Republic of Iraq
Ministry of Housing & Construction
State Organization of Roads & Bridges

HIGHWAY DESIGN MANUAL

1982

Design & Studies Department
Road & Traffic Division

Republic of Iraq
Ministry of Housing & Construction
State Organization of Roads & Bridges

HIGHWAY DESIGN MANUAL

1982

Design & Studies Department
Road & Traffic Division

GENERAL CONTENTS

INTRODUCTION

The objective of the Highway Design Manual is to establish uniform procedure to be followed for the design of a highway. The scope of the manual is limited to highway design disciplines and aimed at meeting the need for an up-to-date design practice in Iraq.

It does not cover the wider field of highway and traffic engineering such as planning and transportation studies, pavement design, pricing, or other matters of general policy.

Up-to-date studies of highway design and construction in the United States and Europe, and design standards established by AASHO, and some European standards which could be applied to Iraqi conditions have been taken into consideration in the preparation of this manual.

The design criteria contained in the manual generally represent minimum acceptable values and should not be considered as inflexible rules. Engineering judgment should be applied in selecting higher design parameters when not restricted by local or economic conditions.

This first printed edition will be improved and supplemented in order to comply with the latest findings and developments in modern highway design. Therefore any suggestions by the users will be appreciated. They shall be given to the head of the Road & Traffic Division of SORB.

PART I

TRAFFIC AS A DESIGN CONTROL

PART I TRAFFIC AS A DESIGN CONTROL

PART I TRAFFIC AS A DESIGN CONTROL

LIST OF TABLES

PART I TRAFFIC AS A DESIGN CONTROL

LIST OF FIGURES

1 GENERAL

Traffic demands govern the various design elements of highways:

CURRENT AND COMPREHENSIVE TRAFFIC DATA is the basis for thorough engineering analyses

FUTURE VOLUMES indicate the purpose for which a highway is being built and largely determine the type of highway needed

TYPE OF TRAFFIC is essential for determining the geometric design features

The traffic flow consists mainly of units of man-machine combinations of driver and vehicle, and/or pedestrians. These units, when using the highway facilities, develop characteristics of speed, volume, origin-destination, traffic flows in the network and at intersections and accident rates. All of these may be influenced by regulatory measures, control devices, physical design, road conditions, and the environment.

A full understanding of traffic elements and characteristics leads to the effective planning and design of highways.

ELEMENTS OF EVERY TRAFFIC PROBLEM

HUMAN (DRIVERS AND PEDESTRIANS)	VEHICLE (CARS, TRUCKS, BUSES, ETC)	ROADS (HIGHWAYS, STREETS, TERMINALS)
PHYSICAL LIMITATION	DESIGN LIMITATION	ROAD DESIGN FEATURES
Vision Judgement Reactions Direction Fatigue	Size, Weight, Power Visibility Safety Equipment	Alignment, Grade Sight Distance Width, Access Intersections Capacity Hazards
FAULTY ACTIONS Violations Bad Practices	PERFORMANCE Speed, Acceleration Deceleration Turning Paths	TERMINAL FACILITIES Loading, Parking
PERSONAL DESIRES	MAINTENANCE	CONTROL DEVICES

2 THE PEDESTRIAN

2.1 DEFINITION

A pedestrian is defined as "any person on foot". His involvement in traffic is a major consideration in many situations.

2.2 CHARACTERISTICS

General characteristics of the pedestrian, which should be taken into account in the design of roads and footways:

(1). Pedestrians' actions are less predictable than those of drivers since many pedestrians consider themselves "outside the law" in traffic matters.
(2). Pedestrians tend to follow a path that represents the shortest distance between two points.
(3). Pedestrians have a natural reluctance to climbing steps or ramps when crossing roadways and tend to avoid special pedestrian underpasses or overpasses.
(4). Pedestrians are more apt to disobey traffic control devices than drivers.
(5). An important factor is age. Very young pedestrians are more often careless in traffic because of ignorance, whereas the elderly appear to be inattentive or defiant toward motor vehicles and drivers.

2.3 WALKING SPEED

Pedestrians vary their speed mainly according to
- type of crowd
- density of pedestrian flow
- kind of footway
- gradient

Relationships between pedestrian walking speeds and some of other features are shown in Figures I–1 to I–3
Average walking speeds range approximately
from 0.75 to 1.80 meters per second, i.e.
from 2.70 to 6.50 kilometers per hour.
For timing of pedestrian signals, a pedestrian walking speed of 1.2 meters per second (4.4 kilometers per hour) is recommended.

2 THE PEDESTRIAN

FIGURE I-1

Relationship between Pedestrian Walking Speeds and Volumes and their Variation by Time of Day in City Centres

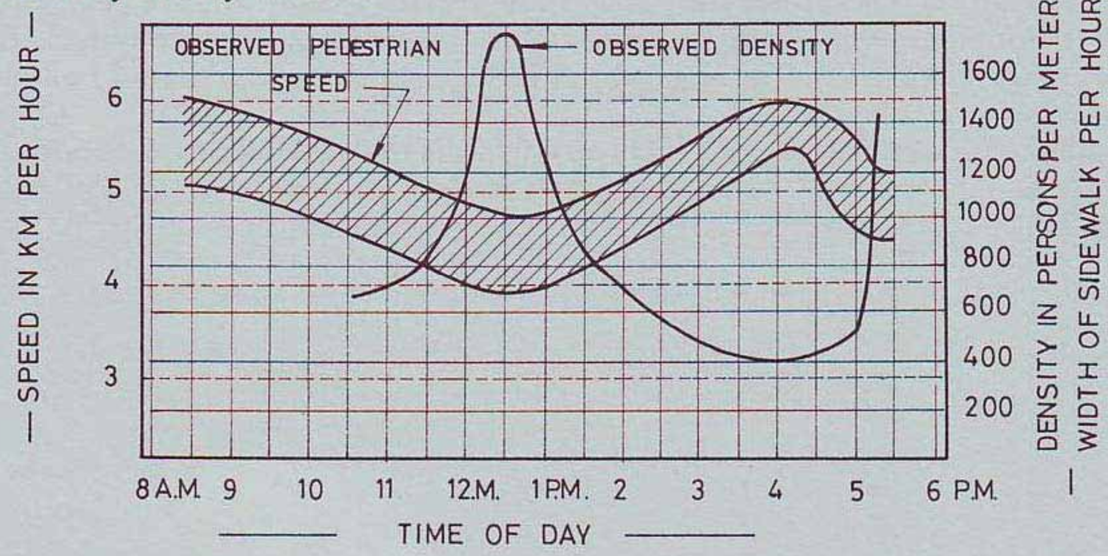


FIGURE I-2 Variation in Pedestrian Speeds when Crossing Streets

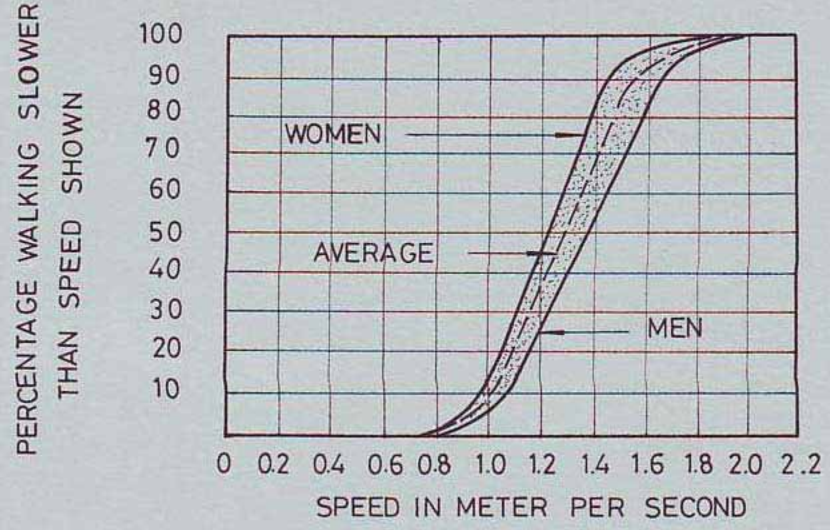
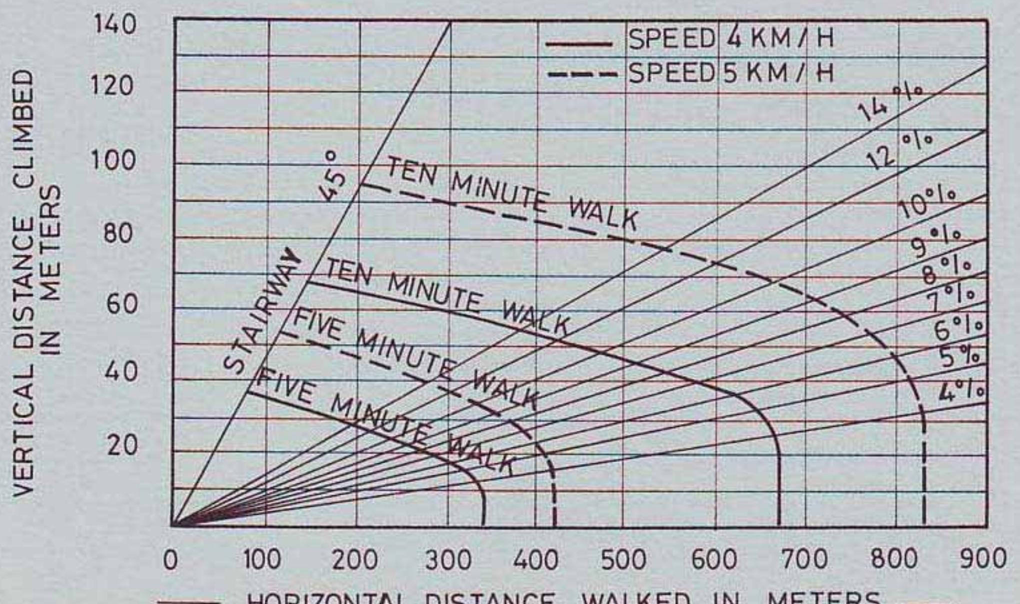


FIGURE I-3 Pedestrian Speeds Decreased by Vertical Grades



2 THE PEDESTRIAN

2.4 FOOTWAY CAPACITY

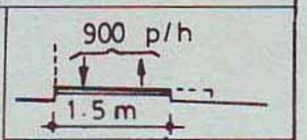
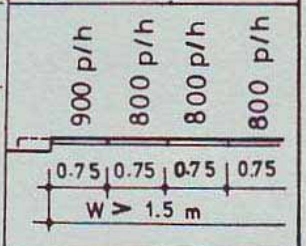
Pedestrian movement patterns are very flexible, and can easily adjust to different space conditions. Therefore, the pedestrian "capacity" controls are very flexible. At peak hours (crowd leaving sporting events, after religious ceremonies, etc.) the density of pedestrian flows can reach about 3000 persons per hour per 0.75 m. wide lane.

As a general design rule, AASTHO recommends that the capacity of footways be 12 persons per foot width of footway per minute after deducting 1.5 feet "dead width" (3.00 feet in shopping areas) from the total width.

Considering full pedestrian comfort and convenience the design capacity presented in Tables I–1 and I–2 may be taken.

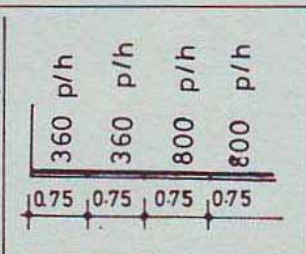
Design Capacities of Footways, Pedestrian Subways and Foot Bridges

TABLE I-1

TYPE OF FOOTWAY	DESIGN CAPACITY PER LANE	
Two-lane, two-way footway of 1.5 m width	900 persons per hour	
First lane of 0.75 width of footways wider than 1.5 m	900 persons per hour	
Second and further lanes of 0.75 m width of footways wider than 1.5 m	800 persons per hour	

Design Capacities of Footways adjoining Shopping Frontage, Large Public Buildings, etc.

TABLE I-2

TYPE OF FOOTWAY	DESIGN CAPACITY PER LANE	
First and second lane of 0.75 m width adjoining frontage	360 persons per hour	
Further lanes of 0.75 m width	800 persons per hour	

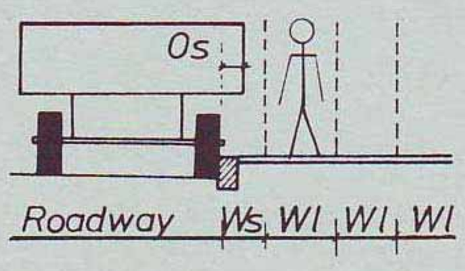
2 THE PEDESTRIAN

2.5 WIDTH CONSIDERATIONS

The factors to be considered in determining footway widths are:

(1) Capacity Controls
These controls determine the number of pedestrian traffic lanes of 0.75 m. standard width required for anticipated pedestrian flows.

(2) Safety Measures
Footways next to roadways should be provided with a safety strip to avoid interference of pedestrians with vehicles moving close to a curbline. Desirable width of the safety strip is 0.5 m.



Ws – width of safety strip (0.5 m.)
Wl – width of pedestrian traffic lane (0.75 m.)
Os – side overhang

(3) Space for Road Equipment
Additional space may be required for the installation of traffic control devices, roadside furniture (lighting, telephones, safety fences, shelters, etc.) and utilities.

(4) Site and Right-of-way Conditions
Since the pedestrian "capacity" controls are flexible, decisions concerning the footway widths are mainly governed by the site and right-of way conditions.

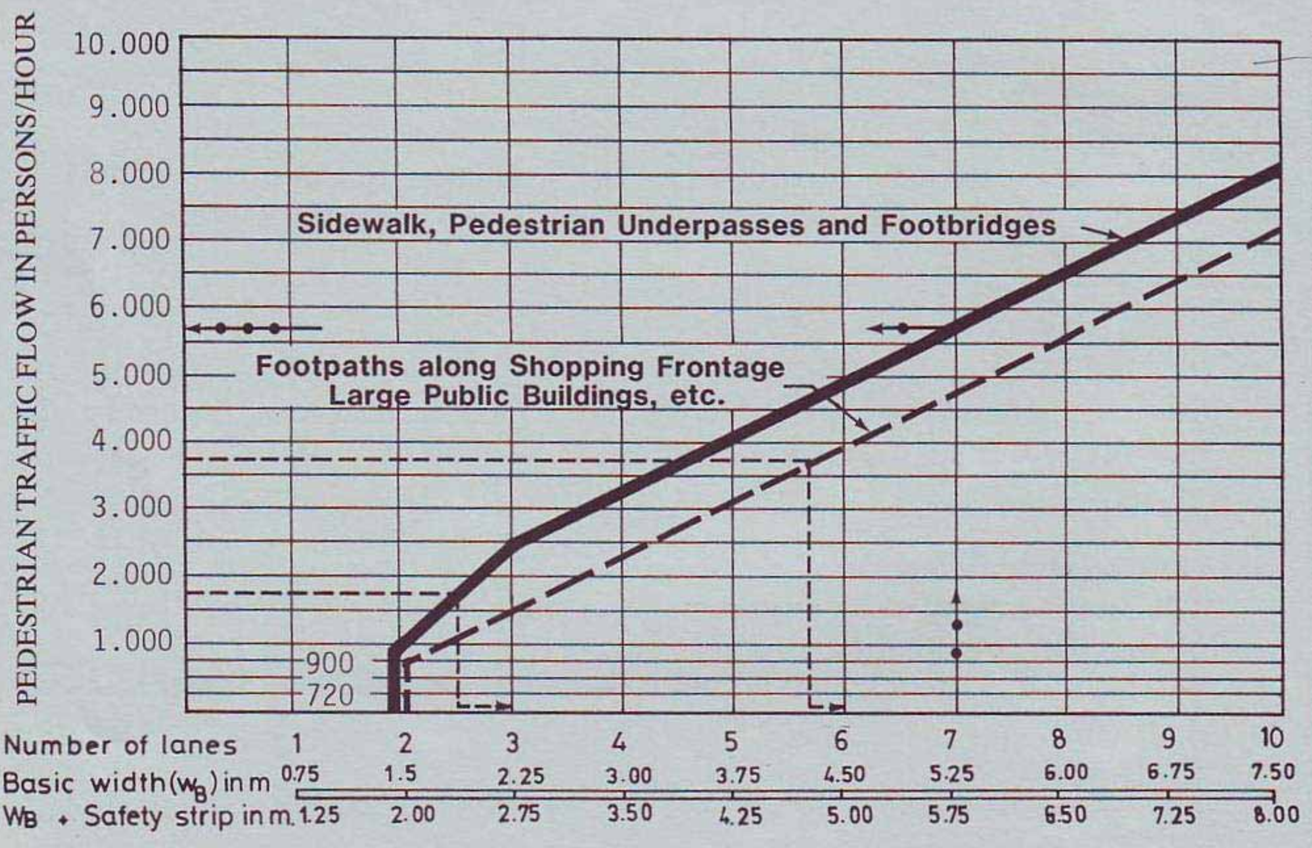
(5) General Requirements
Minimum footway width requirements as stated in Table I–3 should be observed in any situation.
An overall aim should be to keep footway widths as wide as practicable.

Recommended Widths of Footways — TABLE I–3

Number of unit lanes of 0.75 m.	Basic width in m. (nx0.75 m.)	Width including safety strip (of 0.5 m.) in m.	Desirable minimum width recommended for
1	0.75	1.25	Footways for maintenance purposes (bridges, etc.)
2	1.50	2.00	Rural areas, residential areas
3	2.25	2.75	Business and principal industrial areas
4	3.00	3.50	Pedestrian crossings, subways, footbridges
5	3.75	4.25	
6	4.50	5.00	
7	5.25	5.75	
8	6.00	6.50	
9	6.75	7.25	
10	7.50	8.00	

NOTES:
1-Number of lanes should satisfy capacity control and site and right-of-way conditions.
2-Recommended widths may be increased by additional strips for roadside equipment and utilities.

FIGURE I–4 Design Chart for Footway Width



2 THE PEDESTRIAN

2.6 PEDESTRIAN FOOTWAYS

Adequate provisions for the movement and protection of pedestrians should be made in any highway project.

(1) Paved Shoulders
Paved shoulders can be considered as pedestrian footways if:
– the width of shoulders is at least 1.25 m.
– the design speed of the road is not higher than 80 km/h.
– the colour and texture of shoulders are different from those on the through-traffic lanes or if pavement egde lines provide for shoulder delineation.
– the pedestrian flow does not exceed the volumes shown in Table I–4.

Maximum Pedestrian Volumes Allowed on Shoulders — TABLE I–4

ROAD SECTION	DESIGN SPEED OF ROAD IN km/h						
	80	70	60	50	40	30	15
	PEDESTRIAN VOLUME PER HOUR						
RURAL & SUBURBAN	189	218	257	313	400	553	900
VICINITY OF BUS STOP	9	11	13	16	20	28	65

However, footways may be also justified if there are certain hazards (protection of children, elderly persons, etc.), even if the pedestrian volumes are less than those shown in Table I–4.

(2) Footways at the Roadside
Footways along the edge of highway may be adopted if the right-of-way is restricted or where the cross section layout considerably influences the costs, e.g. on or under bridges, in tunnels, etc.

(a) Width:
Control factors for widths are stated in the chapter 2.5 "Width Considerations".

(b) Cross Slope:
Minimum cross slope for good drainage: 2%
Maximum cross slope still providing comfortable walking: 3%

(c) Grade
Grades should be limited to: 9%
Exceptional maximum grade: 12%

(d) Separation of Pedestrians and Vehicles
To prevent vehicles from leaving the pavement and to protect pedestrians, adequate separation should be provided:
Curbs –
Flush curbs – They should be used where design speed does not exceed 20 km/h

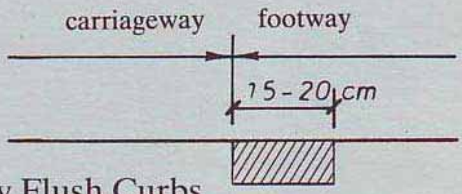


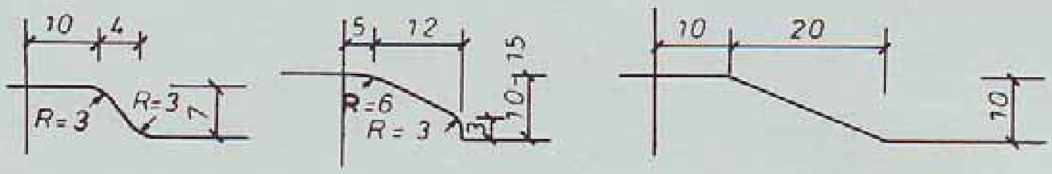
FIGURE I–5A Typical Highway Flush Curbs

2 THE PEDESTRIAN

Mountable curbs – They are designed where vehicles can cross them. They have rounded and sloping face. The height of these kind of curbs should not exceed 15 cm.

FIGURE I–5B Typical Highway Mountable Curbs

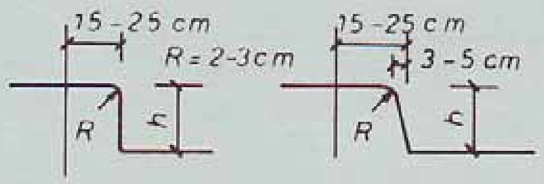
Dimensions are in cm



Barrier curbs – They should inhibit, or at least discourage, vehicles from leaving the roadway. They are 15 to 20 cm high and steep faced.

FIGURE I–6 Typical Highway Barrier Curbs

h	minimum	desirable	maximum
	12 cm	15 cm	20 cm

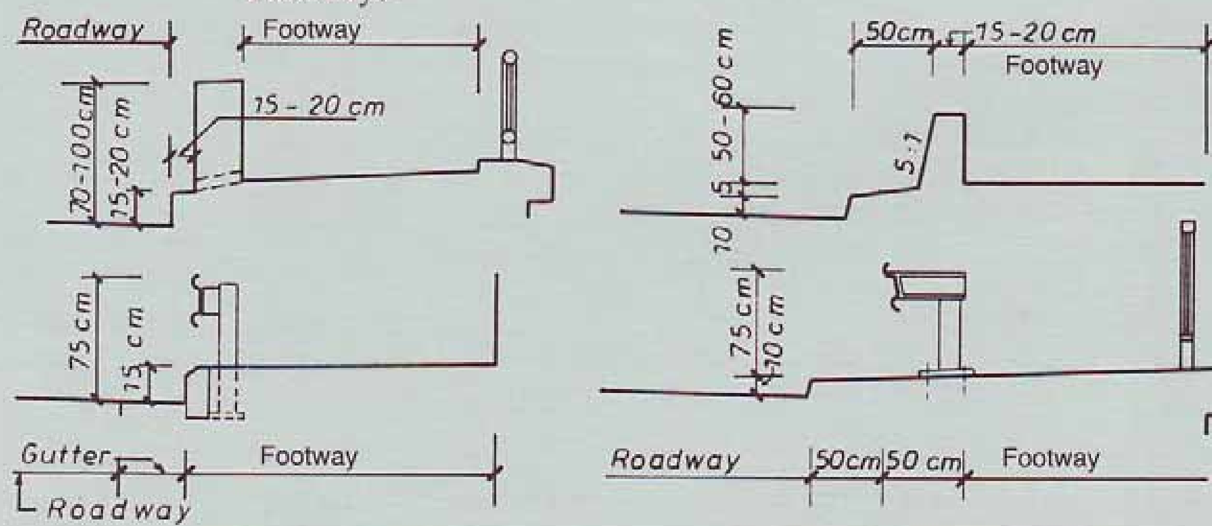


– Barrier curbs should not be used where the design speed is above 80 km/h except where they are provided with guardrails.

Barriers and guardrails

The positive protection of pedestrians is required on high-speed roads, along the faces of long walls, in tunnels and on bridges. Barriers and guardrails provide "safe walking areas" reducing the danger to persons walking from disabled vehicles.

FIGURE I–7 Typical Arrangement of Concrete Barrier and of Guardrails on Footways

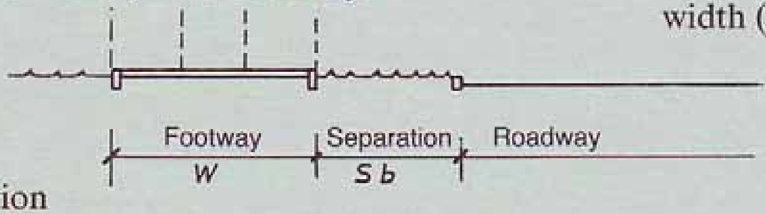


2 THE PEDESTRIAN

(3) Separate Footways
Footways set back from the roadway are strongly recommended since they increase the desirable separation of vehicles and pedestrians.

FIGURE I–8 Separate Footways

width (w) = n x 0.75 m



Separation
2.0 m separation is the desirable minimum
1.0 m separation is the minimum if space is restricted
Other design considerations of separate footways are the same as those for attached footways.

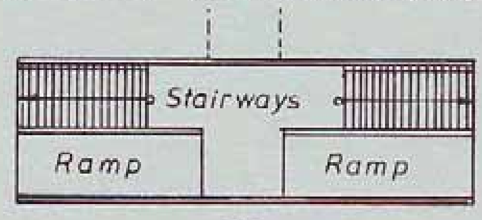
(4) At-grade Pedestrian Crossings
Pedestrians who cross the roadway at random are in far more danger than those who do so at prescribed crossing places. Therefore it is desirable to clearly delineate location where pedestrians are to cross the roadway.

General Requirements
– The crossings should be clearly recognisable by drivers.
– The crossing location should not substantially increase the distance pedestrians must walk.
– The crossing should cross the roadway at right angles, whenever possible.
– The crossing must be placed at safe points, e.g. at places without restricted visibility, with low running speed, etc.
– The crossing should be provided with protective and control devices such as markings, lighting, refuge islands, barriers or fences, and traffic signals.

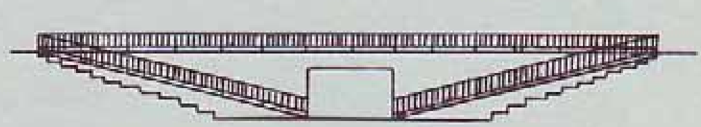
WIDTH
The width of crossings should accommodate pedestrian volumes. For capacity controls see the chapter 2.5 "Width Considerations".
The desirable minimum width of a crossing is 3.0 m. The crossing should be at least as wide as the footways they connect.

(5) Stairways and Ramps
Outdoor stairways and ramps provide access to pedestrian underpasses and footbridges. Stairways are also used at places where footways are very steep. Ramps are generally preferred by users, and are safer; however, they take more ground space than stairways.
Where possible, busy pedestrian footbridges and underpasses should have ramped approaches as well as stairways. For example see Figure I–9.

FIGURE I–9 Stairway Ramp Approach to Pedestrian Underpass



Plan



Elevation

2 THE PEDESTRIAN

(a) Stairways

Outdoor stairways are used for:

– Access to pedestrian underpasses and footbridges where their design is generally governed by building practice, and

– Footways having slopes equal to or steeper than 14%.

(i) Width

The width of stairway should accommodate the pedestrian volume. It should be in units of 0.6 m. The capacity of stairways can be assumed to be the value of 30 persons per minute per traffic lane of 0.6 m width.

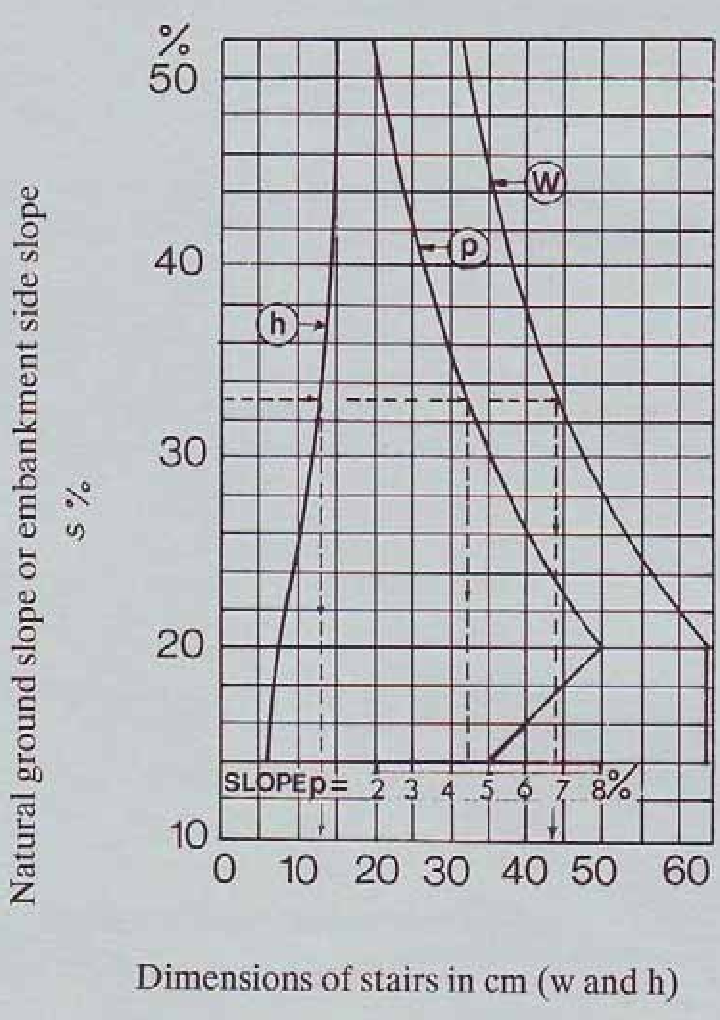
(ii) Stair Design

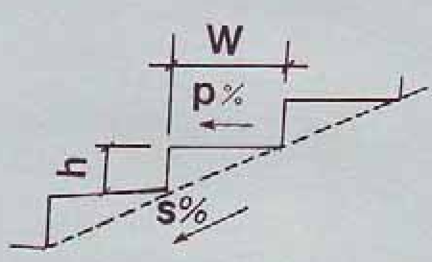
Recommended standards for the stairs to pedestrian underpasses and footbridges are:

Width of step:	28–32 cm
Height of step:	14–16 cm
Landing:	for every 1.5–1.8 m change in elevation
Handrail:	to be provided in all cases – height = 1.1 m

Stair design for footways steeper than 14% is shown in Fig I–10
They should be provided with handrails of 1.1 m height.

FIGURE I–10 Design Chart for Outdoor Footway Staircases





Desirable Design Data:

Width (w)	36 to 46 cm
Height (h)	6 to 16 cm
Slope (p)	2 to 8%

2 THE PEDESTRIAN

(b) Ramps
In general, the design of outdoor ramps is governed by the same considerations as those for footways.

(i) Width
The width of ramps should be in units of 0.75. The number of units should accommodate pedestrian volumes. For details see paragraph 2.5 "Width Considerations".
(ii) Slope
Continuous ramps should not be steeper than 10%. Desirable maximum gradient for ramps is 15%.
(ii) Handrail
Ramps should be provided with handrails when the slope is greater than 8%.

(6) Separation of Vehicular and Pedestrian Traffic

Adequate separation to ensure complete segregation of pedestrians and vehicles is required:
- at very busy intersections;
- at points where pedestrians need to cross the road in large numbers;
- where abnormal hazards or inconveniences to pedestrians would result otherwise;
- where the natural topography and other conditions are favourable.

(a) General Design Considerations

(i) Convenience to pedestrians is an important design aspect, considering the pedestrian's natural aversion to walking up or down when crossing roadways.
(ii) Appropriate arrangements need to be made for drainage, lighting and approach walkways.[1)]
(iii) structures should be attractive in appearance.

(b) Width Requirements
Footways for pedestrians should have a minimum width of 3.0 m and clear height of 2.5 m.

(c) Footbridges
Footbridges are generally cheaper than underpasses. The design of drainage facilities should ensure that no water falls on to the vehicle below. The problem of objects falling into the path of traffic moving under the structure should be considered and protective screens be used if deemed necessary.

(d) Underpasses
Underpasses demand special consideration regarding drainage, lighting and, on lengthy underpasses, ventilation. Conditions are often complicated by underground utility ducts, sewers, water mains, etc.

Note [1)] To limit the space required by staircases or ramps, a more compact arrangement may be achieved by reversing the direction or by a winding alignment.

3 THE VEHICLE

3.1 GENERAL

The traffic unit which is of primary concern is the vehicle. Hence, a thorough knowlege of vehicle characteristics is essential for the design of a highway.

3.2 VEHICLE CHARACTERISTICS

The physical characteristics of vehicles and the proportions of various size vehicles using the highway are important control factors in highway design. To obtain such control factors, all vehicles are grouped in general classes, and representative types within each class are selected as the "design vehicle".

The most important vehicle features for highway design purposes are:
- dimensions and minimum turning radius as a control for the geometric design;
- weight and axle type as a control for pavement design;
- operating characteristics as a control for the geometric design and traffic safety.

The maximum size and weight of vehicles is limited by the state or higway authorities to suit both the transportation services needed and the highway network conditions.

The limits of the main vehicle characteristics valid in Iraq are:
- Maximum allowable width (including load) 2.60 m
- Maximum allowable height (including load) 4.10 m

- Maximum allowable length:

Single truck or bus	12.00 m
Combination of truck-tractor and semitrailer	16.50 m
Combination of truck and trailer	20.00 m

- Maximum axle load:

Front single axle	6.0 metric tons
Rear single axle	12.0 metric tons
Tandem axle	18.0 metric tons
Group of three axles	25.5 metric tons

- Maximum tyre pressure $6.68\ kg/cm^2$ (95 psi)

3.3 DESIGN VEHICLE FOR GEOMETRIC DESIGN

A "design vehicle" is a selected motor vehicle which is used for the determination of design controls.
For the purpose of geometric design the general class groups are:

Passenger cars	P
Single unit trucks and small buses	SU
Large buses	BUS
Truck combinations	WB

3 THE VEHICLE

Design vehicle dimensions and minimum turning paths according to AASHTO are shown in Figures I–11 to I–16. The class of truck combinations is designated by the length of wheelbase in feet. For instance the WB-40 design vehicle has an overall wheelbase of 40 feet.

FIGURE I–11 Minimum Turning Path for P Design Vehicle

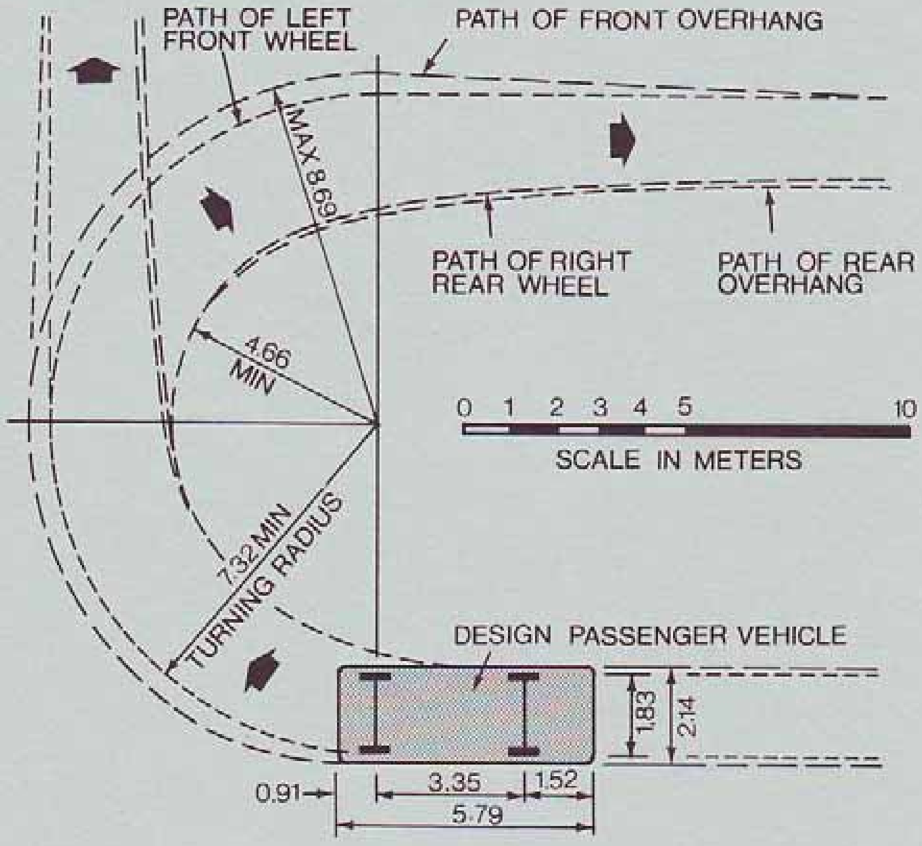
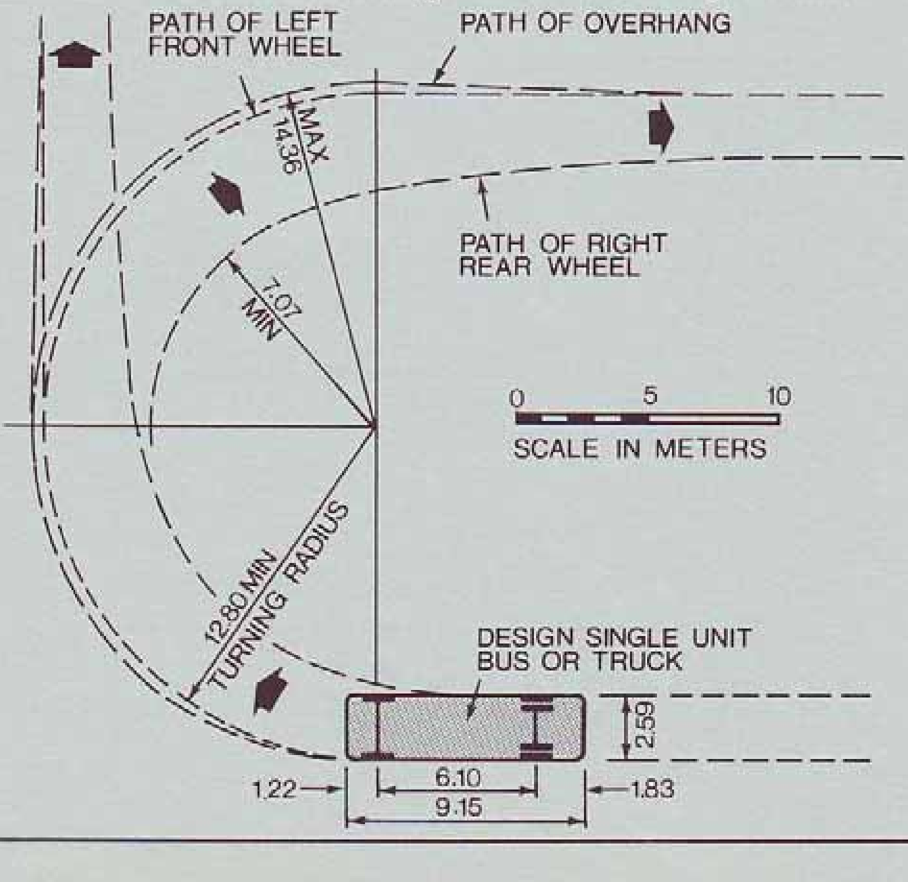


FIGURE I–12 Minimum Turning Path for SU Design Vehicle



3 THE VEHICLE

FIGURE I–13 Minimum Turning Path for BUS Design Vehicle

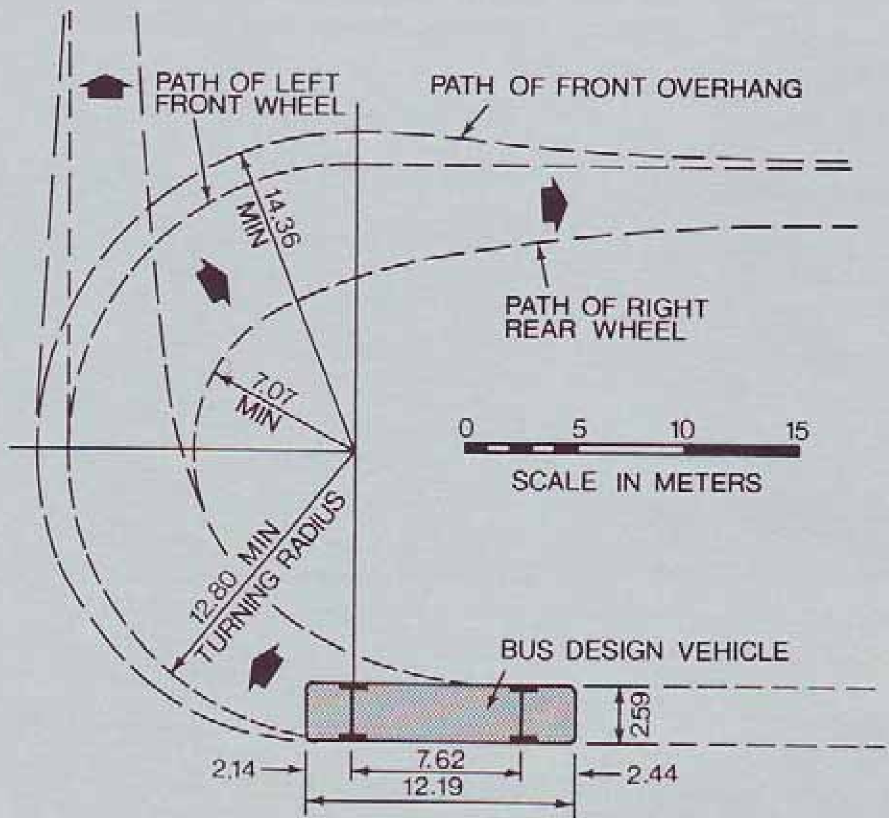
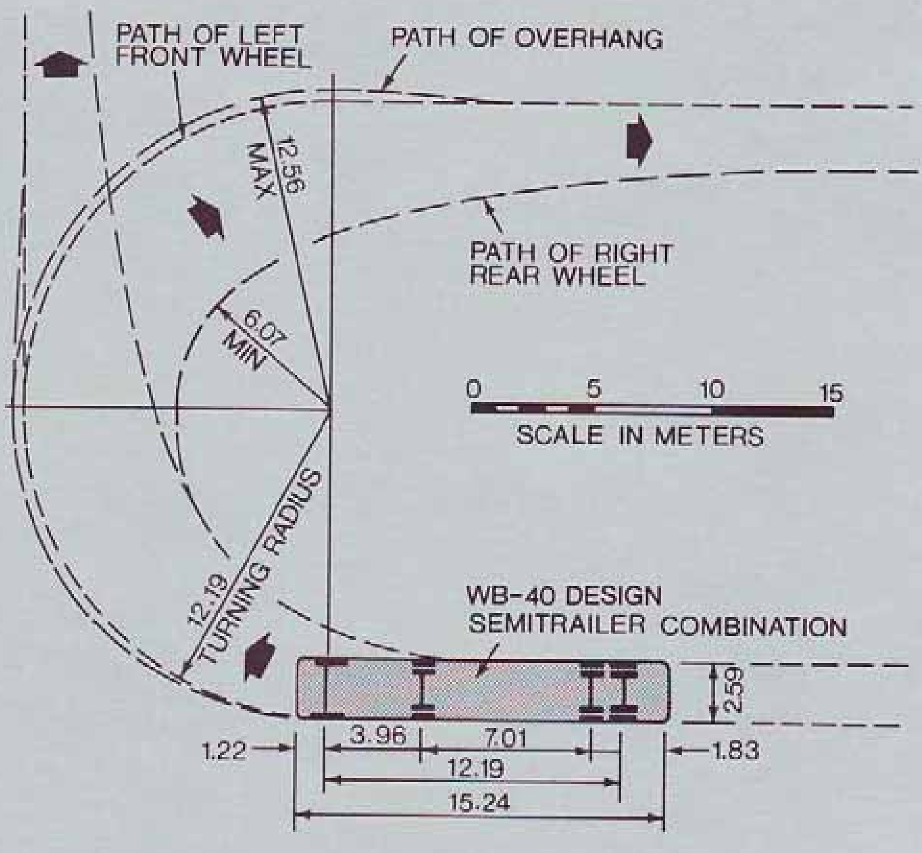


FIGURE I–14 Minimum Turning Path for WB-40 Design Vehicle



3 THE VEHICLE

FIGURE I–15 Minimum Turning Path for WB-50 Design Vehicle

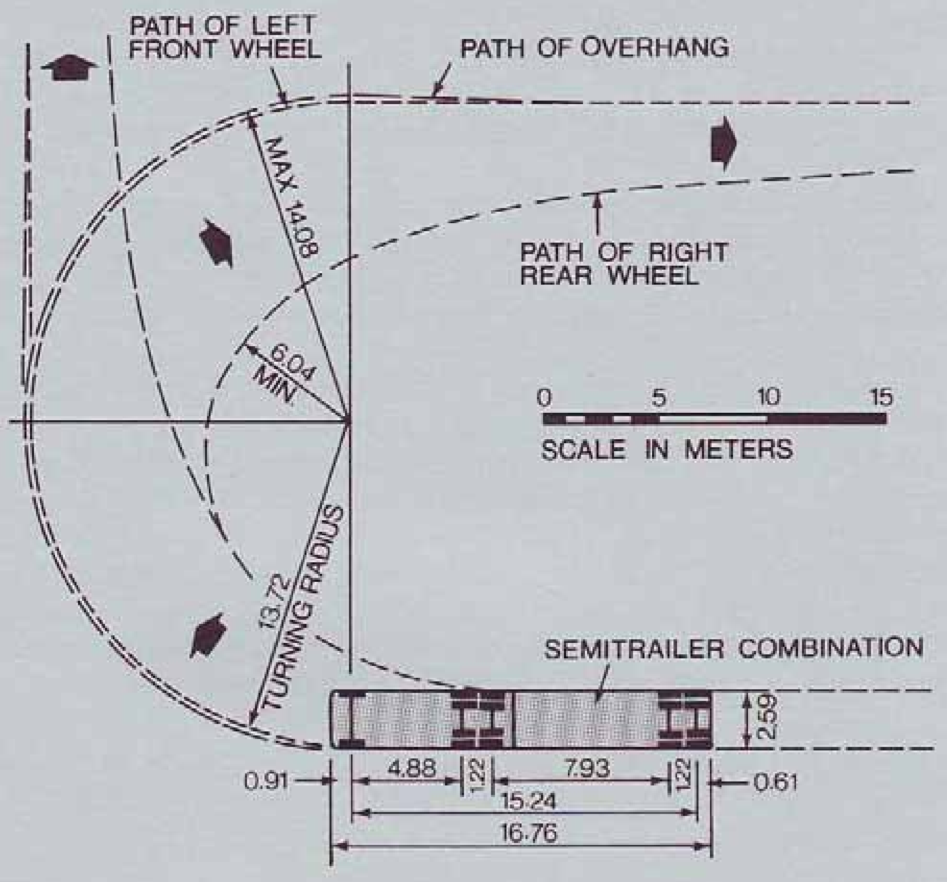
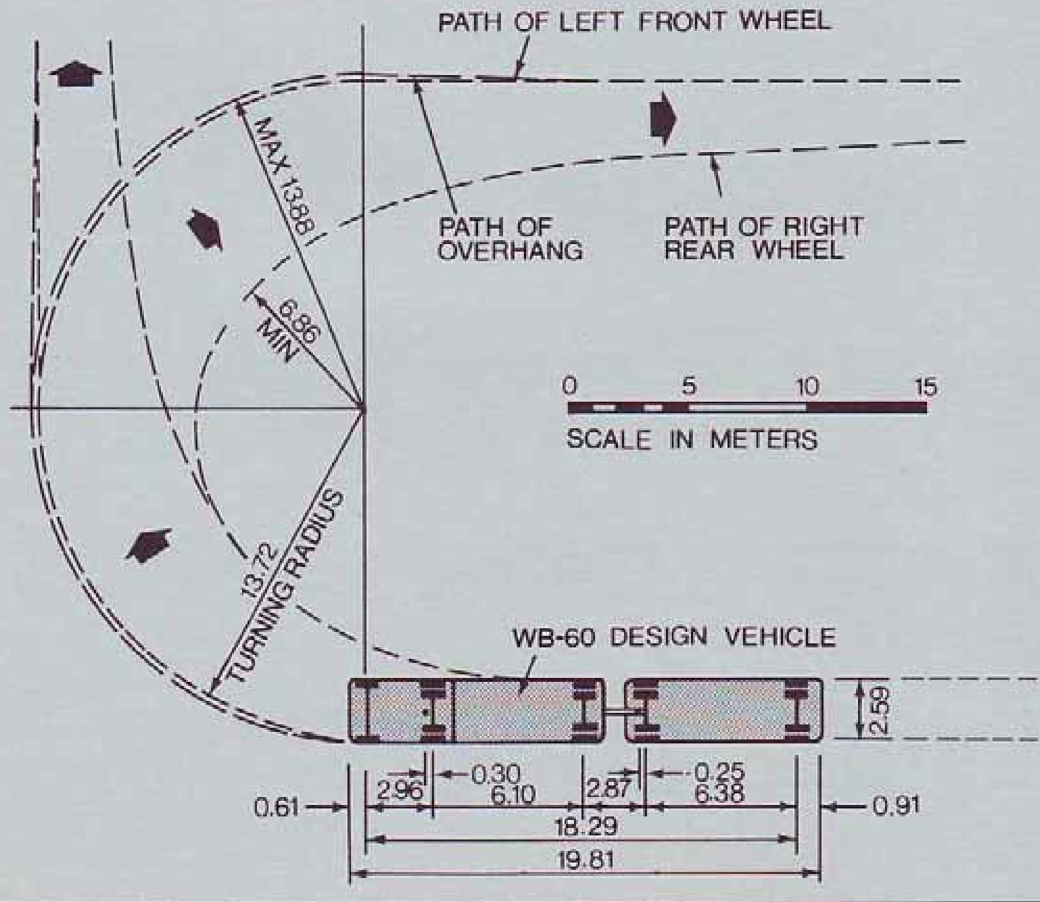


FIGURE I–16 Minimum Turning Path for WB-60 Design Vehicle



3 THE VEHICLE

3.4 LOADS

The live highway loads that generally affect pavement design consist of a combination of single-unit and multiple-unit vehicles. The axles are either single or tandem, and are provided with single or dual tyres. For pavement design the knowledge of vehicle axle arrangements (spacings and loads) is essential.

The loads produced by movement of passenger cars are small compared with those of trucks. The loads produced by heavy trucks are critical to pavement design and thus will be of primary concern. There are different combinations for commercial vehicles, with corresponding different axle arrangements and weights.

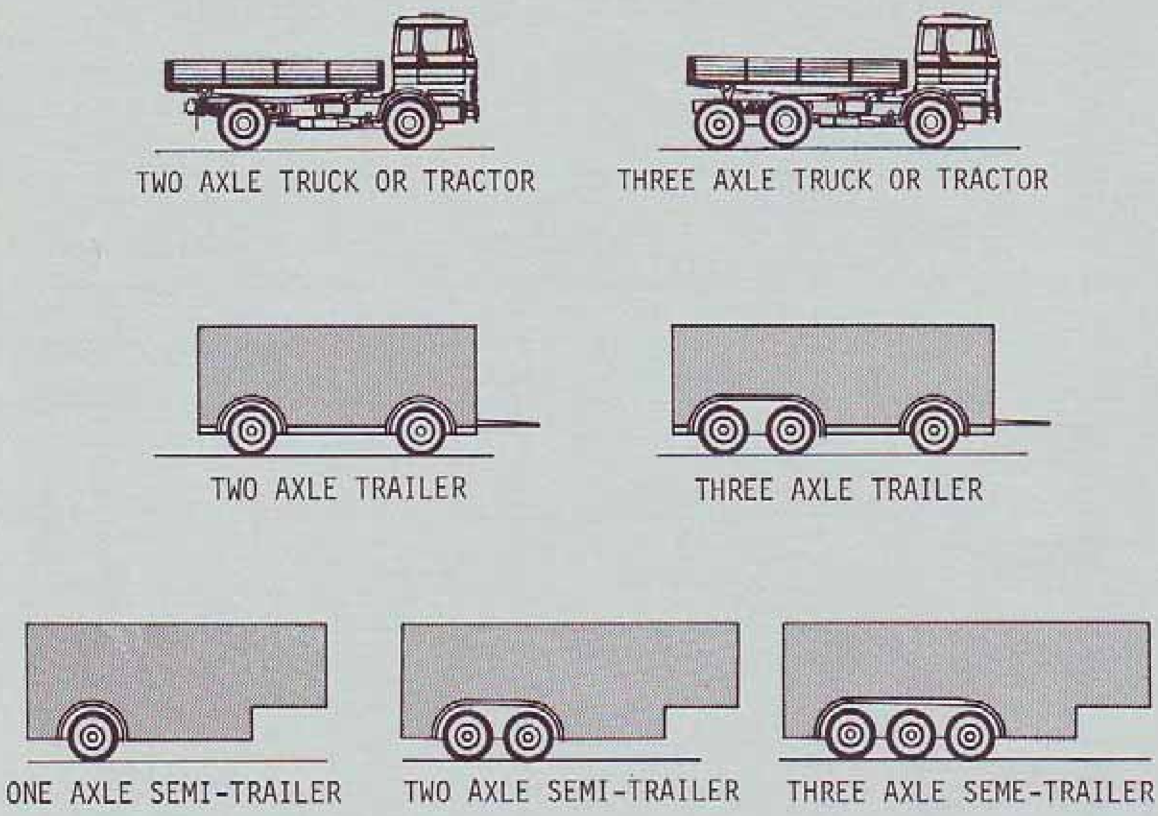
The classification of most basic commercial vehicle types in regular operation, designated by code based on axle arrangement, is shown in Figure I–17. The first digit indicates the number of axles of the truck or truck-tractor. The letter "S" indicates a semitrailer, and the digit immediately following an "S" indicates the number of axles of 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 example, 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.

The maximum allowable axle-loads, gross weights and dimensions, including a load for commercial vehicles in regular operation in Iraq, are shown in Table I–5 and in Figures I–18a and I–18b.

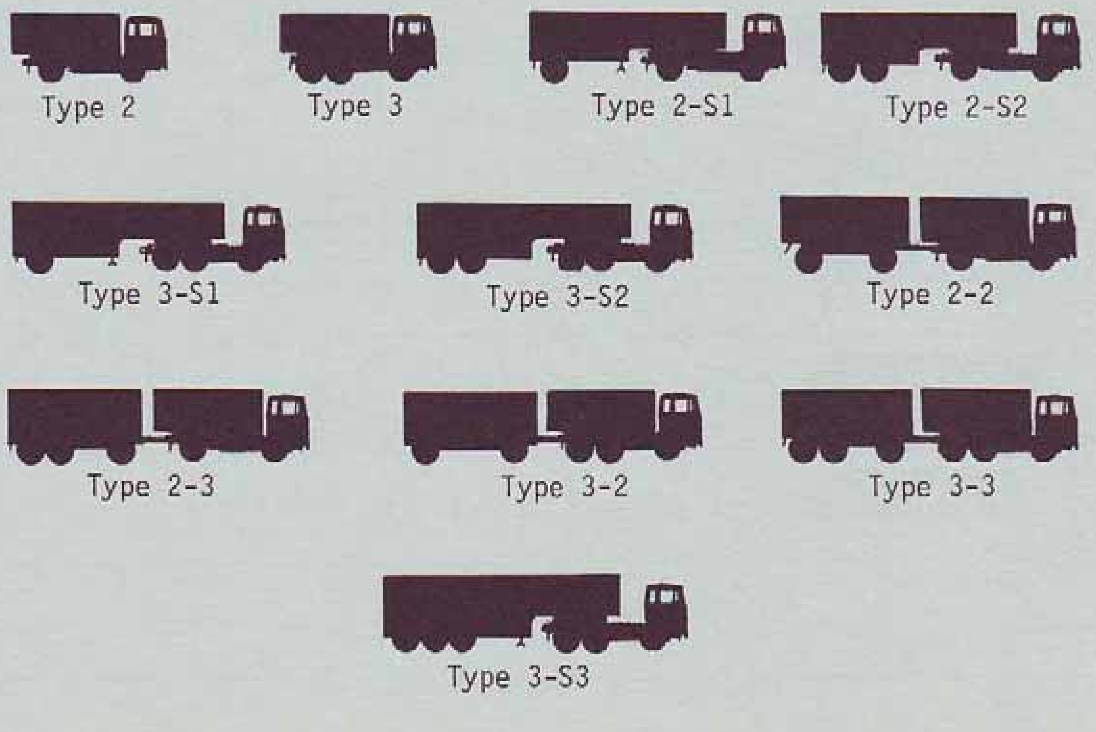
3 THE VEHICLE

FIGURE I–17 Commercial Vehicle Types

Basic Units of Vehicle Types



Silhouettes of most basic Commercial Vehicle Types



3 THE VEHICLE

TABLE I–5

Legal Limits of Commercial Vehicle Weights and Sizes for Iraq

CLASS	VEHICLE SHAPE (SILHOUETTE) AXLE NUMBER	MAXIMUM AXLE LOAD IN METRIC TONS							MAXIMUM VEHICLE DIMENSIONS IN M		
		AXLE						GROSS WEIGHT	LENGTH	WIDTH	HEIGHT
		1	2	3	4	5	6				
A	 2 1	6	12					18	12.00	2.60	4.10
B	 3 2 1	6	← 18 →					24			
C	 3 2 1	6	12	12				30	16.50	2.60	4.10
D	 4 3 2 1	6	12	← 18 →				36			
E	 4 3 2 1	6	← 18 →		12			36			
F	 5 4 3 2 1	6	← 18 →		← 18 →			42			
G	 4 3 2 1	6	12	12	12			42	20.00	2.60	4.10
H	 5 4 3 2 1	6	12	12	← 18 →			48			
I	 5 4 3 2 1	6	← 18 →		12	12		48			
J	 6 5 4 3 2 1	6	← 18 →		12	← 18 →		54			
K	OTHERS										

3 THE VEHICLE

FIGURE I–18a

Legal Axle and Gross Weights Permitted on Motor Vehicles in Regular Operation in Iraq

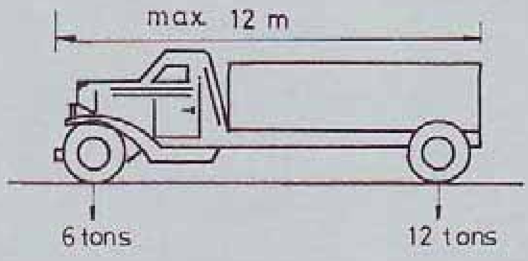
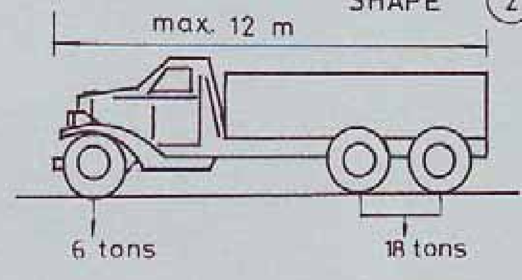
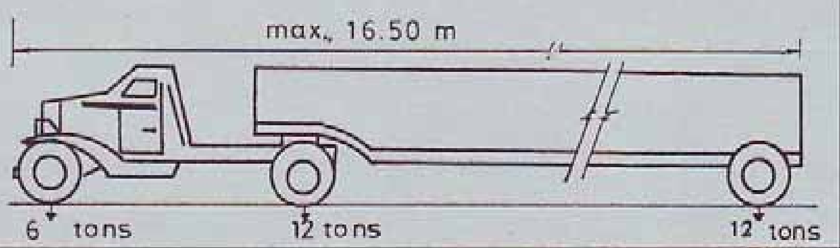
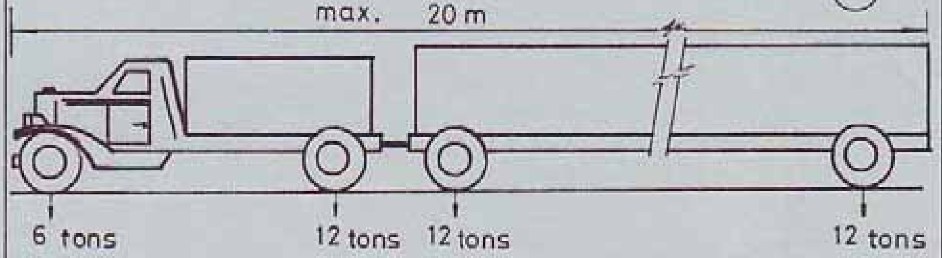
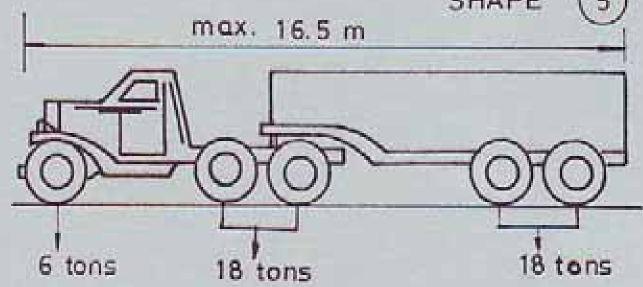
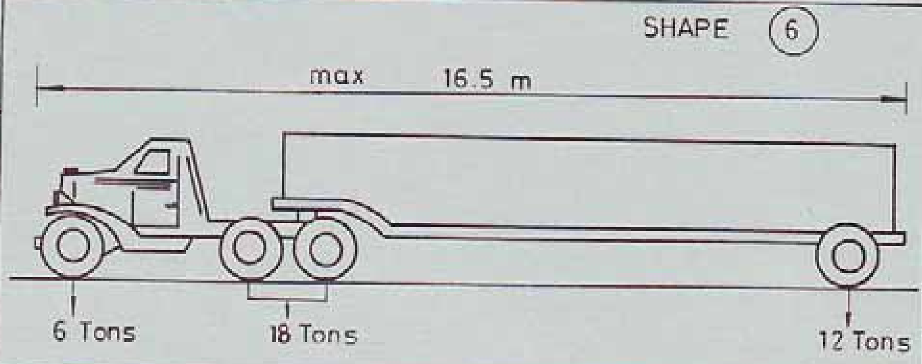
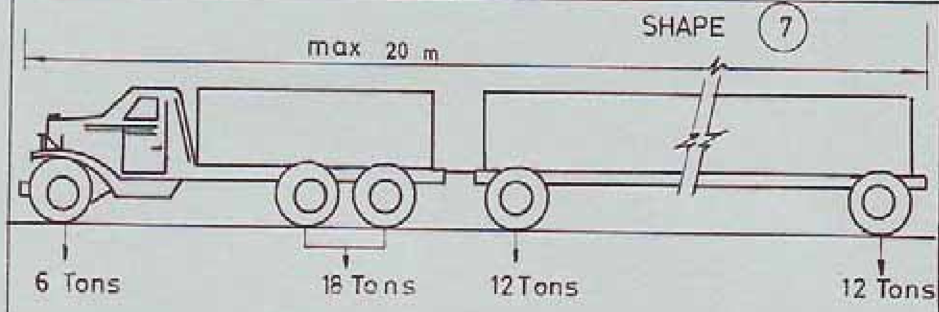
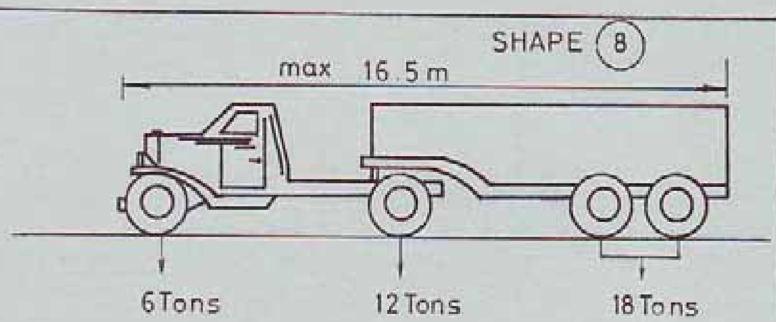
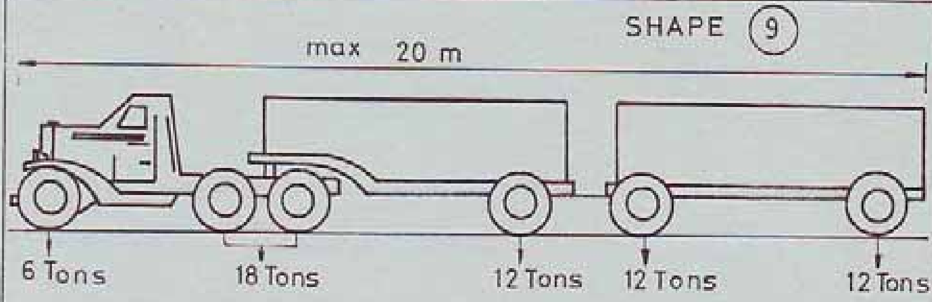
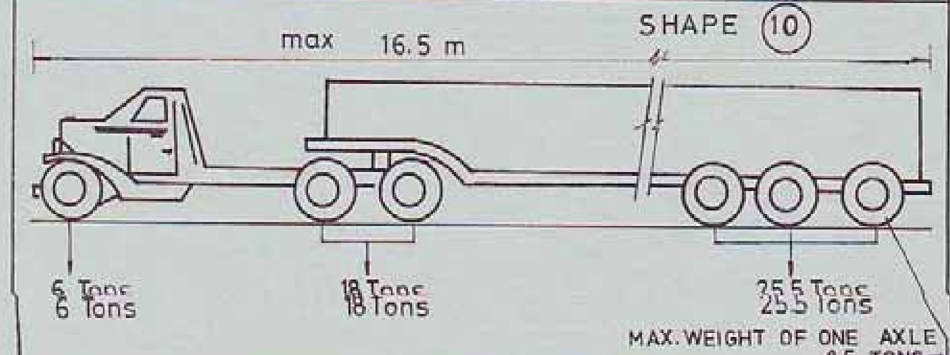
MAXIMUM GROSS WEIGHT	VEHICLE TYPE	SHAPE
18 tons	type 2	SHAPE (1) 
24 tons	type 3	SHAPE (2) 
30 tons	type 2 _ S1	SHAPE (3) 
42 tons	type 2 _ 2	SHAPE (4) 
42 tons	type 3 _ S2	SHAPE (5) 

FIGURE I–18b
Legal Axle and Gross Weights Permitted on Motor Vehicles in Regular Operation in Iraq

MAXIMUM GROSS WEIGHT	VEHICLE TYPE	
36 tons	type 3-S1	
48 tons	type 3_2	
36 tons	type 2_S2	
60 tons	type 3-S1-2	
49.5 tons	type 3_S3	

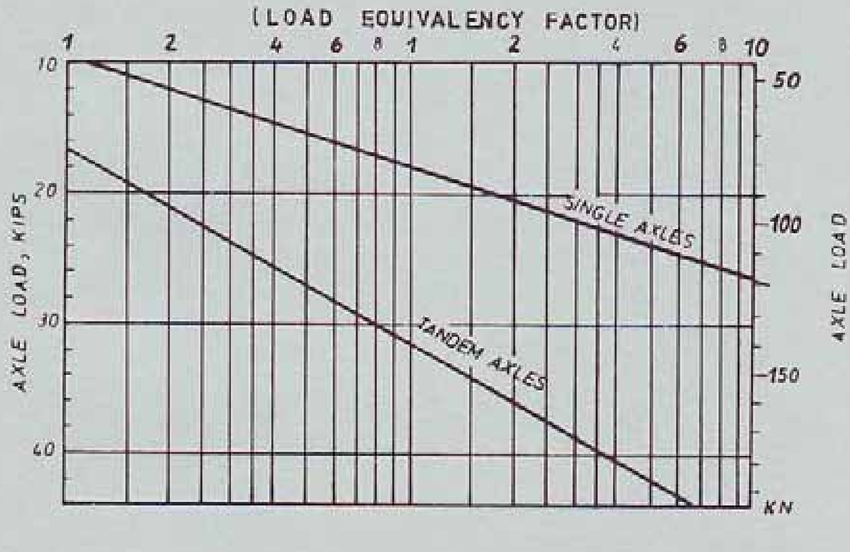
3 THE VEHICLE

The great variety of traffic using highways makes it necessary to replace the traffic loads by a simple loading system that can easily be used in pavement design. In many design procedures a standard load is chosen for the design.

The standard load commonly used in North America is the 18.000 lb (8.170 kg) single-axle load with dual wheels on each side.

