

Highway Geometric Design Code

CHAPTER TITLES	PAGE
<u>CHAPTER 1:</u> FUNCTIONAL CLASSIFICATION OF HIGHWAYS	1/1
<u>CHAPTER 2:</u> DESIGN VEHICLES, DRIVERS, AND PEDESTRIAN CHARACTERISTICS	2/1
<u>CHAPTER 3:</u> TRAFFIC FLOW CHARACTERISTICS	3/1
<u>CHAPTER 4:</u> SIGHT DISTANCES	4/1
<u>CHAPTER 5:</u> HORIZONTAL ALIGNMENT	5/1
<u>CHAPTER 6:</u> VERTICAL ALIGNMENT	6/1
<u>CHAPTER 7:</u> CROSS SECTION ELEMENTS	7/1
<u>CHAPTER 8:</u> GENERAL DESIGN CONSIDERATIONS FOR DIFFERENT HIGHWAY CLASSES AND PARKING FACILITIES	8/1
<u>CHAPTER 9:</u> AT-GRADE INTERSECTIONS	9/1
<u>CHAPTER 10:</u> GRADE SEPARATIONS AND INTERCHANGES	10/1
<u>CHAPTER 11:</u> TRAFFIC CONTROL DEVICES	11/1

HIGHWAY GEOMETRIC DESIGN CODE

CONTENTS	PAGE
CHAPTER 1: FUNCTIONAL CLASSIFICATION OF HIGHWAYS	
1-1	General Definitions 1/1
1-2	Functional Classification as a Design Type 1/2
1-2/1	Local Rural Roads and Urban Streets 1/2
1-2/2	Collectors (Rural and Urban) 1/2
1-2/3	Minor Arterials (Rural and Urban) 1/2
1-2/4	Freeways and Expressways 1/2
1-3	Highway Network in Iraq 1/4
1-3/1	General Rural Highways in Iraq 1/4
1-3/2	Road Lengths Between Main Cities 1/4
1-3/3	Typical Urban Highway Systems, (Baghdad City) 1/4
1-4	References 1/9
CHAPTER 2: DESIGN VEHICLES, DRIVERS, AND PEDESTRIAN CHARACTERISTICS	
2-1	Vehicles 2/1
2-1/1	Design Vehicle Dimensions 2/1
2-1/2	General Guide for Selection of Design Vehicle 2/4
2-1/3	Minimum Turning Paths of Design Vehicle 2/4
2-1/4	Max. Axle Loads and Gross Weights of Trucks 2/11
2-1/5	Passenger Car Unit Equivalents 2/14
2-1/6	Resistances Acting on a Vehicle in Motion 2/15
2-1/7	Acceleration and Deceleration Performance of Vehicles 2/17
2-2	Drivers 2/19
2-2/1	General Drivers Characteristics 2/19
2-2/2	Driver Perception-Reaction Time Performance 2/19
2-3	Pedestrians 2/21
2-3/1	Pedestrian Characteristics 2/21
2-3/2	Level of Service for Pedestrian Walkways 2/22
2-3/3	Pedestrian Crossings 2/24
2-4	References 2/24
CHAPTER 3: TRAFFIC FLOW CHARACTERISTICS	
3-1	Traffic Volume 3/1
3-1/1	Current Annual Average Daily Traffic 3/1

CONTENTS	PAGE
3-1/2 Projection of Future Traffic Demands	3/1
3-1/3 Design Hourly Volume	3/1
3-2 Traffic Composition and Directional Distribution	3/2
3-3 Speed	3/3
3-3/1 Spot Speed	3/3
3-3/2 Overall Travel Speed	3/4
3-3/3 Running (Operating) Speed	3/4
3-3/4 Design Speed	3/4
3-4 Traffic Flow	3/5
3-4/1 Peak Flow Rate and Peak Hour Factor	3/5
3-4/2 Traffic Density, Spacing, and Time Headway	3/6
3-4/3 Traffic Volume-Speed-Density Relationships	3/6
3-5 Highway Capacity	3/8
3-5/1 Highway Capacity as a Design Control	3/8
3-5/2 Levels of Service	3/8
3-5/3 Design Service Flow Rate for Highways	3/9
3-6 References	3/14
CHAPTER 4: SIGHT DISTANCES	
4-1 Stopping Sight Distance	4/1
4-1/1 Perception Reaction Distance	4/1
4-1/2 Braking Distance on Level Roadway	4/1
4-1/3 Braking Distance on a Grade	4/2
4-1/4 Design Values of Stopping Sight Distance	4/2
4-2 Decision Sight Distance	4/4
4-2/1 Distances where a (Stop) is the Appropriate Avoidance Maneuver	4/4
4-2/2 Distances where (Speed, Path, and Direction) Change is the Appropriate Avoidance Maneuver	4/4
4-2/3 Design Values of Decision Sight Distance	4/4
4-3 Passing Sight Distance	4/6
4-3/1 Elements of Passing Sight Distance	4/6
4-3/2 General Equation of Total Passing Sight Distance	4/6
4-3/3 Design Values of Passing Sight Distance	4/7
4-4 Criteria for Measuring Sight Distances	4/8

CONTENTS	PAGE
4-5 References	4/9
CHAPTER 5: HORIZONTAL ALIGNMENT	
5-1 Horizontal Circular Curves	5/1
5-1/1 Minimum Radius of Circular Curves	5/1
5-1/2 Maximum Superelevation rates	5/4
5-1/3 Side Friction Factors	5/4
5-1/4 Compound Circular Curves for Ramps and Intersections	5/4
5-2 Spiral Curve Transitions	5/4
5-2/1 Details of Spiral Compounded with Circular Curve	5/6
5-2/2 Maximum Radius for Use of a Spiral Curve Transition	5/6
5-2/3 Minimum Length of Spiral Curve	5/6
5-2/4 Maximum Length of Spiral Curve	5/8
5-2/5 Desirable Length of Spiral Curve	5/8
5-3 Superelevation on Curves	5/10
5-3/1 Methods of Attaining Superelevation	5/10
5-3/2 Minimum Length of Superelevation Runoff (Tangent-To-Curve Transition Design)	5/10
5-3/3 Minimum Length of Tangent Runout (Tangent-To-Curve Transition Design)	5/12
5-3/4 Length of Superelevation Runoff and Tangent Runout for Spiral-Curve Transition	5/15
5-4 Widening on Horizontal Curves	5/16
5-4/1 Design Values of Traveled Way Widening	5/16
5-4/2 Application of Widening on Curves	5/20
5-5 Design Widths for Turning Roadways or Ramps at Intersections	5/20
5-6 Sight Distance on Horizontal Curves	5/23
5-7 General Controls for Horizontal Alignment	5/25
5-8 References	5/25
CHAPTER 6: VERTICAL ALIGNMENT	
6-1 Control Grades for Design	6/1
6-1/1 Maximum Grades	6/1
6-1/2 Minimum Grades	6/1
6-1/3 Critical Length of Upgrade	6/3

CONTENTS	PAGE
6-2 Added Lanes and Turnouts on Two-Lane Highways	6/3
6-2/1 Climbing Lanes	6/3
6-2/2 Passing Lanes	6/5
6-2/3 Turnouts	6/6
6-3 Emergency Escape Ramps	6/7
6-4 Vertical Parabolic Curves (Symmetrical and Unsymmetrical)	6/10
6-5 Minimum Lengths of Crest Vertical Curves	6/11
6-5/1 Minimum Lengths to Provide Stopping Sight Distance	6/14
6-5/2 Design Rate of Vertical Curvature of Crest Curves to Provide Safe Stopping	6/14
6-5/3 Minimum Lengths to Provide Passing Sight Distance	6/14
6-5/4 Design Rate of Vertical Curvature of Crest Curves to Provide Safe Passing	6/15
6-6 Minimum Lengths of Sag Vertical Curves	6/16
6-6/1 Safety Criteria (Headlight Sight Distance)	6/16
6-6/2 Design Rate of Vertical Curvature of Sag Curves	6/17
6-6/3 Comfort Criteria	6/18
6-6/4 General Appearance Criteria	6/18
6-6/5 Drainage Control Criteria	6/19
6-7 Minimum Lengths of Sag Vertical Curves Undercrossing a Grade Separation Structure	6/19
6-8 General Controls for Vertical Alignment	6/19
6-9 General Design Controls for Combinations of Horizontal Alignment and Profile	6/21
6-10 References	6/23
CHAPTER 7: CROSS SECTION ELEMENTS	
7-1 Lane Width and Marginal Strip	7/1
7-2 Cross Slope	7/1
7-3 Shoulders	7/2
7-4 Medians	7/3
7-5 Side Slopes	7/3
7-6 Drainage Channels	7/3
7-7 Curbs and Gutters	7/5
7-8 Sidewalks	7/6

CONTENTS	PAGE
7-9 Bikeways	7/6
7-10 Right-of-Way	7/8
7-11 Horizontal and Vertical Clearances	7/8
7-12 On – Street Parking	7/11
7-13 Longitudinal Barriers and Crash Cushions	7/11
7-14 Bus stop Turnouts	7/15
7-15 Typical Cross Sections	7/16
7-15/1 Two-Lane Rural Highways	7/16
7-15/2 Four -Lane Rural Highways	7/16
7-15/3 Six-Lane Rural Highways	7/16
7-15/4 Four-Lane Urban Streets	7/17
7-15/5 Overpassing Bridges	7/17
7-15/6 Interchange Ramp (Turning Roadway)	7/17
7-15/7 Tunnels	7/17
7-16 Utilities	7/21
7-17 References	7/21
CHAPTER 8: GENERAL DESIGN CONSIDERATIONS FOR DIFFERENT HIGHWAY CLASSES AND PARKING FACILITIES	
8-1 Local Rural Roads	8/1
8-1/1 Selected Design Speed	8/1
8-1/2 Maximum Grades	8/1
8-2 Local Urban Streets	8/2
8-2/1 Selected Design Speed	8/2
8-2/2 Maximum and Minimum Grades	8/2
8-2/3 Cul-de-Sacs and Turnarounds	8/2
8-2/4 Minimum Levels of Illumination	8/2
8-3 Rural Collector Highways	8/4
8-3/1 Selected Design Speed	8/4
8-3/2 Maximum Grades	8/4
8-4 Urban Collector Streets	8/5
8-4/1 Selected Design Speed	8/5
8-4/2 Maximum and Minimum Grades	8/5

CONTENTS	PAGE
8-5 Rural Arterials	8/6
8-5/1 Selected Design Speed	8/6
8-5/2 Maximum Grades	8/6
8-5/3 Typical Medians on Divided Arterials	8/6
8-5/4 Attaining Superelevated Cross Sections for Divided Arterials	8/7
8-5/5 Cross Sectional Arrangements for Divided Arterials	8/7
8-6 Urban Arterials	8/9
8-6/1 Selected Design Speed	8/9
8-6/2 Maximum and Minimum Grades	8/9
8-7 Freeways	8/9
8-7/1 Selected Design Speed	8/9
8-7/2 Maximum Grades	8/9
8-7/3 Typical Rural Freeway Medians	8/9
8-7/4 Cross Sections for Depressed Urban Freeways	8/11
8-7/5 Cross Sections for Elevated Urban Freeways	8/11
8-7/6 Cross Sections for Ground-Level Urban Freeways	8/13
8-7/7 Cross Sections for Combination-Type Urban Freeways	8/13
8-8 Off – Street Parking Facilities	8/15
8-8/1 Location of Parking Lots and Garages	8/15
8-8/2 General Design Criteria of Parking Garages	8/15
8-8/3 Parking Stall layout at Various Angles	8/16
8-9 References	8/17
CHAPTER 9: AT-GRADE INTERSECTIONS	
9-1 Basic Types of At-Grade Intersections	9/1
9-1/1 Three-Leg	9/1
9-1/2 Four-Leg	9/2
9-1/3 Multileg	9/2
9-1/4 Roundabouts	9/3
9-2 Island Details	9/6
9-3 Minimum Turning Roadway Design with Corner Islands and Different Angle Turns	9/12

CONTENTS	PAGE
9-4 Development of Superelevation at Turning Roadway Terminals	9/25
9-5 Clear Sight Triangles	9/30
9-6 Design of Median Openings	9/34
9-7 Auxiliary Lane Lengths for Turning Vehicles	9/39
9-7/1 Entering Taper	9/39
9-7/2 Deceleration Length	9/41
9-7/3 Storage Length	9/41
9-8 Minimum Design of U-Turns	9/41
9-9 Railroad-Highway Grade Crossings	9/43
9-10 Signalized Intersection	9/45
9-10/1 Measure of Effectiveness	9/45
9-10/2 Traffic Operations Elements	9/45
9-10/3 Operational Analysis	9/46
9-11 References	9/46
CHAPTER 10: GRADE SEPARATIONS AND INTERCHANGES	
10-1 General Interchange Configurations	10/1
10-1/1 Three-Leg Trumpet and Directional Interchanges	10/1
10-1/2 Diamond Interchanges	10/3
10-1/3 Single-Point Urban Interchanges	10/3
10-1/4 Rotary Interchanges	10/6
10-1/5 One-Quadrant Interchanges	10/6
10-1/6 Partial and Full Cloverleaf Interchanges	10/7
10-1/7 All Directional and Semi-directional Interchanges	10/7
10-2 Freeway Interchanges with other Highway Classes	10/10
10-3 Grade Separation Structures	10/10
10-4 Longitudinal Distance to Attain Grade Separation	10/11
10-5 Effective Distance of Auxiliary Lanes	10/12
10-6 Design Speed and Grades for Ramps	10/13
10-7 Development of Superelevation at Ramp Terminals	10/13
10-8 Recommended Minimum Ramp Terminal Spacing	10/15
10-9 Minimum Acceleration Lengths of Speed-Change Lanes (Single	10/16
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CONTENTS	PAGE
Lane, Entrance Ramp)	
10-10 Minimum Deceleration Lengths of Speed-Change Lanes (Single Lane, Exit Ramp)	10/18
10-11 Typical Two-Lane Entrance and Exit Ramp Terminals	10/19
10-12 References	10/20
CHAPTER 11: TRAFFIC CONTROL DEVICES	
11-1 Traffic Control Signals	11/1
11-1/1 Types of Traffic Control Signals	11/1
11-1/1/1 Pretimed Control	11/1
11-1/1/2 Traffic-Actuated Control	11/1
11-1/2 Traffic Control Signal Features	11/2
11-1/2/1 Signal Indication	11/2
11-1/2/2 Size and Design of Lenses	11/4
11-1/2/3 Sequence of Indication	11/6
11-1/2/4 Illumination of Lenses	11/10
11-1/2/5 Visibility and Shielding of Signal Faces	11/10
11-1/2/6 Number and Location of Signal Faces	11/11
11-1/2/7 Height of Signal Faces	11/13
11-1/2/8 Yellow Change and Red Clearance Interval	11/15
11-1/2/9 Coordination of Signals	11/16
11-1/2/10 Flash Operation of Signal	11/17
11-1/3 Conditions that Warrant the Installation of a Traffic Signal	11/17
11-1/4 Pedestrian control Signals	11/22
11-1/4/1 Pedestrian Signal Head Indications	11/22
11-1/4/2 Pedestrian Signal Phases and Interval Time	11/24
11-1/4/3 Location and Height of Pedestrian Signal Heads	11/24
11-1/5 Flashing Beacons	11/25
11-1/5/1 Warning Beacon	11/25
11-1/5/2 Intersection Control Beacon	11/25
11-1/5/3 Stop Beacon	11/26
11-1/5/4 Speed Limit Beacon	11/26
11-2 Traffic Signs	11/27

CONTENTS	PAGE
11-2/1 Traffic Signs Characteristics	11/27
11-2/1/1 Function and Purpose of Signs	11/27
11-2/1/2 Sign Shapes	11/27
11-2/1/3 Sign Colours	11/28
11-2/1/4 Sign Boarder	11/28
11-2/1/5 Retroreflection and Illumination	11/28
11-2/1/6 Sign Dimensions	11/28
11-2/2 Traffic Signs Placement	11/30
11-2/2/1 Mounting Height	11/30
11-2/2/2 Lateral Offset	11/30
11-2/2/3 Longitudinal Placement	11/33
11-2/2/4 Overhead Sign Installation	11/33
11-2/2/5 Orientation Angle	11/34
11-2/2/6 Post and Mounting	11/34
11-2/3 Regularity Signs	11/36
11-2/3/1 Control Signs	11/36
11-2/3/2 Mandatory Signs	11/38
11-2/3/3 Prohibitory Signs	11/41
11-2/3/4 Parking Control Signs	11/45
11-2/4 Warning Signs	11/47
11-2/4/1 Advanced Warning Signs	11/47
11-2/4/2 Hazard Marker Signs	11/52
11-2/4/3 Diagrammatic Signs	11/54
11-2/4/4 High Vehicle Warning Signs	11/55
11-2/5 Guide Signs	11/55
11-2/5/1 Destination Signs	11/55
11-2/5/2 Distance Signs	11/57
11-2/5/3 Information Signs	11/57
11-2/5/4 General Standards for Guide Signs	11/59
11-2/5/4/1 Language and Letter Style	11/59
11-2/5/4/2 Size of Lettering	11/60

CONTENTS	PAGE
11-2/5/4/3 Sign Boarder	11/62
11-2/5/4/4 Arrows	11/62
11- 3 Road Marking	11/63
11-3/1 Colour, Code and Material	11/63
11-3/2 Longitudinal Lines for Pavement Marking	11/64
11-3/2/1 Center Line	11/64
11-3/2/2 Lane Line	11/65
11-3/2/3 Edge Line	11/66
11-3/2/4 No Passing Line	11/67
11-3/2/5 Safety Line	11/69
11-3/3 Transverse Lines for Pavement Marking	11/70
11-3/3/1 Stop Line	11/70
11-3/3/2 Pedestrian Crossing Lines	11/71
11-3/3/3 Give Way Lines	11/71
11-3/4 Other Pavement Marking	11/72
11-3/4/1 No Overtaking Area	11/72
11-3/4/2 Arrows Marking	11/73
11-3/4/3 Words and Numerals	11/77
11-3/4/4 Approaches to Railway Crossing	11/79
11-3/4/5 Speed Hump Marking	11/80
11-3/4/6 Object Marker	11/81
11-3/4/7 Parking Marking	11/85
11-3/4/8 Raised Pavement Markers	11/86
11-3/4/9 Continuity Marking	11/87
11-3/5 Curb Painting	11/88
11-3/6 Delineators	11/89
11-3/7 Barricades and Channelization Devices	11/91
11-3/8 Control Devices Used in Work Zones	11/92
11-4 References	11/96

CHAPTER1

FUNCTIONAL CLASSIFICATION OF HIGHWAYS

1-1 GENERAL DEFINITIONS

The definitions for specific terms used frequently in conjunction with highway improvements are listed: [2, p.629]

- **Highway, Street, or Road-** A general term denoting a public way for purposes of vehicular travel, including the entire area within the right-of- way. (Recommended usage: In urban areas – highway or street In rural areas – highway or road)
 - Urban Areas: Places within boundaries, having a population of 5000 or more.
 - Rural Areas: Areas outside the boundaries of urban areas
- **Roadway:** The portion of a highway, including shoulders, for vehicular use. A divided highway has two or more roadways.
- **Traveled way:** The Portion of the roadway for the movement of vehicles, exclusive of shoulders and auxiliary lanes.
- **Arterial highway:** A general term denoting a highway primarily for through traffic, usually on a continuous route.
- **Expressway:** A divided arterial highway for through traffic with full or partial control of access and generally with grade separations at intersections.
- **Freeway:** An Expressway with full control of access.
 - **Control of access-** The condition where the right of owners or occupants of abutting land or other persons to access, light, air, or view in connection with a highway, is fully or partially controlled by public authority.
 - **Full control of access** means that the authority to control access is exercised to give preference to through traffic by providing access connections with selected public roads only and by prohibiting crossings at grade or direct driveway connections.
 - **Partial control of access** means that the authority to control access is exercised to give preference to through traffic to a degree that, in addition to access connections with selected public roads, there may be some crossings at grade and some private driveway connection.
- **Local street or Local road:** A street or road primarily for access to residence, business, or other abutting property.
- **Frontage street or Frontage road:** A local street or road auxiliary to and located on the side of an arterial highway for service to abutting property and adjacent areas and for control of access.

- **Cul-de- sac street:** A local street open at one end only and with special provision for turning around.
- **Dead – end street:** A local street open at one end only without special provision for turning around.

1-2 FUNCTIONAL CLASSIFICATION AS A DESIGN TYPE

Functional classification of highways based on geometric features, is the most helpful approach for highway planning, location, and design; taking into consideration land access and travel mobility for the desired level as service [1,p.1-8].

The functional systems consist of local roads and streets, collectors, minor and principal arterials.

1-2/1 LOCAL RURAL ROADS AND URBAN STREETS

Local roads and Streets emphasize the land (Property) access function, with short trip lengths, lower operating speed and lower level of service.

1-2/2 COLLECTORS (RURAL AND URBAN)

The Collectors offer balanced service for both land access and mobility, with intermediate operating speed and level of service. They collect traffic from local system and channels it into arterials.

1-2/3 MINOR ARTERIALS (RURAL AND URBAN)

Minor Arterials or Distributors emphasize a high proportion of mobility for through movement, with longer trip length, high operation speed and level of service.

1-2/4 FREEWAYS AND EXPRESSWAYS

They are considered as (Principal Arterials) for main trip movements, with higher operating speed and higher level of service.

The relationship of functionally classified system of highways, in serving land access and travel mobility is shown in figure (1-2/1).

Access management calls for coordinating the planning and design of highway and its surrounding activities to preserve the capacity of the overall system, and to allow efficient and safe access to and from the activities [1,p.2-71].

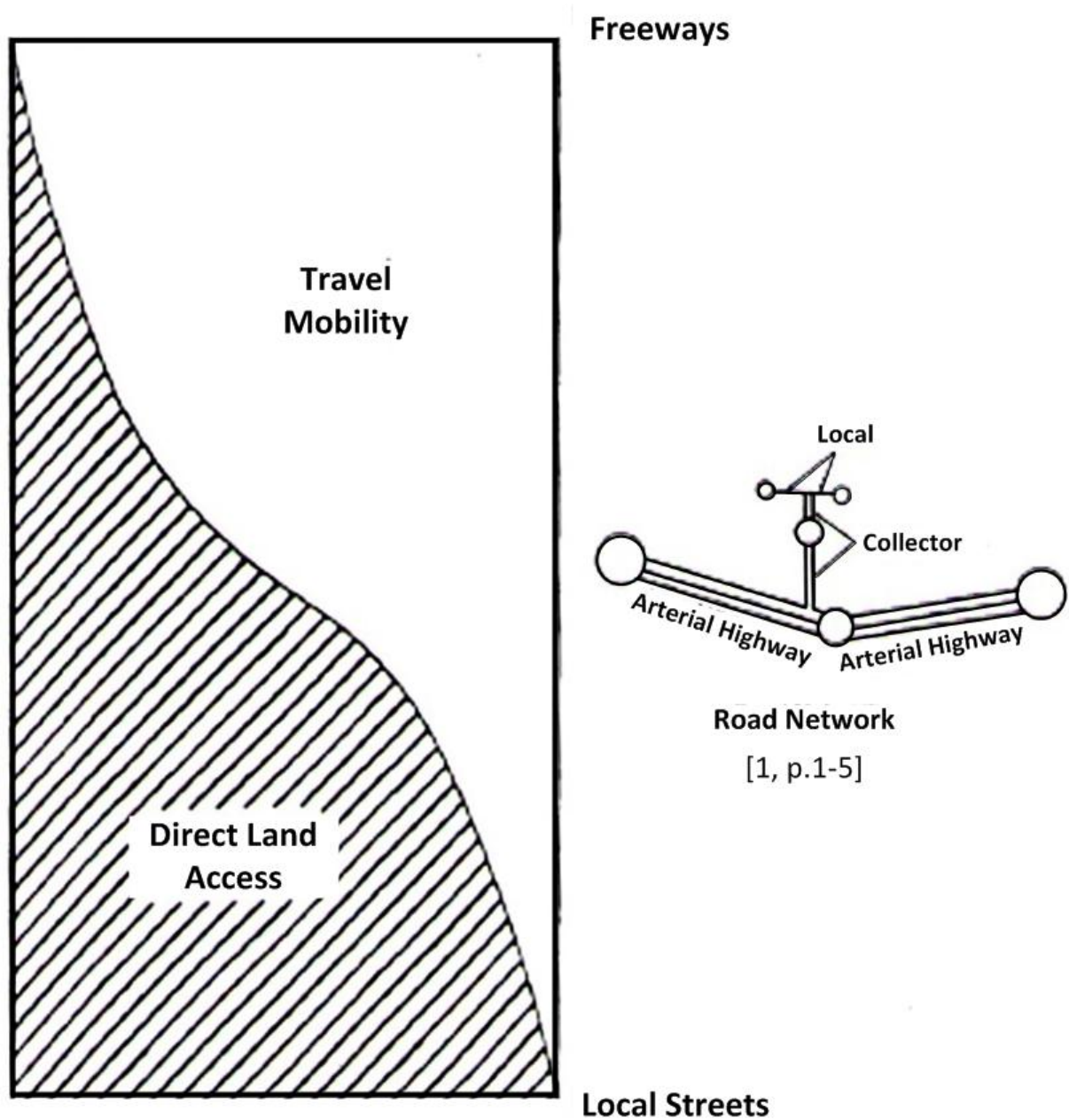


Figure 1-2/1: Relationship of Functionally Classified Systems in Serving Traffic Mobility and Land Access [1, p. 1-7]

1-3 HIGHWAY NETWORK IN IRAQ

1-3/1 GENERAL RURAL HIGHWAYS IN IRAQ

The rural highways connecting the (18) provinces in Iraq, with an approximate total length of (45) thousand kilometers, are shown in figure (1-3/1).

Among the most distinguished highways is Iraq Expressway No. 1, with a length of about (1200) km, connecting the borders with Syria, Jordan, and Kuwait, as shown in figure (1-3/2).

1-3/2 ROAD LENGTHS BETWEEN MAIN CITIES

The detailed lengths of roads between the main cities of the (18) provinces in Iraq are shown in table (1-3/1).

1-3/3 TYPICAL URBAN HIGHWAY SYSTEMS, (BAGHDAD CITY)

The current and recommended highway networks in Baghdad city, together with the suggested designs of Baghdad underground Metro, and Railway loop line, as presented in the comprehensive Transportation Study for Baghdad City, by Scott Wilson Kirkpatrick & Partners, 1986, are shown in figure (1-3/3).

The designs need to be updated in accordance with current demographic and socio-economic data.

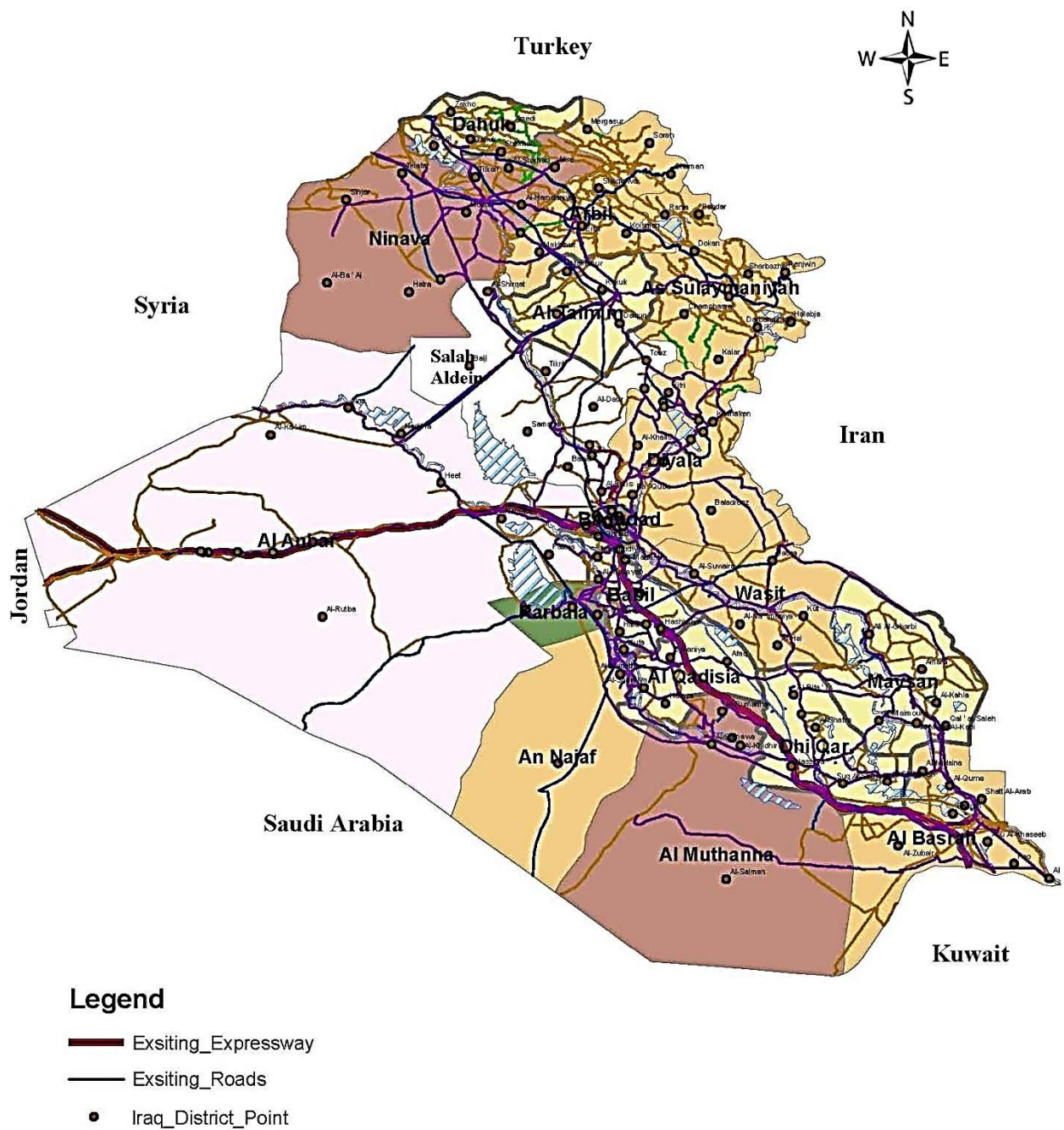


Figure 1-3/1: Roads Map of Iraq [4]

Table 1-3/1: Roads Lengths (km) Between Main Cities [4]

دهوك DOHUK	الموصل MOSUL	اربيل ERBIL	السليمانية SULAIMANIYA	تكريت TKRIT	كركوك KIRKUK	بعقوبة BAQUBA	بغداد BAGHDAD	الرمادي RAMADI	الحلة HILLA	كربلاء KERBALA	الكويت KUT	النجف NAJAF	الناصرية NASIRIYA	العمارة AMARA	الساموة SAMAWA	الديوانية DIWANIYA	البصرة BASRAH
69	153	84	202	112	215	86	110	210	42	280	333	258	384	489	89	402	
153	355	286	205	109	255	176	166	108	218	61	78	187	527	105	194	313	
84	290	221	93	267	365	355	184	172	282	272	317	197	155	65	182		
202	248	177	308	331	285	275	372	238	161	271	485	484	292	292	208		
112	531	393	350	460	442	293	427	227	375	476	466	484	353	377			
215	465	396	308	471	431	275	347	416	441	386	476	484	353	377			
86	575	506	350	496	443	293	347	227	375	476	466	484	353	377			
110	565	496	350	504	488	503	492	336	432	366	476	484	353	377			
210	583	504	460	552	552	503	492	336	432	366	476	484	353	377			
42	637	588	471	583	552	503	492	336	432	366	476	484	353	377			
280	628	557	431	583	552	503	492	336	432	366	476	484	353	377			
333	840	771	443	583	552	503	492	336	432	366	476	484	353	377			
258	831	725	443	583	552	503	492	336	432	366	476	484	353	377			
384	735	666	443	583	552	503	492	336	432	366	476	484	353	377			
489	646	577	443	583	552	503	492	336	432	366	476	484	353	377			
89	1014	945	443	583	552	503	492	336	432	366	476	484	353	377			
402			443	583	552	503	492	336	432	366	476	484	353	377			

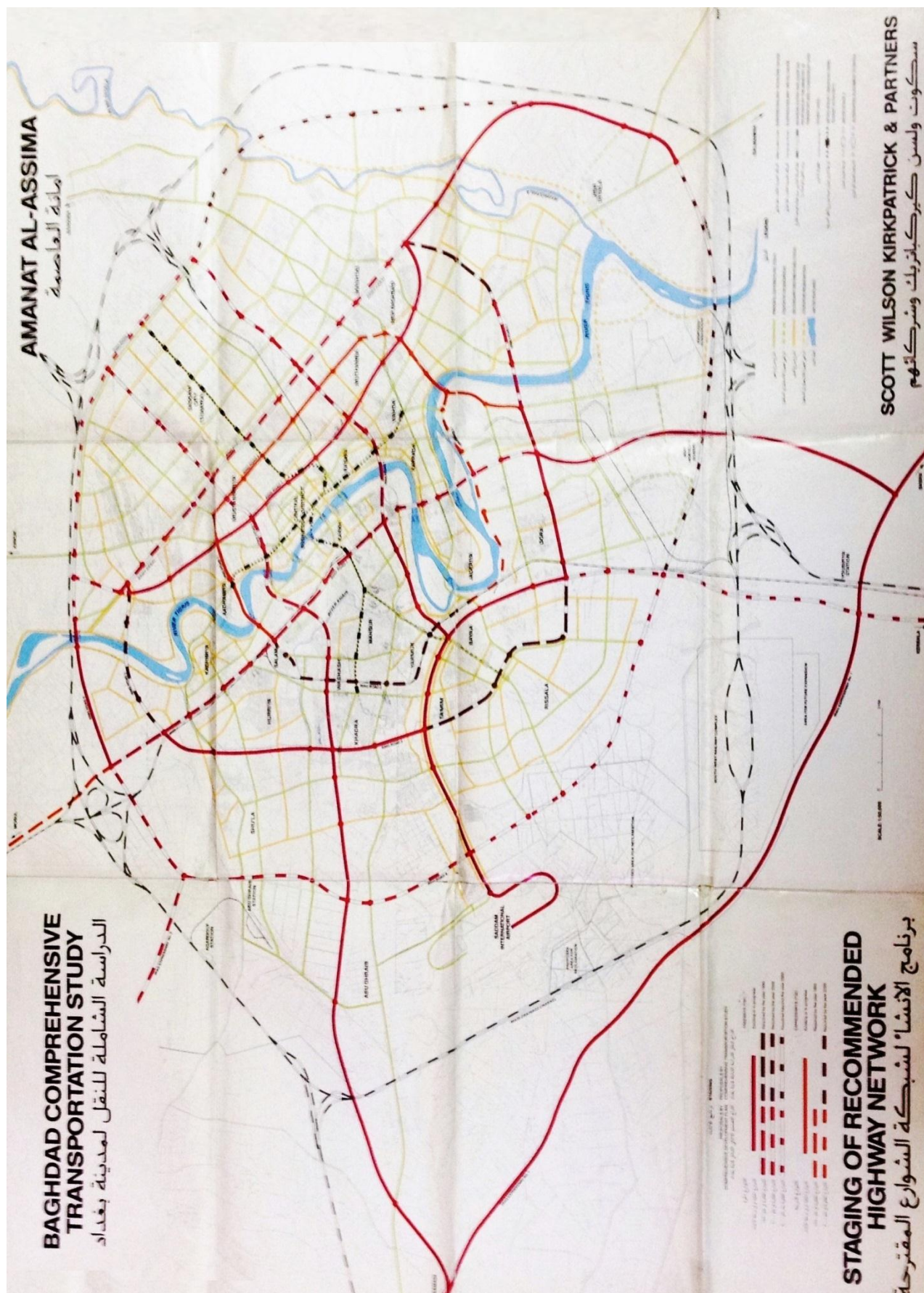


Figure 1-3/3: Recommended Highway Network for Baghdad (1986) [5]

1-4 REFERENCES

- [1] AASHTO, "*A Policy on Geometric Design of Highways and Streets*", American Association of State Highway and Transportation Officials, USA, 2011.
- [2] AASHO, "*A Policy on Geometric Design of Rural Highways*", American Association of State Highway Officials, USA, 1961.
- [3] State Corporation of Roads & Bridges, "*Guide of Rural Roads in Iraq*", 2001.
- [4] State Corporation of Roads & Bridges, "*Roads Map of Iraq*", 2007.
- [5] Scott Wilson Kirkpatrick & Partners, "*Baghdad Comprehensive Transportation Study*", Amanat Al- Assima, Baghdad, 1986).

CHAPTER 2

DESIGN VEHICLES, DRIVERS, AND PEDESTRIAN CHARACTERISTICS

2-1 VEHICLES

Among the main factors affecting highway geometric design, are the physical properties and proportions of different vehicles. From each general class groupings, a selected design vehicle is accommodated with representative weight, dimensions, turning radius, and operating characteristics.

2-1/1 DESIGN VEHICLE DIMENSIONS

Design Vehicles are usually grouped into general classes of Passenger Cars, Buses, Single-Unit Trucks, Combination Trucks, and Recreational Vehicles. The maximum design vehicle dimensions for the main vehicle types are shown in table (2-1/1).

The dimensions of (20) design vehicles given by AASHTO Policy [1, p.2-3] are shown in table (2-1/2).

Table 2-1/1: Maximum Dimension for the Main Vehicle Types

Design Vehicle Type	Maximum Dimensions, Meters		
	Width	Height	Length
• Passenger Cars	2.13	1.30	5.79
• Buses	2.44-2.59	3.20-3.66	10.91-18.29
• Single- Unit Trucks	2.44	3.35-4.11	9.14-12.04
• Semitrailer/ Trailer Combinations	2.44-2.59	4.11	13.87-34.75
• Recreational Vehicles	2.44	3.05-3.66	9.14-16.15

Figure (2-1/1) shows silhouettes of most basic commercial vehicle types in regular operation as designated by axle arrangement code. The first digit indicates the number of axles of the truck or truck- tractor. The letter "S" indicates a semitrailer, and the digit immediately following an "S" indicates the number of axles on the semitrailer. Any digit, other than the first in a combination, when not preceded by an "S" indicates a trailer and the number of its axles. For instance, a 2-S2 combination is a two – axle truck – tractor with a tandem – axle semitrailer. A 3-S1-2 combination is a three- axle, truck – tractor with tandem rear axles, a semitrailer with a single axle, and a trailer with two axles. [2, p.7]

Table 2-1/2: Design Vehicles Dimensions (SI Units) [1, p.2-3]

Design Vehicle Type	Symbol	Dimensions (m)			
		Overall			Overhang
		Height	Width	Length	Front Rear
Passenger Car	P	1.30	2.13	5.79	0.91 1.52
Single-Unit Truck	SU-9	3.35–4.11	2.44	9.14	1.22 1.83
Single-Unit Truck (three-axle)	SU-12	3.35–4.11	2.44	12.04	1.22 3.20
Buses					
Intercity Bus (Motor Coaches)	BUS-12	3.66	2.59	12.36	1.93 2.73 ^a
	BUS-14	3.66	2.59	13.86	1.89 2.73 ^b
City Transit Bus	CITY-BUS	3.20	2.59	12.19	2.13 2.44
Conventional School Bus (65 pass.)	S-BUS 11	3.20	2.44	10.91	0.79 3.66
Large School Bus (84 pass.)	S-BUS 12	3.20	2.44	12.19	2.13 3.96
Articulated Bus	A-BUS	3.35	2.59	18.29	2.62 3.05
Combination Trucks					
Intermediate Semitrailer	WB-12	4.11	2.44	13.87	0.91 1.37 ^a
Interstate Semitrailer	WB-19	4.11	2.59	21.03	1.22 1.37 ^a
Interstate Semitrailer	WB-20	4.11	2.59	22.40	1.22 1.37 ^a
"Double-Bottom" Semitrailer/Trailer	WB-20D	4.11	2.59	22.04	0.71 0.91
Rocky Mountain Double-Semitrailer/Trailer	WB-28D	4.11	2.59	29.67	0.71 0.91
Triple-Semitrailer/Trailers	WB-30T	4.11	2.59	31.94	0.71 0.91
Turnpike Double-Semitrailer/Trailer	WB-33D	4.11	2.59	34.75	0.71 1.37 ^a
Recreational Vehicles					
Motor Home	MH	3.66	2.44	9.14	1.22 1.83
Car and Camper Trailer	P/T	3.05	2.44	14.84	0.91 3.66
Car and Boat Trailer	P/B	—	2.44	12.80	0.91 2.44
Motor Home and Boat Trailer	MH/B	3.66	2.44	16.15	1.22 2.44

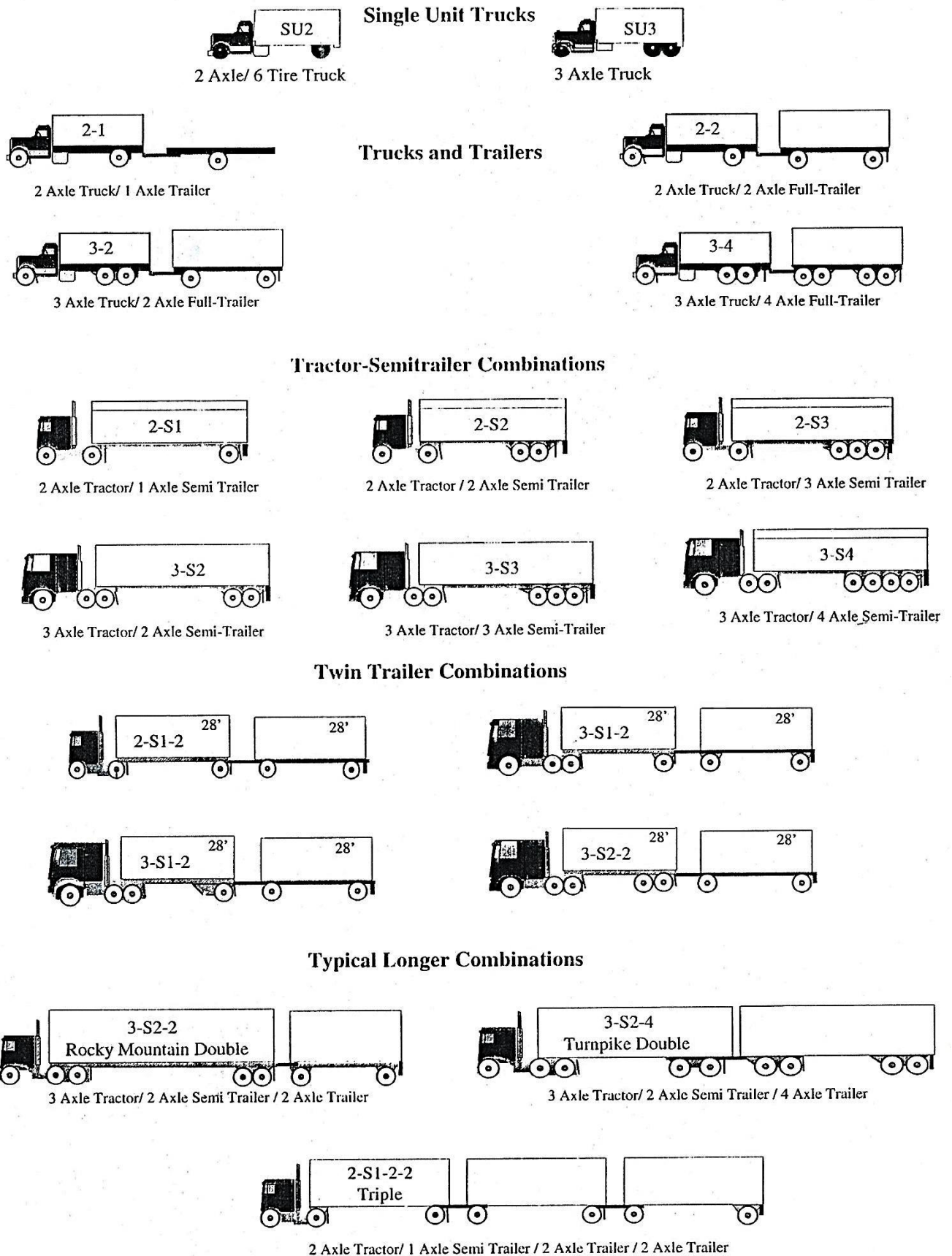


Figure 2-1/1: Common commercial vehicles types as designed by code based on axle arrangement [2, p.8]

2-1/2 General Guide for Selection of Design Vehicle

Design Vehicles: Selected Vehicles, with representative dimensions, weight, and operating characteristics, used to establish highway design controls for accommodating vehicles of designated classes [1, p. 2-1].

For geometric design of highways, each design vehicle has larger physical dimensions and a larger minimum turning radius than most vehicles in its class.

As a general guide, the following may be considered when selecting a design vehicle:

- When the main traffic generator is parking lots, a passenger car may be selected.
- For the design of intersections that mainly serve bus traffic, a city transit bus may be selected.
- For intersection design of residential streets, a single-unit truck may be selected.
- For high volume highways, and for intersections of expressway ramps with arterial crossroads, the WB-20 semitrailer truck should be considered as the minimum size design vehicle.

The selection of a (design vehicle), whose characteristics will encompass nearly all vehicles expected to use the highway, is used to determine criteria for geometric design, intersection design and sight distance requirements. [3, p.63]

2-1/3 Minimum Turning Paths of Design Vehicle

The minimum design turning radii of the different design vehicles are shown in table (2-1/3) together with the centerline and inside turning radius.

Figures (2-1/2) through (2-1/11) present the minimum turning paths for 10 selected design vehicles [1, p.2-5].

Table 2-1/3: Minimum Turning Radii of Design Vehicles [1, p.2-6]

Design Vehicle Type	Pas-senger. Car	Single-Unit Truck	Single-Unit Truck (Three Axle)	Intercity Bus (Motor Coach)		City Tran-sit Bus	Conven-tional School Bus (65 pass.)	Large School Bus (84 pass.)	Articu-lated Bus	Inter-mediate Semi-trailer
Symbol	P	SU-9	SU-12	BUS-12	BUS-14	CITY-BUS	S-BUS11	S-BUS12	A-BUS	WB-12
Minimum Design Turning Radius (m)	7:26	12.73	15.60	12.70	13.40	12.80	11.75	11.92	12.00	12.16
Center-Line Turning Radius (CTR) (m)	6.40	11.58	14.46	11.53	12.25	11.52	10.64	10.79	10.82	10.97
Minimum Inside Radius (m)	4.39	8.64	11.09	7.41	7.54	7.45	7.25	7.71	6.49	5.88
Design Vehicle Type	Interstate Semi-trailer		"Double Bottom" Combina-tion	Rocky Mtn Double	Triple Semi-trailer/trailers	Turnpike Double Semi-trailer / trailer	Motor Home	Car and Camper Trailer	Car and Boat Trailer	Motor Home and Boat Trailer
Symbol	WB-19	W8-20	WB-20D	WB-28D	WB-30T	WB-33D	MH	P/T	P/B	MH/B
Minimum Design Turning Radius (m)	13.65	13.66	13.67	24.98	13.67	18.25	12.11	10.03	7.26	15.19
Center-Line Turning Radius (CTR) (m)	12.50	12.50	12.47	23..77	12.47	17.04	10.97	9.14	6.40	14.02
Minimum Inside Radius (m)	2.25	0.59	5.83	16.94	2.96	4.19	7.92	5.58	2.44	10.67

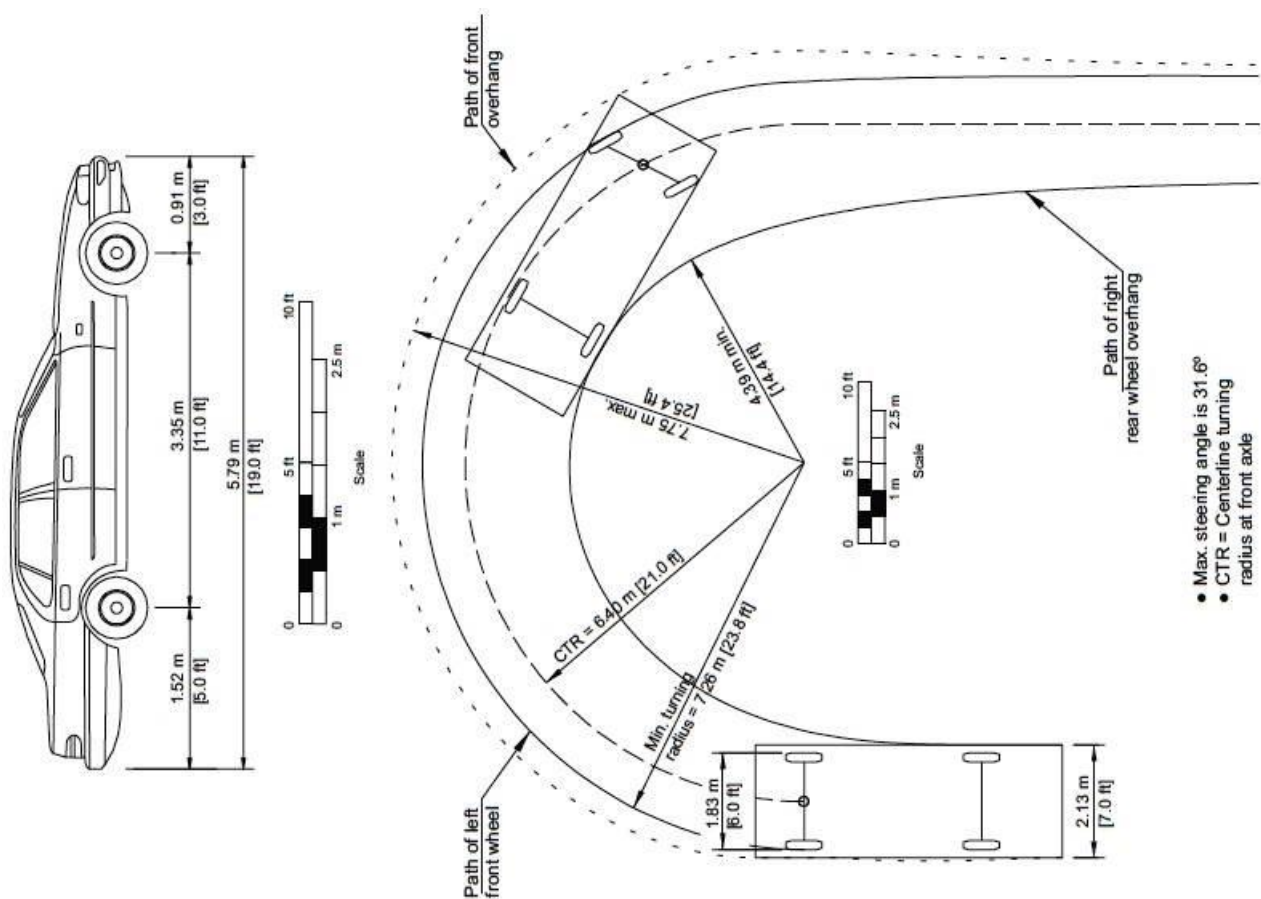


Figure 2-1/2: Minimum Turning Path for Passenger Car (p) Design Vehicles [1, p.2-10]

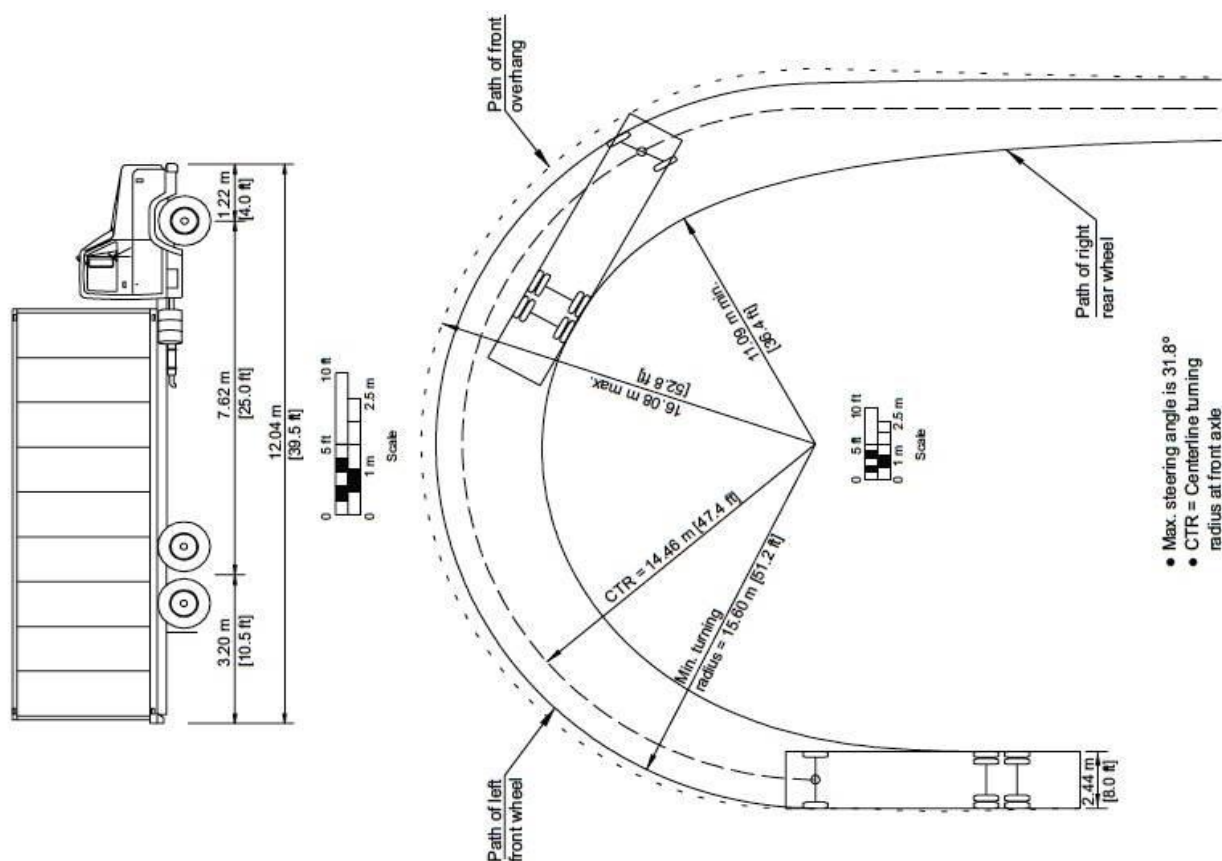
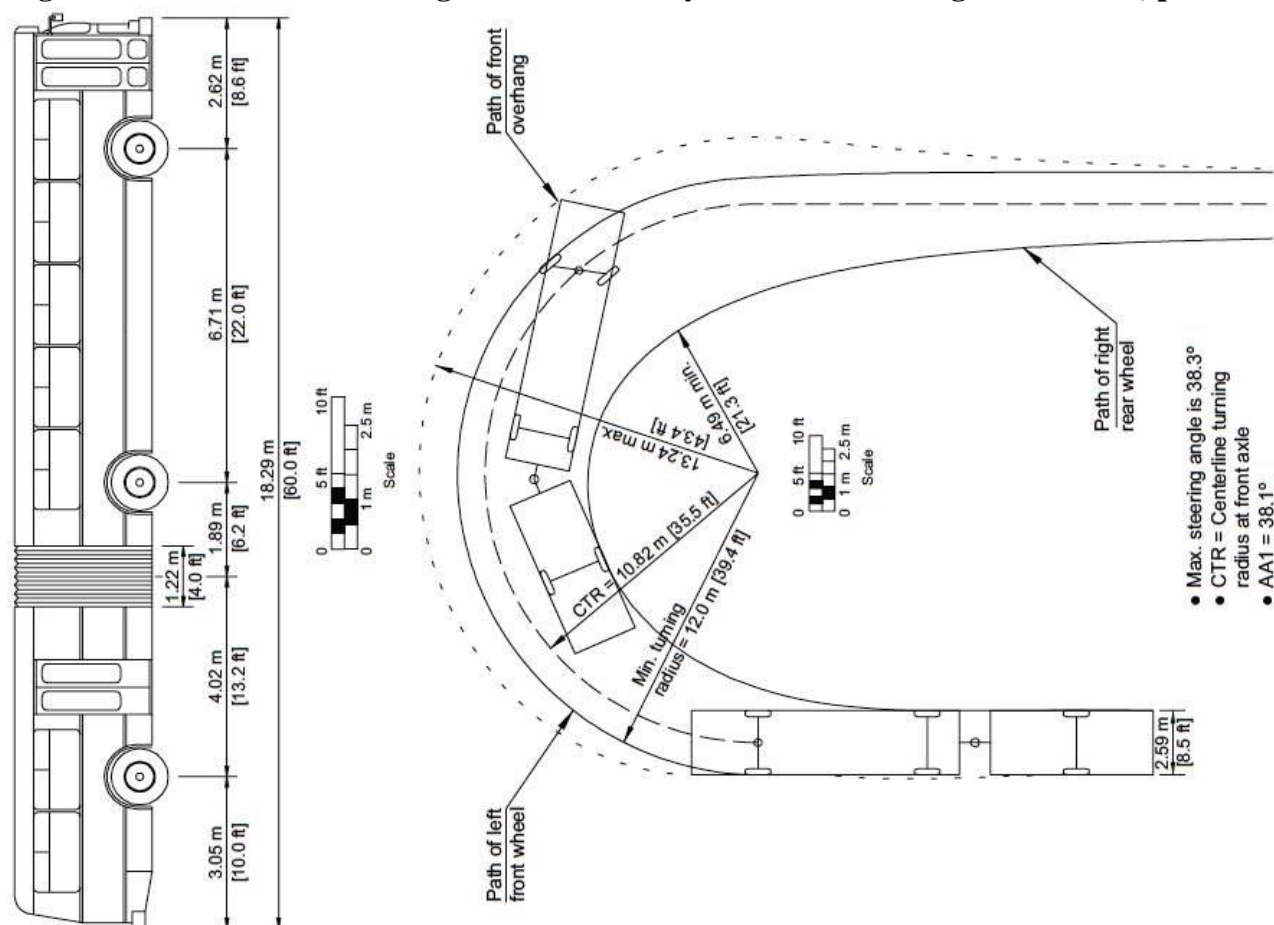
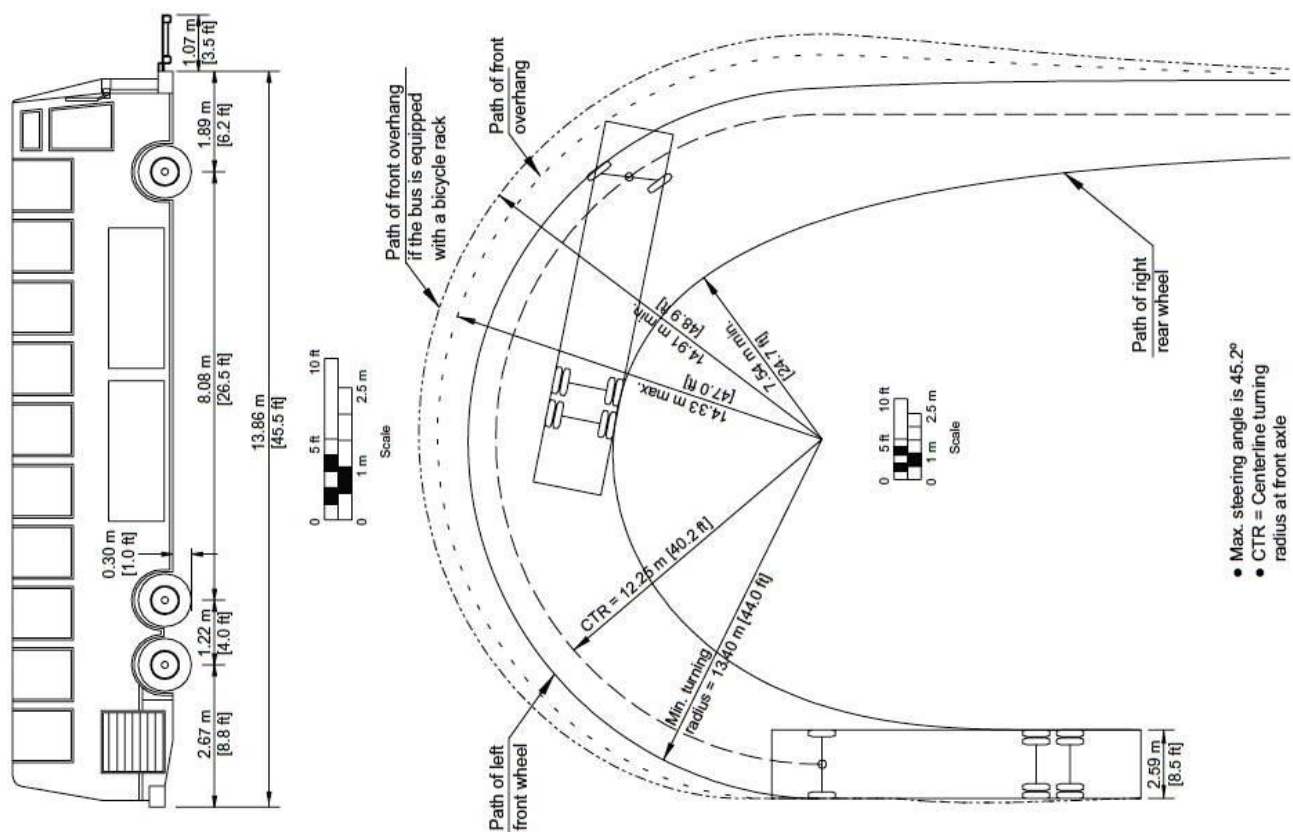


Figure 2-1/3: Minimum Turning Path for single-Units Truck [SU-12] Design Vehicles [1, p.2-12]



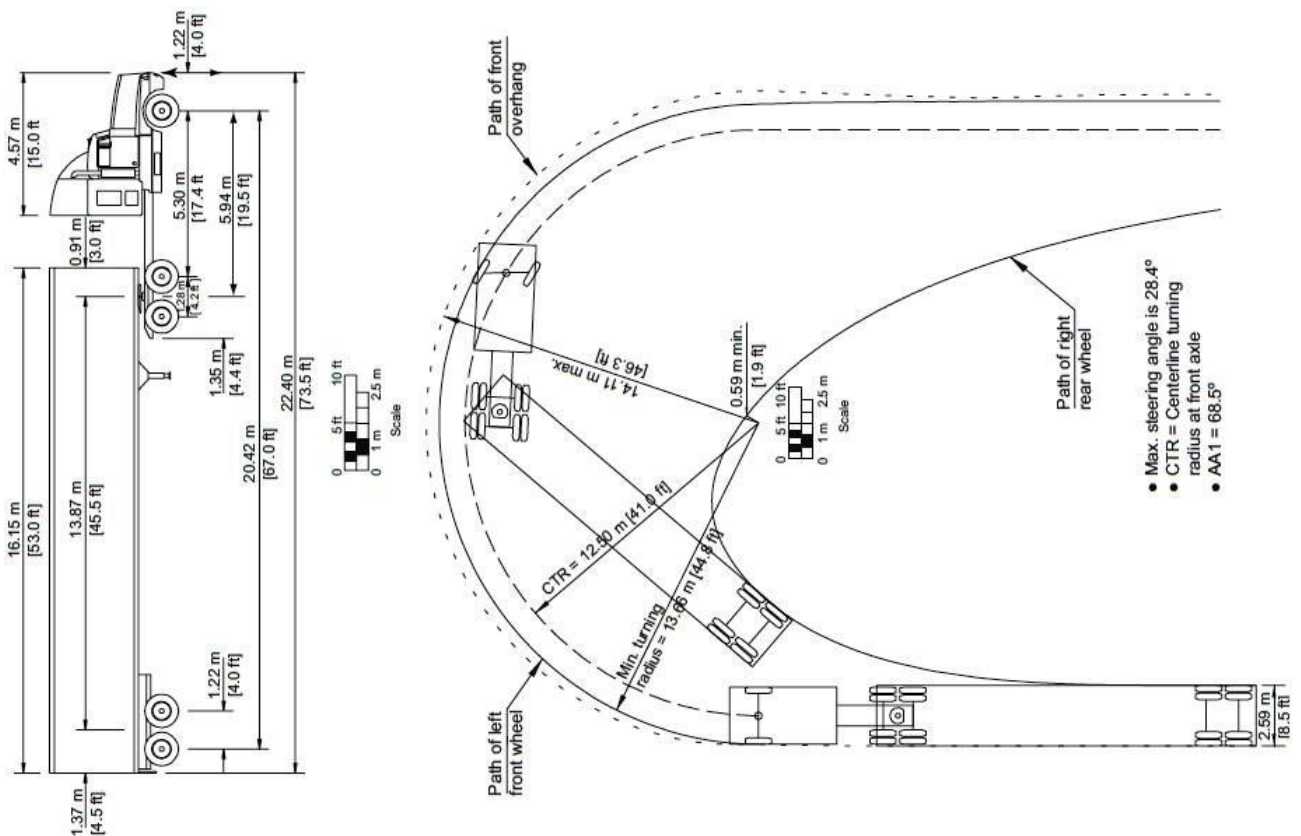


Figure 2-1/6: Minimum Turning Path for Interstate Semitrailer [WB 20] [1, p.2-24]

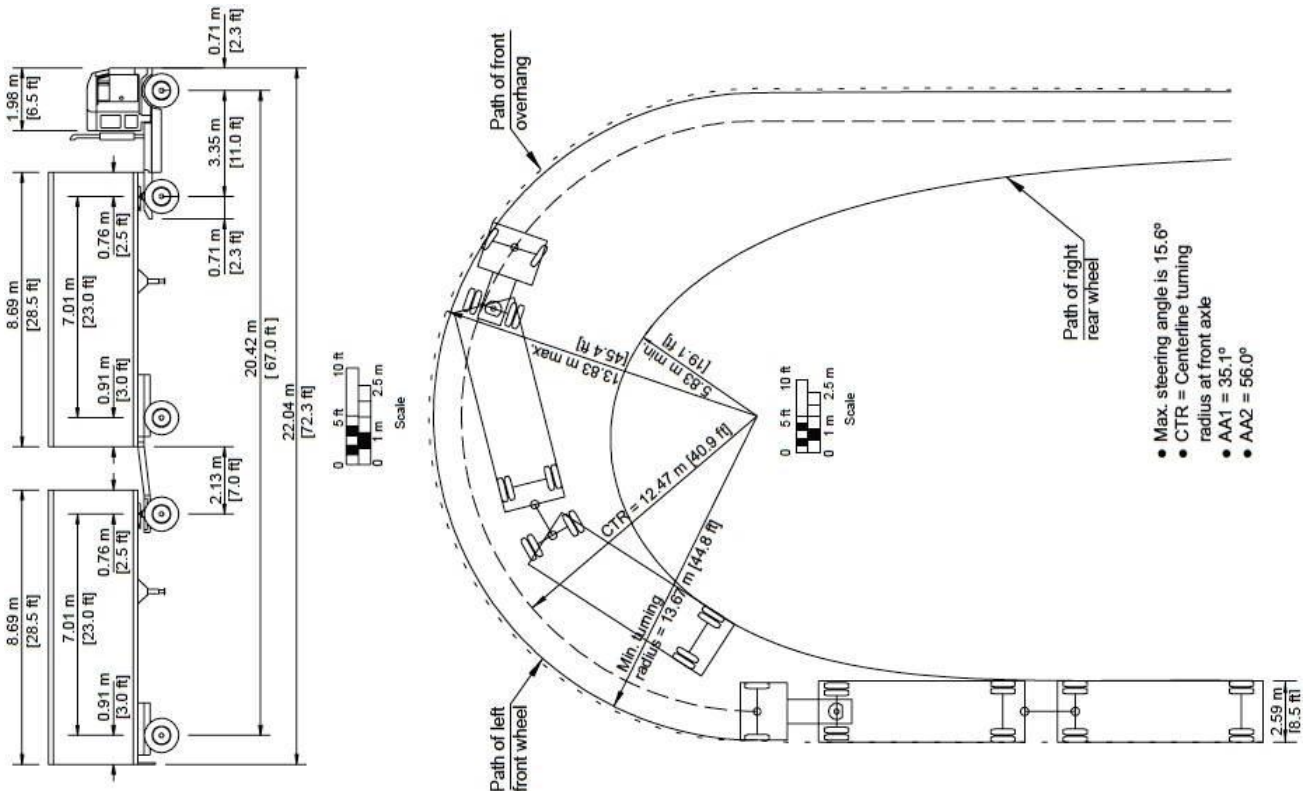


Figure 2-1/7: Minimum Turning Path for Double-Trailer Combination [WB20D] [1, p.2-25]

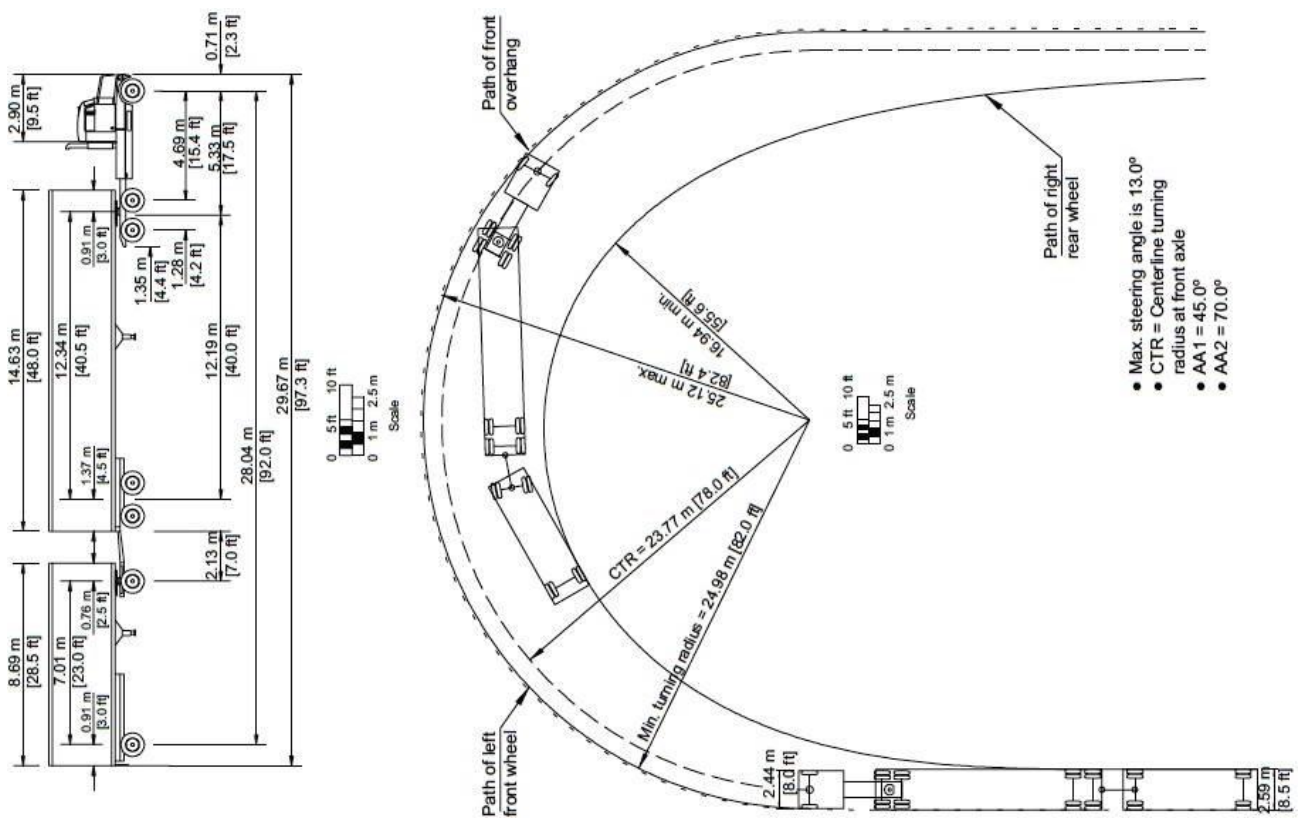


Figure 2-1/8: Minimum Turning Path for Rocky Mountain Double-Trailer Combination [WB-28D] [1, p.2-26]

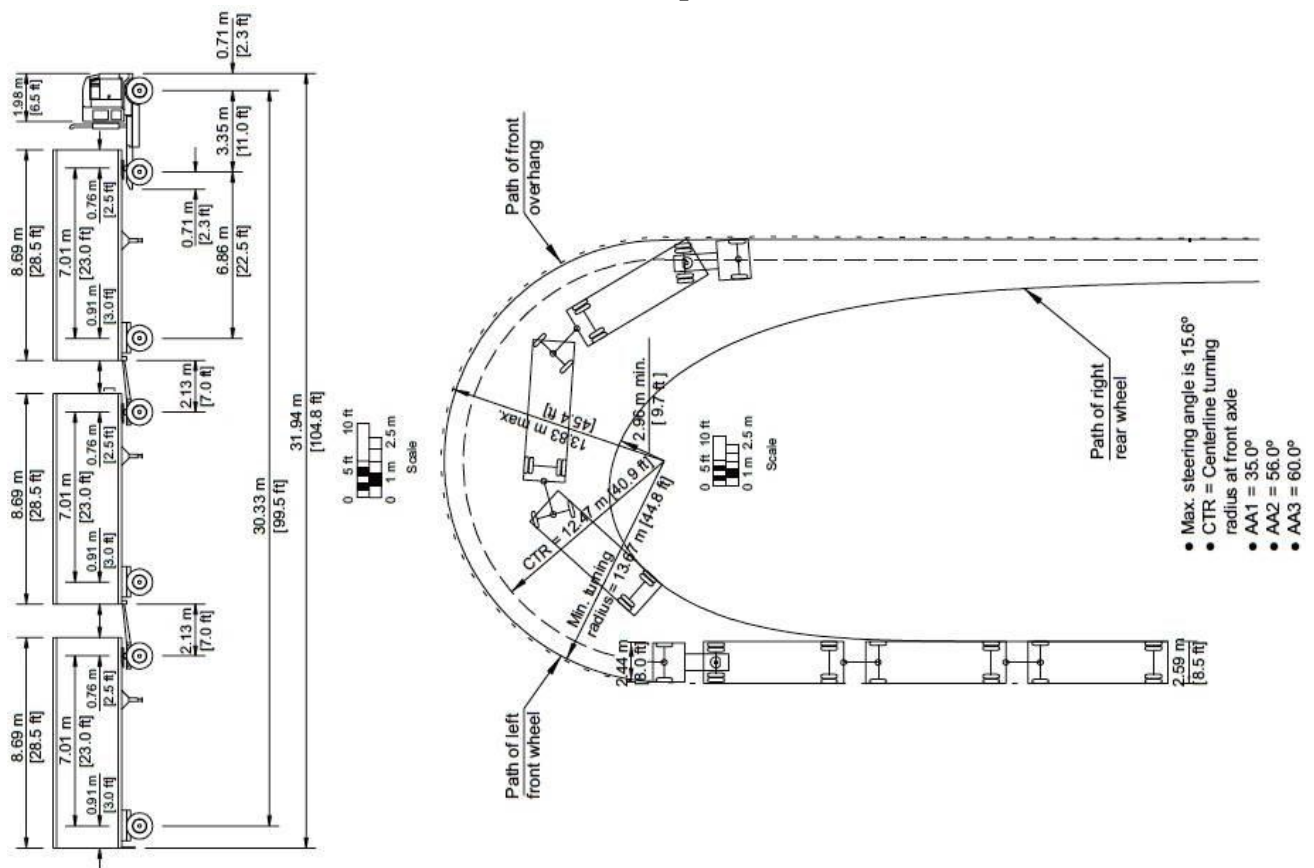
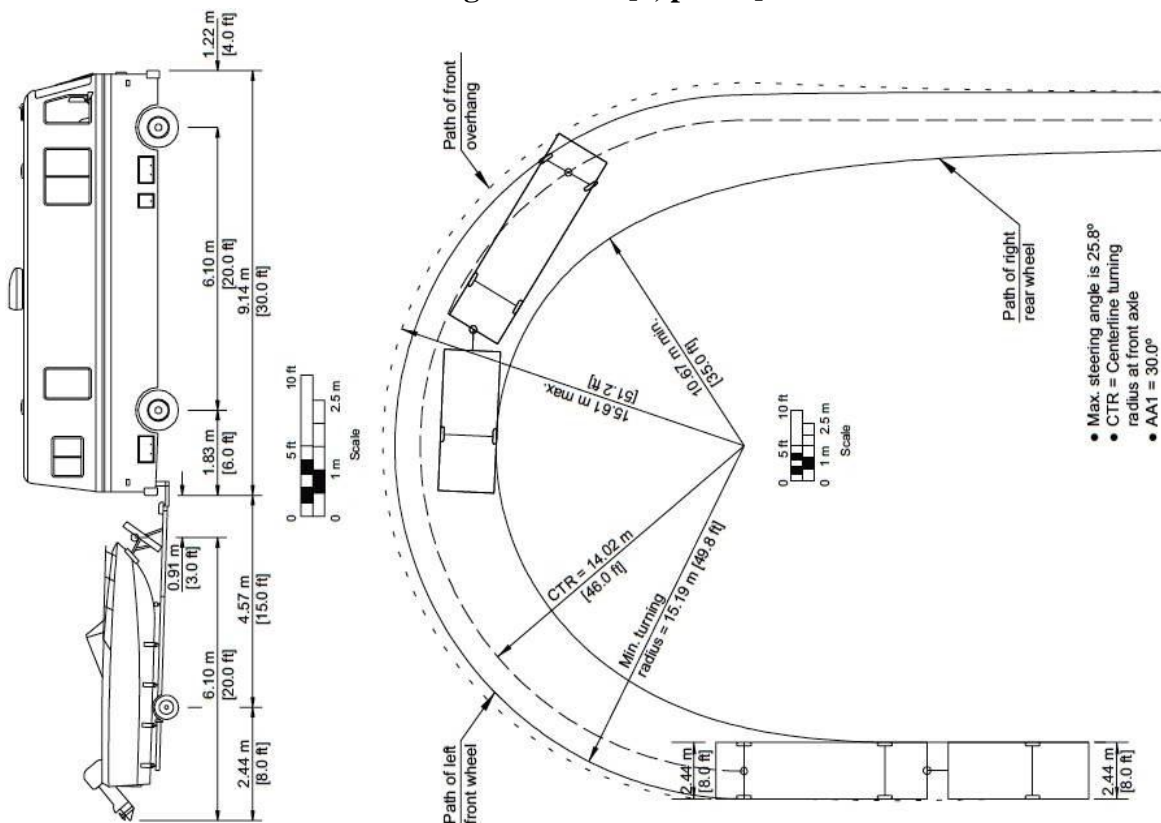
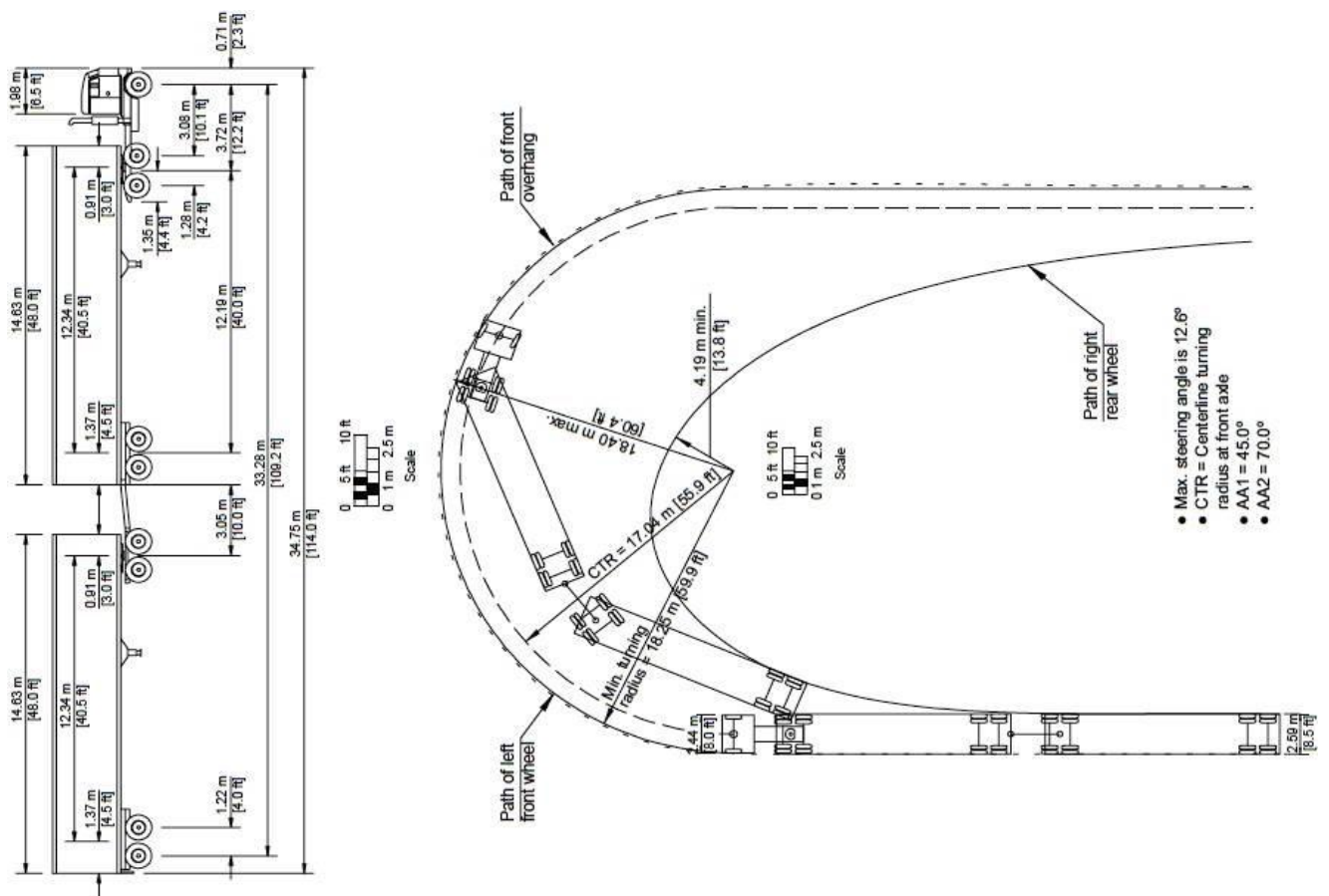


Figure 2-1/9: Minimum Turning Path for Triple – Semi Trailer Combination [WB-30T] [1, p.2-27]



2-1/4 Maximum Axle Loads and Gross Weights of Trucks

The axle weights of vehicles expected on the highway are considered for structural pavement design and maximum grades selection. The legal axle loads permitted on highways in Iraq are given for four axle types are shown table (2-1/4).

Table 2-1/4: Legal Axle Loads Permitted on Highways in Iraq

Axle Type	Max. Axle Load [3] (Metric Tons)
• Front Single Axle	7
• Rear Single Axle	13
• Tandem Axle	20
• Tridem Axle	27

The Legal gross weights of single- unit trucks and semitrailer/ trailer combinations permitted in regular operation in Iraq are shown in figure (2-1/12) for 10 types of heavy vehicles.

The heaviest vehicle (Type 3-S1-2) has a maximum gross weight of 66 tons.

In the AASHTO Guide for vehicle weights and dimensions [2- p.10], the total maximum gross weight imposed on the highway by the wheels of a vehicle has been given shown table (2-1/5):

Table 2-1/5: Total Maximum Gross Weight Imposed on the Highway

Axle Type	Max. Axle Gross Weight, W
Single- Axle	9.07 Tons
Tandem –Axle	15.42 Tons
Group of Two of more Consecutive Axles	$W = 0.745 (L.N / N-1) + 2.724N + 8.171$

Where

W= Max. gross weight on the axle group (Tons)

L= Distance (meters) between the extremes, of the axles group.

N= Number of axles in the group.

The Maximum Permissible Vehicle Gross Weight imposed on the highway by a vehicle or combination of vehicles with two or more consecutive axles should be determined by the application of the maximum permissible axle group weights, formula (W). (Exception: Two consecutive sets of tandem axles may carry a gross load of (15.420 kg) each providing the overall distance between the first and last axles of such consecutive sets of tandem axles is (10.97m) or more.

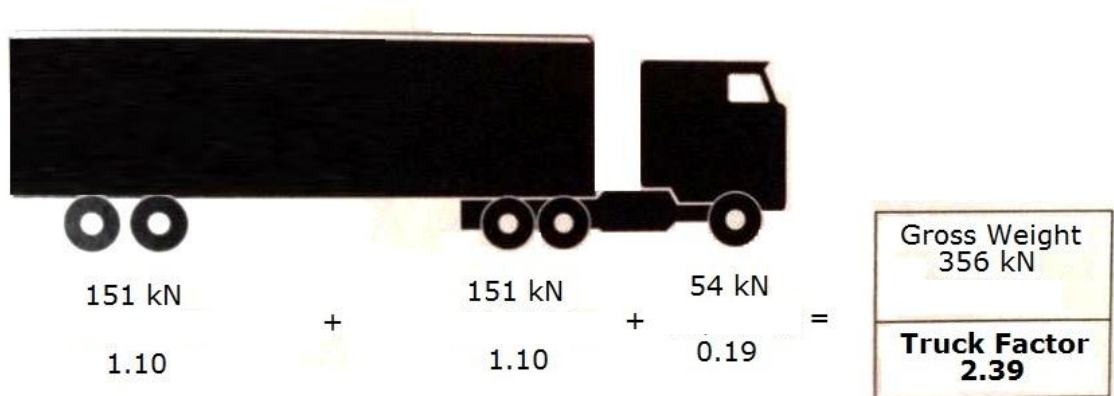
For the structural design of flexible pavements the typical load equivalency factors are shown in table (2-1/6) for different axles.

Legal Axle and Gross Weights Permitted on Motor Vehicles in Regular Operation in Iraq الوزن القصوى المسموح بها لمعابر الشاحنات والوزن الإجمالي (المركبة + الحمولة)		
MAXIMUM GROSS WEIGHT الوزن الإجمالي الأقصى	VEHICLE TYPE صنف الشاحنة	
20 Tons طن ٢٠	Type 2	<p>SHAPE 1</p>
27 Tons طن ٢٧	Type 3	<p>SHAPE 2</p>
33 Tons طن ٣٣	Type 2-S1	<p>SHAPE 3</p>
46 Tons طن ٤٦	Type 2-2	<p>SHAPE 4</p>
47 Tons طن ٤٧	Type 3-S2	<p>SHAPE 5</p>
40 Tons طن ٤٠	Type 3-S1	<p>SHAPE 6</p>
53 Tons طن ٥٣	Type 3-2	<p>SHAPE 7</p>
40 Tons طن ٤٠	Type 2-S2	<p>SHAPE 8</p>
66 Tons طن ٦٦	Type 3-S3	<p>SHAPE 9</p>
54 Tons طن ٥٤	Type 3-S3	<p>SHAPE 10</p>

Figure 2-1/12: Legal Axle and Gross Weights Permitted on Motor Vehicles in Regular Operation in Iraq [3]

Table 2-1/6: Typical Load-Equivalency Factors [4, p.444]

<i>Gross Axle Load</i>			
(kN)	<i>Single Axle</i>	<i>Tandem Axles</i>	<i>Trident Axles</i>
26.7	0.01043	0.001	0.0003
44.5	0.087	0.007	0.002
53.4	0.189	0.014	0.003
62.3	0.360	0.027	0.006
71.2	0.623	0.047	0.011
80.0	1.000	0.077	0.017
89.0	1.51	0.121	0.027
97.9	2.18	0.180	0.040
106.8	3.03	0.260	0.057
115.6	4.09	0.364	0.080
133.4	6.97	0.658	0.145
151.2	11.18	1.095	0.246
178.0	21.08	2.08	0.487
222.4	52.88	4.86	1.22
267.0		9.59	2.51
311.5		17.19	4.52
356.0		29.0	7.45
400.3		46.8	11.6



Example: Load-equivalency factors and the truck factor for a single-tractor semitrailer truck. (Courtesy of Asphalt Institute.)

2-1/5 Passenger Car Unit Equivalents

Vehicles of different sizes and weights have different operating characteristics that should be considered in geometric design of highways. For uninterrupted traffic Flow, the various vehicles affecting operation, can be grouped into two general classes:

- Passenger Cars, including pick-up trucks and vans.
- Trucks, including buses, single-unit trucks, combination trucks, and recreational vehicles. Trucks are normally defined as those vehicles having manufacturers gross vehicle weight of 4 tons or more, having dual tires on at least one rear axle.

The number of equivalent passenger cars equaling the effect on traffic operation of one truck (Passenger Car Unit Equivalent) is dependent on the roadway gradient or terrain.

The passenger car unit equivalents (PCU), for different vehicles as given by HCM 2000 [6], on general highway segments of multilane highways and basic freeway sections are shown in table (2-1/7) [6, p.452]:

Table 2-1/7: Passenger Car Unit Equivalents (PCU) for Different Vehicles

Vehicle Type	Type of Terrain		
	Level	Rolling	Mountainous
• Trucks & Buses	1.5	2.5	4.5
• Recreational	1.2	2.0	4.0

The PCU equivalents used by SCRB since 1982 are shown in table (2-1/8) [7, p.1-24]:

Table 2-1/8: Passenger Car Unit Equivalents (PCU) Used By SCRB

Vehicle Type	Type of Terrain		
	Level	Rolling	Mountainous
• Passenger Cars	1.00	1.00	1.00
• Buses up to 24 passengers	1.25	1.75	3.00
• Buses above 24 passengers	2.00	3.00	6.00
• Trucks & Trailer combination	3.00	5.00	10.00

The PCU values on upgrades are shown in tables (3-5/6) and (3-5/7), for Trucks / Buses and Recreational vehicles respectively.

2-1/6 Resistances Acting on a Vehicle in Motion

Forces that act on a vehicle in motion, to affect the vehicle speed, include: engine power, braking – resistance, and tractive- resistance forces. Tractive resistance may include:

- Inertial resistance: force that maintains vehicle in motion, (Mass x Acceleration)
- Gradient resistance: force needed to move the vehicle through a given vertical distance (due to the effect of gravity). For each 1% grade, the gradient resistance is 10 kg./1000 kg. of total vehicle weight.
- Rolling resistance: resistance to motion at the area of contact between tires and the roadway surface. For asphalt concrete surface, the rolling resistance coefficient is equal to 12 kg/1000 kg. of gross vehicle weight.
- Air resistance: Force resulting from the retarding effect of air on the various surfaces of the vehicle for speeds over 30 km/hr.
- Curve resistance: When a vehicle moves in a circular path, it undergoes a centripetal (lateral) acceleration that acts toward the center of curvature [1, p.3-18].

Figure (2-1/13) shows the different forces acting on a vehicle in motion [1, p.3-141].

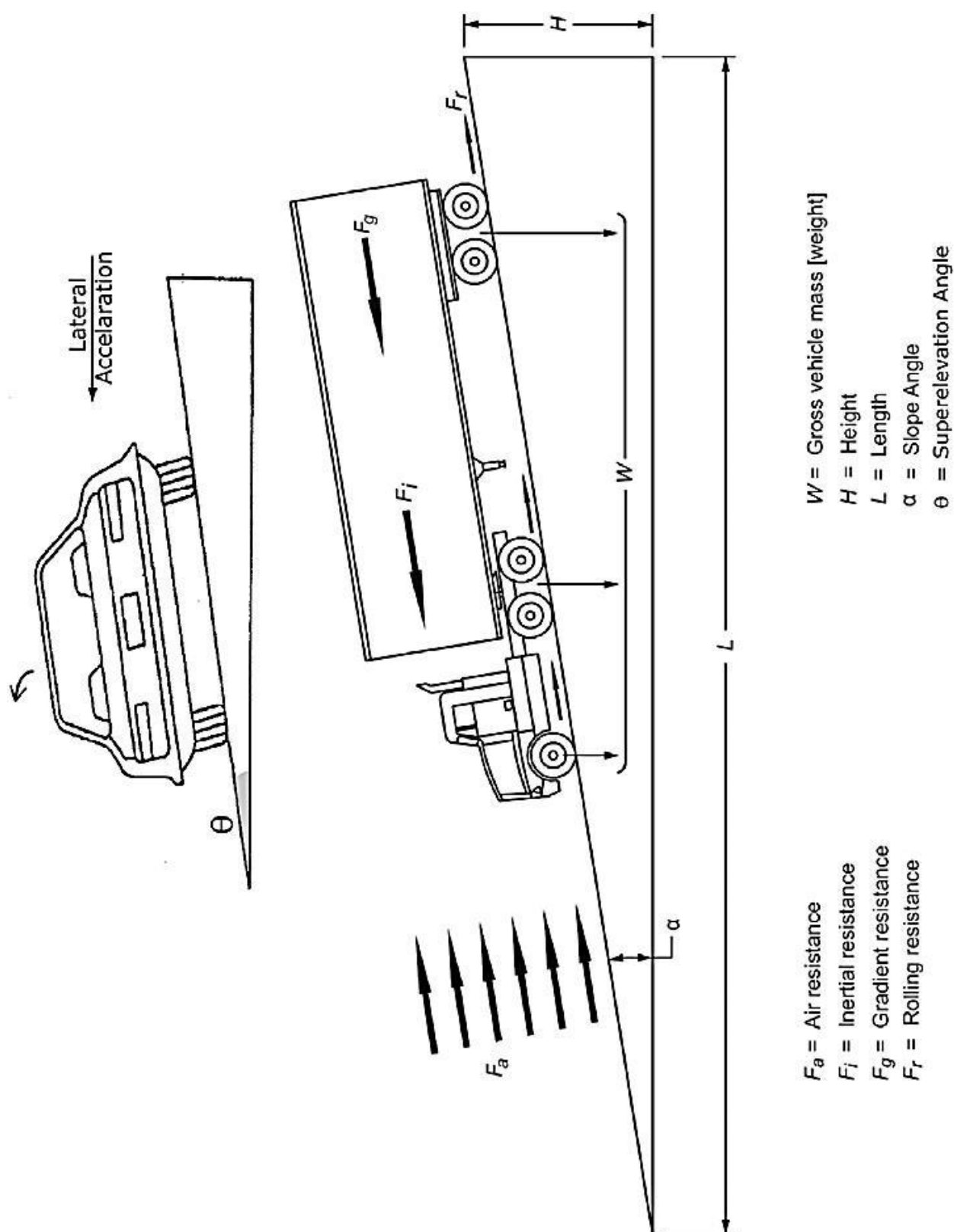


Figure 2-1/13: Forces Acting on Vehicle in Motion [1, p.3-141]

2-1/7 Acceleration and Deceleration Performance of Vehicles

Vehicles move according to fundamental laws of motion [4, p.120]:

$$V_f = V_o + a \times t \quad (2-1/1)$$

$$d = V_o \times t + \frac{1}{2} \times a \times t^2 \quad (2-1/2)$$

$$d = \frac{V_f^2 - V_o^2}{2 \times a} \quad (2-1/3)$$

Where:

V_f = Final velocity (m/sec.)

V_o = Initial velocity (m/sec.)

a = Acceleration or deceleration rate (m/sec.²)

t = time (sec.)

d = distance (meters)

Acceleration and deceleration rates are often critical parameters in determining the dimensions of highway design features, as intersections, ramps, climbing, or auxiliary lanes, and turnouts.

For design applications, lower performance vehicle, such as a low – powered car, or a loaded truck, is usually used [1, p.2-33].

For a speed change of 0 to 48 km/hr., typical maximum accelerations are:

- 0.5 m/sec.² for tractor- semitrailer truck
- 3.1 m /sec.² for large car.
- 4.3 m/ sec.² for sport car.

The deceleration rate for passenger cars under normal braking is about 2.0m/sec² in the range of 0 to 48 km/hr., and about 1.5 m/sec.² in the range of 48 to 112 km/hr.

Figures (2-1/14) and 15 show acceleration and deceleration traveled distances for passenger cars.

Figure (2-1/16) shows typical maximum acceleration rates for 16 km/hr. speed increases at various running speeds, along level roads, for different vehicles.

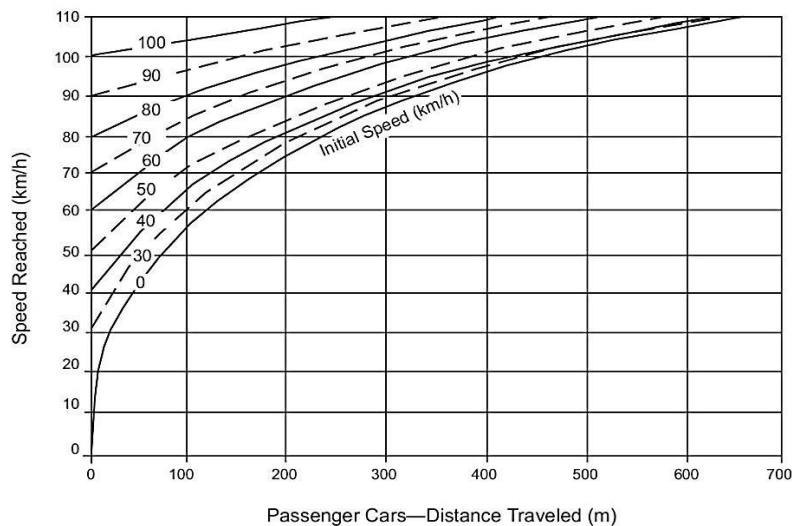


Figure 2-1/14: Acceleration Distances of Passenger Cars, Level Conditions [1, p.2-34]

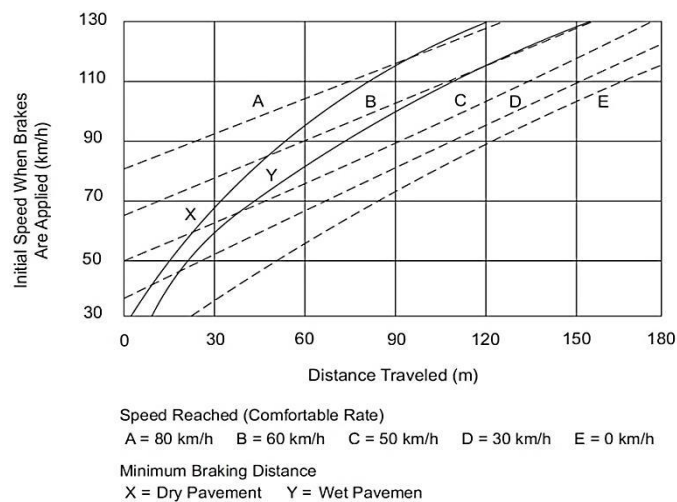


Figure 2-1/15: Deceleration Distances of Passenger Vehicles, Approaching Intersection [1, p.2-35]

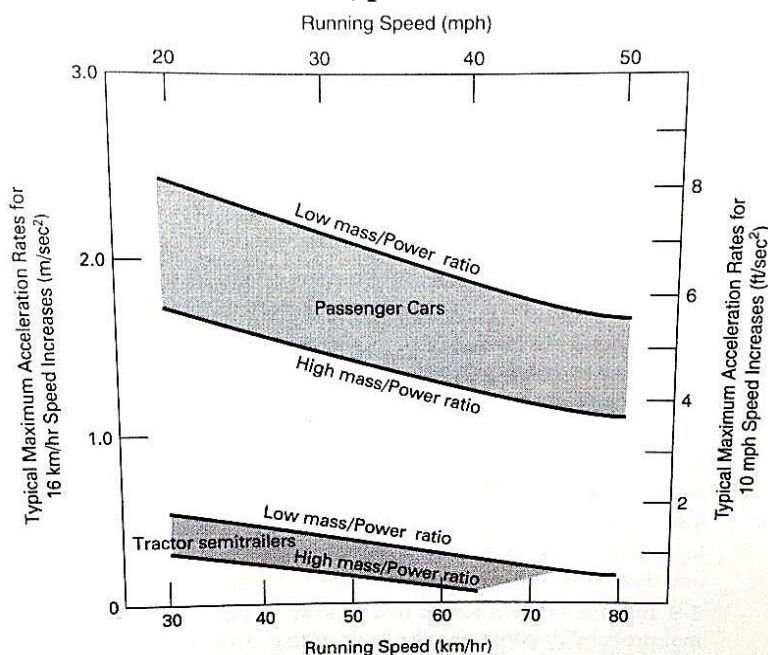


Figure 2-1/16: Typical Maximum Acceleration Rates of Passenger Cars and Tractor-Semitrailers for 16 km/hr. Speed Increases at Various Running Speeds [4, p.121]

2-2 DRIVERS

Driver performance is essential to proper highway design and operation. Highway engineers need to keep in mind that facilities must be designed to accommodate drivers with a wide range of skills and ages.

2-2/1 General Drivers Characteristics

The driver's decisions and actions depend principally on information received through the senses, which include:

- **Visual (sight):** A person with normal vision can perceive peripheral objects within a cone having a central angle ranging up to about 160°. The average range of horizontal fixation locations of experienced drivers varies from 30° to 48°.
- **Kinesthetic (movement):** information about forces associated with change of direction, steering, and braking.
- **Vestibular (equilibrium):** information about forces associated with vibrations and stability of the vehicle.
- **Auditory (hearing):** sounds of horns, skidding tires may alert driver to an impending collision. [4, p. 106].

The driving task depends on drivers receiving and using information for correct decisions. The components of driving task fall into three categories: control, guidance, and navigation as presented in table (2-2/1).

Most information is received visually by drivers from their view of the roadway alignment, markings, and signs.

Many driving errors are caused by deficiencies in driver capabilities, like: insufficient training or experience, poor vision, inappropriate risk taking, wrong judgment, adverse psycho physiological states, fatigue from extended period of driving, violations, and old age. [1, p.2-42]

2-2/2 Driver Perception- Reaction Time Performance

Driver perception – reaction time is defined as the interval between seeing, feeling, or hearing a traffic or highway situation, and making an initial response to what has been perceived.

For braking, it begins when the object or condition first becomes visible, and stops when the driver's foot touches the brake. [4, p.108].

Figure (2-2/1) shows the relationship for 85th- percentile drivers, between reaction time and the quantified amount of perceived information to be processed in bits, for expected and unexpected events. [1, p.2-41]

For the purpose of computing stopping sight distances for highway design, AASHTO recommends to use a brake – reaction time of 2.5 seconds.

Table 2-2/1: Framework for Driving Task Conceptualization [4, p.112]

<i>Subtask Category</i>	<i>Related to</i>	<i>Example of Sources of Information</i>	<i>Importance of Information</i>	<i>Likely Consequence of Failure</i>
Control (micro performance)	Physical operation of vehicle Steering control Speed control	Road edges Lane divisions Warning signs Kinesthesia	Highest	Emergency situation or crash
Guidance (situational performance)	Selecting and maintaining a safe speed and path	Road geometry Obstacles Traffic conditions Weather conditions	Intermediate	Emergency situation or crash
Navigation (macro performance)	Route following Direction finding Trip planning	Experience Directional signs Maps Touring service	Lowest	Delay, confusion, or inefficiencies

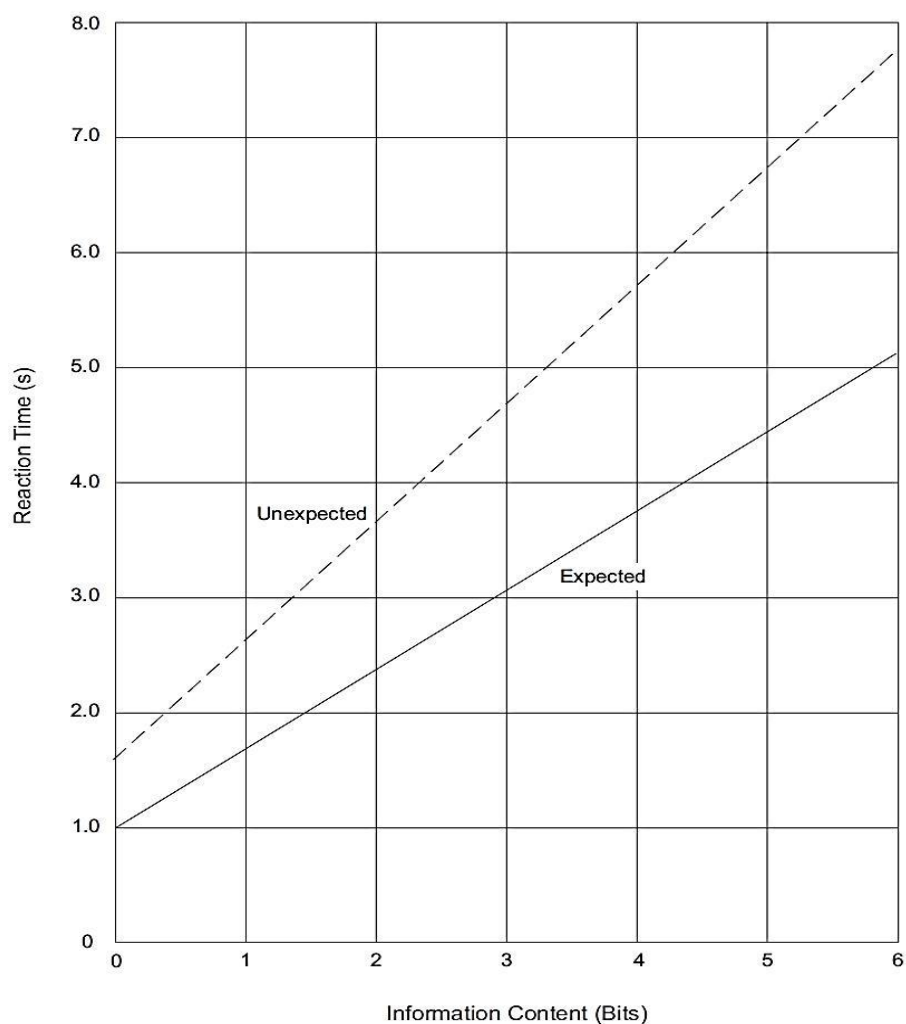


Figure .2-2/1: 85th-percentile Driver Reaction Time to Expected and Unexpected Information [1, p.2-41]

2-3 PEDESTRIANS

In highway planning and design, the interaction of pedestrians with traffic, is of major consideration, especially in urban areas.

2-3/1 Pedestrian Characteristics

The typical pedestrian walking speeds range from 0.8 -1.8 m/sec. A walking speed of 1.2 m/sec. is generally assumed for traffic signal timing.

Pedestrian actions are less predictable, and regulations are not usually enforced. Pedestrians also have a basic resistance to change in grade, and intend to avoid using overpass or underpass pedestrian facilities.

The design of pedestrian facilities (sidewalks, crosswalks, overpass, underpass, etc.), depends on the human body, besides walking space requirements as shown in figure (2-3/1). [4, p.115]

The typical pedestrian will not usually walk over 1.5 km. The definitions related to pedestrian speed, flow, density and space are as follows:

- S_p = **Pedestrian speed**: This is the average walking speed for pedestrians, which is typically about 1.2 m/sec but varies with age and purpose of the walking trip.
- f_p = **Pedestrian flow**: This refers to the number of pedestrians crossing a line of sight across the width of the pedestrian facility perpendicular to the pedestrian path in unit time (p/min) The pedestrian flow/ unit width is equal to the pedestrian flow divided by the effective width of the pedestrian facility, in units of pedestrians /min/ m (p/min/m).
- d_p = **Pedestrian density**: This is computed as the average number of pedestrians/ unit area of the pedestrian facility (p/m²).
- a_p = **Pedestrian space**: This refers to the average area provided for each pedestrian. It is equal to the inverse of the density and is expressed in units of sqm/ pedestrian (m²/p) [8,p.203];

$$a_p = \frac{1}{d_p}$$

$$f_p = S_p \times d_p \quad (2-3/1)$$

Where:

f_p is in (p/min/m)

S_p is in (m/min)

d_p is in (p/m²)

Walking speeds decrease sharply with drop in pedestrian space below 2.3 m².

In areas with large numbers of elderly pedestrians, a walking speed of 0.9 m/ sec. may be used.

2-3/2 Level of Service for Pedestrian Walkways

Level of service (LOS) is a qualitative measure describing pedestrian flow, based on service measures of space per pedestrian, flow rates, and walking speed. Table (2-3/2) shows descriptions of level of service for pedestrian walkways.

Platoons (groups of pedestrians walking together as group) can form when passing is impeded due to insufficient space, and faster pedestrians slow down being slower-moving pedestrians [5, p.4-31].

The level of service for pedestrians at signalized intersections is based upon the average delay experienced by a pedestrian, computed from: [8, p.211]

$$\text{Pedestrian Delay (sec.)} = \frac{0.5 \times (C - g)^2}{C} \quad (2-3/2)$$

Where:

C = Cycle length (sec.)

g = Effective green time for pedestrians (sec.)

Table 2-3/1: Level of Service for Pedestrian at Signalized Intersections

Average delay/pedestrian (sec.)	Pedestrians Level of Service at Signalized Intersection
< 5	A
≥ 5 – 10	B
> 10- 20	C
> 20 -30	D
> 30-45	E
> 45	F

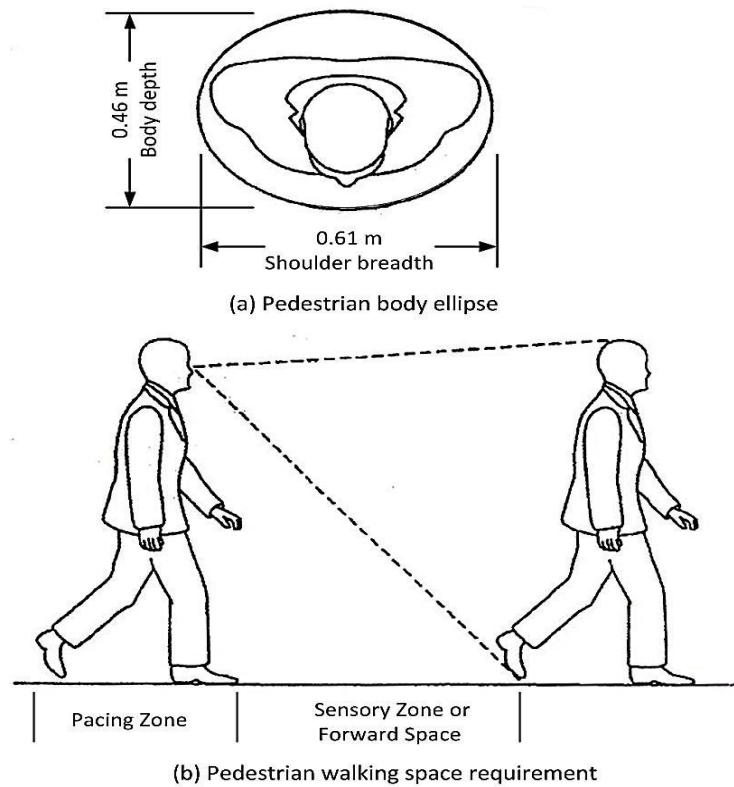
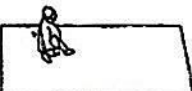
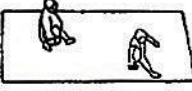

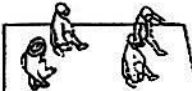




Figure 2-3/1: Pedestrian Spatial Ellipse and Pedestrian Walking Space Requirements
(Courtesy Transportation Research Board) [4, p.115]

Table 2-3/2: Level of Service for Pedestrian Walkways [4, p.665]

Level of Service	Flow Rate (p/min/m)	Space per Person (m²/p)	Description of Flow	
A	≤16	≥5.6	Pedestrians freely select walking speed and are able to bypass slower pedestrians and avoid crossing conflicts.	
B	>16 up to 23	3.7–5.6	Pedestrians begin to be aware of other pedestrians and to respond to their presence when selecting walking paths.	
C	>23 up to 33	2.2–3.7	Space acceptable and sufficient for normal walking speeds and for bypassing other pedestrians but opposing traffic may create minor conflicts.	
D	>33 up to 49	1.4–2.2	The majority of persons have normal walking speeds restricted and reduced; reverse flow and crossing movements are severely restricted.	
E	>49 up to 75	0.75–1.4	Virtually all pedestrians have normal walking speeds restricted, requiring frequent adjusting of gait; reverse flow is very difficult; pedestrians may encounter stoppages and flow interruptions.	
F	Varies	≤0.75	All pedestrian walking speeds are restricted and forward progress is achieved via a shuffling maneuver; reverse flow is virtually impossible.	

2-3/3 Pedestrian Crossings

Pedestrian facilities may include sidewalks, crosswalks on at-grade intersection, crosswalks on grade separations (overpass or underpass), depressed curbs and ramped sidewalks for older walkers. [1, p.2-78]

Sidewalks should have a skid – resistant surface free of holes, bumps and other irregularities with a minimum effective width of 1.50 meters.

Widths of 3.00 meters or greater may be necessary in commercial areas. For increased safety, an extra separation of 1.5 -1.8 meters is desired between sidewalk edge and curb.

The effective width of signalized and unsignalized crosswalks varies according to the desired pedestrian level of service, with a minimum width of 3.75 meters. The following measures need to be considered in crosswalk design: [1, p.2-79]

- Use of simple intersection designs to minimize crosswalk length.
- Provide enhanced markings and delineations.
- Provide refuge islands of sufficient width at wide intersections.
- Use of properly located pedestrian signals.
- Provide lighting and eliminate glare sources.
- Reducing pedestrian- vehicular conflicts, related to left and right turns.
- Providing pedestrian grade separations.
- Enforced regulations for street crossing.

The use of pedestrian overpass or underpass crossings needs to be encouraged by providing suitable and safe escalators and stairways for this purpose.

2-4 REFERENCES

[1] AASHTO, "A Policy on Geometric Design of Highways and Streets", American Association of State Highway and Transportation Officials, USA, 2011.

[2] AASHTO, "Guide for Vehicle Weights and Dimensions", American Association of State Highway and Transportation Officials, USA, 2001.

[3] State Corporation of Roads & Bridges (SCRB), "Legal Axle and Gross Weights Permitted for Vehicles in Regular Operation in Iraq", 2009.

[4] Wright P.H. and Dixon, K.K., "Highway Engineering", John Wiley & Sons, USA, 2004.

[5] "Highway Capacity Manual, HCM 2010", Transportation Research Board, National Research Council, USA, Washington D.C., 2010.

[6] Garber, N.J. and Hoel, L.A., "Traffic & Highway Engineering", Cengage Learning, USA, 2009.

[7] SCRB, "*Highway Design Manual*", State Corporation of Roads and Bridges, Ministry of Construction and Housing, Iraq, 1982.

[8] Hoel, L.A., Garber, N.J. and Sadek, A. W., "*Transportation Infrastructure Engineering/ A Multi – Modal Integration*", Nelson/ Thomson Ltd., USA, 2008.

CHAPTER 3

TRAFFIC FLOW CHARACTERISTICS

3-1 TRAFFIC VOLUME

Traffic Volume is defined as the number of vehicles that pass a point along a roadway or traffic lane per unit of time.

The average daily traffic (ADT), and design hourly volume (DHV) are two measures of traffic volume. The term (demand) relates to vehicles arriving, while the term (volume) relates to vehicles discharging [1, p.3-3]

3-1/1 CURRENT ANNUAL AVERAGE DAILY TRAFFIC

The annual average daily traffic (AADT) is the total volume of traffic passing a point or segment of a highway facility, in both directions, for one year, divided by number of days in the year.

At typical rural locations, the volume on certain days may significantly be higher than the AADT.

3-1/2 PROJECTION OF FUTURE TRAFFIC DEMANDS

The trend analysis of traffic flow that forecasts vehicle movements may be determined directly from observations of prior movement activity, especially where little or no other information related to generated or development traffic is available.

$$\text{Future ADT} = \text{Current ADT} \times \text{Traffic Projection Factor} \quad (3-1/1)$$

$$\text{Traffic Projection Factor} = (1 + r)^n$$

Where:

r = Annual rate of traffic growth, (0.02- 0.12).

n = Traffic analysis period, (15-24 years). [3, p.2-53]

For instance, for $r = 6\%$ and $n = 20$ years, the traffic projection factor = 3.207.

3-1/3 DESIGN HOURLY VOLUME

The design hourly volume (DHV) is a future peak hourly volume used for design. It is usually the two- way (30th) highest hourly volume of the future target year (design year), often assumed as 20 years beyond the road construction year.

The relationship between peak hourly flows, and the annual average daily traffic is shown in figure (3-1 /1), for rural arterials [3, p.2-48]. The curve steepens quickly to the Left of the (30 HV) point indicating few more hours with higher volumes. The curve flattens to the right, indicating many hours in which the volume is not much less than the 30 HV.

On rural roads with average fluctuation in traffic flow, the 30 HV is typically about 15% of the ADT, with a range of 12 to 18 percent for 70% of all locations.

In urban areas, the two- way 30 HV may be determined by applying a representative percentage of 8 to 12 percent to the ADT. [3, p.2-50]

3-2 TRAFFIC COMPOSITION AND DIRECTIONAL DISTRIBUTION

For the geometric design of a highway, it is required to have data on all types of vehicles, and the major types of trucks as a percentage of all traffic expected to use the highway need to be indicated [3, p.2-51]. The passenger car unit equivalents are shown in section (2-1/5), Chapter2.

For 2- lane rural highways, the DHV is the total traffic in both directions of travel.

On multilane highways, the volume of traffic is usually greater in one direction than in the other during a particular hour. During peak hours, from 55 to 80 percent of the traffic, is occasionally traveling in the peak direction of the multilane highway. [3, p.2-50]

The lane distribution factor, as a percent of number of equivalent standard axle load repetitions in each direction, is usually used for design of pavement structures to the design lane as shown in table (3-2/1) [5, p.D2].

