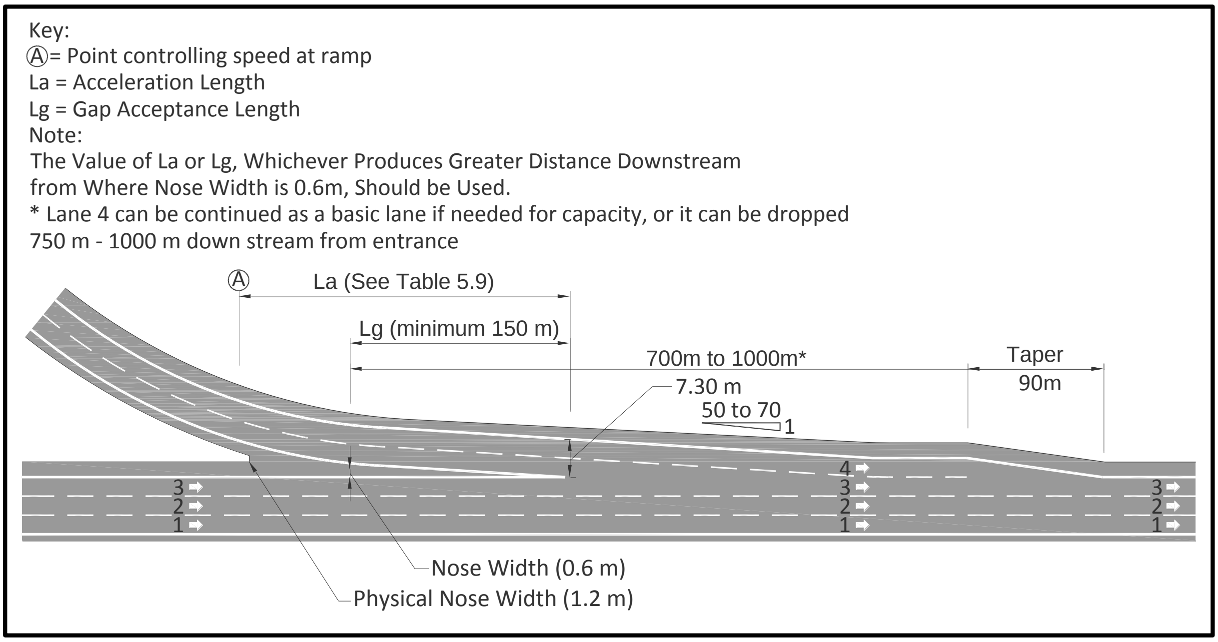


Figure 5.36: Two Lane Entrance Ramp-Taper Design

Research on the safety performance of interchange elements confirms that crash frequencies on entrance ramps are related to the length of acceleration provided. Designers are encouraged to lengthen the acceleration dimension in cases where entering design volumes are higher or the costs of doing so are minimal. Minimum acceleration lengths for various combinations of design speeds for the highway and for the ramp roadway are given in **Table 5.9**. Grade adjustments are given in **Table 5.10**. Where lengths of acceleration lanes exceed 400 m, taper type design is recommended.

Table 5.9: Minimum Acceleration Lengths for Entrance Terminals with Flat Grades of Two Percent or Less

Acceleration Length, L (m) for Design Speed of Entrance Curve, V' (km/h)									
Highway Design Speed, V (Km/h)	Speed Reached, V_a (Km/h)	Stop Condition	20	30	40	50	60	70	80
		Initial Speed V'_a (km/h)							
		0	20	28	35	42	51	63	70
50	37	60	50	30	-	-	-	-	-
60	45	95	80	65	45	-	-	-	-
70	53	150	130	110	90	65	-	-	-
80	60	200	180	165	145	115	65	-	-
90	67	260	245	225	205	175	125	35	-
100	74	345	325	305	285	255	205	110	40
110	81	430	410	390	370	340	290	200	125
120	88	545	530	515	490	460	410	325	245

(Source: *AASHTO's A Policy on Geometric Design of Highways and Streets, 2011* [2])

Table 5.10: Acceleration Lane Adjustment Factors as a Function of Grade

Design Speed of Highway (km/h)	Ratio of Length on Grade to Length of Level for Design Speed of Turning Curve (km/h) [a]					
	40	50	60	70	80	All Speeds
	3 to 4% Upgrade					3 to 4% Downgrade
60	1.3	1.4	1.4	-	-	0.7
70	1.3	1.4	1.4	1.5	-	0.65
80	1.4	1.5	1.5	1.5	1.6	0.65
90	1.4	1.5	1.5	1.5	1.6	0.6
100	1.5	1.6	1.7	1.7	1.8	0.6
110	1.5	1.6	1.7	1.7	1.8	0.6
120	1.5	1.6	1.7	1.7	1.8	0.6
	5 to 6% Upgrade					5 to 6% Downgrade
60	1.5	1.5	-	-	-	0.6
70	1.5	1.6	1.7	-	-	0.6
80	1.5	1.7	1.9	1.8	-	0.55
90	1.6	1.8	2.0	2.1	2.2	0.55
100	1.7	1.9	2.2	2.4	2.5	0.5
110	2.0	2.2	2.6	2.8	3.0	0.5
120	2.3	2.5	3.0	3.2	3.5	0.5

Notes:
1. Ratio from this table multiplied by the length in **Table 5.9** gives length of speed change lane on grade.
(Source: *AASHTO's A Policy on Geometric Design of Highways and Streets, 2011* [2])

5.6.3.4 Entrance Terminals on Curve

Where curves on a freeway are relatively sharp and there are entrances located on these curves, some adjustments in design may be desirable to avoid operational difficulties.

The curves are adequately mild so that either the parallel type or the taper type of speed-change lane can be provided on freeways having design speeds of 100 km/h or more. The added lane should be having the identical curvature as the mainline while using the parallel type. With the taper type, the dimensions applicable to terminals located on tangent alignment are also suitable for use on curves. As demonstrated in **Figure 5.37,** the ramp at exit terminal on curved sections is tapered at the same rate relative to the through-traffic lanes as on tangent sections.

On freeways having design speed of 80 km/h, where the curves are relatively sharp, the parallel type of speed-change lane has an advantage over the taper type. At entrance, the parallel type usually results in smoother merging operations. Parallel-type speed-change lanes at entrance terminals on curves are illustrated in **Figure 5.38**.

Entrances on curved sections of highway generally operate better than exits. It is important that the approach curve on the ramp have a very long radius as it joins the acceleration lane. This aligns the entering vehicle with the acceleration lane and lessens the chances of motorists entering directly onto the through lanes. The taper at the end of the acceleration lane should preferably be about 90 m min. length. When reverse-curve alignment occurs between the ramp and speed-change lane, an intervening tangent should be used to aid in superelevation transition.

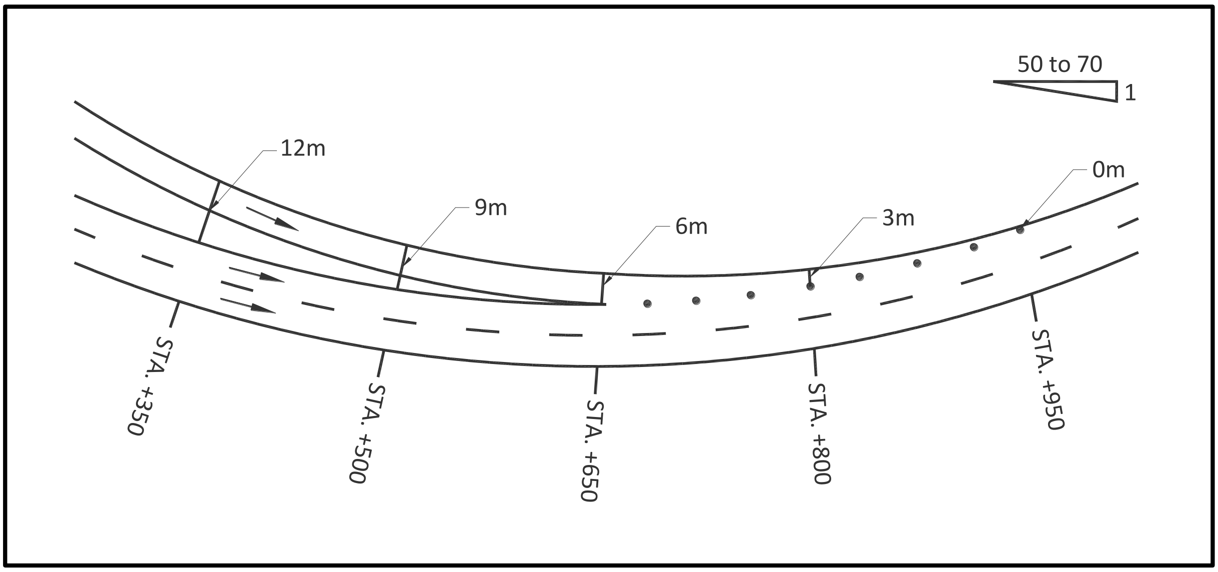


Figure 5.37: Layout of Taper Type Entrance Terminal on Curve

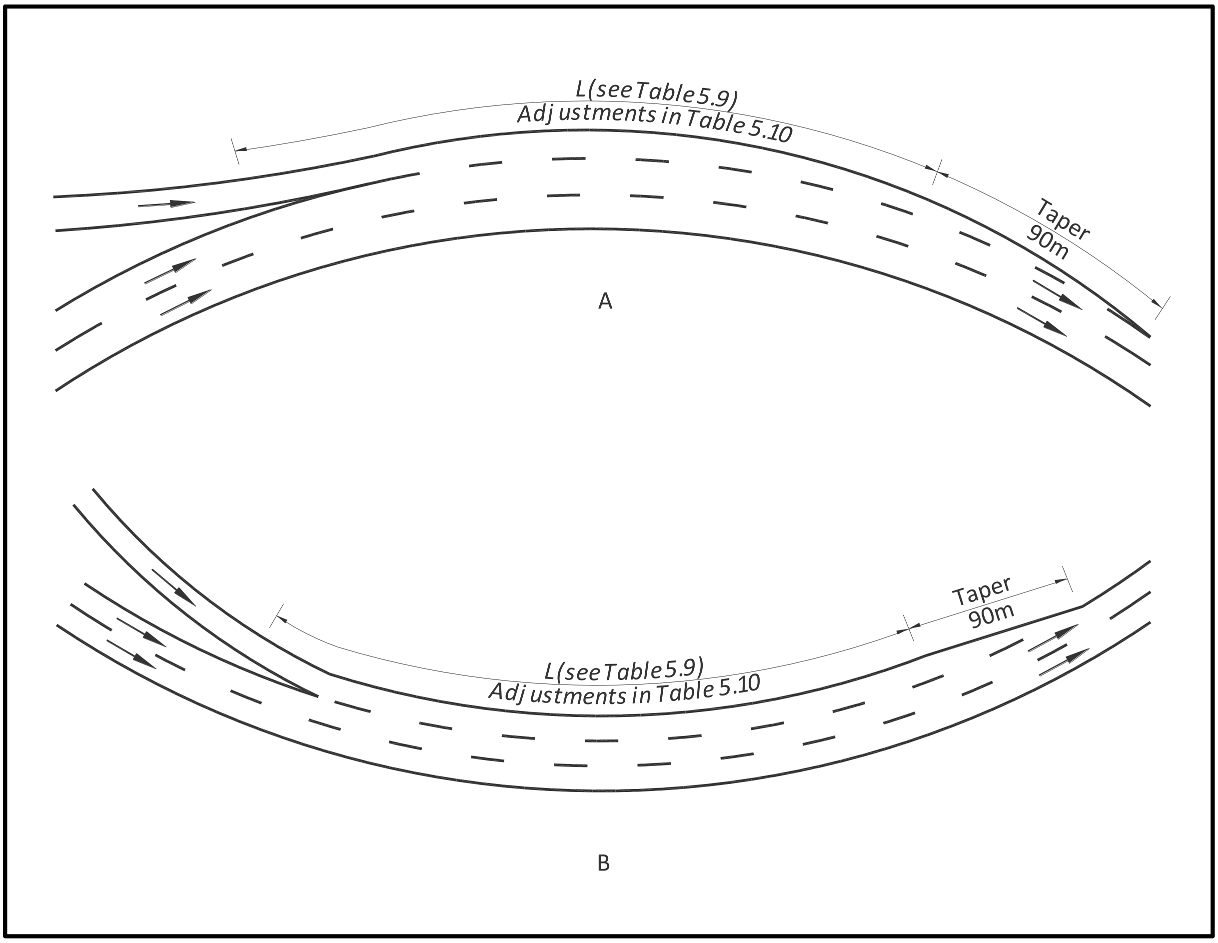


Figure 5.38: Layout of Parallel Type Entrance Terminal on Curve

5.6.4 Major Fork and Branch Connection

Where two roadways are equivalent in importance, their design should reflect such equivalence when they diverge or merge.

A major fork is the splitting of a roadway into two equivalent and separate roadways (**Figure 5.39**). As discussed in “Principle of Lane Balance” of Section 5.4.3.8 of this manual, the design of major forks is subject to the same principles of lane balance as any other diverging area. Major forks should be located on a tangent or very mild curve. The geometry should support operating speeds no less than 20 km/h below the design speed of the pre-split roadway. Decision sight distance should be provided in advance of major forks.

A branch link is defined by:

1. By the convergence of two directional multi-lane ramps from another freeway, the beginning of a directional roadway of a freeway is formed or
2. A single freeway route is formed by the converging of two freeway routes.

Figure 5.40 shows the merging of two major roadways. The same design principles apply with respect to the alignment at the point of merge.

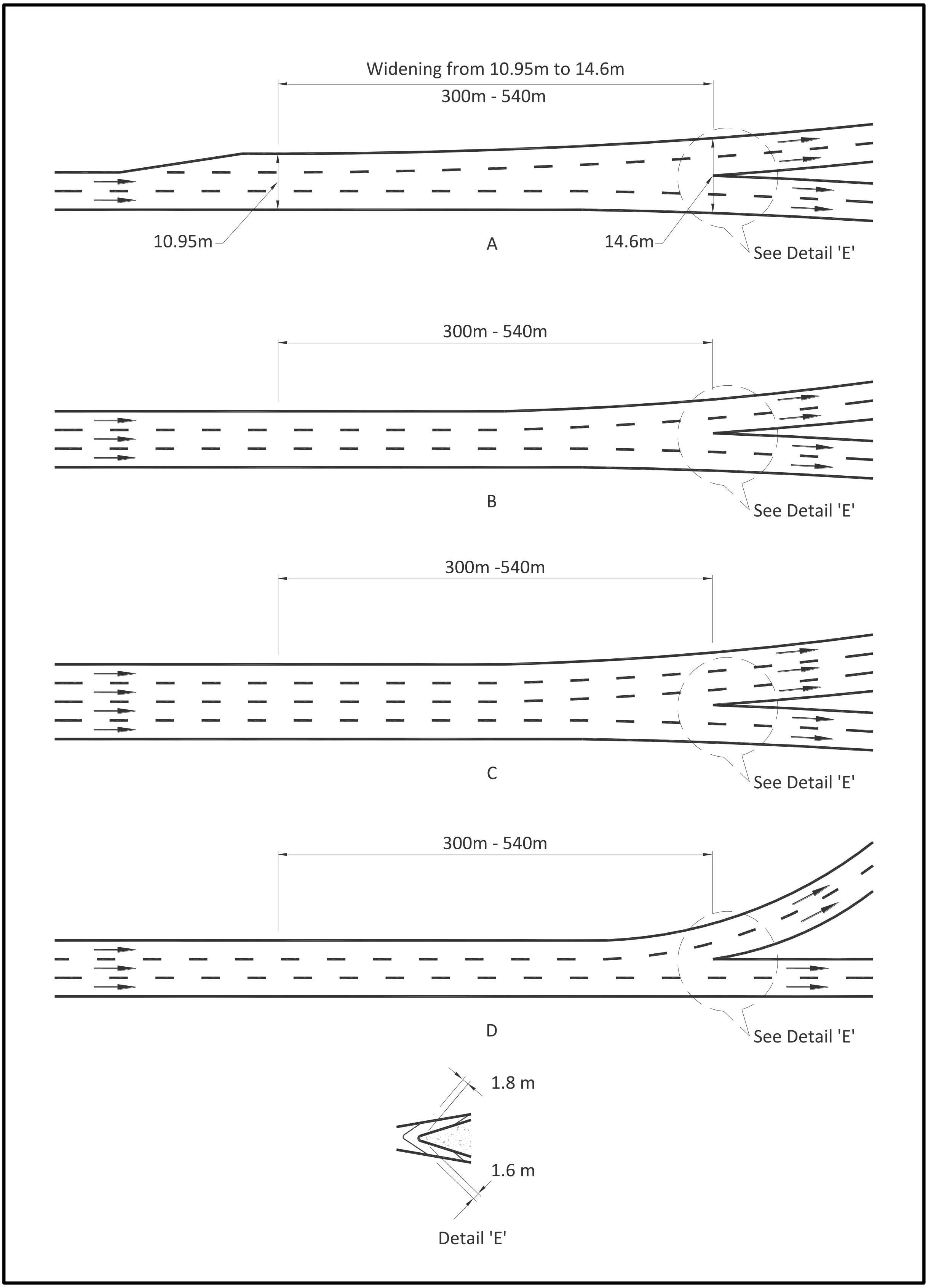


Figure 5.39: Major Fork

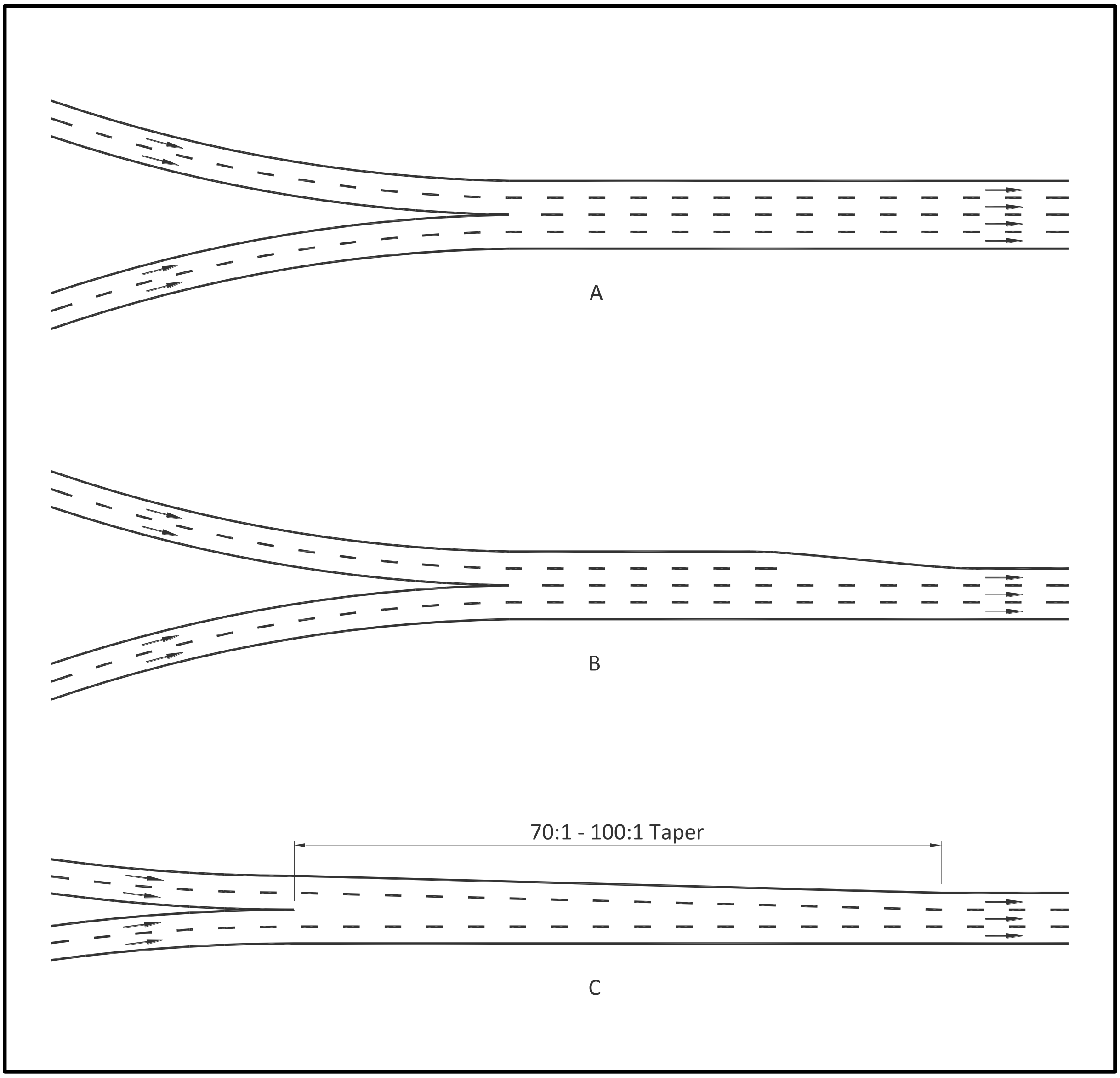


Figure 5.40: Branch Connections

6 ROADSIDE/HIGHWAY FACILITIES

6.1 GENERAL

This chapter deals with a variety of facilities associated with the road and its corridors, which are not addressed elsewhere in this manual. These are:

- Pedestrian Facilities
- Kerb and Edging
- Fences
- Safety Barriers
- Parking Facilities
- Cul-de-Sac / Turnarounds
- Cycle Facilities
- Driveways
- Public Transport Facilities
- Landscaping

6.2 PEDESTRIAN FACILITIES

Pedestrian facilities shall be given full consideration in the early stages of planning and development of future provincial, regional, and local transportation facilities. The design of pedestrian ways shall be included at the construction, reconstruction stage, or other change in existing transportation facility, emphasising areas within 1.6 km of an urban area.

All new roads, except limited access roads and some rural roads, should be designed and constructed under the assumption that they will be used by pedestrians. Therefore, all roadside features associated with pedestrians should be incorporated into the roadway design.

The planning and design of new roads shall include provisions that minimize vehicle-pedestrian conflicts. Features requiring special attention include:

1. Pedestrian Sidewalks/Footpaths
2. Pedestrian Crossings; including overpasses and underpasses for Pedestrians.

Other features such as traffic control devices and kerb-cuts (depressed kerbs and ramped sidewalks) shall also be incorporated along roadsides to facilitate the senior citizens and those physically impaired and handicapped.

6.2.1 Pedestrian Envelopes

The physical characteristics of humans, their capabilities in walking, and their comfort levels and desires with respect to privacy and proximity to other pedestrians are all inputs to design for pedestrian. The term “pedestrian envelope” describes these considerations. **Figure 6.1** shows typical minimum pedestrian envelopes. The pedestrian envelop encompasses the different needs of pedestrians, be they semi-mobile, on wheels, or completely or slightly ambulatory.

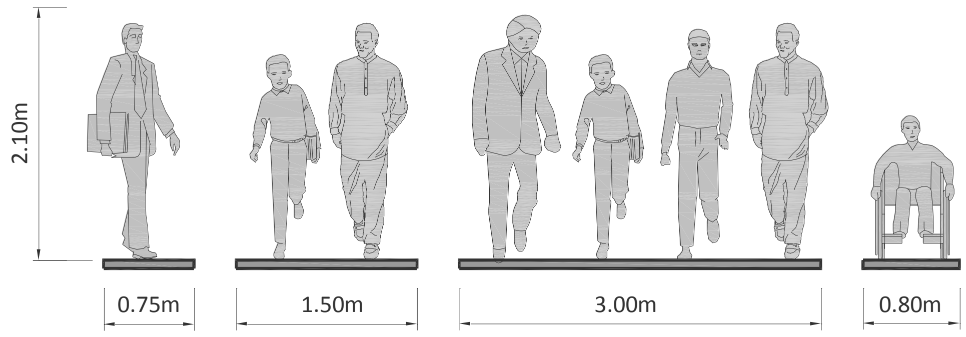


Figure 6.1: Minimum Pedestrian Envelopes

6.2.2 Sidewalks

Sidewalks are an integral part of urban roads. All urban roads should provide space for sidewalks, unless they are being specifically designed to prohibit walking. In rural areas sidewalks are rarely provide, except along section of roads where there is substantial residential or commercial development.

Sidewalk pavement should be smooth and level and should not be interrupted by steps or abrupt changes in levels of more than 6 mm. Changes in sidewalk level due to various building entrance levels (for the existing buildings) should be fixed or mitigated by engineering solutions. Vertical separation of pedestrian paths from vehicle roadways should be maintained. Typical kerb height should be 15 cm for all roads.

6.2.2.1 Cross Slope

The minimum cross slope of sidewalk should be to be 2.0 % towards the roadway to facilitate drainage, but should not be greater than 3 %.

6.2.2.2 Sidewalk Width and Capacity

The minimum width of sidewalk should be 1.8 m. The width of a planted strip between the sidewalk and roadway kerb, if provided, should be a minimum of 0.6 m to allow for maintenance activities. The width of sidewalk should be 0.6 m wider than the minimum required width, if provided adjacent to kerb line.

Table 6.1: Sidewalk Width to Accommodate Pedestrian Flows

Sidewalk Width (m)	Maximum Pedestrian Flow (person/min)
3	55
4	90
5	130
6	170
8	260
10	360

(Source: *Geometric Design Manual for Dubai Roads* [7])

The width of the sidewalk should accommodate the predicted pedestrian volumes. **Table 6.1** shows the recommended standards for the design of sidewalks. The design of pedestrian flow is the number of pedestrians per minute averaged over the busiest 15-minutes period. The sidewalk width relates to the clear and available width, which should be clear of trees, planters, or street furniture such as lighting columns or road signs. Where the back of sidewalk is walled, the available width should be reduced by 0.5 m. Where shop windows form the back boundary of the sidewalk, a total reduction of 1.0 m should be made.

6.2.2.3 Gradients

Where pedestrian paths are provided adjacent to roads, the longitudinal gradient should be consistent with that of the road. Pedestrian path gradients of up to 1:20, or 5 percent, are considered acceptable. Where the gradient exceeds 5 percent, the designer should consider the need to separate the pedestrian path from the road and should consider the pedestrian path as a ramp for design purposes.

6.2.3 Pedestrian Ramps

Where there is a significant change in level between two sections of pedestrian path, it may be necessary to construct steps and ramps. Steps are inaccessible to wheelchair users; therefore, ramps are preferable to steps to provide access for people with disabilities. Ramps should be built with a minimum width of 1.8 m, preferably 2.0 m, and have a non-slip surface. Rest areas or flat sections (with very shallow gradients) of a minimum 1.5-m length should be provided over the full width of the ramp when:

- The ramp is continuous over a length of 10 m
- There is a change of direction

Level areas, known as landings, at least 1.2 m in length should be provided at the bottom and top of the ramp and over the full width of the ramp. The maximum permitted gradient for ramp is 1:12.

6.2.4 Steps

Steps may be the only option at locations where it is not practicable to provide ramps at suitable gradients. However, alternative means of access for persons with disabilities shall be provided such as stair lifts. **Table 6.2** lists the dimensions recommended for steps.

Table 6.2: Recommended Dimensions for Steps

Design Parameter	Dimensions
Width	Preferred 1.2 m a. m where there is two-way movement 3.0 m where the steps lead to a platform where people carrying luggage
Height of Riser	150 mm
Tread Depth	300 mm
Maximum number of risers in a flight	12
Minimum number of steps in a flight	3

(Source: *Pedestrian and Cyclist Design Manual, Dubai Municipality, 2003* [8])

6.2.5 Pedestrian Crossings

Pedestrian movement is relatively compatible with the traffic movement on local streets, but becomes less compatible on roads of higher category in the hierarchy. In particular, the presence of pedestrians making at-grade crossings on expressway or freeway would be extremely hazardous and should be prevented.

The following guidelines should be considered while providing pedestrian crossings on different classes of roads:

- Pedestrian crossings are not generally required on local roads.
- On collector roads, crossings incorporated with signalized intersections should be provided, especially where they intersect with arterials. Grade separated crossings (bridges or underpasses) are only allowed where justified. Zebra Crossings are only appropriate on urban collectors with posted speed of 60 km/h or less.
- It can be hazardous for pedestrians to cross arterial roads. Zebra crossing are only acceptable on minor arterials with a posted speed of 60 km/h or less. Crossings incorporated within intersections are normally provided on Arterials. Grade separated crossings are always acceptable but may not always be cost-justified.
- Expressways and freeways are no pedestrian zones; therefore, pedestrian crossings are always grade separated.

Pedestrian Crossings should be located at a maximum of 150 m intervals along streets and should relate to areas where the pedestrians desire to cross. The choice of pedestrian crossing facilities can be as follows.

- Pedestrian Crosswalk / Zebra Crossing
- Grade Separated Crossings

6.2.5.1 Pedestrian Crosswalk / Zebra Crossing

Zebra crossings do not use signal control and consist of broad bands of white lines across the road, supplemented by pedestrian signs in the verge and, if applicable, in the median or refuge. Drivers must stop ahead of a zebra crossing when a pedestrian steps onto the broadband area. Zebra crossings are applicable only where the crossing is not signalized. **Figure 6.2** shows a typical layout. Details are provided in the *Manual of Uniform Traffic Control Devices (MUTCD)* [1].

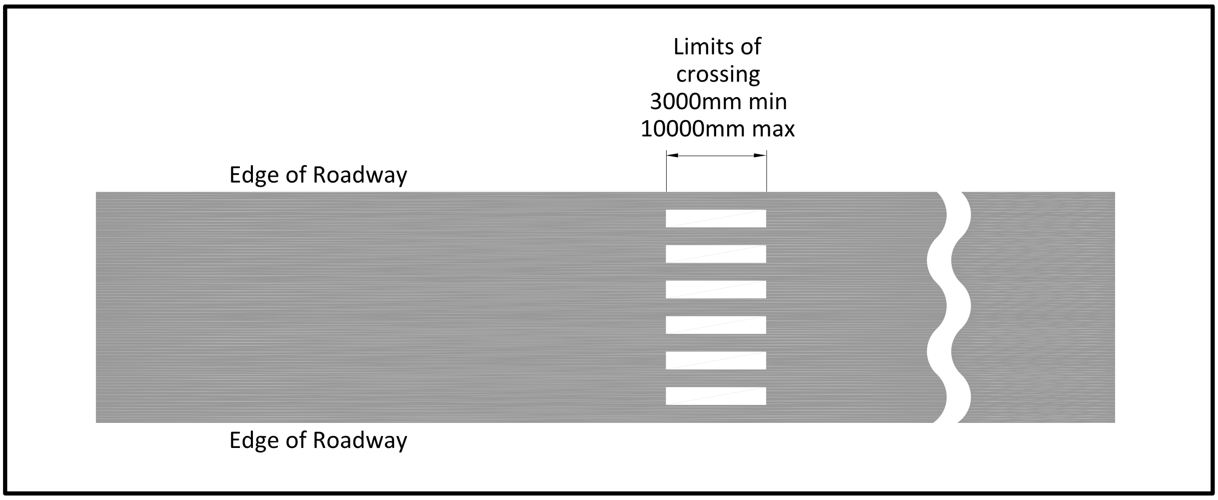


Figure 6.2: Typical Zebra Crossing Layout

It is vital that road markings for crossings be kept clear. If the quality or visibility of the markings reduces significantly, drivers may ignore the crossing, to the detriment of pedestrian safety. Regular maintenance of road markings is essential.

The minimum width of a crossing should be 3.0 m. Width may be increased 0.5 m for each 125 pedestrians above 600 pedestrians/hour averaged over the 4 busiest hours, up to a maximum of width of 10 m. Where the roadway is more than 10 m wide and a straight crossing would be more than 10 m long, a central pedestrian refuge should be provided on the crossing.

It is important that pedestrians be able to see approaching traffic. Their views should not be restricted by parked or stationary vehicles. Zebra crossings generally are more cost-effective to install than signal-controlled crossings and have less ongoing operation costs. However, a signal-controlled pedestrian crossing would be preferable where the following conditions apply:

- Vehicle speeds are high, because pedestrians find it difficult to judge the speeds and stopping distances of approaching vehicles. Zebra crossings should never be used where the posted speed exceeds 60 km/h or where the posted speed is less than 60 km/h but the road layout is such that speeds in excess of 60 km/h are likely to occur.
- A significant number of pedestrians are elderly or have disabilities.
- High and continuous pedestrian flows in busy commercial areas or adjacent to a bus or train station may cause excessive delays to traffic.
- Traffic is particularly heavy. Pedestrians may then be unwilling to step onto the zebra crossing because of the lack of positive control for vehicles.