Values of the axle and wheel loads in European countries used for pavement design are given in Table I–6.
The numbers of actual axle loads derived from traffic studies must be converted to numbers of standard single axle load applications by use of respective empirical conversion factors, for example, as shown in Fig. I–19 below.
The effect on pavement performance of one application of the given axle load is expressed as the number of standard single-axle load applications. The load equivalency factors for flexible pavements are different from those for rigid pavements. Examples of these factors for flexible pavements are given below:

FIGURE I–19 Load Equivalency Factors for Flexible Pavements (According to Asphalt Institute Manual MS1)



3 THE VEHICLE

Design Loads for Roads in Europe

TABLE I–6

Country	Freeways & Expressways		Main Highways		Secondary Highways	
	Design traffic (commercial vehicles/day)	Design axle or wheel load kip (ton)	Design traffic (commercial vehicles/day)	Design axle or wheel load kip (ton)	Design traffic (commercial vehicles/day)	Design axle or wheel load kip (ton)
Austria	N	Single axle 22.5 (10) Tandem axle 36 (16)	N	Single axle 22.5 (10) Tandem axle 36 (16)		
Belgium	N	Axle load 29.25 (13)	No design	Axle load 29.25 (13)	No design	Axle load
Czechoslovakia	1000	Single axle 22.5 (10)	500–1000	Single axle 22.5 (10)	100–500	Single axle 22.5 (10)
Denmark	>(4000 sum in both directions)	Single axle 18 (8)				
France		Single axle 29.25 (13) Tandem axle 45 (20)		Single axle 29.25 (13) Tandem axle 45 (20)		
Great Britain	Design based on total commercial vehicles carried during total life	Single axle 22.5 (10)	Design based on total commercial vehicles carried during total life	Single axle 22.5 (10)	Design based on total commercial vehicles carried during total life	Single axle 22.5 (10)
Netherlands	Approx. 3000–5000	Maximum wheel load 11.25 (5) converted to 22.5 kip (10 ton) axle	Approx. 2000–3000	Maximum wheel load 11.25 (5)	Up to 2000 vehicles also bicycles	Maximum wheel load 11.25 (5)
Italy	N	Single axle 22.5 (10) Tandem axle 32.6 (14.5)	N	Single axle 22.5 (10) Tandem axle 32.6 (14.5)	N	Single axle 22.5 (10) Tandem axle 32.6 (14.5)
Spain	N	Single axle 29.25 (13) Tandem axle 47.25 (21)	N	Single axle 29.25 (13) Tandem axle 47.25 (21)	Not used	Not used
Sweden	>900	Single axle 22.5 (10) Tandem axle 36 (16)	300–900	Single axle 22.5 (10) Tandem axle 36 (16)	Not used	
Switzerland	2000–4000 (sum in both directions)	Single axle 22.5 (10) Tandem axle 36 (16)	1000–2000 (sum in both directions)	Single axle 22.5 (10) Tandem axle 36 (16)	100–1000 (sum in both directions)	Single axle 22.5 (10) Tandem axle 36 (16)
West Germany	1000	Single axle 22.5 (10)	500–1000	Single axle 22.5 (10)	500–1000	Single axle 22.5 (10)

N = not included in specifications

3 THE VEHICLE

3.5 PASSENGER CAR UNITS

Vehicles of different types have different road space requirements and different effects on the capacity of highway and intersection because of variations in size and peformance.

The overall effect on traffic operations by any vehicle can be expressed in terms of the effect of the basic unit – usually a passenger car. Equivalency between the Passenger Car Unit (PCU) and other vehicles grouped in classes is determined by conversion factors expressing the relative effect of different types of vehicles on traffic flow.

The passenger car unit equivalents for different types of vehicles under different conditions as used in Great Britain are shown in Table I–7.

TABLE I–7

Passenger Car Unit Equivalents – Great Britain

Class of vehicle	Equivalent value in passenger car units			
	Urban standards	Rural standards	Roundabout design	Traffic signal design
Private car, taxi, motor cycle combination, light goods vehicle	1.00	1.00	1.00	1.00
Pedal cycle	0.33	0.50	0.50	0.20
Motor cycle (solo) motor scooter, moped	0.75	1.00	0.75	0.33
Medium or heavy goods vehicle, horse-drawn vehicle	2.00	3.00	2.80	1.75
Bus, coach, trolley bus, tram	3.00	3.00	2.80	2.25

3 THE VEHICLE

The factors for converting various types of vehicles into PCU equivalents, developed by the Ministry of Transport for the "Road Transport Study, Iraq, 1972", are presented in Table I–8

TABLE I–8

Conversion Factors to PCU			
Class of Vehicle	Terrain		
	Flat	Hilly	Mountains
Motorcycle	0.50	0.50	0.50
Private car; taxi	1.00	1.00	1.00
Bus up to 24 passengers	1.25	2.00	4.00
Bus 25 passengers and up	2.00	4.00	6.00
Pick-up, van	1.25	1.50	2.50
Lorry	2.00	4.00	8.00
Lorry with trailer, heavy veh.	2.50	5.00	10.00

For traffic count and design purposes, Conversion Factors similar to those of "Road Transport Study, Iraq, 1972" are used by SORB. These factors are shown in Table I–9.

TABLE I–9

SORB – Conversion Factors to PCU			
Class of vehicle	Terrain		
	Flat	Hilly	Mount.
Private car, taxi, motorcycle	1.00	1.00	1.00
Pick-up, van, bus up to 24 passengers	1.25	1.75	3.00
Truck, bus above 24 passengers	2.00	3.00	6.00
Truck and trailer combination, Heavy vehicle	3.00	5.00	10.00

3 THE VEHICLE

3.6 OPERATING CHARACTERISTICS

This part is devoted to the presentation of the more important operation characteristics of vehicles for the geometric design of the highway. The characteristics presented include speed, acceleration and deceleration. Other characteristics, such as fuel economy, operating costs, etc, are not considered in this chapter.

(a) Laws of Motion

(1) Uniform Motion (constant velocity)

$$v = \frac{l}{t}$$

where: v = velocity in meters per second

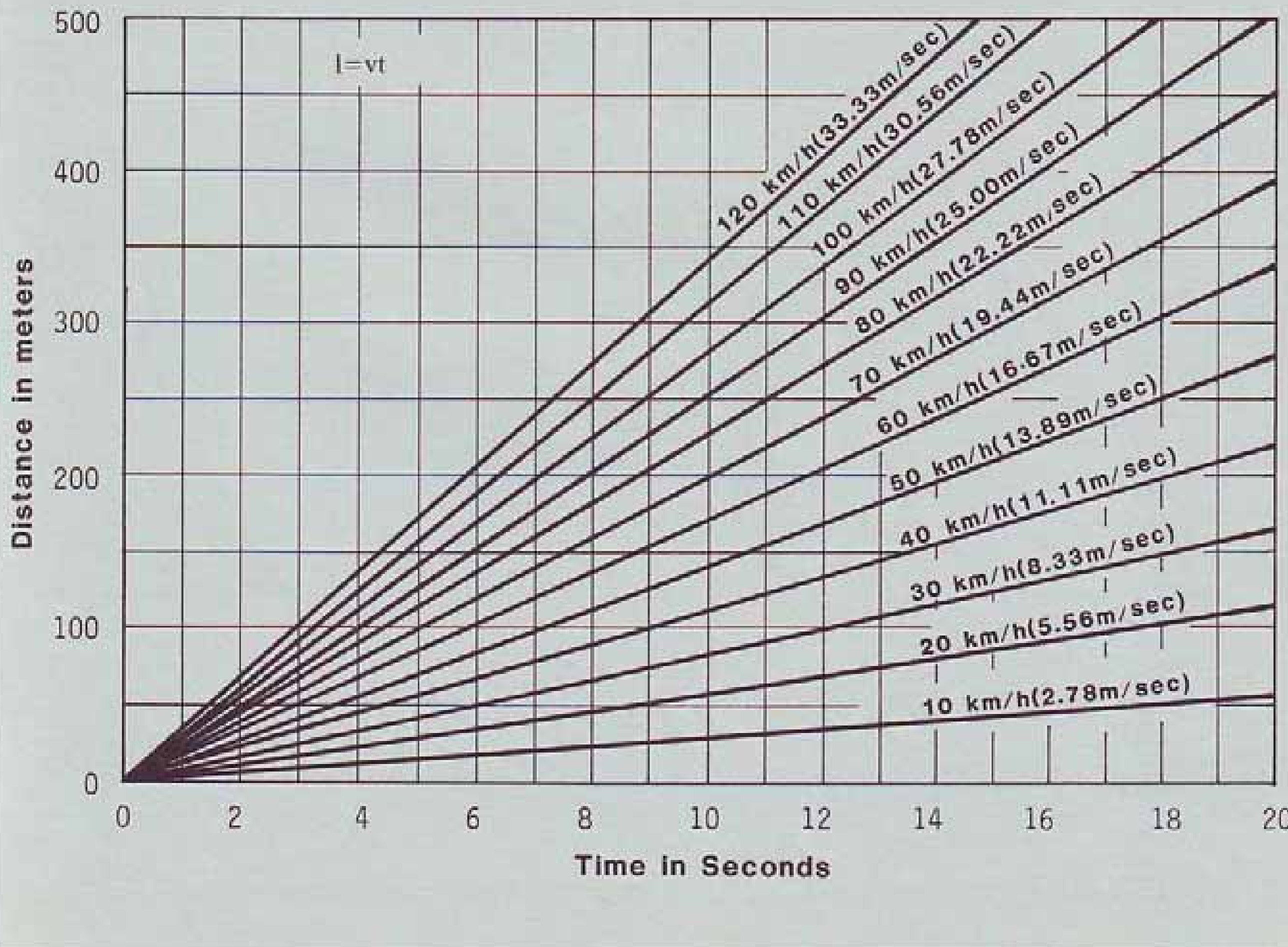
l = distance in meters

t = time in seconds

Conversion of velocity from meters per second (v) to kilometers per hour (V)

$$v = \frac{V}{3.6} = 0.2778\ V$$

FIGURE I–20 Relation of Speed, Distance and Time



3 THE VEHICLE

(2) Uniformly Accelerated Motion

$(v_0 > 0)$	$(v_0 = 0)$
$v_f = v_0 + a\,t$	$v_f = a\,t$
$l = v_0\,t + \frac{1}{2}\,a\,t^2$	$l = \frac{1}{2}\,a\,t^2$
$v_f = v_0 + \sqrt{2\,a\,t}$	$v_f = \sqrt{2\,a\,t}$

Where a = acceleration rate in meters per second per second
l = distance in meters
v_0 = initial velocity in meters per second
f_0 = final velocity in meters per second
t = time in seconds

Conversion of acceleration rate from m/sec^2 (a) to km/h/second (A)

$$A = \frac{a}{3.6} = 0.2778\,a$$

FIGURE I–21

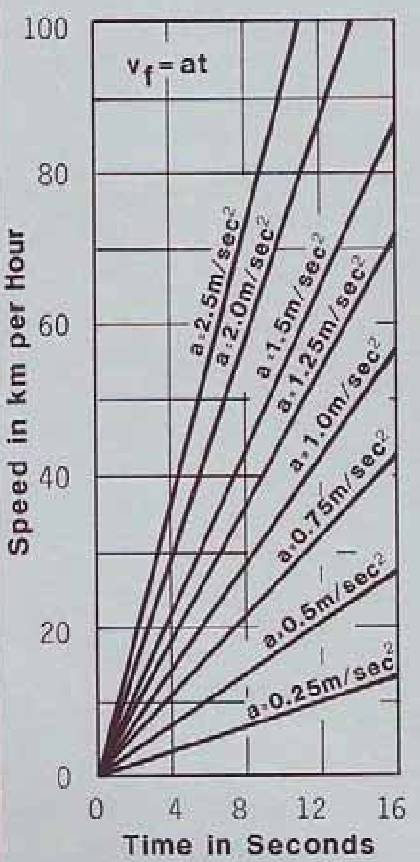


FIGURE I–22

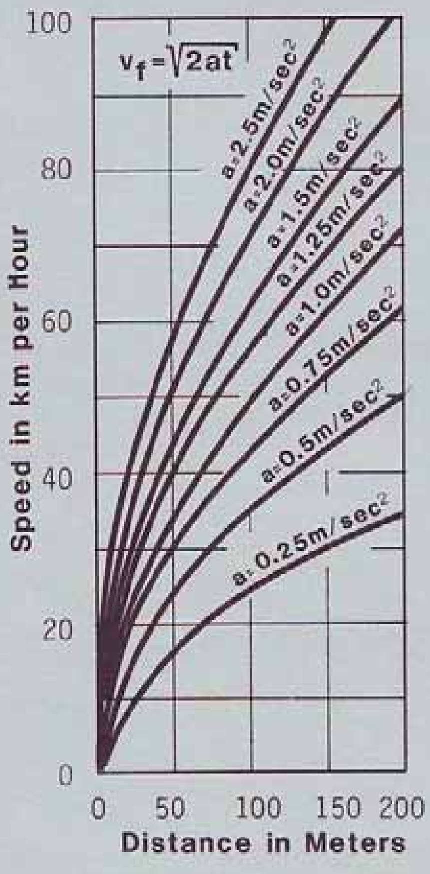


FIGURE I–23

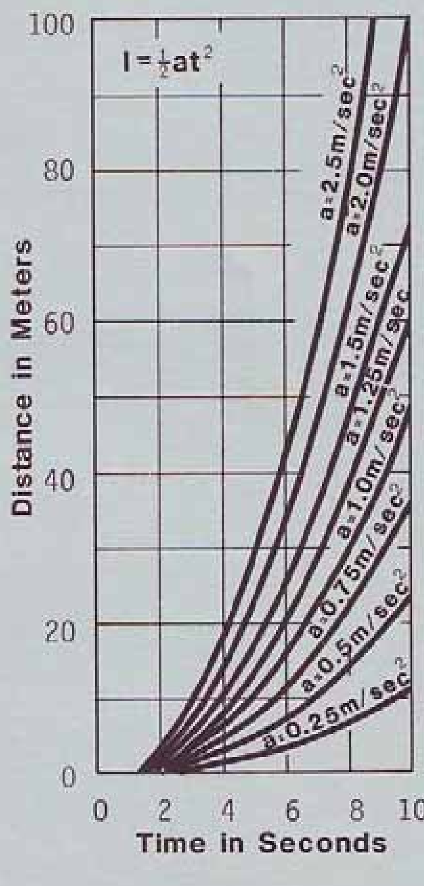


FIGURE I–21 Speed reached at various times after start with constant acceleration
FIGURE I–22 Speed reached at various distances after start with constant acceleration
FIGURE I–23 Distance travelled during periods of time with constant acceleration

3 THE VEHICLE

(3) Uniform Deceleration Motion

$(v_f > {}_o)$	$(v_f = {}_o)$
$v_f = v_o - dt$	
$l = v_o - \frac{1}{2} dt$	$v_o = dt$
$v_f = v_o - \sqrt{2\,dl}$	$v_f = \sqrt{2\,dt}$

Where d = deceleration rate in meters per second per second
l = distance in meters
v_o = initial velocity in meters per second
v_f = final velocity in meters per second
t = time in seconds

Conversion of deceleration rate from m/sec^2 (d) to km/hour/second (D)

$$D = \frac{d}{3.6} = 0.2778d$$

FIGURE I–24

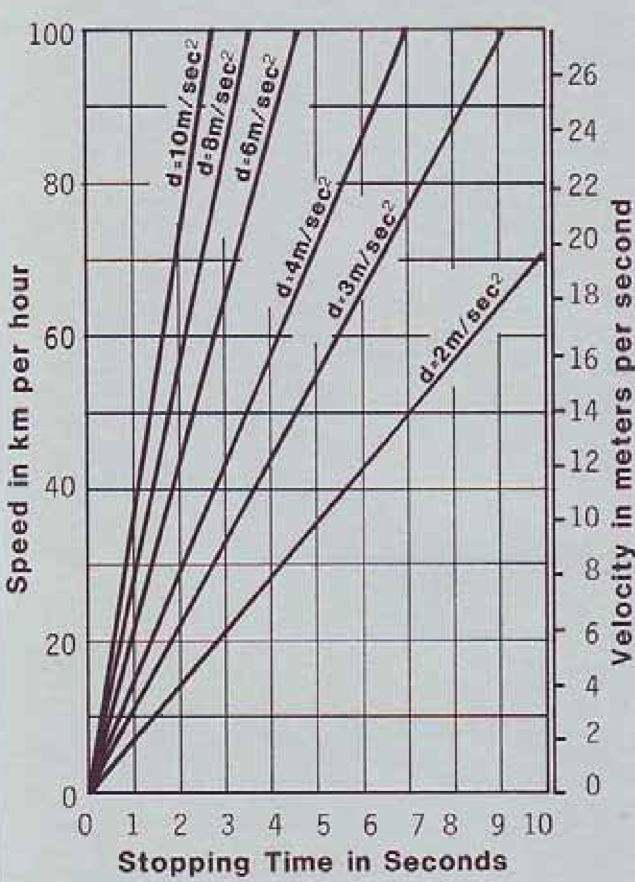


FIGURE I–25

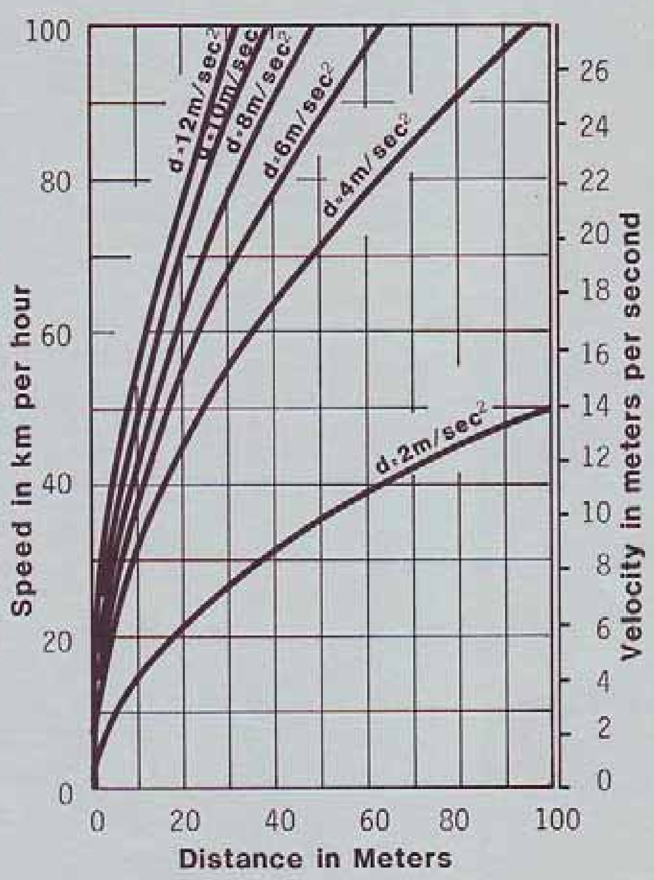


FIGURE I–24

Stopping Time from Various Initial Speeds with Constant Deceleration

FIGURE I–25

Distance Travelled from Various Initial Speeds with Constant Deceleration

3 THE VEHICLE

(4) Normal Roadway Acceleration Peformance

Acceleration performance of vehicles under normal roadway conditions and on relatively level sections of streets and highways is shown in Figures I–26, I–27 and I–28. The three curves for passenger cars were obtained while accelerating up to a running speed of 160, 96, and 56 km per hour.

FIGURE I–26

Distance-time Relationship during Normal Acceleration from a Standing Start

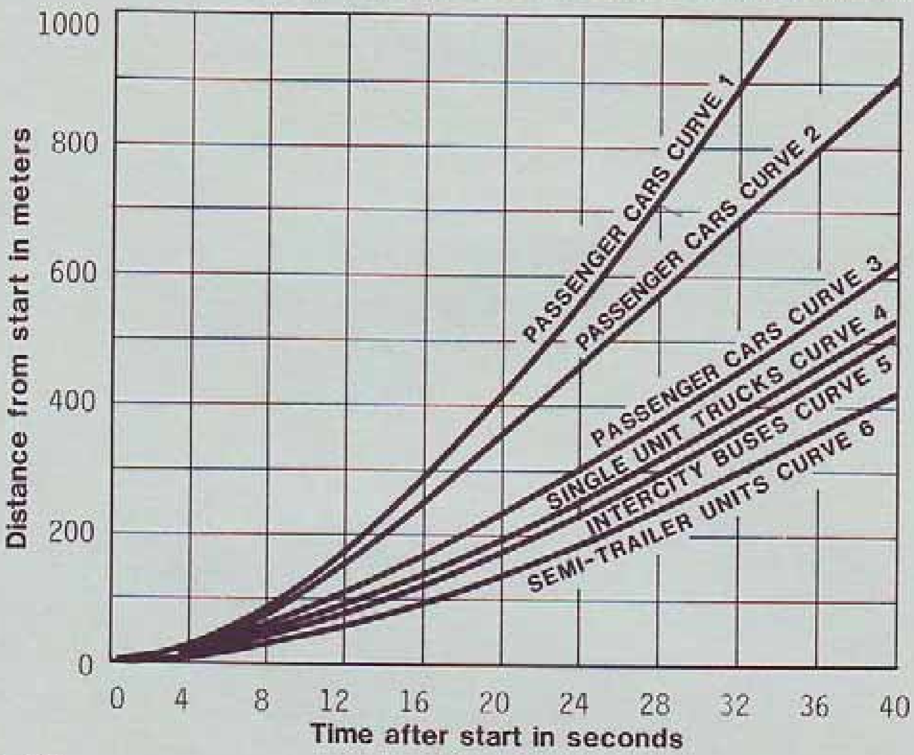
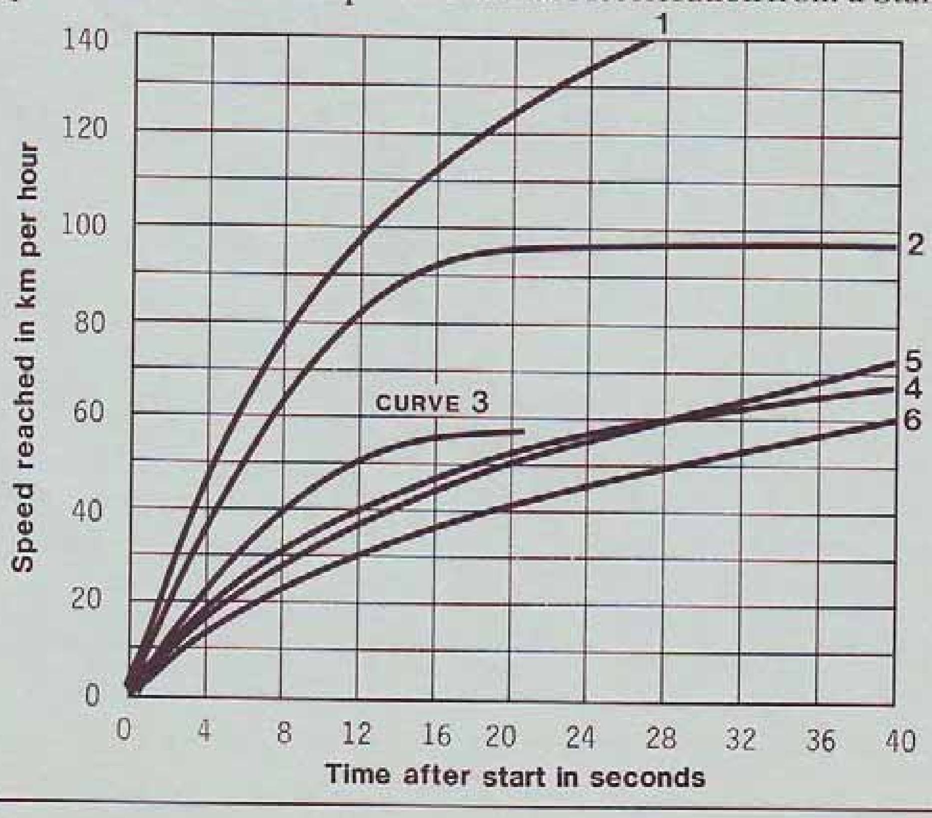


FIGURE I–27

Speed-time Relationship with Normal Acceleration from a Standing Start



Passenger cars = Curves 1, 2 and 3
Single-unit trucks = Curve 4
Intercity buses = Curve 5
Semi-trailer units = Curve 6

3 THE VEHICLE

FIGURE I–28
Speed-distance Relationship during Normal Acceleration from a Standing Start

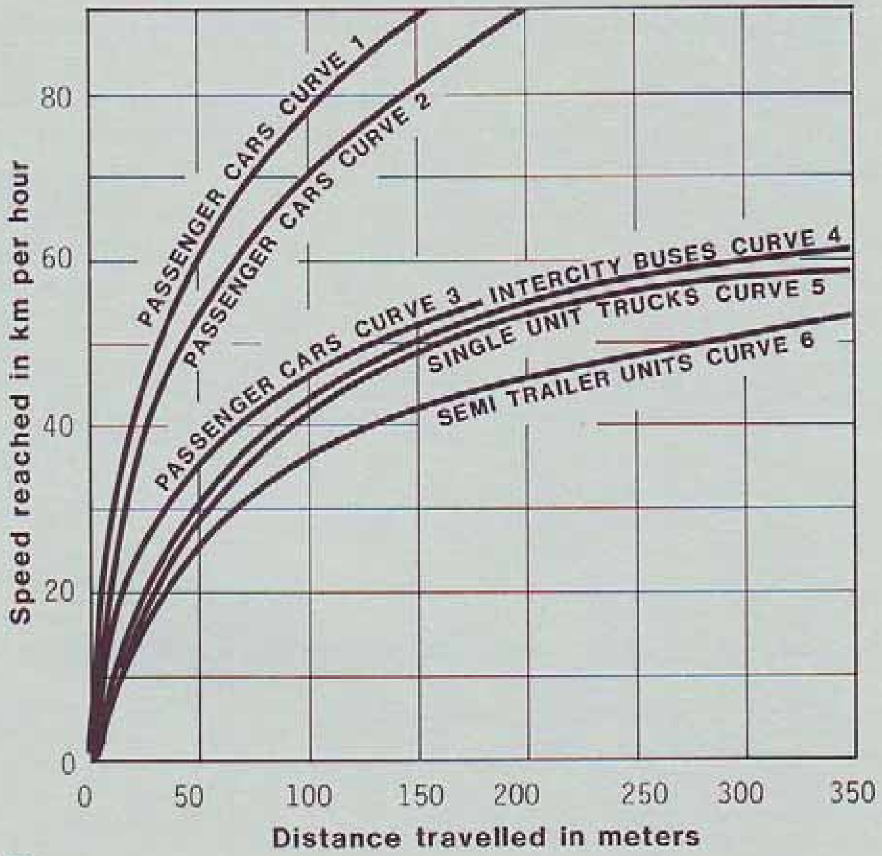
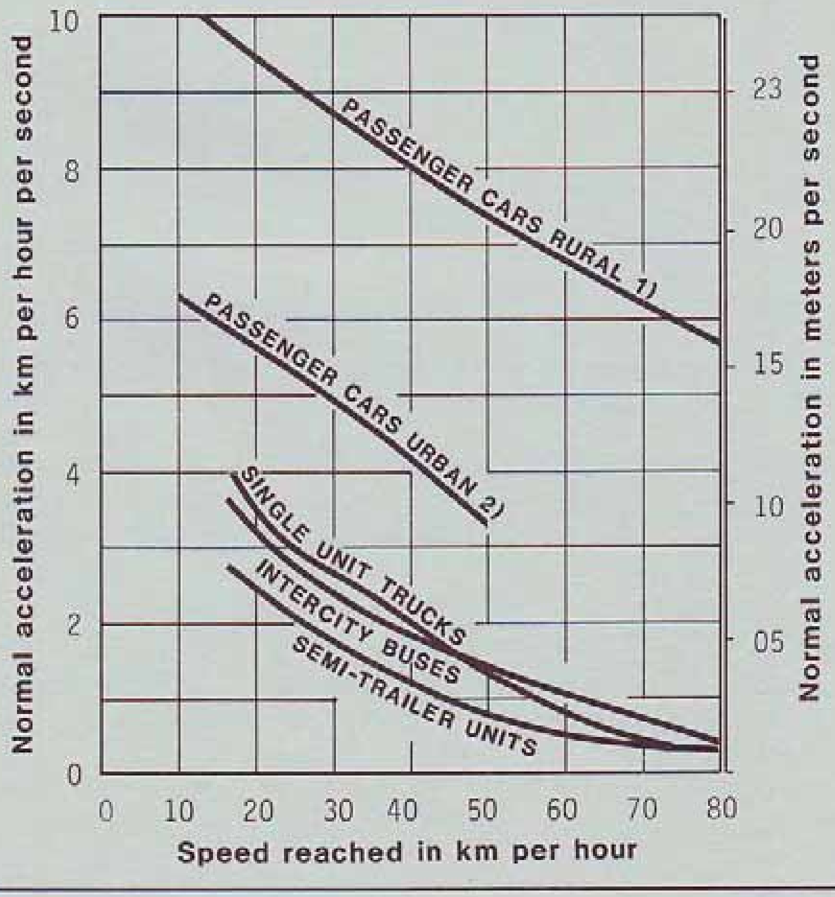


FIGURE I–29
Speed-Acceleration Rate Relationship during Normal Acceleration



1) Obtained while accelerating up to running speed of 96 km/h
2) Obtained while accelerating up to a running speed of 50 km/h

3 THE VEHICLE

(5) Deceleration Performance

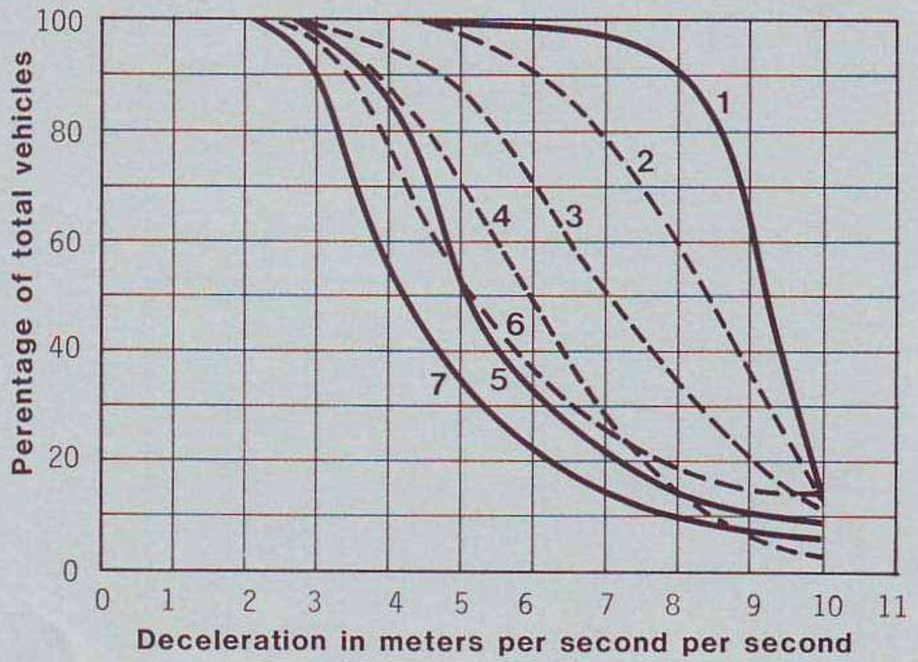
A vehicle coasting (with motor disengaged) to a stop on a level roadway will be decelerated by tractive resistance alone. This is composed of rolling resistance plus air resistance.
Vehicles being stopped by braking are decelerated by the force of friction between tyres and road. The maximum amount of braking force is limited by the maximum coefficient of friction between road surface and tyres.
Deceleration in meters per second^2 caused by braking is equal to 9.81f, where f is the coefficient of friction.

(6) Maximum Deceleration Rate

Maximum brake deceleration rates from 30 km per hour are given in Figure I–30. These results were obtained from 1,200 vehicles selected at random from roadway traffic.

FIGURE I–30

Percentage of Vehicles Capable of a Given or Greater Deceleration Rate



Maximum brake deceleration rates from 80 km per hour for passenger cars vary from 6.0 m per second^2 to 9.0 per second^2.
Deceleration rates greater than 6.0 m per second^2 will be used only in emergency situations.
At deceleration rates above this value, the occupants must brace themselves firmly to avoid being thrown off the seats. At deceleration rates of 4.25 m per second^2, packages may slide off the seats, and occupants will find this rate uncomfortable.
A deceleration rate of 3.4 m per second^2 is considered undesirable but not alarming to passengers.
Practical values of deceleration used under every-day traffic conditions rarely exceed 2.4 to 2.7 m per second^2.

3 THE VEHICLE

(7) Performance on Grades

The effect of upgrades up to 7% on passenger cars is generally negligible. It is the effect of long steep upgrades on the speed of trucks which has to be considered for highway design.

FIGURE I–31

Effect of Length and Steepness of Climb on Speed of Average Trucks on
(a) Two-lane and
(b) Multilane Highways

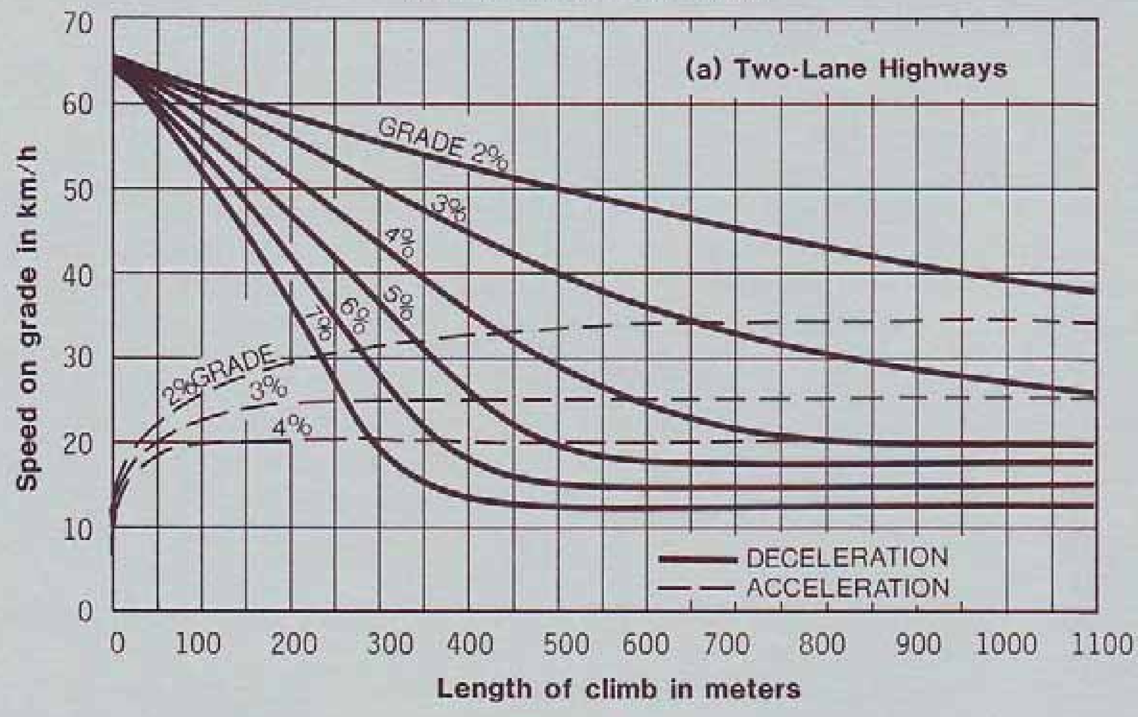


FIGURE 31a

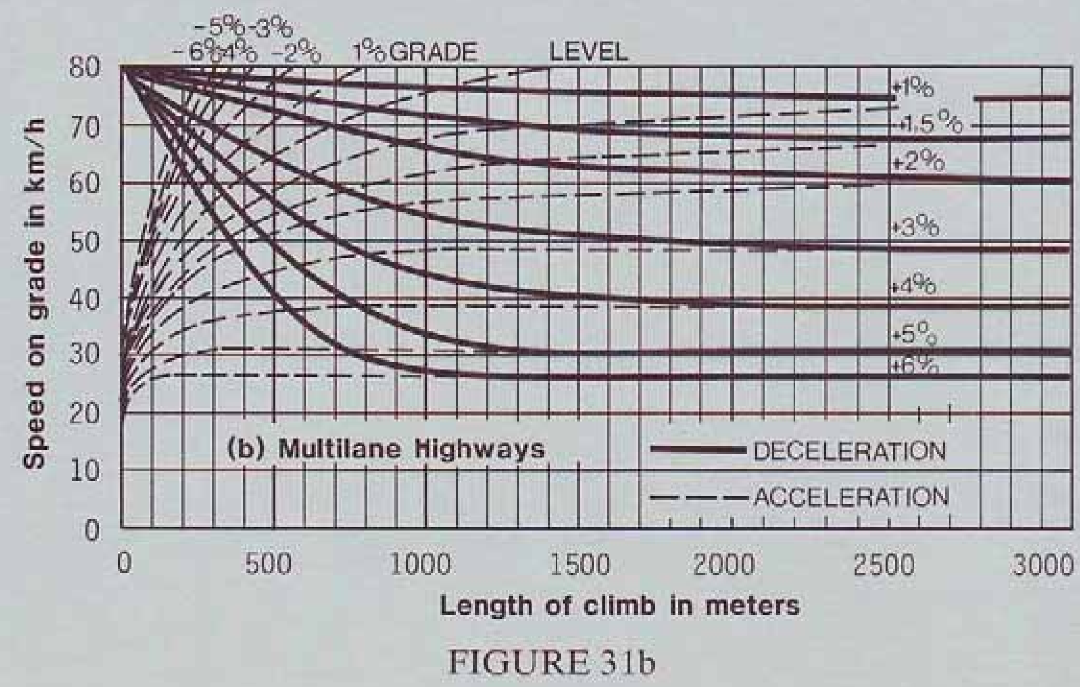


FIGURE 31b

3 THE VEHICLE

The relationship between the speed of average trucks at the bottom of a hill, percentage of grade, and the speed at any distance upgrade is shown in Figure I–31.

From these graphs it is possible to determine how far a vehicle starting at speeds of up to 65 km/h and 80 km/h, respectively, can climb various grades or combinations of grades before a particular crawl speed is reached.
The solid curves indicate the performance that can be expected when the starting speed is above the crawl speed. The curves also show the speed reduction due to any length and steepness of grade for other approach speeds. For example, given a typical two-lane condition and 4% grade and assuming an approach speed of 50 km/h (initial chart distance 220 m), the speed at a point 280 m up the grade will be about 30 km/h (final chart distance 500 m).

The broken lines show what performance may be expected when starting on the hill or approaching the hill at a speed lower than the crawl speed, so that the vehicle accelerates to eventually reach the sustained crawl speed.
These curves show that long distances are required to accelerate on grades when the approach speed is below the crawl speed. For example, to increase the speed of a typical truck on a two-lane road with a 3% grade from 10 km/h to the crawl speed of 26 km/h, the vehicle would have to travel about 400 m.

4 TRAFFIC DATA

4.1 INTRODUCTION

The design of a highway or any part thereof should be based on factual traffic data. Financing, quality of foundations, availability of materials and other factors have an important bearing on the design, but traffic indicates the service for which the improvement is being made, and its characteristics directly affect both the geometric design features and the pavement design such as width, alignment, grades, etc.

4.2 DEFINITIONS

The following sections explain the most commonly used terms concerning traffic data. A summary of definitions including common standard values is shown in Table I–10.

4.2.1 Vehicle Types and their Conversion into Passenger Car Units

Vehicles of different sizes and weights have different operating characteristics. Besides being heavier, trucks generally are slower and occupy more road space and, consequently, impose a greater traffic effect on the highway than do passenger vehicles. Vehicles of various sizes and weights can be grouped into two general classes according to their effect on operations.

Passenger Cars:

All free-wheeled, self-propelled vehicles generally designed for the transportation of persons, but limited in seating capacity to not more than nine passengers, including taxicabs, limousines, and station wagons. Also included, for capacity purposes, are two-axle, four-tyred pickups, panels and light trucks which have operating characteristics similar to those of passenger cars, but not motorcycles.

Trucks:

All free-wheeled, self-propelled vehicles, designed for the transportation of persons and having a seating capacity of ten or more passengers.
All free-wheeled vehicles having dual tires on one or more axles, or having more than two axles, designed for the transportation of freight rather than passengers. Includes tractor-trucks, trailers and semitrailers when used in combination. Excludes those two-axle, four-tyred vehicles that may be classified as a truck for registration purposes, but which have operating characteristics similar to those of a passenger car.
For particular purposes a more detailed breakdown of freight transport vehicles into light trucks, medium trucks and heavy trucks incl. trailers is required.

4 TRAFFIC DATA

Passenger Car Unit (PCU):

Vehicles of different types require different amounts of road space because of variations in size and performance. To allow for this in capacity measurements for roads and junctions, traffic volumes are expressed in passenger car units (PCU's). The basic unit is a car (taxis, light vans or three-wheeled vehicles are also one unit). As different types of vehicles affect the capacity of rural roads, urban roads, roundabouts and traffic signals in varying degrees, the weighting for each class of vehicle has to be varied to suit the purpose for which they are to be used. For example, on fairly level open roads, the following values apply.

1 passenger car	= 1 PCU
1 light truck	= 1.25 PCU
1 medium (average) truck	= 2.0 PCU
1 heavy goods vehicle	= 3 PCU

4.2.2 Traffic Volumes and Characteristics*

(1) General
In order to avoid any misinterpretation of data concerning traffic volumes and operation, the time period and direction of travel to which they relate need to be stated clearly.

(2) Average Annual Daily Traffic (AADT)

The general unit of measures of traffic flow on a highway is the average annual daily traffic volume (AADT). This is the total number of vehicles that pass over a given section of a lane or a roadway during one year divided by 365. Where only sporadic counts are taken (which is common), the AADT can be derived approximately by the calculation of the weighted mean of volumes of representative days. The direct use of AADT in the geometric design of rural highways is not appropriate because it does not indicate the significant variations of traffic occurring during the various months of the year, days of the week and hours of the day. At the average rural location, the volume on certain days may be double the AADT.

(3) Average Daily Traffic (ADT)

The ADT is often used for measuring traffic volumes when traffic volumes vary little from day to day over the course of the year. It is calculated by dividing the total volume of traffic over the count period by the number of days in that period. The count period should be greater than one day and less than one year.

(4) Peak Hour Traffic

Traffic flows on a highway vary during the course of a day reflecting the different operating conditions. The highest hourly volume which occurs during the daily rush hour period is called the peak hour volume and is required for the design of traffic facilities. It is usual to grade the hourly volumes over the designated year so that the 20th and 30th highest annual hourly volumes may be found.

---

*Note: Examples of traffic volumes and characteristics observed on IRAQI roads are shown in Table I–12.

4 TRAFFIC DATA

(5) Design Hour Volume

The hourly traffic flow used for design purposes may occasionally be exceeded and past experience has indicated that an economical and technically sound design may be achieved by using the 30th highest hourly volume (abbreviated to 30 HV) as the design hour volume (DHV). The DHV is normally expressed as a percentage (K) of the ADT.

(6) Directional Distribution

Although traffic flows in each direction on a highway tend to be approximately the same over each 24-hour period, during peak hours or on specific days there is likely to be a higher percentage of traffic in one direction. This is usually expressed as a percentage (D) of the two-directional volume for the particular peak hour and is calculated for both morning and evening peaks.
In designing at-grade intersections and interchanges it is necessary to know the traffic flows on all legs during the peak hours together with the percentage of right and left turning traffic for each leg.
Fig. I–32 illustrates typical intersection turning movement diagrams.

(7) Composition of Traffic

To account for the different operating characteristics of passenger cars and trucks, the composition of the traffic flows is defined by either the percentage of trucks and buses during the peak hour or during the day (T_H or T_D respectively), or by detailing the percentage of all vehicle classes.
Representative values for Iraqi roads are shown in Table I–12.

4.3 TRAFFIC FORECAST

4.3.1 General

The design of new highways or of improvements to existing highways should not be based on current traffic volumes, but on the future traffic expected to use the facilities. The forecast year for design traffic should not be so far ahead that estimates cannot be made with reasonable accuracy. A period of 15 to 20 years is widely used. Future traffic volumes for the design are normally derived from the current traffic level and the traffic increase expected by the design year. There are many methods of traffic forecasting and their selection and application depend on the nature of the project.
For important projects traffic forecasts should be based on thorough-traffic studies considering the analysis of zonal socio-economic variables (e.g. population, jobs, employees, schoolplaces, car ownership), mobility, development of transport facilities and modal split. Here, only the method for simple cases is described.

In any case where a traffic forecast is required, there should be a careful check made for any existing volume projections. An important source of future traffic data may be the Road Transport Study (Vol. III Traffic Forecast, Nov. 1979) published by the Ministry of Planning, Transport and Communications Department, which presents traffic volumes for 1985 and 1995.

4 TRAFFIC DATA

4.3.2 Components of Future Traffic

Current Traffic: Existing and Attracted

Current traffic is the volume of traffic that would use a new or improved highway if it were open to traffic, i.e. traffic already using the route plus traffic transferring to the new highway from less attractive routes.

Normal Traffic Growth

Normal traffic growth is the increase of current traffic due to general increase in number and usage of motor vehicles.

Generated Traffic

Generated traffic consists of motor vehicle trips that would not have been made if the new facility had not been provided. Generated traffic is made up of three categories: new trips not previously made by any mode of travel, trips that previously were made by public transport and trips that were previously made to a different destination, but for which the change is attributable to the attractiveness of the new or improved highway.

Generated traffic should be included even though it may be based on assessment only. For most rural highway projects, generated traffic is likely to be 5 per cent or more of current traffic but rarely as much as 25 per cent.

Development Traffic

Development traffic is traffic due to improvements on adjacent land over and above the development which would have taken place had the new or improved highways not been constructed. Increased traffic due to normal development of adjacent land is included in normal traffic growth. Rural highways leading to a recreation area, which is within reasonable distance of an urban area, might need a larger development factor.
The method of estimating development traffic may not be precise and the volumes obtained are only approximate, but in many cases the volume is substantial.

4.3.3 Traffic Forecast Factor

The traffic forecast factor is a ratio of future traffic to current traffic. The traffic increases reflected by this factor combine those due to normal traffic growth, generated traffic and development traffic. The future year for design should be specified with every traffic forecast. Traffic forecast factors for various traffic analysis periods and annual rates of traffic increase are shown in Table I–13.
General annual rates of traffic increase, which may be used for simple rural highway projects in Iraq, are as follows:

10% for 1980 to 1985
8% for 1986 to 1995
6% for 1996 to 2005

4 TRAFFIC DATA

4.4 EXAMPLE OF TRAFFIC DATA CALCULATION FOR ROAD DESIGN BY MEANS OF FACTORS

Design data is required for the improvement of a two-lane, two-way highway with central reserve. The current traffic in 1979 expressed by the AADT is 3000 vehicles in both directions. The improved road with a design life of 20 years will be opened to traffic in 1984. The assumed design year is therefore 2004. The estimated annual growth rate of traffic is 8% p.a. This results in a total growth factor of 6.8 for a period of 25 years (from 1979 to 2004). Thus the AADT 2004 is 20 400 vehicles per day. Local studies indicate that:

K (DHV percentage) = 12 (% of ADT)
D (direction distribution) = 55 (% of total volume)
T_D (truck percentage) = 25 (% of daily volume)
T_H (peak hour truck percentage) = 18 (% of hourly volume)

In order to get the DHV (design hour volume) in the predominant direction, the following formula applies:

$$DHV^{\longrightarrow} = ADT^{\leftrightarrows} \times K \times D \quad [\text{v.p.h.}]$$

In the example

$$\overrightarrow{DHV}_{2004} = 20\,400 \times 0.12 \times 0.55 = 1346 \ [\text{v.p.h.}]$$

To take into account the given composition of traffic, the respective number of PCUs has to be calculated as

$$DHV\ [\text{PCUs p.h.}] = DHV\ [\text{v.p.h.}] \times \Big[\Big(\underbrace{\frac{100-T_H}{100} \times 1.0}_{\text{Pass. Cars}}\Big) + \Big(\underbrace{\frac{T_H \times 2.0}{100}}_{\text{Trucks}}\Big)\Big]$$

In the example

$$DHV_{2004} = 1346 \times \Big[\Big(\frac{100-18}{100} \times 1.0\Big) + \Big(\frac{18}{100} \times 2.0\Big)\Big] \quad [\text{PCUs p.h.}]$$
$$= 1588 \text{ PCUs p.h.}$$

4 TRAFFIC DATA

TABLE I–10

Definitions

Subject	Explanation	Usual Value
V.p.h. Vehicles per hour	Measure of traffic volume	–
PCU Passenger Car Unit	Operational equivalent of one passenger car	1 Passenger car = 1 PCU 1 Average truck or 1 medium truck or 1 bus = 2 PCU 1 Light truck = 1.25 PCU 1 Heavy truck or trailer = 3 PCU 1 Motorbike = 0.5 PCU
AADT Average Annual Daily Traffic	The total yearly traffic volume for a given section of a lane or roadway divided by 365. Unless specially noted or clearly understandable, the total of both directions is meant.	–
ADT Average Daily Traffic	The total volume during a given time period in whole days greater than one day and less than one year, divided by the respective number of days. Unless specially noted or clearly understandable, the total of both directions is meant.	Representative values see Table I –12
DHV Design Hour Volume	DHV is an hourly volume that occurs during traffic peaks. Unless specially noted or clearly understandable, the total of both directions is meant.	The 30th highest hourly volume of the design year is normally used as DHV
K Percentage of DHV against ADT	$K = \frac{DHV}{ADT} \cdot 100\%$	DHV two-way traffic volume ranges from 10 to 17 per cent of two-way ADT. DHV one-way traffic volume ranges from 7 to 18 per cent of one-way ADT.
D Percentage of predominant direction	D indicates the directional distribution of traffic volumes	On main highways D = c. 55 (% of DHV) On secondary highways D = c. 60 (% of DHV)
T_H, T_D Truck percentage	T_H is the percentage of trucks within peak hour traffic, T_D within daily traffic	$T_H = 0.7\ T_D$ Representative values are shown in Table I–12

4 TRAFFIC DATA

Traffic Forecast Factors

TABLE I-11

Traffic Analysis Period n (Years)	Annual Rate of Traffic Increase in %									
	3	4	5	6	7	8	9	10	11	12
1	1.030	1.040	1.050	1.060	1.070	1.080	1.090	1.100	1.110	1.120
2	1.061	1.082	1.103	1.124	1.145	1.166	1.188	1.210	1.232	1.254
3	1.093	1.125	1.158	1.191	1.225	1.260	1.295	1.331	1.368	1.405
4	1.126	1.170	1.216	1.262	1.311	1.360	1.412	1.464	1.518	1.574
5	1.159	1.217	1.276	1.338	1.403	1.469	1.539	1.611	1.685	1.762
6	1.194	1.265	1.340	1.419	1.501	1.587	1.677	1.772	1.870	1.974
7	1.230	1.316	1.407	1.504	1.606	1.714	1.828	1.949	2.076	2.211
8	1.267	1.369	1.477	1.594	1.718	1.815	1.993	2.144	2.305	2.476
9	1.305	1.423	1.551	1.689	1.838	1.999	2.172	2.358	2.558	2.773
10	1.344	1.480	1.629	1.791	1.967	2.159	2.367	2.594	2.839	3.106
11	1.384	1.539	1.710	1.898	2.105	2.332	2.580	2.853	3.152	3.479
12	1.426	1.601	1.796	2.012	2.252	2.518	2.813	3.138	3.498	3.896
13	1.469	1.665	1.886	2.133	2.410	2.720	3.066	3.452	3.883	4.363
14	1.513	1.732	1.980	2.261	2.579	2.937	3.342	3.797	4.310	4.887
15	1.558	1.801	2.079	2.397	2.759	3.172	3.642	4.177	4.785	5.474
16	1.605	1.873	2.183	2.540	2.952	3.426	3.970	4.595	5.311	6.130
17	1.653	1.948	2.292	2.693	3.159	3.700	4.328	5.054	5.895	6.866
18	1.702	2.026	2.407	2.854	3.380	3.996	4.717	5.560	6.544	7.690
19	1.754	2.107	2.527	3.026	3.617	4.316	5.142	6.116	7.263	8.613
20	1.806	2.191	2.653	3.207	3.870	4.661	5.604	6.727	8.062	9.646
21	1.860	2.279	2.786	3.400	4.141	5.034	6.109	7.400	8.942	10.804
22	1.916	2.370	2.925	3.604	4.430	5.437	6.659	8.140	9.934	12.100
23	1.974	2.465	3.072	3.820	4.741	5.871	7.258	8.954	11.026	13.552
24	2.033	2.563	3.225	4.050	5.072	6.341	7.911	9.850	12.239	15.179
25	2.094	2.666	3.386	4.292	5.427	6.848	8.623	10.835	13.586	17.000

$F = (1+i)^n$; where

F = Traffic forecast factor (Compound growth factor)

i = Annual rate of traffic increase

n = Traffic analysis period

4 TRAFFIC DATA

Traffic Volume Characteristics on Iraqi Roads, 1979–1980 TABLE I-12

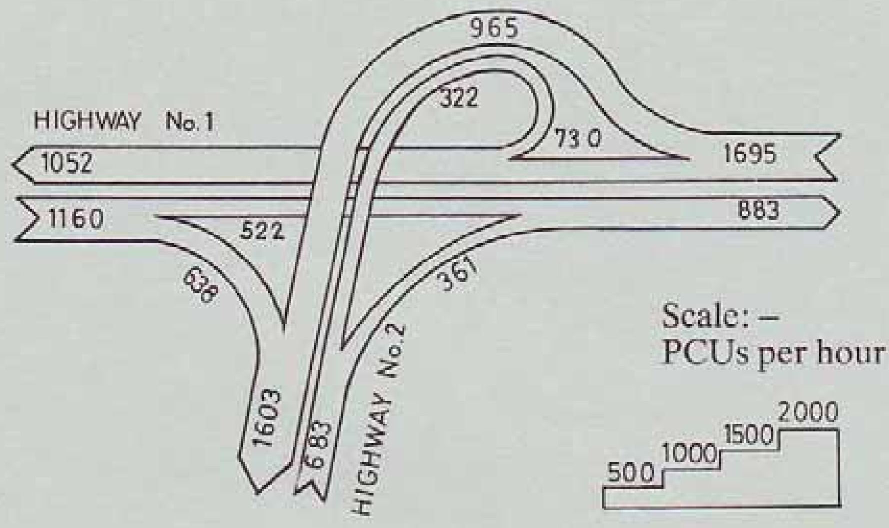
Type of Road			ADT (vph)	Friday Traffic (% of ADT)	Night Traffic (% of ADT)	Peak Hour Traffic (% of ADT)	Percentage of Vehicle Distrubution: Cars Taxi	Pick-ups Small Buses	Trucks Big Buses	Heavy Vehicles	Trucks Heavy Veh.	ADT p.c.u. factor (flat country)	Directional Distribution: ADT	Directional Distribution: P.H.T.
Main Roads, Dual Carriageway	Governorate: Baghdad	Max.	24.250	165.7	28.1	17.9								
		Min.	8.000	48.2	5.2	5.8								
		Average	14.300	83.1	23.6	10.5							51/49	51/49
	Governorate: Others	Max.	11.700	95.4	10.3	10.4	62.6	34.5	32.7	3.9	35.0	1.45		
		Min.	6.900	84.8	6.0	8.8	35.2	21.5	11.2	2.0	14.0	1.29		
		Average	11.600	92.4	9.1	9.4	44.7	30.3	22.0	3.0	25.0	1.36	52/48	55/44
Main Roads, Single Carriageway	Governorate: Baghdad	Max.	6.450	89.7	17.5	13.5								
		Min.	4.600	57.2	7.9	8.4								
		Average	5.350	76.5	11.6	10.2							51/49	51/49
	Governorate: Others	Max.	4.650	104.9	12.3	12.1	44.1	33.0	52.3	10.2	60.7	1.69		
		Min.	2.000	78.5	6.5	7.1	25.7	11.0	23.0	3.3	29.9	1.29		
		Average	3.200	89.5	9.0	10.1	33.1	24.7	36.6	5.6	42.2	1.52		
Secondary Roads single Carriageway		Max.	4.100	106.6	9.4	17.0	56.4	38.3	61.8	12.2	65.5	1.65		
		Min.	550	71.8	4.5	9.5	15.0	18.0	14.6	1.8	16.5	1.19		
		Average	2.000	76.5	6.9	12.5	36.0	27.0	31.6	5.4	37.0	1.34		

4 TRAFFIC DATA

FIGURE I–32 Typical Intersection Turning Movement Diagrams

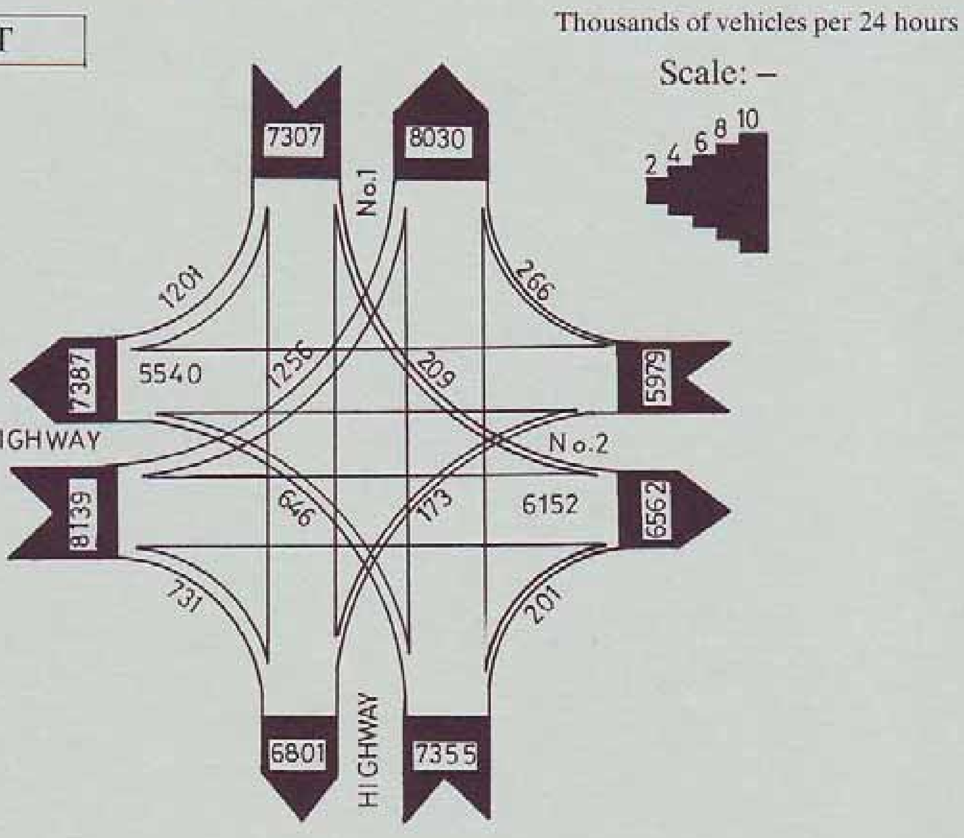
A–Grade separated intersection

AM PEAK



B– At-grade intersection

ADT



PART II

HIGHWAY DESIGN

PART II HIGHWAY DESIGN

PART II HIGHWAY DESIGN

Contents (continued)

PART II HIGHWAY DESIGN

LIST OF TABLES

PART II HIGHWAY DESIGN

LIST OF FIGURES

PART II HIGHWAY DESIGN

LIST OF FIGURES (continued)

1 SYSTEM, CLASSIFICATION, CAPACITIES

1 SYSTEM, CLASSIFICATION AND CAPACITIES OF HIGHWAYS

1.1 SYSTEM OF HIGHWAYS

System designation is as follows:
(1) – Primary system
(2) – Secondary system
(3) – Tertiary system

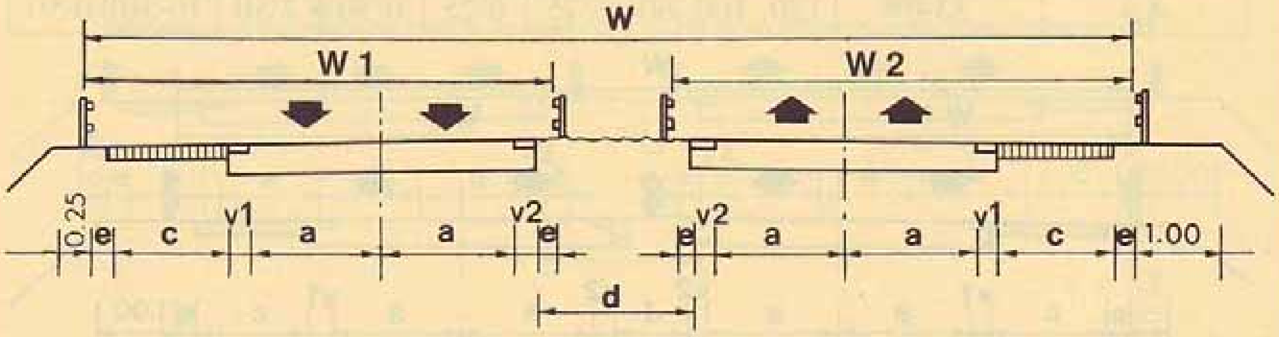
(1) Primary System
Highways of international importance (the main highways connecting main cities) and highways of special importance should form the primary system of national highways. These highways are to be designed to the highest standards.

(2) Secondary System
Highways connecting major cities of economic or other importance, highways connecting agricultural, commercial, recreational or tourist areas should form the secondary system of highways.

(3) Tertiary System
Highways of district and local importance should form the tertiary system.

GENERAL
Basic classes of highways are shown in the following sketches. Their characteristics are mentioned in Tables II–1, 2, 3.

HIGHWAY CHARACTERISTICS:



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

1 SYSTEM, CLASSIFICATION, CAPACITIES

Explanation of class designation:

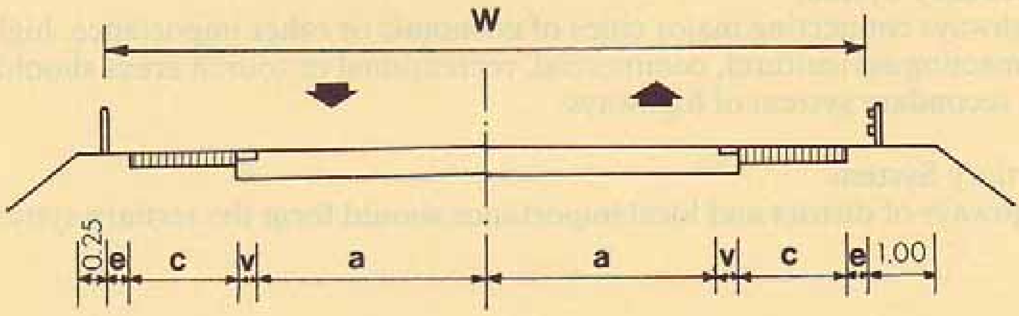
Class: A4/25.5

A – letter symbol of class
4 – number of traffic lanes
25.5 – total width of highway

Highway Characteristics – Two Lane

TABLE II–1

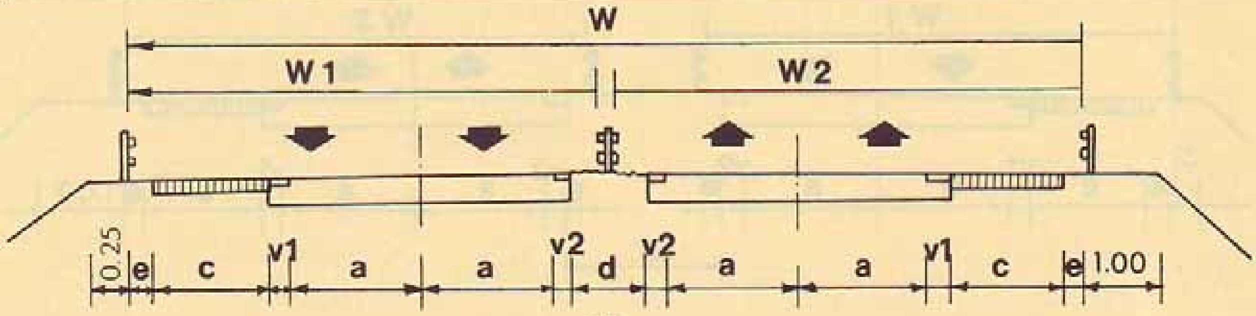
CLASS			WIDTH IN m.			
Letter symbol	W in m.	Design speed km/h	a	v	c	e
D 2	9·5	70, 60, 50	3·00	0·25	1·25	0·25
C 2	11	80, 70, 60	3·50	0·25	1·25	0·50
B 2	12	80, 70, 60	3·50	0·25	1·75	0·50
A 2	13·5	100, 80, 70	3·75	0·25	2·25	0·50

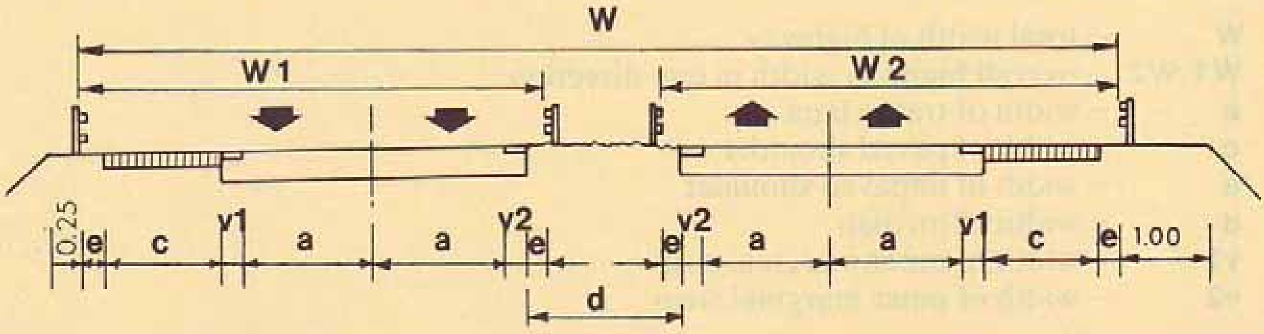


Highway Characteristics – Four Lane

TABLE II–2

CLASS			WIDTH IN m.					
Letter symbol	W in m.	Design speed in km/h	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



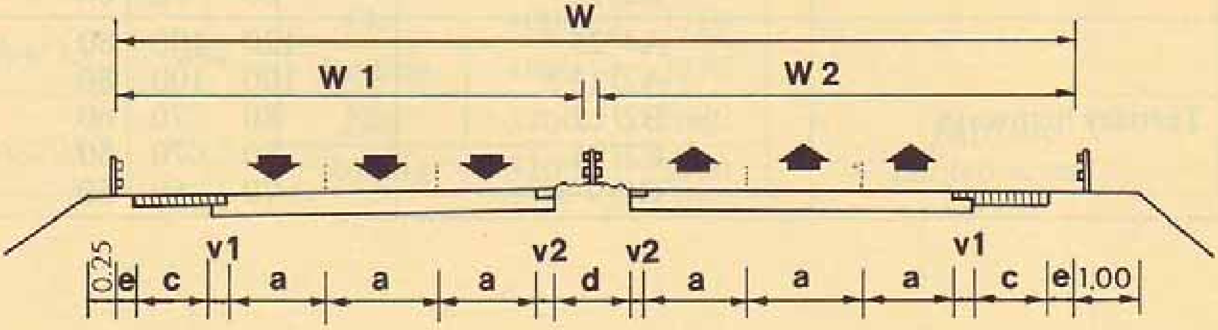


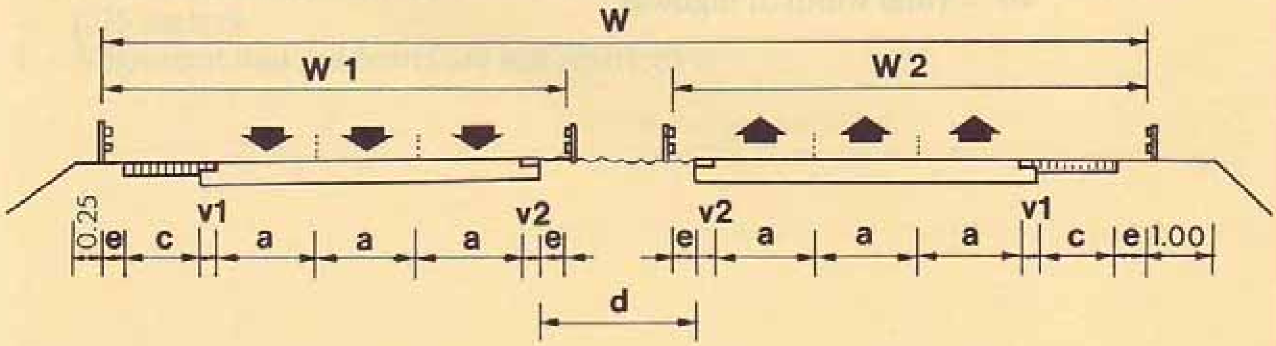
1 SYSTEM, CLASSIFICATION, CAPACITIES

Highway Characteristics – Six Lane

TABLE II–3

CLASS			WIDTH IN m.					
Letter symbol	W in m.	Design speed in km/h	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





1 SYSTEM, CLASSIFICATION, CAPACITIES

1.2 HIGHWAY CLASSES

TABLE II–4

System of highway	Corresponding highway classes	
	Letter symbol	Design speed km/h
Primary highways	A6/40 * A6/33 A4/33 A4/25·5 A2/13·5	120 100 80 120 100 80 120 100 80 120 100 80 100 80 70
Secondary highways	A6/33 A4/33 A4/25·5 A2/13·5 B2/12	120 100 80 120 100 80 120 100 80 100 80 70 80 70 60
Tertiary highways	A4/25·5 A2/13·5 B2/12 C2/11 D2/9·5	120 100 80 100 100 80 80 70 60 80 70 60 70 60 50

Explanation of class designation:
*Class A6/40

A – Letter symbol of class
6 – Number of traffic lanes
40 – Total width of highway

1 SYSTEM, CLASSIFICATION, CAPACITIES

1.3 DESIGN CAPACITIES OF HIGHWAY CLASSES

(1) Multilane Two-Way Highways
Design capacity, total in both directions in VPH

TABLE II–5

Class	Lanes	Terrain	Percentage of trucks in peak hour		Note
			0%	20%	
A6/40	2x3	Flat	6000	4980	Limited access
		Rolling	6000	3780	
A6/33	2x3	Flat	6000	4980	
		Rolling	6000	3780	
A4/33	2x2	Flat	4000	3320	
		Rolling	4000	2520	
A4/25·5	2x2	Flat	3200	2680	Partially limited access
		Rolling	3200	2000	

The values given in Table II–5 presuppose that:
1 – Traffic lanes are 3·75 meters wide.
2 – Shoulders are 3·0 meters wide.
3 – Lateral clearances from pavement edge to roadside obstructions are at least 1.75 meters.
4 – Alignment and gradients are not restrictive.