Table 3-2/1: Lane Distribution Factor

Lanes per direction	Lane distribution factor , percent
1	100
2	80-100
3	60-80
4 or more	50-75

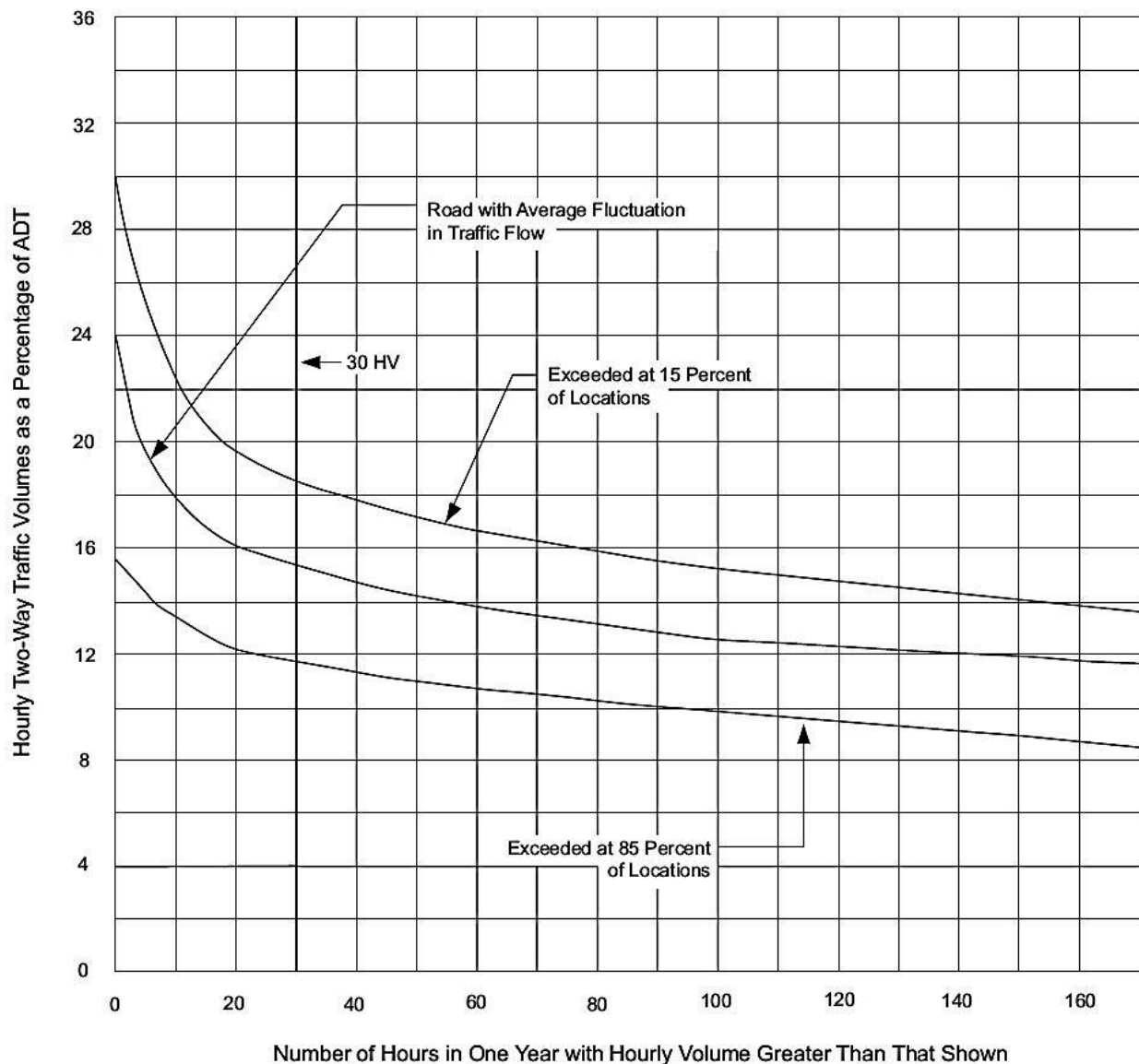


Figure: 3-1/1 Relationship between Peak-Hour and Average Daily Traffic Volumes on Rural Arterials [3, p.2-48]

3-3 SPEED

The objective in design of a highway facility is to satisfy the public's demand for service, in a safe and economical manner. The speed of vehicles on a road of highway depends on: physical characteristics of the highway, roadside interference, weather, presence of other vehicles speed limitations, and capability of the driver and his vehicle. [3,p.2-53]

3-3/1 SPOT SPEED

Spot speed: is the instantaneous speed of a vehicle as it passes a specified point along a street or highway.

Time- mean speed: is the arithmetic mean of speeds of all vehicles passing a point during a specified time interval.

Space- mean speed: is the arithmetic mean of speeds of vehicles occupying a section of street or highway at a given instant. [4, p.125]

3-3/2 OVERALL TRAVEL SPEED

Overall speed: is defined as the total distance traveled, divided by the total time required, including traffic delays.

3-3/3 RUNNING (OPERATING) SPEED

Operating speed: is the speed at which drivers are observed operating their vehicles during free- flow conditions. The 85th percentile of observed speeds is the most frequently used measure.

Running Speed: is the length of the highway section divided by the running time for the vehicle to travel through the section, excluding stop- delay time. The average running speed: is the sum of the distances traveled by vehicles on a highway section during a specified time period, divided by the sum of their running times.

The average spot speeds at several points, may be averaged to provide the average running speed. [3, p.2-54]

3-3/4 DESIGN SPEED

Design Speed: is a selected speed, used to determine the various geometric design features of the roadway, to attain the desired combination of safety, mobility, and efficiency, within constraints of environment, economy, aesthetics, and social impacts.

The selected design speed should depend on the anticipated operating speed, topography, adjacent land use, and the functional classification of the highway. The selected design speed should fit the travel desires and habits of most drivers, exceeding the average running speed.

For urban arterial streets and highways, the appropriate design speeds range from 50 to 100 km/ hr. For high- speed design of rural highways, the usual design speeds have the range of 80 to 130 km/ hr. [3, p.2-58]

The average running speed varies from 78 to 100 percent of design speed are shown in table (3-3/1) [3, p.3-29].

Table 3-3/1: Average Running Speed

Design Speed (km/hr.)	Average Running Speed (km/hr.)
20	20
30	30
40	40
50	47
60	55
70	63
80	70
90	77
100	85
110	91
120	98
130	102

The selected speed limits are not the highest speeds that might be used by drivers. Instead, the 85th percentile speed of traffic, for a sizable sample of vehicles, within the (Pace) of 15-km/hr. speed range used by most drivers, is usually selected. [3, p.2-57]

3-4 TRAFFIC FLOW

Traffic Flow may be either interrupted or uninterrupted.

Interrupted Flow: A category of traffic facilities characterized by traffic signals, stop signs, or other fixed causes of periodic delay or interruption to the traffic stream [1, p.5-8]

3-4/1 PEAK FLOW RATE AND PEAK- HOUR FACTOR

Total Hourly Volume (V): is the total number of vehicles that pass over a given point or section of a lane or roadway during one hour: (Vehicles/ hr.).

Peak Flow Rate (U): is the equivalent hourly rate, at which vehicles pass over a given point or section of a lane or roadway during 15 minutes. The largest flow value of the four consecutive 15- min. periods is selected to determine the peak flow rate (Vehicles/hr).

$$U = \max. V_{15 \text{ minutes}} \times 4 \quad (3 - 4/1)$$

Consideration of peak flow rates, U, (Vehicles/hr.), is important in capacity analysis.

Peak – Hour Factor (PHF): is the ratio of total hourly Volume (V) to the peak flow rate (U) within the hour;

$$PHF = \frac{V}{U} \quad (3 - 4/2)$$

Where:

V = Peak hour volume, (Vehicles/ hr.).

U = Flow rate for a peak 15min. period, (Vehicles/hr.). [1, p.7-1]

Typical PHF for 2-Lane roadways ranges from 0.88 (Rural) to 0.92 (Urban). For Freeways, the range is 0.80 to 0.95 respectively. If there is no variability in flow rate during the hour, the PHF would be 1.00. [4, p.130]

3-4/2 TRAFFIC DENSITY, SPACING, AND TIME - HEADWAY

- **Traffic Density:** is the average number of vehicles occupying a unit length of roadway at a given instant (Vehicles/km/instant.).

$$\text{Density} = \frac{\text{Flow rate}}{\text{Space mean speed}} \quad (3 - 4/3)$$

Where:

Density in (Veh./km)

Flow rate in (Veh./hr.)

Space mean speed in (km/hr.)

- **Spacing (Space Headway):** is the distance between successive vehicles, typically measured from front bumper to front bumper, (meters/ vehicle).
- **The average spacing** is the reciprocal of density.
- **Time-Headway:** is the time between the arrival of successive vehicles at a specified point; (sec./Veh.).
- **The average time-headway** is the reciprocal of volume. [4, p.131].

3-4/3 TRAFFIC VOLUME – SPEED- DENSITY RELATIONSHIPS

The generalized relationships between volume, speed, and density for uninterrupted flow facilities are shown in figure (3-4/1).

When speeds decrease, increased crowding can occur (increased density). Traffic volumes vary with density from zero to maximum flow rate. The two points of zero flow represent either no vehicles at all, or so many vehicles that flow has stopped.

Interference may be caused by weather, cross traffic, disabled vehicles, crashes and other conditions. With more interference, flow rates can still be maintained but with reduced speed, closer spacing, and greater density. With great interference, a rapid decrease in speed and traffic flow, with severe congestion occur. [3, p.2-58]

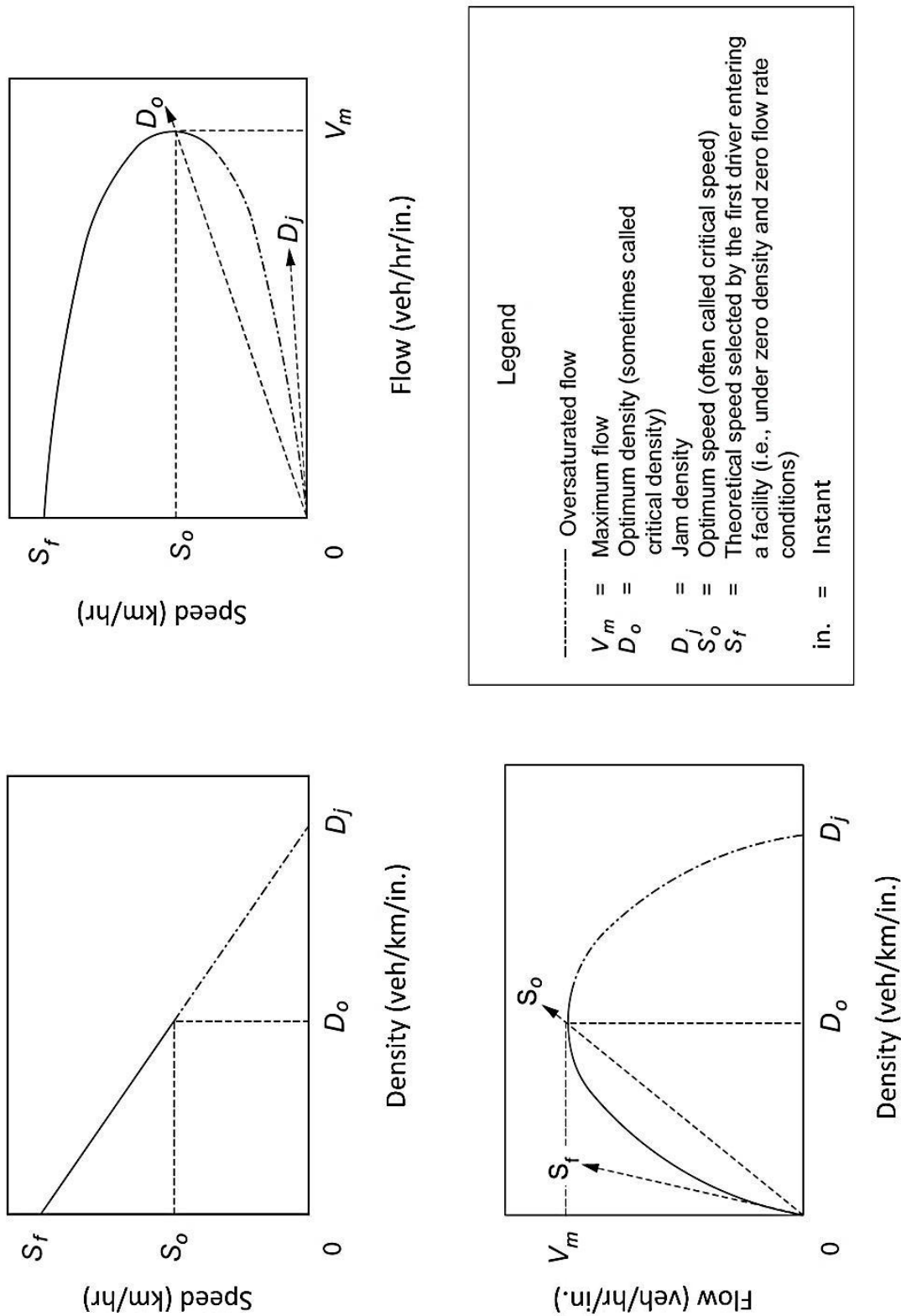


Figure 3-4/1: Generalized Speed-Volume-Density Relationships [3, p.2-59]

3-5 HIGHWAY CAPACITY

3-5/1 HIGHWAY CAPACITY AS A DESIGN CONTROL

Highway Capacity: is the maximum flow rate at which vehicles can reasonably be expected to traverse a point or uniform segment of a lane or roadway during one hour, under prevailing roadway, geometric, traffic, environmental, and control conditions. [1, p.5-2]

Operation at capacity provides the maximum volume, but as both volume and congestion decrease, the level of service (LOS) is improved.

Highway capacity analysis serves three general purposes including: transportation planning studies, highway design, and traffic operational analysis. In highway design, capacity analysis is used both to select highway type, and to determine dimensions (number of lanes, minimum length of weaving sections, etc.).

3-5/2 LEVELS OF SERVICE

Level of Service: is a qualitative measure, describing operational conditions, within a traffic stream, based on service measures, such as speed and travel time, freedom to maneuver, traffic interruptions, comfort, and convenience. [1, p.5-8]

The levels of service range from level A (least congested) to level F (most congested). Table (3-5/1) shows the general operating conditions of the different levels of service.

In table (3-5/2), guidance that may be used by designers in selecting an appropriate level of service has been provided. The various levels of service for the associated density values of freeways are shown in table (3-5/3).

The level of service for signalized intersections is defined in terms of control delay per vehicle as specified in table (3-5/4).

Table 3-5/1: General Definitions of Levels of Service [3, p.2-66]

Level of Service	General Operating Conditions
A	Free flow
B	Reasonably free flow
C	Stable flow
D	Approaching unstable flow
E	Unstable flow
F	Forced breakdown flow

Table 3-5/2: Guidelines for Selection of Design Levels of Service [3, p.2-67]

Functional Class	Appropriate Level of Service for Specified Combinations of Area and Terrain Type			
	Rural Level	Rural Rolling	Rural Mountainous	Urban and Suburban
Freeway	B	B	C	C or D
Arterial	B	B	C	C or D
Collector	C	C	D	D
Local	D	D	D	D

Table 3-5/3: Freeway Level of Service and Density [4, p.138]

<i>Level of Service</i>	<i>Density Range (pc/km/ln)</i>
A	0-7
B	> 7-11
C	> 11-16
D	> 16-22
E	> 22-28
F	> 28

Table 3-5/4: Signalized Intersection Level of Service and Control Delay [4, p.149]

<i>Level of Service</i>	<i>Control Delay Per Vehical (Sec./Veh)</i>
A	≤ 10
B	> 10-20
C	> 20-35
D	> 35-55
E	> 55-80
F	> 80

3-5/3 DESIGN SERVICE FLOW RATE FOR HIGHWAYS

Service flow rates: are termed for the maximum traffic flow rates that can be served at each level of service (Veh/hr.) or (Veh./hr./lane). Once a level of service has been selected, all elements of the roadway are designed consistent to the level.

The base conditions for uninterrupted flow facilities, such as freeways, include:

- Lane width: 3.60 m.
- Lateral clearance between edge of the right lane and nearest obstruction: $\geq 1.80\text{m}$.
- Median lateral clearance: $\geq 0.60\text{m}$.
- Grades: $\leq 2\%$ (level terrain).
- Interchange spacing: $\geq 3\text{ km}$.
- Traffic Stream: All Passenger Cars.
- Driver: of regular and familiar users of the facility.
- In urban areas only: $\geq 5\text{ Lanes/direction}$. [4, p.136]

Since prevailing conditions seldom reflect base conditions, computations of capacity, service flow rate, or level of service must be adjusted to account for departures from base conditions.

Table 3-5/5 demonstrates variability in minimum speed, maximum volume –to – capacity (v/c) ratio, and maximum service flow rate, for different levels of service of freeway segments under base conditions. The actual hourly flow rate must be adjusted to account for any prevailing conditions using the equations (3-5/1) and (3-5/2) presented with table (3-5/5) together with tables (3-5/6) and (3-5/7).

The driver population adjustment factor ranges from 0.85 (unfamiliar driver) to 1.00 (familiar driver). [4, p.143]

Freeway adjustments to determine reductions in free flow speed, as influenced by number of lanes/ direction, lane width, and right shoulder lateral clearance, are presented in table (3-5/8).

Table 3-5/5: Level of Service for Basic Freeway Segments and Varying Free-Flow Speed
[4, p.653]

<i>Level of Service</i>	<i>Maximum Density (pc/km/ln)</i>	<i>Minimum Speed (km/hr.)</i>	<i>Maximum v/c Ratio</i>	<i>Maximum Service Flow Rate (pc/hr./ln)</i>
<i>Free-Flow Speed = 120 km/hr.</i>				
A	7	120.0	0.35	840
B	11	120.0	0.55	1320
C	16	114.6	0.77	1840
D	22	99.6	0.92	2200
E	28	85.7	1.00	2400
F	varies	varies	varies	varies
<i>Free-Flow Speed = 110 km/hr.</i>				
A	7	110.0	0.33	770
B	11	110.0	0.51	1210
C	16	108.5	0.74	1740
D	22	97.2	0.91	2135
E	28	83.9	1.00	2350
F	varies	varies	varies	varies
<i>Free-Flow Speed = 100 km/hr.</i>				
A	7	100.0	0.30	700
B	11	100.0	0.48	1100
C	16	100.0	0.70	1600
D	22	93.8	0.90	2065
E	28	82.1	1.00	2300
F	varies	varies	varies	varies
<i>Free-Flow Speed = 90 km/hr.</i>				
A	7	90.0	0.28	630
B	11	90.0	0.44	990
C	16	90.0	0.64	1440
D	22	89.1	0.87	1955
E	28	80.4	1.00	2250
F	varies	varies	varies	varies

Equivalent passenger-car flow rate (adjusted to account for any prevailing conditions):

$$v_p = \frac{V}{PHF \times N \times f_{HV} \times f_p} \quad (3 - 5/1)$$

Where:

v_p = Hourly adjusted peak 15-minute flow rate (pc/hr./ln)

V = Hourly traffic volume (veh/hr.)

PHF = Peak hour factor (section 3-4/1)

N = Number of freeway lanes in one direction of travel.

f_{HV} = Heavy vehicle adjustment factor

f_p = Driver population factor

$$f_{HV} = \frac{1}{1 + P_T (E_T - 1) + P_R (E_R - 1)} \quad (3 - 5/2)$$

Where:

f_{HV} = Adjustment factor for combined effect of trucks, recreational vehicles, and buses on the traffic stream

E_T, E_R = Passenger-car equivalents for trucks/buses and recreational vehicles, per tables (3-5/6) and (3-5/7) respectively.

P_T, P_R = Proportion of trucks/buses and recreational vehicles, respectively, in the traffic stream [4, p.141]

Table 3-5/6: Passenger-Car Unit Equivalents for Trucks and Buses on Upgrades [4, p.142]

<i>Grade</i> (%)	<i>Length</i> (km)	<i>E_T</i>								
		<i>Percent Trucks and Buses</i>								
		2	4	5	6	8	10	75	20	25
< 2	All	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5
≥ 2-3	0.00-0.40	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5
	> 0.40-0.80	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5
	> 0.80-1.20	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5
	> 1.20-1.60	2.0	2.0	2.0	2.0	1.5	1.5	1.5	1.5	1.5
	> 1.60-2.40	2.5	2.5	2.5	2.5	2.0	2.0	2.0	2.0	2.0
	> 2.40	3.0	3.0	2.5	2.5	2.0	2.0	2.0	2.0	2.0
≥ 3-4	0.00-0.40	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5
	0.40-0.80	2.0	2.0	2.0	2.0	2.0	2.0	1.5	1.5	1.5
	> 0.80-1.20	2.5	2.5	2.0	2.0	2.0	2.0	2.0	2.0	2.0
	> 1.20-1.60	3.0	3.0	2.5	2.5	2.5	2.5	2.0	2.0	2.0
	> 1.60-2.40	3.5	3.5	3.0	3.0	3.0	3.0	2.5	2.5	2.5
	> 2.40	4.0	3.5	3.0	3.0	3.0	3.0	2.5	2.5	2.5
≥ 4-5	0.00-0.40	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5
	> 0.40-0.80	3.0	2.5	2.5	2.5	2.0	2.0	2.0	2.0	2.0
	> 0.80-1.20	3.5	3.0	3.0	3.0	2.5	2.5	2.5	2.5	2.5
	> 1.20-1.60	4.0	3.5	3.5	3.5	3.0	3.0	3.0	3.0	3.0
	> 1.60	5.0	4.0	4.0	4.0	3.5	3.5	3.0	3.0	3.0
≥ 5-6	0.00-0.40	2.0	2.0	1.5	1.5	1.5	1.5	1.5	1.5	1.5
	> 0.40-0.48	4.0	3.0	2.5	2.5	2.0	2.0	2.0	2.0	2.0
	> 0.48-0.80	4.5	4.0	3.5	3.0	2.5	2.5	2.5	2.5	2.5
	> 0.80-1.20	5.0	4.5	4.0	3.5	3.0	3.0	3.0	3.0	3.0
	> 1.20-1.60	5.5	5.0	4.5	4.0	3.0	3.0	3.0	3.0	3.0
	> 1.60	6.0	5.0	5.0	4.5	3.5	3.5	3.5	3.5	3.5
≥ 6	0.00-0.40	4.0	3.0	2.5	2.5	2.5	2.5	2.0	2.0	2.0
	> 0.40-0.48	4.5	4.0	1.5	3.5	3.5	3.0	2.5	2.5	2.5
	> 0.48-0.80	5.0	4.5	4.0	4.0	3.5	3.0	2.5	2.5	2.5
	> 0.80-1.20	5.5	5.0	4.5	4.5	4.0	3.5	3.0	3.0	3.0
	> 1.20-1.60	6.0	5.5	5.0	5.0	4.5	4.0	3.5	3.5	3.5
	> 1.60	7.0	6.0	5.5	5.5	5.0	4.5	4.0	4.0	4.0

Table 3-5/7: Passenger-Car Unit Equivalents for Recreational Vehicles on Specific Upgrades [4, p.143]

<i>Grade</i> (%)	<i>Length</i> (km)	<i>E_R</i>								
		<i>Percent Recreational Vehicles</i>								
		2	4	5	6	8	10	15	20	25
≤ 2	All	1.2	1.2	1.2	1.2	1.2	1.2	1.2	1.2	1.2
≥ 2-3	0.00-0.80	1.2	1.2	1.2	1.2	1.2	1.2	1.2	1.2	1.2
	> 0.80	3.0	1.5	1.5	1.5	1.5	1.5	1.2	1.2	1.2
≥ 3-4	0.00-0.40	1.2	1.2	1.2	1.2	1.2	1.2	1.2	1.2	1.2
	0.40-0.80	2.5	2.5	2.0	2.0	2.0	2.0	1.5	1.5	1.5
	> 0.80	3.0	2.5	2.5	2.5	2.0	2.0	2.0	1.5	1.5
≥ 4.5	0.00-0.40	2.5	2.0	2.0	2.0	1.5	1.5	1.5	1.5	1.5
	0.40-0.80	4.0	3.0	3.0	3.0	2.5	2.5	2.0	2.0	2.0
	> 0.80	4.5	3.5	3.0	3.0	3.0	2.5	2.5	2.0	2.0
≥ 5	0.00-0.40	4.0	3.0	2.5	2.5	2.5	2.0	2.0	2.0	1.5
	0.40-0.80	6.0	4.0	4.0	3.5	3.0	3.0	2.5	2.5	2.0
	> 0.80	6.0	4.5	4.0	4.0	3.5	3.0	3.0	2.5	2.0

Table 3-5/8: Freeway Reduction in Free-Flow Speed [4, p.654]

(a) Freeway Adjustments for Number of Lanes

<i>Number of Lanes (One Direction)</i>	<i>Reduction in Free-Flow Speed, f_N (km/hr.)</i>
≥ 5	0.0
4	2.4
3	4.8
2	7.3

(b) Freeway Adjustments for Lane Width

<i>Lane Width (m)</i>	<i>Reduction in Free-Flow Speed, F_{LW} (km/hr.)</i>
3.6	0.0
3.5	1.0
3.4	2.1
3.3	3.1
3.2	5.6
3.1	8.1
3.0	10.6

(c) Freeway Adjustment for Right-Shoulder Lateral Clearance

<i>Right-Shoulder Lateral Clearance (m)</i>	<i>Reduction in Free-Flow Speed, f_{LC} (km/hr.)</i>			
	<i>Lanes in One Direction</i>			
	<i>2</i>	<i>3</i>	<i>4</i>	<i>≥ 5</i>
≥ 1.8	0.0	0.0	0.0	0.0
1.5	1.0	0.7	0.3	0.2
1.2	1.9	1.3	0.7	0.4
0.9	2.9	1.9	1.0	0.6
0.6	3.9	2.6	1.3	0.8
0.3	4.8	3.2	1.6	1.1
0.0	5.8	3.9	1.9	1.3

3-6 REFERENCES

- [1] "*Highway Capacity Manual, HCM 2010*", Transportation Research Board, National Research Council, USA, Washington D.C., 2010.
- [2] Garber, N.J. and Hoel, L.A, "*Traffic & Highway Engineering*", Cengage Learning, USA, 2009.
- [3] AASHTO, "*A Policy on Geometric Design of Highways and Streets*", American Association of State Highway and Transportation Officials, USA, 2011.
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CHAPTER 4

SIGHT DISTANCES

The design of highways, need to provide for drivers a sufficient distance of clear vision ahead, so they can avoid hitting unexpected obstacles, and can pass slower vehicles without danger.

Sight Distance: is the length of highway visible ahead to the driver of a vehicle. The three general types of sight distances (stopping, decision, and passing) need to be considered. [3, p.187]

4-1 STOPPING SIGHT DISTANCE

The stopping sight distance: is the minimum distance required to stop a vehicle traveling near the design speed, before it reaches a stationary object in the vehicles path. It is the sum of two distances: perception- reaction, and braking.

4-1/1 PERCEPTION – REACTION DISTANCE

Perception – Reaction Distance (d_1): Distance traveled from the time the object is sighted to the instant the brakes are applied.

For design purposes, the combined perception and brake- reaction time of 2.5 seconds is usually recommended.

$$d_1 = 0.278 \times V \times t \quad (4 - 1/1)$$

Where:

d_1 = Perception- reaction distance, meters

V = Design speed, km/hr.

t = Perception- reaction time, sec., [1, p.3-4].

4-1/2 BRAKING DISTANCE ON LEVEL ROADWAY

Braking Distance (d_2): is the distance required for stopping a vehicle after the brakes are applied.

$$d_2 = \frac{0.039 \times V^2}{a} \quad \text{For Level Roadway} \quad (4 - 1/2)$$

Where:

d_2 = Braking distance, meters.

V = Design speed, km/hr.

a = Deceleration rate of vehicle, m/sec.²

A deceleration rate of 3.4 m/sec² is generally recommended for calculations of stopping sight distance). [1, p.3-3]

4-1/3 BRAKING DISTANCE ON A GRADE

When a highway is on a grade, the braking distance (d_2) is determined as follows: [1, p.3-5]

$$d_2 = \frac{V^2}{254 \left[\frac{a}{9.81} \pm \frac{G}{100} \right]} \quad \text{For Roadway on Grade} \quad (4 - 1/3)$$

Where:

G = Grade in percent

+ For upgrade

– For downgrade

Some designers do not generally adjust stopping distance for grade, unless the roads are one-way, as on divided highways with separate vertical profiles for each direction of travel. [1, p.3-6]

4-1/4 DESIGN VALUES OF STOPPING SIGHT DISTANCE

The general design equation for determining minimum stopping sight distance (S_{stopping}) in meters is:

$$S_{\text{stopping}} = 0.278 \times V \times t + \frac{V^2}{254 \left[\frac{a}{9.81} \pm \frac{G}{100} \right]} \quad (4 - 1/4)$$

Where:

V = design speed, km/hr.

t = perception- reaction time, seconds

a = deceleration rate, m/sec²

G = grade in percent

The minimum design distances on level roadways are shown in table (4-1/1), and for roadways on grade in table (4-1/2). [1, p.3-5]

Table 4-1/1: Minimum Stopping Sight Distance on Level Roadways [1, p.3-4]

Design Speed (km/hr.)	Brake Reaction Distance (m)	Braking Distance on Level (m)	Stopping Sight Distance	
			Calculated (m)	Design (m)
20	13.9	4.6	18.5	20
30	20.9	10.3	31.2	35
40	27.8	18.4	46.2	50
50	34.8	28.7	63.5	65
60	41.7	41.3	83.0	85
70	48.7	56.2	104.9	105
80	55.6	73.4	129.0	130
90	62.6	92.9	155.5	160
100	69.5	114.7	184.2	185
110	76.5	138.8	215.3	220
120	83.4	165.2	248.6	250
130	90.4	193.8	284.2	285

Note: Brake reaction distance predicated on a time of 2.5 sec.; deceleration rate of 3.4 m/sec²

Table 4-1/2: Minimum Stopping Sight Distance on Grades [1, p.3-5]

Design Speed (km/hr.)	Stopping Sight Distance (m)					
	Downgrades			Upgrades		
	3%	6%	9%	3%	6%	9%
20	20	20	20	19	18	18
30	32	35	35	31	30	29
40	50	50	53	45	44	43
50	66	70	74	61	59	58
60	87	92	97	80	77	75
70	110	116	124	100	97	93
80	136	144	154	123	118	114
90	164	174	187	148	141	136
100	194	207	223	174	167	160
110	227	243	262	203	194	186
120	263	281	304	234	223	214
130	302	323	350	267	254	243

4-2 DECISION SIGHT DISTANCE

Decision Sight Distance: is the distance needed for a driver to detect unexpected or difficult condition, select an appropriate speed and path, and complete avoidance maneuvers at complex highway locations, such as interchanges or intersections.

4-2/1 DISTANCES, WHERE A (STOP) IS THE APPROPRIATE AVOIDANCE MANEUVER

Among the five general categories for avoidance maneuvers (A ,B, C, D ,E); the maneuvers A and B include stops on rural and urban highway respectively.

The decision sight distance is the combination of pre- maneuver distance and braking distance, similar to stopping sight distance, but with longer reaction time as shown in table (4-2/1).

4-2/2 DISTANCES WHERE (SPEED, PATH, AND DIRECTION) CHANGE IS THE APPROPRIATE AVOIDANCE MANEUVER

Avoidance maneuvers C, D, E include speed, path, and direction change on rural, suburban, and urban roads respectively.

The decision sight distance is based on the distance traveled as the vehicle executes the required maneuver (during the total pre-maneuver and maneuver time), as shown in table (4-2/1).

4-2/3 DESIGN VALUES OF DECISION SIGHT DISTANCE

Decision sight distance values, for various situations, rounded for design, are presented in table (4-2/1), together with assumed criteria and equations.

Shorter distances are generally needed for rural roads, and for locations where a stop is the appropriate maneuver.

Table 4-2/1: Decision Sight Distance [1, p.3-7]

Design Speed (km/hr.)	Decision Sight Distance (m)				
	Avoidance Maneuver				
	A	B	C	D	E
50	70	155	145	170	195
60	95	195	170	205	235
70	115	235	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

Avoidance Maneuver A: Stop on rural road— $t = 3.0$ sec.

Avoidance Maneuver B: Stop on urban road— $t = 9.1$ sec.

Avoidance Maneuver C: Speed/path/direction change on rural road— t varies between 10.2 and 11.2 sec.

Avoidance Maneuver D: Speed/path/direction change on suburban road— t varies between 12.1 and 12.9 sec.

Avoidance Maneuver E: Speed/path/direction change on urban road— t varies between 14.0 and 14.5 sec.

Calculation of the decision sight distance for avoidance maneuvers A and B

$$DSD = 0.278Vt + 0.039 \frac{V^2}{a} \quad (4-2/1)$$

Where:

DSD = decision sight distance, m

t = pre-maneuver time, sec.

V = design speed, km/hr.

a = driver deceleration, m/sec.

$$= 3.4 \text{ m/sec.}^2$$

Calculation of the decision sight distance for avoidance maneuvers C, D, and E

$$DSD = 0.278Vt \quad (4-2/2)$$

Where:

DSD = decision sight distance, m

t = total pre-maneuver and maneuver time, sec.

V = design speed, km/hr.

4-3 PASSING SIGHT DISTANCE

Passing Sight Distance: The length of highway required to complete normal passing maneuvers in which the passing driver can determine that there are no potentially conflicting vehicles ahead before beginning the maneuver. [2, p.9-13]

4-3/1 ELEMENTS OF PASSING SIGHT DISTANCE

The minimum passing sight distance for 2-lane highways is determined as the sum of the four distances shown in figure (4-3/1).

4-3/2 GENERAL EQUATION OF TOTAL PASSING SIGHT DISTANCE

The total minimum passing sight distance;

$$\text{Minimum } S_{\text{passing}} = d_1 + d_2 + d_3 + d_4 \quad (4-3/1)$$

Where:

d_1 = Initial maneuver distance, meters.

$$= 0.278 \times t_i \left(V - m + \frac{a \times t_i}{2} \right) \quad (4-3/2)$$

d_2 = Occupation of left lane distance, meters.

$$= 0.278 \times V \times t_2 \quad (4-3/3)$$

d_3 = Clearance distance (30-90 meters).

d_4 = Opposing vehicle distance, meters.

$$= 0.67 \times d_2 \quad (4-3/4)$$

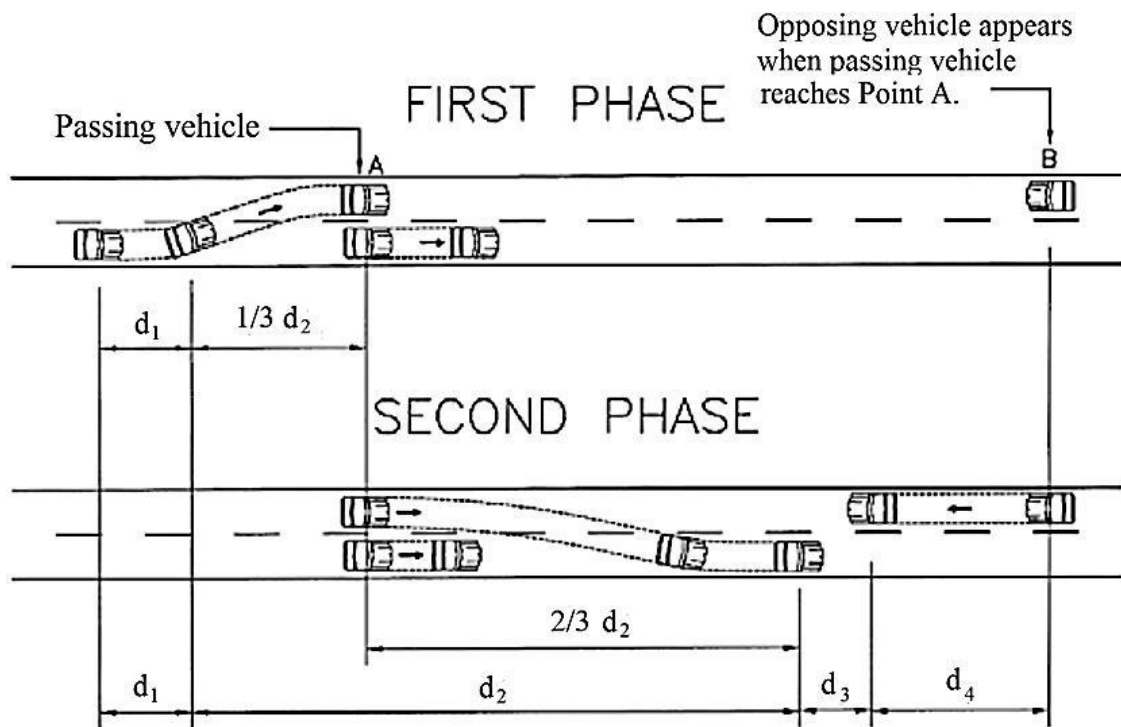
t_i = Time of initial maneuver, seconds; (2.25-2.41 sec.)

V = Average speed of passing vehicle, km/hr.

m = Difference in speed of passed and passing vehicles, km/hr. (15-19 km/hr.).

a = Average acceleration, (2.25-2.41) km/hr./sec.

t_2 = Time passing vehicle occupies the left Lane, sec. (9.3 – 11.3 sec.).



d_1 – Distance traversed during perception and reaction time and during the initial acceleration to the point of encroachment on the left lane.

d_2 – Distance traveled while the passing vehicle occupies the left lane.

d_3 – Distance between the passing vehicle at the end of its maneuver and the opposing vehicle.

d_4 – Distance traversed by an opposing vehicle for two-thirds of the time the passing vehicle occupies the left lane, or $2/3$ of d_2 above.

Figure 4-3/1: Elements of Passing Sight Distance for Two-Lane Highways [4, p.119]

4-3/3 DESIGN VALUES OF PASSING SIGHT DISTANCE

Actual driver behavior in passing maneuvers varies widely. The design criteria for passing sight distance should accommodate the behavior of a high percentage of drivers rather than just the average driver [1, p.3-12]

The design values for minimum passing sight distance are presented in table (4-3/1), and are shown in comparison to stopping sight distance criteria in figure (4-3/2).

The passing sight distance values in table (4-3/1) are sufficient for a single or isolated P- vehicle passing. It is not practical to assume multiple passing in developing minimum design criteria.

Longer sight distances are often used in design to accommodate an occasional multiple passing maneuver, or a passing maneuver involving a truck, [1, p.3-12]

4-4 CRITERIA FOR MEASURING SIGHT DISTANCES

The sight distance along a roadway, throughout which an object of specified height is continuously visible to the driver, is dependent on height of the drivers eye above road surface, the object height above road surface, and height of lateral obstructions within the drivers line of sight.

The height of drivers eye is considered to be 1.08 m for passenger cars and 2.33m for trucks.

The height of object is considered to be 0.60m for both stopping and decision sight distances, and 1.08m for passing sight distance.

The headlight mounting height of 0.60 m, with an upward angle of 1 degree along vehicles longitudinal axis, is used for stopping sight distance control on sag vertical curves.

Table 4-3/1: Passing Sight Distance for Design of Two-Lane Highway [1, p.3-9]

Design Speed (km/hr.)	Assumed Speeds (km/hr.)		Passing Sight Distance (m)
	Passed Vehicle	Passing Vehicle	
30	11	30	120
40	21	40	140
50	31	50	160
60	41	60	180
70	51	70	210
80	61	80	245
90	71	90	280
100	81	100	320
110	91	110	355
120	101	120	395
130	111	130	440

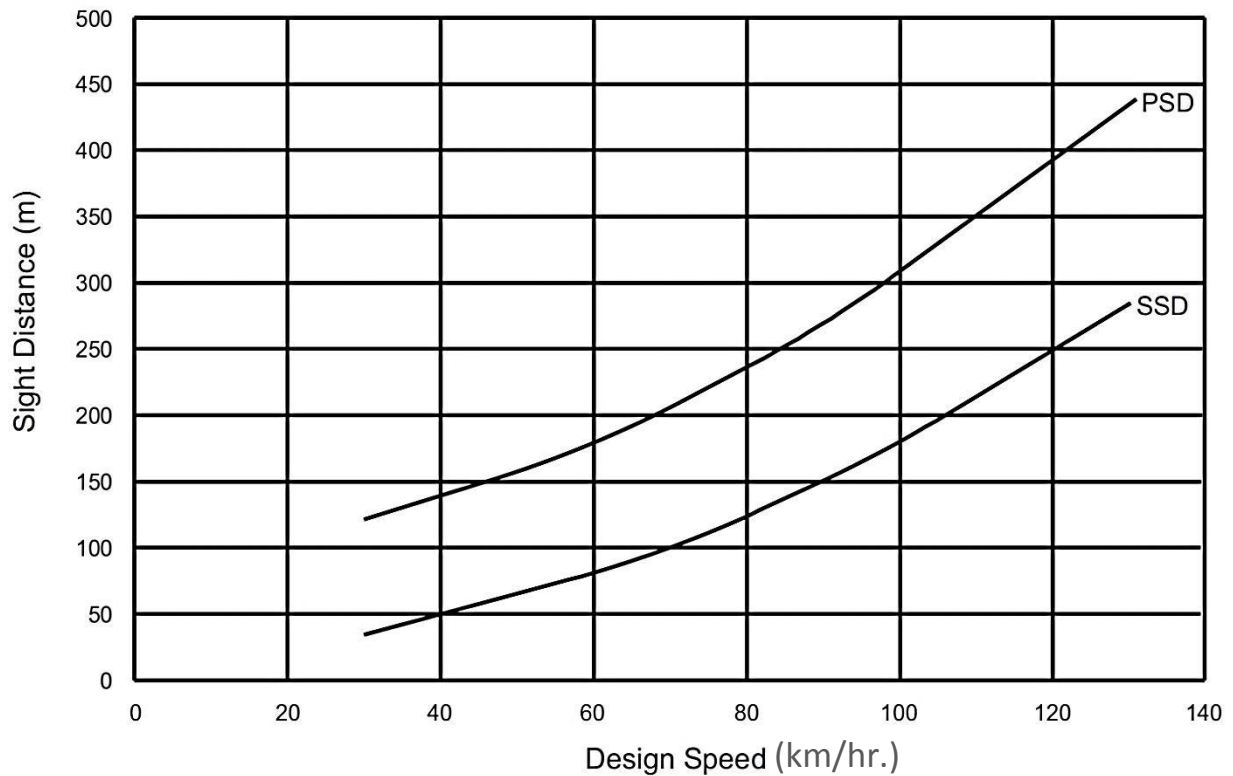


Figure 4-3/2: Comparison of Design Values for Passing Sight and Stopping Sight Distance
[1, p.3-11]

4-5 REFERENCES

- [1] AASHTO, "*A Policy on Geometric Design of Highways and Streets*", American Association of State Highway and Transportation Officials, USA, 2011.
- [2] "*Highway Capacity Manual, HCM 2010*", Transportation Research Board, National Research Council, USA, Washington D.C., 2010.
- [3] Wright, P.H. and Dixon, K. K., "*Highway Engineering*", John Wiley & Sons, USA, 2004.
- [4] AASHTO, "*A Policy on Geometric Design of Highways and Streets*", American Association of State Highway and Transportation Officials, USA, 2004.

CHAPTER 5

HORIZONTAL ALIGNMENT

Highways have usually tangent distances, that are connected by very gradual circular curves, that may be complemented by transitional spiral curves, to accommodate a given design speed with comfort and safety.

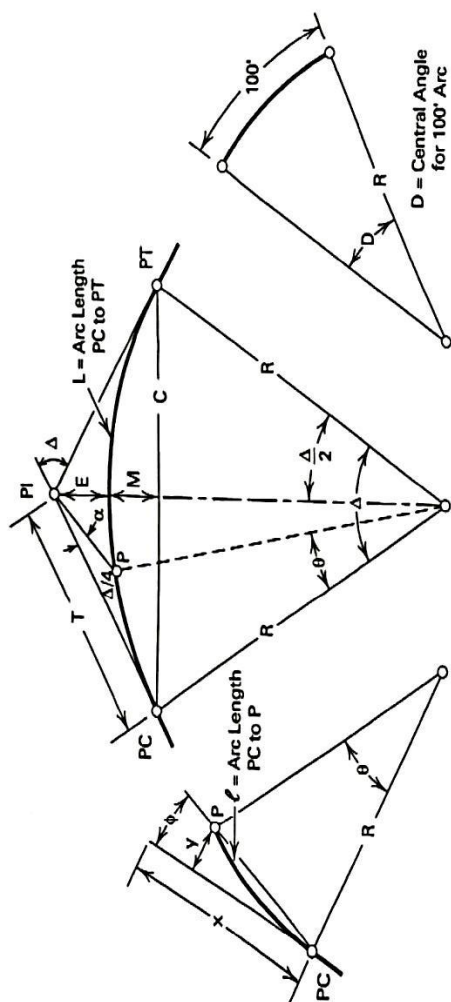
5-1 HORIZONTAL CIRCULAR CURVES

The four types of circular curve applications are shown in figure (5-1/1). The (broken-type) arrangement should be avoided in horizontal alignment of highway except for very unusual topographical or right – of- way conditions. Compound highway curves should also be used with caution, and when conditions make their use necessary, the radius of the flatter curve should not exceed 1.5 times the radius of the sharper curve [1, p.3-112]. Reverse curves would be used only for low-speed roads, such as those in mountainous terrain.

The properties of a simple circular curve are shown in figure (5-1/2), assuming a station length of 100m.

5-1/1 MINIMUM RADIUS OF CIRCULAR CURVES

The minimum radius of circular curve ($R_{\min.}$) for a given design speed (V), is determined from the maximum rate of superelevation ($e_{\max.}$), and the maximum side friction factor ($f_{\max.}$), using the simplified curve equation, as shown in table (5-1/1).



VARIABLES

PC = Point of curvature (Beginning of curve)

PT = Point of tangency (End of curve)

PI = Point of intersection

Δ = Central angle

L = Length of curve (PC to PT) (2)

l = Length of arc (PC to P) (2)

θ = Central angle for arc length l

T = Tangent length (PC to PI & PT to PI) (2)

φ = Deflection angle at PC between tangent and chord for P

α = Deflection angle at PI between tangent and line from PI to P

x = Tangent distance from PC to P (2)

y = Tangent offset P (2)

PI = Point of intersection

D = Degree of curvature (1)

R = Radius of curve (2)

E = External distance (2)

M = Middle ordinate (2)

C = Chord length (2)

D = Central Angle
for 100' Arc

CIRCULAR CURVE EQUATIONS

$$D = \frac{5729.57795}{R} \quad (\text{arc def.}) \quad C = 2 R \sin \frac{\Delta}{2}$$

$$L = \frac{2\pi R \Delta}{360}$$

$$l = \frac{100 \theta}{D}$$

$$T = R \tan \frac{\Delta}{2}$$

$$E = R \left(\sec \frac{\Delta}{2} - 1 \right)$$

$$M = R \left(1 - \cos \frac{\Delta}{2} \right)$$

$$\phi = \frac{\theta}{2} = \frac{ID}{200}$$

For any tangent distance x,

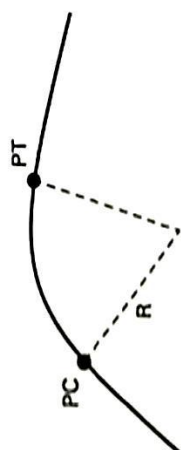
$$y = R - [R^2 - x^2]^{1/2}$$

For any arc length l,

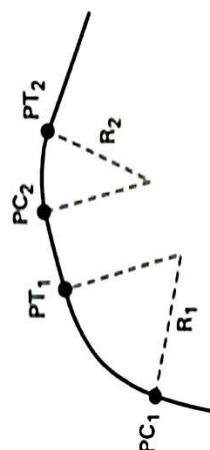
$$x = R \sin \theta$$

$$y = R (1 - \cos \theta)$$

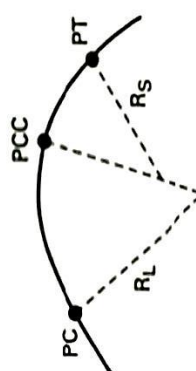
NOTES: (1) This variable used only for curve definition in traditional US units.
(2) Units for these variables can be expressed in either feet or meters.



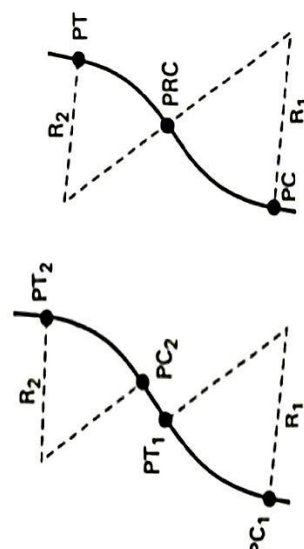
(a) Simple Horizontal Curve



(b) Broken Back Curve



(c) Compound Curve



(d) Reverse Curve

Figure 5-1/1: Four Types of Circular Curve Application [2, p.175]

Table 5-1/1: Minimum Radius Using Limiting Values of e and f [1, p.3-32]

Design Speed (km/hr.)	Maximum e (%)	Maximum f	Total (e/100+f)	Calculated Radius (m)	Rounded Radius (m)
15	4.0	0.40	0.44	4.0	4
20	4.0	0.35	0.39	8.1	8
30	4.0	0.28	0.32	22.1	22
40	4.0	0.23	0.27	46.7	47
50	4.0	0.19	0.23	85.6	86
60	4.0	0.17	0.21	135.0	135
70	4.0	0.15	0.19	203.1	203
80	4.0	0.14	0.18	280.0	280
90	4.0	0.13	0.17	375.2	375
100	4.0	0.12	0.16	492.1	492
15	6.0	0.40	0.46	3.9	4
20	6.0	0.35	0.41	7.7	8
30	6.0	0.28	0.34	20.8	21
40	6.0	0.23	0.29	43.4	43
50	6.0	0.19	0.25	78.7	79
60	6.0	0.17	0.23	123.2	123
70	6.0	0.15	0.21	183.7	184
80	6.0	0.14	0.20	252.0	252
90	6.0	0.13	0.19	335.7	336
100	6.0	0.12	0.18	437.4	437
110	6.0	0.11	0.17	560.4	560
120	6.0	0.09	0.15	755.9	756
130	6.0	0.08	0.14	950.5	951
15	8.0	0.40	0.48	3.7	4
20	8.0	0.35	0.43	7.3	7
30	8.0	0.28	0.36	19.7	20
40	8.0	0.23	0.31	40.6	41
50	8.0	0.19	0.27	72.9	73
60	8.0	0.17	0.25	113.4	113
70	8.0	0.15	0.23	167.8	168
80	8.0	0.14	0.22	229.1	229
90	8.0	0.13	0.21	303.7	304
100	8.0	0.12	0.20	393.7	394
110	8.0	0.11	0.19	501.5	501
120	8.0	0.09	0.17	667.0	667
130	8.0	0.08	0.16	831.7	832
15	10.0	0.40	0.50	3.5	4
20	10.0	0.35	0.45	7.0	7
30	10.0	0.28	0.38	18.6	19
40	10.0	0.23	0.33	38.2	38
50	10.0	0.19	0.29	67.9	68
60	10.0	0.17	0.27	105.0	105
70	10.0	0.15	0.25	154.3	154
80	10.0	0.14	0.24	210.0	210
90	10.0	0.13	0.23	277.3	277
100	10.0	0.12	0.22	357.9	358
110	10.0	0.11	0.21	453.7	454
120	10.0	0.09	0.19	596.8	597
130	10.0	0.08	0.18	739.3	739
15	12.0	0.40	0.52	3.4	3
20	12.0	0.35	0.47	6.7	7
30	12.0	0.28	0.40	17.7	18
40	12.0	0.23	0.35	36.0	36
50	12.0	0.19	0.31	63.5	64
60	12.0	0.17	0.29	97.7	98
70	12.0	0.15	0.27	142.9	143
80	12.0	0.14	0.26	193.8	194
90	12.0	0.13	0.25	255.1	255
100	12.0	0.12	0.24	328.1	328
110	12.0	0.11	0.23	414.2	414
120	12.0	0.09	0.21	539.9	540
130	12.0	0.08	0.20	665.4	665

Note: In recognition of safety considerations, use of $e_{max} = 4.0\%$ should be limited to urban conditions.

$$R_{min} = \frac{V^2}{127 (0.01 e_{max} + f_{max})} \quad (5 - 1/1)$$

5-1/2 MAXIMUM SUPERELEVATION RATES

The maximum rates of superelevation for highway design, are controlled by:

- Climatic conditions (rain, snow...)
- Terrain type (Flat, Rolling, Mountainous).
- Type of area (Urban, Rural).
- Frequency of slow moving vehicles.

The highest superelevation rate for highways in common use is 10%, although 12% is used in low volume unpaved road to facilitate drainage. Generally, 8% is recognized as a reasonable maximum value for superelevation rate of highways.

A maximum superelevation rate of 4-6% is applicable for urban design with traffic congestions, [1, p.3-30]

5-1/3 SIDE FRICTION FACTORS

The side friction factor is the friction force divided by the component of the weight perpendicular to the pavement surface.

The upper limit of the side friction factor (f) is the point at which the tire would begin to skid, and the values used in curve design should be less than the coefficient of friction at impending skid. The recommended side friction factor, for use in horizontal curve design of highways, at different vehicle speeds, is shown in figure (5-1/3) [1, p.3-25].

The (f) values vary with the design speed from 0.40 (at 15 km/hr.), 0.15 (at 70 km/hr.), to 0.075 (at 130 km/hr.).

5-1/4 COMPOUND CIRCULAR CURVES FOR RAMPS AND INTERSECTIONS

For turning roadways, compound curves can be used for design speeds not exceeding 70 km/hr. for higher speeds, a large amount of right-of-way is needed, to provide driver comfort and safety. For compound curves on turning roadways, it is preferable that the ratio of the flatter radius to the sharper radius not exceed 2:1 [1, p.3-58]

Curves that are compounded should not be too short, to enable drivers to decelerate at a reasonable rate (about 3 km/hr./sec.). The minimum compound circular curve lengths for different radii are presented in table (5-1/2).

5-2 SPIRAL CURVE TRANSITIONS

The incorporation of spiral curve transition (Euler or Clothoid), between tangent and sharp circular curve, as well as between circular curves of different radii, in horizontal alignment of highways, have the following advantages:

- Simulates the natural turning path of a vehicle to promote a uniform speed, with gradual change of lateral force.

- Provides a suitable length for superelevation runoff, and widening of sharp curves.
- Provides better appearance for the highway, avoiding noticeable breaks in alignment.

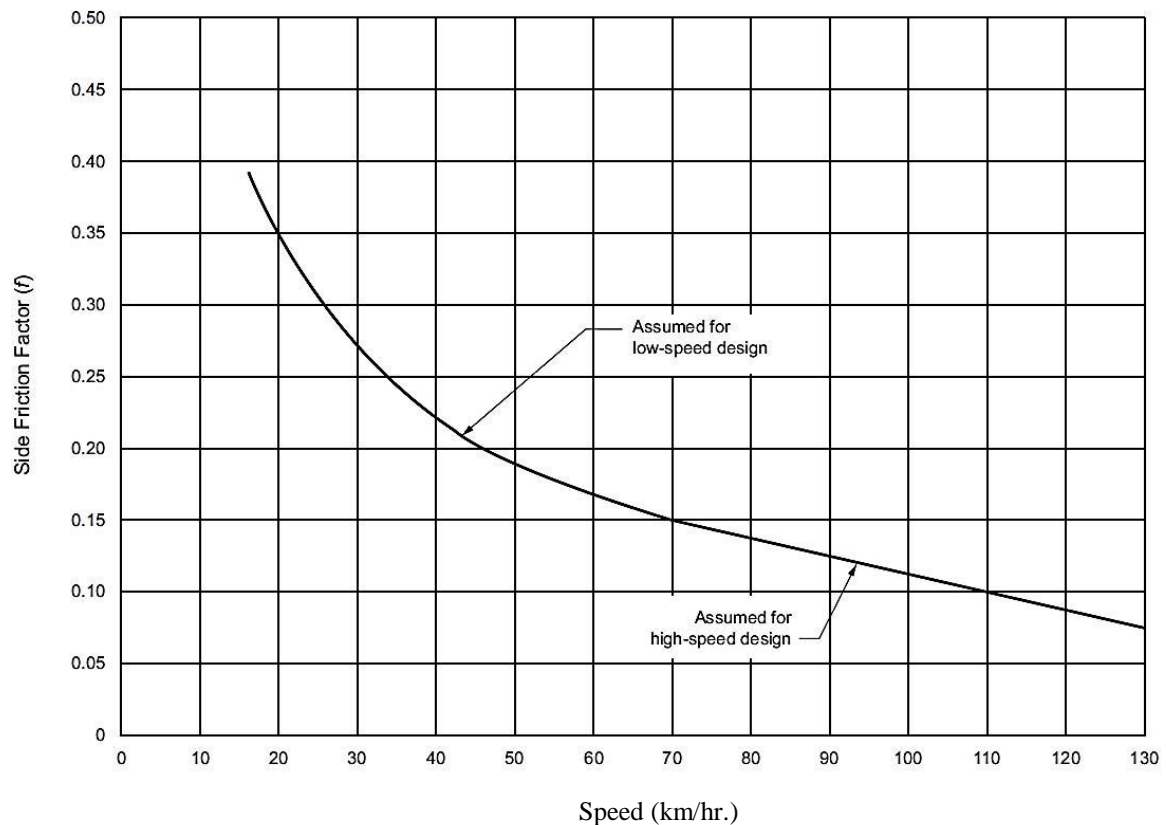


Figure 5-1/3: Side Friction Factors Assumed for Design [1, p.3-25]

Table 5-1/2: Length of Circular Arcs for Different Compound Curve Radii [1, p.3-58]

Radius (m)	Minimum Length of Circular Arc (m)	
	Acceptable	Desirable
30	12	20
50	15	20
60	20	30
75	25	35
100	30	45
125	35	55
150 or more	45	60

5-2/1 DETAILS OF SPIRAL COMPOUNDED WITH CIRCULAR CURVE

The details of a transition spiral (clothoid) connecting the tangent at point (A); where the radius is infinite, with point (C) on a circular curve with radius (R_c), are shown in figure (5-2/1) together with related formulas.

5-2/2 MAXIMUM RADIUS FOR USE OF A SPIRAL CURVE TRANSITION

The maximum radius for use of a spiral should be based on a minimum lateral acceleration rate of 1.3 m/sec^2 . Only radii below this maximum are likely to obtain safety and operational benefits from the use of spiral transition curves. [1, p.3-70]

The values of maximum radius of circular curves for the desired use of spiral are shown in table (5-2/1).

5-2/3 MINIMUM LENGTH OF SPIRAL CURVE

Two criteria are used together to determine the minimum length of spiral, based on driver comfortable increase in lateral acceleration, and shift in a vehicle lateral position (due to steering) within its lane, consistent with that produced by the spiral path. The minimum spiral length can be computed as shown in equations (5-2/2) and (5-2/3).

Besides, the minimum length of spiral should be set equal to the length of superelevation runoff.

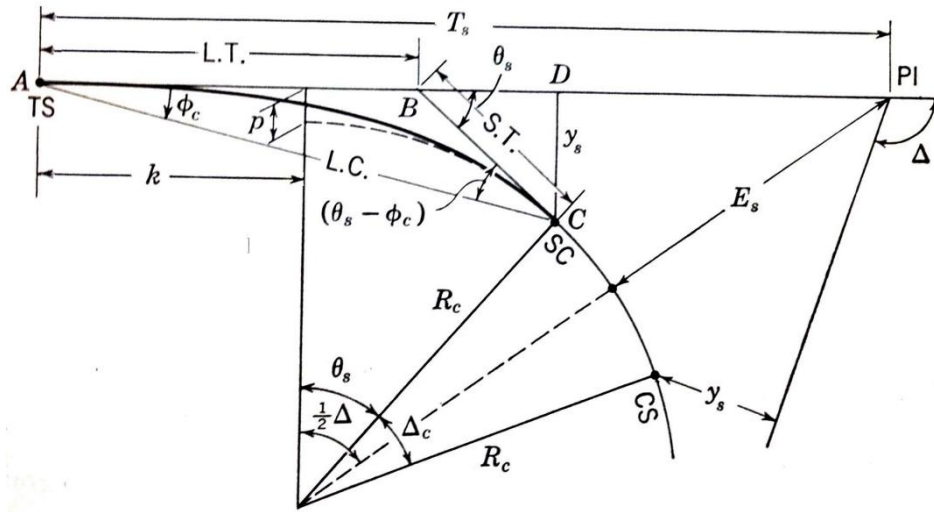


Figure 5-2/1: Parts of the Spiral and Osculating Circle [3, p.153]

Notation and Formulas

TS = Point of change from tangent to spiral;

SC = Point of change from spiral to circle;

CS = Point of change from circle to spiral;

l = Spiral arc from the TS to any point on spiral;

l_s = Total length of spiral from TS to SC ;

θ = Central angle of spiral arc l ;

θ_s = Central angle of spiral arc l_s , called "spiral angle";

ϕ_c = Spiral deflection angle at the TS from initial tangent to any point on spiral;

Δ = Total central angle

Δ_c = Central angle of circular arc of length L_c extending from SC to CS ;

y = Tangent offset of any point on spiral with reference to TS and initial tangent;

y_s = Tangent offset at the SC ;

x = Tangent distance of any point on spiral with reference to TS and initial tangent;

x_s = Tangent distance at the SC ;

p = Offset from the initial tangent to the PC of the shifted circle; $p = y_s - R_c \text{ vers } \theta_s$

k = Abscissa of the shifted PC referred to the TS ; $k = x_s - R_c \sin \theta_s$

T_s = Total tangent distance = distance from PI to TS , or from PI to ST ;

$$T_s = (R_c + p) \tan \frac{1}{2} \Delta + k$$

$$E_s = \text{Total external distance} = (R_c + p) \text{exsec} \frac{1}{2} \Delta + p$$

LC = Long chord; LT = Long tangent; ST = Short tangent.

$$l_s = 2 \theta_s R_c ; \theta_s = \text{radians}$$

$$\theta = \left(\frac{l}{l_s} \right)^2 \theta_s ; \phi = \frac{\theta}{3} - C_s$$

$$\sqrt{L_s R_c} = \text{Flatness Parameter of Spiral}$$

R_c = Radius of Circular Curve.

Table 5-2/1: Maximum Radius for Use of a Spiral Curve Transition [1, p. 3-71]

Design speed (km/hr.)	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

Note: The effect of spiral curve transitions on lateral acceleration is likely to be negligible for larger radii.

5-2/4 MAXIMUM LENGTH OF SPIRAL CURVE

Spirals should not be so long, relative to the length of circular curves, that drivers are misled about the sharpness of the approaching curve [1, p.3-72].

The maximum length of the spiral can be computed as shown in equation (5-2/1).

5-2/5 DESIRABLE LENGTH OF SPIRAL CURVE

The desirable length of the spiral is that length corresponding to 2 seconds of travel time at the design speed of the roadway, as shown in table (5-2/2).

If the desirable spiral length is less than, the minimum spiral curve length, the minimum length should be used in design.

Spiral lengths longer than desirable length may be needed at turning roadways, to develop the desired superelevation, provided that the maximum spiral length values are not exceeded.

Table 5-2/2: Desirable Length of Spiral Curve Transition [1, p.3-73]

Design Speed (km/hr.)	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

Maximum Length of spiral [1, p.3-72]

$$L_{s,\max} = \sqrt{24(p_{\max})R} \quad (5 - 2/1)$$

where:

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

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

R = radius of circular curve, m

Minimum Length of spiral [1, p. 3-71]

$L_{s,\min}$ should be the larger of:

$$L_{s,\min} = \sqrt{24(p_{\min})R} \quad (5 - 2/2)$$

or

$$L_{s,\min} = 0.0214 \frac{V^3}{RC} \quad (5 - 2/3)$$

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;

V = design speed, km/h

C = maximum rate of change in lateral acceleration (1.2 m/s³)

5-3 SUPERELEVATION ON CURVES

The transition from a tangent with either normal crown, or straight cross slope section, to a full curve superelevation section, must be accomplished without any reduction in speed to insure safety and comfort to the occupants of vehicle.

5-3/1 METHODS OF ATTAINING SUPERELEVATION

Four methods are commonly used to attain the full superelevation transition, as shown in figure (5-3/1). The change to a uniformly inclined section, need to be accomplished gradually as a constant rate along the centerline.

The first method which revolves the traveled way about the centerline is the most widely used, because of the less distortion to edge elevations.

5-3/2 MINIMUM LENGTH OF SUPERELEVATION RUNOFF (TANGENT-TO- CURVE TRANSITION DESIGN)

Superelevation runoff: is the distance required to accomplish transition from zero percent (Flat) section to a fully superelevated section.

The maximum grade difference (relative gradient) need to be limited for each design speed to provide longer runoff lengths at higher speeds. Relative gradients of 0.35 to 0.80 are usually used as shown in figure (5-3/2).

The minimum length of superelevation runoff can be determined from equation (5-3/1), using the adjustment factor for the number of rotated lanes of undivided highways (without a median), [1.p.3-62]. In divided highways (with a median), the location of axis of rotation depends on the median width as shown in the following three cases:

CASE 1: For narrow medians ($\leq 4\text{m}$), the whole of the traveled way, including the median is superelevated as a plane section.

CASE2: For medians with widths (4-18m), the median is held in a horizontal plane, and the two traveled ways are rotated separately around the median edges.

CASE3: For median widths of (18m or more), the two traveled ways are treated separately for runoff, which results in variable elevations at median edges (sloped median). [1, p.3-80].

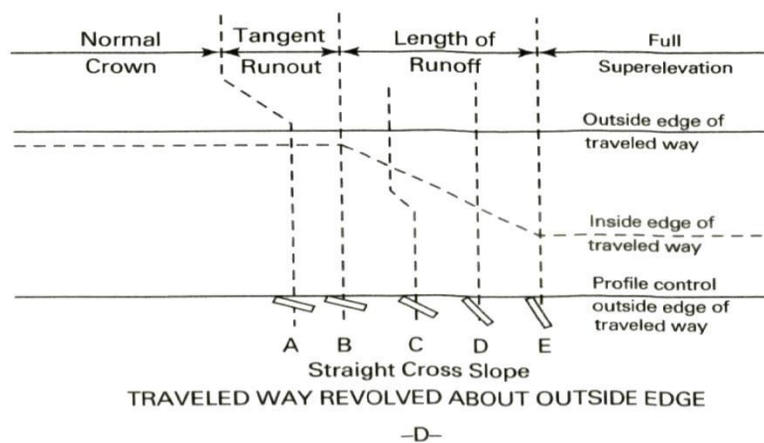
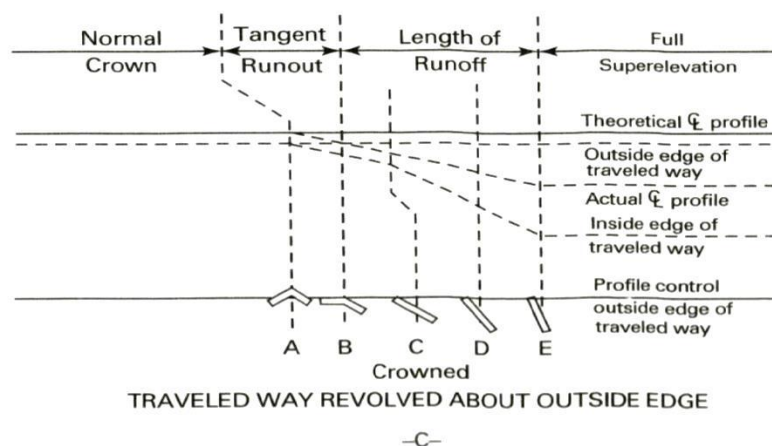
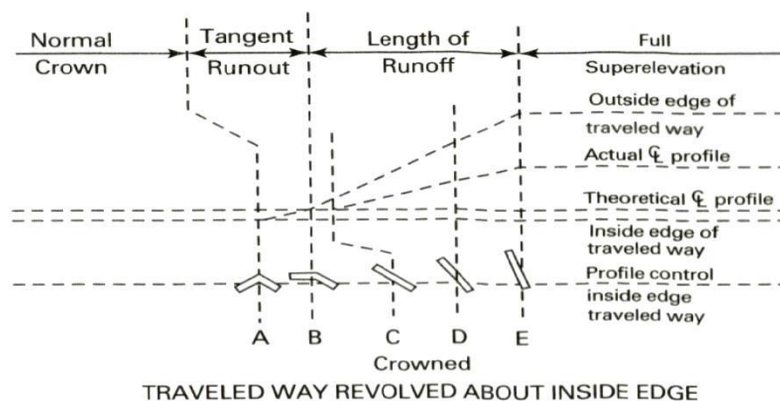
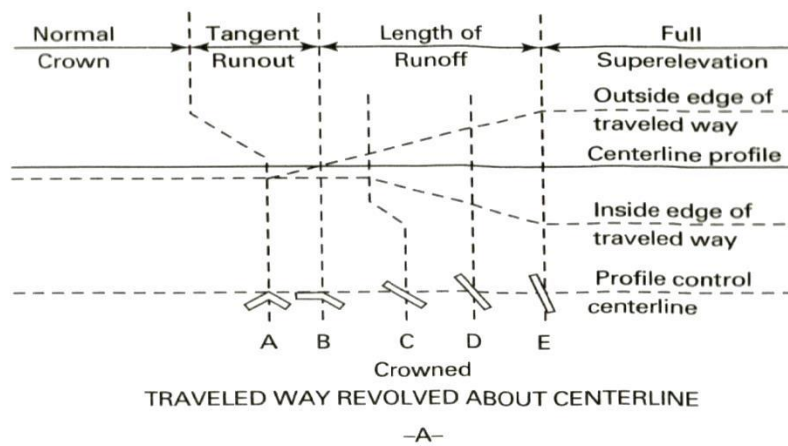


Figure 5-3/1: Methods of Attaining Superelevation [2. p.180]

Normal practice is to divide the superelevation runoff length between the tangent and curve, and to avoid placing the entire runoff length on either the tangent or the curve in order to minimize lateral acceleration and the vehicles lateral shift. The design runoff locations are shown in table (5-3/1).

Table 5-3/1: Design Runoff Locations

Design speed (km/hr.)	Portion of Runoff Located prior to the Curve			
	Number of Lanes Rotated			
	1.0	1.5	2.0-2.5	3.0-3.5
20-70	0.80	0.85	0.90	0.90
80-130	0.70	0.75	0.80	0.85

Typical minimum superelevation runoff lengths are presented in table (5-3/2), where one or two lanes are rotated, assuming a 3.60 m lane width.

5-3/3 MINIMUM LENGTH OF TANGENT RUNOUT (TANGENT-TO-CURVE TRANSITION DESIGN)

Tangent Runout: is the length of tangent required to transition from normal crown to zero percent (Flat) section.

The minimum length of tangent runout should be determined from equation (5-3/2) presented in figure (5-3/2), and typical values are shown in the 2.0 percent row of (e) values in table (5-3/2).

$$L_r = \frac{(wn_1) e_d}{\Delta} (b_w) \quad (5-3/1)$$

where:

L_r = minimum length of superelevation runoff, m

w = width of one traffic lane, m
(typically 3.6 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

Number of Lanes Rotated, n_1	Adjustment Factor,* b_w	Length Increase Relative to One-Lane Rotated, $(= n_1 b_w)$
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

$$L_t = \frac{e_{NC}}{e_d} L_r \quad (5-3/2)$$

where:

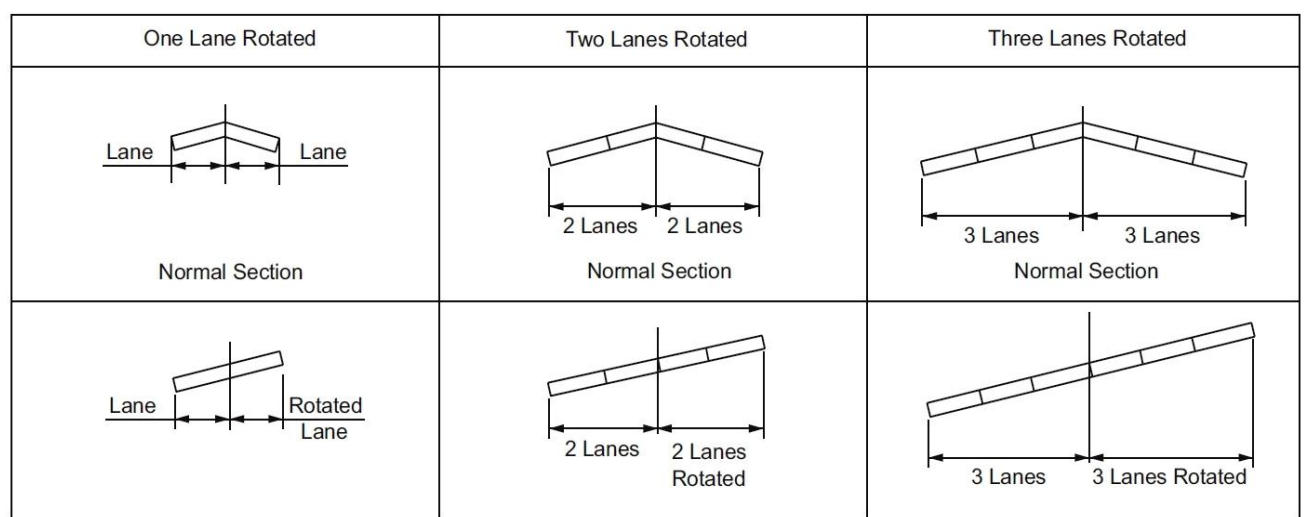
L_t = minimum length of tangent runoff, m

e_{NC} = normal cross slope rate, percent

e_d = design superelevation rate, percent

L_r = minimum length of superelevation runoff, m

Design Speed (km/hr.)	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



$$* b_w = [1 + 0.5 (n_1 - 1)] / n_1$$

Figure 5-3/2: Minimum Length of Superelevation runoff & Tangent runoff
[1, p.3-60], [1, p.3-66]

Table 5-3/2: Superelevation Runoff Lr (m) for Horizontal Curves [1, p.3-64]

$V_d=20$ km/hr.		$V_d=30$ km/hr.		$V_d=40$ km/hr.		$V_d=50$ km/hr.		$V_d=60$ km/hr.		$V_d=70$ km/hr.		$V_d=80$ km/hr.		$V_d=90$ km/hr.		$V_d=100$ km/hr.		$V_d=110$ km/hr.		$V_d=120$ km/hr.		$V_d=130$ km/hr.	
1	2	1	2	1	2	1	2	1	2	1	2	1	2	1	2	1	2	1	2	1	2	1	2
Number of Lanes Rotated. Note that 1 lane rotated is typical for a 2-lane highway, 2 lanes rotated is typical for a 4-lane highway, etc.																							
e (%)	L_r (m)	L_r (m)	L_r (m)	L_r (m)	L_r (m)	L_r (m)	L_r (m)	L_r (m)	L_r (m)	L_r (m)	L_r (m)	L_r (m)	L_r (m)	L_r (m)	L_r (m)	L_r (m)	L_r (m)	L_r (m)	L_r (m)	L_r (m)	L_r (m)	L_r (m)	L_r (m)
1.5	7	10	7	11	8	13	9	14	10	15	11	16	12	17	12	18	13	20	14	21	15	23	15
2.0	9	14	10	14	10	17	12	18	13	20	14	22	15	23	16	25	18	26	19	28	21	31	23
2.2	10	15	11	16	11	18	13	20	14	22	16	24	17	25	18	27	19	29	21	31	23	34	25
2.4	11	16	12	17	12	19	13	20	14	22	16	24	17	25	18	28	20	29	21	32	23	34	25
2.6	12	18	12	19	13	20	14	22	16	23	17	26	19	28	20	30	21	32	23	34	25	37	27
2.8	13	19	13	20	14	22	16	23	17	25	18	27	20	30	21	32	23	34	25	37	27	40	29
3.0	14	20	14	22	15	23	17	25	18	29	20	32	23	34	25	37	26	40	28	43	31	46	31
3.2	14	22	15	23	16	25	18	27	19	31	22	33	24	35	26	39	28	42	30	45	33	49	33
3.4	15	23	16	24	17	26	19	28	20	32	23	34	25	37	26	39	28	42	30	45	33	49	33
3.6	16	24	17	26	19	28	20	30	22	35	26	39	28	41	29	44	32	47	34	51	37	56	37
3.8	17	26	18	27	20	31	22	33	24	37	27	41	29	44	31	47	35	50	36	54	39	59	39
4.0	18	27	19	29	21	32	24	36	26	39	29	43	31	46	33	49	35	53	38	57	41	62	41
4.2	19	28	20	30	22	33	25	38	27	41	30	45	32	48	34	52	37	55	40	60	43	65	43
4.4	20	30	21	32	23	34	24	37	26	43	32	48	34	51	36	54	39	58	42	63	45	68	45
4.6	21	31	22	33	24	35	25	38	28	45	33	50	35	53	38	56	40	61	44	65	47	71	47
4.8	22	32	23	34	25	37	27	40	29	47	35	52	37	55	39	59	42	63	45	68	49	74	49
5.0	23	34	24	36	26	39	28	42	30	49	36	54	38	57	41	61	44	66	47	71	51	77	51
5.2	23	35	25	37	27	40	29	43	31	51	37	56	40	60	44	64	46	68	49	74	53	80	53
5.4	24	36	26	39	28	42	30	45	32	53	39	58	41	62	44	66	47	71	51	77	56	83	56
5.6	25	38	27	40	29	43	31	47	34	55	40	60	43	64	46	69	49	74	53	80	58	86	58
5.8	26	39	28	42	30	45	32	48	35	57	42	63	44	67	47	71	51	76	55	82	60	89	60
6.0	27	41	29	43	31	46	33	50	36	59	43	65	46	69	49	74	53	79	57	85	62	93	62
6.2	28	42	30	45	32	48	34	52	37	61	45	67	47	71	51	76	54	82	59	88	64	96	64
6.4	29	43	31	46	33	49	35	53	38	63	46	69	49	74	52	79	56	84	61	91	66	99	66
6.6	30	45	32	48	34	51	37	55	40	65	48	71	51	76	54	81	58	87	63	94	68	102	68
6.8	31	46	33	49	35	52	38	56	41	67	49	73	52	78	56	83	60	90	64	97	70	105	70
7.0	31	47	34	50	36	54	39	58	42	69	50	76	54	80	57	86	61	92	66	99	72	108	72
7.2	32	49	35	52	37	56	40	60	43	71	52	78	55	83	59	88	63	95	68	102	74	111	74
7.4	33	50	36	53	38	57	41	61	44	73	53	80	57	85	61	91	65	97	70	105	76	114	76
7.6	34	51	36	55	39	59	42	63	46	75	55	82	58	87	62	93	67	100	72	108	78	117	78
7.8	35	53	37	56	40	60	43	65	47	77	56	84	60	90	64	96	68	103	74	111	80	120	80
8.0	36	54	38	58	41	62	44	66	48	79	58	86	61	92	65	98	70	105	76	114	82	123	82
8.2	37	55	39	59	42	63	45	68	49	81	59	89	63	94	67	101	72	108	78	117	84	127	84
8.4	38	57	40	60	43	65	47	70	50	82	60	91	64	97	69	103	74	111	80	119	86	130	86
8.6	39	58	41	62	44	66	48	71	52	84	62	93	66	99	70	106	76	113	81	122	88	133	88
8.8	40	59	42	63	45	68	49	73	53	86	63	95	67	101	72	108	77	116	83	125	91	136	91
9.0	40	61	43	65	46	69	50	75	54	88	65	97	69	103	74	110	79	119	85	128	93	139	93
9.2	41	62	44	66	47	71	51	76	55	90	66	99	70	106	75	113	81	121	87	131	95	142	95
9.4	42	63	45	68	48	73	52	78	56	92	68	102	72	108	77	115	83	124	89	134	97	145	97
9.6	43	65	46	69	49	74	53	80	58	94	69	104	74	110	79	118	84	126	91	136	99	148	99
9.8	44	66	47	71	50	76	54	81	59	96	71	106	75	113	80	120	86	129	93	139	101	151	101
10.0	45	68	48	72	51	77	55	83	60	98	72	108	77	115	82	123	88	132	95	142	103	154	103
10.2	46	69	49	73	52	79	56	85	61	100	73	110	78	117	83	125	90	134	97	145	105	157	105
10.4	47	70	50	75	53	80	58	86	62	102	75	112	80	119	85	128	91	137	99	148	107	160	107
10.6	48	72	51	76	55	82	59	88	64	104	76	114	81	122	87	130	93	140	100	151	109	164	109
10.8	49	73	52	78	56	83	60	90	65	106	77	116	83	124	88	133	95	142	102	153	111	167	111
11.0	50	74	53	79	57	85	61	91	66	108	79	118	84	126	90	135	97	145	104	156	113	170	113
11.2	50	76	54	81	58	86	62	93	67	110	81	121	86	129	92	137	98	148	106	159	115	173	115
11.4	51	77	55	82	59	88	63	95	68	112	82	123	87	131	93	140	100	150	108	162	117	176	117
11.6	52	78	56	84	60	89	64	96	70	114	84	125	89	133	95	142	102	153	110	165	119	179	119
11.8	53	80	57	85	61	91	65	98	71	116	85	127	90	136	97	145	104	155	112	168	121	182	121
12.0	54	81	58	86	62	93	66	100	72	118	86	130	92	138	98	147	105	158	114	171	123	185	123

5-3/4 LENGTH OF SUPERELEVATION RUNOFF AND TANGENT RUNOUT FOR SPIRAL- CURVE TRANSITION

The change in highway cross slope begins by introducing a (tangent runout) section just in advance of the spiral curve. The superelevation runoff should be accomplished over the length of the spiral, and the whole of the circular curve has full superelevation.

The tangent runout lengths for spiral –curve transition design are shown in table (5-3/3), based on 2.0 percent normal cross slope, together with the equation used for computation. The length of the spiral should be set equal to the length of superelevation runoff. [1, p.3-73]

Table 5-3/3: Tangent Runout Length for Spiral Curve Transition Design [1, p.3-75]

Design Speed (km/hr.)	Tangent Runout Length (m)				
	Superelevation Rate				
	2	4	6	8	10
20	11	—	—	—	—
30	17	8	—	—	—
40	22	11	7	—	—
50	28	14	9	—	—
60	33	17	11	8	—
70	39	19	13	10	—
80	44	22	15	11	—
90	50	25	17	13	10
100	56	28	19	14	11
110	61	31	20	15	12
120	67	33	22	17	13
130	72	36	24	18	14

$$L_t = \frac{e_{NC}}{e_d} L_S \quad (5-3/3)$$

where:

L_t = length of tangent runout, m

L_S = length of spiral, m

e_d = design superelevation rate, percent

e_{NC} = normal cross slope rate, percent

5-4 WIDENING ON HORIZONTAL CURVES

Widening of the traveled way on a horizontal curve is the difference between the width needed on the curve (W_c) and the width used on a tangent (W_n).

Widening is needed on certain curves for one of the following reasons:

- Rear wheels of the design vehicle track inside front wheels (offtracking) while rounding curves.
- Difficulty in steering vehicles in the center of lane on curves.

5-4/1 DESIGN VALUES OF TRAVELED WAY WIDENING

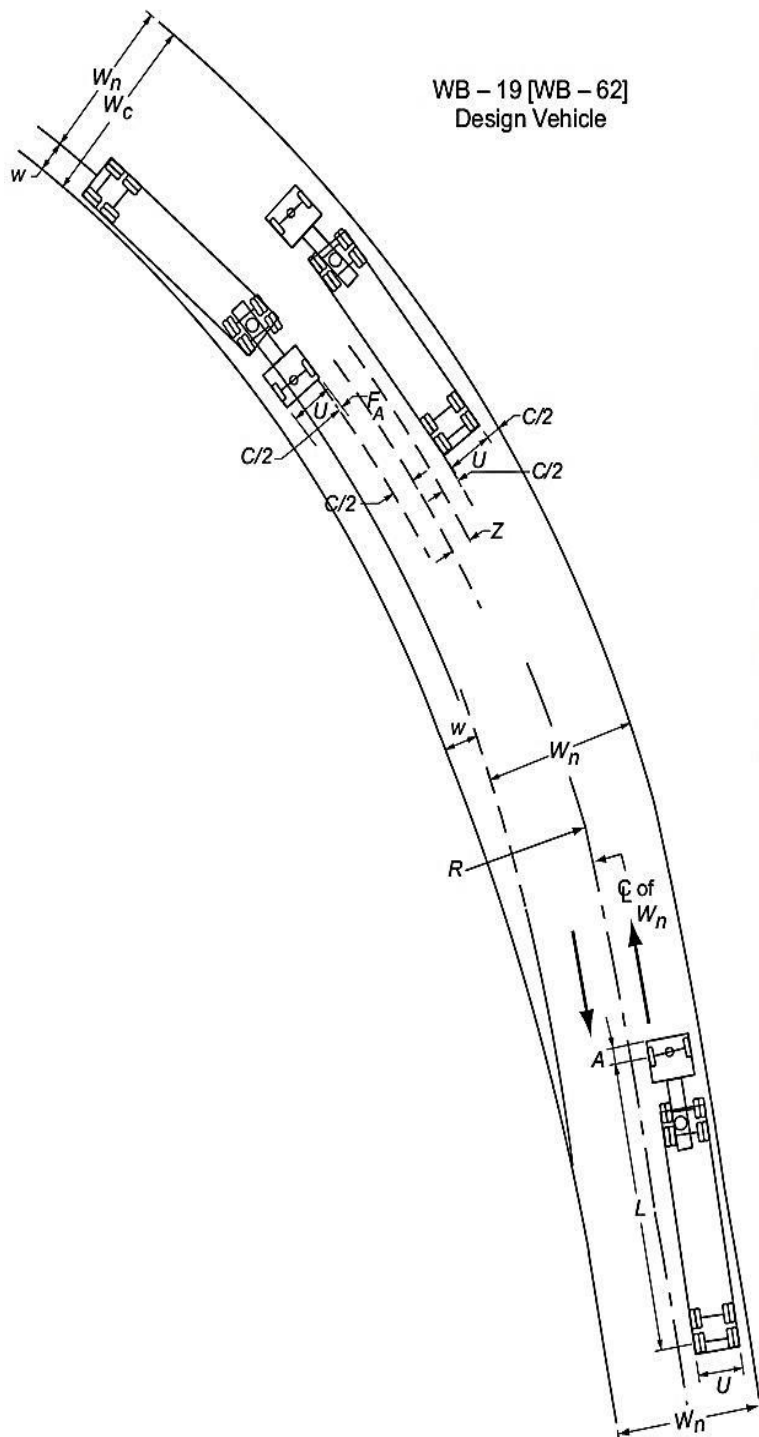
Widening components of the traveled way width on curves (W_c), as calculated by the related equation, are shown in figure (5-4/1). The track width values (U) of design vehicles of different circular curves are shown in figure (5-4/2).

The front overhang (FA) of the vehicle occupying the inner lanes is the radial distance between the outer edge of tire path of the outer front wheel, and the path of the outer front edge of the vehicle body. Figure (5-4/3) illustrates front overhang values (FA) of different design vehicles.

The lateral clearance allowance (C) provides clearance between vehicles passing or meeting, which is assumed to be (0.60, 0.75, and 0.90 m) for tangent lane widths of (3.00, 3.30, and 3.60m) respectively.

The extra width allowance (Z) for the difficulty of maneuvering on curves, is shown in figure (5-4/4) for different speeds and curves.

The widening values on curves, ($W_c - W_n$) for WB-19 design vehicle, on two-lane highways are presented in table (5-4/1).



$$W_c = N(U + C) + (N - 1)F_A + Z$$

where:

W_c = width of traveled way on curve, m

N = number of lanes

U = track width of design vehicle (out-to-out tires) on curves, m

C = lateral clearance, m

F_A = width of front overhang of inner-lane vehicle, m

Z = extra width allowance, m

Figure 5-4/1: Widening Components on Open Highway Curves (Two-Lane Highways, One-Way or Two-Way) [1, p.3-91]

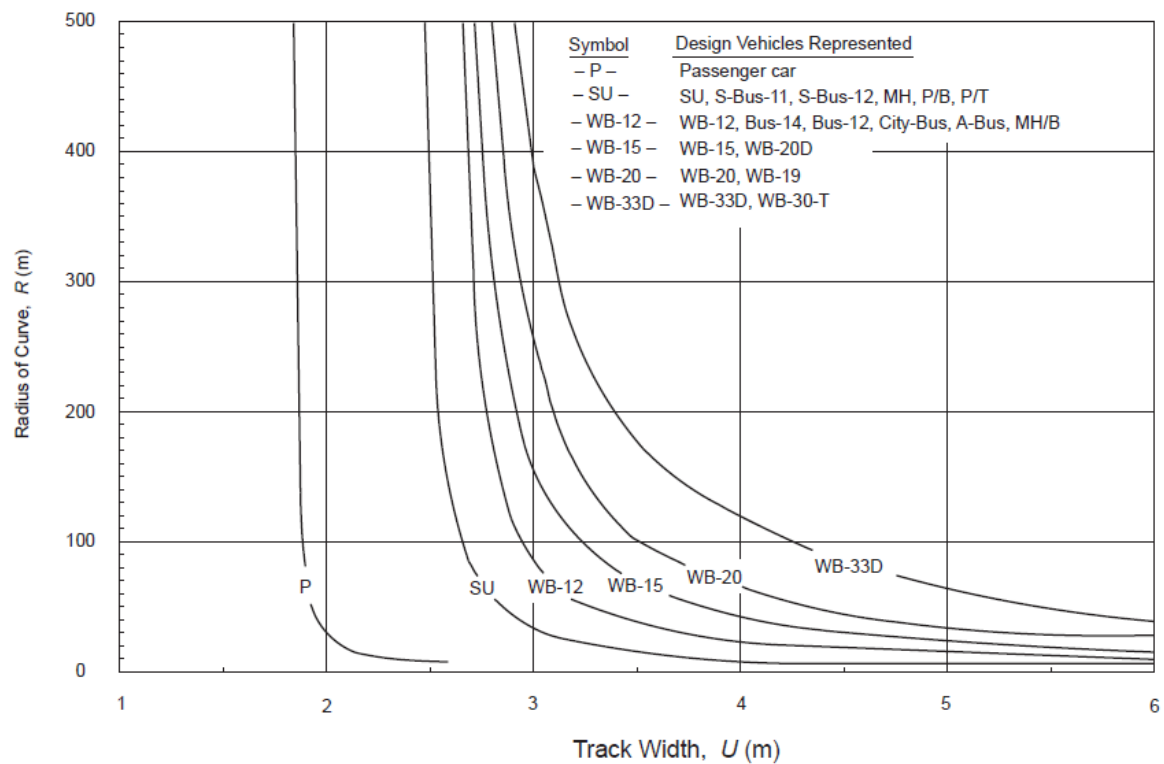


Figure 5-4/2: Track Width for Widening of Traveled Way on Curves [1, p.3-87]

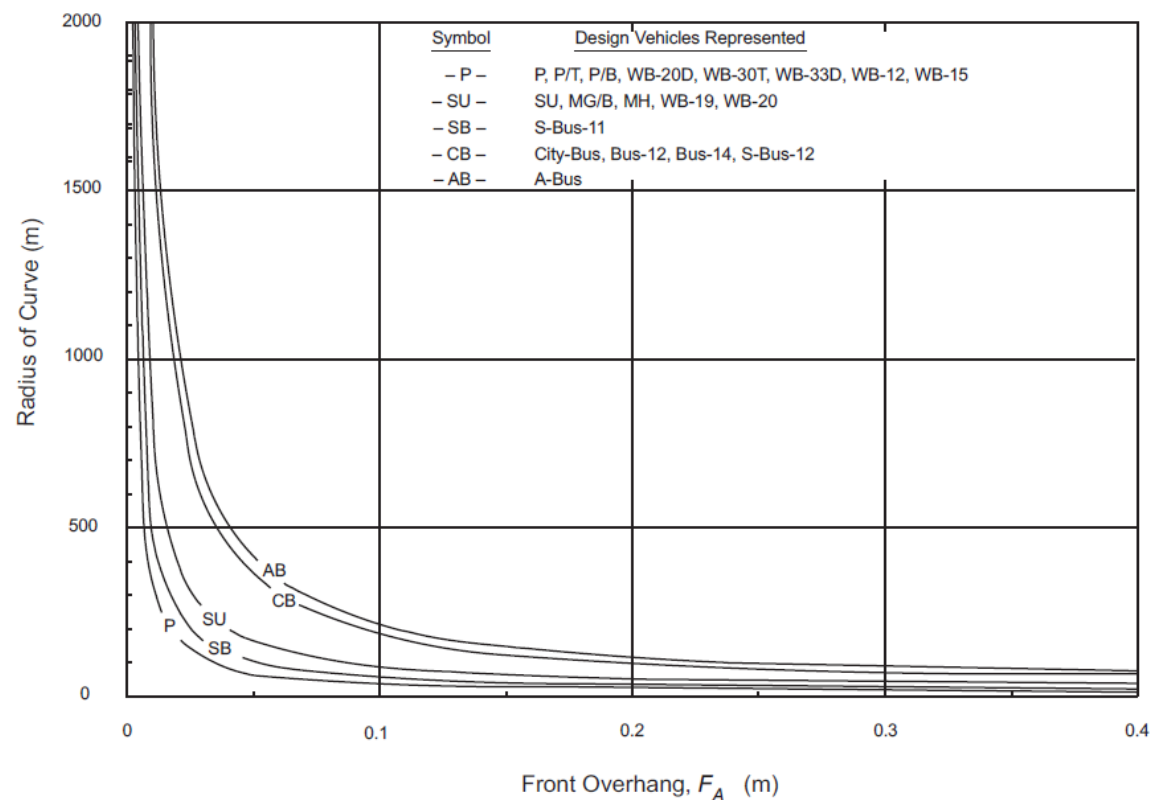


Figure 5-4/3: Front Overhang for Widening of Traveled Way on Curves [1, p.3-88]

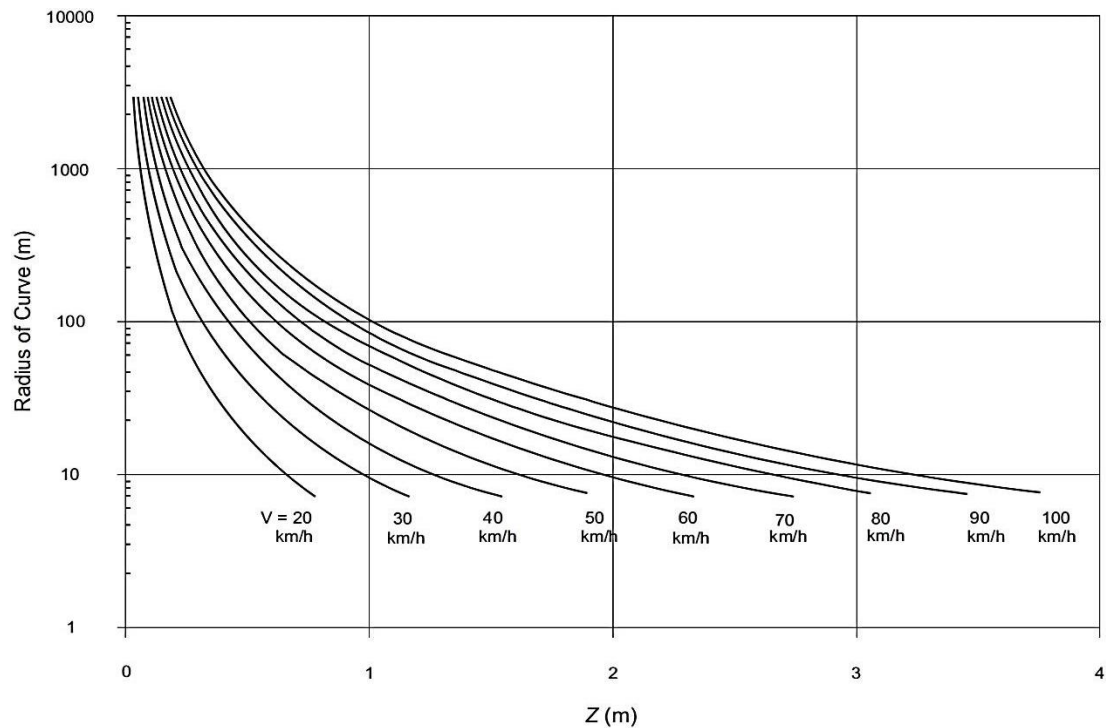


Figure 5-4/4: Extra Width Allowance for Difficulty of Driving on Traveled Way on Curves [1, p.3-89]

Table 5-4/1: Calculated and Design Values for Traveled Way Widening on Open Highway Curves (Two-Lane Highways, One-Way or Two-Way) [1, p.3-93]

Radius of Curve (m)	Roadway width = 7.2 m						Roadway width = 6.6 m						Roadway width = 6.0 m					
	Design Speed (km/h)						Design Speed (km/h)						Design Speed (km/h)					
	50	60	70	80	90	100	50	60	70	80	90	100	50	60	70	80	90	100
3000	0.0	0.0	0.0	0.0	0.0	0.0	0.2	0.3	0.3	0.3	0.3	0.3	0.5	0.6	0.6	0.6	0.6	0.6
2500	0.0	0.0	0.0	0.0	0.0	0.1	0.3	0.3	0.3	0.3	0.3	0.4	0.6	0.6	0.6	0.6	0.6	0.7
2000	0.0	0.0	0.0	0.1	0.1	0.1	0.3	0.3	0.3	0.4	0.4	0.4	0.6	0.6	0.6	0.7	0.7	0.7
1500	0.0	0.1	0.1	0.1	0.1	0.2	0.3	0.4	0.4	0.4	0.4	0.5	0.6	0.7	0.7	0.7	0.7	0.8
1000	0.1	0.2	0.2	0.2	0.3	0.3	0.4	0.5	0.5	0.5	0.6	0.6	0.7	0.8	0.8	0.8	0.9	0.9
900	0.2	0.2	0.2	0.3	0.3	0.3	0.5	0.5	0.5	0.6	0.6	0.6	0.8	0.8	0.8	0.9	0.9	0.9
800	0.2	0.2	0.3	0.3	0.3	0.4	0.5	0.5	0.6	0.6	0.6	0.7	0.8	0.8	0.9	0.9	0.9	1.0
700	0.3	0.3	0.3	0.4	0.4	0.4	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	0.3	0.4	0.4	0.4	0.5	0.5	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	0.4	0.4	0.5	0.5	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	0.5	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.

Values less than 0.6 m may be disregarded.

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

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

5-4/2 APPLICATION OF WIDENING ON CURVES

Widening should transition gradually on the approaches to the curve over a length, (30-60m), sufficient to make the whole traveled way usable and preferably over the superelevation runoff length.

On un-spiraled curves, widening should be applied on the inside edge of the traveled way only. On curves designed with spirals, widening may be applied either on the inside edge, or divided equally on both sides of the centerline. The final marked centerline should be placed midway between the edges of the widened traveled way on curves.

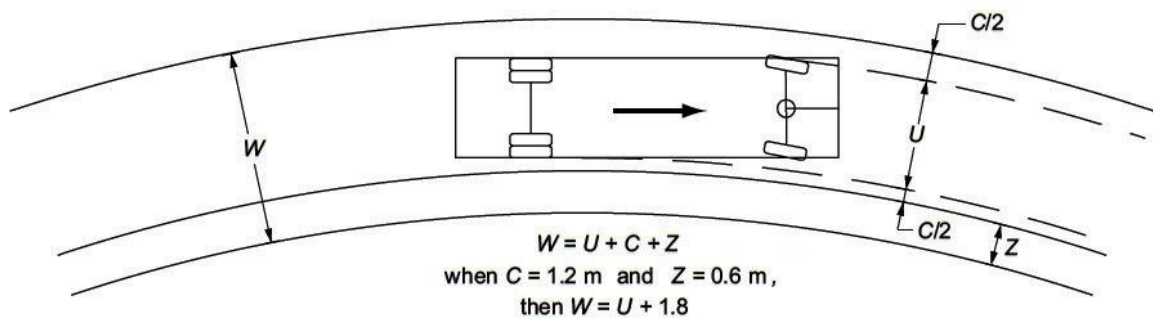
5-5 DESIGN WIDTHS FOR TURNING ROADWAYS OR RAMPS AT INTERSECTIONS

The radius of curvature and track width of the design vehicle determine the width of a turning roadway or ramp at intersections. Figure (5-5/1) shows the equations for determining the required turning roadway width, for each of the three cases of operation, on intersection curves.

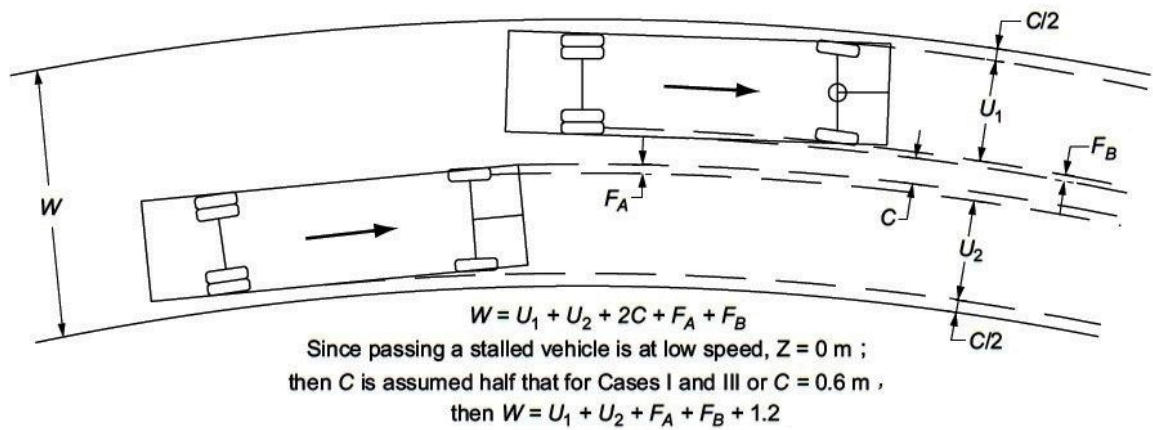
The minimum design widths of turning roadways for the three operation cases, different radii on inner pavement edge, and traffic condition, with modifications for edge conditions are presented in table (5-5/1). The three traffic conditions are assumed to have only an occasional large trucks for (A), a moderate volume of trucks (5-10%) for (B), and more and larger trucks for (C).

The types of design vehicles with full clearance, together with larger vehicles that can be operated on turning roadways, but with partial clearance are shown in table (5-5/1).

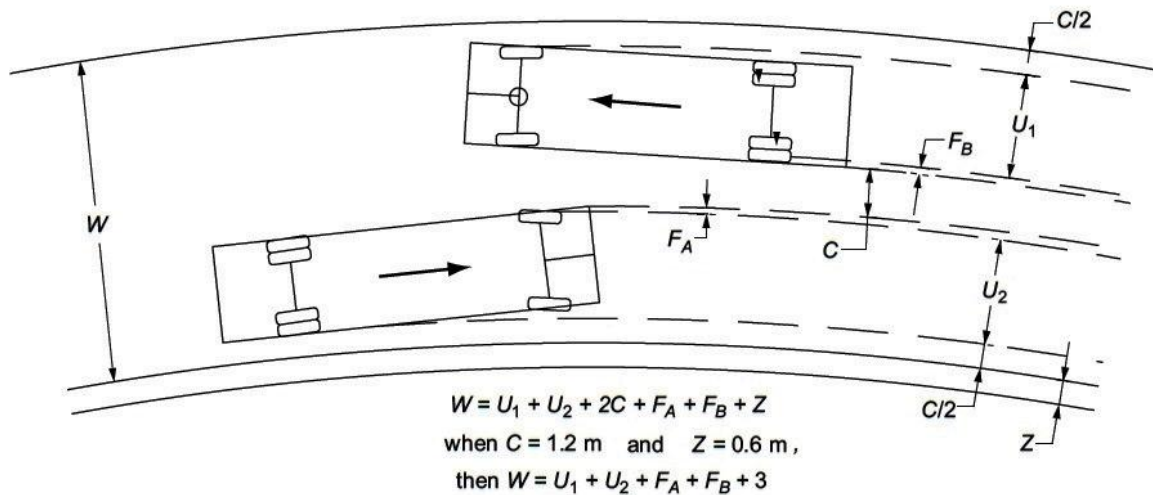
The shoulder width or lateral clearance outside the traveled- way edge ranges from 1.20 to 3.6 meters. Where roadside barriers are provided, the width should be about 0.60 m greater, [1, p.3-106].



CASE I
One-Lane One-Way Operation — No Passing



CASE II
One-Lane One-Way Operation Provision for Passing Stalled Vehicle



CASE III
Two-Lane Operation — One or Two Way

U = Track Width of Vehicle (Out-to-Out Tires), m
 F_A = Width of Front Overhang, m
 F_B = Width of Rear Overhang, m

C = Total Lateral Clearance per Vehicle, m
 Z = Extra Width Allowance Due to Difficulty of Driving on Curves, m

Figure 5-5/1: Derivation of Turning Roadway Widths on Curves at Intersections [1, p.3-98]

Table 5-5/1: Design Width of Pavements for Turning Roadways [1, p.3-103]

Radius on Inner Edge of Pave- ment, <i>R</i> (m)	Pavement Width (m)								
	Case I One-Lane, One-Way Operation—no provision for passing stalled vehicle			Case II One-Lane, One-Way Op- eration—with provision for passing stalled vehicle			Case III Two-Lane Operation—ei- ther one-way or two-way operation		
	Design Traffic Conditions								
	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 for Edge Conditions									
No stabilized shoulder		None			None			None	
Sloping curb		None			None			None	
Vertical curb:									
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	

Note:

- A = predominantly P vehicles, but some consideration for SU 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

Full clearance for the design vehicles			
Case	Design Traffic Condition		
	A	B	C
I	P	SU-9	WB-12
II	P-P	P-SU-9	SU-9-SU-9
III	P-SU-9	SU-9-SU-9	WB-12-WB-12

Partial clearance for the larger vehicles			
Case	Design Traffic Condition		
	A	B	C
I	WB-12	WB-12	WB-19
II	P-SU-9	P-WB-12	SU-9-WB-12
III	SU-9-WB-12	WB-12-WB-12	WB-19-WB-19

5-6 SIGHT DISTANCE ON HORIZONTAL CURVES

The presence of sight obstructions on the inside of curves may need adjustments in horizontal alignment to satisfy sight distance criteria.

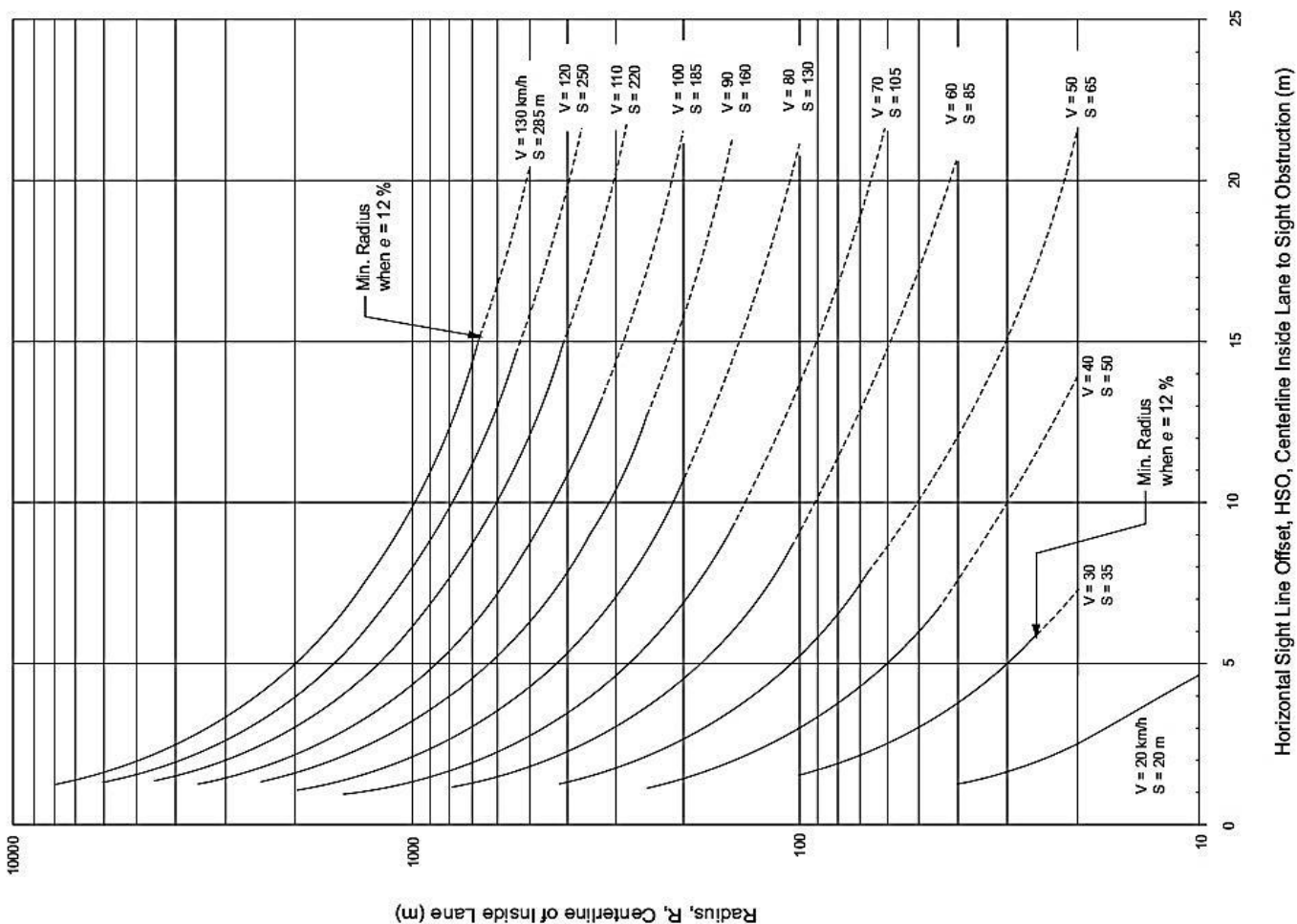
The horizontal sight line offset (HSO) for circular curves longer than the sight distance, can be determined by the equation presented in figure (5-6/1).

The design controls for stopping sight distance (S), on horizontal circular curves (along centerline of inside Lane) with radius (R) to determine (HSO) for the pertinent design speed (V), are indicated in figure (5-6/1).

For a circular curve length (L), smaller than the sight distance (S), the following relationship may be used:

$$HSO = \frac{L(2S - L)}{8R} \quad (5 - 6/1)$$

The minimum passing sight distance for a 2- lane highway is about twice the minimum stopping sight distance at the same design speed, thus clear sight areas on the inside of curves should have widths in excess of those for stopping sight distances. [1, p.3-110]



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

where:

HSO = Horizontal sight line offset, m

S = Stopping sight distance, m

R = Radius of curve, m

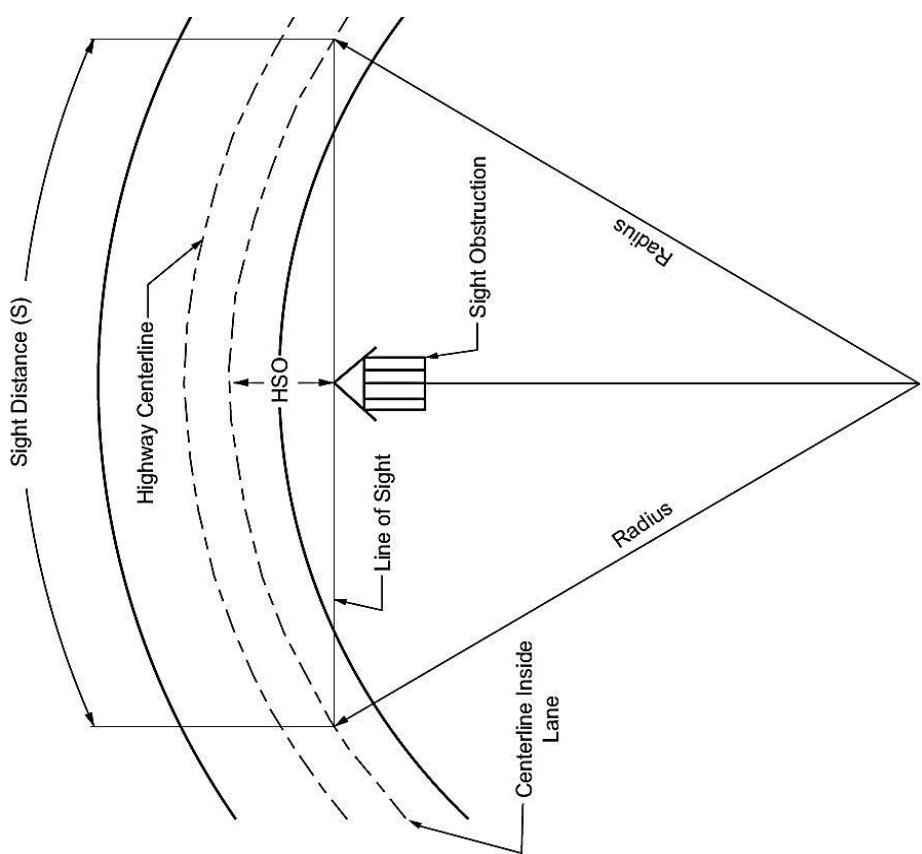


Figure 5-6/1: Design Controls for Stopping Sight Distance on Horizontal Curves
[1, p.3-107, 109]

5-7 GENERAL CONTROLS FOR HORIZONTAL ALIGNMENT

For efficient and smooth – flowing highways, the general controls that follow should be used where practical, [1, p.3-111]:

- Alignment should be consistent with the topography (conforms generally to natural contours). Long tangents are needed on 2- lane highways for safe passing.
- Horizontal alignment should be coordinated with profile design.
- Flat curves need to be used, saving the minimum radius for the most critical conditions.
- Sudden changes from areas of tangents or flat curvature, to areas of sharp curvature should be avoided.
- The minimum length of horizontal curves (L) in meters should be (3 to 6) times the design speed (V) in km/hr. on main highways.

For small deflection angles, the curves should be at least 150m long for a central angle of 5 degrees, and increased 30 m for each 1 degree decrease.

- Compound circular curves should be used with caution. The radius of the flatter curve should not exceed 1.5 times the radius of the sharper curve.
- Abrupt reversals in alignment should be avoided. The distance between reverse curves should be the sum of the superelevation runoff lengths, and the tangent runout lengths.
- The broken –back arrangement (a short tangent between two curves in the same direction), should be avoided, except for very unusual topographical or right- of – way conditions.
- Changing median width, on tangent alignments should be avoided, due to distorted appearance.

5-8 REFERENCES

- [1] AASHTO, "*A Policy on Geometric Design of Highways and Streets*", American Association of State Highway and Transportation Officials, USA, 2011.
- [2] Wright, P.H. and Dixon, K. K., "*Highway Engineering*", John Wiley & Sons, USA, 2004.
- [3] T.F. Hickerson, "*Route Surveys and Design*", McGraw- Hill Book Co., USA, 1959.

CHAPTER 6

VERTICAL ALIGNMENT

6-1 CONTROL GRADES FOR DESIGN

Highways and streets should be designed to encourage uniform operation in both horizontal and vertical alignments.

The effect of grades on truck speeds is much more pronounced than on speeds of passenger cars.

The effect of rate and length of grade on the speed of a typical heavy truck is shown in figure (6-1/1) for deceleration on upgrades, and in figure (6-1/2) for acceleration on upgrades and downgrades.

6-1/1 MAXIMUM GRADES

Maximum grades of 5% are usually considered for a design speed of 110 km/hr. maximum grades of 7-8% are used in current practice for a 50 km/hr. design speed. [1, p.3-119].

Maximum grade controls for each functional class of highway are presented in chapter 8. For short grades (<150m) in length, and for one- way down grades, the maximum grade may be increased by 1% (steeper). In design practice, grades should be frequently less than the maximum design grade.

6-1/2 MINIMUM GRADES

A minimum longitudinal grade of 0.5% should be provided on curbed highways or streets to facilitate surface drainage. Lower grades (0.3%) may be used for the accurately sloped paved surfaces.

Particular attention should be given to design of storm water inlets spacing and roadside channels.