Installing a raised zebra crossing will reduce vehicle speeds approaching the crossing and makes it safer for pedestrians to cross the road. Additional benefits may be realized in providing speed hump or table because vehicle speeds will be slower on the approach and pedestrians can cross the roadway at the same level as the pedestrian path. Zebra Crossings can be provided at intersections and mid-blocks.

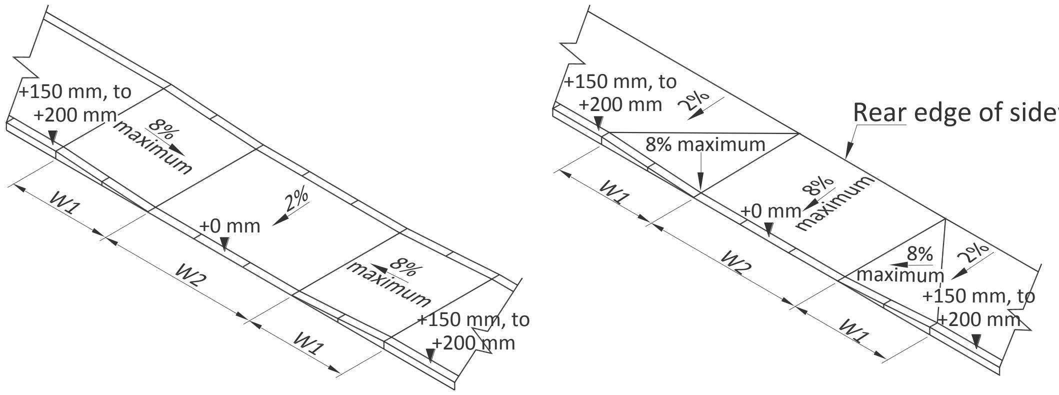
The provision of un-signalized mid-block crosswalk should be carefully considered. When used, mid-block crosswalks should be illuminated, marked and outfitted with advanced warning signs. Pedestrian-activated signalized crosswalks may be appropriate at some locations but the locations must be approved authority.

6.2.5.2 Kerb Ramps

Longitudinal ramps (pedestrian ramps) should not exceed a maximum of 8.3% gradient. Kerb Ramps shall be provided at all pedestrian crossing points, to allow a smooth transition from raised pedestrian path level to road roadway level, as shown in **Figure 6.3**. This enables safe and convenient movement of pedestrians, people pushing prams or strollers, and persons in wheelchairs. The dimensions W1 and W2 may be amended to suit site-specific circumstances.

A mountable/sloping kerb flush with the roadway shall be provided across the entire width of the crossing point. Generally, the ramp gradient across the pedestrian path to a dropped kerb should be between 1:12 and 1:20. For narrow pedestrian paths, the steeper gradient will allow the level strip at the back of the pedestrian path to be maximized.

Drainage equipment such as gratings should not be placed in ramp areas where they may cause an interference to wheelchair movement. At signalized urban priority intersections with no formal pedestrian facility, mountable kerbs should be provided on both sides of the minor road to provide a continuous pedestrian route parallel to the major road.



Kerb and Footpath Details-Dropped Kerb and Median Crossing

Kerb and Footpath Details-Dropped Kerb

Key:

Kerb transition: W1 = 1,800 mm for asphalt or precast concrete slabs
W1 = 3,000 mm for block paving

Dropped kerb: W2 = 1,800 mm for all types of surfacing

Note: The dimensions W1 and W2 may be amended to suit site-specific circumstances.

Figure 6.3: Kerb Ramp and Median Crossing

6.2.5.3 Pedestrian Refuge Islands

Pedestrian refuges, also known as traffic islands, are a relatively inexpensive method of improving crossing facilities for pedestrians. They allow pedestrians to concentrate on crossing one stream of traffic at a time, and they create a relatively safe waiting area between traffic lanes in urban areas.

Where the roadway is more than 10 m wide and a straight crossing would be more than 10 m long, a central pedestrian refuge should be provided on the crossing. Refuges should be at least 2 m wide, preferably 2.5 m wide to accommodate wheelchairs, prams, and strollers. The width of the crossing should be maintained across the full roadway. This width should include the refuge island that will have either openings or median crossings as shown in **Figure 6.3**.

The roadway width at the crossing should be sufficient to prevent vehicles passing too closely to the refuge or the pedestrian path, which can be intimidating for pedestrians. Pedestrians should be discouraged from crossing divided roadways unless signal control is provided. Special pedestrian refuge sections should be provided at selected points, or ideally at intersection locations, and preferably incorporated in the median.

6.2.6 Grade Separated Pedestrian Crossings

Grade separation of pedestrians and vehicles is achieved by the use of pedestrian bridge or underpasses. Pedestrian bridge or underpass provide full segregation of pedestrian and vehicular movements, provide pedestrian/bicycle continuity to sidewalks, bicycle lanes and shared use paths and they are potentially a safe form of crossing facility. They also result in minimum disruption to traffic flow; however, the cost of such facilities can be high.

Justification for pedestrian grade separation structures derives from a detailed study of present and future community needs. Each situation should be studied separately and the study should include pedestrian generating sources, travel patterns, crossing volumes, roadway classification, and location/circuitry of adjacent crossings, land uses, sociological and cultural factors, and the predominant type and age of users.

Grade separation involves pedestrians having to change levels by the use of steps and ramps. This situation may not be convenient because the route up or down to change levels is usually longer than a direct crossing. The situation can lead to pedestrians choosing to use a direct and potentially hazardous crossing as an alternative route.

Furthermore, the need to change levels and the longer pathway can pedestrian bridge or underpass challenging for the elderly or persons with disabilities to use. Careful consideration should be given to these conditions when opting for a grade-separated facility.

The choice between a bridge and underpass should be based on relative costs, groundwater influence, drainage, existing utilities, current and future land use, visibility, topography and the surrounding architecture. **Table 6.3** lists minimum vertical clearances required for grade separated facilities. **Table 6.4** lists the minimum widths.

Table 6.3: Vertical Clearances for Pedestrian Overpass and Underpass

Minimum Clearance Required below Pedestrian Overpass	6.5 m
Minimum Clearance Required in Underpass	3.5 m

Table 6.4: Minimum Width of Pedestrian Overpass and Underpass

	Width (m)
Pedestrian Only	2.4
Shared-Use Path	3.65

The overpass or underpass shall be provided according to the following criteria:

- If the approach sidewalk or path is wider than these minimums, the clear width of the structure should match the approach width. The desirable clear width should include additional 0.6 m wide clear area on each side.
- In addition to stairs, ramps should be provided at all grade-separated structures as per Section 6.2.3 of this Chapter.
- Provide full-length pedestrian handrails on both sides of pedestrian ramps.
- There should be no alternate provision of pedestrian crossing within a distance of 200 m on either side of the proposed location.
- Fencing/Railing should be provided on both side of the bridge structure.
- Provide full or partial screening on pedestrian bridges in order to reduce the likelihood of objects being dropped or thrown onto the roadway below.

- For pedestrians to use the underpass it must be clean, well-lit and clear visibility of the Exit must be offered to the pedestrians before entering.
- Underpass should not be located at places, which are subjected to drainage issues.

6.3 Kerbs and Edging

A kerb is a raised stone or concrete edging at the edge of pavement that separates the roadway from another feature, such as a roadside, island, or median. Kerbs provide structural side-support for pavement layers, delineate the edges of a road, collect and channelize storm runoff at the kerb face, and confine vehicles to the roadway area. They can also serve to delineate and protect pedestrian paths and provide erosion protection from storm runoff. Kerbs will be provided along all edges of pavement in urban areas.

6.3.1 Types of Kerbs

There are generally three types of kerbs:

1. Upstand Kerbs/Vertical Kerbs
2. Mountable Kerbs/Sloping Kerbs
3. Flush Kerbs

Figure 6.4 presents various types of kerb used for the different purposes.

6.3.1.1 Upstand Kerbs/Vertical Kerbs

Vertical kerbs may be either vertical or nearly vertical and are intended to discourage vehicles from leaving the road. They are used to provide protection to footways, traffic islands, pedestrian guardrail, traffic signs, etc. Kerbs on footways should have a height of 150 – 200 mm above road level. If they are higher than this, pedestrians may prefer to walk in the road. The upstand kerbs are available in a range of sizes and shapes, allowing installation on curves of various radius. The designer is advised to check the availability and dimensions of kerbs with the suppliers. Vertical Kerbs and safety walks may be desirable along the faces of long walls and tunnels, particularly if full shoulders are not provided. These Kerbs tend to discourage vehicles from driving close to the wall, and thus ensure the safety walk, reducing the risk to persons walking from disabled vehicles.

6.3.1.2 Mountable Kerbs/Sloping Kerbs

Sloping kerbs are designed so vehicles can cross them readily when the need arises. Sloping kerbs are low with flat sloping faces. For ease in crossing, sloping kerbs should be well rounded. Sloping kerbs can be used at median edges, to outline channelizing islands in intersection areas, or at the outer edge of the shoulder.

6.3.1.3 Flush Kerbs

Flush kerbs are used where a paved area joins an unpaved area, and it is laid level with the surface of paved are. The errant vehicles are faced by less danger of damage because of these kerbs. The also define the carriageway edge and are useful in considerable snow-ploughing areas. In flat areas where it is impractical to create longitudinal gradients for channels or drainage, flush kerbs can be provided to allow for continuous over-edge drainage of carriageway run-off.

Gutter sections may be provided on the carriageway side of a vertical or sloping kerb to form the principal drainage system for the roadway. Inlets are provided in the gutter or kerb, or both. Gutters are generally 0.3 to 1.8 m wide, with a cross slope of 5 to 8 percent to increase the hydraulic capacity of the gutter section. The gutter section cannot take all the runoff, some overflow is expected on the surface. The spread of water on the carriageway is kept within tolerable limits by the proper size and spacing of inlets. Grate inlets and depressions for kerb-opening inlets should not be placed in the lane as the drivers veer away from them, due to their adverse effect. Bicycle-compatible grates should be used everywhere bicyclists are permitted. Warping of the gutter for kerb-opening inlets should be limited to the portion within 0.6 to 0.9 m of the kerb to minimize adverse driving effects.

Where there is a need to install a safety barrier alongside a kerbed section of roads, the barrier design, kerb design and drainage design should be carried out together. The kerb may affect the choice of safety barrier type and it is important to ensure that the combined drainage/kerb arrangement does not impair the safe operations of the safety barrier.

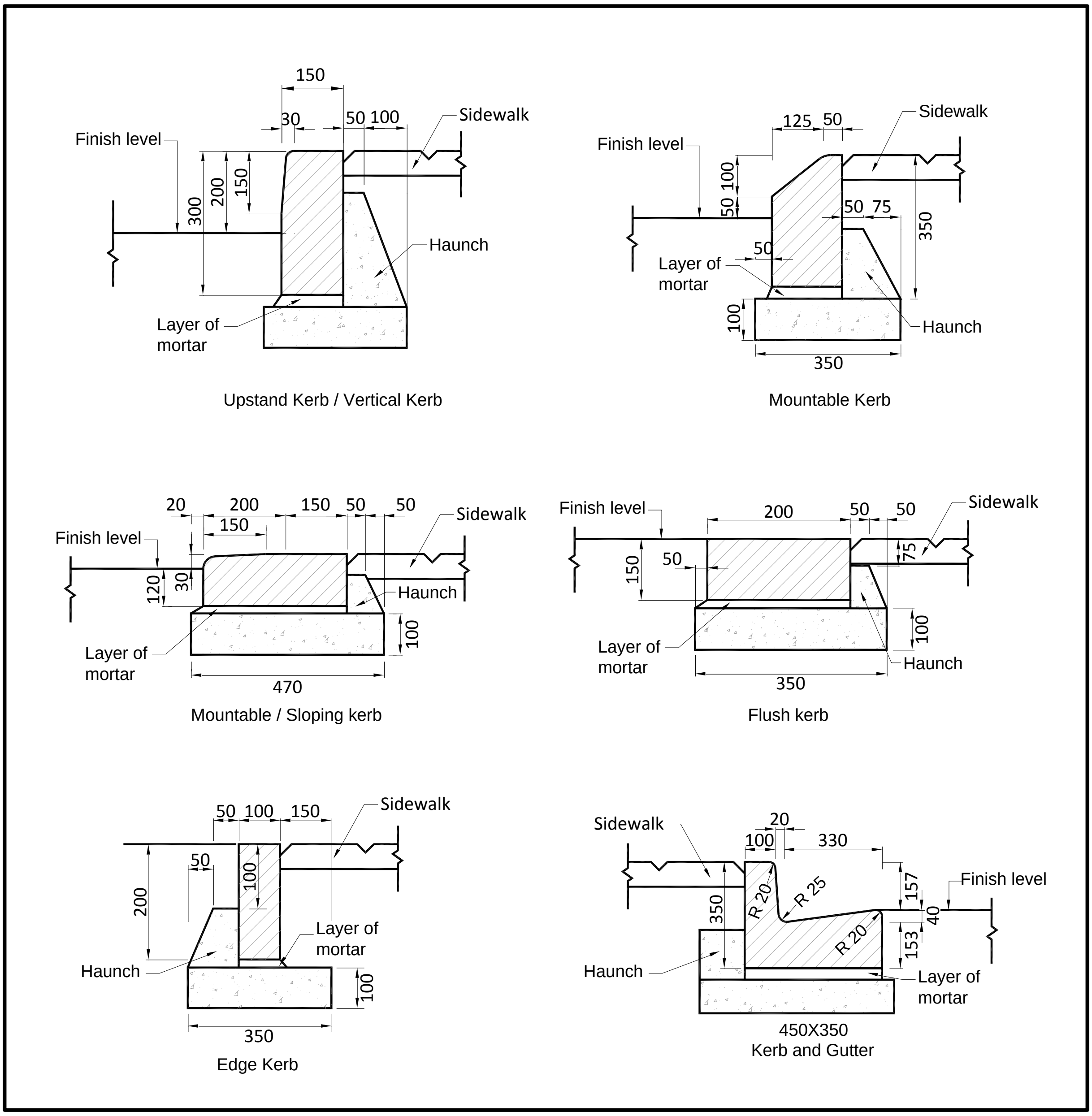


Figure 6.4: Standard Kerb Types

6.3.2 Placement

Kerbs should be positioned to provide the same unobstructed roadway width that is normally provided. All roadway width dimensions are measured to the front face of the kerb. On low speed urban arterials with posted speed less than 80 km/h, kerbs are normally place adjacent to the carriageway. Kerbs should be offset at least 0.3 m from the edge of the roadway. Kerbs are not recommended along roadways with posted speed of 80 km/h or greater. If kerbs are to provide on such roads to control drainage, they should be placed at least 1.2 m from the edge of carriageway or at the edge of paved shoulder, whichever is farther form the carriageway edge.

Vertical kerbs should not be used along motorways or other rural high-speed roads with posted speeds of 80 km/h or greater because an out-of-control vehicle may overturn or become airborne because of an impact with such a kerb. If kerbs are used in conjunction with posted speeds of 80 km/h or greater to control drainage and minimize right-of-way impacts, they should be placed at least 3.0 m from the edge of the carriageway or behind a guardrail. The use of kerbs should be limited to isolated locations such as lighted interchange locations to control drainage and minimize right-of-way. For posted speeds less than 80 km/h, kerbs should be placed at the edge of the paved shoulder.

6.3.3 End Transitions

A transition from one kerb type to another shall be done over a length of between 1.2 m and 1.8 m. At kerb termini, the kerb should transition from normal kerb height to zero in 3.0 m.

6.4 Fences

Highway agencies use fencing extensively to delineate the control of access acquired for a highway. While provision of fence is not a necessary, it may serve to reduce the likelihood of encroachment onto the highway right-of-way. Any portion of a highway with full control of access may be fenced, except in areas of steep slopes or natural barriers or areas where it can be established that fencing is not needed to preserve access control.

Fencing for access control is usually owned by the highway agency so that the agency has control of the type and location of fence. The type of fence that is most cost-effective yet best suited to the specific adjacent land use is should be selected.

There are many different types of fence used within the road right of way, each having its own particular application. The main types and their uses are listed below:

6.4.1 Boundary Fences

Boundary fence delineates and separate private property from the road right of way.

6.4.2 Animal Fences

Animal fences are provided to prevent animals from entering the road right of way. The height and nature of the chosen fence depends in the type of animal to be contained.

6.4.3 Headlight Fences

Headlight fences may be introduced, generally in the median, at location where it is desirable to minimize the glare of the headlight of oncoming vehicles. This is likely to occur at bends in unlit rural freeways and expressways, especially on crest curves.

6.4.4 Pedestrian Fences

Pedestrian fences may be required where there are significant numbers of pedestrians in a sidewalk or at venues where crowds may gather. The fence is designed to channel the movement of pedestrian traffic, and when used near kerb, reduces the risk of a pedestrian to accidently stepping from the sidewalk into a traffic lane. It is particularly useful in discouraging pedestrians from crossing at hazardous locations. It is also very effective near at-grade intersections, channelling pedestrians to designated crossing points.

Pedestrian fences can also be used away from the road edge, for example to direct pedestrians along a footpath to a grade separated crossing of a freeway. In this circumstance, the fence needs to be around 2 m high, typically 500 m on both side of the crossing facility and strong enough to withstand wilful damage. The fence in the median should also be high enough to discourage the pedestrians from climbing over it. Where main risk is form errant vehicles rather than straying pedestrians, safety barriers should be used rather than pedestrian fence, as pedestrian fences are not designed to withstand significant vehicle impact.

6.5 SAFETY BARRIERS

Traffic barriers are used to prevent vehicles that leave the carriageway from colliding with objects that have greater crash severity potential than the barrier itself. Because barriers are themselves a source of crash potential, their use should be carefully considered. For details of specific safety barriers, the manufacturer's technical literature should be referred. Roadside barriers are categorized in terms of deflection characteristics in the case of vehicular impact, and are listed below:

- Flexible Barrier
- Semi-Rigid Barrier
- Rigid Barrier

The selection of the most appropriate type of barrier depends on the available distance between the barrier and the hazard. This is important, as sufficient clearance must be provided to ensure that the anticipated deflection of the barrier system is less than the distance to the hazard.

The expected dynamic deflection of different barrier systems can be found in the *AASHTO's Roadside Design Guide, 2011 (4th Edition).* [5]

6.5.1 Warrants for Use of Safety Barriers

When determining barrier requirements, the following factors must be considered:

- Risks involved with hitting the hazard versus colliding with the barrier.
- Evaluating roadway design (speed and traffic volumes) and roadside design (e.g. hazard lateral offset and side slope geometry) to barrier need.
- Evaluating costs of installing and maintaining a barrier system versus not installing a barrier system.
- Costs of accidents involving barriers versus not involving barriers.

Figure 6.5 indicates when a barrier is warranted, based on height and fill slope of embankments associated with constructed roadways. Embankment height and slope that falls out of the shaded region does not require barrier. However, barrier need for other roadside hazards within clear zone, installation and maintenance cost of barrier and accident cost involving barriers should be evaluated.

Head-on impact with an opposing vehicle often leads to fatalities, therefore, a continuous safety barrier is often provided in the median of a divided roadway. The provision of safety barrier at the edge of roadways should be considered when height of the embankment is greater than 6 m. It should always be located on the verge and not on the slope. When barriers are not provided, rounding of the top of the slope should be done to reduce the chance of an errant vehicle becoming airborne. Safety barriers are seldom required in cutting. Exceptions are where there is a steep rock face or where large boulders or other obstacles are located in the cutting slope. For roadside obstacles, a safety barrier should only be installed if it is clear that the result of a vehicle striking the barrier would be less severe than the accident resulting from hitting the unprotected object.

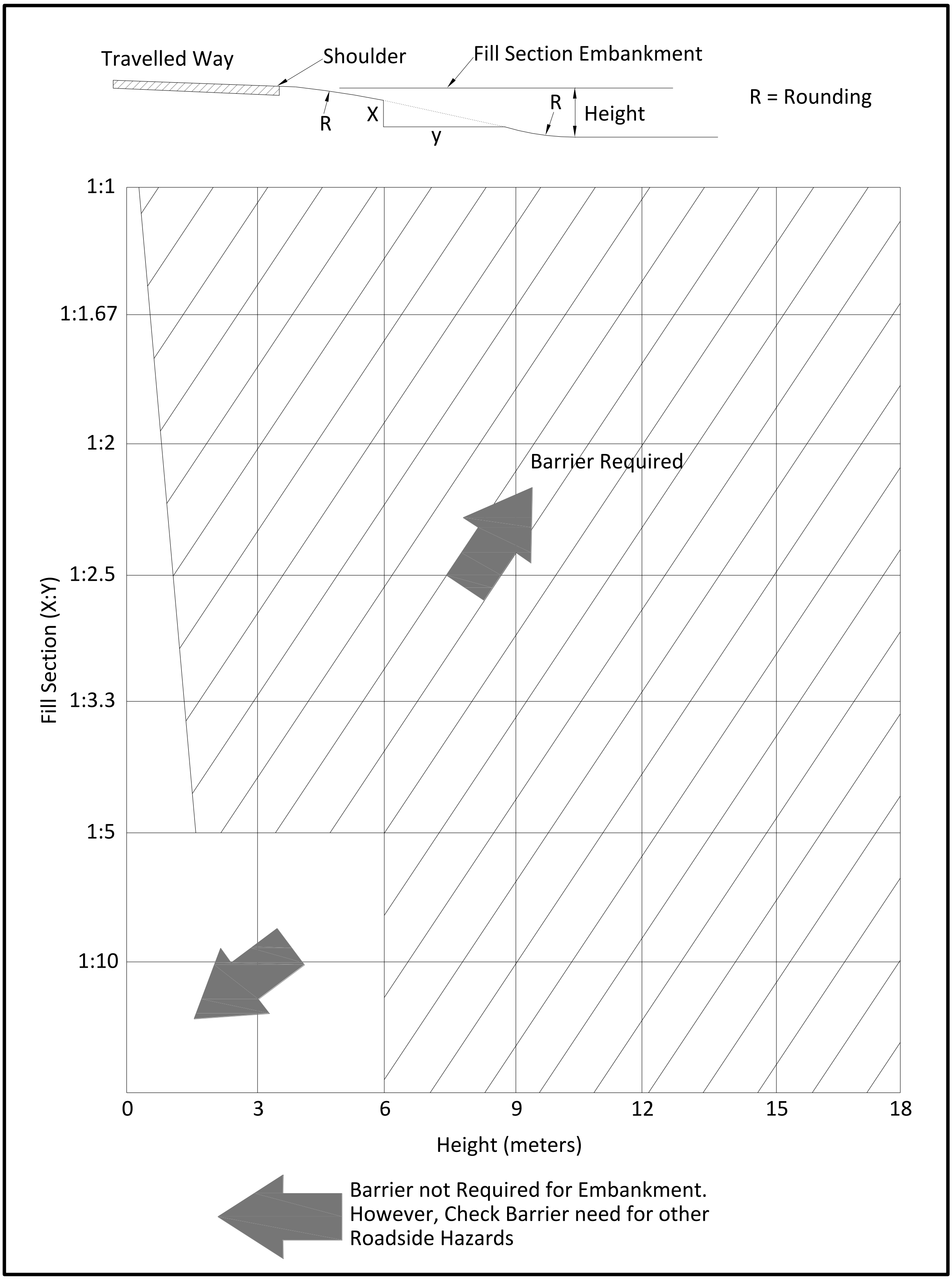


Figure 6.5: Requirement of Barriers on Embankments

6.5.2 Flexible Barriers

Flexible systems are generally more forgiving than other categories, because these dissipate the energy of a crash by providing a relatively high deflection upon impact and lower impact forces are imposed on the vehicle. Vehicles will tend to be redirected along the barrier after impact and there is a lower injury risk to occupants compared to other barrier types.

Furthermore, installation costs associated with flexible barriers are lower than semi-rigid and rigid barriers. Examples of this type of barrier include Weak-Post W-Beam and Low-Tension/High-Tension Wire Rope Safety Barrier.

When stiffer rail is used, the post spacing may be increased. The weak post barriers and cable barrier should be limited to areas with adequate clear space, as they can deflect up to 2.44 to 3.65 m.

6.5.3 Semi-Rigid Barriers

Semi-Rigid barriers deflect upon impact, but to a lesser degree than flexible barriers. This is achieved by the support posts bending and the barrier rail deforming to absorb the force of the impact. Examples of this type of system include Blocked-Out W-Beam (Strong-Post), Blocked-Out Thrie Beam (Strong-Post) and Modified Thrie Beam (Strong-Post). Lateral deflection of a semi-rigid barrier may typically be as much as 1.5 m. It usually remains functional after moderate collision.

6.5.4 Rigid Barriers

Rigid systems offer no deflection when hit by a vehicle. The impact energy is entirely absorbed by the vehicle. However, because of this the injury risk to vehicle occupants is higher than the other types. For high-speed impacts, vehicles have a higher probability of overturning or vaulting over these rigid barriers. For these reasons, rigid barrier system is not generally recommended on roads with design speed over 100 km/h. Their adoption on higher speed roads should be carefully evaluated by the designer. The basic roadside barrier is designed to be 810 mm high, but taller designs are available to counteract the overturning moments of trucks with higher centres of gravity. The most common examples of this type include Concrete Barriers (F-Shape and Vertical Profile).

6.5.5 End Treatment

The untreated end of any safety barrier is extremely hazardous if hit, as the beam element can penetrate the passenger compartment and will generally stop rather than redirecting the vehicle. All roadside and median barriers terminating within the clear zone and/or located where they have a high probability of being hit, shall have a crashworthy terminal installed at their upstream end. Crashworthy is defined as having been tested, approved and met the performance criteria of the *AASHTO's Manual for Assessment of Safety Hardware (MASH)* [9].

Leading Terminals - All leading approach end terminals shall be tested and approved and have met the performance criteria of MASH. Safety performance should meet the appropriate test level criteria, which is dependent on operating speeds and traffic composition of the roadway. The leading terminal is the approach end of a guardrail installation.

Note that on undivided roads, both ends of a barrier installation shall be considered as being leading ends, due to the probability of an impact from the opposing direction.

Trailing Terminals - Trailing terminals may be either crashworthy terminals or other suitably designed and approved terminal arrangement. The trailing terminal is the departure end of a guardrail installation. However, see the paragraph above regarding terminals on undivided roads.

6.5.6 Transition

Transitions shall be properly designed to avoid situations where vehicles may pocket or snag on stiffer sections, due to deflection of a more flexible system. For example, this is the case where a semi-rigid approach barrier joins a rigid barrier.

The transition should be designed to provide a gradual stiffening of the overall approach protection system, which can be achieved by reducing post spacing, providing larger or longer posts or strengthening the barrier (such as the use of nested rails). Generally, the length of the transition shall be 10 to 12 times the difference in the lateral deflections of the two adjoining systems. For example, the transition between a beam with a design deflection of 1.5 m and a rigid barrier or abutment, the transition length should be around 15 to 18 m.

All transitions between different barrier types and systems shall be tested and approved and have met the performance criteria of *MASH* [9]. Safety performance should meet the appropriate test level criteria, which is dependent on operating speeds and traffic composition of the roadway.

For more information on transition designs, refer to Chapter 7 of the *AASHTO's Roadside Design Guide, 2011* [5].

6.5.7 Placement

Placement of a barrier system shall be determined in a manner that increases motorist safety, decreases accident frequency and minimizes injury severity.

6.5.7.1 Lateral Offset from the Road

A barrier system shall shield the motorist from roadway hazards. It is therefore a standard rule that the barrier system shall be placed as far from the edge of carriageway as possible. This allows drivers space to regain control of their vehicle and possibly avoid an accident. It is important to note, however, as the distance between the edge of carriageway and the barrier increases, the potential vehicular impact angle also increases. Barriers that expose motorists to high impact angles tend to produce unacceptably high injury levels. **Table 6.5** gives suggested lateral offsets related to design speed.

Table 6.5: Suggested Setback from Edge of Carriageway

Design Speed (Km/h)	Setback from Edge of Carriageway (m)
50	1
60	1.5
70	1.7
80	2
90	2.2
100	2.5
120	3
140	3.7

(Source: *Geometric Design Manual for Dubai Roads* [7])

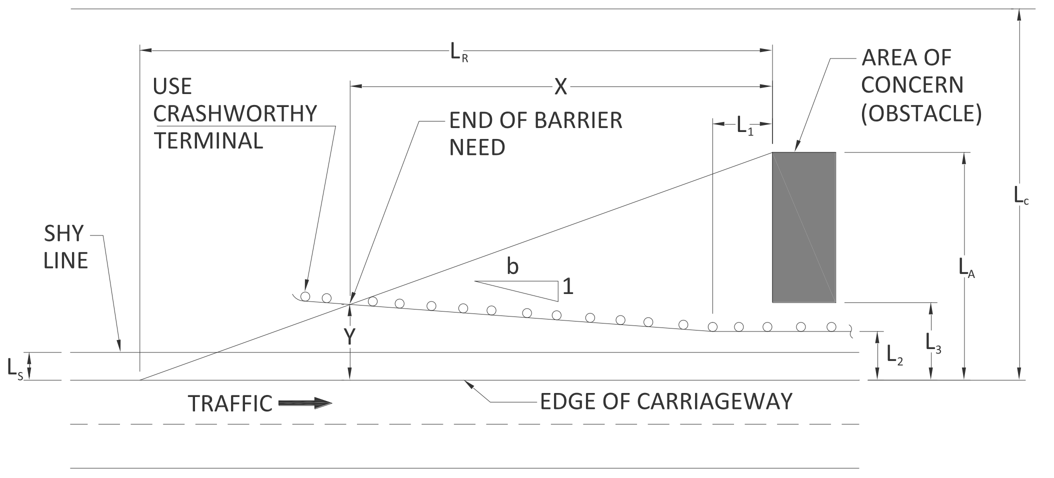
6.5.7.2 Clearance between Barrier and Object Being Protected

The desirable minimum distance between back of barrier and rigid hazards should not be less than the dynamic deflection of the safety barrier at impact condition of approximately 25° and 100 km/h. For minimum clearance, the manufacturer's specific requirements must be followed. On embankments care should be taken to ensure that at full deflection of the barrier, the wheels of the vehicle do not overhang the edge of the slope.

6.5.7.3 Flare

The flared portion of the barrier is not parallel to the roadway. The flare rates, as shown in **Figure 6.6**, are given in **Table 6.6**. The main advantages of using flare are as follows:

- Reduces the barrier length-of-need.
- Reduce the perception that the barrier is a hazard
- Minimizes the risk of vehicular impact due to the larger lateral offset.
- Lower barrier implementation cost.
- Minimize a driver's reaction to the introduction of an object near the carriageway.



X = barrier length of need
Y = distance from edge of carriageway to end of barrier need
L_A = distance from edge of carriageway to lateral extent of obstruction
L_1 = length of parallel barrier in advance of obstruction
L_2 = barrier lateral offset
L_3 = obstruction offset from edge of carriageway
L_c = distance from edge of carriageway to outside edge of the clear zone
L_R = runout length needed for a vehicle leaving the roadway to stop
1:b = flare rate

Figure 6.6: Barrier Layout Diagram

Table 6.6: Typical Flare Rates

Design Speed (km/h)	Flare Rate for Barrier Within Setback	Flare Rate for Barrier beyond Setback (1:b)	
		Rigid System	Semi-Rigid System
60	1:13	1:8	1:7
70	1:17	1:11	1:9
80	1:21	1:14	1:11
90	1:23	1:16	1:12
100	1:26	1:18	1:13
120	1:30	1:20	1:15
140	1:35	1:23	1:17

Refer to manufacturer's technical specifications for special conditions
(Source*: Geometric Design Manual for Dubai Roads* [7])

6.5.7.4 Runout (L_R)

The runout length (L_R) is the distance, parallel to the roadway, which a vehicle may require to stop after leaving the roadway, prior to hitting a hazard. Runout length requirements vary according to the roadway design speed (see **Table 6.7**). By measuring a distance in advance of the hazard equal to the runout length, the point at which an errant vehicle leaves the carriageway is identified. A straight line is drawn from the outer edge of the carriageway at this point to the furthest point of the hazard form the road. This defines the triangle of need.