1 SYSTEM, CLASSIFICATION, CAPACITIES

(2) Two-lane Two-way Highways
Design capacity, total in both directions in VPH

TABLE II–6

DESIGN SPEED 100 km/h					
Class	Lanes	Alignment: percentage of total length of highway on which sight distance is restricted to less than 450 meters		Percentage of trucks in peak hour	
				0%	20%
A2/13·5	2	0%	Flat	900	690
			Rolling	860	485
DESIGN SPEED 80 km/h					
A2/13·5	2	0%	Flat	900	690
		20%	Rolling	810	450
B2/12	2	20%	Flat	810	625
		40%	Rolling	700	390
C2/11	2	40%	Flat	700	540
		60%	Rolling	585	325

The values given in Table II–6 presuppose that:
1 – Traffic lanes have a min. width of 3·50 meters
2 – Shoulders have a min. width of 2·00 meters
3 – Lateral clearance from pavement edge to roadside obstructions is at least 1·75 meters.
4 – Alignment and gradients are not restrictive.

2 DESIGN ELEMENTS

2.1 DESIGN SPEED

Design speeds for different highway classes are mentioned in Table II–7. These speeds may be decreased only in special cases (near cities etc.)

Design Elements for Different Classes

TABLE II–7

CLASS OF HIGHWAY	TYPE OF TOPOGRAPHY								
	FLAT			ROLLING			MOUNTAINOUS		
	g	s	r	g	s	r	g	s	r
A6/40·5		120			100			80	
A4/33	3	4·5	5	4	5	6	5	5·5	6·5
A2/13·5		100			80			70	
	3·5	4·5	5·5	4·5	5·5	6·5	5·5	6	7
C2/11		80			70			60	
B2/12	4	5	6	5·5	6	7·5	7	6·5	8·5
D2/9·5		70			60			50	
	5	5·5	6·5	6·5	6·5	8·5	8	7	10
NOTE: The values given in this table are not valid for interchange ramps									

g – Gradient in %
s – Superelevation in %
r – Resulting grade in %
Design speed in km/h

2.2 SIGHT DISTANCE

2.2 SIGHT DISTANCE

(1) General

It is of utmost importance for the safe and efficient operation of a highway to offer the road user optimum visibility. For the sake of highway safety, the designer must provide for sight distance of sufficient length in which drivers can control the speed of their vehicles so as to avoid hitting an unexpected obstacle in their path. Also, at frequent intervals and for substantial portions of their length, two-way carriageways should permit a good view of oncoming traffic so as to enable drivers to use the opposite traffic lane unhindered while overtaking other vehicles.

(2) Stopping Sight Distances

Stopping Sight Distances for different design speeds and different gradients are given in Table II–8. The stopping sight distance has to be ensured on any highway over its entire lenght.

(3) Overtaking Sight Distances

Overtaking Sight Distances for design speeds of 2-lane two-way highways or ramps of interchanges, are given in Table II–9. On sections where it is not possible to ensure overtaking sight distance, the centerline of the highway must be marked by an uninterrupted line in addition to the "no overtaking" signs in order to prevent overtaking manoeuvres. The length of such sections in one direction should not exceed 500 m.

2.2 SIGHT DISTANCE

Stopping Sight Distance

TABLE II–8

Gradient of lane in %		DESIGN SPEED IN km/h								25 20
			120	100	80	70	60	50	40	30	
Downgrade	−8	–	–	–	–	–	45			
	−7	–	–	–	–	60	45			
	−6	–	–	100	80	60	45			
	−5	–	–	100	80	60	45			
	−4·5	–	150	95	80	60	40			
	−4	210	145	95	75	60	40			
	−3	205	145	95	75	55	40			
	−2	200	140	95	75	55	40			
	−1	195	140	90	75	55	40			
	0	190	135	90	75	55	40	30	20	15
Upgrade	1	190	135	90	75	55	40			
	2	185	130	90	70	55	40			
	3	180	130	85	70	55	40			
	4	175	125	85	70	55	40			
	4·5	–	125	85	70	55	40			
	5	–	–	85	70	55	40			
	6	–	–	85	75	50	40			
	7	–	–	–	–	50	40			
	8	–	–	–	–	–	40			

NOTE: – Stopping sight distance in meters
– A perception brake reaction time of 1.5 sec. has been taken into consideration

TABLE II–9

Overtaking Sight Distance						
DESIGN SPEED IN km/h	100	80	70	60	50	40
OVERTAKING SIGHT DISTANCE m.	590	440	370	300	240	180

2.3 HORIZONTAL ALIGNMENT

2.3.01 General

The alignment should be as direct as possible after making allowance for the topography. A flowing line that conforms generally to the natural contours of the terrain is preferable to one with long straights that "slashes" through the terrain. In general, flat curves should be used and curves of minimum radius should be avoided, except under critical conditions.
Alignments should avoid sharp curves at the ends of long straights and sudden changes of curvature. Curves should be at least 150 meters long for a central angle of 5 degrees, and should be increased by 30 meters for each 1-degree decrease in central angle (AASTHO). Compound curves with large differences in curvature should be avoided. In compounding, the radius of the flatter circular curve should not be more than 50 per cent greater than the radius of the sharper circular curve (AASTHO). Where this is not practicable, an intermediate curve or spiral curve should be used to provide the necessary transition.
Short straights between two curves in the same direction should be avoided.
The horizontal alignment and vertical alignment should be well co-ordinated.

2.3.02 Straight Between Two Curves in the Same Direction

A straight (between inter-visible curves) should be avoided if its length is shorter than indicated in Table II–10.

Permissible Straight Between Two Unidirectional Curves

TABLE II–10

speed (km/h)	120	100	80	70	60	50
length (m.)	600	400	300	200	100	70

2.3.03 Horizontal Curves

Changes of direction in the horizontal alignment may be designed as follows:
a) the circular curve with two transition curves
b) the simple circular curve
c) the compound curve
d) the compound clothoid
The circular curve with clothoid transition curves is the most common method. In the case of two reverse curves with transition curves, the ratio of the radii should be $\frac{R_2}{R_1} \leq 2$ and the ratio of the parameters of clothoids should be $\frac{A_2}{A_1} \leq 1.5$.
The simple circular curve without transition curve can be used only for curves with radii larger than (or degree of curves less than) or equal to the values shown in Table II–11.
The relationship of the radius of the circular curve to design speed, superelevation and length of the transition curve is shown in Table II–11.

2.3 HORIZONTAL ALIGNMENT

2.3.04 Minimum Radii of Horizontal Circular Curves

TABLE II–11

DESIGN SPEED (km/h)	MINIMUM RADIUS OF CIRCULAR CURVES IN METERS												
	SUPERELEVATION IN %											NO SUPER-ELE-VATION	NO TRANSI-TION-CURVE
	2	2·5	3	3·5	4	4·5	5	5·5	6	6·5	7		
120	2200	1750	1450	1250	1100	975	–	–	–	–	–	3500	5400
	0°16′	0°20′	0°24′	0°28′	0°31′	0°35′	–	–	–	–	–	0°10′	0°06′
100	1500	1200	1000	875	750	675	600	–	–	–	–	2500	3800
	0°23′	0°29′	0°34′	0°39′	0°45′	0°51′	0°57′	–	–	–	–	0°14′	0°09′
80	1000	775	650	550	500	450	400	350	–	–	–	1500	2400
	0°34′	0°44′	0°53′	1°02′	1°09′	1°16′	1°26′	1°38′	–	–	–	0°23′	0°14′
70	750	600	500	425	375	330	300	270	250	–	–	1500	1850
	0°46′	0°57′	1°09′	1°21′	1°32′	1°44′	1°54′	2°07′	2°18′	–	–	0°23′	0°18′
60	550	450	375	325	270	240	220	200	180	170	–	1500	1350
	1°02′	1°16′	1°32′	1°46′	2°07′	2°23′	2°36′	2°52′	3°11′	3°22′	–	0°23′	0°25′
50	375	300	250	220	190	170	150	140	125	120	110	1500	1000
	1°32′	1°54′	2°18′	2°36′	3°01′	3°22′	3°49′	4°06′	4°35′	4°46′	5°12′	0°23′	0°34′

2.3.05 Turning Roadways

Turning roadways should be avoided except for critical conditions in mountainous terrain.

Minimum Radii Allowed for Turning Roadways

TABLE II–12

SUPERELEVATION IN %	DESIGN SPEED IN km/h			
	45	40	35	30
7	75	60	45	35
8	65	50	40	30

2.3.06 Transition Curves

A transition curve is used either between a straight and a circular curve or two straights or between two circular curves of different direction or of same direction but with different radii.
The length of the superelevation-runoff should be the same as the length of the transition curve.
Only clothoids should be used as transition curves. The minimum radii (or maximum degrees of curvature) for circular curves without transition curve are shown in Table II–11.

2.3 HORIZONTAL ALIGNMENT

The minimum length of transition curve is as follows:
a) $L = 1{\cdot}5\ Va$
where the superelevation is applied about the outer edge of outer marginal strip.
b) $L = Va$
where the superelevation is applied about the center line.

L = length of transition curve in meters
Va = design speed in km/h.

From an aesthetic point of view the recommended minimum length of transition curve is:

$$L_{min} = \frac{R}{9}$$

Minimum length of the transition curve in case of compound circular arcs is:

$$L_m = \frac{L\,(R_2 - R_1)}{R_2}$$

where: L_m = length of transition curve between circular curves
L = length of theoretical transition curve between larger radius and a straight
R_1 = the smaller radius
R_2 = the larger radius

Various applications of horizontal curves (circular, clothoid, compound etc.) are shown in the examples (Fig. II–39 onwards).

2.3.07 Superelevation

Superelevation of the carriageway on horizontal curves must be related to the design speed and radius. Maximum values of superelevation according to design speed and minimum radii are given in Table II–11.

2.3.08 Resultant Gradient

Resultant gradient is determined by the formula:

$$r = \sqrt{g^2 + s^2}$$

r – resultant gradient in %.
g – longitudinal gradient in %.
s – superelevation in %.
The computed resultant gradient must not exceed the rates given in Table II–7.
The resultant gradient should not be less than 0.5% and may not be less than 0.3%.

2.3 HORIZONTAL ALIGNMENT

2.3.09 Superelevation Runoff

A gradual change from a normal camber section to a cross section with full superelevation is necessary between tangent and a circular curve. This is known as superelevation runoff. This change in superelevation should be distributed uniformly along the transition curve.
The gradient of superelevation runoff, expressed by ratio 1:n, should have the minimum values of "n" as shown in Table II–13.

A) rotation of carriageway about outer edge of inner marginal strip
B) rotation about outer edge of outer marginal strip (undivided highways).
C) rotation about outer edge of outer marginal strip (divided highways).
D) rotation about center line.

TABLE II–13

Minimum Gradient (n) of Superelevation Runoff

Design speed in km/h	Method of rotation			
	A	B	C	D
Less than 30	150	150	200	75
More than 30	300	300	400	150

The minimum length of superelevation runoff is determined by the ratio 1:n.
In cases where the length of the clothoid transition is greater than the calculated length of superelevtion runoff, the application of superelevation will be made along the length of the clothoid transition curve.

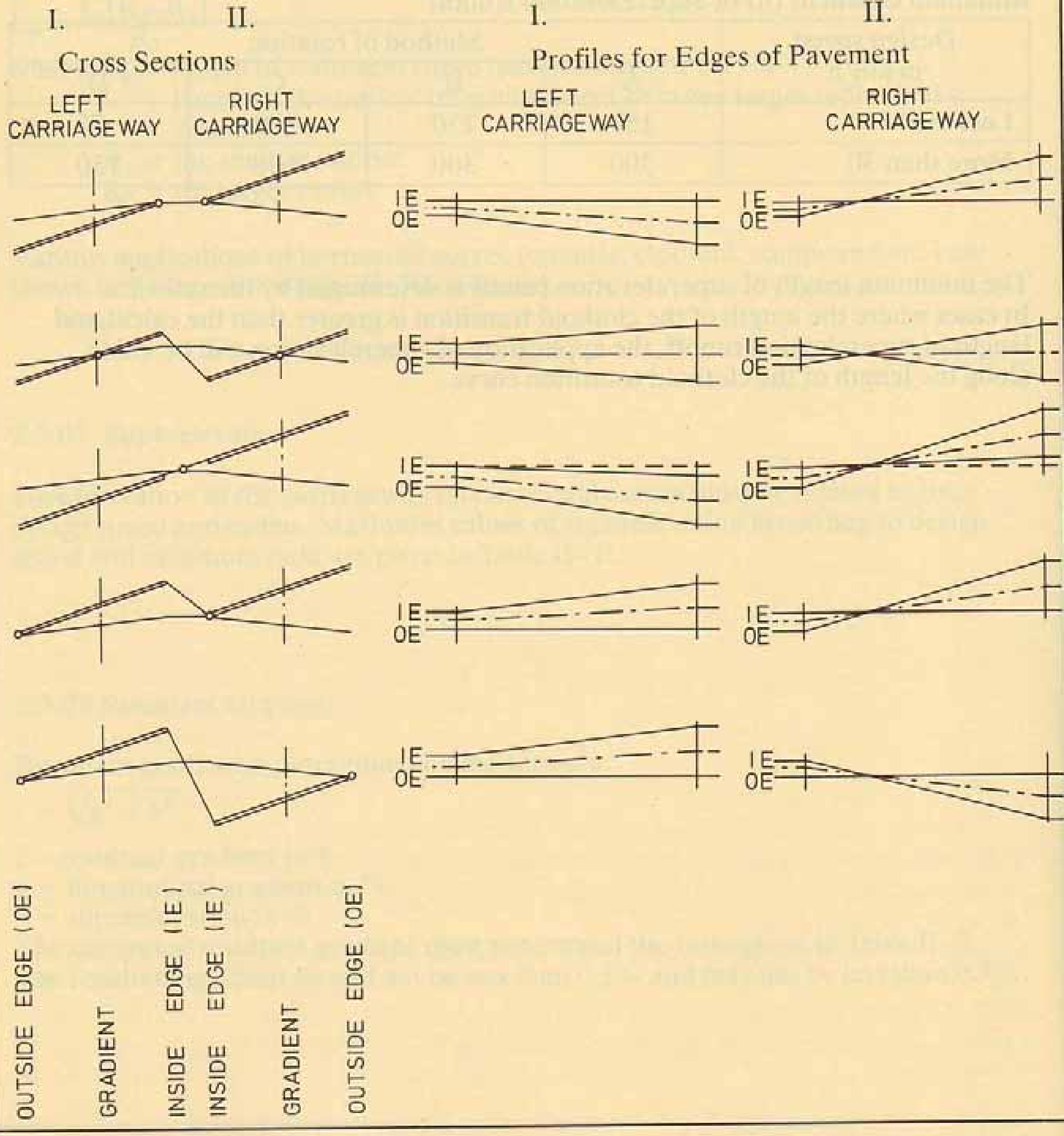
2.3 HORIZONTAL ALIGNMENT

2.3.10 Axis of Rotation

For undivided highways the axis of rotation for superelevation is usually the center line of the road. However, in special cases where curves are preceded by long relatively level straights, the plane of superelevation may be rotated around the inside edge of the pavement to improve perception of the curve. In flat terrain, areas of poor drainage caused by application of superelevation may be avoided by changing the axis of rotation from the center line to the inside edge of the pavement. For a highway with a median that is both narrow and traversable the axis of rotation usually lies in the center of the median.

Other methods of rotation are shown in Figure II–1.

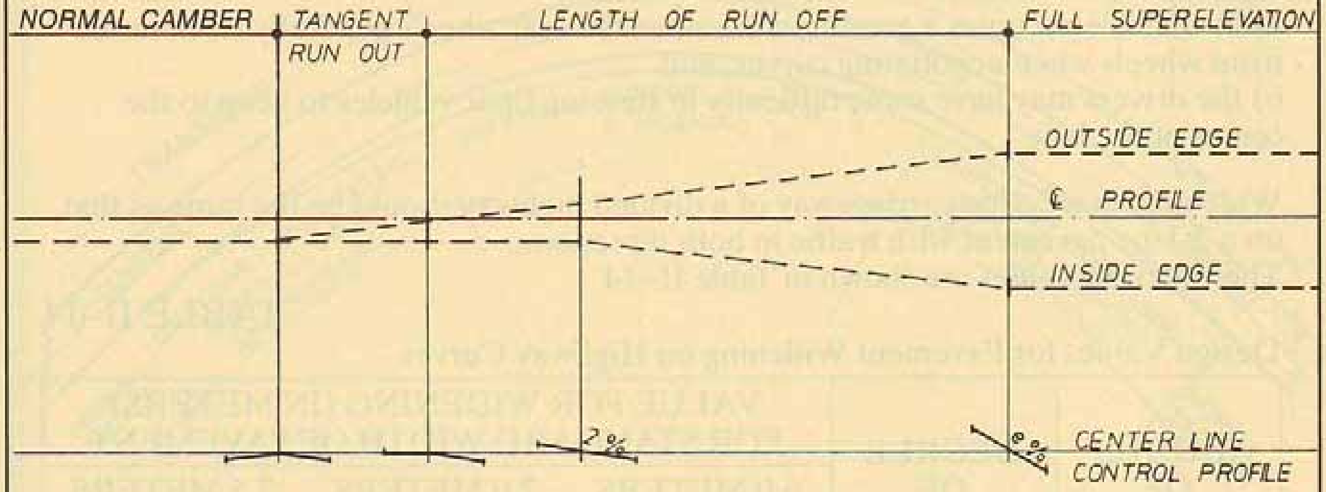
FIGURE II-1 Methods of Rotation of Pavements



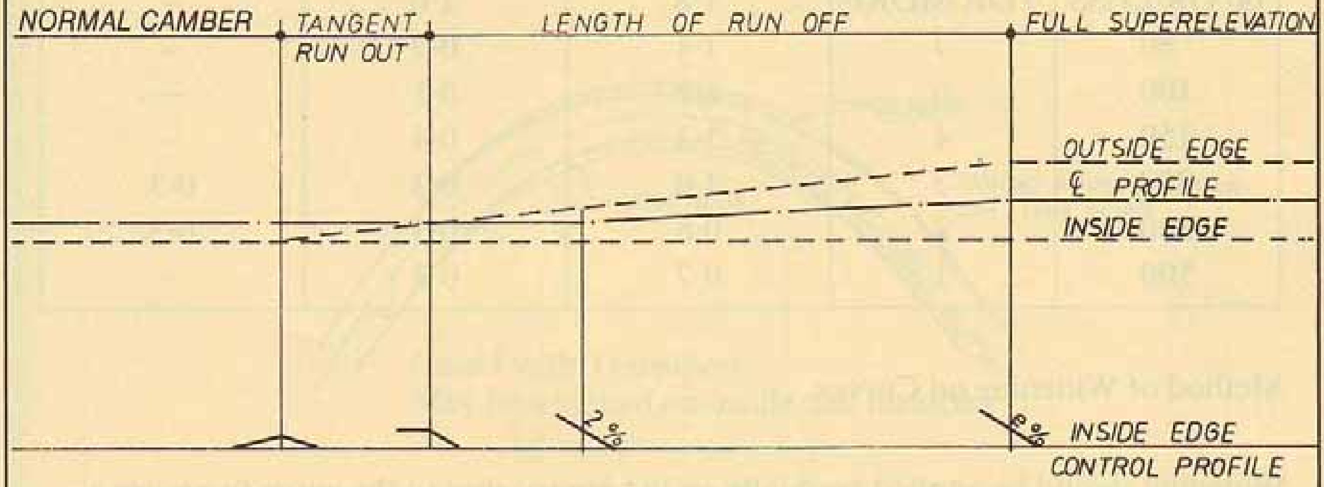
2.3 HORIZONTAL ALIGNMENT

2.3.11 Methods of Attaining Superelevation

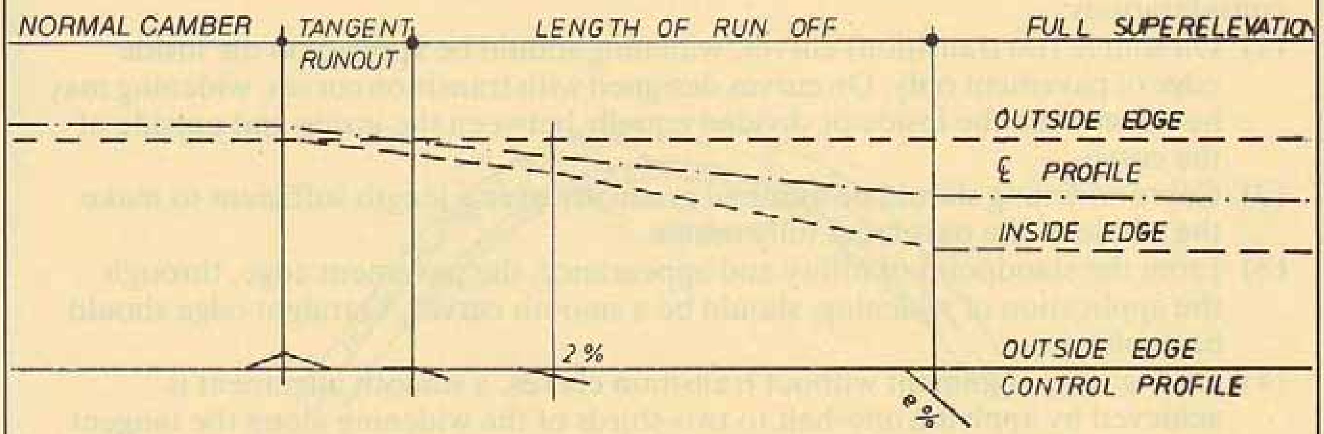
Pavement Rotated about Center line



Pavement Rotated about Inside Edge



Pavement Rotated about Outside Edge



2.3 HORIZONTAL ALIGNMENT

2.3.12 Pavement Widening on Curves

Pavements on curves are widened to make operating conditions on curves comparable to those on straights. Carriageway widening is needed on certain open highway curves because:
a) the vehicle occupies a greater width since the rear wheels generally track inside front wheels when negotiating curves, and
b) the drivers may have some difficulty in steering their vehicles to keep to the center of the lane.

Widening on a 2-lane carriageway of a divided highway should be the same as that on a 2-lane pavement with traffic in both directions.
The widening values are shown in Table II–14

TABLE II–14

Design Values for Pavement Widening on Highway Curves

RADIUS OF CURVE, m	DEGREE OF CURVE	VALUE FOR WIDENING (IN METERS) FOR STANDARD WIDTH OF PAVEMENT		
		6.0 METERS 50–70 km/h	7.0 METERS 60–80 km/h	7.5 METERS 70–120 km/h
60 OR LESS	9 OR MORE	1·8	1·0	–
80	7	1·4	0·7	–
100	6	1·2	0·5	–
150	4	1·1	0·4	–
200	3	1·0	0·3	0·3
300	2	0·8	0·3	0·3
500	1	0·7	0·3	–

Method of Widening on Curves

Widening should be applied gradually on the approaches to the curve to ensure a reasonably smooth alignment on the edge of the pavement and to suit the paths of vehicles entering or leaving the curve. The following are the principal design considerations:

(1) On simple (no transition) curves, widening should be applied on the inside edge of pavement only. On curves designed with transition curves, widening may be effected on the inside or divided equally between the inside and outside of the curve.

(2) Curve widening should be attained gradually over a length sufficient to make the whole of the pavement fully usable.

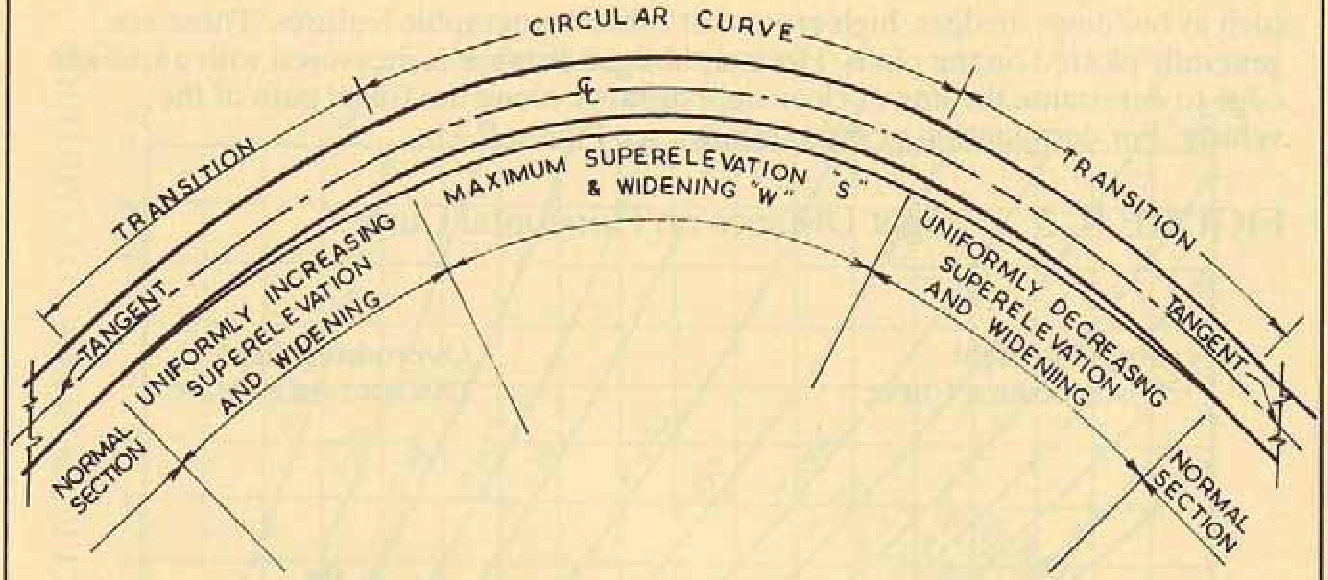
(3) From the standpoint of utility and appearance, the pavement edge, through the application of widening, should be a smooth curve. A straight edge should be avoided.

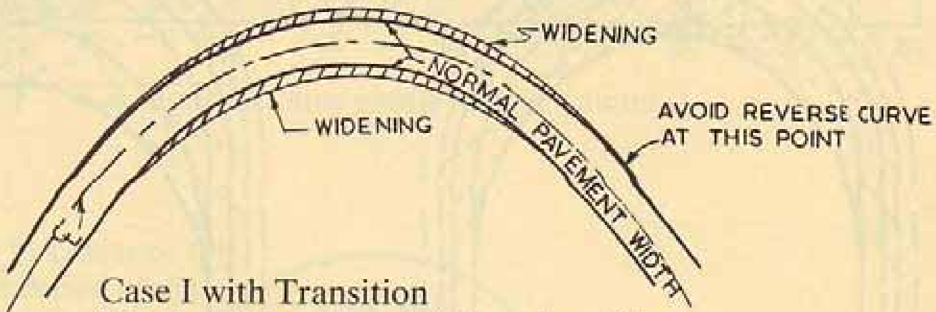
(4) On highway alignment without transition curves, a smooth alignment is achieved by applying one-half to two-thirds of the widening along the tangent and the balance along the curve. This method is commonly used for applying superelevation.

See Figure II–2.

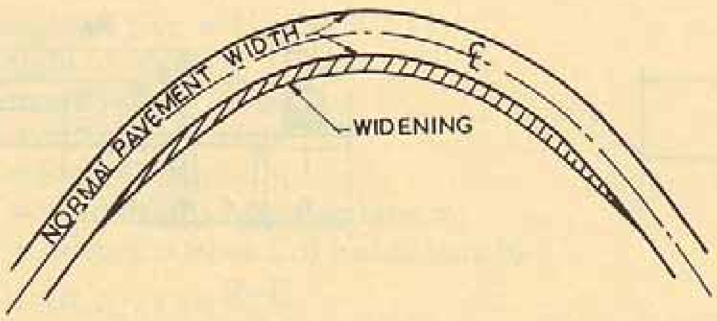
2.3 HORIZONTAL ALIGNMENT

FIGURE II-2 Widening of Pavement in Curves





Case I with Transition
May be widened on inside and outside.



Case II Simple Curve
May be widened on inside only. Usual method.

2.3 HORIZONTAL ALIGNMENT

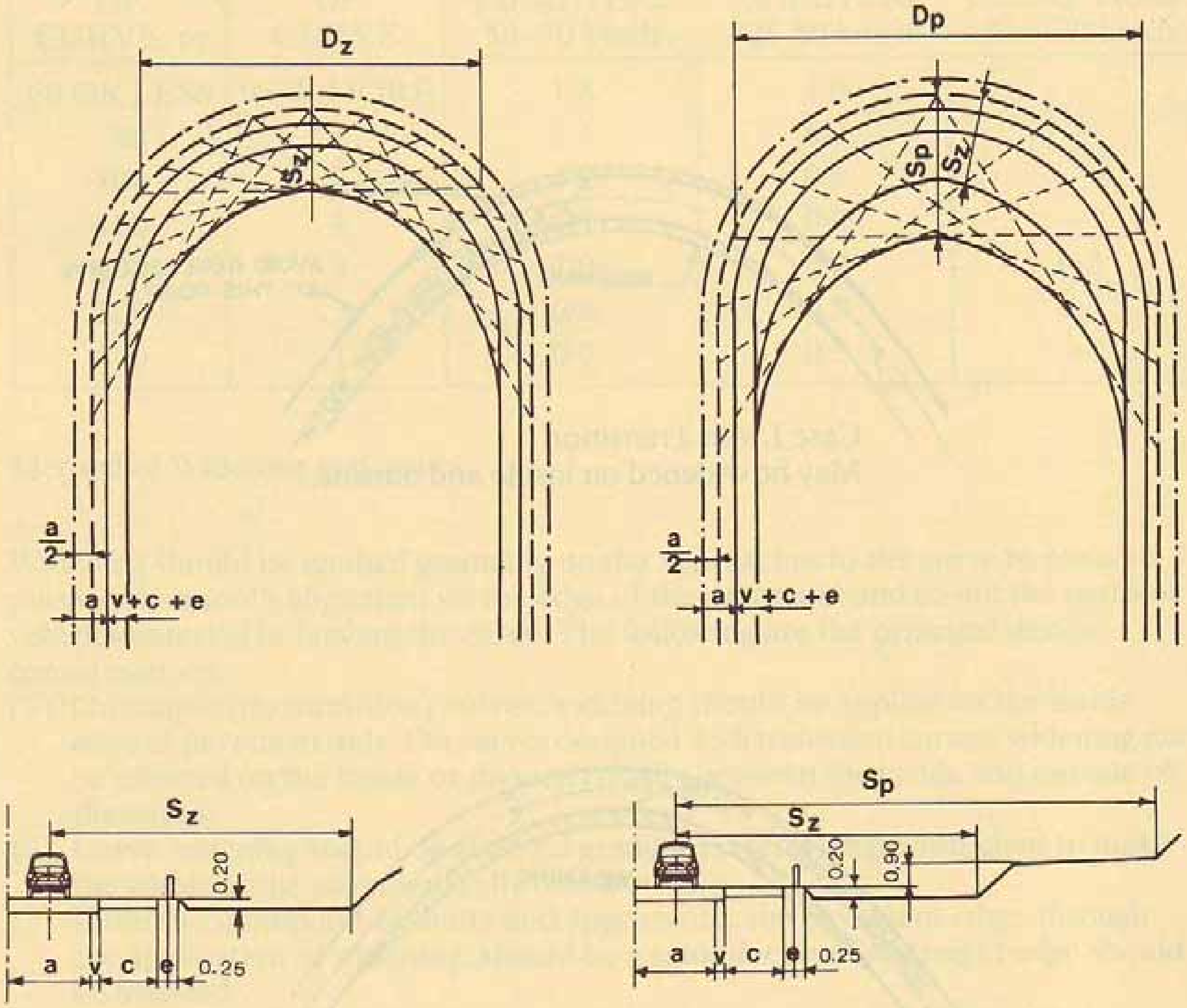
2.3.13 Sight Distance on Horizontal Curves

Horizontal sight distance on the inside of a curve may be limited by obstructions such as buildings, hedges, high ground or other topographic features. These are generally plotted on the plans. Horizontal sight distance is measured with a straight edge to determine the line of clear sight distance along the travel path of the vehicle. For dermination of sight distance see Figure II–3.

FIGURE II–3 Sight Distance on Horizontal Curves

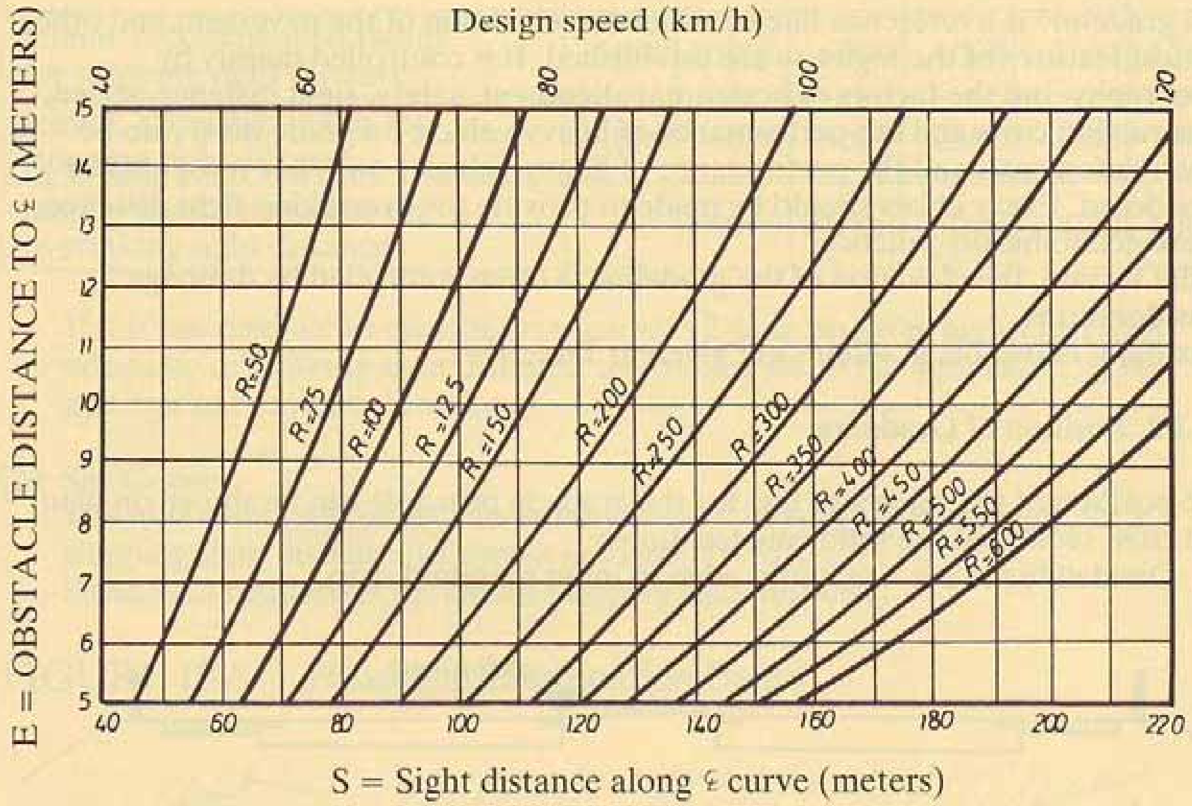
Stopping Sight Distance on a Curve

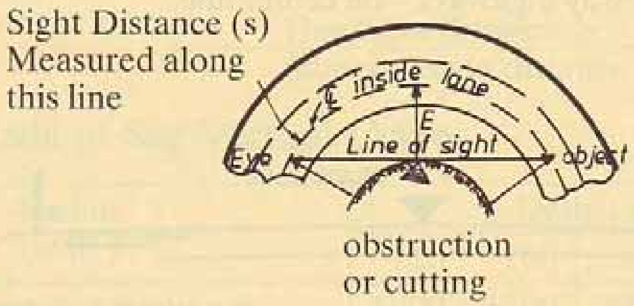
Overtaking Sight Distance on a Curve



2.3 HORIZONTAL ALIGNMENT

FIGURE II–4 Stopping Sight Distance along Horizontal Curves





Height of Eye = 1.15 m.
Height of object = 15 cm.
Line of Sight is 76 cm.
above ℄ of inside lane
S = Sight Distance m.
R = Radius of ℄ of inside lane m.
E = Distance from ℄ of inside lane m.

$$S = 2R \text{ Arc Cos } \frac{R-E}{R}$$

(Arc Cos Expressed in Radians.)

2.4 VERTICAL ALIGNMENT

2.4 VERTICAL ALIGNMENT

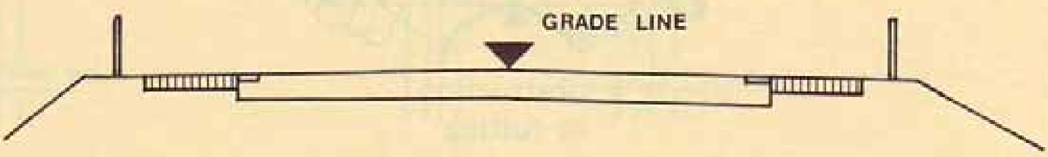
2.4.01 Grade Line

The gradeline is a reference line by which the elevation of the pavement and other vertical features of the highway are established. It is controlled mainly by topography, but the factors of horizontal alignment, safety, sight distance, speed, construction costs and the performance of heavy vehicle on grade must also be construction costs and the performance of heavy vehicles on grade must also be considered. Every effort should be made to provide for overtaking sight distance where economically practical.
In flat terrain, the elevation of the gradeline is often controlled by drainage considerations.
Maximum allowable gradients are given in Table II–7.

2.4.02 Position of Gradeline

The position of the reference line for the grade in principle can be shown on plan and cross section at the following locations:
(1) Divided highways – on outer edge of inner marginal strip.

(2) Undivided 2-lane two-way highways – on center line.



2.4.03 Vertical Curves

The gradual change between longitudinal grades is generally made by a simple parabola (the preferable curve in highway profile design, it closely approximates the circular curve). The parabola is determined by the radius of the osculating circular curve.

(1) Crest Curves
If possible the crest curves of 2-lane two-way highways should have adequate overtaking sight distance.
Crest curves of highways must always provide stopping sight distance.
Minimum values of radii of crest curves are given in Table II–15.

2.4 VERTICAL ALIGNMENT

Minimum Radii of Crest Vertical Curves

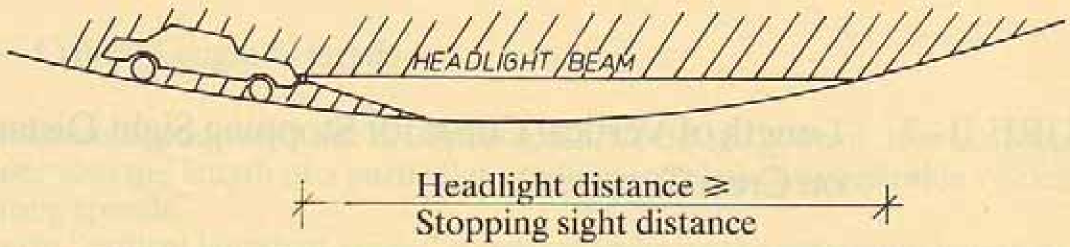
TABLE II–15

Radius in m.	Design speed in km/h					
	120	100	80	70	60	50
minim. recommended radius for stopping sight distance	15000	10000	5000	4000	2500	1500
minim. allowable radius for stopping sight distance	11000	6000	3000	2500	1500	1000
minim. allowable radius for overtaking sight distance	—	36000	21000	15000	10000	6000

If it is not possible to provide crest curves (2-lane two-way highways) with adequate overtaking sight distance, overtaking should be prevented by the appropriate pavement marking.

(2) Sag Curves
The minimum radii of sag curves must ensure headlight illumination for a stopping sight distance (as shown in Figure II–5).
Head light distance ≥ minimum stopping sight distance.

FIGURE II–5 Headlight Illumination Beam



Minimum Radii of Sag Vertical Curves

TABLE II–16

Radius (in m.)	Design speed (in km/h)					
	120	100	80	70	60	50
recommended minimum	6000	4200	3000	2000	1500	1200
allowable minimum	5000	3400	2000	1500	1000	700

Radii of vertical curves should be as large as possible. The smaller the difference in longitudinal grades, the greater the radius that should be used.
A straight line between two crest or between two sag curves is aesthetically poor and should be replaced by a vertical curve of large radius.
The length of straight grade between reverse vertical curves shall be designed according to the formula:

$$Lm = \frac{100\,Vn^2}{Rv}$$

Lm = the horizontal length of straight line in m
Vn = design speed in km/h
Rv = radius of crest curve in m

2.4 VERTICAL ALIGNMENT

FIGURE II–6 Length of Vertical Curve for Overtaking Sight Distance on Crest Vertical Curves

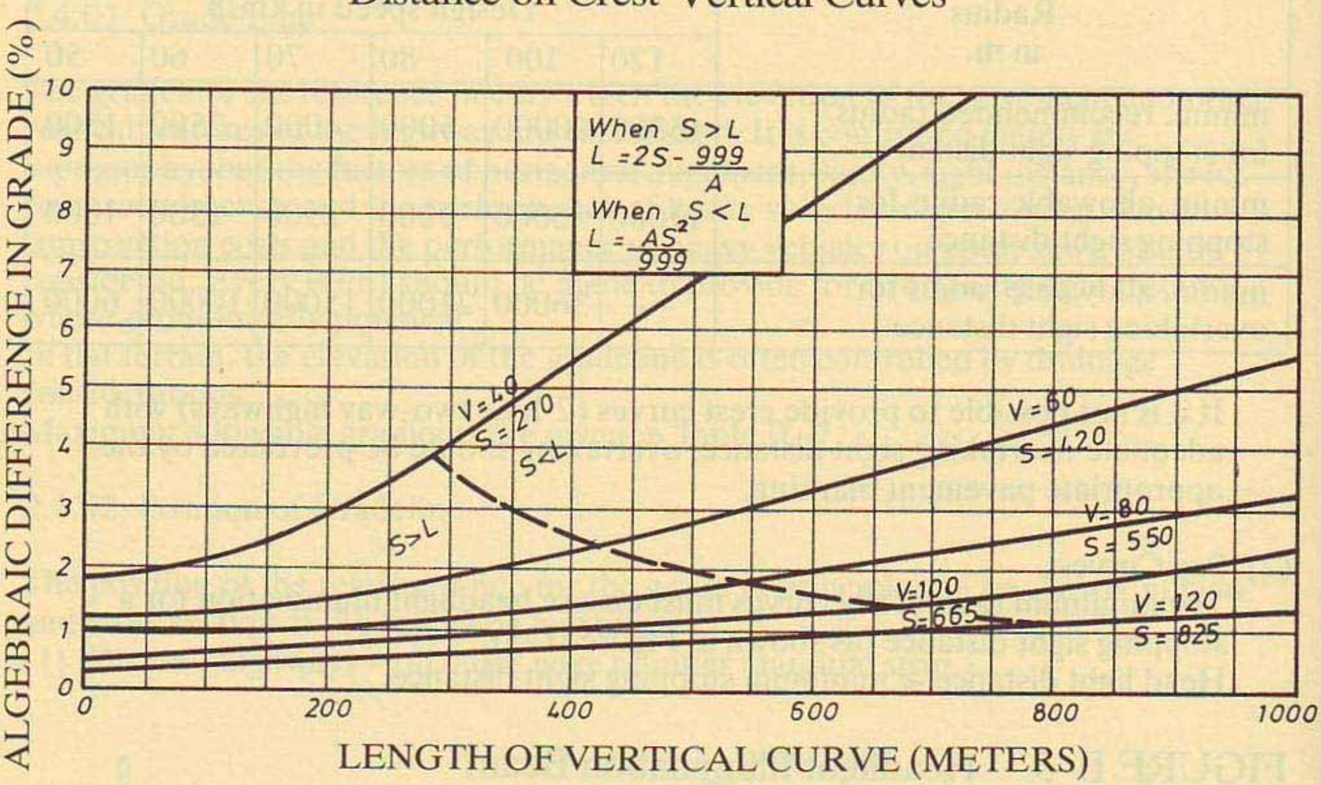
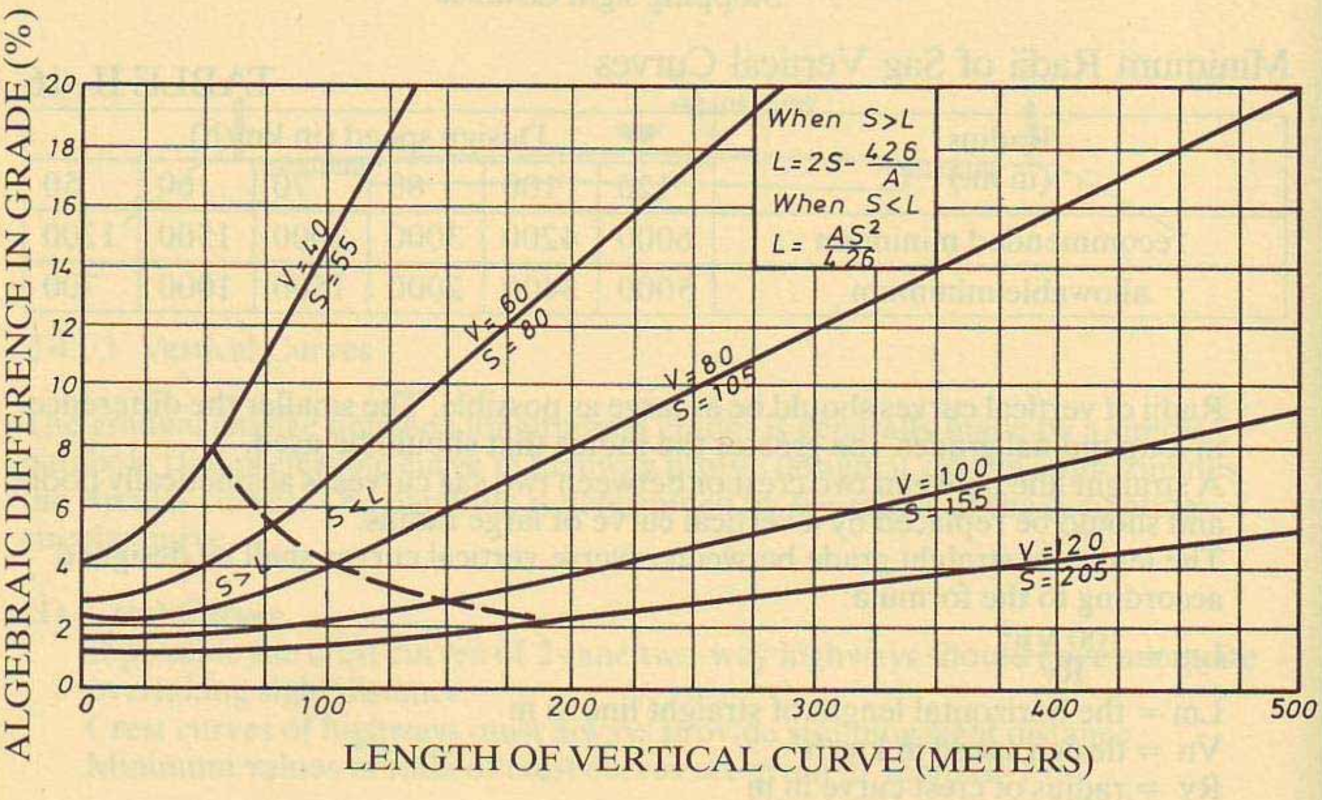
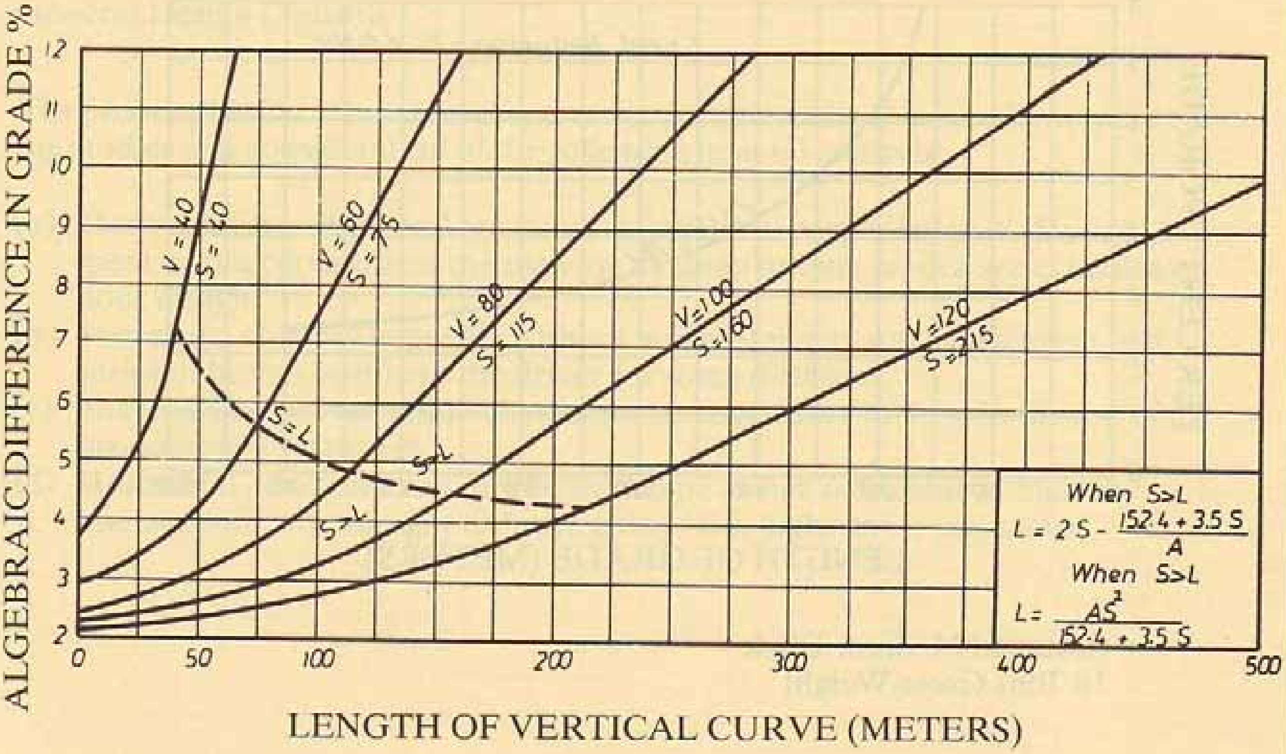


FIGURE II–7 Length of Vertical Curve for Stopping Sight Distance on Crest Curves



2.4 VERTICAL ALIGNMENT

FIGURE II–8 Length of Vertical Curve for Headlight Sight Distance on Sag Vertical Curves



2.4.04 Critical Length of Grade

The maximum gradient is not in itself a complete design control. It is necessary to consider also the length of a particular gradient in relation to desirable vehicle operating speeds.

The term "critical length of grade" is used to indicate the maximum length of climb upon which a loaded truck can operate without an unreasonable reduction in speed. For a given gradient, lengths less than ,,critical" ensure acceptable operation in the desired range of speeds. If the desired freedom of operation is to be maintained on grades longer than "critical", design adjustments such as change of alignment to reduce grades or addition of extra lanes, should be made. The critical length of gradient criterion is used with other pertinent considerations, such as traffic volume in relation to capacity, to determine where climbing lanes are warranted.

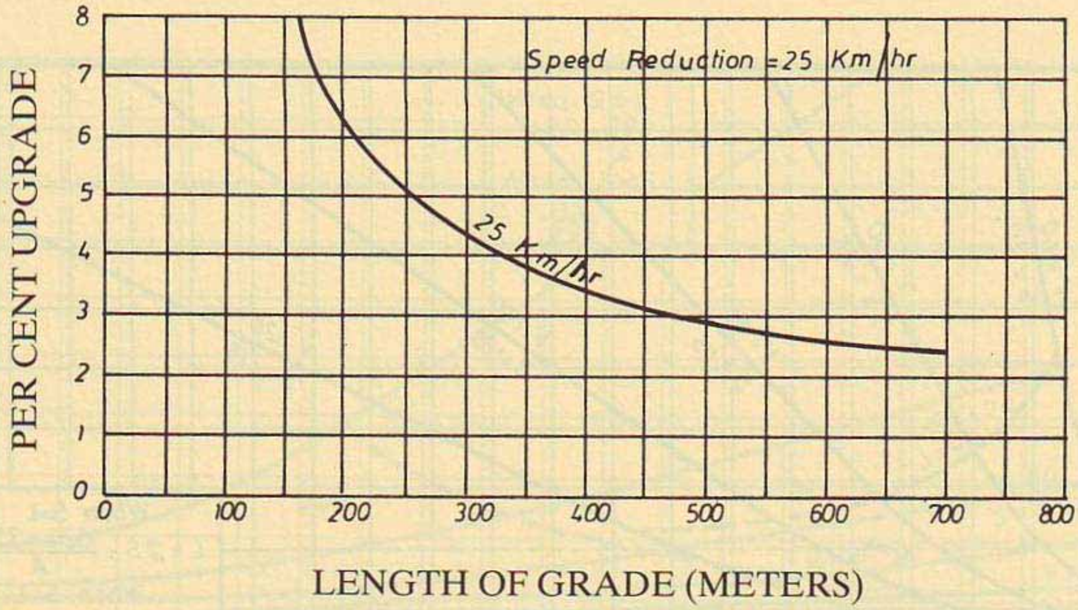
Design values for critical length of gradient, for which speed operation of trucks is the determining factor, are set by Figure II–9.

The design criterion for determining the critical length of grade is not intended as a strict control, but as a guide. In some instances the terrain or other physical controls may preclude shortening or flattening grades to meet these controls. Where a speed reduction greater than the suggested design guide cannot be avoided, undesirable types of operation may result on roads with numerous trucks, particularly on 2-lane roads with volume approaching capacity and, in some instances, on multilane highways.

Where the length of critical grade is exceeded, consideration should be given to providing an added uphill climbing lane for slow moving vehicles, particularly where the volume is at or near capacity and the truck volume is high.

2.4 VERTICAL ALIGNMENT

FIGURE II–9 Critical Length of Grade



Assumed Medium Truck
18 Tons Gross Weight

2.5 PHASING OF HORIZ. & VERT. ALIGNMENT

2.5 Phasing of Horizontal and Vertical alignment

General Design Controls

The proper phasing of horizontal and vertical alignment is achieved by engineering studies and consideration of the following general controls:

(a) Horizontal curvature and gradients should be in proper balance. Straight alignment or flat curvature at the expense of steep or long grades are evidence of poor design.
(b) Successive changes in profile without horizontal curvature may result in a series of humps visible to the driver for some distance.
(c) Sharp horizontal curvature should not be introduced at or near the top of a pronounced crest curve.
(d) Horizontal curvature and profile should be as flat as feasible at highway intersections, where sight distance along both highways is important.

2.5 PHASING OF HORIZ. & VERT. ALIGNMENT

Figure II–10 Compound Curve with Ratio of the Radii 1:10

INCORRECT DESIGN

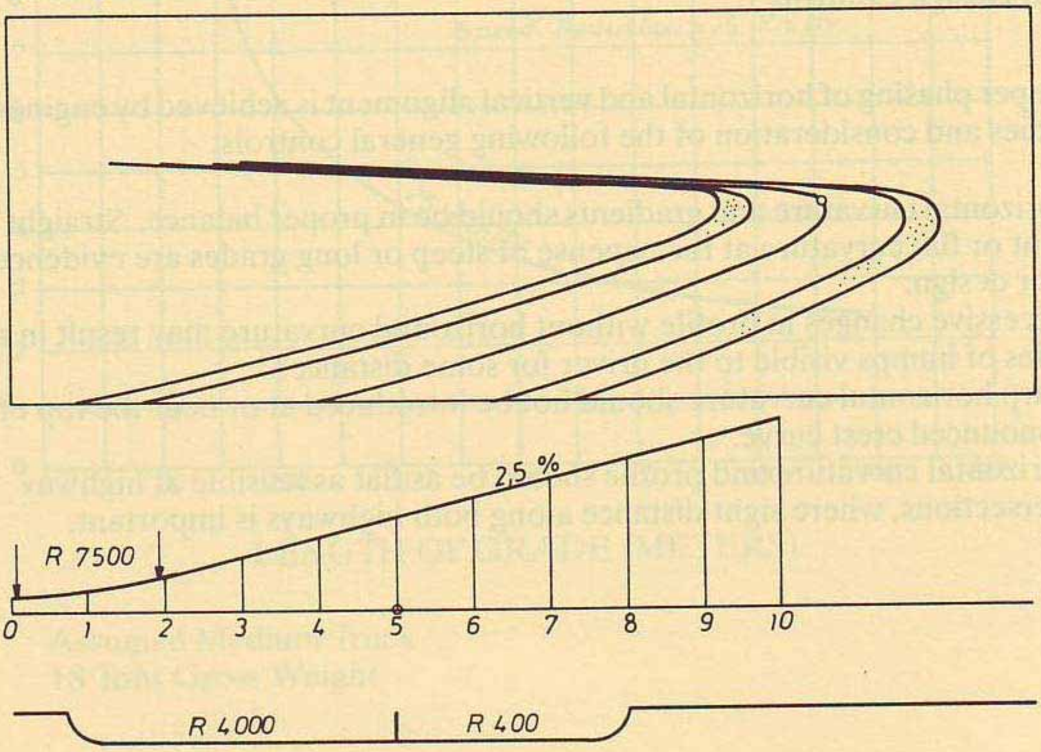
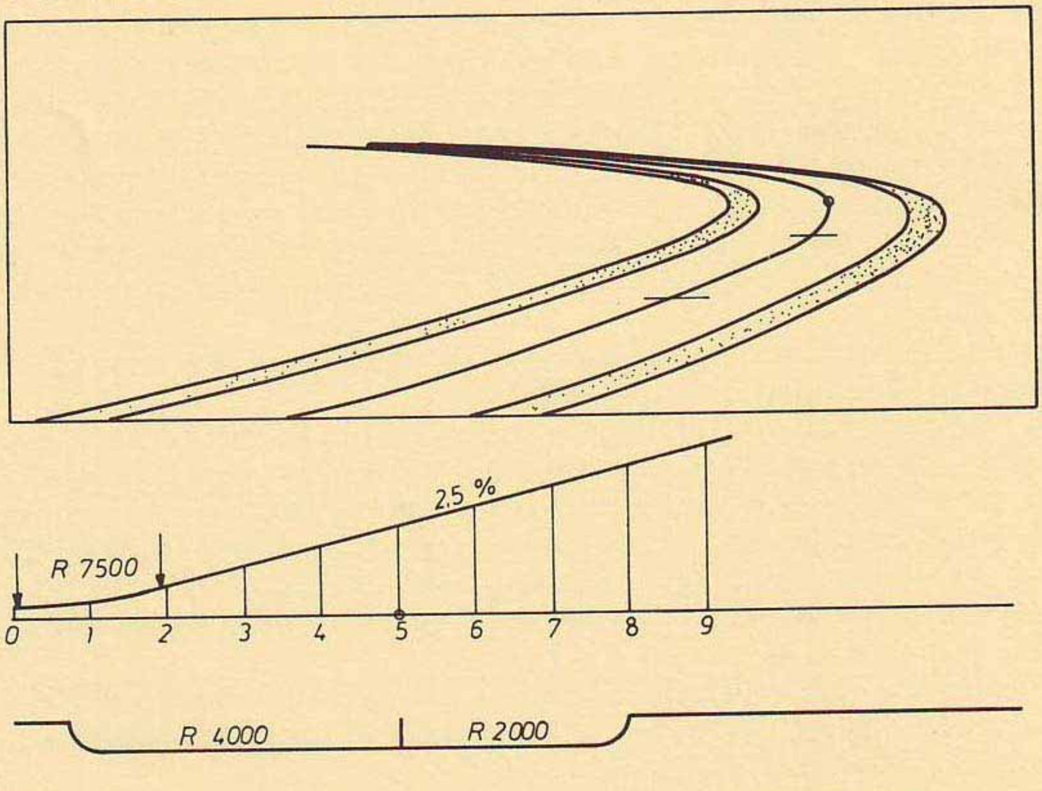


FIGURE II–11 Compound Curve with Ratio of the Radii 1:2

CORRECT DESIGN



2.5 PHASING OF HORIZ. & VERT. ALIGNMENT

FIGURE II–12 Radius R= 1000 m at a Small Centre Angle 5° Looks Like a Break in the Alignment

INCORRECT DESIGN

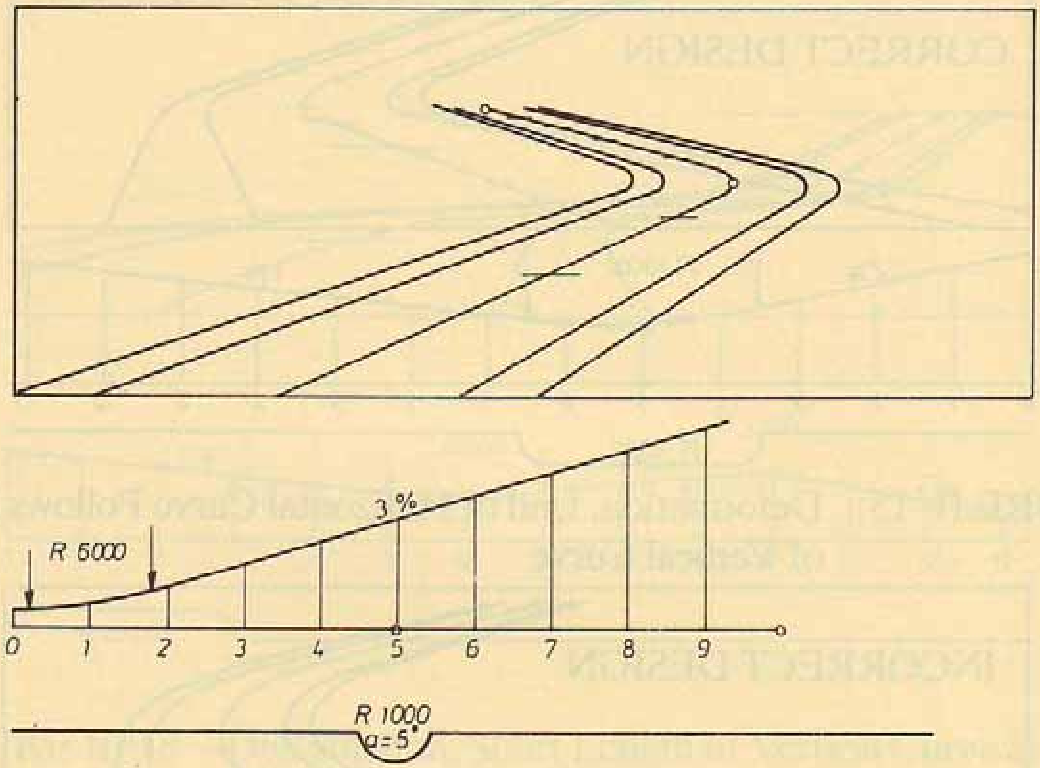
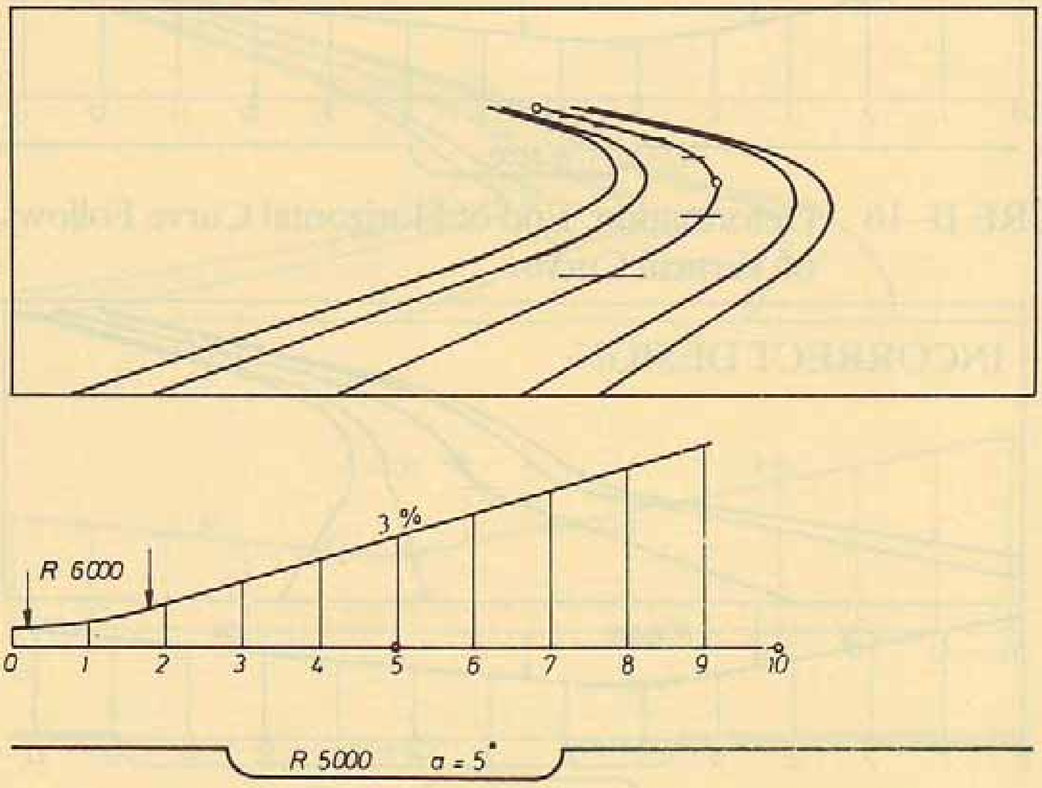


FIGURE II–13 Radius R= 5000 m Appears Natural

CORRECT DESIGN



2.5 PHASING OF HORIZ. & VERT. ALIGNMENT

FIGURE II–14 No Deformation, Horizontal and Vertical Curves in Accord

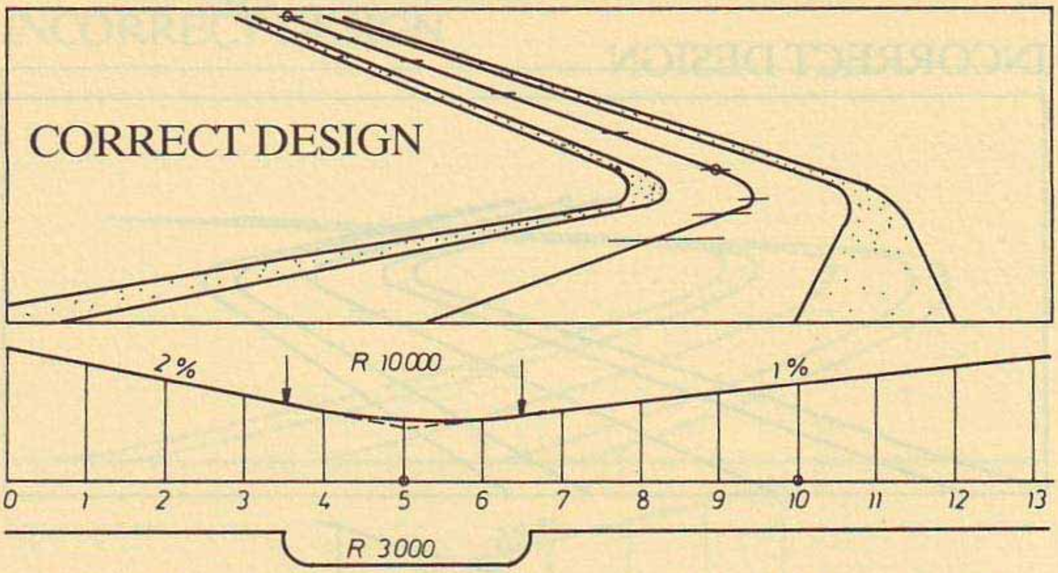


FIGURE II–15 Deformation, End of Horizontal Curve Follows End of Vertical Curve

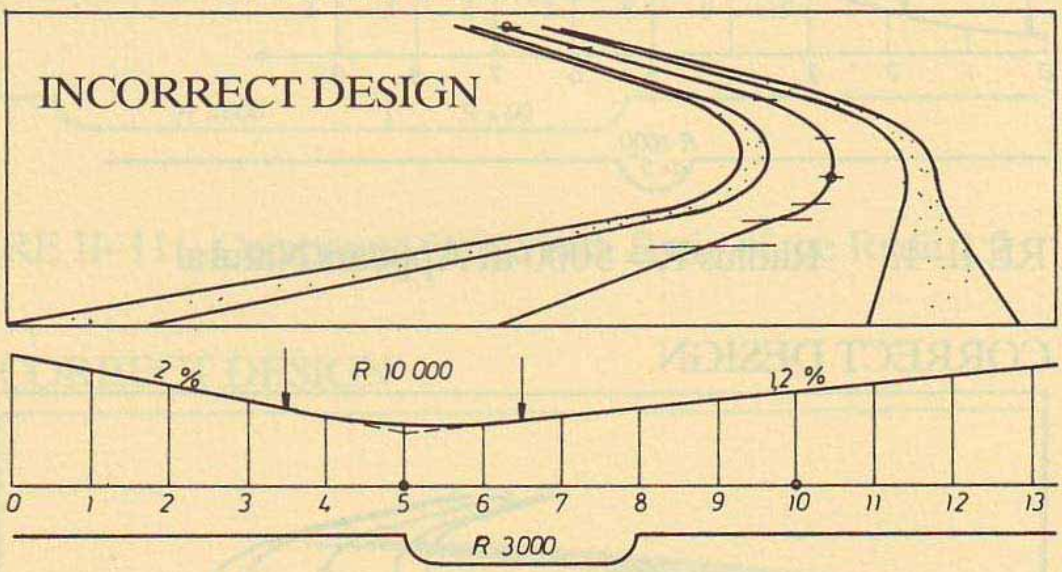
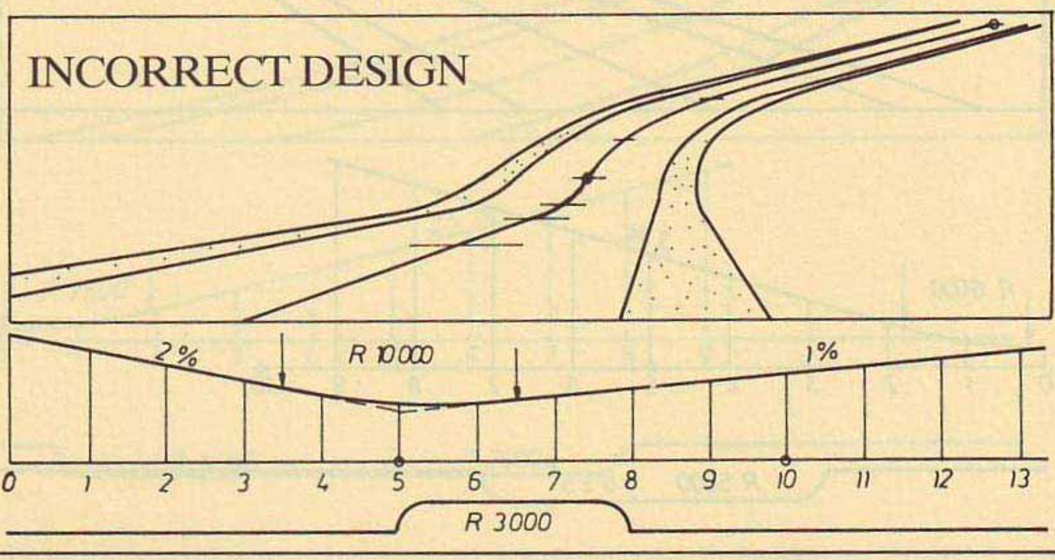