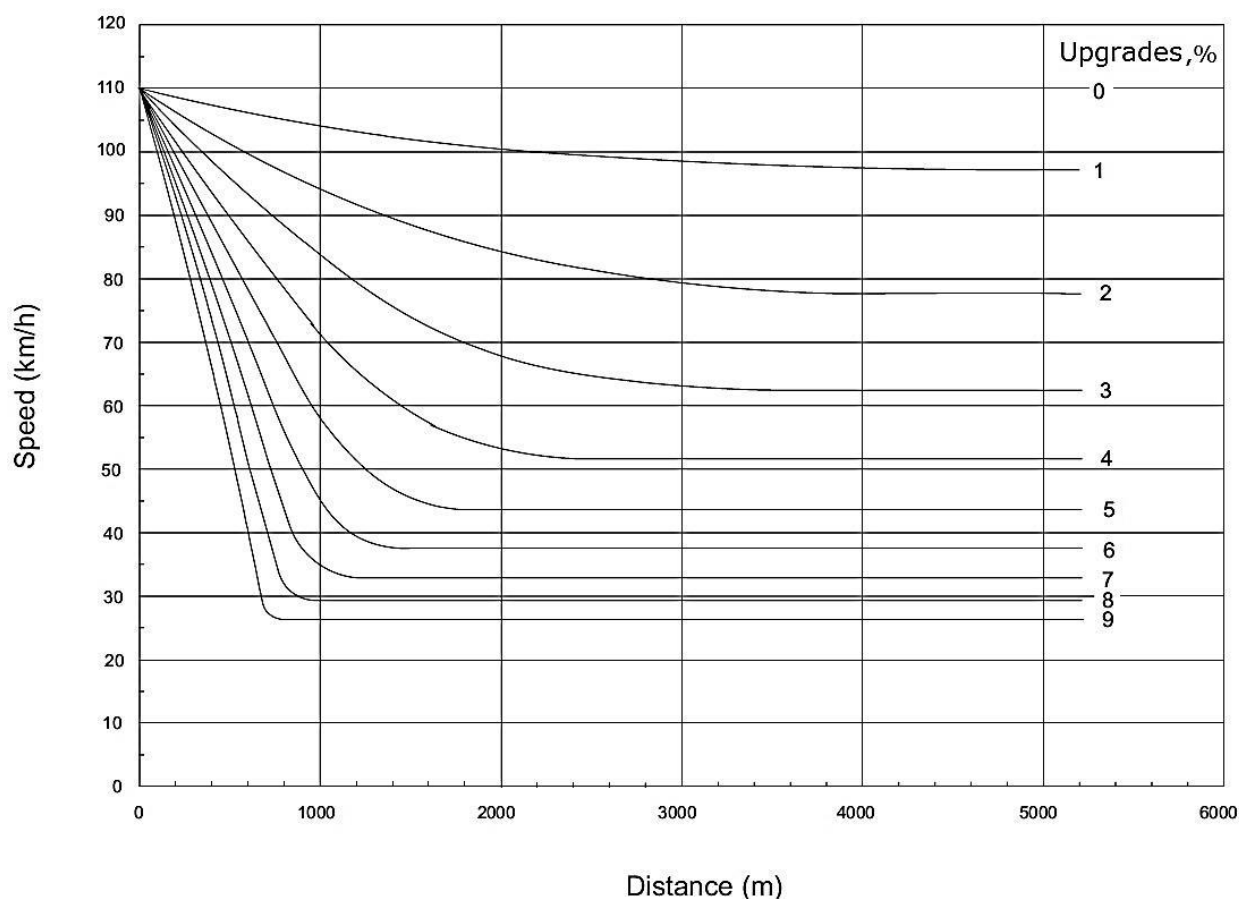


Figure 6-1/1: Speed-Distance Curve for a Typical Heavy Truck of 120 kg/kw for Deceleration on Upgrades. [1, p.3-115]

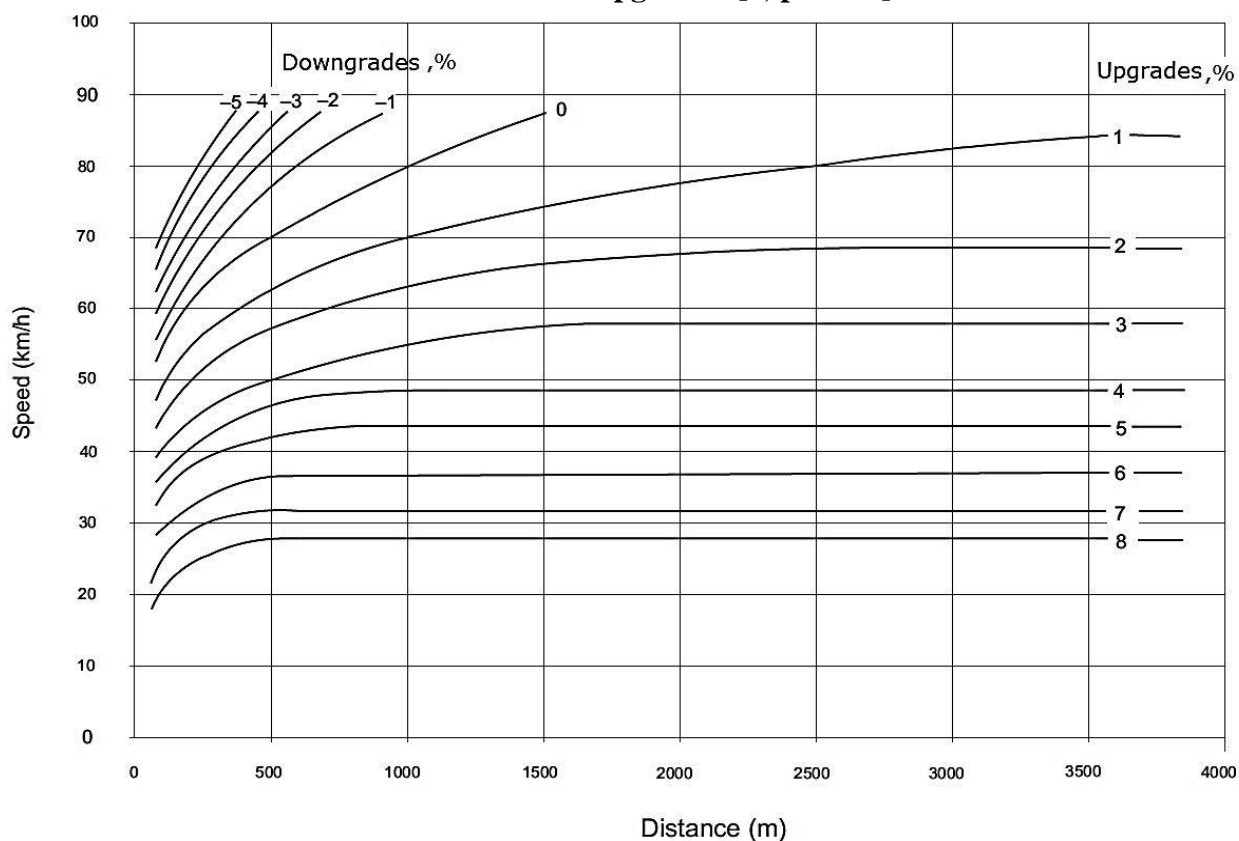


Figure 6-1/2: Speed-Distance Curve for Acceleration of a Typical Heavy Truck of 120 kg/kw on Upgrades and Downgrades. [1, p.3-116]

6-1/3 CRITICAL LENGTH OF UPGRADE

Critical length of upgrade: is the maximum length of an upgrade, on which a loaded truck can operate without an unreasonable reduction in speed (15 km/hr.). [1, p.3-119]

On upgrade length greater than critical, design adjustments such as: changes in location to reduce grades, or addition of extra climbing lane should be considered. The critical length of upgrades, that will cause the speed of a typical heavy truck entering the grade at 110 km/hr., to be reduced by various values, is shown in figure (6-1/3), including the design curve for the 15 km/hr. reduction.

For entering speeds less than 110km/hr., the same speed reduction will occur over shorter lengths of upgrade. The crash involvement rates for truck speed reductions, assuming a 30% reduction for other vehicles on the same grade are shown in figure (6-1/4).

6-2 ADDED LANES AND TURNOUTS ON TWO- LANE HIGHWAYS

An added lane for vehicles moving slowly uphill is sometimes needed, so that other vehicles using the normal lane to the right of the centerline are not delayed.

6-2/1 CLIMBING LANES

Climbing lane is commonly used on two- lane highways for freedom and safety of operations.

The following criteria should be satisfied to justify a climbing lane:

- Upgrade flow rate in excess of 200 vph
- Upgrade truck flow rate in excess of 20 vph.
- One of the following conditions exists:
 - Level of service E or F on the grade.
 - A 15 km/hr. or greater speed reduction for heavy truck.

Two typical designs of climbing lanes (without and with overlapping) are shown in figure (6-2/1).

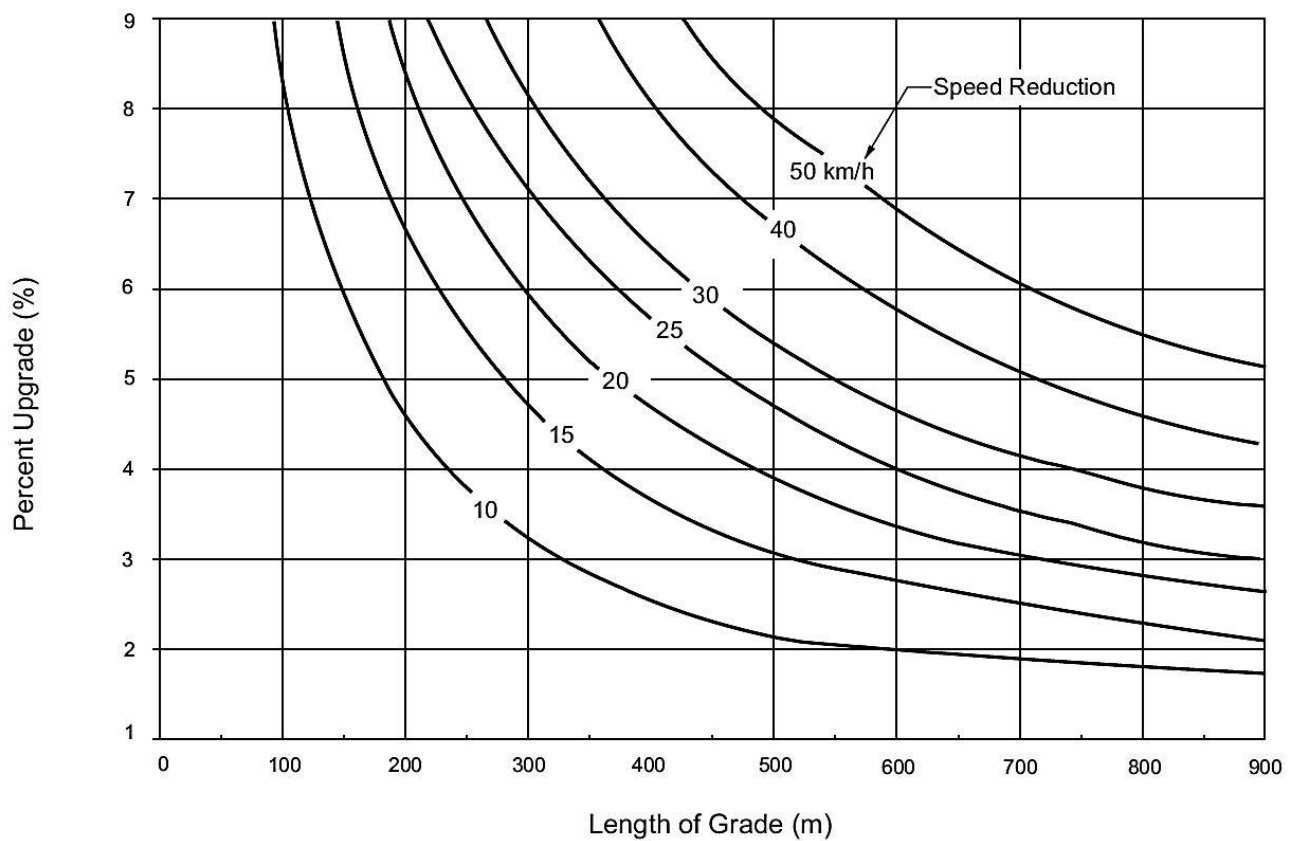


Figure 6-1/3: Critical Lengths of Grade for Design, Assumed Typical Heavy Truck of 120 kg/kw, Entering Speed = 110 km/hr.; [1, p.3-123]

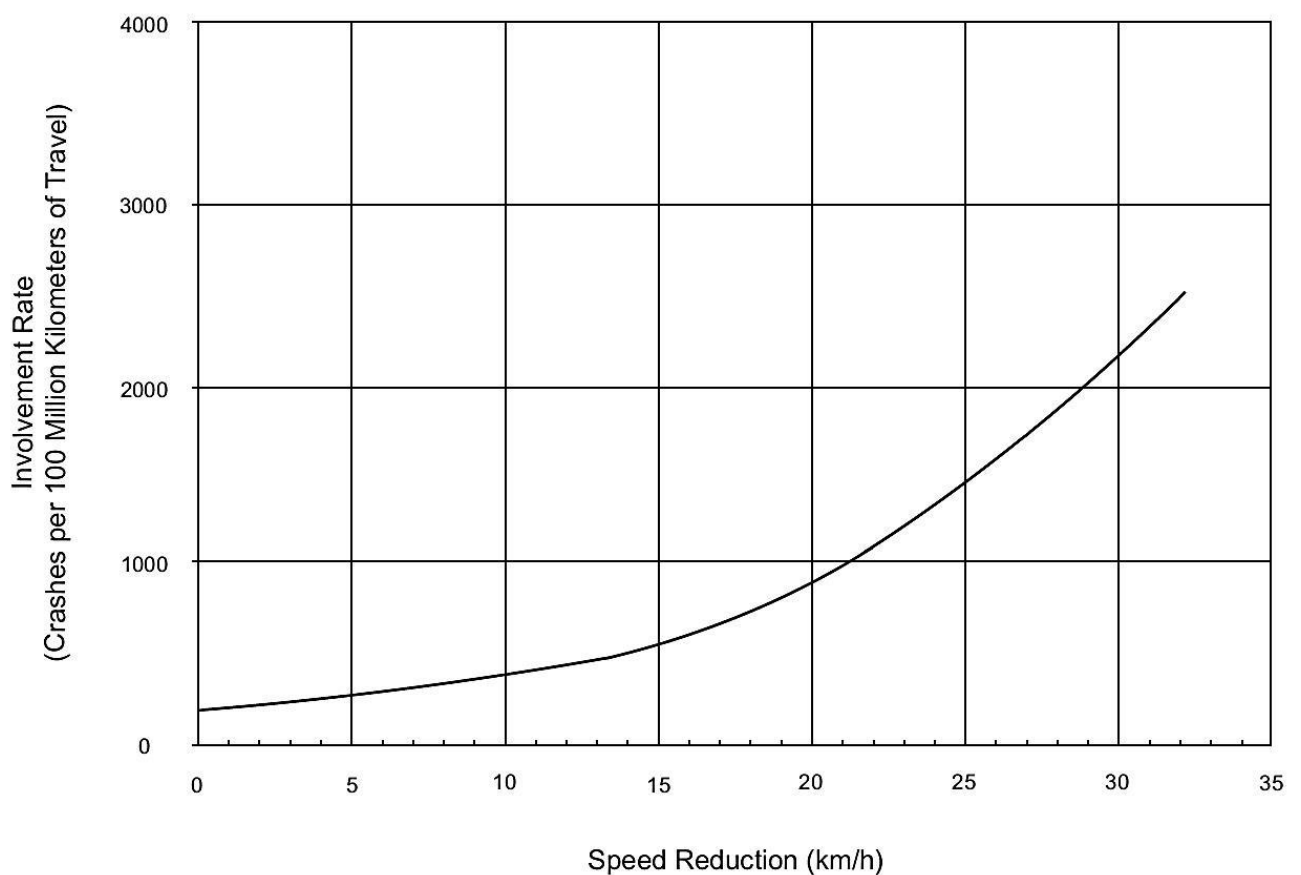


Figure 6-1/4: Crash Involvement Rate of Trucks for Which Running Speeds Are Reduced Below Average Running Speed of All Traffic; [1, p.3-120]

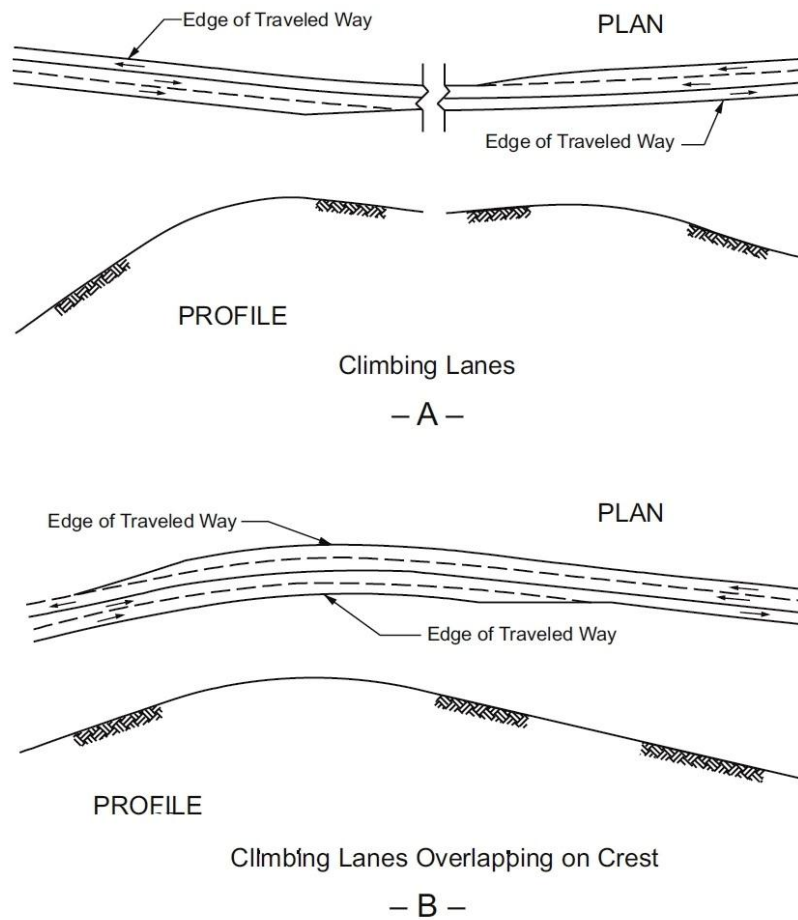


Figure 6-2/1: Climbing Lanes on Two-Lanes Highways [1, p.3-126]

6-2/2 PASSING LANES

Where a sufficient number and length of passing sections cannot be obtained in the design of horizontal and vertical alignment alone, an occasional added lane in one or both directions of travel may be introduced to provide more passing opportunities for 2- lane highway, especially in rolling terrain figure (6-2/2) [1, p. 3-133].

The optimal design lengths for passing lanes with traffic operational efficiency are shown in table (6-2/1), together with equations for determining lane- drop taper lengths at each end of passing lane length.

For roadways with traffic volumes higher than what can be served by 2- lane highways (>1200 vph / direction), but not high enough to justify a 4- lane highway, the (2+1) configuration may be a suitable treatment.

A (2+1) road will generally operate at least two levels of service higher than conventional 2- lane highway serving the same traffic volume [1, p.3-136].

The (2+1) concept provides a continuous 3- lane cross section, striped in a manner to provide for passing lane in alternating directions throughout the section, as shown in figure (6-2/3).

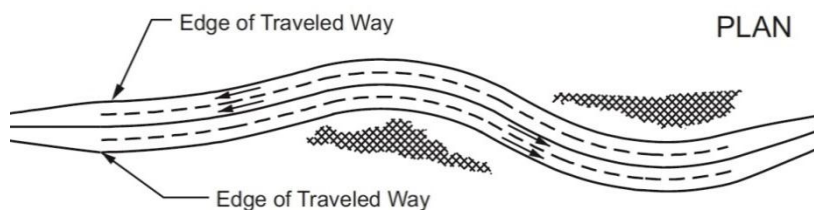
A (2+1) road should only be used in level or rolling terrain, and climbing lanes are suggested on upgrades of mountainous terrain.

6-2/3 TURNOUTS

Turnout: is a widened, unobstructed shoulder area that allows slow- moving vehicles to pull out of the through lane, to give passing opportunities to following vehicles.

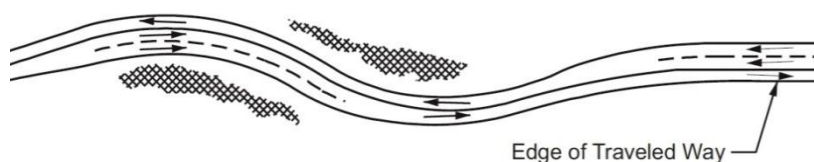
Turnouts are most frequently used in difficult terrain with steep grades, where construction of an additional lane may not be cost – effective [1, p.3-138].

The recommended lengths of turnouts are shown in table (6-2/2). The entry and exit taper lengths range from 15 to 30m. The minimum width of turnout is 3.60m. Up to 5.00 meter width may be used for the turnouts.



Four-Lane Passing Section on Two-Lane Highway

– A –



Three-Lane Passing Section on Two-Lane Highway

– B –

Figure 6-2/2: Passing Lanes Section on Two-Lane Roads [1, p.3-133]

Table 6-2/1: Optimal Passing Lane Lengths [1, p.3-135]

One-Way Flow Rate (veh/hr.)	Passing Lane Length (km)
100-200	0.8
201-400	0.8-1.2
401-700	1.2-1.6
701-1200	1.6-3.2

Lane-Drop Taper Length	
$L = 0.62 WS$	($S \geq 70$)
$L = \frac{WS^2}{155}$	($S < 70$)
where:	
L	= Length of taper, m
W	= Width, m
S	= Speed, km/h

(6 - 2/1)

(6 - 2/2)

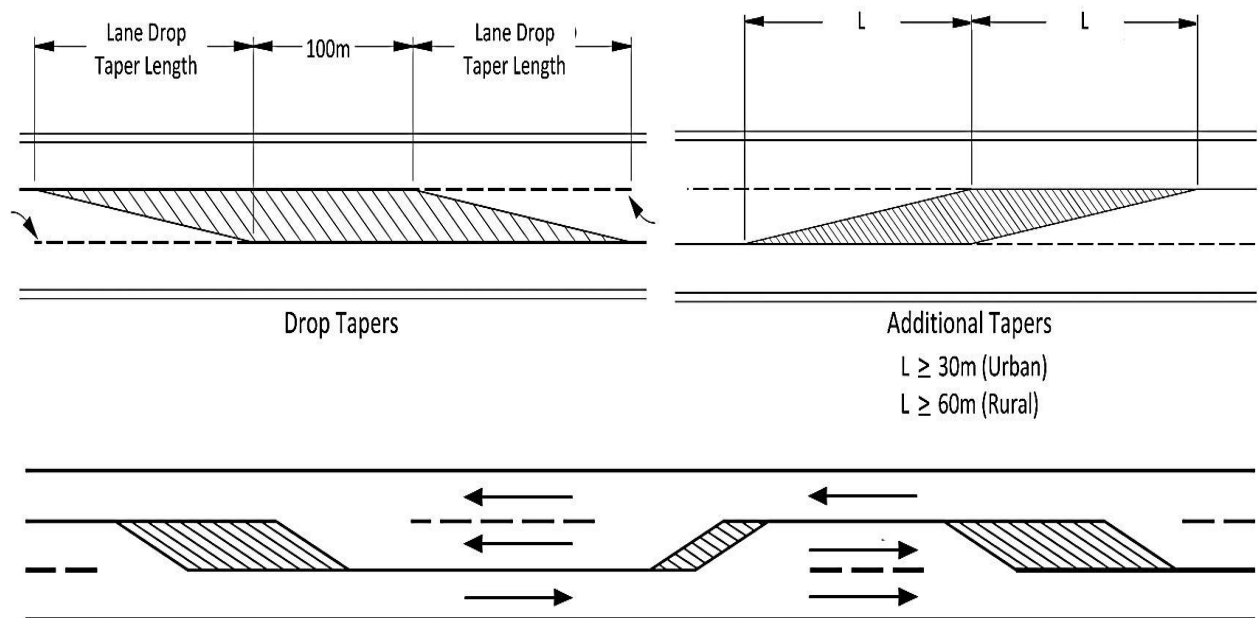


Figure 6-2/3: Schematic for 2 + 1 Roadway [1, p.3-136]

Table 6-2/2: Recommended Lengths of Turnouts Including Taper [1, p.3-139]

Approach Speed (km/hr.)	Minimum Length (m) ^a
30	60
40	60
50	65
60	85
70	105
80	135
90	170
100	185

^a Maximum length should be 185 m to avoid use of the turnout as a passing lane.

6-3 EMERGENCY ESCAPE RAMPS

Where long descending grades exist, it is desirable to provide a location for out- of- control vehicles,(losing braking ability), particularly trucks, to slow and stop away from the main traffic stream [1, p.3-140].

The four basic types of emergency escape ramps are illustrated in figure (6-3/1). Each type is applicable to a particular topographic situation. The most commonly used escape ramp, is the (Ascending- Grade) arrested bed, where the gradient – resistance act downgrade, in the opposite direction of vehicle movement.

The use of single size, rounded aggregate, with a top size of 40 mm, which is free from fine- size material, in the arrester bed, with a minimum depth of 1.00 meter, shall increase rolling resistance, to slow and stop the out – of- control vehicle.

Sand piles, composed of loose, dry sand, dumped at the ramp site (120m length), have severe deceleration characteristics, making its use less desirable, except at locations where inadequate space exists.

The alignment of the escape ramp should be tangent, or on very flat curvature, with a minimum width of 8.00 meters, and a service road need to be located adjacent to the arrested bed, for use of two trucks and maintenance vehicles. Anchors are needed to secure a ton truck when removing a vehicle from the arrested bed. A typical emergency escape ramp is illustrated in figure (6-3/2).

Rolling resistance: is the resistance to motion at the area of contact between a vehicles tire and the roadway surface, which may be expressed as (equivalent gradient) as shown in table (6-3/1).

The length of arrester bed needed to bring the vehicle to a stop, with consideration of the rolling resistance and gradient resistance, may be determined from the equation presented with table (6-3/1).

When the arrester bed is constructed using more than one grade along its length, the final speed at each change in grade may be determined, and the calculation is repeated for other grades, until sufficient length is provided to reduce the speed of vehicle to zero.

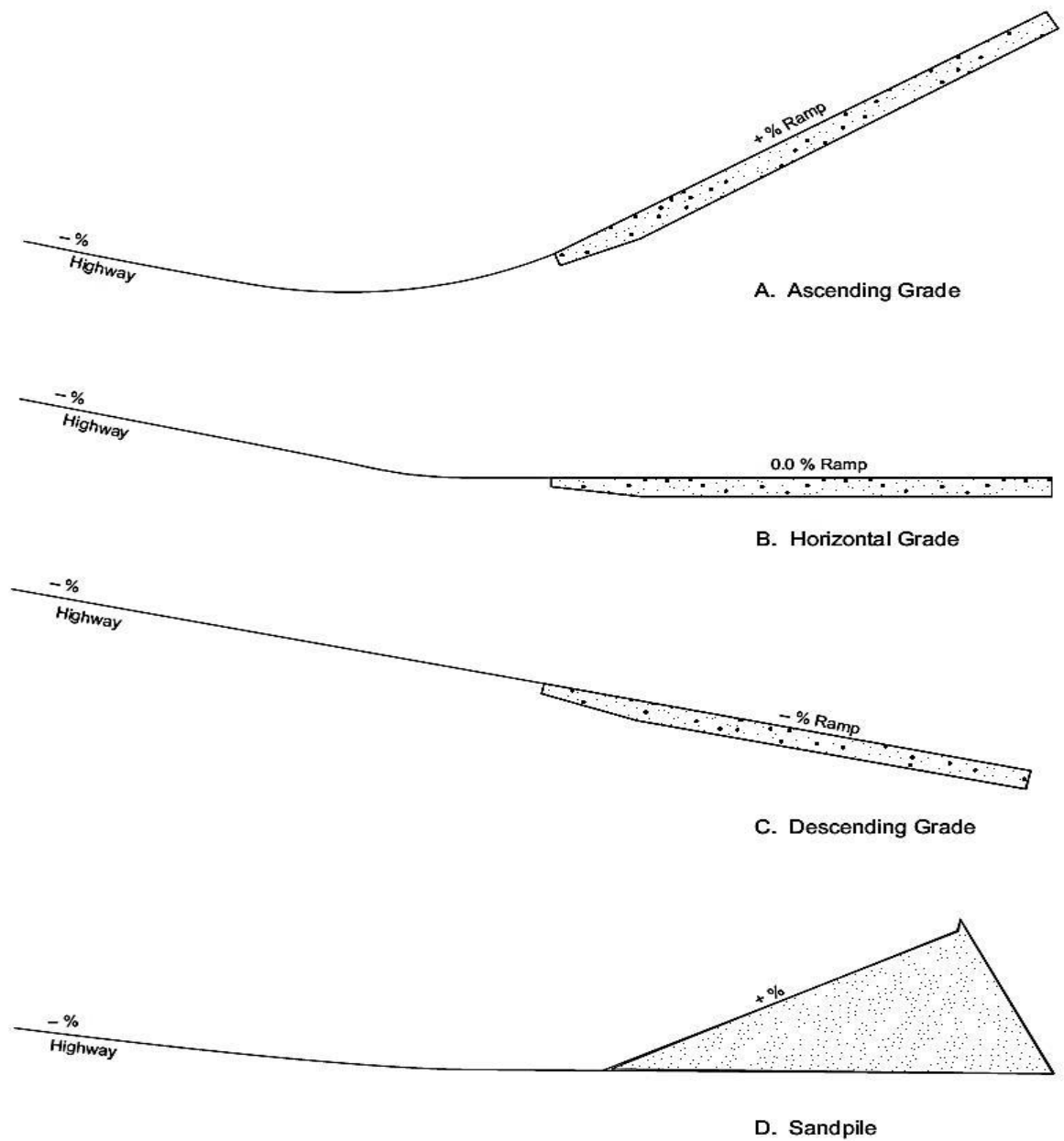


Figure 6-3/1: Basic Types of Emergency Escape Ramps [1, p.3-144]

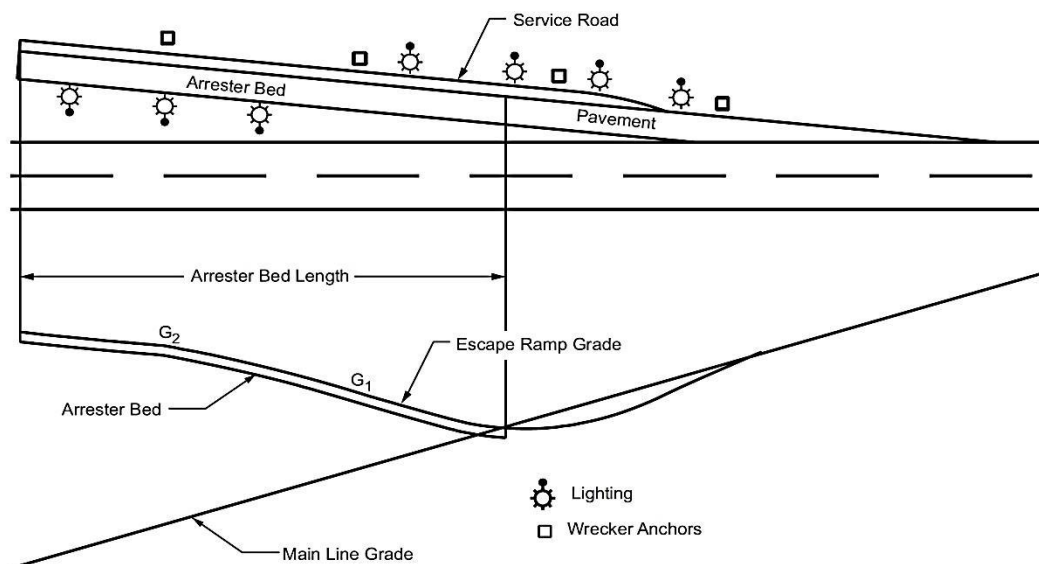


Figure 6-3/2: Typical Emergency Escape Ramp [1, p.3-148]

Table 6-3/1: Rolling Resistance of Roadway Surfacing Materials (R) [1, p.3-142]

Surfacing Material	Rolling Resistance (kg/1 000 kg GVM)	Equivalent Grade (%) ^a
Portland cement concrete	10	1.0
Asphalt concrete	12	1.2
Gravel, compacted	15	1.5
Earth, sandy, loose	37	3.7
Crushed aggregate, loose	50	5.0
Gravel, loose	100	10.0
Sand	150	15.0
Pea gravel	250	

^a Rolling resistance expressed as equivalent gradient.
GVM: Gross Vehicle Mass

Length of Arrester Bed
$L = \frac{V^2}{254(R \pm G)} \quad (6-3/2)$

where:

L = length of arrester bed,

V = entering velocity, km/hr.

R = rolling resistance, expressed as equivalent percent gradient divided by 100 (see table(6-3/1)).

G = percent grade divided by 100

Speed at End of Grade
$V_f^2 = V_i^2 - 254L(R \pm G) \quad (6-3/2)$

where:

V_f = speed at end of grade, km/hr.

V_i = entering speed at beginning of grade, km/hr.

L = length of grade, m

R = rolling resistance, expressed as equivalent percent gradient divided by 100 (see table(6-3/1)).

G = percent grade divided by 100

6-4 VERTICAL PARABOLIC CURVES (SYMMETRICAL AND UNSYMMETRICAL)

In alignment design of highways, crest or sag vertical parabolic curves are used to effect gradual changes between tangent grades. The rate of changes of grade at successive points on the curve is a constant amount for equal increments of horizontal distance.

The types of vertical curves are illustrated in figure (6-4/1). The properties of typical symmetrical and unsymmetrical vertical parabolic curves are shown in figures (6-4/2) and (6-4/3) respectively. Symmetrical parabolic curves are frequently used in vertical alignments of highways.

6-5 MINIMUM LENGTH OF CREST VERTICAL CURVES

Minimum lengths of crest vertical curves based on sight distance criteria, are generally satisfactory for safety, comfort and appearance [1,p.3-151]

Sight distances (S), are usually measured from height of drivers eye (h_1) above roadway surface (1.08m.), to the height of an object (h_2) which is used as 0.60m for stopping sight distance and 1.08m for passing sight distance.

The basic equations for determining the length (L) of parabolic crest vertical curves, in terms of algebraic difference in grade (A) and sight distance are shown in figure (6-5/1).

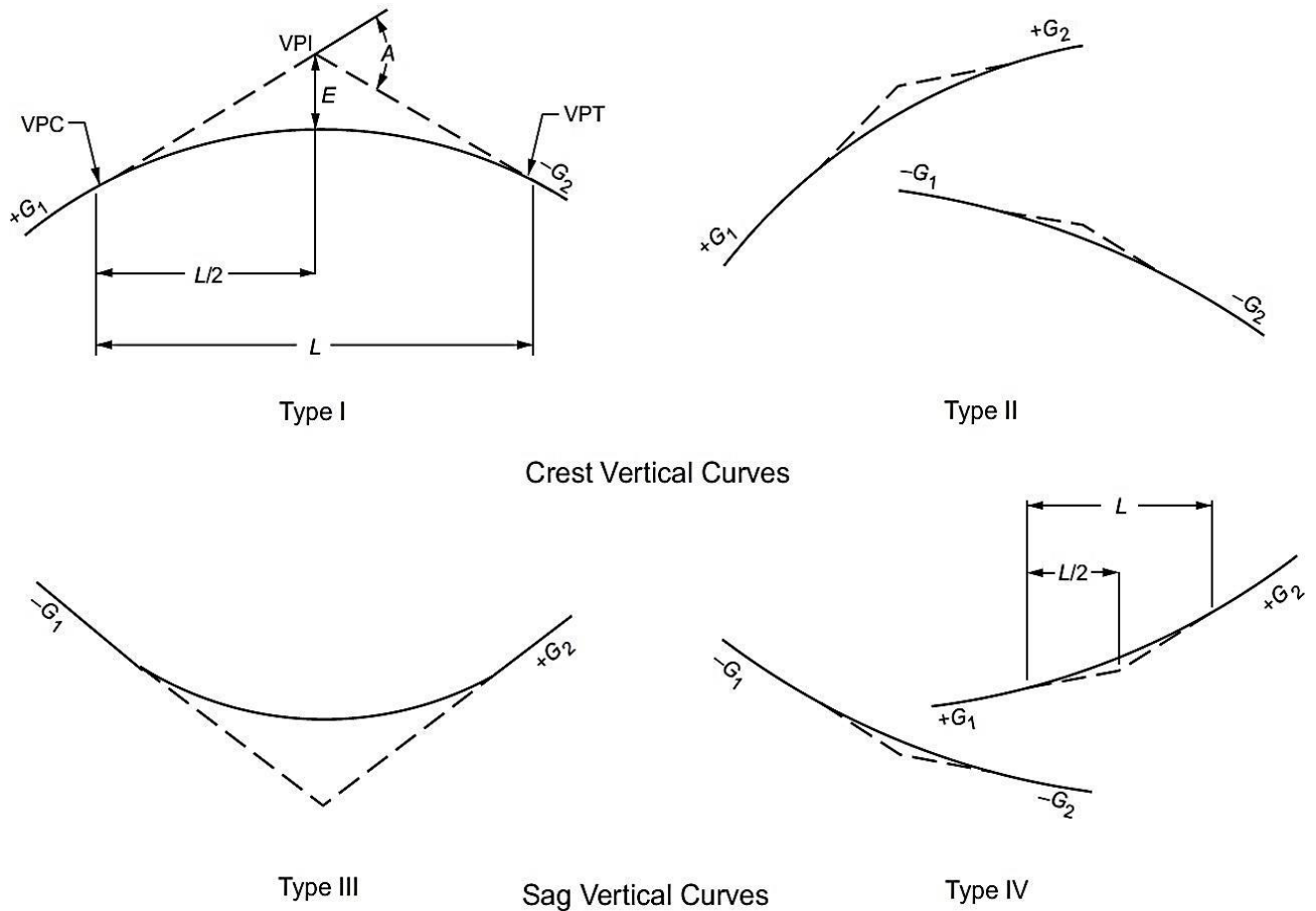
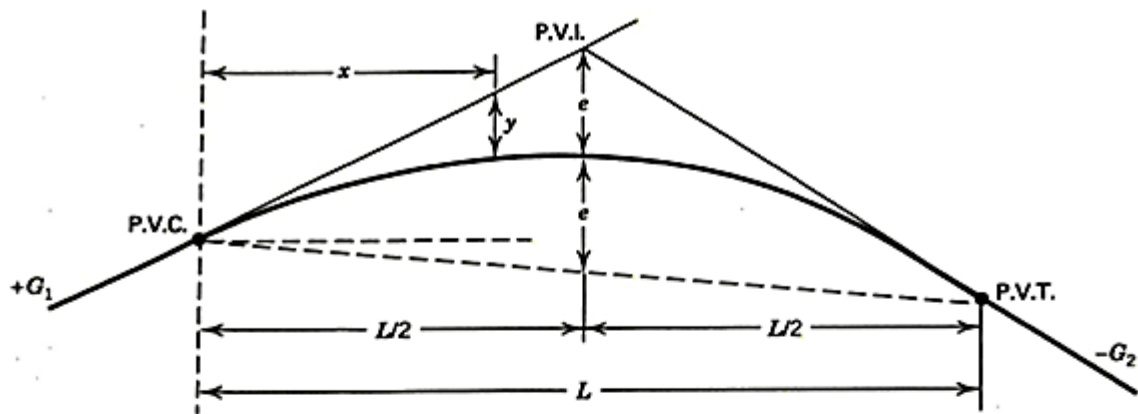


Figure 6-4/1: Basic Types of Vertical Curves [1, p.3-150]



VARIABLES

PVI = Point of vertical intersection
 PVC = Point of vertical curvature
 PVT = Point of vertical tangency
 G_1 = Grade of initial tangent in percent
 G_2 = Grade of final tangent in percent
 L = Length of vertical curve, meters
 A = Algebraic difference in grade between G_1 and G_2 in percent
 K = Vertical curve length coefficient as determined for sight distance, meters per %
 x = Horizontal distance to point on curve, measured from PVC, meters
 E_x = Elevation of point on curve located at distance x from PVC, meters
 x_m = Location of min/max point on curve, measured from PVC, meters
 E_m = Elevation of min/max point on curve at distance x_m from PVC, meters
 e = External distance = middle ordinate, meters
 y = Offset of curve from initial grade line, meters

E_{PI} = Elevation of PVI, meters
 E_{PC} = Elevation of PVC, meters
 E_{PT} = Elevation of PVT, meters

VERTICAL CURVE EQUATIONS

$$A = G_2 - G_1$$

$$K = \frac{L}{A}$$

$$e = \frac{AL}{800}$$

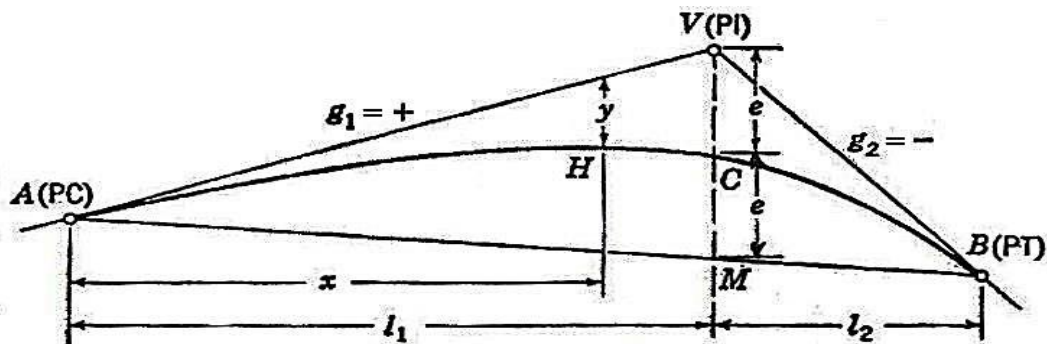
For any point p on curve,

$$y = \frac{(G_2 - G_1)x^2}{200L}$$

$$E_x = E_{PC} + \left(\frac{G_1}{100}\right)x + \frac{(G_2 - G_1)x^2}{200L}$$

For high (low) point on curve, $x_m = \left| \frac{G_1 L}{A} \right|$

Figure 6-4/2: Properties of a Typical Symmetrical Vertical Curve (), [2, p.185]



$$r_1 = \frac{g_2 - g_1}{L} \frac{l_2}{l_1} \quad \text{and} \quad r_2 = \frac{g_2 - g_1}{L} \frac{l_1}{l_2}$$

$$e = \frac{1}{2} r_1 l_1^2 \quad \text{and} \quad e = \frac{1}{2} r_2 l_2^2$$

Where:

g_1, g_2 in (%)
 L, l_1, l_2 in (100 meters)
 e in (meters)
 r_1, r_2 in (% per meter)

Figure 6-4/3: Properties of Unsymmetrical Vertical Curve [3, p. 139]

When S is less than L ,

$$L = \frac{AS^2}{100(\sqrt{2h_1} + \sqrt{2h_2})^2} \quad (6-5/1)$$

When S is greater than L ,

$$L = 2S - \frac{200(\sqrt{h_1} + \sqrt{h_2})^2}{A} \quad (6-5/2)$$

where:

L = length of vertical curve, m

A = algebraic difference in grades, percent

S = sight distance, m

h_1 = height of eye above roadway surface, m

h_2 = height of object above roadway surface, m

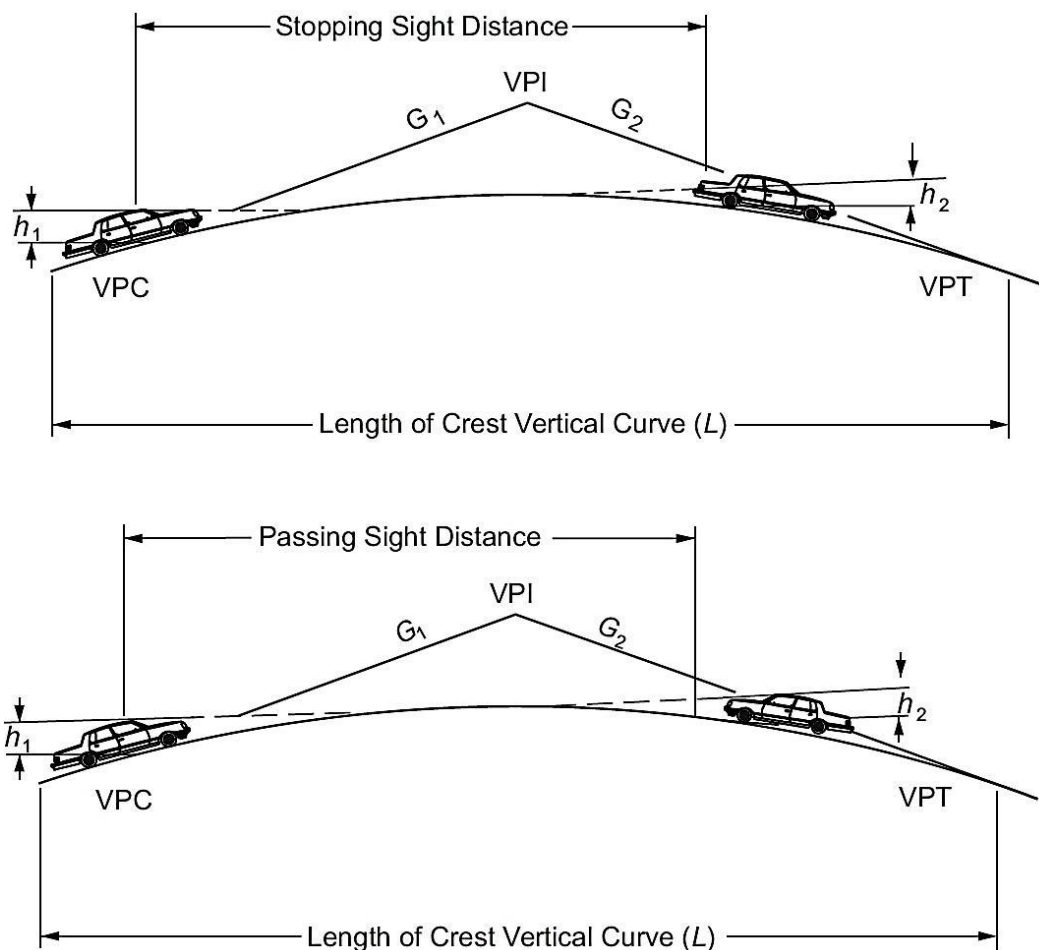


Figure 6-5/1: Parameters Considered in Determining the Length of a Crest Vertical Curve to Provide Sight Distance [1, p.3-152]

6-5/1 MINIMUM LENGTHS TO PROVIDE STOPPING SIGHT DISTANCE

The minimum lengths (L) of crest vertical curves, for different values of (A), to provide the minimum stopping sight distances (S) for each design speed, are shown in figure (6-5/2). The related equations for use with stopping sight distances (S) are:

When S is less than L ,

$$L = \frac{AS^2}{658} \quad (6-5/3)$$

When S is greater than L ,

$$L = 2S - \frac{658}{A} \quad (6-5/4)$$

6-5/2 DESIGN RATE OF VERTICAL CURVATURE OF CREST CURVES TO PROVIDE SAFE STOPPING

The rate of vertical curvature (K), as a length of vertical curve per one percent change in grades (A), is usually used in design for determining minimum vertical curve length (L).

$$L = K \cdot A \quad (6-5/5)$$

Where:

L in meters

K in meters per percent

A = $G_2 - G_1$ in meters

The design rates of vertical curvature (K), for crest curves based on stopping sight distance requirements, are shown in table (6-5/1), for different design speeds.

6-5/3 MINIMUM LENGTHS TO PROVIDE PASSING SIGHT DISTANCE

For the minimum passing sight distance (S), the minimum lengths (L) of crest vertical curves are longer than those for stopping sight distance, with different object height criteria ($h_2 = 1.08\text{m}$), and may be determined using the formulas:

When S is less than L ,

$$L = \frac{AS^2}{864} \quad (6-5/6)$$

When S is greater than L ,

$$L = 2S - \frac{864}{A} \quad (6-5/7)$$

6-5/4 DESIGN RATE OF VERTICAL CURVATURE OF CREST CURVES TO PROVIDE SAFE PASSING

The design rates of vertical curvature (K), for crest curves based on passing sight distance, are shown in table (6-5/2), for different design speeds, in order to determine the required lengths of crest curves. Generally, it is impractical to design crest vertical curves that provide passing, because of high construction cost where crest cuts are involved [1, p. 3-157].

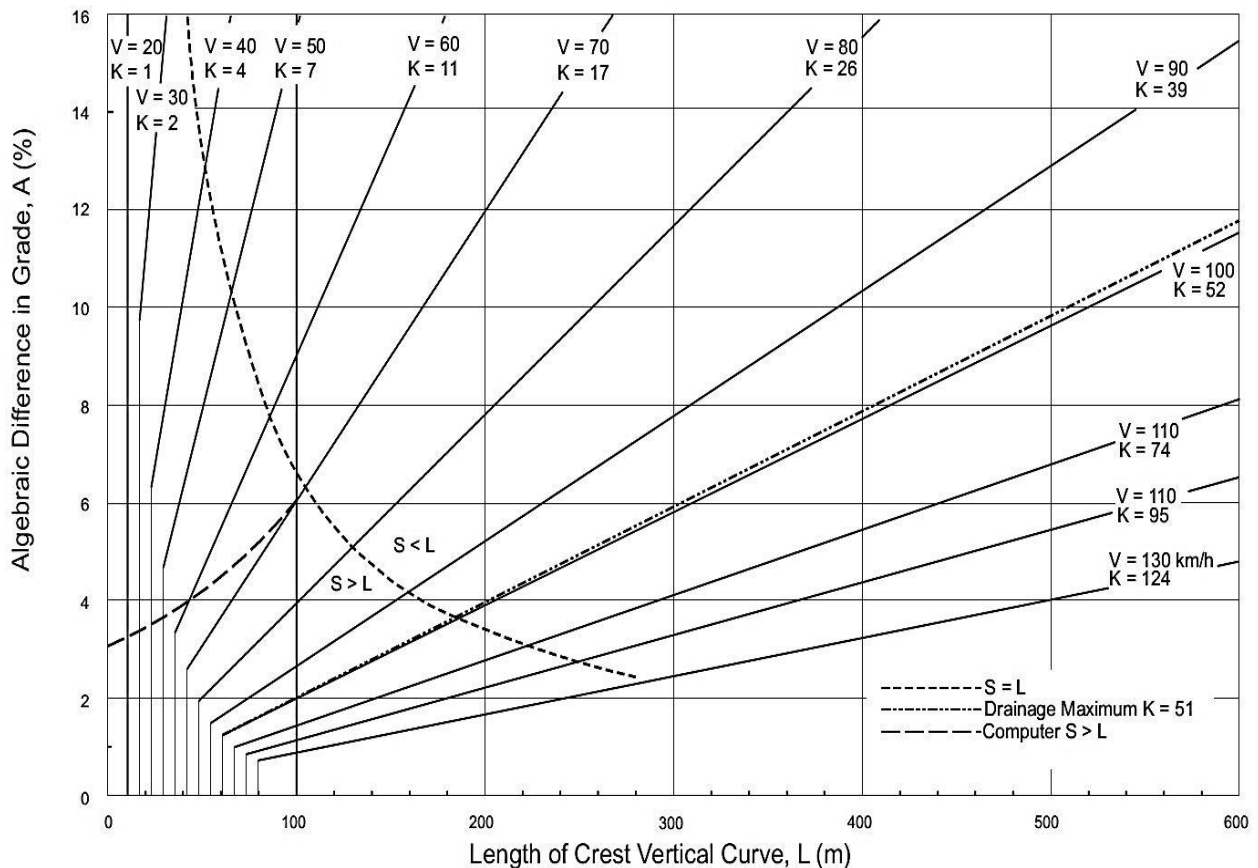


Figure 6-5/2: Design Controls for Crest Vertical Curves-Open Road Conditions [1, p.3-154]

Table 6-5/1: Design Controls for Crest Vertical Curves Based on Stopping Sight Distance [1, p.3-155]

Design Speed (km/hr.)	Stopping Sight Distance (m)	Rate of Vertical Curvature, $K = L/A$	
		Calculated	Design
20	20	0.6	1
30	35	1.9	2
40	50	3.8	4
50	65	6.4	7
60	85	11.0	11
70	105	16.8	17
80	130	25.7	26
90	160	38.9	39
100	185	52.0	52
110	220	73.6	74
120	250	95.0	95
130	285	123.4	124

Table 6-5/2: Design Controls for Crest Vertical Curves Based on Passing Sight Distance
[1, p.3-157]

Design Speed (km/hr.)	Passing Sight Distance (m)	Rate of Vertical Curvature, K_a Design
30	120	17
40	140	23
50	160	30
60	180	38
70	210	51
80	245	69
90	280	91
100	320	119
110	355	146
120	395	181
130	440	224

^a Rate of vertical curvature, K , is the length of curve per percent algebraic difference in intersecting grades (A), $K = L/A$.

6-6 MINIMUM LENGTHS OF SAG VERTICAL CURVES

The minimum length of sag parabolic vertical curves is established on the basis of four criteria, including:

Headlight sight distance (safety), Passenger comfort;
General appearance; and Drainage control.

6-6/1 SAFETY CRITERIA (HEADLIGHT SIGHT DISTANCE)

The portion of highway lighted ahead, is dependent on the headlight height (0.60m), and the upward divergence of the light beam from longitudinal axis of vehicle (1 degree).

A sag vertical curve should be long enough, that light beam distance is the same as stopping distance (S). On this basis, the minimum lengths of sag vertical curves may be determined using the following formulas presented together with table (6-6/1):

When S is less than L ,

$$L = \frac{AS^2}{200[0.6 + S(\tan 1^\circ)]}$$

or,

$$L = \frac{AS^2}{120 + 3.5S} \quad (6-6/1)$$

where:

L = length of sag vertical curve, m

A = algebraic difference in grades, percent

S = light beam distance, m

When S is greater than L ,

$$L = 2S - \frac{200[0.6 + S(\tan 1^\circ)]}{A}$$

or,

$$L = 2S - \frac{120 + 3.5S}{A} \quad (6-6/2)$$

The resulting lengths of sag vertical curves, for the recommended stopping sight distances, for each design speed are shown in figure (6-6/1).

6-6/2 DESIGN RATE OF VERTICAL CURVATURE OF SAG CURVES

The headlight sight distance is the most logical criterion to establish design values of lengths for sag vertical curves.

The design controls in terms of the (K) rate for sag curves are shown in table (6-6/1), for different design speeds, in order to determine minimum sag curves length: $L = K \cdot A$. Longer curves are desired wherever practical, but special attention to drainage needs to be considered where (K) values exceed 51 meters per percent change in grade [1, p.3-161]

Table 6-6/1: Design Controls for Sag Vertical Curves [1, p.3-161]

Design Speed (km/hr.)	Stopping Sight Distance (m)	Rate of Vertical Curvature, K^a	
		Calculated	Design
20	20	2.1	3
30	35	5.1	6
40	50	8.5	9
50	65	12.2	13
60	85	17.3	18
70	105	22.6	23
80	130	29.4	30
90	160	37.6	38
100	185	44.6	45
110	220	54.4	55
120	250	62.8	63
130	285	72.7	73

^a Rate of vertical curvature, K , is the length of curve per percent algebraic difference in intersecting grades (A), $K = L/A$.

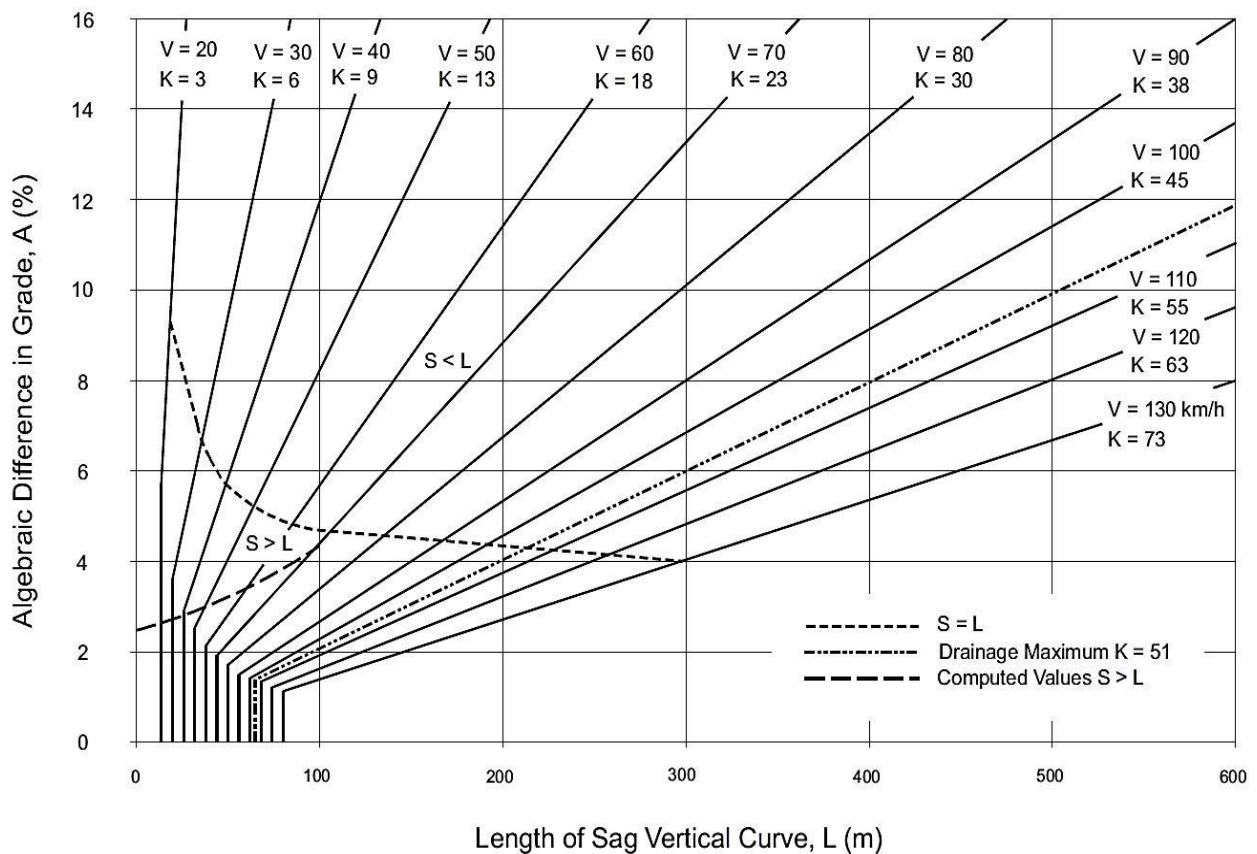


Figure 6-6/1: Design Controls for Sag Vertical Curves-Open Road Conditions [1, p.3-159]

6-6/3 COMFORT CRITERIA

On sag curves, gravitational and centripetal forces are combining, and thus passenger comfort due to the change in vertical direction is greater, as compared to crest curves.

Riding is comfortable on sag vertical curves when the centripetal acceleration does not exceed 0.3 m/sec.^2 . On this basis, the minimum length of sag vertical curve (L), to satisfy comfort may be determined from the following formula [1.p.3-160]:

$$L = \frac{A \times V^2}{395} \quad (6 - 6/3)$$

Where:

L = Length of sag vertical curve, m

A = Algebraic difference in grades, percent

V = Design speed, km/hr.

6-6/4 GENERAL APPEARANCE CRITERIA

For improved appearance on sag vertical serves:

$$\text{Minimum Length (meters)} = 30 \times A \quad (6 - 6/4)$$

Where: A in percent

Longer curves are appropriate to improve appearance for high type highways [1, p.3-160].

6-6/5 DRAINAGE CONTROL CRITERIA

Drainage affects design of sag vertical curve with (plus and minus) grades, where curbed sections are used. A minimum grade of 0.3% should be provided within 15m of the level point of the sag curve. This criteria corresponds to (K) of 51 meters per percent change in grade. Therefore;

$$\text{Maximum L (meters)} = 51 \times A \quad (6 - 6/5)$$

Where: A in percent

6-7 MINIMUM LENGTHS OF SAG VERTICAL CURVES UNDERCROSSING A GRADE SEPARATION STRUCTURE

The minimum length of sag vertical curve undercrossing a grade separation structure need to provide a sight distance which should not be less than the minimum stopping sight distance.

In figure (6-7/1), the general equations for minimum length determination of sag curve are presented at undercrossings, including the equations related to truck drivers.

6-8 GENERAL CONTROLS FOR VERTICAL ALIGNMENT

- Grade lines with numerous breaks and short lengths need to be avoided.
- The "hidden- dip" or "roller- coaster" profiles in straight alignment of a rolling terrain should be avoided, by use of horizontal curves with more gradual grades.
- A "broken- back" grade line (two vertical curves in the same direction, separated by a short grade) should be avoided.
- A uniform sustained long grade slightly below maximum grades, needs breaking the grade with flatter grades, placing the steepest grade at the bottom.
- Sag vertical curves should be avoided in cuts, unless adequate drainage can be provided.
- It is desirable to use very flat grades at sections of at- grade intersections of the roadway. [1,p.3-163].

The general equation for sag vertical curve length at undercrossings:

For $S > L$

$$L = 2S - \frac{800 \left[C - \left(\frac{h_1 + h_2}{2} \right) \right]}{A} \quad (6-8/1)$$

where:

L = length of vertical curve, m
 S = sight distance, m
 C = vertical clearance, m
 h_1 = height of eye, m
 h_2 = height of object, m
 A = algebraic difference in grades, percent

For $S < L$

$$L = \frac{AS^2}{800 \left[C - \left(\frac{h_1 + h_2}{2} \right) \right]} \quad (6-8/2)$$

where:

L = length of vertical curve, m
 A = algebraic difference in grades, percent
 S = sight distance, m
 C = vertical clearance, m
 h_1 = height of eye, m
 h_2 = height of object, m

For trucks ($h_1 = 2.40$ m, $h_2 = 0.60$):

For $S > L$

$$L = 2S - \frac{800(C - 1.5)}{A} \quad (6-8/3)$$

For $S < L$

$$L = \frac{AS^2}{800(C - 1.5)} \quad (6-8/4)$$

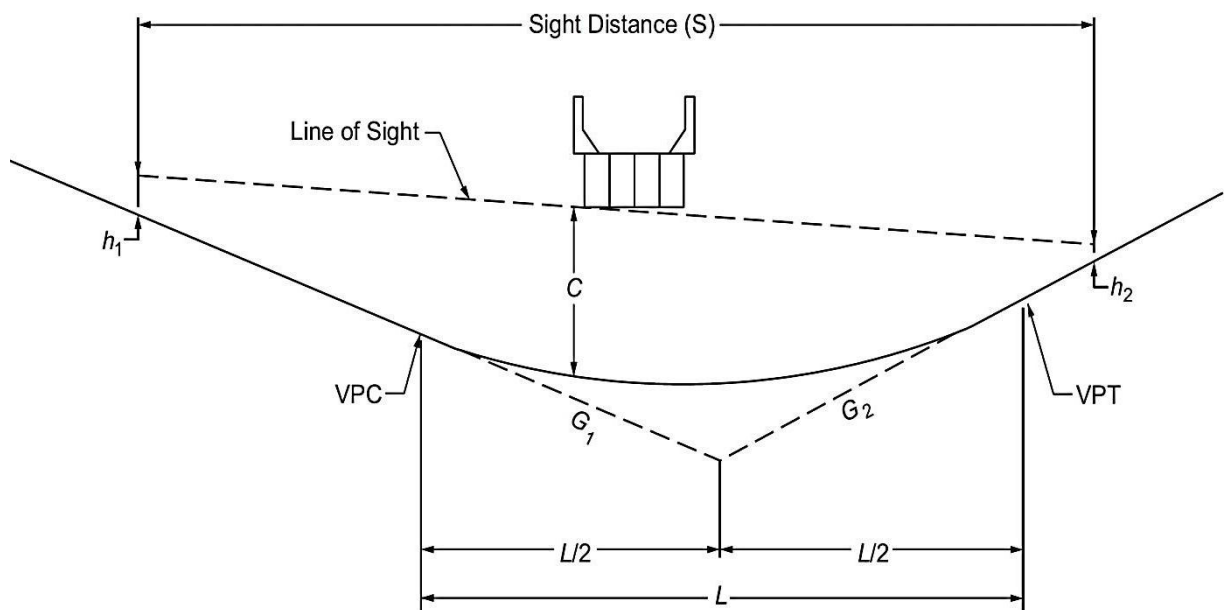


Figure 6-7/1: Sight Distance at Undercrossings [1, p.3-162]

6-9 GENERAL DESIGN CONTROLS FOR COMBINATIONS OF HORIZONTAL ALIGNMENT AND PROFILE

- Horizontal and vertical alignment should not be designed independently. Design of the combination offers safety, capacity, uniformity of operation, and pleasing appearance within the practical limits of terrain, nearly always without additional cost.
- Flat curvature with steep and long grades, or excessive curvature with flat grades, both represent poor design.
- Vertical curvature superimposed on horizontal curvature, without successive changes in profile (humps), generally results in a more pleasing facility.
- Sharp horizontal curvature should not be introduced at the top of a crest vertical curve or near the low point of a sag vertical curve.
- Both horizontal curvature and profile should be made as flat as practical at intersections.
- To minimize the nuisance to the neighborhood, a depressed alignment makes the highway less visible and less noisy to adjacent area.
- The alignment design should enhance attractive views towards, (rather than away), of the natural or manmade features (reverses, parks, outstanding structures).

Some examples of poor and good alignment design are shown in figure (6-9/1).

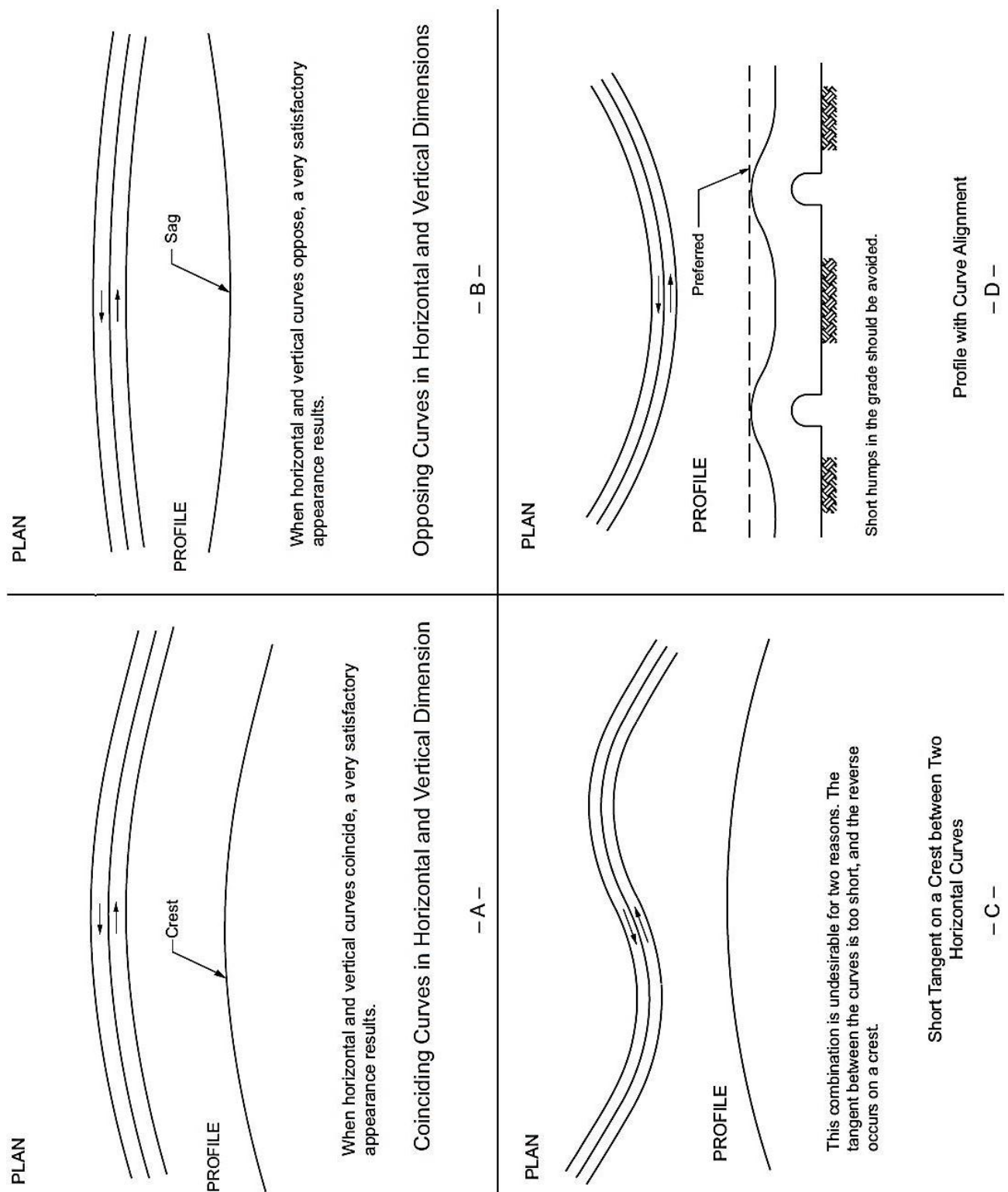


Figure 6-9/1: Alignment and Profile Relationships in Roadway Design [1, p.3-168]

6-10 REFERENCES

- [1] AASHTO, "*A Policy on Geometric Design of Highways and Streets*", American Association of State Highway and Transportation Officials, USA, 2011.
- [2] Wright, P.H. and Dixon, K. K., "*Highway Engineering*", John Wiley & Sons, USA, 2004.
- [3] T.F. Hickerson, "*Route Surveys and Design*", McGraw- Hill Book Co., USA, 1959.

CHAPTER 7

CROSS SECTION ELEMENTS

Cross Section: is a vertical sectional section of the ground and roadway, at right angles to the interline, including all elements of a highway or street, between right- of- way lines.

Roadway: is the portion of highway, including shoulders, for vehicular use.

Traveled way: is the portion of roadway, exclusive of shoulders, for the movement of vehicles.

7-1 LANE WIDTH AND MARGINAL STRIP

Lane width of a roadway affects level of service, comfort of driving, safety, and operational characteristics. A lane width of 3.75m is generally used for multilane highway in Iraq. The standard lane width is 3.60m.

Lane width, of 3.30m or 3.00m may be used at low speed facilities in urban areas. [1,p.4-7]

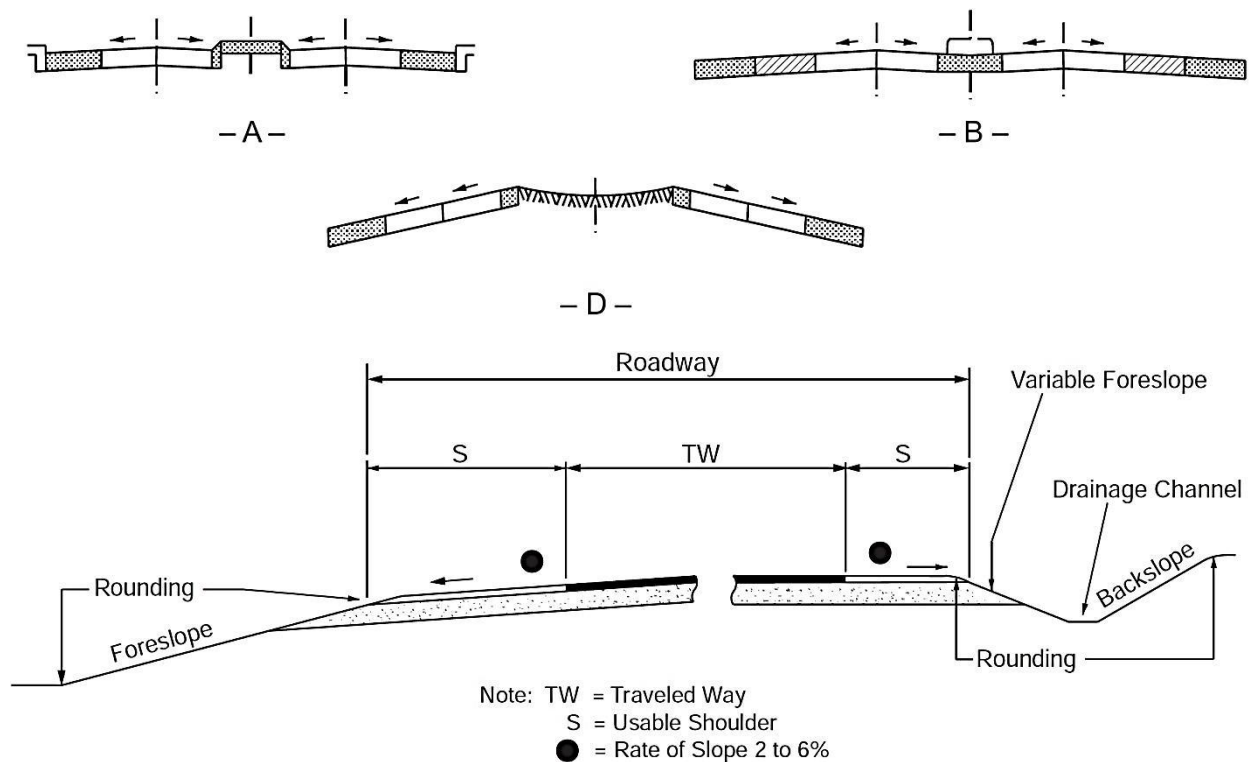
Marginal strips need to be provided with widths of 0.25m to 0.50m, and should be highly visible, using the same colour of pavement marking. [4, p. II-34]

7-2 CROSS SLOPE

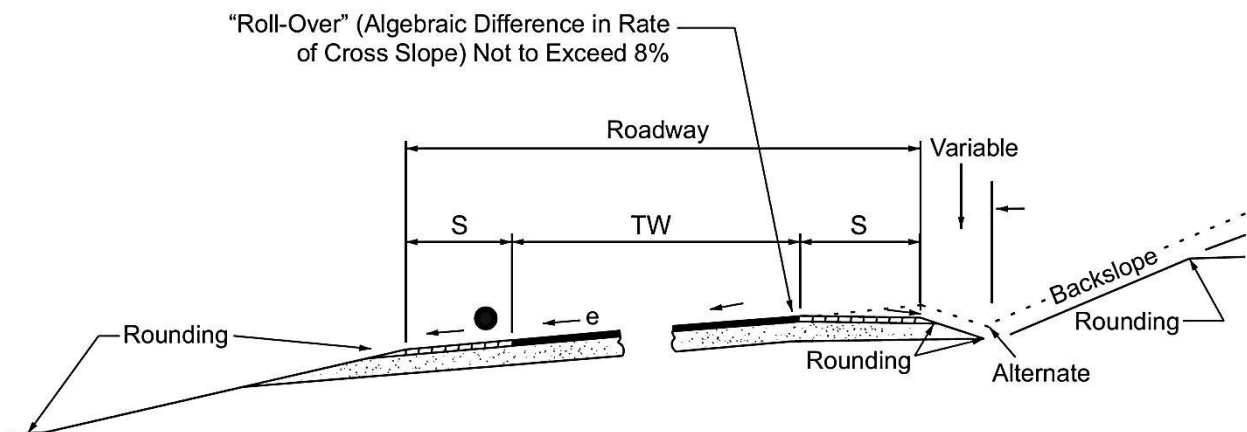
On paved two- lane roadways, a crown or high point in the middle and a cross slope (1.5- 2.0) percent downward toward both edges, is normally used.

When three or more lanes are inclined in the same direction on multilane highways, the cross slope may be increased by 0.5 to 1.0 percent for each successive pair of lanes outward from the top two lanes. On tangent alignment, the cross slope should not exceed 3% unless there are three or more lanes in one direction of travel.

A greater cross slope (2 to 6) percent should be utilized for unpaved surfaces figure (7-2/1) shows typical cross slope arrangements.



Typical Cross Section, Normal Crown



Typical Cross Section, Superelevated

Figure 7-2/1: Basic Cross Slope Arrangements [1, p.4-3]

7-3 SHOULDERS

Shoulder: is the portion of roadway, contiguous with the traveled way that accommodates stopped vehicles, emergency use, and lateral support of pavement layers. [1, p. 4-8].

The normal usable shoulder width is 3.00 meters and preferably 3.60 meters. A minimum shoulder width of 1.80-2.40 m may be considered for low- volume highways. Shoulders on structures should normally have the same width as usable shoulders on the approach roadways.

Asphalt and concrete-surfaced shoulders should be sloped from 2 to 6 percent, aggregate shoulders from 4 to 6 percent, and turf shoulders from 6 to 8 percent. [1, p.4-11]

It is desirable that texture of shoulders be different from those of traveled way.

7-4 MEDIANS

Median is the portion of a highway separating opposing directions of the traveled way. They provide a space for emergency stops, allow space for storage of left running diminish headlight glare, provide widths for future lanes, and provide refuge area for pedestrian crossings.

Median widths are in the range from 1.20 to 24.00m, and should be highly visible both night and day. Medians may be depressed, raised, or flush with the traveled way surface. A depressed median is generally preferred on rural highways for more efficient drainage with side slope of 1V: (4 to 6) H

Raised medians are used on arterial streets to regulate left- turn movements. A median barrier may be needed where flush medians are used in urban streets. When medians are 12m or wider, drivers have a sense of separating from opposing traffic, with freedom of operation.

Wide median (18m or more) may not be desirable at urban intersections that are signalized. [1, p.4-34]

7-5 SIDESLOPES

Sideslopes should be designed to provide roadway stability, with a reasonable opportunity for recovery of an out- of- control vehicle. The top of the slope (hinge point) need to be rounded to reduce the loss of steering control, because vehicles tend to become airborne in crossing this point.

Foreslopes steeper than 1V: 4 H are nor desirable. When slopes steeper than 1V: 3 H are used, consideration should be given to the use of roadside barrier.

Backslopes should be 1V: 3H or flatter. Retaining walls should be considered where space restrictions result in slopes steeper than 1V: 2H.

A smooth transition between foreslope and backslope should be provided (ditch bottom) at the toe of the slope, in case the out- of- control vehicle with reach the ditch. [1,p.4-24]

For slope stability of soils that are mainly clay or susceptible to erosion, slopes of 1V: 3H or flatter should be used. A commonly used slope for rock cuts is 2V:1H, and slopes as steep as 6V: 1H may be used in good- quality rock.

7-6 DRAINAGE CHANNELS

Drainage channels are used to collect and convey surface water from the highway right- of- way. Roadside channels should be located and shaped to provide a smooth transition from the roadway to the backslope.

Foreslope and backslope combination forming the channel (width: 1.20-2.40m), can be selected to produce cross sections that can be safely traversed by an unrestrained vehicle occupant. Flatter foreslopes (1V:4H), permit greater flexibility in the selection of backslopes to permit safe traversal figure 7-6/1.

The depth of channel should be sufficient to remove surface water, based on a desirable grade needed to avoid sedimentation.

Median drainage channels are generally shallow depressed areas, located at or near the center of the median and formed by flat sideslopes. The channel is sloped longitudinally for drainage, and water is intercepted at intervals by inters and discharged from the roadway in a storm drain or culvert. Flumes are used to carry the water collected by shoulder curb down the slopes, and can be either pipes or open channels. See figure (7-6/2)

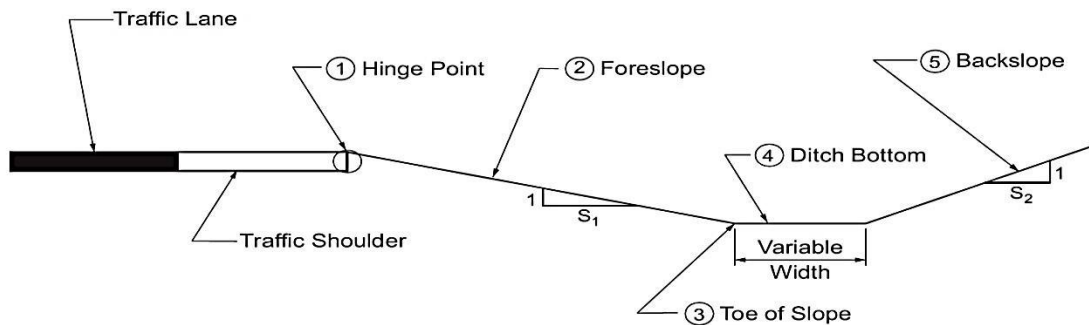


Figure 7-6/1: Designation of Roadside Regions [1, p.4-24]

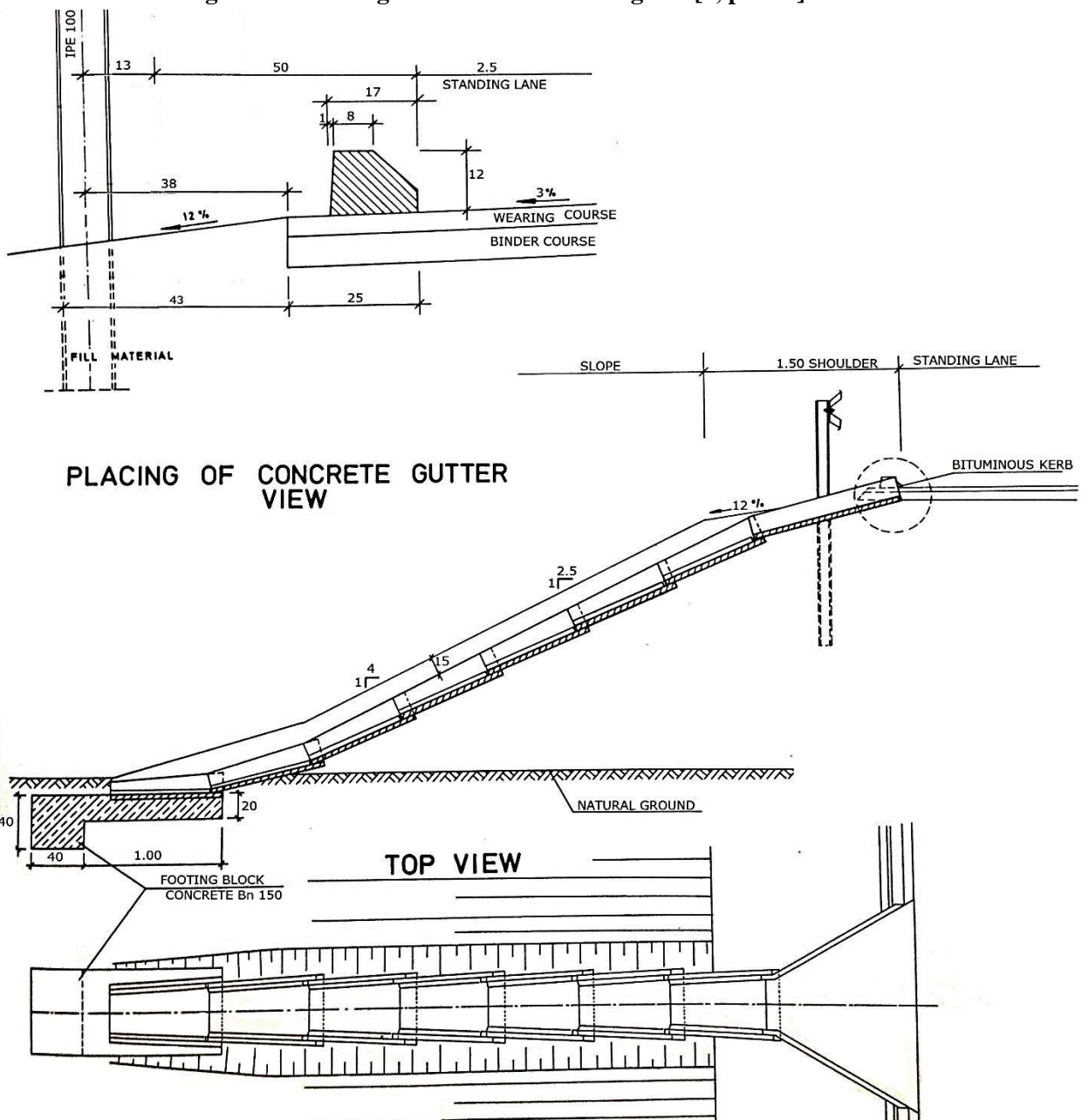


Figure 7-6/2: Flumes of Iraq Expressway No. One [2, R9/A]

7-7 CURBS AND GUTTERS

Curbs are used for drainage control, edge delineation, safety of pedestrian walkways, and reduction of maintenance operations.

Curb configurations include both vertical and sloping curbs as illustrated in figure (7-7/1). Concrete or asphaltic curbs need to be visible particularly at night which may be improved through the use of reflectorized markers attached to the top of the curb.

Vertical and sloping curb design may include a gutter, forming a combination curb and gutter section. Vertical curbs are intended to discourage vehicles from leaving the roadway, with a range from 15 to 20 cm in height. [1, p.4-16]

Sloping curbs are designed so vehicles can cross them readily when the need arises. When the slope of the curb face is steeper than 1V: 1H, vehicles can mount the curb when the height is not exceeding 10 cm. When the face slope is between 1V: 1H and 1V: 2H, the height should be limited to 15cm. When the total curb height exceeds 15cm, it may be considered a vertical curb rather than a sloping curb. Gutter section are provided to form the principal drainage system, 0.3 to 1.8m wide, with a cross slope of 5 to 8 percent, and should be of contrasting color and texture.

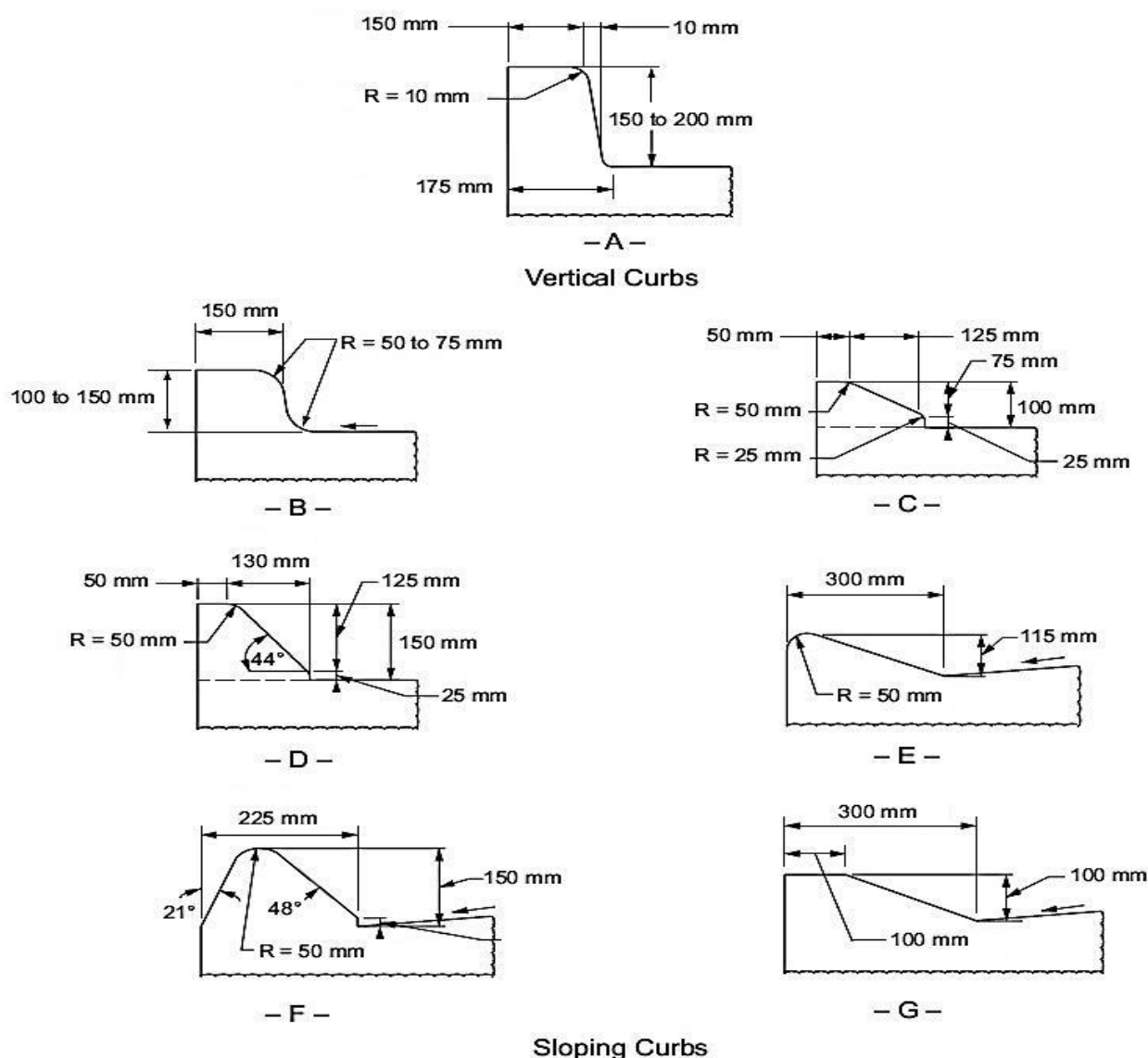


Figure 7-7/1: Typical Highway Curbs [1, p. 4-17]

7-8 SIDEWALKS

Sidewalks along city streets are often justified by pedestrian concentration, such as residential area, schools, businesses, and industrial plants. Shoulders may obviate the need for sidewalks in rural area, in all weather conditions.

In urban area, a border area, (2.4m) minimum width, separates the roadway from community's homes and businesses, to provide space for sidewalks, vegetation, street lights, and street hardware.

Sidewalk minimum width in residential area may vary from 1.20 to 2.40m. The width of planted strip between sidewalk and traveled- way curb, if provided, should be a minimum of 0.60m.

The cross slope of sidewalks is not permitted to exceed 2%. On long bridges (greater than 60m), a single walkway may provide.

A grade- separated pedestrian facility, either over or under a roadway, allows crossing at different levels with safe refuge. Overcrossings are more preferred by pedestrian. Pedestrian ramps in addition to stairway should be provided at all pedestrian separation structures [1,p.4-56].

7-9 BIKEWAYS

There are two basic infrastructure approaches for bicycling as a mode of travel within transportation system:

- a) On –street improvements, which may include:
 - Share motor vehicle/ bicycle use of a standard lane, with no bikeway designation.
 - Shared motor vehicle/ bicycle use with designated bike route sign.
 - A portion of roadway is designated by marking for exclusive use of bicycles (figure 7-9/1).
- b) Separate shared- use path, which is facility physically separated from the roadway, and intended for a variety of path users including cyclists (figure 7-9/2).

The classification groups for bicyclists may include: group A (advanced or experienced), group B (basic or casual bicyclists), and group C (Children).

The minimum width of a bicycle lane at roads is 1.20 to 1.50 meters, and some departments may specify 1.80 meters. For the separate shared- use path, the absolute minimum width for a two-direction bike path is 2.40 to 3.00m, with a desirable width of 3.60 to 4.20 meters. The minimum shoulder width of 0.60m must be graded to drain water away.

Most cyclists travel within a range of 11-24 km/hr., with an average of 16-18 km/hr. The minimum design speed for bicycle path is 30 km/hr. for most eases. The superelevation rate for most bike path designs will vary from a minimum of 2% to a maximum of 3%.

The minimum radii for paved shared –use paths are presented in table (7-9/1) for the different design speeds and friction factors. Grades greater than 5% are undesirable for bicyclists and when the 5% grade is exceeded, the length of the grade should not exceed the vales listed in table (7-9/2).

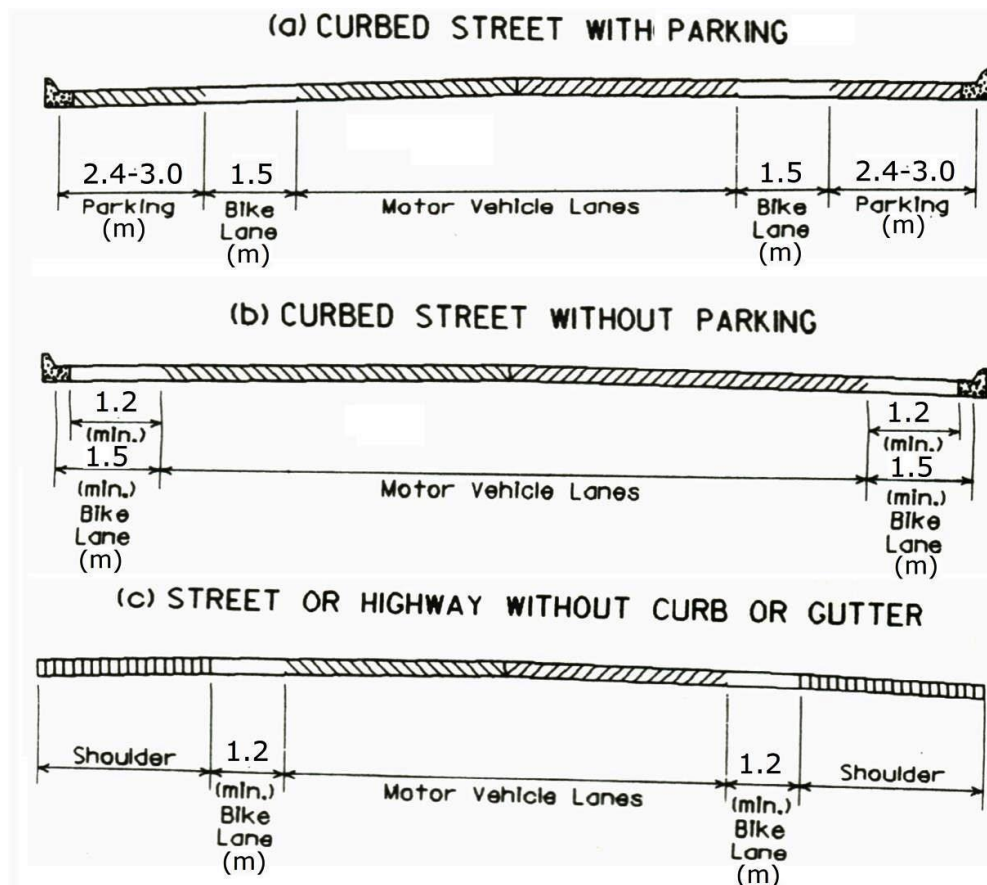


Figure 7-9/1: Typical Bicycle Lane Cross Section [3, p.202]

Table 7-9/1: Minimum Radii for Paved Shared-Use Path (2% Superelevation and 20° Lean Angle) [3, p.205]

Design Speed, V (km/hr.)	Friction Factor, f	Minimum Radius, R (m)
20	0.31	10
30	0.28	24
40	0.25	47
50	0.21	86

Table 7-9/2: Maximum Length of Grades Exceeding 5%

Grade %	Maximum Length, meters
5-6	240
7	120
8	90
9	60
10	30
≥ 11	15

Minimum vertical curve lengths may be determined using an eye height for bicyclist $h_1 = 1.40\text{m}$, and object height, $h_2 = \text{zero}$

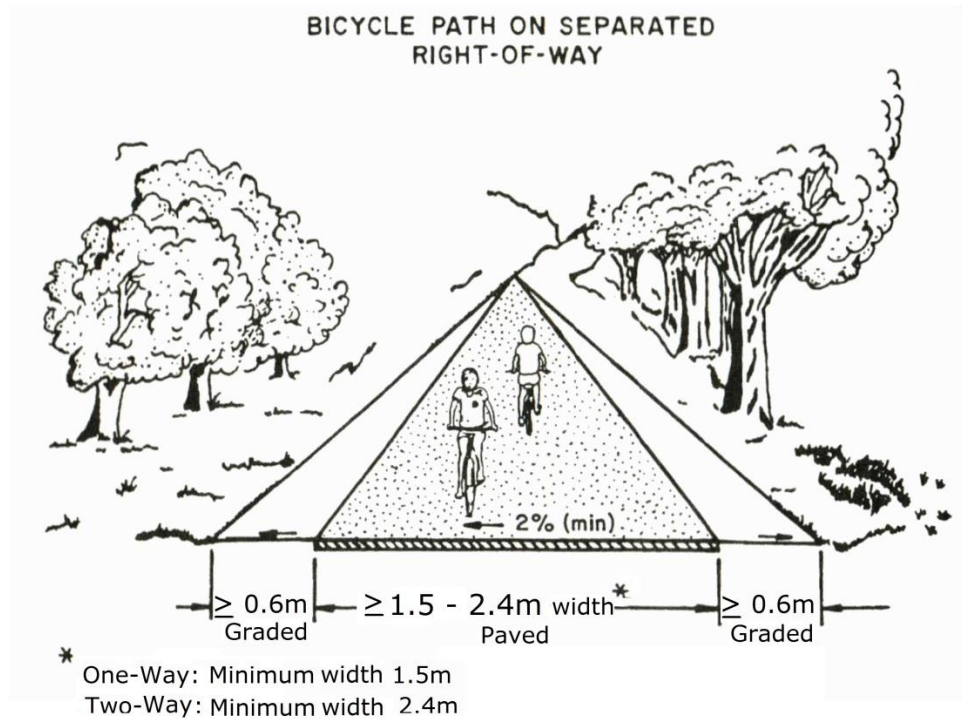


Figure 7-9/2: Two-way shared-use path on separated right-of-way [3, p.204]

7-10 RIGHT-OF-WAY

Sufficient right – of- way should be acquired in order to avoid the expense of purchasing developed property, with varying widths depending on local conditions.

The right-of- way for a 2- lane highway in rural areas is recommended to have a minimum width of 30m, with 37 m desirable. A minimum right- of- way width of 45m, and a desirable width of 76m are recommended for divided highways. Widths of 60 to 90m have been used for divided highways without frontage roads.

For Iraq Expressway No. One, a right- of- way width of 260 m has been provided, which included service roads. A typical cross section dimensions within the right- of- way width is presented in figure (7- 10/1), for a two- lane highway.

7-11 HORIZONTAL AND VERTICAL CLEARANCES

Lateral (horizontal) clearances and vertical clearances usually used in design in Iraq, are presented in table (7-11/1), including the following:

- a) Minimum vertical clearances to the underside of over passing structure.
- b) Minimum horizontal clearances from roadway edge to obstruction.
- c) Minimum vertical clearances for high tension lines.
- d) Minimum horizontal clearances for high tension lines.

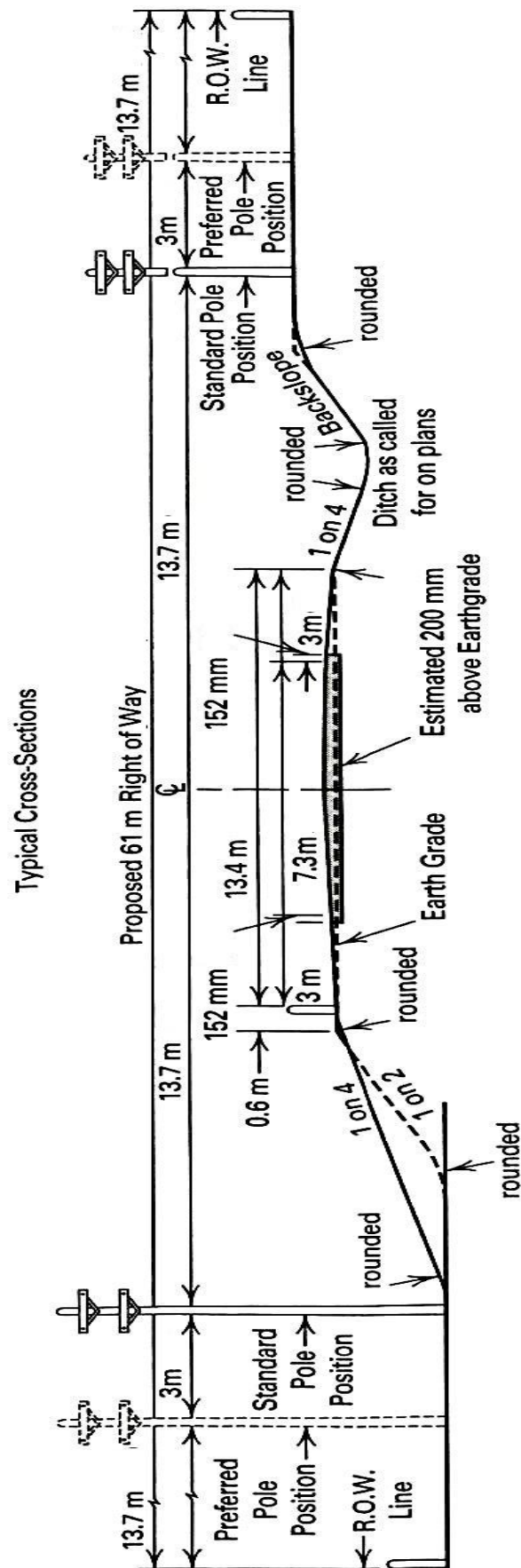


Figure 7-10/1: Typical cross-section dimensions for arterial streets . (Courtesy American Association of State Highway and Transportation Officials) [3, p. 170]

Table 7-11/1: Vertical and Horizontal Clearances [4, P.II-43]

a) Minimum Vertical Clearance

OVER	MINIMUM VERTICAL CLEARANCE
Road	5.20 m
Footway	2.50 m
Railway	6.50 m
River	
– group 1: Tigris, Euphrates	6.25 m
– group 2: Diyala, Gharraf, Great Majar, Shatt-al-Hilla, Shamiya, Suwerra, Qadissiya	3.50 m
– group 3: Kahla'a, Musharah, Butera, Bahriya	2.50 m
– others	1.50 m

b) Minimum Horizontal Clearance (D) from Roadway Edge to Obstruction

Design speed	D
120 km/h.	15 m
100 km/h.	10 m
80 km/h.	6.5 m
70 km/h.	5.0 m
60 km/h.	4.0 m
50 km/h.	3.0 m
40 km/h.	2.5 m
30 km/h.	2.0 m
(c) Along rivers or lakes if the distance to the nearest water edge is less than given above (absolute min: 10 m).	

c) Minimum Vertical Clearance for High Tension Lines

Type of road and other constructions	Minimum clearance required for 400 thousand volt transmission line	Minimum clearance required for 132 thousand volt transmission line
Natural ground	8.25 meters	6.0 meters
Major roads	10.0 meters	8.8 meters
Minor roads	9.00 meters	8.8 meters
Railway lines	13.75 meters	11.5 meters
Oil & gas pipe lines	10.00 meters	8.8 meters
Highest water level while crossing river	14.75 meters	10.5 meters

d) Minimum Horizontal Clearance Required for High Tension Lines

Type of Construction	Minimum horizontal distance required
From the centre line of railway line	50 meters
From centre line of telephone and other low and medium voltage electric lines	50 meters
From centre line of road	100 meters
From centre line of the bridge	100 meters
From centre line of gas and pipe line	100 meters

7-12 ON-STREET PARKING

Stalls for on- street (curb) parking are especially recommended for locations where there is great demand for curb parking spaces, and where parking meters are used.

On- street parking generally, decreases through- traffic capacity, impedes traffic flow, and increases crash potential. Parallel parking is usually considered for on- street parking and angle parking should be allowed only under certain circumstances.

Most drivers will parallel park within 15 to 30 cm of the curb face. The desirable minimum width of a parking stall is 2.40m, and widths of 3.00 to 3.60m may be used. Stall lengths vary from 6.60 to 7.80m.

Curb parking stalls should not be placed closer than 6.0m to the nearest sidewalk edge at unsignalized intersections. At signalized intersections, a clearance to the nearest sidewalk edge of 15.2m and preferably 30m should be considered for parking stalls. Figure (7-12/1) shows an example for parking stall treatment.

7-13 LONGITUDINAL BARRIERS AND CRASH CUSHIONS

Longitudinal barriers are systems used to prevent vehicles from leaving the roadway, and crashing into roadside obstacles, overturning, or crossing into the path of vehicles in the opposite direction. Longitudinal barriers include three classes: roadside barriers, median barriers, and bridge railings.

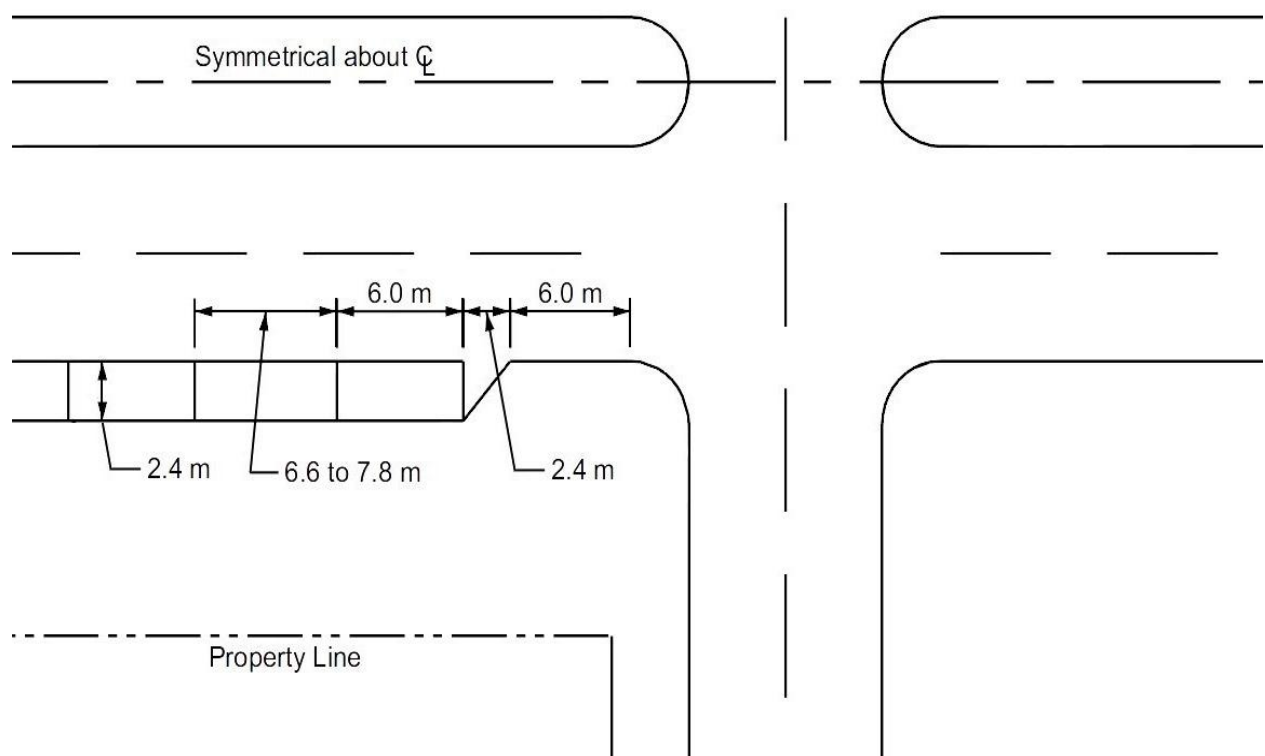


Figure 7-12/1: Parking Lane Transition at Unsignalized Intersection [1, p.4-74]

Roadside barriers may include flexible or semirigid system (Guardrails), and rigid systems (concrete and stone masonry walls). Design features of some guardrail system are given in table (7-13/1).

The height and slope of embankments are the primary factors that determine whether guardrails should be used along embankments. Barriers are not usually warranted on embankments with sideslopes flatter than 1V: 3H. See figure (7-13/1) [3, p.225].

The barrier should be placed far enough from the pavement edge, beyond (Shy Line) offset values given in table (7-13/2). Special care should be considered when designing and installing barrier ends, and barrier transitions. Guardrails ends may be flared away from pavement, or design a loader terminal that collapse upon impact, to absorb the energy of the crash, figure (7-13/2). An example of barrier transition design of connections between guardrails and bridge parapets is shown in figure (7-13/3).

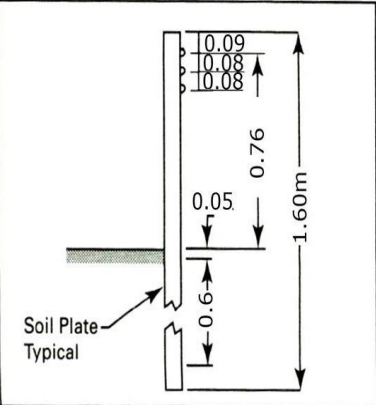
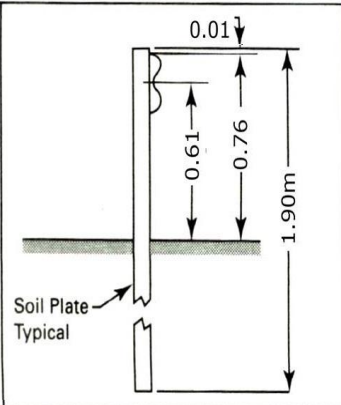
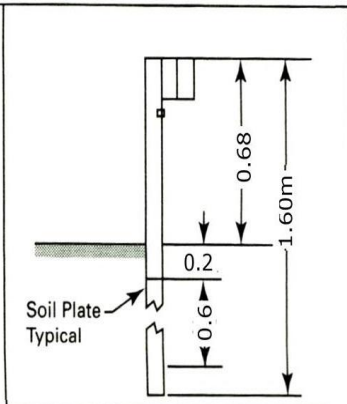
The design features of median barrier system are presented in table (7-13/3). Barrier system performs test when a level surface is provided in front of the barrier (without curbs or stepped medians. For narrow medians, concrete barriers, such as (New Jersey) is recommended figure (7-13/4).

Bridge railings are designed to prevent vehicles from running out of the edge of bridges or culverts, after connected to the structure, using large number of systems, including rail and posts or reinforced concrete railing, with common heights of 0.81 to 1.00 meter.

Crash cushions are protective systems that prevent errant vehicles from impacting roadside obstacles, by decelerating the vehicle to a safe stop when head-on, or redirecting vehicles away from the obstacle [1, p.4-33]. Crash cushions are usually used in front of retaining walls, bridge piers, and abutments.

Rows of barrels and arrays of containers filled with sand may be used for this purpose. The design of crash cushions usually employs one of two concepts of mechanics including the kinetic energy principle or the conservation of momentum principle, with or without rigid support to resist the vehicle impact force. An example of the configuration of sand-filled plastic barrels is shown in figure (7-13/5).

Table 7-13/1: Design Features of Guardrails That Have Performed Satisfactorily [3, p.224]

			
System	Three-Strand Cable	W-Beam (weak post)	Weak Post Box Beam
Deflection	3.50m	1.98m	1.5m
Post spacing	4.87m	3.66m	1.83m
Post Type	S3 x 5.7 steel	S3 x 5.7 steel	S3 x 5.7 steel
Beam type	1.9cm dia steel cables	12-gage W-beam	(0.15x0.15x0.05) steel tube

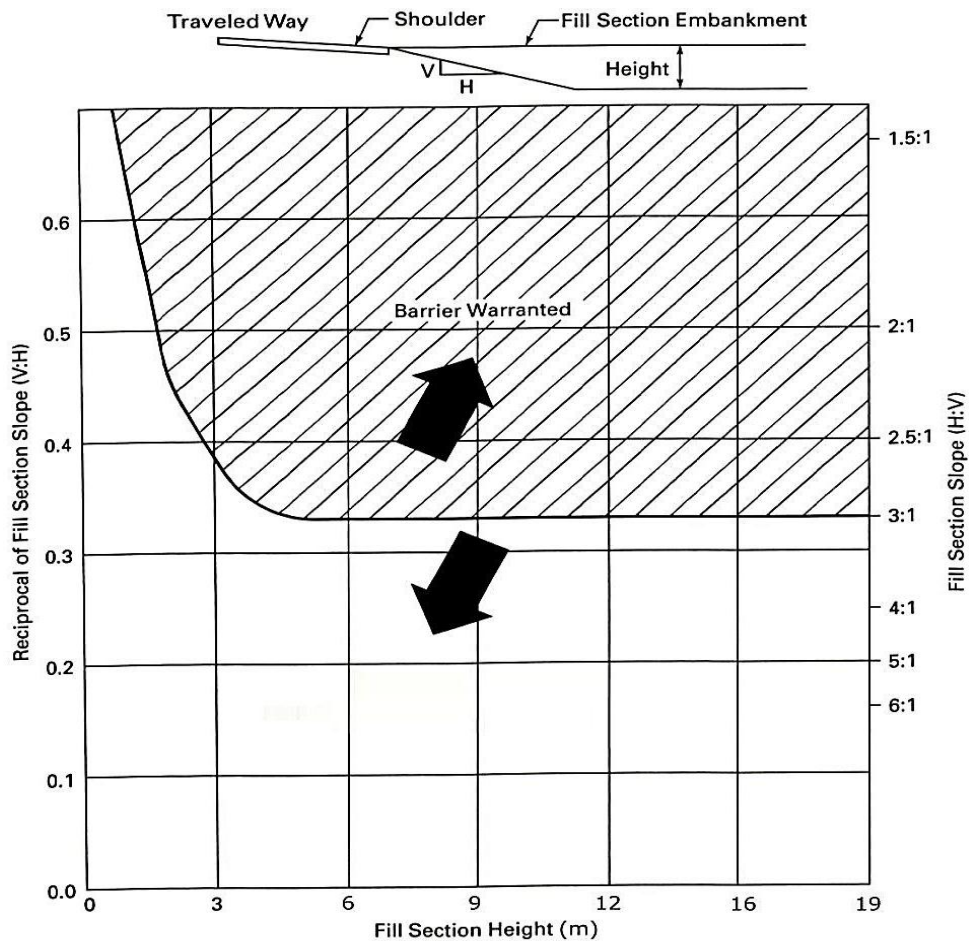


Figure 7-13/1: Comparative risk warrants for embankments [3, p.666]

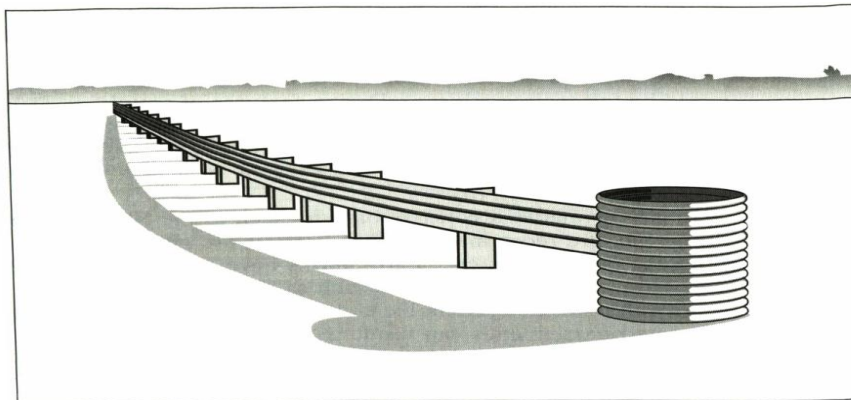


Figure 7-13/2: Sketch of an eccentric loader terminal [3, p.227]

Table 7-13/2: Recommended Shy Line Offset Values [3, p.226]

Design Speed, (km/hr.)	Shy Line Offset Values, (m)
130	3.7
120	3.2
110	2.8
100	2.4
90	2.2
80	2.0
70	1.7
60	1.4
50	1.1

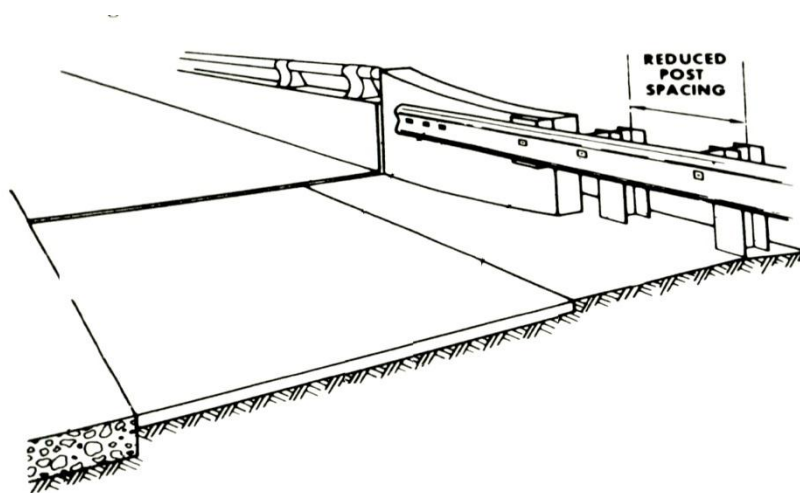


Figure 7-13/3: Barrier transition design (courtesy Federal Highway Administration [3, p.228]

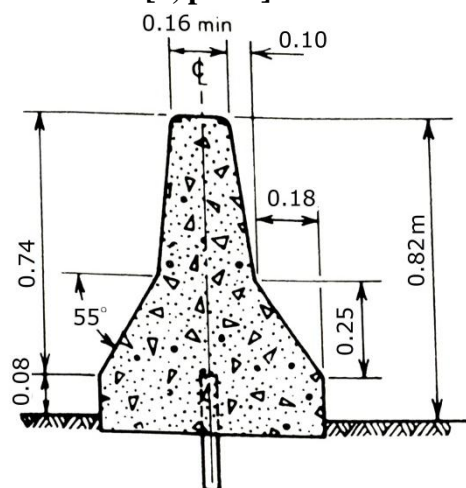


Figure 7-13/4: Concrete New Jersey median barrier (Courtesy Transportation Research Board [3, p.230]

Table 7-13/3: Design Features of Median Barriers That Have Performed Satisfactorily [3, p.229]

<i>System</i>	<i>Three-Strand Cable</i>	<i>W-Beam (weak post)</i>	<i>Weak Post Box Beam</i>	<i>Blocked-Out W-Beam (strong post)</i>
Deflection	3.50m	2.13 m	1.68m	1.22m
Post spacing	4.87m	3.80m	1.83m	1.90m
Post Type	S3 × 5.7 steel	S3 × 5.7 steel	S3 × 5.7 steel	W6 × 9
Beam type	1.9cm dia steel cables	Two steel W sections, 12 gage	0.20x0.20x0.06 steel tube	Two steel W sections

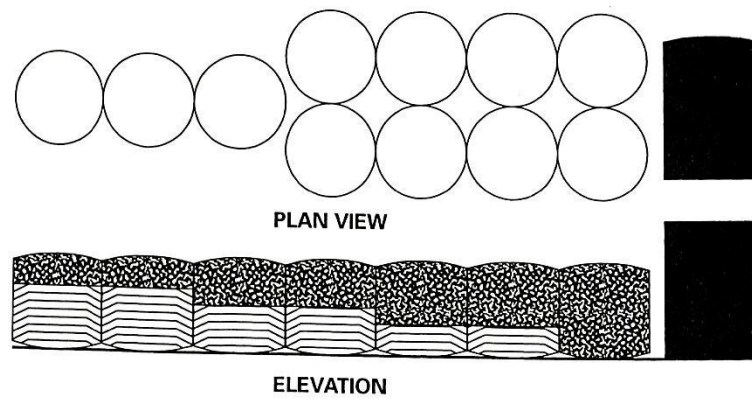


Figure 7-13/5: Configuration of Example sand-Filled Plastic Barrels. (Crash Cushions)
[3, p.232]

7-14 BUS STOP TURNOUTS

Bus stop turnouts serve to remove the bus from the traveled way, for deceleration standing, and acceleration. Preferably, the frequency of stops should not exceed 5 or 6 per kilometer [3,p.255]. The bus stops may be located on the (near side) or (far side) of intersection, or at the (mid-block) as shown in figure (7-14/1).

The recommended dimensions for bus bays to accommodate one standard 12-m bus are presented in figure (7-14/2). Where two or more buses are expected to use a bus stop simultaneously, 14m. should be added to the lengths shown for each additional bus.

The bus loading area can have a (parallel) or a (sawtooth) design. A recommended design of a sawtooth arrangement is shown in figure (7-14/3) [1,p.4-71].

The pavement areas of turnouts should contrast in color and texture with the traveled way, and pointed with (BUS LANE).