Table 6.7: Runout Lengths

Design Speed (km/h)	Runout Length (L_R) (m)
50 and 60	60
70 and 80	90
90 and 100	110
120 and 140	140

(Source: *Geometric Design Manual for Dubai Roads* [7])

6.5.7.5 Hazard Lateral Distance (L_A)

The hazard lateral distance (L_A) is the distance between the edges of carriageway to the back of the hazard, if the hazard is a point obstacle. However, if the hazard is continuous or spans larger distances, such as an embankment, river, slope or culvert, the lateral distance should be extended to the edge of the clear zone. If the hazard extends beyond the clear zone, the minimum lateral distance would only be to the edge of the clear zone.

6.5.7.6 Length of Need (X)

The barrier length of need (X) is the length of barrier within the triangle of need. The farther the setback from the road and sharper the flare rate, the shorter is the length of need. The length of need can determined from analysis of the plan of the road and obstruction. For a straight road, it can be calculated form the following formula:

$$\boldsymbol{L_N = \frac{L_A + bL_1 - L_2}{X + (L_A/L_R)}} \qquad \textit{Equation 6.1}$$

Where,

L_1 = Length of parallel barrier in advance of obstruction

L_2 = Lateral offset from edge of carriageway to face of parallel barrier

L_A = Lateral extent of obstruction from edge of carriageway

X = Length of need

L_R = Runout Length (from **Table 6.7**)

b = Flare Rate (1:b) (from **Table 6.6**)

6.6 PARKING FACILITIES

The need for on- and off-street parking is determined by the existing and future development of the immediate surrounding area. To maximize the effective capacity of roadway improvements, sufficient off-street parking facilities should be provided to avoid the need for kerb line parking along primary roadways and main roads. In urban locations, parking may be provided contiguous with the road in designated parking lanes. Parallel parking lanes should be provided only on roads with posted speeds of 60 km/h or less. On-street parking is most appropriate on local roads and service roads.

Parking facilities are of four general types:

1. Parking areas located parallel to, but physically separated from the main road.
2. On-street parking spaces, developed adjacent to the travelled lanes of local roads.
3. Independent parking lots developed off local roads.
4. Parking structures.

Each facility consists of an “aisle” area and a “standing area” (parking stalls). In the case of on-street parking, the moving lanes of the local road also serve as the aisle.

The layout of on-street parking should support the functionality of the road. Parking should be prohibited:

- Within sight triangles at intersections, in order to maintain visibility.
- Within 20 m of an intersection, measured between the centreline of the side road and the end of the parking lane taper on the principal road (see **Figure 6.7**).
- Opposite vehicle or pedestrian access points to properties.
- On bends, so that adequate forward visibility can be maintained.
- At and ahead of pedestrian crossing points.
- Within 5 m of fire hydrants.
- At any other location where it would create unsafe condition.

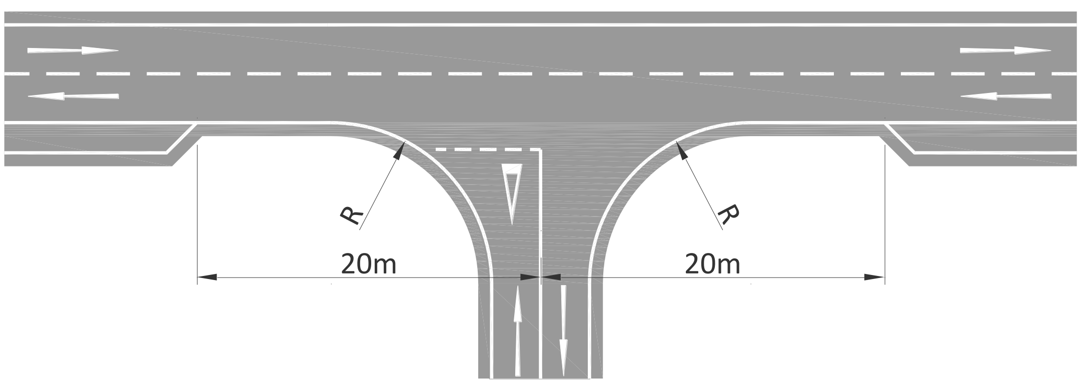


Figure 6.7: Minimum Clearance of Parking Lane from Intersection

The following guidelines should be followed concerning the design of parking facilities:

1. Aisle widths for 2-way roads should be a minimum of 6.0 m. Minimum aisle widths for 1-way roads vary dependent upon associated parking provision (See **Table 6.8**)
2. For one-way roads, diagonal parking is preferred.

3. A sufficient number of parking spaces for disabled persons should be provided, based on the International Building Code (IBC). See **Table 6.8** below.

4. Parking spaces for disabled persons should be located as close to building entrances and facilities as possible.

5. Provide mid-block pedestrian crossing(s) for large parking areas.

6. Consider parking spaces for bicycles and motorcycles.

7. Consider garbage bin locations in parking areas.

Table 6.8: Parking Provision Requirements for Disabled Persons

Total Parking Bays Provided	Required Minimum Number of Bays for Disabled Persons
1-25	1
26-50	2
51-75	3
76-100	4
101-150	5
151-200	6
201-300	7
301-400	8
401-500	9
501-1000	2% of the total
1001 and over	20, plus 1 no. for each 100 over 1000

Source: International Building Code (IBC) 2015

Disabled parking spaces must be designed so that a disabled person does not travel within the manoeuvring lane for vehicle traffic to reach the safe travel path to a building or other site location. Disabled parking spaces should be 90 degrees, minimum of 3.5 m wide and 6 m long with a minimum of 1.6 m wide space between the two 3.5 m spaces. The 1.6 m space must have diagonal striping on a 45-degree angle. Concrete kerb stops should be provided for each parking space. A minimum 1.6 m area must be provided in front of each parking space leading to handicap ramp. There must be an additional 1.6 m diagonal striped area between a regular parking space and an adjacent handicap space.

6.6.1 On-Street Parking

Two types of on-street parking are used, parallel and angle parking. Guidance on design for each is provided in the following sections. The dimensions are the minimum requirements and consideration should be given to increasing these depending on the size and class of vehicle anticipated.

6.6.1.1 Parallel Parking

Parallel parking may be provided adjacent to the left lane of the roadway. Parallel parking should be provided on roads with posted speed of 50 km/h or less. The bay dimensions for parallel parking bays should be a minimum of 2.5 m wide by 6.0 m long. This is illustrated in **Figure 6.8**.

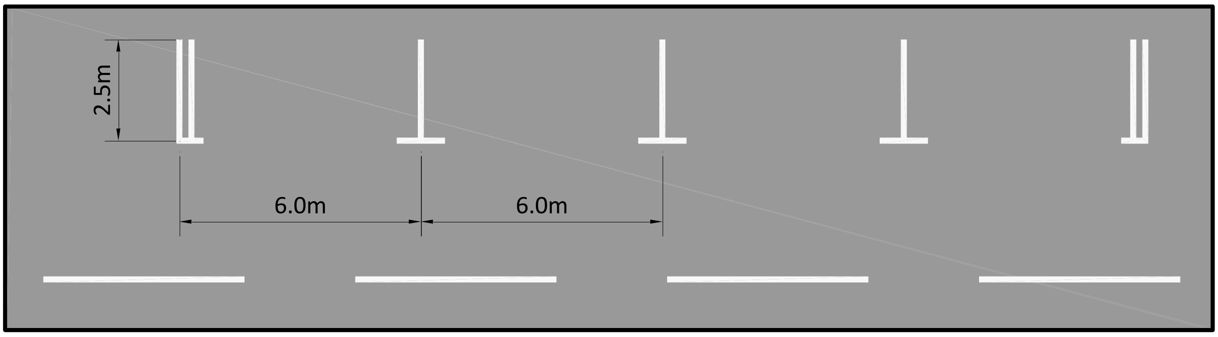


Figure 6.8: On-Street Parallel Parking Bay Dimensions

6.6.1.2 Angled Parking

Angle parking on public streets must be pre-approved by the Overseeing Organization. The decision whether to use angle parking on street should be based on safety and consideration of:

- Width of road
- Traffic volume
- Type of traffic
- Traffic speed characteristics
- Vehicle dimensions
- Expected turnover
- Land use served
- Functional road classification.

A buffer lane between the edge of the carriageway and the nearest part of the parking bay of 1.0 m should be provided. **Figure 6.9** and **Table 6.9** shows the angled parking width perpendicular to the road, buffer lane width requirements, the minimum width of the through lane (in addition to the buffer lane width), for one-way operation. For two-way operations, the absolute minimum width of the through lane is 6.0 m.

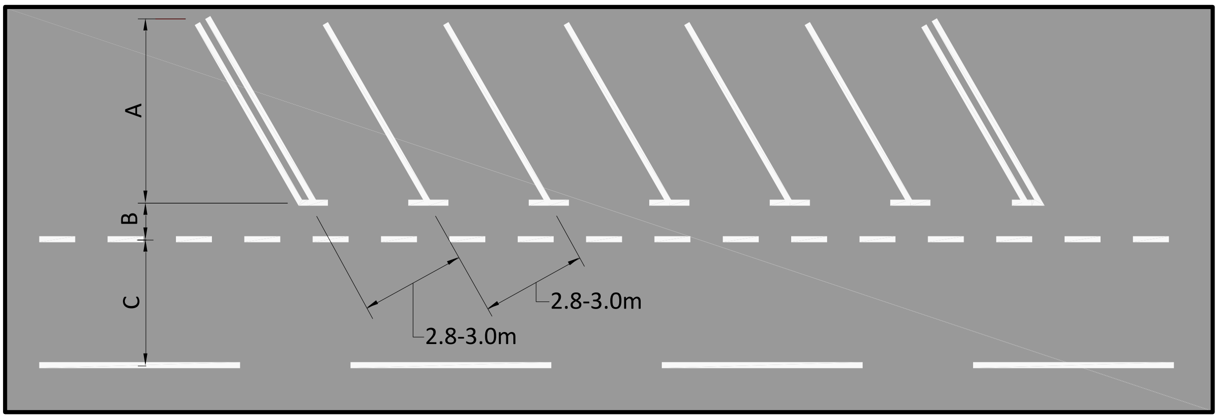


Figure 6.9: On-Street Angled Parking Bay Dimensions

Table 6.9: Roadside Angled Parking-Minimum Width for Adjacent Through Lane for One-Way Operations

Angle of Parking	Width Occupied (A)	Buffer Lane (B)	Minimum Through Lane (C)
45	5.1	1.0	3.8
60	5.4	1.0	4.5
75	5.3	1.0	6.5
90	4.8	1.0	7.0

(Source: *Qatar Highway Design Manual (QHDM)* [10])

6.6.2 Off-Street Parking

Off-street parking areas are located outside the roadway right-of-way in parking lots or garages. Generally off street parking lots are designed for angled parking. The dimensions for various angles of parking, depending on the geometry of the layout are shown in **Figure 6.10** and are given in **Table 6.10**.

For trucks and other large vehicles, it is normally convenient to provide a large, preferably paved area with no obstruction. This area may be marked to permit vehicles to drive forwards from an aisle into a marked bay and pull forwards into the aisle when setting off again. Bay dimensions are dictated by the size of the design vehicle and relevant swept path templates. The bay width should be 1 m wider than the width of the vehicle. Shallow parking angles of 30 ° to 45 ° are generally appropriate with aisle width dependent on the design vehicle, but typically around 15 to 20 m.

Table 6.10: Off-Street Parking Lot Dimensions

Dimensions (for a bay size of 2.5 m x 5.0 m)	On Figure 6.10	30 °	45 °	60 °	90 °
Bay width (m)	A1	2.50	2.50	2.50	2.50
Bay width, parallel to aisle (m)	A2	5.60	3.50	2.80	2.50
Bay length (m)	B1	5.00	5.00	5.00	5.00
Length of line between bays (m)	B2	10.00	7.50	6.25	5.00
Bay depth to wall (m)	C1	4.50	5.30	5.60	5.00
Bay depth to kerb (m)	C2	4.15	4.70	4.90	4.25
Bay depth to interlock (m)	C3	3.40	4.40	5.05	5.00
Aisle width between bay lines (m)	D	3.50	3.75	4.50	7.00
Bumper overhang (typical) (m)	E	0.35	0.60	0.70	0.75
Module, wall to interlock (m)	F1	11.40	13.45	15.65	18.00
Module, kerb to interlock (m)	F2	11.05	12.85	14.95	17.25
Module, interlock to interlock (m)	F3	10.30	12.55	14.80	18.00

(Source: *Geometric Design Manual for Dubai Roads* [7])

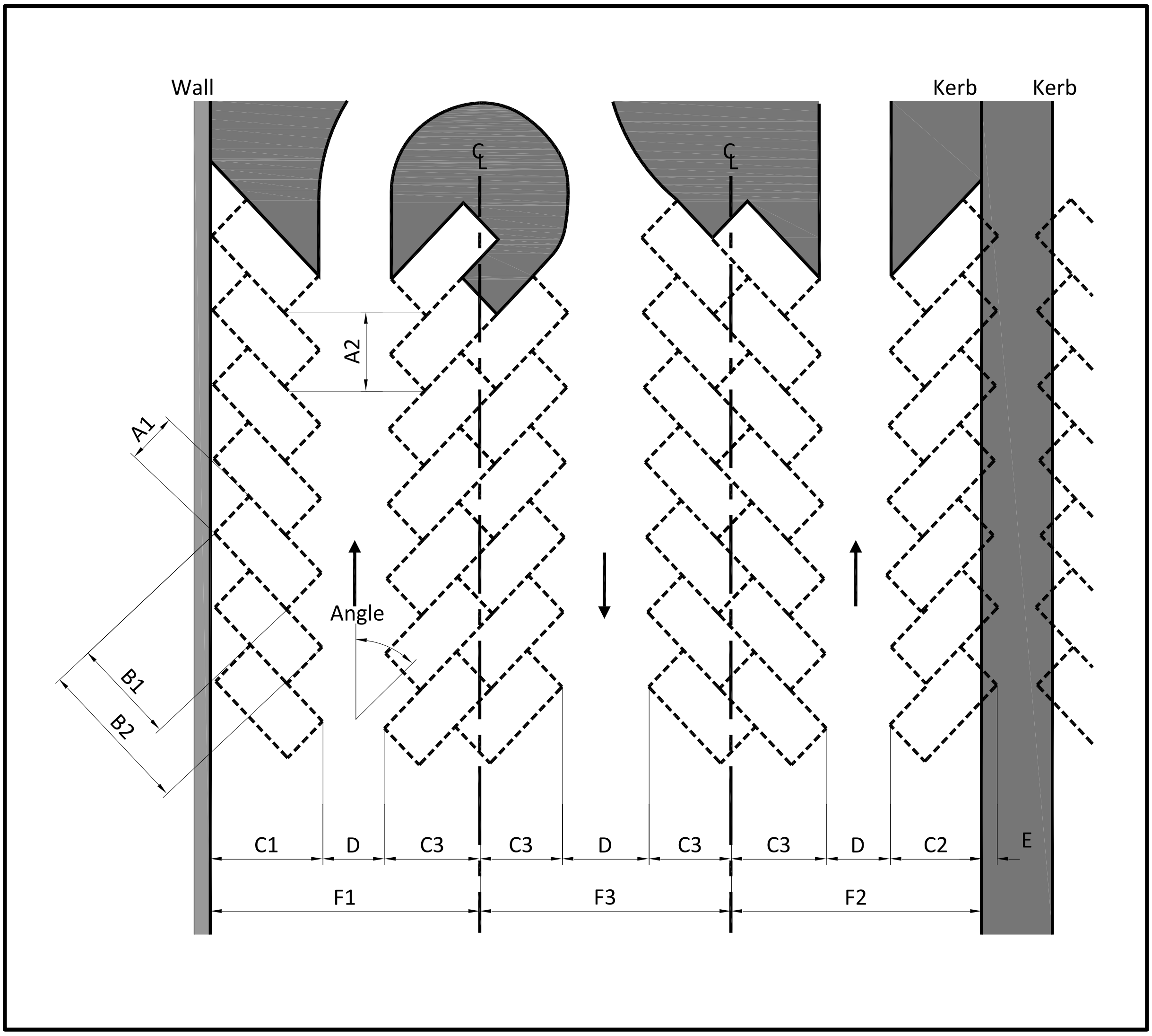


Figure 6.10: Off-Street Parking Bay Dimensions

6.7 CUL-DE-SACS / TURNAROUNDS

If a local street has one end closed, a special turning area should be provided at the closed end. The geometry will depend on the available right-of-way and the design vehicle expected to use the road. The turning areas can be of various shapes, as shown in **Figure 6.11**, the dimensions are shown for information only. A circular turning area is desirable, but the other shapes shown in **Figure 6.11** are also acceptable. Minimum outside radii of 10 m in residential areas and 15 m in commercial and industrial areas should be used. The choice and layout of the turning area depends on the width of the roadway and the position of the properties that need to the accessed and the available right-of-way.

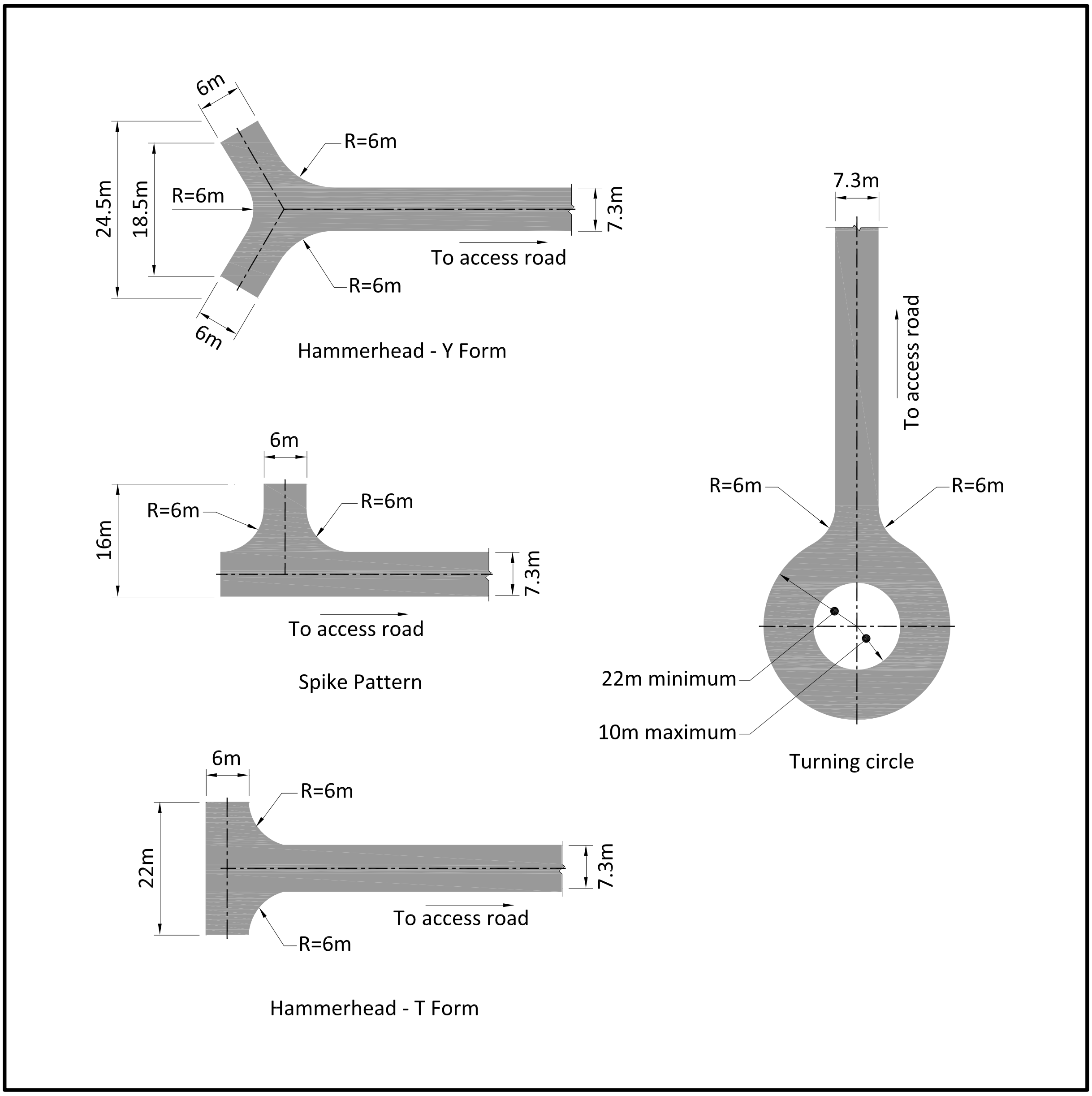


Figure 6.11: Cul-de-Sac and Turning Head Layouts

6.8 Facilities For Cyclists

Provision for bicycles has become an important factor requiring consideration in the highway design process. Fortunately, most of the mileage needed for bicycle travel is provided by the street and highway system. While many highway agencies allow bicycles on partial-access controlled facilities, most agencies do not allow bicycles on full-access controlled facilities. The use of bicycles in KP is limited and there is no practice to provide separate facilities for cyclist in the province. However, for new or improved highway and traffic management system, the provision of separate facilities for cyclists can be considered at planning stages in consultation and coordination with stakeholders. *AASHTO's Guide for the Development of Bicycle Facilities* [11] and *Roadside Features, DMRB, Volume 6, Section 3, Part 3, TA 57/87* [12] can be referred for further information.

Cyclists have same rights and duties as motor vehicles (for example, same merging and turning movement, need of adequate sight distance, access to all destinations etc.) when using the highway system. The main differences are lower speed and acceleration capabilities, and greater sensitivity to steep uphill grades.

Roads designed according to the standards provided in this manual should not pose hazards for cyclist, and thus no cycle specific design standards are required. In areas of heavy cycle traffic, following measures, which are generally low cost, can be helpful in enhancing the safety and capacity for cycle traffic:

- Where shoulders are provided, consider providing paved shoulder. If feasible, a wider than minimum shoulder width should be provided.
- On kerbed roads with no shoulder, consider providing a wider outside traffic lane (preferably 4.2 m).
- Drainage gullies and manhole covers should be flushed with pavement, preferably not in cycle travelled way.
- Maintaining a smooth, clean riding surface particularly by regular sweeping.

6.9 DRIVEWAYS

Driveways are access points that connect adjacent properties to the roadway. Driveway geometry depends on driveway type and the volume of traffic. Driveway types include residential, commercial, and industrial. The type of driveway will dictate the type of design vehicle to be used. The design vehicle establishes the minimum width of the driveway and radius dimensions. In commercial areas, the volume of the traffic using the driveway determines its number of lanes and establishes the overall driveway width. **Table 6.11** lists the recommended dimensions based on international best practices.

Table 6.11: Recommended Driveway Geometric Elements

Type of Driveway	Design Vehicle	Width of Driveway (m)		Radius of Driveway (m)	Distance between Driveways (m)	
		One-way	Two-Way		30 km/h	50 km/h
Residential	P	3.65	7.3	4.5	Less than 50	80
Commercial	SU-12	5.0	8.0	7.5		
Industrial	WB-12	5.0	8.0	10.5		

Consideration should be given to the following elements when designing driveways:

- **Location of driveways:** The location of a driveway affects the operation of an adjacent roadway. Driveways should be located where adequate sight distance could be provided, and they should be visible to the drivers on the roadway. When proposed drives are added to an existing corridor, the proposed drives should be located such that there is an adequate distance between the existing drive and the proposed drive to provide required sight distance.
- **Spacing between driveways: Table 6.11** lists the recommended spacing between driveways. However, the designer should coordinate with the Overseeing Organization and establish the spacing as required.

- **Driveway profile and cross slope:** Driveway profile should be smooth and provide efficient movements in and out of the driveway. The profile should be flat and still provide adequate drainage.

6.10 Bus Stops

Bus travel is an increasingly important mode of mass transportation. Bus stops serve to remove the bus from the carriageway. The location and design of bus stops should provide ready access in the safest and most efficient manner practical. The location of bus stops is primarily the concern of the transport operator, who will seek to provide stops within reasonable walking distance of trip generators and attractors. The resultant bus stop spacing is normally three to four stops per kilometre in urban area. Decisions on appropriate pedestrian and bicycle facilities to connect with transit service shall be determined with input from the Transport Company, or any other concerned department and/or agencies.

The basic requirement is that the deceleration, standing and acceleration of the buses be effected on pavement areas clear of and separated from the through traffic lanes. The locations of bus stops are important so as not to impede the normal flow of traffic.

Bus bays can be designed for one or more buses. When possible, bus bays should be located on the far side of a signalized intersection, so that the traffic signal will create a gap into the traffic needed for bus re-entry.

Table 6.12: Bus Bays Dimensions for Urban Roads

Road Classification	Single Bus				Dual Bus			
	a	b	c	d	a	b	c	d
Local Roads	15	14	20	3.3	15	28	20	3.3
Arterial and Collector Roads	20	15	20	3.65	20	30	20	3.65

Note: For Each additional bus add 14 meter in Bus Bay Length

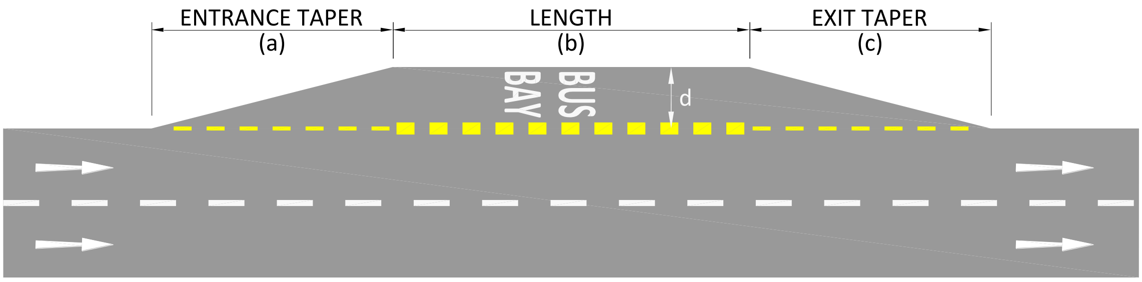


Figure 6.12: Bus Stop Layout Plan for Urban Roads

6.11 Landscaping

There are practical advantages of providing landscape treatment on medians, intersections and verges. The ground shaping, in addition to planting helps to make the alignment of road more obvious to the drivers. Hard landscaping can help in the protection of embankment. Additionally, landscaping can also act as windbreak protection, noise barrier, and reduces headlight glare. Landscaping can also be provided on retaining walls in order to avoid graffiti. The provision of plants in central island of roundabout screens the traffic on the opposite side, which are of no concern to the driver, avoiding distraction, without restricting the necessary visibility.

Grade separated Interchanges provide extensive area for landscaping due to their sheer size. By proper landscaping, the visual impact of interchanges can be improved. In rural areas, the landscaping should be limited to native species and should be related to the surrounding landscape.

The landscaping design should also follow the same criteria for sight distance as to geometric design. The following general guidelines should be noted, while designing landscaping.

- The trees, shrubs other landscaping material should not be provided in any location that will interfere with the safety and visibility of traffic.
- The horizontal and vertical lines of sight along the road, at intersections or interchanges should not be restricted by trees, shrubs or graded earth mounds.
- The plants and trees should not be planted too close to the road, such that the root system can damage the roadway structure at any time during plant's life.
- Only grass or low mature height plants should be provided in gore areas of the intersections and interchanges. The higher and denser species should only be provided outside the visibility envelope.
- In roundabouts, higher and denser plants should be limited to central island. Due to the requirement of adequate forward visibility, the central islands of roundabouts having diameter less than 10 m should not be planted.

7 BLACK SPOT MITIGATIONS

7.1 INTRODUCTION

Transport system is the major part to our life today. They also performance as substance to the accidents taking place on roads. Accident can be defined as "an unanticipated and detrimental event, a misfortune unexpected and with no obvious cause". Traffic safety can be contain of three E's example Engineering, Education and Enforcement. Road traffic crashes (RTC) and the related injuries are a major reason of death in developing countries. Really, more than 90% of RTC happened in those countries have low and middle-income. The major reasons of road accidents are road environment, vehicle condition, and road user.

Accident-prone spots are circumstances that are significantly affected by geometric design and traffic factors in accidents and that they would be reduced by means of engineering counter-measures. It may have happened for a selection of reasons, such on the straight road have sharp drop or corner, so oncoming traffic is obscured, a hidden junction on a fast road, poor or obscured warning signs at a crossroads. In order to study the speeds at the black spots accident locations, spot speed study should be conducted. The important factors considered for analysis include:

- Weather conditions
- timely variation of accidents,
- Gender wise analysis,
- Vehicle wise distribution,
- Monthly variation of accidents
- Age limit variation of accidents, and
- Daily variation of accidents,
- Monthly variation of accidents

7.2 BLACK SPOT SAFETY WORKS

Black spot safety work can be described as the task of improving road safety through alterations of the geometrical and environmental characteristics of the problematic sites in the existing road network. More specifically, this task involves targeting and treating intersections and road sections with an unusual high number of accidents, the so-called black spots.

This work may be divided into three phases:

- Targeting hot spots on the road network.
- Prioritizing the hot spots to treat with safety improving measures.
- Before and after studies of the effect of treatment.

Furthermore, there are four basic approaches to reducing crashes by applying engineering treatments or countermeasures:

- Single sites or black spots-treating specific sites or short sections of road;
- Route action-applying known remedies on a route with an abnormally high crash rate;

- Area-wide action-applying several treatments over a wide area; and
- Mass action applying a known remedy to locations with common crash problems or causal factors.

7.3 Targeting And Ranking Black Spots

The rules for targeting black spots could be based on the total number reported accidents at a site. However, road accidents are connected with a number of features, which may be considered to analyse the occurrence in variance between accident frequencies on different locations:

- The severity of the accident, e.g. fatal, injury or property damage only.
- The accident contributory factors present in the accident, e.g. ice on the road. These factors may be assigned to the driver, the location or the vehicle.
- The accident category, e.g. pedestrian, left angle, left-turn, rear-end, head-on, and various runoff-road collisions.

7.3.1 Black Spot Criteria

Highway black spots are highway locations where the potential for accidents is unacceptably high. The most common assumption for a black spot location is that there should be any road environmental or geometric issues resulting in the repetition of accidents.

7.3.2 Root Causes of Traffic Accidents in Pakistan

The root causes of the accidents in Pakistan are based on the real world data of traffic accidents, from National Highways and Motorways Police, Islamabad. The following are the main causes of severe road accidents:

- Careless driving
- Dozing at wheel
- Tire burst
- Brake failures
- Lack of training institutes
- Unskilled and uneducated drivers
- Poor road conditions
- Excessive use of cell phone during driving
- Use of intoxicants and drugs
- Overloading
- Poor implementation of Traffic by-Laws

7.4 Step By Step Procedure For Black Spot Identification

Identification

- Finding sites with highest number of accidents
- Weighing sites for severity and exposure
- Initial accident investigation and site visit i.e. detective work
- Rank sites for in depth investigation

Diagnosis

- Collection of further data from accident forms i.e. site studies
- Analysis of the data i.e. more detective work
- Human factors i.e. further site visit

Selection

- Select and package of potential counter measures
- Rank sites for treatment

Finding Countermeasures

- Implies a methodical analysis to design suitable countermeasures for each black spot, based on actual problems and deficiencies.

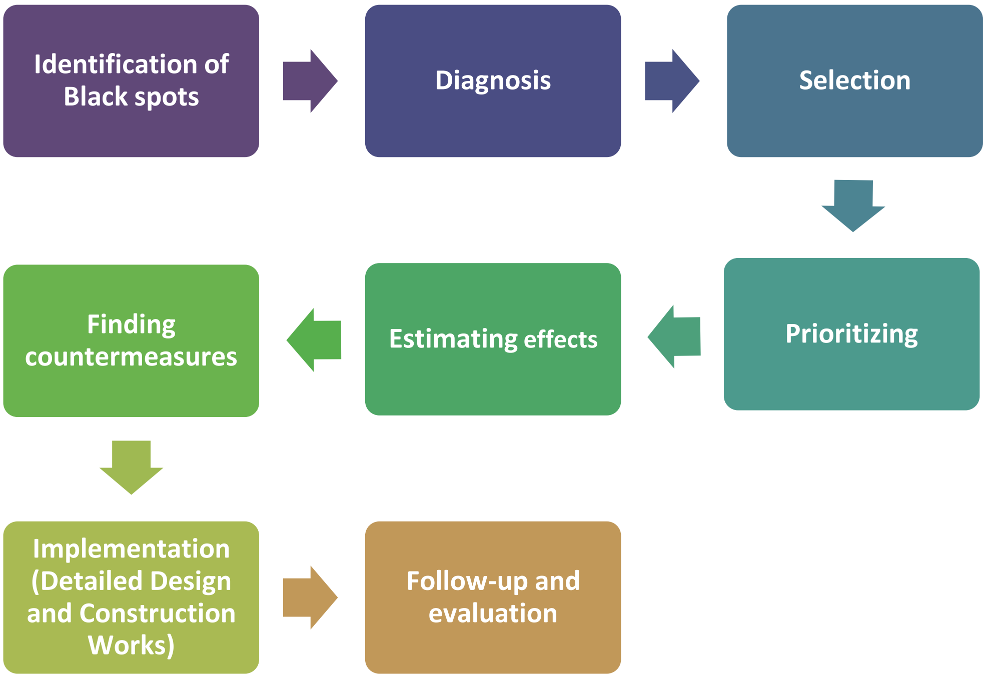


Figure 7.1: Systematic Procedure for Black Spot Identification

Estimating Effects

- The process to estimate the safety effects (and if necessary also other effects) and costs of suitable countermeasures.