FIGURE II–16 Deformation, End of Horizontal Curve Follows End of Vertical Curve



2.5 PHASING OF HORIZ. & VERT. ALIGNMENT

FIGURE II–17 Deformation, Vertical Curve Follows Horizontal Curve

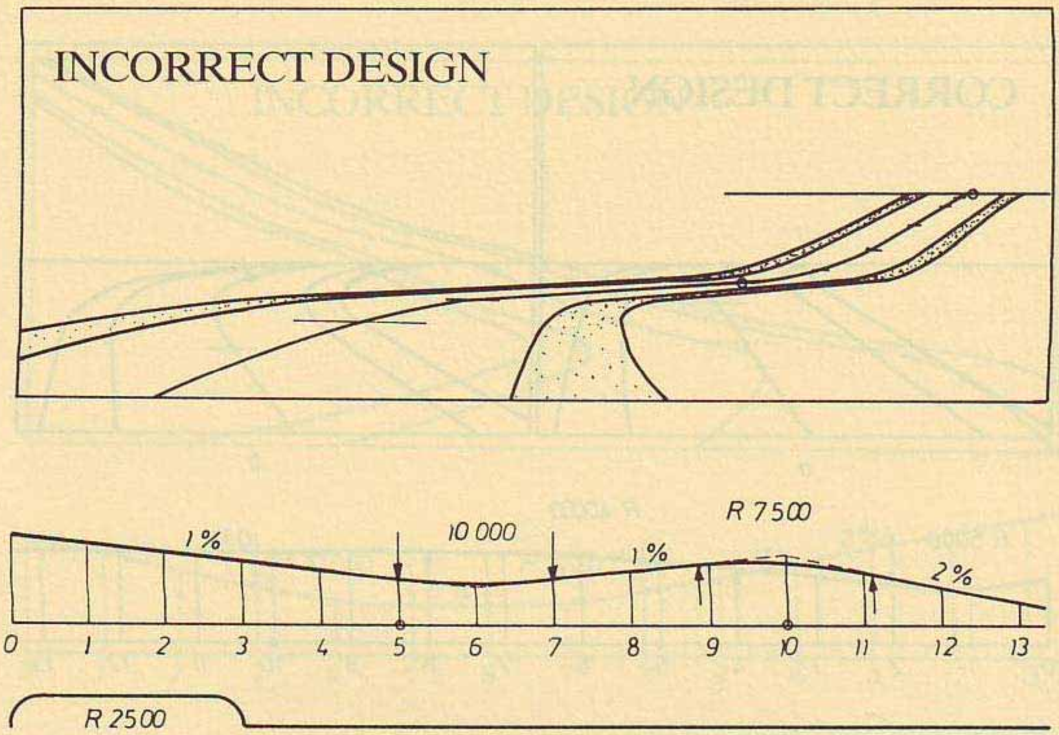
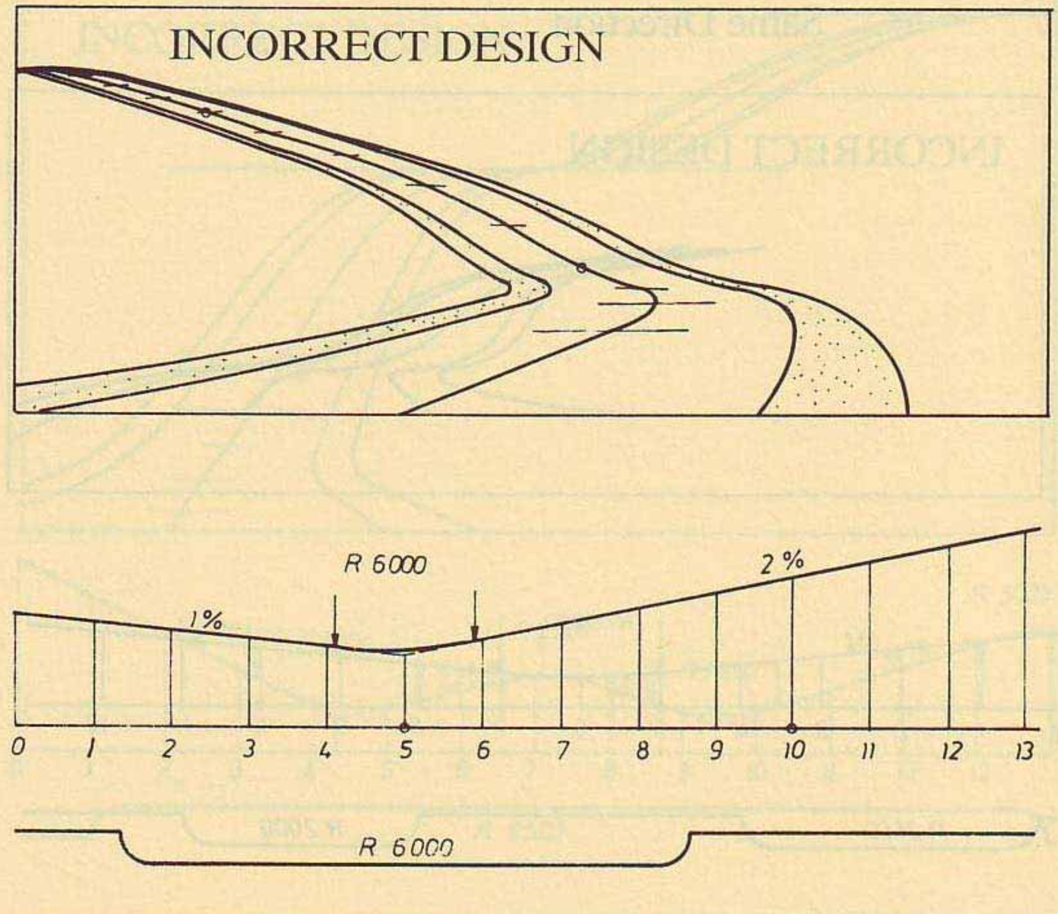


FIGURE II–18 Deformation, Short Length of Vertical Curve at Long Horizontal Curve



2.5 PHASING OF HORIZ. & VERT. ALIGNMENT

FIGURE II–19 No Deformation, Beginning of Horizontal Curve is before Vertical Curve

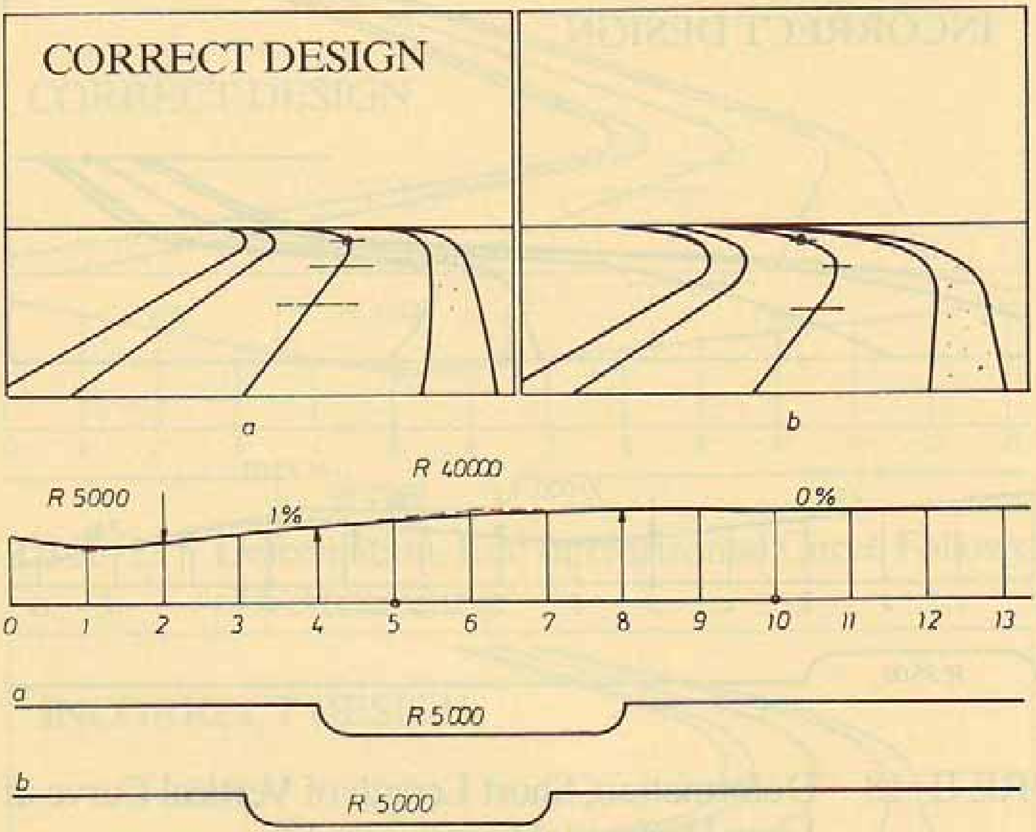
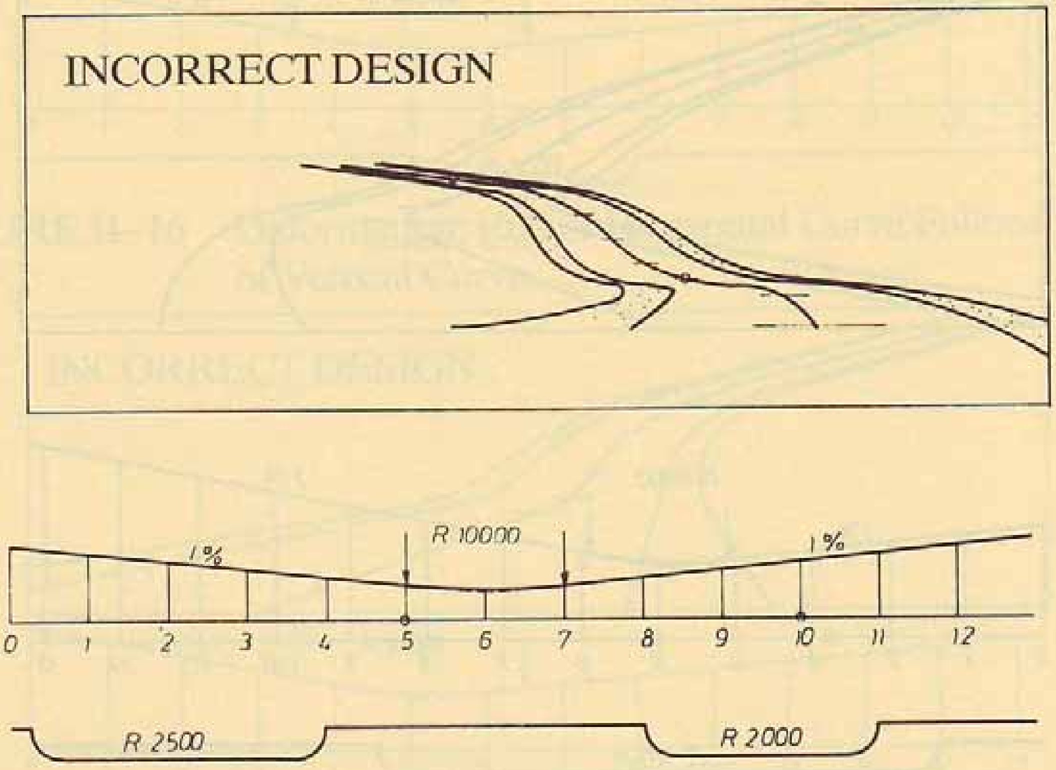


FIGURE II–20 Deformation, Sag Curve between Two Curves of the Same Direction



2.5 PHASING OF HORIZ. & VERT. ALIGNMENT

FIGURE II–21 Deformation, Double Sag Curves at One Horizontal Curve

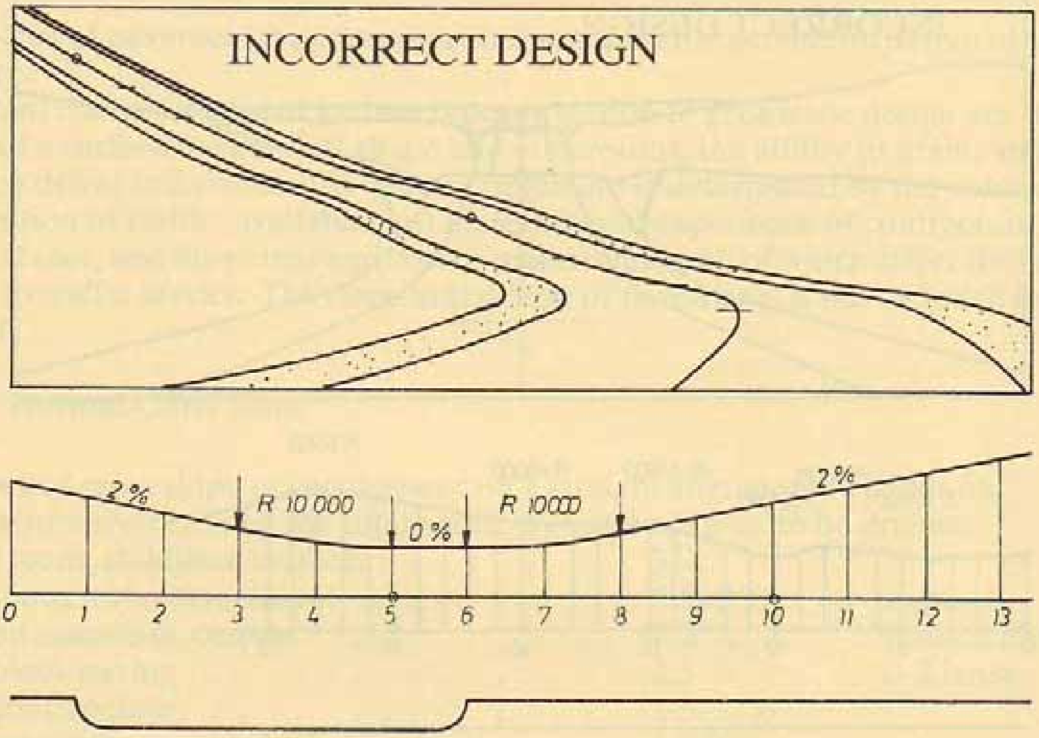
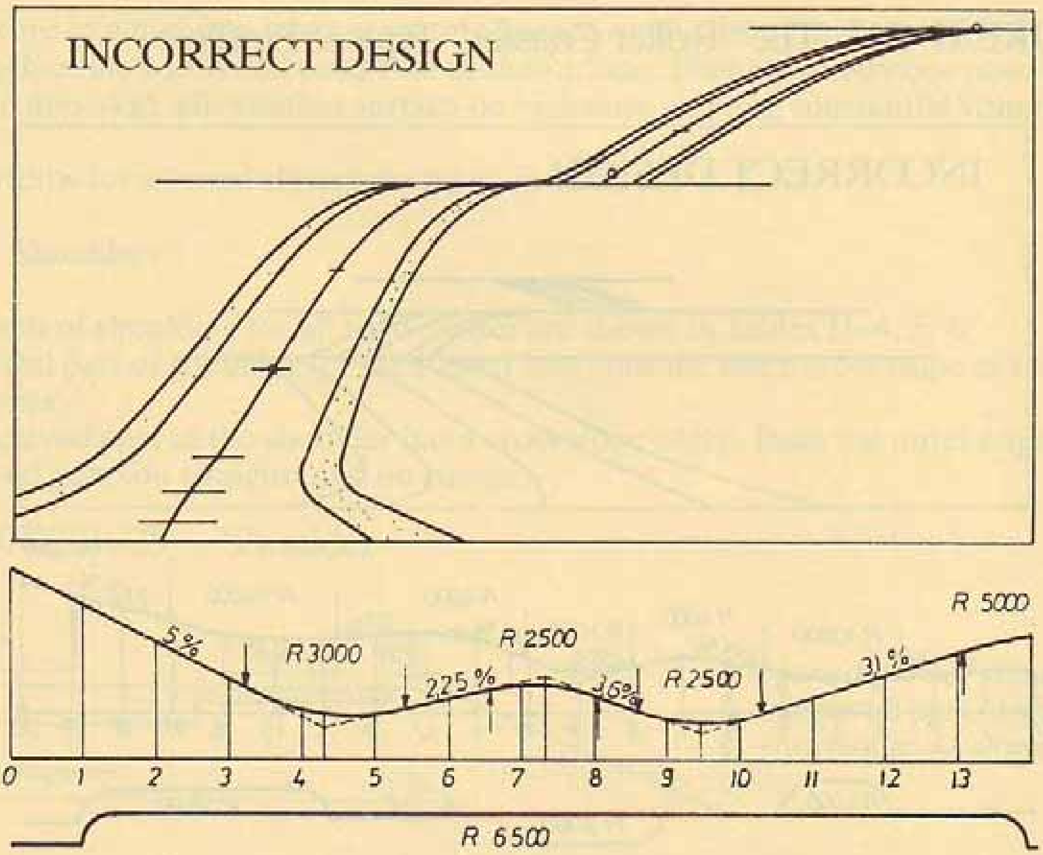


FIGURE II–22 No Phasing of Horizontal and Vertical Curves



2.5 PHASING OF HORIZ. & VERT. ALIGNMENT

FIGURE II–23 The "Hidden-dip" Type of Profile

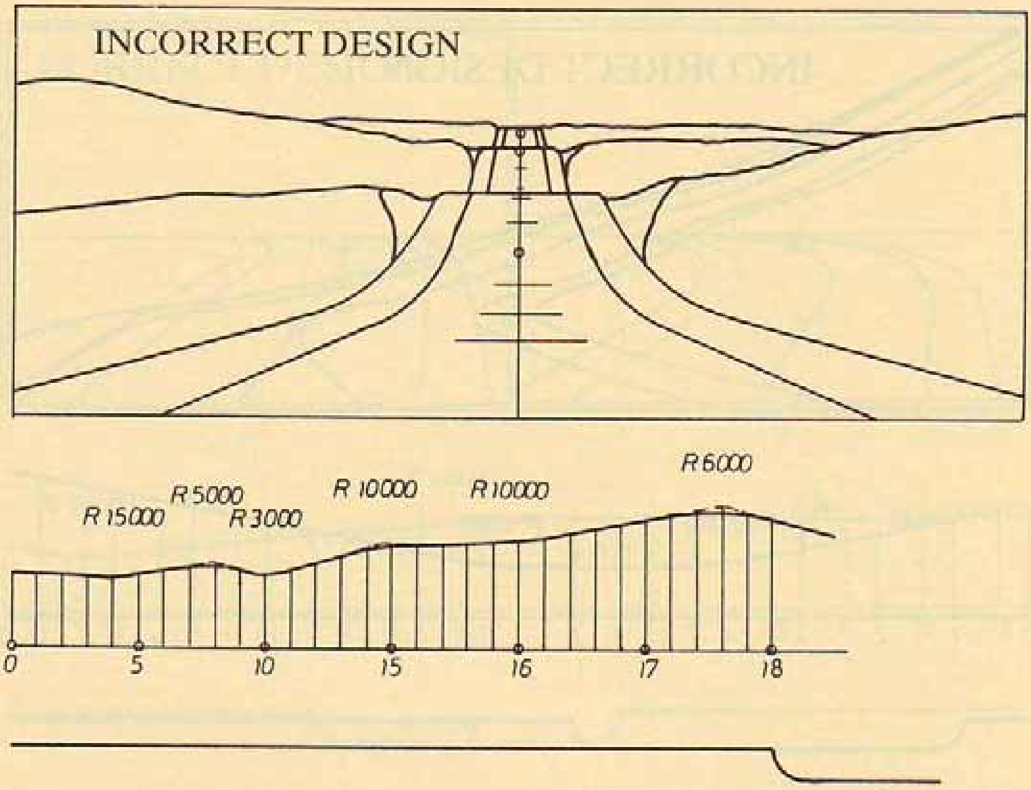
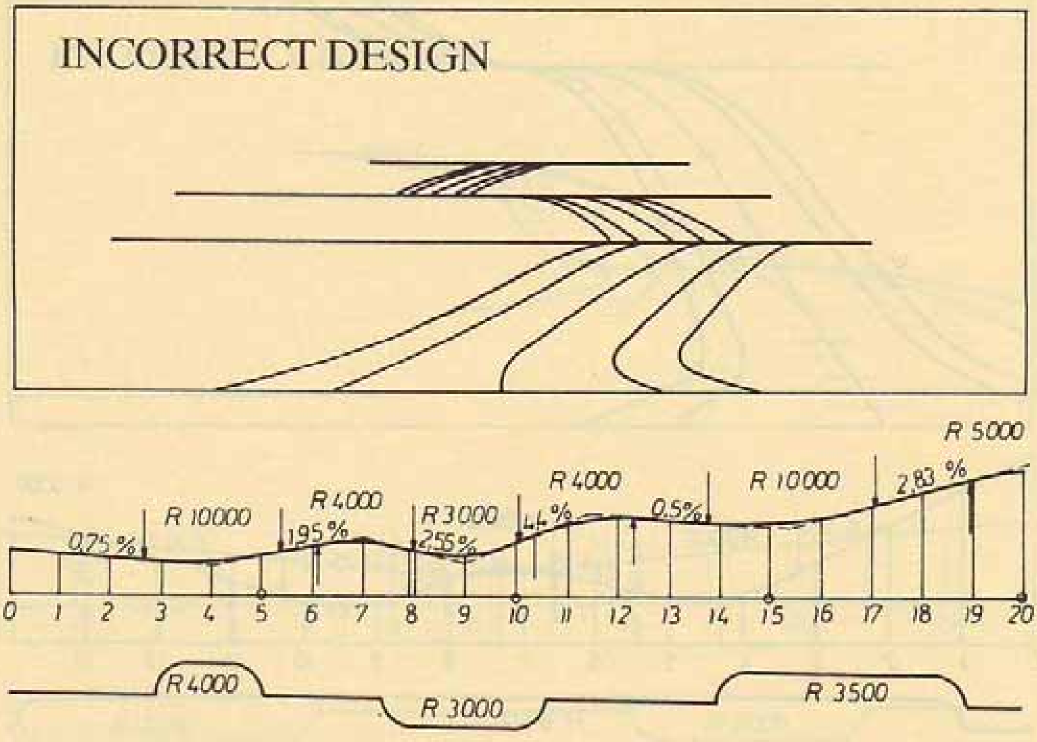


FIGURE II–24 The "Roller Coaster" Type of Profile



2.6 CROSS SECTION ELEMENTS

2.6.01 Pavement

The choice of pavement type has some influence on the geometric design of the highway.
Important characteristics of surface type in relation to geometric design are the ability of a surface to retain its shape and dimensions, the ability to drain, and the effect on driver behaviour. The type of pavement is determined by the volume and composition of traffic, availability of materials and experience of contractors, the inital cost, and the extent and cost of maintenance, all of which affect the relation of cost to traffic service. The structural design of pavements is not included in this manual.

2.6.02 Normal Cross Slope

The normal cross slope of carriageway on a straight alignment depends on the type of pavement and the total width of the paved area to be drained:

Gravel roads, stabilized surfaces	3%	
Bituminous surface treatment	2.5–3%	width of
Bitumen macadam, carpets	2.5 %	drained area:
Stone block paving	2.5 %	1–2 lanes
Asphaltic concrete	1.5–2 %	
Concrete road	1.5 %	

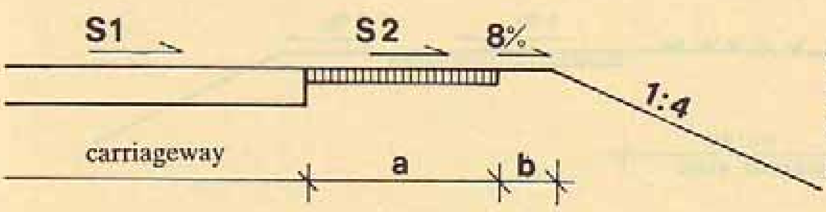
2.6.03 Lane Width

No feature of a highway has a greater influence on the safety and comfort of driving than the width and condition of the surface. There is an obvious need for a smooth, non-skid, all-weather surface on highways carrying substantial volumes of traffic.
Lane widths for all road classes are given in Tables II–4, 5, 6.

2.6.04 Shoulders

The width of shoulders for all road classes are shown in Tables II–4, 5, 6.
The paved part of shoulder (or additional lane) has the same cross slope as the carriageway.
The unpaved part of the shoulder has a cross slope of 8% from the outer edge of the paved part (on straights and on curves).

FIGURE II–25 Shoulder



a – paved part of shoulder
b – unpaved part of shoulder
s_1 – normal cross slope
$s_2 - s_1$

2.6 CROSS SECTION ELEMENTS

If shoulders are to receive use, they must be sufficiently stable to support occasional vehicle loads in all kinds of weather without rutting. Unstabilized shoulders are frequently hazardous. Skidding or overturning of vehicles are not uncommon accidents resulting from loose gravel, sandy, muddy, soft or spongy shoulders.

2.6.05 Curbs

Curbs are used extensively on all types of urban highways to control drainage, to prevent vehicles from leaving the pavement at hazardous points, to protect pedestrians, to delineate the edge of pavement, to present more finished appearance, and to assist in the orderly development of the roadside.
In strictly rural areas, there are long stretches without curbs. The two general classes of curbs are "barrier" and "mountable" curbs and each has numerous variations of type and design.
Barrier curbs are relatively high and steep faced, designed to inhibit, or at least discourage, vehicles from leaving the pavement.
Their height is from 15 to 25 cm.
Mountable curbs are so designed that vehicles can cross them readily when required. They are low and have sloping faces.

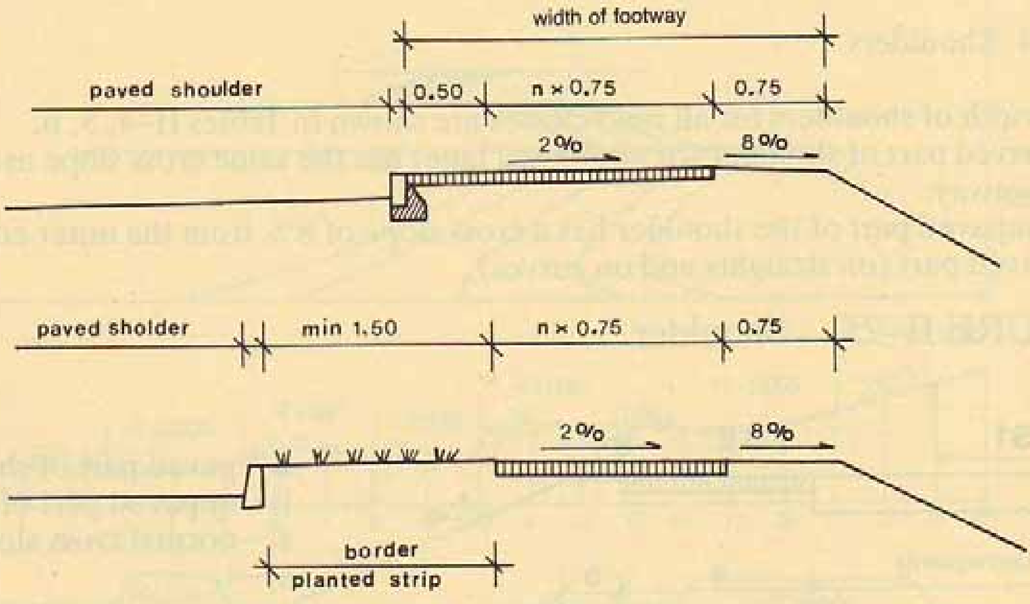
2.6.06 Marginal Strips

Marginal strips have to be provided for all road classes. To be fully effective they should be highly visible (even during bad weather, rain, fog). Their colour will be the same as the colour of pavement marking.
Widths of marginal strips are given in Table II–1, 2, 3.

2.6.07 Footways

In cases where footways are to be provided, their width is determined as follows:

FIGURE II–26 Footways

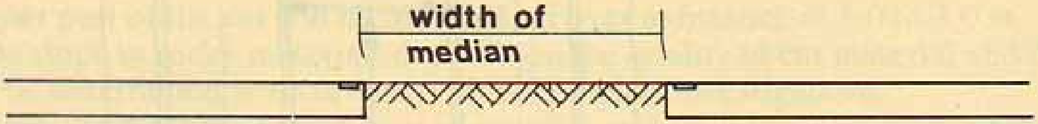


n–number of strips for pedestrians

2.6 CROSS SECTION ELEMENTS

2.6.08 Medians

The median is used for separating opposing traffic lanes on multilane highways. Medians on rural highways should provide the desired protection from interference by opposing traffic, should minimize headlight glare, should render space for safe use by crossing and turning vehicles at at-grade intersections, and should provide a layby in case of emergency.

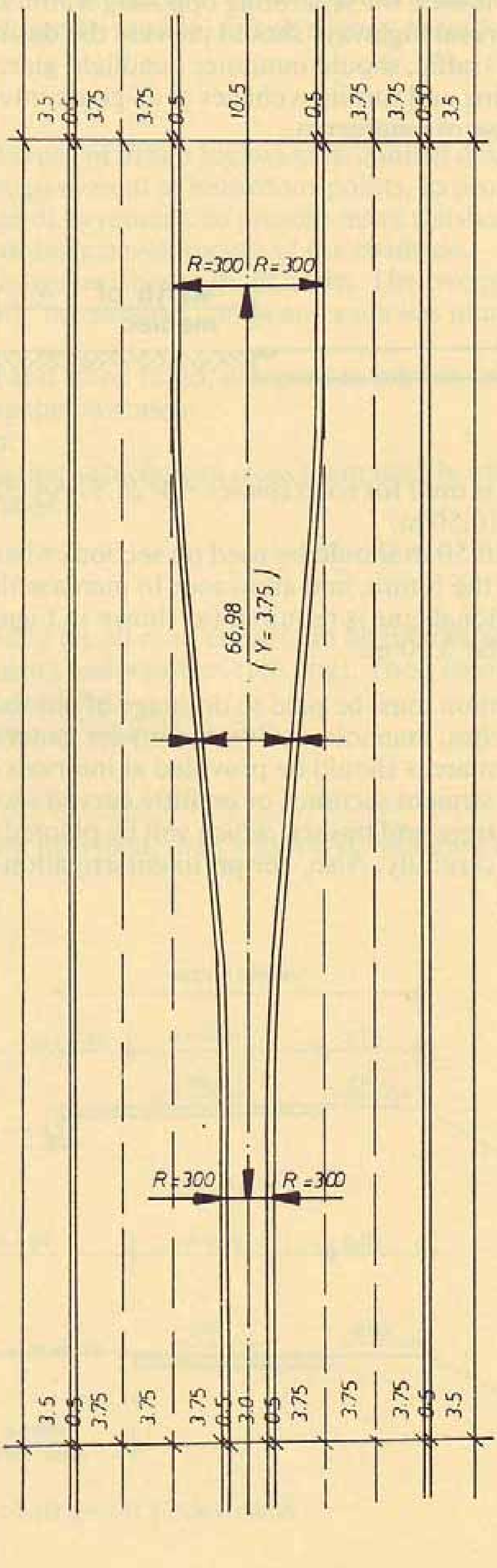


The median is used for road classes A4/25.5; A4/33; A6/33; A6/40.5 in widths of 3.00 m and 10.50 m.
A width of 10.50 m should be used on sections where the central reserve is necessary in the future, and allowance to increase the width of both carriageways by one additional lane is required (as shown in Figure II–27); the final width of the median will be 3.00 m.

Special attention must be paid to drainage of medians. For this purpose, drainage systems (ditches, manholes, culverts etc.) are generally used.
Paved U-turn areas should be provided at intervals of about 2 km; these should be provided on straight sections, or on little curved sections, only.
All kinds of trees and bushes, which will be planted in the median, have to be chosen very carefully. Also, the pertinent irrigation system must not damage the road construction.

2.6 CROSS SECTION ELEMENTS

FIGURE II–27 Change of Median Width to Provide Additional Lanes



Dimensions in meters

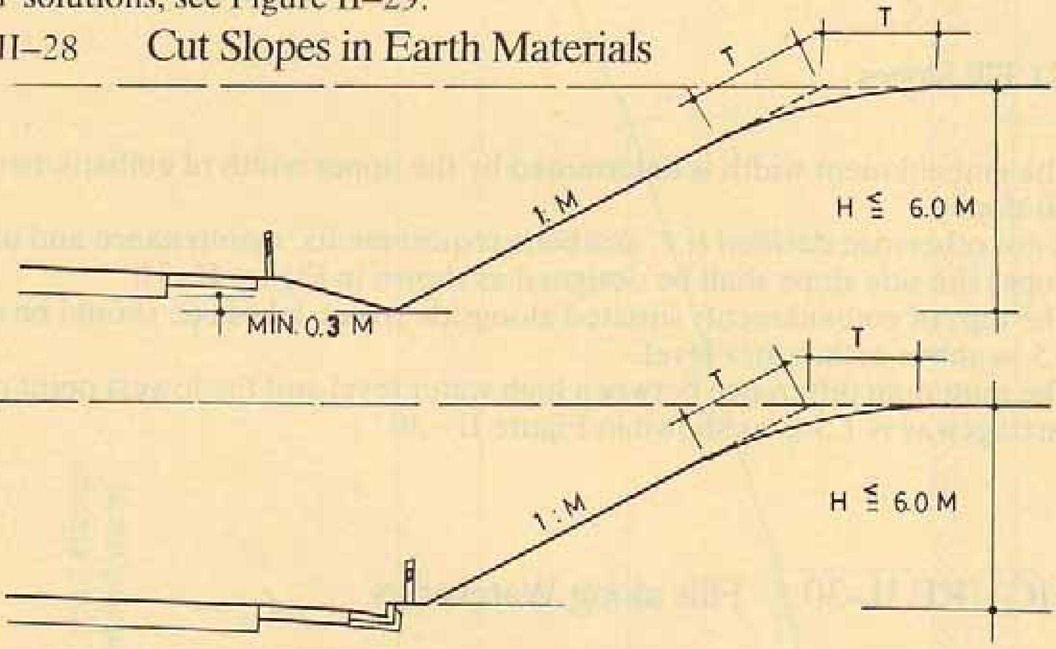
2.6 CROSS SECTION ELEMENTS

2.6.09 Slopes

(1) Cut Slopes

The width of cut sections is determined by the crown width of ditches and by *designed side slopes.*
For cut sections in horizontal curves it is necessary to calculate the width for horizontal sight distance.
If not otherwise decided (geological and hydrological conditions, aesthetic requirements etc.), the maximum side slopes will be as shown in Figure II−28.
The side slopes for cuts deeper than 6 m. should be designed according to soil investigations.
The upper part of the cut will be rounded off over a distance of 1.0 to 3.0 m.
The side slope in rocky materials depends on the quality of cut material and it should be determined in accordance with geological investigations.
At cuttings deeper than 6 meters or in unstable materials it is normal to provide a berm of 1.0 m or greater width.
For typical solutions, see Figure II–29.

FIGURE II–28 Cut Slopes in Earth Materials

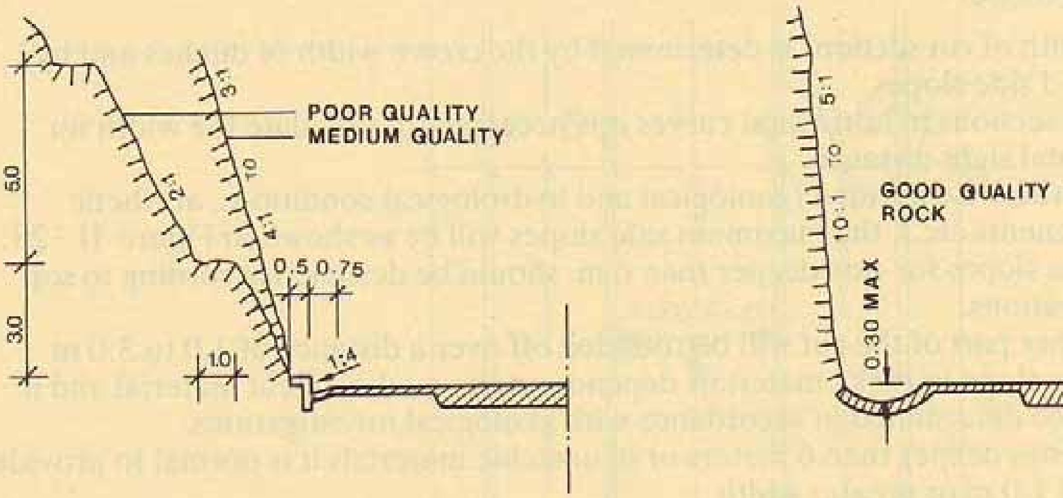


MAIN SOIL TYPES	SLOPES FOR H ≤ 6.0 M			SLOPES FOR H > 6.0 M
	1:M	ROUNDING		
Gravel Sand Silt-dry Clay-dry[2)]	1:2[1)]	H>3.0M	T−3.0M	To be designed according to analysis for slope stability and economics of material usage.
		H=1.0-3.0M	T−2.0M	
		H<1.0M	T−1.0M	
Sandy soil-wet Silty soil-damp Clay soil-damp	1:3	H>2 0M	T−3.0 M	
		H=1.0-2.0M	T−2.0M	
		H<1.0M	T−1.0M	
Silt-wet Clay-wet	1:4	H>2.0M	T−3.0M	
		H=1.0-2.0M	T−2.0M	
		H<10M	T−1.0M	

NOTE: [1)]Slopes of 1:2 and flatter can be formed and maintained by mechanical plant travelling up the slope. For this reason a side slope 1:2 is preferable to that determinated purely by stability consideration.
[2)]Clay-dry can be considered in arid area only.

2.6 CROSS SECTION ELEMENTS

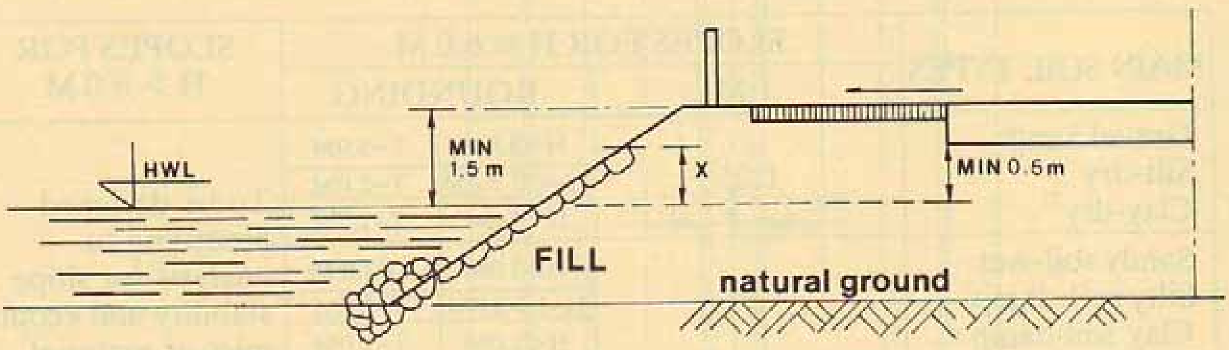
FIGURE II–29 Cut Slopes in Rocky Materials



(2) Fill Slopes

The embankment width is determined by the upper width of embankment and the fill slopes.
If not otherwise decided (i.e. aesthetic requirements, maintenance and use of side slope) the side slope shall be designed as shown in Figure II–31.
The top of embankments situated alongside rivers, lakes etc. should be at least 0.5 m above max. water level.
The minimum difference between high water level and the lowest point on the carriageway is 1.5 m as shown in Figure II–30.

FIGURE II–30 Fills along Waterways



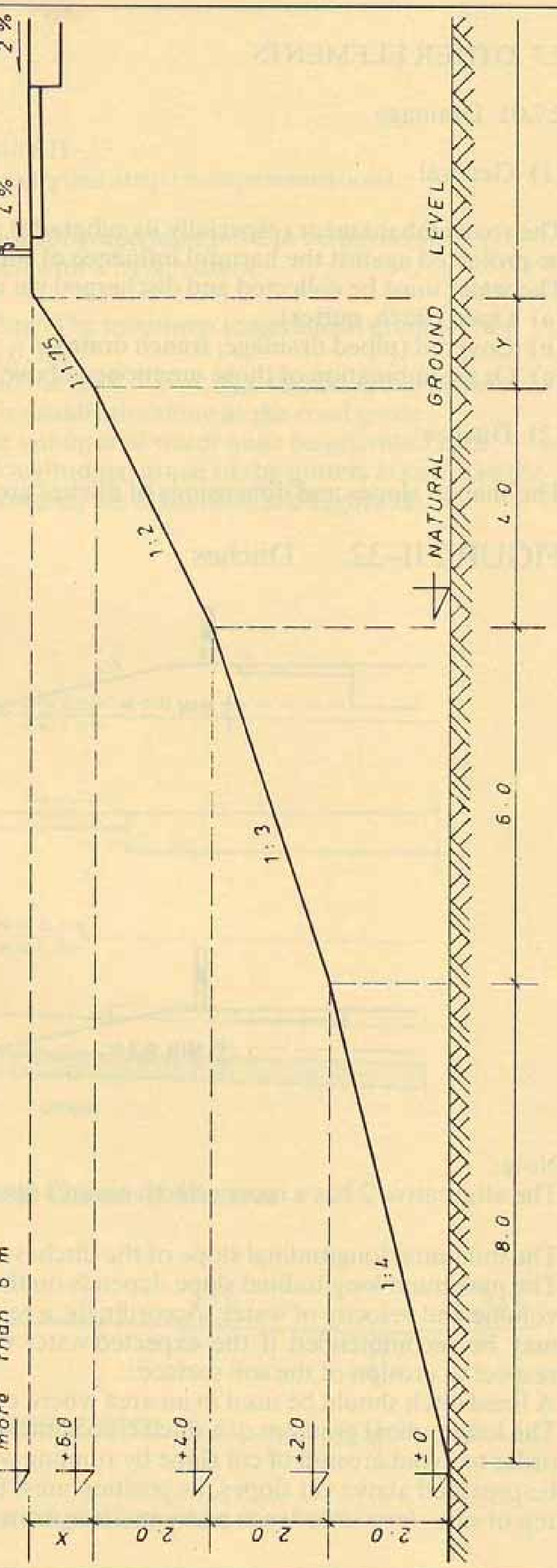
HWL – high water level
X – height of stone pitching shall be at least 0.5 m above HWL or up to maximum height of expected waves.
In such cases, the velocity of water, depth of water, the period of HWL, and height of waves should be taken into consideration.

2.6 CROSS SECTION ELEMENTS

FIGURE II–31

Typical Section – Fill Slopes

Dimensions in meters



NOTE: These side slopes are valid for all road classes
X – height of upper part of the embankment
Y – will be determined by x

2.7 OTHER ELEMENTS

2.7 OTHER ELEMENTS

2.7.01 Drainage

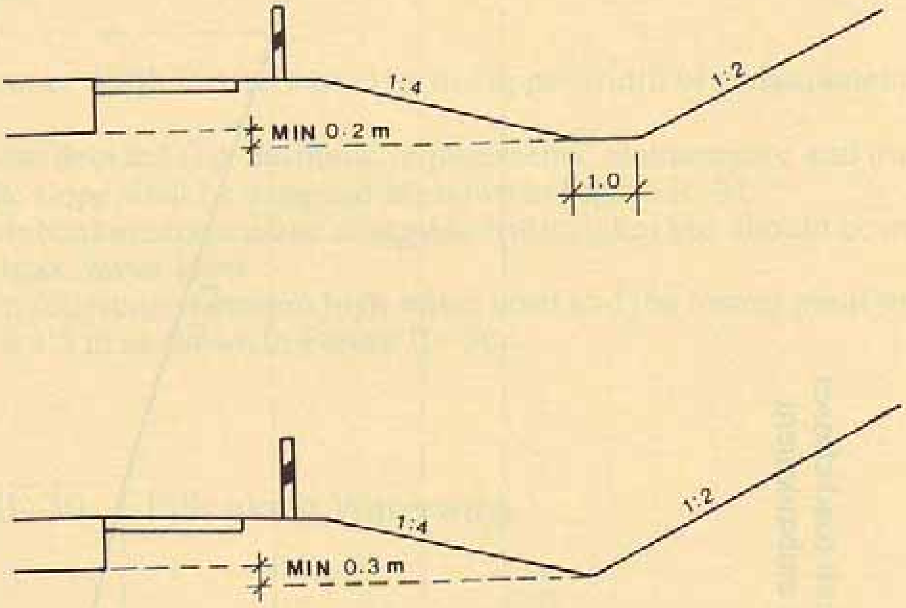
(1) General

The road embankment (especially its subgrade) and the surrounding area have to be protected against the harmful influence of surface and underground water.
The water must be collected and discharged via a proper drainage system such as:
(a) Open (ditch, gutter)
(b) Covered (piped drainage, french drain)
(c) Or a combination of those mentioned above.

(2) Ditches

The shapes, slopes and dimensions of ditches are shown in Figure II–32.

FIGURE II–32 Ditches



Note:
The alternative 2 has a more effective water discharge.

The minimum longitudinal slope of the ditch is 0.5% (for paved ditches 0.3%). The maximum longitudinal slope depends on the road gradient, soil conditions, volume and velocity of water. Accordingly, a reinforcement of the ditch bottom may be recommended if the expected water velocity is above tolerable limits with respect to erosion of the soil surface.
A lined ditch should be used in an area where erosion may occur.
The longitudinal gradient of a ditch (recommended) should not exceed 3% in order to avoid erosion of cut slope by running water, an intercepting ditch should be provided above cut slopes, its position must be at sufficient distance from the top of side slope in order to prevent water from penetrating the cut slope.

2.7 OTHER ELEMENTS

(3) Gutters

Gutters are designed mainly:
(a) In cut (rocky materials) see Figure II–33
(b) In the medians along the inner marginal strip (in superelevation)
(c) In other cases
Gutters which have the bottom above subgrade level have to be paved and, if necessary, they have to be provided with a french drain.

The maximum depth of gutter is 0.3 m. The minimum longitudinal gradient of a gutter is 0.5%, in exceptional cases 0.3%.
The min. width of gutter is 0.5 m and the max. 1.0 m. The surface cross-fall should be 10% and the longitudinal grade is usually the same as the road grade.
Gutters which are designed for large volumes of water must be provided with manholes and french drains. If the longitudinal grade of the gutters is less than the minimum, the slopes will be 0.5% towards the manholes. See Figure II–34.

FIGURE II–33 Gutters

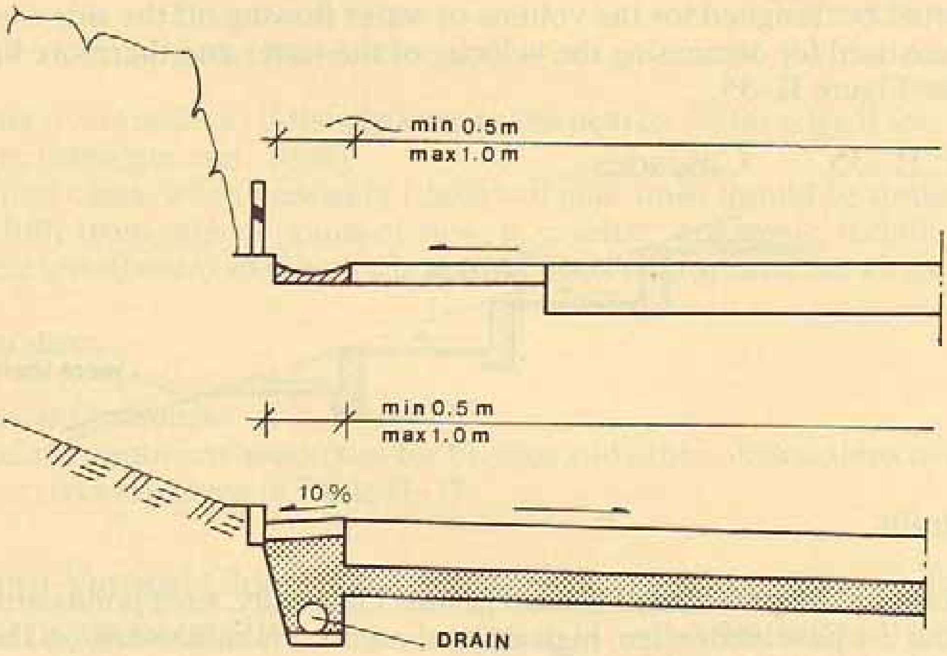
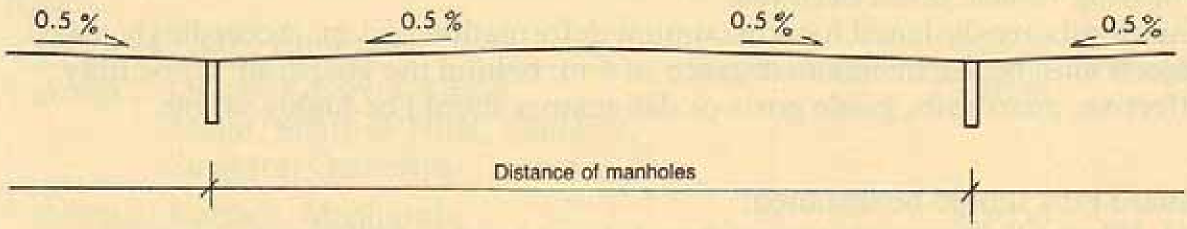


FIGURE II–34 Longitudinal Grade of Gutters



Longitudinal slope of gutters when road gradient is less than 0.3%.

2.7 OTHER ELEMENTS

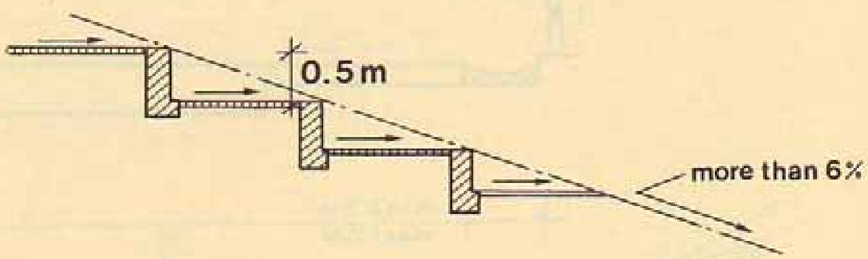
(4) French Drains

They are used in special cases, mainly:
(a) In the cut.
(b) Along ditches whose bottom is above natural ground level.
Minimum diameter of pipes is 100 mm.
Minimum longitudinal slope 0.5%. Manholes are placed approx. every 150 meters.
Drainage is usually proposed:
(a) In the median (on horizontal curves).
(b) Under the ditches in the cut or under the gutters which are designed for large water volumes.
(c) In other cases, when necessary.
The drainage has to be designed for a maximum discharge according to hydraulic calculations. Minimum diameter: 30 cm. If manholes are placed in the shoulder, a manhole cover shall be provided.
Drainage outlets will be connected to an open ditch or stream.

(5) Chutes, Cascades

Chutes should be designed for the volume of water flowing off the side slopes. Cascades are used for decreasing the velocity of the water and therefore limiting erosion. See Figure II–35.

FIGURE II–35 Cascades



(6) Guardrails

Guardrails should be provided at hazard points. Generally, such points are fixed objects along the pavement edge, high embankments, embankments on sharp curves, along water courses, along deep ditches in cuttings and similar locations.

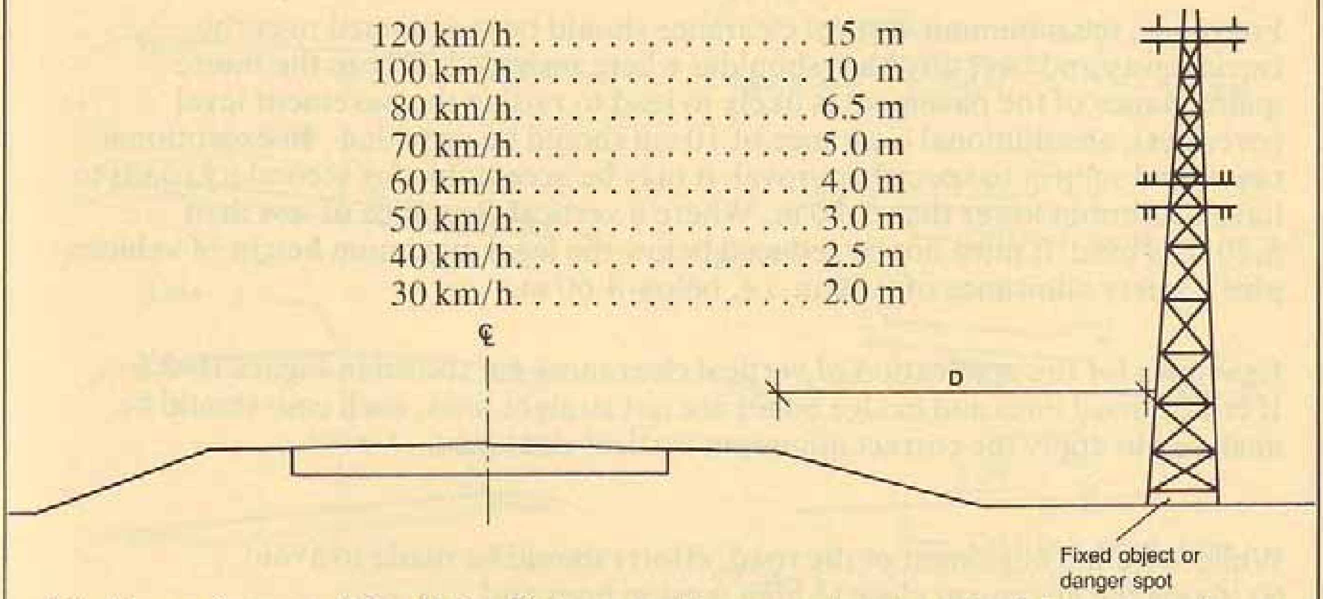
Guardrails are designed to resist impact by deflecting the vehicle so that it continues to move at reduced speed along the guardrail. Any abrupt stop of a vehicle is dangerous, and a guide post or projection on guardrails which might halt a moving vehicle is not desirable.
Guardrails are designed for a maximum deformation of 1 m. Accordingly, fixed objects must be at a minimum distance of 1 m. behind the guardrail. To be fully effective, guardrails, guide posts or delineators should be highly visible.

Guard rails should be installed:
(a) When fill slopes are steeper than 1:4 and height of fill is over 3 meters.
(b) When distance D to the fixed object or danger spot is less than the minimum given below.

2.7 OTHER ELEMENTS

FIGURE II–36 Minimum Space between Pavement and Danger Objects

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).

(d) In other cases, when necessary (design of guardrails should be studied very carefully from various points of view, e. g. safety, economic, technical etc.).

Types and installations of guardrails shall be shown on special drawings.

(7) Clearances

(a) Vertical Clearances

The standard minimum headroom for bridges and other obstructions over a road, railway or river are given in Table II–17.

Minimum Vertical Clearances — Table II–17

OBSTRUCTION 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

2.7 OTHER ELEMENTS

Approval shall be obtained from the competent authorities before finalizing the respective design documents.
The minimum vertical clearance should be used on the lowest edge or point to provide for at least the minimum clearance between any point of obstruction and the pavement, rails or high water level.

For roads, the minimum vertical clearance should be maintained over the carriageway and over any hard shoulder where provided. Where the future maintenance of the pavement is likely to lead to raising the pavement level (overlays), an additional clearance of 10 cm should be provided . In exceptional cases, and subject to special approval, it may be acceptable for secondary roads to have headroom lower than 5.20 m. Where a vertical clearance of less than 5.20 m is used, it must not be reduced below the legal maximum height of vehicles plus a safety allowance of 0.50 m, i.e. below 4.60 m.

Examples for the application of vertical clearances are shown in Figure II–37.
If critical road lines and bridge edges are not straight lines, each case should be analysed to apply the correct minimum vertical clearance.

While selecting alignment of the road, efforts should be made to avoid
(i) fixing the alignment close to high tension lines and
(ii) crossing it at many places.
Wherever the road has to cross overhead high tension lines, the following vertical clearance is to be provided for:

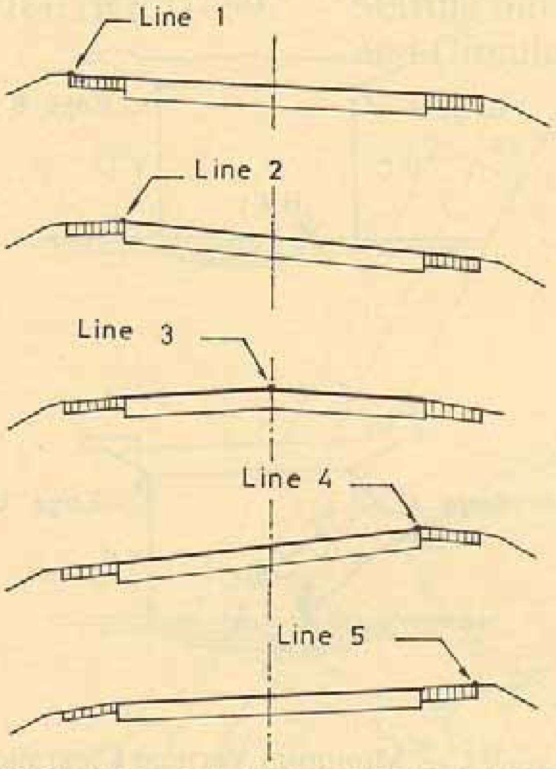
Minimum Vertical Clearances for High Tension Lines TABLE II–18

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

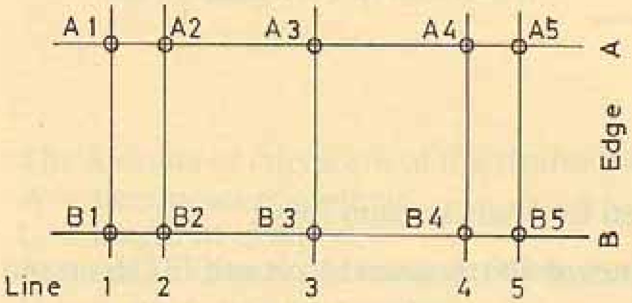
2.7 OTHER ELEMENTS

FIGURE II–37 Critical Line or Point for Application of Minimum Vertical Clearance

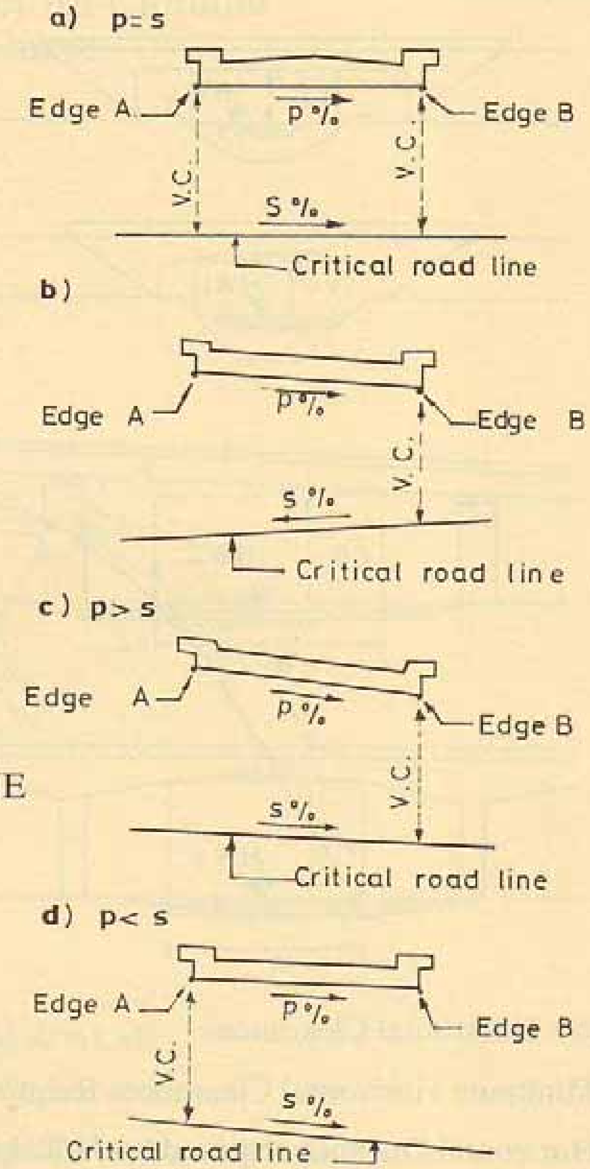
1. STRUCTURE OVER ROAD



POSITION OF CRITICAL ROAD LINE



LOCATION OF CRITICAL POINTS AND LINES



POSITION OF CRITICAL BRIDGE EDGE

CRITICAL POINTS AND LINES

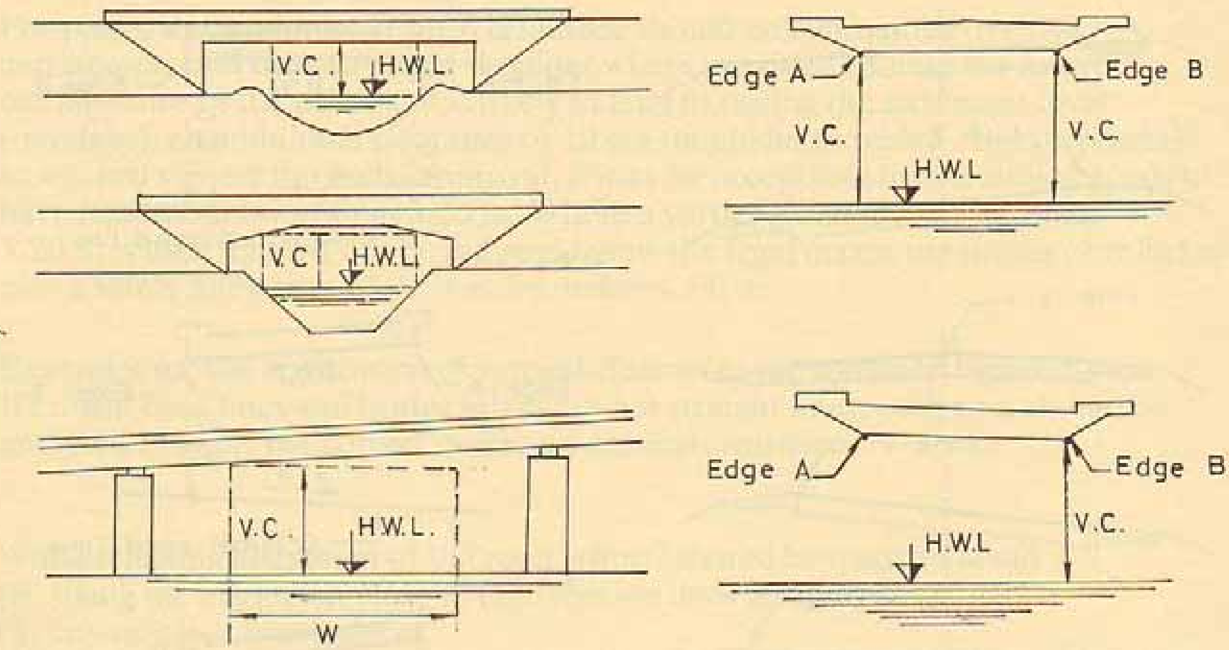
CRITICAL ROAD LINE	BRIDGE ARRANGEMENT			
	a	b	c	d
1	Line A1–B1	Point B1	Point B1	Point A1
2	Line A2–B2	Point B2	Point B2	Point A2
3	Line A3–B3	Point B3	Point B3	Point A3
4	Line A4–B4	Point B4	Point B4	Point A4
5	Line A5–B5	Point B5	Point B5	Point A5

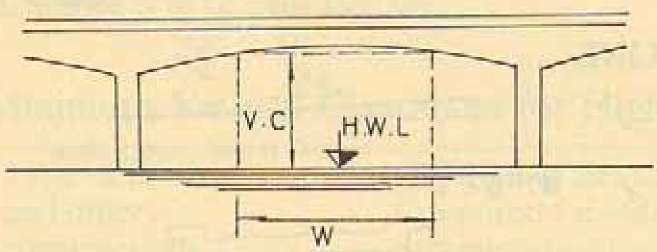
2.7 OTHER ELEMENTS

FIGURE II–38 Critical Line or Point for Application of Minimum Vertical Clearance

2. STRUCTURE OVER RIVER

ELEVATION POSITION OF BRIDGE





V.C… Minimum Vertical Clearance
H.W.L… High Water Level
Level of Edge A ≥ Level of Edge B
W ≥ Width of Navigation Path

(b) Horizontal Clearances

Minimum Horizontal Clearances Required for High Tension Lines

Horizontal Distance required for H.T. Lines of 400 thousand Volt and 132 thousand Volt

Minimum horizontal distance required from the centre line of the transmission lines to the centre line of various types of constructions is given in the following table: –

TABLE II–19

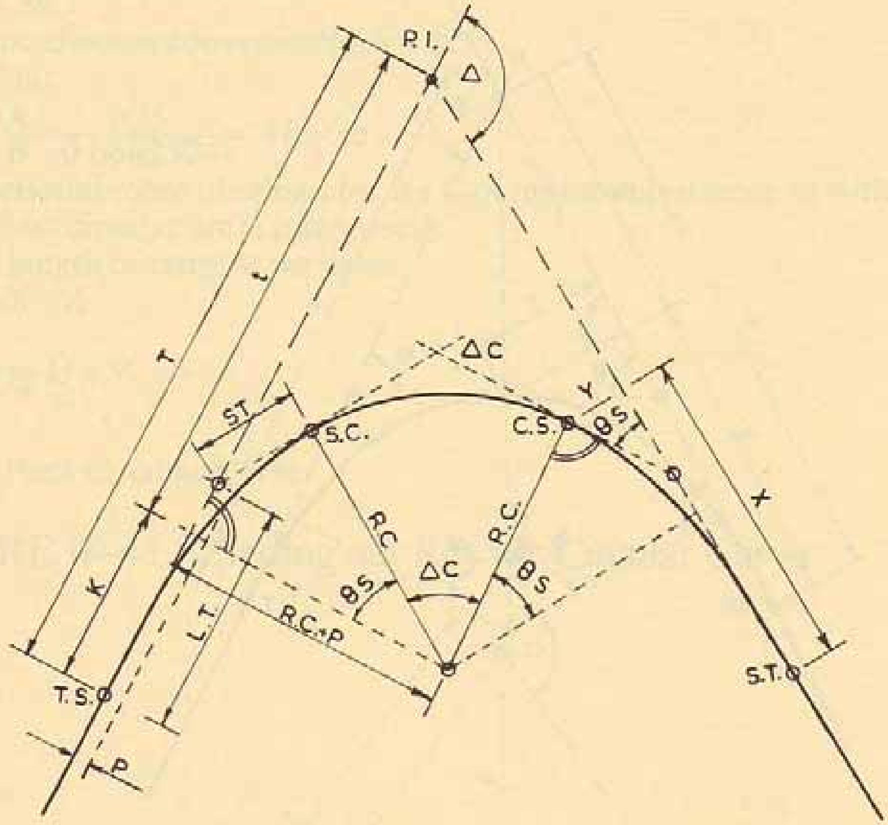
Type of Construction	Minimum horizontal distance required
From the centre line of railway line	50 meters
From centre line of telephone and other low and medium voltage electric lines	50 meters
From centre line of road	100 meters
From centre line of the bridge	100 meters
From centre line of gas and pipe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

2.8 ENCLOSURES – TABLES

2.8 Enclosures – Tables

2.8.01 Clothoid as Transition Curve

FIGURE II–39 Setting out Key for Clothoid and Circular Curve



The formula of curvature of the clothoid is: $A^2 = L x R$

A = parameter of clothoid

L = length of clothoid

R = radius of curve at the point of the length L

Given: ∢ △ by observation

R by graphical solution

A by graphical solution

Required: The clothoid elements L, LT, P, etc;

The total tangent T

The central angle = △c of the circular arc

The length of circular arc Lc

Solution: The clothoid elements are obtained from the table

The table is divided through: $\frac{A}{R}$

from Fig. II–39 it follows that:

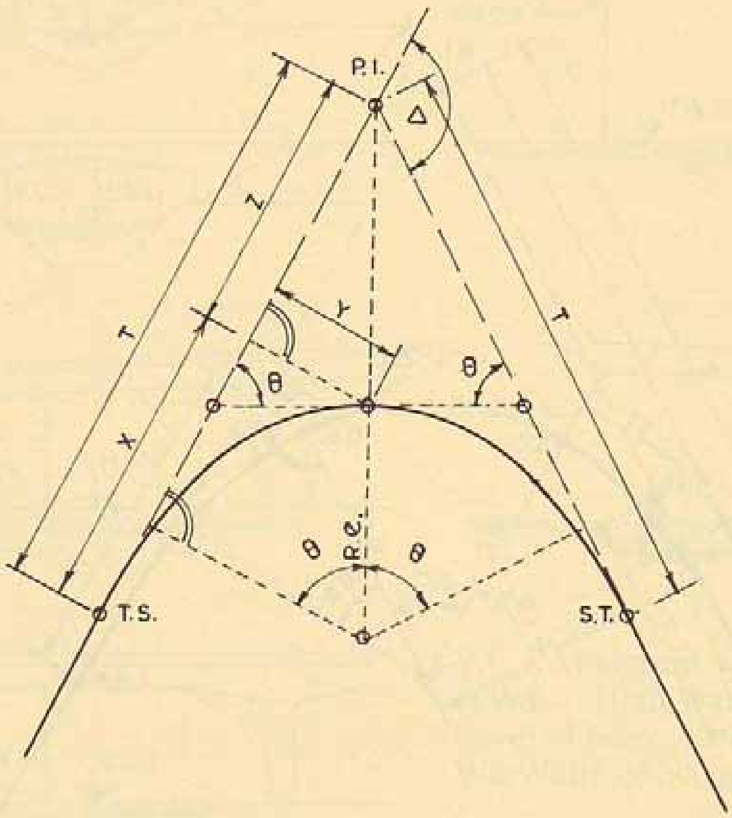
$T = t + K \qquad t = tg\frac{L}{2}(R+P)$

$\triangle c = \triangle - 2\theta s \qquad Lc = \frac{R x \pi x \triangle c^{g}}{200^{g}} = \frac{R x \pi x \triangle c^{\circ}}{180^{\circ}}$

2.8 ENCLOSURES – TABLES

2.8.02 Compound Clothoid

FIGURE II–40 Setting out Key for Compound Clothoid



Given: ∢ Δ, R by graphical solution

Required: The clothoid elements L, A, K, etc., the total tangent T

Solution: The values $\frac{L}{R}$, $\frac{A}{R}$, etc. are taken from table obtaining it through the angle $\theta(\theta=\frac{\Delta}{2})$. If θ ist not contained directly in the table, the values $\frac{L}{R}$, $\frac{A}{R}$ etc. are found by interpolation. In this way the table values which yield the required clothoid elements are determined by multiplication by Re. If Re is given only approximately, a round value for $A = Re \times \frac{A}{R}$ is preferable.