Passenger-loading areas should be provided with shelters, benches, route information, and sufficient lighting intensity. [1, p.4-71]

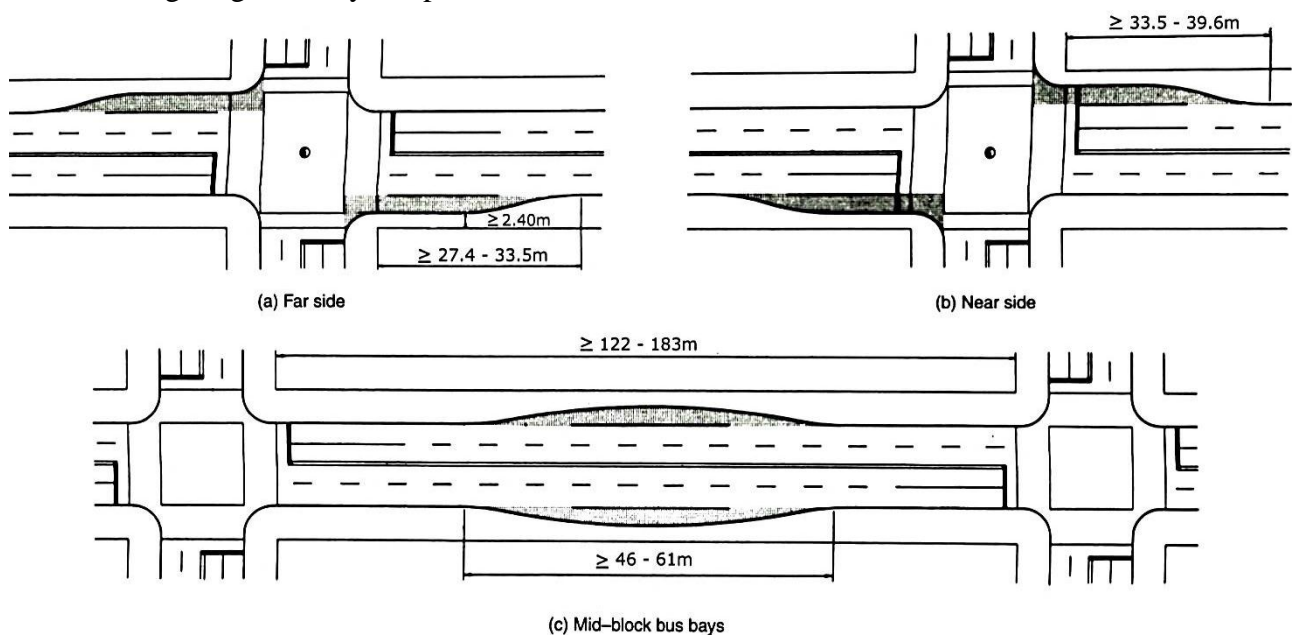


Figure 7-14/1: Typical Bus Stop Turnouts on Arterial Streets. [3, p.259]

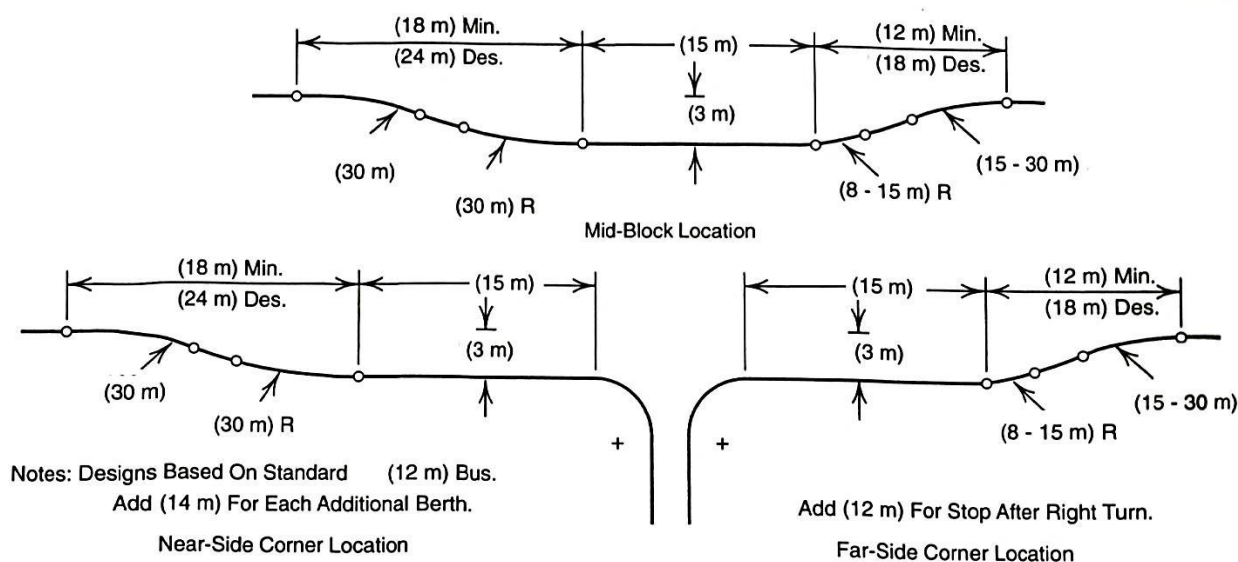


Figure 7-14/2: Recommended Dimensions for Bus Bays to Accommodate One Standard 12-m Bus [3,p260]

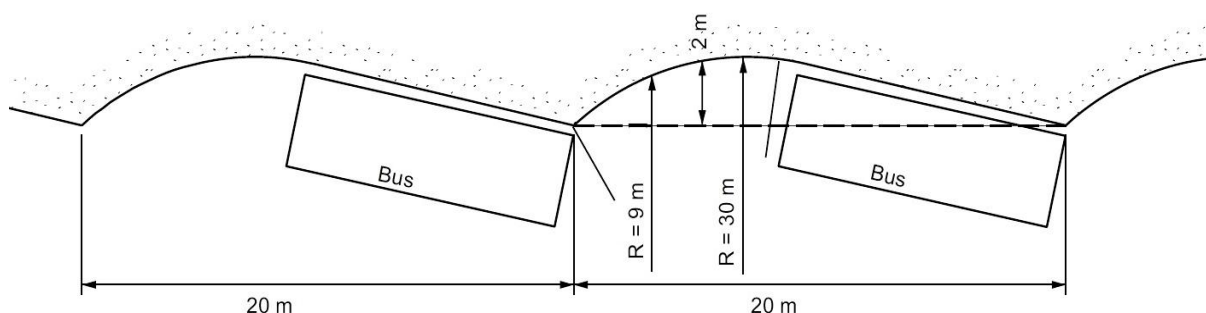


Figure 7-14/3: Sawtooth Bus Loading Area [1, p.4-71]

7-15 TYPICAL CROSS SECTIONS

7-15/1 TWO-LANE RURAL HIGHWAYS

The typical details of four types of two- lane rural highways are presented in figure (7-15/1) with roadway widths of 9.5, 11.0, 12.0 and 13.5 meters.

7-15/2 FOUR-LANE RURAL HIGHWAYS

The typical details of two types of four- lane rural highways are shown in figure (7-15/1), with roadway widths of 25.5 and 33.0 meters.

7-15/3 SIX-LANE RURAL HIGHWAYS

The typical details of two types of six- lane rural highways are illustrated in figure (7-15/1), with roadway widths of 33.0 and 40.5 meters.

A typical cross section of Iraq Expressway No. one with six lanes plus two standing lanes is presented in figure (7-15/2), including all details within the right- of- way of 260m.

7-15/4 FOUR-LANE URBAN STREETS

A typical cross section for a four-lane urban street with curbed pavements, median, and sidewalks is shown in figure (7-15/3) with a total width of 26.4 meters.

7-15/5 OVER PASSING BRIDGES

A typical cross section of an overpassing bridge used for Iraq expressway No. one project, is presented in figure (7-15/4), with two travel lanes and a total width of 14.0 meters.

7-15/6 INTERCHANGE RAMP (TURNING ROADWAY)

Typical details of an interchange ramp junction used in Iraq Expressway No. one, is shown in figure (7-15/5).

7-15/7 TUNNELS

The desired typical two-lane tunnel section is illustrated in figure (7-15/6), with a total width of 13.10 meters.

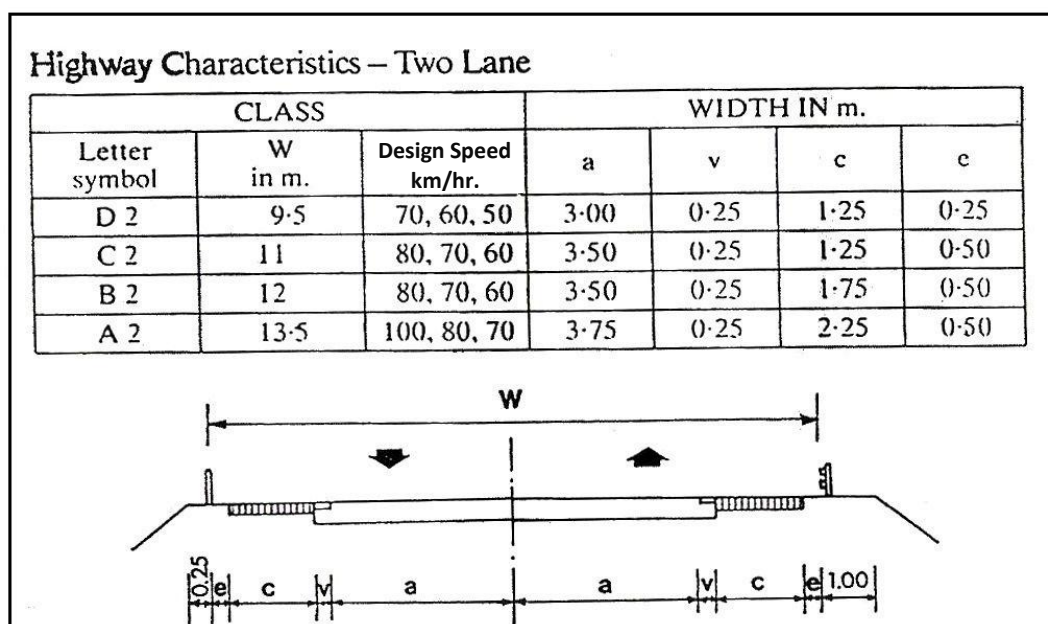
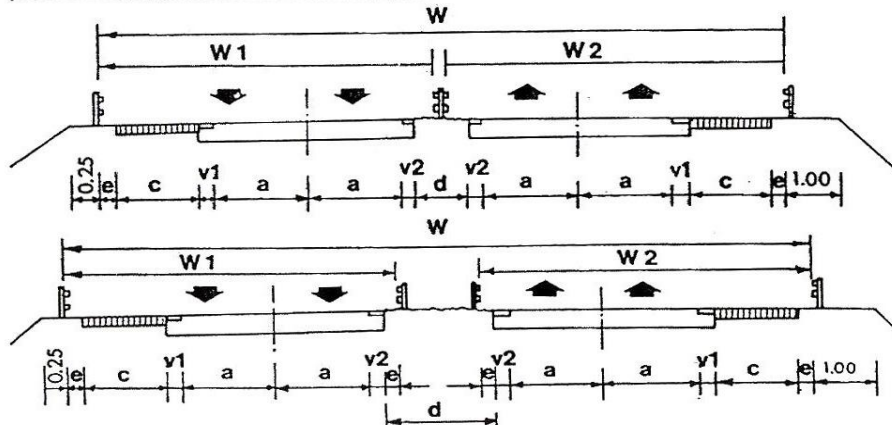


Figure 7-15/1: Typical Details of Rural Highways [4, p.II-2]

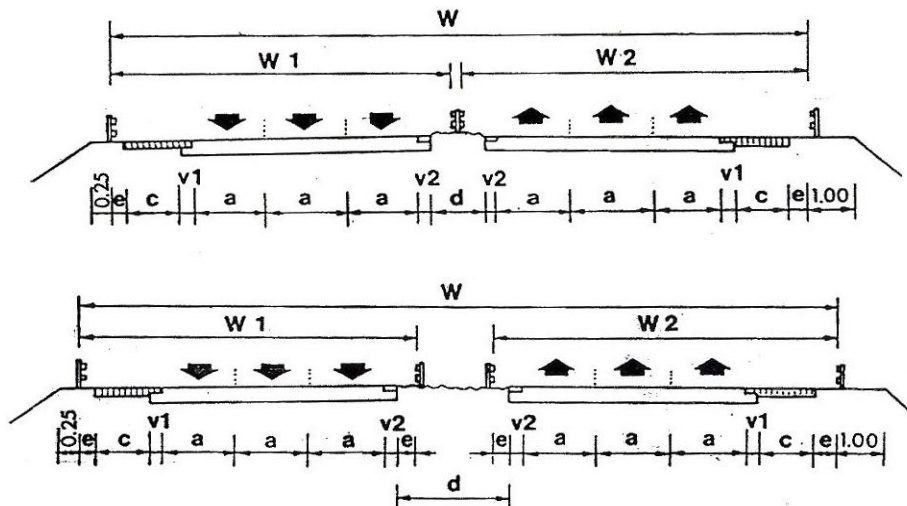
Highway Characteristics – Four Lane

CLASS			WIDTH IN m.					
Letter symbol	W in m.	Design Speed km/hr.	a	v1	v2	c	d	e
A 4	25.5	120, 100, 80	3.75	0.25	0.50	2.50	3.00	0.50
A 4	33.0	120, 100, 80	3.75	0.25	0.50	2.50	10.50	0.50



Highway Characteristics – Six Lane

CLASS			WIDTH IN m.					
Letter symbol	W in m.	Design Speed km/hr.	a	v1	v2	c	d	e
A 6	33	120, 100, 80	3.75	0.25	0.50	2.50	3.00	0.50
A 6	40.5	120, 100, 80	3.75	0.25	0.50	2.50	10.5	0.50



- W – total width of highway
- W1 W2 – overall highway width in one direction
- a – width of traffic lane
- c – width of paved shoulder
- e – width of unpaved shoulder
- d – width of median
- v1 – width of outer marginal strip
- v2 – width of inner marginal strip

Figure 7-15/1: Typical Details of Rural Highways [4, p.II-2] (Contd.)

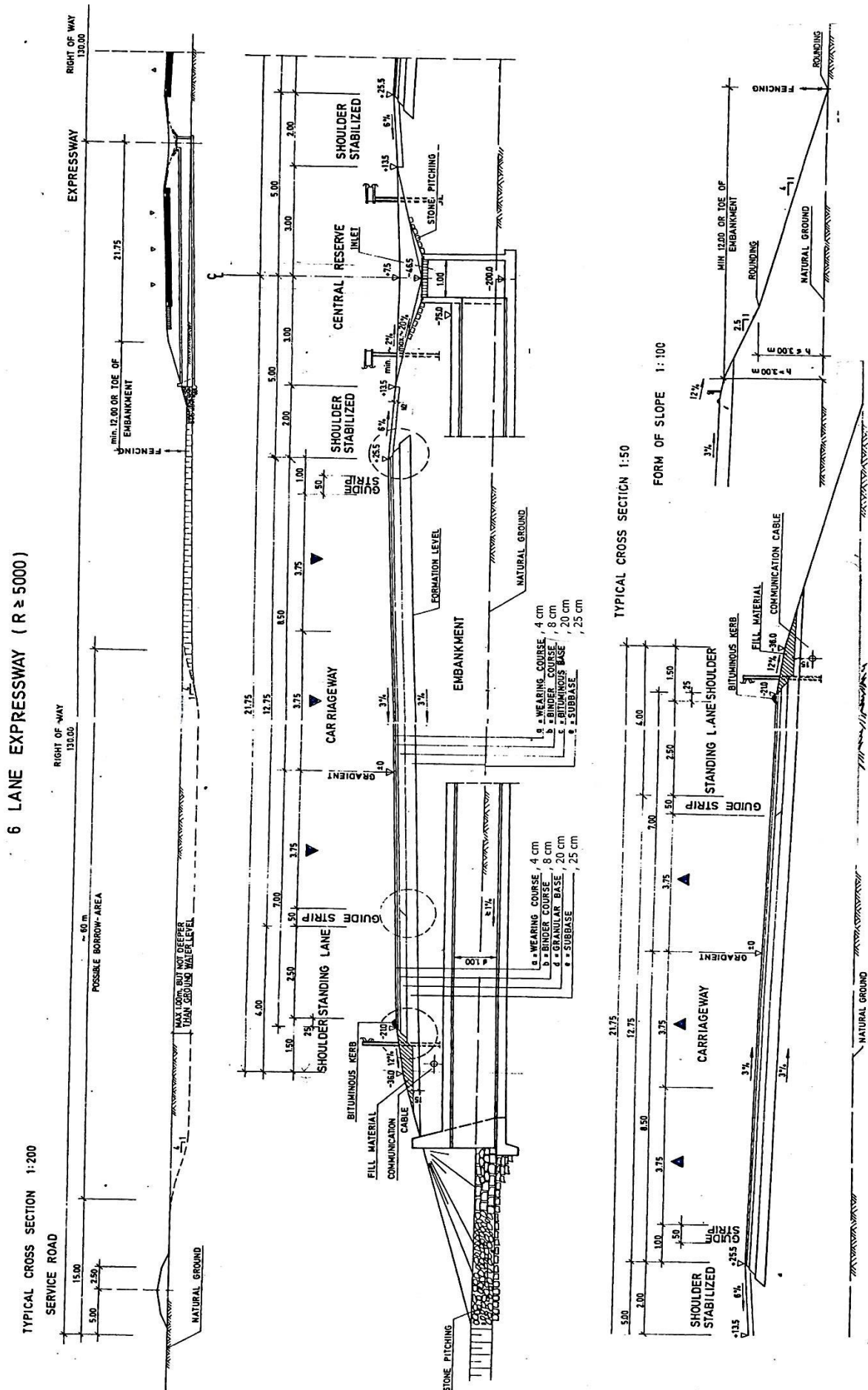
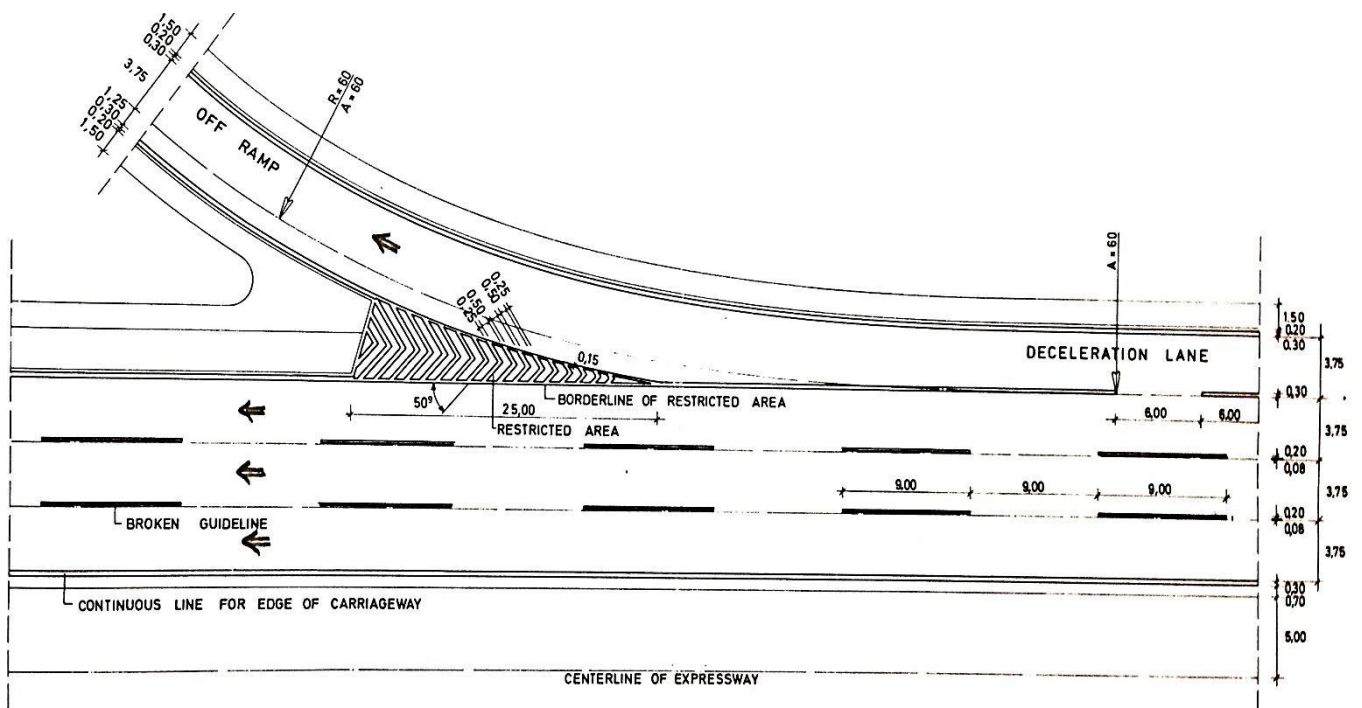
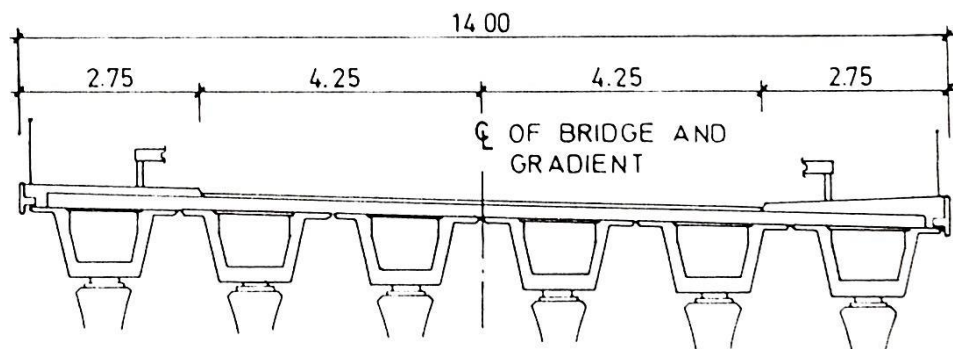
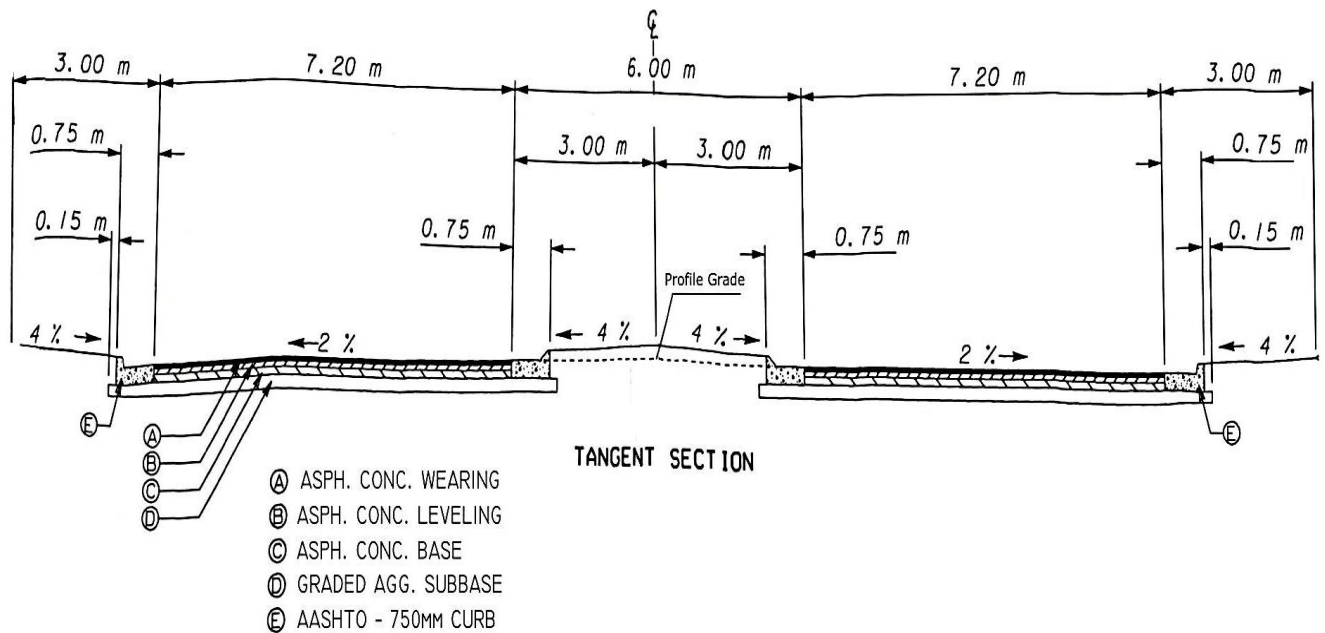


Figure 7-15/2: Iraq Expressway No. One/ Typical Cross Section [2, R/9A]



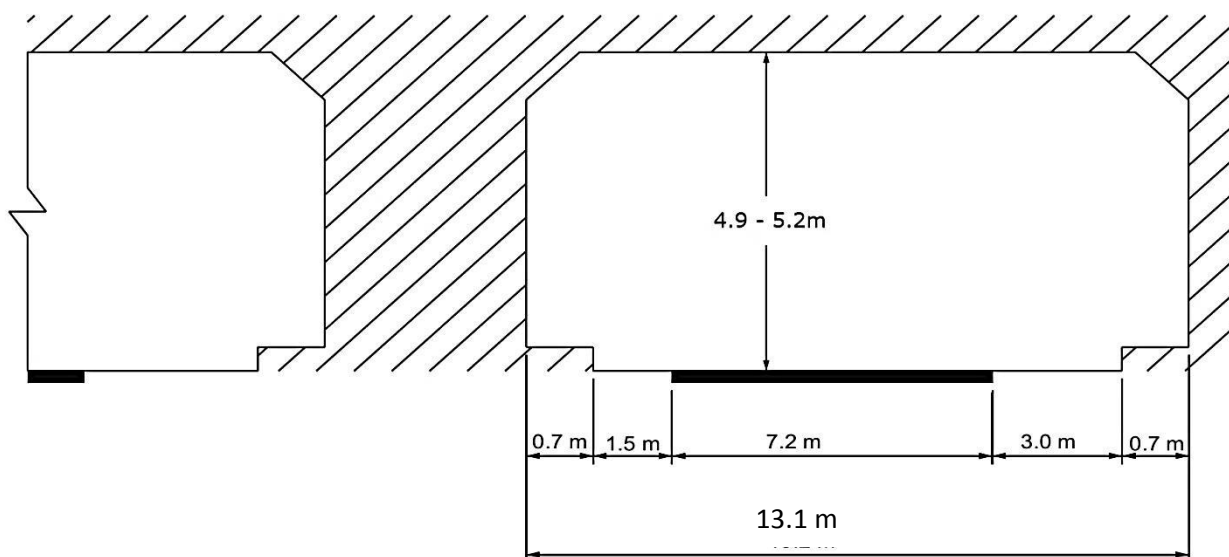


Figure 7-15/6: Typical Two-lane Tunnel Section [1, p.4-53]

7-16 UTILITIES

Adjustment of utility facilities is needed to preserve and protect the integrity and visual quality of the roadway, its maintenance efficiency, and the safety of traffic.

Utilities involve sanitary sewers, water supply lines, pipelines, power and communication lines, drainage and irrigation lines. Utility lines should be located to minimize need for later adjustments, to accommodate future highway improvements, and to permit servicing such lines with minimum interference with traffic.

Longitudinal installation should be located on uniform alignment as near as practical to the right-of-way line so as not to interfere with traffic operation and to preserve space for future highway and street improvements or other utility installations [1, p.3-174].

Aboveground utilities should not be placed within the highway clear zones. No poles should be located in the median of divided highways. In curbed section of urban areas, underground utilities should be located in the border areas between curb and sidewalks, at least 0.5m behind the face of curb.

On high-speed, heavily traveled roadways, the unobstructed clear roadside area, need to be extended to a minimum of 9.0 meters from edge of the driving lane.

7-17 REFERENCES

- [1] AASHTO, "A Policy on Geometric Design of Highways and Streets", American Association of State Highway and Transportation Officials, USA, 2011.
- [2] Dorsch Consult, "Iraq Expressway No. One Contract Drawings", Munich- Germany, 1976.
- [3] Wright, P.H. and Dixon, K.K, "Highway Engineering", John Wiley & Sons, USA, 2004.
- [4] SCRB, "Highway Design Manual", State Corporation of Roads and Bridges, Ministry of Construction and Housing, Iraq, 1982.

CHAPTER 8

GENERAL DESIGN CONSIDERATIONS FOR DIFFERENT HIGHWAY CLASSES AND PARKING FACILITIES

8-1 LOCAL RURAL ROADS

A local road services primarily to provide access to farms, residences, businesses, or other abutting properties. [1, p.5-1]

8-1/1 SELECTED DESIGN SPEED

The selected design speed is used to determine the various geometric design features of the roadway, for the design year, which is about 20 years into the future. The design features should be appropriate for environmental and terrain conditions and consistent with the selected design speed.

The minimum design speeds, as appropriate for traffic volumes of local rural roads, and types of terrain are shown in table (8-1/1) [1, p.5-2].

8-1/2 MAXIMUM GRADES

The maximum grades for local rural roads as a function of type of terrain and selected design speed are presented in table (8-1/2) [1.p.5-3].

Table 8-1/1: Minimum Design Speeds for Local Rural Roads [1, p.5-2]

Type of Terrain	Design Speed (km/hr.) for Specified Design Volume (veh/day)					
	under 50	50 to 250	250 to 400	400 to 1500	1500 to 2000	2000 and over
Level	50	50	60	80	80	80
Rolling	30	50	50	60	60	60
Mountainous	30	30	30	50	50	50

Table 8-1/2: Maximum Grades for Local Rural Roads [1, p.5-3]

Type of Terrain	Maximum Grade (%) for Specified Design Speed (km/hr.)								
	20	30	40	50	60	70	80	90	100
Level	9	8	7	7	7	7	6	6	5
Rolling	12	11	11	10	10	9	8	7	6
Mountainous	17	16	15	14	13	12	10	10	—

8-2 LOCAL URBAN STREETS

A local urban street is a public roadway for traffic, including vehicles, public transit, bicycles, and pedestrians. The main function of local street, is to provide access to adjacent property.

8-2/1 SELECTED DESIGN SPEED

The closely spaced intersections in urban areas, usually limit vehicular speeds. Therefore, design speed is not a major factor for design of local urban streets.

For consistency in geometric design elements, design speeds ranging from 30 to 50 km/hr. may be used depending on available right- of – way, terrain, pedestrian presence, adjacent development, and other area controls and development [1, p.5-11].

8-2/2 MAXIMUM AND MINIMUM GRADES

Grades for local urban streets should be as level as practical. Where grades of 4 percent or steeper are necessary, the drainage design may become critical.

Streets should have grades less than 8 percent in commercial and industrial areas, and less than 15 percent in residential areas. For proper drainage of curbed pavements, a minimum grade of 0.3 percent should be used for local urban streets. [1, p.5-12]

8-2/3 CUL-DE-SACS AND TURNAROUNDS

For local urban streets open at one end only, a special turning area at the closed end, need to be provided, with a minimum outside radius of 10 m in residential areas, and 15m in commercial and industrial areas. The different types of Cul –de –Sacs and dead- end local streets are illustrated in figure (8-2/1) [1, p.5-17].

Alleys with narrow widths, from 5 to 6m in residential areas, and up to 19m in industrial areas, usually provide access to the side or rear of individual land parcels. Dead – end alleys should include a turning area as shown in figure (8-2/2) [1, p.5-18].

8-2/4 MINIMUM LEVELS OF ILLUMINATION

Good visibility under day or night is one of the fundamental needs for safe driving. Properly designed and maintained street lighting will provide efficient night use of streets.

The suggested minimum levels of illumination, expressed in average maintained lux, are presented in table (8-2/1).

Illumination levels at intersections should be the sum of illumination levels on intersecting streets. Where only intersections are lighted, a graduate lighting transition from dark to light should be provided, so that drivers may have time to adapt their vision.

Uniformity of lighting can be represented by a uniformity ratio of the average –to- minimum lux values which is recommended to be: 6:1 for residential roadways, 3:1 for commercial roadways, 10:1 for residential walkways, and 4:1 for commercial walkways. The designer should minimize visual discomfort and impairment of driver and pedestrian vision due to glare.

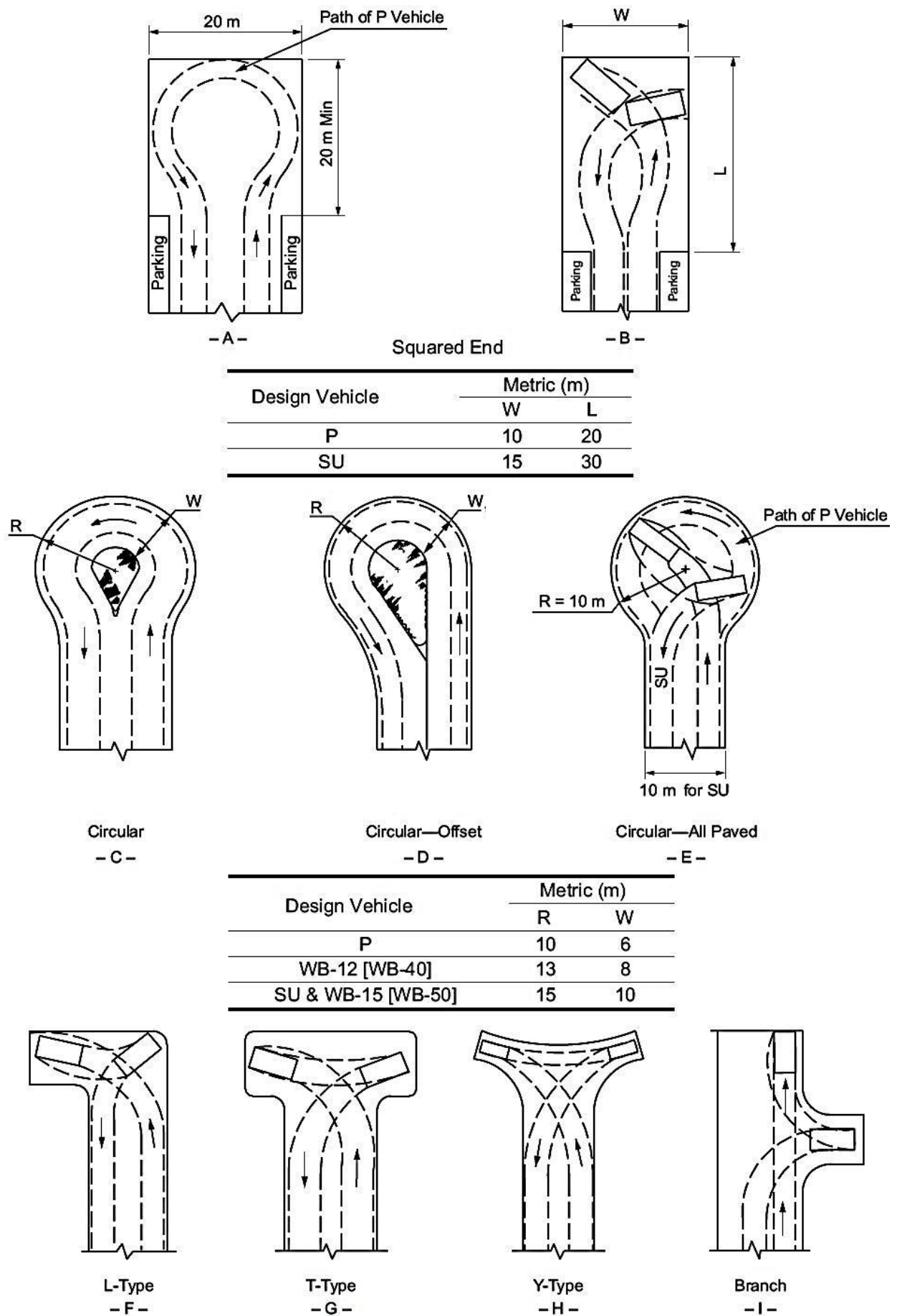


Figure 8-2/1: Types of Cul-de-Sacs and Dead-End Streets [1, p.5-17]

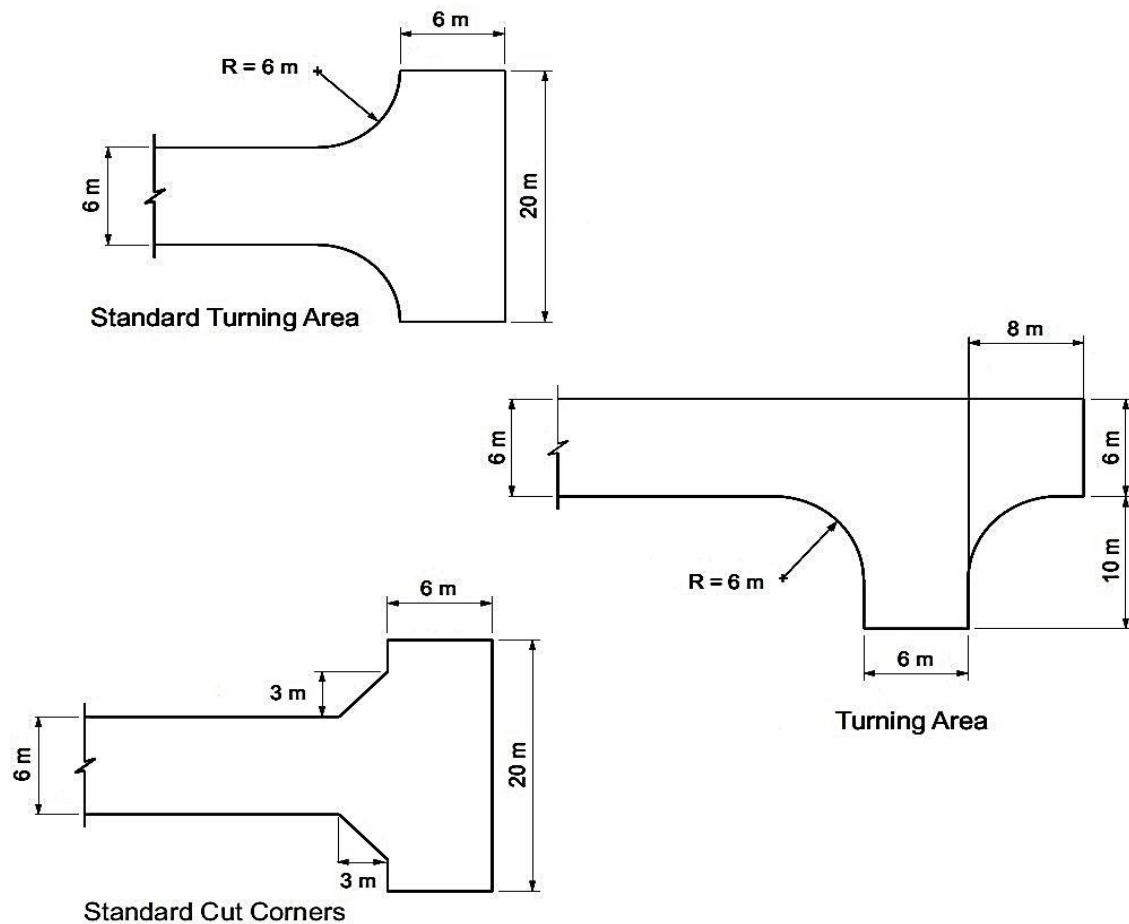


Figure 8-2/2: Alley Turnarounds [1, p.5-18]

Table 8-2/1: Minimum Illumination Levels [4, p.402]

Classification	Industrial/ Commercial (lux)	Residential (lux)
Local	9.7	4.3
Alleys	6.5	2.2
Sidewalks	9.7	2.2

8-3 RURAL COLLECTOR HIGHWAYS

Collector highways serve a dual function of collecting traffic for movement between arterials and local roads, and providing access to abutting properties. Collectors usually serve moderate traffic volumes.

8-3/1 SELECTED DESIGN SPEED

The minimum design speeds for rural collector roads, as a function of the type of terrain and the design traffic volumes are indicated in table (8-3/1) [1, p.6-2].

8-3/2 MAXIMUM GRADES

The maximum grades for rural collectors as a function of type of terrain and design speed are presented in table (8-3/2) [1, p.6-3].

8-4 URBAN COLLECTOR STREETS

A collector street is a public facility providing both traffic mobility and land access within residential, commercial, and industrial areas.

8-4/1 SELECTED DESIGN SPEED

For consistency in geometric design of collector streets, a design speed of 50 km/hr. or higher should be used, depending on available right- of – way, terrain, adjacent development, pedestrian presence, and other site controls.

8-4/2 MAXIMUM AND MINIMUM GRADES

Grades for urban collector streets should be as level as practical. Where adjacent sidewalks are present, a maximum grade of 5 percent is recommended. The maximum grades for urban collector streets are shown in table (8-4/1).

For drainage purposes, it is recommended that a grade of 0.5 percent or more is used, with a minimum grade of 0.3 percent.

Table 8-3/1: Minimum Design Speeds for Rural Collectors [1, p.6-2]

Type of Terrain	Design speed (km/h) for Specified Design Volume (veh/day)		
	0 to 400	400 to 2000	over 2000
Level	60	80	100
Rolling	50	60	80
Mountainous	30	50	60

Note: Where practical, design speeds higher than those shown should be considered

Table 8-3/2: Maximum Grades for Rural Collectors [1, p.6-3]

Type of Terrain	Maximum Grade (%) for Specified Design Speed (km/hr.)							
	30	40	50	60	70	80	90	100
Level	7	7	7	7	7	6	6	5
Rolling	10	10	9	8	8	7	7	6
Mountainou	12	11	10	10	10	9	9	8

Table 8-4/1: Maximum Grades for Urban Collectors [1, p.6-12]

Type of Terrain	Maximum Grade (%) for Specified Design Speed (km/hr.)							
	30	40	50	60	70	80	90	100
Level	9	9	9	9	8	7	7	6
Rolling	12	12	11	10	9	8	8	7
Mountainous	14	13	12	12	11	10	10	9

Note: Short lengths of grade in urban areas, such as grades less than 150 m in length, one-way downgrades, and grades on low-volume urban collectors may be up to 2% steeper than the grades shown above.

8-5 RURAL ARTERIALS

The principal and minor arterial systems, provide a high- speed, high- volume network, for travel between major points, covering a broad range of roadways from two – lane to multilane highways, to provide the desired safe and efficient operations.

8-5/1 SELECTED DESIGN SPEED

Rural arterials, (other than freeways), should be designed for speeds of 60 to 120 km/hr., depending on terrain, as shown in table (8-5/1)[1, p.7-2]

Table 8-5/1: Rural Arterials Selected Depending on Design Speed

Type of Terrain	Design Speed (km/hr.)
Level	100-120
Rolling	80-100
Mountainous	60-80

8-5/2 MAXIMUM GRADES

The recommended maximum grades for rural arterials are presented in table (8-5/2), for different types of terrain and desired design speeds.

8-5/3 TYPICAL MEDIANS ON DIVIDED ARTERIALS

Typical median configurations that may be used on divided arterials are shown in figure (8-5/1). Configurations A, B, F and G are appropriate for rural arterials, while configurations C, D and E, are more appropriate for urban arterials.

Table 8-5/2: Maximum Grades for Rural Arterials [1, p.7-4]

Type of Terrain	Maximum Grade (%) for Specified Design Speed (km/hr.)							
	60	70	80	90	100	110	120	130
Level	5	5	4	4	3	3	3	3
Rolling	6	6	5	5	4	4	4	4
Mountainous	8	7	7	6	6	5	5	5

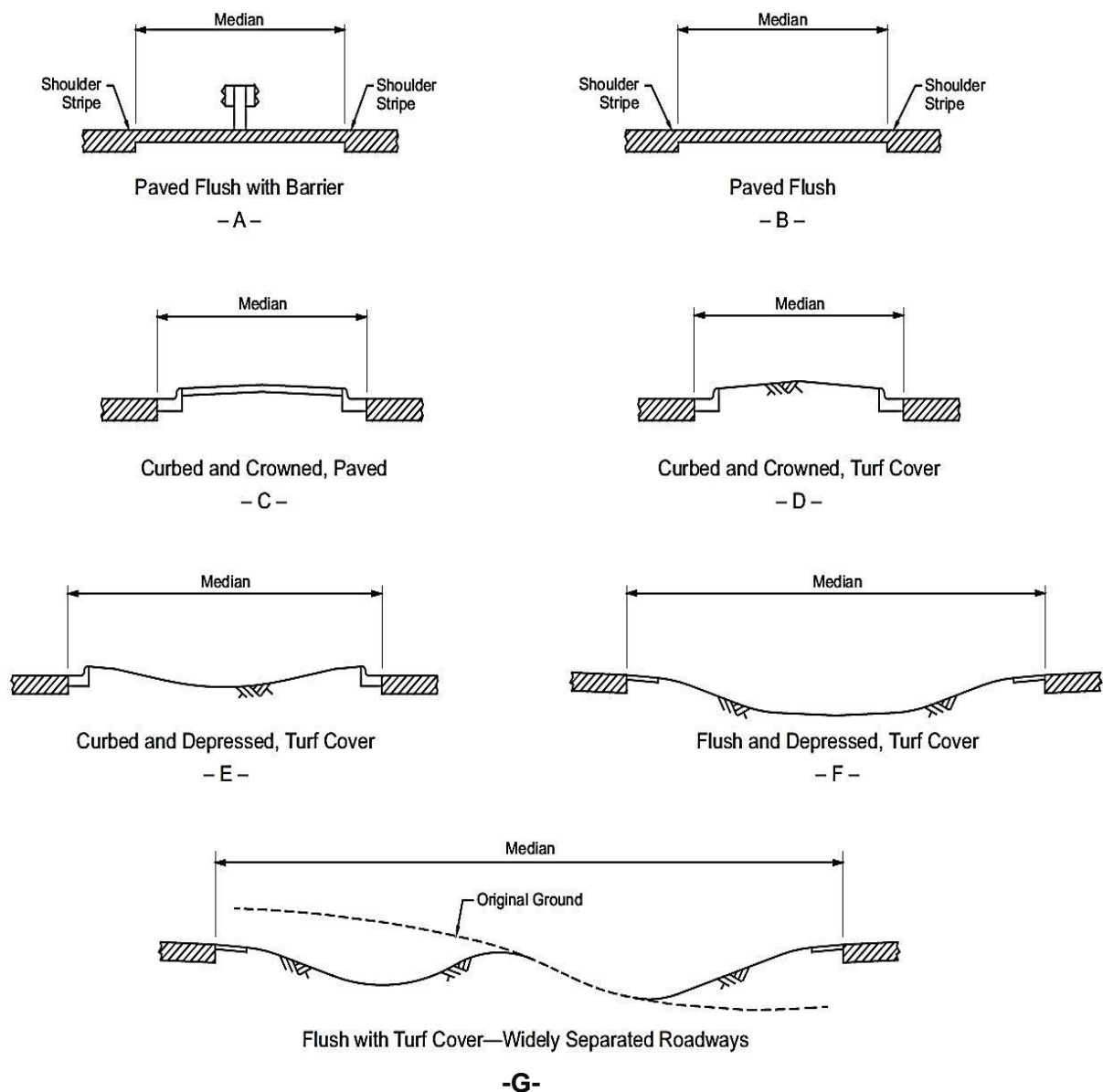


Figure 8-5/1: Typical Medians on Divided Arterials [1, p.7-20]

8-5/4 ATTAINING SUPERELEVATED CROSS SECTIONS FOR DIVIDED ARTERIALS

A divided arterial on a curve should be superelevated to ensure safe operation, and pleasing appearance.

The three general methods for attaining superelevation on divided arterials are presented in figure (8-5/2), including cross section rotation about median centerline, median edges, and roadway centerlines.

8-5/5 CROSS SECTIONAL ARRANGEMENTS FOR DIVIDED ARTERIALS

Sufficient width of right- of- way borders is desirable for divided arterials to provide cross-sectional elements of traveled ways, shoulders, wide median 9m or more, foreslopes, backslopes, drainage channels, clear zones, and frontage roads. A typical cross sectional arrangement on divided arterials with desirable widths is shown in figure (8-5/3).

The cross sectional arrangement with frontage roads is presented in figure (8-5/4). Frontage roads serve to collect and distribute local traffic to and from adjacent development, thus freeing the divided arterial from the disturbance of local operation.

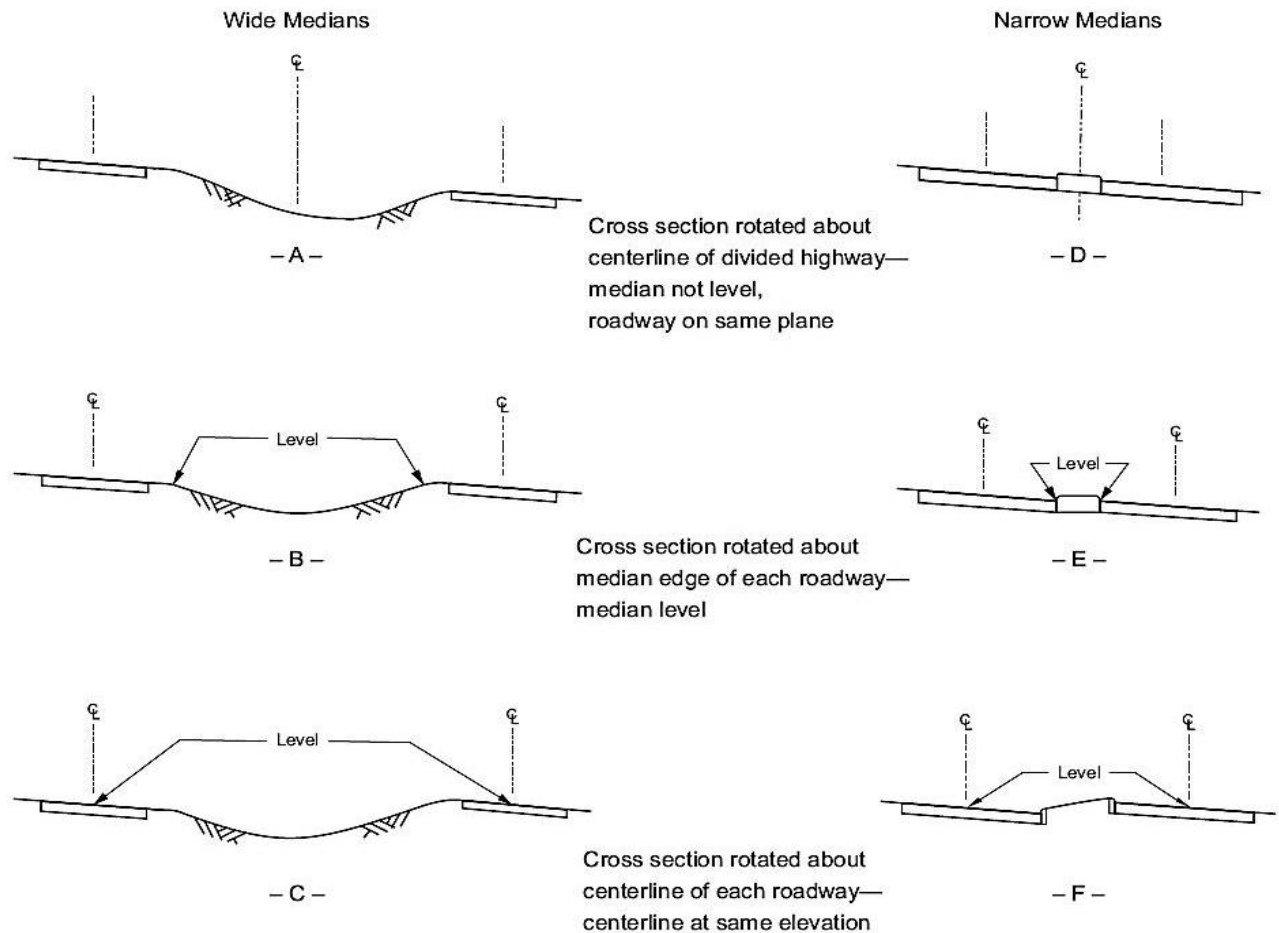


Figure 8-5/2: Methods of Attaining Superelevation on Divided Arterials [1, p.7-18]

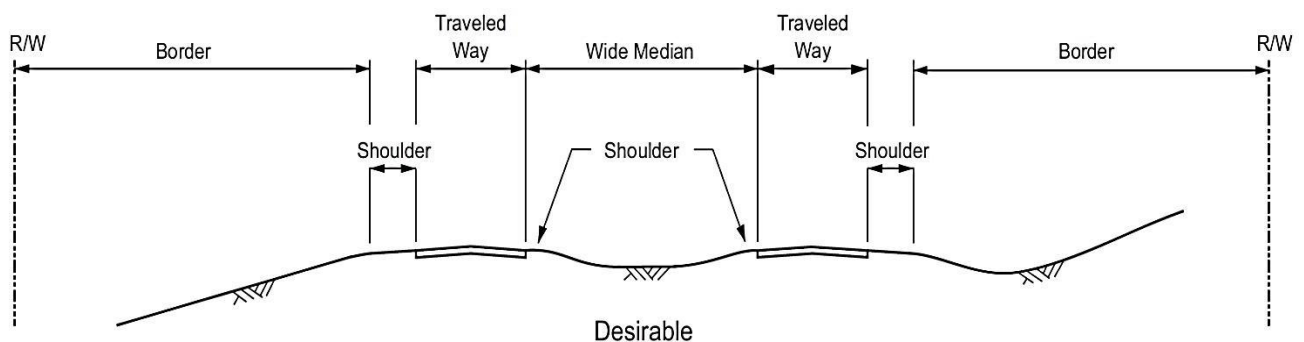


Figure 8-5/3: Cross Sectional Arrangement on Divided Arterials [1, p.7-21]

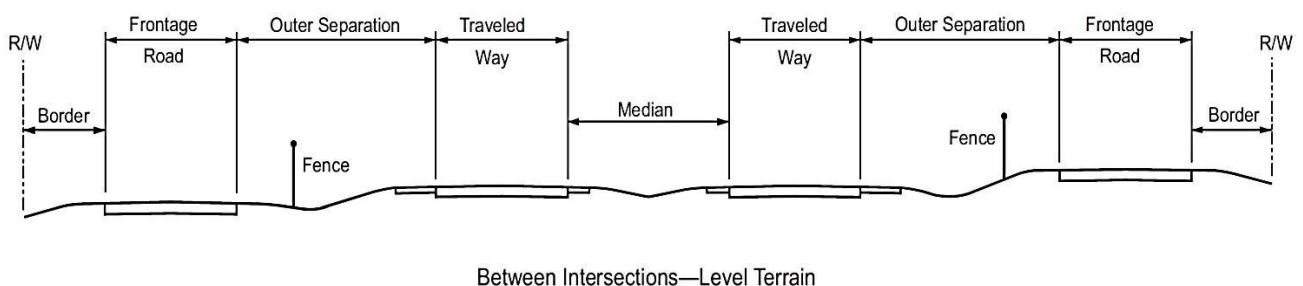


Figure 8-5/4: Cross Sectional Arrangement on Divided Arterials With Frontages Roads. [1, p.7-23]

8-6 URBAN ARTERIALS

The principal objective for an urban arterial is traffic mobility of all users, with some degree of access to abutting property. The urban arterial system serves the major activity centers of a metropolitan area, the highest traffic volume, and the longest trips.

8-6/1 SELECTED DESIGN SPEED

Design speeds for urban arterials generally range from 50 to 100 km/hr., with lower speeds in central business districts, and higher speeds in suburban areas.

8-6/2 MAXIMUM AND MINIMUM GRADES

It is always desirable to provide the flattest grades practical. The recommended maximum grades for urban arterials are presented in table (8-6/1) [1, p.7-29].

Where steep grades cannot be flattened, climbing lanes may be considered. In order to provide adequate longitudinal drainage in curbed sections of urban arterials, it is desirable to provide 0.5 percent gradient minimum.

8-7 FREEWAYS

Freeways are arterial highways with full control of access, intended to provide high levels of safety and efficiency for large volumes of traffic at high speeds.

Urban freeways are classified as: depressed, elevated, ground- level, or combination-type.

8-7/1 SELECTED DESIGN SPEED

For rural freeways, a design speed of 110 km/hr. should be used. On urban freeways, a design speed of 100 km/hr. or higher may be used. In mountainous terrain, a design speed of 80 to 100 km/hr. may be used for design of freeways. [1, p.8-1]

8-7/2 MAXIMUM GRADES

The maximum grades for freeways are presented in table (8-7/1).

8-7/3 TYPICAL RURAL FREEWAY MEDIANS

On rural freeway, median widths of 15 to 30 meters are common. In rolling terrain, a wide variable median with an average width of 45m or more may be used. In mountainous terrain, paved medians may be needed with widths of 3 to 9 meters flush with barrier figure (8-7/1) shows some typical rural medians.

Emergency crossovers may be placed in wide medians on rural freeways, where interchange spacing exceeds 8 km at intervals of 5 to 6.5 km, or as needed.

Maintenance or emergency crossovers should not be placed unless the median width is sufficient to accommodate the vehicle length (7.5m or more). [1, p.8-9]

Table 8-6/1: Maximum Grades for Urban Arterials [1, p.7-29]

Type of Terrain	Maximum Grade (%) for Specified Design Speed (km/hr.)					
	50	60	70	80	90	100
Level	8	7	6	6	5	5
Rolling	9	8	7	7	6	6
Mountainou	11	10	9	9	8	8

Table 8-7/1: Maximum Grades for Rural and Urban Freeways [1, p.8-4]

Type of Terrain	Design Speeds (km/hr.)					
	80	90	100	110	120	130
	Grades (Yo) ^a					
Level	4	4	3	3	3	3
Rolling	5	5	4	4	4	4
Mountainous	6	6	6	5	—	—

^a Grades 1% steeper than the value shown may be provided in urban areas with right-of-way constraints or where needed in mountainous terrain.

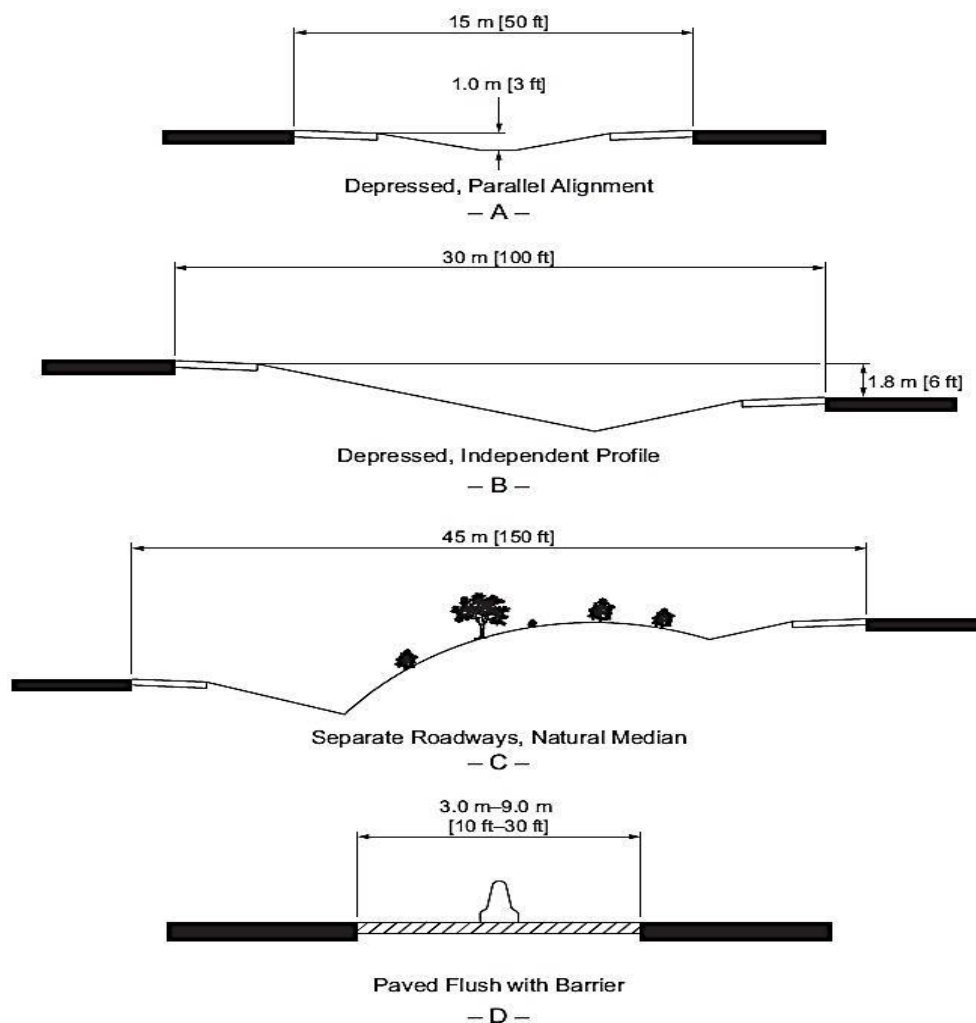


Figure 8-7/1: Typical Rural Medians [1, p.8-8]

8-7/4 CROSS SECTIONS FOR DEPRESSED URBAN FREEWAYS

The roadways of depressed urban freeways are located at a minimum depth of 5.20m in addition to the expected structural depth, below the surface of adjacent streets.

Figure (8-7/2) shows a typical cross section for depressed freeways, with a 3.0 to 6.6m median width. Depressed freeway with walled cross section is shown in figure (8-7/3).

8-7/5 CROSS SECTIONS FOR ELEVATED URBAN FREEWAYS

An elevated freeway may be constructed on either a viaduct or an embankment. Typical cross sections for elevated freeways are shown in figures (8-7/4) and (8-7/5). The double-deck design figure (8-7/4 B) may be used to narrow rights – of- way, particularly where few ramps are needed, in order to convert the two- way one level structure, to a two- level structure.

Elevated freeways may be constructed on earth embankments, and the outer separation may permit the use of earth slopes at locations without ramps, and retaining walls at ramp, as illustrated in figure (8-7/6).

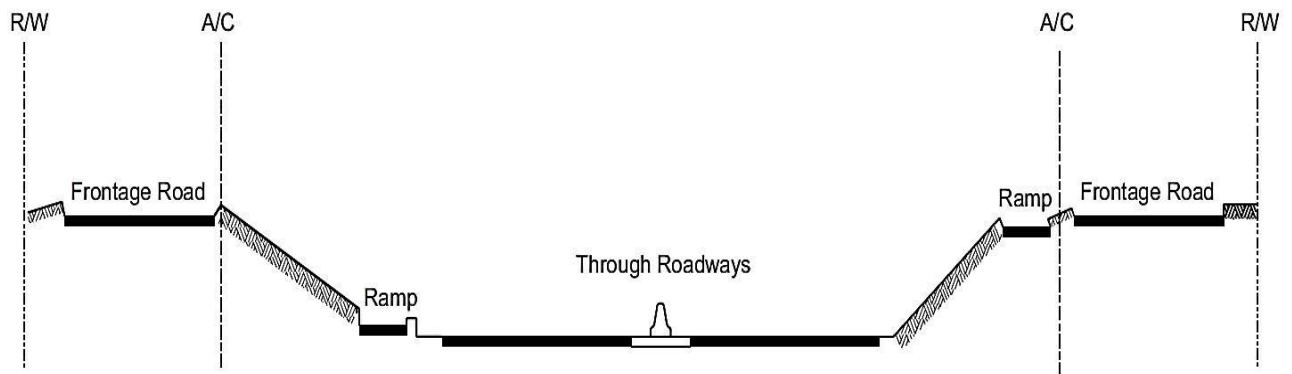


Figure 8-7/2: Typical Cross Section for Depressed Freeways [1, p.8-13]

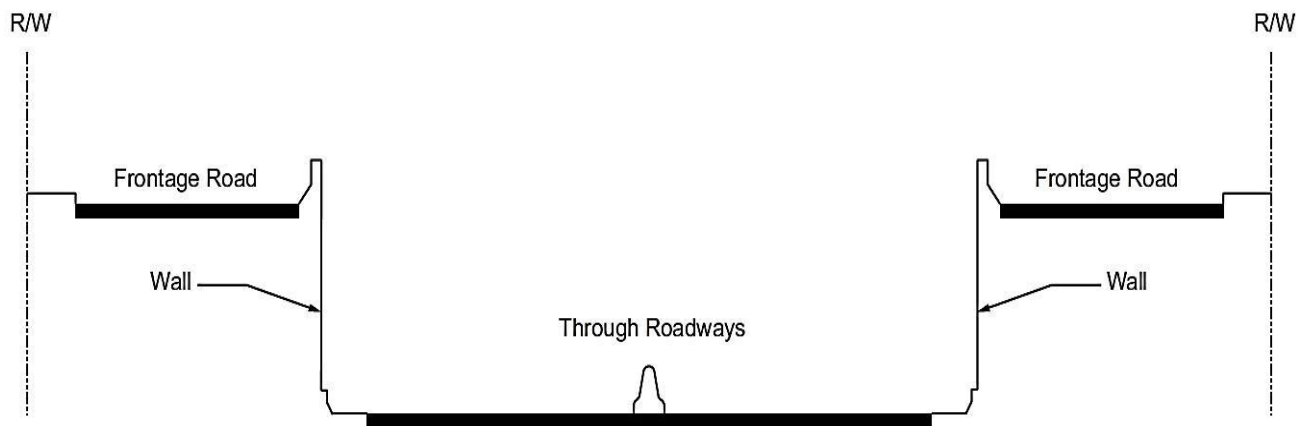


Figure 8-7/3: Cross Section with Retaining Walls on Depressed Freeways without Ramps [1, p.8-14]

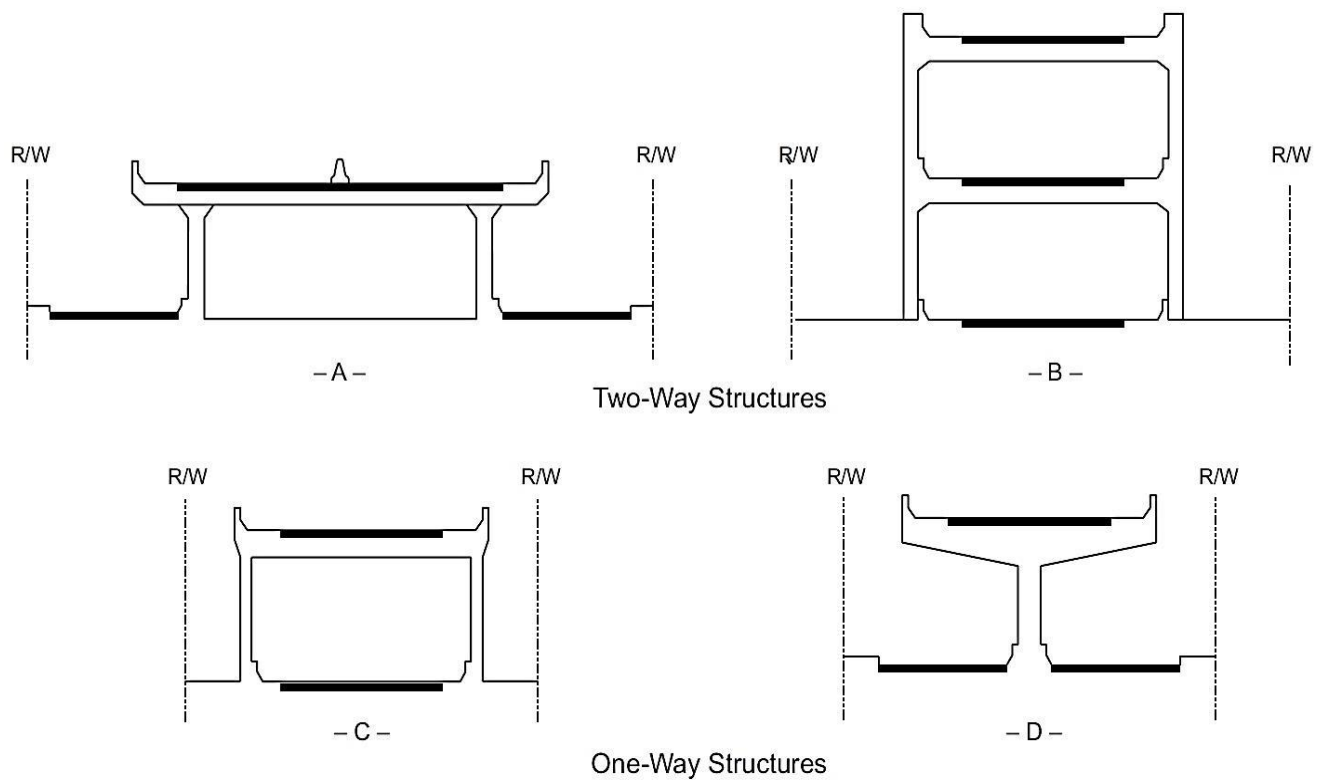


Figure 8-7/4: Typical Cross Section for Elevated Freeways on Structures without Ramps [1, p.8-18]

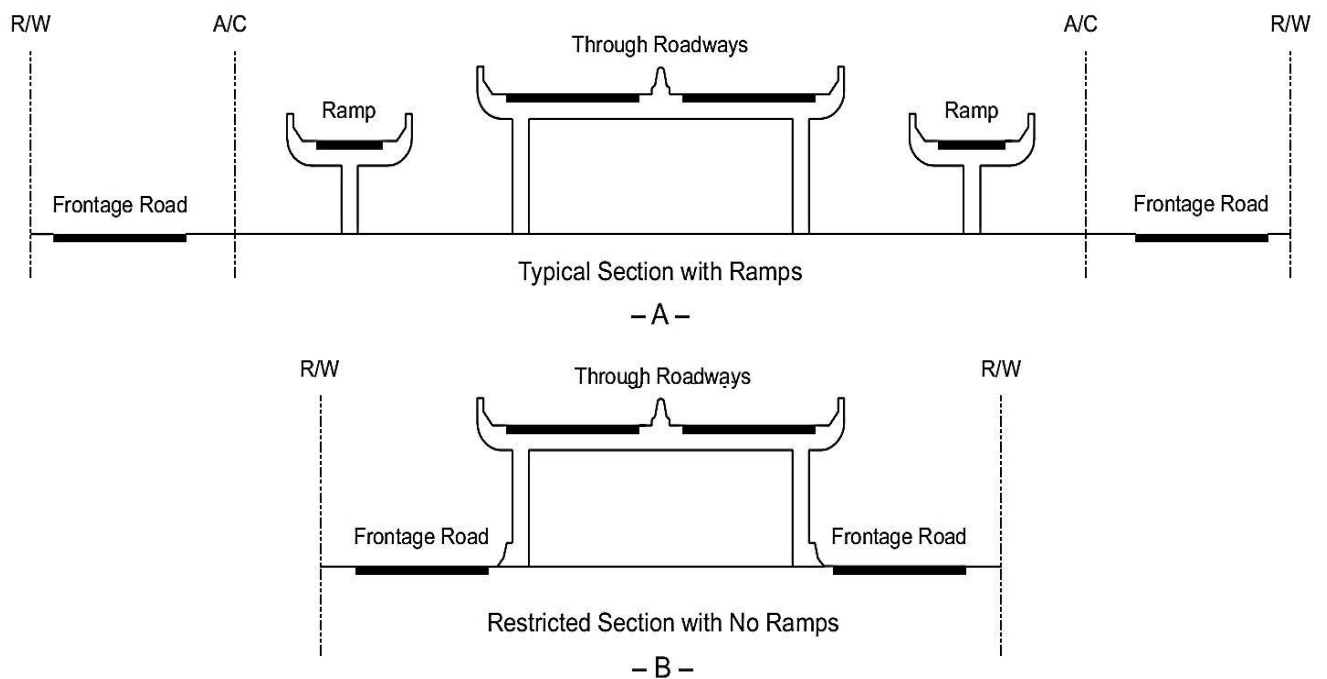


Figure 8-7/5: Typical and Restricted Cross Sections for Elevated Freeways on Structures with Frontage Roads [1, p.8-19]

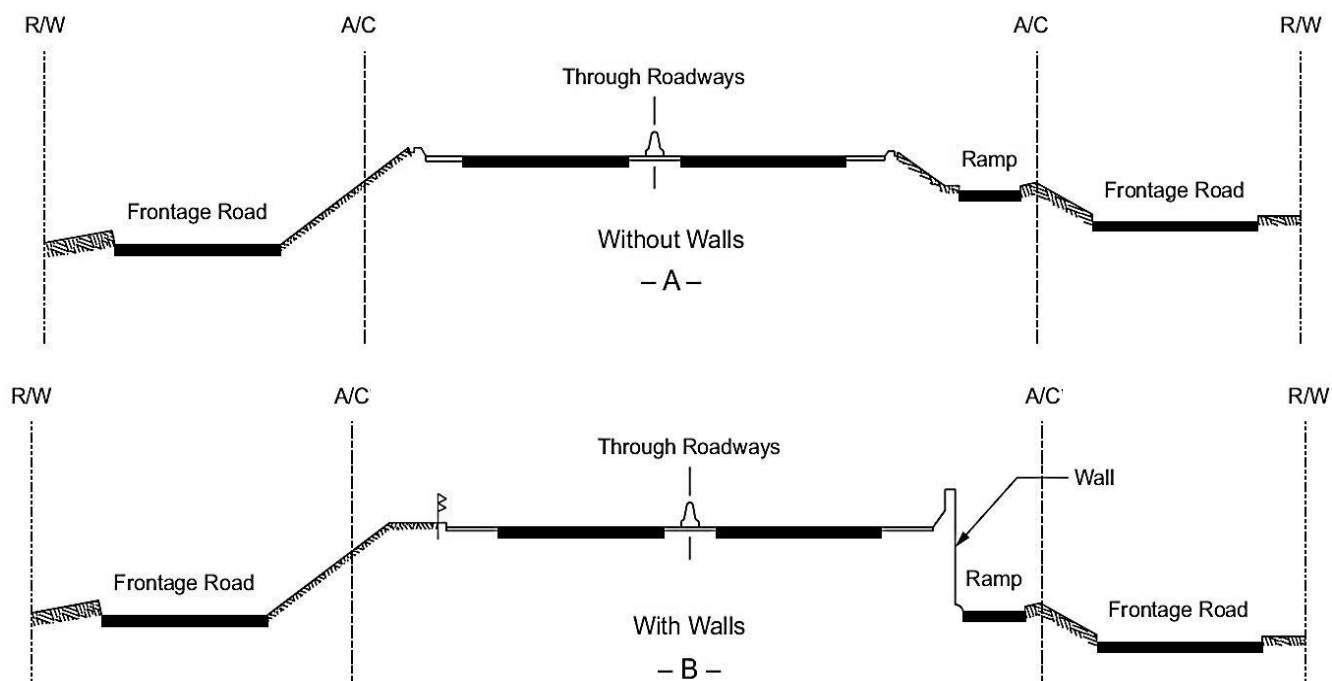


Figure 8-7/6: Typical and Restricted Cross Sections for Elevated Freeways on Embankment [1, p.8-20]

8-7/6 CROSS SECTIONS FOR GROUND- LEVEL URBAN FREEWAYS

Ground- level freeway are used in flat terrain, and along water courses, where right- of – way is not expensive. Crossroads need to change their profile, over or under the freeway.

Typical cross sections for ground- level freeways are shown in figure (8-7/7), with and without frontage roads.

Restricted cross sections for ground- level freeways are illustrated in figure (8-7/8). With restricted cross sections, both the narrow median and outer separation should be paved.

Where there is no fixed- source lighting, a glare screen may be desirable in the outer separation.

8-7/7 CROSS SECTIONS FOR COMBINATION-TYPE URBAN FREEWAYS

Urban freeway may incorporate some combination of depressed, elevated, or ground- level designs, which result in variations of cross section profiles.

These special designs usually apply to relatively short lengths of roadway to meet specific conditions, as illustrated in figure (8-7/9).

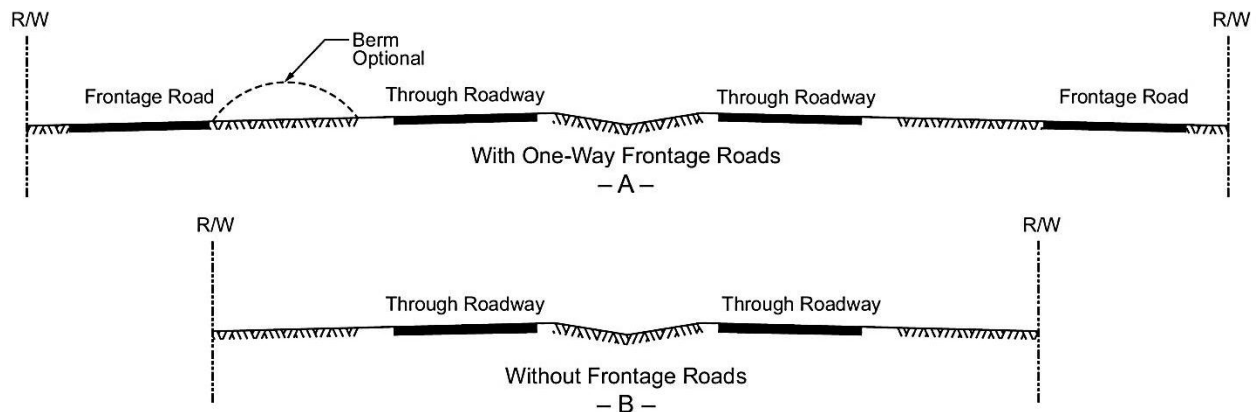


Figure 8-7/7: Typical Cross Section for Ground-Level Freeway [1, p.8-23]

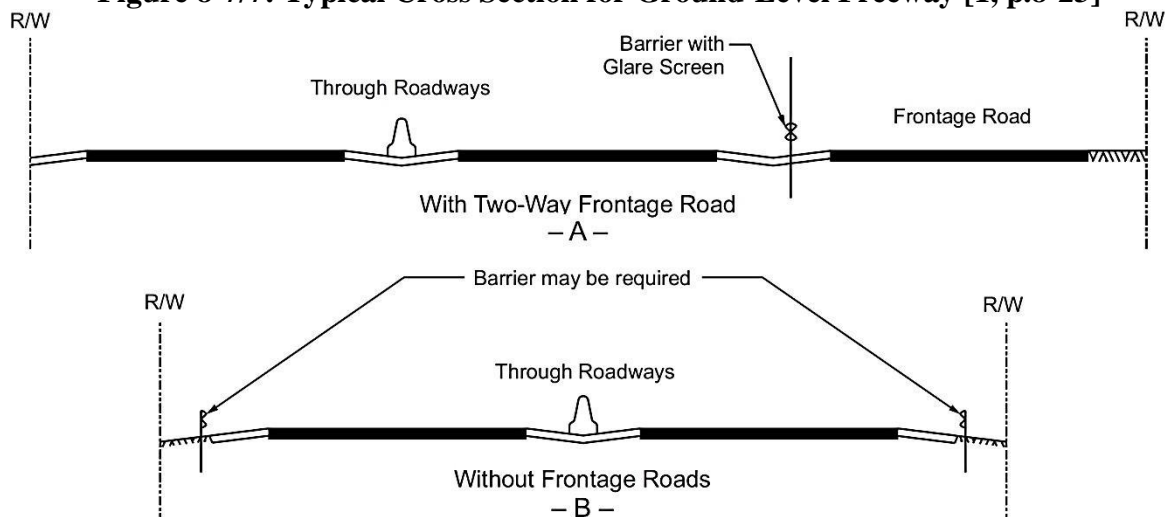


Figure 8-7/8: Restricted Cross Section for Ground-Level Freeways [1, p.8-23]

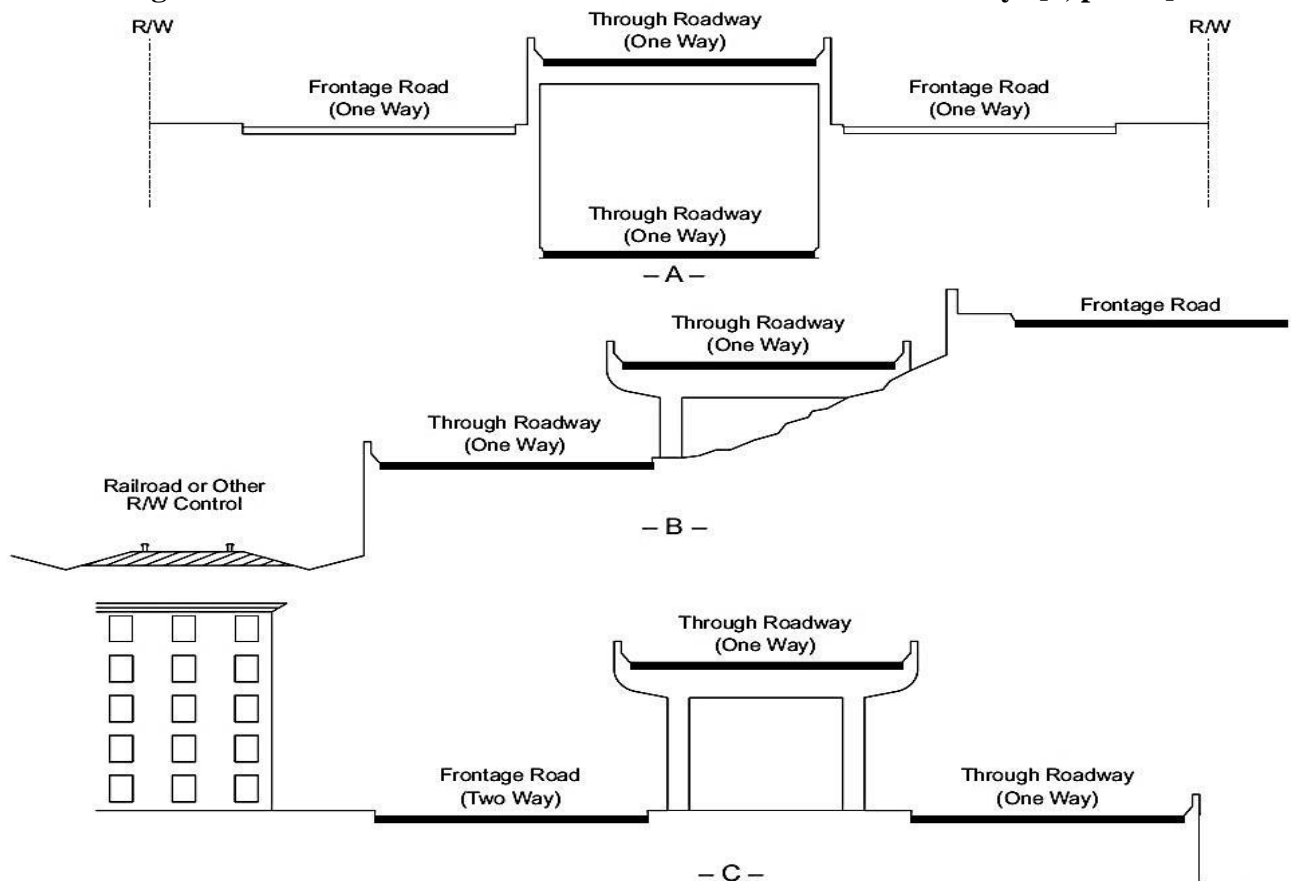


Figure 8-7/9: Cross Section-Control Combination-Type Freeway [1, p.8-26]

8-8 OFF-STREET PARKING FACILITIES

Parking problems have been aggravated in main cities, especially in business districts, commercial areas, and college complexes. In many locations curb parking has been eliminated to improve traffic flow for streets.

On – street parking has been discussed in chapter seven (7-12). Off- street parking may include parking lots, and garages.

8-8/1 LOCATION OF PARKING LOTS AND GARAGES

Improper location of a parking lot or garage is likely to have them of limited use, even if located very close to parking demand. Parking lots and garages should be located near major arterials, and desirably accessed by right- hand turning movements. Garages should have access to two or more streets.

Parking for short- term parker should be located within 30m of the building entrance. Long- term parkers may walk a maximum distance of 600m. Where parkers are carrying baggage, a maximum walking distance should not exceed 300m. [2, p.249]

Safe and convenient pedestrian access to parking facilities should be provided, and conflicts with vehicular traffic should be avoided.

8-8/2 GENERAL DESIGN CRITERIA OF PARKING GARAGES

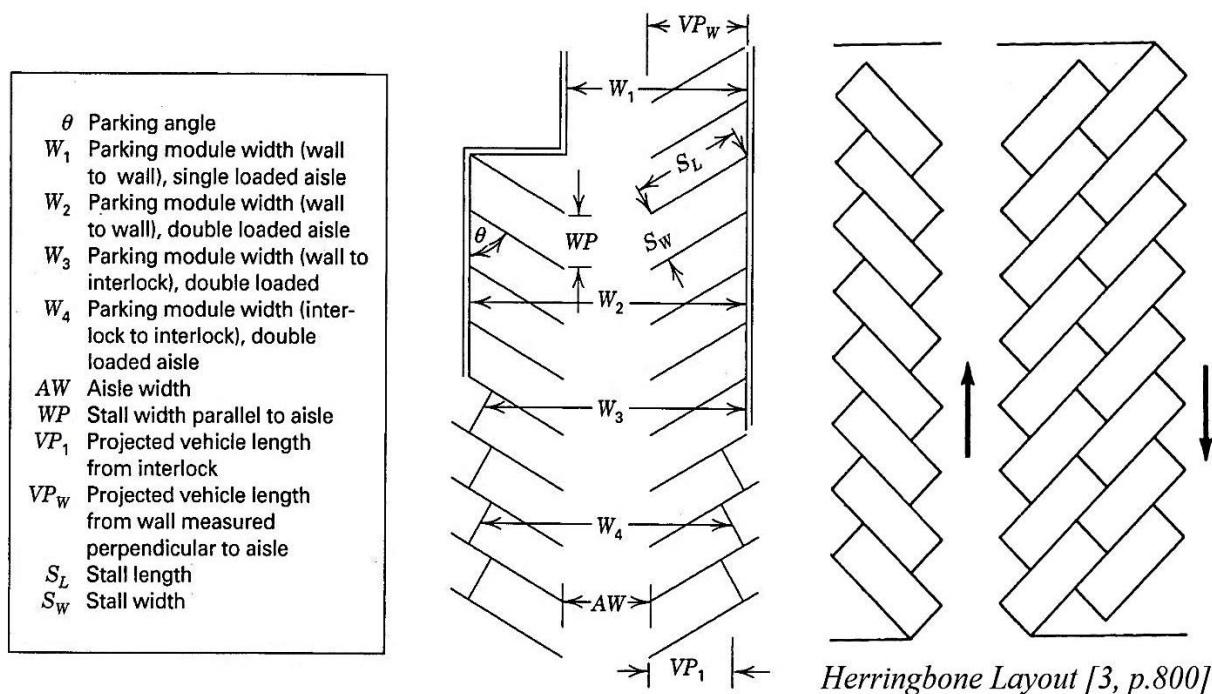
Special design criteria for parking garages may include the following:

- Single entrances and exits, with multiple lanes are preferable.
- Entrances and exits should be located away from street intersections to avoid congestion.
- Lane widths of 3.6 to 4.3m are typically used for entrances and exits.
- One entrance lane for every 300 to 500 spaces is typically provided.
- One exit lane should be provided for every 200 to 250 spaces depending on average parking duration.
- Column spacing need to be equal to the unit parking depth (module width), if possible.
- A clear ceiling height of 2.3 m should be provided.
- Vehicular access between floors should be provided by sloped floors or ramps.
- Floor slopes should not exceed 3 to 4 percent for self- park facilities, and 10 percent for attendant park facilities.
- Ramp slopes should preferably not exceed 10 percent.
- One – way curved ramps should be at least 3.6 to 4.0 m in width.
- A minimum radius of curvature of 10m is recommended, measured at the face of the outer curb of the inside lane.
- The capacity of an entrance lane ranges up to 700 vph without a ticket dispenser and 400 vph with an automatic ticket dispenser. [2,p.251]

8-8/3 PARKING STALL LAYOUT AT VARIOUS ANGLES

A parking lot or garage should preferably be rectangular. Right- angled parking tends to require slightly less area per parked car than other configurations, and generally used for two- way movements.

Parking stalls oriented at angles of 45° to 75° to the aisles are often used with one- way circulation. Parking stall layout dimensions for stalls of (2.8 x 5.5m), arranged at various angles are given in figure (8-8/1).



Parking Angle and Projected Vehicle Length		Stall Widths		Aisle Widths	Module Widths			
		Sw	WP	AW	W ₁	W ₂	W ₃	W ₄
Large-car vehicle 1,956 mm by 5,461 mm								
90°	VP _w – 5.61 VP ₁ – 5.61	2.59	2.59	7.33	12.94	18.56	18.56	18.56
75°	VP _w – 5.93 VP ₁ – 5.68	2.59	2.68	6.45	12.38	18.31	18.06	17.80
60°	VP _w – 5.84 VP ₁ – 5.35	2.59	2.99	4.29	10.13	15.97	15.48	14.99
45°	VP _w – 5.25 VP ₁ – 4.55	2.59	3.66	3.35	8.60	13.84	13.15	12.46
Small-car vehicle 1,676 mm by 4,445 mm								
90°	VP _w – 4.60 VP ₁ – 4.60	2.29	2.29	6.79	11.38	15.98	15.98	15.98

*Small car spaces normally are considered only for 90° layouts.

Figure 8-8/1: Parking Stall Layout Elements Expressed in Meters [2, p.250]

8-9 REFERENCES

- [1] AASHTO, "*A Policy on Geometric Design of Highways and Streets*", American Association of State Highway and Transportation Officials, USA, 2011.
- [2] Wright, P.H. and Dixon, K.K., "*Highway Engineering*", John Wiley & Sons, USA, 2004.
- [3] Garber, N.J. and Hoel, L.A. "*Traffic & Highway Engineering*", Cengage Learning, USA, 2009.
- [4] AASHTO, "*A Policy on Geometric Design of Highways and Streets*", American Association of State Highway and Transportation Officials, USA, 2004.

CHAPTER 9

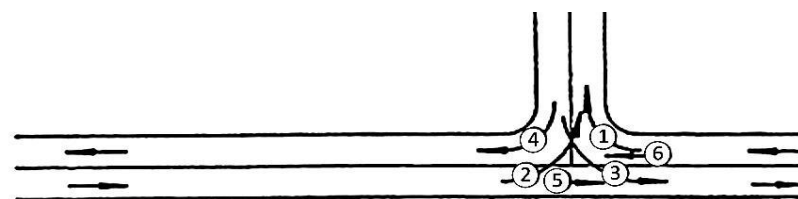
AT-GRADE INTERSECTIONS

An intersection is defined as the general area where two or more highways join or cross, including the roadway and roadside facilities for traffic movements within the area.

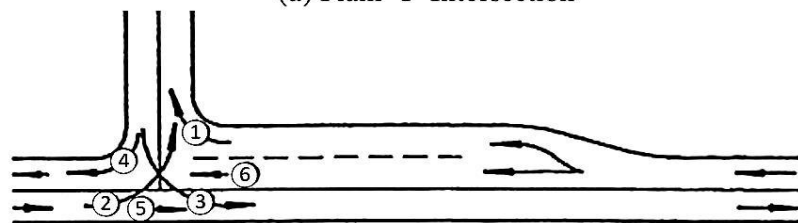
9-1 BASIC TYPES OF AT-GRADE INTERSECTIONS

9-1/1 THREE-LEG

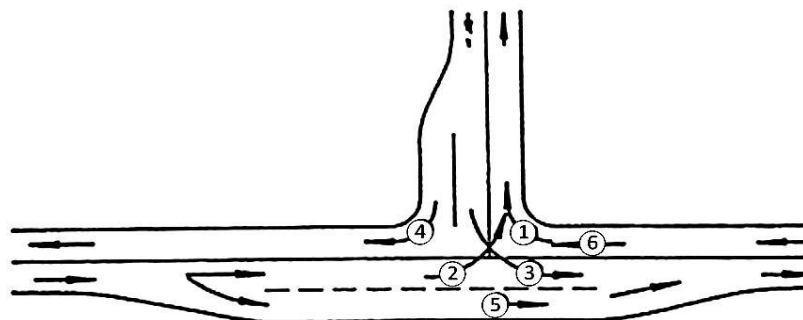
Figure (9-1/1) below shows different types of T intersections ranging from the simplest shown in figure (9-1/1a) to a channelized one with divisional islands and turning roadways shown in figure (9-1/1d).



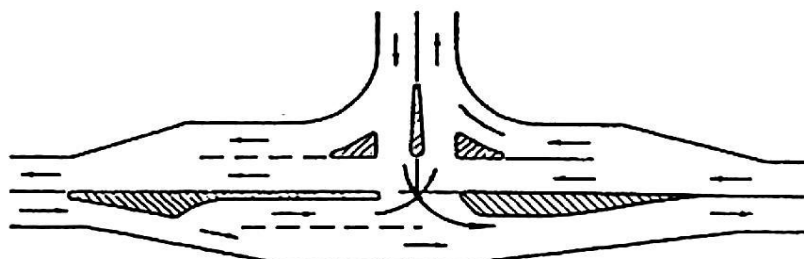
(a) Plain 'T' Intersection



(b) 'T' Intersection (With Right Turn Lane)



(c) 'T' Intersection (With Right-Hand Passing Lane)



(d) 'T' Intersection (With Divisional Island and Turning Roadways)

Figure 9-1/1: Examples of T- Intersection, [2, p.270]

9-1/2 FOUR-LEG

Figure (9-1/2) shows examples of four-leg intersection.

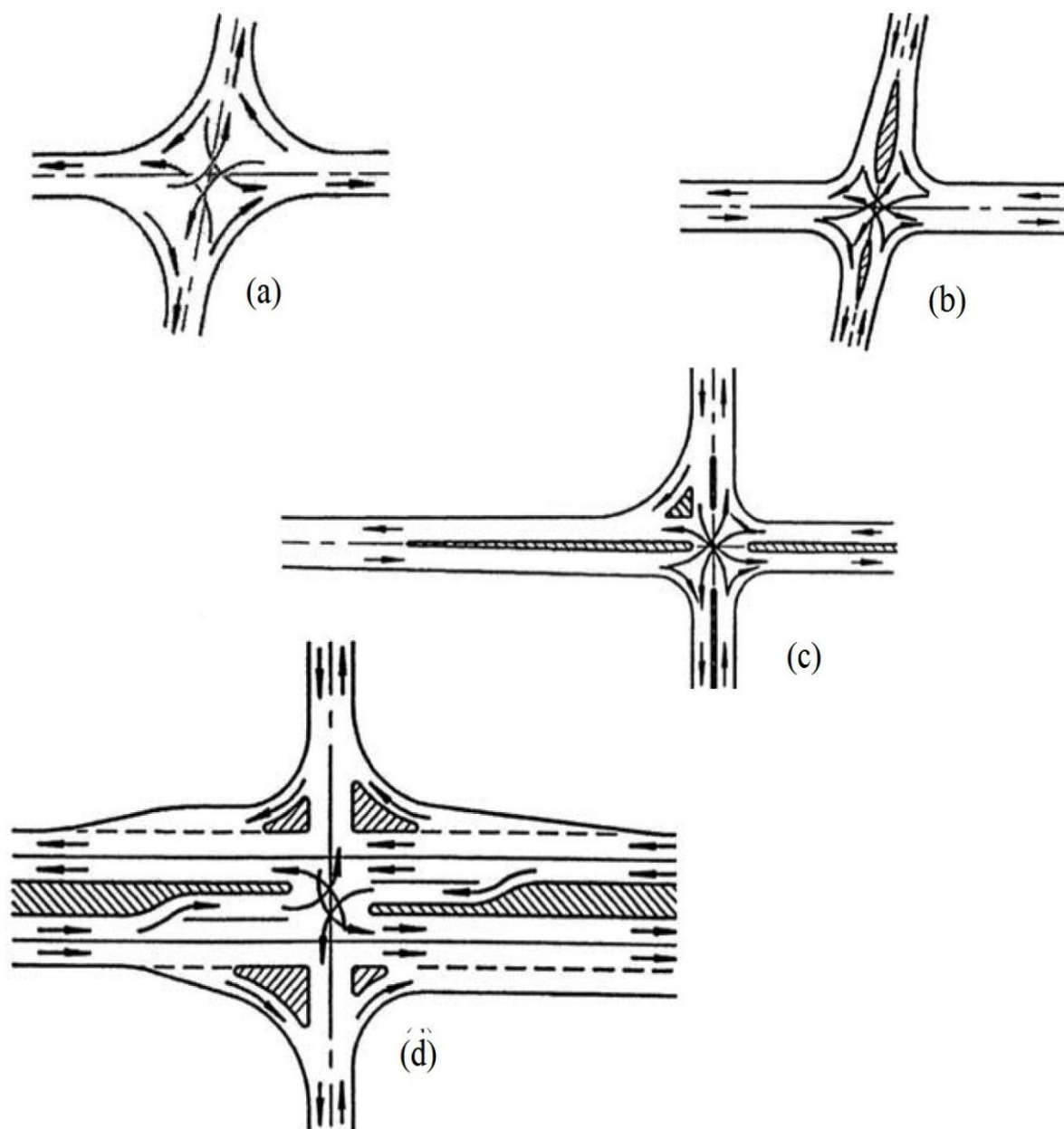


Figure 9-1/2: Examples of Four-Leg Intersection, [2, p. 272]

9-1/3 MULTILEG

Multileg intersections have five or more approaches, as shown in figure (9-1/3). Whenever possible, this type of intersection should be avoided. In order to remove some of the conflicting movements from the major intersection and thereby increase safety and operation, one or more of the legs are realigned.

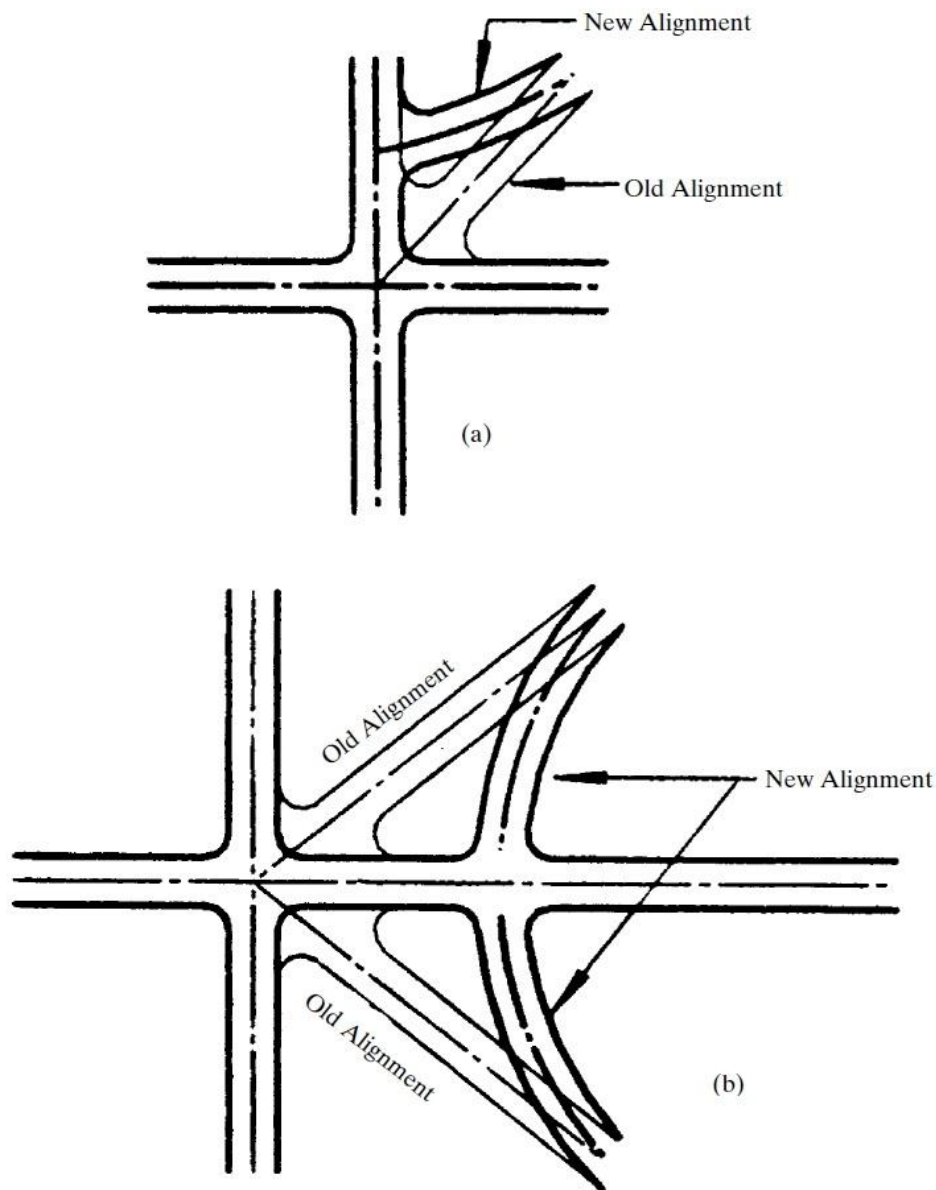


Figure 9-1/3: Examples of Multileg Intersections, [2, p.273]

9-1/4 ROUNDABOUTS

A roundabout is an intersection with a central island around where traffic must travel counterclockwise and in which entering traffic must yield to circulating traffic. Roundabouts can be classified into three basic categories according to size and number of lanes

- Mini-roundabouts
- Single-lane roundabouts
- Multilane roundabouts

Table (9-1/1) summarizes and compares some fundamental design and operational elements for each of the three roundabout categories discussed herein. The following paragraphs provide a brief discussion of each category

Table 9-1/1: Comparison of Roundabout Types [1, p. 9-22]

Design Element	Mini-Roundabout	Single-Lane Roundabout	Multilane Roundabout
Recommended maximum entry design speed	25 to 30 km/hr.	30 to 40 km/hr.	40 to 50 km/hr.
Maximum number of entering lanes per approach	1	1	2+
Typical inscribed circle diameter	13 to 27 m	27 to 46 m	40 to 76 m
Central island treatment	Mountable	Raised	Raised
Typical daily volumes on 4-leg round- about (veh/day)	0 to 15,000	0 to 20,000	2Q000+

9-4/1/1 MINI-ROUNDBABOUTS

Mini-roundabouts are small roundabouts used in low-speed urban environments, with average operating speeds of 50 km/h or less. Figure (9-1/4) provides an example of a mini-roundabout.

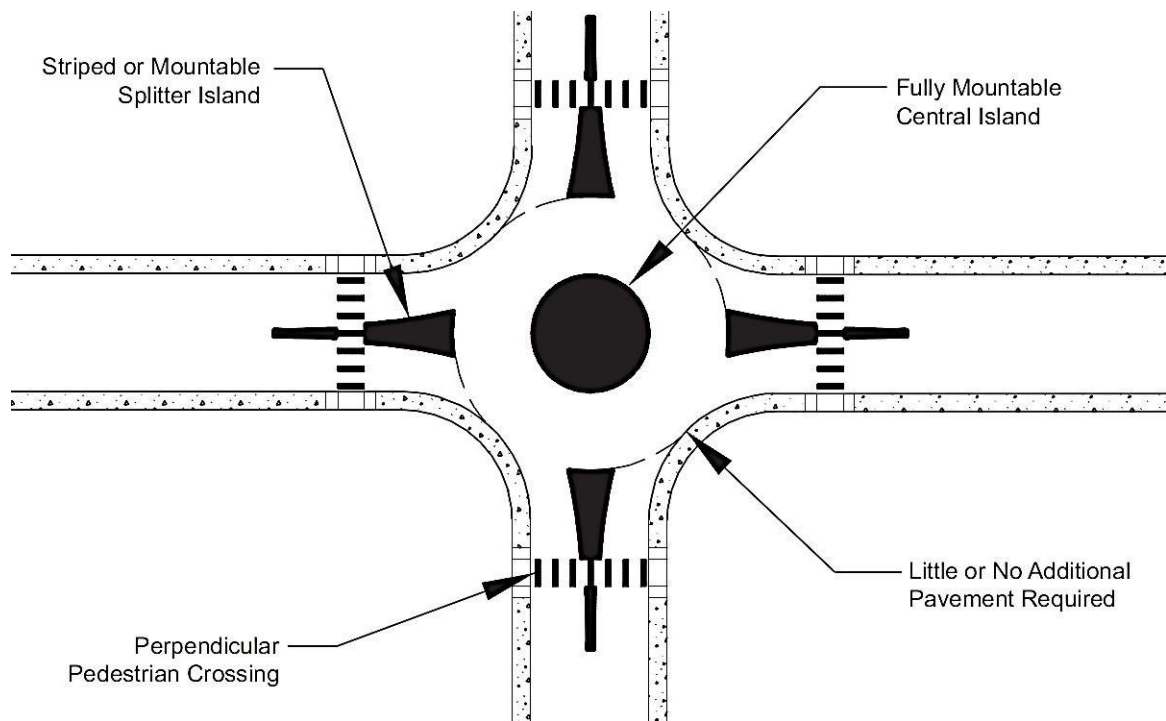


Figure 9-1/4: Typical Mini-Roundabout, [1, p. 9-23]

9-1/4/2 SINGLE-LANE ROUNDBABOUTS

This type of roundabout is characterized as having a single entry lane at all legs and one circulatory lane. Figure (9-1/5) provides an example of a typical urban single-lane roundabout. They are distinguished from mini-roundabouts by their larger inscribed circle diameters and non-mountable central islands.

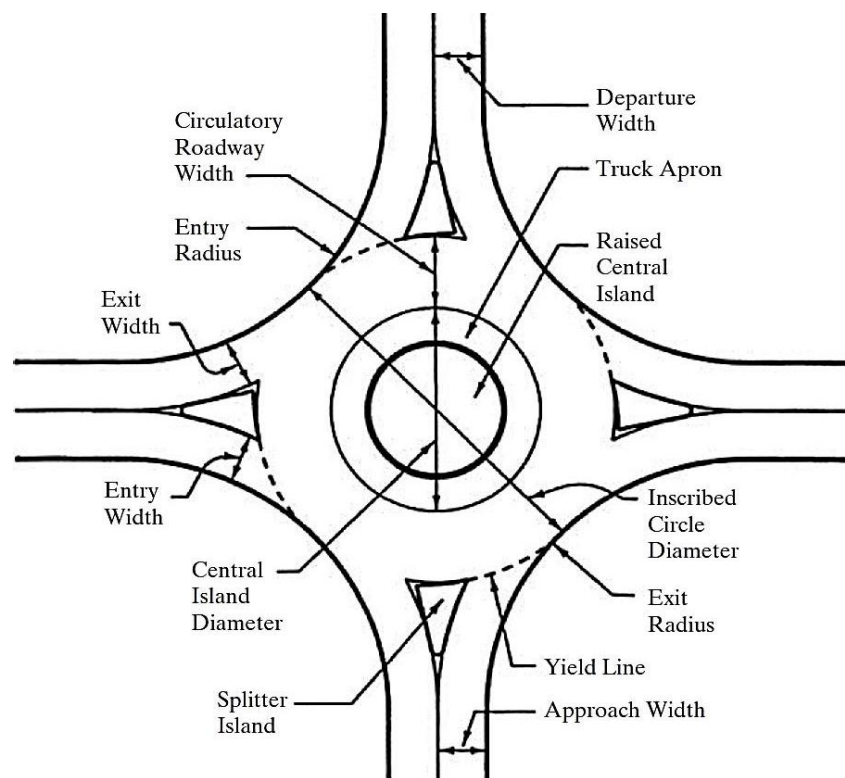


Figure 9-1/5: Geometric Elements of a Single-Lane Roundabout [2, p. 275]

9-1/4/3 MULTILANE ROUNDABOUTS

Multilane roundabouts include all roundabouts that have at least one entry with two or more lanes. They also include roundabouts with entries on one or more approaches that flare from one to two or more lanes. These need wider circulatory roadways to accommodate more than one vehicle travelling side-by-side. Figure (9-1/6) provides an example of a typical multilane roundabout.

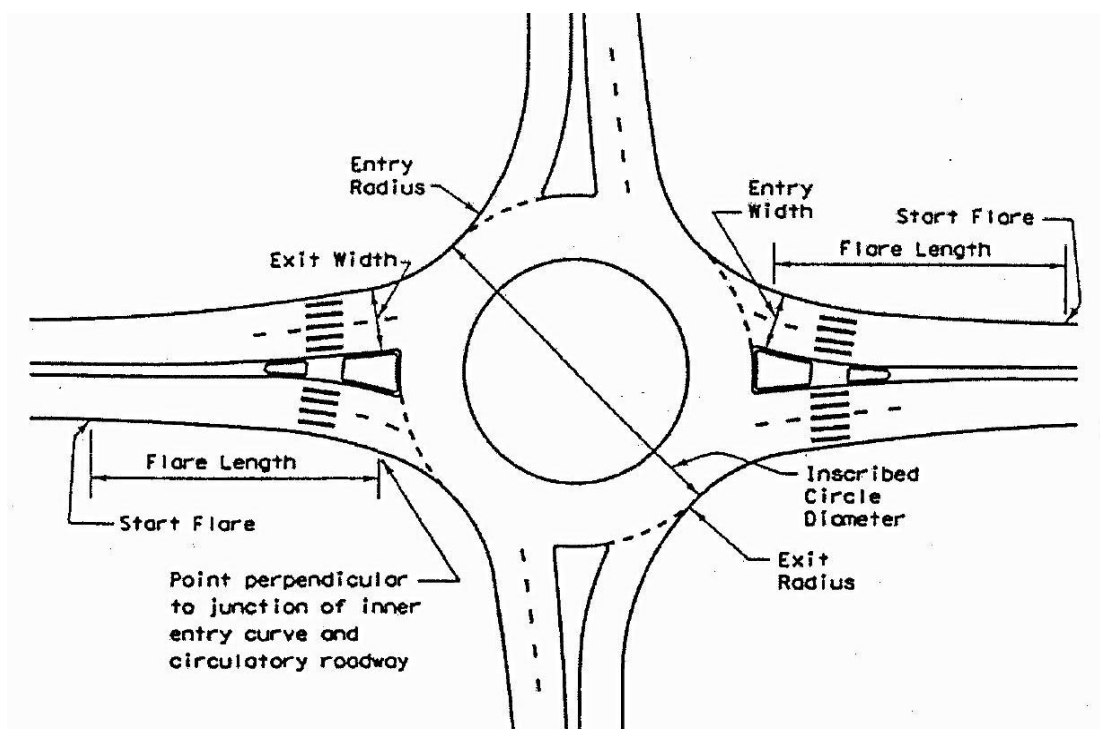


Figure 9-1/6: Roundabout with Entry Flaring in Two Quadrants [4, p582]

9-2 ISLAND DETAILS

An island is a defined area between traffic lanes used for control of vehicle movements. Islands also provide an area for pedestrian refuge and traffic control devices. Within an intersection, a median or an outer separation is also considered an island. Where traffic entering an intersection is directed into definite paths by islands, this design feature is termed a channelized intersection.

Channelizing islands generally are included in intersection design for one or more of the following purposes:

- Separation of conflicts
- Control of angle of conflict
- Reduction in excessive pavement areas
- Regulation of traffic and indication of proper use of intersection
- Arrangements to favor a predominant turning movement
- Protection of pedestrians
- Protection and storage of turning and crossing vehicles
- Location of traffic control devices

Islands serve three primary functions: (1) channelization—to control and direct traffic movement, usually turning; (2) division—to divide opposing or same direction traffic streams, usually through movements; and (3) refuge—to provide refuge for pedestrians. Most islands combine two or all of these functions.

9-2/1 CHANNELIZING ISLANDS

Channelizing islands that control and direct traffic movements into the proper paths for their intended use are an important part of intersection design. Channelizing islands may be of many shapes and sizes, depending on the conditions and dimensions of the intersection. Some of those conditions are illustrated in figure (9-2/1).

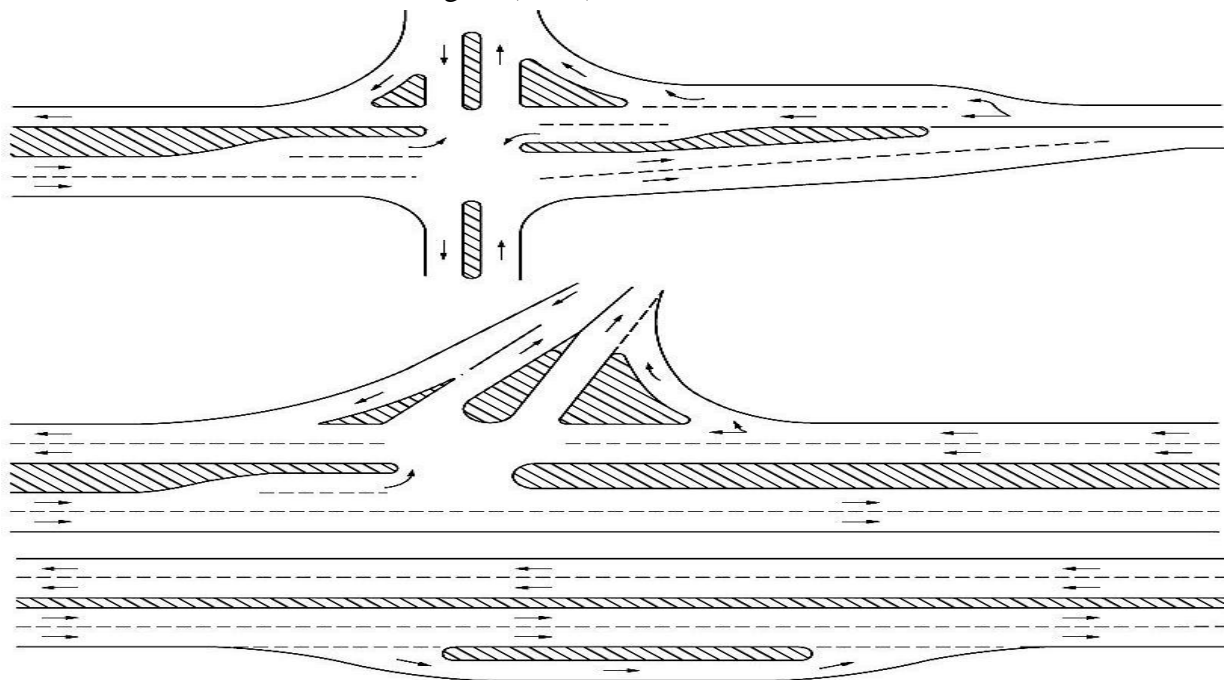


Figure 9-2/1: General Types and Shapes of Islands and Medians [1, p.9-96]

9-2/2 DIVISIONAL ISLANDS

These are frequently used at intersections of undivided highways to alert drivers that they are approaching an intersection and to control traffic at the intersection. They also can be used effectively to control left turns at skewed intersections. Examples of divisional islands are shown in figure (9-2/2). It is sometimes necessary to use reverse curves (two simple curves with opposite curvatures, forming a compound curve) when divisional islands are introduced, particularly when the location is at a tangent. At locations where speeds tend to be high, particularly in rural areas, reversals in curvature should preferably be with radii of 1 165 m or greater. Sharper curves may be used on intermediate-speed roads (up to 70 km/h) with radii of 620 m or greater.

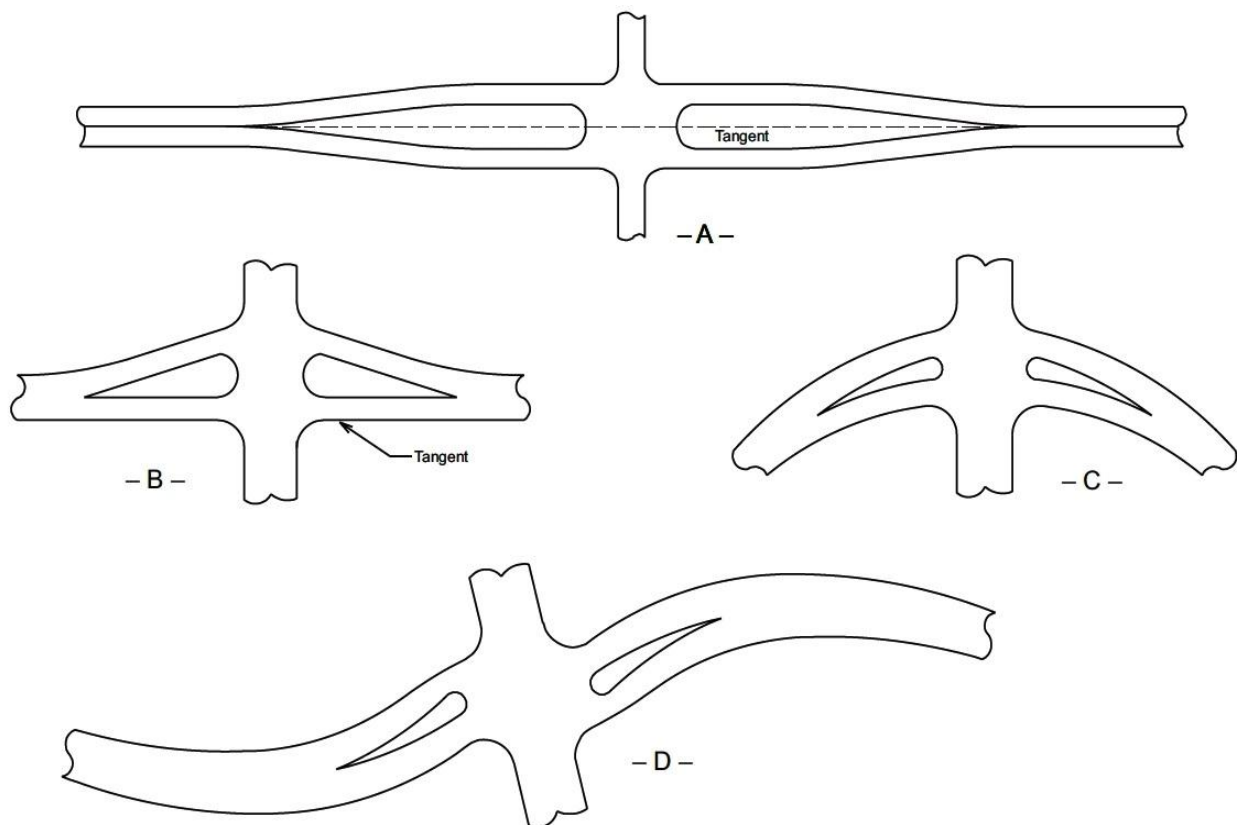


Figure 9-2/2: Examples of Divisional Islands [1, p. 9-98]

9-2/3 REFUGE ISLANDS

Refuge islands, sometimes referred to as pedestrian islands, are used mainly at urban intersections to serve as refuge areas for wheelchairs and pedestrians crossing wide intersections. They also may be used for loading and unloading transit passengers. Figure (9-2/1) shows examples of islands that provide refuge as well as function as channelized islands. Refuge islands should be a minimum of 1.8 m wide when they will be used by bicyclists.