Prioritizing

- Implies finding the best action plan (or investment program), according to some defined criteria, and based on estimated effects and costs as well as budget restrictions.

Implementation (Detailed Design and Construction Works)

- Monitor the behaviour during the first few days and months
- Evaluate the effects of accidents
- Cost benefit analysis

Follow-Up and Evaluation

- The last and very important step, which aim is to assess the actual results (effects and costs).

7.5 METHODS FOR EVALUATING BLACK SPOTS

7.5.1 Spot Speed Study

Spot speed analysis is essential to understand the speeds of different vehicles traveling in between a certain stretched length. Hence, we can compare the highest speeds to the design speed and hence suggest some speed control measures.

7.5.2 Site Specific Accident Analysis

Accident-prone areas data required for the analysis is as follows:

- Date of accident
- Day of accident
- Month of accident
- Time of accident
- Site visits
- Number of fatalities and injuries
- Place of accident
- Age and Gender
- Type of vehicle involved in the accident
- Light
- Atmospheric and Road conditions
- Classifying accidents and severity
- Local government area of crash

7.5.3 Development of Different Statistics for Road Crashes

- Satellite image of study area
- Area wise Statistics
- Day wise accidents Statistics
- Time wise statistics
- Gender Wise Statistics
- Statistics based on accident impact
- Monthly Statistics
- Vehicle Types Causing Accidents
- Vehicle Types Damaged by Accidents

7.5.4 Different Techniques Useful For Identification of Road Crashes

GIS is useful in identifying crash-prone areas. The information obtained is necessary in the development of strategies to reduce road traffic crashes and the associated injuries. There is scarcity of data on locational characteristics of RTC sites in most developing countries. This could be attributed to the life threatening nature of crashes which typically makes interrogating affected patients directly impossible since most of them arrive hospitals either unconscious or in pain.

Geographic Information Systems (GISs) provide tools and techniques for identifying and analysing the influence of location on phenomenon. GIS applications in road crash analysis offers data management system as well as cartographic and analytical functions in support of RTCs management GIS has been widely used in crash analysis to identify high risk neighbourhoods, areas of vehicle collision and pedestrian black spots. The growing use of GIS is based on the increasing availability of digital data coupled with the need to increase precision in the identification of crash locations while reducing time and money expended on such analysis. RTC prevention and mitigation should therefore be accorded greater attention to reduce the increasing human loss and injury.

7.6 POSSIBLE BLACK ZONES OF ROAD TRAFFIC ACCIDENTS

- Overloaded Traffic vehicles
- Inappropriate vertical curves
- Lack of traffic signs
- Inappropriate sight distance
- Lack of road shoulder
- Inappropriate and sub-standard horizontal curves
- Lack of adequate lighting at night
- Lack of interchanges at residential areas
- Lack of guardrail and passage of animals across the road

7.6.1 Mitigation of Black Spots (Black Zones)

- Traffic Signs
- Improve sight distances
- Treatment of high crash risk locations
- Road safety awareness campaigns
- Establishment of driver’s training institutions
- Improve road geometry
- Construction of emergency response centres
- Imposing heavy penalties on users
- Technological solutions for user information
- Minor road safety treatments
- Pavement Markings
- Establishment of driving licensing authority
- Enforcement of traffic laws strictly

7.6.2 Mitigation Measures at Signalized Intersections

Problem solving for intersection-related crashes:

- Provision of protected right turn facilities.
- Staggered cross intersections
- New or revised traffic signals
- Grade separation
- Conversion of un-signalized cross intersections to roundabouts
- Extension of median through intersections (turn prohibited)

Problem solving for non-intersection-related crashes:

- Road delineation
- Pedestrian facilities
- Overtaking lanes
- Central median to divide the road
- Shoulder sealing
- Removal and/or shielding of roadside hazards

7.7 METHODS OF IDENTIFICATION OF BLACK SPOTS IN LONG TUNNELS

- Breakdown with fire
- Heads-on collision (with/without secondary collision)
- Breakdown with rear end collision (with/without secondary fire)
- Single accident (with/without secondary fire)
- Rear end collision (with/without secondary fire)
- Accidents with injured persons
- Fires in the vehicles
- Accidents with casualties
- Fires in the facilities of the tunnels

The two main reasons of tunnel fires are technical faults and accidents. Technical faults include vehicle breakdowns and overheated vehicles or vehicle parts (brakes, tires, motor, etc.), in particular in tunnels with steep gradient. Another source of fires can be identified in heavy goods vehicles with loads not classified as dangerous goods, but highly flammable (e.g. new energy carriers like Ethanol and Biogas. All accident scenarios are further distinguished by the type of involved vehicles (car, HGV, bus) and a possible participation of dangerous goods.

7.7.1 Mitigation Measures of Black Spots in Long Tunnels

The following are the measures required to mitigate black spots:

- Fire resistance of the equipment
- Emergency power supply
- Emergency exits
- Fire safety in tunnels (Fixed fire fighting systems - FFFS)
- Communication systems (alarm systems, emergency telephones, loudspeakers, fire or smoke detectors, GSM coverage inside tunnels)
- Mechanical Ventilation
- Monitoring systems (CCTV, automatic incident detection and or fire detection)
- Fire hydrants (Fire extinguishers, automatic sprinklers system)
- Control centre
- Approach to tunnels with higher gradients should be reduced
- Road signs (speed controls both outside and inside the tunnel along with Pedestrians rescue and exits)
- Water supply
- Periodic inspections of Electrical, Mechanical installations and Tunnel Structures
- Emergency stations
- Lighting (normal, safety, evacuating e.g. LED to reduce maintenance and operating costs)
- Structural measures (Passive fire protection to mitigate damage to structures)
- Drainage (flammable and toxic liquids)
- Emergency walkways
- Reduction in noise and vibrations of traffic for roadside telephones
- Equipment to close the tunnels (variable message signs, traffic lights lane, etc.)
- Safety crash barriers for lay-bys inside tunnels

7.8 IMPROVEMENTS FOR ACCIDENT PRONE BLACK SPOTS

Improvements related to accident-prone locations (black spots), traffic and road engineering measures, and emergency maintenance processes mainly related to public safety and uninterrupted flow of traffic includes the following:

- Immediate removal of any debris (e.g., glass, parts of a blown tire etc.) left on the road from an accident or other incident,
- Provision of properly designed U-turns for the highway police, and emergency rescue/para-medical vehicles,
- Provision of other traffic safety devices where not installed.
- Provision of properly constructed and marked bays for the parking of highway police vehicles along the roadside,
- Provision of overhead pedestrian crossings, and
- Provision of anti-glare devices particularly in the case of New Jersey median barrier,

7.9 CONCLUSION TO AVOID BLACK SPOTS

- Lane and shoulder conditions directly affect run-off road (ROR) and opposite direction (OD) accidents. Other accident types, such as rear-end and angle accidents, are not directly affected by these conditions.
- The presence of a median has the effect of reducing specific types of accidents, such as head-on collisions. Medians, particularly with barriers, reduce the severity of accidents.
- Rates of ROR and OD accidents decrease with increasing lane and shoulder width. However, the marginal effect increasing the widths diminishes as the lane width or shoulder width is increased.
- On multi-lane roads, the more lanes that are provided in the carriageway, the lower the accident rates.
- Shoulder wider than 2.5 m give little additional safety. As the median shoulder width increase, accidents increase.
- From the limited information available, it appears that climbing lanes can significantly reduce accident rates.
- Lane width has a greater effect on accident rates than shoulder width.
- Larger accident rates are exhibited on un-stabilized shoulder, including loose gravel, crushed stone, raw earth or turf, than on stabilized (e.g. tar plus gravel) or paved (e.g. bituminous or concrete) shoulders.
- The probability of an accident on a two-lane rural road is highest at intersections, horizontal curves and bridges. The average accident rate for highway curves is about three times the average accident rate for highway tangents.
- Horizontal curves are more dangerous when combined with gradients and surfaces with low coefficients of friction. Horizontal curves have higher crash rates than straight sections of

similar length and traffic composition; this difference becomes apparent at radii less than 1000 m. The increase in crash rates becomes particularly significant at radii below 200 m. Small radius curves result in much shorter curve lengths and overall implications for crashes may not be as severe as would first appear.

- There is only a minor decrease in the speed adopted by drivers approaching curves of radii significantly less than the minimum radii specified for the design speed. However, curve radii below 200 m have been found to limit the mean speed to 90 km/h.
- The average single vehicle accident rate for highway curves is about four times the average single vehicle accident rate for highway tangents. Regarding general terrain descriptions it was found that accident rates in mountainous terrain can be 30 percent higher than in flat terrain.
- Crashes increase with gradient and down-gradients have considerably higher crash rates than upgradients. However, the overall crash implications a steep gradients may not be severe since steeper gradients are shorter. The geometry of vertical curves is not known to have a significant effect on crashes severity.
- There appears to be little erosion of safety resulting from the use of sight distances below the minimum values specified in geometric design standards, although there is a significant increase in the accident rate for sight distances below 100 m.

8 ROADWAY SIGNAGE AND MARKINGS

8.1 INTRODUCTION

Traffic signs consist of a specific message in the form of symbols or text mounted on fixed or portable posts. The main purpose of traffic signs is to provide guidance and warnings to the road user. There are other forms of signing including pavement markings, studs, traffic cones, bollards, traffic signals and other devices.

This manual contains only general information of different types of traffic signs and markings, their shapes, siting and colour. For detailed design guidelines, it is recommended to prepare a separate manual for traffic control devices containing full and comprehensive information about dimensions of all signs and text heights. Until such manual is prepared, signs available with local highway authorities and their current practice shall be used. For further guidance, *Manual of Uniform Traffic Control Devices* [1], can be referred. All sign and marking usage shall be approved by the overseeing authority. Examples of traffic signs and pavement marking applications are provided at the end of this chapter from **Figure 8.27** to **Figure 8.30**.

8.2 TRAFFIC SIGNS

In order to maximize the ability of a traffic control device to prove efficient, aspects like design, placement, operation, maintenance, and uniformity should be carefully considered. The sign should not be larger than necessary. The number of sign types should be kept to the minimum for safe and efficient functioning of the road network. Signs other than traffic signs as described in this section may only be placed within the right of way with the approval of the overseeing authority. No advertisement message or any other message not related to traffic control shall not be allowed on traffic signs or their support.

8.2.1 Classification of Signs

There are three main classes of road signs, each having different basic shape and colour. The three classes are:

1. **Regulatory Signs**
   Regulatory signs include all signs that inform road user of requirements or prohibitions on the speed, movement and waiting times of vehicles. They may be either mandatory or prohibitory. They are circular in shape and may be supplemented by plates beneath them supporting the message given by the sign.

2. **Warning Signs**
   Warning signs are used when vehicles need to warned of a potential hazard ahead. Most warning signs are triangular, sometimes supplemented by rectangular plates giving additional information.

3. **Informatory Signs**
   Informatory signs normally give road users information about the route designations and about places and facilities of particular value and interest. They are generally rectangular.

8.2.2 General Standards

8.2.2.1 Legibility

All traffic signs should be legible. The legibility is obtained by the size of lettering or symbols used, and colour contrast between lettering and symbols with their background.

8.2.2.2 Uniformity

The design of traffic signs should be uniform, including uniformity in use, shape, size, colour, lettering and reflectorization.

8.2.2.3 Excessive Use of Signs

Excessive use of regulatory and warning signs tends to lose their effectiveness. They should be used conservatively. Informative signs should be used frequently, as they keep the driver informed of their location.

8.2.2.4 Maintenance

All traffic signs shall be physically maintained to retain position, legibility and visibility. Damaged signs shall be replaced without undue. The signs face should not be obstructed by weeds, trees or shrubbery. The traffic signs should also be checked to determine if there is a need to change the sign to meet the change in traffic operations.

8.2.2.5 Illumination and Reflectorisation

All warning and regulatory signs shall be retro-reflective to show the same colour and shape in both day and night. All informative signs shall be retro-reflective and in certain instances shall be illuminated as well. All overhead signs shall be illuminated where possible.

8.2.2.6 Shapes

Standard sign shapes are as follows:

- All warning signs shall be equilateral triangular pointing upwards.
- All regulatory signs shall be circular in shape, except Give Way Sign which shall be equilateral triangle pointing downwards.
- All informatory signs shall be rectangular. Some direction signs may have one pointed end.

8.2.2.7 Colours

Different classes of signs have different distinctive colour combinations. Colours are an important aid to understanding signs. The details are provided below:

- All regulatory sign shall have a black symbol within a red border on white background.
- All warning signs shall have a black symbol within a red border on white background.
- The colour of informatory signs depends on the road hierarchy or sign function which are as follows:
    - All informatory signs on Motorways shall green background with white border symbols and text.
    - All informatory signs on National Highways and other routes shall have blue background with white border symbols and text.

- Signs for Public and private facilities (e.g. hotels, parks, museum, zoo, arts centre etc.) shall have Brown background and white border, symbols and text.
- Pedestrian route signs shall have blue background with white border symbols and text.

8.2.2.8 Dimensions

The sign sized described in subsequent parts of this chapter shall be used. Where engineering judgement determines different size than the dimensions prescribed as appropriate, then the standard shape, colour shall be used and the proportion shall be maintained as much as practical. The prescribed dimensions shall be used as minimum.

8.2.2.9 Lettering

Proper names and street names shall be transliterated in English with Urdu or Pushto Text as approved by the authority.

8.2.2.10 Sign Borders

All traffic signs panels and borders shall be rounded. The sign borders for each classification of sign are provided below:

- Borders on regulatory and warning signs shall be red and shall have the dimensions to approximate the widths illustrated for each sign.
- Information signs shall be white border.

8.2.2.11 Standardization of Location

Generally, all signs should be located at the left side of the road or highway. On wider facilities, or where the space is not available, overhead signs can be used. A supplementary warning sign may be provided on right median of divided road or right side of a one-way road, if the initial sign is not in direct line of sight of driver. Spacing between signs must be sufficient to allow time for the driver to make the required decisions safely.

Signs should be located such that they:

- are outside the clear zone,
- do not obscure each other,
- do not obstruct the sight distance of approaching vehicles at the intersection,
- are not hidden from view
- are visible in night time

8.2.2.12 Overhead Installations

On freeways and expressways to control lane-use as desirable and at locations where space is not available at roadside. In certain situations, over crossing structures, such as flyovers, elevated pedestrian crossing etc. may be used to support overhead signs eliminating the need for foundation and sign supports at roadside. The following conditions provide a basis for determining the benefit of erection of overhead sign:

- Traffic volume at or near capacity for highways with two or more lanes.
- Complex interchanges.
- Three or more lanes in each direction.
- Restricted sight distance.
- Multi-lane exits.
- Large percentage of trucks.

- Street lighting background.
- High-speed traffic.
- Consistency of sign message location through a series of interchanges.
- Insufficient space for ground mounted signs.
- Junction of one major road with another.

8.2.2.13 Height

On rural roads, signs shall be mounted at a height of minimum 1.5 m measured vertically from the bottom of the sign to the nearest edge of the carriageway. In urban areas, including central business district, commercial and residential areas where the pedestrians movement and parking are likely to obstruct the view of sign , the minimum vertical height from the bottom of sign to the nearest edge of carriageway or to the top of kerb (if provided) shall be 2.0 m.

On major roads, the advance direction signs, direction signs and confirmation signs shall be installed at a height of minimum 2.0 m measured vertically from the bottom of the sign to the nearest edge of the carriageway. Overhead signs shall provide a clearance of not less than 5.5 m. See **Figure 8.1** for further details.

8.2.3 Lateral Clearance

Signs should have the maximum practical lateral clearance from the edge of the carriageway to increase safety of motorists who may accidently runoff the road striking the sign supports. Overhead sign supports shall have a barrier or crash cushion to shield them if they are within the clear zone. Post-mounted sign and object marker supports shall be crashworthy (breakaway, yielding, or shielded with a longitudinal barrier or crash cushion) if within the clear zone.

Sign supports should not be closer than 1.8 m from the edge of the shoulder or if no shoulder exists, 3.5 m from the edge of the carriageway. In urban areas, 0.60 m is recommended as a clearance from the kerb edge. Overhead sign supports shall not be erected in gore areas or other exposed locations at exit and entry points of interchanges. In many instances, the location of the informative and warning signs can be shifted to take advantage of existing roadway devices such as guardrails or overcrossing structures to minimize the exposure of sign supports to traffic. See **Figure 8.1** for further details.

8.2.3.1 Position

- A warning sign is placed in advance of the condition that calls for attention.
- A regulatory sign normally is placed where its mandate or prohibition applies or begins.
- Informative signs are placed where needed, to keep drivers well informed as to their route and destination.

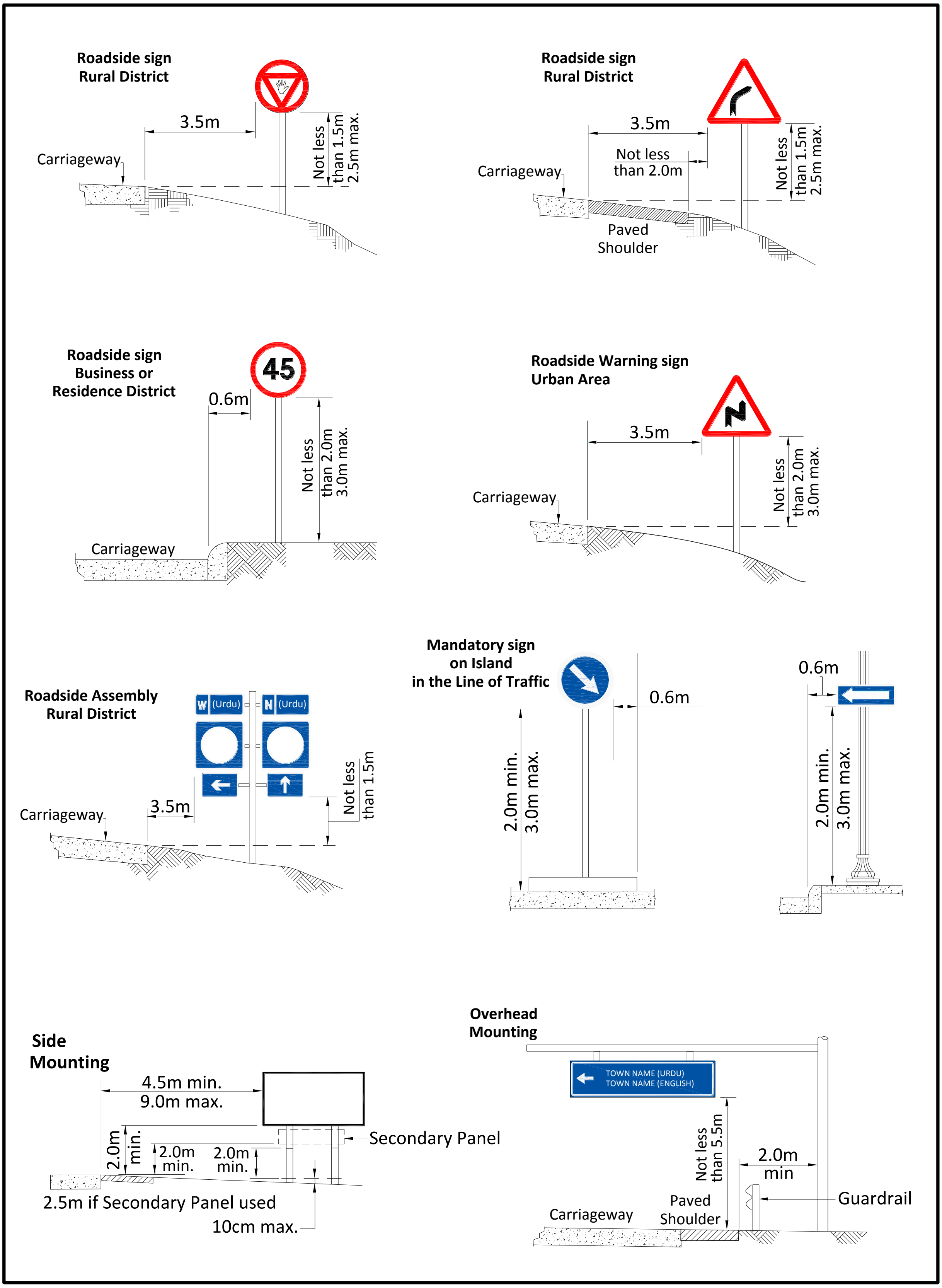


Figure 8.1: Height and Lateral Locations of Signs - Typical Installations

8.2.3.2 Orientation of Signs

Signs shall be mounted at approximately right angles to the direction of, and facing the traffic for which the signs are intended to serve, unless otherwise stated in this manual. In case where mirror reflection from the sign is faced, such that the legibility is reduced, the sign should be turned slightly away from the road. Signs that are placed more than 9 m away for road edge shall be turned towards the road. On curved alignment, the angle of sign should be determined by the direction of approaching traffic rather than from the edge of carriageway.

Sign faces may be tilted forward or back from the vertical position on grades to improve the viewing angle. **Figure 8.2** shows typical sign orientation.

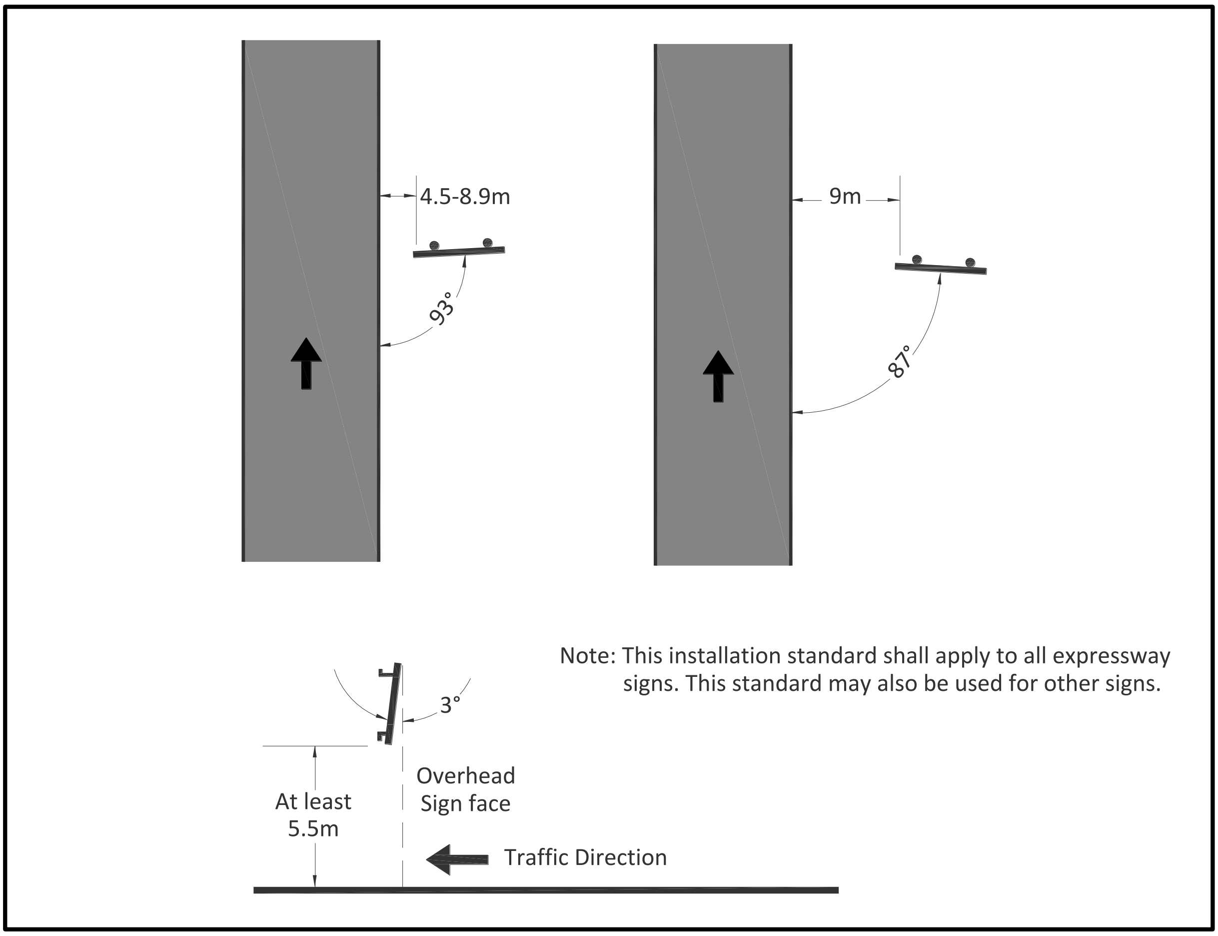


Figure 8.2: Typical Sign Orientation

8.2.3.3 Posts and Mountings

Signposts, foundation and mounting shall be able to hold signs in a proper and permanent position. Sign supports shall be of a suitable breakaway or yielding design. Overhead sign supports are non-breakaway type and shall be protected by the appropriate barriers. In urban areas, signs may be placed on supports such as traffic signals, streetlights and utility poles. **Figure 8.3** shows back of signs fixing details.

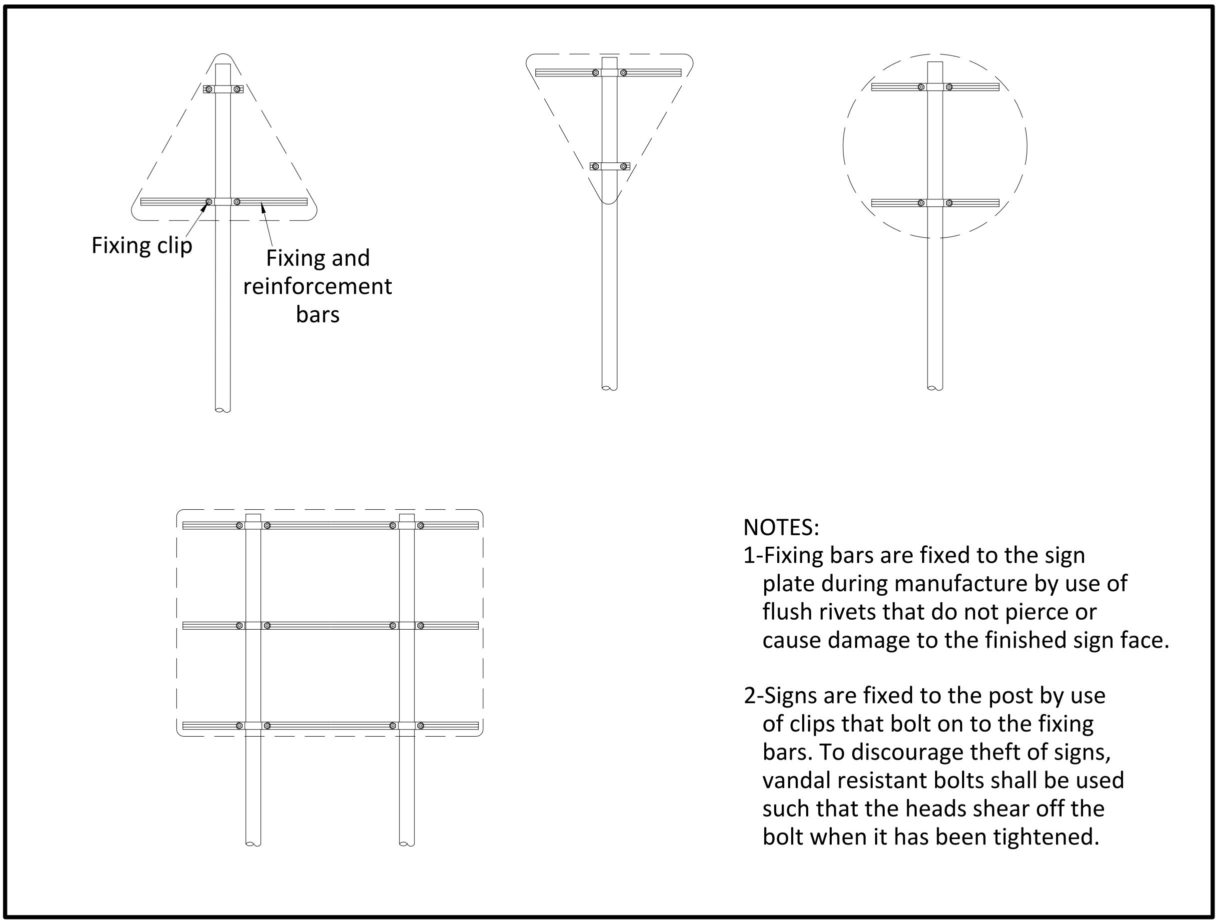


Figure 8.3: Back of Sign Fixing Detail

8.2.4 Regulatory Signs

8.2.4.1 Application of Regulatory Signs

Regulatory signs shall be used to inform drivers of selected traffic law, legal requirements, restrictions and prohibitions. They shall be installed at or near the location where regulation apply. They shall be designed and installed to provide clear visibility and legibility and clearly indicate the requirement imposed by the regulation. In certain cases, supplementary plates (rectangular) may be added to provide extra information.

Figure 8.5 and **Figure 8.6** shows the regulatory signs commonly used in Pakistan, which are also described ahead. The sizing and siting of the circular regulatory signs are shown in **Table 8.1.**

Table 8.1: Size and Siting of Circular Regulatory Signs

Speed Limit (km/h)	Functional Class	Diameter of Circular Sign (mm)	Clear visibility distance to sign (m)
30 – 60	Local	600	60
30 – 100	Collector	750	60
60 – 100	Minor Arterial	900	75
80 – 110	Major Arterial	900	90
100 – 130	Motorways / Freeways	1200	120

8.2.4.2 Stop Sign

Stop sign requires that all vehicles shall come to a full and complete stop at the transverse road marking and the driver shall not proceed until it is safe to do so. STOP signs should only be installed at junctions where the visibility criteria. They should be positioned between 1 and 3 m in advance of the road marking, as shown in **Figure 8.4.**

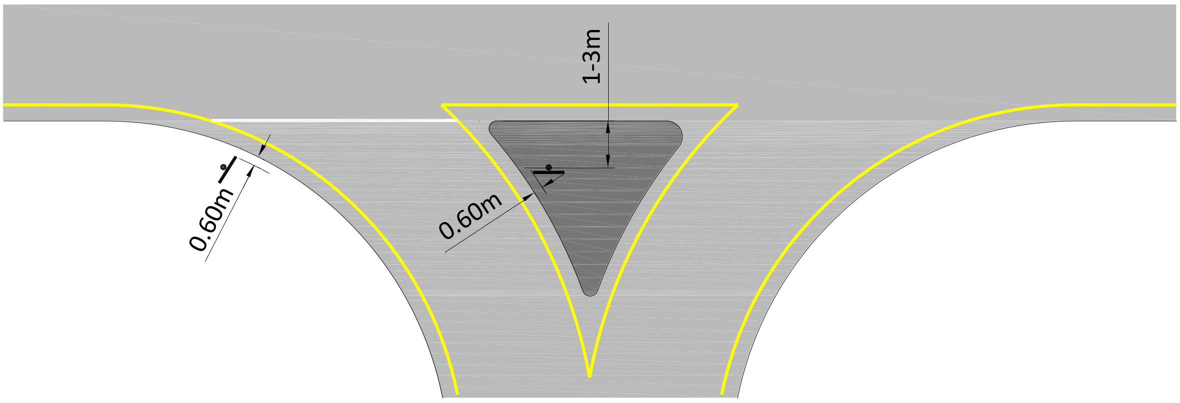


Figure 8.4: Siting of Stop & Give Way Signs

The minimum visibility below which a 'stop' sign should be provided is given in **Table 8.2**.