2.8 ENCLOSURES – TABLES

2.8.03 Numerical Example

$\theta = \frac{\Delta}{2} = 14{\cdot}2137^{g}$, $Re \approx 500$

Table value $\frac{A}{R} = 0{\cdot}6682324$ (by interpolation)

Hence it follows:

$A = Re \times \frac{A}{R} = 500 \times 0{\cdot}6682324 = 334{\cdot}12$

Now, one chooses conveniently $A = 325$

Obtaining:

$Re = A : \frac{A}{R} = \frac{325}{0{\cdot}6682324} = 486{\cdot}36$

The fractional value obtained for Re is of minor importance as with the compound clothoid no circular arc is interposed.

For the length of tangent we have:

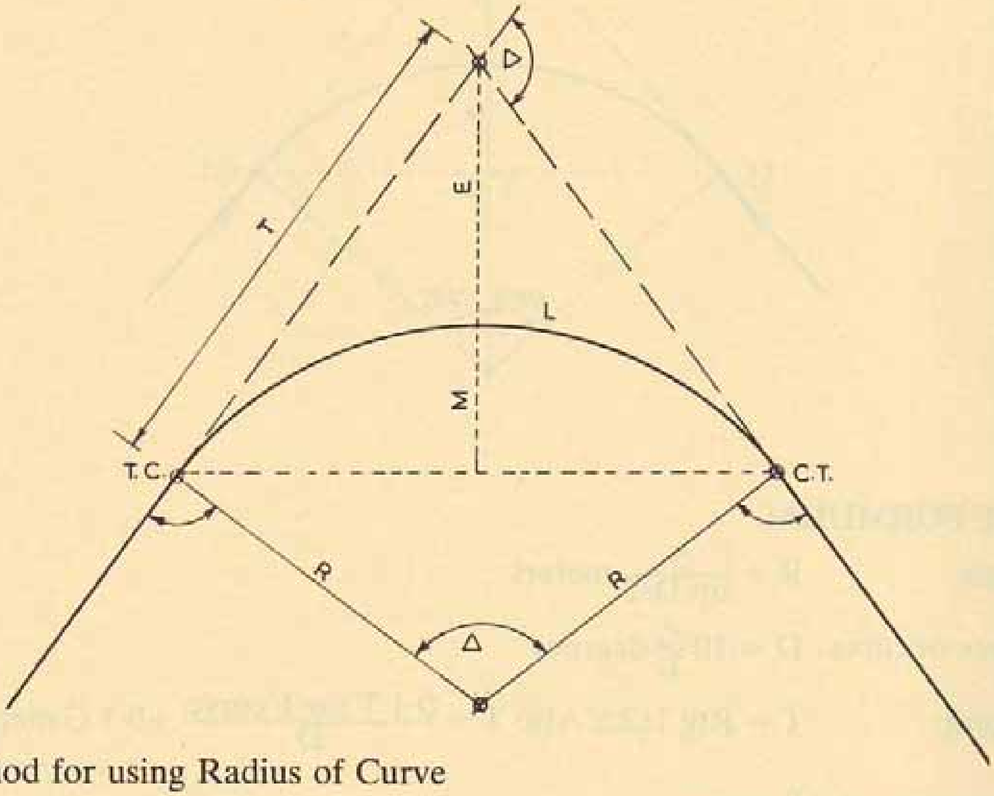
$T = X + Z$

and

$Z = tg\frac{\Delta}{2} \times Y$

2.8.04 Pure Circular Curve

FIGURE II–41 Setting out Key for Circular Curve



(1) Method for using Radius of Curve

$T = R \times tg\frac{\Delta}{2}$

$L = R\frac{\pi \times \Delta^{\circ}}{180} = R\frac{\pi \times \Delta^{g}}{200}$

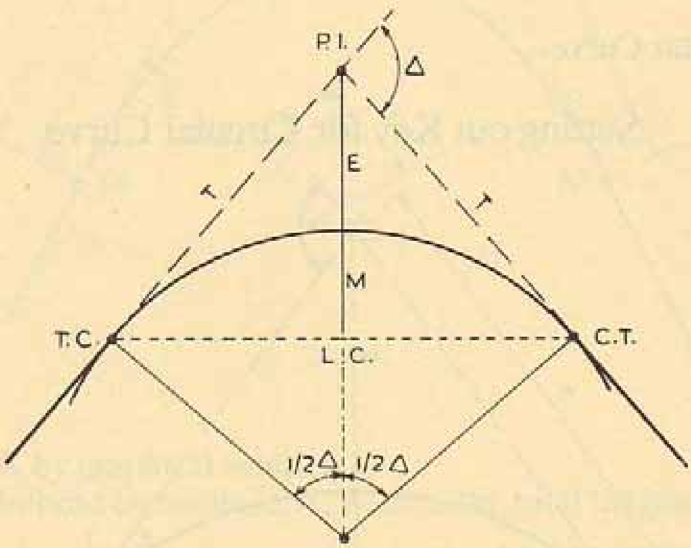
$E = \frac{R}{\cos\frac{\Delta}{2}} - R = T \times tg\frac{\Delta}{4}$ $\qquad M = R\left(1 - \cos\frac{\Delta}{2}\right)$

2.8 ENCLOSURES – TABLES

MINUTES IN DECIMALS OF DEGREE

1′	0·0167	11′	0·1833	21′	0·3500	31′	0·5167	41′	0·6833	51′	0·8500
2	0·0333	12	0·2000	22	0·3667	32	0·5333	42	0·7000	52	0·8667
3	0·0500	13	0·2167	23	0·3833	33	0·5500	43	0·7167	53	0·8833
4	0·0667	14	0·2333	24	0·4000	34	0·5667	44	0·7333	54	0·9000
5	0·0833	15	0·2500	25	0·4167	35	0·5833	45	0·7500	55	0·9167
6	0·1000	16	0·2667	26	0·4333	36	0·6000	46	0·7667	56	0·9333
7	0·1167	17	0·2833	27	0·4500	37	0·6167	47	0·7833	57	0·9500
8	0·1333	18	0·3000	28	0·4667	38	0·6333	48	0·8000	58	0·9667
9	0·1500	19	0·3167	29	0·4833	39	0·6500	49	0·8167	59	0·9833
10	0·1667	20	0·3333	30	0·5000	40	0·6667	50	0·8333	60	1·0000

(2) Method for Using Degree of Curve



CURVE FORMULAS

1. Radius: $R = \frac{5}{\sin D/2}$ meters

2. Degree of curve: $D = 10 \frac{\Delta}{L}$ degrees

3. Tangent: $T = R\,tg\ 1/2\Delta$ Also $T = \frac{0{\cdot}1\ T\ \text{for}\ 1^{\circ}\text{curve}}{D} + 0{\cdot}1\ C$ meter

4. Length of curve: $\frac{\Delta}{D}$ meters

5. Long chord: L.C. $= 2\,R\sin 1/2\Delta$ meters

6. Middle Ordinate: $M = R\,(1 - \cos 1/2\Delta)$ meters

7. External: $E = \frac{R}{\cos 1/2\Delta} - R$. Also $E = T\ tg\ 1/4\Delta$

2.8 ENCLOSURES – TABLES

Explanations and Use of Tables (use field book)

Given P.I. sta. 83+40·7, $\triangle = 45°20'$ and $D = 6°30'$ find:

Stations: T.C. = P.I. − T.T = $\frac{0{,}1 \text{ T for } 1° \text{ curve}}{D}$ + C from Tables V and VI.

$$T = \frac{0{\cdot}1 \text{x} 2392{\cdot}8}{6{\cdot}5} + 0{\cdot}1\text{x}0{\cdot}197 = 36{\cdot}81 + 0{\cdot}02 = 36{\cdot}83 \text{ m.}$$

$$TC = 83+40{\cdot}7 - 36{\cdot}83 = 83+03{\cdot}87$$

$$CT = 83+03{\cdot}87 + L \text{ and } L = 10\frac{\triangle}{D} = 10\frac{45{\cdot}3333}{6{\cdot}5} = 69{\cdot}74 \text{ m.}$$

Therefore, CT = 83+03·87+69·74 = 83+73·61

External, from table V for 1° curve, with central angle of 45°20′
E = 0·1x479·6 therefore, for 6°30′ curve

$$E = \frac{47{\cdot}96}{6{\cdot}5} + 0{\cdot}1 \text{ correction from table* VI} = 7{\cdot}38 + 0{\cdot}004 = 7{\cdot}38 \text{ m.}$$

Note:
* See field book

2.8 ENCLOSURES – TABLES

2.8.05 CONVERSION CENTIGRADES INTO DEGREES

g	°	′	g	°	′	c	′	″	c	′	″	cc	″	cc	″
1	0	54	51	45	54	1	0	32·4	51	27	32·4	1	0·3	51	16·5
2	1	48	52	46	48	2	1	04·8	52	28	04·8	2	0·6	52	16·8
3	2	42	53	47	42	3	1	37·2	53	28	37·2	3	1·0	53	17·2
4	3	36	54	48	36	4	2	09·6	54	29	09·6	4	1·3	54	17·5
5	4	30	55	49	30	5	2	42·0	55	29	42·0	5	1·6	55	17·8
6	5	24	56	50	24	6	3	14·4	56	30	14·4	6	1·9	56	18·1
7	6	18	57	51	18	7	3	46·8	57	30	46·8	7	2·3	57	18·5
8	7	12	58	52	12	8	4	19·2	58	31	19·2	8	2·6	58	18·8
9	8	08	59	53	06	9	4	51·6	59	31	51·6	9	2·9	59	19·1
10	9	00	60	54	00	10	5	24·0	60	32	24·0	10	3·2	60	19·4
11	9	54	61	54	54	11	5	56·4	61	32	56·4	11	3·6	61	19·8
12	10	48	62	55	48	12	6	28·8	62	33	28·8	12	3·9	62	20·1
13	11	42	63	56	42	13	7	01·2	63	34	01·2	13	4·2	63	20·4
14	12	36	64	57	36	14	7	33·6	64	34	33·6	14	4·5	64	20·7
15	13	30	65	58	30	15	8	06·0	65	35	06·0	15	4·9	65	21·1
16	14	24	66	59	24	16	8	38·4	66	35	38·4	16	5·2	66	21·4
17	15	18	67	60	18	17	9	10·8	67	36	10·8	17	5·5	67	21·7
18	16	12	68	61	12	18	9	43·2	68	36	43·2	18	5·8	68	22·0
19	17	06	69	62	06	19	10	15·6	69	37	15·6	19	6·2	69	22·4
20	18	00	70	63	00	20	10	48·0	70	37	48·0	20	6·5	70	22·7
21	18	54	71	63	54	21	11	20·4	71	38	20·4	21	6·8	71	23·0
22	19	48	72	64	48	22	11	52·8	72	38	52·8	22	7·1	72	23·3
23	20	42	73	65	42	23	12	25·2	73	39	25·2	23	7·5	73	23·7
24	21	36	74	66	36	24	12	57·6	74	39	57·6	24	7·8	74	24·0
25	22	30	75	67	30	25	13	30·0	75	40	30·0	25	8·1	75	24·3
26	23	24	76	68	24	26	14	02·4	76	41	02·4	26	8·4	76	24·6
27	24	18	77	69	18	27	14	34·8	77	41	34·8	27	8·7	77	24·9
28	25	12	78	70	12	28	15	07·2	78	42	07·2	28	9·1	78	25·3
29	26	06	79	71	06	29	15	39·6	79	42	39·6	29	9·4	79	25·6
30	27	00	80	72	00	30	16	12·0	80	43	12·0	30	9·7	80	25·9
31	27	54	81	72	54	31	16	44·4	81	43	44·4	31	10·0	81	26·2
32	28	48	82	73	48	32	17	16·8	82	44	16·8	32	10·4	82	26·6
33	29	42	83	74	42	33	17	49·2	83	44	49·2	33	10·7	83	26·9
34	30	36	84	75	36	34	18	21·6	84	45	21·6	34	11·0	84	27·2
35	31	30	85	76	30	35	18	54·0	85	45	54·0	35	11·3	85	27·5
36	32	24	86	77	24	36	19	26·4	86	46	26·4	36	11·7	86	27·9
37	33	18	87	78	18	37	19	58·8	87	46	58·8	37	12·0	87	28·2
38	34	12	88	79	12	38	20	31·2	88	47	31·2	38	12·3	88	25·5
39	35	06	89	80	08	39	21	03·6	89	48	03·6	39	12·6	89	28·8
40	36	00	90	81	00	40	21	36·0	90	48	36·0	40	13·0	90	29·2
41	36	54	91	81	54	41	22	08·4	91	49	08·4	41	13·3	91	29·5
42	37	48	92	82	48	42	22	40·8	92	49	40·8	42	13·6	92	29·8
43	38	42	93	83	42	43	23	13·2	93	50	13·2	43	13·9	93	30·1
44	39	36	94	84	36	44	23	45·6	94	50	45·6	44	14·3	94	30·5
45	40	30	95	85	30	45	24	18·0	95	51	18·0	45	14·6	95	30·8
46	41	24	96	86	24	46	24	50·4	96	51	50·4	46	14·9	96	31·1
47	42	18	97	87	18	47	25	22·8	97	52	22·8	47	15·2	97	31·4
48	43	12	98	88	12	48	25	55·2	98	52	55·2	48	15·6	98	31·8
49	44	06	99	89	06	49	26	27·6	99	53	27·6	49	15·9	99	32·1
50	45	00	100	90	00	50	27	00·0	100	54	00·0	50	16·2	100	32·4

2.8 ENCLOSURES – TABLES

2.8.06 CONVERSION DEGREES INTO CENTIGRADES

o	g	c	cc	o	g	c	cc	o	g	c	cc
1	1	11	11·1	31	34	44	44·4	61	67	77	77·8
2	2	22	22·2	32	35	55	55·6	62	68	88	88·9
3	3	33	33·3	33	36	66	66·7	63	70	00	00·0
4	4	44	44·4	34	37	77	77·8	64	71	11	11·1
5	5	55	55·6	35	38	88	88·9	65	72	22	22·2
6	6	66	66·7	36	40	00	00·0	66	73	33	33·3
7	7	77	77·8	37	41	11	11·1	67	74	44	44·4
8	8	88	88·9	38	42	22	22·2	68	75	55	55·6
9	10	00	00·0	39	43	33	33·3	69	76	66	66·7
10	11	11	11·1	40	44	44	44·4	70	77	77	77·8
11	12	22	22·2	41	45	55	55·6	71	78	88	88·9
12	13	33	33·3	42	46	66	66·7	72	80	00	00·0
13	14	44	44·4	43	47	77	77·7	73	81	11	11·1
14	15	55	55·6	44	48	88	88·9	74	82	22	22·2
15	16	66	66·7	45	50	00	00·0	75	83	33	33·3
16	17	77	77·8	46	51	11	11·1	76	84	44	44·4
17	18	88	88·9	47	52	22	22·2	77	85	55	55·6
18	20	00	00·0	48	53	33	33·3	78	86	66	66·7
19	21	11	11·1	49	54	44	44·4	79	87	77	77·8
20	22	22	22·2	50	55	55	55·6	80	88	88	88·9
21	23	33	33·3	51	56	66	66·7	81	90	00	00·0
22	24	44	44·4	52	57	77	77·8	82	91	11	11·1
23	25	55	55·6	53	58	88	88·9	83	92	22	22·2
24	26	66	66·7	54	60	00	00·0	84	93	33	33·3
25	27	77	77·8	55	61	11	11·1	85	94	44	44·4
26	28	88	88·9	56	62	22	22·2	86	95	55	55·6
27	30	00	00·0	57	63	33	33·3	87	96	66	66·7
28	31	11	11·1	58	64	44	44·4	88	97	77	77·8
29	32	22	22·2	59	65	55	55·6	89	98	88	88·9
30	33	33	33·3	60	66	66	66·7	90	100	00	00·0

2.8 ENCLOSURES – TABLES

2.8.07 CONVERSION MINUTES INTO CENTIMINUTES

′	c	cc	′	g	c	cc	″	cc	″	c	c
1	1	85·2	31		57	40·7	1	3·1	31		95·7
2	3	70·4	32		59	25·9	2	6·2	32		98·8
3	5	55·6	33		61	11·1	3	9·3	33	1	01·9
4	7	40·7	34		62	96·3	4	12·3	34	1	04·9
5	9	25·9	35		64	81·5	5	15·4	35	1	08·0
6	11	11·1	36		66	66·7	6	18·5	36	1	11·1
7	12	96·3	37		68	51·9	7	21·6	37	1	14·2
8	14	81·5	38		70	37·0	8	24·7	38	1	17·3
9	16	66·7	39		72	22·2	9	27·8	39	1	20·4
10	18	51·9	40		74	07·4	10	30·9	40	1	23·5
11	20	37·0	41		75	92·6	11	34·0	41	1	26·5
12	22	22·2	42		77	77·8	12	37·0	42	1	29·6
13	24	07·4	43		79	63·0	13	40·1	43	1	32·7
14	25	92·6	44		81	48·1	14	43·2	44	1	35·8
15	27	77·8	45		83	33·3	15	46·3	45	1	38·9
16	29	63·0	46		85	18·5	16	49·4	46	1	42·0
17	31	48·1	47		87	03·7	17	52·5	47	1	45·1
18	33	33·3	48		88	88·9	18	55·6	48	1	48·1
19	35	18·5	49		90	74·1	19	58·6	49	1	51·2
20	37	03·7	50		92	59·3	20	61·7	50	1	54·3
21	38	88·9	51		94	44·4	21	64·8	51	1	57·4
22	40	74·1	52		96	29·6	22	67·9	52	1	60·5
23	42	59·3	53		98	14·8	23	71·0	53	1	63·6
24	44	44·4	54	1	00	00·0	24	74·1	54	1	66·7
25	46	29·6	55	1	01	85·2	25	77·2	55	1	69·8
26	48	14·8	56	1	03	70·4	26	80·2	56	1	72·8
27	50	00·0	57	1	05	55·6	27	83·3	57	1	75·9
28	51	85·2	58	1	07	40·7	28	86·4	58	1	79·0
29	53	70·4	59	1	09	25·9	29	89·5	59	1	82·1
30	55	55·6	60	1	11	11·1	30	92·6	60	1	85·2

2.8 ENCLOSURES – TABLES

2.8.08 TRIGONOMETRIC FUNCTIONS

Degree	0°	30°	45°	60°	90°	180°	270°	360°
Arc	0	$\frac{\pi}{6}$	$\frac{\pi}{4}$	$\frac{\pi}{3}$	$\frac{\pi}{2}$	π	$\frac{3}{2}\pi$	2π
Sin	0	$\frac{1}{2}$	$\frac{1}{2}\sqrt{2}$	$\frac{1}{2}\sqrt{3}$	1	0	-1	0
Cos	1	$\frac{1}{2}\sqrt{3}$	$\frac{1}{2}\sqrt{2}$	$\frac{1}{2}$	0	-1	0	1
tg	0	$\frac{1}{3}\sqrt{3}$	1	$\sqrt{3}$	$\pm\infty$	0	$\pm\infty$	0
cotg	$\pm\infty$	$\sqrt{3}$	1	$\frac{1}{3}\sqrt{3}$	0	$\pm\infty$	0	$\pm\infty$

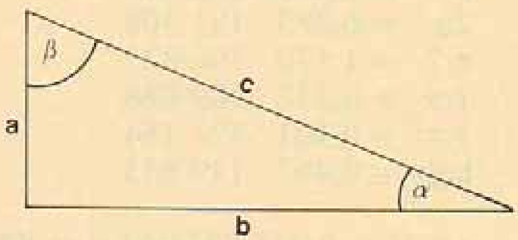
For angle α

$\sin(90°\pm\alpha) = +\cos\alpha$
$\cos(90°\pm\alpha) = \pm\sin\alpha$
$tg(90°\pm\alpha) = \pm cotg\,\alpha$
$cotg(90°\pm\alpha) = \pm tg\,\alpha$
$\sin(180°\pm\alpha) = \pm\sin\alpha$
$\cos(180°\pm\alpha) = \pm\cos\alpha$
$tg(180°\pm\alpha) = \pm tg\,\alpha$
$cotg(180°\pm\alpha) = \pm cotg\,\alpha$

$\sin(270°\pm\alpha) = -\cos\alpha$
$\cos(270°\pm\alpha) = \pm\sin\alpha$
$tg(270°\pm\alpha) = \pm cotg\,\alpha$
$cotg(270°\pm\alpha) = \pm tg\,\alpha$
$\sin(360°\pm\alpha) = \pm\sin\alpha$
$\cos(360°\pm\alpha) = +\cos\alpha$
$tg(360°\pm\alpha) = \pm tg\,\alpha$
$cotg(360°\pm\alpha) = \pm cotg\,\alpha$

Solution of Triangle

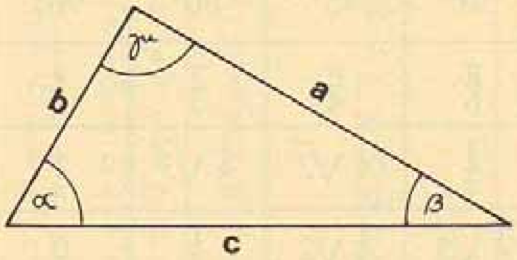
Rectangular Triangle:



$a = c\sin\alpha = b\,tg\,\alpha = c\cos\beta = b\,cotg\,\beta$
$b = c\cos\alpha = a\,cotg\,\alpha = c\sin\beta = a\,tg\,\beta$
$c = \frac{a}{\sin\alpha} = \frac{b}{\cos\alpha} = \frac{a}{\cos\beta} = \frac{b}{\sin\beta}$

2.8 ENCLOSURES – TABLES

Oblique Triangle:



$$\frac{a}{\sin\alpha} = \frac{b}{\sin\beta} = \frac{c}{\sin\gamma}$$

$$a = b\cos\gamma + c\cos\beta$$

$$a^2 = b^2 + c^2 - 2bc\cos\alpha$$

$$s = \frac{1}{2}(a+b+c)$$

$$\mathrm{tg} \quad = \frac{a\sin\gamma}{b - a\cos\gamma} \quad / \quad \sin\frac{\alpha}{2} = \sqrt{\frac{(s-b)(s-c)}{bc}}$$

$$\cos\frac{\alpha}{2} = \sqrt{\frac{s(s-a)}{bc}}$$

2.8.09 IMPORTANT DATA

π = 3,141 592 654
π^2 = 9,869 604 401
π^3 = 31,006 276 680
2π = 6,283 185 308
π:2 = 1,570 796 327
1:π = 0,318 309 886
1:π^2 = 0,101 321 184
logπ = 0,497 149 873

arc 1^g = 0,015 707 963 = π:200
arc 1^c = 0,000 157 079 = π:20000
arc 1^{cc} = 0,000 001 570 = π:2000 000

PART III

INTERSECTION & INTERCHANGE DESIGN

PART III INTERSECTION & INTERCHANGE DESIGN

PART III INTERSECTION & INTERCHANGE DESIGN

LIST OF TABLES

PART III INTERSECTION & INTERCHANGE DESIGN

LIST OF FIGURES

1 GENERAL

1.1 DEFINITION

An intersection is the general area where two or more highways join or cross at grade, within which are included the roadway and roadside facilities for traffic movements in that area. Each highway radiating from an intersection and forming part of it is an intersection leg.
An interchange is the general area where two or more highways are connected with each other without at-grade crossing of traffic streams.

1.2 INTERSECTIONS

Most highways intersect at grade. The geometric forms are 3-leg (T or Y), 4-leg, multi-leg and roundabouts. A further classification includes such variations as unchannelized, flared and channelized intersections.
The traffic volume is the most significant factor in the choice of the appropriate type of intersection although the composition of traffic and the design speed can affect many design details.
The alignment and gradient intersecting roads and the angle of intersection may make it advisable to channelize or use auxiliary pavement areas, regardless of the traffic densities. For the design of intersections, careful consideration should be given to the appearance of the intersection as the driver will see it.
The combination of vertical and horizontal curves as well as sight distance should also be considered : e. g. a sharp horizontal curve following a crest (vertical) curve is very undesirable.

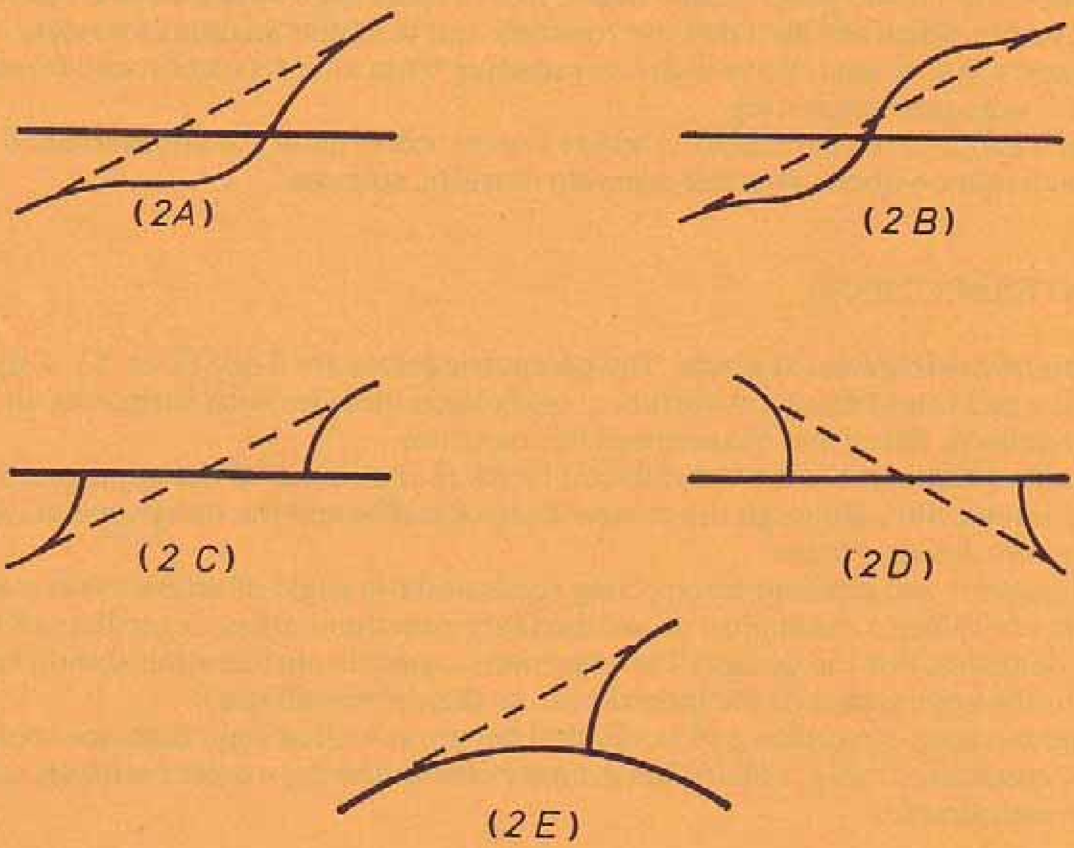
(1) Modifying the Alignment

At many places, site conditions dictate definite alignment and gradient limitations on the intersecting roads. Often it is possible to modify both alignment and gradient to suit traffic conditions better and to reduce hazards, particularly on rural highways.
Regardless of the intersection type, it is desirable that intersecting roads meet at (or nearly at) a right angle. The practice of realigning roads intersecting at an acute angle as shown in Figures III–1–(2A) and III–1–(2B) has proved beneficial.
Another method of realigning an acute angled intersection is to apply a stagger as shown in Figures III–1–(2C) and III–1–(2D). Where the direction of the minor road is as shown in Figure III–1 (2C), the result is poor because a crossing vehicle must re-enter the minor road by a left turn from the major highway. This arrangement should therefore not be used.

Where the direction of the minor road is as shown in Figure III–1–(2D), the result is better because a crossing vehicle first turns left onto the major highway. This can be done safely by waiting for a gap in the through-traffic stream. It then turns right to re-enter the minor road.
Highway intersections on sharp curves should be avoided wherever possible.
Where the main highway is curving and a minor highway constitutes an extension of one tangent, it may be advantageous to realign the minor highway as shown in Figure III–1–(2E).

1 GENERAL

FIGURE III–1 Modifying the Alignment of Intersecting Roads



(2) Profiles

Although it is desirable to avoid substantial gradient changes at intersections, it is not always practicable to do so.
At all intersections where there are YIELD signs STOP signs or traffic signs, the gradient of the intersecting roads should be as flat as practicable on those sections that are to be used as storage space for stopped vehicles. Gradients in excess of 3 per cent should be avoided. Where conditions make such design unduly expensive, gradients should not exceed about 6 per cent. The gradient of the main highway should be carried through the intersection and that of the cross road adjusted to it. Gradients of separate turning roadways should be calculated to fit the cross slopes and longitudinal grades of the intersection legs.

(3) Speed/Curvature Relation

Vehicles turning at intersections designed for minimum turns have to operate at low speed, perhaps less than 15 km/h. While it is desirable and often feasible to design for turning vehicles operating at higher speeds, in most cases of at-grade rural intersections low turning design speeds will be necessary for safety and economy.
Curves at intersections need not be considered in the same category as curves on the open highway because of various warnings being provided. Obviously, different rates of superelevation will produce somewhat different radii for a given design speed. For the intersection curve design, a single minimum radius is established for each design speed. This is done by assuming a likely minimum rate of superelevation.

1 GENERAL

Minimum Radii for Intersection Curves

TABLE III–1

Design (turning) speed km/h	25	30	40	50	60
Assumed min. superelevation	2%	3%	4%	6%	8%
Suggested radius for design R meters	20	35	50	75	120

Note: For design speed greater than 60 km/h, use values for open highway conditions.

The minimum radii established above should be used for the design of the inner edge of pavement. In all cases as much superelevation as practicable should be developed.

1.3 INTERCHANGES

The traffic volume that can pass through an intersection can approach or equal the sum of the open road capacities of two intersecting highways if the roadways are at different levels. The design-hour volumes, character or composition of traffic and design speed are of the greatest importance in designing facilities with a capacity adequate to ensure the traffic safety.

Interchanges are adaptable to all kinds of traffic. The presence in the traffic stream of a high proportion of heavy trucks makes the provision of interchanges especially desirable.

(1) General Types of Interchanges

There are several basic patterns of ramps for turning movements at an interchange, as illustrated in Figure III–2.

(2) Adaptability of Interchanges

(a) Traffic and Operation
Turning traffic can have little or great effect on operation and is accommodated to varying degrees on at-grade intersections and interchanges. At interchanges, ramps are provided for the turning movements. Where turning movements are light and some provision must be made for all turning movements, a one-quadrant ramp design may suffice. Ramps provided in two quadrants may be located so that crossings of through movements occur only at the crossroad, and the major highway is free of such interference. An interchange with a ramp for every turning movement is suitable for heavy through-traffic volumes. Right turning movements at interchanges follow simple direct paths. Cloverleaf interchanges require looped paths for the left turning movements, manoeuvres that confuse many drivers.
The diamond pattern of ramps is simple and more adaptable than a cloverleaf in cases where direct left turns are fitting on the minor road. But where traffic on the minor road is sufficient to justify the expenditure of eliminating the at-grade left turns, a cloverleaf or higher type interchange should be considered.

1 GENERAL

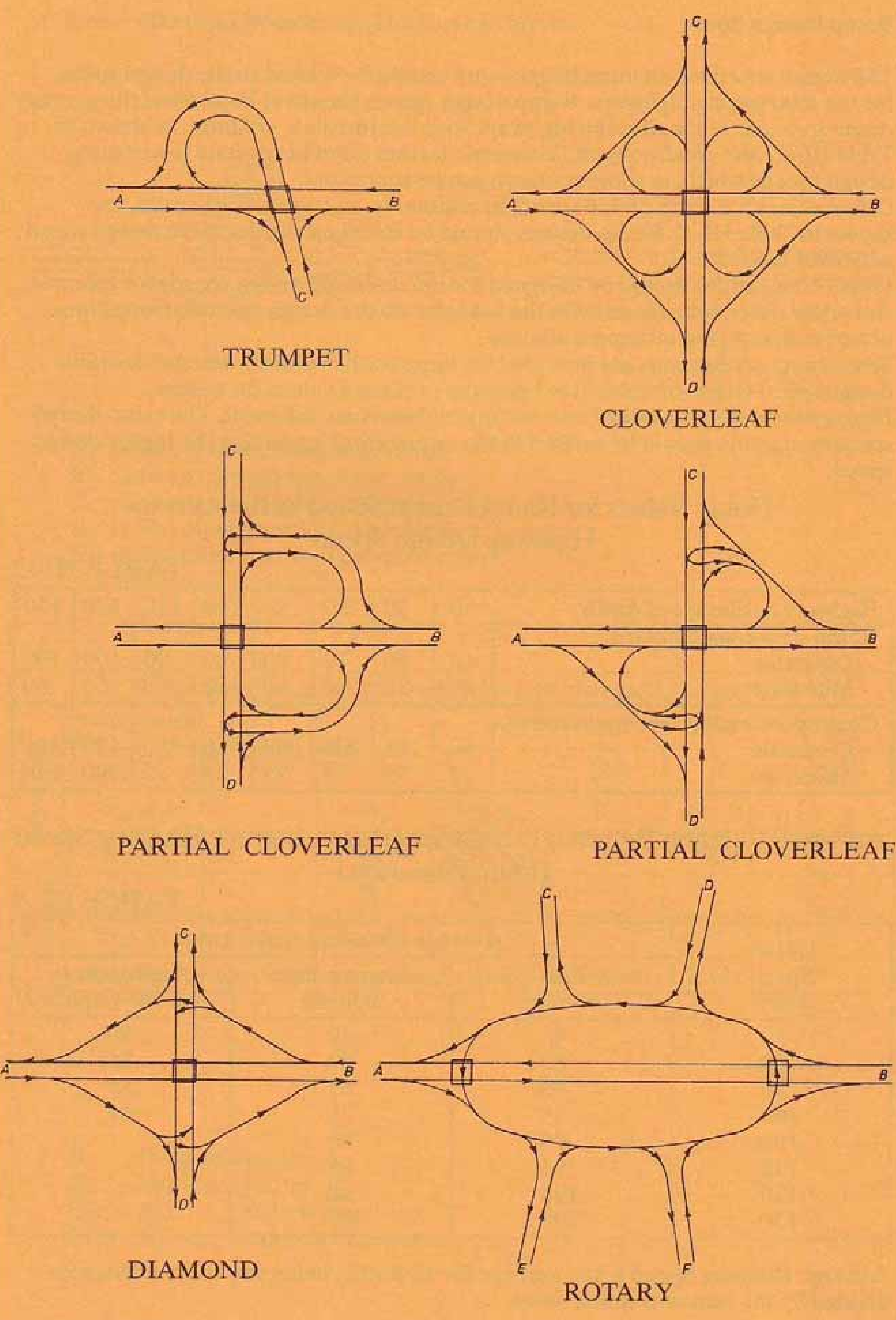
(b) Site Conditions

In rolling and hilly terrain, interchanges can usually be well adapted to the existing ground. Interchange design is simple in flat terrain, although such interchanges are not generally as pleasing to the eye as those in rolling terrain.

(c) Safety

All types of accidents are inherent to at-grade intersections, and they cannot be eliminated completely by design and signalization. Grade separation, on the other hand, reduces drastically the possibility of accidents between the intersecting through-traffic.

FIGURE III–2 Basic Types of Interchanges



2 DESIGN ELEMENTS

Ramp Design Speed

The design speed for an interchange ramp should be related to the design speed for the intersecting highways. Ramp design speeds should at least equal the average running speeds of the through highways with intermediate volumes, as shown in Table III–2. Site conditions and economic factors often necessitate lower ramp design speeds which, in some respects, can be supported.
Guide values for ramp design speed, in relation to highway design speed, are shown in Table III–2. Ramp designs should be based on the desirable design speed wherever feasible.
Direct connections should be designed for the desirable design speed; for example, cloverleaf outer connections. On the other hand, the design speed for loops must often be close to the minimum allowed.
Semi-direct connections are provided for large traffic volumes and the desirable design speed values of Table III–2 give the preferred values for design.
Frequently, design speeds of intersecting highways are different. The ramp design speed preferably should be related to the intersecting leg having the higher design speed.

Guide Values for Ramp Design Speed in Relation to Highway Design Speed

TABLE III–2

Highway design speed km/h	50	70	80	90	100	110	120	130
Ramp design speed km/h								
Desirable	40	60	70	80	90	90	100	100
Minimum	25	30	40	50	50	50	60	60
Correspondending minimum radius								
Desirable	50	135	200	280	350	350	420	420
Minimum	17	35	50	75	75	75	100	140

Assumed Relation Between Design Speed and Average Running Speed (Main Highways)

TABLE III–3

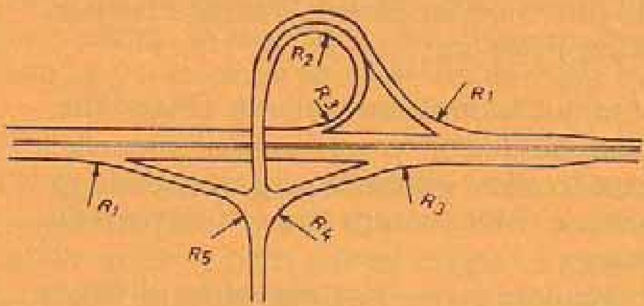
Design Speed km/h	Average operating speed km/h		
	Low Volume	Intermediate Volume	Approaching Possible Capacity
50	45	40	40
70	60	60	55
80	70	65	55
100	85	75	60
100	90	80	
110	95	85	
120	100	90	
130	105	95	

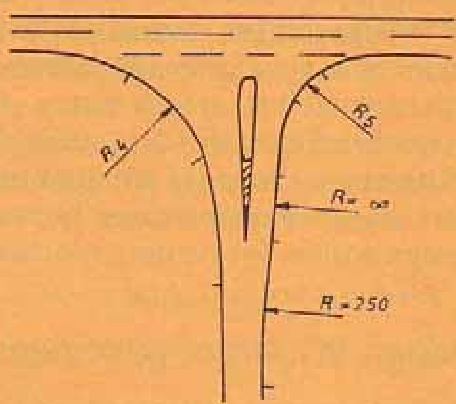
Average Running Speed is the average for all traffic, being the sum of distances divided by the sum of running times.

2.1 HORIZONTAL CURVES

2.1 HORIZONTAL CURVES

Intersection and Interchange Horizontal Curves





R_1 radius of interchange exit ramps
R_2 radius of interchange loop ramps
R_3 radius of interchange entrance ramps
R_4 radius of intersection exit ramps
R_5 radius of intersection entrance ramps

Ramp design speeds of interchanges and intersections (km/h)							
	class \ V (km/h)	20	25	30	40	50	60
Interchanges	A_4/A_4				Ⓡ$_2$	R_2	R_1 R_3
	A_4/A_2		R_5	Ⓡ$_2$ R_4	R_2	R_1 R_3	
	A_2/A_2		R_5	Ⓡ$_2$ R_4	R_2	R_1 R_3	
Intersections	A_4/A_2		Ⓡ$_5$	Ⓡ$_4$ R_5			
	A_2/A_2	Ⓡ$_5$	Ⓡ$_4$ R_5	R_4			
Radii corresponding to ramp design speeds are shown in Table III−16							

R Desirable radius
Ⓡ Minimum radius
Class A_4 .... Dual-carriageways
Class A_2 .... Single carriageways

2.2 PAVEMENT WIDTH

2.2 PAVEMENT WIDTH

Pavement widths of turning roadways and ramps may be designed for one-way or two-way operation.
Widths of pavement for turning roads and ramps are classified for the following types of operation:

Case I 1-lane, one-way operation, no provision for passing.
Case II 1-lane, one-way operation with provision for passing a stalled vehicle.
Case III 2-lane operation, either one-way or two-way.

Widths under case I are usually used for minor turning movements. One of the carriageway edges should be flush with the shoulder.
Under case II, widths are determined so as to allow operation at low speed and with restricted clearance past a stalled vehicle. Many ramps and connections at channelized intersections are in this category.
Widths under case III are applicable where there is two-way operation or where there is one-way operation but two lanes are needed to handle the traffic volume.
Design widths for turning roadways are shown in Table III–4.

Design Widths of pavement for Turning Roadways

TABLE III-4

R Radius on inner edge of pavement (meters)	PAVEMENT WIDTH IN METERS FOR:								
	CASE I 1-lane, one-way operation – no provision for passing			CASE II 1-lane, one-way operation – with provision for passing a stalled vehicle			CASE III 2-lane operation either one-way or two-way		
	DESIGN TRAFFIC CONDITION								
	A	B	C	A	B	C	A	B	C
15	5.50	5.50	7.00	7.00	7.60	8.80	9.50	10.70	12.80
25	4.90	5.20	5.80	6.40	7.00	8.20	8.80	10.10	11.30
30	4.60	4.90	5.50	6.10	6.70	7.60	8.50	9.50	10.70
45	4.30	4.90	5.20	5.80	6.40	7.30	8.20	9.20	10.10
60	4.00	4.90	4.90	5.80	6.40	7.00	8.20	8.80	9.50
90	4.00	4.60	4.90	5.50	6.10	6.70	7.90	8.50	9.20
120	4.00	4.60	4.90	5.50	6.10	6.70	7.90	8.50	8.80
150	3.70	4.60	4.60	5.50	6.10	6.70	7.90	8.50	8.80
Tangent	3.70	4.60	4.60	5.20	5.80	6.40	7.60	8.20	8.20

Design Traffic Condition:
A – having a small volume of trucks or a large truck only occasionally.
B – having a moderate volume of trucks, say in the range of 5 to 10 per cent of the total traffic.
C – having more than 10 per cent heavy vehicles.

2.3 SHOULDER WIDTH

2.3 SHOULDER WIDTH

Clearance Outside Pavements Edges

The roadway width for a turning roadway, distinct from pavement width, includes the shoulders or equivalent additional width outside the edges of pavement.
Within a channelized intersection, shoulders for turning roadways usually are unnecessary. In most instances, these roadways are relatively short, and it is not necessary to provide shoulder sections for the temporary storage of vehicles.

At a separate roadway for right turns, the left edge of pavement outlines the triangular corner area of island, and a developed left shoulder is normally unnecessary.
On the inner or right side of a right turning roadway, a shoulder is usually provided. In cross-section the right shoulder is essentially the same as the outer portion of the open highway section, possibly somewhat less in width because of the intersection conditions.
Table III–5 is a summary of design values for the above general turning roadway conditions.

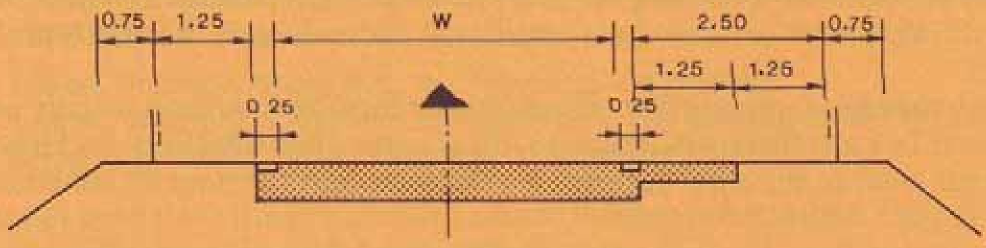
Where guardrails are provided, the width indicated should be measured to the face of guardrail, and the graded width should be 0·75 meters more.
For other than low conditions, the right shoulder should be surfaced for a width of 1·25 meters or more.
A full shoulder width along a speed-change lane is not necessary.

Shoulder Width of Turning Roadways — TABLE III–5

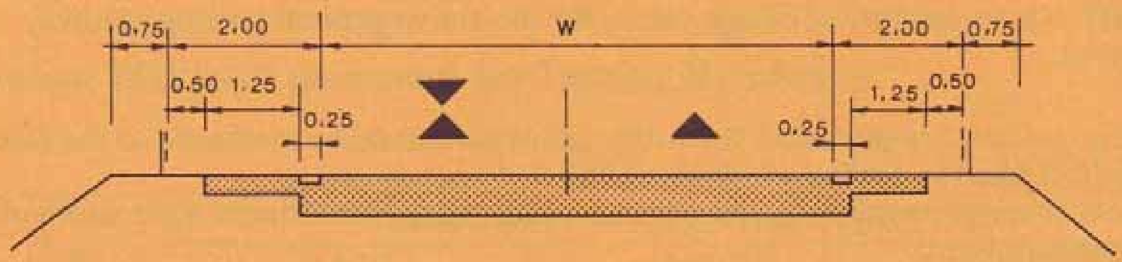
Turning roadway condition	Design	Shoulder width outside of pavement edge (meters)	
		At left	At right
Short length, usually within channelized intersection	Minimum	space to posts or rails at least 0·75 m	
	Desirable	1·25	1·25
Intermediate to large length, or in cut and fill sections	Minimum	1·25	1·75
	Desirable	1·75	2·50

2.3 SHOULDER WIDTH

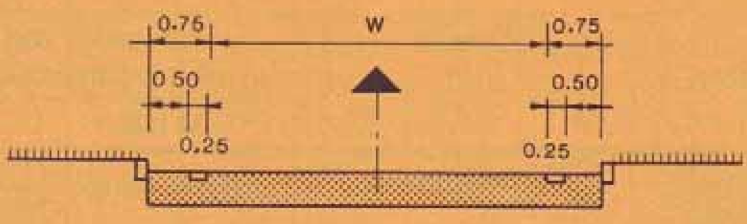
Example of a Turning Roadway Cross Section



One-way one-lane turning roadway



Two-lane one-way or two-way turning roadway



One-way one-lane turning roadway between islands

w = width of pavement according to Table III–4.

2.4 TRANSITION CURVES

2.4 TRANSITION CURVES

Drivers turning on at-grade intersections and on interchanges naturally follow transitional paths just as they do at higher speeds on the open highway. If facilities are not provided for driving in this natural manner, many drivers deviate from the intended path and develop their own transition. Provision for the natural path is best achieved by the use of transition curves, which may be inserted between a tangent and a circular curve. Practical designs fitting transitional paths should be developed by use of clothoid curves.

Length of Clothoid

Lengths of clothoids for use at intersections and interchanges are determinded in the same manner as for open highways. On intersection curves, lengths of clothoids may be shorter than on the open highway curves.
Suggested lengths of clothoids are shown in Table III–6.

Minimum Length of Clothoid for Intersections and Interchanges

TABLE III–6

DESIGN (TURNING) SPEED (km/h)	30	40	50	60	70
MINIMUM RADIUS (m)	25	45	75	120	185
SUGGESTED MIN. LENGTH OF CLOTHOID (m)	20	25	40	50	65
CORRESPONDING CIRCULAR CURVE OFFSET FROM TANGENT (m)	0·66	0·81	0·83	0·83	0·94

2.5 SUPERELEVATION

2.5 SUPERELEVATION

It is desirable to provide as much superelevation as practicable on intersection curves, particulary where the intersection curve is sharp and downhill.
Where radii greater than the minimum for a given design speed are used, they should be superelevated at less than the maximum rate to affect a balance in design as between sharper and flatter curves.
The principles of superelevation run-off ruling for open highway conditions apply generally to intersection and interchange curves, too.
The rate of change of crossfall should be provided according to the radius value of the inner edge of the pavement and according to ramp design turning speed.

The straight part of the ramps may be at a constant crossfall.
If there is a different crossfall between ramp terminals and through-highways, a crown will be provided. The difference between the two slopes should not exceed 4%.
Superelevation rates for curves at intersections are shown in Table III–7.

Superelevation for Curves at Intersections and Interchanges (in %)

TABLE III–7

Radius meters	Range of superelevation rate (%) for intersection and interchange curves with design speed (km/h) of:					
	25	30	40	50	60	70
15	2–12	–	–	–	–	–
25	2–7	2–12	–	–	–	–
45	2–5	2–8	4–12	–	–	–
70	2–4	2–6	3–8	6–12	–	–
90	2–3	2–4	3–6	5–9	8–12	–
130	2–3	2–3	3–5	4–7	6–9	9–12
180	2	2–3	2–4	3–5	5–7	7–9
300	2	2–3	2–3	3–4	4–5	5–6
450	2	2	2	2–3	3–4	4–5
600	2	2	2	2	2–3	3–4
900	2	2	2	2	2	2–3

NOTE: Preferably use superelevation rate in upper half or third of indicated range

2.5 SUPERELEVATION

Superelevation Run-off

The principles of superelevation run-off for open highway conditions generally apply to intersection and interchange curves as well. For through-highway profiles the slope of one edge of pavement with respect to the center line should be no greater than 1:150 for a design speed of 30 km/h or less, and 1:300 for a design speed of more than 30 km/h. These rotation rates are desirable, but values as high as 1:125 may be used on turning roadways without distortion of appearance or hazard in operation.

Design Rate of Change in Crossfall for Curves at Intersections and Interchanges

TABLE III–8

Design speed (km/h)	Less than 30	More than 30
Change in superelevation rate	1:150	1:300
Per 25 meter length	0.17 m	0.34 m

Normally the profile zone edge of pavement is established first and the profile of the other edge is developed by stepping up or down from the first-edge and by plotting a few control points using values from Table III–8 to approximate the change in superelevation rate from one point to the next. A smooth profile can then be plotted.

2.6 GRADE LINE

2.6 GRADE LINE

Gradients on turning roadways should take into account the grade conditions of the intersecting highways and their traffic importance. Uphill gradients of 6% should be taken as a maximum, but for one-way ramps 8% downhill can be acceptable.

Vertical Curves

Only parabolic curves should be used. The parabola is determined by the radius of the osculating circular curve.
Minimum radii for vertical curves are shown in Table III–9.

Minimum Radii for Vertical Curves

TABLE III–9

DESIGN SPEED (TURNING) (km/h)	70	60	50	40	30
CREST CURVE (m)	2500	1400	1000	600	400
SAG CURVE (m)	1500	1200	800	500	300

It is recommended that greater radii be used than those indicated in Table III–9 wherever it is practicable.

2.7 SPEED-CHANGE LANES

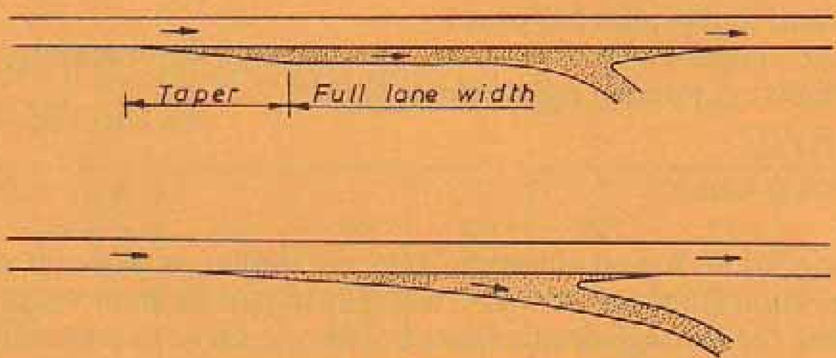
2.7 SPEED-CHANGE LANES

A speed-change lane is an auxiliary lane including tapered areas, primarily for the acceleration or deceleration of vehicles entering or leaving the through-traffic lanes. A speed-change lane should be of sufficient width and length to enable a driver to make the necessary change between the speed of operation on the highway and the lower speed on the turning roadway.

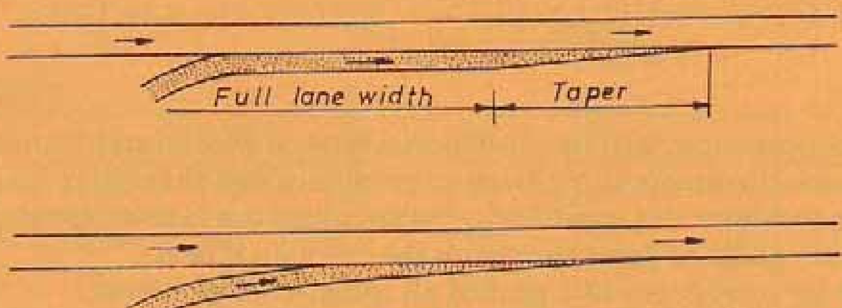
Speed-change lanes may have different forms depending on the alignment of the highway, frequency of intersections and the distance required to effect the necessary speed change.

Forms of speed-change lanes are shown in Figure III−3.

FIGURE III−3 Forms of Speed-change Lanes



DECELERATION LANES



ACCELERATION LANES

A speed-change lane of uniform width should not be less than 3·50 m wide.

2.7 SPEED-CHANGE LANES

(1) Deceleration Lanes:

The deceleration lane consists of a taper and a full lane width. The length of deceleration lanes should be determined according to the design speed of the highway and the design speed of the turning roadway.
The greater the difference between these speeds the longer the deceleration lane should be. Deceleration lanes on approaches to at-grade intersections also function as storage lanes for turning traffic in some cases.
The deceleration lane for left turning traffic should be designed at the left side of the highway only. This additional lane is not to be provided in cases where the left turning traffic is of little importance.
The deceleration lane for the right turning traffic should be designed at the right side of the highway only.

This lane is to be provided in the following cases:
(a) For dual and multi-lanes highways.
(b) At two-lane highways having a design speed of 80 km/h or more.
(c) For the higher volume of right turning traffic.

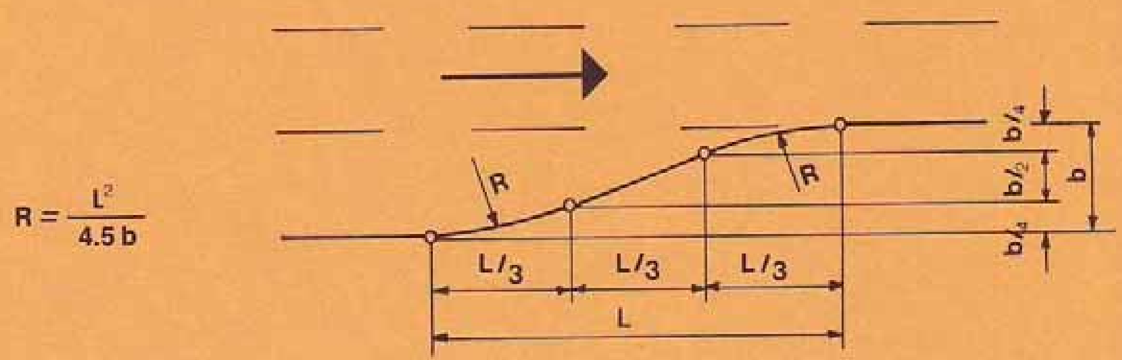
(2) Acceleration Lanes

Design considerations for acceleration lanes are similar to those for deceleration lanes. Acceleration lanes are provided not only to permit an increase in speed before entering the through-traffic lanes but also to serve as manoeuvering space, so that a driver can take advantage of an opening in the adjacent stream of through-traffic and join it from the side. At the far end there should be no barriers such as a curb between lane and shoulder.

(3) Taper

A taper, in conjunction with the directional type of acceleration lane, joins the carriageway for the through-highway so gradually that there may be no positive indication as to where the taper ends. On deceleration lanes, the taper should show a definite point of departure from the through lanes.
A taper is to be considered as a part of all speed-change lanes.
A taper should be developed using two reverse circular curves as shown in Figure III–4.

FIGURE III–4 Form of Taper



2.7 SPEED-CHANGE LANES

(4) Length

The lengths of acceleration and deceleration lanes are based on passenger vehicle operation. Although trucks require longer distances to decelerate for the same change in speed, longer lanes are not justified because average speeds of trucks are generally lower than those of passenger cars. For acceleration lanes, trucks generally require a much longer distance to accelerate.
Where a substantial number of large vehicles are to enter a high-speed highway, the length should be increased.
The resulting lengths of speed-change lanes are shown in Table III–10.
These lengths are recommended for the design of level sections or gradients of up to 2 per cent. Where the gradient on the speed-change lane is steeper than 2 per cent, an adjustment in length should be made in accordance with Table III–11.
Lengths given for speed-change lanes generally should be considered as minimum values for all main highways.

Design Length of Speed-change Lanes for Gradients of 2% or Less

TABLE III–10

Design speed of turning roadway curve (km/)		Stop condition	25	30	40	50	60	70	80
Minimum curve radius (m)			17	35	50	75	120	180	230
Design speed of highway (km/h)	Length of taper (meters)	TOTAL LENGTH OF DECELERATION LANE INCLUDING TAPER (METERS)							
70	70	110	100	90	80	80	–	–	–
80	80	140	135	125	115	105	90	–	–
90	90	165	165	155	145	135	120	100	–
100	95	185	185	175	165	155	130	110	–
110	100	200	190	180	180	170	150	130	115
120	105	215	210	200	200	190	170	150	130
130	110	235	225	215	215	200	180	160	145
Design speed of highway (meters)	Length of taper (meters)	TOTAL LENGTH OF ACCELERATION LANE INCLUDING TAPER (METERS)							
70	70		110	85	75	–	–	–	–
80	80	–	235	210	200	165	135	–	–
90	90	–	375	360	335	300	270	200	135
100	95	–	515	500	465	440	410	335	275

2.7 SPEED-CHANGE LANES

Ratio of Lengths of Speed-change Lanes for Gradients >2%

TABLE III–11

Deceleration lanes

Design speed of highway (km/h)	Ratio of lengths for gradients > 2% to lengths on level for:	
All	3 to 4% up grade 0·9	3 to 4% down grade 1·2
All	5 to 6% up grade 0·8	5 to 6% down grade 1·35

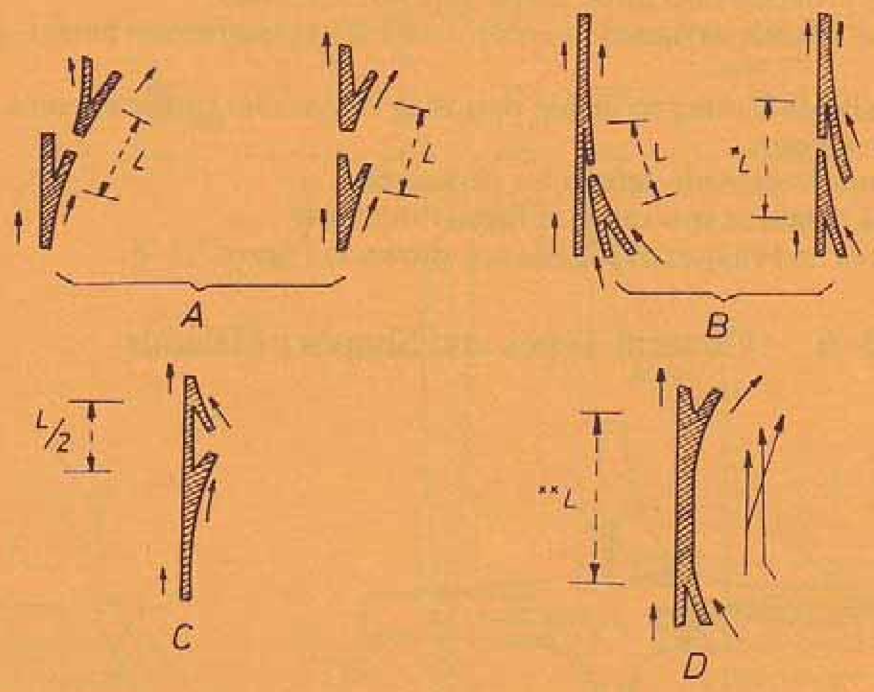
Acceleration lanes

Design speed of highway (km/h)	Ratio of lengths for gradients > 2% to lengths on level* for:				
	Design speed of turning roadway curve (km/h)				
	30	50	60	80	All speeds
	3 to 4% up grade				3 to 4% down grade
70	1·3	1·3	–	–	– 0·7
80	1·3	1·4	1·4	–	– 0·65
90	1·4	1·5	1·5	1·6	– 0·6
100	1·5	1·6	1·7	1·8	– – 0·6
	5 to 6% up grade				5 to 6% down grade
70	1·5	1·5	–	–	– 0·6
80	1·5	1·7	1·9	–	– 0·55
90	1·7	1·9	2·3	2·5	– 0·5
100	2·2	2·2	2·7	3·0	– 0·5

* Ratio from this table multiplied by length in Table III–10 gives length of speed-change lane for the respective gradients.

2.7 SPEED-CHANGE LANES

FIGURE III–5 Distances between Successive Ramp Terminals



A SUCCESSIVE EXIT TERMINALS
B SUCCESSIVE ENTRANCE TERMINALS
C EXIT TERMINAL FOLLOWED BY ENTRANCE TERMINAL
D ENTRANCE TERMINAL FOLLOWED BY EXIT TERMINAL

*L As in table but not less than length required for manoeuvering or speed change as shown in Table III–10.
**L As in table but not less than length required for weaving.

Distance between Successive Ramp Terminals TABLE III–12

DESIGN (km/h)	50 or less	70 TO 80	100 TO 110	120 TO 130
Av. operating (km/h)	35 TO 50	50 TO 80	80 TO 110	110
DISTANCE L (meters)				
Minimum	70	135	170	300
Desirable	135	230	300	400

3 TRAFFIC ISLANDS

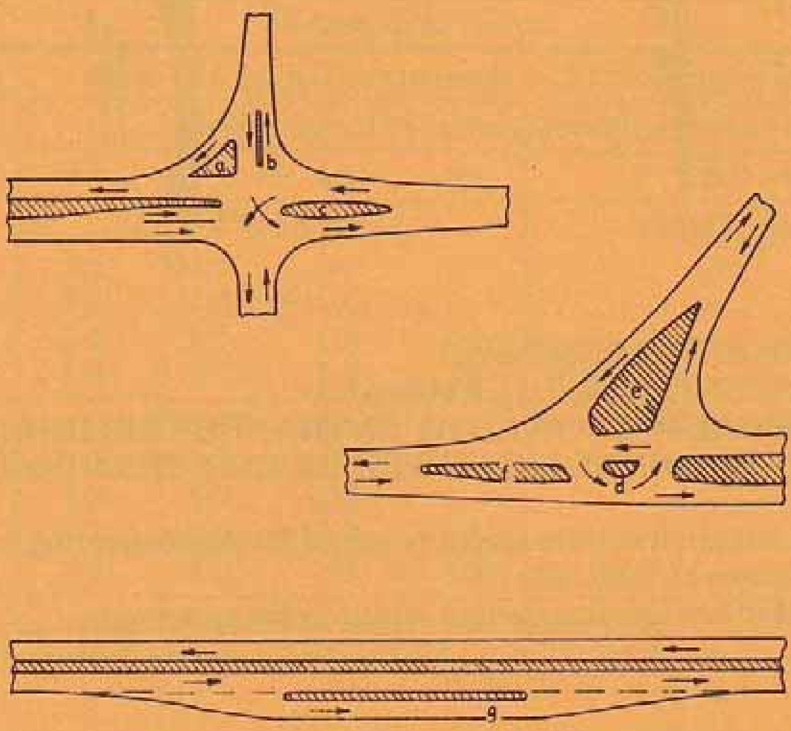
In general, islands are either elongated or triangular in shape and are situated in areas not normally used as vehicle paths.

Islands may be grouped into three major functional classes:

1 Channelizing islands designed to control and direct traffic movement, usually turning.
2 Divisional islands serving to divide opposing or parallel traffic streams, usually through movements.
3 Refuge islands to provide refuge for pedestrians.

Most islands combine two or all of these functions.
General types and shapes of islands are shown in Figure III–6.

FIGURE III–6 General Types and Shapes of Islands



Channelizing Islands

Triangular shape "a" separates right turning from through-traffic.
Central islands "c" or "d" may serve as a guide around which turning vehicles operate.

Divisional Islands

Varieties of divisional islands are indicated by letters b, c, e, f and g.

Refuge Islands

Islands a, b, c, e and f are examples. The general principles of the island design apply directly to refuge islands, except that barrier curbs are usually considered necessary.

3 TRAFFIC ISLANDS

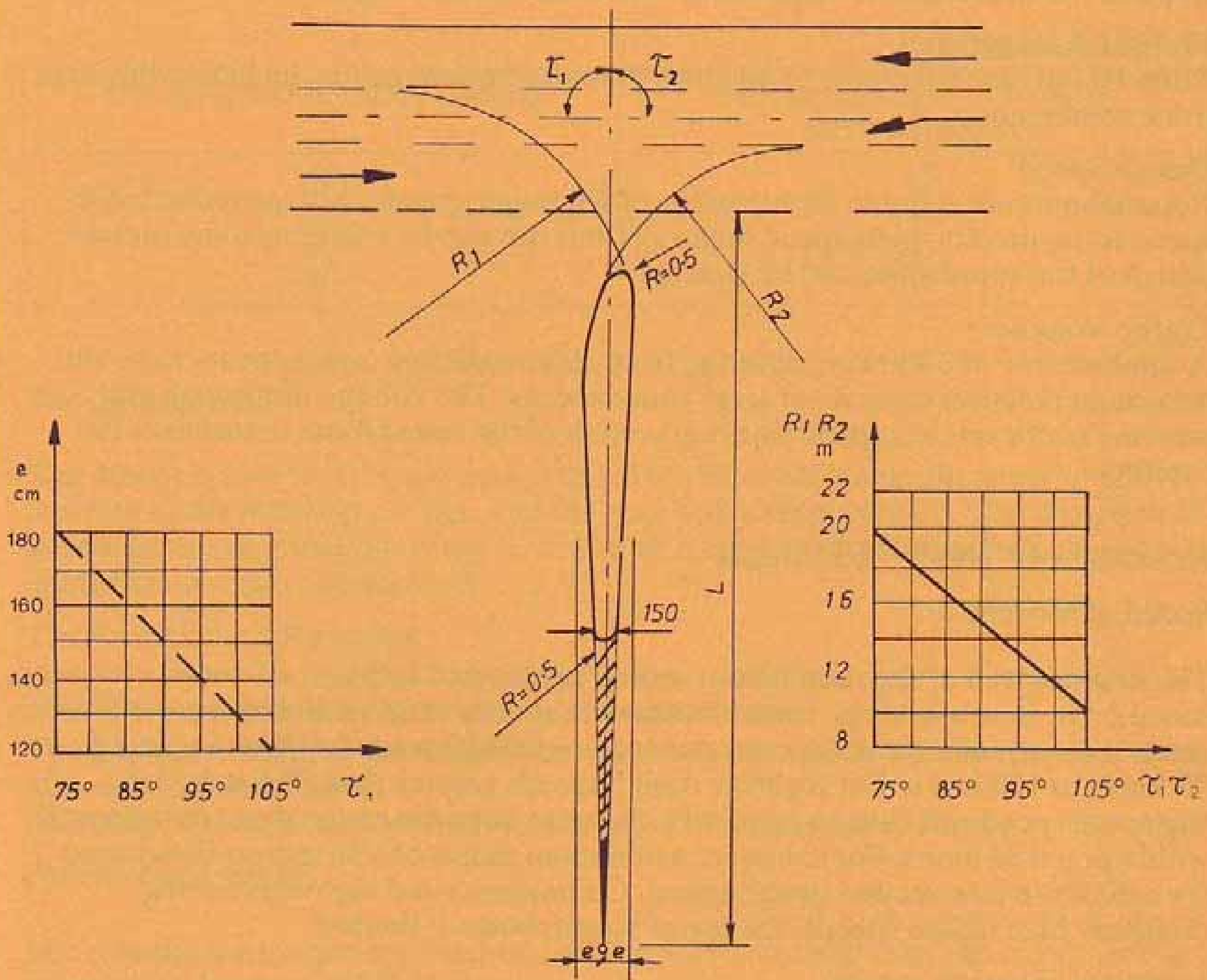


FIGURE III–7 Form of Divisional Islands

	KIND OF MAIN HIGHWAY	
	2 Lanes	2 lanes with the left turning lane
L	20·0 m	40·0 m

4 ROUNDABOUTS

Traffic Conditions Favourable to Roundabout Design

Traffic Characteristics
Roundabouts are adaptable to all kinds of motor vehicle traffic, including the large truck combination.