9-2/4 MINIMUM SIZES OF ISLANDS

Island sizes and shapes vary materially from one intersection to another. Islands should be sufficiently large to command attention. The smallest curbed corner island normally should have an area of approximately 5 m² for urban and 7 m² for rural intersections. However, 9 m² is

preferable for both. Accordingly, corner triangular islands should not be less than 3.5 m a side, and preferably should be 4.5 m on a side after the rounding of corners.

Elongated or divisional islands should be not less than 1.2 m wide and 6 to 8 m long. In special cases where space is limited, elongated islands may be reduced to a minimum width of 0.5 m. In general, introducing curbed divisional islands at isolated intersections on high-speed highways is undesirable unless special attention is directed to providing high visibility for the islands. Curbed divisional islands introduced at isolated intersections on high-speed highways should be 30 m or more in length. When situated in the vicinity of a high point in the roadway profile or at or near the beginning of a horizontal curve, the approach end of the curbed island should be extended to be clearly visible to approaching drivers.

Islands having side lengths near the minimum are considered to be small islands whereas those with side lengths of 30 m or greater are considered to be large. Those with side lengths less than those for large islands but greater than the minimum are considered to be intermediate islands.

9-2/5 LOCATION AND TREATMENT OF APPROACH ENDS OF CURBED ISLANDS (AASHTO 2011)

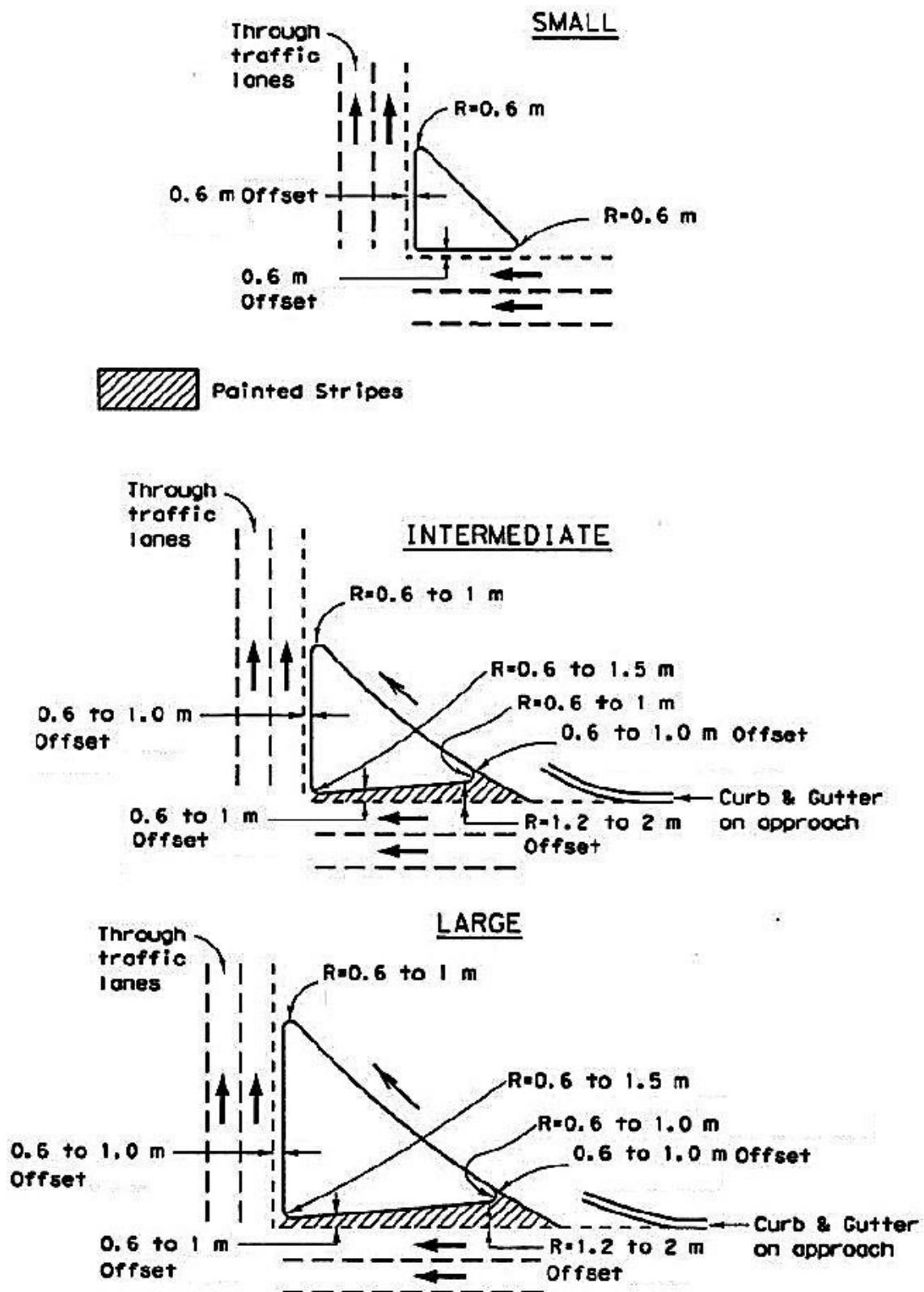
The location of a curbed island at an intersection is dictated by the edge of the through traffic lanes and the turning roadways. Figures (9-2/3) and (9-2/4) show design details of curbed islands at urban and rural intersections without and with shoulders, respectively.

All curbed islands in figures (9-2/3) and (9-2/4) are shown with approach noses and merging ends rounded with appropriate radii of 0.6 to 1 m. The approach corner is rounded with a radius of 0.6 to 1.5 m.

The approach nose of a curbed island should be conspicuous to approaching drivers and should be definitely clear of vehicle paths, physically and visually, so that drivers will not shy away from the island. Reflectorized markers may be used on the approach nose of the curbed island. The offset from the travel lane to the approach nose should be greater than that to the face of the curbed island, normally about 0.6 m. For curbed median islands, the face of curb at the approach island nose should be offset at least 0.6 m and preferably 1.0 m from the normal median edge of the traveled way. The island should then be gradually widened to its full width. For other curbed islands, the total nose offset should be 1 to 2 m from the normal edge of through lanes and 0.6 to 1 m edge of the traveled way of a turning roadway. Large offsets should be provided where the curbed corner island is preceded by a right turn deceleration lane.

Where a curbed corner island is proposed on an approach roadway with shoulders, the face of curb on the corner island should be offset by an amount equal to the shoulder width. If the corner island is preceded by a right-turn deceleration lane, the shoulder offset should be at least 2.4 m.

Curbed corner islands and median noses should be ramped down as shown in figure (9-2/5) and provided with devices to give advance warning to approaching drivers and especially for nighttime driving. Pavement markings in front of the approach nose are particularly advantageous on the areas shown as stippled in figure (9-2/3)



TRIANGULAR CURBED ISLAND ON URBAN STREETS

Figure 9-2/3: Layouts of curbed island without shoulders [2, p. 292]

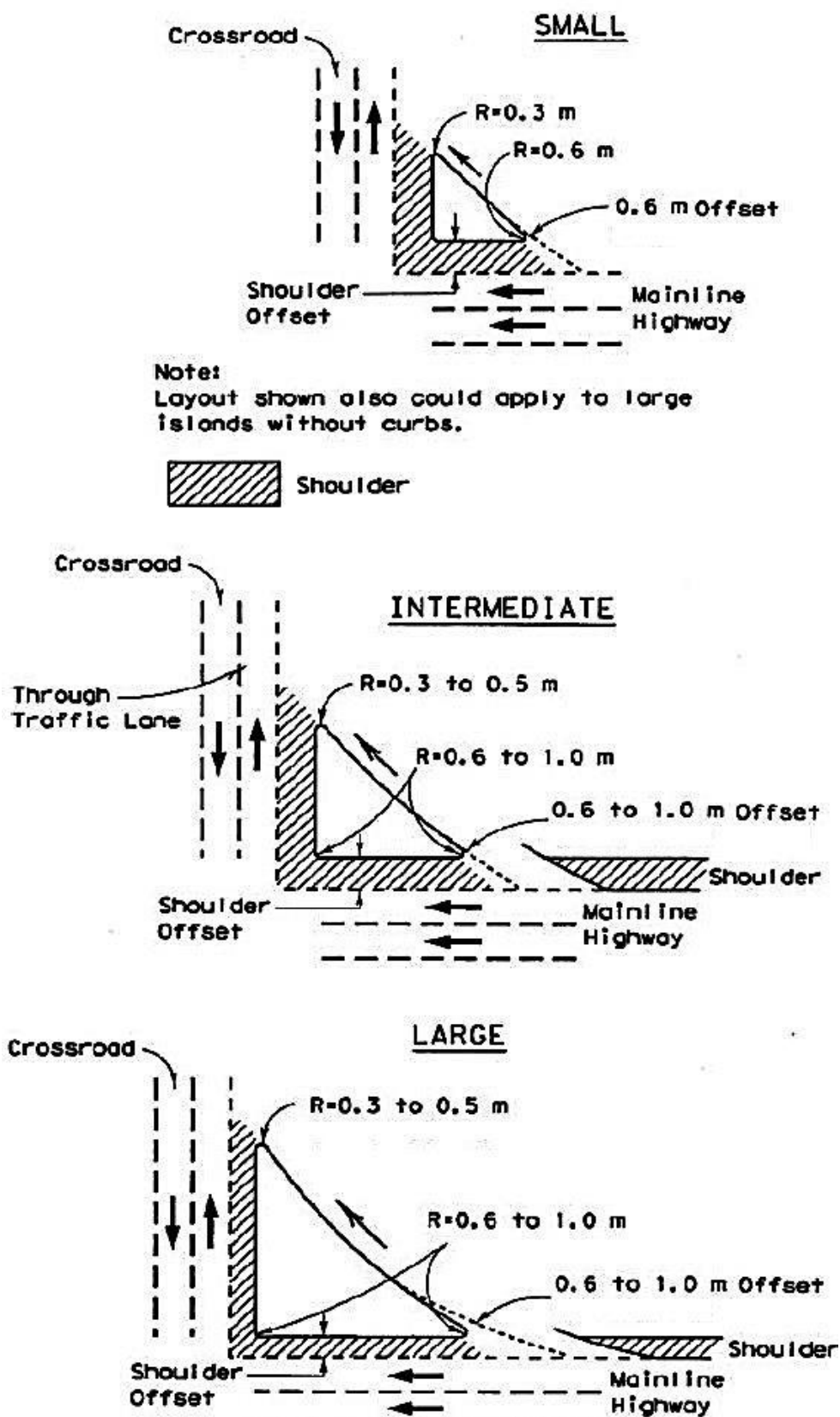


Figure 9-2/4: Layouts of curbed island with shoulders [2, p. 293]

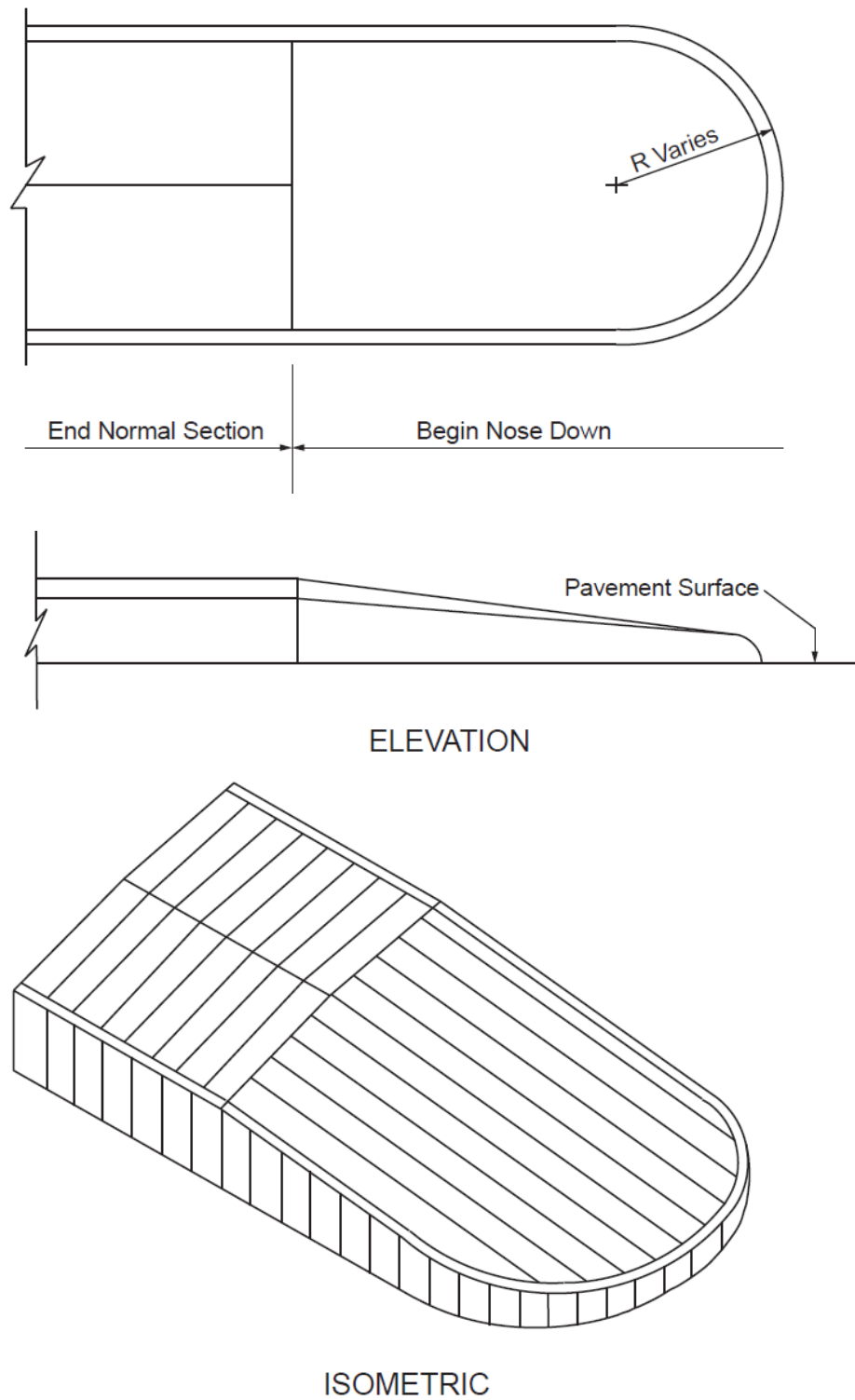


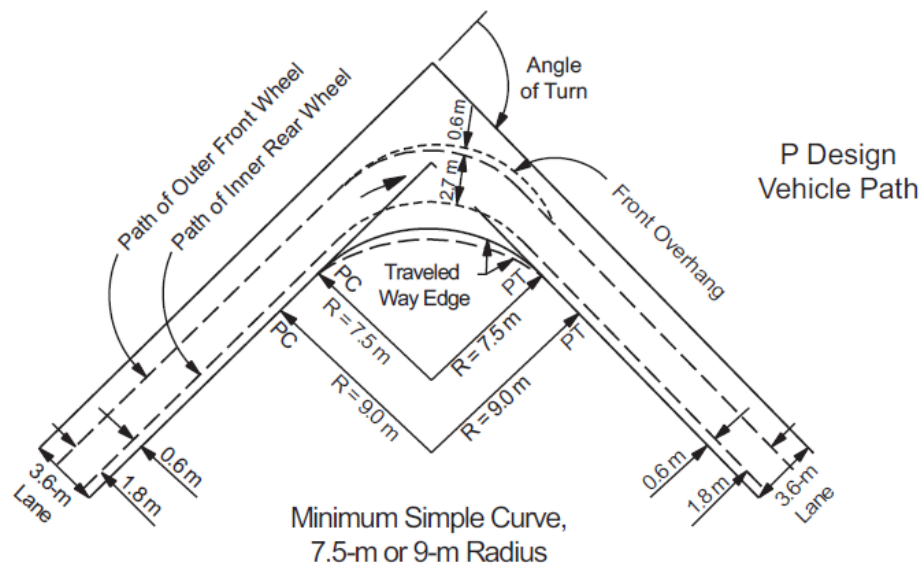
Figure 9-2/5: Nose Ramping at Approach End of Median or Corner Island [1, p. 9-104]

9-3 MINIMUM TURNING ROADWAY DESIGN WITH CORNER ISLANDS AND DIFFERENT ANGLE TURNS

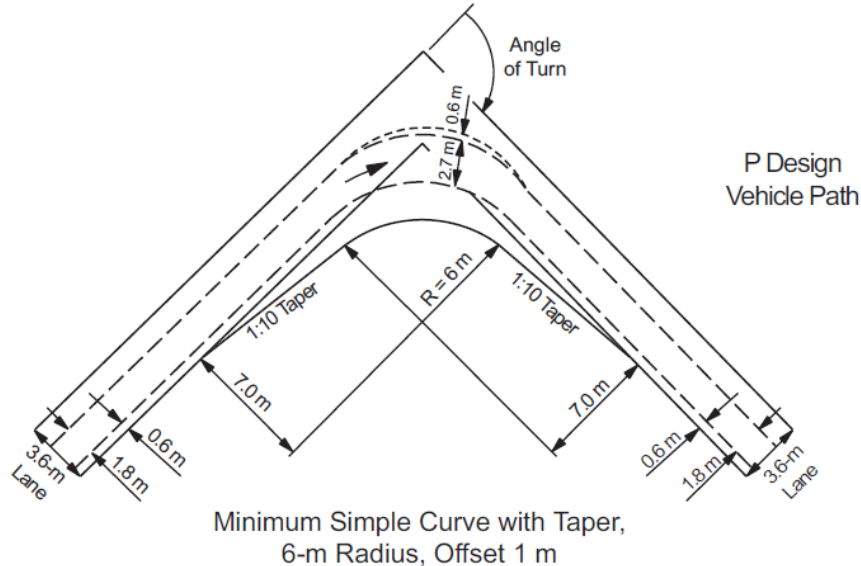
The angle of turn, turning speed, design vehicle, and traffic volume are the main factors governing the design of curves at at-grade intersections. When the turning speed at an intersection is assumed to be 24 km/hr. or less, the curves for the pavement edges are designed to conform to at least the minimum turning path of the design vehicle. The three types of design commonly used when turning speeds are 24 km/h or less are the simple curve (an arc of a circular curve), the simple curve with taper, and the three-centered compound curve (three simple curves joined together and turning in the same direction)

Figure (9-3/1) shows the minimum designs necessary for a passenger car making a 90-degree right turn. Similar designs for single-unit (SU) trucks in figure (9-3/2) and for tractor semi-trailer are shown in figures (9-3/3) through (9-3/5). The minimum edge-of-traveled-way designs for turns may be considered at an intersection based on the turning paths of the design vehicles identified below:

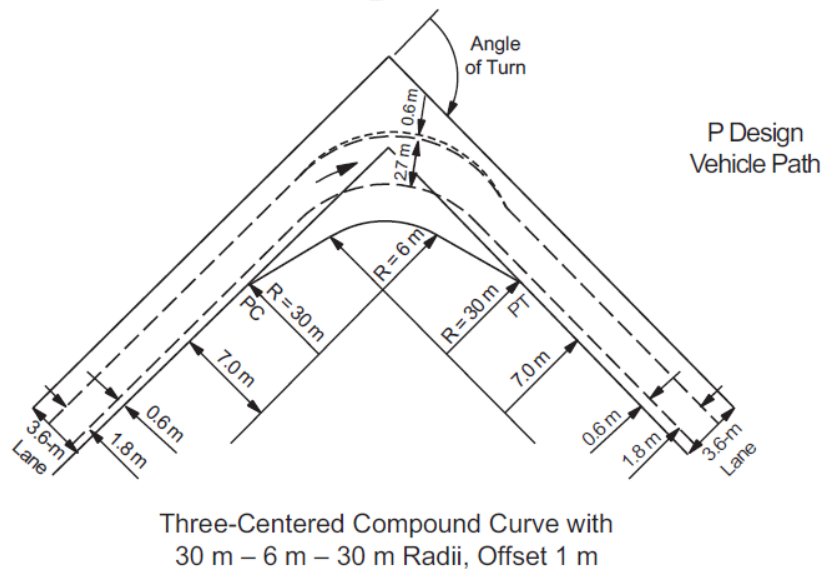
- P design vehicle (figure 9-3/1). This design vehicle is used at intersections in conjunction with parkways where minimum turns are appropriate, at local road intersections with major roads where turns are made only occasionally, and at intersections of two minor roads carrying low volumes. However, if conditions permit, the SU vehicle (figure 9-3/2) is the preferred design vehicle.
- SU design vehicle (figure 9-3/2). Generally, this design vehicle provides the recommended minimum edge-of-traveled-way design for rural highways other than those described above. Turning movements for urban conditions are discussed in a separate section of this chapter. Important turning movements on major highways, particularly those involving a large percentage of trucks, should be designed with either larger radii, or speed-change lanes, or both.
- Semitrailer combination design vehicles (figures (9-3/3) through (9-3/5)). These design vehicles should be used where truck combinations will turn repeatedly. Where designs for such vehicles are warranted, the simpler symmetrical arrangements of three-centered compound curves (shown in figures (9-3/3) and (9-3/4)) are generally preferred if these smaller truck combinations make up a sizable percentage of the turning volume. Because designs for semitrailer combination vehicles, particularly when used in two or more quadrants of an intersection, produce large paved areas, it may be desirable to provide somewhat larger radii and use a corner triangular island.



- A -



- B -



- C -

Figure 9-3/1: Minimum Traveled Way (Passenger Vehicles) [4, p. 598]

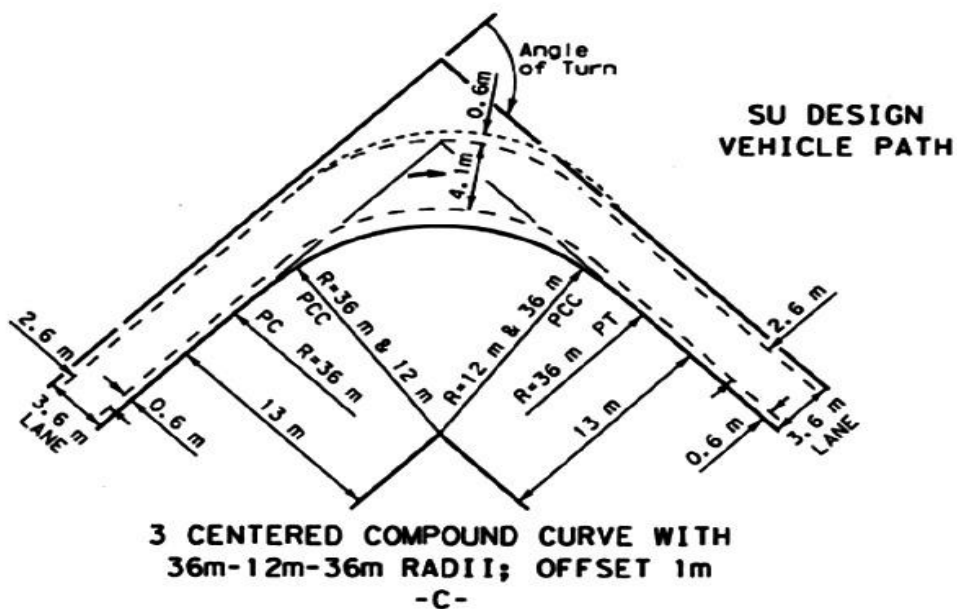
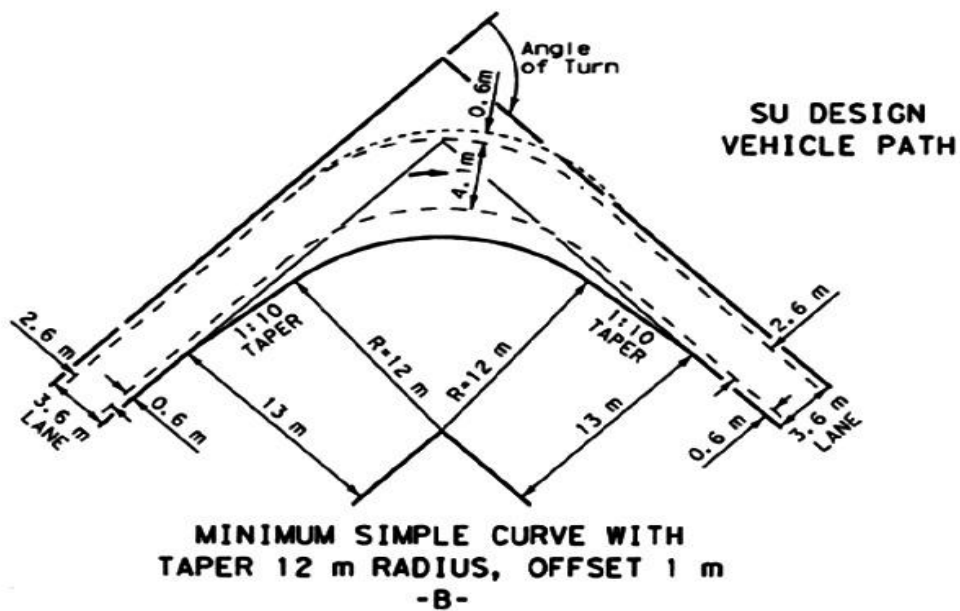
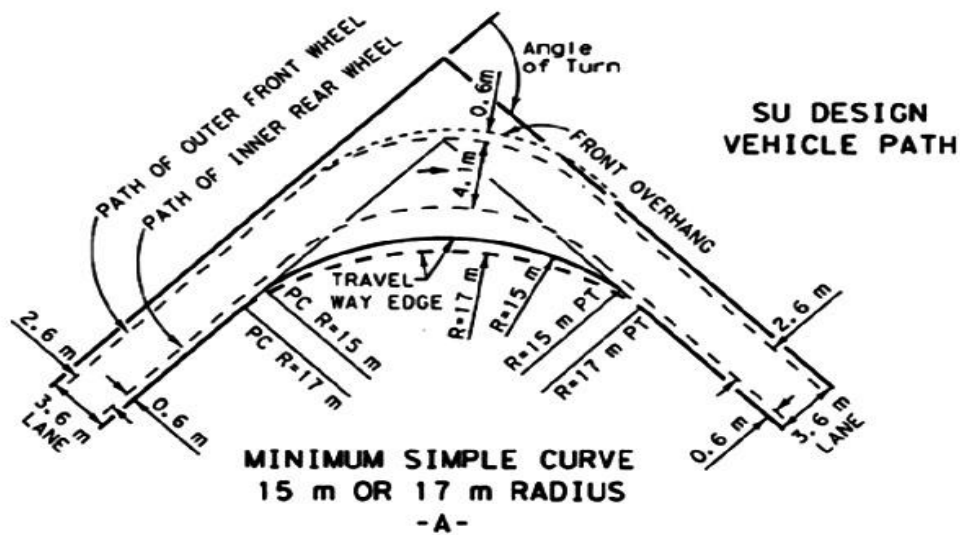
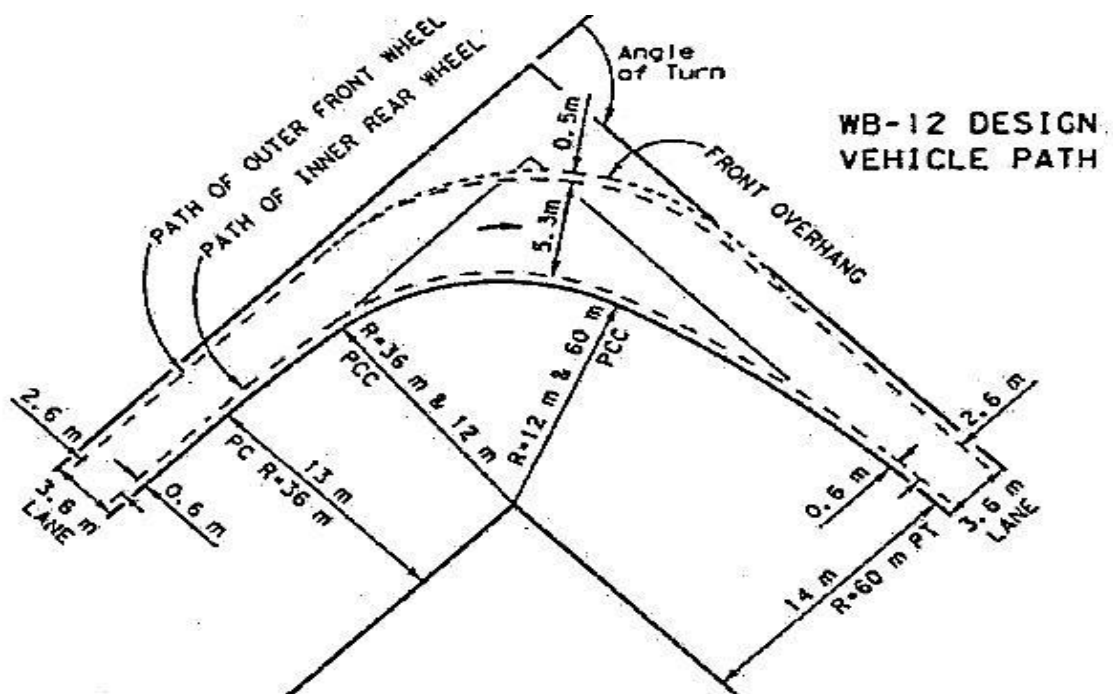
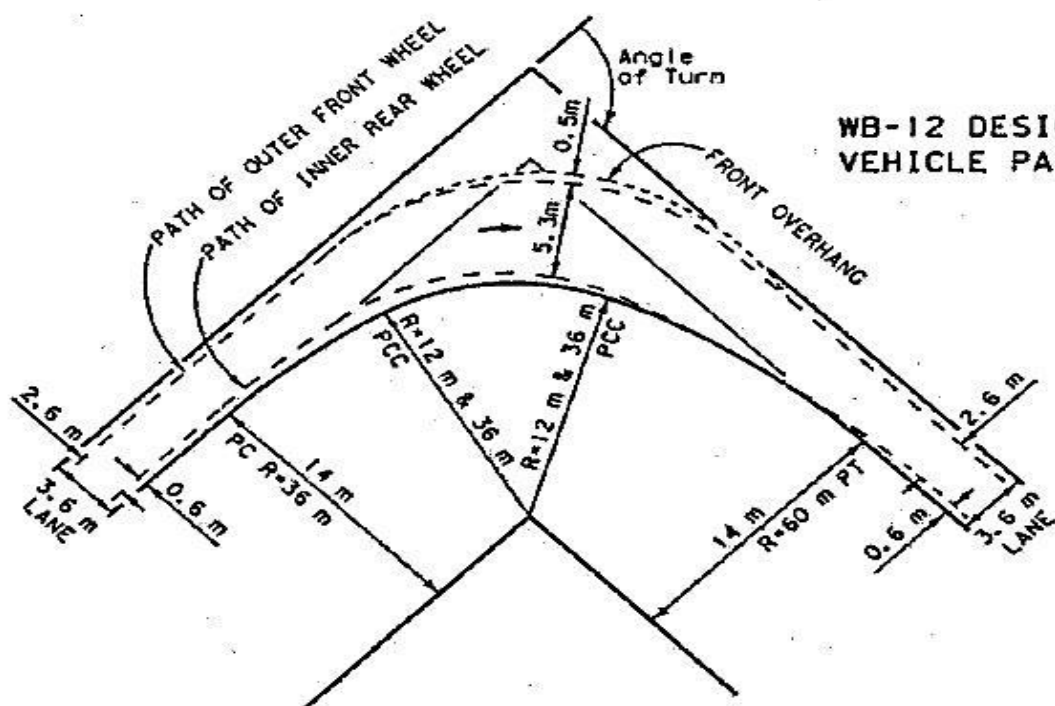


Figure 9-3/2: Minimum Traveled Way Designs (Single Unit Trucks and City Transit Buses)
[4, p. 600]



**3 CENTERED COMPOUND CURVE WITH
36m-12m-60m RADII; OFFSET 1m AND 2m**
-A-



**3 CENTERED COMPOUND CURVE WITH
36m-12m-36m RADII; OFFSET 2m**
-B-

Figure 9-3/3: Minimum Edge-of-Traveled Way Designs (WB-12 Design Vehicle Path)
[4, p. 602]

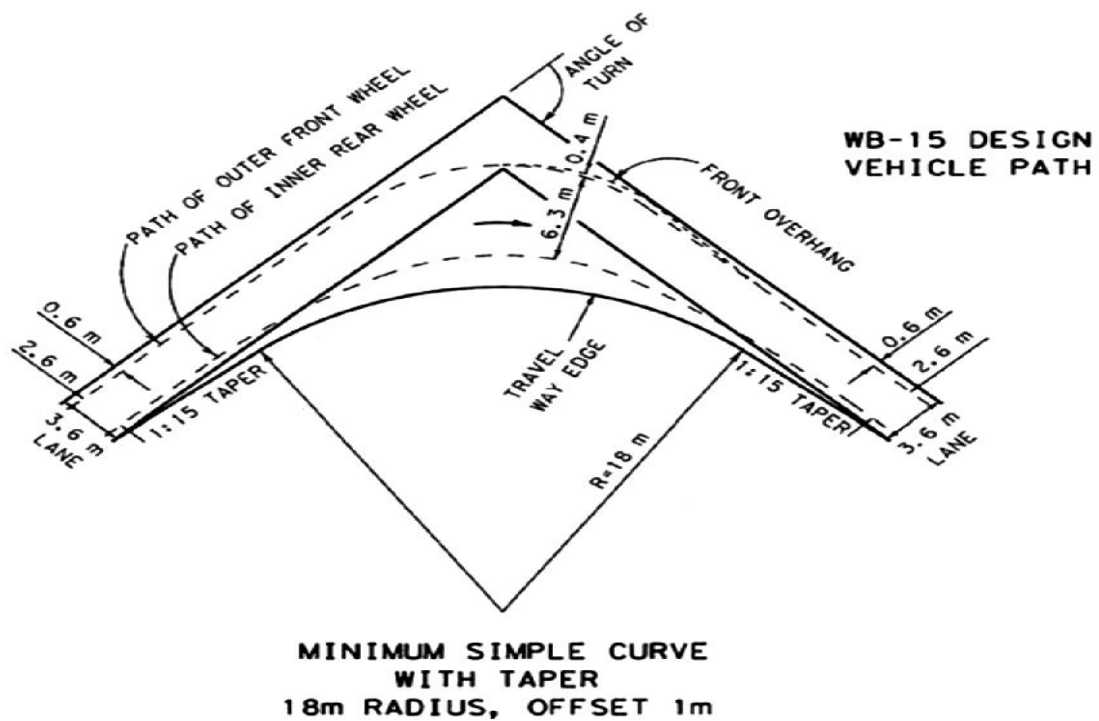


Figure 9-3/4: Minimum Edge-of-Traveled Way Designs (WB-15 Design Vehicle Path)
[4, p. 604]

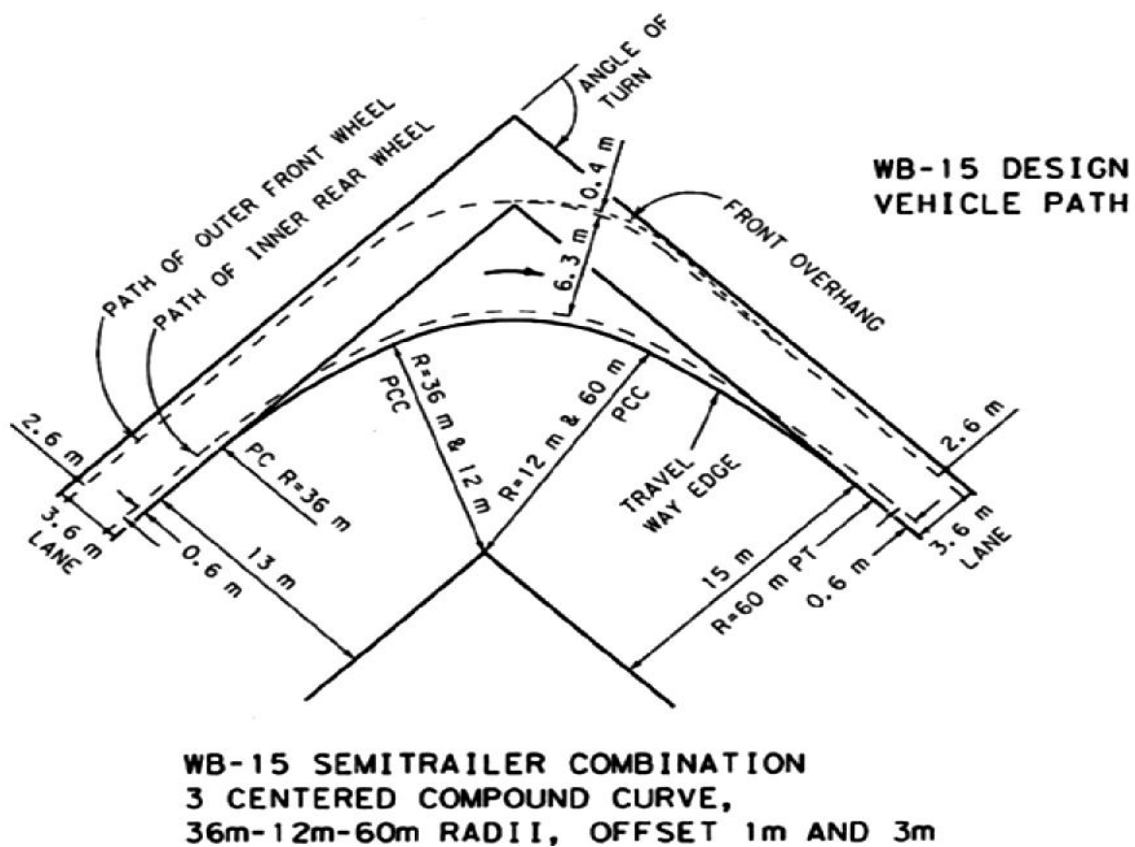


Figure 9-3/5: Minimum Edge-of-Traveled Way Designs (WB-19 Design Vehicle Path)
[4, p. 605]

Minimum edge-of-pavement designs for different angles of turn and design vehicles are given in table (9-3/1) for simple curves and simple curves with taper and in table (9-3/2) for symmetric and asymmetric three-centered curves.

Table 9-3/1: Edge-of-Traveled-Way Designs for Turns at Intersection [4, p. 588]

Angle of turn (degrees)	Design vehicle	Simple curve radius (m)	Simple curve radius with taper		
			Radius (m)	Offset (m)	Taper H:V
30	P	18	—	—	—
	SU	30	—	—	—
	WB-12	45	—	—	—
	WB-15	60	—	—	—
	WB-19	110	67	1.0	15:1
	WB-20	116	67	1.0	15:1
	WB-30T	77	37	1.0	15:1
	WB-33D	145	77	1.1	20:1
45	P	15	—	—	—
	SU	23	—	—	—
	WB-12	36	—	—	—
	WB-15	53	36	0.6	15:1
	WB-19	70	43	1.2	15:1
	WB-20	76	43	1.3	15:1
	WB-30T	60	35	0.8	15:1
	WB-33D	—	60	1.3	20:1
60	P	12	—	—	—
	SU	18	—	—	—
	WB-12	28	—	—	—
	WB-15	45	29	1.0	15:1
	WB-19	50	43	1.2	15:1
	WB-20	60	43	1.3	15:1
	WB-30T	46	29	0.8	15:1
	WB-33D	—	54	1.3	20:1

Table 9-3/1: Edge-of-Traveled-Way Designs for Turns at Intersection, Continued [4, p 589]

Angle of turn (degrees)	Design vehicle	Simple curve radius (m)	Simple curve radius with taper		
			Radius (m)	Offset (m)	Taper H:V
75	P	11	8	0.6	10:1
	SU	17	14	0.6	10:1
	WB-12	—	18	0.6	15:1
	WB-15	—	20	1.0	15:1
	WB-19	—	43	1.2	20:1
	WB-20	—	43	1.3	20:1
	WB-30T	—	26	1.0	15:1
	WB-33D	—	42	1.7	20:1
90	P	9	6	0.8	10:1
	SU	15	12	0.6	10:1
	WB-12	—	14	1.2	10:1
	WB-15	—	18	1.2	15:1
	WB-19	—	36	1.3	30:1
	WB-20	—	37	1.3	30:1
	WB-30T	—	25	0.8	15:1
	WB-33D	—	35	0.9	15:1
105	P	—	6	0.8	—
	SU	—	11	1.0	—
	WB-12	—	12	1.2	—
	WB-15	—	17	1.2	15:1
	WB-19	—	35	1.0	15:1
	WB-20	—	35	1.0	15:1
	WB-30T	—	22	1.0	15:1
	WB-33D	—	28	2.8	20:1

Table 9-3/1: Edge-of-Traveled-Way Designs for Turns at Intersection, Continued
[4, p.590]

Angle of turn (degrees)	Design vehicle	Simple curve radius (m)	Simple curve radius with taper		
			Radius (m)	Offset (m)	Taper H:V
120	P	—	6	0.6	—
	SU	—	9	1.0	—
	WB-12	—	11	1.5	—
	WB-15	—	14	1.2	15:1
	WB-19	—	30	1.5	15:1
	WB-20	—	31	1.6	15:1
	WB-30T	—	20	1.1	15:1
	WB-33D	—	26	2.8	20:1
135	P	—	6	0.5	10:1
	SU	—	9	1.2	10:1
	WB-12	—	9	2.5	15:1
	WB-15	—	12	2.0	15:1
	WB-19	—	24	1.5	20:1
	WB-20	—	25	1.6	20:1
	WB-30T	—	19	1.7	15:1
	WB-33D	—	25	2.6	20:1
150	P	—	6	0.6	10:1
	SU	—	9	1.2	8:1
	WB-12	—	9	2.0	8:1
	WB-15	—	11	2.1	6:1
	WB-19	—	18	3.0	10:1
	WB-20	—	19	3.1	10:1
	WB-30T	—	19	2.2	10:1
	WB-33D	—	20	4.6	10:1

Table 9-3/1: Edge-of-Traveled-Way Designs for Turns at Intersection, Continued
[4, p. 591]

Angle of turn (degrees)	Design vehicle	Simple curve radius (m)	Simple curve radius with taper		
			Radius (m)	Offset (m)	Taper H:V
180	P	—	5	0.2	20:1
	SU	—	9	0.5	10:1
	WB-12	—	6	3.0	5:1
	WB-15	—	8	3.0	5:1
	WB-19	—	17	3.0	15:1
	WB-20	—	16	4.2	10:1
	WB-30T	—	17	3.1	10:1
	WB-33D	—	17	6.1	10:1

Table 9-3/2: Edge-of-Traveled-Way for Turns at Intersection [4, p. 592]

Angle of turn (degrees)	Design vehicle	3-centered compound		3-centered compound	
		Curve radii (m)	Symmetric offset (m)	Curve radii (m)	Asymmetric (m)
30	P	—	—	—	—
	SU	—	—	—	—
	WB-12	—	—	—	—
	WB-15	—	—	—	—
	WB-19	—	—	—	—
	WB-20	140-53-140	1.2	91-53-168	0.6–1.4
	WB-30T	67-24-67	1.4	61-24-91	0.8–1.5
	WB-33D	168-76-168	1.5	76-61-198	0.5–2.1
45	P	—	—	—	—
	SU	—	—	—	—
	WB-12	—	—	—	—
	WB-15	60-30-60	1.0	—	—
	WB-19	140-72-140	0.6	36-43-150	1.0–2.6
	WB-20	140-53-140	1.2	76-38-183	0.3–1.8
	WB-30T	76-24-76	1.4	61-24-91	0.8–1.7
	WB-33D	168-61-168	1.5	61-52-198	0.5–2.1
60	P	—	—	—	—
	SU	—	—	—	—
	WB-12	—	—	—	—
	WB-15	60-23-60	1.7	60-23-84	.06–2.0
	WB-19	120-30-120	4.5	34-30-67	3.0–3.7
	WB-20	122-30-122	2.4	76-38-183	0.3–1.8
	WB-30T	76-24-76	1.4	61-24-91	0.6–1.7
	WB-33D	198-46-198	1.7	61-43-183	0.5–2.4

Table 9-3/2: Edge-of-Traveled-Way for Turns at Intersection, continued [4, p. 593]

Angle of turn (degrees)	Design vehicle	3-centered compound		3-centered compound	
		Curve radii (m)	Symmetric offset (m)	Curve radii (m)	Asymmetric (m)
75	P	30-8-30	0.6	—	—
	SU	36-14-36	0.6	—	—
	WB-12	36-14-36	1.5	36-14-60	0.6–2.0
	WB-15	45-15-45	2.0	45-15-69	0.6–3.0
	WB-19	134-23-134	4.5	43-30-165	1.5–3.6
	WB-20	128-23-128	3.0	61-24-183	0.3–3.0
	WB-30T	76-24-76	1.4	30-24-91	0.5–1.5
	WB-33D	213-38-213	2.0	46-34-168	0.5–3.5
90	P	30-6-30	0.8	—	—
	SU	36-12-36	0.6	—	—
	WB-12	36-12-36	1.5	36-12-60	0.6–2.0
	WB-15	55-18-55	2.0	36-12-60	0.6–3.0
	WB-19	120-21-120	3.0	48-21-110	2.0–3.0
	WB-20	134-20-134	3.0	61-21-183	0.3–3.4
	WB-30T	76-21-76	1.4	61-21-91	0.3–1.5
	WB-33D	213-34-213	2.0	30-29-168	0.6–3.5
105	P	30-6-30	0.8	—	—
	SU	30-11-30	1.0	—	—
	WB-12	30-11-30	1.5	30-17-60	0.6–2.5
	WB-15	55-14-55	2.5	45-12-64	0.6–3.0
	WB-19	160-15-160	4.5	110-23-180	1.2–3.2
	WB-20	152-15-152	4.0	61-20-183	0.3–3.4
	WB-30T	76-18-76	1.5	30-18-91	0.5–1.8
	WB-33D	213-29-213	2.4	46-24-152	0.9–4.6

Table 9-3/2: Edge-of-Traveled-Way for Turns at Intersection, continued [4, p. 594]

Angle of turn (degrees)	Design vehicle	3-centered compound		3-centered compound	
		Curve radii (m)	Symmetric offset (m)	Curve radii (m)	Asymmetric (m)
120	P	30-60-30	0.6	—	—
	SU	30-9-30	1.0	—	—
	WB-12	36-9-36	2.0	30-9-55	0.6–2.7
	WB-15	55-12-55	2.6	45-11-67	0.6–3.6
	WB-19	160-21-160	3.0	24-17-160	5.2–7.3
	WB-20	168-14-168	4.6	61-18-183	0.6–3.8
	WB-30T	76-18-76	1.5	30-18-91	0.5–1.8
	WB-33D	213-26-213	2.7	46-21-152	2.0–5.3
135	P	30-6-30	0.5	—	—
	SU	30-9-30	1.2	—	—
	WB-12	36-9-36	2.0	30-8-55	1.0–4.0
	WB-15	48-11-48	2.7	40-9-56	1.0–4.3
	WB-19	180-18-180	3.6	30-18-195	2.1–4.3
	WB-20	168-14-168	5.0	61-18-183	0.6–3.8
	WB-30T	76-18-76	1.7	30-18-91	0.8–2.0
	WB-33D	213-21-213	3.8	46-20-152	2.1–5.6
150	P	23-6-23	0.6	—	—
	SU	30-9-30	1.2	—	—
	WB-12	30-9-30	2.0	28-8-48	0.3–3.6
	WB-15	48-11-48	2.1	36-9-55	1.0–4.3
	WB-19	145-17-145	4.5	43-18-170	2.4–3.0
	WB-20	168-14-168	5.8	61-17-183	2.0–5.0
	WB-30T	76-18-76	2.1	30-18-91	1.5–2.4
	WB-33D	213-20-213	4.6	61-20-152	2.7–5.6

Table 9-3/2: Edge-of-Traveled-Way for Turns at Intersection, continued [4, p. 595]

Angle of turn (degrees)	Design vehicle	3-centered compound		3-centered compound	
		Curve radii (m)	Symmetric offset (m)	Curve radii (m)	Asymmetric (m)
180	P	15-5-15	0.2	—	—
	SU	30-9-30	0.5	—	—
	WB-12	30-6-30	3.0	26-6-45	2.0–4.0
	WB-15	40-8-40	3.0	30-8-55	2.0–4.0
	WB-19	245-14-245	6.0	30-17-275	4.5–4.5
	WB-20	183-14-183	6.2	30-17-122	1.8–4.6
	WB-30T	76-17-76	2.9	30-17-91	2.6–3.2
	WB-33D	213-17-213	6.1	61-18-152	3.0–6.4

Table (9-3/1) indicates that it is not feasible to have simple curves for large trucks such as WB-12, WB-15 and WB-19 when the angle of turn is 75 degrees or greater

9-3/1 EFFECT OF CURB RADII ON TURNING PATHS

The effect of curb radii on the right-turning paths of various design vehicles turning through an angle of 90 degrees (on streets without parking lanes) is shown in figures (9-3/6) and (9-3/7). Figure (9-3/6) shows the effects of a 4.5-m radius. With 3.6-m lanes, the design passenger vehicle can turn with no encroachment on an adjacent lane at the end of the turn, but the SU and BUS design vehicles will swing wide on both streets and will occupy two lanes at the end of the turn.

To turn into two lanes on the cross street, the WB-15 [WB-50] design vehicle will occupy an area wider than those two lanes (i.e., the design vehicle would encroach on a shoulder or curb area, as well).

Figure (9-3/7) shows vehicle operation at a 12-m curb radius. The P vehicle can easily make the turn around this radius. The SU and BUS design vehicles can turn around the radius into one lane on the cross street by beginning their turn adjacent to the centerline of the major street. The WB-15 [WB-50] design vehicle needs the entire two-lane width of the cross street to complete the turn.

Table (9-3/3) shows the effect of the angle of intersection on turning paths of various design vehicles on streets without parking lanes. The dimensions d_1 and d_2 are the widths occupied by the turning vehicle on the major street and cross street, respectively, while negotiating turns through various angles. Both dimensions are measured from the right-hand curb to the point of maximum overhang. These widths, shown for various angles of turn and curb radii, and for two types of maneuvers, generally increase with the angle of turn.

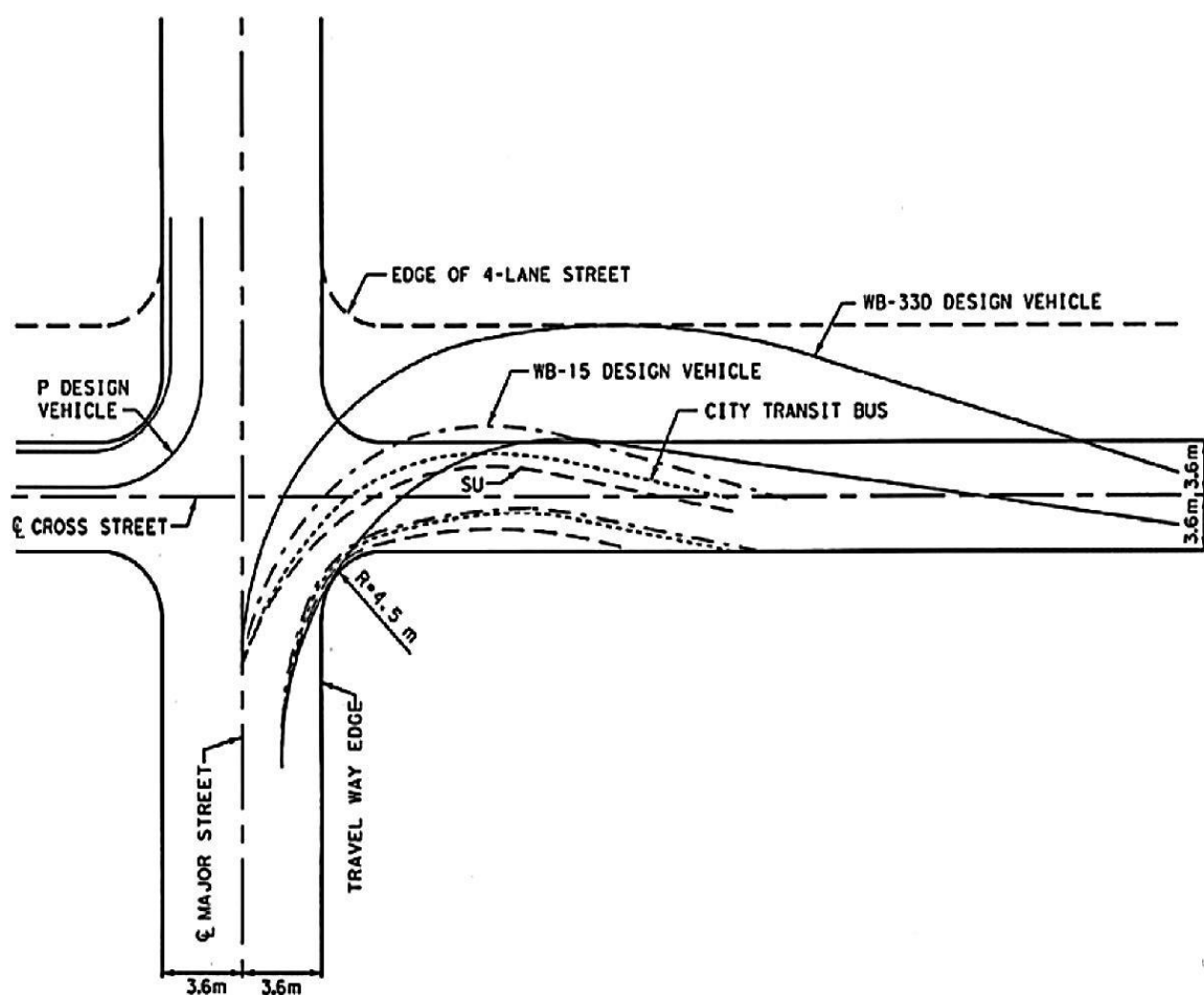


Figure 9-3/6: Effect of Curb Radii on Right Turning Paths of Various Design Vehicles
[4, p. 616]

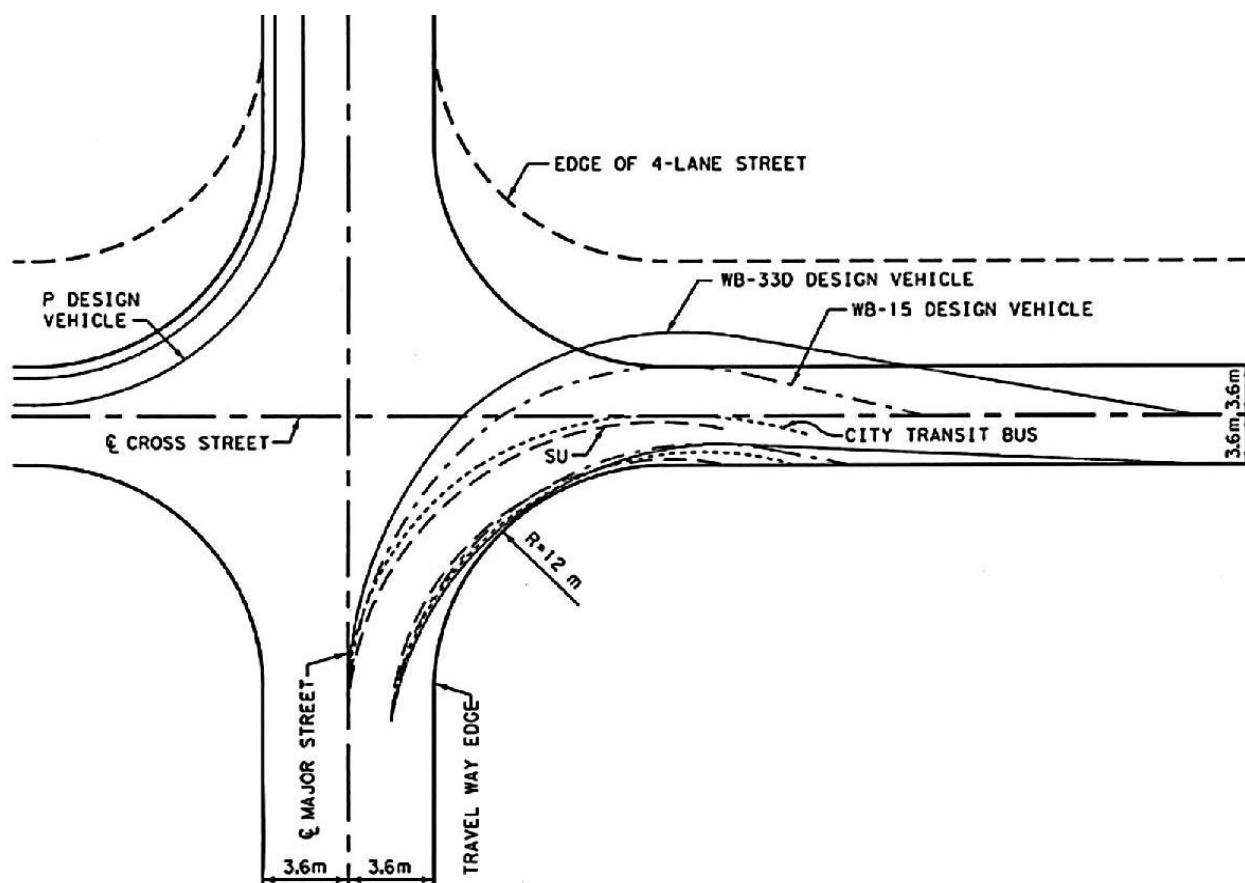


Figure 9-3/7: Effect of Curb Radii on Right Turning Paths of Various Design Vehicles
[4, p. 617]

Table 9-3/3: Cross Street Width Occupied by Turning Vehicle for Various Angles of Intersection and Curb Radii, continued [4, p. 619]

		d ₂ for cases A and B where:									
		R=4.5 m		R=6 m		R=7.5 m		R=9 m		R=12 m	
Design	vehicle	A	B	A	B	A	B	A	B	A	B
30°	SU	4.3	4.0	4.3	4.0	4.0	4.0	4.0	4.0	4.0	4.0
	BUS	6.7	5.2	5.8	5.2	5.8	5.2	5.8	5.2	5.5	5.2
	WB-12	4.3	4.3	4.3	4.3	4.3	4.3	4.3	4.3	4.3	4.3
	WB-15	6.1	5.2	6.1	5.2	6.1	5.2	5.8	4.9	5.5	4.9
	WB-19	-	-	-	-	-	-	-	-	8.2	5.2
	WB-20	-	-	-	-	-	-	-	-	8.5	5.5
60°	SU	5.8	4.9	5.8	4.9	5.2	4.6	4.9	4.6	4.3	4.3
	BUS	8.5	6.4	7.9	6.1	7.3	6.1	7.0	5.8	6.7	5.5
	WB-12	7.3	5.8	6.7	5.8	6.4	5.8	5.8	5.5	5.2	4.9
	WB-15	9.4	6.7	8.2	6.4	8.5	6.1	7.6	5.8	6.7	5.5
	WB-19	-	-	-	-	-	-	-	-	9.1	6.7
	WB-20	-	-	-	-	-	-	-	-	11.3	7.3
90°	SU	7.9	6.1	7.0	5.5	5.8	4.9	5.2	4.6	4.0	4.0
	BUS	11.6	7.0	10.0	6.7	9.1	6.7	7.6	6.4	6.7	5.5
	WB-12	9.4	6.7	8.2	6.4	7.0	6.4	5.8	5.5	5.2	4.9
	WB-15	12.8	6.7	11.3	7.3	9.8	6.7	8.8	6.4	6.7	5.5
	WB-19	-	-	-	-	-	-	-	-	11.9	7.0
	WB-20	-	-	-	-	-	-	-	-	11.9	7.6

		d ₂ for cases A and B where:									
		R=4.5 m		R=6 m		R=7.5 m		R=9 m		R=12 m	
Design	vehicle	A	B	A	B	A	B	A	B	A	B
120°	SU	10.4	6.7	8.2	5.8	6.4	5.5	5.2	4.9	4.0	4.0
	BUS	14.0	8.5	12.2	7.6	9.8	7.0	7.9	5.8	5.8	5.5
	WB-12	11.3	7.0	8.8	6.7	7.3	6.7	5.8	5.5	5.2	4.9
	WB-15	15.2	8.8	13.1	8.5	11.0	8.2	9.1	7.9	6.7	5.5
	WB-19	-	-	-	-	-	-	-	-	7.9	6.7
	WB-20	-	-	-	-	-	-	-	-	9.1	7.0
150°	SU	12.2	7.6	9.8	6.4	6.7	5.8	5.2	4.9	3.6	3.6
	BUS	14.6	8.5	12.2	7.6	9.8	7.0	6.7	5.5	5.2	4.9
	WB-12	11.9	7.3	8.8	6.7	7.0	6.7	5.8	5.5	5.2	4.9
	WB-15	16.2	9.4	14.0	8.5	11.0	8.2	8.5	7.9	6.7	5.5
	WB-19	-	-	-	-	-	-	-	-	6.1	5.5
	WB-20	-	-	-	-	-	-	-	-	8.2	5.5

Note: P Design Turns within 3.6m width where R=4.5m or more. No parking on either street.

CASE A
Vehicle turns from proper lane and swings wide on cross street
d₁=3.6m d₂ is variable

CASE B
Turning vehicle turns swings equally wide on both streets
d₁=d₂ Both variable

With parking allowed along a curbed street, vehicles (except for WB-19 [WB-62] and larger vehicles) are able to turn without encroachment onto adjacent lanes, even where curb radii are relatively small. As shown in figure (9-3/8), the SU and WB-12 [WB-40] design vehicles are able to turn at a 4.5-m curb radius with little, if any, encroachment on adjacent lanes. However, parking should be restricted for a distance of at least 4.5 m in advance of the right-turning radius and at least 9 m beyond the radius on the exit.

The BUS and WB-15 [WB-50] design vehicles will encroach onto the opposing lanes in making a turn unless the turning radius is at least 7.5 m and parking is restricted at the far end of the turn for at least 12 m beyond the radius.

Because traffic volumes may increase to the point where all parking is prohibited either during rush hours or throughout the day, caution is advised in the use of radii of 4.5 or 7.5 m where parking is permitted. If parking is prohibited, the same turning conditions prevail as shown in figures (9-3/6), (9-3/7), and table (9-3/3).

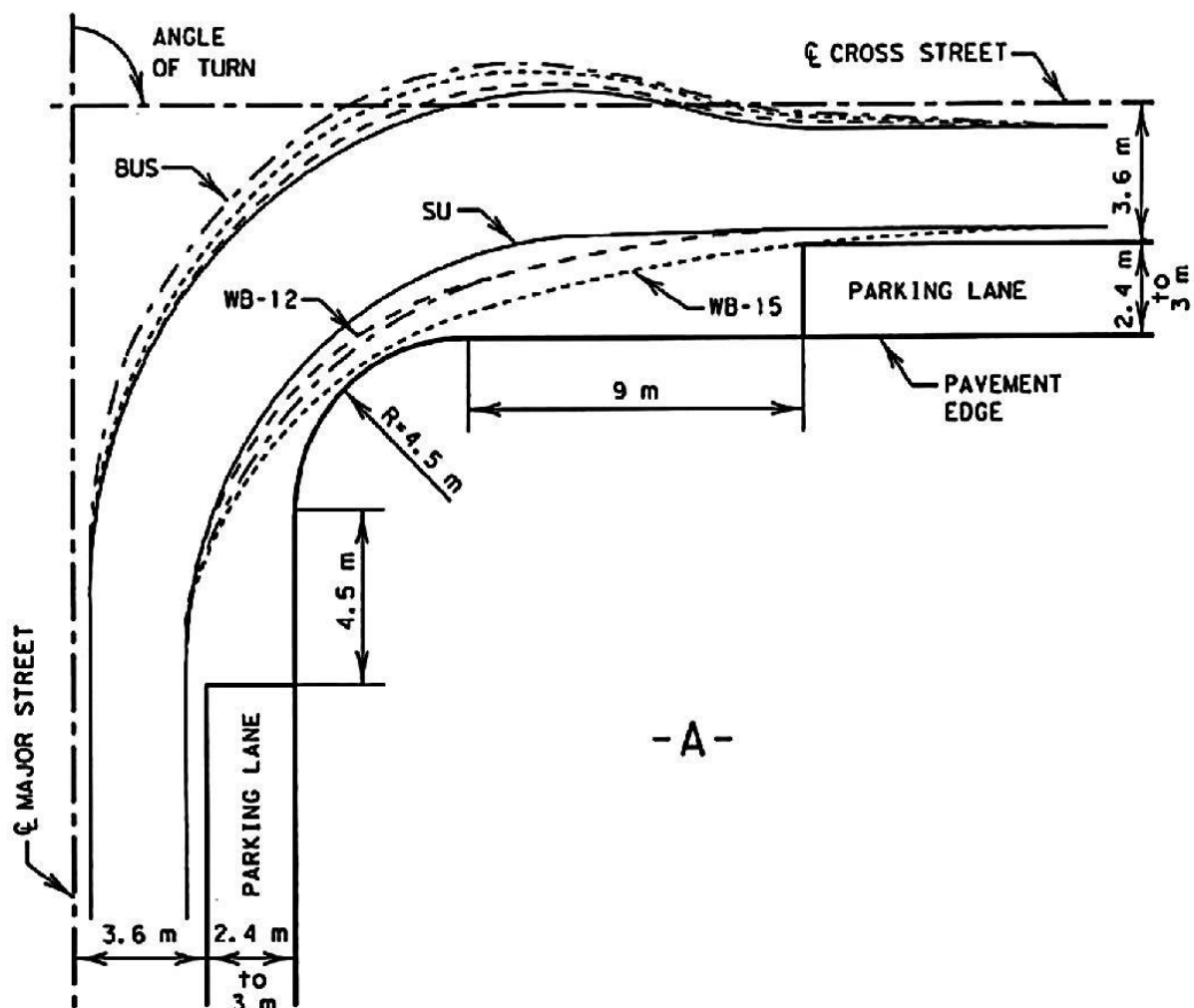


Figure 9-3/8: Effect of Curb Radii and Parking on Right Turning Paths [4, p.621]

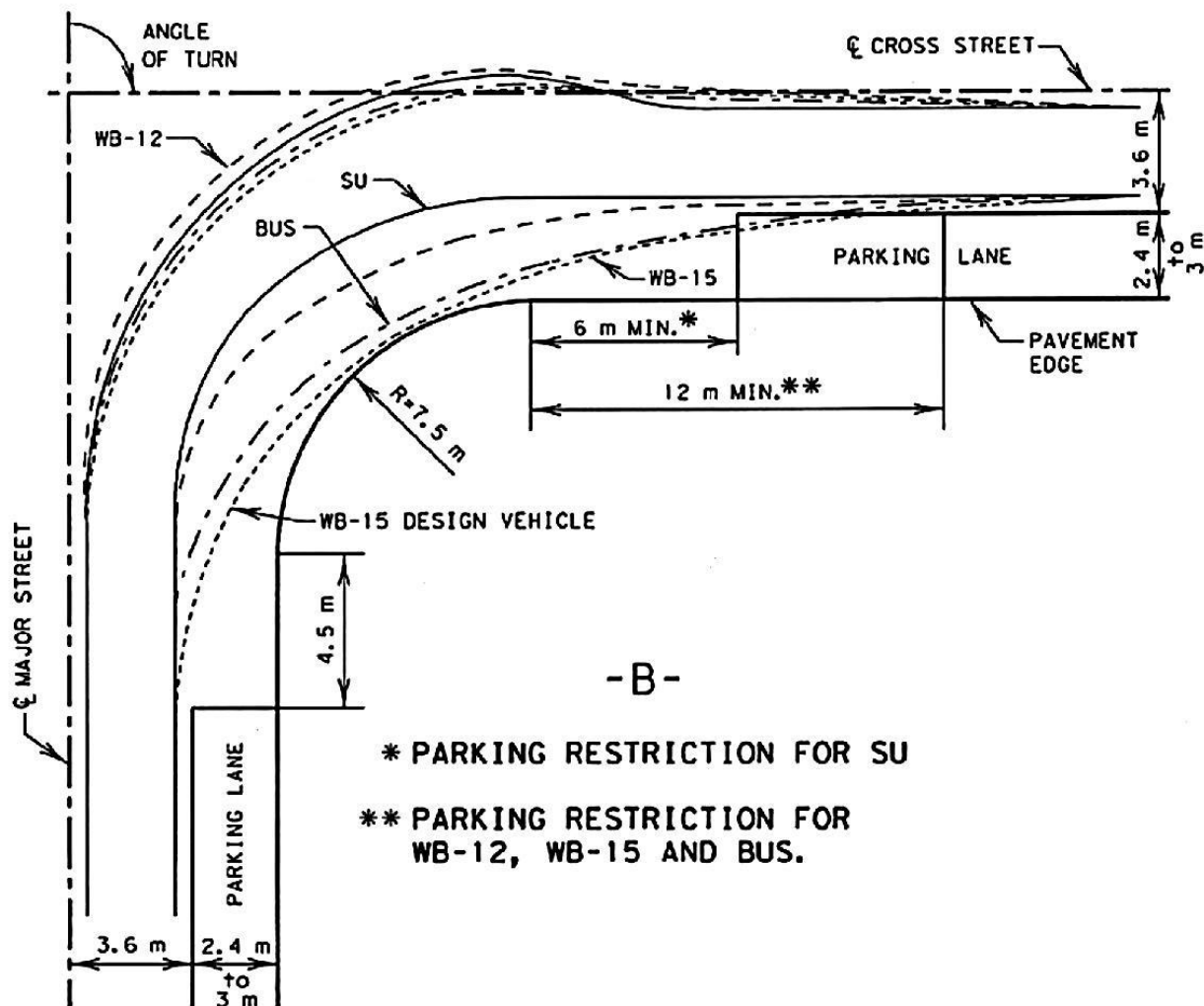


Figure 9-3/8: Effect of Curb Radii and Parking on Right Turning Paths [4, p.621]
(Continued)

9-3/2 CORNER RADII INTO LOCAL URBAN STREETS

Because of space limitations, presence of pedestrians, and generally lower operating speed in urban areas, curve radii for turning movements may be smaller than those normally used in rural areas.

Guidelines for right-turning radii into minor side streets in urban areas usually range from 1.5 to 9 m and most are between 3 and 4.5 m. Where a substantial number of pedestrians are present, the lower end of the ranges described below may be appropriate. Most passenger cars operating at very low speed on lanes 3 m or more in width are able to make a right turn with a curb radius of about 4.5 m with little encroachment on other lanes. However, operation of these vehicles at increased speeds or of larger vehicles even at a very low speed generally results in substantial encroachment on adjacent lanes at either the beginning or the end of the turn, or both.

Where there are curb parking lanes on both of the intersecting streets and parking is restricted for some distance from the corner, the extra width provided by the restriction serves to increase the usable radius. On most streets, curb radii of 3 to 4.5 m are reasonable because streets and sidewalks are generally confined within the public right-of-way, and larger radii can be obtained only by narrowing sidewalks at corners and increasing the length of pedestrian crosswalks.

However, to ensure efficient traffic operation on arterial streets carrying heavy traffic volumes, it is desirable to provide corner radii of 4.5 to 7.5 m for passenger vehicles and 9 to 15 m for most trucks and buses, provided there are no significant pedestrian conflicts. Where large truck combinations turn frequently, somewhat larger radii should be provided for turns.

9-4 DEVELOPMENT OF SUPERELEVATION AT TURNING ROADWAY TERMINALS

The general factors that control the maximum rates of superelevation for open highway conditions as discussed in Chapter 5 also apply to turning roadways at intersections.

For the superelevation runoff design in turning roadways, usually, the profile of one edge of the traveled way is established first, and the profile on the other edge is developed by stepping up or down from the first edge by the amount of desired superelevation at that location. This step is done by plotting a few control points on the second edge by using the maximum relative gradients in table (9-4/1) and then plotting a smooth profile for the second edge of traveled way. Drainage may be an additional control, particularly for curbed roadways.

Table 9-4/1: Effective Maximum Relative Gradients [4, p. 647]

Design speed (km/hr.)	Effective maximum relative gradient (%)		
	Rotated width (m)		
	3.6 m	5.4 m	7.2 m
20	0.80	0.96	1.00
30	0.75	0.90	1.00
40	0.70	0.84	0.93
50	0.65	0.78	0.87
60	0.60	0.72	0.80
70	0.55	0.66	0.73
80	0.50	0.60	0.67
90	0.47	0.57	0.63
100	0.44	0.53	0.59
110	0.41	0.49	0.55
120	0.38	0.46	0.51
130	0.35	0.42	0.47

The method of developing superelevation at turning roadway terminals is illustrated diagrammatically in figures (9-4/1) through (9-4/4).

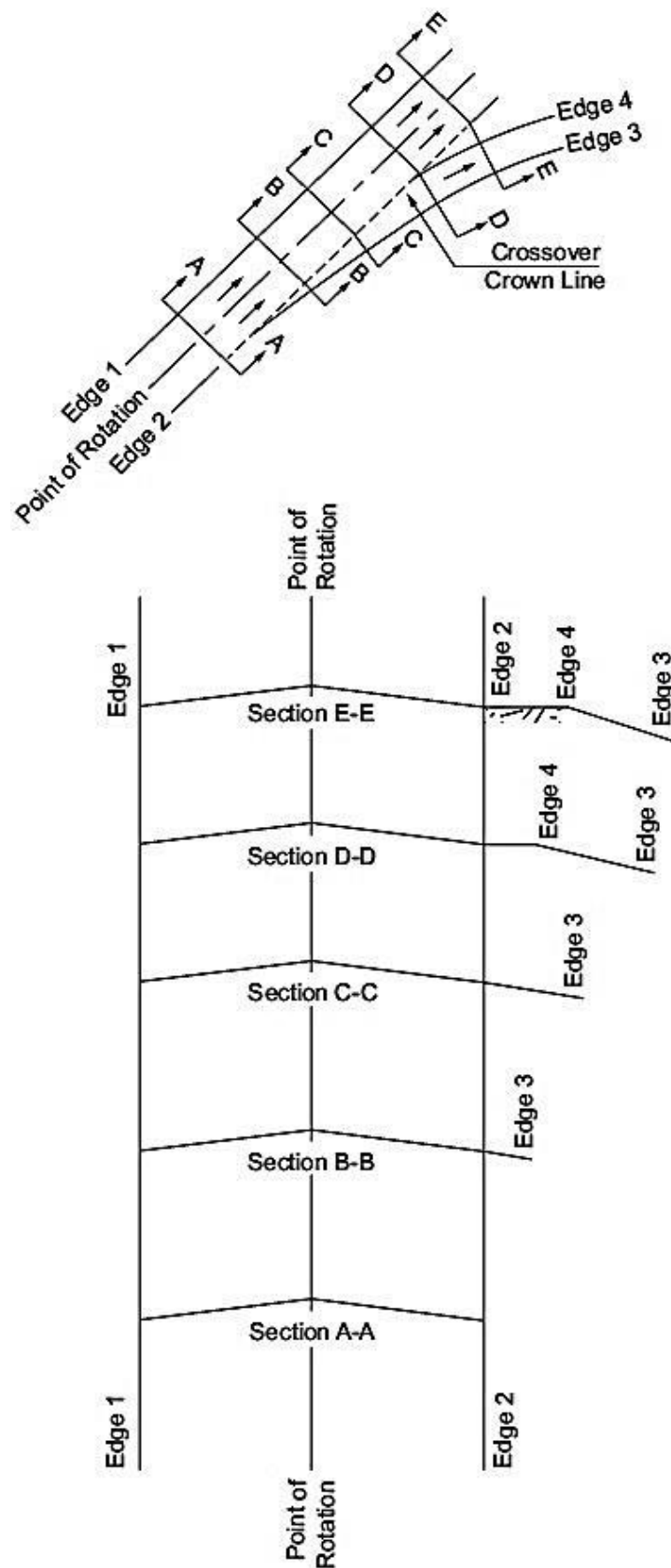


Figure 9-4/1: Development of Superelevation at Turning Roadway Terminals [4, p. 648]

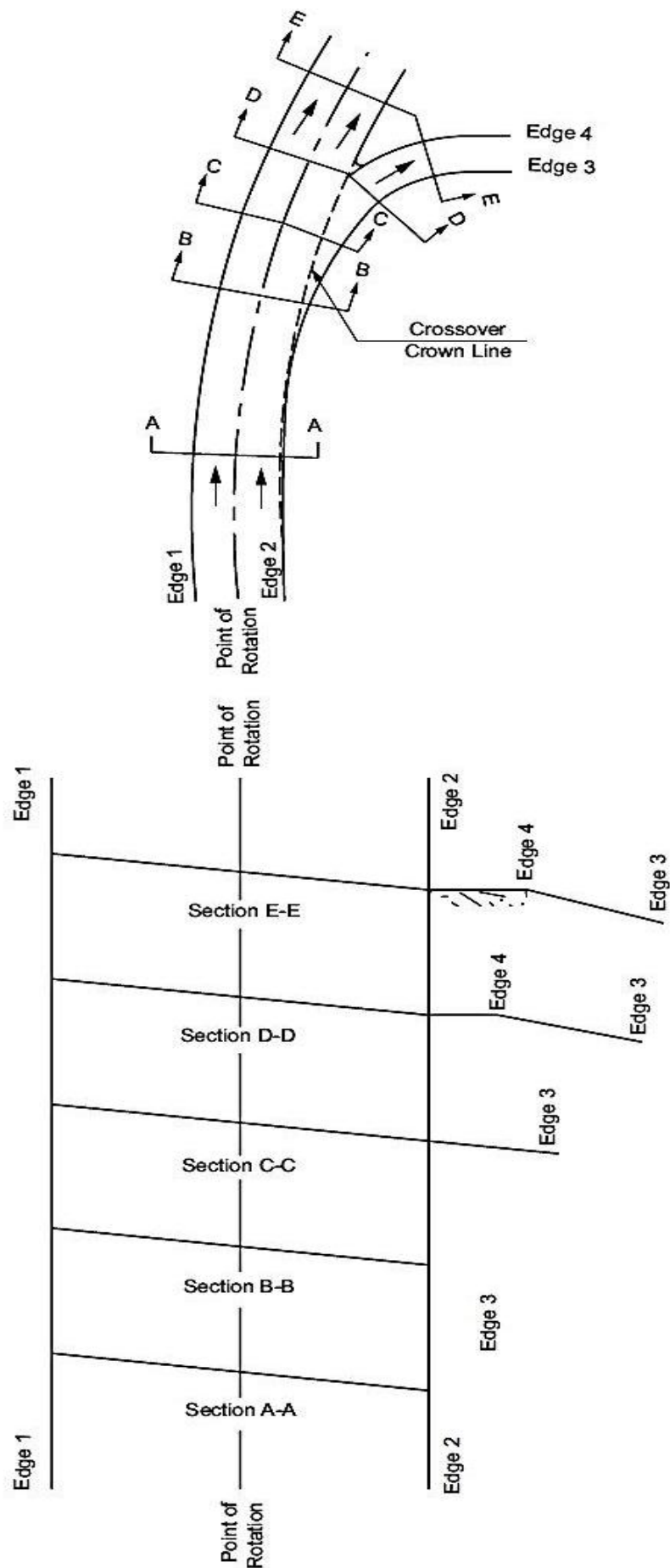


Figure 9-4/2: Development of Superelevation at Turning Roadway Terminals [4, p. 649]

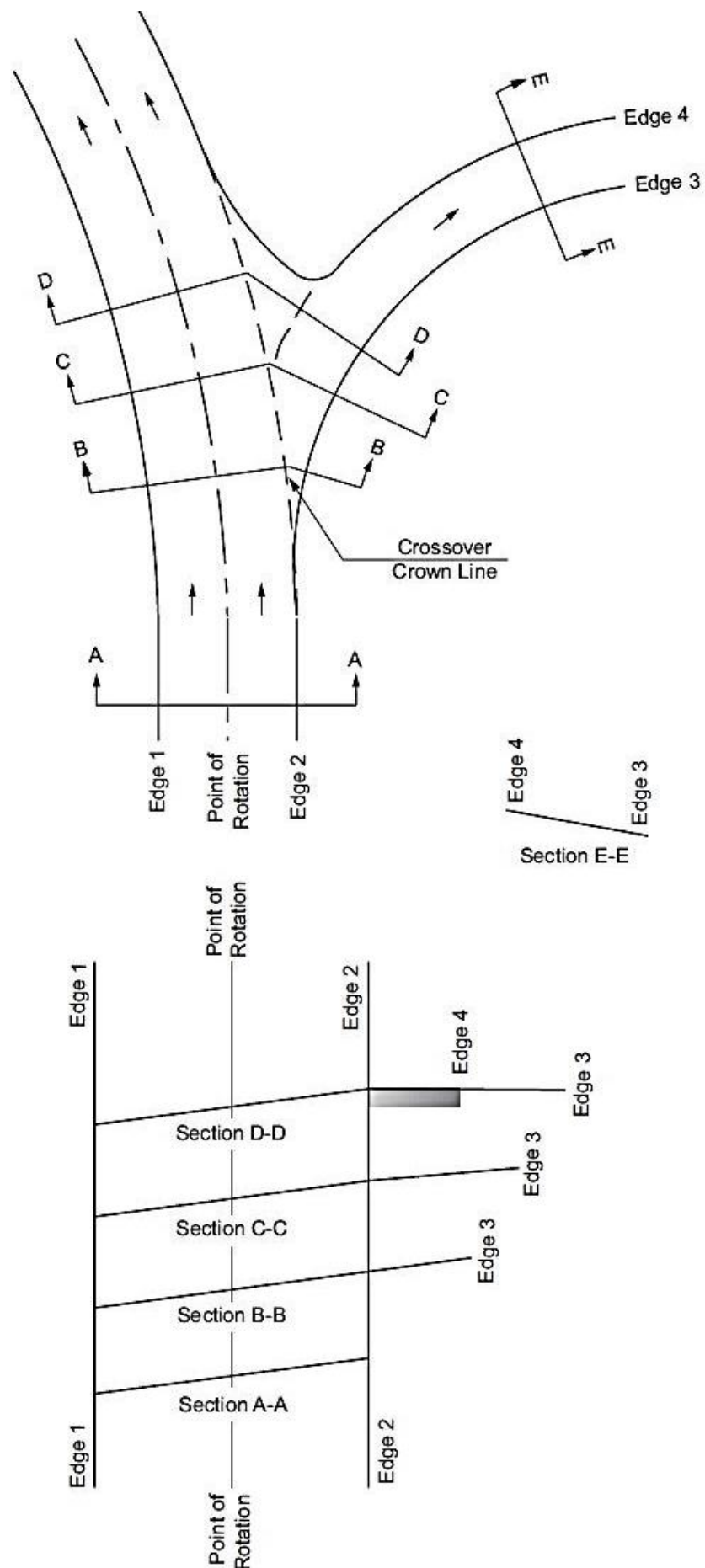


Figure 9-4/3: Development of Superelevation at Turning Roadway Terminals [4, p. 650]

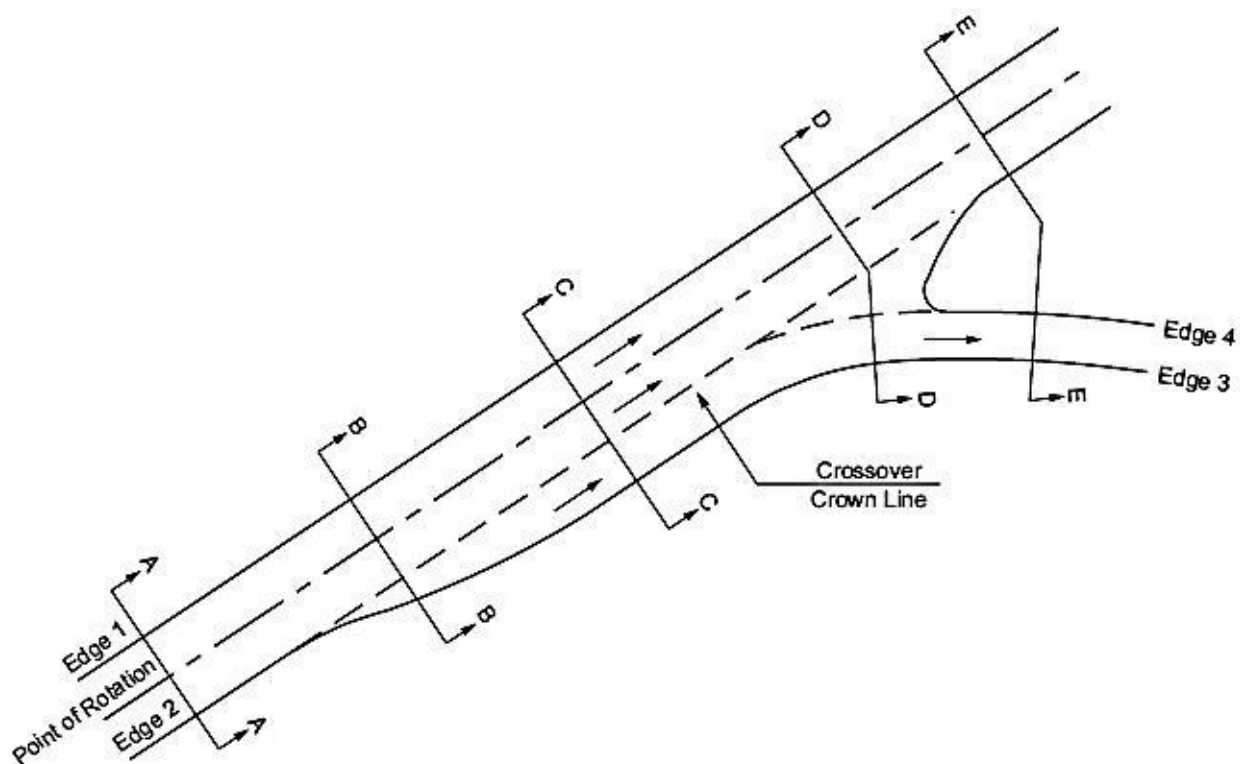


Figure 9-4/4: Development of Superelevation at Turning Roadway Terminals [4, p. 651]

9-5 CLEAR SIGHT TRIANGLES

In general, intersection sight distance (ISD) refers to the corner sight distance available in intersection quadrants that allows a driver approaching an intersection to observe the actions of vehicles on the crossing leg(s). ISD evaluations involve establishing the needed sight triangle in each quadrant by determining the legs of the triangle on the two crossing roadways.

The height of eye for passenger cars is assumed to be 1.08 m above the surface of the minor road. The height of object (approaching vehicle on the major road) is also assumed to be 1.08 m. An object height of 1.08 m assumes that a sufficient portion of the oncoming vehicle must be visible to identify it as an object of concern by the minor road driver.

The necessary clear sight triangle is based on the type of traffic control at the intersection and on the design speeds of the two roadways. The types of traffic control and maneuvers are as follows:

- Case A – Intersections with no control,
- Case B – Intersections with Stop control on the minor road:
- Case C – Intersections with Yield control on the minor road:
- Case D – Intersections with traffic signal control,
- Case E – Intersections with all-way Stop control, and
- Case F – Left turns from the major road.

9-5/1 CASE A – INTERSECTIONS WITH NO CONTROL

Intersections between low-volume and low-speed roads/streets may have no traffic control. At these intersections, sufficient corner sight distance should be available to allow approaching vehicles to adjust their speed to avoid a collision, typically a reduction to 50% of their mid-block running speed. Figure (9-5/1) illustrates the corner sight distance triangles for intersections with no traffic control. Table (9-5/1) provides the ISD criteria for these intersections.

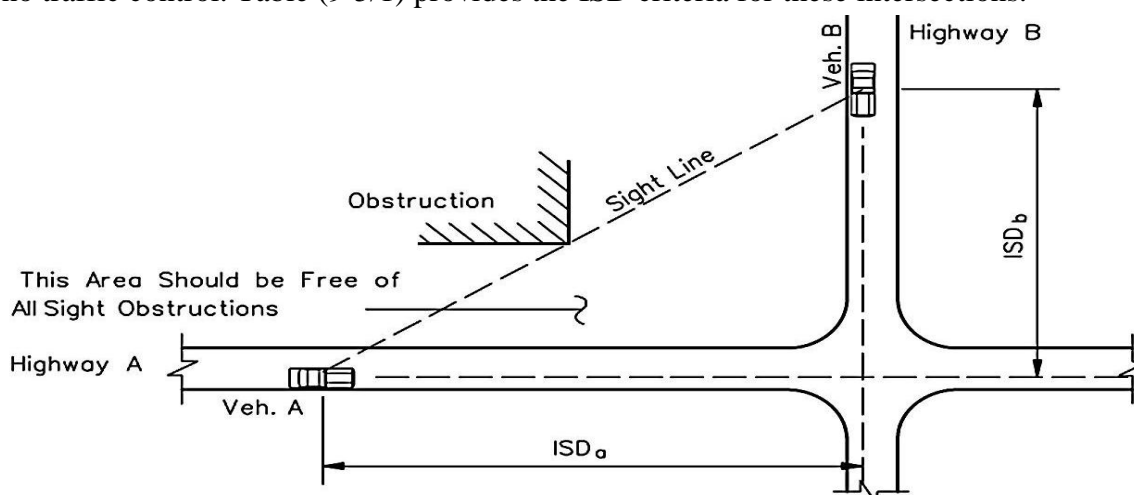


Figure 9-5/1: Measurement of intersection sight distance (No traffic control) [4, p. 656]

Table 9-5/1: Intersection Sight Distance (No Traffic Control) [4,p. 659]

Design Speed (km/hr.)	30	40	50	60	70	80
Intersection Sight Distance (m)	25	35	45	55	65	75

Note: For approach downgrades greater than 3.0%, increase the ISD value by 10%.

9-5/2 CASE B – INTERSECTIONS WITH STOP CONTROL ON THE MINOR ROAD

Where traffic on the minor road of an intersection is controlled by stop signs, the driver of the vehicle on the minor road must have sufficient sight distance for a safe departure from the stopped position assuming that the approaching vehicle comes into view as the stopped vehicle begins its departure.

The intersection sight distance is obtained by providing clear sight triangles both to the right and left as shown in figure (9-5/2). The length of legs of these sight triangles is determined as follows:

The length of the sight triangle leg or ISD along the major road is determined using the following equation:

$$ISD = 0.278 V_{major} t_g \quad (9 - 5/1)$$

Where:

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

V_{major} = Design speed of major road, (km/hr.)

t_g = Gap acceptance time for entering the major road, (sec.)

The critical gap time (t_g) can be obtained from table (9-5/2).

Table 9-5/2: Time Gap for Case B, Left Turn From Stop [4, p. 963]

Design Vehicle	Time Gap (t_g)(sec.) at Design Speed of Major Road
Passenger car	7.5
Single-unit	9.5
Combination	11.5

Note: Time gaps are for a stopped vehicle to turn left onto a two-lane highway with no median and with grades of 3 percent or less. The table values should be adjusted as follows:

For multilane highways-For left turns onto two-way highways with more than two lanes, add 0.5 sec. for passenger cars or 0.7 for trucks for each additional lane, from the left, in excess of one, to be crossed by the turning vehicle.

For minor road approach grades -If the approach grade is an upgrade that exceeds 3 percent, add 0.2 sec. for each percent grade for left turns.

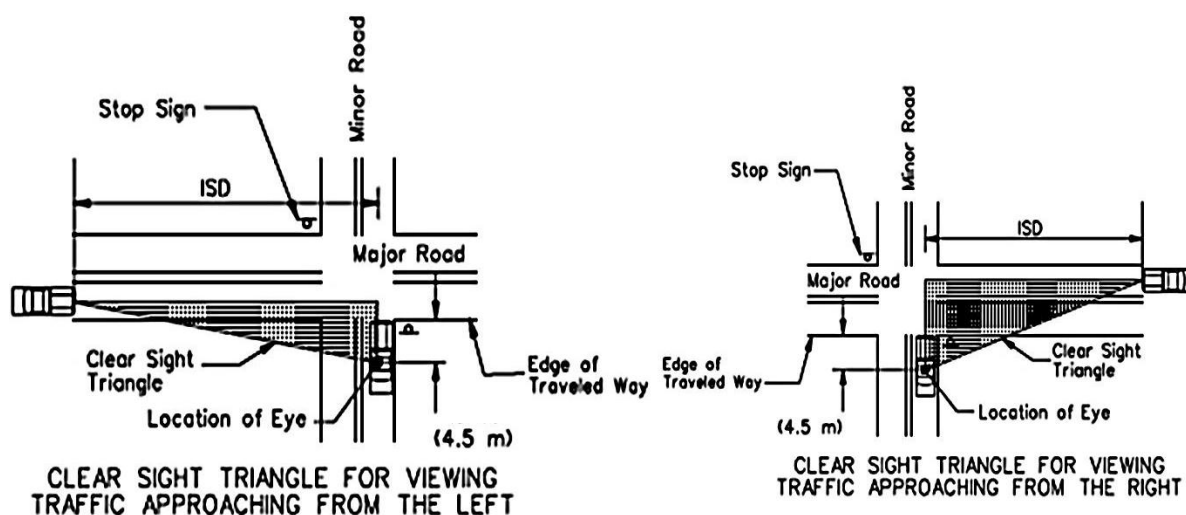


Figure 9-5/2: Clear sight triangle (stop-controlled) intersections [4, p. 963]

9-5/3 CASE C – INTERSECTIONS WITH YIELD CONTROL ON THE MINOR ROAD

At intersections controlled by a yield sign, drivers on the minor road will typically:

- Slow down as they approach the major road to approximately 60% of the approach speed;
- Based on their view of the major road, make a stop/continue decision; and
- Either brake to a stop or continue their crossing or turning maneuver onto the major road.

To determine the applicable clear sight triangles for a yield-controlled intersection, see figure (9-5/3).

Design Speed (km/hr.)	Minor Road Approach (a) (m) ^{1,2}	Major Road Approach (b) (m)
30	30	55
40	40	75
50	55	95
60	65	110
70	80	130
80	100	145
90	115	165
100	135	185

Notes:

1. For "T" intersections, use 25 m .
2. Values shown are for passenger cars crossing a 2-lane facility with no median and grades 3.0% or less. Increase ISD by 10% on minor roads with approach grades exceeding 3.0%.

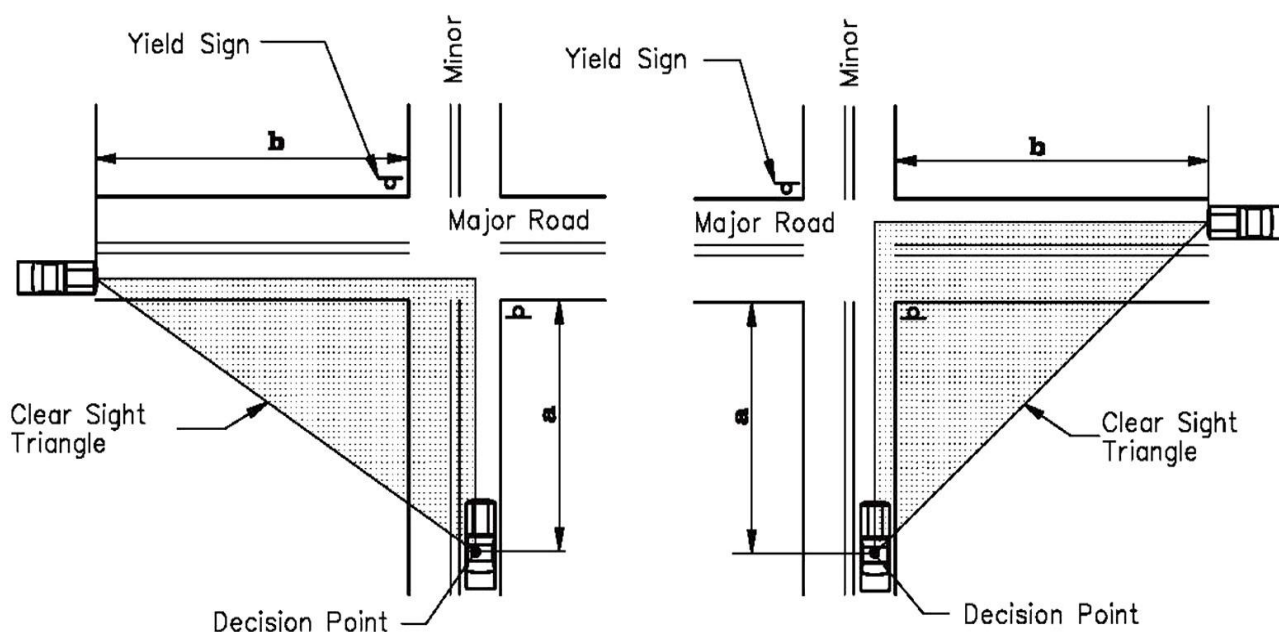


Figure 9-5/3: Intersection sight distance guidelines (yield control) [4, p. 970]

9-5/4 CASE D – INTERSECTIONS WITH TRAFFIC SIGNAL CONTROL

At signalized intersections, provide sufficient sight distance from the stop bar so that the first vehicle on each approach is visible to all other approaches. However, where right-turn-on-red is allowed, check to ensure that the ISD for a stop-controlled right-turning vehicle is available to the left. If it is not, consider restricting the right-turn-on-red movement. In addition, if the traffic signal is placed on two-way flash operation (i.e., flashing amber on the major-road approaches and flashing red on the minor-road approaches) under off-peak or nighttime conditions, consider providing the ISD criteria as discussed in case B for a stop-controlled intersection.

9-5/5 CASE E – INTERSECTIONS WITH ALL-WAY STOP CONTROL

At intersections with all-way stop control, provide sufficient sight distance from the stop bar so that the first stopped vehicle on each approach is visible to all other approaches. Often, intersections are converted to all-way stop control to address limited sight distance at the intersection. Therefore, providing additional sight distance at the intersection is unnecessary.

9-5/6 CASE F – LEFT TURNS FROM THE MAJOR ROAD

At all intersections, regardless of the type of traffic control, consider the sight distance needs for a stopped vehicle turning left from the major road. This situation is illustrated in figure (9-5/4). The driver will need to see straight ahead for a sufficient distance to turn left and clear the opposing travel lanes before an approaching vehicle reaches the intersection. Sight distance for opposing left turns may be increased by offsetting the left-turn lanes. Figure (9-5/4) provides ISD values for passenger cars turning left from the major road.

Design Speed (km/hr.)	ISO Crossing 1-Lane (m)	ISD Crossing 2-Lanes (m)
30	50	55
40	62	69
50	75	81
60	87	94
70	99	108
80	111	122
90	123	136
100	136	149

Note: Assumes no median on major road.

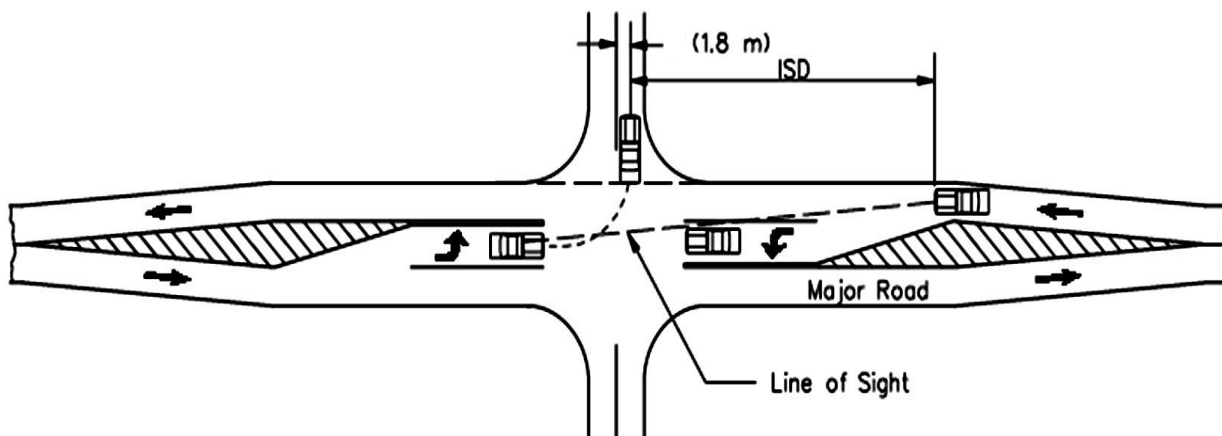


Figure 9-5/4: Intersection Sight Distance for Stopped Vehicle Turning Left (on Major Road) [4, p. 978]

9-6 DESIGN OF MEDIAN OPENINGS

For any three- or four-leg intersection on a divided highway, the minimum length of median opening should be as great as the width of the crossroad traveled way plus shoulders. Where the crossroad is a divided highway, the length of opening should be at least equal to the width of the crossroad traveled ways plus that of the median. The use of a minimum length of opening without regard to the width of median or the control radius should not be considered except at very minor crossroads

An important factor in designing median openings is the path of each design vehicle making a minimum left turn at 15 to 25 km/h. Where the volume and type of vehicles making the left-turn movement call for higher than minimum speed, the design may be made by using a radius of turn corresponding to the speed deemed appropriate. However, the minimum turning path at low speed is needed for minimum design and for testing layouts developed for one design vehicle for use by an occasional larger vehicle. The control radii required for each design vehicle are shown in table 9-6/1 below.

Table 9-6/1: Design Controls for Minimum Median Openings [1, p. 9-154]

Design Vehicles Accommodated	Control Radius (m)			
	12	15	23	40
Predominant	P	SU-9	WB-12	WB-19
Occasional	SU-9	SU-12	—	WB-20

The length of a median opening should properly accommodate the control radius of the design vehicle. Tables (9-6/2) through (9-6/4) and figures (9-6/1) through (9-6/4) illustrate median-opening criteria for various design vehicles. It should be noticed at a 4-leg intersection, the offset between the nose and the through travel lane (extended) should be at least 0.6m . Also, the shape of median end is desired to be semicircular for median width equal or less than 3m, for wider median the bullet-nose can be used.

Table 9-6/2: Minimum Design of Median Openings (P Design Vehicle, Control Radius of 12 m) [1, p. 9-145]

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

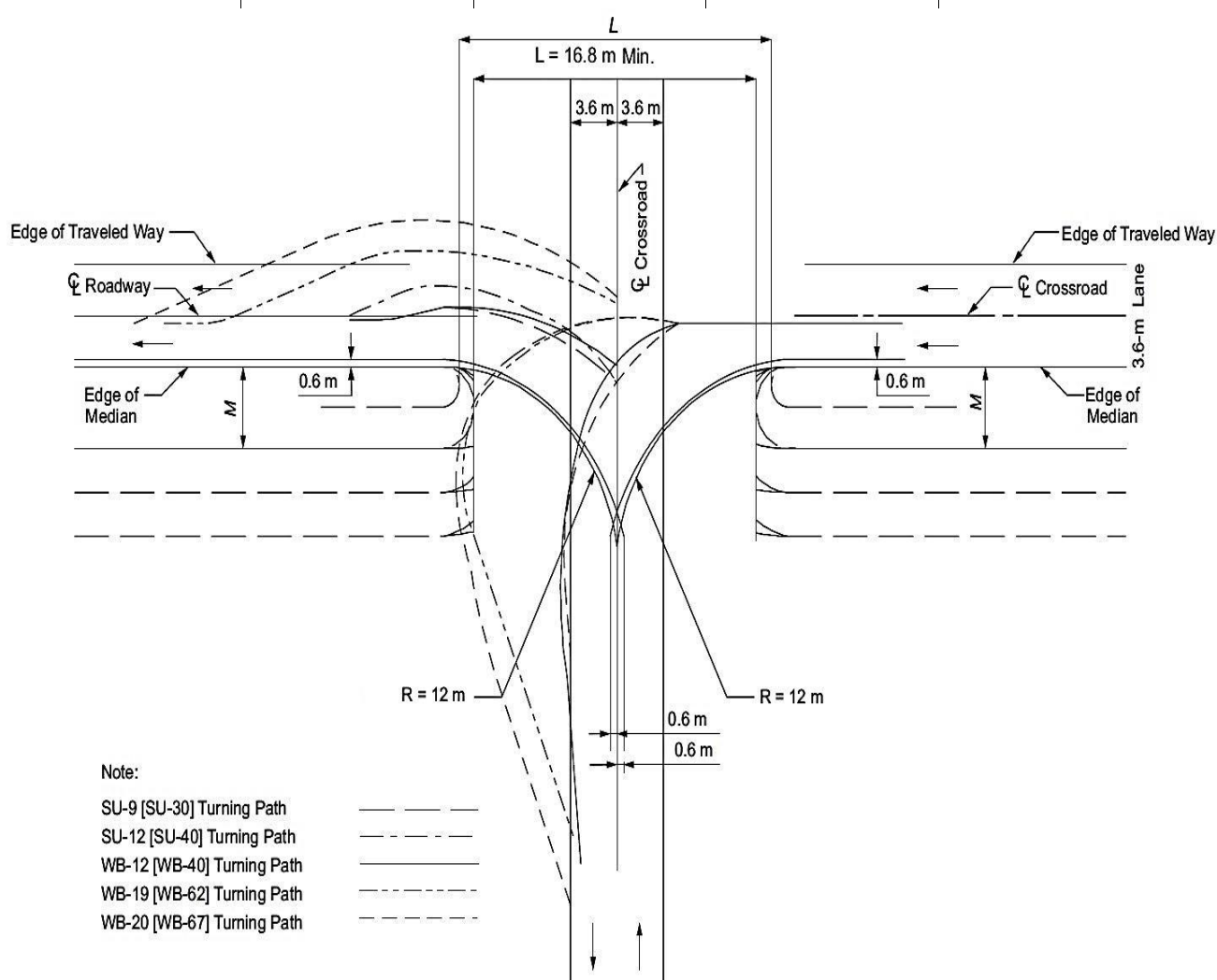


Figure 9-6/1: Minimum Design of Median Openings (P Design Vehicle, Control Radius of 12 m) [1, p. 9-145]

Table 9-6/3: Minimum Design of Median Openings (SU-9 [SU-30] Design Vehicle, Control Radius of 15 m) [1, p. 9-146]

Width of Median, M (m)	Minimum Length of Median Opening, L (m)	
	Semicircular	Bullet Nose
1.2	28.8	28.8
1.8	28.2	22.8
2.4	27.6	20.4
3.0	27.0	18.6
3.6	26.4	17.4
4.2	25.8	16.8
4.8	25.2	16.8
5.0	24.0	16.8
7.2	22.8	16.8
8.4	21.6	16.8
9.6	20.4	16.8
10.8	19.2	16.8
12.0	18.0	16.8

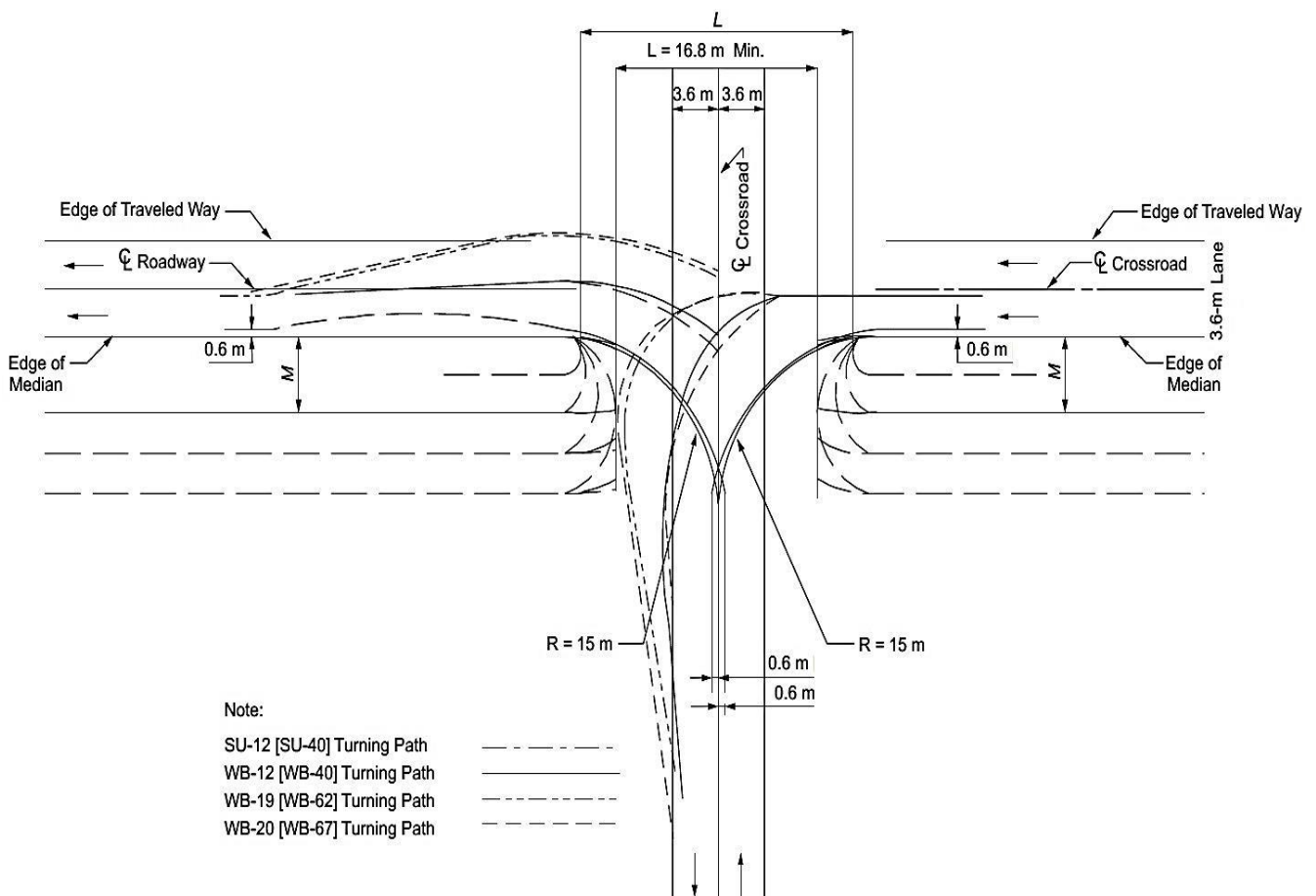


Figure 9-6/2: Minimum Design of Median Openings (SU-9 [SU-30] Design Vehicle, Control Radius of 15 m) [1, p. 9-146]

Table 9-6/4: Minimum Design of Median Openings (WB-12 [WB-40] Design Vehicle, Control Radius of 23 m) [1, p. 9-147]

Width of Median, M (m)	Minimum Length of Median Opening, L (m)	
	Semicircular	Bullet Nose
1.2	43.8	36.6
1.8	43.2	36.3
2.4	42.6	33.6
3.0	42.0	31.2
3.6	41.4	29.4
4.2	40.8	27.6
4.8	40.2	26.4
6.0	39.0	23.4
7.2	37.8	21.6
8.4	36.6	19.5
9.6	35.4	18.0
10.8	34.2	16.2
12.0	30.0	14.7
13.0	27.0	13.3
24.0	21.0	13.2
30.0	15.0	13.2

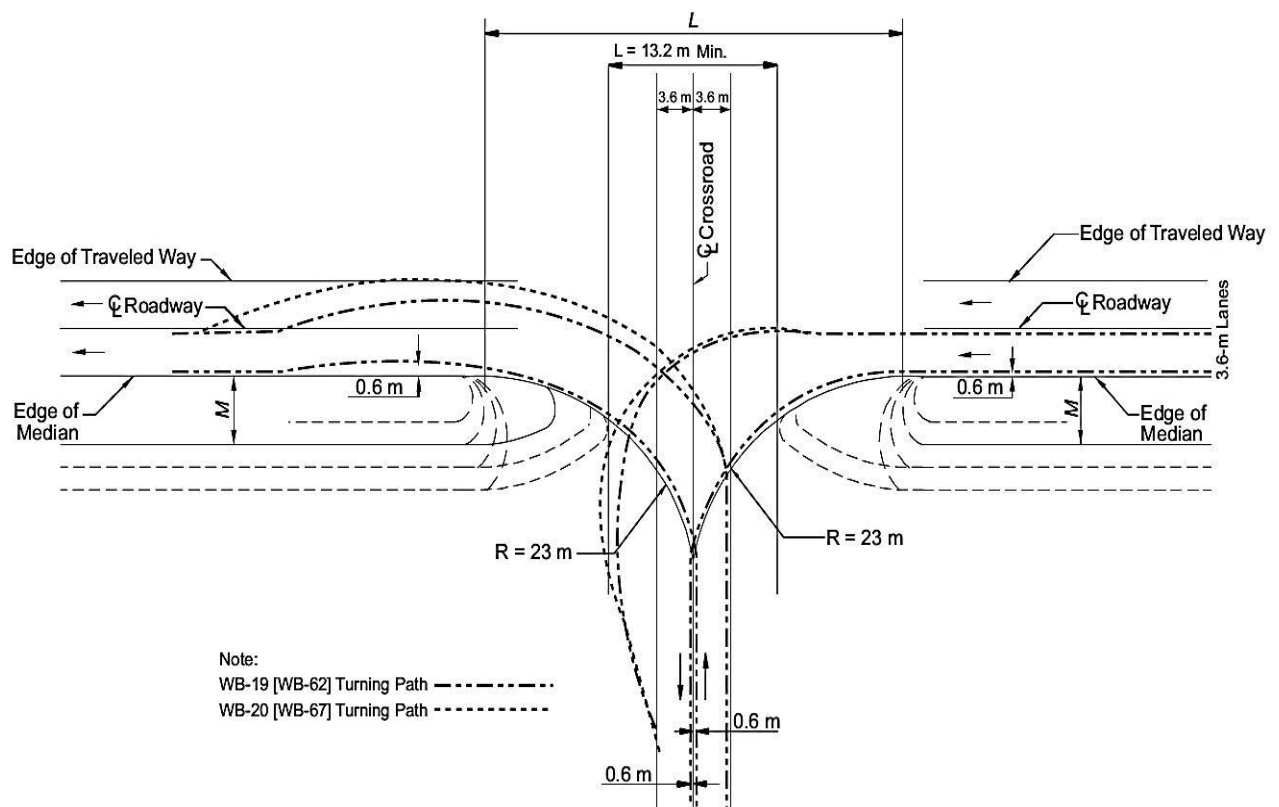


Figure 9-6/3: Minimum Design of Median Openings (WB-12 [WB-40] Design Vehicle, Control Radius of 23 m) [1, p. 9-147]

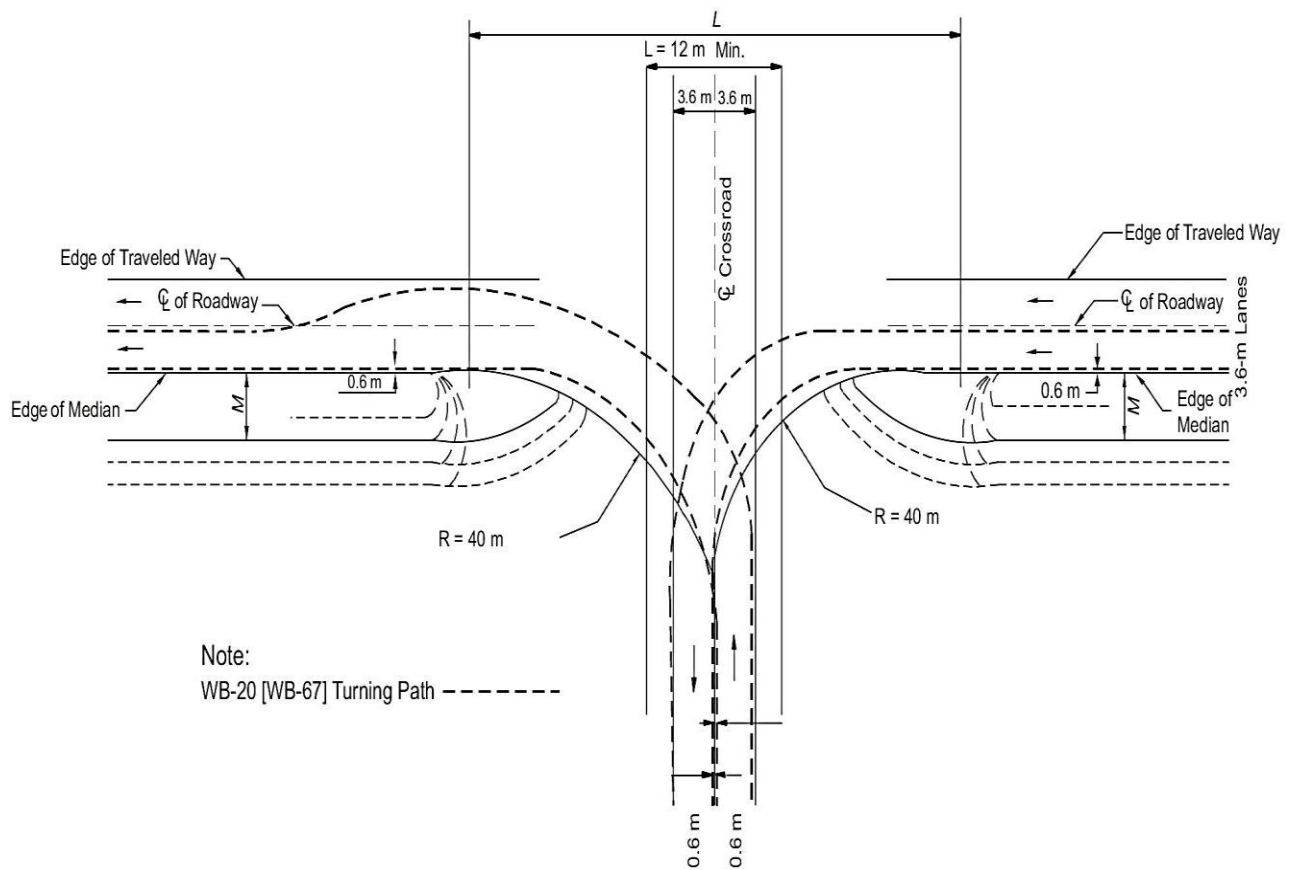
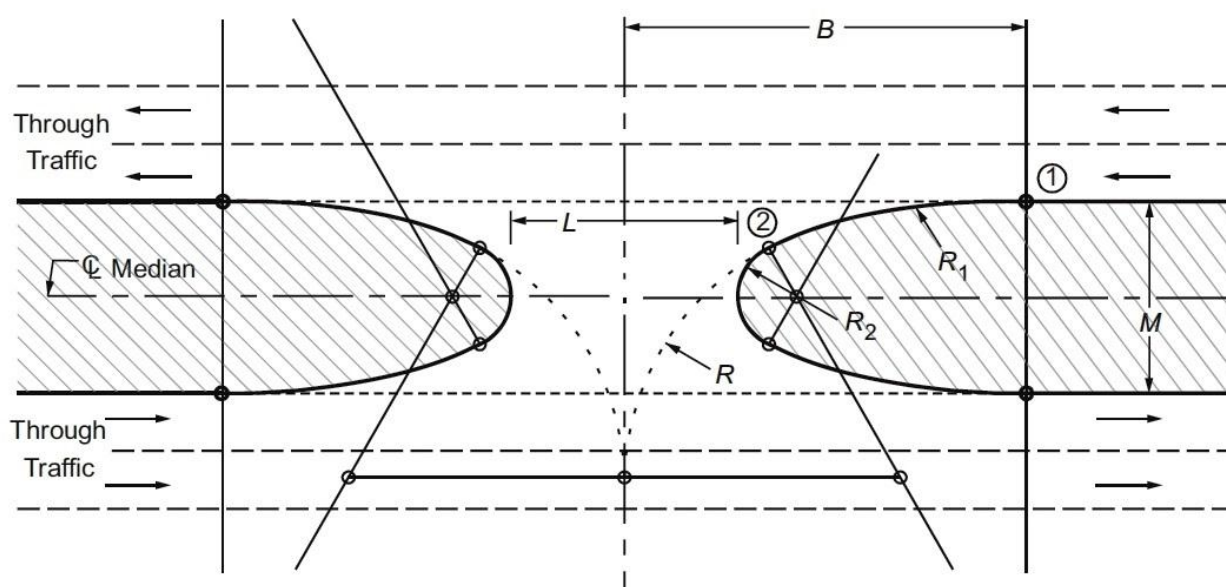


Figure 9-6/4: Minimum Design of Median Openings (WB-20 [WB-67] Design Vehicle, Control Radius of 40 m) [1, p. 9-148]

The median openings (shown in figure above) that enable vehicles to turn on minimum paths and at 15 to 25 km/hr. are adequate for intersections where traffic for the most part proceeds straight through the intersection. Where through-traffic volumes and speeds are high and left-turning movements are important, undue interference with through traffic should be avoided by providing median openings that permit turns without encroachment on adjacent lanes. In this case the "above-minimum design is required.