Table 8.2: Minimum Visibility below which a Stop Sign should be provided

Speed Limit (km/h)	Visibility Distance from Centre Line of Minor Road at 3 m Back from Edge of Major Road (m)
50	30
60	40
70	50
80	70
100	90
130	120

8.2.4.3 Give Way Sign

The Give Way Sign shall be downward-pointing equilateral triangle with red border and white background. The Give Way Sign indicates driver approaching the intersection to stop whenever necessary to avoid interference with traffic that has the right-of-way. The sign should be located between 1 m to 3 m advance of the give way road marking. An additional sign shall be provided on the central median.

8.2.4.4 Maximum Speed Limit

Speed Limit sign indicates the speed limit in km/h established by the relevant highway authority for the road on which the sign is located. The sign shall be provided after the point of access. It shall be located on both sides of the carriageway at each point where the maximum speed limit changes or at every five kilometres on longer routes with same posted-speed.

Figure 8.5: Regulatory Signs (1 of 2)

Figure 8.6: Regulatory Signs (2 of 2)

8.2.4.5 No Entry Sign

No Entry Sign indicates the drivers that the entry is prohibited by all vehicles unless supplemented by exempting certain categories of vehicles from restriction. **Figure 8.7** shows no entry for any vehicle Except Buses sign. Other permitted legends are:

- Except buses
- Except buses and taxis
- Except for access

Figure 8.7: No Entry for Any vehicle Except Buses Sign

There are other No Entry Signs that imposes restrictions only to certain kind of vehicles. Other vehicles are allowed to enter the road. Some of the examples are (See **Figure 8.5**).

- **No Motor Vehicles** - Prohibits entry by any motor vehicle except two wheeled motorcycles.
- **No Motor Cycles -** Prohibits entry by any motor cycle.
- **No Buses -** Prohibits entry by buses.
- **No Trucks (Lorries) -** Prohibits entry by any goods vehicle.
- **No Animal Drawn Vehicles -** Prohibits entry by any animal drawn vehicle.
- **No Hand-drawn Carts -** Prohibits entry by any handcart.
- **No Tractors -** Prohibits entry by any power driven agricultural vehicle.
- **No Pedestrians -** Prohibits entry by pedestrians.

8.2.4.6 Gross Weight Limit

Gross Weight Limit sign gives notification to drivers prohibiting vehicles having gross weight, in tons, greater than that indicated on sign. **Figure 8.5** shows Weight Limit Sign and Axle Weight Limit Sign.

8.2.4.7 Height Limit

This Sign prohibits vehicle exceeding the indicated height. The height indicated on the sign face should be at least 100 mm less than measured minimum clearance on the road to be regulated.

8.2.4.8 Width Limit

Width limit sign prohibits entry by any vehicle exceeding the indicated width on sign. The indicated width should be at least 200 mm less than the minimum available clear width of the road.

8.2.4.9 Length Limit

Length limit sign prohibits entry by any vehicle or combination of vehicles exceeding the indicated length on sign.

8.2.4.10 Roundabout

Roundabout sign requires that the driver shall proceed only in clockwise direction at the roundabout. This sign is located below Give Way sign in advance of roundabout entry point.

8.2.4.11 Turn Left (or Right) Ahead Only

Turn Left Ahead Only sign requires that the driver of a vehicle shall turn only to the left on the junction ahead. This sign should be placed on left side of a two-way road and right side of a one-way road at a distance approximately 50 m in advance of the junction to which it applies. The symbol can be reversed for Turn Right Ahead Only.

8.2.4.12 Turn Left (or Right) Only

Turn Left Only Sign indicates that the driver shall turn left only at the junction. This sign should be located at the far side of the roadway to which it applies facing the driver. The symbol can be reversed for Turn Right Only.

8.2.4.13 Straight Ahead Only

Straight Ahead sign requires the driver to proceed ahead only at the junction. It should be located on left side of a two-way road and right side of a one-way road.

8.2.4.14 Keep to Left (or Right)

Keep to Left sign indicates the driver to pass to the left of an obstruction in the roadway. The symbol can be reversed for Keep to Right sign. It should be positioned as near as possible to the leading edge of the obstruction. This sign typically applies to traffic islands or refuges on two-way roads or the beginning of the median strip where a single carriageway widens to become dual carriageway.

8.2.4.15 Pass Either Side

Pass either side sign indicates the driver that he can pass to either the left or right of an obstruction in the roadway. This sign should be located on traffic island near the nose.

8.2.4.16 Straight or Left (or Right) Only

Straight or Left Only Sign indicates the driver that he may proceed straight ahead or to the left only. Side road sign is used on the nose of traffic Islands within junctions where traffic may proceed ahead or to the left only. The symbol can be reversed for Straight or Right Only Sign.

8.2.4.17 Prohibition of Turning (No Left Turn, No Right Turn, No U-Turn)

Prohibition of turning signs indicates the driver that they should not turn to the left, right or make U-Turn at the junction or entrance at which the sign is displayed. The sign should be located on the side of the roadway to which the prohibition applies not more than 25 m in advance of the point where prohibition applies.

8.2.4.18 No Overtaking

No Overtaking Sign requires that the driver of the vehicle shall not overtake any other vehicle travelling in the same direction within next 500 m.

8.2.4.19 No Overtaking by Heavy Vehicles

No Overtaking by Heavy Vehicles (Truck) Sign requires that the driver of heavy vehicle shall not overtake any other vehicle travelling in the same direction within next 500 m.

8.2.4.20 No Parking

No Parking Sign prohibits parking of any vehicle at any time along a section.

8.2.4.21 End of Restriction Signs

These signs shall be placed at the locations where the specific prohibition imposed on vehicles ceases to apply. Examples of End of Restriction Signs are: (See **Figure 8.6**)

- End of Speed Limit
- End of Overtaking Restriction
- End of All Restrictions.

8.2.4.22 Motorway Signs

The Motorway signs are provided to indicate road user which road are the classified as freeways. **Figure 8.8** illustrates some of the signs used on motorways. Motorway signs usually have green background with white text, symbols and border.

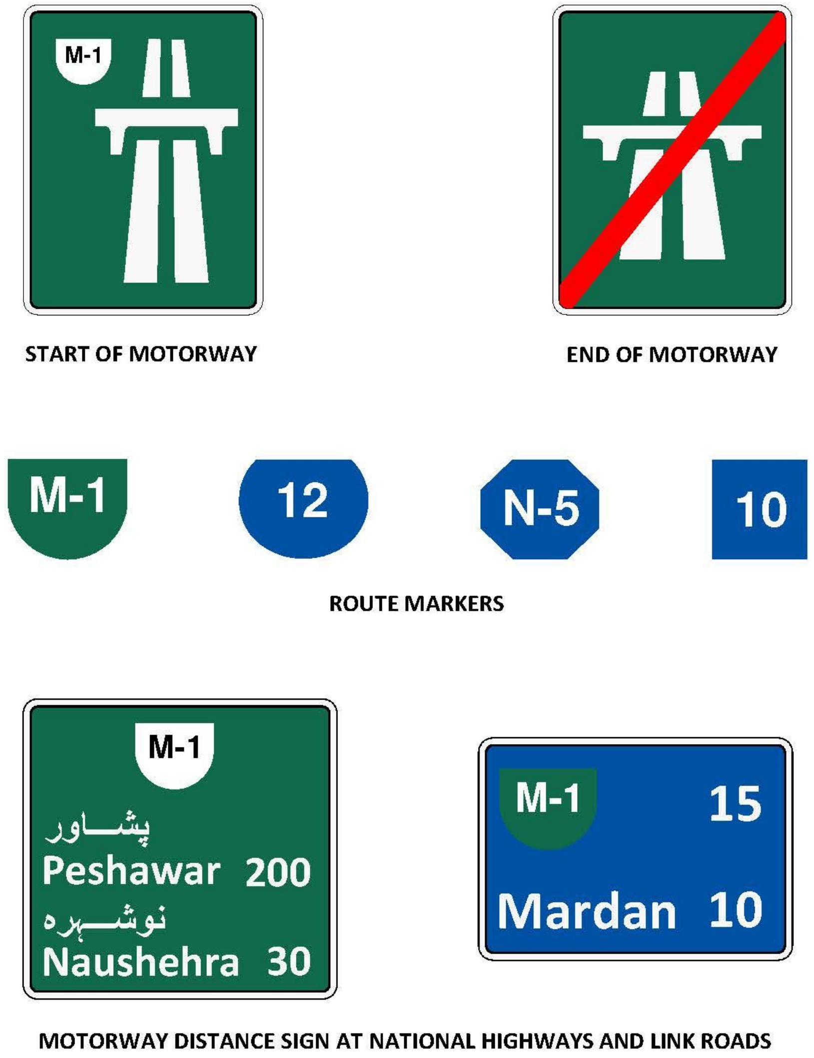


Figure 8.8: Motorway Signs

8.2.5 Warning Signs

8.2.5.1 Application of Warning Signs

Warning signs are used to alert drivers to the hazard or potential hazards to which they relate, on the road ahead. These signs are triangular having a red border surrounding a black symbol on a white background. The symbols give a pictorial indication of the likely hazard. In certain cases, supplementary plates (rectangular) may be added to provide extra Information. Warning signs require drivers to be cautious and may call for reduction in speed or a manoeuvre for their own safety or that of other drivers and pedestrians.

Temporary warning signs are also triangular having a black symbol within a red border but with a yellow background.

These signs should be kept to the minimum, and should only be provided at locations of dangerous hazards. If warning signs other than listed in this manual are required, the standard shape, sizes, colour and positioning should be followed, and shall be approved by the authority. **Figure 8.9** and **Figure 8.10** shows the warning signs commonly used in Pakistan, which are also described ahead. The sizing and siting of the warning signs are provided in **Table 8.3.**

Table 8.3: Size and Siting of Triangular Warning Signs

Speed Limit (km/h)	Functional Class	Height of Triangular Warning Sign (mm)	Desirable distance to Hazard (m)	Clear visibility distance to sign (m)
30 – 60	Local	600	50	60
30 – 100	Collector	750	75	60
60 – 100	Minor Arterial	900	150	75
80 – 110	Major Arterial	1200	200	90
100 – 130	Motorways / Freeways	1500	300	125

8.2.5.2 Left (or Right) Curve

The Curve Sign (Left or Right) is used to give advance warning of a severe bend ahead requiring caution. The sign should be located on left side of the road, in advance of a horizontal curve that can only be negotiated comfortably by reducing the speed. Engineering judgment is required, for which curves the sign should be placed. A supplementary plate may be attached below the Curve sign indicating the speed at which the curve can be safely negotiated.

8.2.5.3 Left (or Right) Reverse Curve

The Reverse Curve Sign (Left or Right) warns the driver of sharp reverse direction curves on the road ahead. The symbol direction (Left or Right) shall be according to the direction of the first curve. A supplementary plate may be attached below the Curve sign indicating the speed at which the curve can be safely negotiated. The sign should be located on left side of the road, in advance of the first curve, that can only be negotiated comfortably by reducing the speed. Engineering judgment is required, for which curves the sign should be placed.

8.2.5.4 Steep Grade Signs

Steep Grade signs are used in advance of upgrade or downgrade, where the length and percent of grade and other geometrical and physical features require cautions by drivers.

8.2.5.5 Two Way Traffic

This sign warns the driver traveling on one-way road that the roadway ahead carries the two-way traffic.

8.2.5.6 Junction Ahead Signs

Junction ahead signs warn the driver traveling on major road of a junction ahead and indicate both the layout of the junction and the priority route through it. The sign should be located on left side of the roadway in advance of the point of junction. A supplementary plate may be attached below the junction ahead sign indicating the distance to the junction. The sign is only provided on priority routes. **Figure 8.9** illustrates different types of junction ahead signs.

Figure 8.9: Warning Signs (1 of 2)

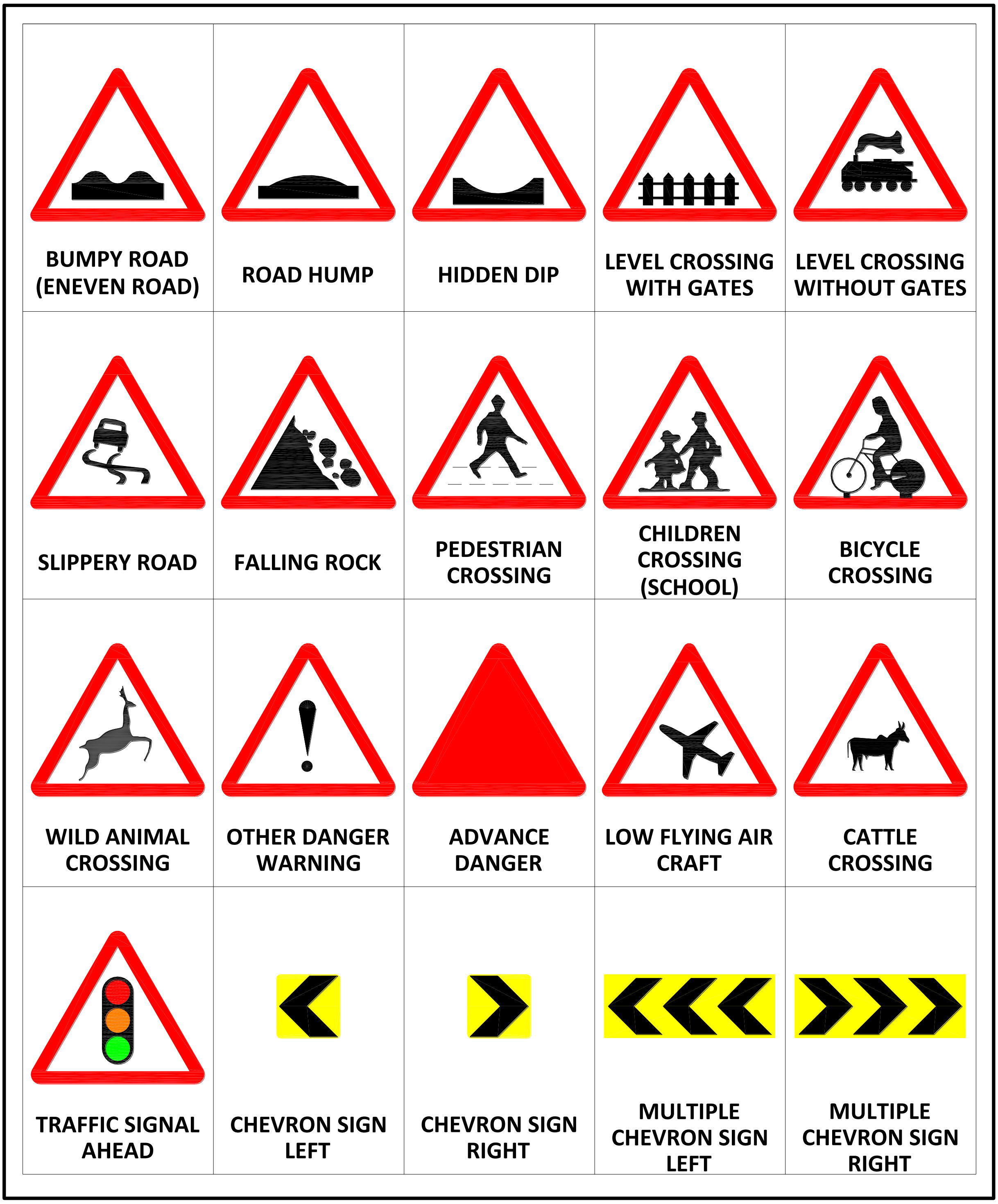


Figure 8.10: Warning Signs (2 of 2)

8.2.5.7 U-Turn Ahead

U-Turn Ahead sign indicates the driver that they are allowed to make U-turn at the junction or median opening ahead. The sign should be located on the median of the divided roadway. A supplementary plate may be attached below the sign indicating the distance to the U-turn.

8.2.5.8 Roundabouts Ahead
Roundabout sign is used to warn drivers of their approach to a roundabout. The sign is located on left side of the roadway in advance of the roundabout. A supplementary plate may be attached below the sign indicating the distance to the Roundabout.

8.2.5.9 Road Narrows Ahead (From Left, Right or Both Sides)
Road Narrow signs are used to warn of reduction in carriageway width from the left, from the right or from both side. The sign should be located on left side of a two-way roadway and, where practical, on both sides of a one-way roadway.

8.2.5.10 Other Warning Signs
Some other warning signs are also illustrated in **Figure 8.9** and **Figure 8.10.**

8.2.5.11 Hazard Markers

Single Chevron Sign (Left or Right)
The chevron sign is intended to provide additional emphasis and guidance to the driver for change in horizontal alignment. It is located at the actual position of the hazard. At a sharp bend in the road where single sign would not be sufficient to convey to the motorist the severity of the bend chevron sign can be provided. If the angle of the bend exceeds 90°, more than one sign may be used.

Multiple Chevron Sign (Left or Right)
Multiple chevron sign warns the motorist of the actual position of a very sharp bend or change in the direction of curve. They comprises of minimum three chevron modules. The multiple Chevron Sign is usually provided at roundabout on central island opposite each approach. **Figure 8.11** illustrates the placement of multiple chevron sign at sharp bends and at roundabout.

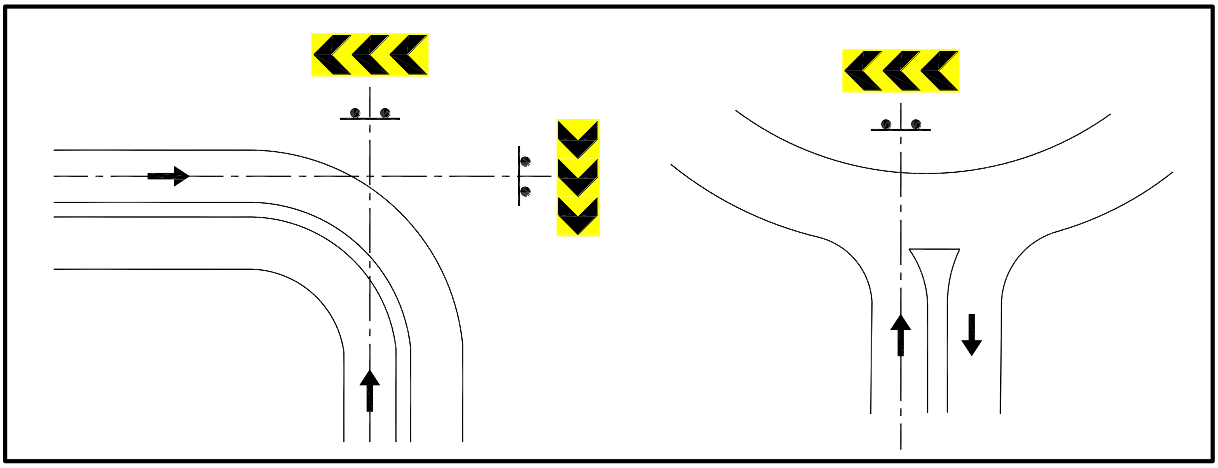


Figure 8.11: Placement of Roundabout / Sharp Curve Chevron Sign

8.2.6 Informatory Signs
Informatory Signs provide information to drivers in the form of symbols and/or texts varying in sizes according to the posted speed of the road. These signs are generally rectangular with white text and border on a blue background.

8.2.6.1 The Direction Signing System
Following functions are performed by the direction signing system:-

1. It provides advance warning to drivers of their approach to a junction.

2. It indicates the type of junction that will be negotiated.
3. It informs of the destinations that may be reached from each exit.
4. It indicates the point of turn to reach a specific destination.
5. It identifies the route and its status within the network.

Directional informatory signs can be further classified as follows:

1. **Advance Direction Signs** – provides route information with respect to the junction ahead.
2. **Direction Signs** - placed at a junction pointing along specific routes.
3. **Route Confirmatory Signs** - placed after a junction confirming the route being followed. It may indicated the destination that can be reached and the distance to the destination.

The lettering sizes of all the direction signs are shown in **Table 8.4.**

Table 8.4: Lettering Sizes for Directions Signs

Speed Limit (km/h)	Functional Class	Type of Sign	Siting distance (m)	Aleph height (mm)	"x" height (mm)	Minimum clear visibility to sign (m)
30 – 60	Local	Advance Direction Sign	45	180	100	60
		Direction Sign	-	135	75	45
		Confirmatory Sign	-	135	75	45
30 – 100	Collector	Advance Direction Sign	90	225	125	75
		Direction Sign	-	180	100	60
		Confirmatory Sign	-	180	100	60
60 – 100	Minor Arterial	Advance Direction Sign	150	270	150	105
		Direction Sign	-	225	125	75
		Confirmatory Sign	-	225	125	75
80 – 110	Major Arterial	Advance Direction Sign	225	360	200	135
		Direction Sign	-	270	150	105
		Confirmatory Sign	-	270	150	105
100 – 130	Motorways / Freeways	Advance Direction Sign	1 Km & 2 Km	540	300	180
		Direction Sign	-	450	250	135
		Confirmatory Sign	-	450	250	135
	Pedestrian			90	50	Not Applicable

Advance Direction Signs

Advance Direction Signs are normally provided on all approaches to a junction to provide driver adequate advance warning of the junction ahead, and the destinations that may be reached from each exit. Either they can be post mounted or gantry mounted. Advance Direction Signs can be of three types:

- Map type,
- Stack type and
- Dedicated lane signs

Advance Direction Signs for Interchanges

Two Advance Direction Signs should be provided at approach o interchange ramps. The first, the Far Advance Direction Sign, is located 1 km before the exit ramp. It is normally ground mounted sign, or gantry mounted if the verge width is limited.

The second, the Advance Direction Sign, is located 500 m before the exit ramp. It is normally gantry mounted. **Figure 8.12** shows the layout of advance direction signs for interchanges.

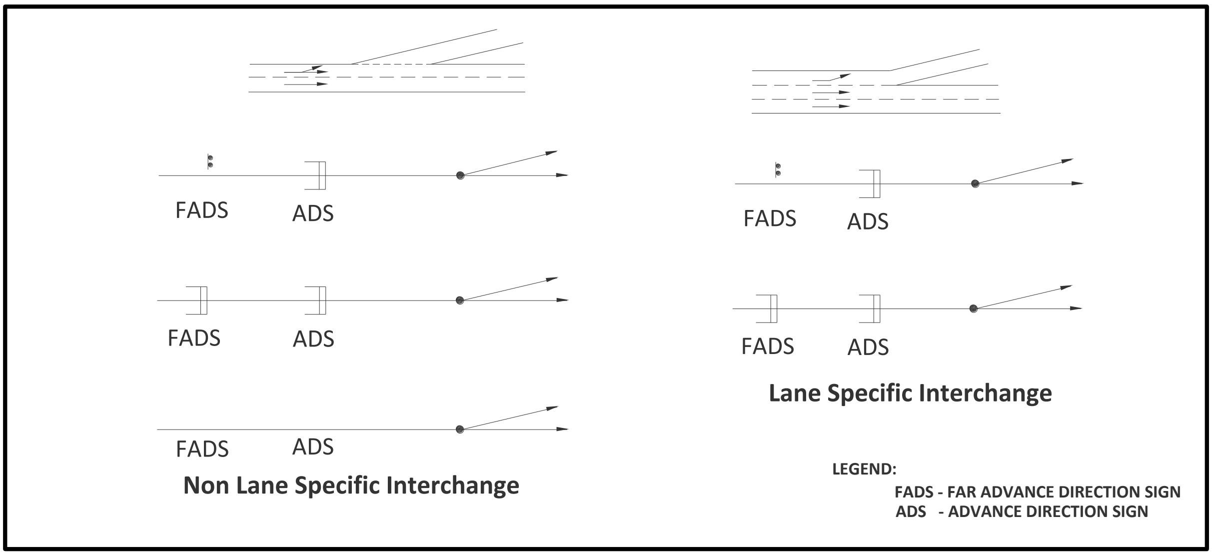


Figure 8.12: Advance Direction Signs (ADS) for Interchanges

Advance Direction Signs for At-Grade Junctions

Only one Advance Direction Sign is provided on each approach of At-grade junctions. 'Stack' or 'Map' type signs displaying the destination(s) together with the route number (where applicable), are provided depending on the configuration of the junction. They are located at a maximum distance of 225 m from the junction.

Stack Type Signs are provided in advance of simple priority or signal controlled junctions, with arrows indicating the direction to reach various destinations. **Figure 8.13** illustrates example of Stack Type Signs.

A Map Type Advance Direction Signs are provided in advance of an at-grade roundabout or complex priority junction. The map symbol indicates the layout of junction showing exits in correct orientation with respect to the approach, as near as possible. **Figure 8.14** show typical map type signs.

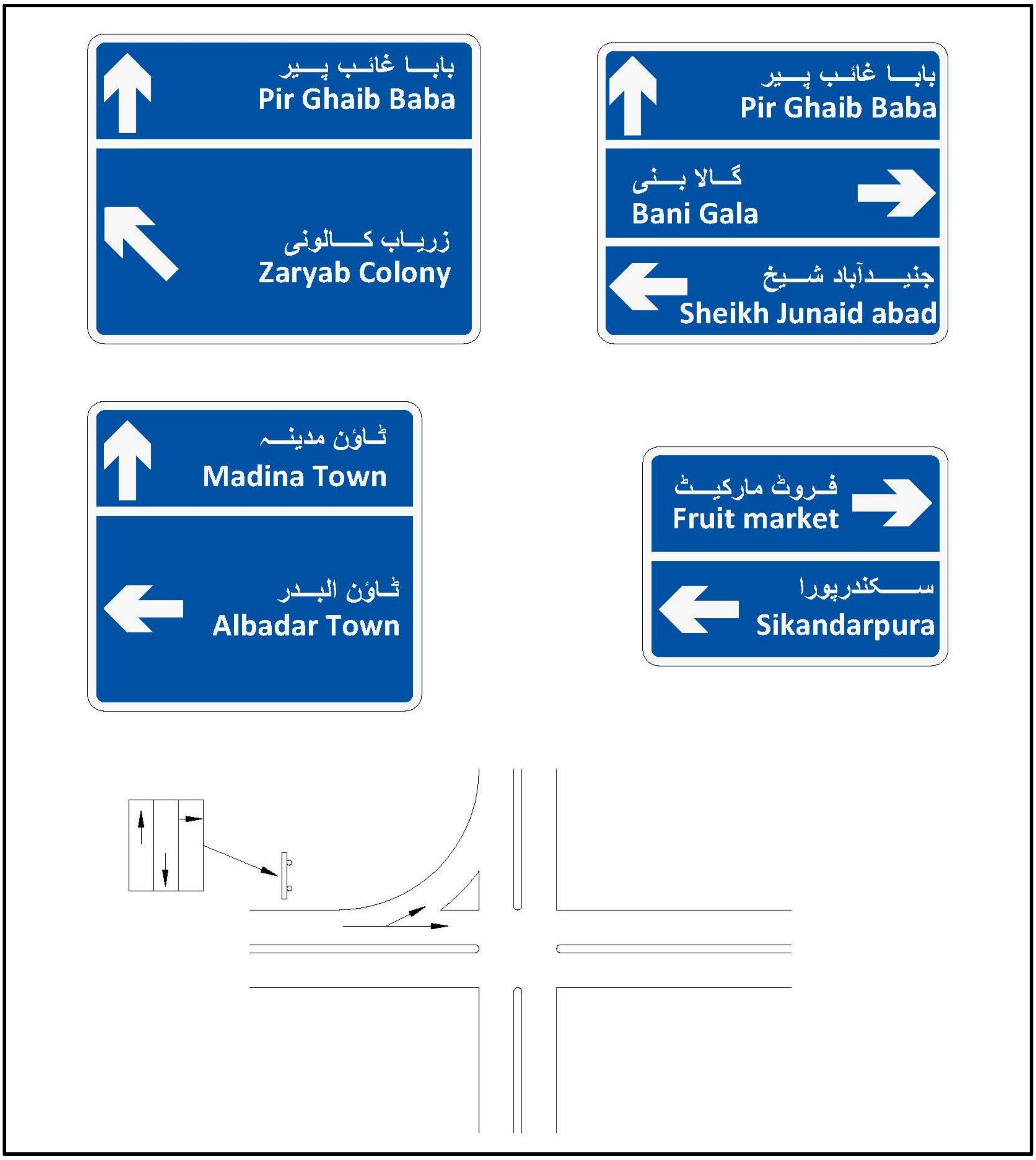


Figure 8.13: Stack Type Sign

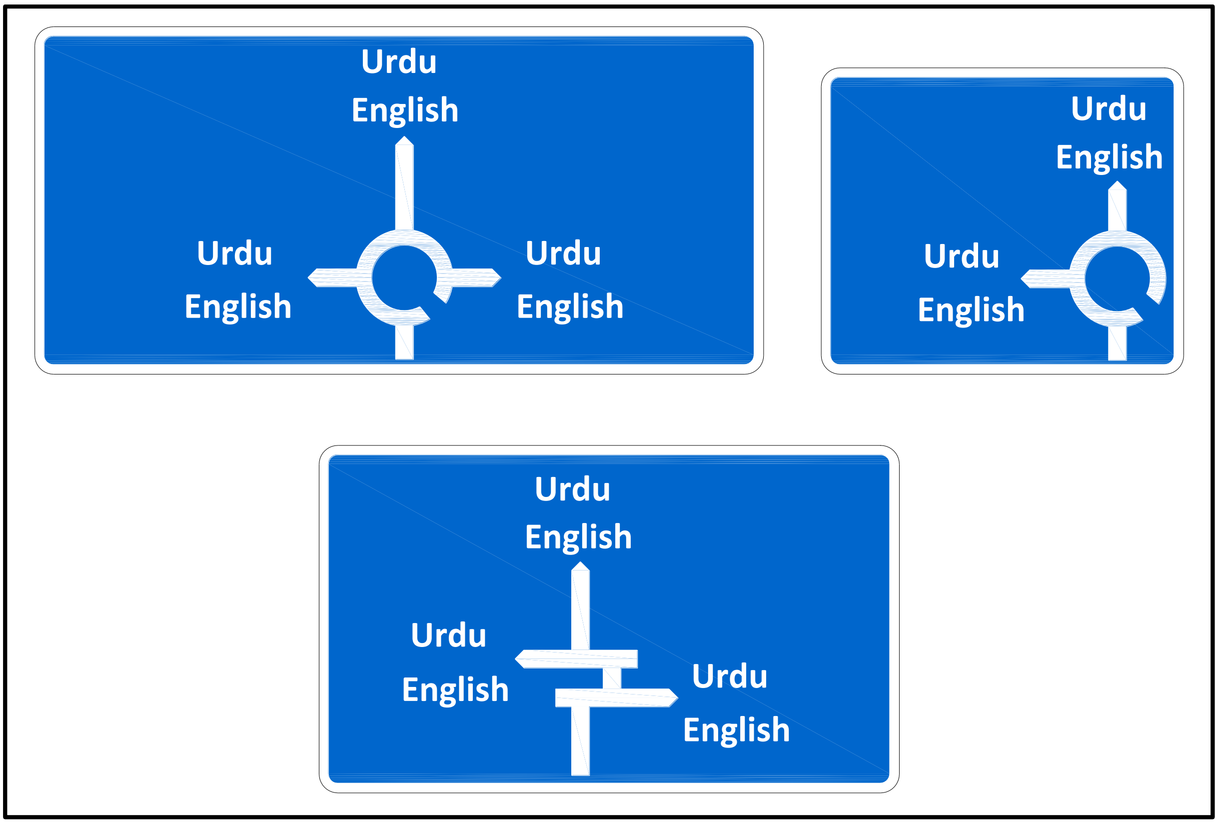


Figure 8.14: Typical Map Type Signs

Direction Signs

Direction Signs for Interchanges

Direction Signs for interchanges can be Non-Lane Specific or Lane Specific. Non Lane Specific signs are provided where the number of through lanes remains constant. They are provided at interchanges where it is impractical to gantry signs. They may be substituted by Flag Type signs.

Lane Specific Signs are generally provided where the number of through lanes decreases. **Figure 8.15** shows different types of interchange direction signs.