Design Speed
Roundabouts are suitable for highways of all design speeds. A large reduction in speed is required on high-speed highways, but this can be encouraged by proper design of the approaches and by signing.

Traffic Volume
A total volume of 3000 vph, entering from all intersection legs, appears to be the maximum practical capacity of large roundabouts. The volume of through and weaving traffic on the critical weaving section of the roundabout determines its capacity.

Roundabout Design Elements

Speed of Movement

The design speed of the roundabout should be selected first and all design elements based on it. In urban areas, roundabouts with speeds of 25 to 40 km/h are efficient. In rural areas, such speeds are not satistactory for highways designed for speeds 70 km/h to 120 km/h. For highway design speeds greater than 70 km/h, the suggested speed will have to be relatively low to keep the roundabout dimensions within practical limits. For example, a minimum radius of 130 meters is required for a 60 km/h intersection design speed. On modern rural highways having relatively high design speeds, the use of roundabouts is limited.

Length of Weaving Section

The weaving length is the distance between ends of directional islands. Table III–13 shows the relation between length of section, running speed and volume of weaving vehicles.

Relation between Number of Weaving Vehicles and Length of Weaving Section for Various Speeds

TABLE III–13

Length of weaving section (meters)	Number of weaving vehicles per hour at average operating speed of	
	50 km/h	70 km/h
30	750	350
60	1100	600
90	1350	750
120	1600	900
150	1750	1050
180	1900	1200

Note: An average operating speed of 50 km/h is normally used as a basis for the design of roundabouts.

4 ROUNDABOUTS

Suggested Minimum Lengths of Weaving Sections

TABLE III–14

Design speed of rotary (km/h)	Minimum length of weaving section (meters)
40	45
50	55
60	65
70	75

The Central Island

The design is governed by the design speed of the roundabout, the number and location of the intersection legs, and the required weaving length. The provision of adequate weaving sections often results in an unsymmetrical design utilising an elongated or oval central island.

The Roundabout Pavement

This is the one-way pavement around the central island

The minimum width of the roundabout pavement should be at least the equivalent of two 3.75 meters lanes.

Normally, the maximum width recommended in rural areas is 4 lanes.

Pavement crossfalls

For roundabouts, a greater algebraic difference in crossfall over the crown line is practicable because of low and uniform speeds.

Figure III–8 demonstrates the normal method of applying crossfall to roundabout pavements.

Crossfall of Roundabouts

TABLE III–15

Design speed of roundabout (km/h)	Maximum algebraic difference in crossfall over the crown line
40–50	6%–7%
60–70	5%–6%

4 ROUNDABOUTS

CROSSFALLS AND CROWN LINES

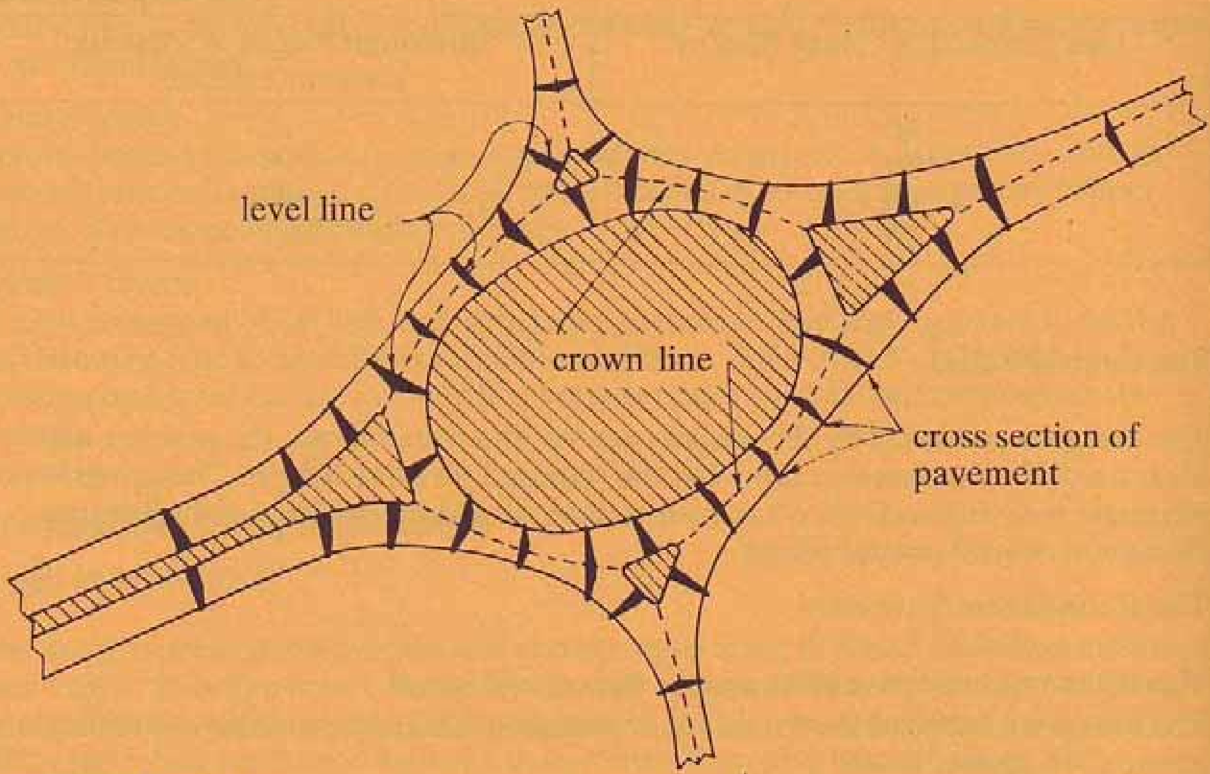
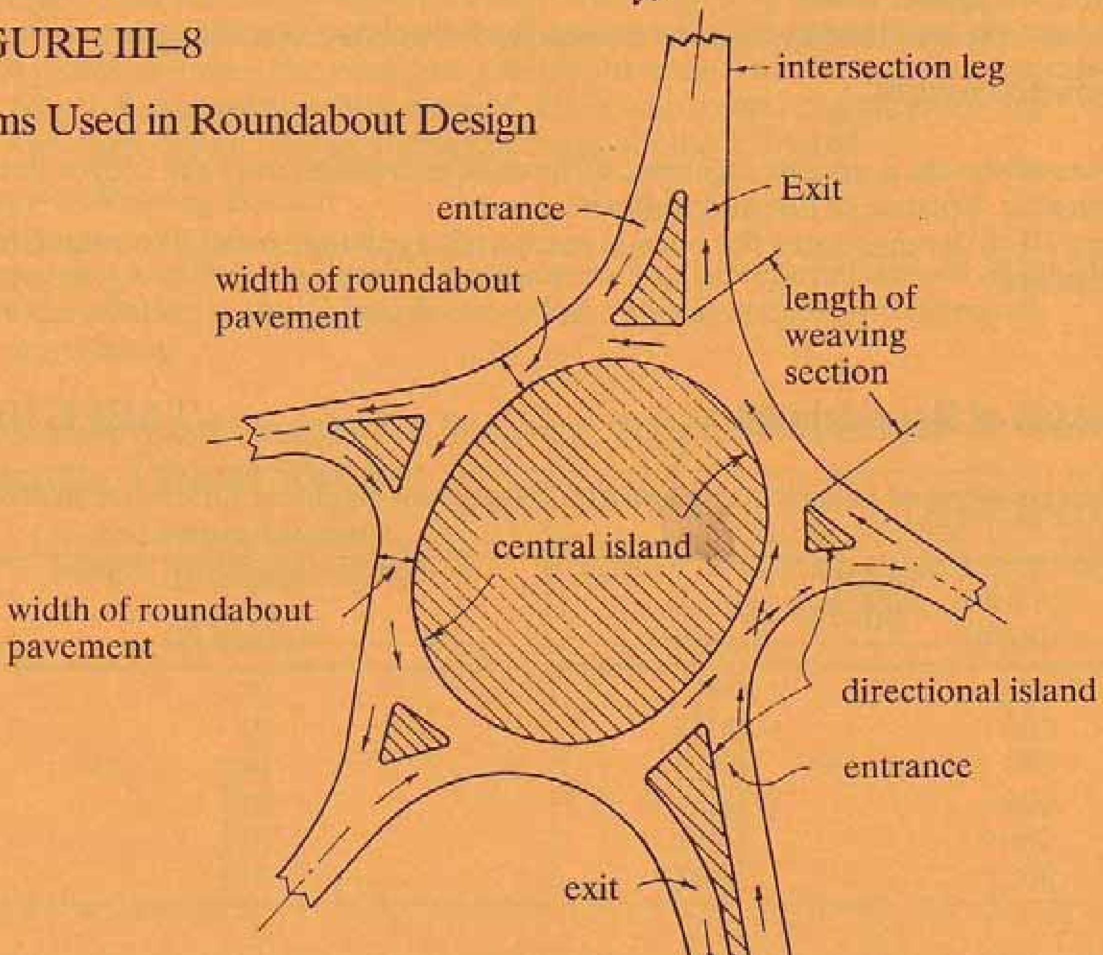


FIGURE III–8

Terms Used in Roundabout Design



5 ENCLOSURES

Minimum Radii in Relation to Design Speed and Rate of Superelevation

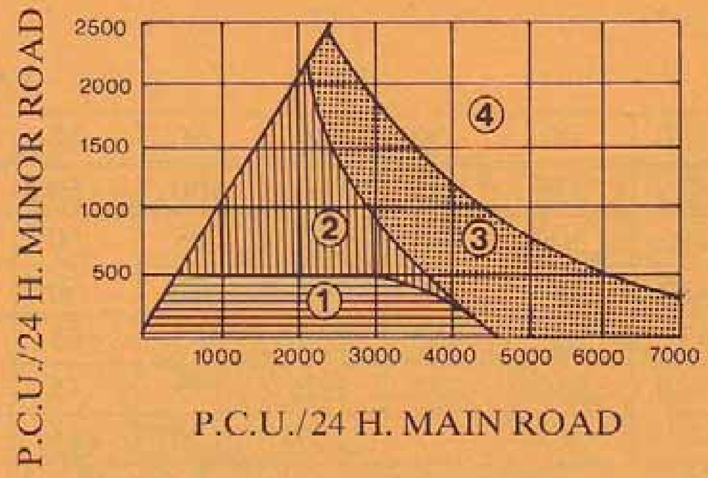
TABLE III–16

S \ V	120	100	80	70	60	50	40	30	25
2%	875	525	315	230	160	100	50	35	(20)
3%	810	495	300	215	150	95	50	35	(20)
4%	760	460	280	205	145	90	50	30	(20)
5%	710	420	265	195	135	75	45	25	(17)
6%	670	400	255	185	130	75	45	25	(17)
7%	–	–	–	–	(120)	(70)	(45)	(25)	(17)
minimum radii if superelevation is not developed									
2%		2300	1500	1100	800	500	400	250	
Note: ( ) for intersection design only. Radii indicated cannot be used for through-traffic lanes or interchanges.									

V = ramp design speed in km/h
S = superelevation in %
Intersections: Radii given for the inner edge of pavement.
Interchanges: Radii given for the center of pavement.

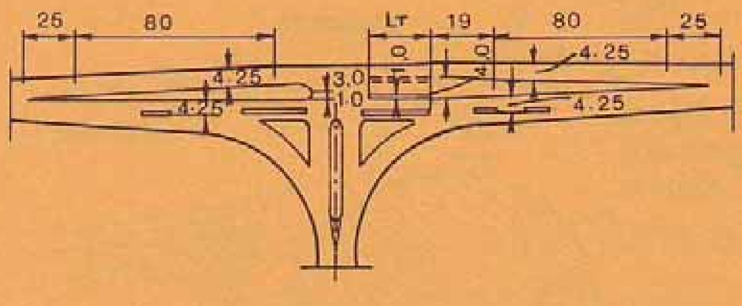
5 ENCLOSURES

5.1 Determination of a Type of Intersection



1–Intersection without traffic islands.
2–Divisional and channelizing trangular islands on the minor road.
3–Divisional and channelizing triangular islands on the minor road and divisional island on the main road
4–Control intersection or interchange

5.2 Dimension of Divisional Islands on Main Road

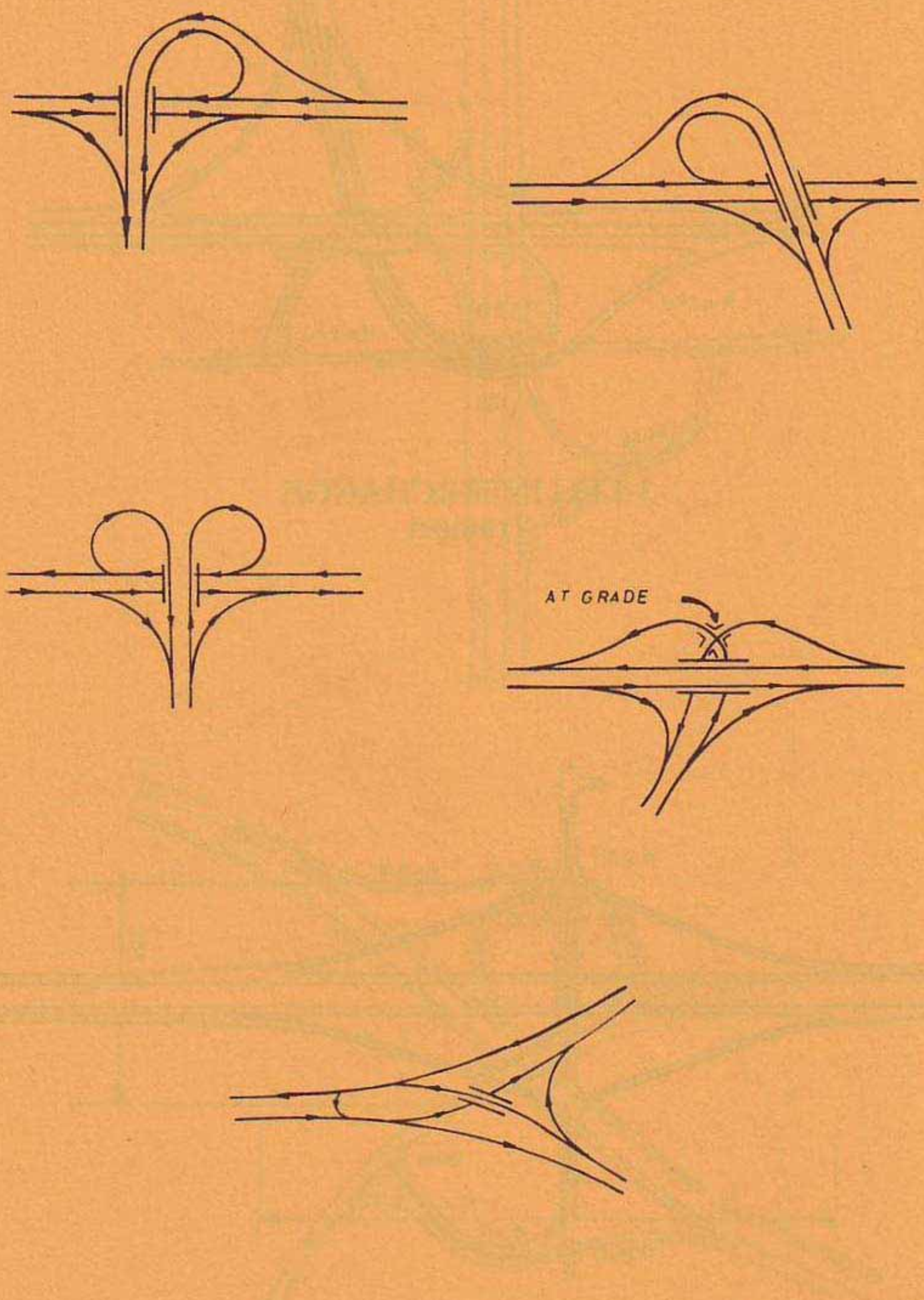


VOLUME OF TRAFFIC ON MAIN ROAD P.C.U./24 H	LENGTH OF THE PART LT METERS LEFT TURNING FROM MAIN ROAD%			
	10	20	30	40
2000	40	40	60	90
3000	40	50	70	110
4000	50	70	90	130
5000	70	90	120	160
6000	100	120	160	210

5 ENCLOSURES

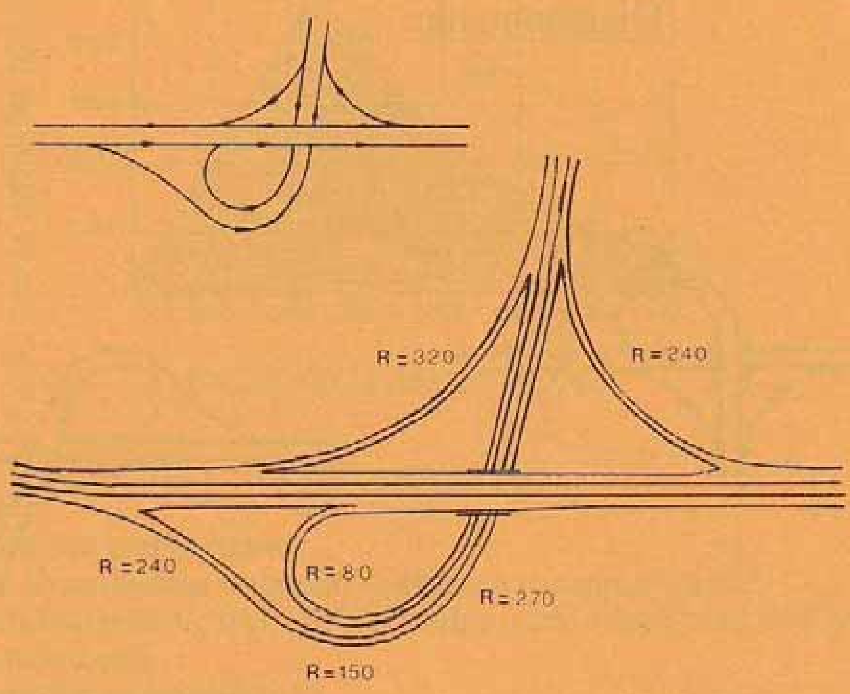
3-LEG INTERCHANGES

FIGURE III–9   3-leg Interchanges with a Single Structure, Diagrammatic

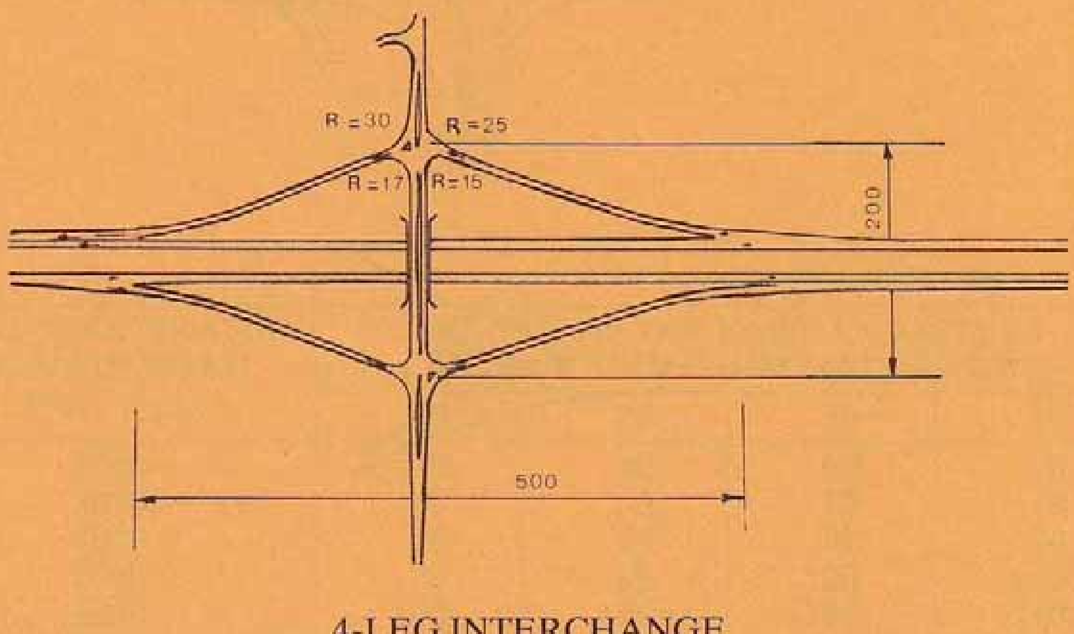


5 ENCLOSURES

FIGURE III–10 3-leg and 4-leg Interchanges



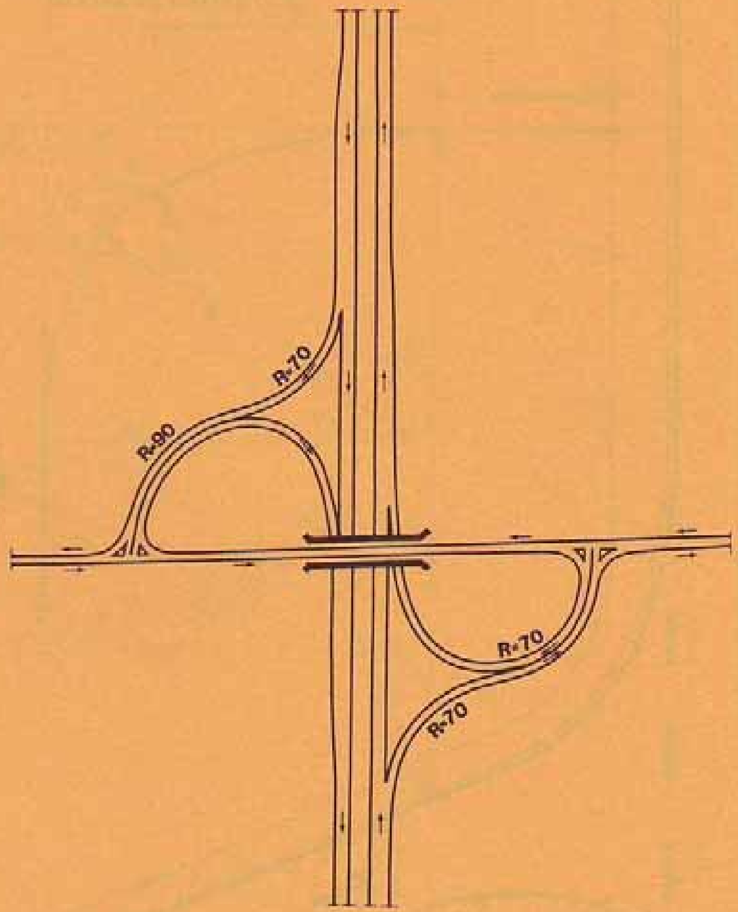
3-LEG INTERCHANGE
Trumpet

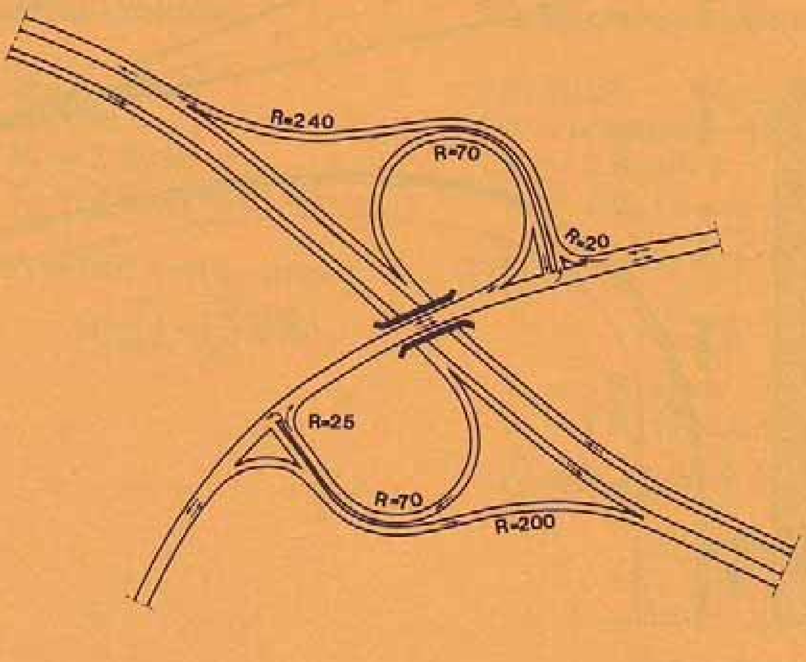


4-LEG INTERCHANGE
Diamond

5 ENCLOSURES

FIGURE III–11 4-leg Interchange 2 Quadrant Cloverleaf



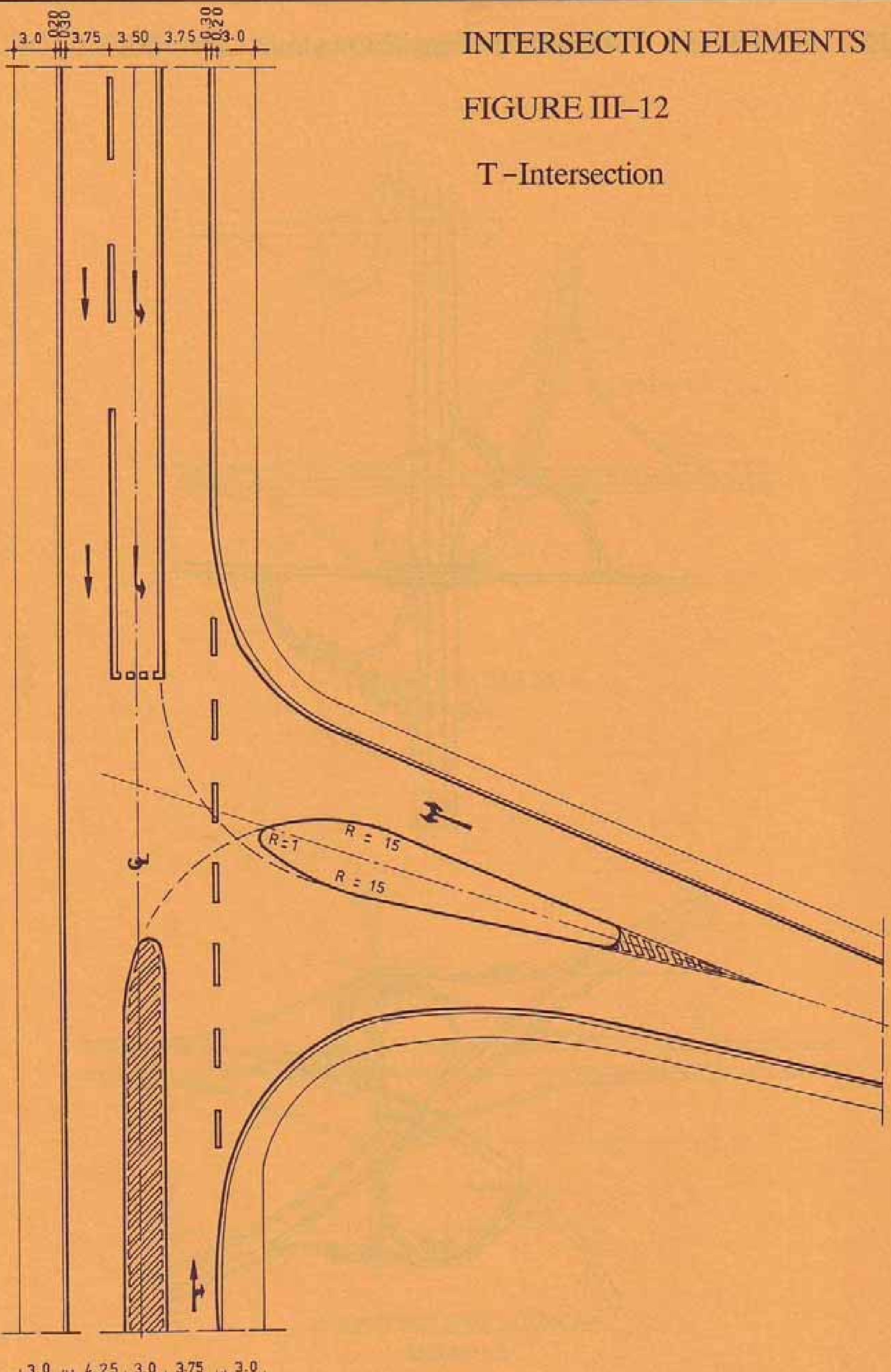


5 ENCLOSURES

INTERSECTION ELEMENTS

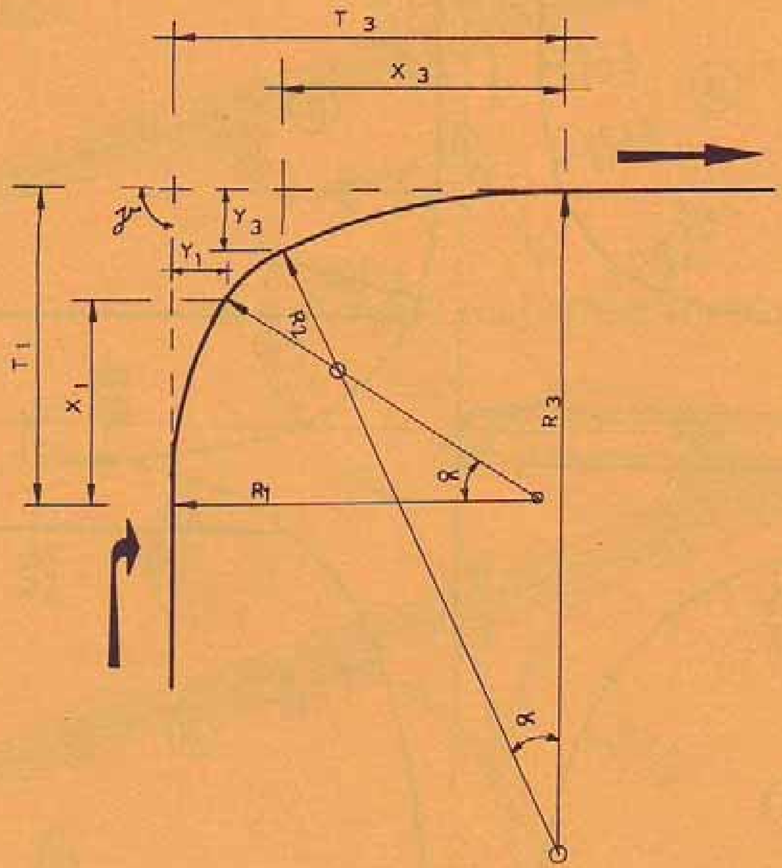
FIGURE III–12

T –Intersection



5 ENCLOSURES

FIGURE III–13 Compound Curve at Intersections



Symmetrical Solution

$R_1 : R_2 : R_3 = 2:1:2$

$\alpha = \text{constant} = 15°$

$X_1 = X_3 = 2\, R_2 \sin\alpha$

$Y_1 = Y_3 = 2\, R_2\, (1 - \cos\alpha)$

$$T_1 = T_3 = \left[\sin\alpha + (2 - \cos\alpha)\left(\frac{2}{\sin\gamma} - \frac{1}{\mathrm{tg}\,\gamma}\right)\right] R_2$$

Unsymmetrical Solution

$R_1 : R_2 : R_3 = 2:1:5$

$\alpha = \text{constant} = 15°$

$X_1 = 2\, R_2 \sin\alpha$

$Y_2 = 2\, R_2\, (1 - \cos\alpha)$

$$T_1 = R_2\left(\sin\alpha + \frac{5 - 4\cos\alpha}{\sin\gamma} - \frac{2 - \cos\alpha}{\mathrm{tg}\,\gamma}\right)$$

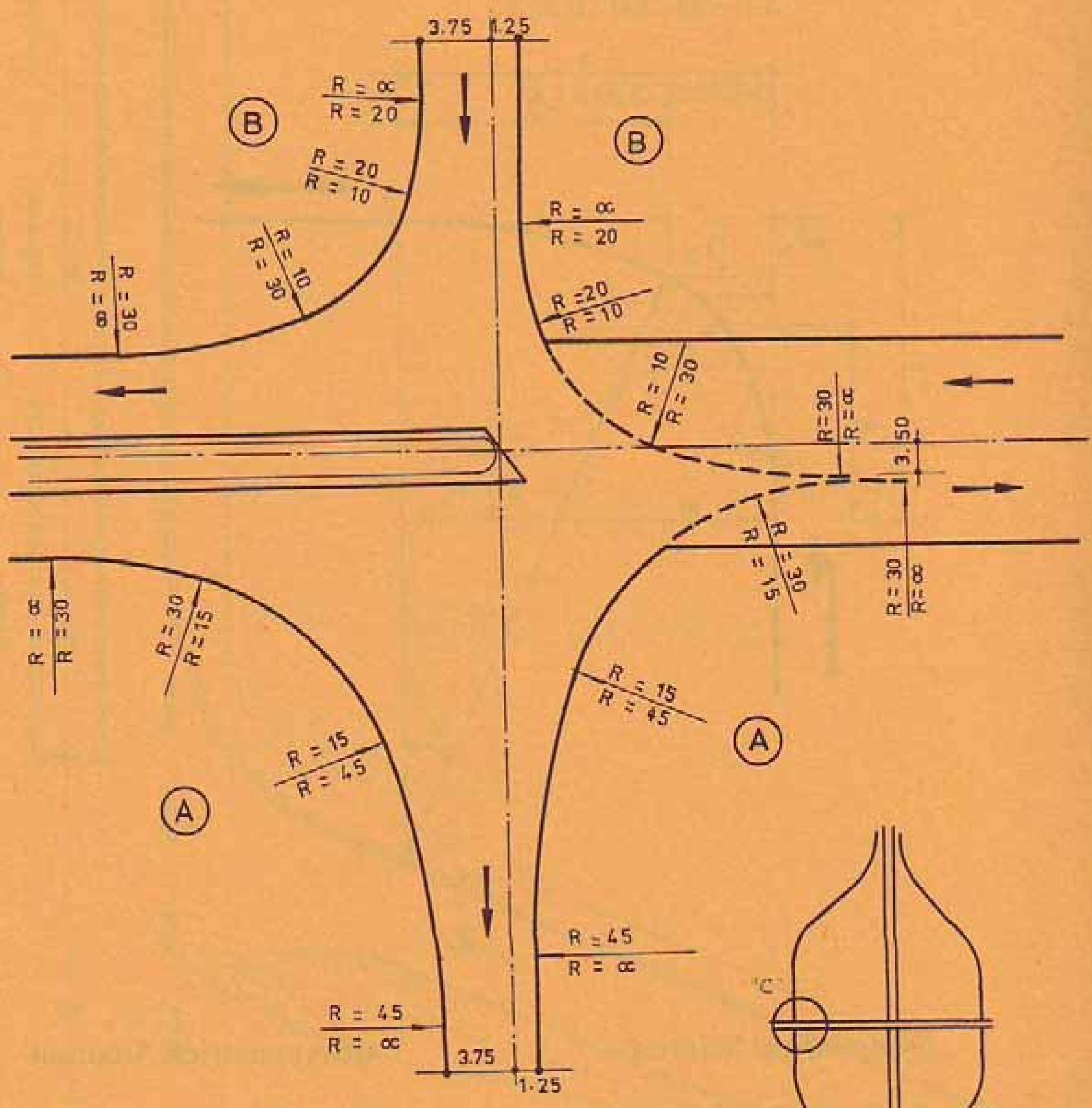
$X_3 = 5\, R_2 \sin\alpha$

$Y_3 = 5\, R_2\, (1 - \cos\alpha)$

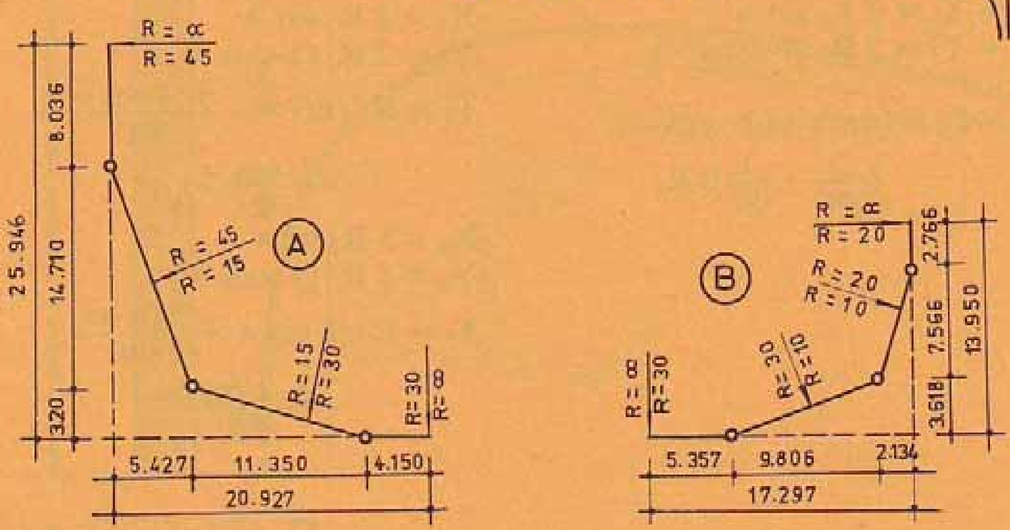
$$T_3 = R_2\left(4\sin\alpha + \frac{2 - \cos\alpha}{\sin\gamma} - \frac{5 - 4\cos\alpha}{\mathrm{tg}\,\gamma}\right)$$

5 ENCLOSURES

FIGURE III–14 Compound Curve at Intersections



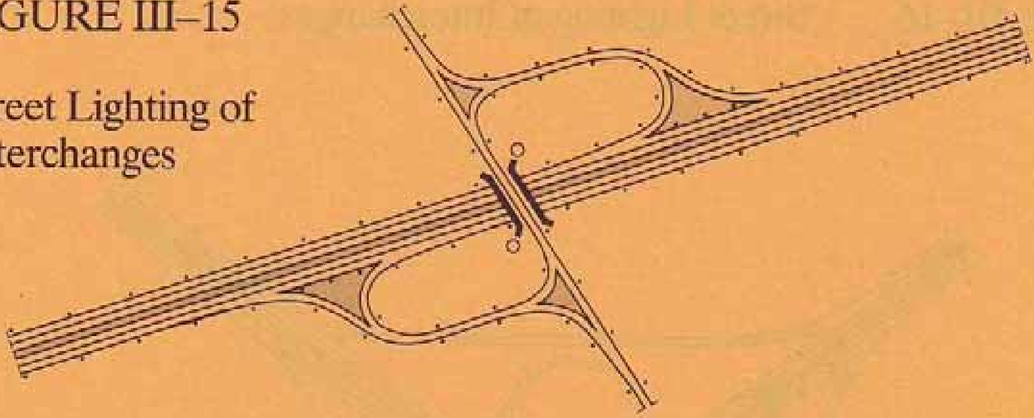
DETAIL AT "C"



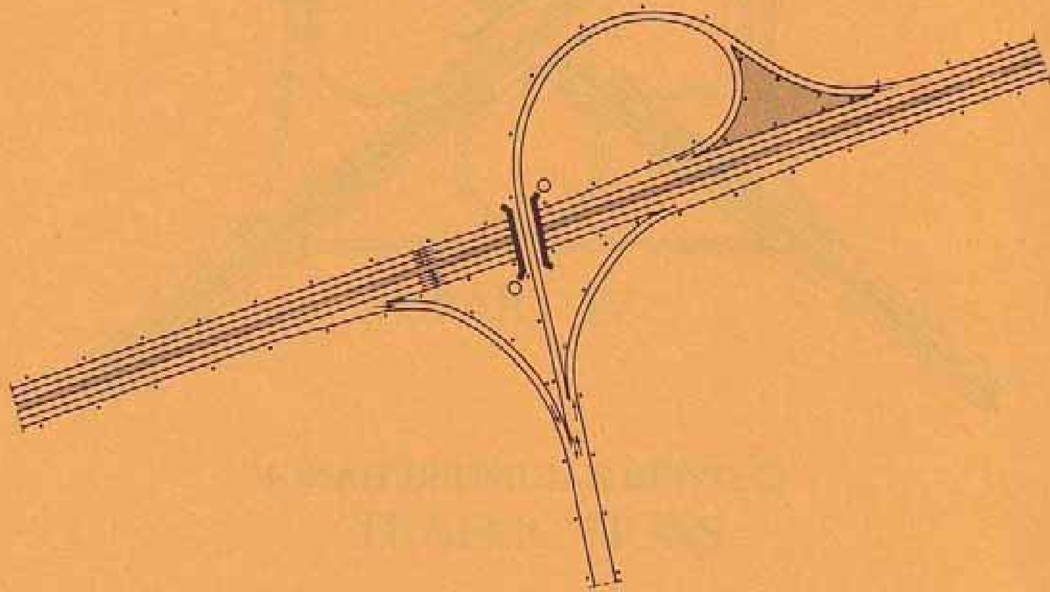
6 STREET LIGHTING

FIGURE III–15

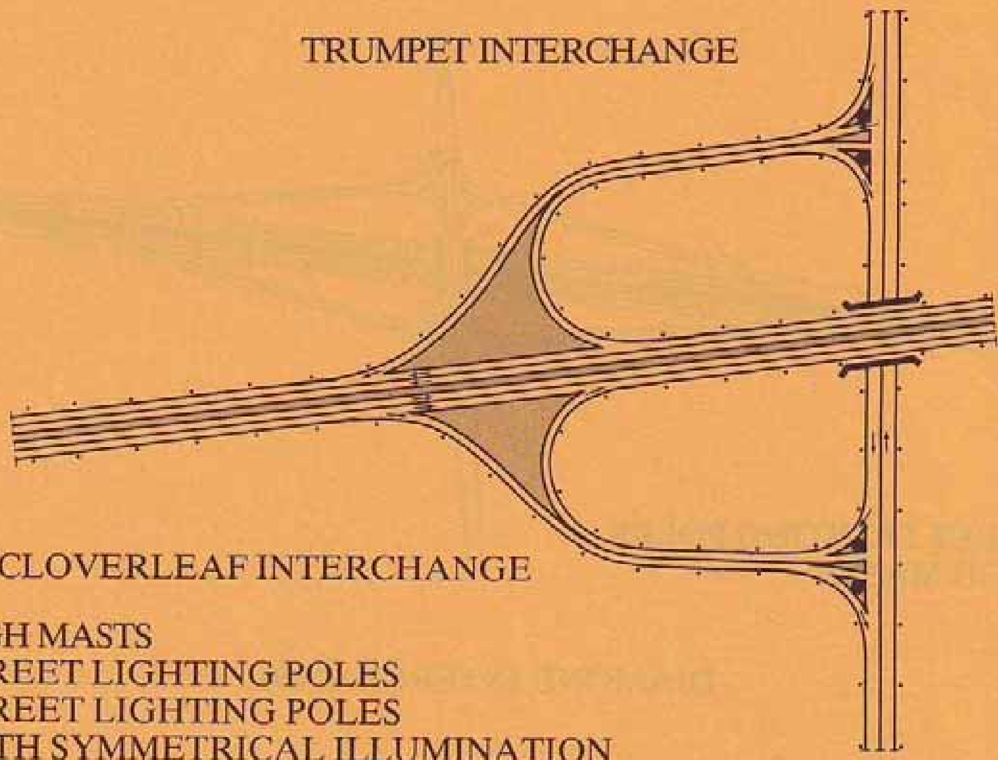
Street Lighting of Interchanges



SEMI-CLOVERLEAF INTERCHANGE



TRUMPET INTERCHANGE

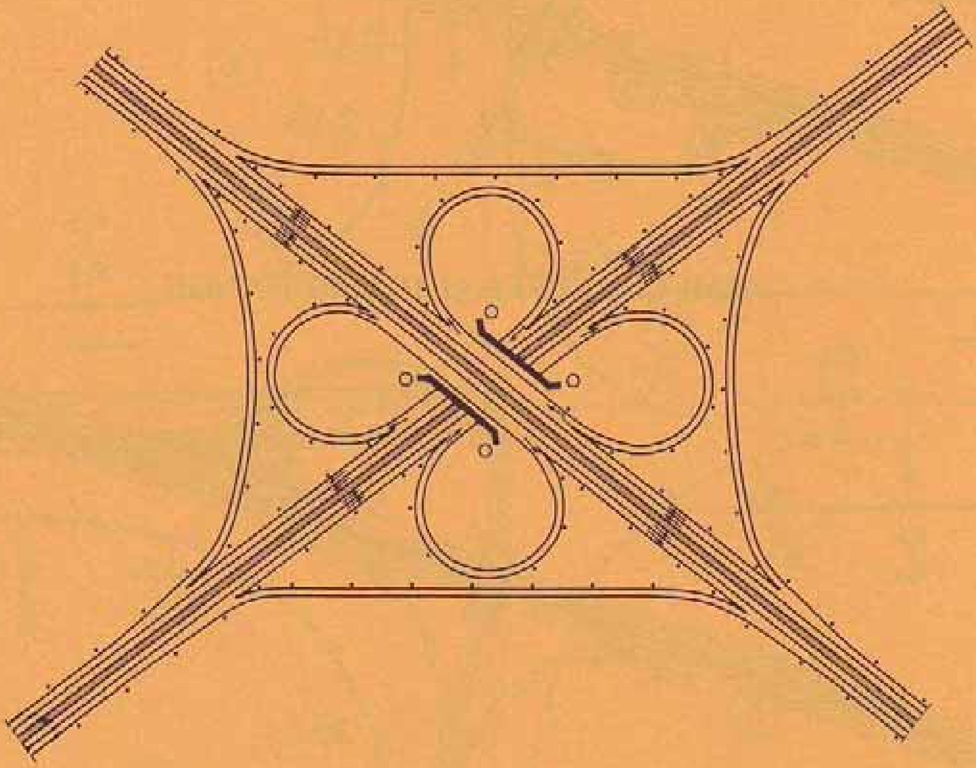


SEMI-CLOVERLEAF INTERCHANGE

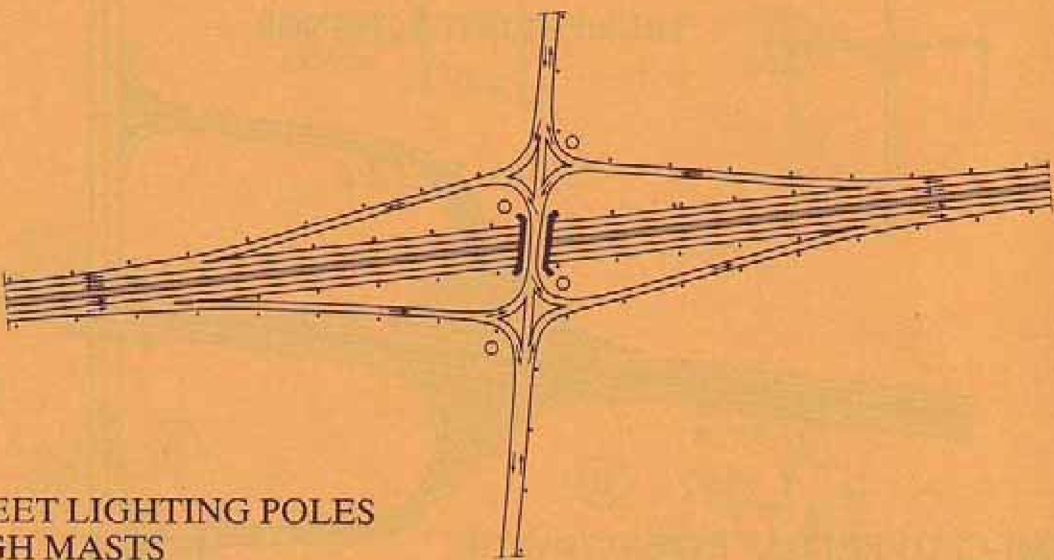
- ○ HIGH MASTS
- · STREET LIGHTING POLES
- ● STREET LIGHTING POLES WITH SYMMETRICAL ILLUMINATION OF THE AREA

6 STREET LIGHTING

FIGURE III–16 Street Lighting of Interchanges



CLOVERLEAF INTERCHANGE



· STREET LIGHTING POLES
○ HIGH MASTS

DIAMOND INTERCHANGE

PART IV

TRAFFIC SIGNS

PART IV TRAFFIC SIGNS

PART IV TRAFFIC SIGNS

LIST OF TABLES

PART IV TRAFFIC SIGNS

LIST OF FIGURES

1 TRAFFIC SIGN CHARACTERISTICS

1.01 FUNCTION 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.

The effectiveness of a sign depends mainly on the following factors:
a) The attention value of the sign
b) The legibility of the sign
c) The ease of comprehending the sign message
d) The appropriateness of the sign message

1.02 LEGAL AUTHORITY

Traffic signs shall be placed only on the authority of a public body or official with jurisdiction for the purpose of regulating, warning or guiding traffic. Neither a traffic sign nor its support shall bear any commercial advertising. Any other organization using temporary signs to protect equipment and workmen engaged in construction, maintenance or repair work on a public highway should see that signs conform to the standard in size, shape and colour.

1.03 CLASSIFICATION OF SIGNS

There are three sign classifications based on their intended function:
1) WARNING signs – call attention to conditions in or adjacent to a highway or street that are potentially hazardous to traffic operations.
2) REGULATORY signs – give the highway user notice of traffic regulations that apply at any given place or on a given highway. It is an offence to disregard such signs.
3) GUIDE signs – show route designations, destinations, directions, distances, points of interest and other geographical or cultural information.

1.04 SIGN SHAPES

Standard sign shapes are:
The octagon – reserved exclusively for the STOP sign.
The triangular shape with the point facing downwards – reserved for the GIVE WAY sign.
The triangular shape with the point facing upwards
– used for WARNING signs.
The round shape – used for REGULATORY signs.
The diamond shape – used for PRIORITY and END of PRIORITY signs.
GUIDE signs should be rectangular in shape. Any other sign plates with messages which are needed to supplement warning and regulatory signs should be rectangular in shape.

1 TRAFFIC SIGN CHARACTERISTICS

1.05 SIGN COLOURS

The colours 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 expressways.
   For expressways the direction signs should have a blue background with a white message or symbols.

1 TRAFFIC SIGN CHARACTERISTICS

1.06 SIGN BORDERS

The signs which are rectangular in shape (mostly guide signs) should have a narrow border of the same colour as the message. This improves the appearance. For 60 cm signs the border should be in the range of 1.0 to 1.5 cm set 1.0 cm 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 2 m by 3 m in size, the border should be approximately 5 cm wide or, on unusually large signs, 7.5 cm.
The corners of the border should be rounded.
Where practicable, the corners of the sign panels should also be rounded to fit the border.

1.07 ILLUMINATION AND REFLECTORISATION

Regulatory, warning and guide signs should be reflectorised or illuminated to show the same shape and colour both at day and night. Where an engineering study shows that reflectorisation will not perform effectively, all overhead sign installations should be illuminated. Illumination may be obtained by means of:
a) A light behind the sign face illuminating the main message or symbol, or the sign background, or both, through a translucent material.
b) An attached or independently mounted light source designed to provide uniform illumination over the entire face of the sign.
c) Some other effective device such as luminous tubing shaped to the lettering or symbols, a pattern of incandescent light bulbs or luminescent panels that will make the sign clearly visible at night.

1.08 STANDARD LOCATION OF SIGNS

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. 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 Fig IV–1.

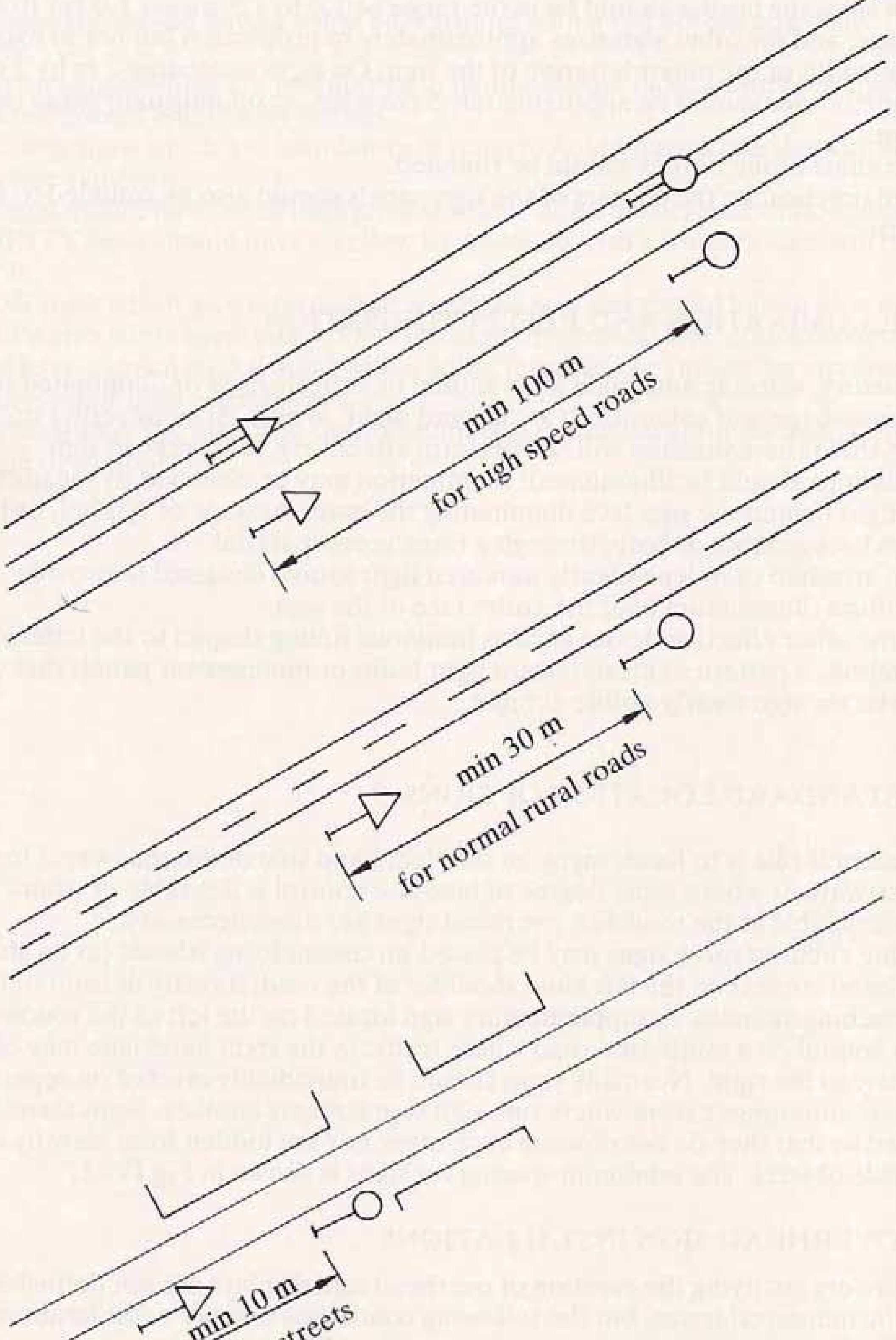
1.09 OVERHEAD SIGN INSTALLATIONS

The factors justifying the erection of overhead sign displays are not definable in specific numerical terms, but the following conditions deserve consideration:

1. Traffic volume at or near capacity.
2. Complex interchange design.
3. Three or more lanes in each direction.
4. Restricted sight distance.
5. Closely spaced interchanges.
6. Multi-lane exits.
7. Large percentage of trucks.
8. Background of street lighting.
9. High speed of traffic.
10. Insufficient space for ground signs.

1 TRAFFIC SIGN CHARACTERISTICS

FIGURE IV–1 Minimum Spacing of Traffic Signs



1 TRAFFIC SIGN CHARACTERISTICS

1.10 HEIGHT OF SIGNS

Signs erected at the side of the road in rural areas should be mounted at a height of at least 1.5 m measured from the bottom of the sign to the nearside edge of pavement.
In business, commercial and residential areas where parking and/or pedestrian movement is likely to occur, the clearance to the bottom of sign should be at least 2 m. The height to the bottom of a secondary sign mounted below another sign may be 30 cm less than the appropriate height specified above.
Overhead signs should provide a vertical clearance of not less than 5.2 m over the entire width of pavement and shoulders except where a lesser vertical clearance is used for the design of other structures. Suggested heights of signs for different kinds of roads are shown in Fig. IV–2.

1.11 LATERAL CLEARANCE

Signs should have the maximum practical lateral clearance from the edge of the roadway for the safety of the motorist. As illustrated in Fig. IV–3, signs should be placed so that the distance from the near edge of the nearest traffic lane to the near sign edge is not less than 2 m nor more than 4 m. In urban areas a sign should be placed adjacent to the roadway with its nearest edge not less than 30 cm nor more than 2 m from the kerb line.

1.12 POSITION OF SIGNS

A warning sign is placed in advance of the condition to which it calls attention. A regulatory sign is normally placed where its mandate or prohibition applies or begins. Guide signs are placed (usually in advance) to keep drivers well informed as to the route to their destination.

1.13 ERECTION

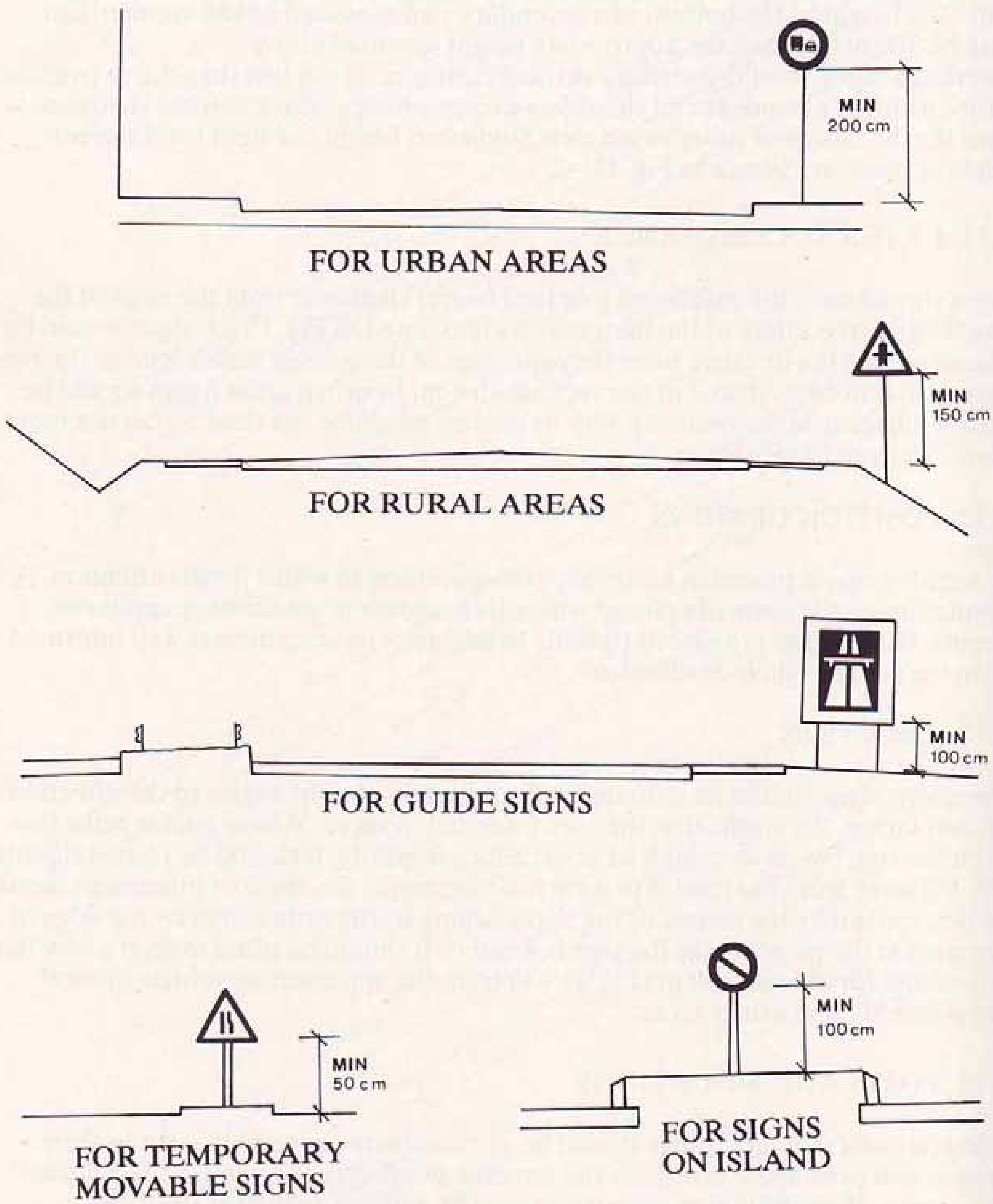
Normally, signs should be mounted approximately at right angles to the direction of, and facing, the traffic that they are intended to serve. Where mirror reflection from the sign face is very high so as to reduce legibility, it should be turned slightly (3°–5°) away from the road. For a curved alignment, the angle of placement should be determined by the course of the approaching traffic rather than by the edge of the road at the point where the sign is located. It should be tilted in such a way that it is visible for at least 100 m (Fig. IV–4) from the approaching vehicle in rural areas and 50 m in urban areas.

1.14 POSTS AND MOUNTINGS

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 sufficently distant from the pavement edge, sign supports should be of a suitable collapsable 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 minimising footpath obstruction.
Figs. IV 5A to D illustrate some features of sign mounting.