Where above-minimum designs are required, the general pattern for the simple design is used, but with larger dimensions. For example, in lieu of using a 12m control radius for single-unit vehicles, a 15 m radius could be used. Also, a bullet-nose median end can be used in lieu of the semicircular end. This end treatment improves vehicle travel paths, lessens intersectional pavement, and shortens the length of median opening. Typical bullet-nose ends for above-minimum design of median openings are shown below.



Assumed: $R = 15 \text{ m}$

$$R_2 = \frac{M}{5}$$

Width of Median, M (m)	Dimensions in Meters when					
	$R_1 = 30 \text{ m}$		$R_1 = 50 \text{ m}$		$R_1 = 70 \text{ m}$	
	L	B	L	B	L	B
6.0	18.0	20.2	20.2	24.4	21.3	27.6
9.0	15.1	21.4	17.7	26.5	19.0	30.4
12.0	12.8	22.4	15.6	28.3	17.1	32.7
15.0	—	—	13.8	29.9	15.4	34.7
18.0	—	—	—	—	13.8	36.7
21.0	—	—	—	—	12.4	38.4

Figure 9-6/5: Above-Minimum Design of Median Openings (Typical Bullet-Nose Ends)
[1, p. 9-155]

9-7 AUXILIARY LANE LENGTHS FOR TURNING VEHICLES

The length of the auxiliary lanes for turning vehicles consists of three components: (1) entering taper, (2) deceleration length, and (3) storage length.

Desirably, the total length of the auxiliary lane should be the sum of the length for these three components. Common practice, however, is to accept a moderate amount of deceleration within the through lanes and to consider the taper length as a part of the deceleration within the through lanes. Each component of the auxiliary length is discussed in the following clause.

Auxiliary lanes should be at least 3 m wide and desirably should equal the width of the through lanes.

9-7/1 ENTERING TAPER

Table (9-7/1) below provides the recommended taper rates for various design speeds

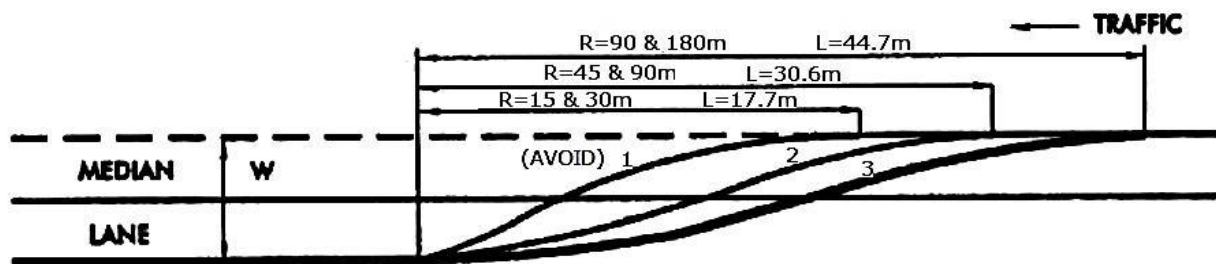
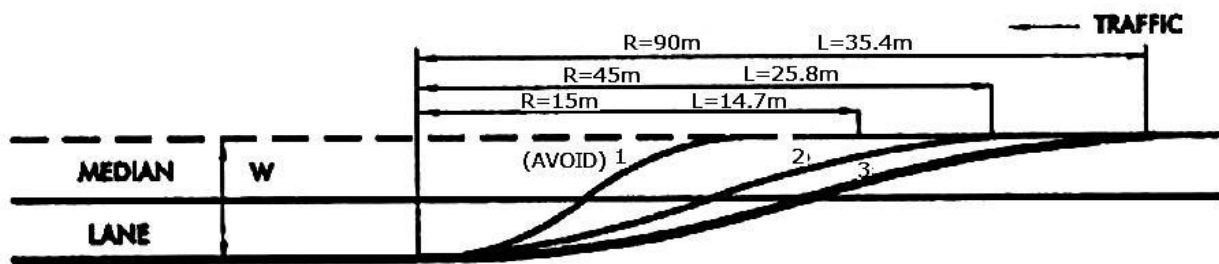
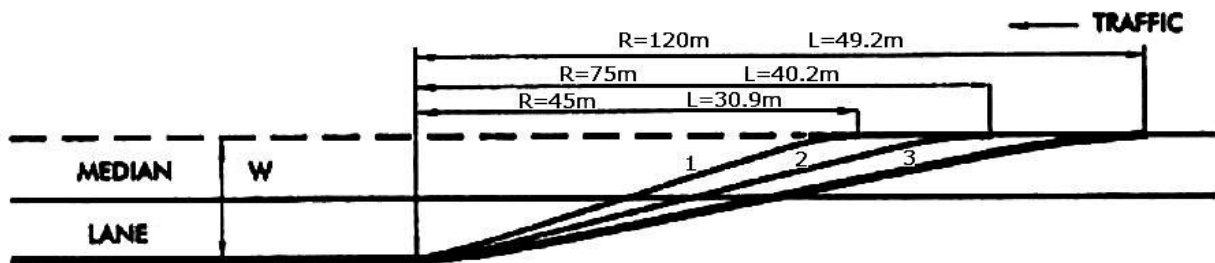
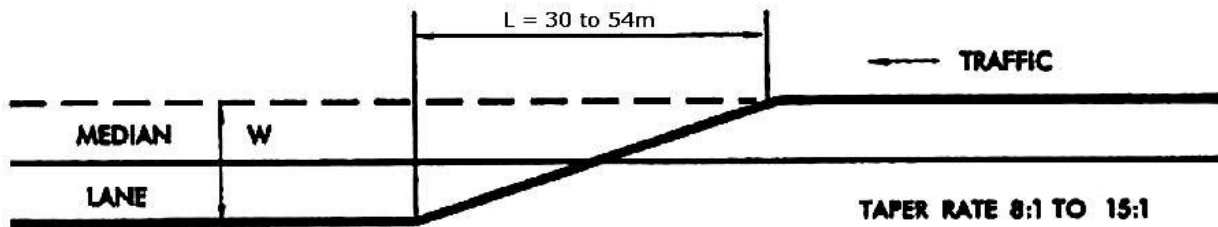
Table 9-7/1: Recommended Taper Rate [1, p. 9-127]

Metric	
Design Speed (km/hr.)	Taper Rate
$V < 60$	8:1
$60 \leq V < 80$	10:1
$80 \leq V < 90$	15:1
$90 \leq V$	18:1

The minimum taper length shall be 30 m for a single-turn lane and 45 m for a dual-turn lane in urban areas and 60 m in rural areas.

Taper can be formed in one of the configurations shown in figure below. Straight-line tapers are particularly applicable where a paved shoulder is striped to delineate the auxiliary lane. Short, straight-line tapers should not be used on curbed urban streets because of the probability of vehicles hitting the leading end of the taper with the resulting potential for a driver losing control.

A short curve is desirable at either end of long tapers as shown in figure (9-7/1B), but may be omitted for ease of construction. Where curves are used at the ends, the tangent section should be about one-third to one-half of the total length. Symmetrical reverse curve tapers are commonly used on curbed urban streets. A more desirable reverse-curve taper is shown in figure (9-7/1D) where the turnoff curve radius is about twice that of the second curve. When 30 m or more in length is provided for the tapers in figure (9-7/1D), tapers 1 and 2 would be suitable for low-speed operations. All the dimensions and configurations shown in figure (9-7/1) are applicable to right-turn lanes as well as left-turn lanes.



DIMENSIONS ARE ALSO APPLICABLE FOR RIGHT-TURN FLARES.

Figure 9-7/1: Taper Design for Auxiliary Lanes [4, p. 721]

9-7/2 DECELERATION LENGTH

Provision for deceleration clear of the through-traffic lanes is a desirable objective on arterial roads and streets and should be incorporated into design, whenever practical. The approximate total lengths needed for a comfortable deceleration to a stop from the full design speed of the highway are as follows: for design speeds of 50, 60, 70, 80, and 90 km/h, the desirable deceleration lengths of the auxiliary lane are 70, 100, 130, 165, and 205 m, respectively. These approximate lengths are based on grades of less than 3 percent.

9-7/3 STORAGE LENGTH

At unsignalized intersections, the storage length, exclusive of taper, may be based on the number of turning vehicles likely to arrive in an average two-minute period within the peak hour. Space for at least two passenger cars should be provided; with over 10 percent truck traffic, provisions should be made for at least one car and one truck.

At signalized intersections the storage length should usually be based on one and one-half to two times the average number of vehicles that would store per cycle, which is predicated on the design volume. As in the case of unsignalized intersections, provision should be made for storing at least two vehicles.

Where turning lanes are designed for two-lane operation, the storage length is reduced to approximately one-half of that needed for single-lane operation

The HCM (3) indicates that exclusive left-turn lanes at signalized intersections should be installed as follows:

1. Where fully protected, left-turn phasing is to be provided;
2. When left-turn volumes are higher than 100 vph, an exclusive left-turn should be considered. Dual left-turn lanes should be considered when left turn hourly volumes exceed 300 vph.

9-8 MINIMUM DESIGN OF U-TURNS

Median openings designed to accommodate vehicles making U-turns only are needed on some divided highways in addition to openings provided for cross and left-turning movements.

For a satisfactory design for U-turn maneuvers, the width of the highway, including the median, should be sufficient to permit the design vehicle to turn from an auxiliary left-turn lane in the median into the lane next to the outside shoulder or outside curb and gutter on the roadway of the opposing traffic lanes.

The minimum widths of median to accommodate U-turns by different design vehicles turning from the lane adjacent to the median are given in figure (9-8/1).

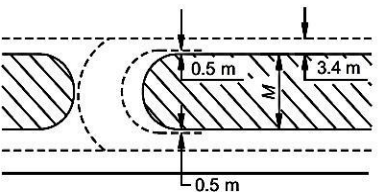
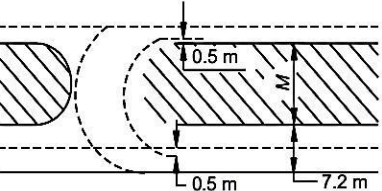
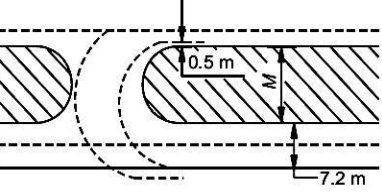
Type of Maneuver		M—Minimum Width of Median (m) for Design Vehicle						
		P	WB-12	SU-9	BUS	SU-12	WB-19	WB-20
		Length of Design Vehicle (m)						
		5.7	15.0	9.0	12.0	12.0	21.0	22.4
Inner Lane to Inner Lane		9	18	19	19	23	21	21
Inner Lane to Outer Lane		5	15	15	16	19	17	17
Inner Lane to Shoulder		2	12	12	12	16	14	14

Figure 9-8/1: Minimum Designs for U-turns [1, p. 9-166]

Figure (9-8/2) illustrates special U-turn designs with narrow medians.

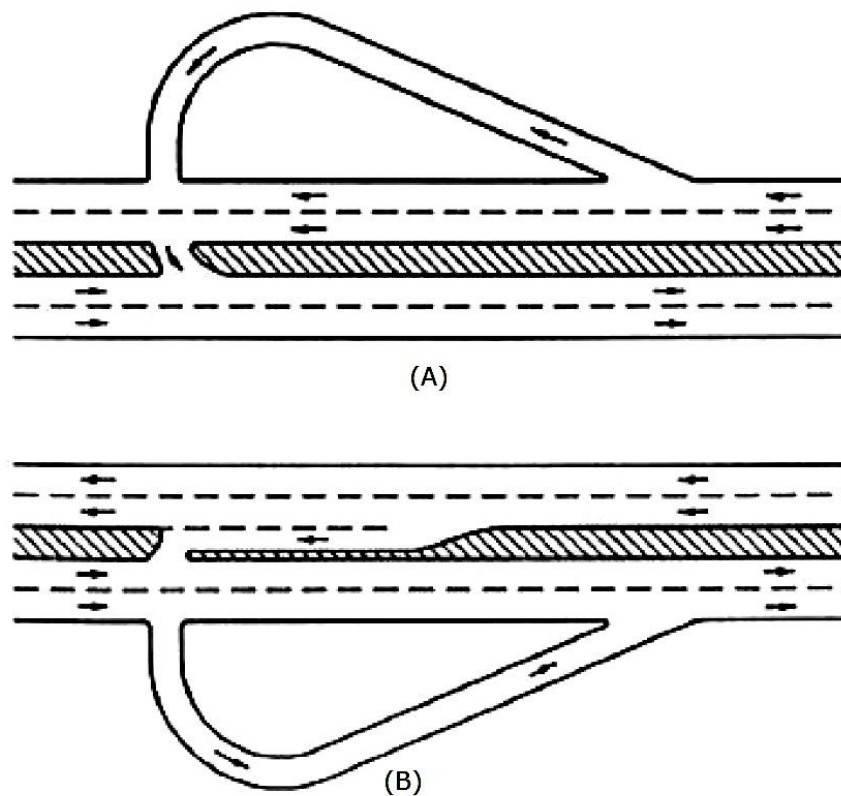


Figure 9-8/2: Special Indirect U-Turn with Narrow Medians [4, p.716]

9-9 RAILROAD-HIGHWAY GRADE CROSSINGS

The following guidelines affect the horizontal alignment of streets at a railroad-highway grade crossing:

- The highway should intersect tracks as near as possible to 90 degrees.
- To the extent practical, crossings should not be located on either highway or railroad curves.
- Ideally, there should not be nearby intersections with streets or driveways. Where it is not possible to provide sufficient distance between the crossing and nearby intersections, traffic signals at the nearby intersection can be interconnected with the grade crossing signal, to enable vehicles to clear the grade crossing as a train approaches.

The following guidelines apply to the vertical alignment of streets at railroad highway grade crossings:

- It is desirable that the intersection of highway and railroad be made as level as practical, otherwise:
- The crossing surface should be at the same plane as the top of the rails for a distance of 0.6 m outside the rails
- The surface of the highway should also not be more than 75 mm higher or lower than the top of nearest rail at a point 9 m from the rail unless track superelevation makes a different level appropriate, as shown in figure (9-9/1).
- Vertical curves should be used to traverse from the highway grade to a level plane at the elevation of the rails

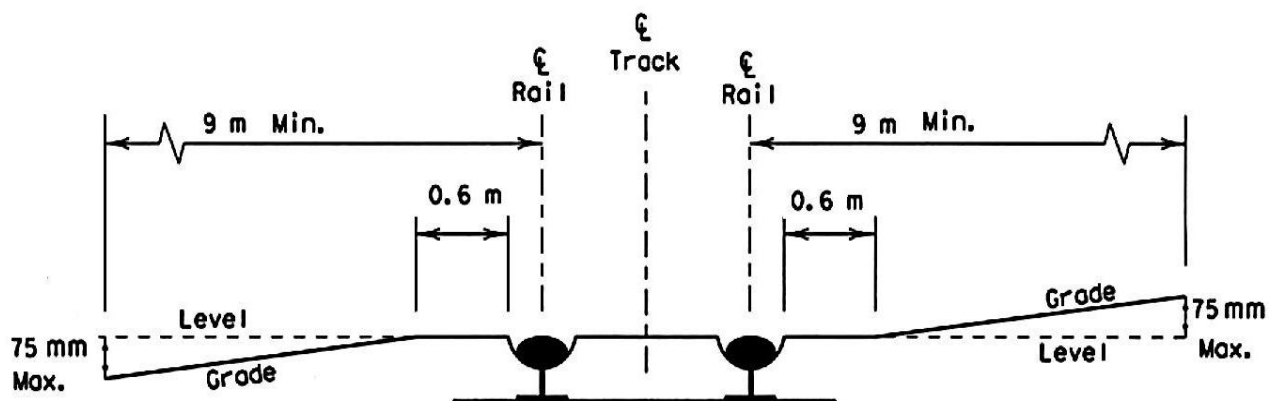


Figure 9-9/1: Railroad-Highway Grade Crossing [4, p. 736]

Traffic control devices for railroad-highway grade crossings range from passive (signs, pavement markings) to active (flashing light signals or automatic gates).

At crossings without traffic control devices, the sight distance requirements are shown in table (9-9/1) for Case A (Moving Vehicle to Safely Cross or Stop at Railroad Crossing) and case B (Departure of Vehicle from Stopped Position to Cross Single Railroad Track), the maneuver of case A and B are shown in figure (9-9/2) below.

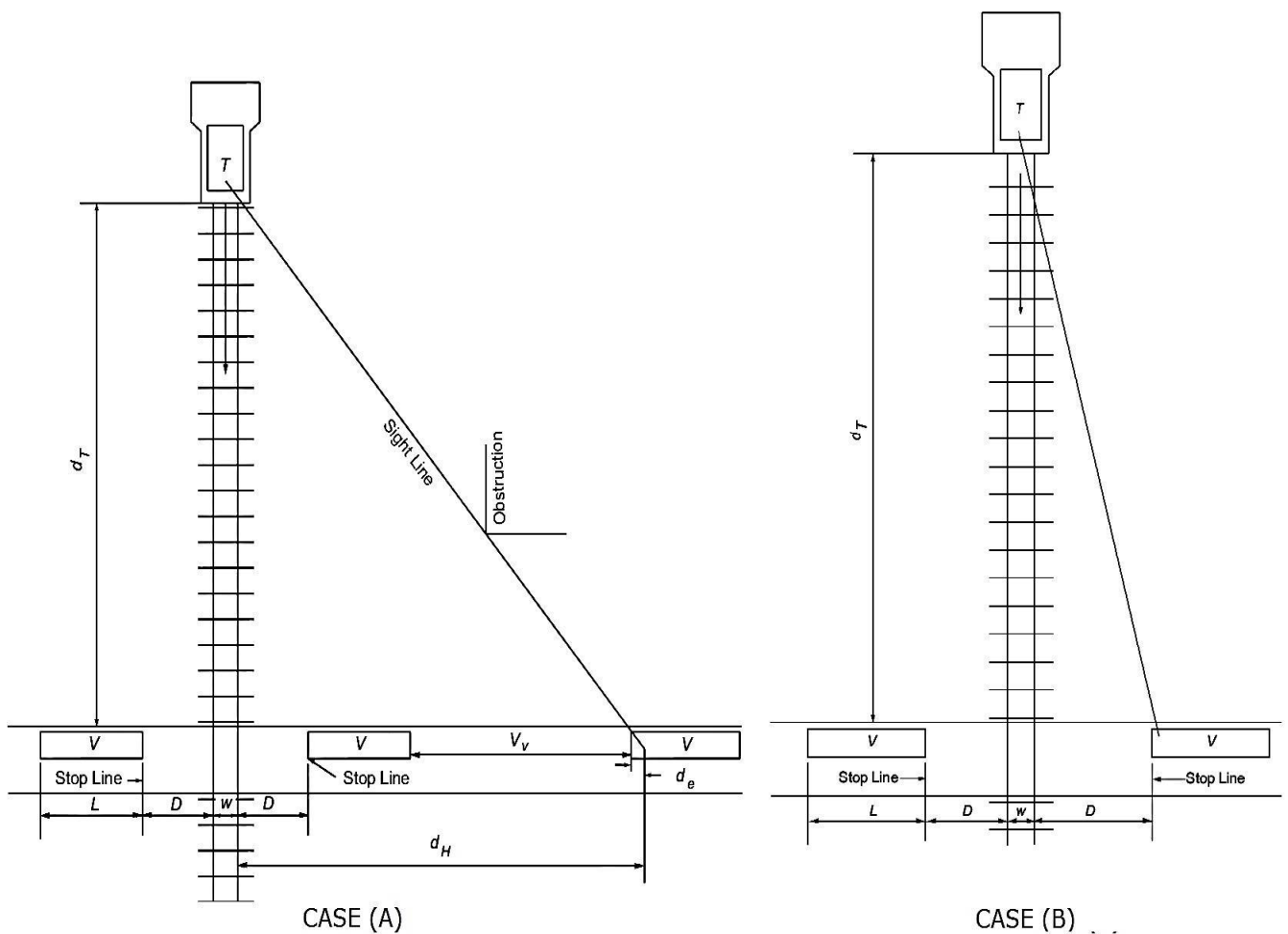


Figure 9-9/2: Maneuver of Cases A and B [1, p.9-190,188]

Table 9-9/1: Design Sight Distance for Combination of Highway and Train Vehicle Speeds; 22.4-m Truck Crossing a Single Set of Tracks at 90 Degrees [1, p.9-191]

Train Speed km/hr.	Case B Departure from Stop	Case A Moving Vehicle												
		Vehicle Speed (km/h)												
		0	10	20	30	40	50	60	70	80	90	100	110	120
		Distance along railroad from crossing, d_T (m)												
10	48	41	26	21	20	19	19	20	20	21	22	23	23	24
20	96	82	51	43	40	39	39	39	40	42	43	45	47	49
30	143	123	77	64	59	58	58	59	61	63	65	68	70	73
40	191	164	103	85	79	77	77	79	81	84	87	90	94	98
50	239	205	128	107	99	96	97	98	101	105	109	113	117	122
60	287	246	154	128	119	116	116	118	121	126	130	135	141	146
70	334	287	180	150	138	135	135	138	142	147	152	158	164	171
80	382	328	206	171	158	154	155	157	162	167	174	180	188	195
90	430	369	231	192	178	173	174	177	182	188	195	203	211	220
100	478	410	257	214	198	193	193	197	202	209	217	226	235	244
110	526	451	283	235	217	212	212	216	223	230	239	248	258	268
120	573	492	308	256	237	231	232	236	243	251	261	271	281	293
130	621	533	334	278	257	250	251	256	263	272	282	293	305	317
140	669	574	360	299	277	270	270	276	283	293	304	316	328	341
		Distance along Highway from Crossing, d_H (m)												
		15	25	38	53	70	90	112	136	162	191	222	255	291

9-10 SIGNALIZED INTERSECTION

9-10/1 MEASURE OF EFFECTIVENESS

Control delay is used as the basis for determining Level of Service (LOS), which can be defined as a qualitative measure describing the operational conditions within the traffic stream. Intersection control delay is generally computed as a weighted average of the average control delay for all lane groups based on the amount of volume within each lane group. Delay thresholds for the various LOS are given in table (9-10/1).

Table 9-10/1: LOS Criteria for signalized intersections [3, p.18-6]

LOS ($V/C \leq 1.0$)	Control Delay per Vehicle (s/veh)
A	≤ 10
B	$> 10-20$
C	$> 20-35$
D	$> 35-55$
E	$> 55-80$
F	> 80

9-10/2 TRAFFIC OPERATIONS ELEMENTS

Signalized intersection operations are affected by the input data elements shown in table (9-10/2).

Table 9-10/2: Traffic Operation Elements [3, P.18-8]

Data Category	Input Data element
Traffic characteristics	Demand flow rate
	Right-turn-on-red flow rate
	Percent heavy vehicles
	Intersection peak hour factor
	Platoon ratio
	Upstream filtering adjustment factor
	Initial queue
	Base saturation flow rate
	Lane utilization adjustment factor
	Pedestrian flow rate
	Bicycle flow rate
	On-street parking maneuver rate
	Local bus stopping rate
Geometric design	Number of lanes
	Average lane width
	Number of receiving lanes
	Turn bay length
	Presence of on-street parking
	Approach grade
Signal control	Type of signal control
	Phase sequence
	Left-turn operational mode
	Dallas left-turn phasing option
	Passage time (if actuated)
	Maximum green (or green duration if pretimed)
	Minimum green
	Yellow change
	Red clearance
	Walk
	Pedestrian clear
	Phase recall
	Dual entry (if actuated)
	Simultaneous gap-out (if actuated)

9-10/3 OPERATIONAL ANALYSIS

The operational analysis of the signalized intersection can be achieved using the highway capacity software, HCS. An example for the software output showing the results of operational analysis is shown in table (9-10/3).

Table 9-10/3: HCS results for level of service determination

Control Appr/ Lane Grp	Delay and Ratios		Unf Del d1	Determination		Incremental Factor k	Del d2	Res Del d3	Lane Group		Approach		
	v/c	g/C		Prog Adj Fact	Lane Grp Cap				Delay	LOS	Delay	LOS	
Eastbound													
L	0.46	0.29	34.8	0.588	729	0.50	2.1	0.0	22.6	C	19.8	B	
T	0.65	0.29	37.2	0.588	1439	0.50	2.3	0.0	24.2	C			
R	0.13	1.00	0.0	0.950	1992	0.50	0.1	0.0	0.1	A			
Westbound													
L	0.56	0.29	36.0	0.588	687	0.50	3.3	0.0	24.5	C	19.9	B	
T	0.55	0.29	35.9	0.588	1536	0.50	1.4	0.0	22.5	C			
R	0.08	1.00	0.0	0.950	2419	0.50	0.1	0.0	0.1	A			
Northbound													
L	0.70	0.13	50.3	0.857	322	0.50	11.8	0.0	54.9	D	43.7	D	
T	0.88	0.13	51.6	1.000	282	0.50	30.5	0.0	82.1	F			
R	0.14	1.00	0.0	0.950	1955	0.50	0.2	0.0	0.2	A			
Southbound													
L	1.01	0.13	52.5	0.857	180	0.50	68.5	0.0	113.5	F	52.0	D	
T	0.82	0.13	51.2	1.000	301	0.50	21.3	0.0	72.4	E			
R	0.14	1.00	0.0	0.950	2302	0.50	0.1	0.0	0.1	A			
Intersection delay = 29.2 (sec/veh) Intersection LOS = C													

9-11 REFERENCES

- [1] AASHTO, "A Policy on Geometric Design of Highways and Streets", American Association of State Highway and Transportation officials, USA, 2011.
- [2] Garber, N.J. and Hoel, L.A, "Traffic & Highway Engineering", Cengage Learning, USA, 2009.
- [3] "Highway Capacity Manual, HCM 2010", Transportation Research Board, National Research Council, USA, Washington D.C., 2010.
- [4] AASHTO, "A Policy on Geometric Design of Highways and Streets", American Association of State Highway and Transportation Officials, USA, 2004.

CHAPTER 10

GRADE SEPARATIONS AND INTERCHANGES

Grade separation structures, permit the cross flow of traffic at different levels without interruption, with increase in safety and saving of time.

The interchange is a grade separation in which vehicles may transfer to other directions of flow by the use of connecting roadways or ramps, on different levels.

Grade separations and interchanges may be justified by the following warrants:

- To carry large volume of traffic of express highways, with full control of access.
- To eliminate bottlenecks, and reduce frequent crashes.
- To reduce delay, and avoid unreasonable size of at- grade intersections, with less road-user cost.
- To fit topography requirements. [1, p.10-4]

10-1 GENERAL INTERCHANGE CONFIGURATIONS

The selection of interchange configuration type, is influenced by many factors including: highway classification, intersection legs, through and turning traffic design speed, terrain, right-of- way, access control, safety needs, and economy.

The basic interchange configurations may include: three- leg T or Y, diamond, single point urban diamond, rotary, one quadrant, partial and full cloverleaf, and directional interchanges.

10-1/1 THREE- LEG TRUMPET AND DIRECTIONAL INTERCHANGES

Figure (10-1/1) shows typical layouts of T and Y interchanges at three- leg junctions with one or more grade separations, to provide for all movements with or without loops.

Using more than one single – structure is more costly, and justified only where all movements are large [1, p.10-30].

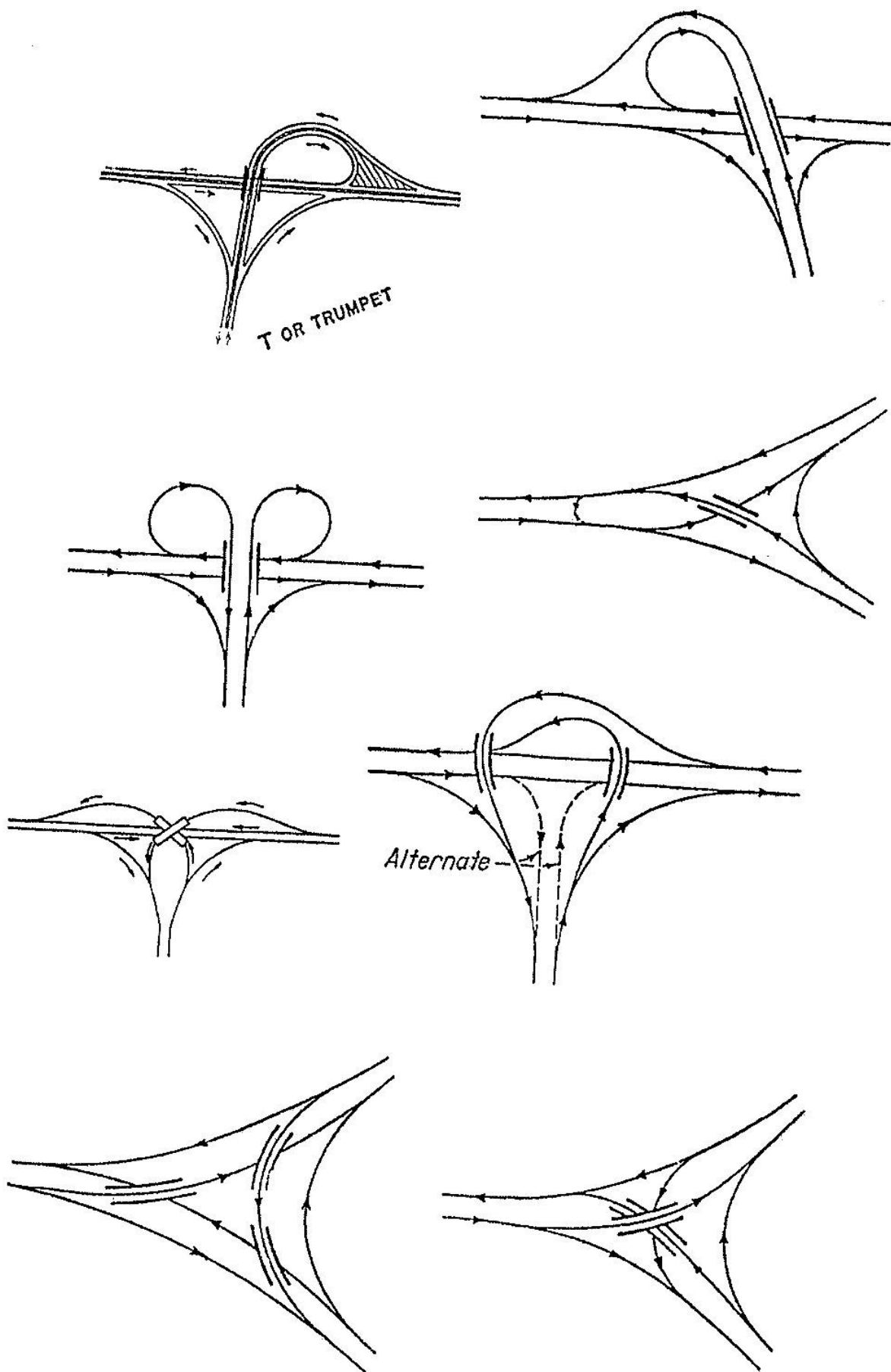


Figure 10-1/1: Three-Leg T and Y Interchanges [1, p.10-11]

10-1/2 DIAMOND INTERCHANGES

Diamond interchange is the simplest 4- Leg interchange particularly adaptable to major – minor crossings, in both rural and urban areas.

The ramps are aligned with free- flow terminals on the major highway, and an at –grade left turn on the minor cross road which may need signalization, [1, p.10-37].

A variety of diamond interchange configurations, are shown in figure (10-1/2 A,B,C) with and without frontage roads including: conventional arrangements, split arrangements to reduce traffic conflicts, and arrangements with more than one structure.

Double roundabouts may be used at both crossroad ramp terminals for the elimination of signal control [1, p.10-42].

10-1/3 SINGLE- POINT URBAN INTERCHANGES

The Single- Point Urban (Diamond) Interchange is characterized by narrow right – of- way, high construction cost of the bridges, and greater capacity than conventional diamond interchange. All four turning movements are controlled by a single traffic signal, and the opposing left turns operate to the left of each other [1, p.10-42].

It is required from the cross- road drivers to rely heavily on guide signing, pavement markings, and land - use signing, in order to travel safely through the intersection area. Long constant radii of left- turning roadways (52-122m), with a minimum of 1.8m clearance between the outside edge lines of opposing left- turning movements are required.

The overpass and underpass Single- Point Urban Interchanges are shown in figures (10-1/3) and (10-1/4) respectively.

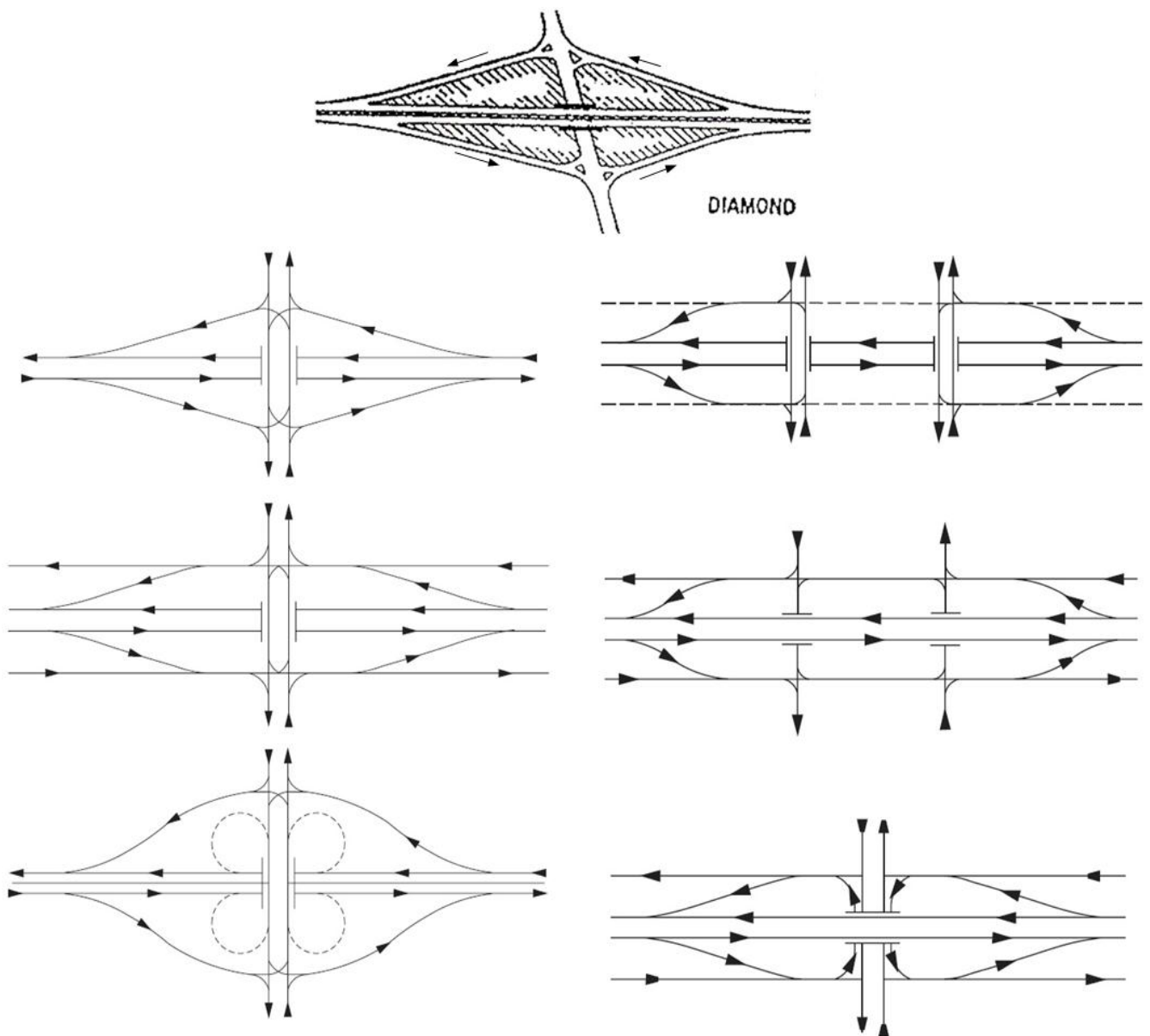


Figure 10-1/2A: Diamond Interchanges, Conventional Arrangements [1, p.10-39] **Figure 10-1/2B: Diamond Interchanges, Split Arrangements to Reduce Traffic Conflicts [1, p.10-39]**

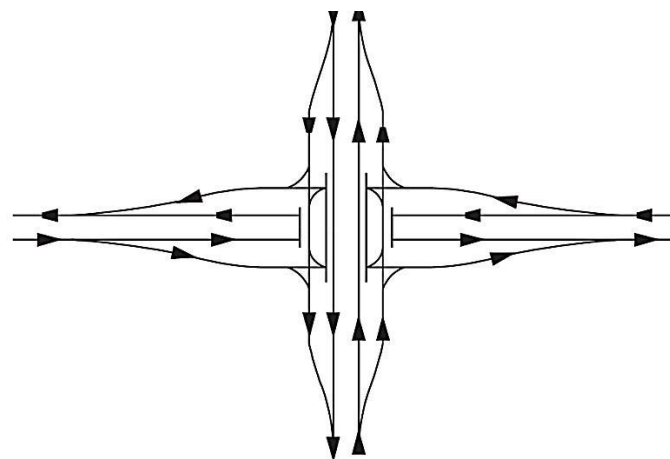


Figure 10-1/2C: Diamond Interchanges, with Additional Structures (Third-Level Structure) [1, p.10-40]

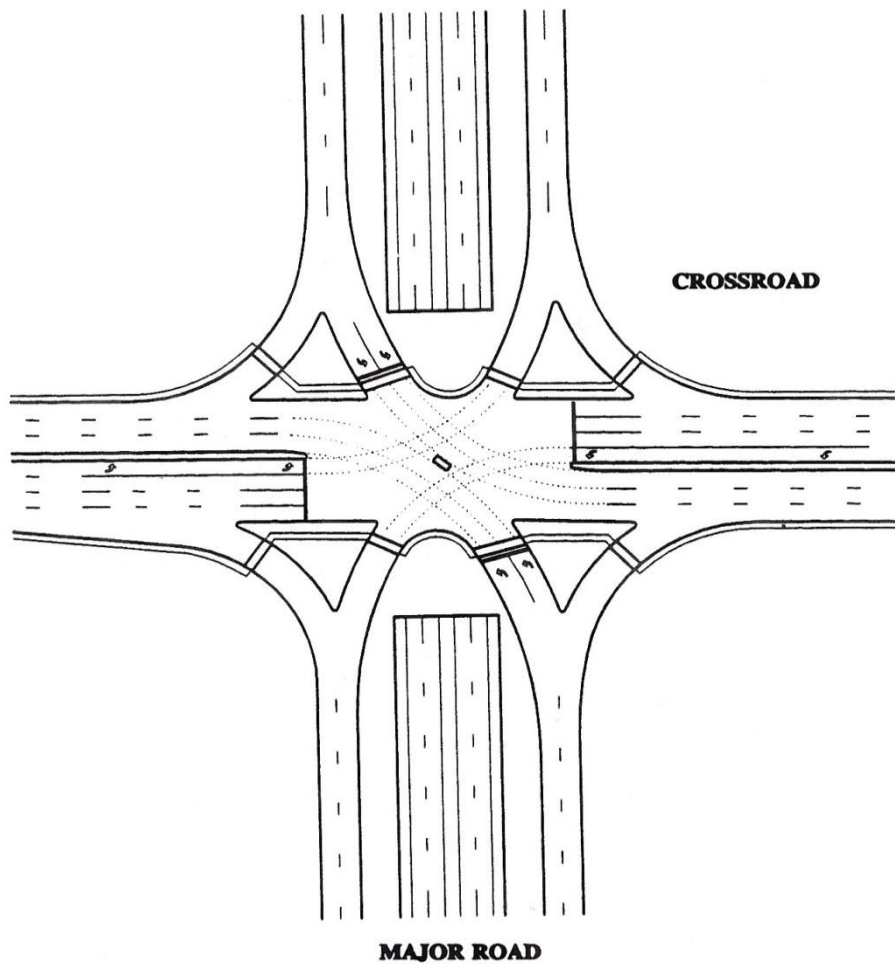


Figure 10-1/3: An Overpass Single-Point Urban Interchange (Courtesy Transportation Research Board) [2, p.244]

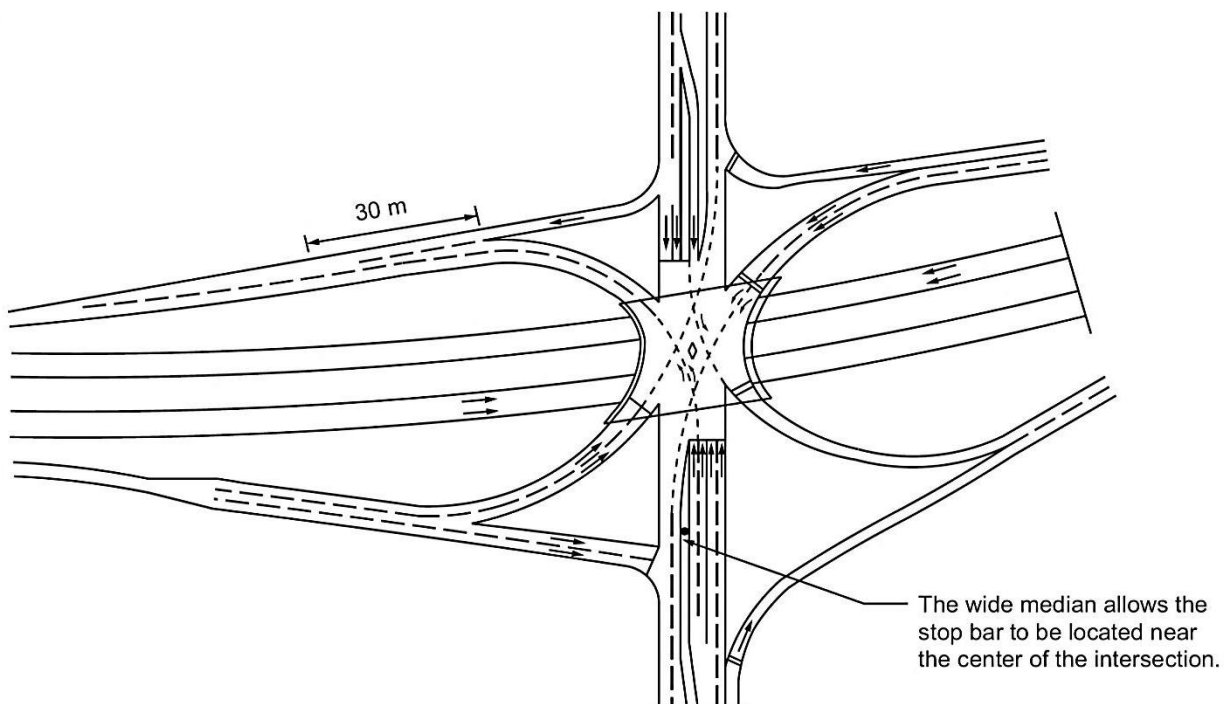


Figure 10-1/4: Underpass Single-Point Diamond Interchange [1, p.10-43]

10-1/4 ROTARY INTERCHANGES

Rotary interchanges are fitting where there are five or more intersection legs, and all movements other than through traffic on the principal highway, can be handled properly on the weaving sections.

The rotary may be overpassed or underpassed by one of the intersecting highways. The design features and operation of the rotary roadway are basically the same as that of an at-grade rotary. A typical rotary interchange is presented in figure (10-1/5) [3, p.558]

10-1/5 ONE-QUADRANT INTERCHANGES

One – quadrant interchanges have application for an intersection of roadways with low traffic volume and minimal truck traffic.

A one- quadrant interchange may be constructed as the first step in a stage construction program, to become a part of a full or partial cloverleaf interchange. Figure (10-1/6) shows a four - leg interchange with ramps in one quadrant, [1,p.10-36].

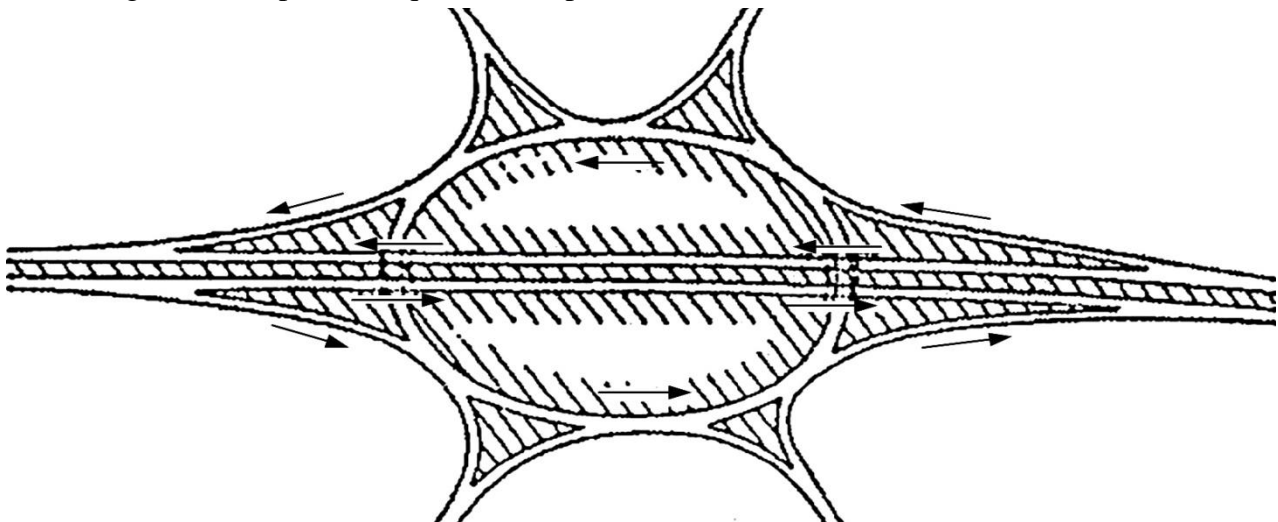


Figure 10-1/5: Rotary Interchange [3, p.558]

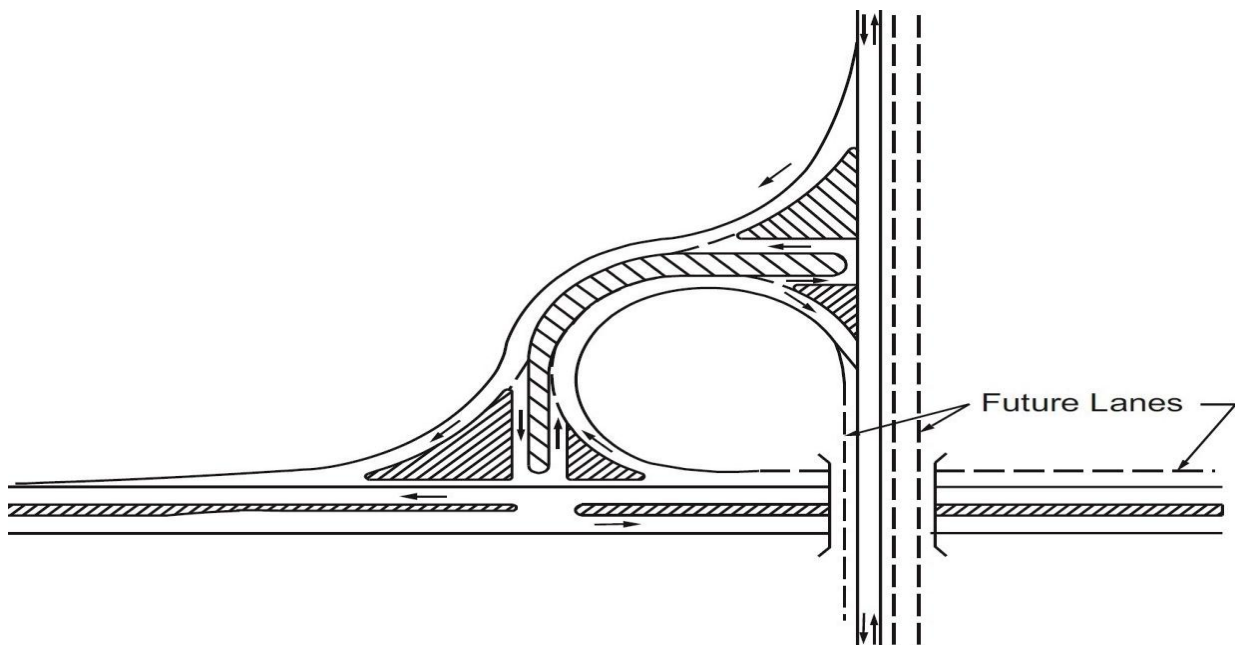


Figure 10-1/6: Four-Leg Interchange, Ramps in One Quadrant [1, p.10-36]

10-1/6 PARTIAL AND FULL CLOVERLEAF INTERCHANGES

Cloverleaf is a 4-leg interchange that employs loop ramps to accommodate left – turning movements, and referred to as “Full Cloverleaf” with loops in all four quadrants, and “partial cloverleaf” for all others [1,p.10-48].

For cloverleaves, large right- of- way areas are needed, with additional travel distances for left turns, and relatively short weaving lengths are typically available.

A single structure is usually required for the cloverleaf, and all crossing movements are eliminated.

For design speeds exceeding 80 km/hr., the practical radii of loops are usually 50 to 75m. On highways with minor movements (design speed less than 80 km/hr.), loops radii of 30 to 50 m may be used.

Ramp arrangements for full and partial cloverleaf interchanges are illustrated in figure (10-1/7).

10-1/7 ALL DIRECTIONAL AND SEMIDIRECTIONAL INTERCHANGES

Direct connection is the ramp that does not deviate greatly from the intended direction of travel. Semidirect connection is the ramp where a driver exits to the right first, heading away from the intended direction of travel, gradually reversing, and then passing around other interchange ramps, before entering the other road.

In comparison to loops, direct or semidirect connections have shorter travel distance, higher speeds, higher level of service, and they often avoid the need for weaving [1, p.10-54].

Fully directional interchanges are used where two high- volume freeways intersect. Semidirect or direct connections for one or more left- turning movements are often appropriate at major interchanges in urban areas. [1,p.10-56].

Basic pattern of selected directional interchanges are illustrated in figure (10-1/8).

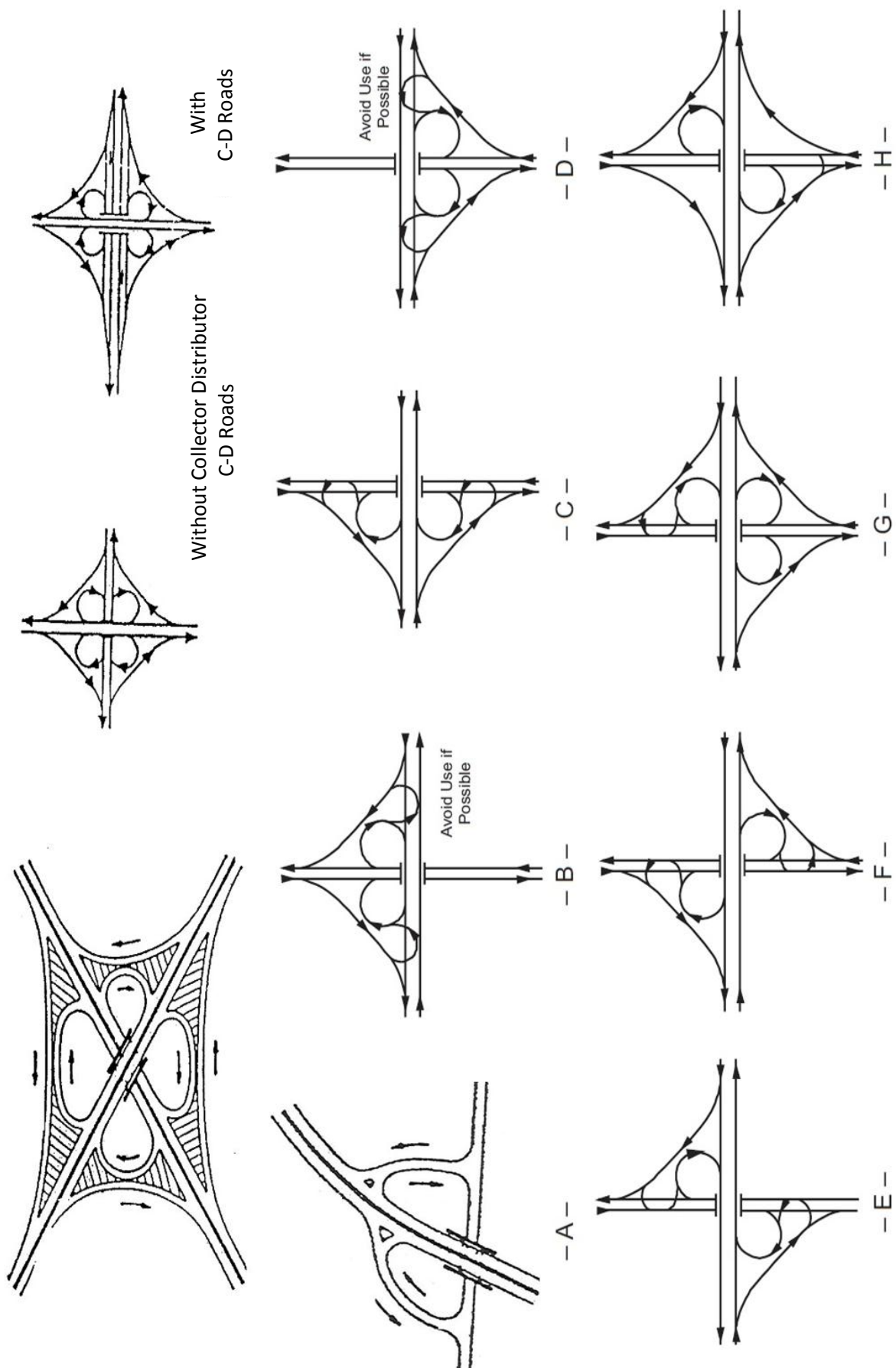
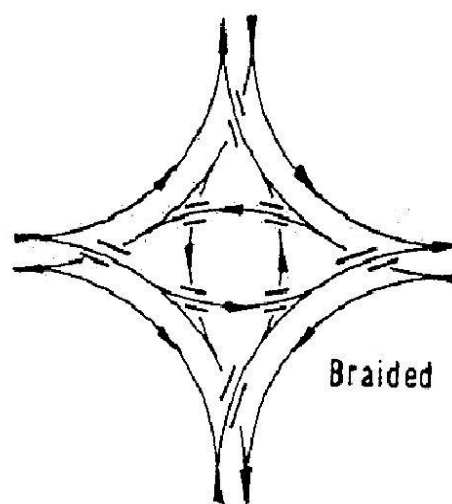
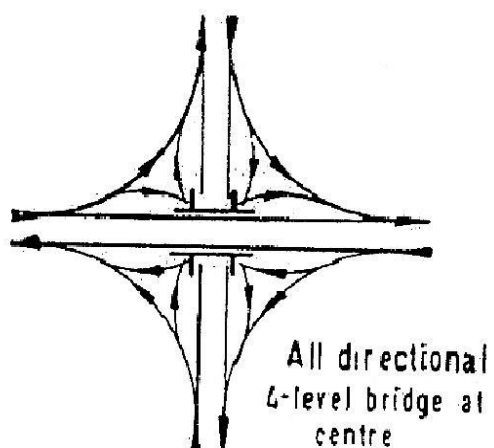


Figure 10-1/7: Cloverleaf Ramp Arrangements, Exit and Entrance Turns [1, p.10-51]



DIRECTIONALS

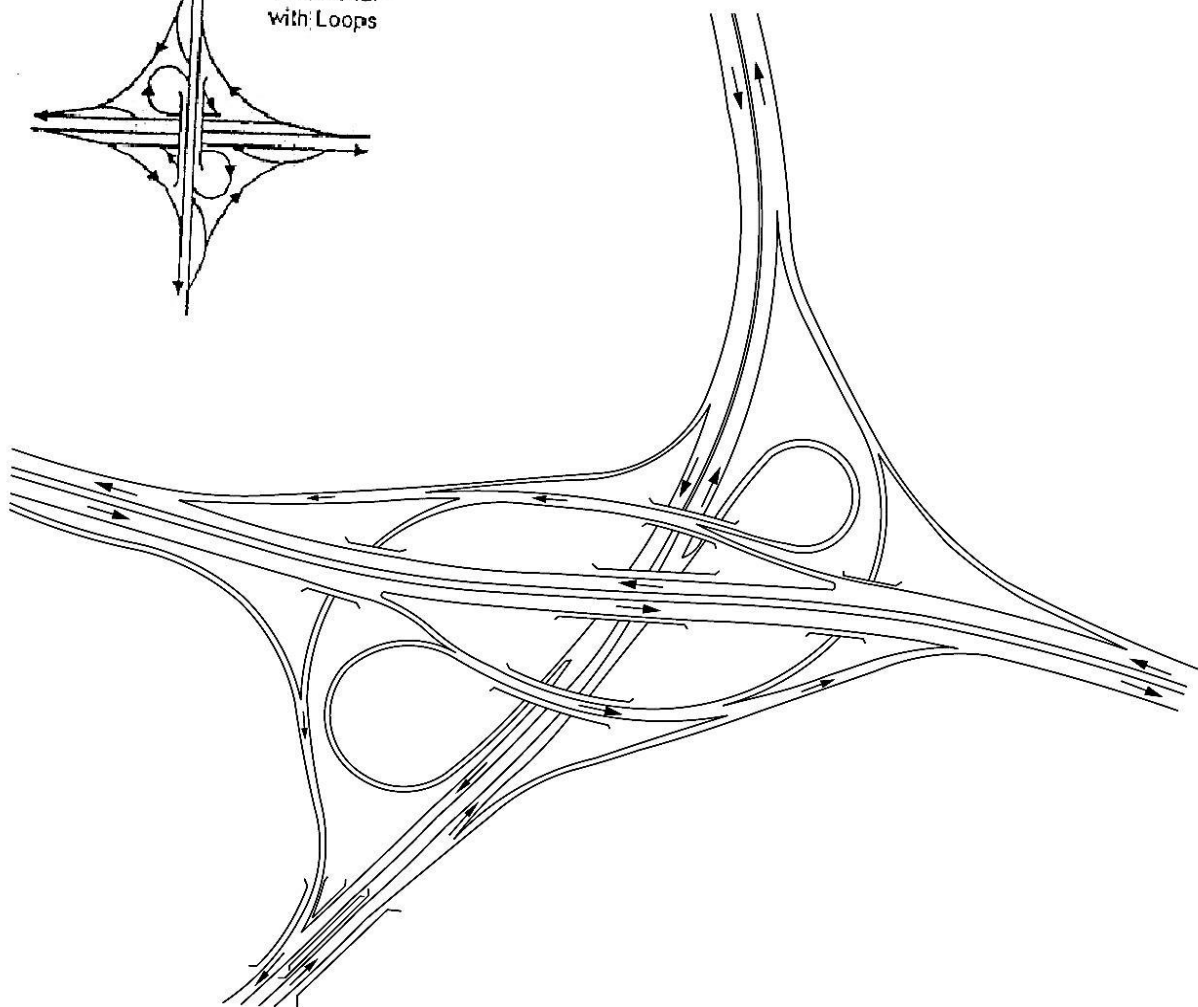
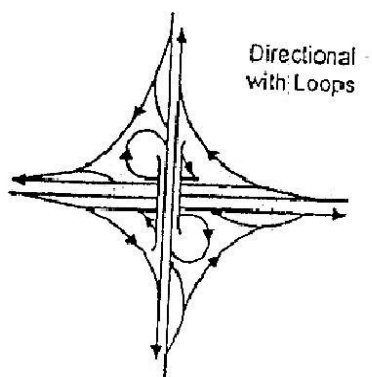


Figure 10-1/8: Directional Interchange, Two Semidirect Connections [1, p.10-58]

10-2 FREEWAY INTERCHANGES WITH OTHER HIGHWAY CLASSES

The selection of appropriate interchange configurations for the intersection of freeways or expressways with other highway classes, are covered in two categories: “service interchanges”, (that connect freeway to lesser facilities), and “system interchanges”, (that connect two or more freeways). [1, p.10-63].

The interchange configurations of diamond, partial cloverleaf, full cloverleaf single- point urban interchange, and directional interchange, may be selected for a freeway intersection, depending on the class of intersecting, roadway (local, collector, arterial or freeway), and compatibility with the environment (rural, suburban, urban), as presented in figure (10-2/1).

10-3 GRADE SEPARATION STRUCTURES

The grade – separation structure should conform to the natural lines of the highway approaches in alignment, profile, and cross section.

A single simple- span girder bridge may be used with spans of up to 45m, and can accommodate conditions of severe skew and horizontal curvature. The structural depth for a single span girder bridge is approximately 1/15 to 1/30 of the span [1,p.10-1] Where spans are long, truss bridges may be used.

On a divided highway with a wide median, the overpass will likely be built as two parallel structures separated by an opening. A study should be made to determine whether the major roadway should overpass or underpass the crossroad, as governed by topography, economy, and design controls.

Where the major road is built close to the ground (underpass) the ramp profiles are best fitted for deceleration and acceleration in turning movements, in addition to resultant economy in construction. On the otherhand, aesthetic preference, and better drainage are obtained by the overpass layout.

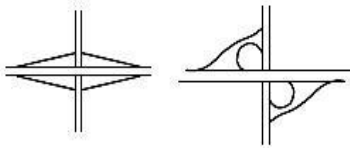
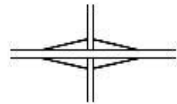
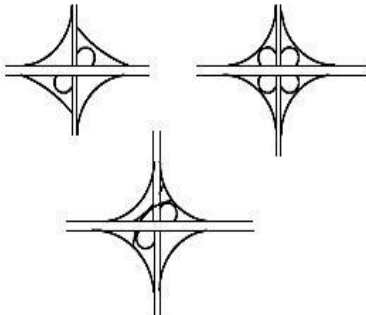
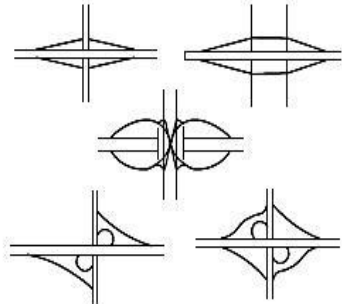
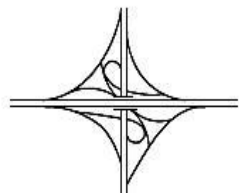
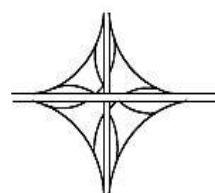
TYPE OF INTER- SECTING FACILITY	RURAL	SUBURBAN	URBAN
LOCAL ROAD OR STREET	 - A -	 - B -	
COLLECTORS AND ARTERIALS	 - C -	 - D -	
FREEWAYS	 - E -	 - F -	

Figure 10-2/1: Adaptability of Interchanges on Freeways as Related to Types of Intersecting Facilities [1, p.10-65]

10-4 LONGITUDINAL DISTANCE TO ATTAIN GRADE SEPARATION

The longitudinal distance needed for adequate design of a grade separation, depends on: the design speed, gradient, and the amount of rise or fall needed to achieve the separation [1,p.10-24].

The minimum distance D, required to effect grade separation (overpass or underpass) in flat terrain, for gradients ranging from 2 to 7 percent, and for design speeds of 50 to 110 km/hr., may be used as a guide for preliminary design, as shown in figure (10-4/1). The difference in elevation (H) for a grade separation, is usually needed for essential vertical clearance and

structural thickness. A minimum vertical clearance of 5.20m may be used for the grade separation of two highways, and 6.50m for a highway overcrossing a railroad.

10-5 EFFECTIVE DISTANCE OF AUXILIARY LANES

The auxiliary lane originating at a two-lane entrance, should be carried along the roadway for an effective distance beyond the merging point. An auxiliary lane introduced for a two-lane exit, should be carried along the roadway for an effective distance in advance of the exit, and then extended onto the ramp [1,p.10-76].

Generally, parallel design of auxiliary lane is preferred in comparison with taper design. The effective lengths of the introduced auxiliary lane of about 750 m, produce the desired operational effect and achieve full capacity of two-lane entrances and exits, as shown in figure (10-5/1).

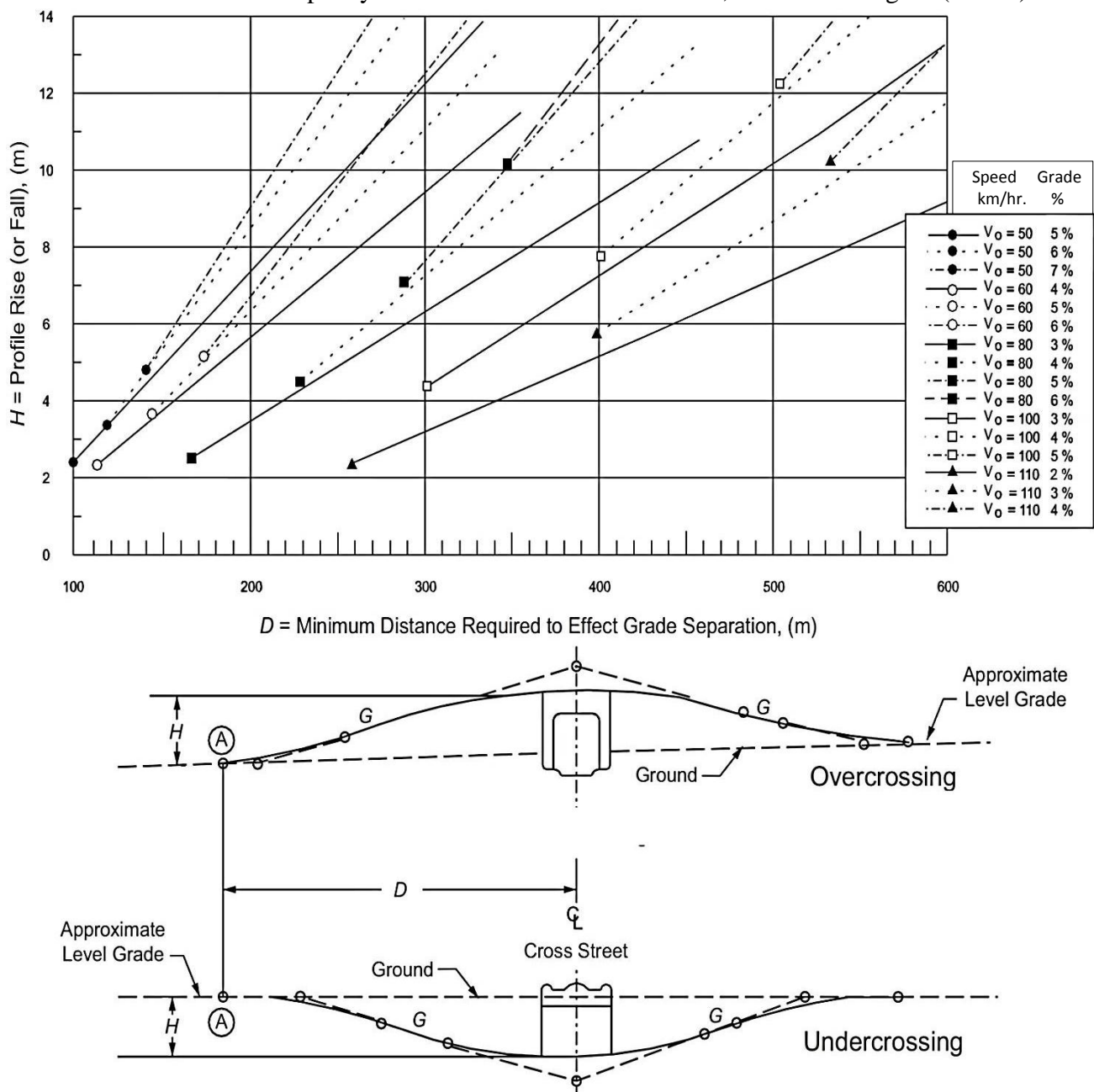
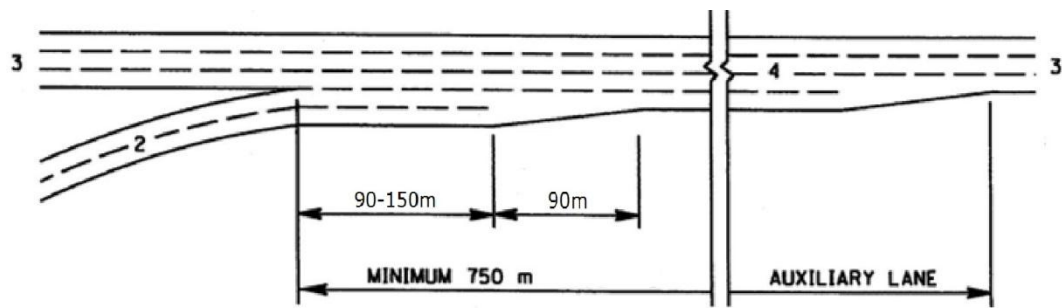
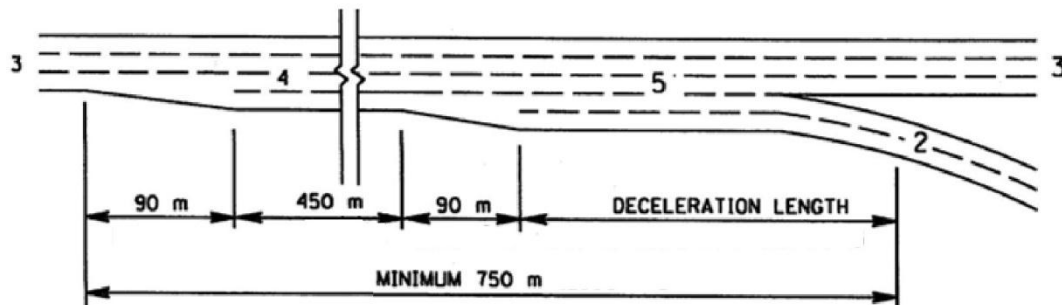


Figure 10-4/1: Flat Terrain, Distance Needed to Achieve Grade Separation [1, p.10-26]



PARALLEL DESIGN (PREFERRED)

AUXILIARY LANE EXTENDED FOR EFFECTIVE DISTANCE BEYOND ENTRANCE



PARALLEL DESIGN (PREFERRED)

AUXILIARY LANE INTRODUCED FOR EFFECTIVE DISTANCE IN ADVANCE OF EXIT

(A) Point controlling speed on ramp

Figure 10-5/1: Coordination of Lane Balance and Basic Number of Lanes through Application of Auxiliary Lanes [1, p.10-78]

10-6 DESIGN SPEED AND GRADES FOR RAMPS

Ramps include all types of turning roadways, shown in figure (10-6/1), that connect two or more legs at an interchange. The guide values for ramp design speed as related to highway design speed are presented in table (10-6/1).

Ramp design speeds of 80 km/hr. or higher, apply to freeway and expressway exits. An upper – range value of design speed is attainable on ramps for right turns.

Ramp design speeds above 50 km/hr. for loops, involve large land areas that are rarely available in urban areas [1, p.10-89].

Downgrades should desirably be limited to 3 or 4 percent on ramps with sharp horizontal curvature and significant heavy trucks. Upgrades on ramps should desirably be limited as shown in table (10-6/2) [1,p.10-93].

10-7 DEVELOPMENT OF SUPERELEVATION AT RAMP TERMINALS

The methods of developing superelvation at free- flow ramp terminals for tangent section (parallel exit), curved section (Parallel exit), curved section (parallel entrance), and tangent section (between entrance and exit), are illustrated in figure (10-7/1).

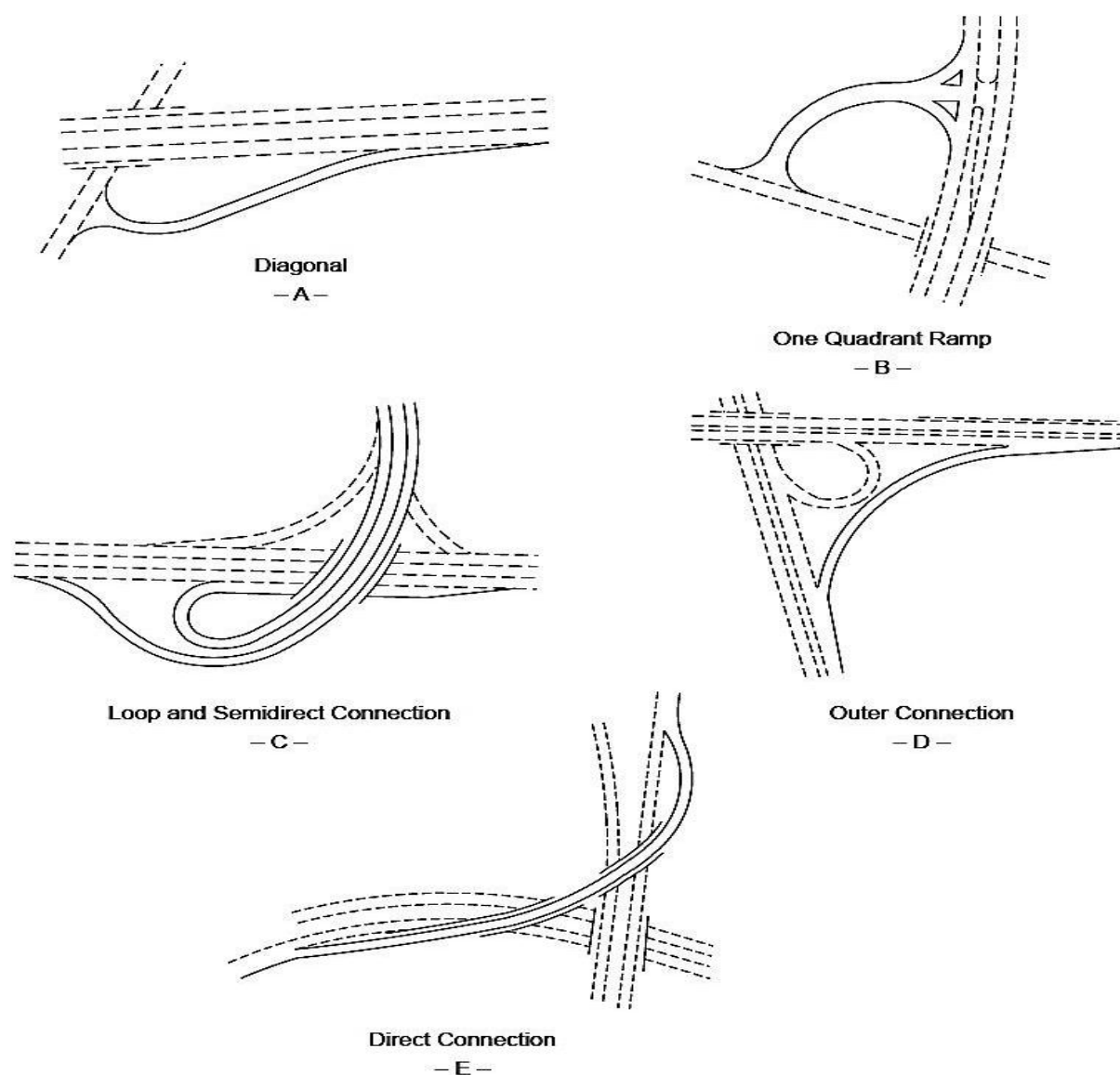


Figure 10-6/1: General Types of Ramps [1, p.10-88]

Table 10-6/1: Guide Values for Ramp Design Speed as Related to Highway Design Speed [1, p.10-89]

Highway design speed (km/hr.)	50	60	70	80	90	100	110	120
Ramp design speed (km/hr.)								
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

Table 10-6/2: Limits of Ramp Upgrade with Respect to Ramp Design Speed

Ramp Design Speed (km/hr.)	Max. Ramp Upgrade (%)
70-80	3-5
60	4-6
40-50	5-7
30-40	6-8

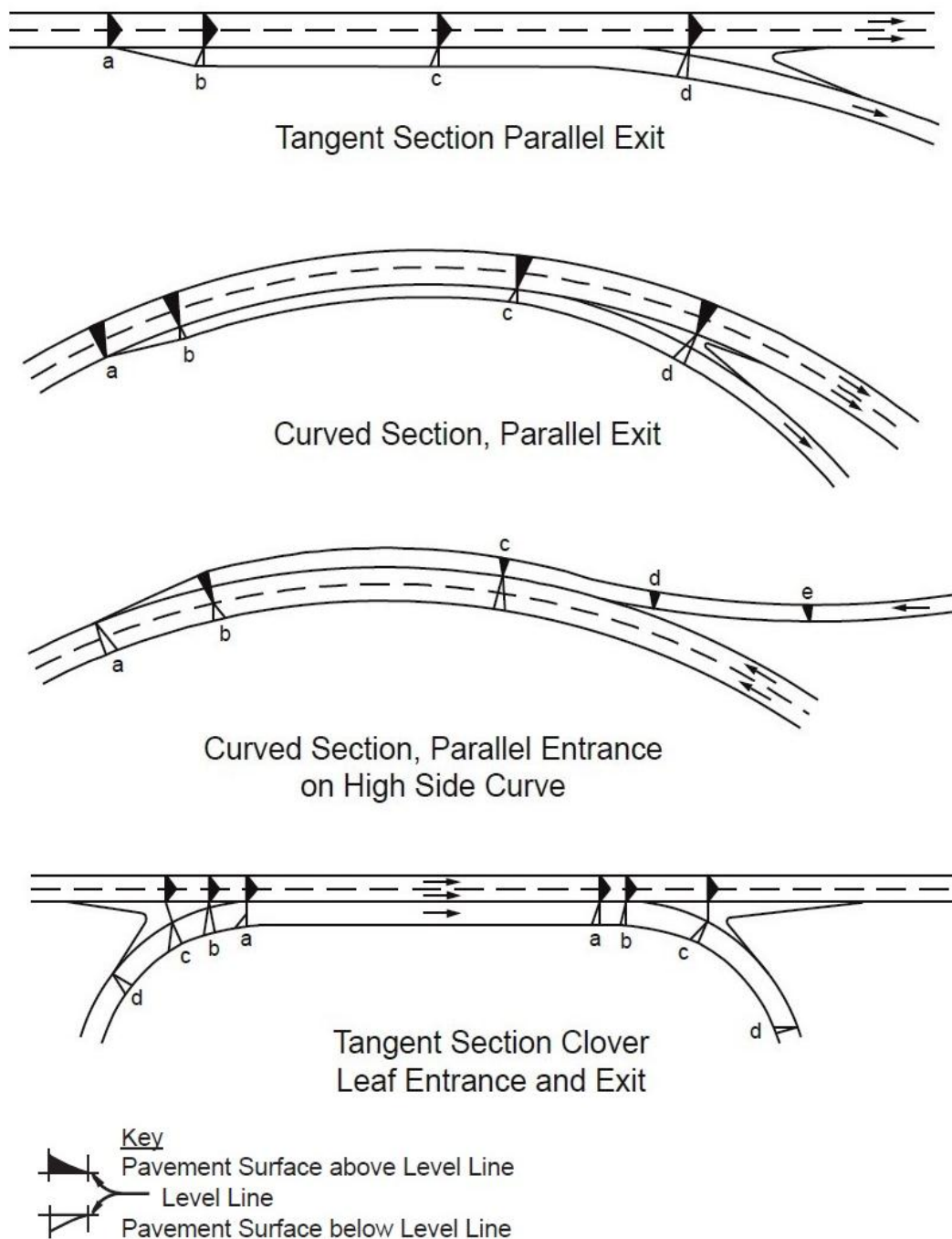


Figure 10-7/1: Development of Superelevation at Free-Flow Ramp Terminals [1, p.10-95]

10-8 RECOMMENDED MINIMUM RAMP TERMINAL SPACING

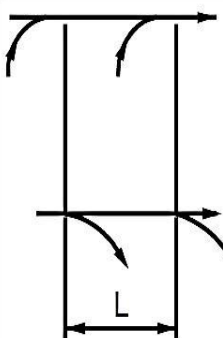
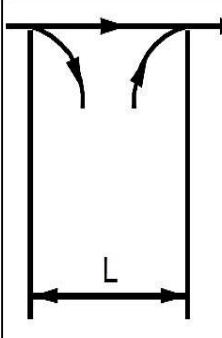
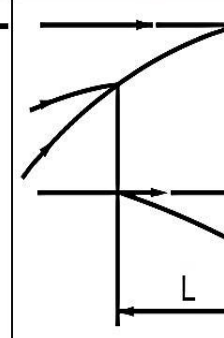
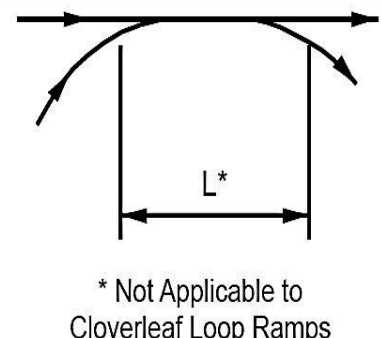
Ramp terminal is that portion adjacent to through traveled way, including speed change lanes, tapers, and islands. Ramp terminals may be (at grade type), as at crossroad terminal of diamond interchange, or (free-flow type), where ramp traffic merges with, or diverges from high-speed through traffic at flat angles [1,p.10-103].

On urban highways, two or more ramp terminals are often located in close succession, and a reasonable spacing should be provided for sufficient weaving length. and adequate space for signing.

- The five possible ramp- pair combinations are:
- An entrance followed by entrance (EN- EN).
- An exit followed by exit (EX- EX).
- An exit followed by entrance (EX- EN).
- Turning roadways.
- An entrance followed by exit (EN- EX).

The recommended minimum ramp terminal spacing (L), between successive painted noses, for the various ramp- pair combination, applicable to interchange classifications, are presented in figure (10-8/1).

When the distance between successive noses is less than 450m, the speed change lanes should be connected to provide an auxiliary lane. [1,p.10-106].

EN-EN or EX-EX		EX-EN		Turning Roadways		EN-EX (Weaving)			
						 * Not Applicable to Cloverleaf Loop Ramps			
Full Freeway	CDR or FDR	Full Freeway	CDR or FDR	System Interchange	Service Interchange	System to Service Interchange		Service to 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

EN—Entrance

CDR—Collector distributor road

EX—Exit

Figure 10-8/1: Recommended Minimum Ramp Terminal Spacing [1, p.10-106]

10-9 MINIMUM ACCELERATION LENGTHS OF SPEED-CHANGE LANES (SINGLE LANE, ENTRANCE RAMP)

The minimum lengths of acceleration distances for ramp entrance terminals of the parallel-type single – lane, are given in table (10-9/1) (with flat grades). Adjustment factors as a function of grade are given in table (10-9/2), [1,p.10-111].

Table 10-9/1: Minimum Acceleration Lengths for Entrance Terminals with Flat Grades of Two Percent or Less [1, p.10-110]

Acceleration Length, L (m) for Entrance Curve Design Speed (km/hr.)									
Highway		Stop Condition	20	30	40	50	60	70	80
Design Speed, V (km/hr.)	Speed Reached, V_a , (km/hr.)	and Initial Speed, V'_a , (km/hr.)							
		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	51	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 400 m.

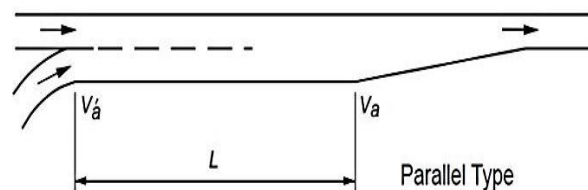


Table 10-9/2: Speed Change Lane Adjustment Factors as a Function of Grade [1, p.10-111]

Design Speed of Highway (km/hr.)	Deceleration Lanes					
	Ratio of Length on Grade to Length on Level for Design Speed of Turning Curve (km/hr.) ^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		
Design Speed of Highway (km/hr.)	Acceleration Lanes					
	Ratio of Length on Grade to Length of Level for Design Speed of Turning Curve (km/hr.) ^a					
	40	50	60	70	80	All Speeds
	3 to 4% Upgrade					3 to 4%
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%
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

^a Ratio from this table multiplied by the length in table (10-9/1) or table (10-10/1) gives length of speed change lane on grade.

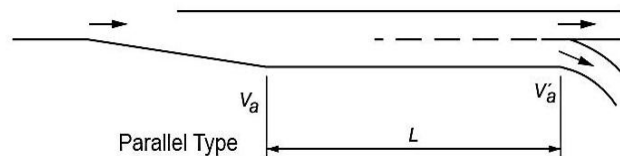
10-10 MINIMUM DECELERATION LENGTHS OF SPEED –CHANGE LANES (SINGLE- LANE, EXIT RAMP)

The minimum deceleration lengths for ramp exit terminals, of parallel- type single lane, with flat grades, are shown in table (10-10/1). Adjustment factors as a function of grade, are presented in table (10-9/2), [1,p.10-113].

Table 10-10/1: Minimum Deceleration Lengths for Exit Terminals with Flat Grades of Two Percent or Less [1, p.10-115]

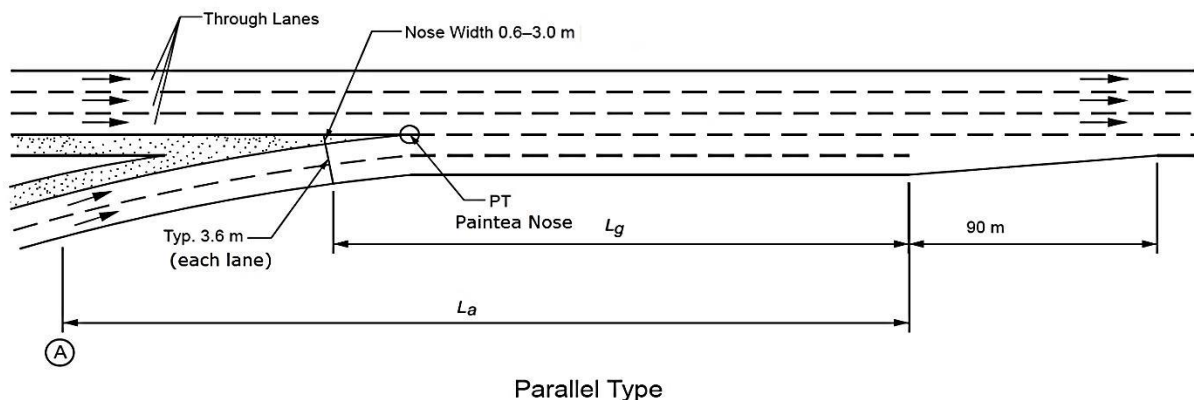
Deceleration Length, L (m) for Design Speed of Exit Curve, V' (km/hr.)									
Highway Design Speed, V (km/hr.)	Speed Reached, V _a (km/hr.)	Stop Condition	20	30	40	50	60	70	80
		For Average Running Speed on Exit Curve V' _a (km/h)							
		0	20	28	35	42	51	63	70
50	47	75	70	60	45	—	—	—	—
60	55	95	90	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	93	200	195	185	175	170	155	110	120

V = design speed of highway (km/hr.).
V_a = average running speed on highway (km/hr.).
V' = design speed of exit curve (km/hr.).
V'_a = average running speed on exit curve (km/hr.).



10-11 TYPICAL TWO- LANE ENTRANCE AND EXIT RAMP TERMINALS

Two- lane entrances are needed either as branch connections, or due to capacity needs on ramps. Two – lane exits are warranted because of capacity needs for the traffic volume leaving the highway. [1,p.10-120]. Figures (10-11/1) and (10-11/2) illustrate typical two- lane entrance and exit ramp terminals respectively.



Notes:

1. L_a is the required acceleration length as shown in Table 10-9/1 or as adjusted by Table 10-9/2
2. Point A controls speed on the ramp. L_a should not start back on the curvature of the ramp unless the radius equals 300 m or more.
3. L_g is the required gap acceptance length. L_g should be a minimum of 90 to 150 m , depending on the nose width.
4. The value of L_a or L_g , whichever produces the greater distance downstream from where the nose equals 0.6 m is suggested for use in the design of the ramp entrance.

Figure 10-11/1: Typical Two-Lane Entrance Ramps [1, p.10-122]

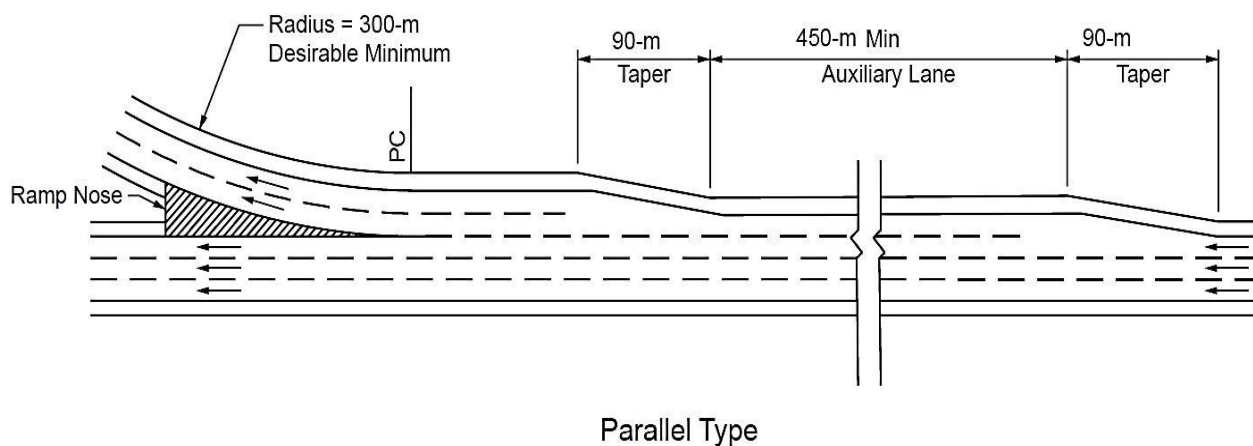


Figure 10-11/2: Two-Lane Exit Terminals [1, p.10-123]

10-12 REFERENCES

- [1] AASHTO, "*A Policy on Geometric Design of Highways and Streets*", American Association of State Highway and Transportation Officials, USA, 2011.
- [2] Wright, P.H. and Dixon, K. K., "*Highway Engineering*", John Wiley & Sons, USA, 2004.
- [3] AASHO, "*A Policy on Geometric Design of Rural Highways*", American Association of State Highway Officials, USA, 1961.

CHAPTER 11

TRAFFIC CONTROL DEVICES

The purpose of traffic control devices on streets and highways is to promote highway safety and efficiency by providing for the orderly movement of all road users. These devices include traffic control signals, traffic signs and pavement markings.

11-1 TRAFFIC CONTROL SIGNALS

A traffic control signal is defined as a power-activated traffic control device by which traffic is warned or is directed to take some specific action, such as stop or proceed. A traffic control signal should control traffic only at the intersection or midblock crosswalk location at which it is placed (Midblock crosswalks shall not be signalized if they are located within 100 m from the nearest traffic control signal).

11-1/1 TYPES OF TRAFFIC CONTROL SIGNALS

11-1/1/1 RETIMED CONTROL

A pretimed control is a type of signal control in which the cycle, phasing, intervals, and indications are predetermined and do not vary. Figure (11-1/1) shows the timing operation for a basic two-phase or two-traffic movement pretimed controller unit.

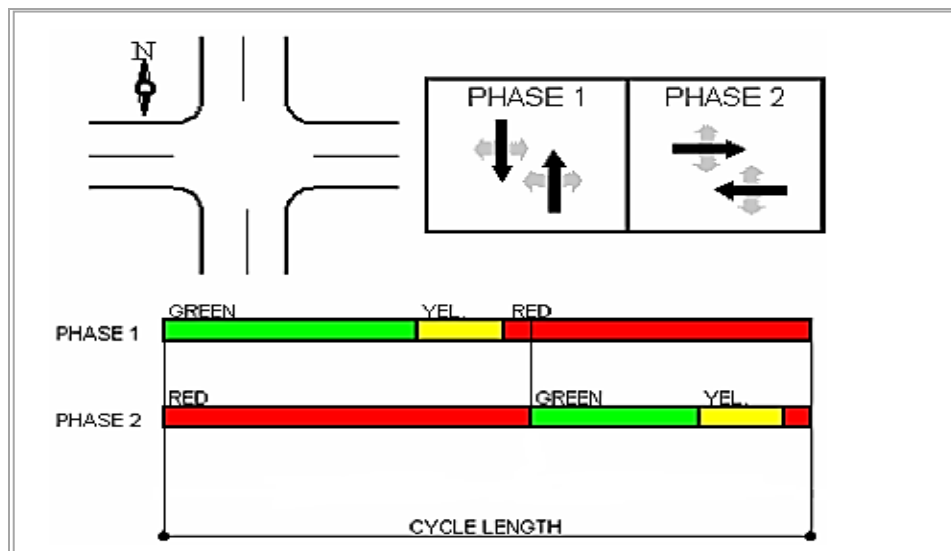


Figure 11-11/1: Pretimed Signal Operation [5, p.44]

11-1/1/2 TRAFFIC-ACTUATED CONTROL

A traffic-actuated signal is a type of signal control in which the length of most intervals and the cycle, and in some types the sequence of phasing, are adjusted continuously in accordance with real time measures of traffic volume obtained from vehicle detectors placed on one or more of the approaches to the intersection.

11-1/2 TRAFFIC CONTROL SIGNAL FEATURES

11-1/2/1 SIGNAL INDICATION

The indications in each traffic control signal face should be arranged in a vertical straight line (horizontal arrangement can be used as an alternative for the vertical arrangement in case of overhead signal).

The relative positions of indications within a signal face (from top to bottom in case of vertical arrangement or from left to right in case of horizontal arrangement) should be as shown in figure (11-1/2).

- Circular RED
- Circular YELLOW
- Circular GREEN

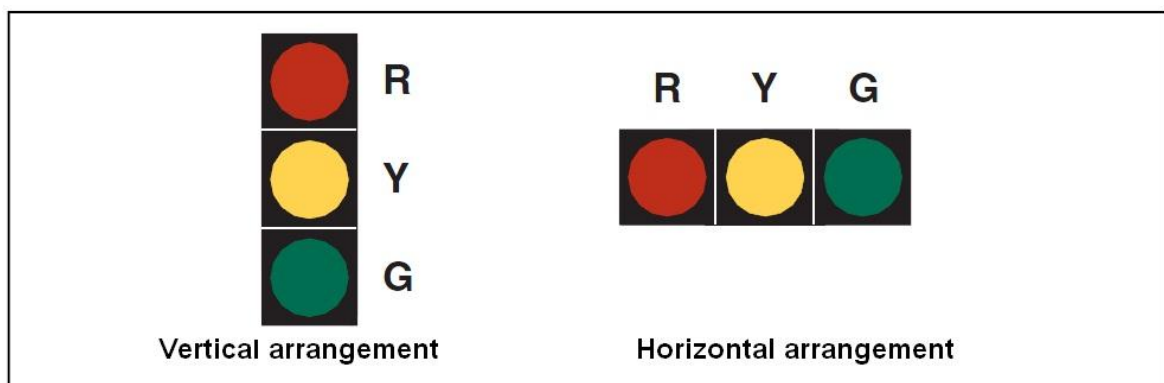


Figure 11-1/2: Arrangement of Indications in Traffic Signal Faces [1, p.455]

The following meanings shall be given to highway traffic signal indications for vehicles:

- **STEADY GREEN SIGNAL:**
 - Traffic, except pedestrians, facing a steady CIRCULAR GREEN indication may proceed straight through an intersection or turn right or left.
- **STEADY YELLOW SIGNAL:**
 - Traffic, except pedestrians, facing a steady CIRCULAR YELLOW indication is warned that the movement which had been allowed by the corresponding green indication is being terminated.
- **STEADY RED SIGNAL:**
 - Traffic, except pedestrians, facing a steady CIRCULAR RED indication should stop at the marked stop line before entering the intersection.
- **FLASHING YELLOW SIGNAL**
 - Vehicular traffic, on an approach to an intersection, facing a flashing CIRCULAR YELLOW signal indication is permitted to cautiously enter the intersection to proceed straight through or turn right or left or make a U-turn. Flashing yellow is typically used to indicate a permissible right turn movement and mid-block pedestrian crossings.

- **FLASHING RED SIGNAL**

- Vehicular traffic, on an approach to an intersection, facing a flashing CIRCULAR RED signal indication shall stop at the marked stop line before entering the intersection.

Note: Arrows can be used instead of circular indication for the same purpose but in the direction of arrows shown in the signal.

Arrows shall be pointed:

- A. Vertically upward to indicate a straight-through movement, or
- B. Horizontally in the direction of the turn to indicate a turn at approximately or greater than a right angle, or
- C. Upward with a slope at an angle approximately equal to that of the turn if the angle of the turn is substantially less than a right angle, or
- D. In a manner that directs the driver through the turn if a U-turn arrow is used, as shown in figure (11-1/3).

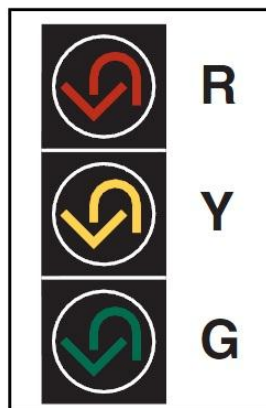


Figure 11-1/3: Example of U-Turn Signal Face [1, p.456]

Each signal face should have at least three indications but not more than five with the following exceptions:

- A single GREEN ARROW indication should be used alone to permit a continuous movement.
- Pedestrian signal faces, which have two indications.
- One or more indications in a signal face may be repeated for safety or increased effectiveness. A typical arrangement of signal sections in signal faces that do not control turning movements is shown in figure (11-1/4).

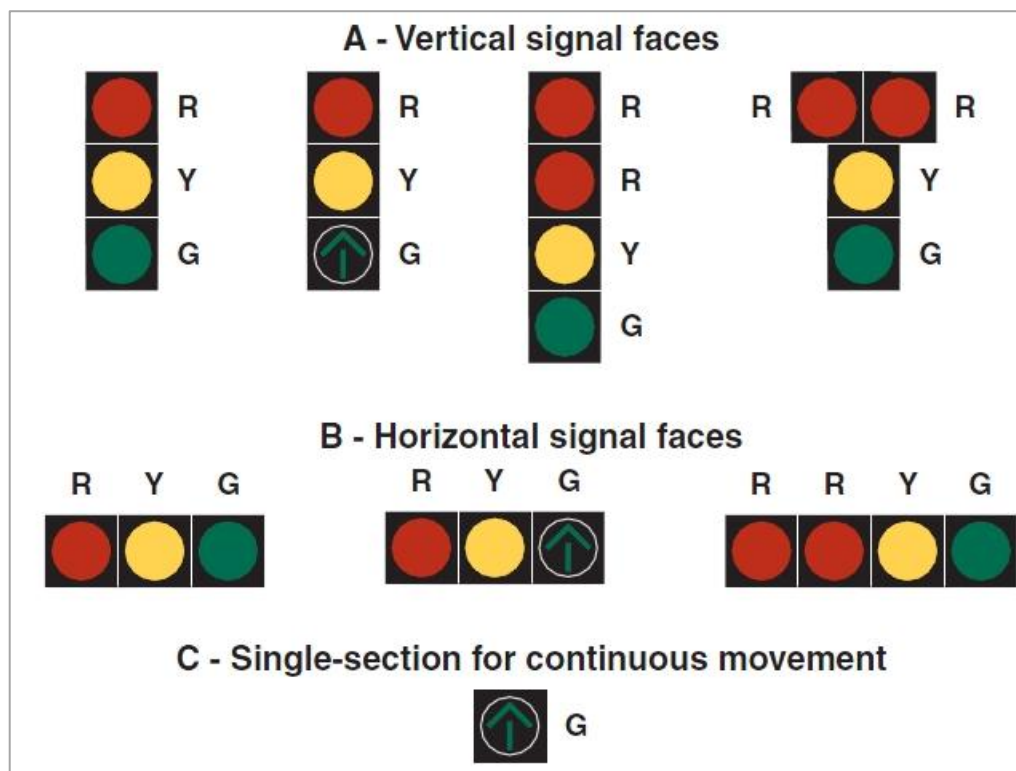


Figure 11-1/4: Typical Arrangements of Signal Sections in Signal Faces that Do Not Control Turning Movement [1, p.458]

11-1/2/2 SIZE AND DESIGN OF LENSES

There shall be two nominal diameter sizes for vehicular signal indications: 200 mm and 300 mm.

300 mm signal lenses shall be used when:

- 1- Approaches with 85-percentile approach speed exceeding 60km/hr.
- 2- For signal faces located more than 45m from the stop line.
- 3- Approaches where a traffic control signal might be unexpected.
- 4- For arrow signal indications
- 5- All approaches without curbs and gutters (rural roads) where only post-mounted signal heads are used.
- 6- If the nearest signal face is between 35m and 45m beyond the stop line, unless a supplemental near-side signal face is provided.
- 7- For approaches to all signalized locations for which the minimum sight distance in table (11-1/1) below cannot be met.

Table 11-1/1: Required Advance Visibility of Traffic Control Signal Indications [3, p.9-12]

85 Percentile Speed (km/hr.)	Minimum Visibility Distance (meters)
30	50
40	65
50	85
60	110
70	135
80	165
90	195
100	230
110	265
120	295

Note: Existing 200 mm circular signal indications that satisfy the above requirements of 1 to 7 may be retained for the remainder of their useful service life.

Each signal lenses, except those used for pedestrian signal heads shall be circular or arrow with following properties

1. each signal lens shall be independently illuminated
2. each circular or arrow signal lens shall emit a single color : red, yellow or green
3. letters or numbers shall not be displayed as a part of a vehicular signal lens.
4. the arrow, which shall show only one direction, shall be the only illuminated part of an arrow signal lens.

11-1/2/3 SEQUENCE OF INDICATION

1- Application of signal indication for left turn

Left turn traffic is controlled by one of the of two modes as follows

- a) Permissive Only Mode: turns made on the CIRCULAR GREEN signal indication, a flashing left turn YELLOW ARROW signal indication, or a flashing left – turn RED ARROW signal indication after yielding to pedestrian if any, and\ or opposing traffic, if any.

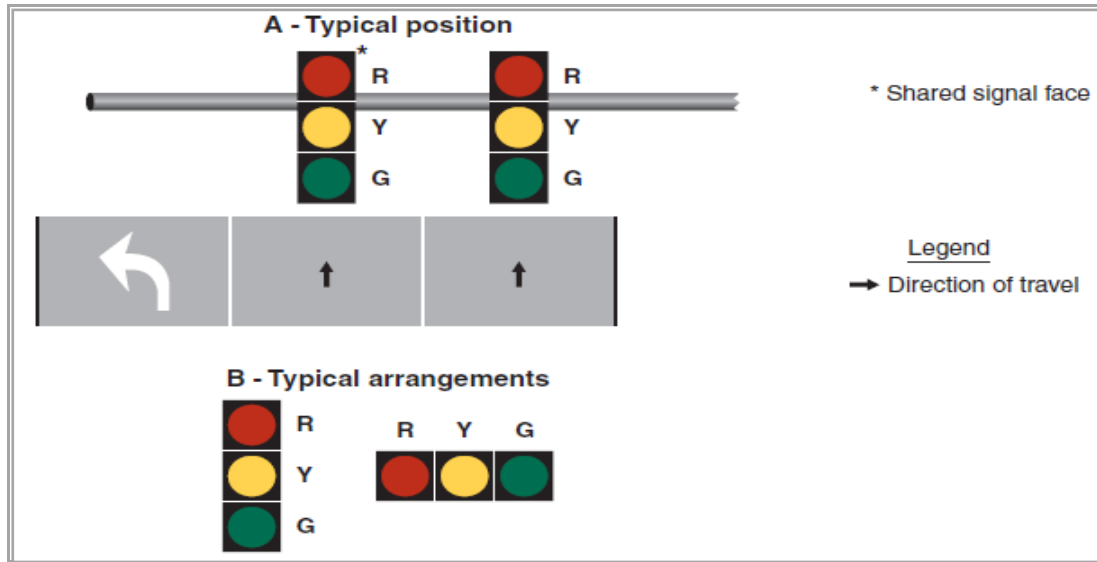


Figure 11-1/5: Typical Position and Arrangements of Shared Signal Faces for Permissive Only Mode Left Turns [1, p.467]

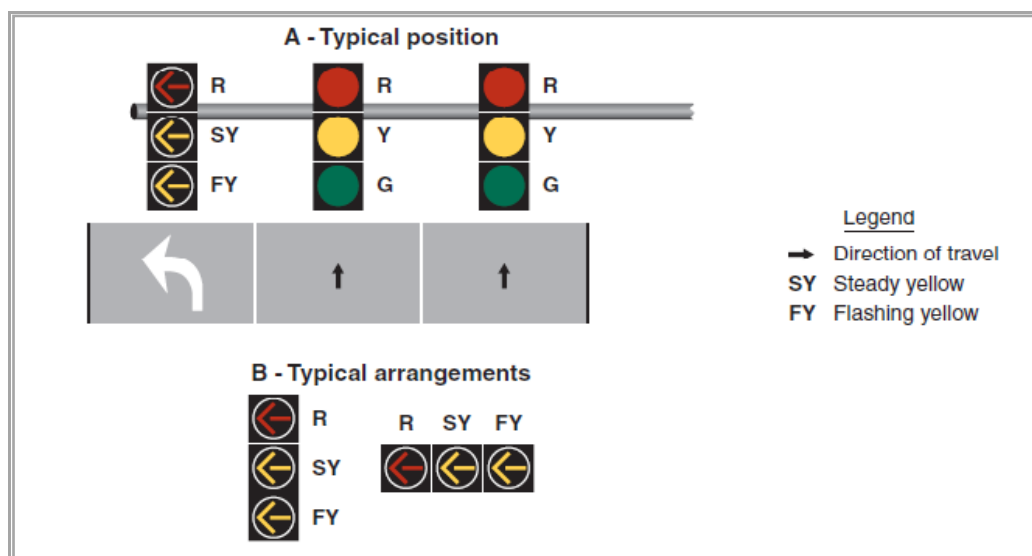


Figure 11-1/6: Typical Position and Arrangements of Separate Signal Faces with Flashing Yellow Arrow for Permissive Only Mode Left Turns [1, p.468]

- b) Protected Only Mode—turns made only when a left-turn GREEN ARROW signal indication is displayed.

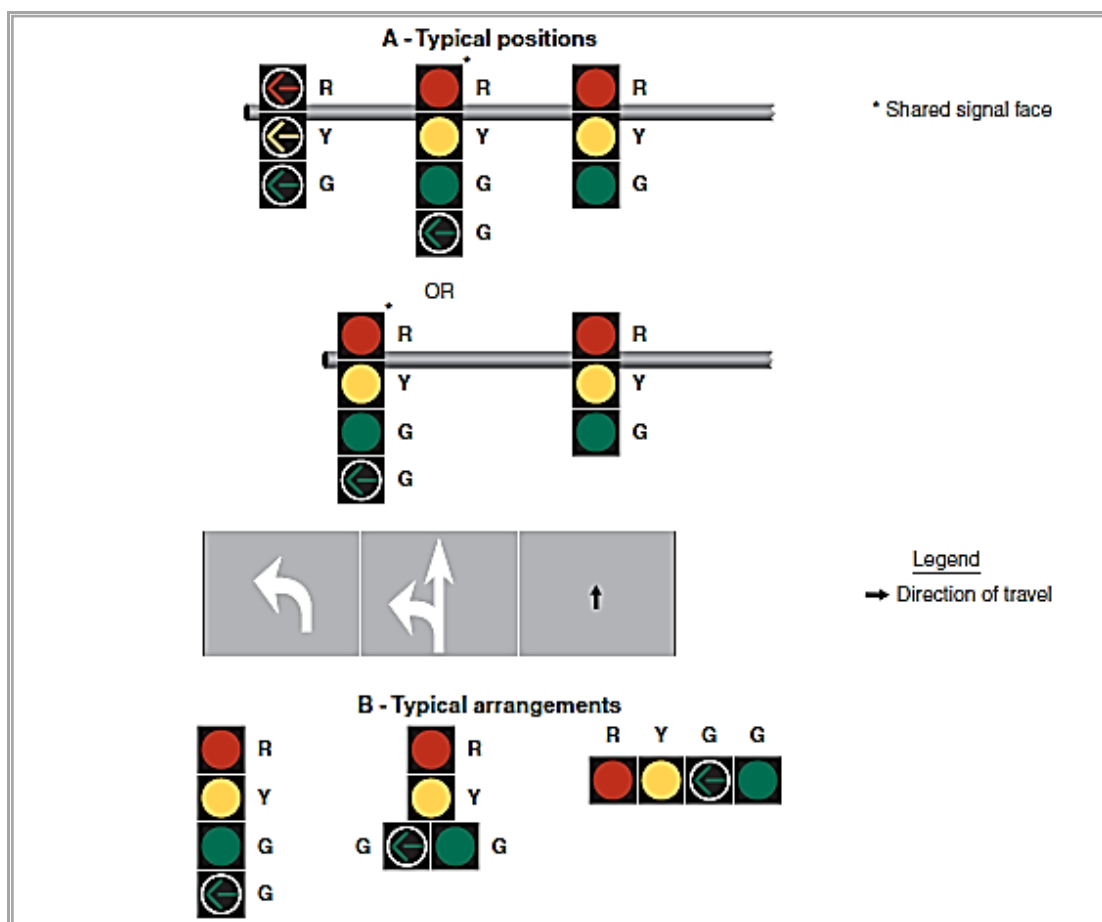


Figure 11-1/7: Typical Positions and Arrangements of Shared Signal Faces for Protected Only Mode Left Turns [1, p.470]

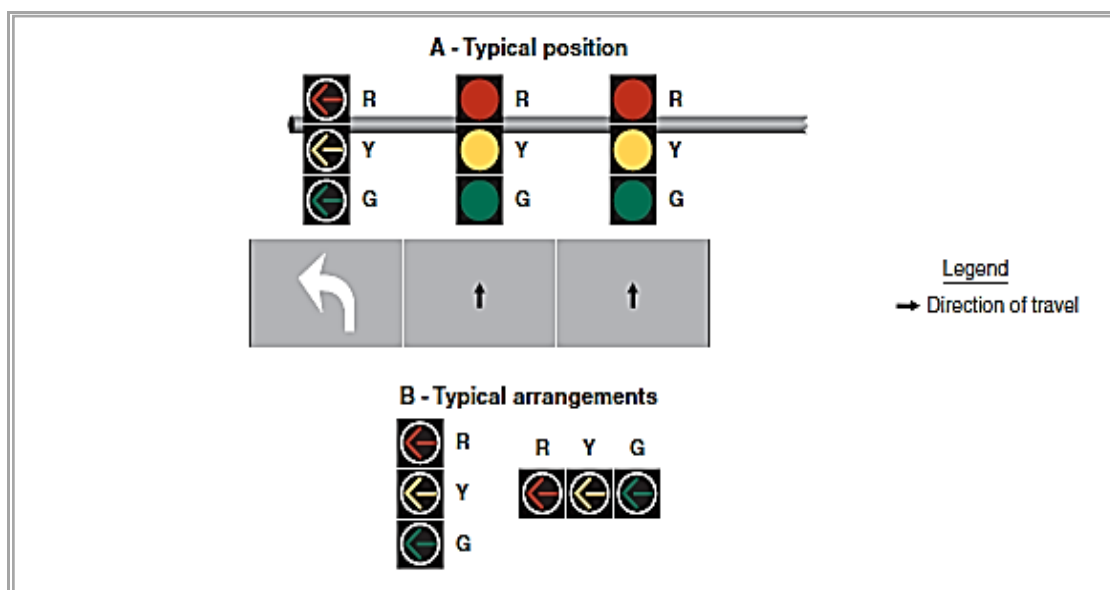


Figure 11-1/8: Typical Position and Arrangements of Separate Signal Faces for Protected Only Mode Left Turns [1, p.471]

Application of signal indication for right turn

Right turn traffic is controlled by one of the two modes as follows

- a) Permissive only mode: turns made on the circular green signal indication a flashing right-turn YELLOW ARROW signal indication or a flashing right- turn RED ARROW signal indication after yielding to pedestrians.

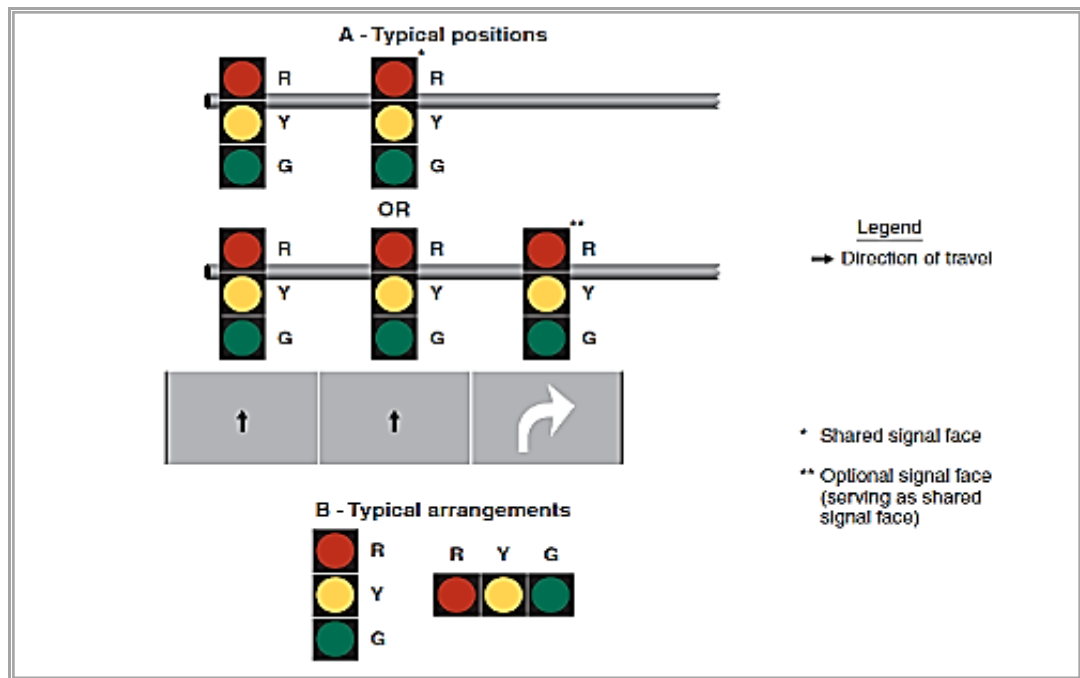


Figure 11-1/9: Typical Positions and Arrangements of Shared Signal Faces for Permissive Only Mode Right Turns [1, p.476]

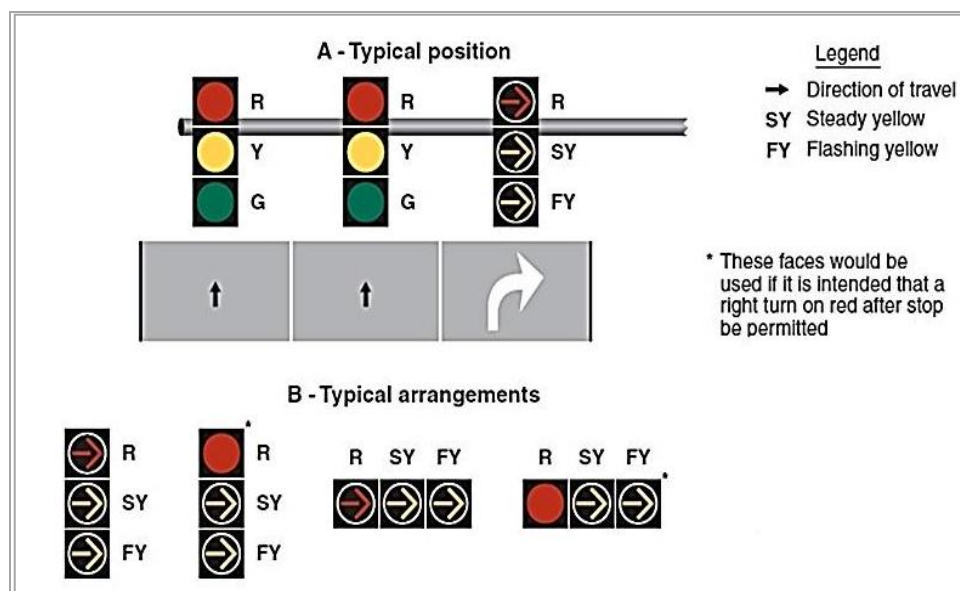


Figure 11-1/10: Typical Position and Arrangements of Separate Signal Faces with Flashing Yellow Arrow for Permissive Only Mode Right Turns [1, p.477]

- b) Protected Only Mode—turns made only when a right-turn GREEN ARROW signal indication is displayed.