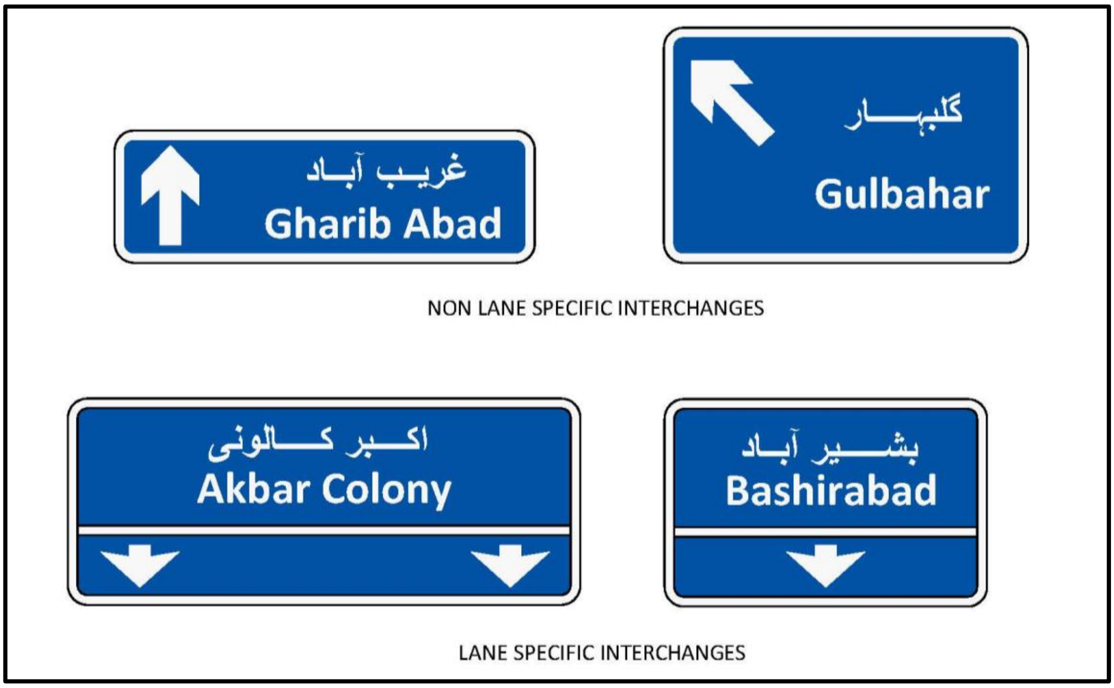


Figure 8.15: Direction Signs for Interchange

Direction Sign for At-Grade Junctions

Flag Type Sign is provided at each exit of an at-grade junction, irrespective of type. **Figure 8.16** shows an example of flag type sign.

Figure 8.16: Flag Type Sign

Route Confirmatory Sign

Route confirmatory signs are provided on major arterials and collector roads and on all exits of major junctions on these routes. It displays the destination as shown in the previous Advance Direction Sign and the associated distance to the nearest kilometre. **Figure 8.17** shows Route Confirmatory Sign.

Figure 8.17: Route Confirmatory Sign

8.2.6.2 One Way Street

One Way Street Signs is located on the either side of the roadway at the point of entry of a one-way street. Repeater signs should be provided on alternate sides, along the length of the street, at a distance not more than 50 m. Additional sign should also be provided immediately downstream of the junction through which traffic enters the one-way street. **Figure 8.18** shows One Way Road sign ahead, One Way Road left side and One Way Road right side.

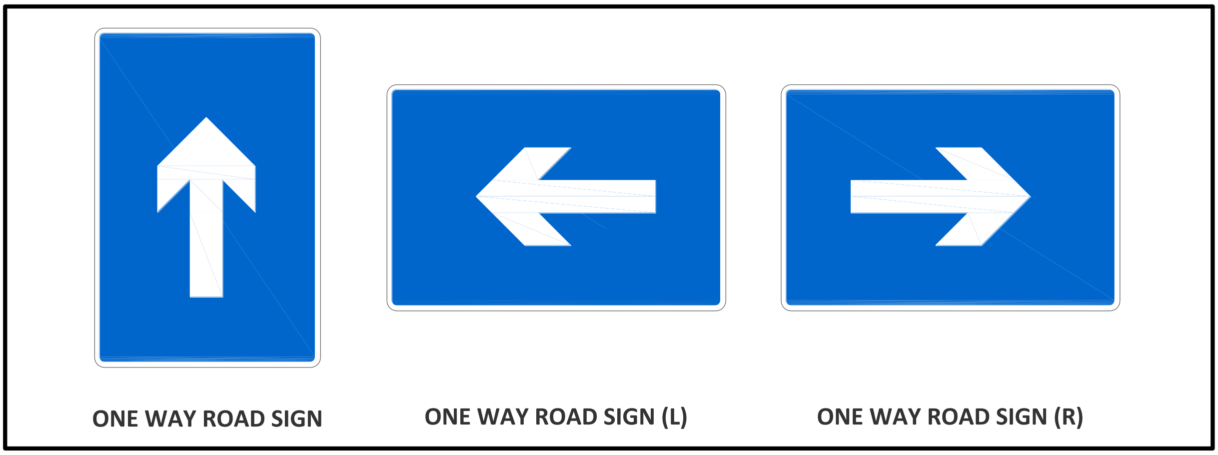


Figure 8.18: One-Way Street Signs

8.2.6.3 Town or Area name

Town or Area Name Sign at the beginning and end of the town are provided to inform the drivers the name and limits of the town they are entering or leaving. The sign should be located at the start of build-up area rather than on the geographical boundary. **Figure 8.19** illustrated the example of Town or Area Nam Signs.

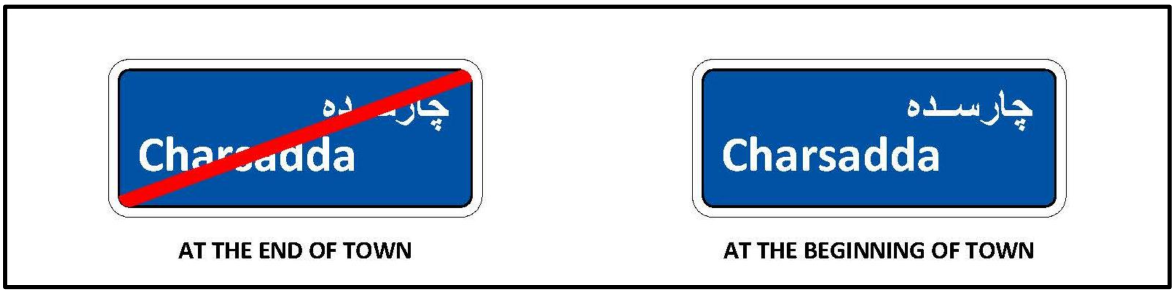


Figure 8.19: Town or Area Name Sign

8.2.6.4 Lane Discipline

Land discipline sign is provided in urban areas in advance of a junction to indicate which lanes are available for particular traffic movements. The number of lanes shown on the sign corresponds with the number of lanes marked on the carriageway at the point where the sign is located. **Figure 8.20** shows lane discipline sign.

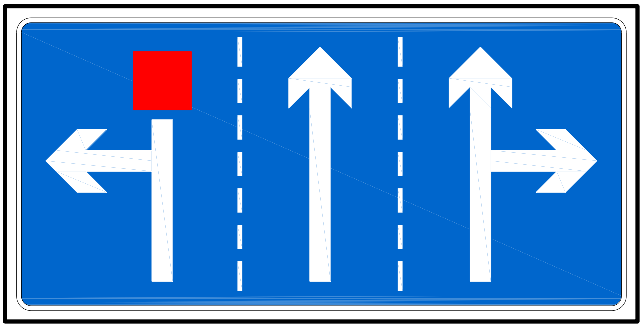


Figure 8.20: Lane Discipline Sign

8.2.6.5 Parking

Parking sign is used to indicate a parking place for vehicles; whether it is on-street parking bays or off-street parking places. The plate may be extended to include the distance or a direction arrow below the text. Alternatively, the symbol may be incorporated in directional signing. **Figure 8.21** shows parking signs.

Figure 8.21: Parking Signs

8.2.6.6 Other Informatory Signs

Some other examples of informatory signs, such as hospital, hotel, and restaurants, auto repair shop, phone, petrol pump, bus stop and mosque signs are shown in **Figure 8.22**. These signs are provided in advance of the facility and additional sign at the point of facility. The sign may be supplement by an additional plate showing the distance and/or direction arrow to the facility.

Figure 8.22: Other Informatory Signs

8.3 ROAD MARKINGS

8.3.1 Introduction to Road Markings

Road markings may be used in combination with road signs to supplement the information given by the sign, signals or on their own to provide information that cannot be properly conveyed by signing. They must function day and night under adverse weather condition. Pavement markings should not be overused. Their maintenance and replacement is necessary, as they are much prone to rapid deterioration.

8.3.2 Marking Materials

Most commonly used material for pavement and curb markings is thermoplastics; however, other suitable marking materials, (used less frequently) including raised pavement markers and coloured pavements, are also used. All markings on all highways shall be retro-reflective.

8.3.3 Colour

All pavement markings shall be either whiter or yellow. In general, the colour of markings shall be as under:

- Yellow shall be used for warning markings, to mark left and right edge of carriageway of a divided or one way road, dividing line on a two way single carriageway road, or any longitudinal line separating the opposing direction of traffic.
- White shall be used for regulatory markings and any other longitudinal line.
- All symbols shall be white.

8.3.4 Functions, Widths, and Patterns of Longitudinal Pavement Markings

The general functions of longitudinal lines shall be:

- A double line indicates maximum or special restrictions,
- A solid line discourages or prohibits crossing, except in case of emergency,
- A broken line indicates a permissive condition, and
- A dotted line provides guidance or warning of a downstream change in lane function.

The widths and patterns of longitudinal lines are generally as follows:

- Normal line — 0.1 to 0.15 m wide.
- Wide line — at least twice the width of a normal line.
- Double line — two parallel lines separated by a visible space.
- Broken line — normal line segments separated by gaps usually in the ratio of 1:2.
- Dotted line — noticeably shorter line segments separated by shorter gaps usually in the ratio of 1:1 than used for a broken line. The width of a dotted line extension shall be at least the same as the width of the line it extends.

8.3.5 Classification of Pavement Marking

Pavement markings can also be further classified as:

- Transverse Lines
- Longitudinal Lines
- Symbols

Transverse Markings

These markings are set across the carriageway and are associated with traffic control by sign or Signal.

Longitudinal Markings
Longitudinal lines run parallel to the roadway. The purpose of longitudinal lines is to convey a continual message to the driver over an extended length of roadway.

White longitudinal lines shall delineate the separation of traffic flows in the same direction, while yellow longitudinal lines shall delineate the separation of traffic traveling in opposite directions and the left and right edges of carriageways of undivided or divided roads.

Symbols
Symbolic marking convey the message using a single or grouped set of markings to indicate the present condition. The examples of symbolic markings are lane and turn arrows. All markings shall be white except mentioned otherwise.

8.3.6 Transverse Markings

The different types of pavement markings are illustrated in **Figure 8.23** and **Figure 8.24**. These figures shall be seen in conjunction with following description.

8.3.6.1 The Stop Line

Stop line imposes a mandatory requirement to all vehicles that the driver must stop completely behind that line. This marking is always used in conjunction with Stop Sign on priority intersections and or traffic signal on signalized intersection. It is a continuous solid white line marked across the full width of the carriageway that is controlled by Stop Sign or traffic signal.

The standard width of stop line shall be 300 mm and it shall be positioned at the location best suited for safe stopping and clear view of conflicting traffic. Preferably, it should be 1 m in advance of and parallel to the near edge of intersecting roadway. If pedestrian crossing provided, the stop line shall be located at least 1 m in advance of the crossing.

8.3.6.2 The Give Way Line

Give Way marking imposes a mandatory requirement that the driver shall, when in conflict, stop at the line and give way to the vehicular and/or pedestrian traffic. This marking shall always be used in conjunction with Give Way sign or Give Way to Pedestrian Sign. The marking shall be broken white line marked across the full width of the carriageway that is controlled by Give Way sign or Give Way to Pedestrian Sign. The width of Give Way line and its position with respect to the edge of the major carriageway is the same as that of (STOP) marking.

8.3.6.3 Pedestrian Crossings Markings

Pedestrian Crossing marking imposes a mandatory requirement that a driver must give way to pedestrians crossing the roadway. This marking may be used in conjunction with Give Way to Pedestrian Sign. Pedestrian Crossing marking shall be white broken line marked across the full width of the carriageway and shoulders. The configuration of the line shall be 500 mm line with 500 mm gap. The minimum width of the line, perpendicular to the axis of pedestrian travel shall be 3 m, and may be increased for large volume of pedestrians. The use of Pedestrian Crossing marking should be carefully considered based on engineering study and judgement.

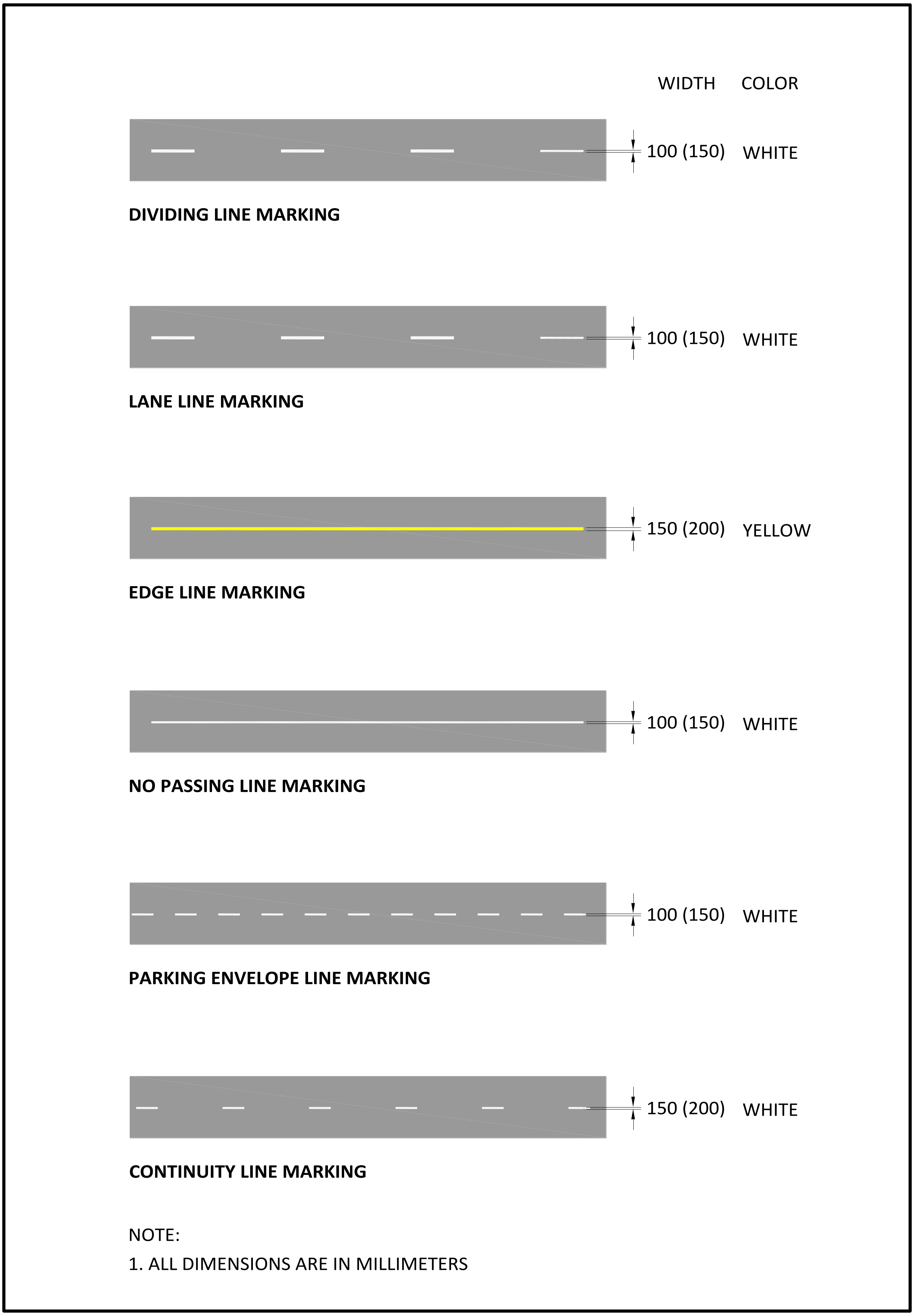


Figure 8.23: Pavement Markings Details-1

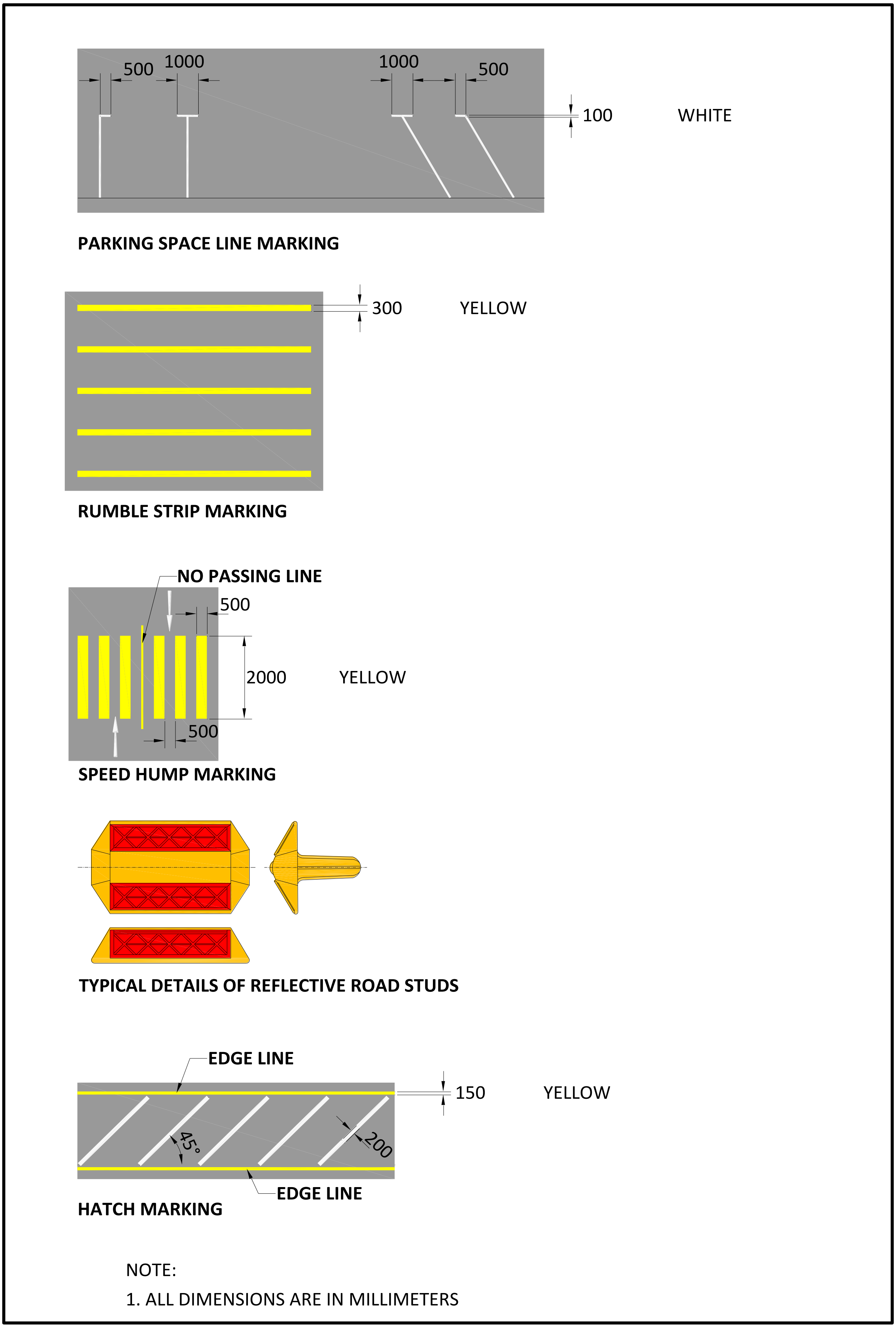


Figure 8.24: Pavement Markings Details-2

8.3.7 Longitudinal Markings

8.3.7.1 Centreline or Dividing Line Marking

Centreline or dividing line pavement marking is used to separate the traffic travelling in opposite directions. Vehicles may cross this line for passing only when it is safe to do so.

On two lane roads, it shall be broken white line. The line shall be 100 mm wide for posted speed below 60 km/h and 150 mm wide for posted speeds of 60 km/h or above. Where passing is permitted only in one direction, double line shall be provided consisting of broken line on side from where passing is permitted and a continuous line on the other side. When passing is prohibited in both directions, the line shall be continuous white line.

On undivided highways with four or more lanes, a double yellow continuous line shall be provided.

8.3.7.2 Lane Line Marking

Lane lines separate lanes is used to separate the traffic travelling in the same direction and allows changing of lanes, where it is safe to do so. . Lane line is provided on all multi-lane highways. It shall be broken white line, running continuously on the line separating two lanes travelling in same direction. The line shall be 100 mm wide for posted speed below 80 km/h and 150 mm wide for posted speeds of 80 km/h or above. This line shall be converted to channelizing line where changing lane is not permitted.

8.3.7.3 Edge Line Marking

Edge line marking is used to define the edge of carriageway. Edge line marking imposes the mandatory requirement to the vehicle that the driver shall not cross the line except in case of emergency. It is also used to separate the carriageway from shoulder. It is a longitudinal solid yellow line running continuously along the edge of carriageway. The line shall be 150 mm wide for posted speed below 80 km/h and 200 mm wide for posted speeds of 80 km/h or above. This line shall not be continued through intersections.

8.3.7.4 No-Passing Line Marking

No passing line imposes a mandatory requirement that the driver shall not cross the line. It is used to replace the dividing line or used with dividing line to prohibit passing on two way roads at approaches to controlled junctions and roundabouts (Signal, Stop, Give Way or Pedestrian), on sharp horizontal and vertical curves and at areas where inadequate sight distance is available. The line shall be continuous solid white line. The line shall be 100 mm wide for posted speed below 60 km/h and 150 mm wide for posted speeds of 60 km/h or above.

8.3.7.5 Channelizing Line

Channelizing line when used in place of Lane Line, imposes a mandatory requirement that the driver shall not change the lane. Channelizing line shall only be used between the lanes of traffic traveling in the same direction. The line shall be continuous solid white line. The line shall be 100 mm wide for posted speed below 60 km/h and 150 mm wide for posted speeds of 60 km/h or above. It shall be used in advance of traffic signals, roundabouts or curved roadway sections.

8.3.7.6 Continuity Line Markings

Continuity Line marking may be provided, for guidance to through traffic, at discontinuities in the delineation of pavement edge. Its use is optional, and should only be considered on long breaks in the marking of edge line. The continuity line shall be longitudinal broken white line, following the edge of through lanes through an exit ramp or an intersection. The line shall be 150 mm wide for posted speed below 80 km/h and 200 mm wide for posted speeds of 80 km/h or above.

8.3.7.7 Parking Envelope Line and Space Line Markings

Parking Envelope Line imposes the mandatory requirement that the driver parking their vehicles in the marked area shall park such that no part of their vehicle shall encroach outside the Parking Envelope line. Parking Envelope Line shall be broken white line 100 mm wide. It shall be located at the edge of pavement adjacent to parking permitted areas where width of parking area is at least 2.2 m or more.

Parking Space Line marking requires that the driver should park their vehicle such that the vehicle is completely within the marked area of parking space. Parking Space Lines shall be solid white lines 100 mm wide.

8.3.7.8 Yellow Box Junctions Markings

Box Junction marking is used at junction where there is a chance that the stationary traffic could block the junction. It warns the driver that they much not enter the painted box unless the exit form it is clear. The junction is marked with yellow cross-hatched lines enclosed in a box, as shown in **Figure 8.25.**

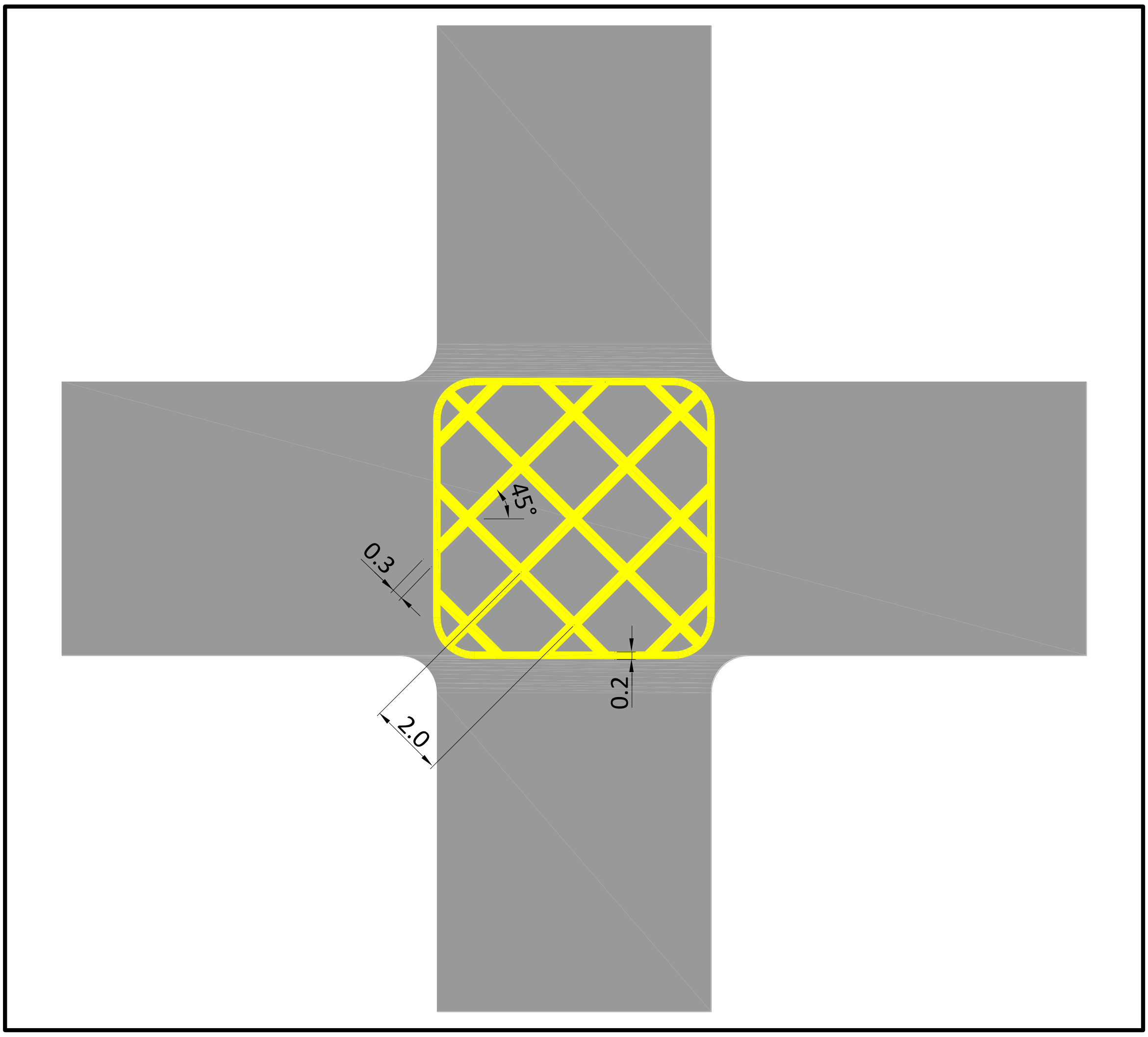


Figure 8.25: Box Junction Markings

8.3.7.9 Lane Direction Arrow Markings

Lane Direction Arrow markings, as shown in **Figure 8.26**, imposes a mandatory requirement that the drivers, if they are in lane marked by an arrow must travel in the direction indicated by the arrow, otherwise the driver is required to change lane, if not prohibited by channelizing line. All lanes at the approach to a roundabout or junction shall be marked by one arrow, positioned at the center of lane, in line with one another, and shall be located as close to the junction as possible. Additional Lane Direction Arrow markings shall be located in advance of this row of marking.

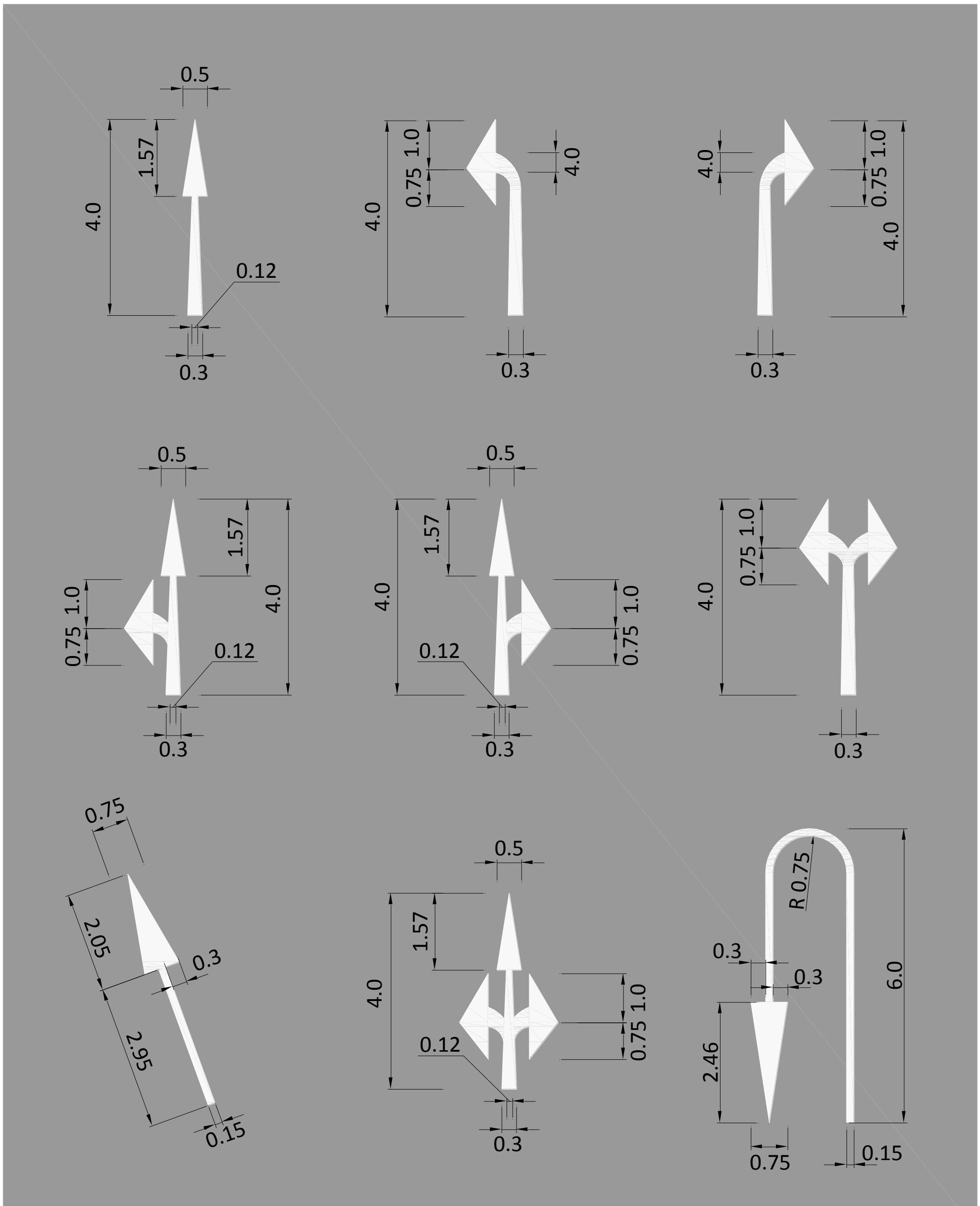


Figure 8.26: Pavement Arrow Markings Details

8.3.7.10 Rumble Strips

Rumble Strip markings warn the driver to the presence of critical regulatory or warning device, through visibility, sound and vibration. Rumble strips are transverse warning pavement markings provided to ensure that the drivers reduce their speed. They are applied to the pavement surface in sets of 5 transverse lines. The distance between each set should equivalent to 1 to 2 seconds travel as speed reduces. These lines are made of yellow thermoplastic material and build-up at least 5 mm above the roadway surface, but not greater than 15 mm. The width, spacing and number of RUMBLE STRIPS used in a single installation may vary to suit conditions.