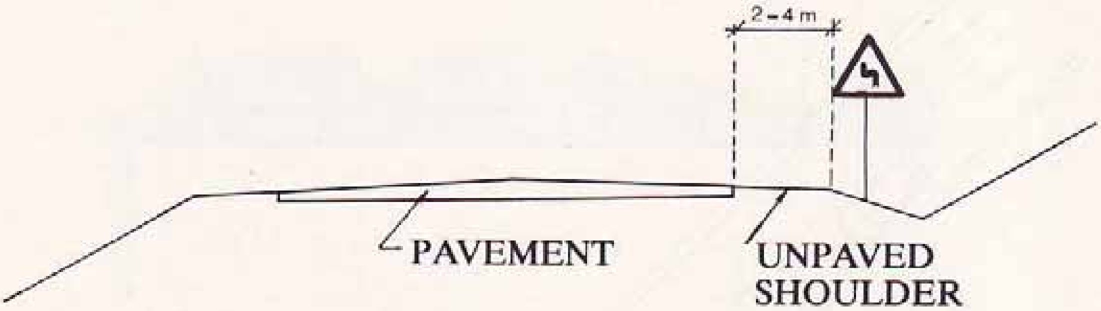
1 TRAFFIC SIGN CHARACTERISTICS

FIGURE IV–2 Minimum Mounting Height for Signs

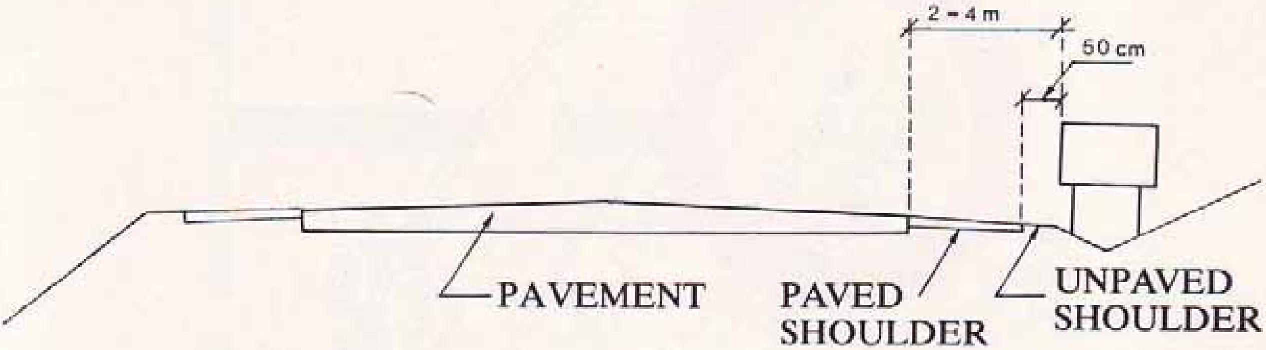


1 TRAFFIC SIGN CHARACTERISTICS

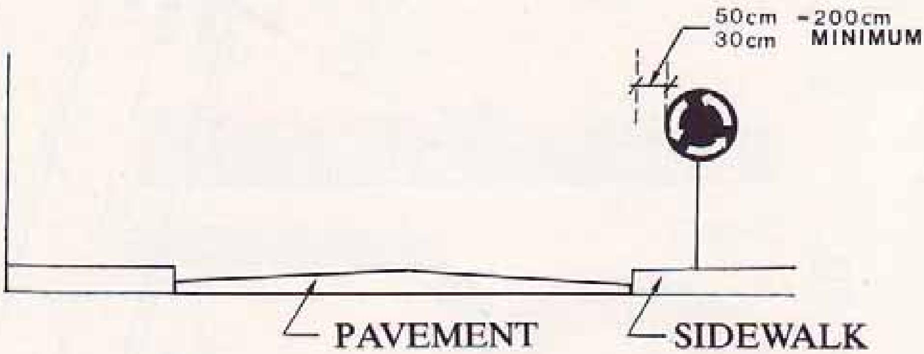
FIGURE IV–3 Lateral Location of Signs



NORMAL RURAL ROAD



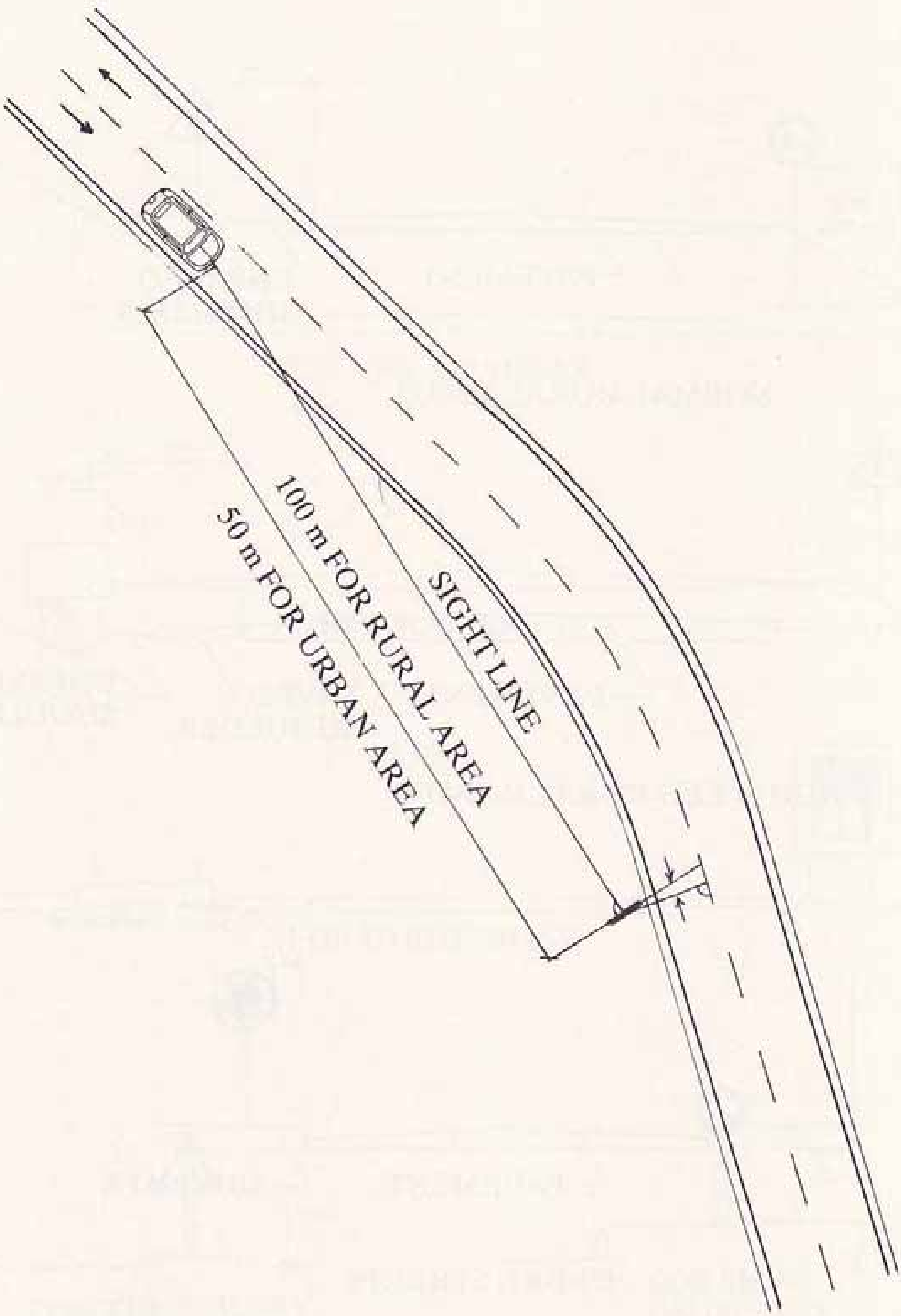
HIGH SPEED RURAL ROAD



URBAN STREETS

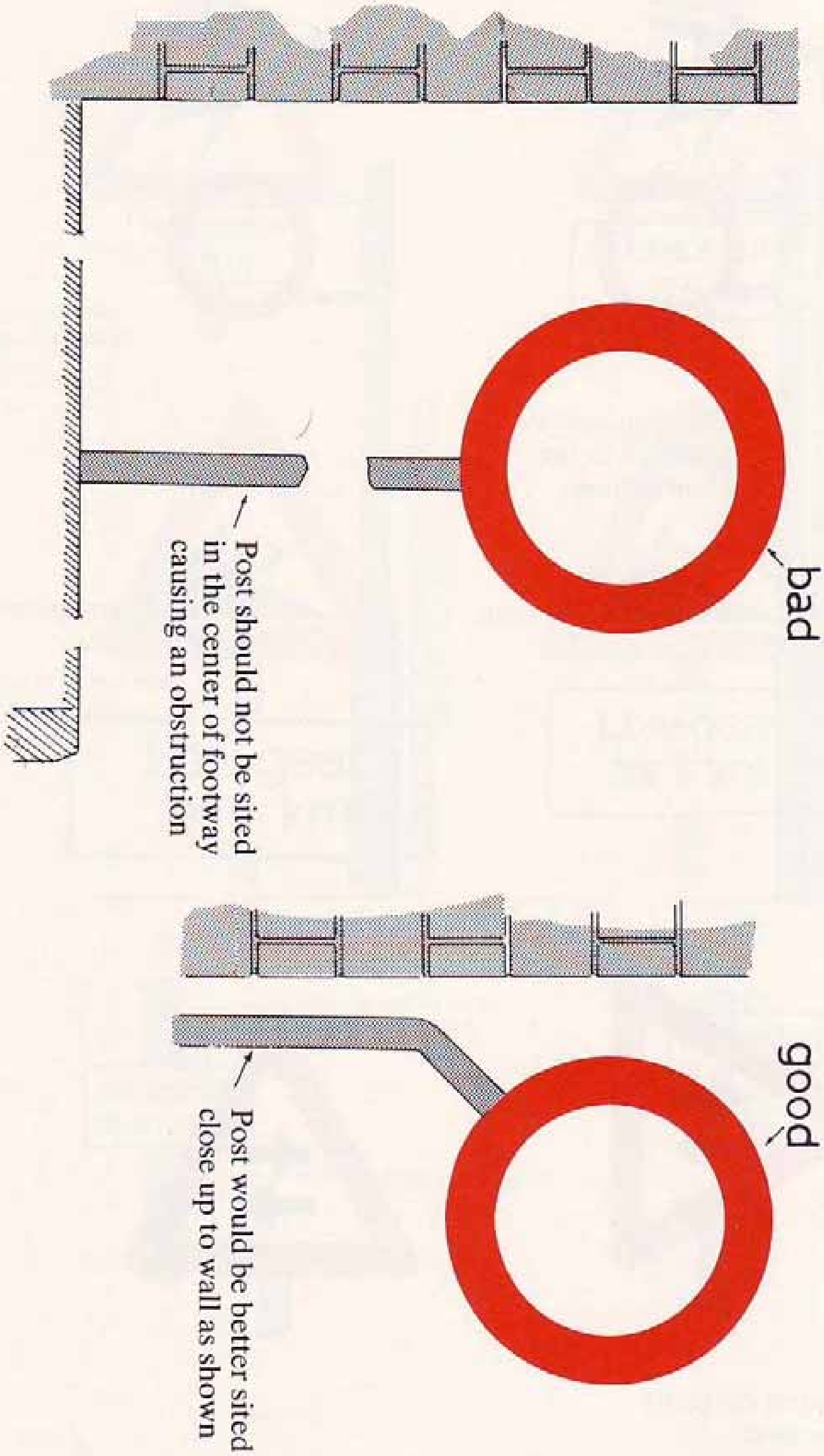
1 TRAFFIC SIGN CHARACTERISTICS

FIGURE IV–4 Location of Signs at Bend



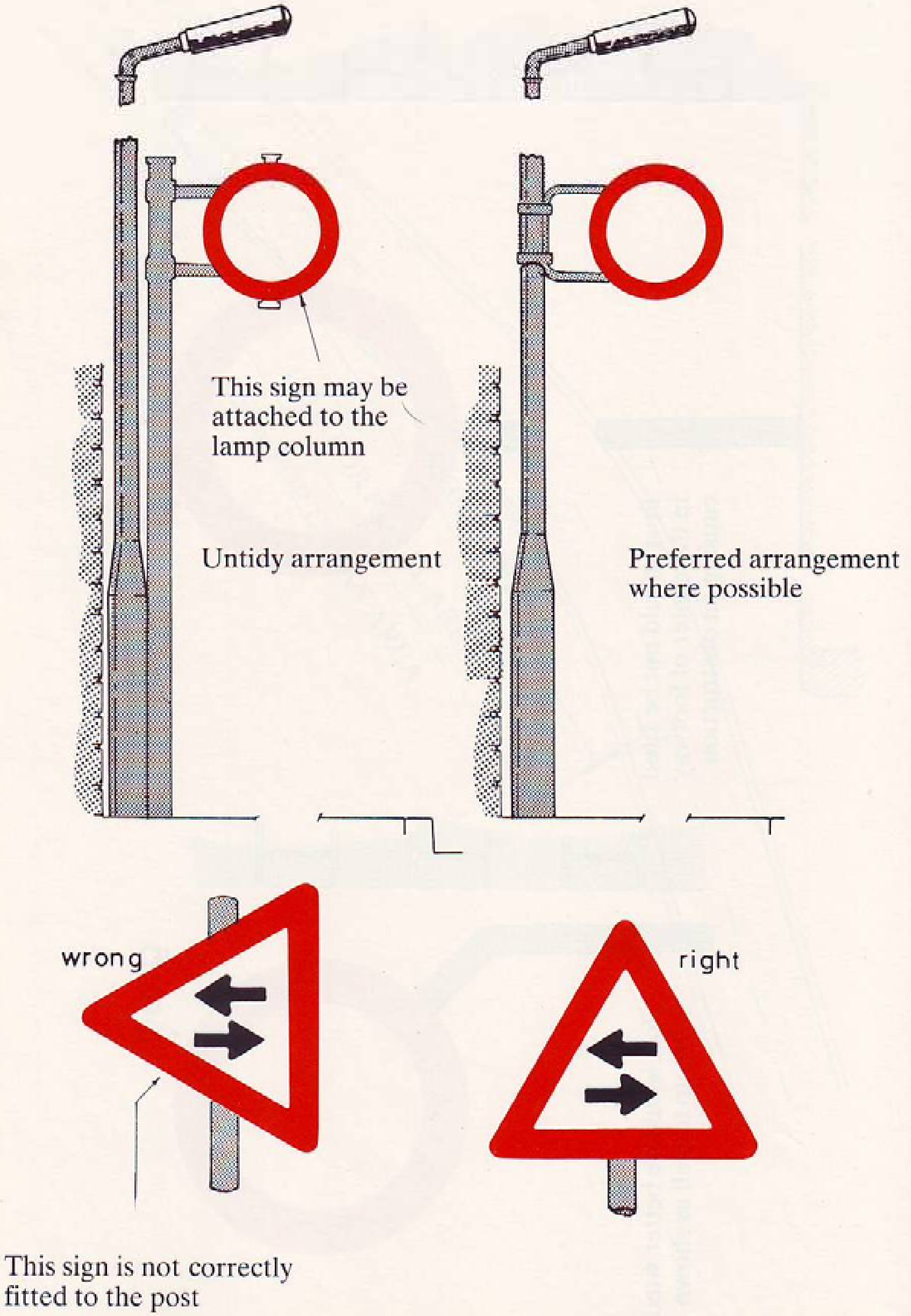
1 TRAFFIC SIGN CHARACTERISTICS

FIGURE IV–5 A Sign Mounting



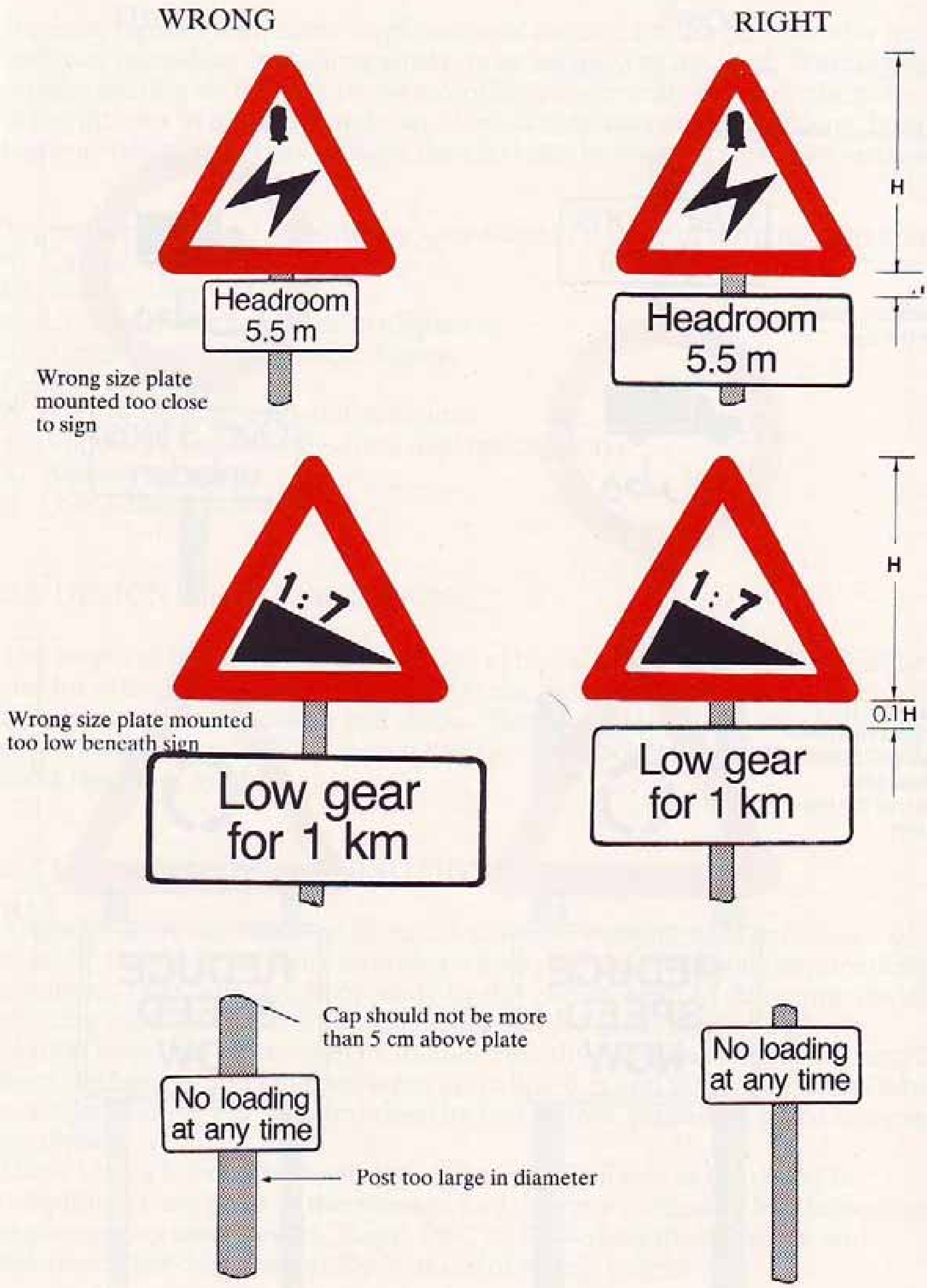
1 TRAFFIC SIGN CHARACTERISTICS

FIGURE IV–5B Sign Mounting



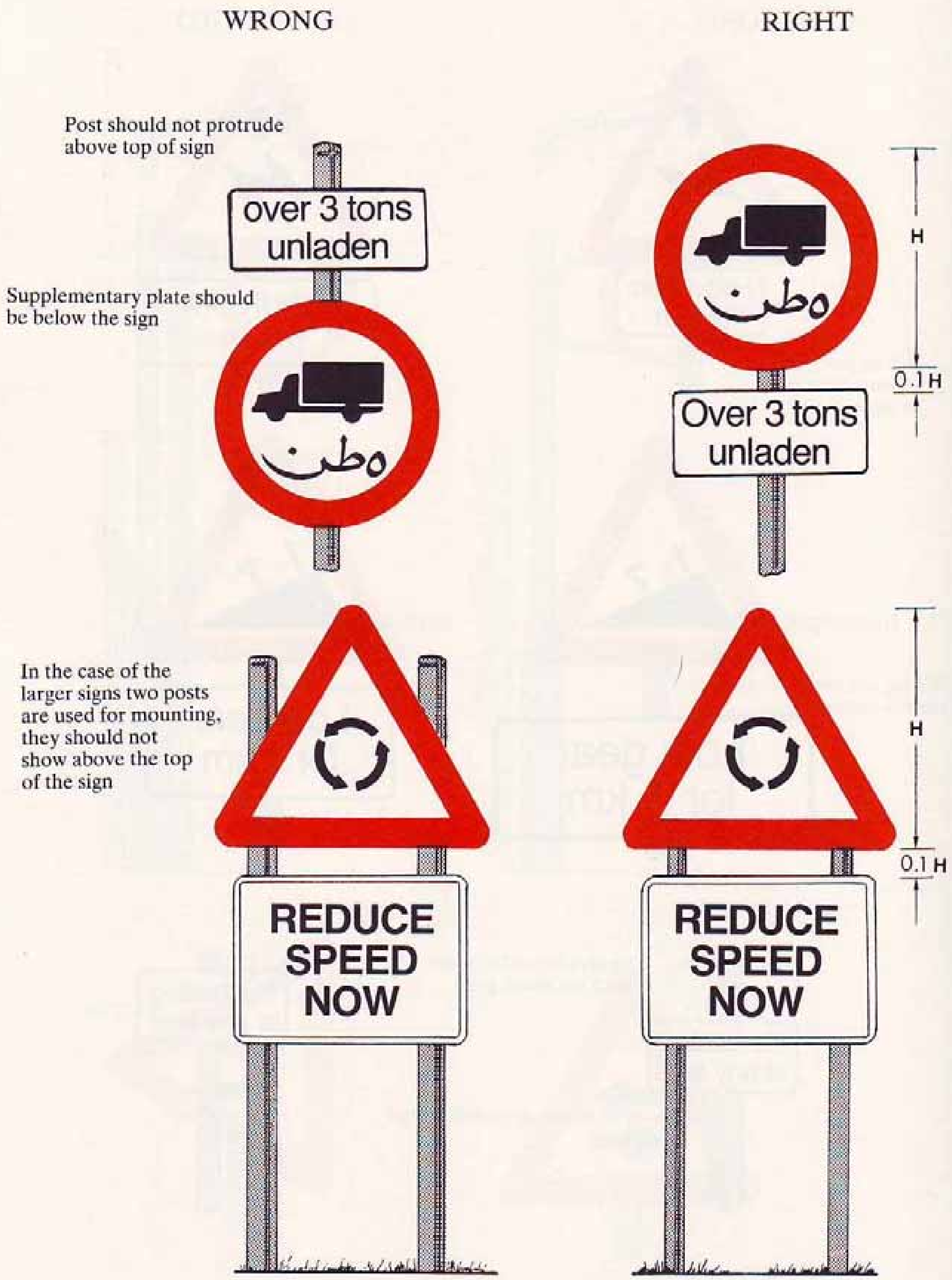
1 TRAFFIC SIGN CHARACTERISTICS

FIGURE IV–5C Sign Mounting



1 TRAFFIC SIGN CHARACTERISTICS

FIGURE IV–5D Sign Mounting



2 WARNING SIGNS

2.1 APPLICATION

Warning signs as their name implies should be used for the purpose of warning traffic of hazardous conditions either on or adjacent to the road. Warning signs require caution on the part of the motorist and generally call for reduction of speed in the interest of his safety and that of other motorists and pedestrians. Improved highway design generally reduces the need for the warning signs.

Typical locations and hazards that may warrant the use of warning signs are:
a) Curves
b) Intersections
c) Unusually sharp changes in alignment
d) Advance warning of traffic signals
c) Hills and bumps
f) Narrow carriageways and structures
g) Approaches to and exits from dual carriageways.
h) Railway crossings
i) Other dangerous spots

2.2 DESIGN OF WARNING SIGNS

The length of the side of a warning sign to be used for primary roads shall be 90 cm and for other lower category roads – 60 cm. Warning signs convey their message by legend or symbol, colour and shape. They should be triangular shaped with the point facing upwards. They should have a white background with a red border and black legend or symbols.

2.3 LOCATION OF WARNING SIGNS

Although there are standard distances to locate warning signs in advance of hazards, there will be many instances where physical conditions require different distances. Trial runs should be made by day and at night to determine the most effective location and erection characteristics for each installation.
In rural areas, the warning signs should normally be placed between 90 and 250 m from the hazard, and in urban areas between 40 m and 90 m. The actual advance warning distance will be determined by two factors, prevailing speed and prevailing conditions.
These have a bearing respectively on the time available to the driver to comprehend and react to the message and the time needed by him to perform any necessary manoeuvres. Tables IV–1 to IV–4 show the minimum and recommended distances for the location of warning signs.

2 WARNING SIGNS

2.4 CURVE SIGNS (A–1a – A–1d)

On rural highways the curve sign is intended for use where engineering investigations of geometric and operating conditions on a roadway show that the recommended speed on the curve is less than 100 km/h and/or equal to or less than the speed limit established by law or regulation for that section of highway. Additional warnings may be provided by the use of the advisory speed limit, distance plate and by the installation of road delineation markers. Where two curves in opposite directions are separated by a tangent less than 250 m long, a reverse curve sign shall be used.

TABLE IV–1

Minimum and Recommended Distances for the Location of Traffic Signs Requiring Vehicles to Stop

GRADIENT OF ROAD IN %	MAX SPEED OF APPROACHING VEHICLE (km/h)									
	120	110	100	90	80	70	60	50	40	
−6	320*	260	215	170	137	105	80	55	35	
−5	310	255	210	165	130	105	75	55	35	
−4	300	245	200	160	125	100	75	55	35	
−3	290	240	195	160	120		75	55	35	
−2	200	230	190	155	120		70	50	35	
−1	270	225	185	150	115		70	50	35	MINIMUM
0	260	220	180	145	115		70	50	35	DISTANCE
+1	250	210	175	145	110		70	50	35	OF SIGN
+2	240	205	170	140	110		65	50	35	(m)
+3	235	200	165	135			65	50	35	
+4	230	195	160	135			65	50	35	
+5	225	190	160	130			65	45	30	
+6	220	180	155	130			60	45	30	
	250		220	180	150	120	100			RECOMMENDED DISTANCE (m)

* THE SIGN SHOULD BE REPEATED IN THIS CASE

2 WARNING SIGNS

TABLE IV–2

Minimum and Recommended Distances for the Location of Traffic Signs Requiring Vehicles to Reduce Speed to 40 (km/h)

GRADIENT OF ROAD IN %	MAX SPEED OF APPROACHING VEHICLE (km/h)							
	120	110	100	90	80	70	60	
−6	300*	240	165	150	110	85	60	MINIMUM DISTANCE OF SIGN (m)
−5	290	235	190	145	110	85	55	
−4	280	225	180	140	110	80	55	
−4	210	220	175	140	105	80	55	
−2	260	210	170	135	100	75	50	
−1	250	205	165	130	100	75	50	
−0	240	200	160	130	95	75	50	
+1	230	190	155	123	95	70	50	
+2	220	185	150	125	90	70	45	
+3	215	180	185	120	90	70	45	
+4	215	180	145	120	90	70	45	
+5	210	175	145	115	90	70	45	
+6	205	170	140	115	90	70	45	
	250	220	180	150	120		100	RECOM-MENDED DISTANCE (m)

* THE SIGN SHOULD BE REPEATED IN THIS CASE

2 WARNING SIGNS

TABLE IV–3

Minimum and Recommended Distances for the Location of Traffic Signs Requiring Vehicles to Reduce Speed to 60 (km/h)

GRADIENT OF ROAD IN %	MAX SPEED OF APPROACHING VEHICLE (km/h)					
	120	110	100	90	80	
−6	270*	210	165	120	85	MINIMUM DISTANCE OF SIGN (m)
−5	265	210	160	120	85	
−4	255	200	155	115	85	
−3	245	195	150	115	80	
−2	235	185	145	110	75	
−1	225	180	140	105	75	
−0	220	180	140	105	75	
+1	210	170	135	100	75	
+2	200	165	130	100	70	
+3	195	160	125	95	70	
+4	195	155	120	95	70	
+5	185	150	120	90	65	
+6	185	150	120	90	65	
	250	220	180	150	120	RECOMMENDED DISTANCE (m)

* THE SIGN SHOULD BE REPEATED IN THIS CASE

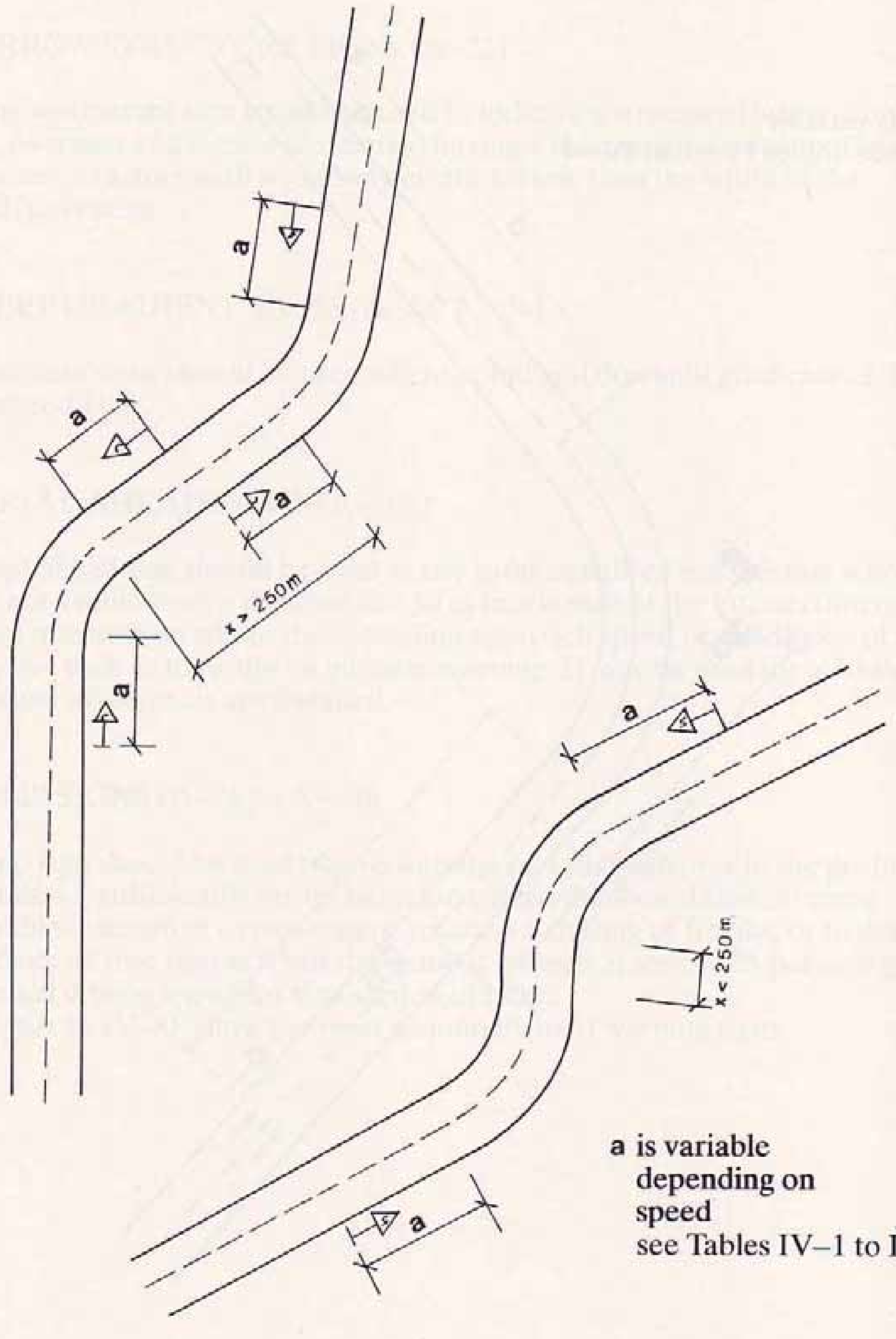
TABLE IV–4

Recommended Distances for the Location of Traffic Signs

SPEED LIMIT NEAR TRAFFIC SIGN (km/h)	MAX SPEED OF APPROACHING VEHICLE (km/h)								
	120	110	100	90	80	70	60	50	40
0	300	250	220 to 250	180 to 220	150 to 180	120 to 150	100 to 120	100	100
40	260 to 300	220 to 250	180 to 220	150 to 180	120 to 150	120	100		
60	250	220 to 250	180 to 220	150	120				

2 WARNING SIGNS

FIGURE IV–6 Criteria for Curve Sign and Reverse Curve Sign

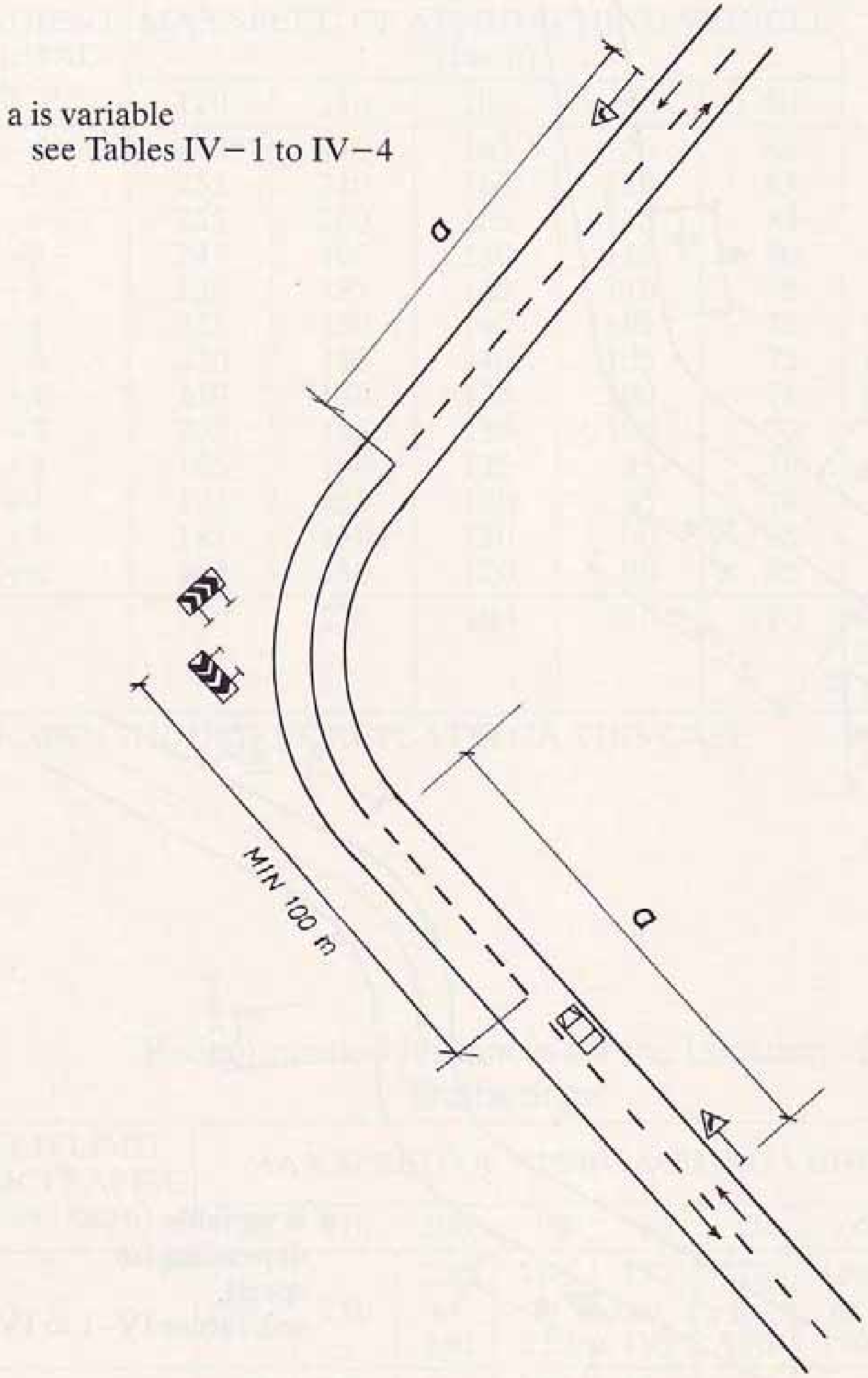


a is variable depending on speed see Tables IV–1 to IV–4

x = Length of tangent

2 WARNING SIGNS

FIGURE IV–7 Application of Delineator on Sharp Bends



2 WARNING SIGNS

2.5 PAVEMENT NARROWS SIGNS (A–4a, A–4b)

The pavement narrows sign should be used to give advance warning of a reduction in the number of lanes.

2.6 NARROW STRUCTURE SIGNS (A–21)

The narrow structure sign should be used to indicate a structure (bridge, culvert, subway, overpass and similar structures) having a clear roadway width of less than 5.5 m or any structure with a roadway clearance less than the width of the approach pavement.

2.7 STEEP GRADIENT SIGNS (A–2a, A–3a)

Steep gradient signs should be used where uphill and downhill gradients of the road ahead exceed 10%.

2.8 SIGNAL AHEAD SIGNS (A–16a)

The signal ahead sign should be used at any rural signalised intersection where the signal is not visible from a distance of 150 m in advance of the intersection or at any signalised intersection where the prevailing approach speed or conditions of visibility are such as to justify an advance warning. It may be used for a 30-day period when new signals are installed.

2.9 BUMP SIGNS (A–7a to A–7c)

The bump sign should be used to give warning of a sharp change in the profile of the road that is sufficiently abrupt to create a hazardous condition, to cause considerable discomfort to passengers, to cause a shifting of freight, or to deflect a vehicle from its true course when the bump is crossed at speeds 25 per cent greater than normal driving speed for that section of road.
Figs. IV–8A to IV–8F show the most commonly used warning signs.

2 WARNING SIGNS

FIGURE IV–8A

A–1a
Left bend

A–1b
Right bend

A–1c
Double bend
first to the left

A–1d
Double bend
first to the right

A–3a
Steep descent

A–2a
Dangerous ascent

A–4a
Carriageway narrows

A–4b
Carriageway narrows (from right)

2 WARNING SIGNS

FIGURE IV–8B

A–5
Opening bridge

A–6
Quayside or riverbank ahead

A–7a
Uneven road

A–7b
Ridge

A–7c
Dip

A–8
Slippery carriageway

A–9a
Loose gravel

A–10a
Risk of falling rocks (R)

2 WARNING SIGNS

FIGURE IV–8C

A–11a
Pedestrian crossing

A–12
Children crossing

A–13
Cyclists entering
or crossing

A–14a
Cattle crossing

A–14b
Animal crossing

A–15
Road works

A–16a
Traffic signals ahead

A–17
Airfield

2 WARNING SIGNS

FIGURE IV–8D

A–18
Cross-wind

A–19
Right hand lane of a three-lane carriageway closed to traffic ahead

A–20
Other dangers

A–21
Narrow structure

A–22
Dual carriageway ends

A–23a
Two-way traffic

A–23b
Two-way traffic across a one-way carriageway

A–24
Tunnel ahead

2 WARNING SIGNS

FIGURE IV–8 E

Warning Signs near Intersection

A–25
Intersection ahead

A–26
Intersection with a non-priority road

A–27
Junction of a non-priority road from the left

A–28
Junction of a non-priority road from the right

A–29
Junction of a non-priority road from the left (or merging traffic)

A–30
Junction of a non-priority road from the right (or merging traffic)

A–31
Roundabout ahead

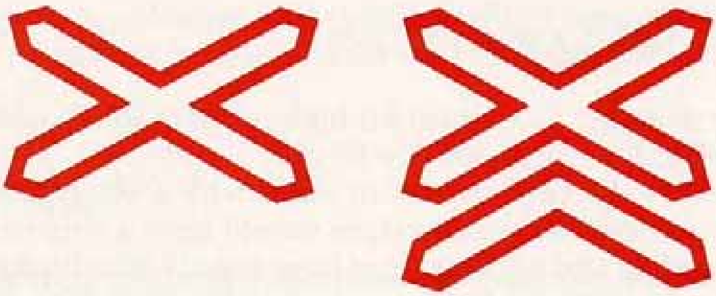
2 WARNING SIGNS

FIGURE IV–8F

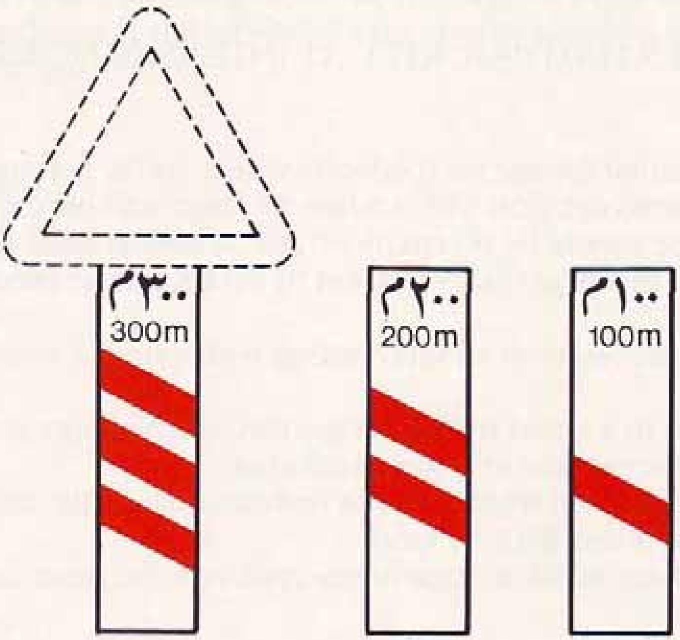
Signs Concerning Level Crossing

A–32a
Level crossing with gates

A–32b
Level crossing without gates



A–32c
Signs placed in the immediate vicinity of a level crossing



A–32d
Additional signal at approaches to level crossing

3 REGULATORY SIGNS

3.1 APPLICATION

Regulatory signs should be used to inform highways users of certain laws and regulations enacted to promote safety and convenience on the streets and highways. Regulatory signs should be erected at those locations where the regulations apply and should be mounted so as to be easily visible to the motorist.
Regulatory signs are classified in the following groups:
(i) Signs regulating priority at intersection
(ii) Prohibitory or restrictive signs
(iii) Mandatory signs
(iv) Standing and parking signs
(v) Miscellaneous signs

3.2 DESIGN OF REGULATORY SIGNS

The side length or diameter to be used for high speed roads should be 90 cm and for urban areas and other low speed roads – 60 cm.
Most of the regulatory signs are circular in shape with a white background, red border and black symbols. Mandatory signs should have a blue background with white symbols. Standing and parking signs have mostly blue backgrounds with red borders and are round in shape. Signs regulating priority at intersections have a distinctive shape and colour compared with other regulatory signs.

3.3 SIGNS REGULATING PRIORITY AT INTERSECTIONS

STOP SIGN
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. The size should be 60 cm by 60 cm. Where greater emphasis or visibility is required, the larger size of 90 cm by 90 cm can be used.

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 intersecting a through highway or through street.
(ii) Unsignalised intersection in a signalised 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. Typical location of STOP signs are illustrated in Fig. IV–9.

3 REGULATORY SIGNS

3.4 GIVE WAY SIGNS

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 15 km/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 roadways exceeds 10 m.
(iv) Where there is a separate or channelised right-turn lane.

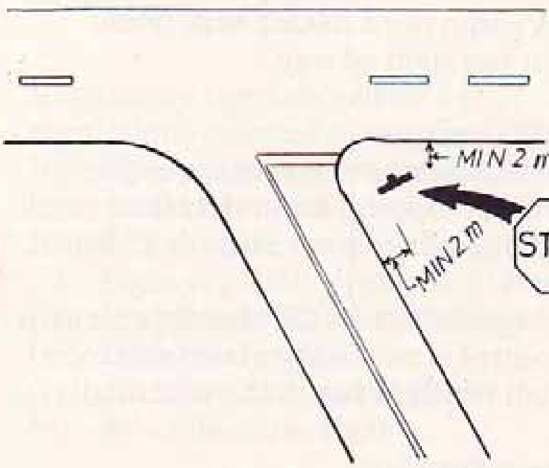
For a YIELD sign to be effective, full visibility will be needed to the right and left between a point 1 m above road level over an area defined by distances x an y as shown in Fig IV–10.

3.5 NO ENTRY SIGNS

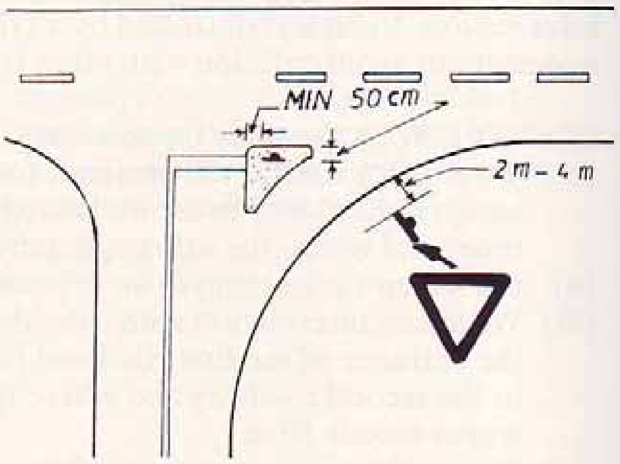
To prohibit traffic from entering a restricted road section, the NO ENTRY sign should be placed in the most appropriate position at the end of a one-way roadway or ramp. The sign should normally be mounted on the right-hand side of the roadway facing traffic that would enter the roadway or ramp in the wrong direction. However, a second sign at the left-hand side may be justified where a vehicle is approaching for a turn.

3 REGULATORY SIGNS

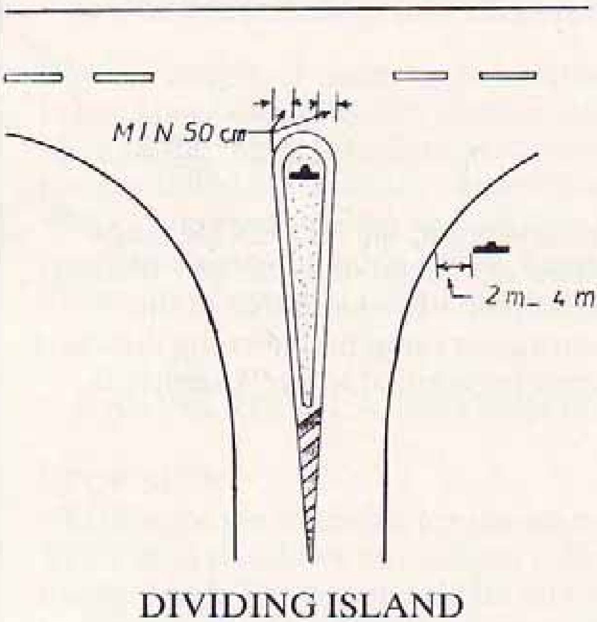
FIGURE IV–9 Typical Location for Stop Signs and Give Way Signs



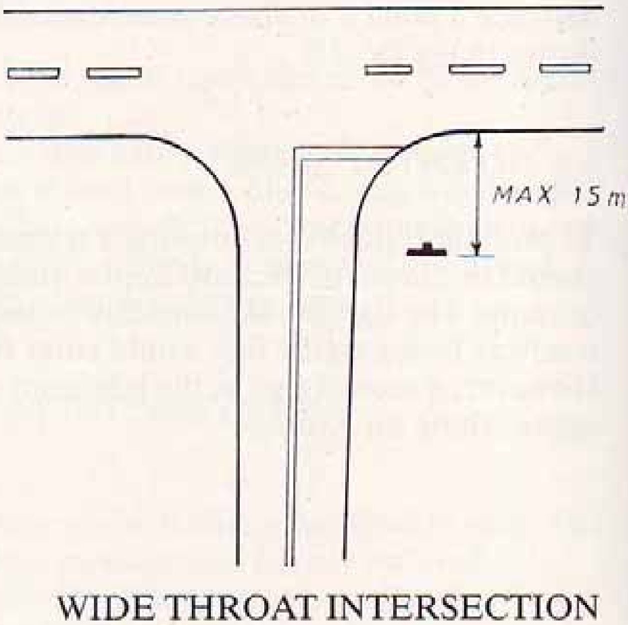
ACUTE ANGLE INTERSECTION



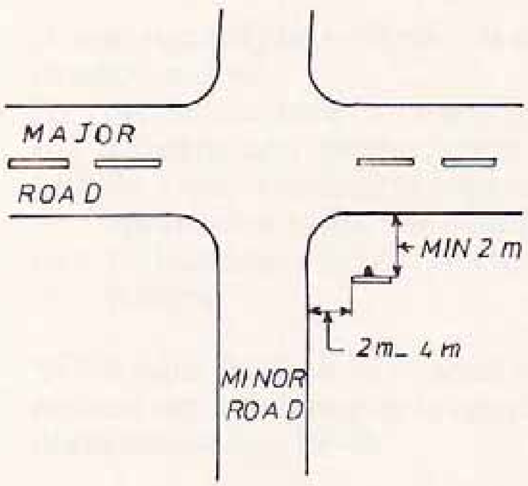
CHANNELISED INTERSECTION



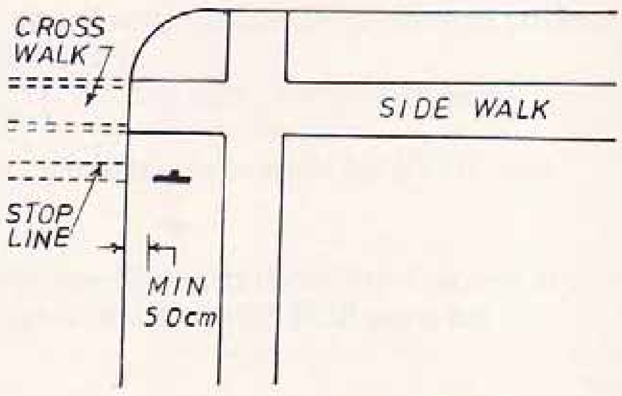
DIVIDING ISLAND



WIDE THROAT INTERSECTION



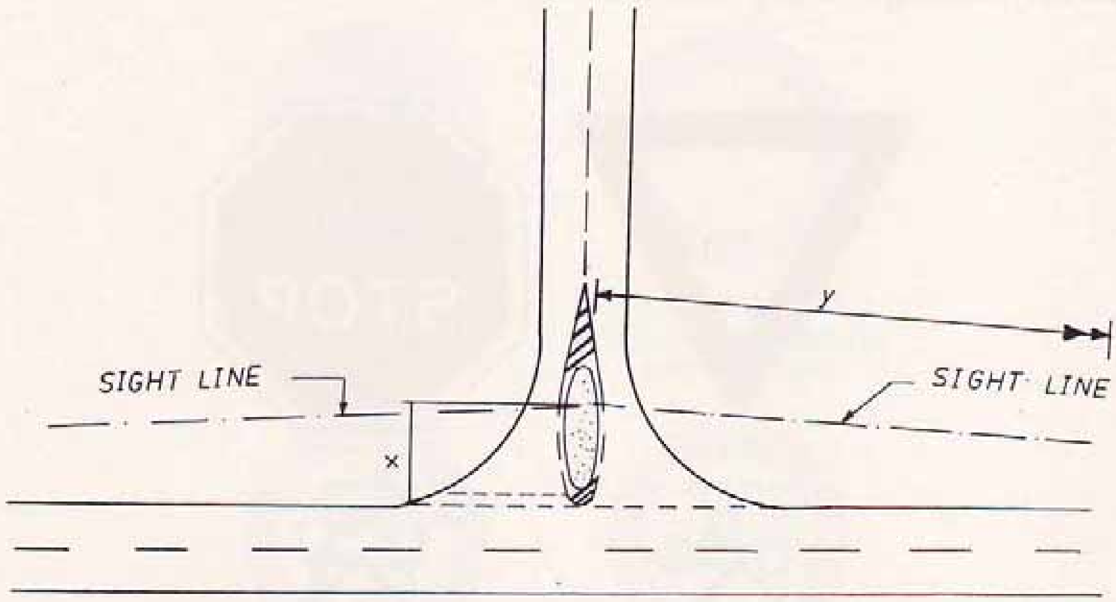
MINOR CROSS ROAD



URBAN INTERSECTION

3 REGULATORY SIGNS

FIGURE IV–10 Visibility Display



Visibility Distances at Priority Junctions TABLE IV–5

type of major road	speed limit km/h	minimum visibility distance (y) m
primary road	80 60	150 m 120 m
secondary road	50	90 m
access road	50	60 m
The x distance should normally be 10 m. For light trafficked roads, and roads in urban areas the x distance may be reduced to 5 m		

3.6 SPEED LIMIT SIGNS

The SPEED LIMIT signs should display the limit established by law or by regulation after an engineering and traffic investigation has been made in accordance with established traffic engineering practices. The speed limit shown should be in multiples of 5 km per hour.

In order to determine the proper numerical value for a speed zone on the basis of an engineering and traffic investigation, the following factors should be considered:

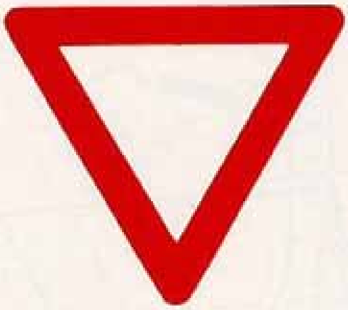
(i) Road surface characteristics, shoulder condition, gradient, alignment and sight distance.
(ii) the 85 percentile speed.
(iii) Roadside development.
(iv) Safe speed for curves or hazardous locations within the zone.
(v) Parking practices and pedestrian activities.
(vi) Reported accident experience for a 12 month period.

Figs IV–11(A) to IV–11(K) show the most commonly used regulatory signs.

3 REGULATORY SIGNS

FIGURE IV–11

Signs Regulating Priority at Intersections



B–1
Give way

B–2
Stop at intersection

B–3
Priority road

B–4
End of priority

FIGURE IV–11A

Signs Regulating Priority on Narrow Sections of Road

B–5
Priority for oncoming traffic

B–6
Priority over oncoming traffic

3 REGULATORY SIGNS

FIGURE IV–11B

C–1a
No entry

C–2
Closed to all vehicles in both directions

C–3a
No entry for all motor vehicles except motor-cycles without sidecar

C–3b
No entry for motor-cycles

C–3c
No entry for cycles

C–3e
No entry for goods vehicles

C–3f
No entry for any vehicles drawing a trailer other than a semi-trailer or single axle trailer

C–3g
No entry for pedestrians

3 REGULATORY SIGNS

FIGURE IV–11C

C-3h
No entry for animal-drawn vehicles

C–3j
No entry for handcarts

C–3k
No entry for power-driven agricultural vehicles

C–3m
No entry for vehicles carrying more than a certain quantity of explosives or readily inflamable Substances

C–3n
No entry for vehicles carrying more than a certain quantity of substances liable to cause water pollution

C–4a
No entry for power-driven vehicles

C–4b
No entry for power-driven vehicles or animal-drawn vehicles

C–5
No entry for vehicles having an over all width exceeding. Leave some space as follows as in C–6...metres

3 REGULATORY SIGNS

FIGURE IV–11D

C–6
No entry for vehicles having an overall height exceeding....metres

C–7
No entry for vehicles exceeding ....tons laden weight

C–8
No entry for vehicles having a weight exceeding ....tons on one axle

C–9
No entry for vehicles or combinations of vehicles exceeding ....metres in length

C–10
Driving of vehicle less than ....meters apart prohibited

C–11a
No left turn

C–11b
No right turn

C–12
No U-turn

3 REGULATORY SIGNS

FIGURE IV–11E

C–13aa
Overtaking prohibited

C–13ba
Overtaking by goods vehicles prohibited

C–14
Maximum speed limited to the figure indicated

C–15
Use of audible warning devices prohibited

C–15a
Photography prohibited

C–16
Passing without stopping prohibited (e.g. for customs)

C–17a
End of all local prohibitions imposed on moving vehicles

C–17b
End of speed limit

C–17c
End of prohibition of overtaking

3 REGULATORY SIGNS

FIGURE IV–11F

Standing and Parking Signs

C–18
Parking prohibited

C–19
Standing and parking prohibited

C–20a
Alternative parking (prohibited on odd number dates)

C–21b
Alternative parking (prohibited on even number dates)

3 REGULATORY SIGNS

FIGURE IV–11G

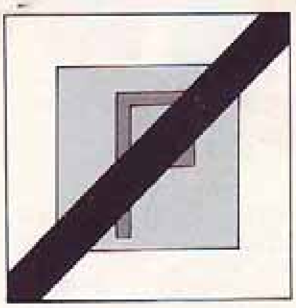
Standing and Parking Signs

C–21
Limited duration parking zone

C–22
Limited duration parking zone

E–23
Parking

E–24
Exit from a limited duration parking zone



E–24
Exit from a limited duration parking zone

3 REGULATORY SIGNS

FIGURE IV–11H

Mandatory Signs

D–1a
Direction to be followed

D–1a
Direction to be followed

D–1a
Direction to be followed

D–1a
Direction to be followed

D–2
Pass this side

D–3
Compulsory roundabout

3 REGULATORY SIGNS

FIGURE IV–11K

Mandatory Signs

D–4
Compulsory cycle track

D–5
Compulsory footpath

D–6
Compulsory track for riders on horseback

D–7
Compulsory minimum speed

D–8
End of compulsory minimum speed

D–9
Snow chains compulsory

E–11a
Pedestrian crossing

4 GUIDE SIGNS

GUIDE SIGNS can be classified in two categories
(i) DIRECTION signs
(ii) INFORMATION signs.
Direction signs are used to: –
(a) Inform the motorist at frequent intervals about the number of the route on which he is travelling.
(b) Inform the motorist about the name (identification or route number) of important crossing streets and highways.
(c) Inform the motorist about cities and towns along the route and also about important towns immediately adjacent to the route.
(d) Identify nearby rivers, streams, lakes, parks, institutions and historical sites.

4.1 DIRECTION SIGNS

Signs for streets and highways should have a green background with a white message or symbol. DIRECTION signs for expressways should have a blue background with a white symbol or letters. The shape of the sign will be rectangular but the size varies (depending on height of letters, no. of words and no. of lines).
If there is more than one destination to be shown on a direction sign, the following order should be maintained (See Sign No. E–5c Fig IV–14B)
(i) Straight ahead destination.
(ii) Left turn destination.
(iii) Right turn destination.

4.2 DESIGN OF LETTER HEIGHT FOR DIRECTION SIGNS

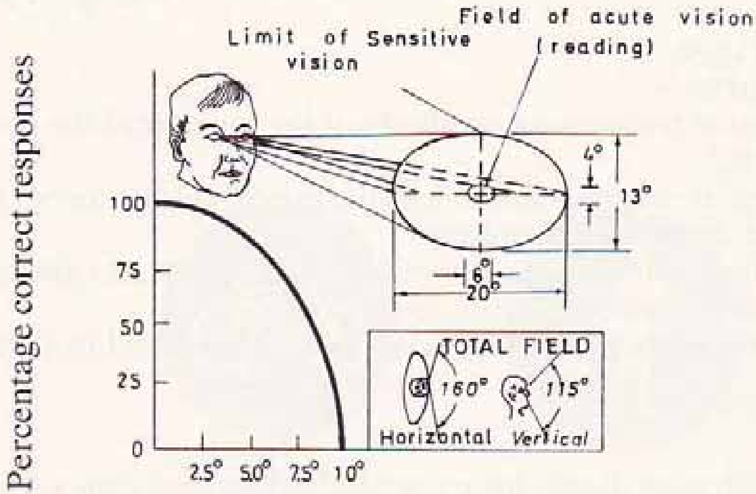
VISUAL FIELD
Because of the variation in types of cell in the retina, the recognition of detail within the field of view is reduced, both in horizontal and vertical plane, for zonal areas set further away from the focal point (See Fig IV–12). While the total field of view for the two eyes with normal sight is approximately 160° horizontally and 115° vertically, anything outside a 20° cone is indistinct. As speed increases, the peripheral field of view tends to shrink and the focal distance increases linearly from about 110° and 150 m at 20 km/h to 40° and 500 m at 90 km/h. Less time can be afforded on higher speed roads and in complex situations for the continual shifting of the eyes to and from the traffic stream.
To overcome this, the letter height must be increased or the sign mounted overhead for high-speed roads.

4.3 SITING OF SIGNS RELATED TO LETTER HEIGHT

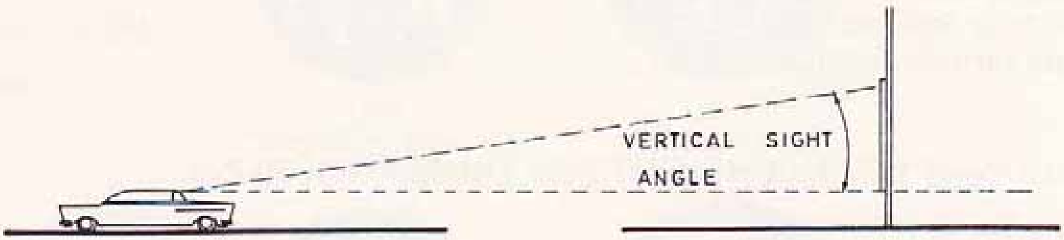
There are three time factors which have to be taken into account when considering the siting of signs:
(a) Reading time (tg) of the message or symbol.
(b) Reaction period (tr) required before acting on the information presented.
(c) In most cases an appropriate action (ta) is called for which requires further time e.g. decelerating to a stop line, making a turn etc. All times are measured in seconds.

4 GUIDE SIGNS

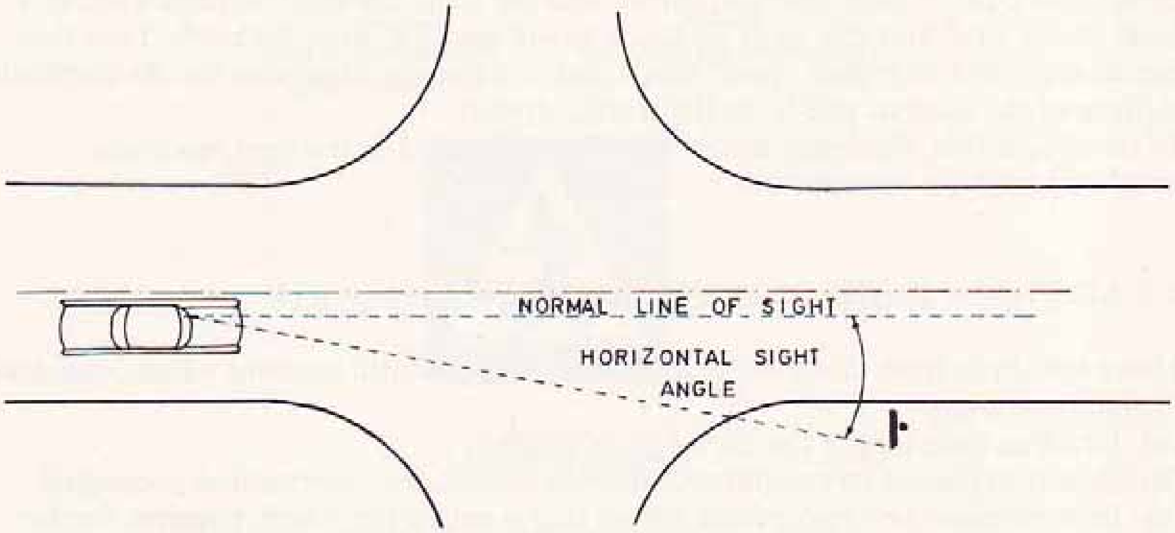
FIGURE IV–12 Cone of Vision



Indirect reading at angular separation
VISUAL FIELD



VERTICAL SIGHT ANGLE

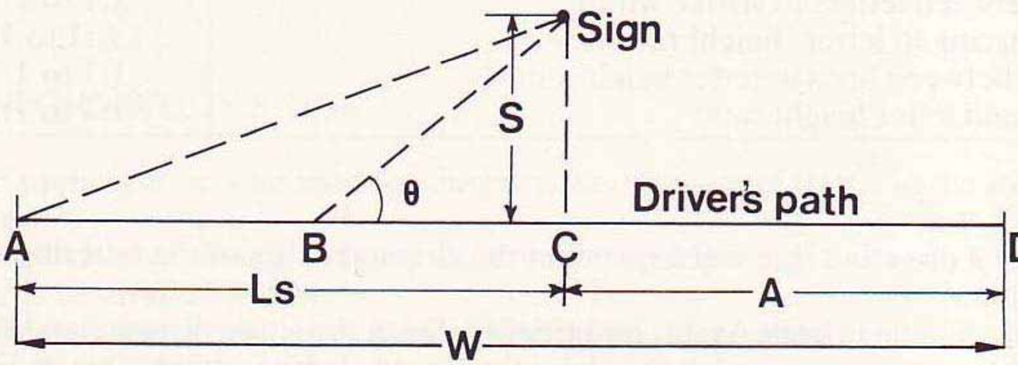


HORIZONTAL SIGHT ANGLE

4 GUIDE SIGNS

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 (tg) can be taken as a minimum of 1 sec for signs with one or two words and N/3 sec for signs with N unfamiliar words. A suitable reaction time (tr) 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 IV–13 Siting of Signs



The distance $A+S\cot\theta$ is equivalent to the minimum safe stopping distance. Selecting suitable values for the parameters enables the siting distances to be solved as shown in Fig IV–13

$$W = V_1(2tg + tr) + ta\frac{(V_1+V_2)}{2}$$

Where V_1 and V_2 are the initial and final velocities (m/sec). Uniform deceleration is assumed.

$$\text{But } Ls = AB + BC = V_1(2tg) + S\cot\theta$$

The larger value of Ls or W–A is used for the determination of letter height. For a displacement of $\theta = 10°$ the required letter size is

$$^{*}\quad H = \frac{2tgV + 5.7\,S}{L}\ \text{cm}$$

Where H = required letter height (cm)

L = legibility in metres 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 4 m per cm of letter height.

*A factor of safety has been introduced by allowing two glances.

4.4 LEGIBILITY DISTANCE

Legibility distance depends on the following factors:

(a) Size of letter-height and width.
(b) Stroke width.
(c) Letter and word spacing both horizontally and vertically.
(d) Size of margins.

4 GUIDE SIGNS

The relationship between these parameters is shown in Table IV–6

Parameter Ratio for Legibility Distance for Latin Characters

TABLE IV–6

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 IV–6.
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 IV–7 gives the suggested letter height for direction signs for different categories of road.

Recommended Letter Heights

TABLE IV–7

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)	25	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.

4 GUIDE SIGNS

Suggested Distances for Siting Direction Signs (m)

TABLE IV–8

Type of sign	Approach speed of vehicle on the road (km/h)			
	≤60	60–80	80–120	>120
Direction sign	30–50 m	50–80 m	80–120 m	120–150 m
Advance direction sign	250 m	250–300 m	300–400 m	400–500 m

Figs. IV–14A to IV–14G show some typical guide signs.

4.5 INFORMATION SIGNS

Information signs may be used for the purpose of directing traffic to the following facilities:

a) Off-road services such as fuel, restaurants, parking areas, lodgings, tent and trailer camp sites or motels.
b) Points of interest such as historical, geological or geographic sites.
c) Emergency services such as hospitals and telephones.

Warrants for Information Signs

a) The distance from the highway to the facility should not exceed 5 km.
b) Adequate standard of service should be provided at the facility. For example the service station should be open 24 hours every day.
c) A point of interest should be one that has been recognised by an appropriate body such as the Ministry of Culture, Ministry of Tourism, etc.

Information signs 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 arrow directed to the location of the facility.

The symbol for a hospital and first aid sign should be red in colour.

4 GUIDE SIGNS

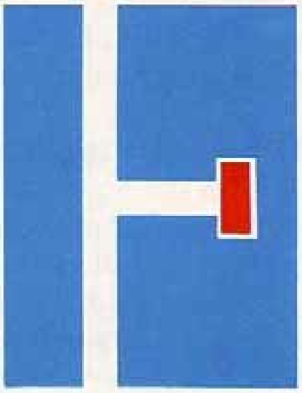
FIGURE IV–14A

Advance Direction Signs

E–1a
Advance direction sign

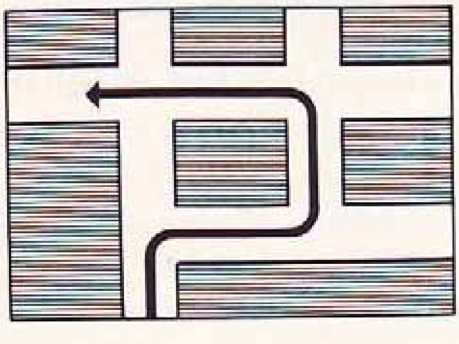
E–1b
Advance direction sign

E–1c
Advance direction sign



E–2a
No through road

E–2b
No through road



E–3
Route to be followed in order to turn left where a left turn at the next intersection is prohibited

4 GUIDE SIGNS

FIGURE IV–14B

Direction Signs

E–4
Preselection at intersection
on multi-lane roads

E–5a
Direction sign

E–5b
Direction sign

E–5c
Direction sign

E–6a
Direction to an airfield

E–6b
Direction to an airfield

E–7
Direction and/or distance to
a camping site

E–8
Direction and/or distance to a
youth hostel

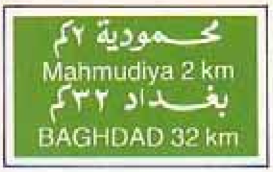
4 GUIDE SIGNS

FIGURE IV–14C

Direction Signs

E–9a
Beginning of a built up area

E–9c
End of a built up area



E–10
Confirmation sign

4 GUIDE SIGNS

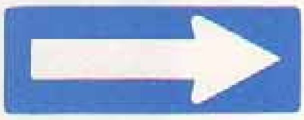
FIGURE IV–14D

Information Signs

E–12a
Hospital

E–12b
Hospital

E–13a
One-way road



E–13b
One-way road

E–14
No through road

E–15
Expressway
Freeway

4 GUIDE SIGNS

FIGURE IV–14E

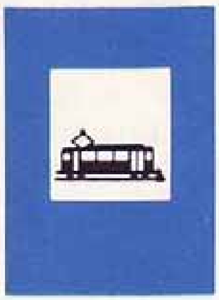
Information Signs

E–16
End of Expressway
Freeway

E–17
Road for motor vehicles

E–18
End of road for motor vehicles

E–19
Bus stop



E–20
Tramway stop

E–21
Mountain road open or closed
with removable panel (1, 2, 3)

4 GUIDE SIGNS

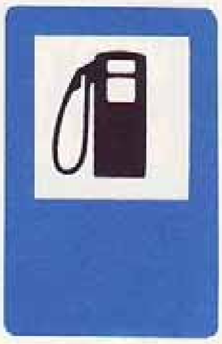
FIGURE IV–14F

Information Signs

F–1a
First-aid station

F–2
Breakdown service

F–3
Telephone



F–4
Filling station

F–5
Hotel or
Motel

F–6
Restaurant

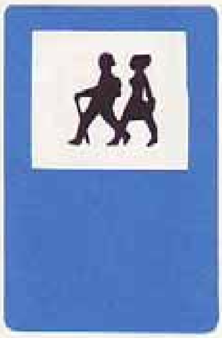
4 GUIDE SIGNS

FIGURE IV–14G

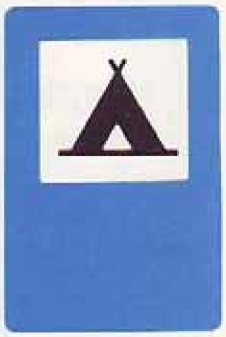
Information Signs

F–7
Refreshments or cafeteria

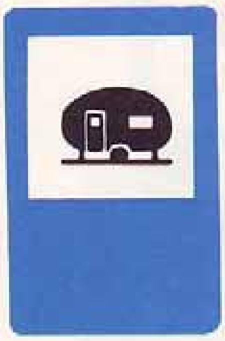
F–8
Picnic site



F–9
Starting point for walks

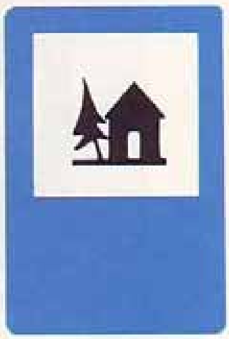


F–10
Camping site



F–11
Caravan site

F–12
Camping and caravan site



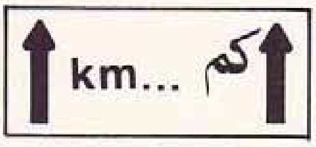
F–13
Youth Hostel

4 GUIDE SIGNS

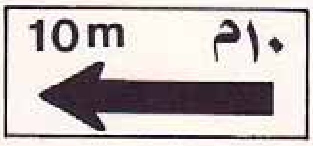
4.6 Additional Panels

FIGURE IV–14 H   Additional Panels

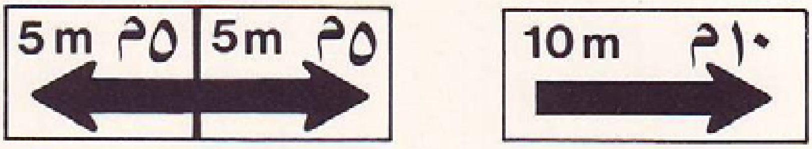
Distance from sign to the beginning of the dangerous section of road or of the zone to which the regulation applies.



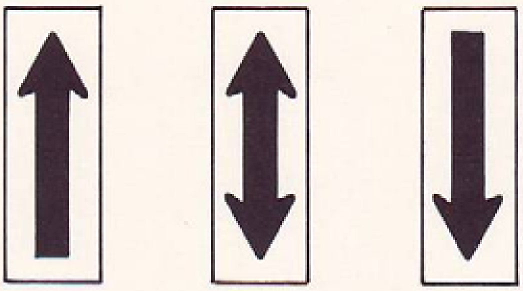
Length of the dangerous section of road or zone to which the regulation applies.



Distance covered by prohibitions or restrictions.



Distance covered by prohibitions or restrictions



Prohibitions or restrictions

begin   continue   end

PART V

ROAD MARKING

PART V ROAD MARKING

Contents

PART V ROAD MARKING

LIST OF FIGURES

PART V ROAD MARKING

LIST OF TABLES

1 MARKING CHARACTERISTICS

1.1 FUNCTIONS OF MARKINGS

Road markings may be defined as markings on the road surface which serve to control, warn, guide or inform road users. All markings have definite functions within a proper scheme of traffic control. In some cases they are used to supplement the traffic control, guidance or warning functions of other devices such as traffic signs or signals. In other instances they obtain results on their own merit, which cannot be achieved using any other device. In such cases, they serve as a very effective means of conveying certain regulatory messages and warnings that otherwise could not be made clearly understandable.

1.2 LEGAL AUTHORITY

Carriageway and curb markings lie exclusively within the boundaries of public highways and should never be applied except by a public authority. Hazard markers/markings are also normally located within the highway boundary and will be subject to the same jurisdictional regulations.

1.3 STANDARDISATION

Markings should be uniform in design, position and application. As in the case of all other traffic control devices, it is imperative that markings be uniform so that they are recognized and understood immediately.

1.4 MATERIAL AND COLOUR

Road markings should be of non-skid materials and should not protude more than 6 mm above the level of carriageway. Studs or similiar devices used for marking should not protrude more than 1.5 cm above the level of the carriageway (or 2.5 cm in the case of studs incorporating reflectors). All pavement markings should be white in colour. Improved night visibility is obtained by the use of reflectorised paint.

Hazard markers may consist of single reflectors, clusters of reflectors or small panels of uniform shape covered with a reflective coating and mounted on separate posts.

1 MARKING CHARACTERISTICS

1.5 TYPES OF MARKING

(1) Pavement Markings

(a) Centre lines
(b) Lane lines
(c) Edge of carriageway lines
(d) Prohibitory lines/No overtaking lines
(e) Safety lines
(f) Stop lines
(g) Give way lines
(h) Channelising and central median island markings
(i) Arrow markings
(k) Pedestrian crossings
(l) Approaches to railway crossings

(2) Hazard Markers

(a) Road edge marker posts
(b) Marking of objects within the roadway
(c) Marking of objects adjacent to the roadway

2 ROAD MARKING

2.01 CENTRE LINES

Centre 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 centre line may be located away from the geometric centre of the road to follow the vehicle path.
Centre line markings are needed where carriageways greater than 6.0 m in width carry an average annual traffic volume exceeding the equivalent of 2000 veh/day or, for carriageways of 5.5 m width, carrying 1000 veh/day.
The centre line should be a continuous solid line for a maximum of 30 m before a junction, a marked pedestrian crossing or prohibitory lines.
The standard centre line markings are shown in Fig. V–1.

2.02 LANE LINES

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 2.75 m. Lane lines should be used irrespective of traffic density.
The standard types of lane lines are shown in Fig V–2.