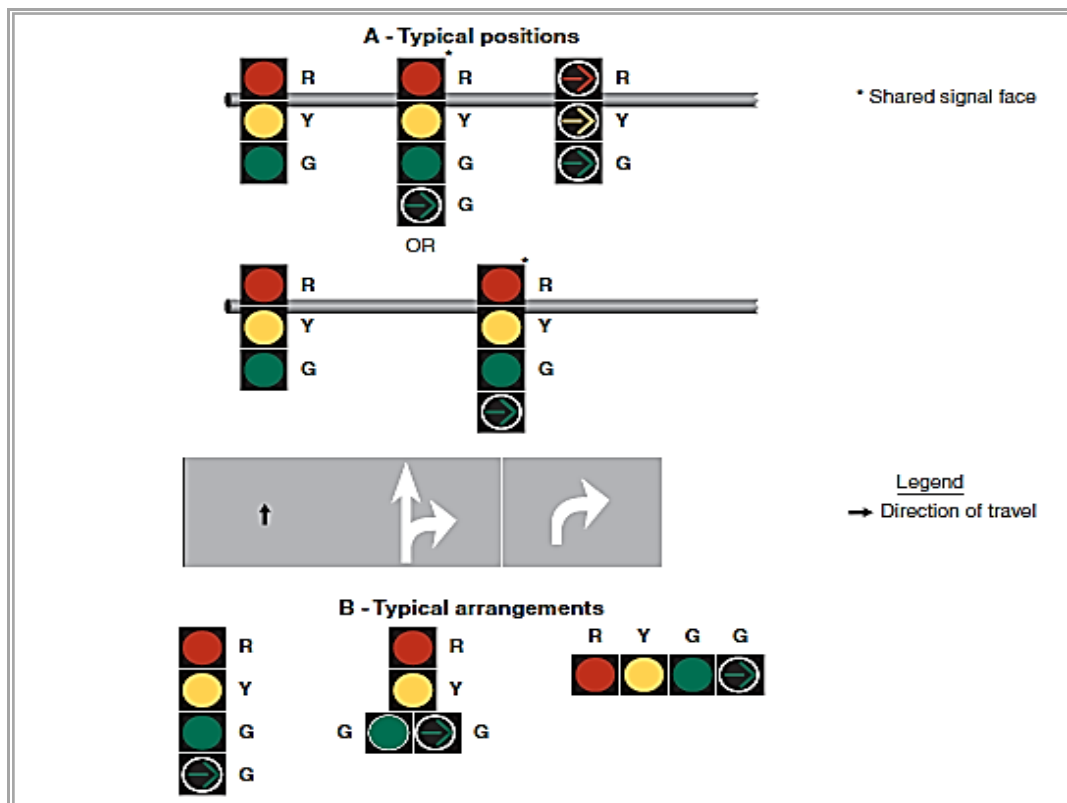


Figure 11-1/11: Typical Positions and Arrangements of Shared Signal Faces for Protected Only Mode Right Turns [1, p.479]

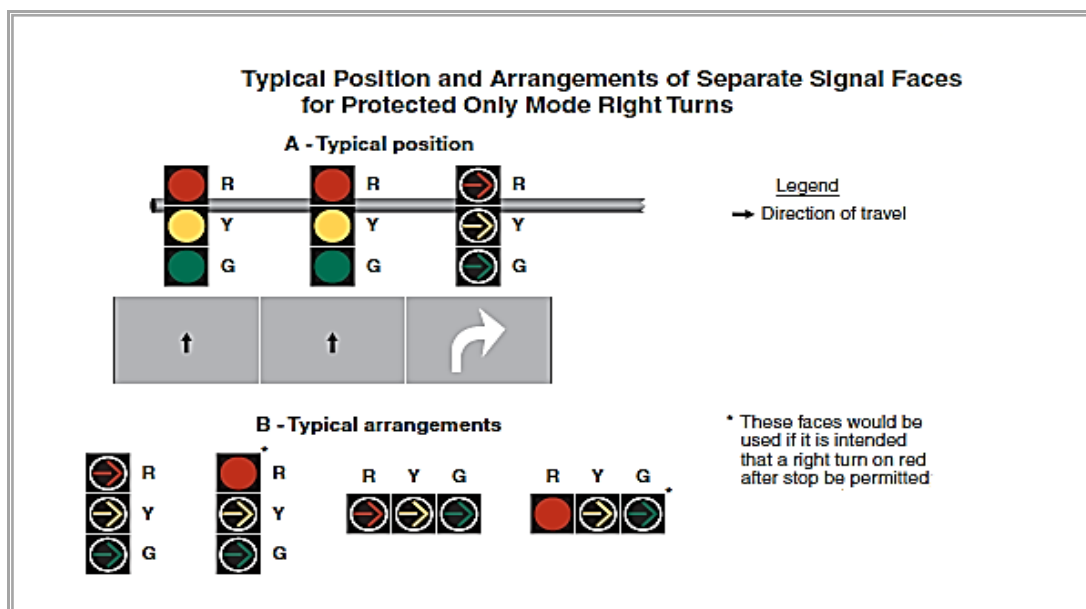


Figure 11-1/12: Typical Position and Arrangements of Separate Signal Faces for Protected Only Mode Right Turns [1, p.480]

- the sequence of signal indication for any phase within the cycle should be in the following order, when there is countdown for the green and red indications to inform the vehicular for the number of seconds remaining to turn the signal ahead

- 1- Red
- 2- Green
- 3- Yellow

If there is no countdown, as an alternative, the following sequence of signal indication can be followed.

- 1- Red
- 2- Red/Yellow
- 3- Green
- 4- Yellow

11-1/2/4 ILLUMINATION OF LENSES

Each traffic signal lens should be illuminated separately. An unobstructed, illuminated vehicular traffic control signal indication should be sufficiently bright to be clearly visible for a distance of at least 400 meters under normal atmospheric conditions. Signal dimmers should be provided for the yellow signal sections. The dimmer should allow the signal lamp to operate at full intensity under daylight conditions and to reduce proportionally to 25 ± 5 percent of full intensity at night. A dimmer should not control more than one yellow section for each direction.

11-1/2/5 VISIBILITY AND SHIELDING OF SIGNAL FACES

Every signal head and its supports should be designed so each signal face may be aimed independently of any other signal face

Every signal face should be aimed so its indications will have maximum visibility to the traffic it is intended to control. Each signal face should normally be aimed at a point approximately one meter above the approach roadway, substantially in advance of the stop line.

The distance from the stop line to this point should be approximately the distance traveled by a vehicle while the driver reacts to the signal indication and stops. This distance is shown in table (11-1/1).

It is important that signal indications not be visible to drivers who are not controlled by those indications. For this reason, visors should be used around all signal lenses. Visors also reduce “sun phantom” which gives an unlighted lens the false appearance of being lighted when it is facing a low sun. The visor should be 127 millimeters thick and not less than 22.86 centimeters in length.

Street, commercial, and advertising lighting behind and in line with traffic signal indications may seriously interfere with signal visibility and effectiveness. Backplates (a strip of thin material such as sheet aluminum or sheet plastic extending outward approximately 127 millimeters parallel to the signal face on all sides of the signal housing) are necessary. Backplates should be used on all signal heads placed over the roadway. Backplates should also be used on all other signal heads located where background colors and lights would interfere substantially with the effectiveness of the traffic signal indications. The front surface of backplates, the inside surfaces of visors, and the entire surface of louvers and fins should have a flat dull black finish. A backplate may have a white or silver border.

11-1/2/6 NUMBER AND LOCATION OF SIGNAL FACES

The signal faces for each approach to an intersection or a midblock location shall be provided as follows:

- a) A minimum of two signal faces shall be provided for the major movement on the approach, even if the major movement is a turning movement.
- b) See Section 11-1/2/3 -1 for left-turn signal faces.
- c) See Section 11-1/2/3 -2 for right-turn signal faces.
- d) Except where the width of an intersecting roadway or other conditions make it physically impractical:
 - 1- at least one and preferably both of the two signal faces required for the major movement on the approach shall be located:
 - Not less than 12 m beyond the stop line.
 - Not more than 55 m beyond the stop line unless a supplemental near side signal face is provided.
 - As near as practical to the line of the driver's normal view, if mounted over the roadway.
 - 2- A signal face installed to satisfy the requirements for left-turn signal faces (see Phrase 11-1/2/3 -1) and right-turn signal faces (see Phrase 11-1/2/3 -2), and at least one and preferably both of the two signal faces required for the major movement on the approach shall be located so that the maximum height to the top of the signal housing mounted over a roadway does not exceed 7.8 m above the pavement (for viewing distances more than 16 m). For viewing distances between 12 m and 16 m from the stop line, the maximum mounting height to the top of the signal housing shall be as shown on figure (11-1/13).
 - 3- At least one and preferably both of the signal faces required by Item A in this cluster shall be located between two lines intersecting with the center of the approach at a point 3 m behind the stop line, one making an angle of approximately 20 degrees to the right of the center of the approach extended, and the other making an angle of approximately 20 degrees to the left of the center of the approach extended (see figure 11-1/14).
 - 4- If both of the signal faces required by Item a in this cluster are post-mounted, they shall both be on the far side of the intersection, one on the right and one on the left of the approach lane(s).
- e) If the minimum sight distance in table (11-1/1) cannot be met, a sign shall be installed to warn approaching traffic of the traffic control signal.
- f) Required signal faces for through traffic on any one approach shall be located not less than 2.4 m apart measured horizontally perpendicular to the approach between the centers of the signal faces.

- g) If more than one turn signal face is provided for a protected-mode turn and if one or both of the signal faces are located over the roadway, the signal faces shall be located not less than 2.4 m apart measured horizontally perpendicular to the approach between the centers of the signal faces.

Supports for post-mounted signal heads and also signal heads at the side of a street should be placed not less than 0.5 meter back from the face of a curb. If there is no curb the supports and signal heads should be placed not less than 0.5 meter back from the edge of the shoulder.

Supports for mast arm pole-mounted signal heads should be placed in the most suitable roadway median facing the controlled approach. A signal should not obstruct a crosswalk.

Overhead (mast arm) indications should be used on intersection approaches where:

- More than 15 percent of the traffic is approaching at speeds in excess of 80 km/hr.
- On any approach where there are three or more approach lanes.
- Where physical conditions prevent drivers from having a continuous view of at least two signal indications.

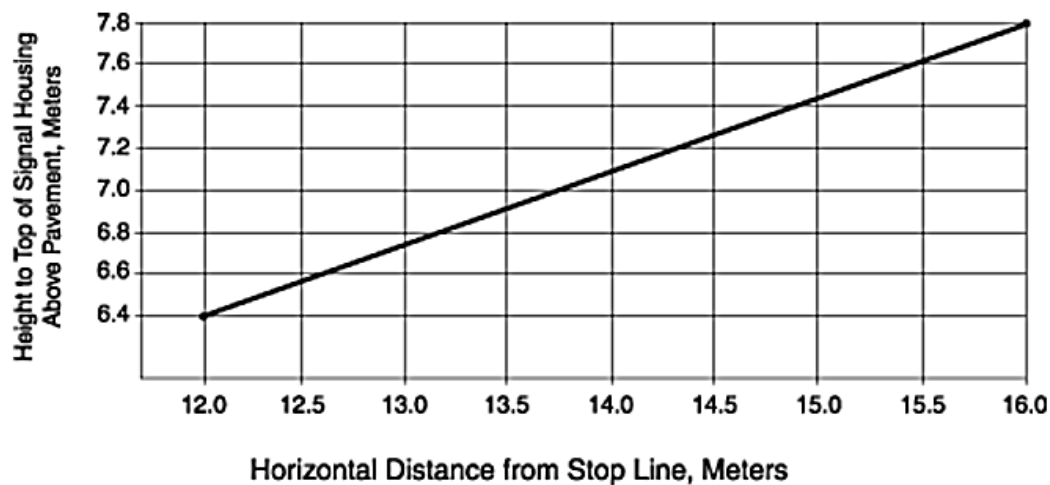


Figure 11-1/13: Maximum Mounting Height of Signal Faces Located Between 12meters and 16meters From Stop Line [2, p.4 D-14]

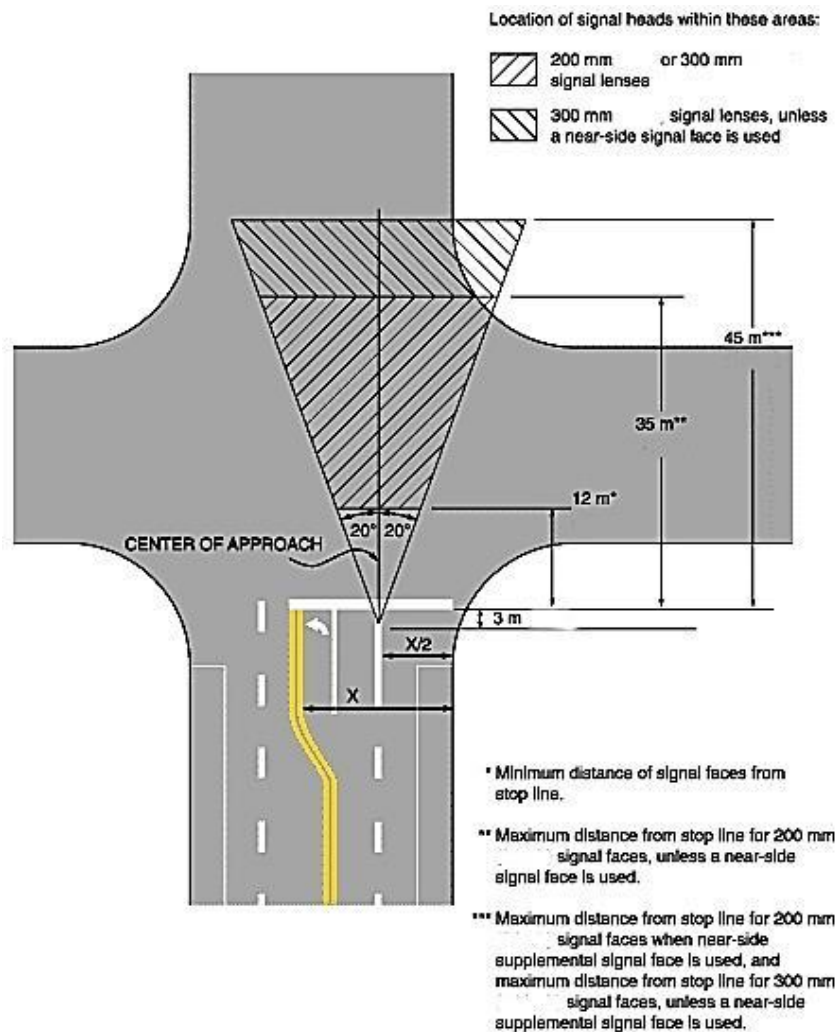


Figure 11-1/14: Horizontal Location of Signal Faces [2, p.4 D-15]

11-1/2/7 HEIGHT OF SIGNAL FACES

A signal face is most visible when installed directly in the driver's line of sight. The bottom of the housing of a signal face placed on a median island on the near side of an intersection approach should not be less than 1.25 meters and not more than 2.5 meters above the top of the median island. The bottom of the housing of signal face supported over a roadway should not be less than 5.5 meters nor more than 6.5 meters above the pavement below the signal. Typical mast arm and bracket mountings of signals are shown in figures (11-1/15) and (11-1/16).

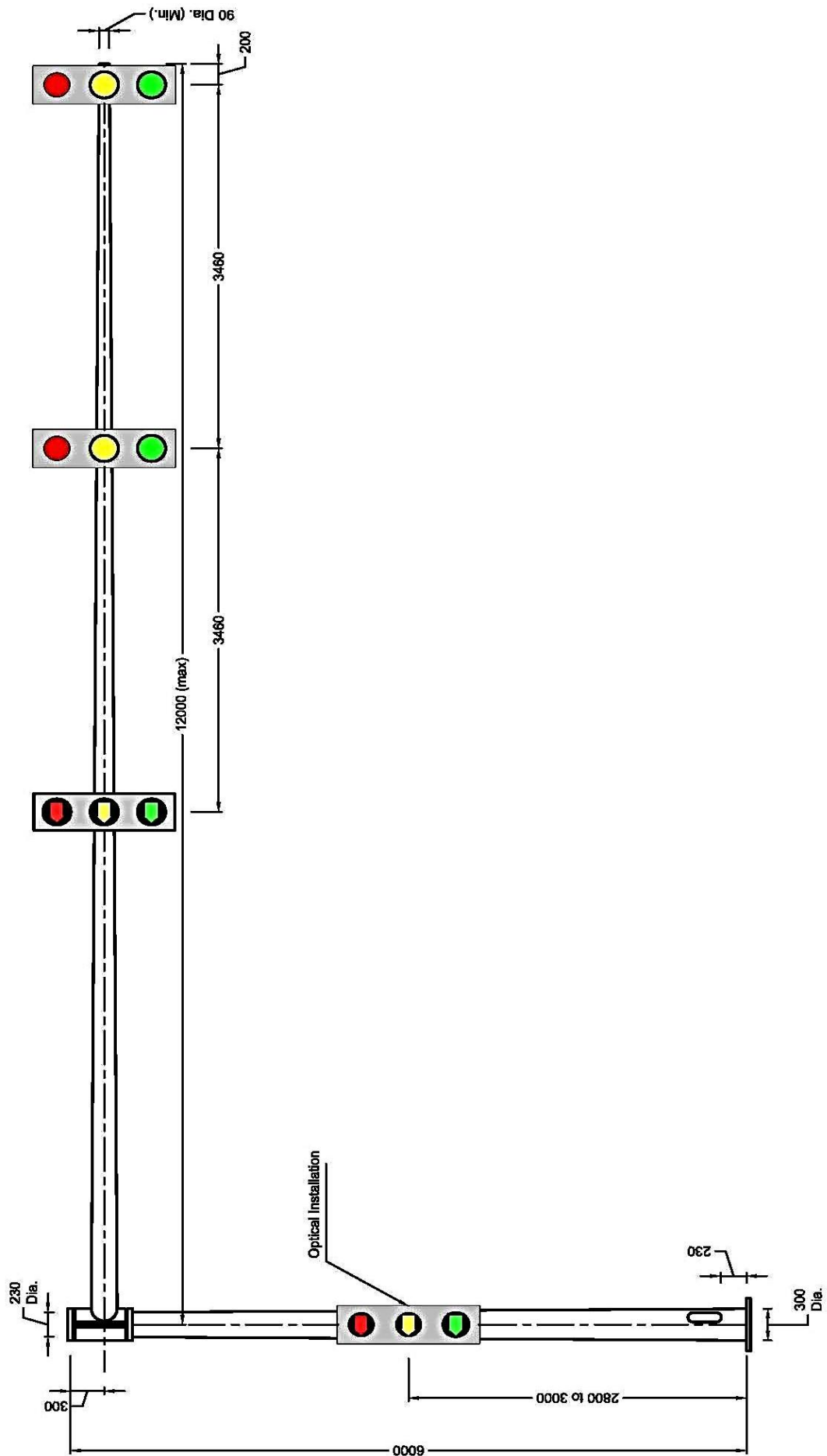


Figure 11-1/15: Typical Mast Arm of Signal (All Dimensions are in Millimeters)

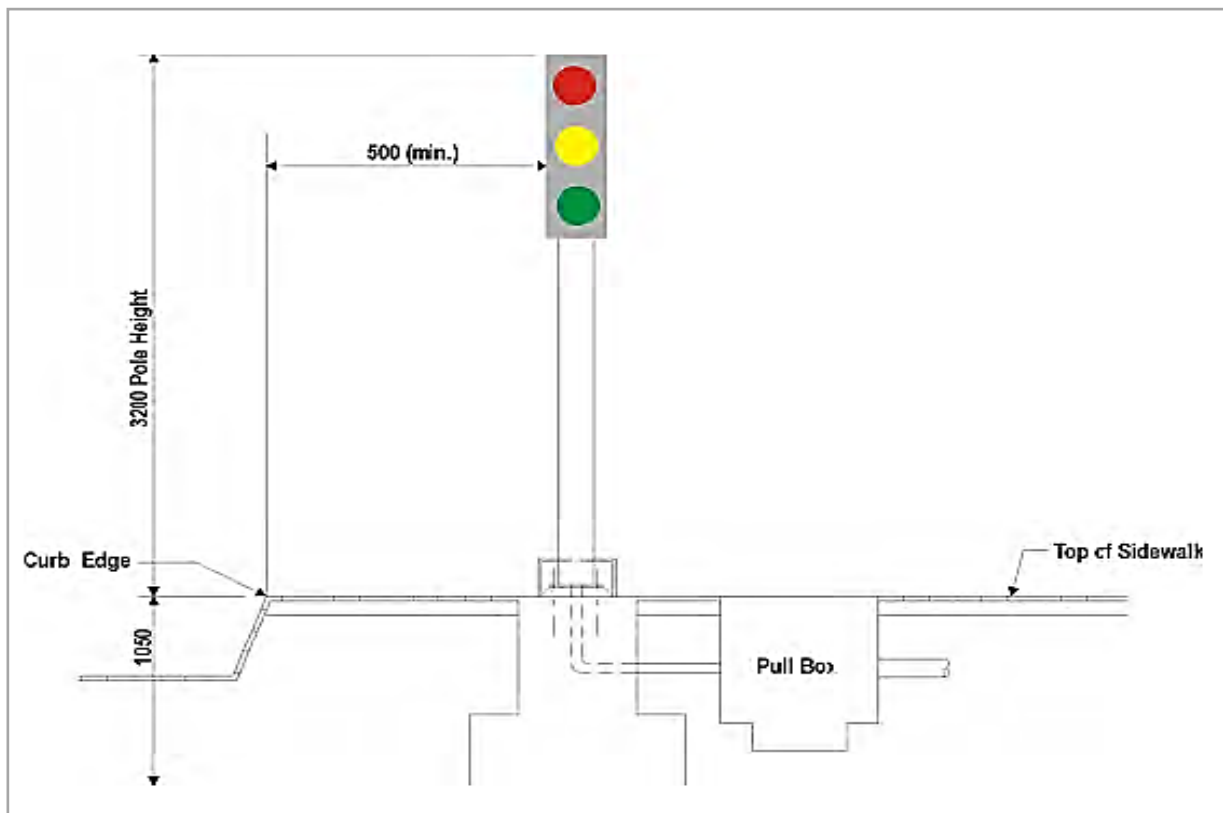


Figure 11-1/16: Typical Pole Mounting of Signal Heads (All dimensions are in millimeters)
[3, p.9-21]

11-1/2/8 YELLOW CHANGE AND RED CLEARANCE INTERVAL

A yellow signal indication shall be displayed following every CIRCULAR GREEN or GREEN ARROW signal indication. The exclusive function of the yellow change interval shall be to warn traffic of an impending change in the right-of-way assignment. The vehicle change interval should not be less than three seconds nor more than seven seconds. It can be calculated according to the following formula.

$$\text{Vehicle Change Interval} = T + \frac{0.91 V}{6.56A + 0.644G} \quad (11 - 1/1)$$

Where:

A= Maximum deceleration rate (m/sec.²), typically 3.048.

T= Reaction time, typically one sec.

G= Gradient of approach.

V= Speed of vehicle (km/hr.).

The yellow change interval may be followed by a red clearance interval to provide additional time before conflicting traffic movements, including pedestrians, are released. A red clearance interval should have a duration not exceeding 6 seconds. The all red clearance time can be calculated according to the following formula.

$$T_{(sec.)} = \frac{3.6 + W + C + L}{V} \quad (11 - 1/2)$$

Where:

W= Street width (meter).

C= Distance from stop line to near side of cross street (meter).

L= Length of the vehicle (meter).

V= Speed of Vehicle (km/hr.).

11-1/2/9 COORDINATION OF SIGNALS

Both pretimed and traffic-actuated signals within 800 meters of one another along a major route or in a network of major routes should normally be operated in coordination to minimize unnecessary delay and accidents. In the coordinated systems, all signals must have the same cycle length. Coordination of traffic signals can be accomplished with the aid of space–time diagram. An example of this diagram is shown in figure below.

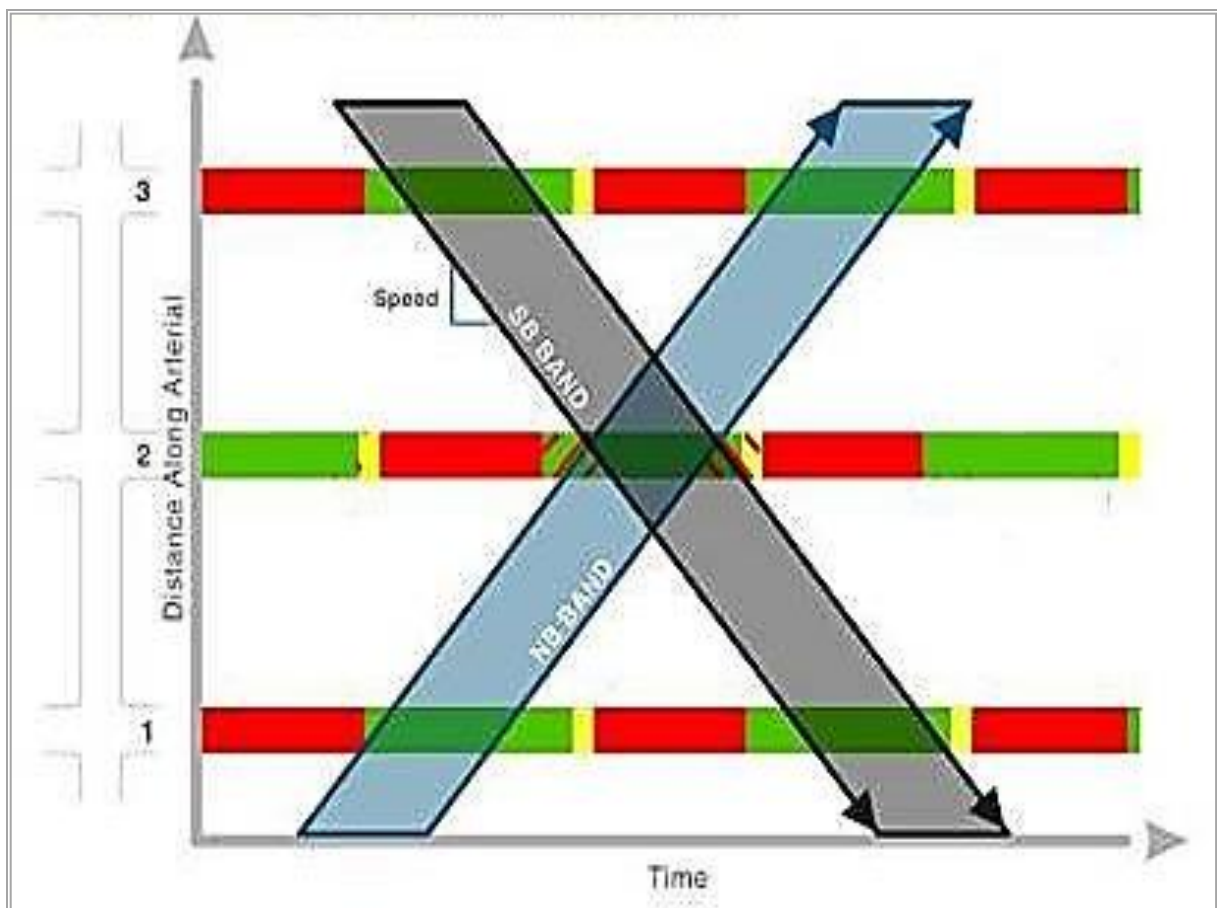


Figure 11-1/17: Time-Space Diagram, Example [5, p.6-21]

11-1/2/10 FLASH OPERATION OF SIGNAL

All traffic control signal installations should have electrical mechanisms which will flash the signal indications when activated by a manual or automatic switch. It should be possible to remove the signal timer without affecting the flashing of the indications. During flashing operation, the indications should be flashed at a rate of not less than one-half nor more than two-thirds of the total flash cycle.

During late night/early morning hours when the traffic volume is less than 120 vehicles per hour in the major direction, a pre-timed signal may be placed on flashing operation. Traffic actuated signals should normally not be placed on flashing operation during such times of lower traffic. In the flashing operation mode, the following signal indications should normally be displayed:

- Flashing yellow in all indications facing traffic on the main street or highway.
- Flashing red in all indications facing traffic on the minor or cross streets.

11-1/3 CONDITIONS THAT WARRANT THE INSTALLATION OF A TRAFFIC SIGNAL

A traffic control signal should not be installed unless one or more of the factors described below are met.

Warrant 1: Eight-Hour Vehicular Volume

The eight-hour vehicular volume warrant addresses the need for signalization for conditions that exist over extended periods of the day (a minimum of eight hours).

Two of the most fundamental reasons for signalization are:

- Heavy volumes on conflicting cross-movements that make it impractical for drivers to select gaps in an uninterrupted traffic stream through which to safely pass. This requirement is often referred to as the “minimum vehicular volume” condition (Condition A).
- Vehicular volumes on the major street are so heavy that no minor-street vehicle can safely pass through the major-street traffic stream without the aid of signals. This requirement is often referred to as the “interruption of continuous traffic” condition (Condition B).

Details of this warrant are shown in table (11-1/2). The warrant is met when:

- Either Condition A or Condition B is met to the 100% level.
- Either Condition A or Condition B is met to the 70% level, where the intersection is located in an isolated community of population 10,000 or less, or where the major-street approach speed is 70 km/hr. or higher.
- Both Conditions A and B are met to the 80% level.

Table 11-1/2: Warrant 1: Eight-Hour Vehicular Volume [1, p.456]

Condition A: Minimum Vehicular Volume							
Number of lanes for moving traffic on each approach		Vehicles per hour on major street (total, both approaches)			Vehicles per hour on higher-volume minor-street approach (one direction only)		
<u>Major Street</u>	<u>Minor Street</u>	<u>100%</u>	<u>80%</u>	<u>70%</u>	<u>100%</u>	<u>80%</u>	<u>70%</u>
1	1	500	400	350	150	120	105
2 or more	1	600	480	420	150	120	105
2 or more	2 or more	600	480	420	200	160	140
1	2 or more	500	400	350	200	160	140

Condition B: Interruption of Continuous Traffic							
Number of lanes for moving traffic on each approach		Vehicles per hour on major street (total, both approaches)			Vehicles per hour on higher-volume minor street approach (one direction only)		
<u>Major Street</u>	<u>Minor Street</u>	<u>100%</u>	<u>80%</u>	<u>70%</u>	<u>100%</u>	<u>80%</u>	<u>70%</u>
1	1	750	600	525	75	60	53
2 or more	1	900	720	630	75	60	53
2 or more	2 or more	900	720	630	100	80	70
1	2 or more	750	600	525	100	80	70

Warrant 2: Four-Hour Vehicular Volume

The Four-Hour Vehicular Volume signal warrant conditions are intended to be applied where the volume of intersecting traffic is the principal reason to consider installing a traffic control signal.

Figure (11-1/18) shows this warrant, which is in the form of a continuous graph.

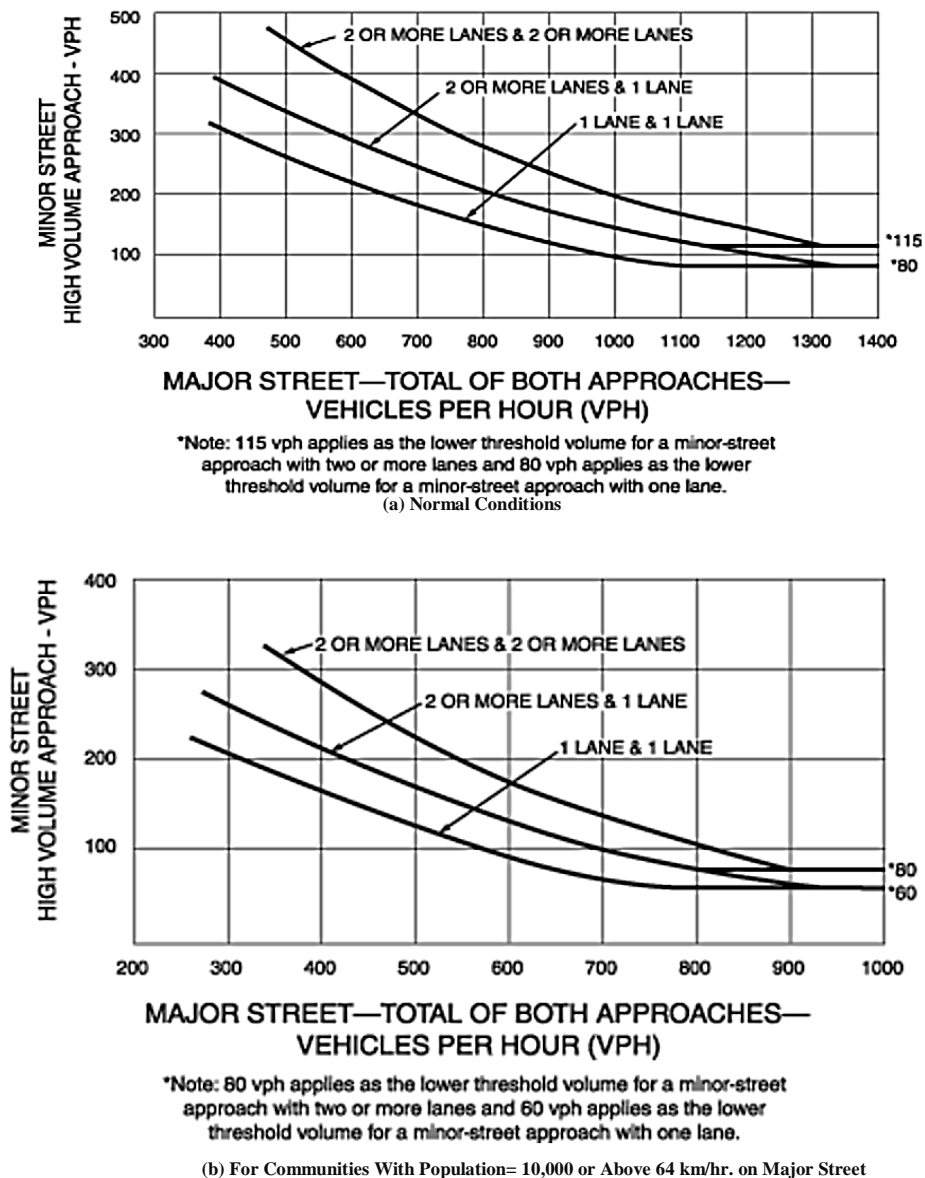


Figure 11-1/18: Warrant 2: Four-Hour Vehicular Volume Warrant [1, p.457]

Warrant 3: Peak Hour

This signal warrant shall be applied only in unusual cases, such as office complexes, manufacturing plants, industrial complexes, or high-occupancy vehicle facilities that attract or discharge large numbers of vehicles over a short time.

The need for a traffic control signal shall be considered if an engineering study finds that the criteria in either of the following two categories are met:

- A. If all three of the following conditions exist for the same 1 hour (any four consecutive 15-minute periods) of an average day:
 1. The total stopped time delay experienced by the traffic on one minor-street approach (one direction only) controlled by a STOP sign equals or exceeds: 4 vehicle-hours for a one-lane approach; or 5 vehicle-hours for a two-lane approach ;and

2. The volume on the same minor-street approach (one direction only) equals or exceeds 100 vehicles per hour for one moving lane of traffic or 150 vehicles per hour for two moving lanes; and
 3. The total entering volume serviced during the hour equals or exceeds 650 vehicles per hour for intersections with three approaches or 800 vehicles per hour for intersections with four or more approaches.
- B. The plotted point representing the vehicles per hour on the major street (total of both approaches) and the corresponding vehicles per hour on the higher-volume minor-street approach (one direction only) for 1 hour (any four consecutive 15-minute periods) of an average day falls above the applicable curve in figure (11-1/19) for the existing combination of approach lanes.

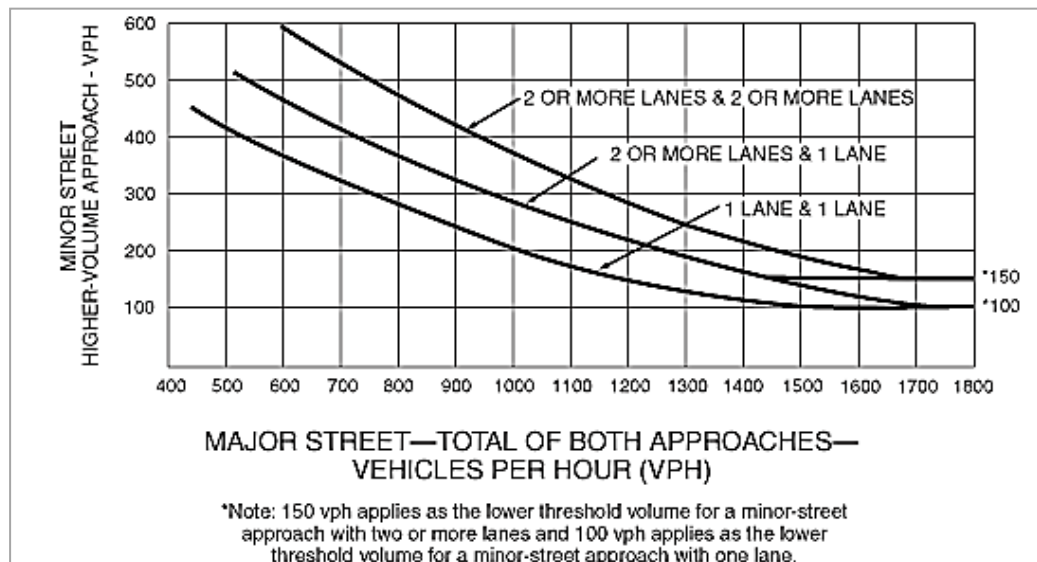


Figure 11-1/19: Warrant 3: Peak Hour [1, p.459]

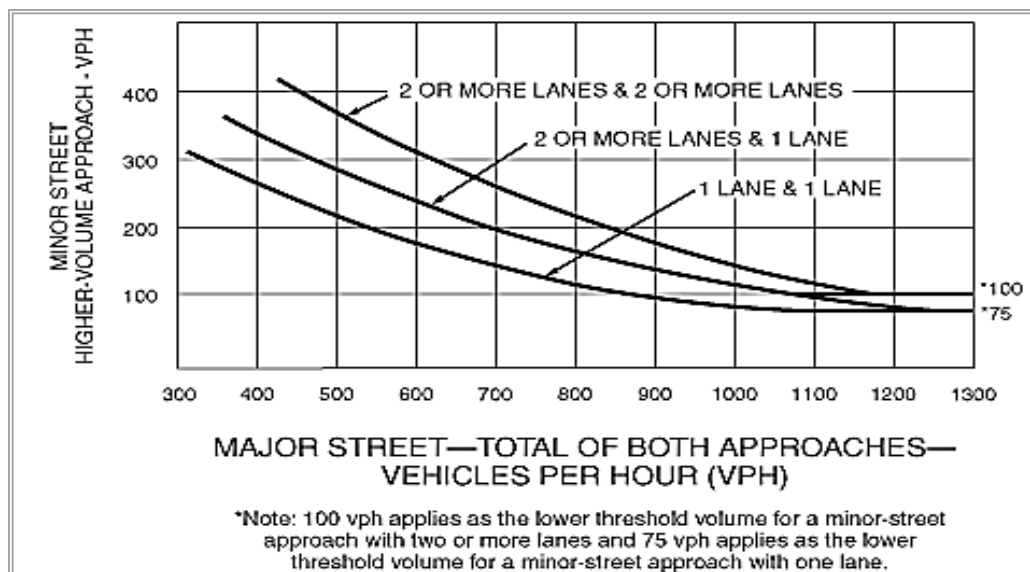


Figure 11-1/20: Warrant 3: Peak Hour (70% Factor) [1, p.459]

Figure (11-1/20) may be used in place of figure (11-1/19) to satisfy the criteria in the second category of this warrant.

Warrant 4: Pedestrian Volume

The need for a traffic control signal at an intersection or midblock crossing shall be considered if an engineering study finds that both of the following criteria are met:

- A. The pedestrian volume crossing the major street at an intersection or midblock location during an average day is 100 or more for each of any 4 hours or 190 or more during any 1 hour; and
- B. There are fewer than 60 gaps per hour in the traffic stream of adequate length to allow pedestrians to cross during the same period when the pedestrian volume criterion is satisfied. Where there is a divided street having a median of sufficient width for pedestrians to wait, the requirement applies separately to each direction of vehicular traffic.

The Pedestrian Volume signal warrant shall not be applied at locations where the distance to the nearest traffic control signal along the major street is less than 90 m, unless the proposed traffic control signal will not restrict the progressive movement of traffic.

If this warrant is met and a traffic control signal is justified by an engineering study, the traffic control signal shall be equipped with pedestrian signal heads conforming to requirements set forth in cluster (11-1/4).

Note: The criterion for the pedestrian volume crossing the major roadway may be reduced as much as 50 percent if the average crossing speed of pedestrians is less than 1.2 m/sec.

Warrant 5: School Crossing

This warrant is similar to the pedestrian warrant but is limited to application at designated school crossing locations, either at intersections or at mid-block locations.

The need for a traffic control signal shall be considered when an engineering study of the frequency and adequacy of gaps in the vehicular traffic stream as related to the number and size of groups of school children at an established school crossing across the major street shows that the number of adequate gaps in the traffic stream during the period when the children are using the crossing is less than the number of minutes in the same period and there are a minimum of 20 students during the highest crossing hour.

Note: The School Crossing signal warrant shall not be applied at locations where the distance to the nearest traffic control signal along the major street is less than 90 m, unless the proposed traffic control signal will not restrict the progressive movement of traffic.

Warrant 6: Coordinated Signal System

Progressive movement in a coordinated signal system sometimes necessitates installing traffic control signals at intersections where they would not otherwise be needed in order to maintain proper platooning of vehicles.

The need for a traffic control signal shall be considered if an engineering study finds that one of the following criteria is met:

- A. On a one-way street or a street that has traffic predominantly in one direction; the adjacent traffic control signals are so far apart that they do not provide the necessary degree of vehicular platooning.
- B. On a two-way street, adjacent traffic control signals do not provide the necessary degree of platooning and the proposed and adjacent traffic control signals will collectively provide a progressive operation.

Warrant 7: Crash Experience

The Crash Experience signal warrant conditions are intended for application where the severity and frequency of crashes are the principal reasons to consider installing a traffic control signal.

The need for a traffic control signal shall be considered if an engineering study finds that all of the following criteria are met:

1. Adequate trial of alternatives with satisfactory observance and enforcement has failed to reduce the crash frequency.
2. Five or more reported crashes of types susceptible to correction by a traffic control signal have occurred within a 12-month period, each involving a personal injury or property damage apparently exceeding the applicable requirements for a reportable crash.
3. For each of any eight hours of the day, vehicular volumes meet either Warrant 1A or Warrant 1B at the 80% level.

Warrant 8: Roadway Network

The need fix- a traffic control signal shall be considered if an engineering study finds that the common intersection of two or more major routes meets one or both of the following criteria:

1. The intersection has a total existing, or immediately projected, entering volume of at least 1,000 veh/hr. during the peak hour of a typical weekday, and has five-year projected traffic volumes, based upon an engineering study, that meet one or more of Warrants 1, 2 and 3 during an average weekday.
2. The intersection has a total existing, or immediately projected, entering volume of at least 1,000 veh/hr. for each of, any five hours of a non-normal business day (Friday or Saturday).

11-1/4 PEDESTRIAN CONTROL SIGNALS

11-1/4/1 PEDESTRIAN SIGNAL HEAD INDICATIONS

Pedestrian signal indications are used to give pedestrians better information for safe crossing than can be given by the vehicular indications alone. These indications consist of the green figure of a walking man, symbolizing permission to walk (WALK) and a representation of

a red-colored hand, symbolizing prohibition of walking (DON'T WALK) (see figure 11-1/21). Another permissible combinations would be a green figure of a walking man and a red figure of standing man, respectively.

All new pedestrian signal head indications shall be displayed within a rectangular background and the Symbols at least 150 millimeters high should be used if the distance from the near curb to the pedestrian signal indication is 35 meters or less. Where that distance is more than 35 m, the symbols should be at least 225 millimeters in height.

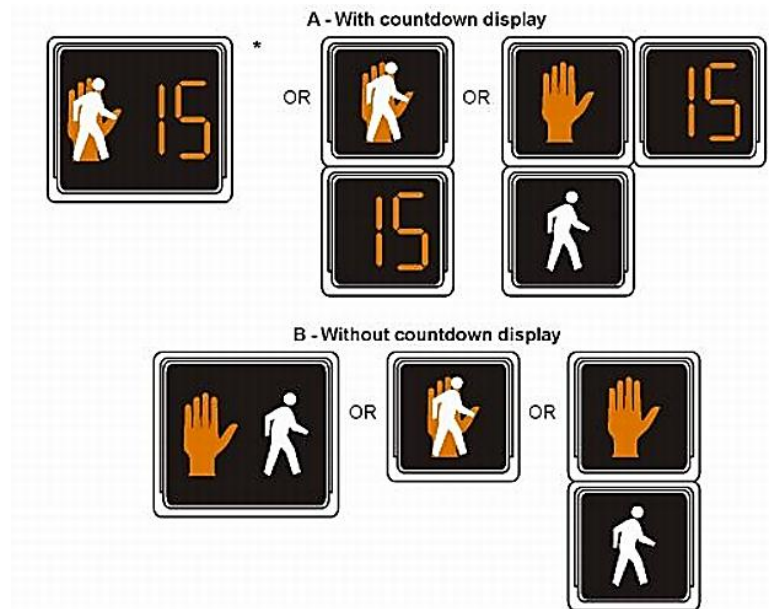


Figure 11-1/21: Typical Pedestrian Signal Indications [1, p.496]

The meanings of pedestrian signal indications are:

- The hand symbol (when steadily illuminated) means pedestrians facing the signal should not enter the roadway.
- The walking man symbol (when flashing) means pedestrians should not start to cross the roadway. However, any pedestrian who has partly completed crossing the roadway should proceed to a sidewalk or to a safety island.
- The walking man pedestrian indication means pedestrians may cross the roadway within the crosswalk limits in the direction of the indication.

Pedestrian signal indications should be installed in conjunction with vehicular traffic control signals under any of the following conditions:

- When a traffic control signal is installed because either the Pedestrian Volume or the School Crossing Criterion is met.
- When it is necessary to give vehicles a signal indication to stop them from crossing the path of a pedestrian movement. A signal indication is then necessary to tell pedestrians when they may walk.
- When vehicular signal indications are not sufficiently visible to pedestrians, particularly on one-way streets or at a "T" intersection.
- At officially designed school crossings at intersection signalized under any criterion.
- At wide intersections where pedestrians must make the crossing in stages.

11-1/4/2 PEDESTRIAN SIGNAL PHASES AND INTERVAL TIME

When pedestrian signal heads are used, a WALKING PERSON (symbolizing WALK) signal indication shall be displayed only when pedestrians are permitted to leave the curb or shoulder.

A pedestrian clearance time shall begin immediately following the WALKING PERSON (symbolizing WALK) signal indication. The first portion of the pedestrian clearance time shall consist of a pedestrian change interval during which a flashing UPRAISED HAND (symbolizing DONT WALK) signal indication shall be displayed. The remaining portions shall consist of the yellow change interval and any red clearance interval (prior to a conflicting green being displayed), during which a flashing or steady UPRAISED HAND (symbolizing DONT WALK) signal indication shall be displayed.

If countdown pedestrian signals are used, a steady UPRAISED HAND (symbolizing DONT WALK) signal indication shall be displayed during the yellow change interval and any red clearance interval (prior to a conflicting green being displayed).

The minimum WALK interval, when the green walking man indication is shown, should be not less than the minimum time required to cross the street at walking speed. The minimum WALK interval is typically four to seven seconds. Additional time, if needed, should be added to the clearance period. The WALK indication may be shown for the vehicular green period minus the time required for the pedestrian change/clearance indication. However, the length of the WALK period need not equal the time required for pedestrians to walk completely across the street, as they can complete their crossing during the change/clearance period. At traffic-actuated signals, the standing man indication should be shown unless there has been a pedestrian actuation.

A pedestrian change/clearance interval should always be provided where pedestrian signal indications are used. This interval should be long enough to allow a pedestrian to walk across the entire roadway or to a median island before vehicles crossing his path receive a green indication.

The normal walking speed is assumed to be 1.20 meters per second. Pedestrian indications should always be displayed when the traffic signal is operating as a stop-and-go device. Pedestrian indications should not be illuminated when the traffic control signal is operating as a flashing device. When pedestrian pushbuttons are provided, the buttons should be operating at any time the pedestrian indications are operating.

11-1/4/3 LOCATION AND HEIGHT OF PEDESTRIAN SIGNAL HEADS

Pedestrian signal heads shall be mounted with the bottom of the signal housing including brackets not less than 2.1 m nor more than 3 m above sidewalk level, and shall be positioned and adjusted to provide maximum visibility at the beginning of the controlled crosswalk.

If pedestrian signal heads are mounted on the same support as vehicular signal heads, there shall be a physical separation between them.

11-1/5 FLASHING BEACONS

A Flashing Beacon is a highway traffic signal with one or more signal sections that operates in a flashing mode. It can provide traffic control when used as an intersection control beacon or warning in alternative uses.

Flashing Beacon units and their mountings shall follow the provisions of clause 11-1/2, except as specified herein.

Beacons shall be flashed at a rate of not less than 50 nor more than 60 times per minute. The illuminated period of each flash shall not be less than one-half and not more than two-thirds of the total cycle.

Note: If used to supplement a warning or regulatory sign, the edge of the beacon signal housing should normally be located no closer than 300 mm outside of the nearest edge of the sign.

11-1/5/1 WARNING BEACON

Typical applications of Warning Beacons include the following:

- A. At obstructions in or immediately adjacent to the roadway;
- B. As supplemental emphasis to warning signs;
- C. As emphasis for midblock crosswalks;
- D. On approaches to intersections where additional warning is required, or where special conditions exist; and
- E. As supplemental emphasis to regulatory signs, except STOP, YIELD, DO NOT ENTER, and SPEED LIMIT signs.

A Warning Beacon shall consist of one or more signal sections of a standard traffic signal face with a flashing CIRCULAR YELLOW signal indication in each signal section. It shall be used only to supplement an appropriate warning or regulatory sign or marker. The beacon shall not be included within the border of the sign except for SCHOOL SPEED LIMIT sign beacons.

If a Warning Beacon is suspended over the roadway, the clearance above the pavement shall be at least 4.6 m but not more than 5.8 m .Warning Beacons should be operated only during those hours when the condition or regulation exists.

11-1/5/2 INTERSECTION CONTROL BEACON

An Intersection control beacon shall consist of one or more signal faces directed toward each approach to an intersection. Each signal face shall consist of one or more signal sections of a standard traffic signal face, with flashing CIRCULAR YELLOW or CIRCULAR RED signal indications in each signal face. They shall be installed and used only at an intersection to control two or more directions of travel.

Intersection control beacons are used at intersections where traffic control signals are not warranted, but accident experience indicates a special hazard. Only the following combinations of signal indications should be used:

- Yellow indications on one route (normally the major route) and red on all other approaches, or
- Red on all approaches to the intersection. This is permissible only where an all-way stop is warranted.

11-1/5/3 STOP BEACON

A stop sign beacon is a signal with one or two flashing RED DISK indications used with a STOP sign. The lens of a stop sign beacon should have a visible diameter of not less than 200 millimeters. Where greater effectiveness is needed, two separate beacons and sing installations may be made. One should be on the right side of the approach and one overhead or on the left of the approach. This will usually be more effective than a beacon with a 300-millimeter lens. If two lenses are used with a STOP sign, the lenses should be placed above and below the sign, and should be alternately illuminated. The standard beacon should have one lens. Two-lens beacons should only be used where accident experience shows single lens beacons have not been effective.

11-1/5/4 SPEED LIMIT BEACON

Speed limit sign beacons are intended for use where signs alone have not been effective in controlling speed. A speed limit sign beacon may be used with a fixed or variable speed Limit sign. If applicable, a flashing Speed Limit Sign Beacon (with an appropriate accompanying sign) may be used to indicate that the displayed speed limit is in effect. Too frequent use of the beacons, however, may reduce their effectiveness. A speed limit sign beacon is a signal having one or two YELLOW DISK lens sections. If one lens is used, it should have a visible diameter of not less than 200 millimeters. If two lenses are used, the lenses should have visible diameter of not less than 150 millimeters.

11-2 TRAFFIC SIGNS

11-2/1 TRAFFIC SIGNS CHARACTERISTICS

11-2/1/1 FUNCTION AND PURPOSE OF SIGNS

The main purpose of highway traffic signs is to aid the safe and orderly movement of traffic. Signs are needed to give information about highway routes, direction, destinations and points of interest. They are needed to give information on special regulations which apply only at specific places or at specific times. They are essential to inform drivers about hazards which are not self-evident.

11-2/1/2 SIGN SHAPES

Standard sign shapes and their function are as shown in figure (11-2/1).


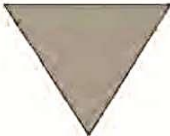




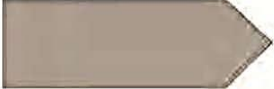
Standard Traffic Sign Shape	Function(s)
	Reserved exclusively for the STOP sign.
	Reserved exclusively for the GIVE WAY (YIELD) sign.
	Reserved exclusively for REGULATORY signs (mandatory and prohibitory).
	Primarily used for REGULATORY SIGNS. Also used for DIAGRAMMATIC WARNING signs.
	Reserved exclusively for ADVANCE WARNING signs.
	Primary shape for GUIDE signs including DIRECTION signs and GENERAL INFORMATION signs. Also used for ONE WAY sign and SUPPLEMENTARY PLATE or QUALIFICATION PLATE below other signs.
	Reserved exclusively for CHEVRON DIRECTION signs.

Figure 11-2/1: Standard Sign Shapes [3, p.2-7]

11-2/1/3 SIGN COLORS

The colors to be used on standard signs should be as follows:

1. Warning signs should have a white background with a red border and black symbols.
2. Regulatory signs which are prohibitive in nature should have a white background with a red border and black symbols.

Regulatory signs which are mandatory in nature should have a blue background with white symbols.

3. Stop signs should have a red background with a white border and white symbols.
4. Priority signs should have a yellow background with a white border and black symbols.
5. GUIDE signs which give information regarding services should have a blue background with a white inset and a black message or symbols. The direction signs should have a green background with a white message or symbols for streets and highways other than expressway.

For expressways, the direction signs should have a blue background with a white message or symbols.

11-2/1/4 SIGN BORDER

The signs which are rectangular in shapes (mostly guide signs) should have a narrow border of the same color as the message. This improves the appearance. For 60cm signs, the border should be in the range of 1.0 to 1.5cm set 1cm from the edge, and for other sign sizes approximately in proportion but not to exceed the stroke-width of the major lettering of the sign. On signs exceeding 2m by 3m in size, the border should be approximately 5cm wide or, on unusually large signs, 7.5cm.

The corners of the border should be rounded except for STOP signs. When practicable, the corner of the sign panels should also be rounded to fit the border.

11-2/1/5 RETROREFLECTION AND ILLUMINATION

Regulatory, warning, and guide signs shall be retroreflective or illuminated to show the same shape and similar color by both day and night. The requirements for sign illumination shall not be considered to be satisfied by street or highway lighting. Sign elements may be illuminated by the means shown in table (11-2/1). Retroreflection of sign elements may be accomplished by the means shown in table (11-2/2).

11-2/1/6 SIGN DIMENSIONS

The side length or diameter of the regulatory and warning signs to be used for arterial roads should be 90 cm and for all other roads 60 cm.

Table 11-2/1: Illumination of Sign Elements [1, p. 29]

Means of Illumination	Sign Element to be Illuminated
Light behind the sign face	Symbol or word message Background Symbol, word message, and background (through a translucent material)
Attached or independently mounted light source designed to direct essentially uniform illumination onto the sign face	Entire sign face
Light emitting diodes (LEDs)	Symbol a word message Portions of the sign border
Other devices, or treatments that highlight the sign shape, color, or message: Luminous Tubing Fiber optic Incandescent light bulbs Luminescent panels	Symbol or word message Entire sign face

Table 11-2/2: Retroreflection of Sign Elements [1, p. 29]

Means of Retroreflection	Sign Element
Reflector "buttons" or similar units	Symbol Word message Border
A material that has a smooth, sealed outer surface over a microstructure that reflects light	Symbol Word message Border Background

Light Emitting Diode (LED) units may be used individually within the face of a sign and in the border of a sign, except for Changeable Message Signs, to improve the conspicuity, increase the legibility of sign legends and borders, or provide a changeable message. Individual LED pixels may be used in the border of a sign.

Note: All overhead signs should be illuminated except in cases where all of the following conditions are met:

- When the roadway is not lighted, and
- When the sign is visible from a distance of 370 meters or more, and
- When the horizontal curvature of the road approaching the sign has no less than a 250 meter radius.

11-2/2 TRAFFIC SIGNS PLACEMENT

The general rule is to locate signs on the right hand side of the roadway. On wide expressways or where some degree of lane-use control is desirable or where space is not available at the roadside, overhead signs are often necessary.

In some circumstances, signs may be placed on channelizing islands (as on sharp right hand curves) on the left hand shoulder of the road, directly in front of the approaching vehicles. A supplementary sign located on the left of the roadway is often helpful on a multi-lane road where traffic in the right hand lane may obstruct the view to the right. Normally, signs should be individually erected on separate posts or mountings except where one sign supplements another.

11-2/2/1 MOUNTING HEIGHT

Signs installed at the side of the road in rural districts shall be at least 1.5 m, measured from the bottom of the sign to the near edge of the pavement. Where parking or pedestrian movements occur, the clearance to the bottom of the sign shall be at least 2.1 m.

Directional signs on freeways and expressways shall be installed with a minimum height of 2.1 m. If a secondary sign is mounted below another sign, the major sign shall be installed at least 2.4 m and the secondary sign at least 1.5 m above the level of the pavement edge. All route signs, warning signs, and regulatory signs on freeways and expressways shall be at least 2.1 m above the level of the pavement edge.

Overhead mounted signs shall provide a vertical clearance of not less than 5.2 m to the sign, light fixture, or sign bridge, over the entire width of the pavement and shoulders except where a lesser vertical clearance is used for the design of other structures.

11-2/2/2 LATERAL OFFSET

The minimum lateral offset is intended to keep trucks and cars that use the shoulders from striking the signs or supports.

For post-mounted signs, the minimum lateral offset should be 3.7 m from the edge of the traveled way. If a shoulder wider than 1.8 m exists, the minimum lateral offset for ground-mounted signs should be 1.8 m from the edge of the shoulder.

For overhead sign supports, the minimum lateral offset from the edge of the shoulder (or if no shoulder exists, from the edge of the pavement) to the near edge of overhead sign supports (cantilever or sign bridges) shall be 1.8 m. Overhead sign supports shall have a barrier or crash cushion to shield them if they are within the clear zone.

In urban areas a minimum lateral offset of 0.6 m from face of the curb should be used.

Note: In areas where it is impractical to locate a sign with the lateral offset prescribed by this clause, a lateral offset of at least 0.6 m may be used for rural area and 0.3m for urban area.

Figures (11-2/2) and (11-2/3) illustrate some examples of the lateral offset and mounting height requirements.

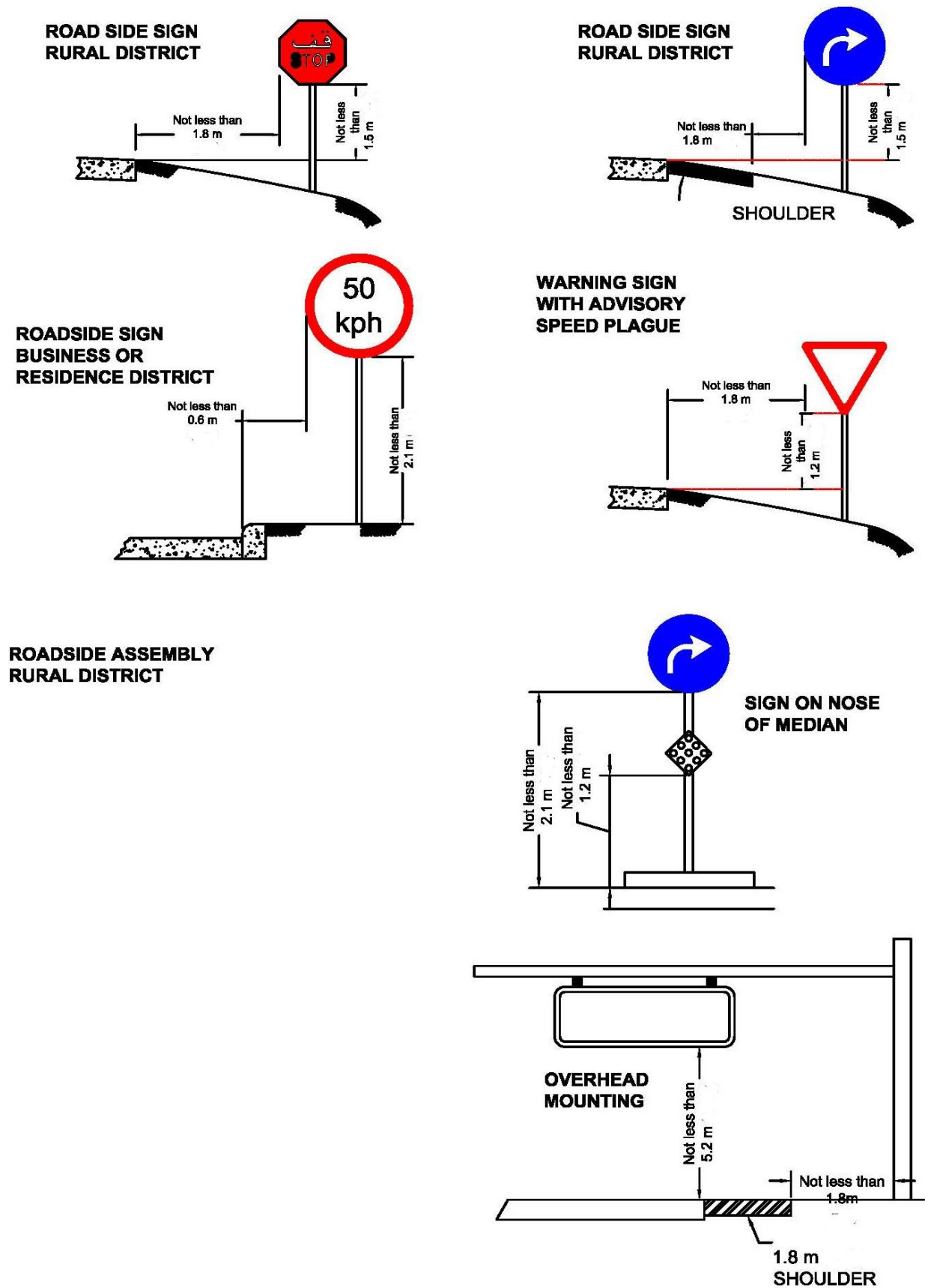


Figure 11-2/2: Examples of Heights and Lateral Locations of Sign Installations [1, p.38]

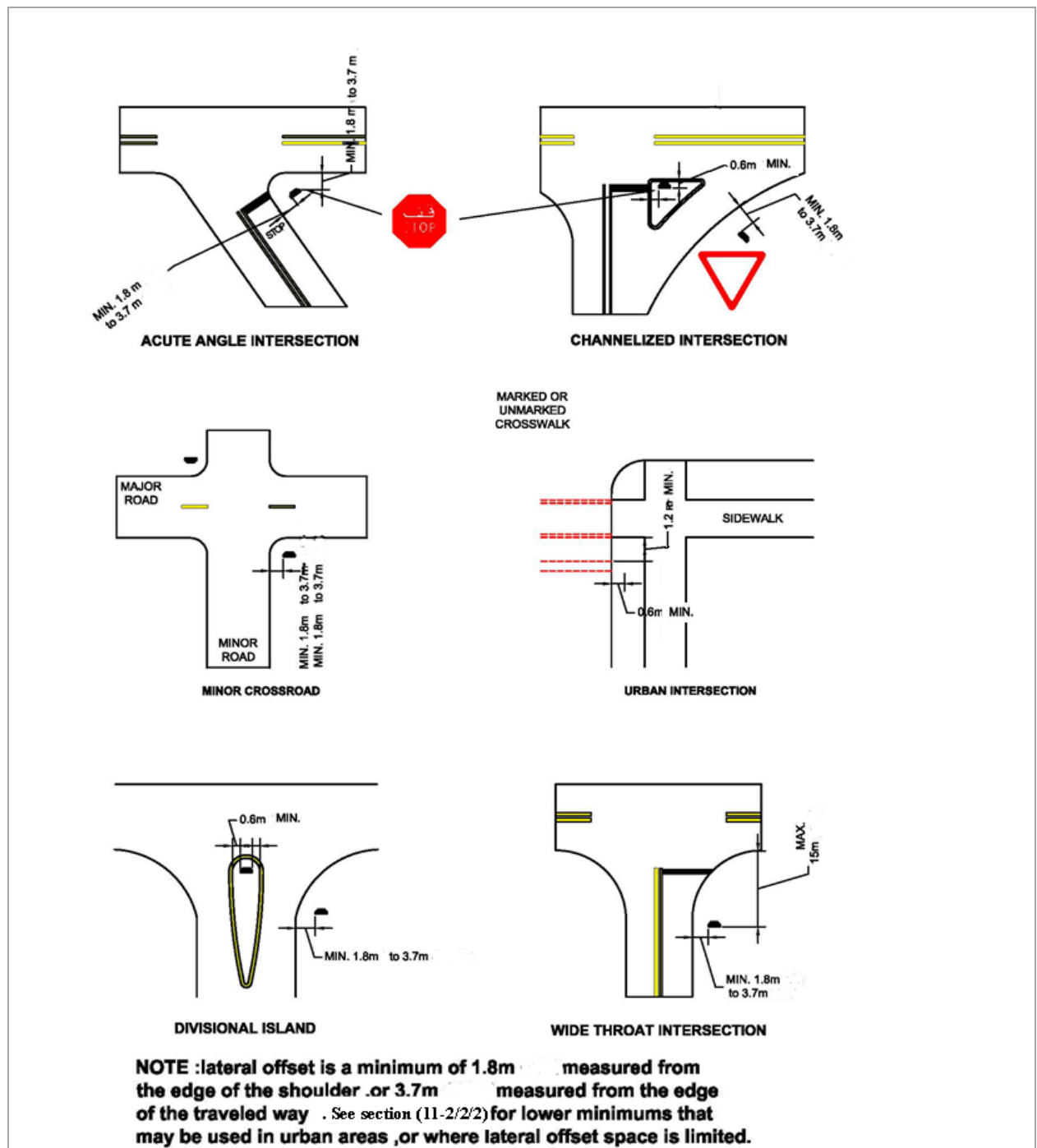


Figure 11-2/3: Examples of Locations for Some Typical Signs at Intersections [1, p.39]

11-2/2/3 LONGITUDINAL PLACEMENT

The longitudinal placement of signs along a road depends on the sign type, criticality of message, and maneuver required. Signs should be located so that they do not obscure each other nor are hidden from view by other roadside objects. The minimum spacing for signs is shown in figure (11-2/4).

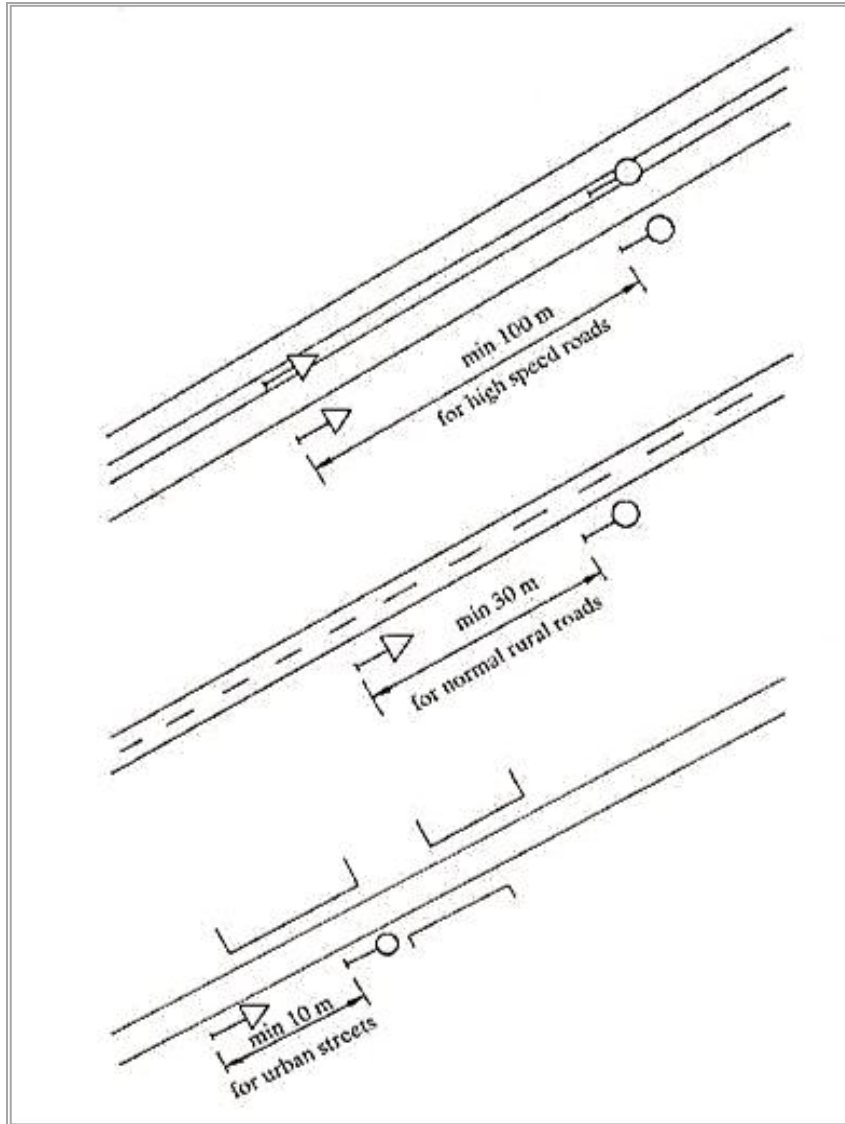


Figure 11-2/4: Minimum Spacing of Traffic Signs [4, p. IV-4]

11-2/2/4 OVERHEAD SIGN INSTALLATION

The factors justifying the erection of overhead sign displays are not definable in specific numerical terms but the following conditions deserve consideration:

- A. Traffic volume at or near capacity,
- B. Complex interchanges design,
- C. Three or more lanes in each direction,
- D. Restricted sight distance,
- E. Closely-spaced interchanges,

- F. Multi-lane exits,
- G. Large percentage of trucks,
- H. Street lighting background,
- I. High-speed traffic,
- J. Consistency of sign message location through a series of interchanges,
- K. Insufficient space for post-mounted signs,
- L. Junction of two freeways, and
- M. Left exit ramps.

11-2/2/5 ORIENTATION ANGLE

Normally, signs should be mounted at right angles to the direction of, and facing, the traffic that they are intended to serve. Where mirror reflection from the sign face is very high so as to reduce legibility, it should be oriented at an angle shown in figures below.

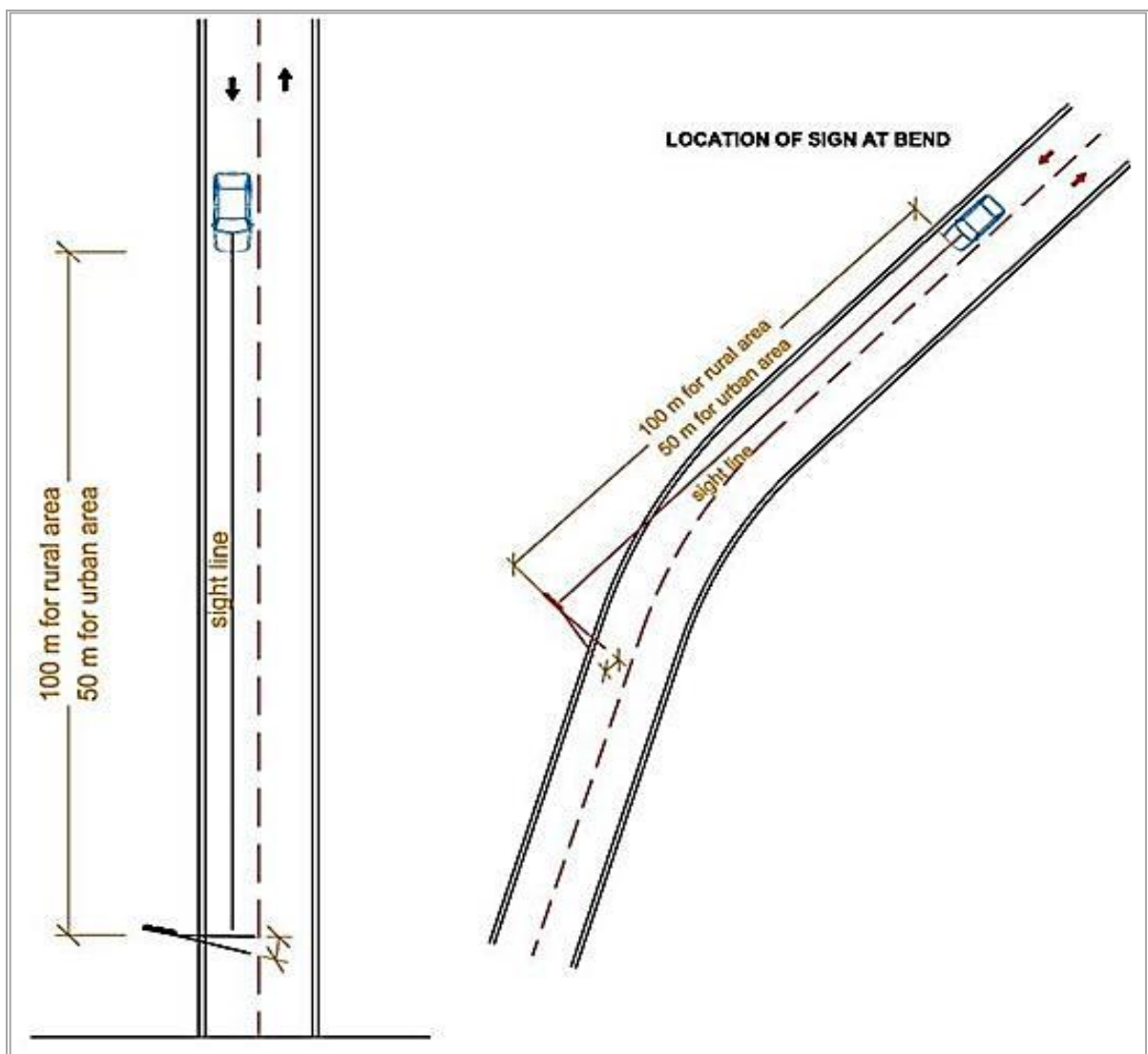
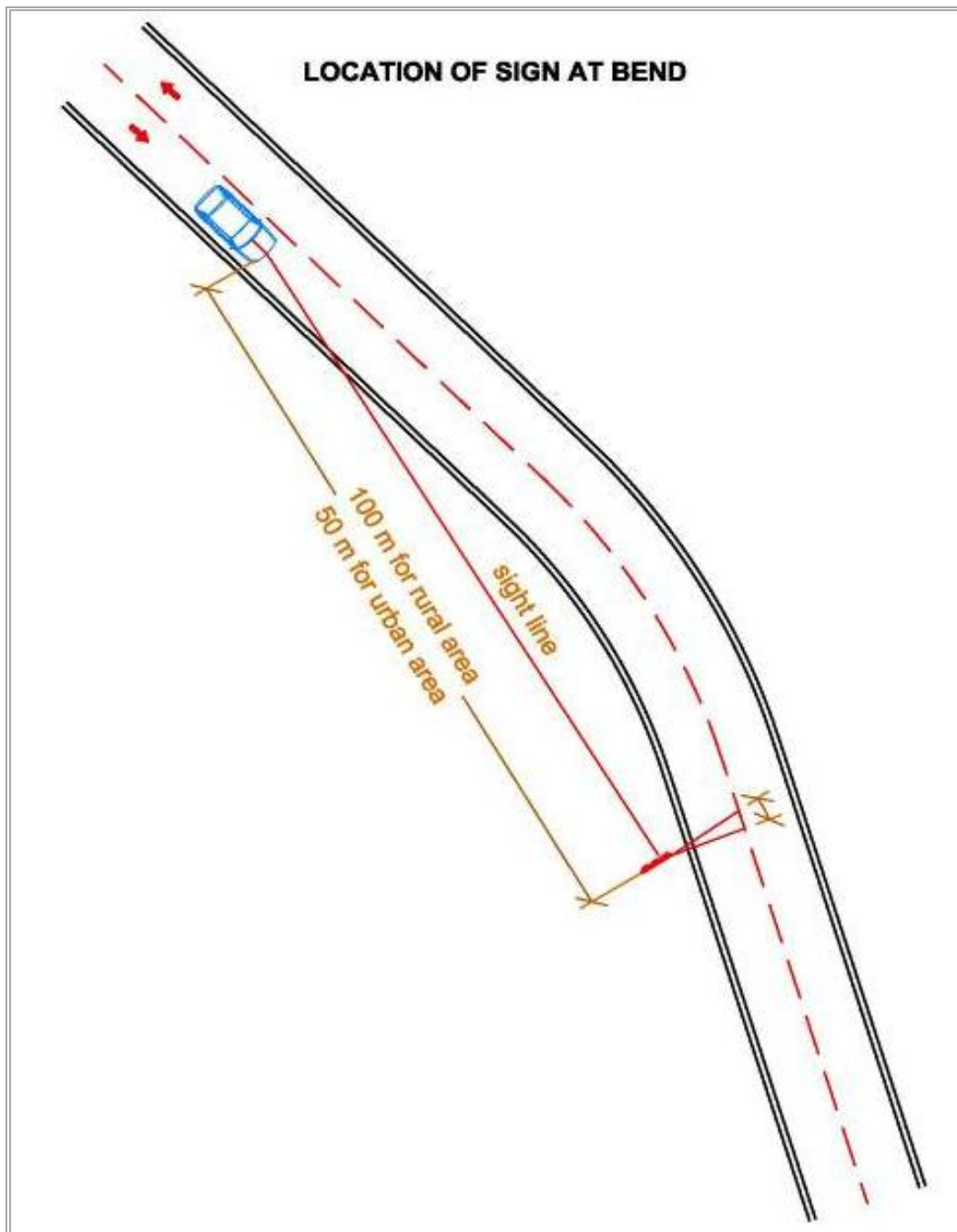


Figure 11-2/5: Orientation Angle of Traffic Sign



Cont'd Figure 11-2/5: Orientation Angle of Traffic Sign

11-2/2/6 POST AND MOUNTING

Signposts and sign mountings should be so constructed as to hold signs in their proper and permanent positions and to resist swaying in the wind. Especially in rural areas, if roadside sign supports cannot be sufficiently distant from the pavement edge, sign supports should be of a suitable anti-collapsible design. In some cases, signs can be placed on existing supports used for other purposes, such as street lights and public utility poles, (in urban areas), thereby minimizing footpath obstruction.

11-2/3 REGULARITY SIGNS

Regulatory signs shall be used to inform road users of selected traffic laws or regulations and indicate the applicability of the legal requirements.

Regulatory signs shall be installed at or near where the regulations apply. The signs shall clearly indicate the requirements imposed by the regulations and shall be designed and installed to provide adequate visibility and legibility in order to obtain compliance.

In order to assist the understanding of the different functions of different types of regulatory sign, the regularity signs are classified into the following groups

1. Control Signs
2. Mandatory Signs
3. Prohibitory Signs
4. Parking Control Signs

11-2/3/1 CONTROL SIGNS

The signs in this group have a common function in that they are used to allocate priority right-of way or direction of movement.

- **STOP sign (R1)**

STOP signs are intended for use on roadways where traffic is required to stop. The STOP sign should be an octagon with a white message and border on a red background.

A stop sign may be necessary at an intersection where one or more of the following conditions exist:

- i. At the entrance to a street intersection a through highway or through street.
- ii. Unsignalized intersection in a signalized area.
- iii. At a minor intersection where, due to restricted view, the safe vehicle approach speed is less than 10 km/h.
- iv. At locations where accident experience indicates the need for STOP sign control.

STOP signs shall not be erected at intersections where traffic control signals are present and continuously in operation



(R1)

- **YIELD sign (R2)**

The GIVE WAY sign indicates right of way to traffic on certain approaches to an intersection. Vehicles controlled by a GIVE WAY sign need to stop only when necessary to avoid collision with other traffic that has right of way.

The GIVE WAY sign may be necessary:

- i. On a minor road at the entrance to an intersection where it is necessary to assign right of way to the major road but where stopping is not necessary at all times and where the safe approach speed on the minor road exceeds 15km/h.
- ii. On the entrance ramp to an expressway.
- iii. Within an intersection with a divided highway where a STOP sign is present at the entrance of the first road and further control is necessary at the entrance to the second roadway and where the median width between the two road-ways exceeds 10m.
- iv. Where there is a separate or channelized right-turn lane.



(R2)

- **NO ENTRY SIGN (R3)**

NO ENTRY sign indicates to drivers of vehicles that entry is prohibited to all vehicular traffic. NO ENTRY sign should be used to prohibit “wrong way” entry to a roadway when confusion may exist as to the direction of travel of traffic in the roadway. NO ENTRY sign should be located on the right and left sides of a one-way roadway. NO ENTRY sign R3 should be oriented at 90 degrees to the direction of “wrong way” travel.



(R3)

- **ONE WAY SIGNS (R4, R5 and R6)**



(R4)



(R5)



(R6)

11-2/3/2 MANDATORY SIGNS

The signs in this group have the function that they are used to indicate to road users actions that they must take or that are mandatory.

- **AHEAD ONLY SIGN (M1)**

Sign M1 requires that the driver of a vehicle should proceed only straight ahead in the direction indicated by the arrow on the sign. AHEAD ONLY sign M1 should be located on the right side of a two-way roadway and on the left side of a one-way roadway. The function of the AHEAD ONLY sign M1 differs from that of the ONE WAY sign M1 in that, while the ONE WAY sign may indicate the mandatory direction in a street at a junction, other directions of travel at the junction may be chosen. AHEAD ONLY sign M1 indicates that drivers have no other choice but to proceed straight ahead.



(M1)

- **TURN RIGHT (OR LEFT) ONLY SIGN M2 (OR M3)**

Signs M2 (or M3) require that the driver of a vehicle should proceed only to the right (or to the left - the arrow direction being reversed) at the junction. TURN RIGHT (or LEFT) ONLY signs M2 (or M3) should be located on the far side of a roadway facing drivers to which they apply.



(M2)



(M3)

- **TURN RIGHT (OR LEFT) AHEAD ONLY SIGN M4 (OR M5)**

Signs M4 (or M5) require that the driver of a vehicle should proceed only to the right (or to the left — the arrow direction being reversed) at the junction ahead. TURN RIGHT (or LEFT) AHEAD ONLY signs M4 (or M5) should be located on the right side of a two-way roadway and on the left side of a one-way roadway at a distance of approximately 50 meters from the junction to which it applies.



(M4)



(M5)

- **PASS EITHER SIDE SIGN (M6)**

Sign M6 indicates that the driver of a vehicle may pass to either side of an obstruction in the roadway, such as a traffic island. PASS EITHER SIDE sign M6 should be located on a traffic divider, near the nose of the divider, so that there is a minimum clearance of 300 millimeters between the edges of the divider and the sign.



(M6)

- **KEEP RIGHT (OR LEFT) SIGN M7 (OR M8)**

Signs M7 (or M8) indicate that the driver of a vehicle should pass to the right (or to the left, the arrow direction being reversed) of an obstruction in the roadway. Use of signs M7 (or M8) most commonly applies to traffic islands or refuges in two-way roadways or at the beginning of median island when a single two-way carriageway widens to become a dual carriageway.



(M7)



(M8)

- **ROUNDAABOUT SIGN (M9)**

Sign M9 requires that the driver of a vehicle should proceed only in a counterclockwise direction at the roundabout ahead.



(M9)

- **U-TURN SIGN (M10)**

Sign M10 notifies the drivers that they can make a U-turn.



(M10)

- **PEDESTRIAN CROSSING SIGN (M11)**

Sign M11 notifies the pedestrian for the road crossing zone.



(M11)

11-2/3/3 PROHIBITORY SIGNS

The signs in this group have the function to indicate to road users actions that they must not take, or which are prohibited. Prohibitions may apply in the form of limits, or to certain actions or objects. Maximum limits such as speed or height limits are indicated in circular signs without a diagonal slash. Prohibitions on actions or objects are indicated in circular signs which include a diagonal slash.



P1
Overtaking prohibited



P2
Overtaking by goods
vehicles prohibited



P3
Maximum speed limited to the
figure indicated



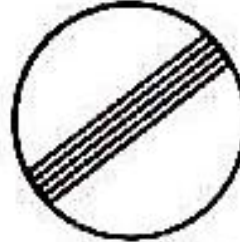
P4
Use of audible warning
devices prohibited



P5
Photography prohibited



P6
Passing without stopping
prohibited (e.g. for customs)



P7
End all local prohibitions
imposed on moving vehicles



P8
End of speed limit



P9
End of prohibition of
overtaking



P10

No entry for vehicles having an overall height exceeding the figure indicated in meters



P11

No entry for vehicles exceeding the figure indicated in tons laden weight



P12

No entry for vehicles having a weight exceeding the figure indicated in tons on one axle



P13

No entry for vehicles or combinations of vehicles exceeding the figure indicated in meters in length



P14

Driving vehicles less than exceeding the figure indicated in meters a part prohibited



P15

No left turn



P16

No right turn



P17

No U-turn



P18
Closed to all vehicles in
in both directions



P19
No entry for all motor
vehicles except motor-cycles
without sidecar



P20
No entry
for motor-cycles



P21
No entry
for cycles



P22
No entry
for goods vehicles



P23
No entry for any vehicles
drawing a trailer other than
a semi-trailer or single axle trailer



P24
No entry
for pedestrians



P25
No entry for animal-drawn
vehicles



P26
No entry for handcarts



P27
No entry for
power-driven
agricultural vehicles



P28
No entry for
vehicles carrying more than a certain
quantity of explosives or readily
flammable substances



P29
No entry for vehicles carrying more
than a certain quantity of
substances liable to cause water pollution



P30
No entry
for power-driven
vehicles



P31
No entry
for power-driven vehicles
or animal-drawn vehicles



P32
No entry for vehicles having
an overall width exceeding
the figure indicated
in meters

11-2/3/4 PARKING CONTROL SIGNS

The signs in this group apply to the control and regulation of stopping and parking. The signs in the group are available to clarify stopping or parking controls in complex environments or to indicate circumstances where limited or part-time restrictions operate for part(s) of a day or apply to specific classes of vehicle.



PC1
Parking prohibited



PC2
Standing and parking prohibited



PC3



PC4



PC5

No Stopping within time limits displayed



PC6



PC7

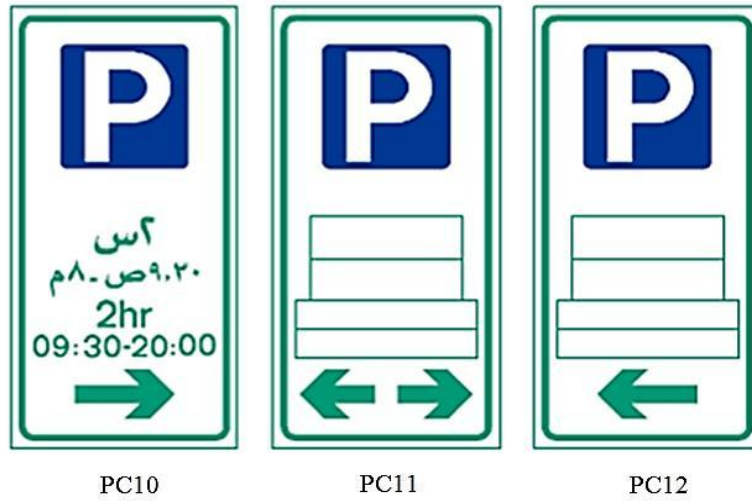


PC8

No parking sign variants



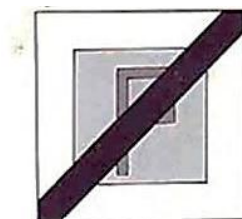
PC9
Parking



Parking is allowed within indicated limits



Parking is only allowed for handicapped



Exit from a limited duration
Parking zone



Exit from a limited duration
Parking zone

11-2/4 WARNING SIGNS

Warning signs are used to make drivers aware of hazardous or potentially hazardous conditions in the roadway which drivers might not otherwise expect to see due to the nature of the hazard, the hazard being hidden or partially hidden, or due to the driver's workload at the time. Advance warning signs have a unique triangular shape and the warning message is given in the majority of instances by a symbolic representation of the hazard or potential hazard.

Most warning signs are located in advance of the hazard to which they refer. To be effective as a class, warning signs must be positioned consistently. The location of warning signs should, in part, be related to the speed with which the hazard can be negotiated. This in turn relates to the difference between the speed at the hazard and the speed when viewing the sign. Table (11-2/3) gives guidelines for the positioning of advance warning signs.

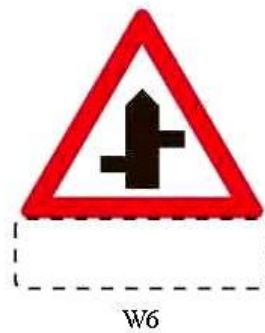
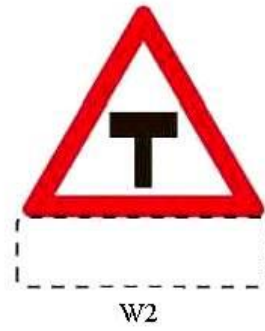
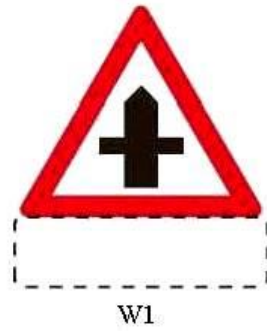
Table 11-2/3: Guidelines for positioning advance warning signs [3, p.4-1]

Approach Speed (km/hr.)	Posted Speed at Hazard (km/hr.)						
	≤20	30	40	50	60	70	130
	Preferred Distance of Sign from Hazard (meters)						
30	Note 1	Note 2	-	-	-	-	-
40	25	Note 1	Note 2	-	-	-	-
50	75	60	Note 1	Note 2	-	-	-
60	125	110	100	50	Note 2	-	-
70	150	140	125	110	50	Note 2	-
80	175	160	150	125	100	60	Note 2
90	200	185	175	150	140	75	65
100	250	230	200	180	175	125	100
110	275	250	225	210	190	160	130
120	300	275	250	230	210	190	175
NOTES: 1. No suggested minimum distances are provided for these speeds, as placement location is dependent on site conditions and other signing to provide an adequate advance warning for the driver. 2. No specific recommended placement distances are provided for these speeds. The Engineer should exercise professional engineering judgement in light of local conditions.							

11-2/4/1 ADVANCED WARNING SIGNS

The signs in this group should all be used in advance of the hazard or potential hazard to which they relate. Consistent with this function they have a unique triangular shape in order to attract the attention of drivers sufficiently early for their message to be effective. Advanced warning signs should be located on the right side of the roadway at a distance from the junction as indicated in table (11-2/3) and with a clear sight distance to the sign. A supplementary plate sign indicating the distance to the junction to the nearest 20 meters can be attached below the advanced warning signs.

- JUNCTION AHEAD SIGNS W1 TO W6



- NO THROUGH ROAD SIGNS W7, W8, AND W9



- MERGING TRAFFIC SIGNS W10 AND W11



- RIGHT (OR LEFT) CURVE SIGN W12 (OR W13)



W12



W13

- BENDS AHEAD SIGNS W14 AND W15



W14



W15

- TWO-WAY TRAFFIC SIGN W16



W16

- LANE ENDS SIGNS W17 AND W18



W17



W18

- U-TURN AHEAD SIGN W19



W19

- ROUNDABOUT AHEAD SIGN W20



W20

- ROAD NARROWS AHEAD SIGNS W21 TO W23



W21



W22



W23

- MAXIMUM HEADROOM SIGN W24

Sign W24 warns drivers of vehicles that the clearance available under an overhead structure ahead is restricted to the amount indicated in meters on the sign. Maximum headroom sign W24 should not be displayed for any structure with minimum clearance of 5.2 meters or greater.



W24

- CHILDREN SIGN W25



W25

- PEDESTRIAN CROSSING AHEAD SIGN W26



W26

- TRAFFIC SIGNALS AHEAD SIGN W27



- QUAYSIDE SIGN W28



- ANIMALS AHEAD SIGN W29



- LOW-FLYING AIRCRAFT SIGN W30



- SPEED HUMP SIGN W31



- TUNNEL SIGN W32



- FALLING ROCKS SIGN W33



- SLIPPERY SURFACE SIGN W34



- GENERAL WARNING SIGN W35



- ROAD WORKS SIGN W36



- **NORROW STRUCTURE SIGN W37 RAILWAY CROSSING SIGNS W38 and W39**



11-2/4/2 HAZARD MARKER SIGNS

The signs in this group have the specific function to mark an actual hazard adjacent to the roadway. The majority of uses therefore relate to identifying the position of physical hazards such as culverts, bridge structures, large sign supports, traffic island gores, guardrails, etc., to drivers. In some instances, such as a sharp curve, hazard markers may be used to delineate the curve and so draw attention to the severity of the curve.

- **HAZARD PLATE SIGNS W40 AND W41**

Signs W40 and W41 warn drivers of the actual position of physical objects which are placed so close to the roadway as to represent a hazard or potential hazard if vehicles should collide with them.

See table (11-2/4) for guidance on spacing. Hazard plate signs W40 and W41 should always be installed so that the “arrow” points towards the roadway and away from the hazard.

Table 11-2/4: Spacing on curves [3, p.4-16]

Curve Radius (meters)	Sign Spacing “S” (meters)
60	8-15
150	15-25
300	25
600	25



- **SINGLE CHEVRON RIGHT (OR LEFT) SIGN W42 (OR W43)**

Signs W42 and W43 warn drivers of the actual position of physical objects or of the actual alignment of the roadway when these conditions represent hazards or potential hazards. Figure (11-2/6) illustrates a typical example of such an installation and table (11-2/4) gives guidance on the spacing of the signs.

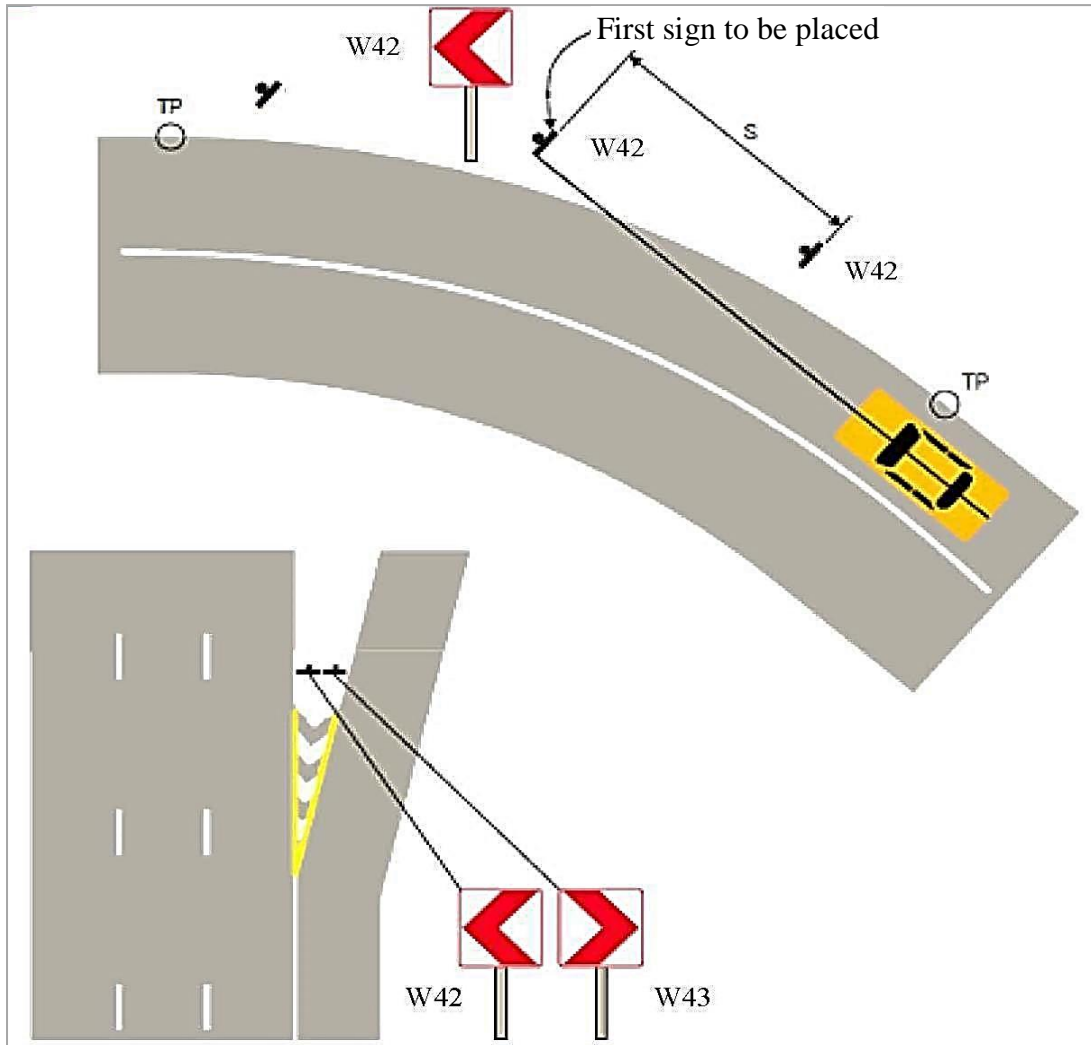
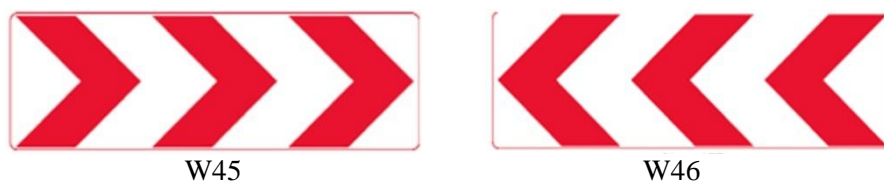


Figure 11-2/6: Example of Application Single Chevron Sign [3, p.4-18]

- **MULTIPLE CHEVRON RIGHT (OR LEFT) SIGN W45 (OR W46)**

Signs W45 and W46 warn drivers of the actual position of a very sharp bend or change in direction in the roadway. Multiple chevron right (or left) signs W45 and W46 may be used at a sharp bend when the severity of the bend is not likely to be adequately conveyed by advance warning sign W12 or W13. Typical example for application of multiple chevron signs W45 and W46 is shown in figure (11-2/7).



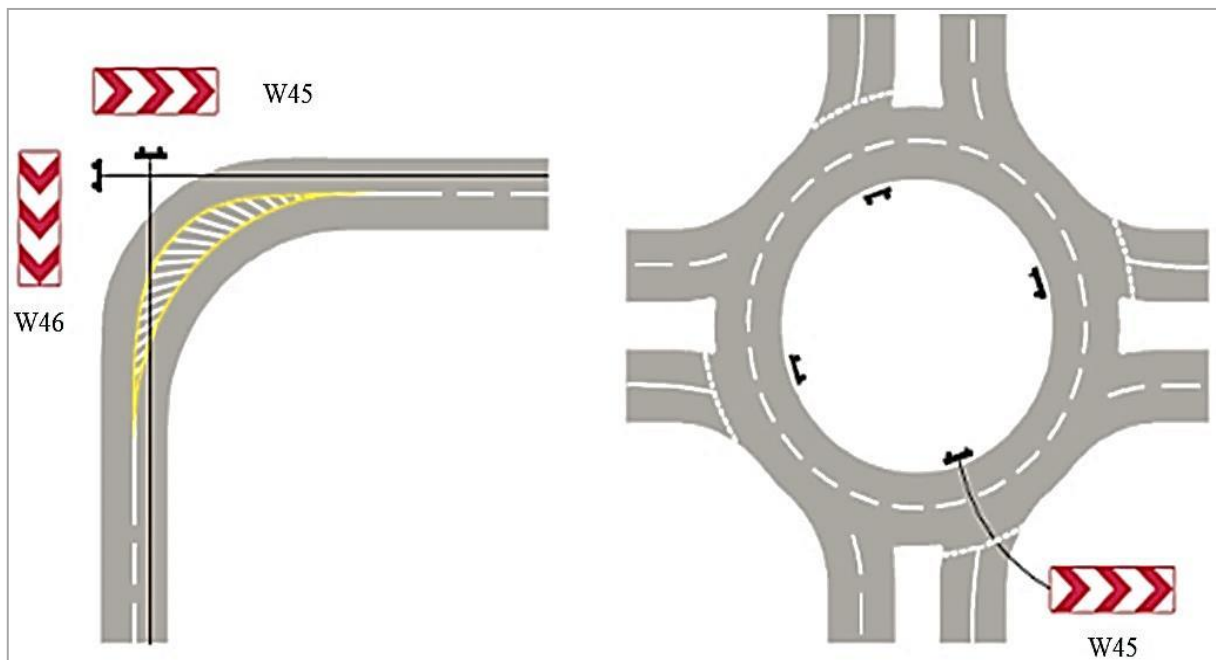
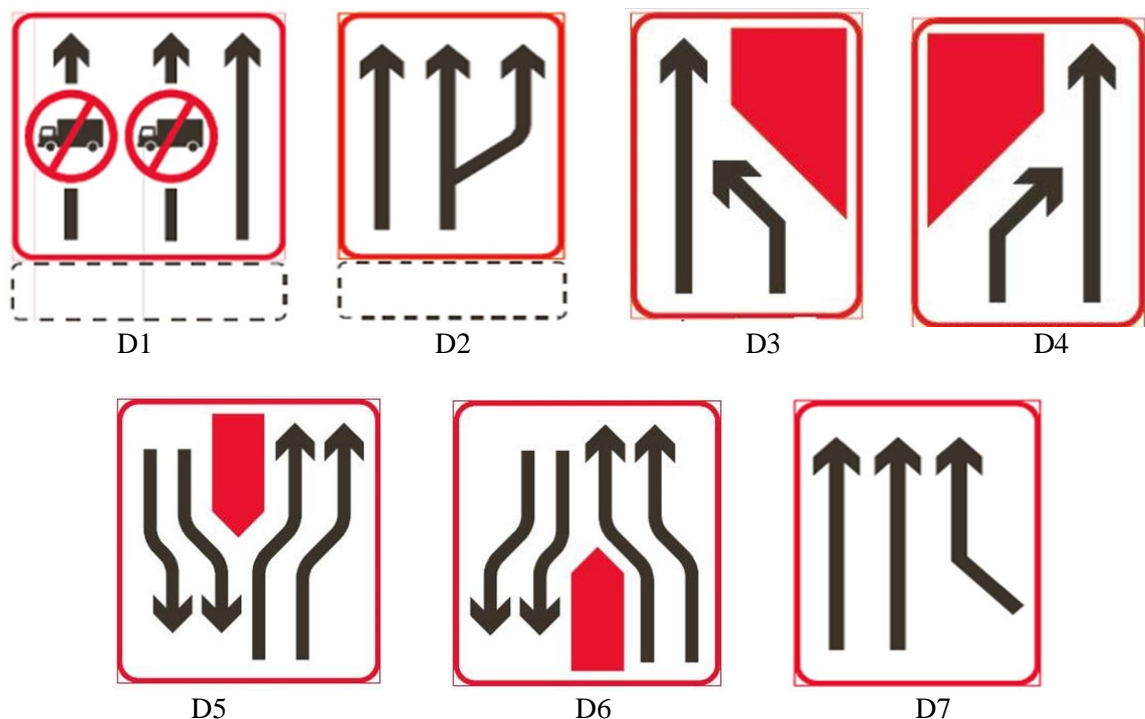


Figure 11-2/7: Typical Application of Multiple Chevron Signs [3, p.4-19]

11-2/4/3 DIAGRAMMATIC SIGNS

The signs in this group are essentially for use in situations where the size and shape of triangular advance warning signs limits the pictorial warning message that can be displayed, and/or the overall conspicuity of the sign. The situations that tend to be indicated on diagrammatic signs are commonly ones that occur on higher speed roads. The diagrammatic nature of the message given by these signs means that they often depict the geometric arrangements of the lanes and/or the whole roadway.



11-2/4/4 HIGH VEHICLE WARNING SIGNS

Sign D8 is typically located in advance of gantry or cantilever signs and points high vehicles to a short, low-grade detour that does not pass underneath the horizontal arm of the sign but instead around the sign's support column. High vehicle warning sign D8 should be located on the right side of the roadway at a distance in advance of the gantry or cantilever structure as indicated in table (11-2/3). If deemed appropriate by the engineer, sign D8 may have flashing yellow caution lights placed at its top.



D8

11-2/5 GUIDE SIGNS

Guide signs are essential to direct road users along streets and highways, to inform them of intersecting routes, to direct them to cities, towns, villages, or other important destinations, to identify nearby rivers and streams, parks, forests, and historical sites, and generally to give such information as will help them along their way in the most simple, direct manner possible.

Guide signing can be divided into the following categories

1. Destination Signs
2. Distance Signs
3. Information Signs

11-2/5/1 DESTINATION SIGNS



D9



D10



D11



D12



D13



D14



D15



D16



D17

11-2/5/2 DISTANCE SIGNS



D18



D19

11-2/5/3 INFORMATION SIGNS

Information signs may be used for the purpose of directing traffic to the following facilities:

- Off-road services such as fuel, restaurants, parking areas, lodgings, tent and trailer camp sites or motels.
- Points of interest such as historical, geological or geographical sites.
- Emergency services such as hospitals and telephones.

Warrants for Information Signs:

- The distance from the highway to the facility should not exceed 5km.
- Adequate standard of service should be provided at the facility. For example, the service station should be open 24 hours every day.
- A point of interest should be one that has been recognized by an appropriate body such as the Ministry of Culture, Ministry of Tourism, etc.



I1

First-aid station



I2

Breakdown Service



I3

Refreshments or Cafeteria



I4

Picnic site



I5
Telephone



I6
Filling station



I7
Starting point for walks



I8
Camping site



I9
Hotel or motel



I10
Restaurant



I11
Caravan site



I12
Camping and caravan site



I13
Youth hostel



I14
Hospital



I15
Hospital



I16
Tramway stop

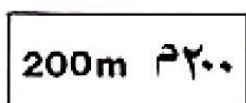


I17
End of a build-up area

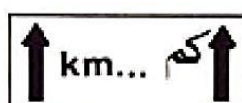


I18
Beginning of a build-up area

Additional Panels



Distance from sign to the beginning of the dangerous section of road or of the zone to which the regulation applies



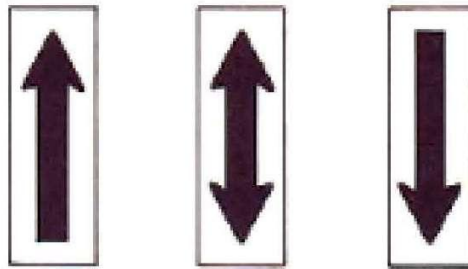
Length of the dangerous section of road or zone to which the regulation applies



Distance covered by prohibition or restrictions



Distance covered by prohibitions or restrictions



Prohibitions or restrictions
Begin Continue End

11-2/5/4 GENERAL STANDARDS FOR GUIDE SIGNS

Guide Signs (destination and distance) for streets and highways should have a green background with a white message or symbol in rural area. Expressway way should have a blue background with a white symbol or letters. For urban area , the guide sign should have a white background with black symbol or letters .For recreational area the sign should have a brown background with white symbol or letters. The shape of the sign will be rectangular but the size varies (depending on height of letters, number of words and number of lines). If there is more than one destination to be shown in guide sign, the following order should be maintained

1. Straight ahead destination
2. Left turn destination
3. Right turn destination

Information sign should have a blue background with a white inset and a black message or symbol. The standard size should be 75 cm (width) by 100 cm (height). It should be placed 2 km to 3 km in advance of the exit junction to the facility with another one about 500 m before the exit junction to the facility. The second sign should be supplemented by a distance plate and an arrow directed to the location of the facility.

11-2/5/4/1 LANGUAGE AND LETTER STYLE

All signs should be in both Arabic and English. For the Arabic message, simplified Arabic font type should be used, whereas for English message, times new roman font type should be used.

11-2/5/4/2 SIZE OF LETTERING

There are three time factors which have to be taken into account when considering the siting of signs:

- Reading time (t_g) of the message or symbol.
- Reaction period (t_r) required before acting on the information presented.
- In most cases an appropriate action (t_a) is called for which requires further time e.g. decelerating to a stop line, making a turn etc. All times are measured in seconds.

The minimum time for message perception will vary according to the individual's vision characteristics, the familiarity of the information and the number of words placed on one sign. The glance reading time (t_g) can be taken as a minimum of 1sec. for signs with one or two words and $N/3$ sec. for signs with N unfamiliar words. A suitable reaction time (t_r) of between 1 and 1.5 sec is allowable in most cases and 1.2 m/sec.^2 as a comfortable rate of deceleration taken from the average running speed of a highway.

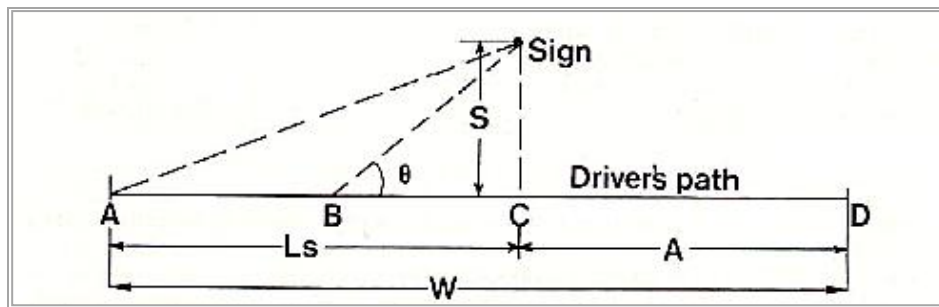


Figure 11-2/8: Siting of Signs [4, p.IV-41]

The distance $A + S \cot \theta$ is equivalent to the minimum safe stopping distance. Selecting suitable values for the parameters enables the siting distance to be solved as shown in figure (11-2/8):

$$W = V_1 (2t_g + t_r) + t_a \left(\frac{V_1 + V_2}{2} \right) \quad (11 - 2/1)$$

Where V_1 and V_2 are the initial and final velocities (m/sec.). Uniform deceleration is assumed.

$$L_s = AB + BC = V_1 (2t_g) + S \cot \theta \quad (11 - 2/2)$$

The larger value of L_s or $W - A$ is used for the determination of letter height. For a displacement of $\theta = 10^\circ$, the required letter size is:

$$H = \frac{2t_g V + 5.7 S}{L} \text{ cm} \quad (11 - 2/3)$$

- H = Required letter height (cm).
- V = Approach Speed (m/sec.).
- L = Legibility in meters per cm of letter height which is taken as x-height for lower-case letters.
- For upper-case letters, multiply H by 1.33.
- An average value of L can be taken as 4m per 1cm of letter height.
- A factor of safety has been introduced by allowing two glances.

Legibility can be defined simply as the ability to read a text message or accurately determine the form of a symbolic message. In this respect, for the long-range legibility required by drivers, the most critical element must be the smallest or thinnest element. The legibility distance depends on the following:

- Size of the letter-height and width.
- Stroke width.
- Letter and word spacing both horizontally and vertically.
- Size of margins.

The relationship between these parameters is shown in table (11-2/5).

Table 11-2/5: Parameter Ratio for Legibility Distance for Latin Characters [4, p.IV-42]

PARAMETER RATIO	RANGE
Height to width ratio	2:1 to 1:1
Height to stroke width ratio	9:1 to 5:1
Space between letters to stroke width	2:1 to 1:1
Word spacing to letter height ratio	1 1/4:1 to 1:2
Spacing between lines to letter height ratio	1:1 to 1:2
Margin and letter height ratio	1:1 to 1:2

The size of a direction sign will depend on the characteristics of the lettering chosen from Table (11-2/5).

All signs should be in both Arabic and English. Since the siting distance and size of direction signs depend on letter height, the larger of either the Arabic or English letters should be used. Table (11-2/6) gives the suggested letter height for direction signs for different categories of road.

Table 11-2/6: Recommended Letter Heights [4, p.IV-42]

Type of Road	X-height of letters (advance direction signs) cm	X-height of letters (direction and route confirmatory signals) cm
Dual carriageway roads (rural)	2.5	20
Single carriageway primary roads (rural)	20	15
Secondary rural roads and wide urban roads	15	10
Other urban roads and minor rural roads	10	10

Although the siting distance of signs will be governed by the factors mentioned above, a general guide line is given in the following table

Table 11-2/7: Suggested Distances for Siting Direction Signs (m) [4, p.IV-43]

Type of sign	Approach speed of vehicle on the road (km/hr.)			
	≤60	60-80	80-120	>120
Guide sign	30-50 m	50-80 m	80-120 m	120-150 m
Advanced guide sign	250 m	250-300 m	300-400 m	400-500 m

11-2/5/4/3 SIGN BORDER

All guide signs should be provided with a contrasting border around the perimeter of the sign. The border color should be the same as the color used for place name lettering on the sign. Sign borders should have the following dimensional characteristics where “x” is the height of the letter used for the sign:

- Width = 0.25x.
- Corner radius = 1.0x (to the outer edge of the border).

An internal dividing border should be used to separate the stacks of a stack type direction sign. This border should have the same width as the outside sign border but should not be provided with radii where it joins the outside border. Where practicable, the corners of the sign panels should also be rounded to fit the border.

11-2/5/4/4 ARROWS

Arrows are used for lane assignment and to indicate the direction toward designated routes or destinations. Figure (11-2/9) shows the up-arrow and the down-arrow designs that have been approved for use on guide signs.

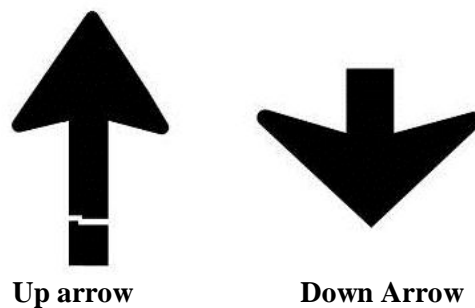


Figure 11-2/9: Arrows for Use on Guide Signs [1, p.141]

On overhead signs where it is desirable to indicate a lane to be followed, a down arrow shall point downward toward the center of that lane. Down arrows shall be used only on overhead guide signs that restrict the use of specific lanes to traffic bound for the destination(s) and/or route(s) indicated by these arrows. Down arrows shall not be used unless an arrow can be pointed to each lane that can be used to reach the destination shown on the sign.

Where a roadway is leaving the through lanes, an up arrow shall point upward at an angle representative of the alignment of the exit roadway, and should be placed at the side of the sign which will reinforce the movement of exiting traffic.

The width across the arrow head should be at least equal to the height of the largest letter on the sign. For short downward pointing arrows on overhead signs, they should be 1.75 times the letter height.

11-3 ROAD MARKING

11-3/1 COLOR, CODE AND MATERIAL

Markings on highways have important functions in providing guidance and information for the road user. Major marking types include pavement and curb markings, object markers, delineators, colored pavements, barricades, channelizing devices and islands. In some cases, markings are used to supplement other traffic control devices such as signs, signals and other markings. In other instances, markings are used alone to effectively convey regulations, guidance, or warnings in ways not obtainable by the use of other devices.

The general functions of longitudinal lines are:

- A. A double line indicates maximum or special restrictions,
- B. A solid line discourages or prohibits crossing (depending on the specific application),
- C. A broken line indicates a permissive condition, and
- D. A dotted line provides guidance.

Markings shall be yellow, white, red, or blue. The colors for markings shall conform to the standard highway colors. Black in conjunction with one of the above colors shall be a usable color.

When used, white markings for longitudinal lines shall delineate:

- A. The separation of traffic flows in the same direction.
- B. The right edge of the roadway.