8.3.7.11 Speed Hump Markings

This marking warns the motorist of the position and extent of a speed hump on the roadway. They are yellow broken lines transverse across the width of roadway, including shoulder. The lines shall be 500 mm with 500 mm gap. The minimum width of the line shall be 2 m. No passing line or channelizing line shall be used on each approach to the sped hump for an appropriate distance.

8.3.7.12 Chevron Markings

Chevron marking is provided to warn the drivers of a divergence or merging of traffic lanes in the same direction. They consists of diagonal white lines arranges in chevron pattern. Chevron points towards the oncoming vehicles and the diverging legs point towards the downstream. Application of Chevron marking at exit and entrance ramp can be seen in **Figure 8.29** and **Figure 8.30**.

8.3.7.13 Hatch Markings

Hatch marking is provided to warn the drivers to stay clear of a physical danger adjacent to their travel lane. It consists of repeated diagonal white or yellow lines. Hatch lines are oriented such that if they were raised barriers, they will reflect the traffic back into the lane from which they came. Common application of hatch marking is in median that separates traffic moving in opposite direction, e.g. from a two-way road to divided road. It may also be used to mark the shoulders at sharp horizontal curves or roadside obstruction.

8.3.7.14 Text and Symbol Markings

Text and symbol marking may be provided of the purpose of emphasising the existing regulatory, warning or guide road sign or pavement marking. These markings are white in colour. Large letters, symbols and numerals should be used. Symbol messages are generally preferable to word messages. Where speeds are low, smaller characters may be used. Common application of text and symbol marking is STOP Symbol marked at the centre of pavement accompanied by Stop Line and Stop Sign.

Figure 8.27: Traffic Signs and Road Markings at Roundabout

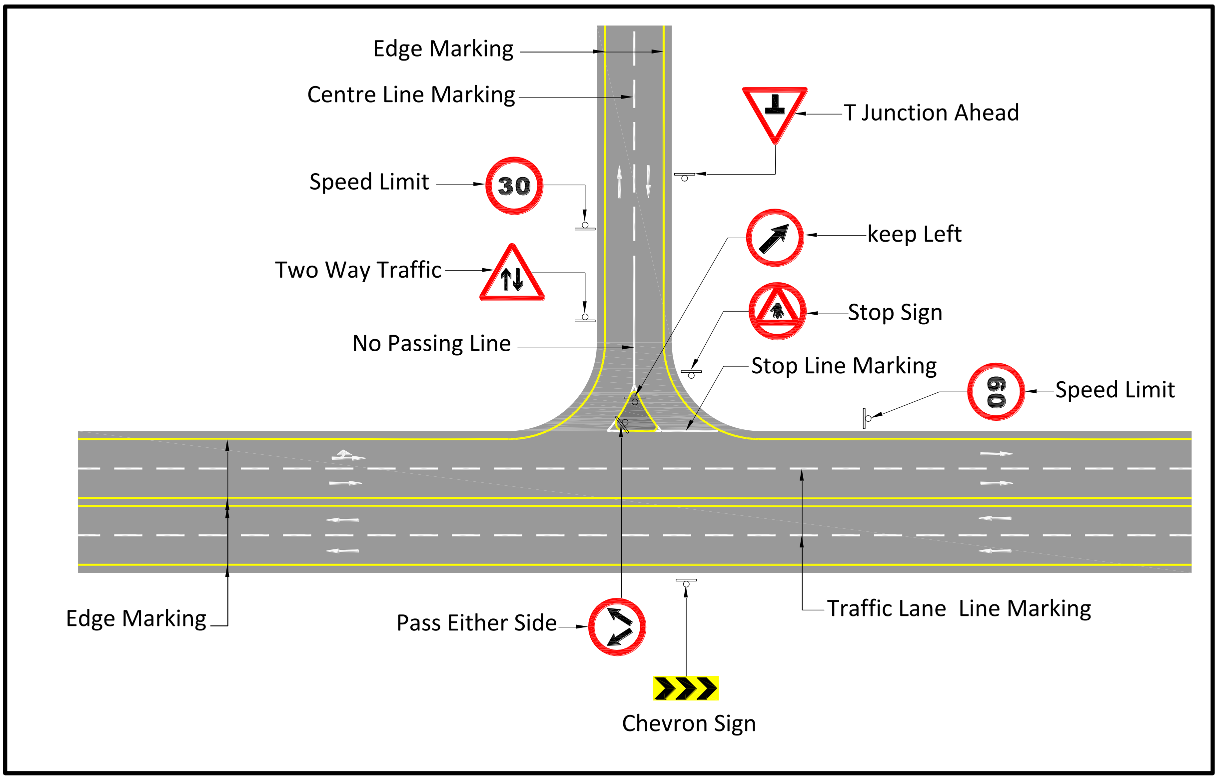


Figure 8.28: Traffic Signs and Road Markings at Rural T- Junction

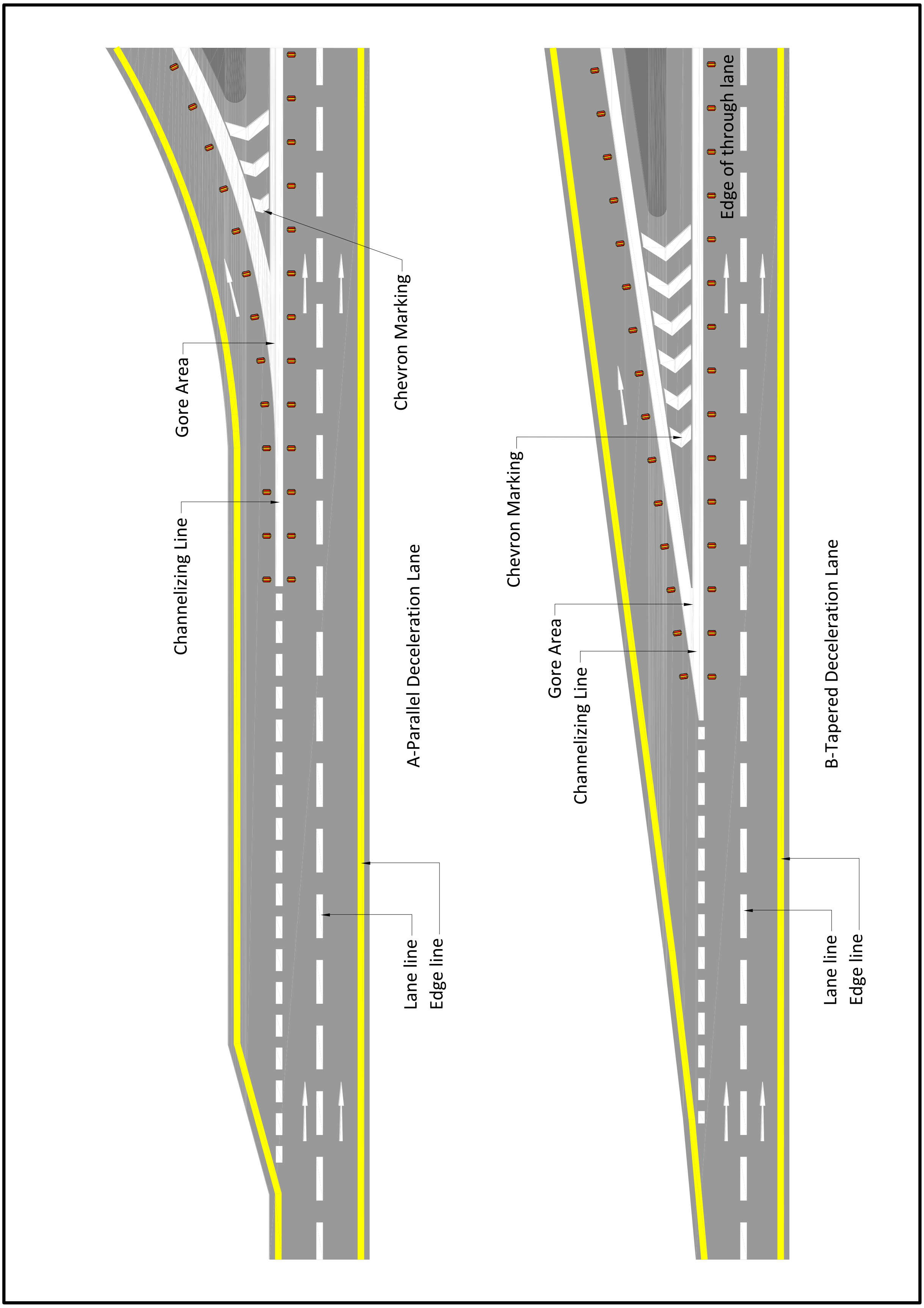


Figure 8.29: Typical Exit Ramp Pavement Markings Details

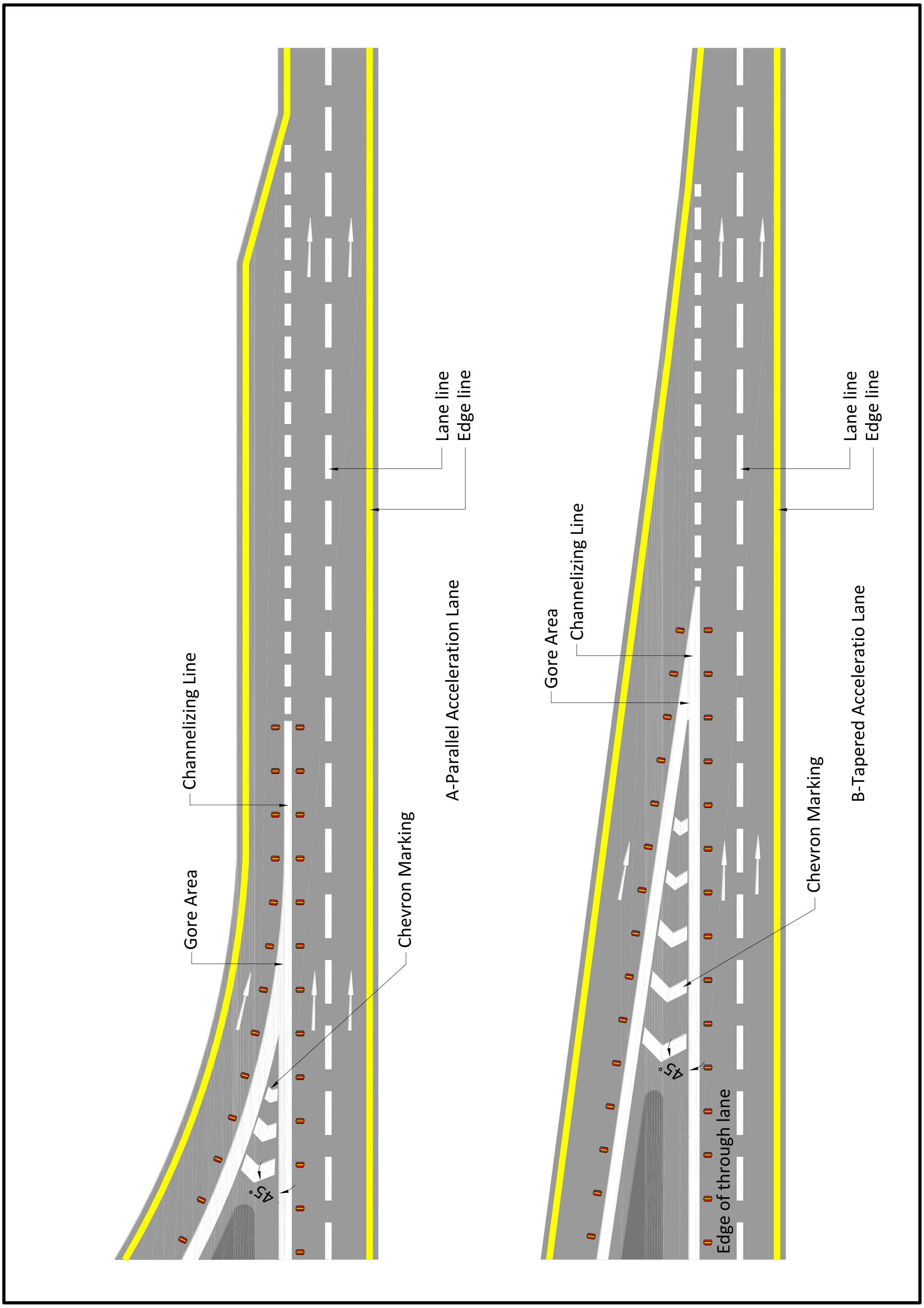


Figure 8.30: Typical Entrance Ramp Pavement Markings Details

9 IMPROVEMENT OF EXISTING ROADS

9.1 GENERAL

This section provides guidance on road improvement. Road improvement means resurfacing, restoration and rehabilitation (3R) work of existing roads. The basic purposes of 3R construction projects are to preserve and extend the service life of existing highways and streets and to enhance safety. Because of limited resources, individual rehabilitation projects may have to be limited in scope in an effort to preserve the mobility function of the entire highway system. The scope of 3R projects varies from thin overlays and safety upgrading to more complete rehabilitation.

- In other words, 3R projects are those that address pavement needs and/or deficiencies and which substantially follow the existing horizontal and vertical alignment.
- An observed or expected traffic operational problem, such as bottlenecks or low LOS
- An observed safety problem, as identified through an assessment of crash frequency and severity
- A need to provide access to a new adjacent development
- Pavement or other road infrastructure in dilapidated state

Project Examples of 3R projects include:

- Pavement that has reached its useful life and requires complete replacement, including potentially the subgrade, shoulders and kerbing
- Removal of a bituminous overlay to a concrete pavement and replacement with a new overlay
- Replacement of roadside barriers such as guardrail
- Bridge re-decking
- Major repairs or replacements to a bridge substructure

In addition to the above, a project may include one or more of the following types of work as a general improvement

- Widen roadway and bridge lanes.*
- Widen or add roadway and bridge shoulders.*
- Provide clear zone.
- Replace bridges rated "insufficient".*
- Upgrade to current Access Management requirements.
- Provide non-vehicular transportation needs.
- Add or extend auxiliary lanes to a roadway.
- Add turn lanes at an intersection or on a roadway.
- Realign an intersection or roadway.
- Replacement of bridges which cannot be widened economically.*
- Upgrade at-grade railroad crossings.
- Intersection improvements.
- Removal of parking lanes.

- Improving skid resistance and other safety improvements.
- Add or upgrade transit stops.

* Major bridge improvements and replacements must be programmed using the appropriate bridge program funds. While the general nature and type of improvements that can be made is extensive, due to the limited availability of funds, the cost of improvements other than those needed for safety and to meet minimum criteria must be carefully considered before including these improvements in the project.

Other certain work efforts related to infrastructure condition are fundamentally preventative maintenance activities. These may include minor pavement repairs such as seal coats, full-width patching, crack sealing, and thin plant mix resurfacing for sealing of the pavement surface, correcting minor surface irregularities, and other similar repairs. Kerb repairs or replacement, replacement of drainage inlets, and other similar activities are also fundamentally preventative maintenance in nature. These repair types are an important part of the road management. Maintenance repair activities are not considered 3R or reconstruction projects.

3R projects differ from reconstruction projects in that reconstruction projects substantially deviate from the existing horizontal and/or vertical alignment and/or add capacity.

9.2 OBJECTIVE OF IMPROVEMENT

The objective of upgrading of the existing road is to restore original service along the road or to consideration of improve it to meet current or future demands. Such upgrading shall be cost effective and within functional limits. This objective applies to all aspects of the road's serviceability, practical and/or structural desires including the following:

- geometric design,
- drainage,
- traffic control
- road safety,
- structural adequacy,
- level of service for the traffic flow,
- slope and embankment stability

9.3 PLANNING IMPROVEMENT

Planning of an improvement projects require balance in a number of competing objectives, the principal ones being the preservation of highways, improved service levels and enhancement of safety. The success in meeting these objectives depends on the quality of project designs and decisions. The majority of existing projects are identified because of deficient pavement condition or a serious safety issue. The budget of the project shall be arranged in a manner that it provides ample fund for the desired improvement along with some limited fund for other works not directly related to the intended improvement but may be required for safety of other reasons.

Apart for road condition, safety and budget availability other important elements in planning are:

9.3.1 Right Of Way Identification and Acquisition

Generally, existing projects do not typically involve Right of Way (ROW) acquisition. However, in case where an additional lane, transit stops, access management, drainage design elements and other improvements are required a review of the existing right of way to identify locations that require additional right of way is necessary. This review shall be done in advance of the actual work and all land acquisition shall be completed before commencing for procurement of work.

9.3.2 Design Exception Design Variation

Existing projects may have features that are not meeting minimum criteria requirement and that may not be improved under the improvement project due to any technical reason (generally restricted ROW) or budget constraint, will require design exception or design variation approval, as appropriate. The competent authority before finalization of such design shall approve such design exception and variations.

9.4 APPLICATION OF IMPROVEMENT

The approach to the designing of the improvement of the existing road projects is to evaluate the existing condition and recommend the improvements to the existing geometrics.

9.4.1 Nature of Improvement

The Designer is required to establish the nature of improvement require for a specific existing project.

- pavement rehabilitation/resurfacing/restoration,
- widening of the roadway including provision of climbing lanes,
- control of land use and access to minimize accidents,
- upgrade roadside safety,
- increase the length of one or more acceleration or deceleration lanes,
- improve a weaving area,
- widen an existing bridge as part of a bridge rehabilitation project,
- improve bridge structural adequacy,
- Improve drainage adequacy and/or provision of traffic control devices, traffic signalization and improve intersection layout.

9.4.2 Design Considerations

The designer shall identify and evaluate any design deficiency that may requires improvement or caused by the road improvement. Such as alignment improvement or installing barrier restricting horizontal sight distance.

9.4.3 Design Exceptions

The designer shall also consider design exceptions that apply equally to the geometric design of the existing projects; this may include things like restricted right of way that hinders in increasing lane or shoulder width.

9.4.4 Safety Review

The designer shall identify geometric and roadside safety design deficiencies within the project. **Table 9.1** summarizes the known relative importance of roadway elements in safety performance, crash frequency, and severity of different roadway types and contexts. Not every geometric element is of equal importance in influencing safety performance. Moreover, the contribution to safety performance of an element varies by type of road. **Table 9.1** serves as a reference in making decisions on retaining existing road geometry to avoid major costs and conflicts. Refer to the *AASHTO's Highway Safety Manual, 2010* [3] for more details on the specific elements and road types.

Table 9.1: Relative Relationship of Geometric Design Features to Crash Frequency or Severity

Roadway Design Elements	Road Type and Intersections						
	Rural 2-lane	Rural Multilane	Multilane Urban Arterials and Collectors	Freeway	Un-signalized intersection	Signalized Intersection	Roundabout
Cross Section							
Lane Width	#	#	—	#	—	—	—
Cross Slope	#	#	—	—	—	—	—
Shoulder Width	✓	✓	—	✓	—	—	—
Shoulder Type (Paved, Unpaved)	#	#	—	#	—	—	—
Presence of Rumble Strips	✓	✓	—	✓	—	—	—
Side slope	#	#	—	✓	—	—	—
Clear Zone	✓	✓	—	✓	—	—	—
Presence of Roadside Barrier	✓	✓	—	✓	—	—	—
Presence of Median	NA	✓	✓	✓	—	—	—
Width of Median	NA	✓	✓	✓	—	—	—
Alignment							
Horizontal Curvature (Radius)	✓	✓	—	✓	NA	NA	✓
Length of Curve	✓	#	—	#	NA	NA	NA
Presence of Spiral	✓	#	—	#	NA	NA	NA
Super elevation	#	—	—	—	NA	NA	NA
Grade	✓	#	—	—	NA	NA	NA
Length of Vertical Curve	#	—	—	—	NA	NA	NA
Stopping Sight Distance	#	—	—	—	NA	NA	NA
Presence of Weaving Sections	NA	NA	NA	✓	NA	NA	NA
Length of Weaving Sections	NA	NA	NA	✓	NA	NA	NA
Location of Ramps (Left vs. Right)	NA	NA	NA	✓	NA	NA	NA
Other							
Frequency of Driveways	✓	—	✓	NA	NA	NA	NA
Frequency of Intersections	✓	✓	✓	NA	NA	NA	NA
Type of Intersections (Traffic Control)	—	—	—	NA	NA	NA	NA
Intersection Elements							
Intersection Sight Distance	NA	NA	NA	NA	✓	—	—
Number of Legs/Approaches	NA	NA	NA	NA	✓	✓	✓
Skew Angle	NA	NA	NA	NA	✓	✓	—
Presence of Left-Turn Lanes	NA	NA	NA	NA	#	✓	—
Presence of Right-Turn Lanes	NA	NA	NA	NA	#	#	—

KEY:
✓ Significant Effect
Minor Effect
— No Effect
NA Not applicable

Based on *AASHTO's Highway Safety Manual, 2010* [3]

9.4.5 Project Evaluation

This includes but may not be limited to pavement condition, geometric design consistency and traffic control devices and accident data as and where applicable.

9.4.6 Reporting

In order to take up an existing road project for improvement it is imperative to analyse the following:

1. The existing conditions, which may include project length, design and operational speed, current and future traffic (for all road users) and percentage of trucks and prepare a report. The report may be based on as built drawing and /or a physical survey actually conducted at site. Following should be included in the report:
   - i. Pavement Condition
   - ii. Structures Condition
   - iii. Intersections Geometrics;
   - iv. Specific areas of failure;
   - v. Presence of underdrains and pipe drain headwalls;
   - vi. Provision of design elements for non-motorized transport;
   - vii. Location and performance level of existing road safety appurtenances;
   - viii. Locations and performance of existing shallow roadside ditches
   - ix. Accident history over the past five years.

   The details of inventory required to be collected for reporting is provided below:
   - i. AADT - Total
   - ii. AADT - Heavy Commercial Vehicles (HCV)
   - iii. Design Speed as designed
   - iv. Posted Speed posted on the road
   - v. Operational Speed possible due to condition factors
   - vi. Accident Frequency preferably for 5 years or more
   - vii. Road Type - Single, double, multilane/ Single or double carriageway/ one way or two way
   - viii. Road width
   - ix. Pavement type- Rigid or Flexible
   - x. Pavement Surfacing- Concrete (RCC, PCC), Asphalt Concrete, Surface treatment, gravel surface, dirt track.
   - xi. Pavement Condition – good, fair, poor
   - xii. Condition of Drainage Structure, functional/ functional with maintenance/ non functional
   - xiii. Bridge - Width and condition
   - xiv. Environmental Impact – Environmental issues and Mitigation plan where required.
   - xv. Horizontal Curves adequacy
   - xvi. Vertical Alignment adequacy
   - xvii. Intersection details – Numbers, issues
   - xviii. Safety requirement – existing and required
   - xix. Sketch and photographs for showing issue in the report

2. Proposed Scope of Improvement Work. The report shall contain a brief description of the proposed scope of work. This description must include but not limited to the following items:

i. A map showing the location of the proposed improvement, the limits of the proposed work
ii. Any omitted section
iii. a typical cross section showing the proposed resurfacing thickness, shoulder widths, pavement and shoulder cross slopes, side slopes, bridge widths; and
iv. exceptions from this Manual

3. Estimated Cost for the Proposed Improvement.

4. Other requirement such as :
   i. Justification for exceptions from design Standard
   ii. Discussion on accident frequency with relation to a specific location with proposed mitigation,
   iii. Proposed geometric changes or super elevation corrections,
   iv. Vertical clearances,
   v. Drainage analyses,
   vi. Any other required attention in improvement

9.5 GEOMETRIC DESIGN

Geometric design criteria applied for new construction of road and for improvement of the existing road projects. However, at request certain decision will required to be made in order to accommodate the design with the limitation of the existing road. Such flexibility is discussed in the sections below:

9.5.1 Design for the Original Construction

In area where the existing road improvement project is taken up for improving the dilapidated pavement and for improvement of drainage etc. in these case design criteria for horizontal and vertical alignment, shoulder, median and lane widths may be kept same when meeting the design criteria as set forth in this manual.

9.5.2 Design Speed

Highway features are generally based on design speed. Design speed is a principal design control that controls the selection of many of the project criteria used to design a roadway. Selection of the design speed must be rational for the type, location and operational conditions of the highway, and the design speed used should be consistent with comparable adjacent roads. Design speed must not be dictated by an isolated geometric feature. Design speed shall not be less than the legal posted speed. The design speed used in the original design of the highway should be used for improvement projects. However, there may be situations where the existing posted speed on the highway is different, then that used in the original design of the highway. Such as where an improvement requires adjustment in the speed limit or where it is required to reduce the speed based on crash history. There are methods that can be used to select the design speed for improvement projects. These may be used alone or in combination.

- Select the design speed used in the original design of the road.
- Select an overall design speed greater than or equal to the posted regulatory speed on the section being improved, if there is a significant crash history associated with a specific highway feature.

- Select the design speed used in the original design of the highway unless a reduced design speed (not less than posted speed) is approved by competent authority.

9.5.3 Design Traffic Volume

The AADT shall be used during design processes. The designer shall consider the use of the forecasted traffic for 10 years beyond the date of completion for the improvement project. It should be noted that the design traffic volume for a given road feature should match the average traffic anticipated over the expected performance period of that feature.

9.5.4 Horizontal and Vertical Alignment

Typically, 3R projects will involve minor or no change in either vertical or horizontal alignment. However, flattening of curves or other improvements may be considered where suggested by accident history, or where existing curvature is inconsistent with prevailing conditions within the project or on similar roadways in the area. Where appropriate, improvements in superelevation may also be a consideration.

Where substantial horizontal or vertical alignment variations are warranted, this shall be considered in reconstruction project rather than improvement of existing road or 3R projects.

Horizontal and vertical alignment must be reviewed together in order to maintain safety and aesthetics.

9.5.4.1 Horizontal Curve

On the existing horizontal curves with a design speed equal to or slightly less than the posted speed, may be retained unless crash record indicate significant unsafe condition through accident records. The procedure and safety should be improved based of the following elements as superelevation amendments, eliminating crown, and improve the stopping sight distance based on removal of any sight obstructions. The traffic control device should be installed, where horizontal cure does not meet the posted speed. A conclusion not to reconstruct an existing horizontal curve where, the curve design speed significantly below the posted speed shall be supported with a design exception.

9.5.4.2 Vertical Alignment

Vertical Curves

Vertical curves shall be analysed to determine if they meet the criteria found in this manual. If the curve is not meeting the minimum criteria as mention in this manual, It is required to determine existence of any operational or safety problems at the location. Where no operational or safety hazard is observed, existing curve shall be kept unaltered.

Vertical Clearances

The desirable vertical clearance over the travel lanes including shoulders for new or reconstructed structures is 5.5 m. where the road vertical clearances is less than 5.5 m will require permission from consent authority before can be implemented.

9.5.5 Bridges

9.5.5.1 Bridge Condition Reports

The improvement of project required bridge condition report and proposed works drawings for every structure within a roadway section. This report will ensure that the bridge meets the minimum desires for width, safety, and structural capacity.

9.5.5.2 Safety Analysis for Bridges

The safety analysis for narrow bridges within the ROW limits must first be examined to determine if widening is necessary a road safety experience or other working problems. This based on accident data for the previous 4 years and, if necessary, a direct field check.

9.5.5.3 Bridge Replacement/Rehabilitation

Bridge replacements is part of Improvement or rehabilitation of road projects but major bridge replacement may not be part of 3R projects or bridge rehabilitation work and, in some circumstances, this may be the complete project scope of work. The following will apply to the geometric design of these projects:

1. **Horizontal and Vertical Alignment**: An alignment of the existing bridge that does not meet the criteria which provided by this manual. For replacement bridge the projects, the designer should evaluate the practicality of realigning the bridge to meet the applicable alignment criteria for new construction/reconstruction. For bridge rehabilitation projects, it is unlikely to be cost effective to realign the bridge in order to correct any alignment deficiencies.

2. **Width**: When the report of bridge condition specifies deck replacement is required, the designer should consider widening the superstructure to the extent possible without needful substructure accompaniments. In no case shall the structure be made narrower than the existing width. The bridge width should be wider than the full approach carriageway width by one meter. Capacity analyses could determine the need for auxiliary lanes and/or the need for wider walkways.

3. **Safety**: It is important to check whether the parapet and any approach safety barrier meets current containment and other safety standards.

9.5.6 Road Safety

9.5.6.1 General

Site-specific conditions should be measured to control the suitability for making improvements in side slope. The considerations should not only limited to economic evaluation i.e. cost and benefit analysis but also requires study of impact of improvement changes. The objective of the workout is to provide a cost effective result within the available budget and to identify dangerous features and determine:

- The hazard to be redesigned for making the facility passable at design speed
- The hazard to be removed,
- The hazard to be shielded with an appropriate barrier, and
- The hazard that is not cost effective to redesign and therefore untreated.

Location where accidents are frequent shall be identified in the early design stage and any action required to remove the cause of such accident should be included in the plan of work. Any item identified, that is shielded by a roadside barrier require to mitigate another hazard may be left untreated. In addition, some hazards may be allowed to remain just inside the clear zone when there are other similar hazards just outside the clear zone that do not require treatment and if the accident history for the facility does not indicate any problem.

9.5.6.2 Side Slopes

In an improvement project, flatting of slope should be undertaken as much as possible provided cost and conditions permit such improvements. Accident history shall be analysed to check the improvement requirements. Consideration should be given to the following:

- In areas where run-off road accidents are likely to occur i.e., outside of sharp horizontal curves, side slopes steeper than 1:3 within existing road reserve should be flattened as much as conditions permit.
- When widening lanes and/or shoulder retain the current rate of side slopes, unless steeper slopes are necessary by special conditions.

9.5.6.3 Cross-Slopes and Superelevation

- This manual provided the guidelines to improvement road projects that include resurfacing pavement, cross-slopes.
- The superelevation rates on horizontal curves should be adjusted as per guidelines provided in this manual for the design speed.

9.5.6.4 Clear Zone

The uniform distance is from the carriageway edge to the utilities poles, tree line and any other object that obstruction site distance to safety is necessary. The special consideration should be given to the following:

- Relocating, removing, and/or protecting remote roadside problems on the fore-slope mostly non-recoverable or roadside ditches.
- Relocation, removal or protecting of roadside problem responsible for recorded accidents.
- Where accident history is not indicating towards a specific location but distributed throughout the road length, an increase of clear zone width should be considered on such roads.

9.5.6.5 Tree Removal

The removal tree will be careful and will usually “fit” situations within the existing road reserve and character of the road. Trees within the clear zone should be considered for removal subject to the following criteria:

- **Accident Frequency** - Where there is evidence of vehicle-tree crashes, either from actual accident reports or from damaging of the trees.
- **Intersections and Crossings** - Trees that are blocking satisfactory sight distance or are mostly exposed to being hit.
- **Natural Tree growth** within the clear zone may be removed especially if a safety hazard is envisaged.

9.5.6.6 Roadside Obstacles

Roadside improvements should be measured to enhance safety. The improvements may include redesign, relocation, removal, or protecting of problems such as culvert headwalls, utility poles, and bridge supports that are within the clear zone.

The information of the accident history will provide guidance for possible treatments. However, treatment of some problems, such as large culverts, can add significantly, to the cost of a project. This means that in most cases only those problems that can be quoted as exactly related to accidents or can be enhanced at low-cost should be included in an improvement project.

9.5.6.7 Guardrails

The assessment of guardrails and bridge rails should include but not only limited to the following:

- Existing bridge rail may remain in place if it meets static load requirements. Otherwise, replaced.
- Blunt ends and turned down endings shall be upgraded.
- Site examination of length height, length, and overall condition should be done to determine guardrail upgrading needs
- Isolated guardrails to bridge rail transitions shall be connected or upgraded.

In the fill sections required special consideration for installation of the guardrails. Where clear zone have a problems and slopes are non-recoverable with hazards at the landing zone, or any other location that requires guardrail based on the traffic accident history analysis.

9.5.6.8 Intersection Design

It is required to evaluate existing intersections for safety where it exceed a certain number of vehicles per day e.g. 1,500 vehicles per-day or where there is proof accident related to the existing condition. Some intersections should be included in the project where practical and feasible. All available accident data should be applied in the field review of the intersection. Safety processes, as discussed in the Additional Safety Measures in this, can be utilized to mitigate safety concerns at intersections. Warning signs/panels should be installed where applicable.

9.5.6.9 Traffic Control Devices

Installation of traffic signs, pavement markings, and traffic signal controls in accordance with this manual and manual of Uniform Traffic Control Device for Streets and Highways 2010.

9.5.6.10 Signing

Consideration should be given to upgrading sign reflectivity, supports, and locations.

9.5.6.11 Supplemental Safety Measures

In the design of roads to be provide the range of the supplemental measures that can be used only or in combination with others to mitigate shortages in controlling elements to provide for safer roadways. Where re-construction of the road feature includes vertical curve, horizontal curve, bridge or intersection, is not reasonable or practical because of social, economic or environmental concerns, another safety measures should be measured. The following of these are shown in **Table 9.2**.

9.5.6.12 Safety Reviews

In all road improvement project a detailed safety review of the project pre and post design is recommended before commencement of actual project procurement and construction.

Table 9.2: Safety Measures for Different Geometric Problems

Geometric Difficulty	Addition Safety Mitigation
Narrow lanes and shoulders	Paved shoulders, Pavement edge lines, Post delineators, Permanent pavement markers, Warning signs
Steep side slopes; roadside obstacles	Install guardrail, Round ditches, Warning signs, Obstacle removal, Slope flattening, Breakaway hardware, Post delineators,
Narrow bridge	Pavement markings, Traffic control devices, Warning signs, Approach guardrail
Poor sight distance at hill crest	Driveway relocation, Traffic control devices, Warning signs Shoulder widening
Sharp horizontal curve	Advisory signs and speed signs, Permanent pavement markers, Pavement anti-skid treatment, Traffic control devices, Slope flattening, Shoulder widening, Appropriate superelevation, safety barrier, Obstacle removal, Post delineators
Problem intersections	Pavement anti-skid treatment, Fixed lighting, Traffic control devices, Speed controls, Advisory signs, channelization using traffic islands, Traffic signalization, Rumble strips

9.6 Reconstruction Projects

Reconstruction projects upgrade the facility to acceptable geometric standards and as a result, provide a greater roadway width. The improvements may be in the form of additional lanes and/or wider shoulders and produce an improvement in the highway's mobility. Reconstruction projects normally include the following types of work: Projects, which alter the original subgrade; those that construct major widening that result in the addition of a new continuous lane; the addition of passing lanes or climbing lanes; structure replacement; and similar projects.

In other words, reconstruction projects will be those, which may fall under the following:

- The project needs more than mere infrastructure repair, to include a known quantitative safety problem or a known operational problem.
- The project involves the designation or reclassification of a roadway to serve new types of trips or travel not previously included along the route, such as bike paths or dedicated transit only lanes; widening; conversion of intersection type such as roundabout to signalized intersection.
- The project is bridge replacement, which specifically includes improvements to vertical clearance.
- The project converts a temporary road to a permanent road. The Overseeing Organization determines that the project shall not be eligible for3R treatment, as described above.

Reconstruction projects will involve substantial revision to the functionality and three-dimensional character of the road. Reconstruction projects shall be designed and reconstructed based on design criteria in this manual.

Challenges unique to reconstruction versus new construction projects include these:

- Right-of-way typically is limited, with adjacent development already established. Even minor land acquisitions may create significant damage to adjacent properties.
- In most cases, it will be necessary to maintain traffic flow along the roadway during reconstruction. This includes through traffic, intersection movements, and access to business, retail, and residential land uses. Existing underground utilities are in place. These constraints will influence the suitability of design solutions, and may limit the ability to make more than minor changes to vertical alignment.

The full design process for new roads applies to reconstruction projects, including development of design alternatives and evaluation of potential deviation from Standards.

9.7 Roadway Drainage

The drainage specialist must evaluate the hydraulic and physical adequacy of the existing drainage system. This requires examination of the existing drainage in the field and by consulting with maintenance personnel and records. If there are apparent problems with the existing drainage system, additional evaluation is required to determine the extent and type of improvements necessary to upgrade the system. Prior to selecting any plan of highway improvement, the designer should consult with drainage and environmental permitting specialists since almost all roadway modifications reduce storage and infiltration, and increase discharge rates and volumes. Storm water retention and detention for quality, rate and volume may be required. Theoretical evaluation of proposed changes to existing and new drainage features necessary to correct operational deficiencies should be referred to a drainage specialist. The drainage specialist will provide the necessary drainage design, flood data information; drainage related information for the Storm water Pollution Prevention Plan (SWPPP) and any storm water permit computations. Due to funding limitations, improvements other than those needed for safety and minimum criteria must be carefully considered before inclusion in the project.

9.8 Highway Lighting

Lighting may be installed at specific locations to improve safety. For example:

- Reducing the effects of ambient light conditions.
- Busy or high crash intersections.
- Transit stops.
- Channelized intersections.
- Car pool parking lots.
- Pedestrian and bicycle crossings.
- Ramp terminals.

Any lighting, existing or proposed, shall be reviewed by the competent authority to determine specific needs. Lighting shall meet new lighting criteria, found in this manual.

9.9 Traffic Management In Work Zones

9.9.1 General

In busy built-up areas, speeding motorists, cyclists, pedestrians and workers all interact with construction vehicles, heavy equipment, trucks and road pavers within the road construction site.

Another scenario is where worker work alone, often not visible, protected only by traffic cones and speed signs.

There are numerous ways to control the risks associated with working on roads or road-related areas. Following are some examples of a number of traffic control measures that may be considered:

- road closures
- footpath closures
- detours
- signing
- traffic controllers

When considering control measures such as road or footpath closures, advice and approvals should be obtained from relevant authorities.

This section provide some guidance on traffic management however for details, refer to *MUTCD* [1] that provides the technical background and guidance for the placement of temporary traffic control signs and devices used at road works. The *MUTCD* [1] also provides standard diagrams for traffic guidance schemes across a range of work activities and worksites. Selection processes are provided that use tables of key site information including:

- lane configuration
- traffic volume
- approach speed
- type of work

These selection processes are used to guide a principal contractor or relevant person to select the most appropriate traffic guidance scheme for a particular situation. However, it should be noted that the traffic guidance schemes provided here or in the *MUTCD* [1] might not be appropriate for every situation, and it may be necessary to design a scheme that is suitable for the individual worksite in question.

9.9.2 Planning

Careful consideration should be given to the signing of the worksite, no matter how brief the occupation of the site may be. This should include:

- protection of workers
- provision of adequate warning of changes in surface condition, and the presence of personnel or plant engaged in work on the road
- adequate instruction of road users and their safe guidance through, around or past the worksite, and
- safe access and egress to and from the worksite

Five important basic principles, to be observed, are as follows:

- Signs and devices shall be appropriate to the conditions at the worksite, and shall be used in accordance with the MUTCD Part 3 guidance, unless a risk assessment by a competent person indicates that an alternative arrangement is satisfactory.
- Signs and devices shall be erected and displayed before work commences at a worksite.

- Signs and devices shall be regularly checked and maintained in a satisfactory condition.
- Signs and devices shall be removed from a worksite as soon as practicable. However, appropriate signs should remain in place until all work (including loose stone removal and line marking following bituminous surfacing) has been completed.
- Records shall be kept of all work's signing and delineation at roadway or part-roadway closures.

A variety of standard plans/diagrams are provided for selection in the *MUTCD* [1], designed to illustrate the application of traffic control devices as they apply to various worksite situations and circumstances. These diagrams indicate the appropriate positions of the signs and devices required to guide traffic safely around, through or past the worksite. Worksite situations that are not specifically covered by a diagram should be designed according to the principles outlined in the *MUTCD* [1]. A sample standard diagram (Traffic Guidance Scheme) is provided in **Figure 9.1** at the end of this chapter.

Plans should be prepared for:

- **Short-term and mobile work**. Planning in these cases should comprise the development of procedures and the provision of appropriate sets of signs and devices to cover all of the routine tasks the workers will encounter.
- **Work involving relatively simple part-roadway closures**. Planning in these cases should comprise a minimum requirement to sketch the protective devices and delineation required on a road construction or similar plan, and to prepare a list of devices required for the job.
- **Complex traffic arrangements**. Planning in these cases should comprise:
    - plans showing temporary traffic paths, their delineation and the position of traffic control or warning devices, or on multi-stage works, a separate set of plans for each stage
    - details of afterhours traffic arrangements, on separate plans, if required, and
    - all necessary instructions for the installation, operation, between-stage rearrangement and ultimate removal of devices at the conclusion of the job, planned well before the job starts, or before the start of the stage to which they apply, so that there is enough time to obtain any special devices or approvals needed

All essential aspects of traffic plans are to be considered in the following order, and incorporated into the plan, if relevant.

9.9.2.1 Traffic Demand

Determine the capacity required to accommodate traffic demand at an acceptable level of service and convenience to road users and to decide on the amount of road space, which must remain open, and where applicable, the times of day when greater amounts of road space are needed to handle higher traffic volumes (e.g. peak hours).

9.9.2.2 Traffic Routing

Select the appropriate means of routing traffic at the site, i.e. through, around or past the site, or a combination of these, and ensuring that all required traffic movements are provided for.

9.9.2.3 Traffic Control

Determine the need for traffic control by:

- traffic controller
- traffic signals (portable or permanent)
- police
- other means

9.9.2.4 Other Road Users

Determine the need to make provisions for road users, other than vehicular traffic, including:

- pedestrians, including people with disabilities, where appropriate
- bicycles
- school children
- local residents
- emergency vehicles
- Cattle or animal carts

9.9.2.5 Special Vehicle Requirements

Determine the need to provide for vehicles, such as:

- buses, including stops and terminals
- over-dimensional vehicles (e.g. vehicles which, together with their load, are wider or longer than standard)
- restricted vehicles (e.g. vehicles which, although within legal limits, are permitted to use only specified routes)

Depending on circumstances, movement of traffic may be achieved in one of the following ways:

- through the work area, by intermingling with workers or plant
- past the work area by means of a delineated path alongside, but clear of the work area, and
- around the work area by a detour, which may be via a side track, or an existing road

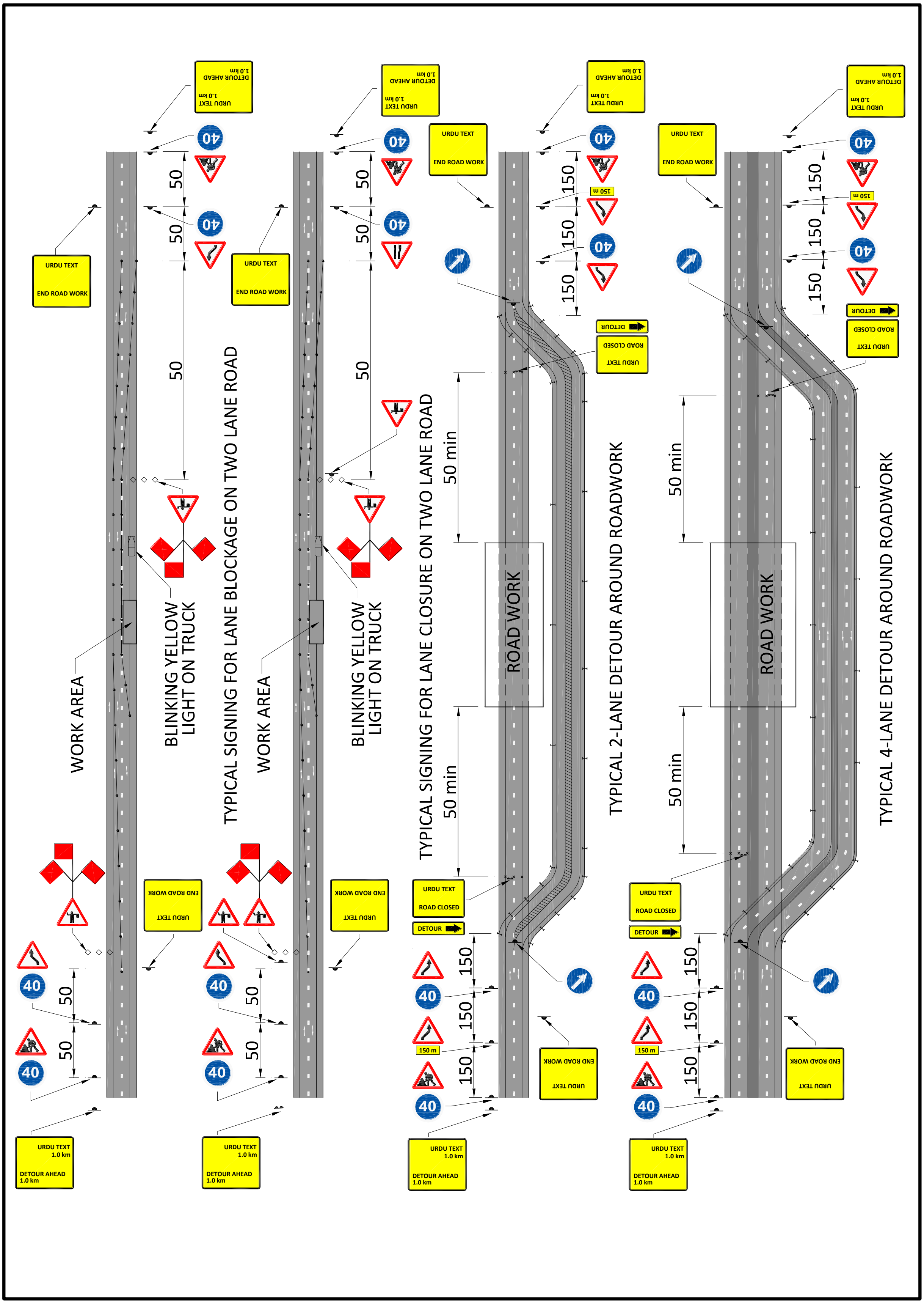


Figure 9.1: Typical Signage and Lane Detour Plans around Work Zone

10 SPECIFIC DESIGN DRAWING REQUIREMENT

10.1 INTRODUCTION

The purpose of this chapter is to provide guidance on the contents and presentation of the design drawings. This chapter include standard numbering for drawings, drawing presentation, units of measurements and symbols and abbreviations.

The drawings shall be clear, accurate and with details that meet its intended purpose. The drawings can be classified as Preliminary Design Drawings for feasibility studies, Detailed Engineering Drawings for tendering and construction purposes and As-built Drawings for archive purposes. This chapter provides guidance on standards of preparing drawings for different purposes. However, it is emphasized that Project TOR may also be referred and followed along with these standards.

Intermediate drawings submitted as Preliminary Drawings and Draft Final Drawings in the Detailed Engineering Design Projects shall be stamped as shown below to indicate that they are yet not ready for implementation.

DRAFT DRAWINGS

Figure 10.1: Stamp on Draft Drawings

Drawings that are approved for Tendering shall be stamped as:

DETAILED ENGINEERING DRAWINGS
ISSUED FOR TENDER

Figure 10.2: Stamp on Tender Drawings

Final Design Drawings for Construction shall be stamped as shown below for construction of the project:

DETAILED ENGINEERING DRAWINGS
ISSUED FOR CONSTRUCTION

Figure 10.3: Stamp on Construction Drawings

As-built drawings shall be stamped and signed by the Project Director and Resident Engineer responsible for supervision of projects:

AS BUILT DRAWINGS

RESIDENT ENGINEER	*PROJECT DIRECTOR*
(NAME)	*(NAME)*
DATE: XX-XX-XXXX	*DATE: XX-XX-XXXX*

Figure 10.4: Stamps on As-Built Drawings

10.2 Standard Numbering of Drawings

In order to easily identify and store, the drawing should have a standard number. The following structure is to be used to allocate reference numbers to all drawings.

Table 10.1: Standard Drawing Numbers Structure

Project Reference		Organization Name		Task		Zone		Drawing Status		Drawing Type		Unique Number		Revision
ABCD-01	-	KPH	-	HGW	-	PEW	-	PR	-	GEN	-	4 Digits	-	Rev

Example of a Drawing reference number:

ABCD-01-KPH-HGW-PEW-PR-GEN-0001-00

In assignments whereby the respective roads have no numbers, the designer shall consult the client for agreement on a number to be utilized.

The road number, ABCD-01 stands for the Khyber-Pakhtunkhwa Roads and Highways. Organization name is the name of organization undertaking the work or Owner, such as Khyber Pakhtunkhwa Highways (KPH). The task shall be Highway (HGW), Civil (CIV), Drainage (DRN), Electrical (ELE) or any other details, first three letters of the task or popular abbreviation of the task may be used. Zone may be a city a district or any specific place, full name or abbreviation may be used, for example Peshawar (PEW), Dera Ismail Khan (DIK), Chitral (CHT), and Kohat (KOH). The letters following the zone will show the status of the drawing such as Preliminary Drawings (PR), Detailed Engineering Drawings (DD) and As-Built Drawings (AB). Following status of drawing, type of drawing is mentioned, this include General (GEN) Drawings, Plan and Profile (PP), Cross Section (CS) Drawings, Road Furniture Details (RF), Structure (STR) Drawings, Land Acquisition (LA) Plans, Grading (GR) Plans, Traffic Sign (TS), Road Marking (RM) etc. Next is a four digit drawing number and finally the revision number.

Clearly show the title blocks of all drawings which includes: - Designed by, approved by, including the name and signature of the responsible engineer and the issue date clearly displayed. The Final Detailed Engineering Drawings shall be approved and signed by Authorized person which responsible for design of roads.

The drawings includes the symbol for the responsible Authority and should have the following text "THIS DRAWING IS THE PROPERTY OF (RESPECTIVE AUTHORITY) AND THEREFORE COPYING OF THIS DRAWING IS NOT ALLOWED UNLESS AUTHORIZED IN WRITING BY THE (RESPECTIVE AUTHORITY)".

10.3 Drawings Template

The drawing template shall be consistent with the details provided below:

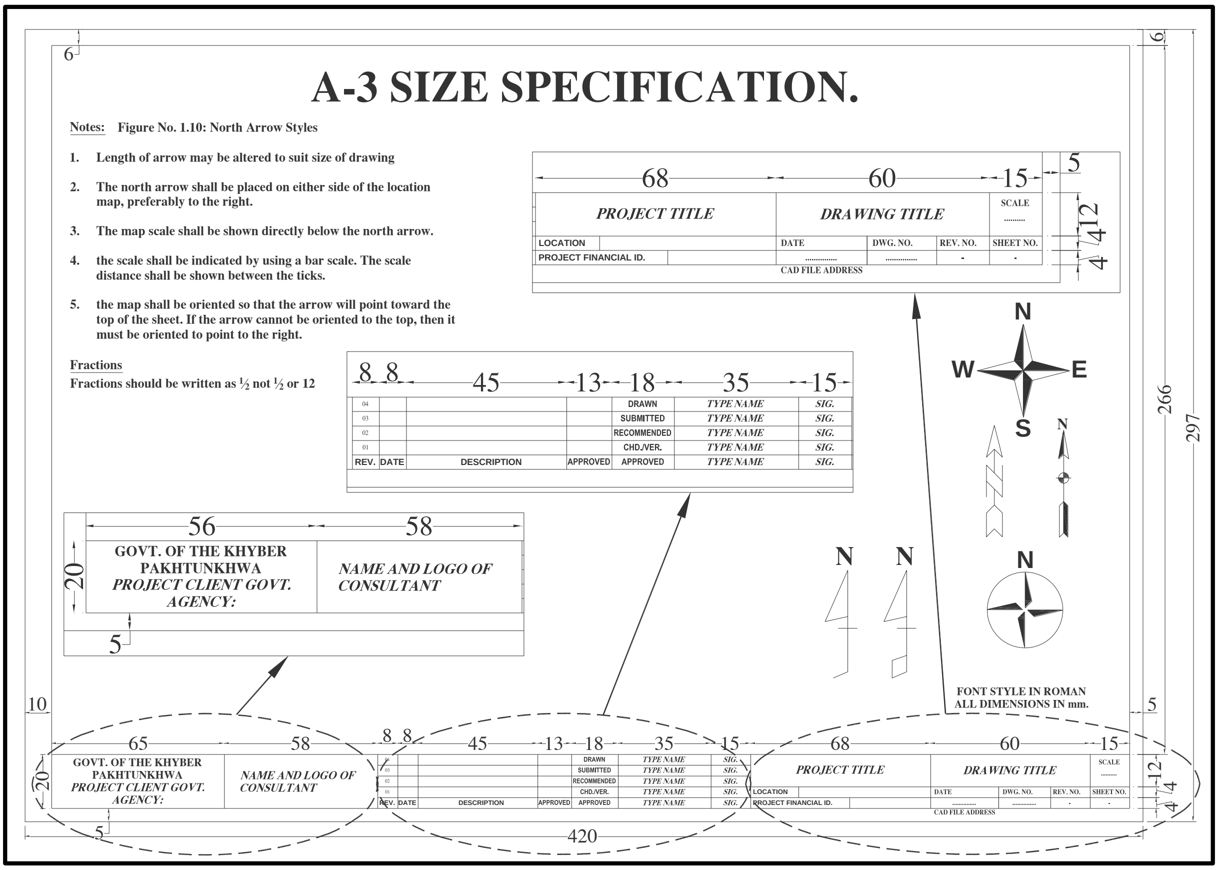


Figure 10.5: Sample of a Title Block

10.4 Presentation

The following factor are very important in the final drawings must be well detailed, concise, clear, unambiguous, readable and consistent to serve the future purposes.

The drawings symbols used to represent different features to confirm uniform clarification of the information showed and the legends which shall explain the lines in the drawings.

All the drawings should be prepared in a scale sufficient to show all the important features for the future determinations such as drawings for landscaping, land acquisition, utilities etc. For drawings prepared to show the location of related facilities, the plan can appear as a background and shown as faint lines while the future data is drawn using more visible lines. This can be applicable to longitudinal drainage, traffic signs, road accessories, traffic signals, utilities, structures and land acquisitions, landscaping etc.

The project drawings are included in the following types list of drawings; but its depends on the size and difficulty of the projects:

10.4.1 Title Sheet

The title sheet has no drawing number. This sheet provides an simply recognisable cover that supports protect the document subjects. The details controlled on the title sheet should permit a reader to classify the work, without the need to open the document set.

10.4.2 List of Drawings

The list of drawing is a summary index listing all relevant final drawings included in a contract. The List of Drawings is used as an easy guide to referencing a particular final plan of interest to a relevant sheet number. The List of Drawings contains a listing of all final plans in sequential order of sheet numbers followed by drawing numbers and description can be divided into various drawing types.

10.4.3 Location Plan

The purpose of Location plan is to show the site of the proposed road in relation to the surrounding area and geographical features. The scale of the plan is variable depending on the size and complexity of the project.

10.4.4 Typical Cross Sections

The Typical Cross Section drawing illustrates the structural elements of the roadway, lateral distance, cross slopes, batter slopes and subsurface drains. The pavement detail drawing provides the pavement structural thickness and materials, and may include the location of kerb and channel, subsurface and surface drainage.

The Pavement Detail drawings are not usually drawn to any nominal scale, but should be visually proportional with the drawing scale being specified as '*Not to Scale'*. Where there is a need to provide more than one typical cross section, a constant nominal scale should be adopted for visual consistency between drawings.

Typical cross sections should be provided at locations where the road formation is consistent and applies over a reasonable length. Specific typical sections whose application is restricted to a limited and specific area may be shown when the section is relevant.

10.4.5 Plan and Profiles

There are two types of drawings are included:

First drawings for built the main road, and the second drawings for secondary roads associated to the main road like service roads, separate roads for pedestrians/non-motorized traffic, ramps, etc. For each type of drawings are specific number or abbreviation to be used.

The plan and profile drawings establish the basis for almost all other drawings associated to the project. The drawings contain of geometrical plans and vertical profiles of the road.

Geometric plans are used to create a reference line for the location and setting out drawings of construction works. The setting out used to establish the connection between the design line and other design lines and/or traverse lines.

The purposes of the longitudinal sections are used to obtain the vertical geometry of the road. The cross fall information included in the longitudinal section are used by the surveyors and contractors in various programs to calculate the earthworks and pavement construction for cut and fill volumes.

In detailed engineering design shall applied the following scales, unless otherwise stated in the terms of reference:

Table 10.2: Scales for Plans for Preliminary Design Drawings

Drawing Type	Scale
Rural type location, with sparse details and straight forward alignment	1:2500
Rural or urban location with construction restrictions and straight forward alignment	1:2000
Urban type alignment with complex alignment and important details	1:1000

Table 10.3: Scales for Plans for Detailed Engineering Design Drawings

Drawing Type	Scale
Rural type location, with sparse details and straight forward alignment	1:2000
Rural or urban location with construction restrictions and straight forward alignment	1:1000
Urban type alignment with complex alignment and important details	1:500

For Profile or longitudinal section drawings, the following scales in **Table 10.4** shall be applied unless otherwise stated.

Table 10.4: Scales for Profiles

Drawing Type	Scale
Detailed Engineering drawings for Highways, main roads, ramps for interchanges, access roads and other uses	Hor. 1:2000, Vert. 1:200 or Hor. 1:1000, Vert. 1:100
Preliminary Design drawings for Highways, main roads, ramps for interchanges, access roads and other uses	Hor. 1:2000, Vert. 1:200

In the plans and profile drawings are labelling of the chainages shall be done at every 100 m interval and at the important points. The drawings should have tables brief all the important factors required for setting out details of the curves including the chainages and coordinates of start and end of circular curves and spirals, the tangents points of intersection, the deflection angles for tangents and the curves radii. At the suitable interval, grids with Easting and Northing coordinates should be provided as backgrounds to the plans to facilitate easy clarification of the drawings.

The orientation of the chainages start from left towards to the right direction for each of the plan drawings and also mark the orientation of the North direction.

In the profile drawings are the following information should provide which included the finished road levels and existing ground levels at every 25m interval or as per mention in terms of reference.

Some of more important profile parameters such as the gradients, K-vales and beginnings and ends of vertical curves should be shown in the drawings. The superelevation diagram show the values of maximum superelevation for each curve should be shown in the profiles.

The following below mentioned standard symbols and parameters shall be commonly used in the drawings:

- TS - Tangent to Spiral
- SC - Spiral to Curve
- CS - Curve to Spiral
- ST - Spiral to Tangent
- TC - Tangent to Curve
- CT - Curve to Tangent
- PI - Point of Intersection
- PVC - Point of Vertical Curvature
- PVI - Point of Vertical Intersection
- PVT - Point of Vertical Tangent
- K - Rate of Vertical Curvature

The chainages in the plan and profile drawings shall be obtainable using kilometres with a "+" sign in between to separate the values of kilometre and the meter. For example, if the chainage at 02+100 means 2 kilometre and 100 meters from the start point.

The additional information show in the detailed engineering drawings which included the road widening, climbing, acceleration and deceleration lanes shall provide the required data to allow for construction.

10.4.6 Intersections, Roundabouts and Accesses

The plan and profile drawings also show the location of the major and minor intersections, buses lay byes, accesses, roundabouts, rest areas, etc. However, the details of these facilities shall be shown on the large scale separate drawings. These drawings will include details of the major and minor intersections, roundabouts found in a actual project and typical uniform solutions to accesses, buses lay byes, etc.

10.4.7 Cross Sections

Cross sections drawings are useful for the purpose of computation of volumes and for setting out of the roads. For the detailed design, the designer will be required to provide cross sections at an interval of 25 m unless otherwise specified in the Terms of Reference. Cross sections shall also be provided at location of culverts for providing the invert levels. For preliminary design purposes, cross-sections may not be needed in some cases, however if there is a need, the intervals shall be as stipulated in the terms of reference. The cross section drawings shall indicate all the necessary information for the road embankments, cuttings and drainage structures. The information to be provided includes the offsets and levels for road centrelines, ends of carriageways and shoulders, inverts for ditches and culverts, edges and toes for embankments and cuttings. Each cross section drawings shall be associated with areas for cut and fill.

10.4.8 Road Furniture

Road Furniture includes road signs, kerbstones, noise barriers and guardrails. These drawings shall include but not limited to the necessary information for the road construction accessories including noise barriers, road marking, guardrails, road signs, and kerbstones.

Signs shall be referred to by their name or type numbers, together with a small-scale illustration of the sign face. Road markings shall be designated by their respective type.

10.4.9 Longitudinal Drainage

All drainage services along the road shall show in longitudinal drainage drawing. A scale exaggeration of 5:1 is regularly draw the drainage longitudinal profile drawings. Regularly the normal scale is horizontal 1:1000 and vertical 1:200.

10.4.10 Utilities

For existing utilities such as electric cable, water, communication cables and sewerage, etc. shall be shown and overlaid on the plan and profile drawings. Existing services that are to be retained must be recognised together with services that require special construction treatment, to be relocated, to be replaced or new services to be constructed.

10.4.11 Structures

For drainage structures and amenities drawings such as bridges should be clear and adequate to allow for their construction. The bridges and cross drainage structures schedule should be provided to show, the proposed and the existing structures and their location and features. For the smooth construction of work provided the invert levels for culvert, side drains and other drainage structures. Information on existing bridges should include their bridge numbers.

The structural drawings should include the following:

10.4.11.1 Bridge

Final drawings for a bridge should consist of a site plan, a general arrangement plan and elevation drawing, foundation plan, substructure drawings, superstructure drawings, deck elevation plan and tabulation, and boring logs. They should be assembled in that general order.

The deck elevation plan shall show finished deck elevations along the centrelines of longitudinal beams or girders, gutter lines, breaks in roadway cross slope, and on tops of parapets. The Bar Bending Schedules for all reinforcements to be used in the bridge shall be provided in the bridge drawings.

10.4.11.2 Culvert

Final drawings for a culvert should consist of a site plan, a general arrangement plan and elevation drawing, culvert cross-section, wing wall cross sections, detail drawings, roadway surface elevation plan and tabulation (if top of culvert is roadway surface), and boring logs (if any). It must also show any inlet and outlet protection measures proposed.

10.4.11.3 Retaining Wall

Final drawings for a retaining wall should consist of a site plan, construction plans and elevation sheets, detail drawings, and boring logs (if available).

10.4.11.4 Numbers for Structural Drawings

A sub type may be used after the drawing type in structure drawings; this may include the following abbreviation for different structural element:

- GEN - General Arrangement
- FON - Foundation Layout
- RCP - Reinforced Concrete Pile
- R - Reinforcement
- SS - Superstructure
- MD - Details for Bearing, Expansion Joint, Drain Pipes, Approach Slab, Railing, etc.
- BB - Bar Bending Schedule

Table 10.5: Scales for Structural Drawings

Drawing Type	Scale
General Arrangement and Foundation Layout	1:50 / 1:100 / 1:200
Reinforced Concrete Pile, Abutment and Pier Reinforcement, Superstructure	1:25 / 1:50
Details for Bearing, Expansion Joint, Drain Pipes, Approach Slab, Railing etc.	1:5 / 1:10 /1:20

10.4.12 Traffic Signals, Street Lighting and Electrical Works

The drawings that provide all details required by a contractor to construct and erect traffic signals, street lighting and all other electrical works required of the project road. The drawings scale are normally variable depending to the amount of details required. However, the drawings should be visually proportional and legible.

10.4.13 Landscaping

The landscaping drawings are used to show the planned roadside beautification to be implemented. Such drawings will normally show:

- Existing trees or vegetation which shall be protected during the construction period
- Which areas shall undergo a special landscape beautification and a specification of the details of the beautification measures?

The scale of the drawings is normally variable depending to the amount of details required.

10.4.14 Soils and Geological maps and details

Soil and Geological maps/drawings assist the contractor to identify designed homogenous sections of subgrade of the road project. It also enables contractors to identify the locations for good construction materials. The drawings shall also include drawings that indicate positions and details obtained during geotechnical investigations activity at bridge location and at any other location where such investigations were carried out. The drawings scale are normally variable depending to the amount of details required.

10.4.15 Land Acquisitions

The basis for the drawings showing new land to be acquired are the plan and profile drawings. The boundary of the land needed for the project may be shown as a dotted line with some color. Existing property boundaries must be shown together with an identification of the owner together with the plot number, if possible. Land to be purchased may also be identified on the ground.

The drawing should also show a list containing all plots that are shown on the drawing together with the names and addresses of the owners where possible. The same list shall also show the area to be acquired.

10.4.16 Standard Drawings

These are standard drawings and information which are already available to the road authority and are required to be included in the design drawings e.g. Consultant’s office and accommodation, weighbridge stations, pits, guard fencing etc.

10.4.17 Conflict areas

These are drawings indicating conflict areas between different activities or facilities. These drawings help the designers and clients to visualize possible conflict interaction between different facilities in the construction areas and plan construction on the best way to minimize overlapping of activities and facilities to enable smooth construction of roads with minimum effect to facilities and activities.

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