When used, yellow markings for longitudinal lines shall delineate:

- A. The separation of traffic traveling in opposite directions.
- B. The left edge of the roadways of divided and one-way highways and ramps.
- C. The separation of two-way left turn lanes from other lanes.

When used, red raised pavement markers shall delineate roadways that shall not be entered or used.

When used, blue markings shall supplement white markings for parking spaces for persons with disabilities. When used, blue raised pavement markers shall indicate locations of fire hydrants along a roadway.

Road marking should be of non-skid material and should not protrude more than 6 mm above the level of pavement. Studs or similar device used for marking should not protrude more than 1.5 cm above the level of pavement (or 2.5 cm in the case of studs incorporating reflectors). Road markings are applied using thermoplastic or cold paint. Thermoplastic shall be used for the arterial highways. Paint can be applied for local and collector roads. For both cases, reflectorisation should be used.

Reflectorisation is achieved by adding glass beads to the markings. These reflect light from vehicle headlamps back towards the driver, making the markings much brighter than they otherwise be. Beads are normally premixed into the material.

11-3/2 LONGITUDINAL LINES FOR PAVEMENT MARKING

11-3/2/1 CENTER LINE

Center lines are used to divide carriageways to facilitate two-way traffic use. They comprise narrow broken lines which become continuous solid lines where vehicles are officially prohibited from crossing. In special cases (e.g. change in road width or on a sharp bend), the center line may be located away from the geometric center of the road to follow the vehicle path.

Center line markings are needed where carriageways greater than 6.0m in width carry an average annual traffic volume exceeding the equivalent of 2000 veh/day or, for carriageways of 5.5m width, carrying 1000 veh/day.

The center line should be a continuous solid line for a maximum of 30m before a junction, a marked pedestrian crossing or prohibitory lines. The standard center line markings are shown in figure (11-3/1).

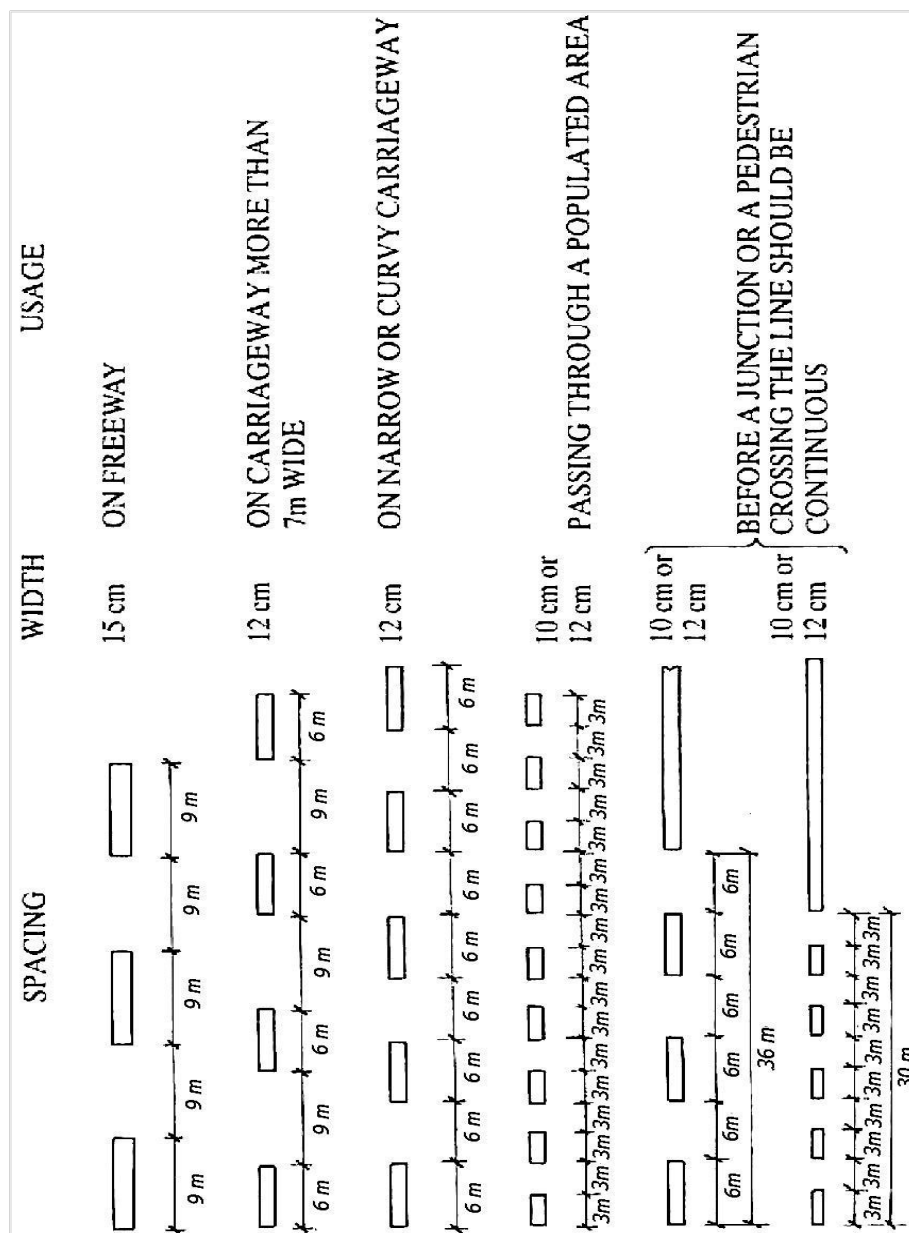
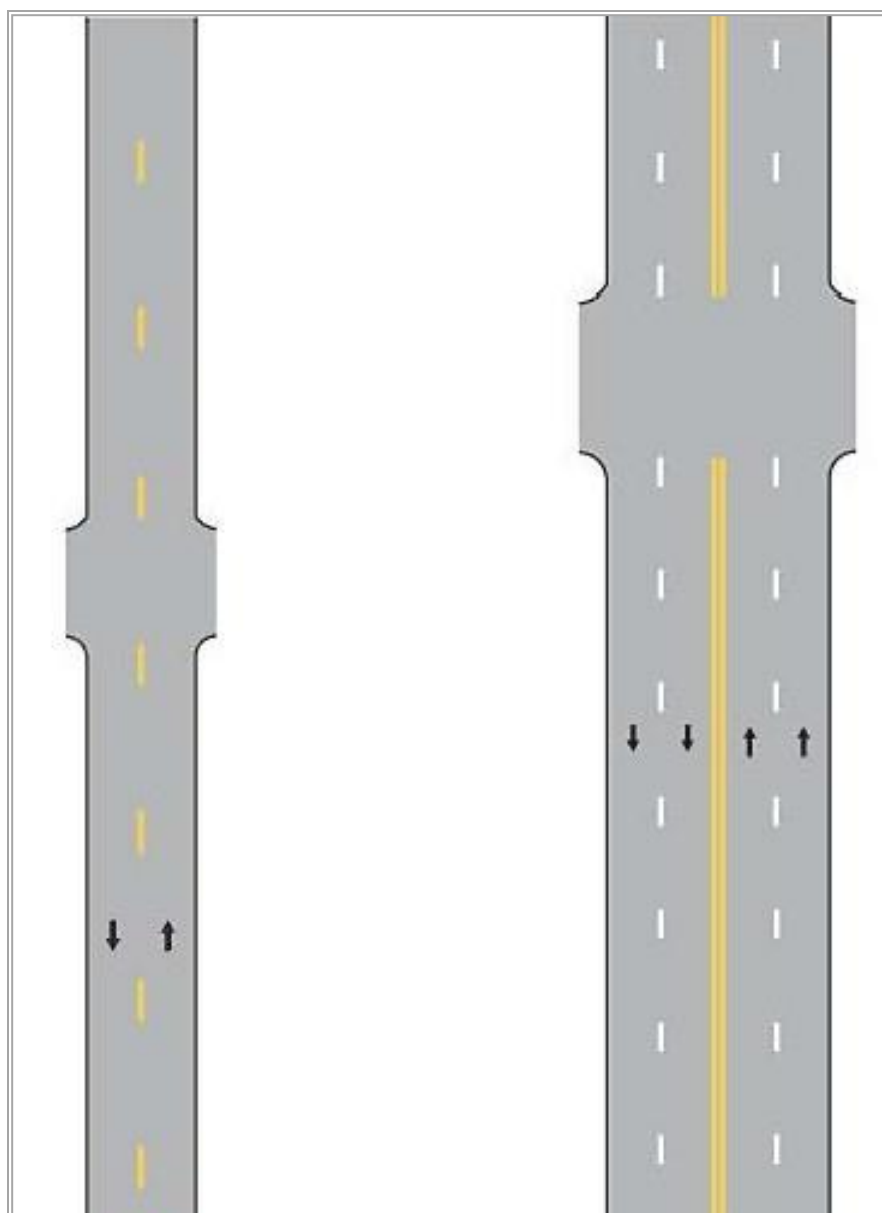


Figure 11-3/1: Center Lines [4, p.V-4]



Typical two-lane, two-way marking
with passing permitted in both direction

Typical multi-lane,
two-way marking

Figure 11-3/2: Examples of Two Lanes and Four Lanes Markings Applications [1, p.351]

11-3/2/2 LANE LINE

Lane lines are used on roads having three or more traffic lanes to regulate the traffic in the same direction and to mark lanes designed for specific traffic streams. The minimum width of each lane is 2.75m. Lane lines should be used irrespective of traffic density. The standard types of lane lines are shown in figure (11-3/3).

- c) Rural arterials and collectors with a traveled way of 6.1 m or more in width and an ADT of 3,000 vehicles per day or greater.
- d) At other paved streets and highways where an engineering study indicates a need for edge line markings.

Edge line markings may be used where edge delineation is desirable to minimize unnecessary driving on paved shoulders or on refuge areas that have lesser structural pavement strength than the adjacent roadway.

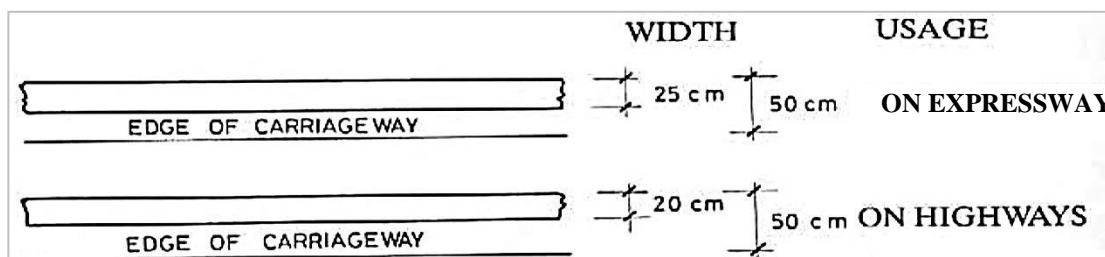


Figure 11-3/4: Edge of Carriageway Lines [4, p.V-6]

11-3/2/4 NO PASSING LINE

No passing line should be a longitudinal solid yellow line, running continuously on or near the centerline of the roadway. Where the distance between successive no-passing zones is less than 120 m, no-passing markings should connect the zones.

On roadways with centerline markings, no-passing zone markings shall be used at horizontal or vertical curves where the passing sight distance is less than the minimum necessary for reasonably safe passing at the 85th-percentile speed or the posted or statutory speed limit as shown in table (11-3/1).

Table 11-3/1: Minimum Passing Sight Distance [1, p.352]

85 Percentile or Statutory Speed Limit (km/hr.)	Minimum Passing Sight Distance (meters)
40	140
50	160
60	180
70	210
80	245
90	280
100	320
110	355
120	395

Prohibitory line visibility may be improved by installing reflective studs at 4m intervals along the continuous line. Standard prohibitory lines are shown in figure (11-3/5).

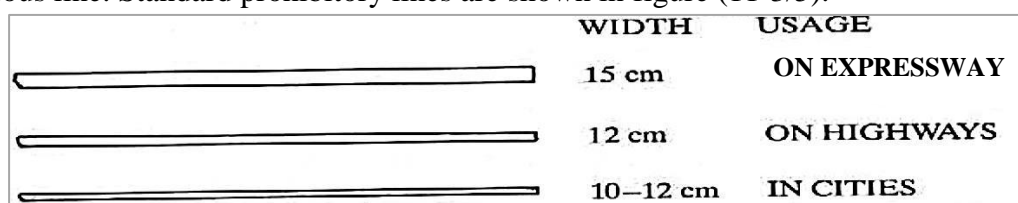
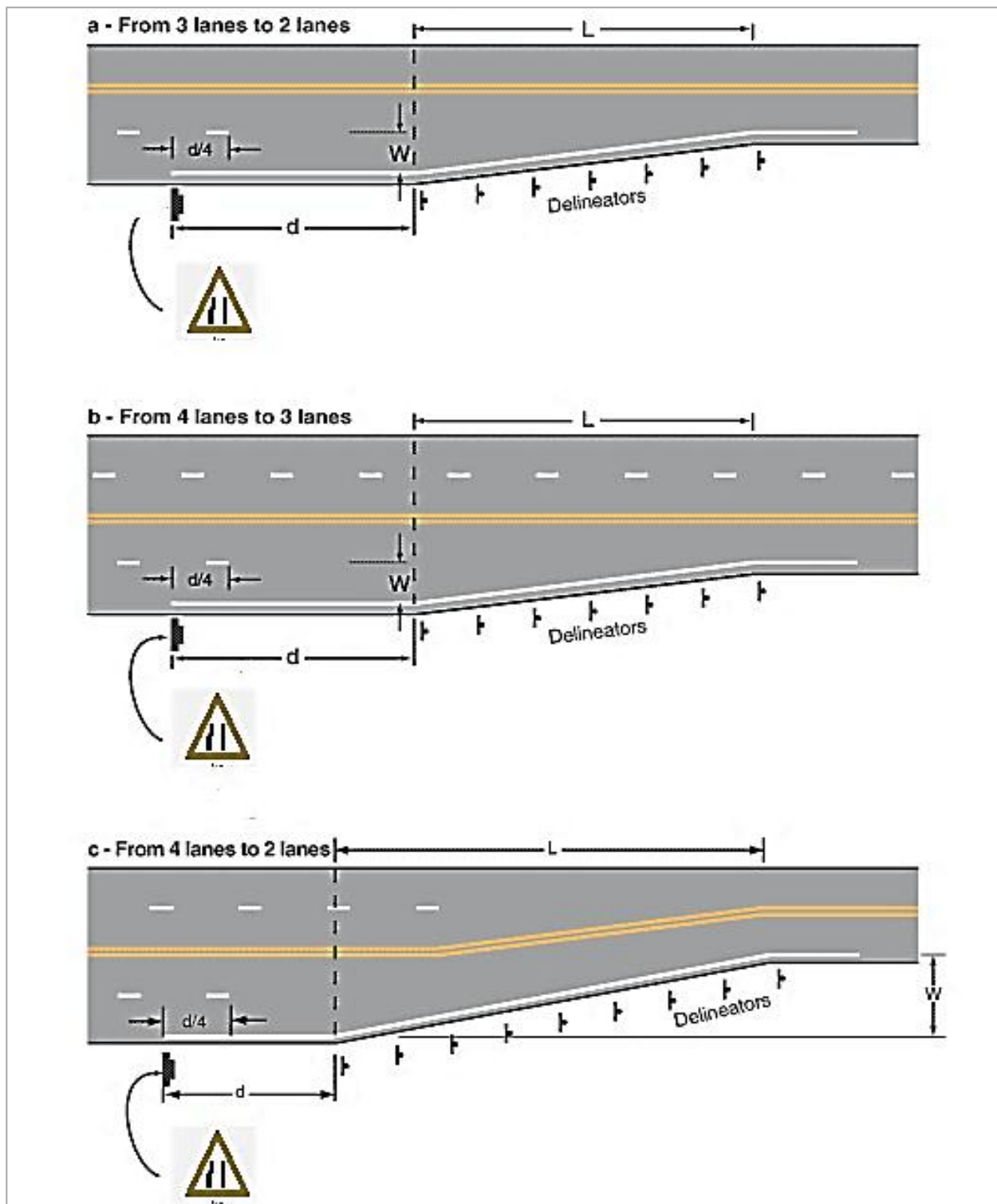


Figure 11-3/5: Prohibitory Lines/No Overtaking Lines [4, p.V-6]

No-passing zone markings shall also be used to prohibit passing through transition areas where the number of through lanes is reduced as shown in figure (11-3/6).



L= Length in meters
S= Posted, 85th -percentile, or statutory speed in km/hr.
W= Offset in meters
d= Advance warning distance (see paragraph 11-2/4/1)

For speeds 70km/hr. or more
 $L=0.62WS$
For speeds less than 70 km.hr.
 $L= WS^2/155$

Figure 11-3/6: Examples of Line Reduction Markings [1, p.370]

11-3/2/5 SAFETY LINE

Pavement markings extended into or continued through an intersection or interchange area shall be the same color and at least the same width as the line markings they extend (see figure 11-3/7).

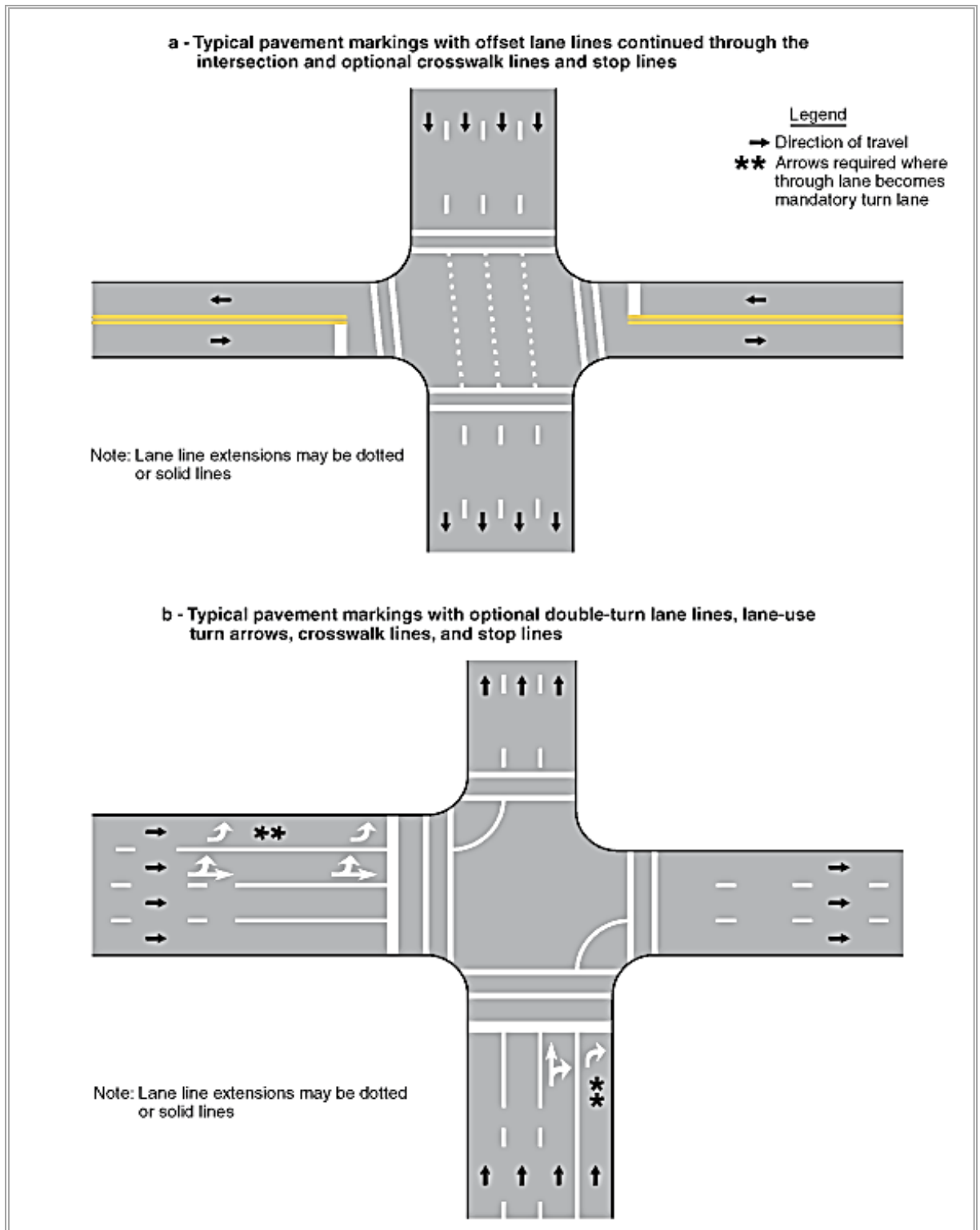


Figure 11-3/7: Examples of Extensions through Intersections [2, p.3B-23]

11-3/3 TRANSVERSE LINES FOR PAVEMENT MARKING

Transverse markings, which include, Stop Line, Pedestrian Crossing Lines, and Give Way Line shall be white unless otherwise specified herein.

11-3/3/1 STOP LINE

Stop line imposes a mandatory requirement that a driver come to a full and complete stop immediately behind that line and is always used in conjunction with a STOP sign or a red traffic signal. STOP LINE marking should be a continuous solid white line transverse and completely across the full width of the traveled portion of the roadway that is controlled by a STOP sign or traffic signal. Stop lines at midblock signalized locations should be placed at least 12 m in advance of the nearest signal indication.

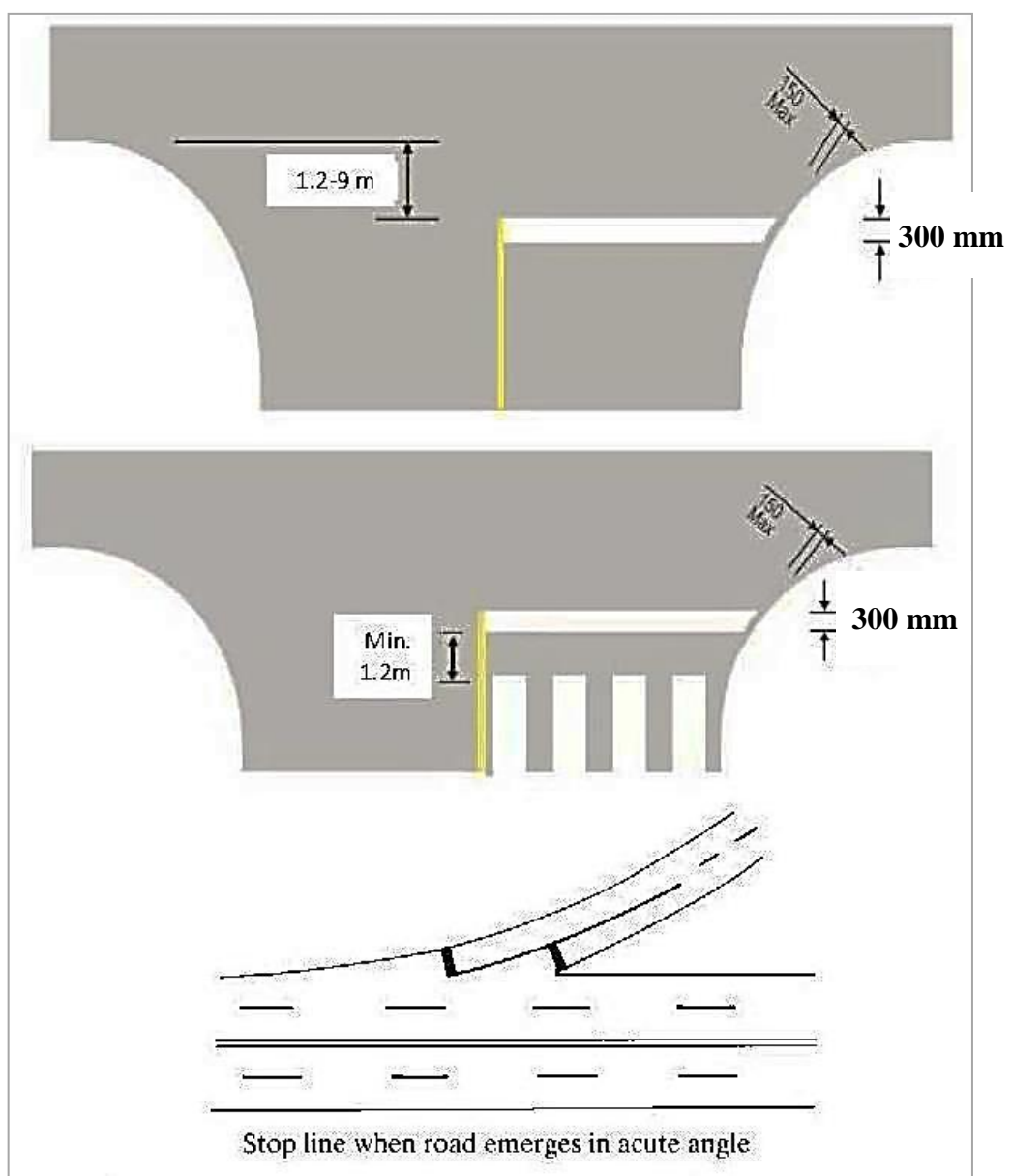


Figure 11-3/8: Stop Line Marking

11-3/3/2 PEDESTRIAN CROSSING LINE

Pedestrian crossings should be marked at all intersections where there is material conflict between vehicular and pedestrian movement. The details of markings are shown in figure (11-3/9).

For safety reasons, where the approach speed of vehicles is more than 60 km/h, pedestrian crossings on such roads should be equipped with traffic signals. At intersections, it is desirable that the pedestrian crossing be located 50cm away from the nearest edge of intersecting roadways. In rural areas and in relatively high-speed urban streets, it is desirable that to give increased visibility to the drivers by making one or two additional lines ahead of and parallel to the standard crosswalk on the approach side at an intersection and on both sides at a midblock crosswalk. These additional lines should have the same width as the crosswalk stripes separated by a space equivalent to one line width. Fencing should be installed along the carriageway where needed to guide the pedestrian traffic.

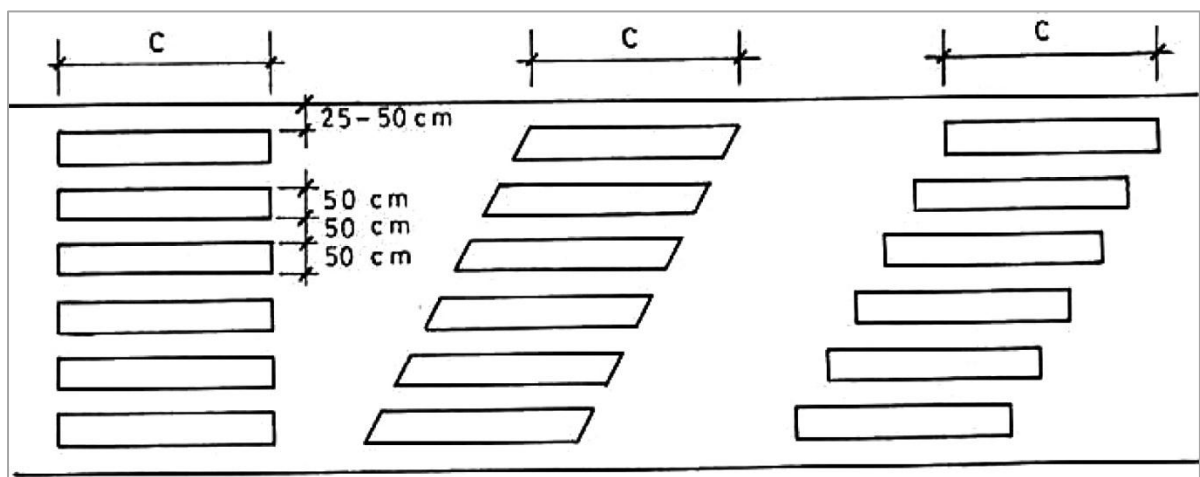


Figure 11-3/9: Marking of Standard Pedestrian Crossing [4, p.V 26]

11-3/3/3 GIVE WAY LINE

Give Way Marking imposes a mandatory requirement that a driver should, when in conflict, stop at the point marked by the line and yield right of way to vehicular and/or pedestrian traffic crossing his intended path. It should always be used in conjunction with GIVE WAY sign. GIVE WAY LINE marking should be a broken white line transverse and completely across the full width of the traveled portion of the approach roadway that is controlled by a GIVE WAY sign. The standard width of a GIVE WAY LINE is 300 millimeters. On high-speed rural roads where higher conspicuousness is desired, its width may be increased to 600 millimeters. The configuration of the GIVE WAY LINE should be a repeated pattern of 600 millimeters of line separated by 300 millimeters of gap.

If used at an unsignalized midblock crosswalk, yield lines should be placed adjacent to the Yield sign located 6.1 to 15 m in advance of the nearest crosswalk line, and parking should be prohibited in the area between the yield line and the crosswalk.

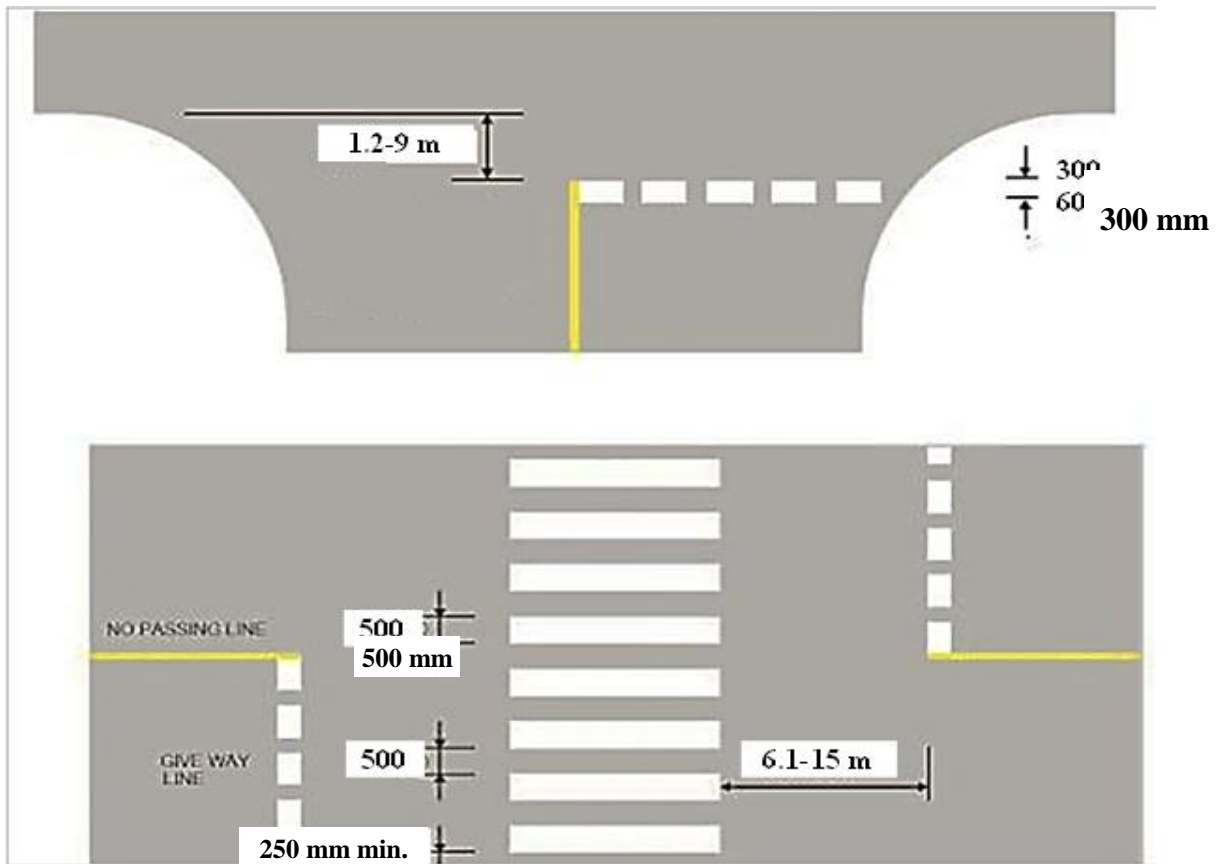


Figure 11-3/10: Giveway Lines Application [3, p.6-5]

11-3/4 OTHER PAVEMENT MARKING

11-3/4/1 NO OVERTAKING AREA

Areas of carriageway marked by solid lines and whose use is prohibited by vehicular traffic are called “no overtaking” areas. In order to clearly mark such islands within the carriageway, they are provided with diagonal lines in such way that they will be transverse to a vehicle crossing the island. No overtaking areas may be used by pedestrians. Standard marking for no overtaking areas is shown in figure (11-3/11).

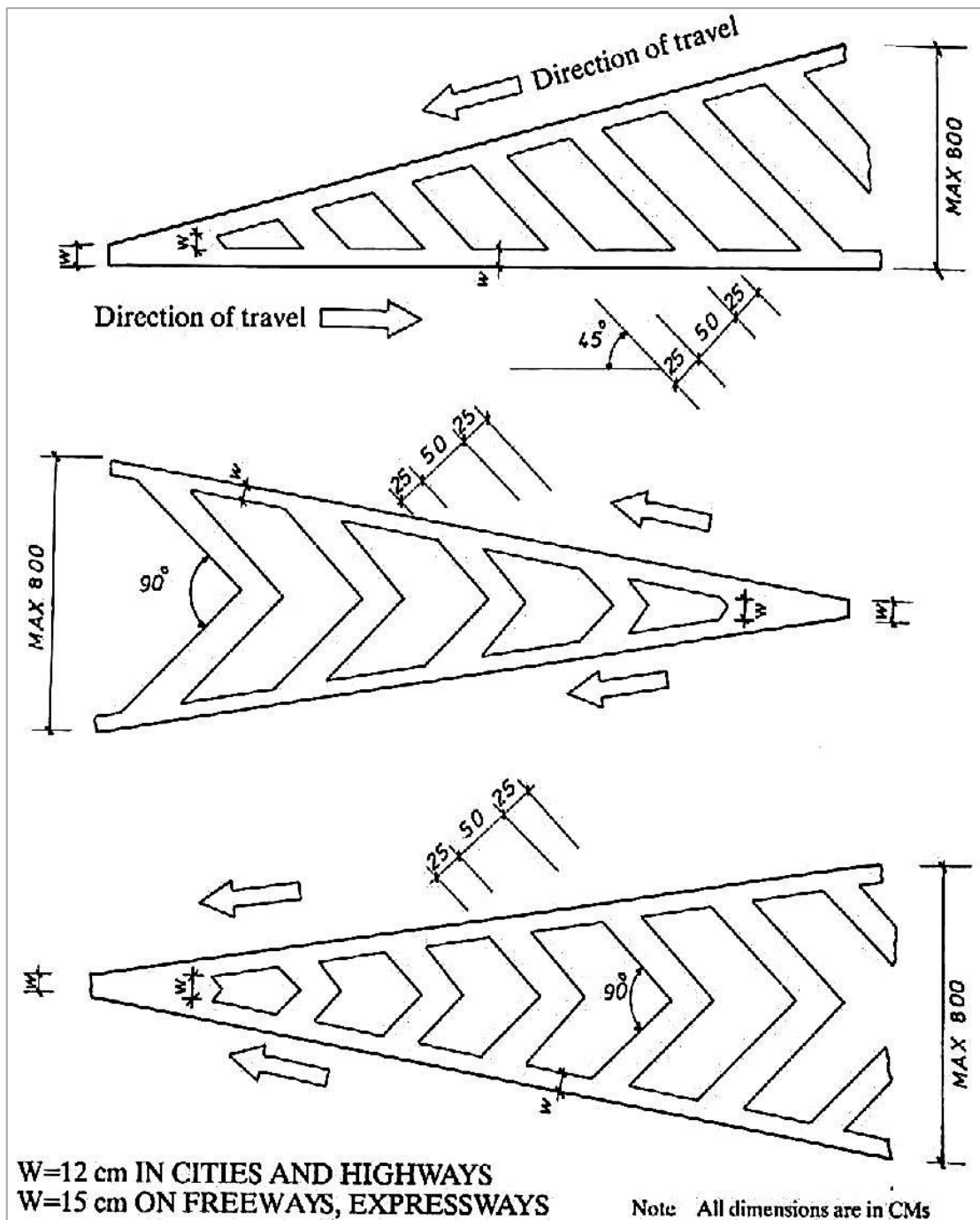


Figure 11-3/11: Marking of No Overtaking/Restricted Area [4, p.V-20]

11-3/4/2 ARROWS MARKING

On roads with traffic lanes to separate vehicles approaching an intersection, the lanes may be indicated by lane selection arrow marking. Lane selection arrows may be used on a one-way road to confirm the direction of traffic. The length of arrow should be 5m for speeds over 50 km/h and 3m for speeds under 50 km/h (populated areas).

There are some special types of arrows which should be used for acceleration lanes and deceleration lanes. Five deflecting arrows should be used at a spacing of 10m, 20m, 30m and 40m in the acceleration lane before the start of taper. One elongated arrow should be used at the beginning of the deceleration lane. Standard arrow markings are shown in figures (11-3/12) to (11-3/15).

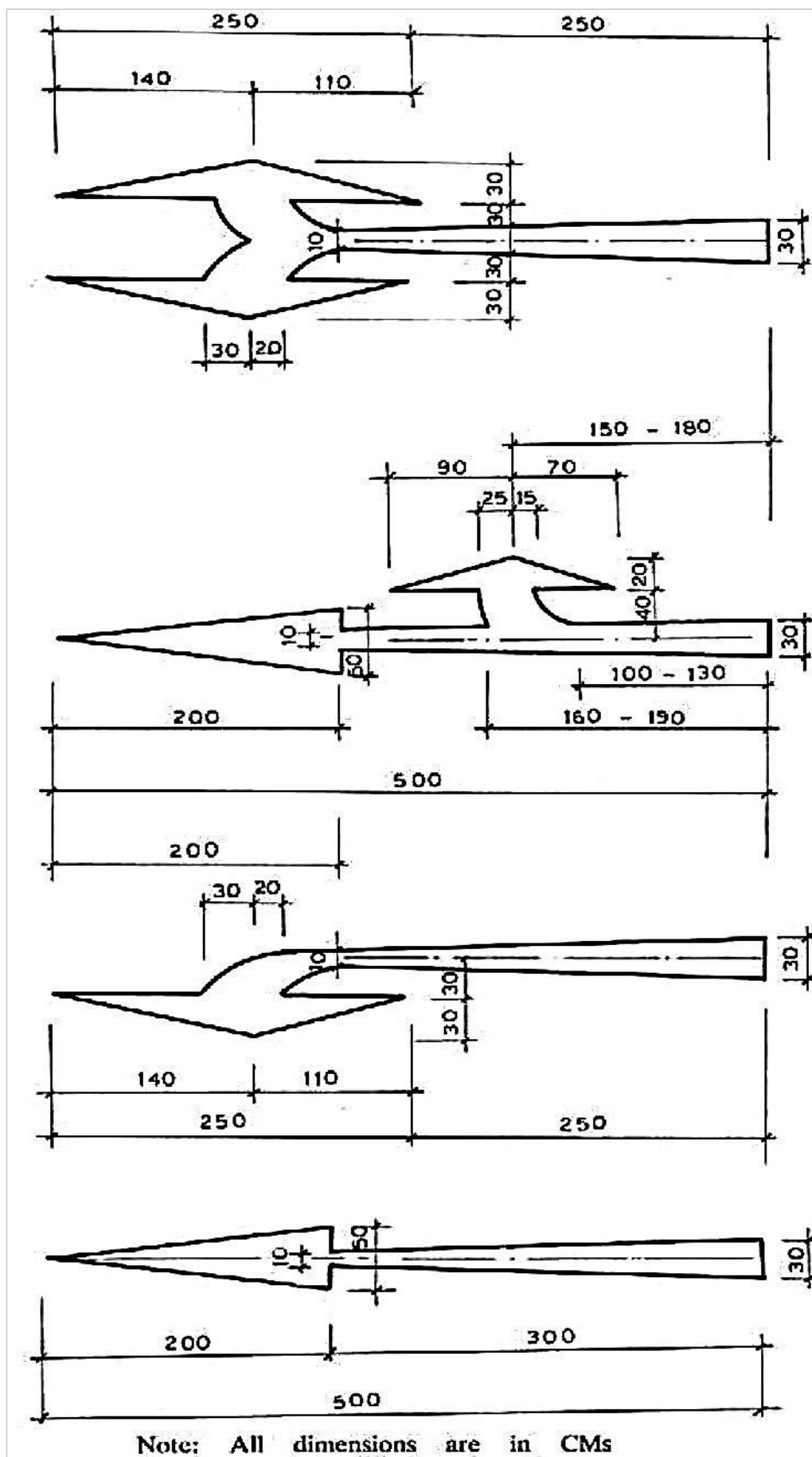


Figure 11-3/12: Arrow Marking for Speeds over 50 km/hr. [4, p.V-22]

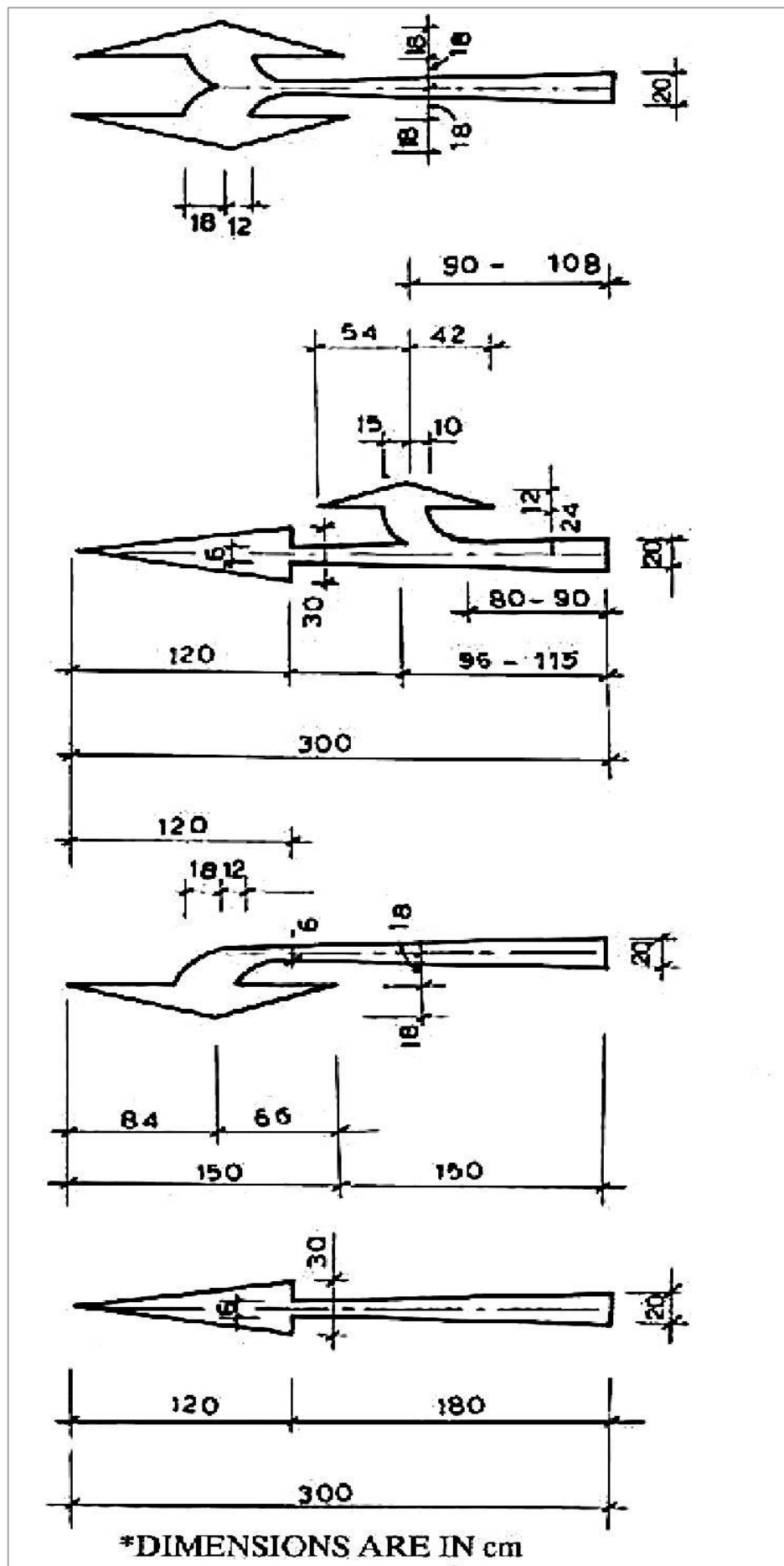


Figure 11-3/13: Arrow Marking for Speeds under 50 km/hr. [4, p.V-23]

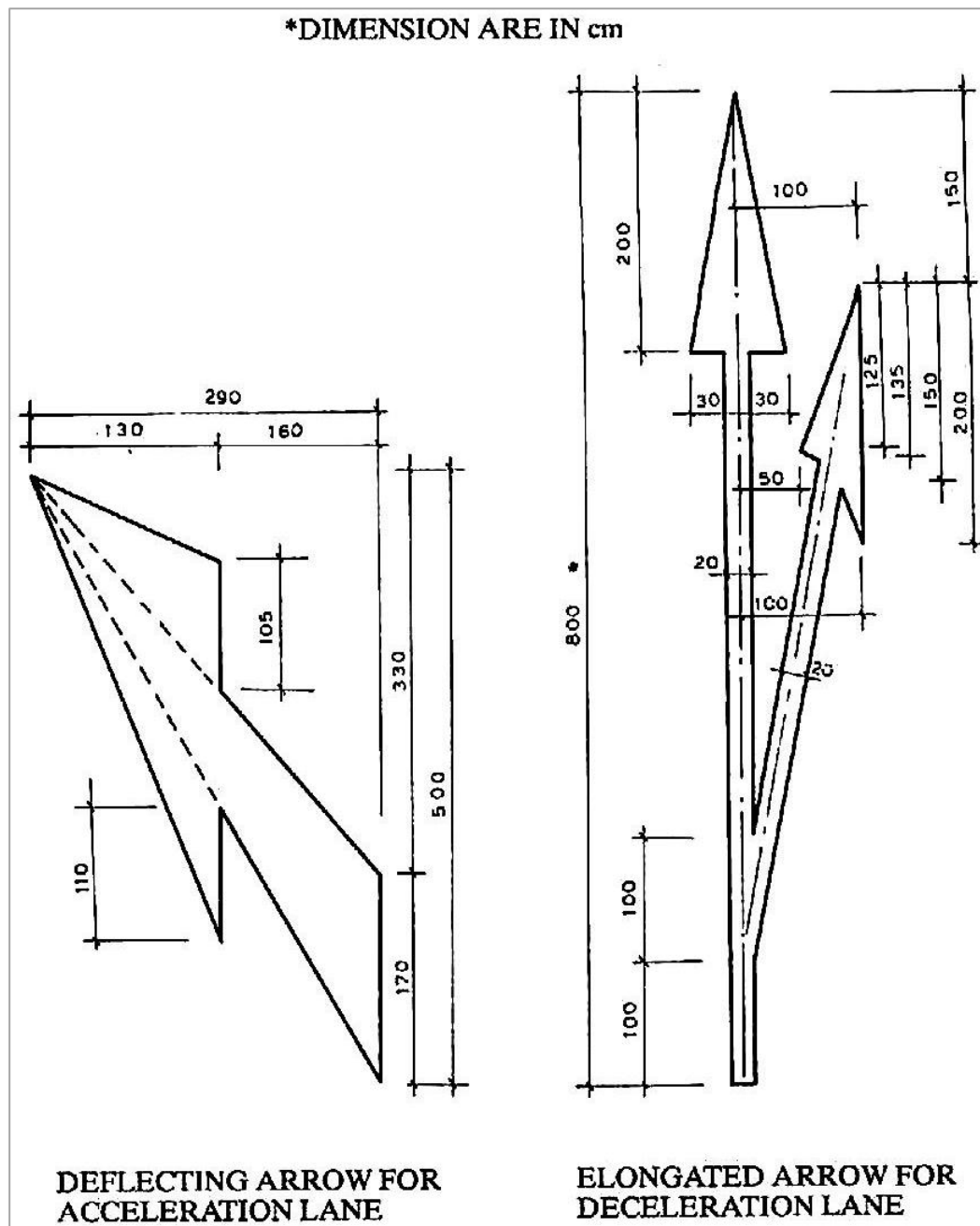


Figure 11-3/14: Arrow Marking for Speeds over 50 km/hr. [4, p.V-24]

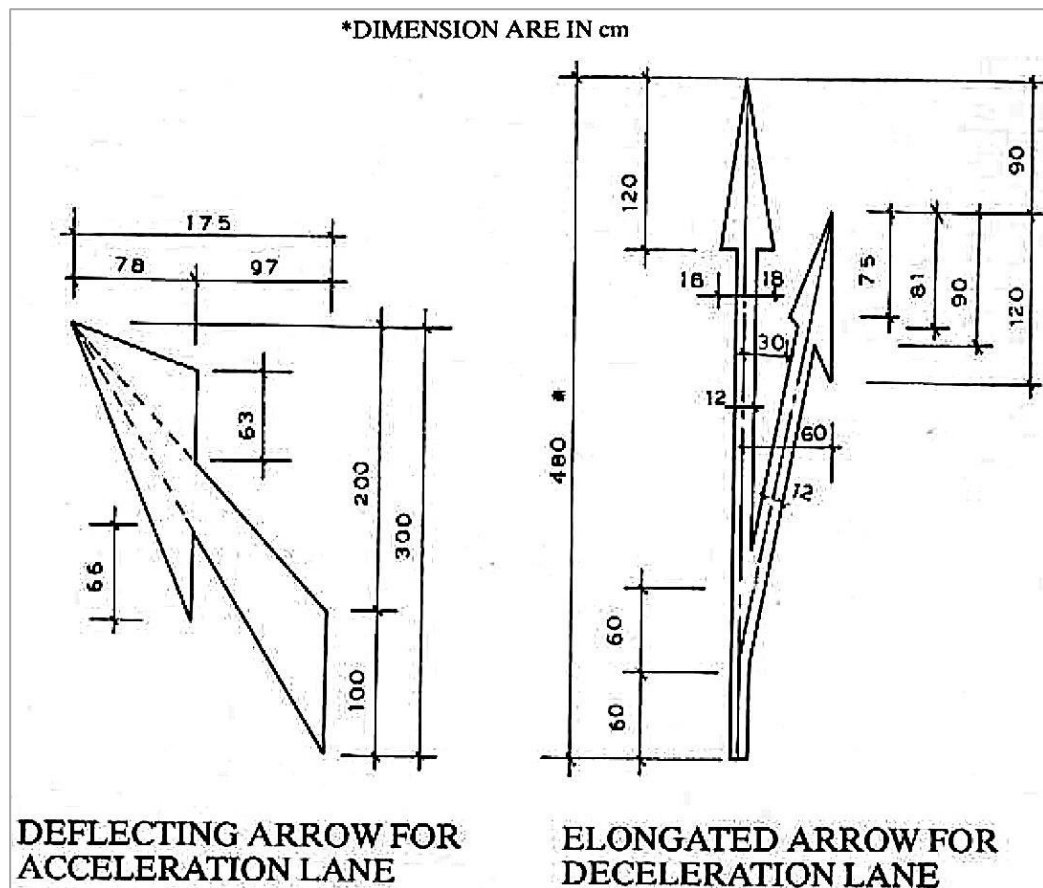


Figure 11-3/15: Arrow Marking for Speeds under 50 km/hr. [4, p.V-25]

11-3/4/3 WORDS AND NUMERALS

Word and symbol markings on the pavement are used for the purpose of guiding, warning, or regulating traffic. Example of standard word is shown in figure (11-3/16).



Figure 11-3/16: Example of Elongated Letters for Word Pavement Markings [2, p.3B-31]

Letters and numerals should be 1.8 m or more in height and shall be white in color. Word and symbol markings should not exceed three lines of information. If a pavement marking word message consists of more than one line of information, it should be read in the direction of travel. The first word of the message should be nearest to the road user.

The longitudinal space between word and symbol message markings should be at least four times the height of the characters for low speed roads, but not more than ten times the height of the characters. Pavement word and symbol markings should be no more than one lane in width except The SCHOOL word marking which may extend to the width of two approach lanes. When the SCHOOL word marking is extended to the width of two approach lanes, the characters should be 3m or more in height. On narrow, low-speed shared-use paths, the pavement words and symbols may be smaller than suggested, but to the relative scale

The International Symbol of Accessibility parking space markings may be placed in each parking space designated for use by persons with disabilities. A blue background with white border may supplement the wheelchair symbol as shown in figure (11-3/17).

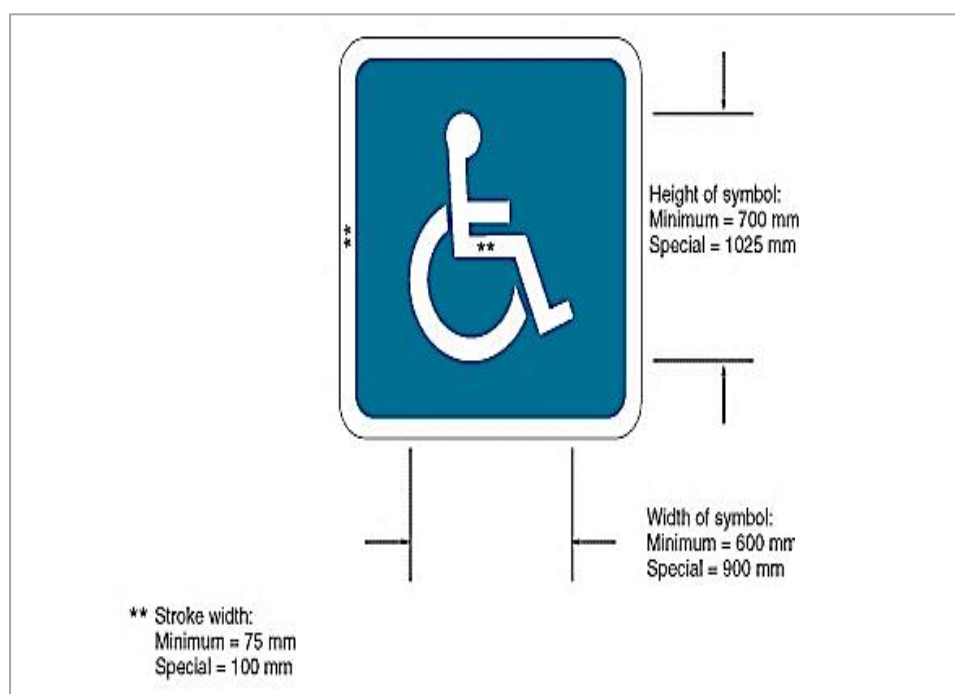


Figure 11-3/17: International Symbol of Accessibility Parking Space Marking With Blue Background and White Border Options [2, p. 3B-31]

Word and numerals markings may include, but are not limited to, the following. Other words or symbols may also be used under certain conditions.

- A) Regulatory:
 1. Stop
 2. Right (left) turn only
 3. 40 , 60, 80 or 100 km/hr.
 4. Arrow Symbols
- B) Warning:
 1. Stop ahead
 2. Yield ahead
 3. Yield ahead triangle symbol
 4. School xing
 5. Signal ahead
 6. Ped xing
 7. School

8. Sump

C) Guide:

1. Road number
2. Destination
3. Distance for destination

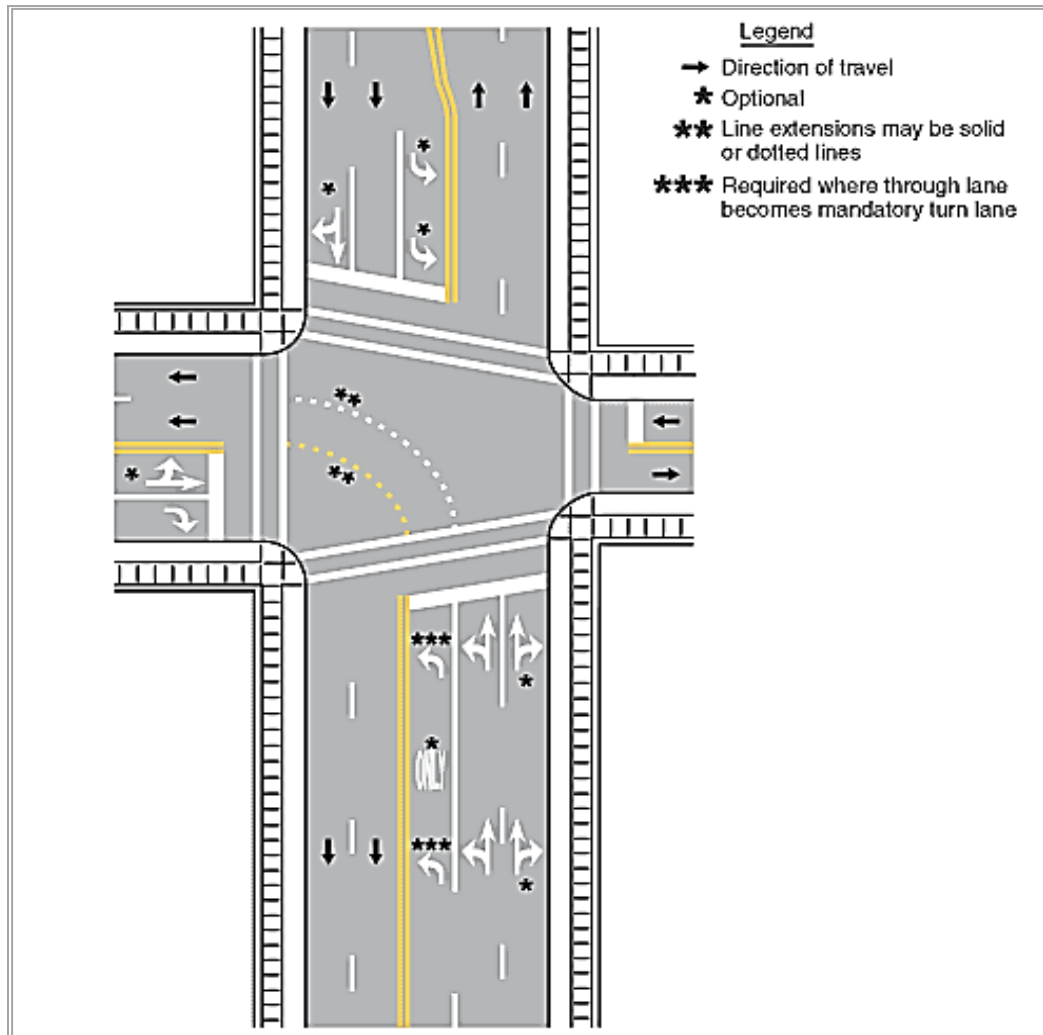


Figure 11-3/18: Examples of Lane Use Control Word and Symbol Markings [2, p.3B-34]

11-3/4/4 APPROACHES TO RAILWAY CROSSING

At level crossings where rail traffic has priority over road traffic, a warring line (1.5m strip, 1.5 gap) should be marked in the center of the carriageway over a length of about 160m from the 2nd ban warning beacon.

If in special cases overtaking is to be prohibited at railway crossing a prohibitory line shall be provided on use right side next to the center line departing from the 2nd bar beacon. If no center line is provided over the whole road section. A center line should begin at the 3rd bar beacon. These markings are shown in figure (11-3/19). These markings are not sufficient protection by themselves and must always be used in conjunction with signs and other devices.

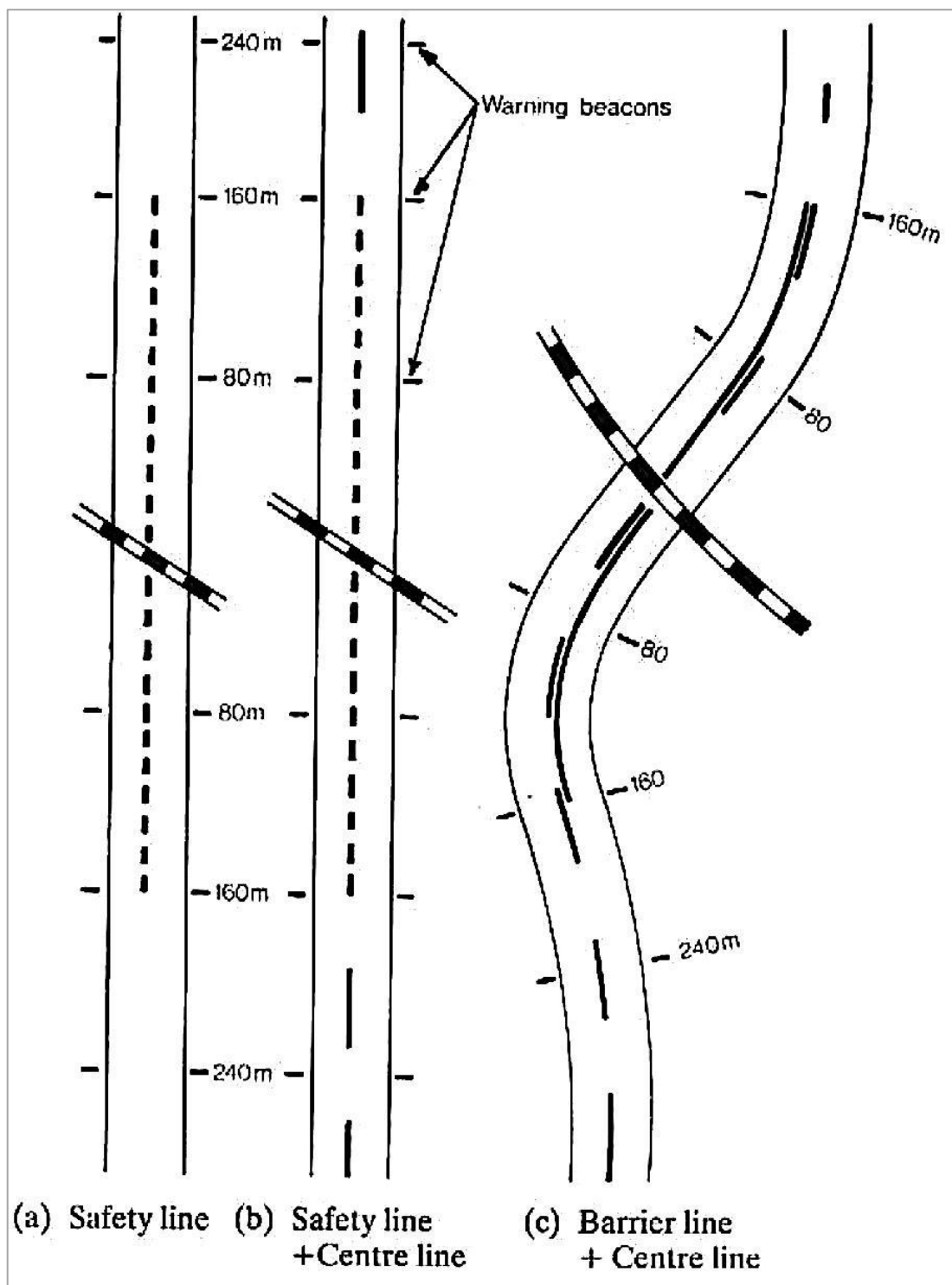


Figure 11-3/19: Marking of railway crossings [4, p.V-27]

11-3/4/5 SPEED HUMP MARKING

Speed hump marking should be used to warn drivers of the presence of a speed hump in the roadway. If speed hump markings are used, they shall be a series of eight white 300 mm transverse lines that become longer and are spaced closer together as the vehicle approaches the speed hump or other deflection. If advance markings are used, they shall comply with the detailed design shown in figure (11-3/20).

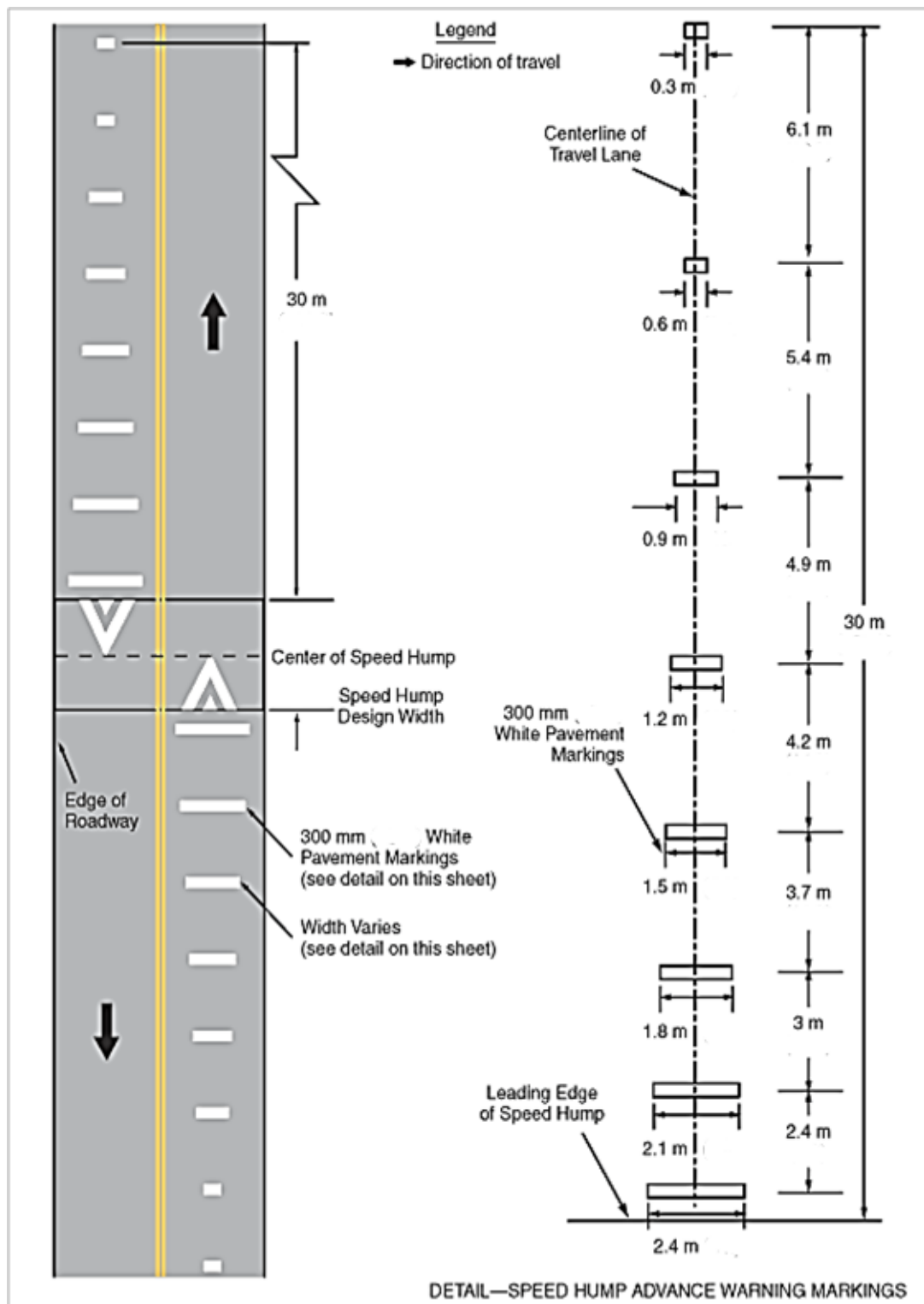


Figure 11-3/20: Examples of Advance Warning Marking for Speed Humps [2, p.3B-49]

11-3/4/6 OBJECT MARKER

Pavement markings shall be used to guide traffic away from fixed obstructions within a paved roadway. Approach markings for bridge supports, refuge islands, median islands, and raised channelization islands shall consist of a tapered line or lines extending from the centerline or the lane line to a point 0.3 to 0.6 m to the right side, or to both sides, of the approach end of the obstruction (see figure 11-3/21).

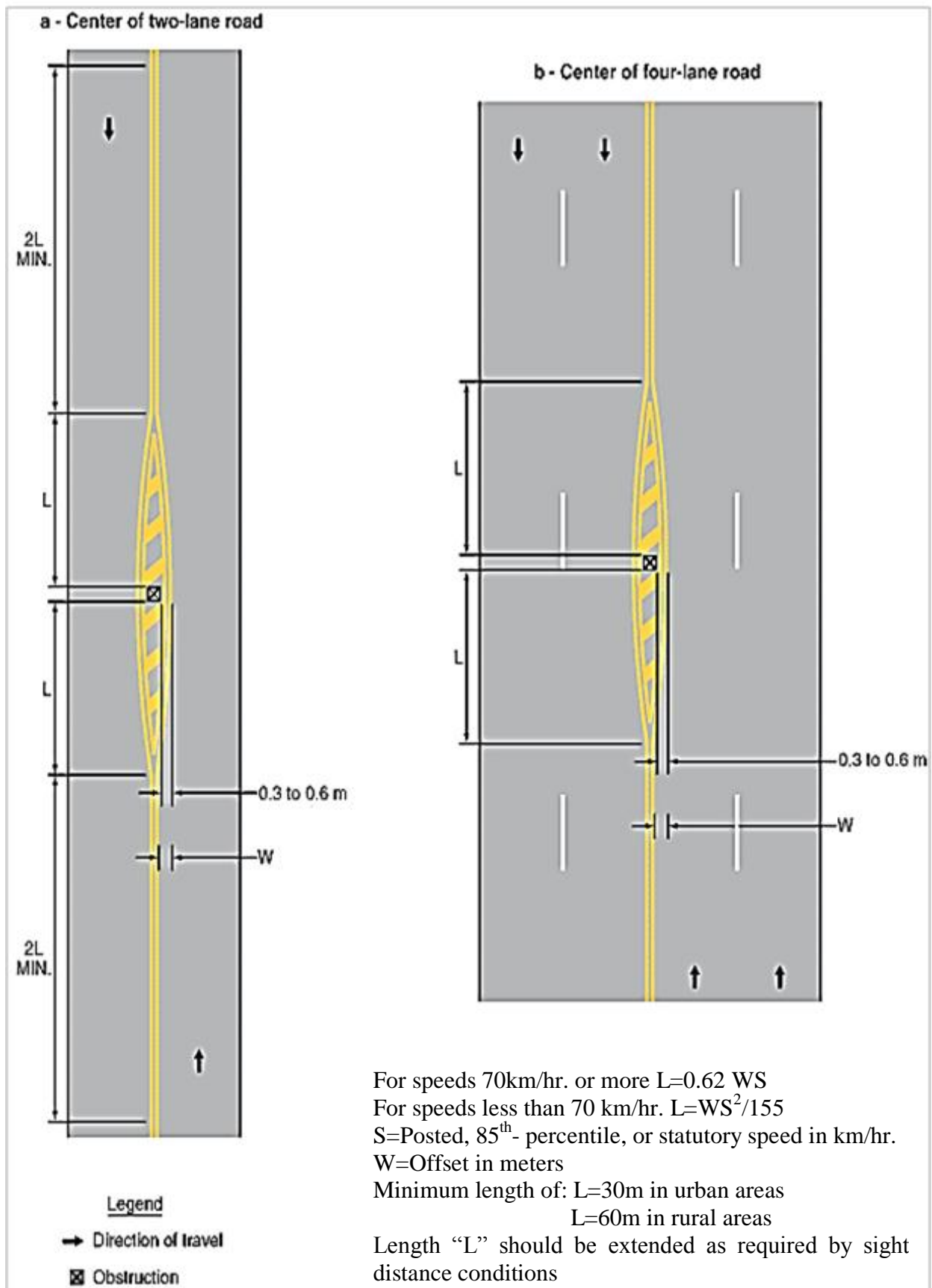


Figure 11-3/21: Examples of Markings for Obstructions in the Roadway [2, p.3B-21]

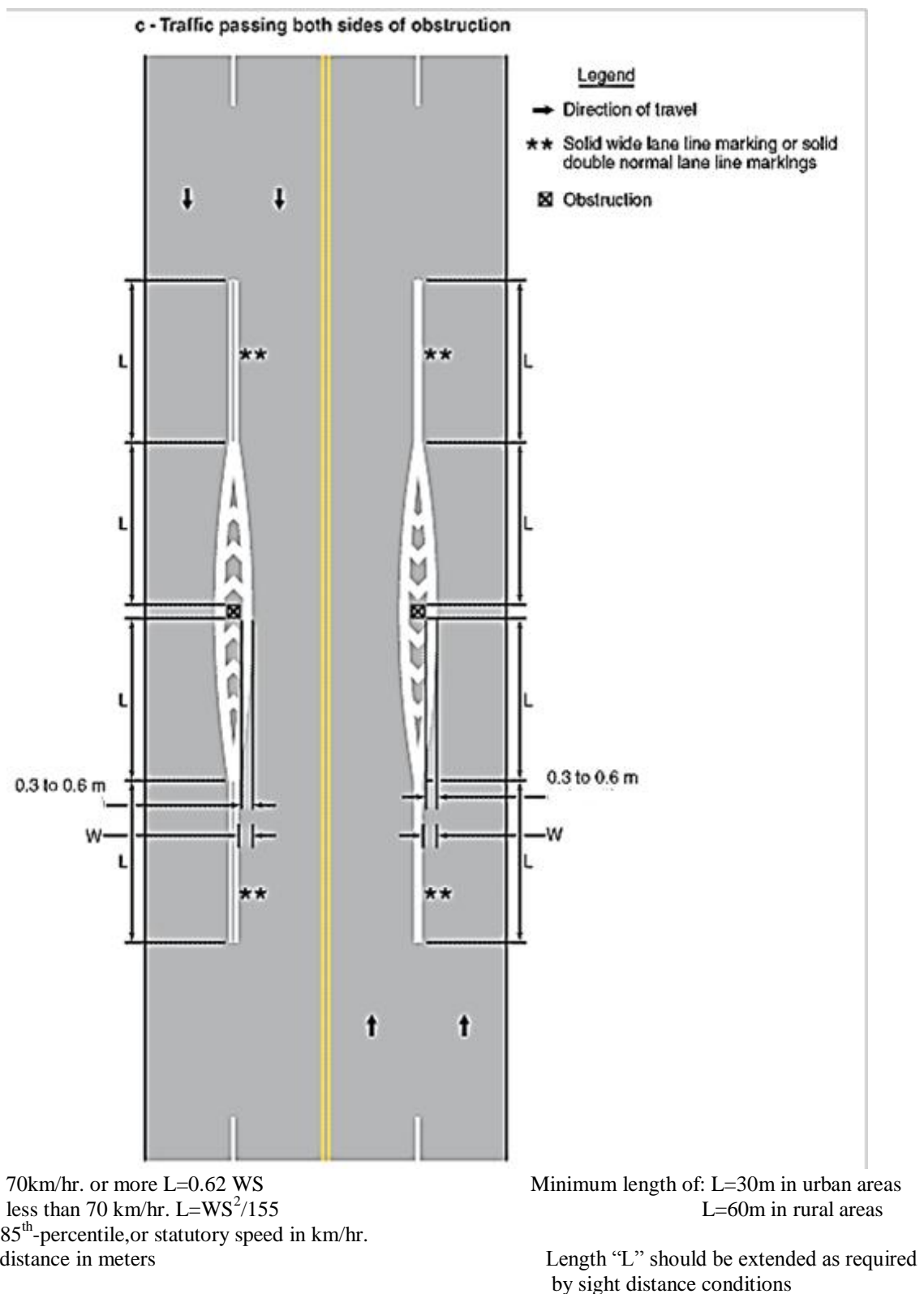


Figure 11-3/21: Examples of Markings for Obstructions in the Roadway [2, p.3B-21]

Object markers are used to mark obstructions within or adjacent to the roadway. When used, these markers should consist of an arrangement of one or more of the following designs:

- Type 1 markers consist of an all-yellow reflective diamond panel 450 mm in size. A variant of this marker type incorporates nine yellow reflector units in the panel. Each reflector unit should have a dimension of approximately 75 mm mounted symmetrically

on a 450-mm diamond-shaped yellow panel. Type 1 markers may be larger if conditions warrant (see figure 11-3/22, type 1).

- Type 2 is a striped vertical rectangle approximately 300 mm by 900 mm in size with alternating black and reflectorized yellow stripes sloping downward at an angle of 45° toward the side of the obstruction on which traffic is to pass. The minimum width of the yellow stripe should be 75 mm. Type 2 object markers with stripes that begin at the upper right side and slope downward to the lower left side are to be designated as “right” object markers (see Figure 11-3/22, type 2).
- Type 3 markers indicate the end of a roadway. When it is determined that markers should be placed at the end of a roadway where there is no alternative vehicular path, a marker consisting of nine red reflectors, each with a minimum dimension of approximately 75 mm, mounted symmetrically on a 450-mm red diamond panel; or a 450-mm diamond reflectorized red panel should be used. More than one marker or a larger marker may be used at the end of the roadway where conditions warrant. The minimum mounting height of this marker should be 1.20 m.

Appropriate advance warning signs should be used (see figure 11-3/22, type 3).

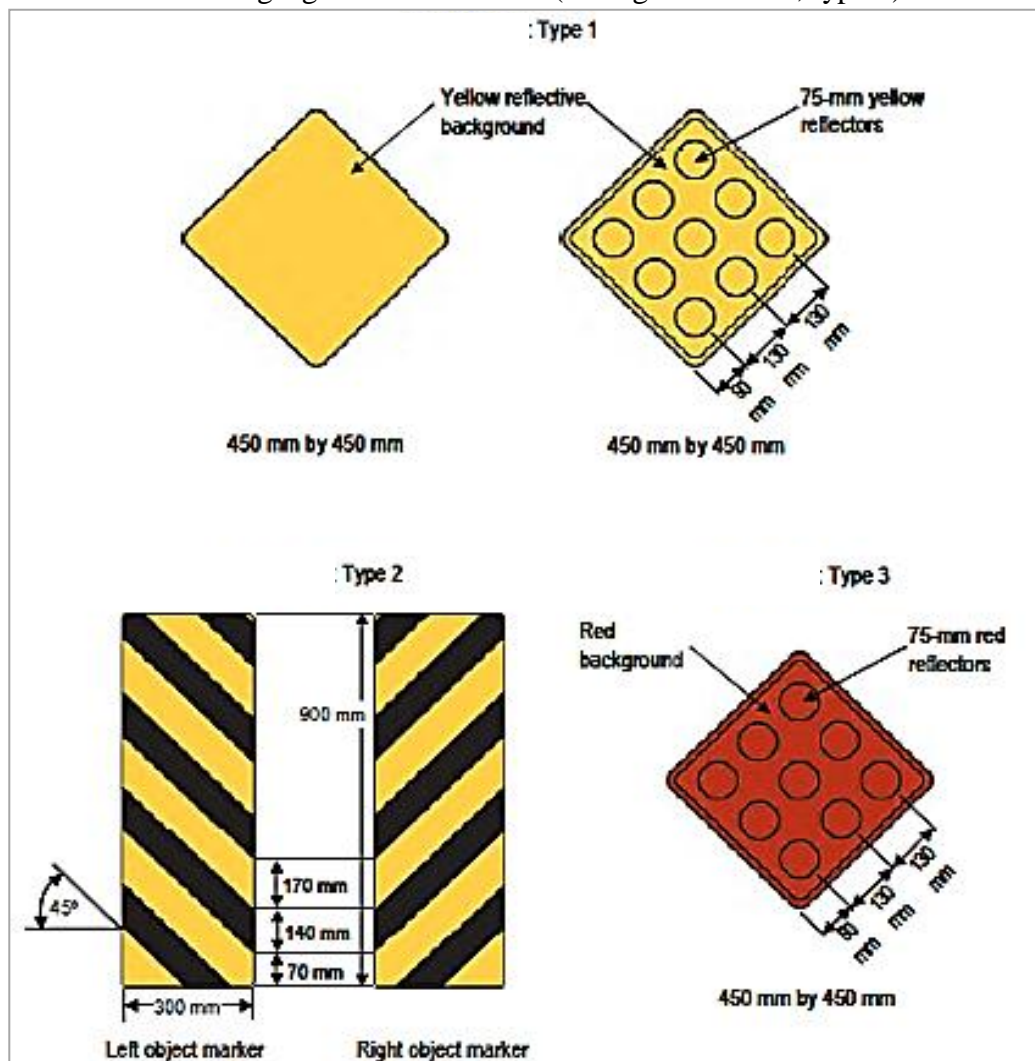


Figure 11-3/22: Object Markers [4, p.6B-25]

- **OBJECTS IN THE ROADWAY**

Obstructions within the roadway should be marked with a Type 1 or Type 2 object marker. For additional emphasis, a large surface such as a bridge pier may be painted with diagonal stripes, 300 mm or more in width, similar in design to the Type 2 object marker. The alternating black and reflectorized yellow stripes should be sloped down at an angle of 45° toward the side of the obstruction that traffic is to pass. The minimum mounting height should be 1.20 m. In addition to markers on the face of the obstruction, warning of approach to the obstruction shall be given by appropriate pavement markings as shown in figure (11-3/21).

- **OBJECTS ADJACENT TO THE ROADWAY**

Objects not actually in the roadway may be so close to the edge of the road that a marker is required. These include guardrail ends, underpass piers, bridge abutments, handrails, and culvert headwalls. In some cases, a physical object may not be involved, but other roadside conditions such as narrow shoulder drop-offs, gores, small islands, and abrupt changes in the roadway alignment may make it undesirable for a driver to leave the roadway. Type 2 object markers are intended for use at such locations. The inside edge of the marker should be in line with the inner edge of the obstruction. Typical applications of markers for roadside obstructions are shown in Figure below.

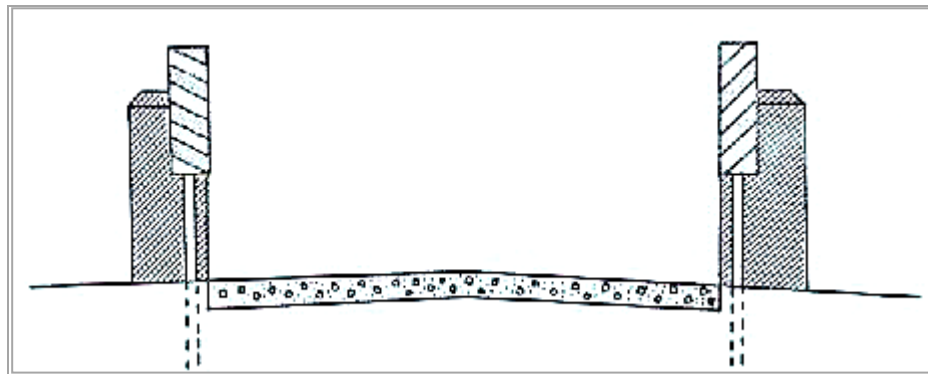


Figure 11-3/23: Object Marker Location at Bridge End [4, p.V-32]

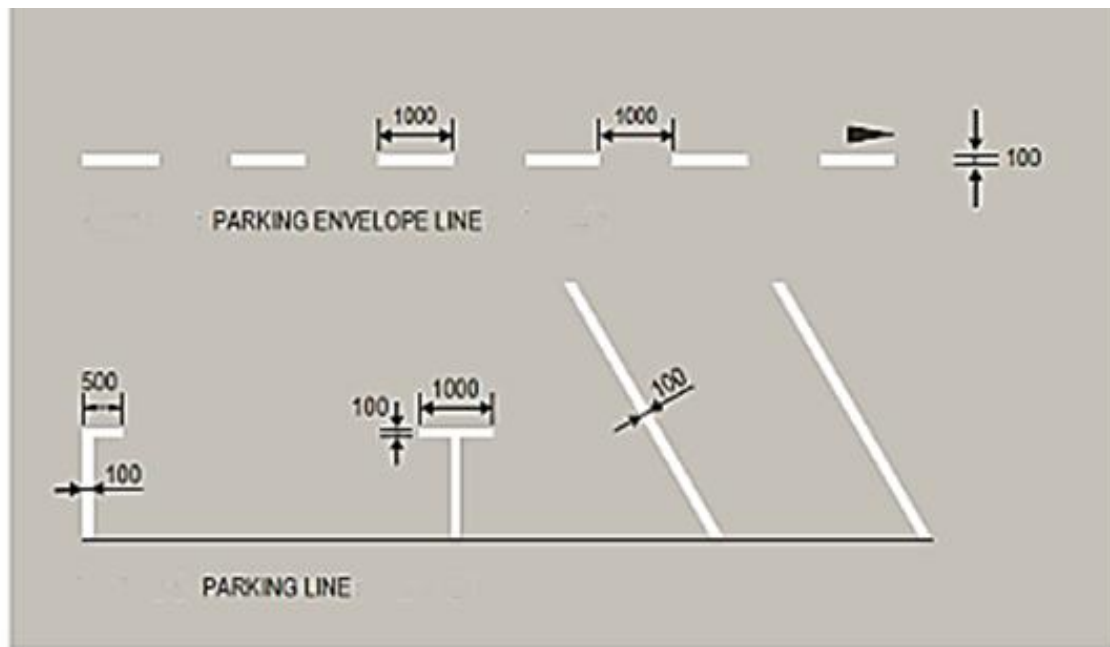
11-3/4/7 PARKING MARKING

- **PARKING ENVELOPE LINE**

Parking envelope line marking imposes a mandatory requirement that drivers parking their vehicles within a marked area park such that no part of their vehicle encroaches upon the pavement area outside a parking envelope so marked. Parking envelope line marking should be a longitudinal broken white line 100 millimeters wide. The configuration of the parking envelope line should be a repeated pattern of one meter of line separated by a one-meter gap. It should be located at the edge of pavement adjacent to areas where parking is permitted, but should not be used if the physical space available for parking is less than 2.2 meters wide (see Figures 11-3/24).

- **PARKING SPACE LINE MARKING**

Parking space line marking imposes a mandatory requirement that drivers parking their vehicles within a marked parking space park such that their vehicle is wholly within the lines defining the limits of the parking space. Parking space line marking should be a solid white line 100 millimeters in width. It should extend from the curb line (if a curb is present) and end at the edge of the designated parking bay. For parallel parking space delineation the end of the parking space line should have a 500-millimeter long L-shape for end-marking of a parallel parking bay, and a 1000 millimeter long T-shape for intermediate lines. Perpendicular and angled parking space markings generally do not require end shapes since the ends of the more closely spaced parking space lines should clearly delineate the parking bay limits.



(All Dimensions are in Millimeters) [3, p.6-9]

11-3/4/8 RAISED PAVEMENT MARKERS

A raised pavement marker is a device with a height of at least 10 mm mounted on or in a road surface that is intended to be used as supplement or substitute for pavement markings. The color of raised pavement markers under both daylight and nighttime conditions shall conform to the color of the marking for which they supplement or substitute. The side of a raised pavement marker that is visible to traffic proceeding in the wrong direction may be red.

There are two basic classifications of raised pavement markers: Retroreflective and Nonretroreflective. The following conditions may warrant the use of raised pavement markers:

- Areas regularly subjected to fog, dust, or blowing sand resulting in reduced visibility.
- Areas of heavy traffic volumes that rapidly deteriorate painted markings and that are disruptive to regularly maintain.
- Isolated areas that have low geometric-roadway design standards for the traffic conditions prevailing and that are not scheduled in the near term for improvement.

- Isolated areas with documented high incidence of collision and/or low levels of lane discipline by drivers, particularly in curved or complex roadway geometry conditions.
- Long-term roadworks sites.
- Freeways (all markings on freeways should be supplemented by Retroreflective markings).
- Unlighted rural roadways.
- Within nonweaving sections of roundabouts.
- the entire curved section of roadway as well as for a distance in advance of the curve that approximates 5 seconds of travel time

Figure (11-3/25) shows example of raised pavement marking application.

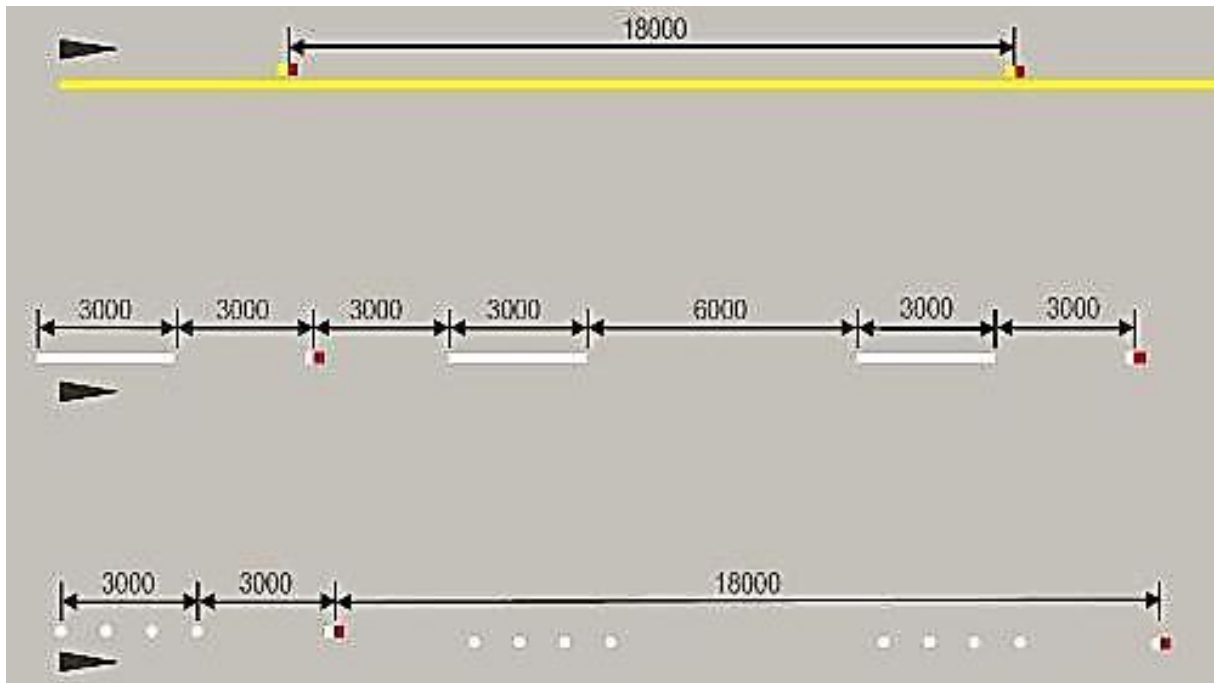


Figure 11-3/25: Examples of Raised Pavement Marking Applications

(All Dimensions are in Millimeters) [3, p.6-21]

11-3/4/9 CONTINUITY MARKING

Continuity line may be used to provide guidance for through traffic at discontinuities in the pavement edge delineation, its use is optional. Continuity line marking should be a longitudinal broken white line that follows the edge of the through-lane of traffic across an exit ramp, a slip road exit, or an intersection. Its configuration should be a repeated pattern of 1 meter of line followed by 3 meters of gap. It should be 150-millimeters wide for posted speeds below 70 km/h and 200-millimeters wide for posted speeds above 70 km/hr.

Figure (11-3/26) below shows example of continuity marking applications.

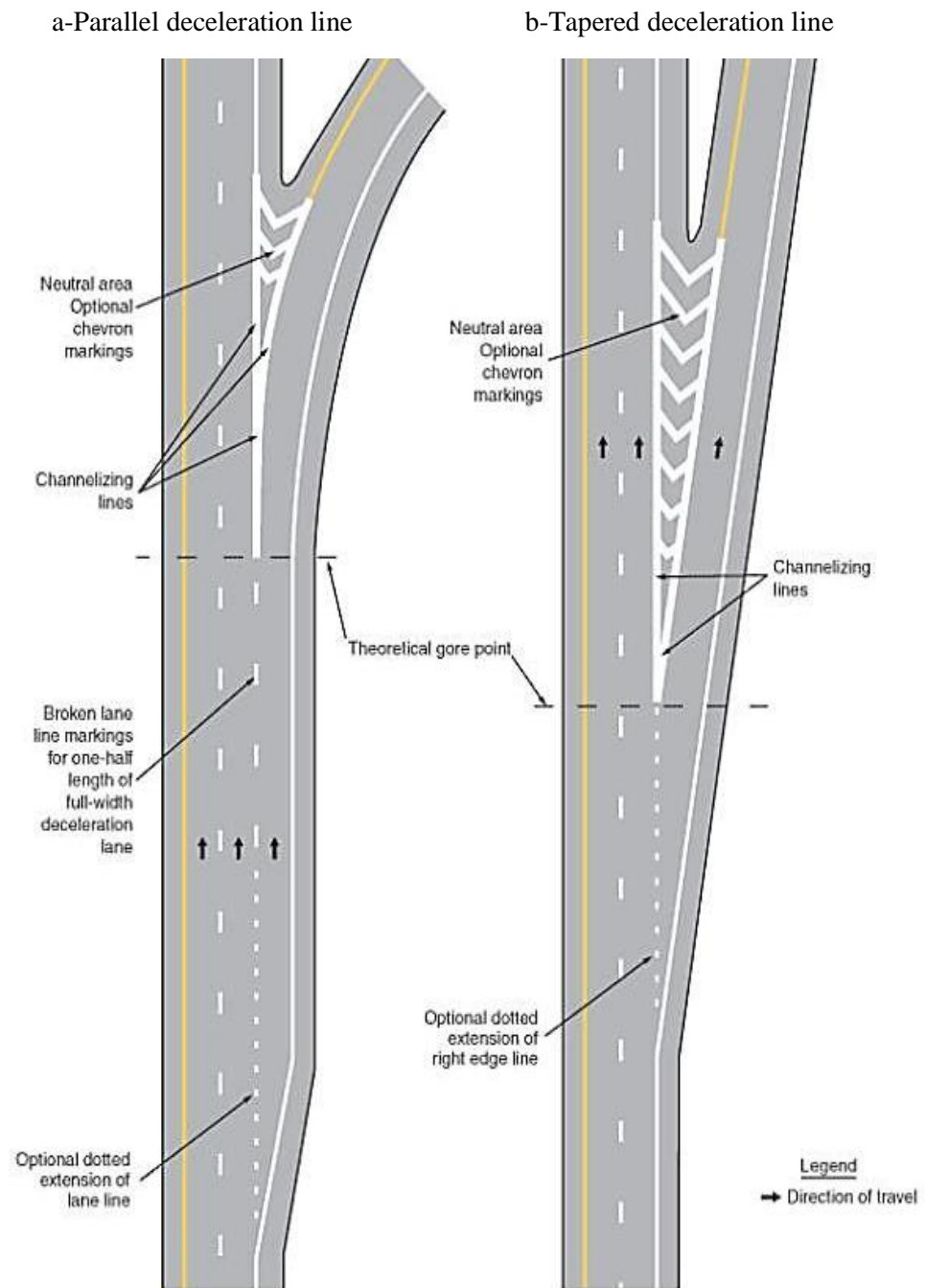


Figure 11-3/26: Examples of Channelizing Line Applications for Exist Ramp Markings [2, p.3B-12]

11-3/5 CURB PAINTING

Curb markings are most often used to indicate parking regulations or to delineate the curb. Since yellow and white curb markings are frequently used for curb delineation and visibility, it is advisable to establish parking regulations through the installation of standard signs. Curb delineation and visibility should consist of painting the top and front face of a curb with alternating sections of yellow and white-colored paint. The length of a yellow section and the length of a white section should be equal to each other. When precast curbs are used, each section may be painted with either yellow or white such that the length of each color will match the length of individual curbs.

Reflectorized, continuous yellow paint should be placed on the curbs of intersection islands located in the line of traffic flow where the curb serves to channel traffic to the right or to the left of the island. Reflectorized, continuous white paint should be used when traffic may pass on either side of the divisional island. In front of fire hydrants, curbstones should be painted red for a length of 12 meters, six meters to either side of the fire hydrant and the fire hydrant combination sign.

11-3/6 DELINEATORS

Road delineators are light-retroreflective devices mounted in series at the side of the roadway to indicate the roadway alignment. Delineators are effective aids for night driving and considered as guidance devices rather than warning devices. Delineators may be used on long, continuous sections of highway or through short stretches where there are changes in horizontal alignment, particularly where the alignment might be confusing or at pavement-width transitions.

Delineators should consist of reflector units capable of clearly reflecting light under normal atmospheric conditions from a distance of 300 meters when illuminated by the upper beam of standard automobile lights. Reflective elements for delineators should have a minimum area of approximately 100 cm². Double delineators consist of two reflector units, one mounted above the other. Elongated reflective units of appropriate size may be used in place of the two reflectors.

The color of delineators should, in all cases, conform to the yellow or white color of edge lines. Single delineators should be provided on the right side of expressway roadways and on at least one side of interchange ramps. These delineators may be provided on other classes of roads. Single delineators may be provided on the left side of roadways and should be provided on the outside of bends on interchange ramps. Where median crossovers are provided for official or emergency use on divided highways and these crossovers are to be marked, a double-yellow delineator should be placed on the left side of the through roadway and on the far side of the crossover for each roadway.

Red delineators may be used on the reverse side of any delineator whenever it would be viewed by a drivers traveling in the wrong direction on that particular ramp or roadway. Delineators of the appropriate color may be used to indicate the narrowing of a pavement. The delineators should be used adjacent to the lane affected for the full length of the convergence and should be so placed and spaced to show the width reduction. Delineation is not necessary for the traffic moving in the direction of a wider pavement or on the side of the roadway where the alignment is not affected by the convergence. On a highway with continuous delineation on either or both side, delineators should be carried through the transition and a closer spacing may be warranted.

Delineation is optional on sections of roadway between interchanges where fixed-source lighting is in operation. Delineators, if used, should be mounted on suitable supports so the top of the reflecting head is approximately 1.20 m above the near roadway edge. Delineators should be placed not less than 1.0 m or more than 2.0 m outside the outer edge of the shoulder, or if appropriate, in the line of the guardrail. Delineators may be mounted on the guardrail at a height less than 1.2 m.

Delineators should be placed at a constant distance from the edge of the roadway. However, where a guardrail or other obstruction intrudes into the space between the pavement edge and the extension of the line of delineators, the delineators should be in line with or inside the innermost edge of the obstruction. Typical delineator installations are shown in figure (11-3/27).

Normally, delineators should be spaced at 60 m to 160 m. When normal uniform spacing is interrupted by driveways, crossroads, or similar interruptions, delineators falling within such areas may be moved in either direction, a distance not exceeding one-quarter of the normal spacing. Delineators still falling within such areas should be eliminated. On expressways, a normal delineator spacing is 100 m. Double or vertically elongated delineators should be installed at 30-meter intervals along acceleration and deceleration lanes. Spacing should be adjusted on approaches and throughout horizontal bends so that several delineators are always visible to the driver. Table (11-3/2) shows suggested maximum spacing for delineators at bends.

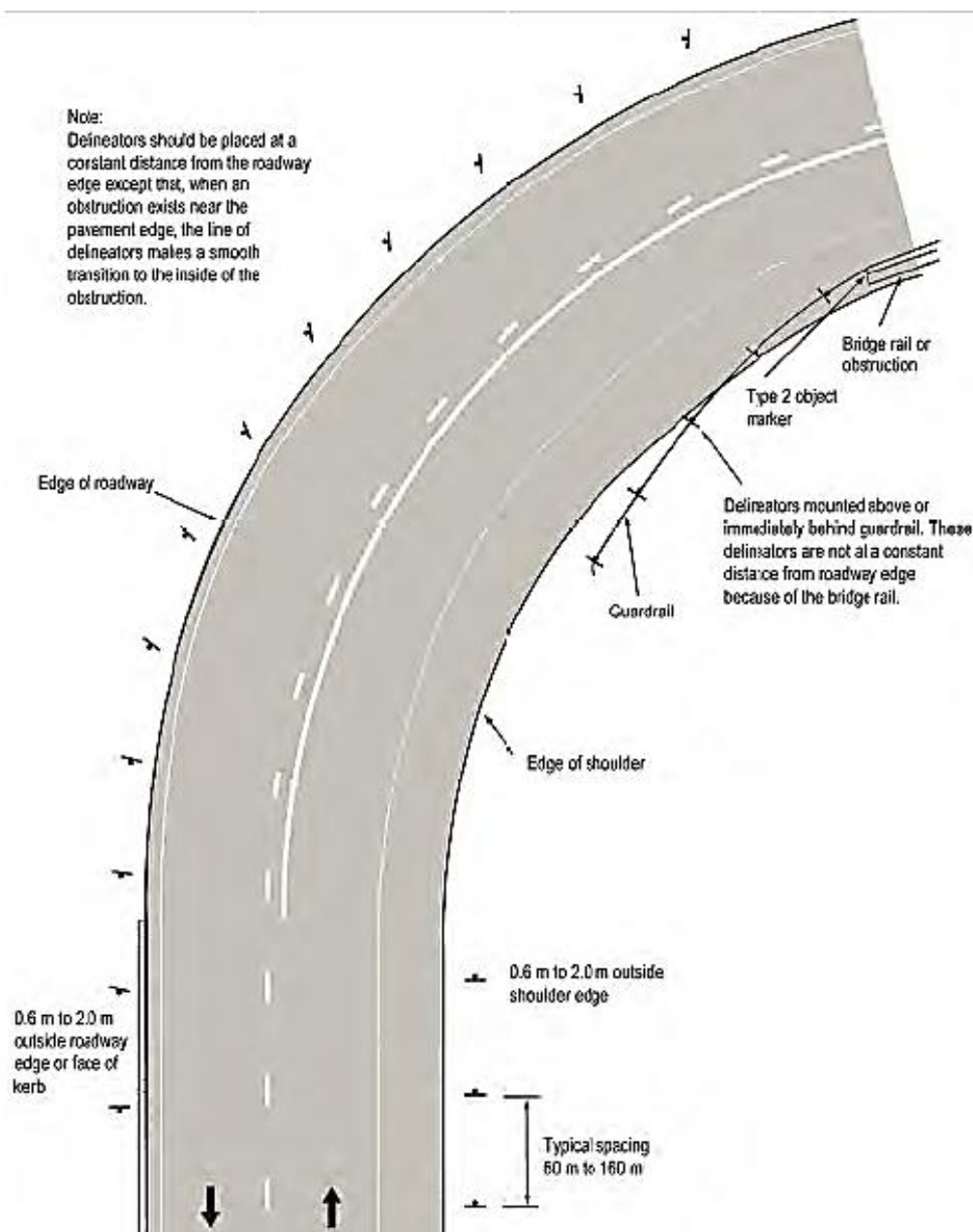


Figure 11-3/27: Typical Delineators Installations [3, p.6-26]

Table 11-3/2: Suggested Maximum Spacing for Highway Delineators on Bends [3, p.6-26]

Radius of Bend. (R) (meters)	Spacing on Bend (S) (meters)
15	6
30	7.5
45	9
60	10.5
75	12
90	13.5
120	16.5
150	19.5
180	21
210	22.5
240	24
270	25.5
300	27
Spacing for radii not shown may be interpolated from the table. The minimum spacing should be 6.0m. The spacing of the first delineator on a tangent adjacent to a bend should be 2S, the second, 3S, and the third, 6S, but not to exceed 100 meters.	

11-3/7 BARRICADES AND CHANNELIZATION DEVICES

• BARRICADES

Red-and-white barricades are to warn and alert drivers of the terminus of a road, street, or highway in other than construction or maintenance areas. The stripes on the barricades should be reflectorized white and reflectorized red. These devices may be used to mark any of the following type locations:

1. Roadway ends in a dead end or cul-de-sac with no outlet.
- 2 A ramp or lane closed for operational purposes.
3. The permanent or semipermanent closure or termination of a roadway.

A typical barricade is illustrated in figure (11-3/28).

• CHANNELIZING DEVICES

Traffic cones and tubular markers are sometimes used outside of construction and maintenance areas for general traffic control purposes. Such uses include adding emphasis to channelizing lines or islands. Two typical channelizing devices (a tube and a cone) are illustrated in figure (11-3/28).

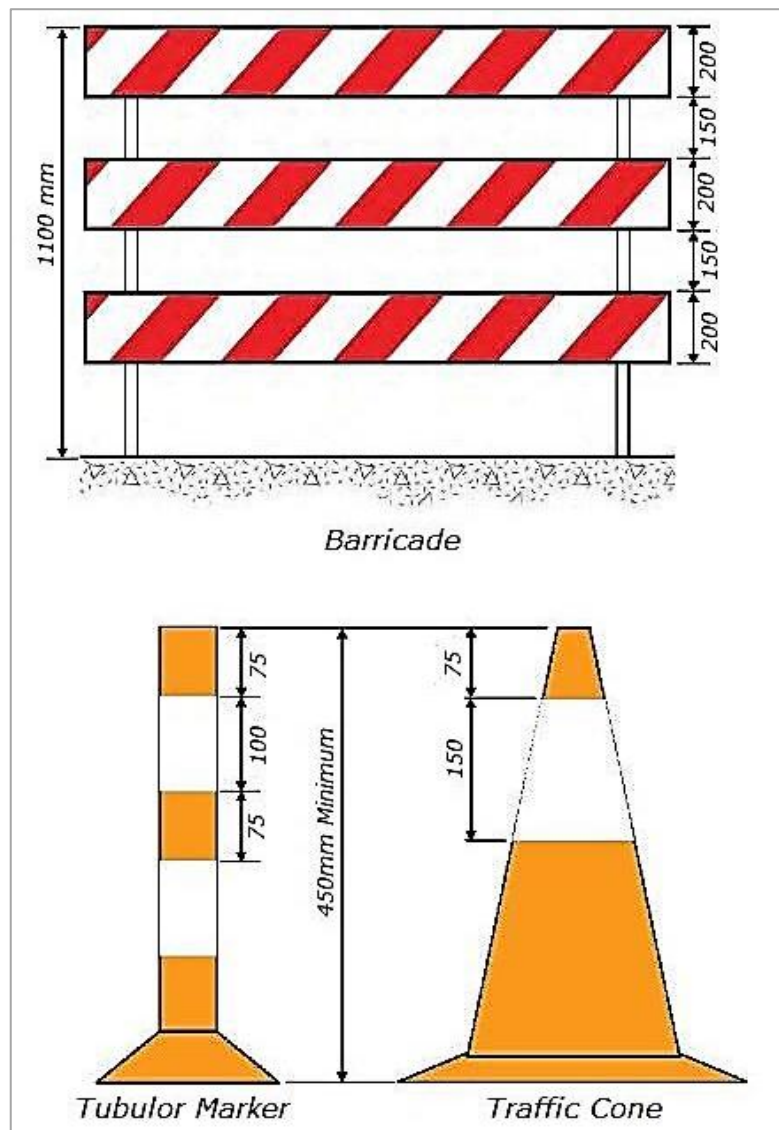


Figure 11-3/28: Barricades and Channelizing Devices [3, p.6-29]

11-3/8 CONTROL DEVICES USED IN WORK ZONES

Roadworks are defined as any roadway or utility construction, maintenance, or repair works occurring within or near a road right of way. Incident areas (traffic accidents, spillage, etc.), police-control points (check points, traffic surveys, etc.), and special-event management (major sports or cultural events) and other short-term disruptions to normal roadway operations also fit within the functional definition of roadworks traffic control.

- **TRAFFIC SIGNS**

Regulatory, warning, and guidance traffic signs comprise a major part of the temporary traffic control devices used at roadworks sites. The majority of signs covered in part (2) may be used in a temporary capacity for work zones but with a yellow background. The most common of these are illustrated in Figures below.

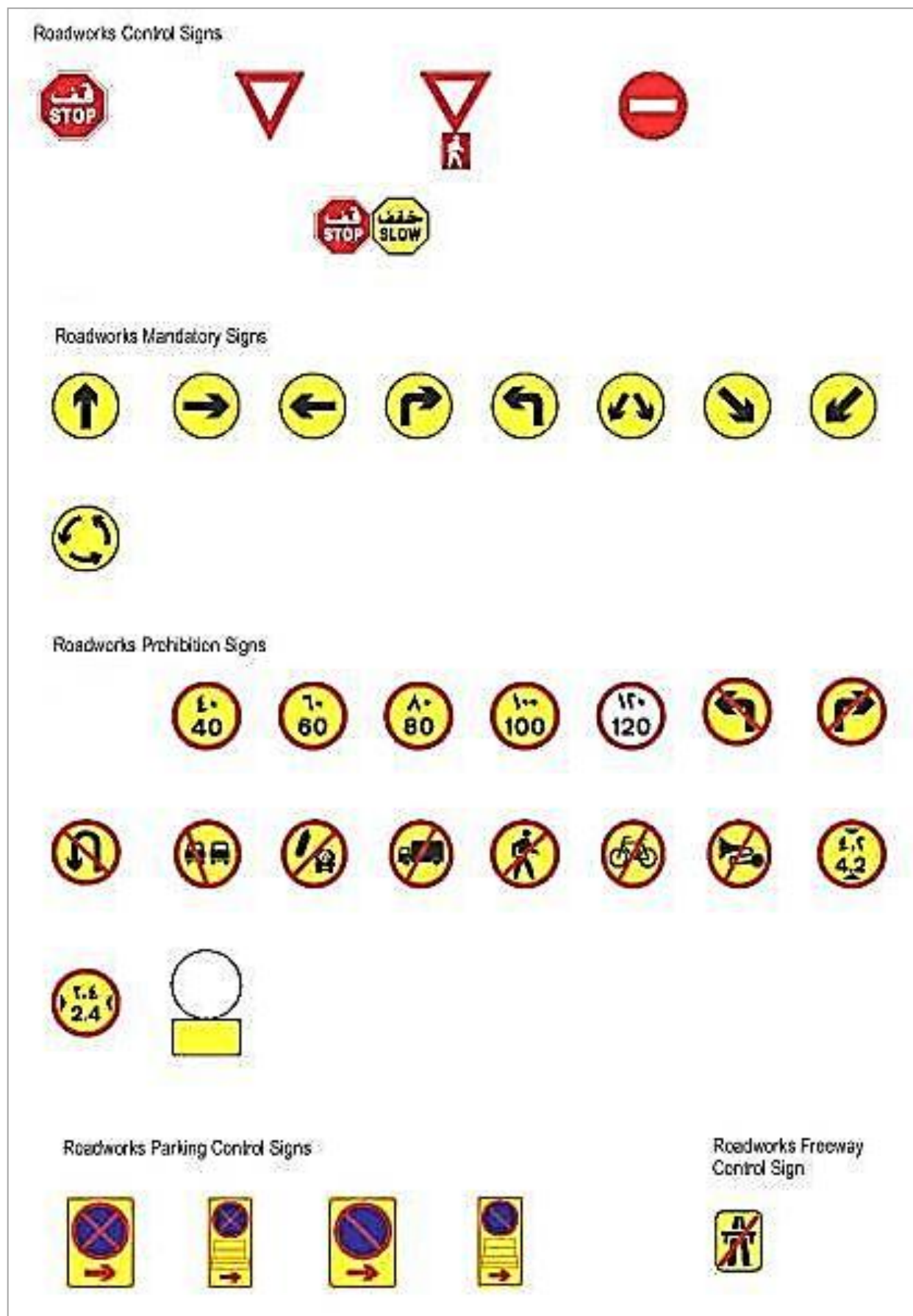
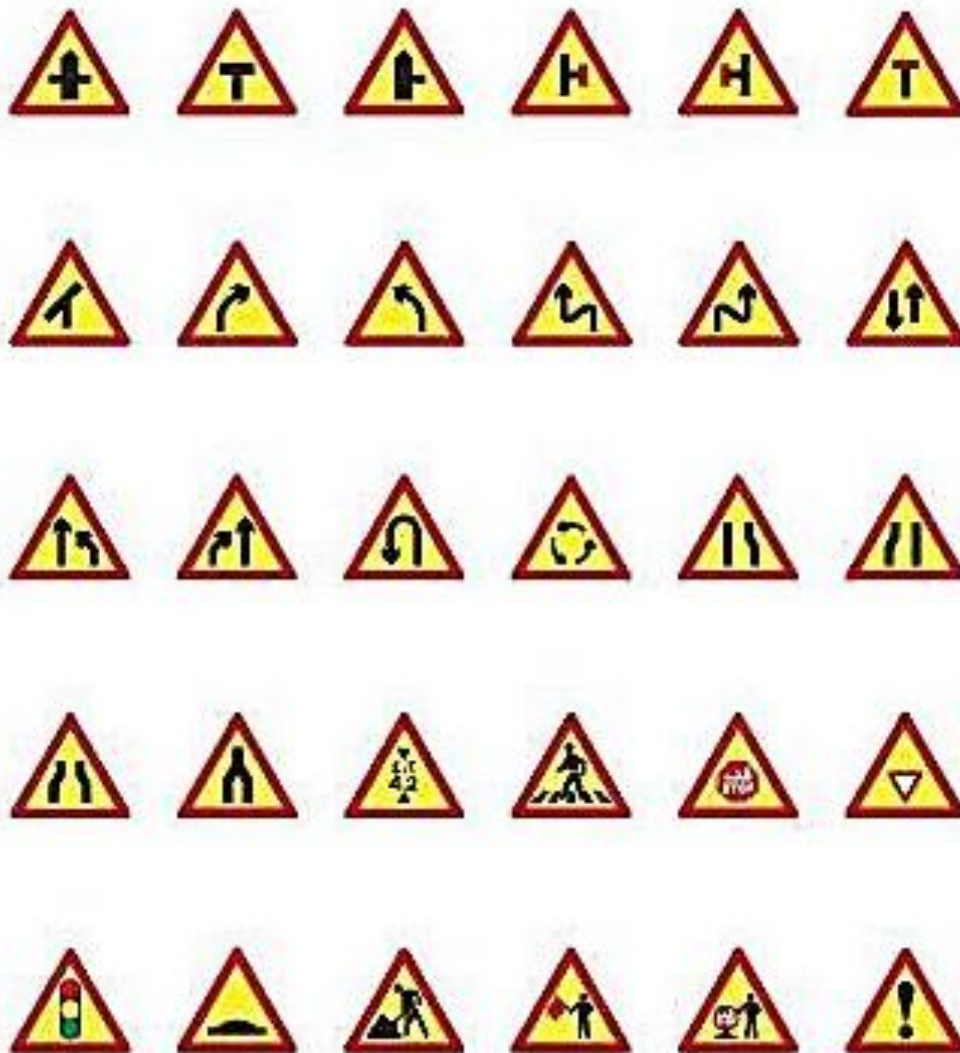


Figure 11-3/29: Regulatory signs at roadworks [3, p.7-7]

Roadworks Advance Warning Signs (not all signs shown)



Roadworks Hazard Marker Signs and Devices

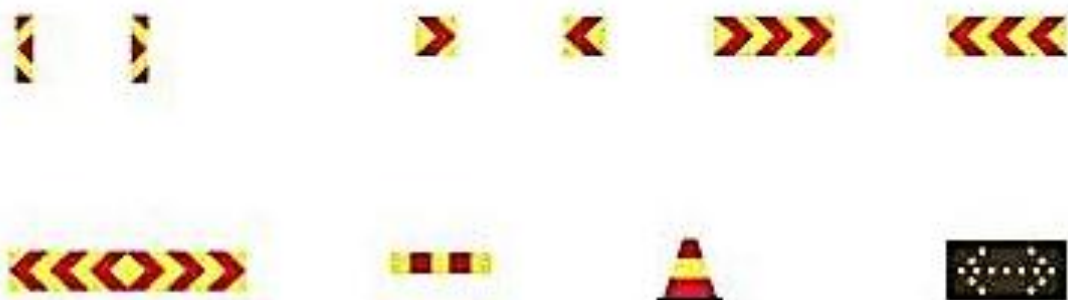
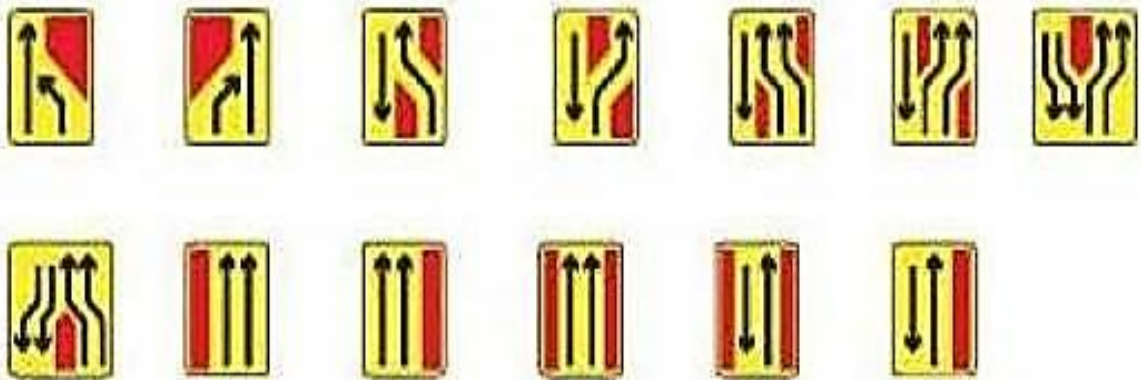


Figure 11-3/30: Warnings signs at roadworks [3, p.7-8]

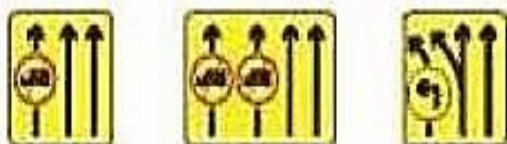
Roadworks Traffic Movement Affected by Obstruction Signs



Roadworks — Additional Lane Signs



Roadworks — Lane Control Signs



At-Grade Vertical Stack Sign

Roadworks Direction Signs



Chevron Detour Sign



Supplemental Plate

Figure 11-3/31: Guide signs at roadworks [3, p.7-9]

11-4 REFERENCES

- [1] MUTCD, "*Manual on Uniform Traffic Control Devices for Streets and Highways*", Federal Highway Administration. U.S Department of Transportation, USA, 2009.
- [2] MUTCD, "*Manual on Uniform Traffic Control Devices for Streets and Highways*", Federal Highway Administration. U.S Department of Transportation, USA, 2003.
- [3] "*Traffic Control Devices Manual*", Road Department, Abu Dhabi, United Arab Emirates, Version 0.1, 2004.
- [4] SCRB, "*Highway Design Manual*", State Corporation of Roads and Bridges, Ministry of Construction and Housing, Iraq, 1982.
- [5] "*Traffic Signal Timing Manual*", Publication No. FHWA-HOP-08-024, Federal Highway Administration, U.S Department of Transportation, USA, June, 2008.