2.03 EDGE LINES

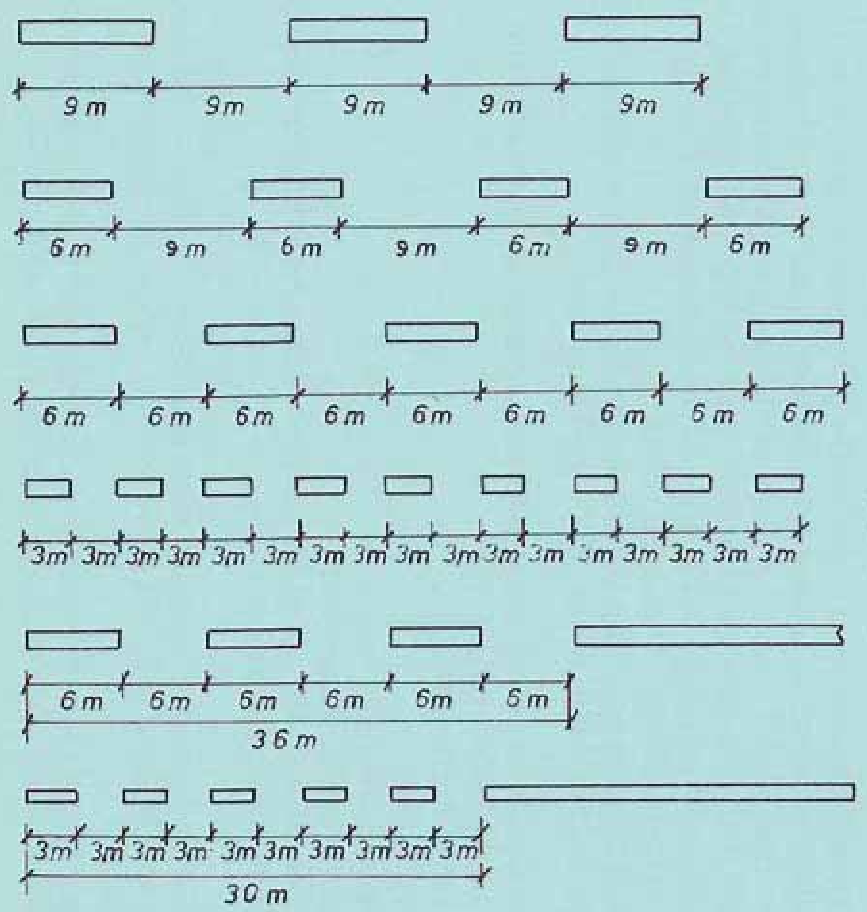
Edge of carriageway lines are used to define the pavement edge or, in instances where the shoulder is paved, to separate the shoulder from the carriageway. They form one of the most useful types of pavement markings to guarantee traffic safety. On narrow roads with two-way traffic, where a centre line is not feasible, they enable the vehicles to keep close to the edge of the carriageway, thereby reducing the possibility of collision with oncoming vehicles.
Although edge of carriageway lines should preferably be used for all highways, there are definite requirements at the following locations:

(a) Where the shoulder is paved and is similar in texture and colour to the pavement on the main carriageway.
(b) Before and over narrow bridges.
(c) Before sharp curves.
(d) On merging and diverging lanes.
(e) On carriageway width transitions.
(f) Where obstructions on the shoulder are close enough to the carriageway to constitute a hazard to the motorist.

At junctions and minor road entries, the edge of carriageway lines to be used are similar to those for majors roads, but they should comprise broken lines at bifurcation points.
Edge of carriageway lines are generally broader than other lines to avoid any confusion. Standard edge of carriageway lines are shown in Fig. V–3.

2 ROAD MARKING

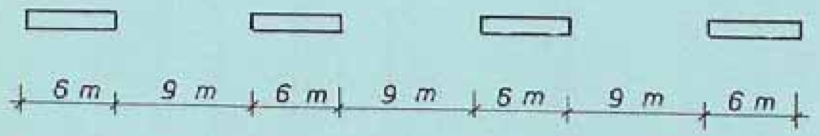
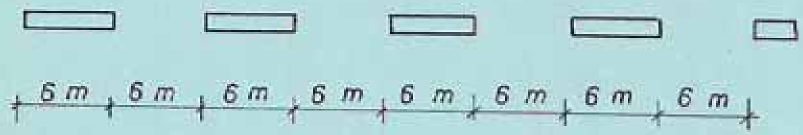
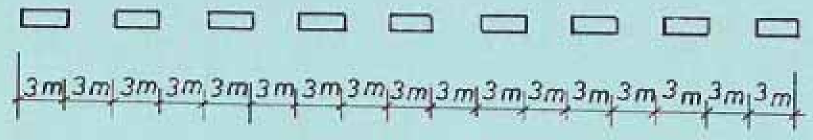
FIGURE V–1 Centre Lines



SPACING	WIDTH	USAGE
(row 1)	15 cm	ON FREEWAY
(row 2)	12 cm	ON CARRIAGEWAY MORE THAN 7m WIDE
(row 3)	12 cm	ON NARROW OR CURVY CARRIAGEWAY
(row 4)	10 cm or 12 cm	PASSING THROUGH A POPULATED AREA
(row 5)	10 cm or 12 cm	BEFORE A JUNCTION OR A PEDESTRIAN CROSSING THE LINE SHOULD BE CONTINUOUS
(row 6)	10 cm or 12 cm	

2 ROAD MARKING

FIGURE V–2 Lane Lines

SPACING	WIDTH	USAGE
	15 cm	MOTORWAY SECTIONS
	12 cm	DUAL-CARRIAGEWAY SECTIONS
	12 cm	ON SINGLE CARRIAGEWAY ROADS
	12 cm	ON PASSING THROUGH POPULATED AREAS, AT ROAD JUNCTIONS AND IN ACCELERATION AND DECELERATION LANES.

2 ROAD MARKING

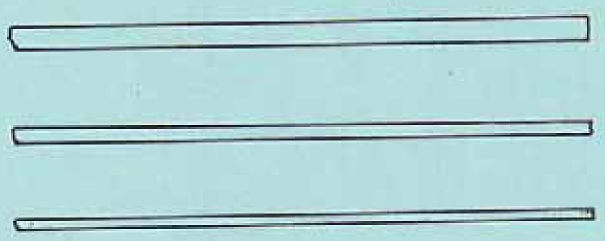
FIGURE V–3 Edge of Carriageway Lines

WIDTH	USAGE
25 cm / 50 cm	ON MOTORWAYS
20 cm / 50 cm	ON HIGHWAYS

2.04 PROHIBITORY LINES

(1) No Overtaking Lines
Prohibitory lines or no overtaking lines should be applied where driving onto the opposite lane or changing to an adjacent lane is dangerous or will impede traffic and is therefore officially prohibited. Consequently, prohibitory lines are chiefly used at places where there is a change of carriageway width, ahead of obstructions on the carriageway, at hills, bends and at road junctions.
Prohibitory lines should comprise narrow continuous lines, which must not be crossed by the wheels of a vehicle.
A prohibitory line located in the middle of a 4-lane two-way carriageway should be indicated by a double line comprising two narrow solid lines, so that it can be recognised more easily from the outer lanes.
A prohibitory line comprising one continuous and one broken line is located on those stretches of the road where vehicles are permitted to cross the prohibitory line from the broken line side of the carriageway but not from the solid line side.
Prohibitory line visibility may be improved by installing reflective studs at 4 m intervals along the continuous line. Standard prohibitory lines are shown in Fig V–4.

FIGURE V–4 Prohibitory Lines/No Overtaking Lines



WIDTH	USAGE
15 cm	ON MOTORWAYS
12 cm	ON HIGHWAYS
10–12 cm	IN CITIES

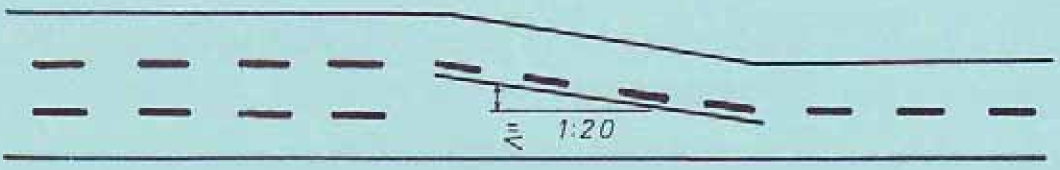
2 ROAD MARKING

(2) Prohibitory Lines at Changes in Carriageway Width

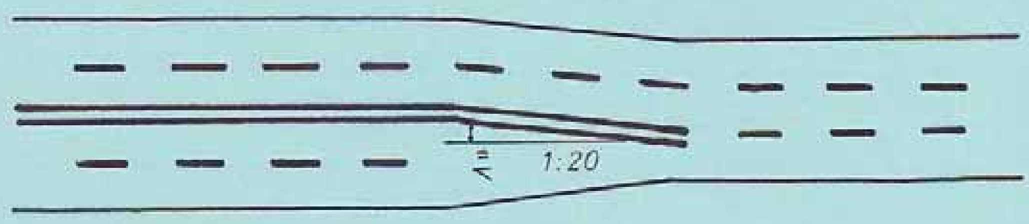
A change in carriageway width from 2 to 3/4 lanes and from 3 to 4 lanes is dangerous. Proper traffic guidance is very important and can be accomplished with the aid of prohibitory lines. The angle of the prohibitory line should not exceed a ratio of 1:20.
Changes of traffic lane width should be marked with the help of a prohibitory line on the edge of the road. On open road sections, such prohibitory lines should begin 50 m ahead of any narrowing and in populated areas 20 m. Standard prohibitory lines or no overtaking lines at changes in road width are shown in Fig V–5.

FIGURE V–5 No Overtaking Lines at Change in Road Width

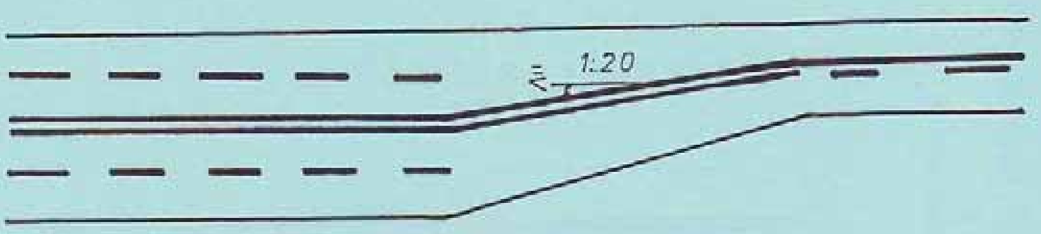
CHANGES IN NUMBER OF LANES



From 3 lanes to 2 lanes

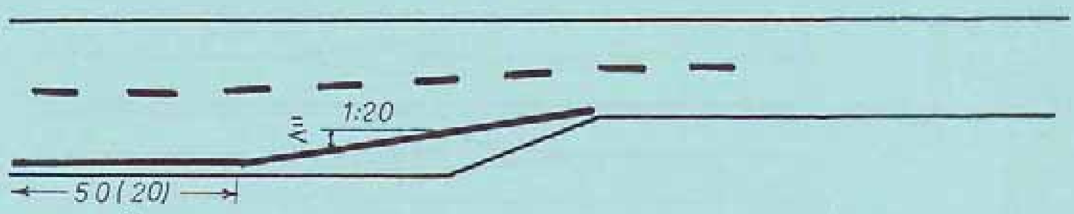


From 4 lanes to 3 lanes



From 4 lanes to 2 lanes

CHANGES IN WIDTH OF LANES



*) 50 m for open areas
20 m for populated areas

2 ROAD MARKING

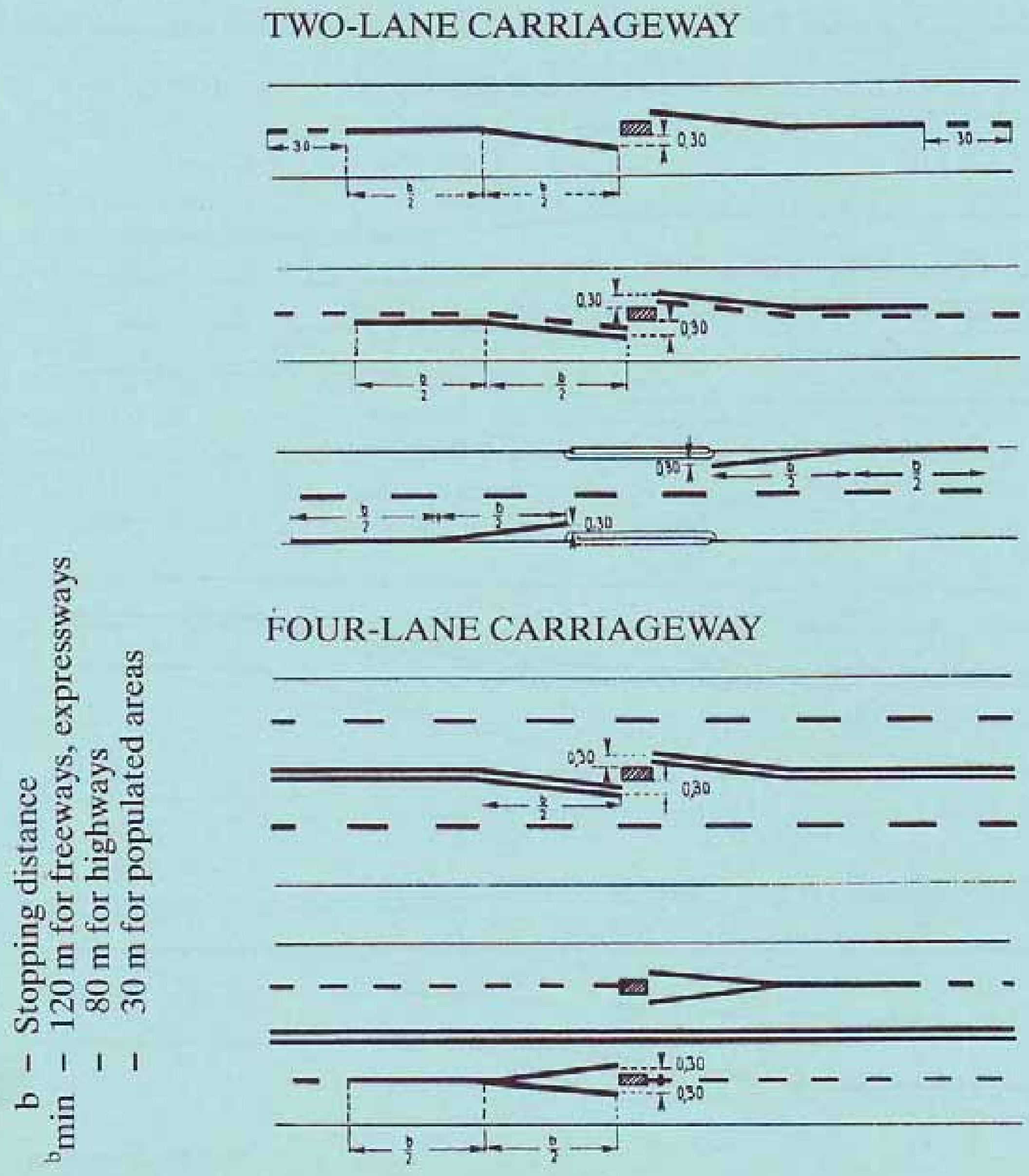
(3) Prohibitory Lines ahead of Obstructions on the Carriageway

Obstructions within the carriageway are dangerous and should be avoided wherever possible. If removal of the obstruction is not possible, carriageway markings should be provided to prevent the traffic from touching or colliding with such obstructions. For this purpose, prohibitory lines are provided. These should begin at a distance ahead of the obstruction equal to the length of the stopping distance. The stopping distance should be calculated according to the speed limit. The length of prohibitory lines generally should not be less than

120 m for motorways
80 m for highways
30 m for populated areas.

Standard prohibitory lines or no overtaking lines for use in advance of obstructions on the carriageway are shown in Fig V–6.

FIGURE V–6 No Passing Lines ahead of Obstructions on the Carriageway



2 ROAD MARKING

(4) Prohibitory Lines at Hills

Prohibitory lines are required at hills on 2 and 3-lane roads if the sight distance is insufficient over a crest curve. In this case, the use of the opposite carriageway must be restricted.
On a vertical curve, the sight distance is the distance at which an object on the carriageway 1.2 m in height (vehicle A) can just be seen from another point located ahead of 1.2 m (eye of driver of vehicle A). The sight distance is considered to be dangerously reduced if it is below the sum of the stopping distances of two vehicles aproaching each other on one lane. Prohibitory lines are needed if the sight distance is less than the minimum sight distance (minimum sight distance is equal to twice the stopping distance at the design speed).

Referring to Fig V–7, the prohibitory line should start from point A (in the direction of travel, prevailing sight distance being less than minimum sight distance) to point B, where the prevailing sight distance (from opposite direction) again reaches minimum. The same applies to the opposite direction from point B' to point A'.
Where zones of insufficient sight distances overlap, the prohibitory line should be a single, solid line with no overtaking effect in both directions. The prohibitory line should be set out according to Fig V–7 even if a section of road is not provided with a centre line or lane lines on its full length.
The length of prohibitory lines should not be less than 50 m if the requirement is between 20 and 50 m, the line should be increased to make it 50 m long. If it is under 20 m, no line shall be applied.
The distance between prohibitory lines in the same direction must never be less than 150 m. If it is less, the lines must be joined.
The relationship between approach speed and sight distance is shown in Table V–1.

Relationship between Approach Speed and Sight Distance

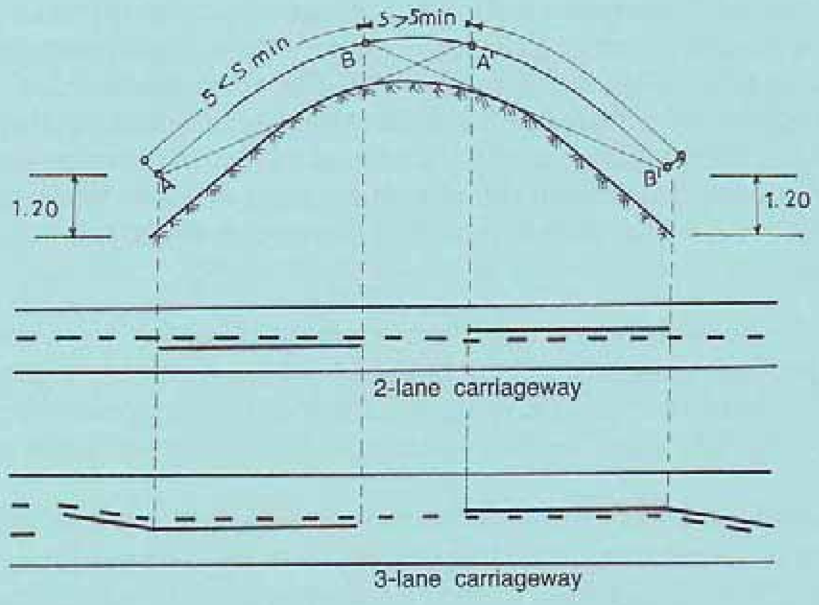
TABLE V–1

Approach speed (km/h)	Sight distance (m)
100	320
80	260
65	180
50	120

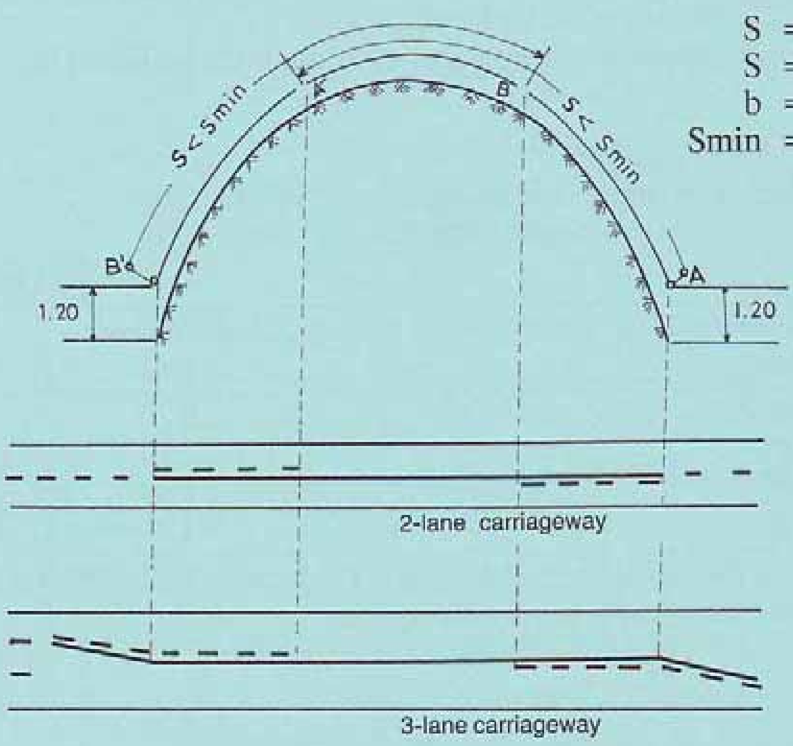
For speeds not given in this table the value of sight distance can be interpolated.
Speed used in the calculation is 85 percentile speed.

2 ROAD MARKING

FIGURE V−7 No Overtaking Line on Hills



CASE I: The areas of insufficient sight distance are separately located



S = Sight distance
S = 2b
b = stopping distance
Smin = minimum sight distance

CASE 2: The areas of insufficient sight distance are overlapped

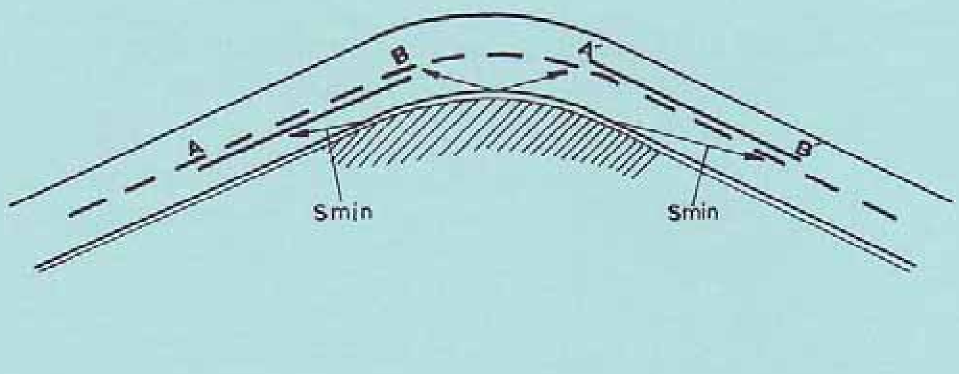
2 ROAD MARKING

(5) Prohibitory Lines at Bends

Prohibitory lines shall be applied at bends on 2 and 3-lane roads if the sight distance is dangerously reduced due to sight impediments on the inside of the bend and the use of the opposite carriageway must thus be limited.
The length and design of prohibitory lines at bends is determined according to the same criteria as hills. Minimum sight distances according to Table V–1 should be used for determining the length of the prohibitory line. The line should be set out according to Fig V–8.

For bends where the radius is less than 50 m the prohibitory line should not be placed at the geometrical centre but set out according to Fig. V–8.

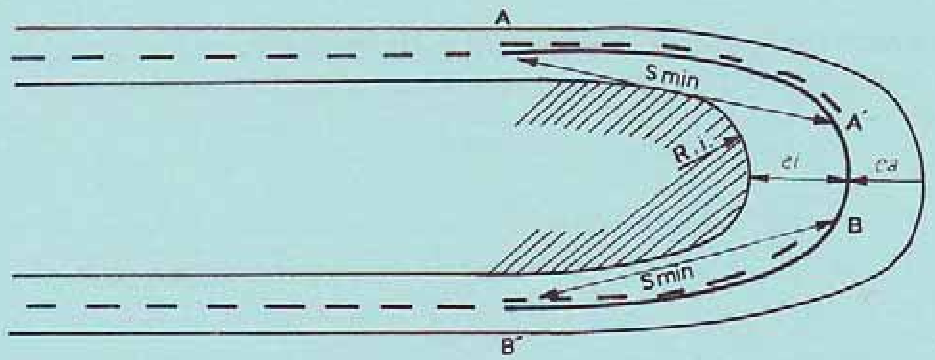
FIGURE V–8a No Overtaking Lines at Bends



CASE 1: Separately located areas of insufficient sight distance for both driving directions

2 ROAD MARKING

FIGURE V–8b No Overtaking Lines at Bends



Ri in m	10–15	15–20	20–30	30–50	over 50
$\frac{ei}{ea}$	1.4	1.3	1.2	1.1	1.0

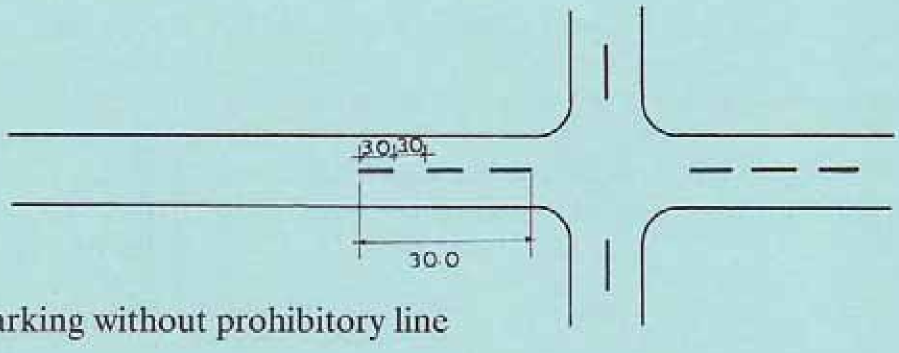
CASE 2: Overlapping areas of insufficient sight distance for both driving directions

(6) Prohibitory Lines at Road Junctions

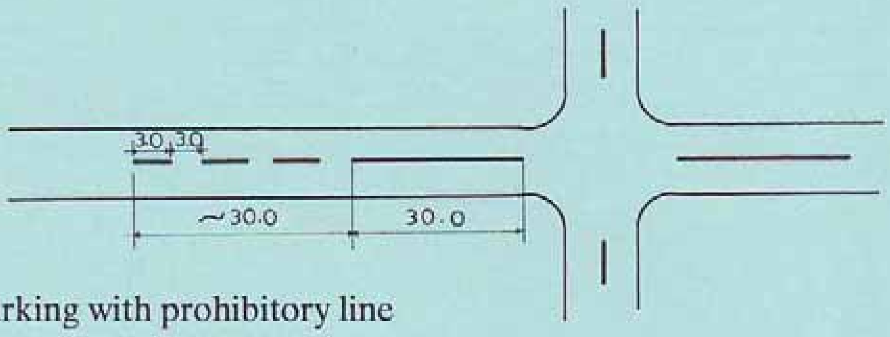
To assist traffic movements at road junctions, lines are needed to direct traffic into assigned lanes.
At simple road junctions and bifurcations (see Fig V–9) with single-lane access, lines of 30 m length should be laid to divide opposite traffic streams at the junction. If an access (see Fig V–10) has two or more lanes, lane lines should be laid for 30 m at the approach to the junction which will divide the traffic according to branching directions. The length of line may be increased depending on the number of vehicles queueing.

2 ROAD MARKING

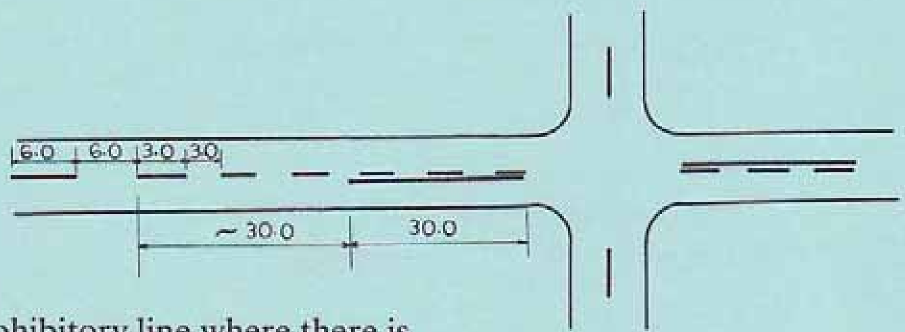
FIGURE V–9 No Overtaking Lines at Road Junctions with One Lane Access



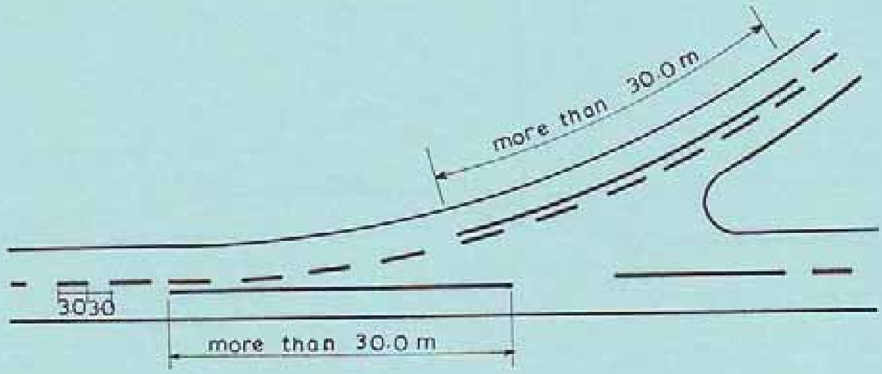
a. Marking without prohibitory line



b. Marking with prohibitory line



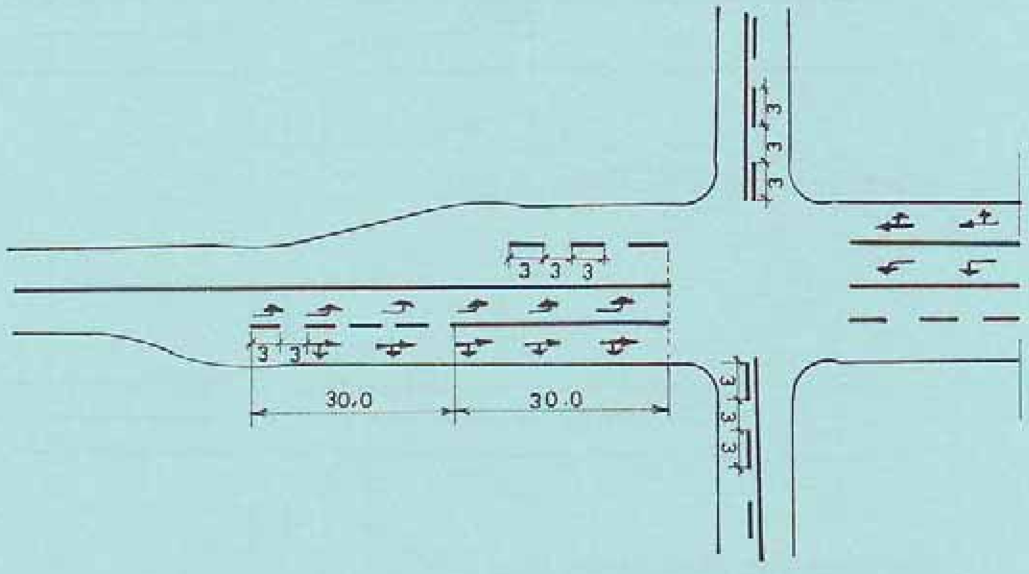
c. Prohibitory line where there is centre line near the junction



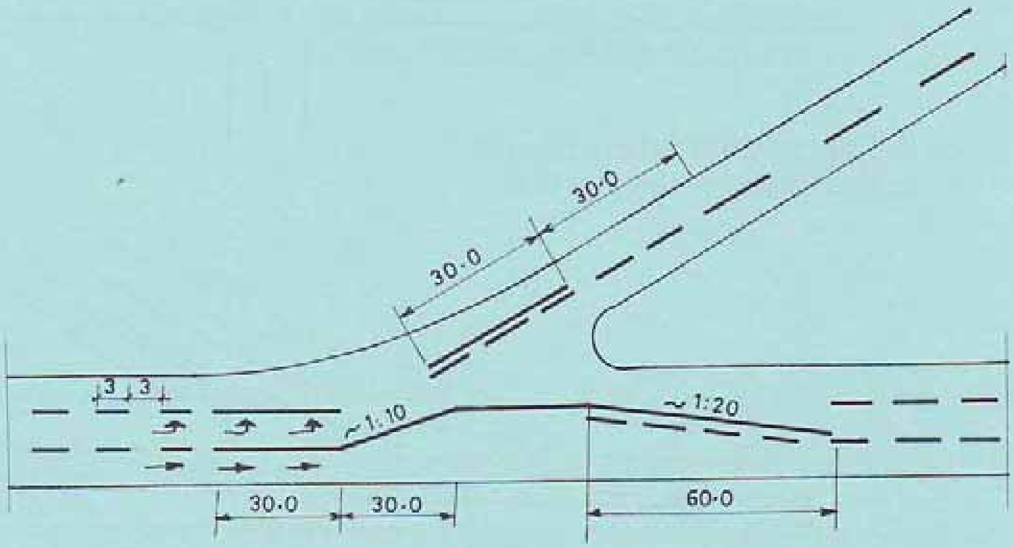
Marking when acute angled corner traffic is prohibited

2 ROAD MARKING

FIGURE V–10 No Overtaking Lines at Road Junctions with Two Lane Access



a. Emergence of one road into another



b. No overtaking line when acute angled traffic is not allowed

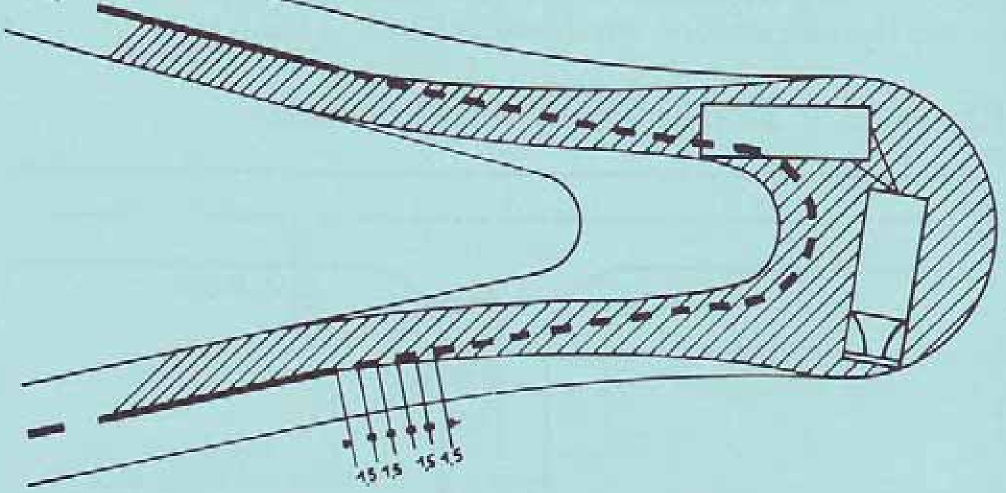
2 ROAD MARKING

2.05 SAFETY LINES

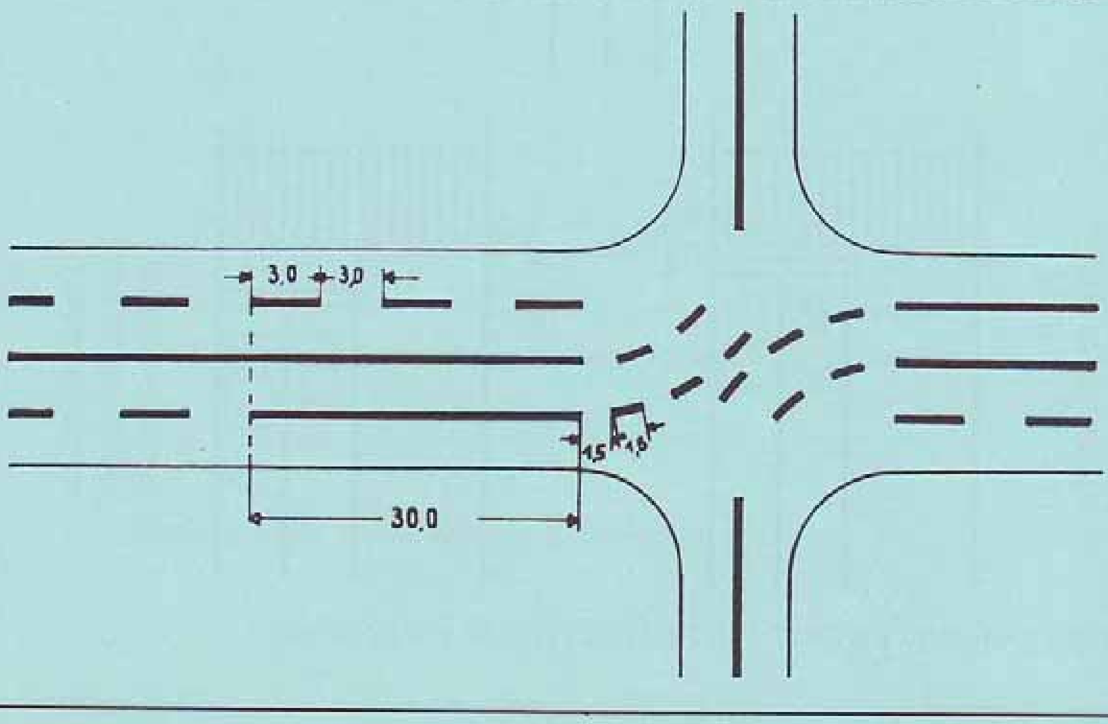
Safety lines are applicable where lane-changing is dangerous and hinders the traffic flow, but where it cannot be officially prohibited by means of prohibitory lines.
This is the case if long vehicles, due to their construction, cannot keep to the prescribed limit of a traffic lane (for instance at bends) or if traffic lanes that are reserved for specific traffic movements must be passed or crossed by vehicles from other directions, so that a prohibitory line would not be suitable.
Safety lines should be laid as narrow broken lines at a ratio of 1.5 m line and 1.5 m gap. The length of safety lines at bends must be determined by tests. Examples of safety lines ares shown in Fig. V–11.

FIGURE V–11 Safety Lines

a. AT NARROW BENDS AS A SUBSTITUTE FOR A PROHIBITORY LINE



b. AT ROAD JUNCTIONS AS A CONTINUATION OF A PROHIBITORY LINE



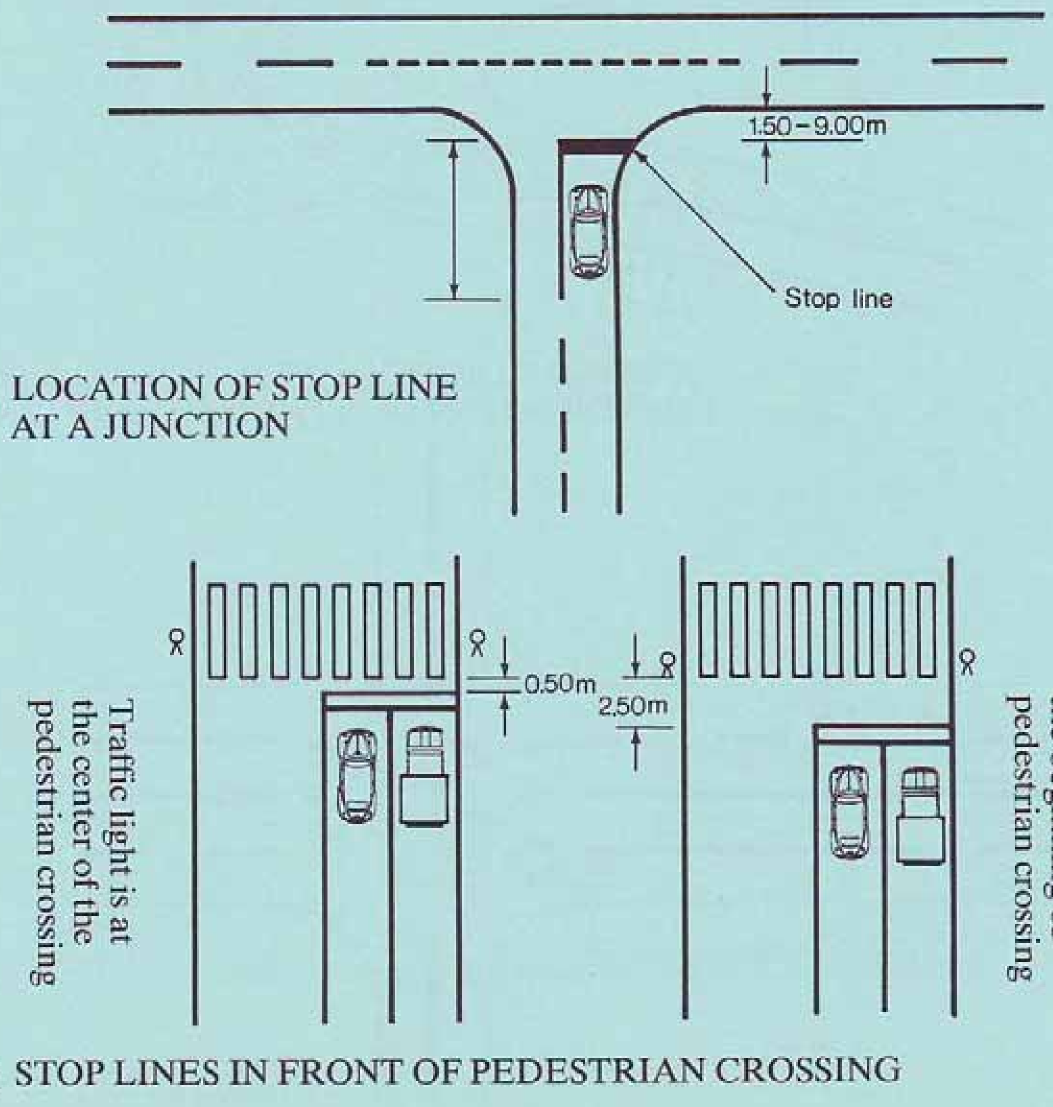
2 ROAD MARKING

2.06 STOP LINES

Stop lines must be laid on free road junctions as well as in villages or urban areas where traffic must be brought to a stop for observation of priority regulations (stop sign) or by the colour of a traffic signal or the signs given by a policeman. A stop line is 50 cm wide.

Stop lines should be placed in such a position (see Fig V–13) that a driver who stops behind this line has the clearest possible view of the traffic on the other legs of the intersection. He should also be able to see the signal or the policeman clearly. Minimum sight distance from stopping place to the right on the other road should be 120 m on free road sections, 80 m in populated areas. On the left it should be 80 m and 50 m respectively. Stop lines should be supplemented by a warning line of at least 30 m in length. It may also be supplemented by the word STOP inscribed on the carriageway.
If a stop line is used in conjunction with a stop sign it should generally be placed in line with the stop sign. However, if the sign cannot be placed exactly where vehicles stop, the stop line should be placed at the stopping point.
Stop lines and their application are shown in Figs V–12 and V–13.

FIGURE V–12 Stop Lines and their Location

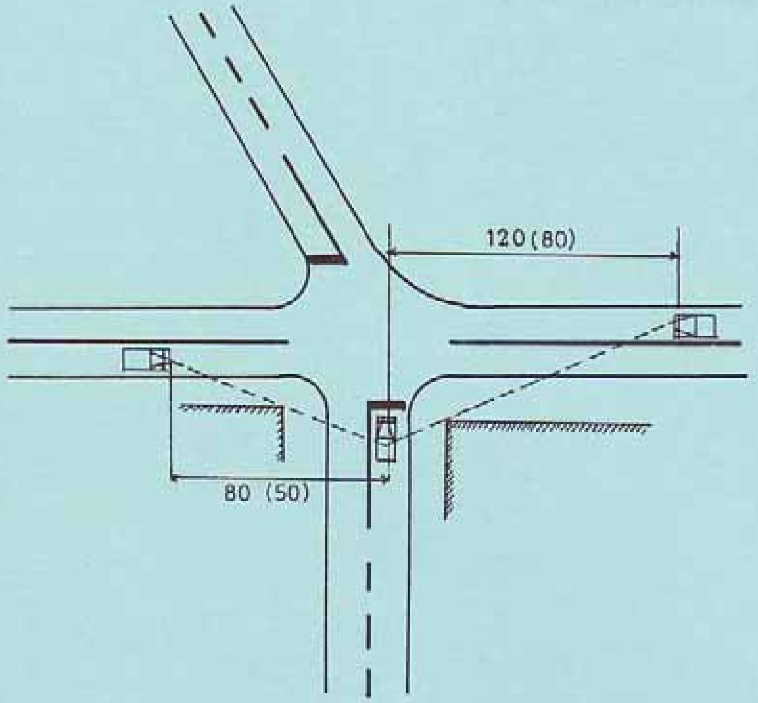


(a) LOCATION OF STOP LINE AT A JUNCTION

(b) STOP LINES IN FRONT OF PEDESTRIAN CROSSING

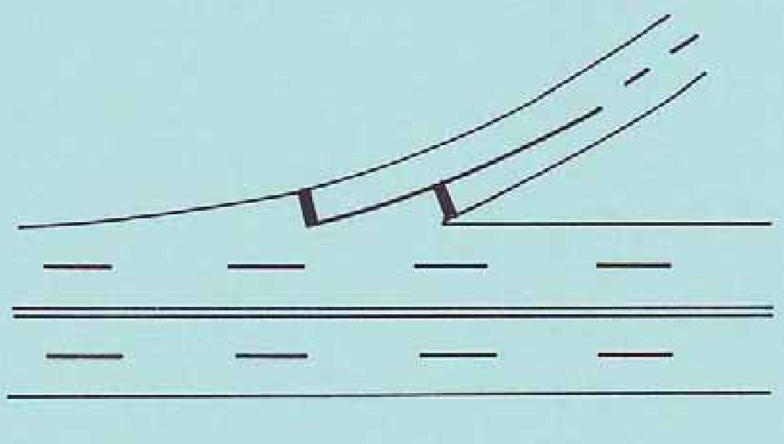
2 ROAD MARKING

FIGURE V–13 Stop Lines



120 minimum sight distance in free road section
(80) minimum sight distance in inhabited area

a. Minimum sight distance for stop lines



b. Stop line when road emerges in acute angle

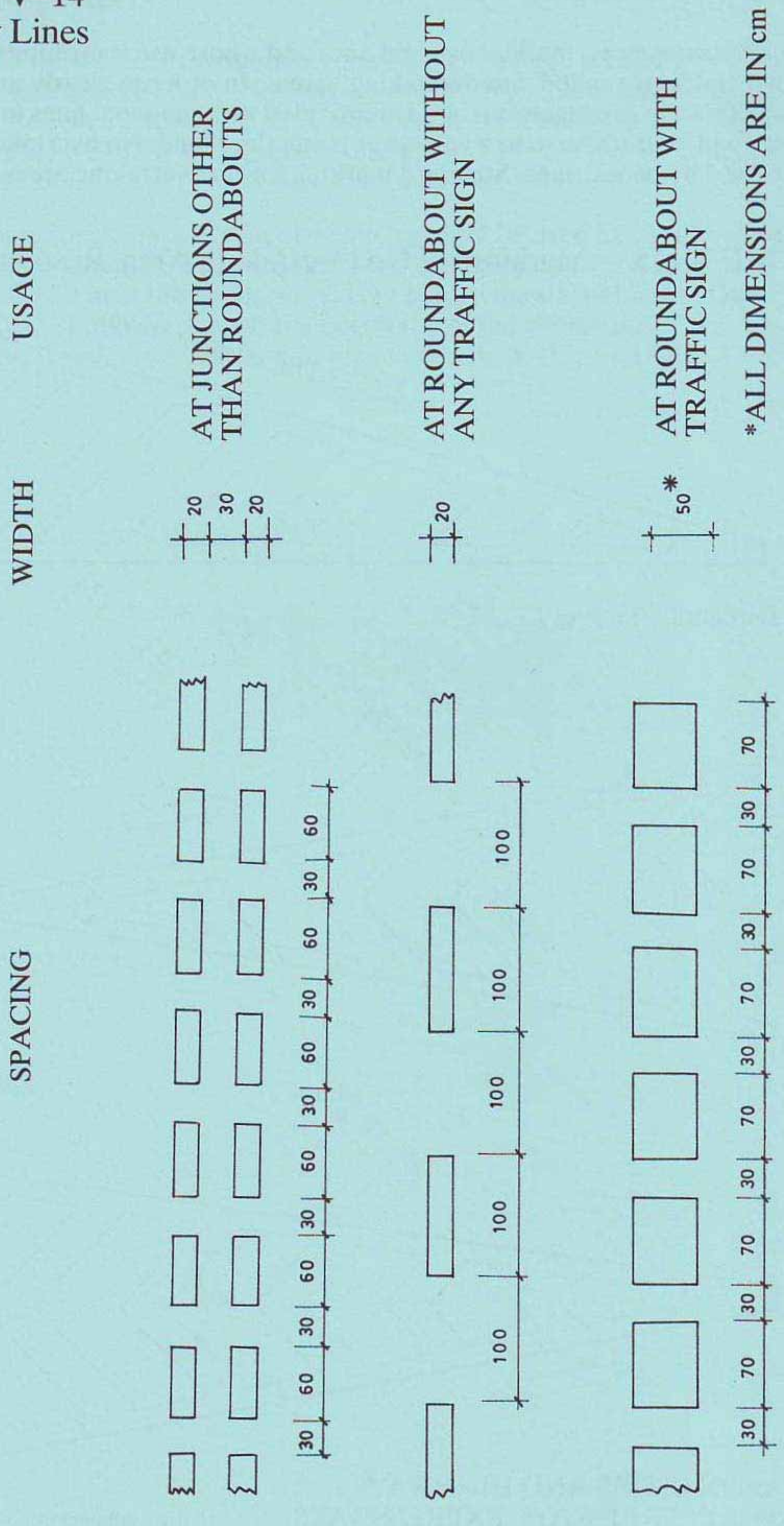
2 ROAD MARKING

2.07 GIVE WAY LINES

Give way lines should be used in conjunction with give way signs. Vehicles approaching a give way line should stop only when necessary to avoid collision with other traffic on the main road. Different types of give way lines and their application are shown in Fig V–14.

2 ROAD MARKING

FIGURE V–14
Give Way Lines

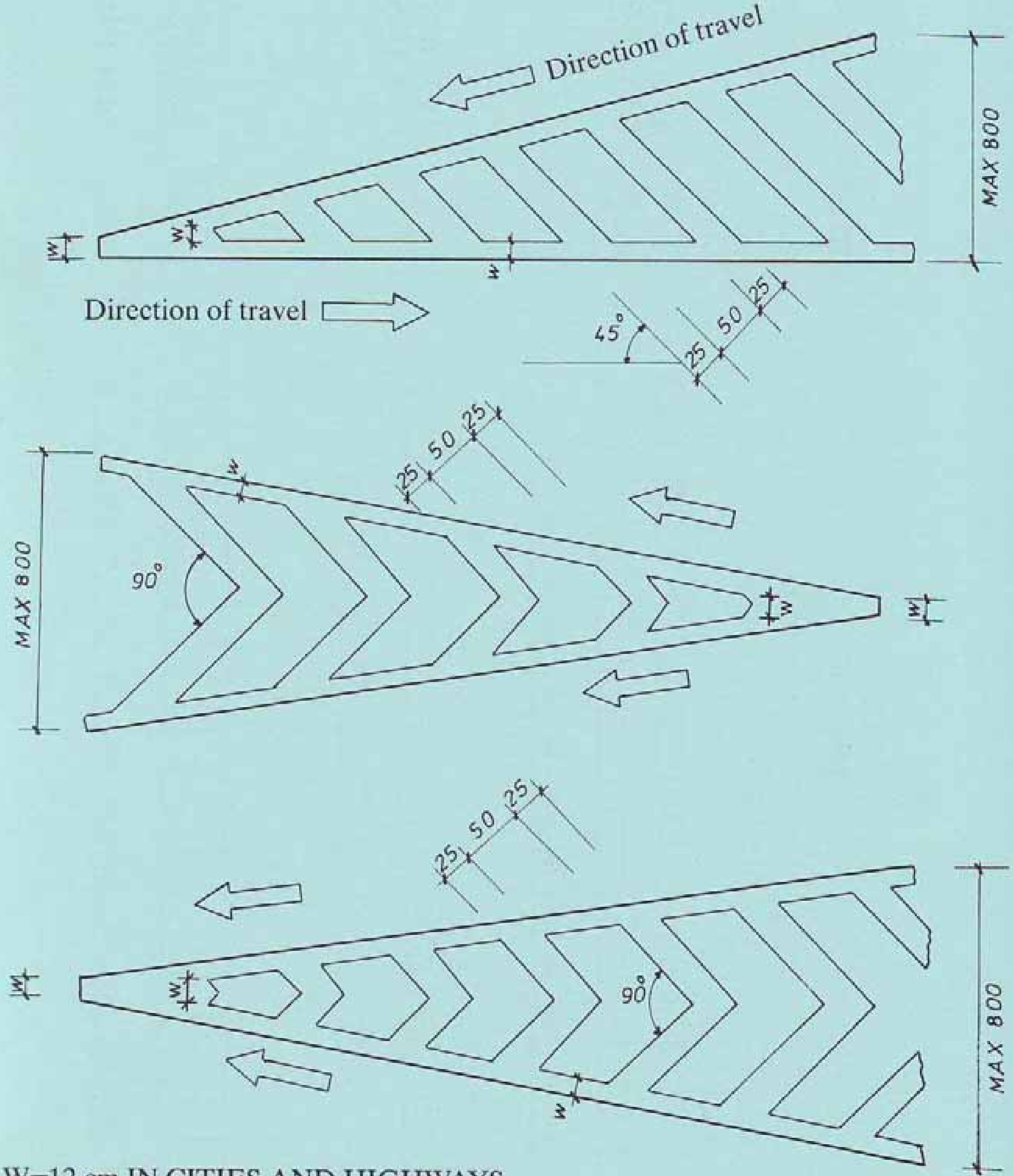


2 ROAD MARKING

2.08 NO OVERTAKING AREAS

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 a 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 Fig V–15.

FIGURE V–15 Markings of No Overtaking Area/Restricted Area



W=12 cm IN CITIES AND HIGHWAYS
W=15 cm ON FREEWAYS, EXPRESSWAYS

Note All dimensions are in CMs

2 ROAD MARKING

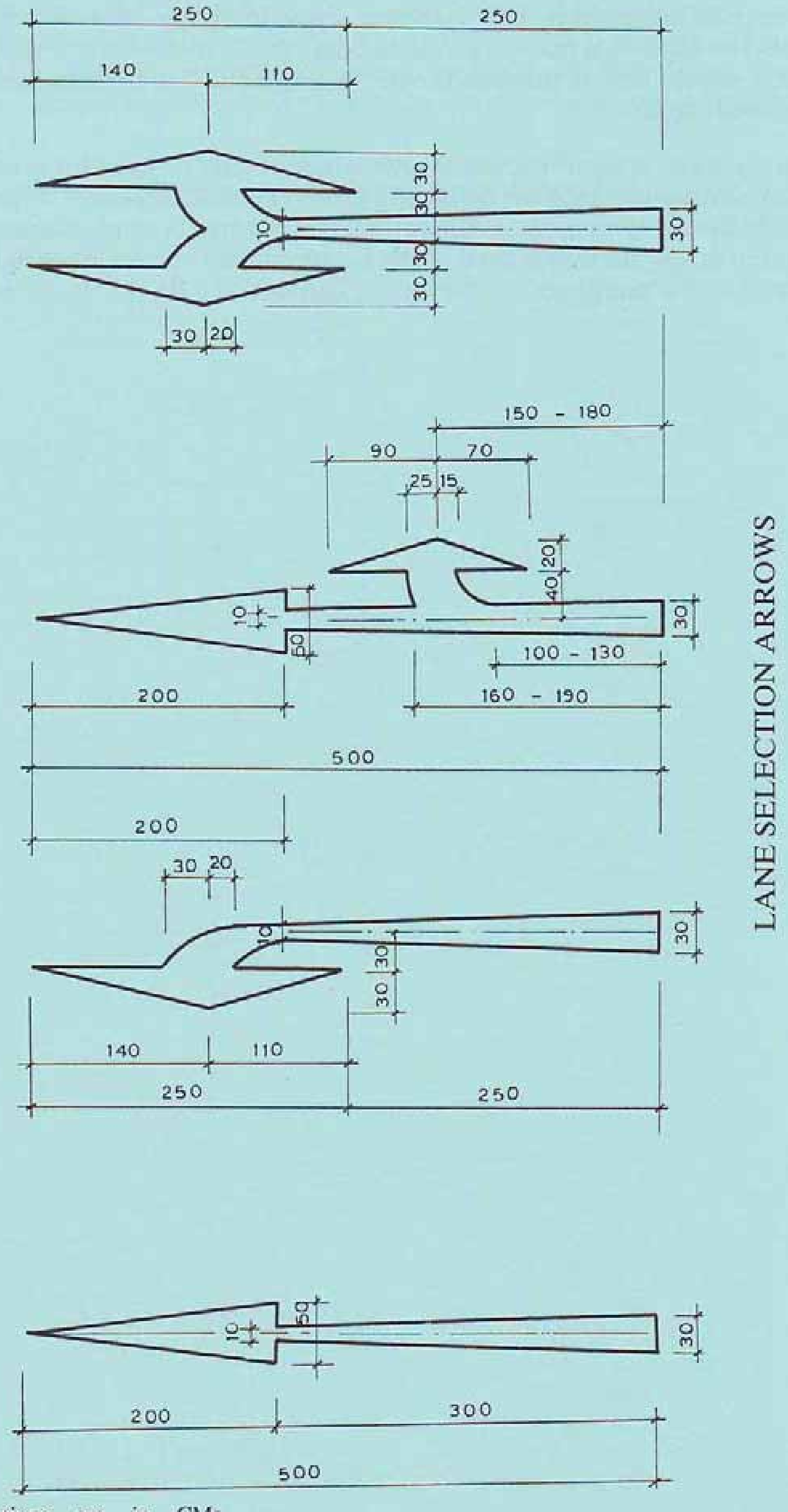
2.09 ARROW MARKINGS

On roads with traffic lanes to separate vehicles approaching an intersection, the lanes may be indicated by lane selection arrow markings. Lane selection arrows may also be used on a one-way road to confirm the direction of traffic. The length of arrow should be 5 m for speeds over 50 km/h and 3 m 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 10 m, 20 m, 30 m and 40 m 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 V–16, V–17, V–18 and V–19.

2 ROAD MARKING

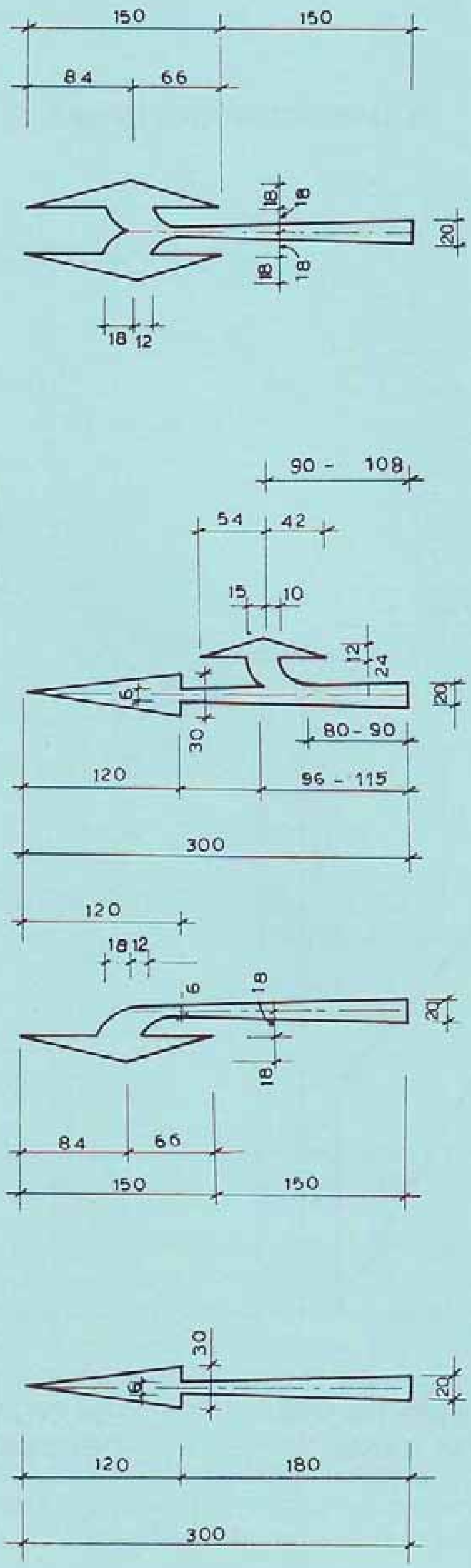
FIGURE V–16 Arrow Markings for Speeds over 50 km/h



Note: All dimensions are in CMs

2 ROAD MARKING

FIGURE V–17 Arrow Markings for Speeds under 50 km/h



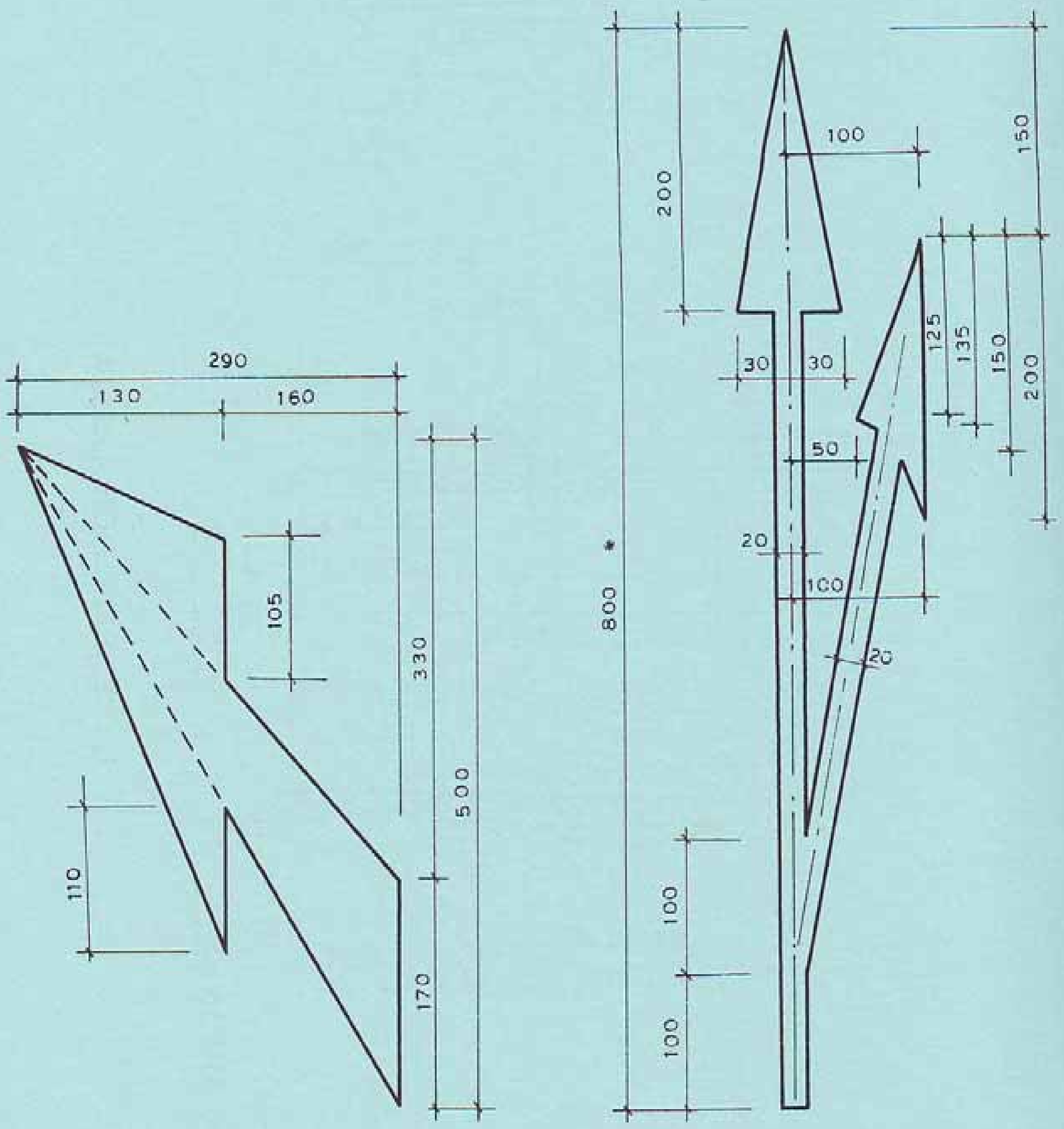
LANE SELECTION ARROWS

*DIMENSIONS ARE IN cm

2 ROAD MARKING

FIGURE V–18 Arrow Markings for Speeds over 50 km/h

*DIMENSION ARE IN cm



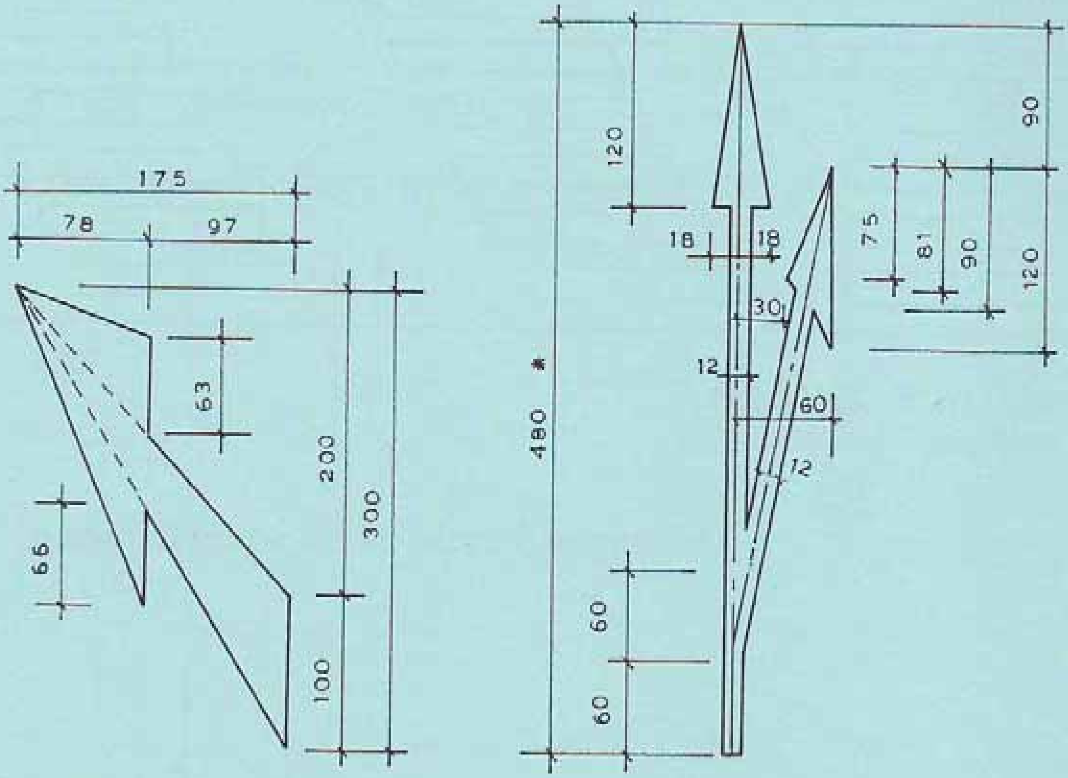
DEFLECTING ARROW FOR ACCELERATION LANE

ELONGATED ARROW FOR DECELERATION LANE

2 ROAD MARKING

FIGURE V–19 Arrow Markings for Speeds under 50 km/h

*DIMENSIONS ARE IN cm



DEFLECTING ARROW FOR ACCELERATION LANE

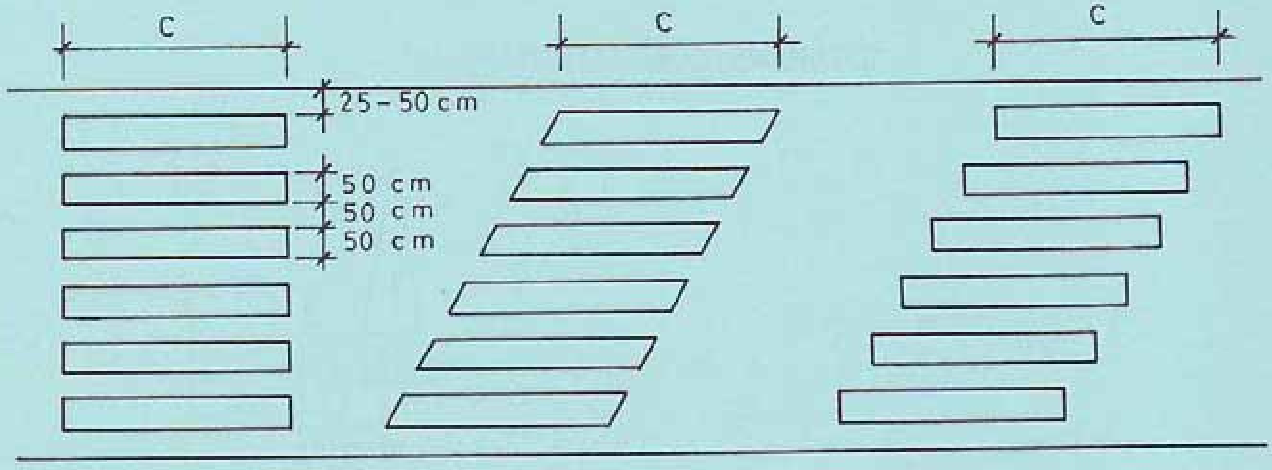
ELONGATED ARROW FOR DECELERATION LANE

2.10 PEDESTRIAN CROSSINGS

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 Fig V–20.
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 50 cm away from the nearest edge of intersecting roadways. In rural areas and on relatively high-speed urban streets it is desirable to give increased visibility to the motorist 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.

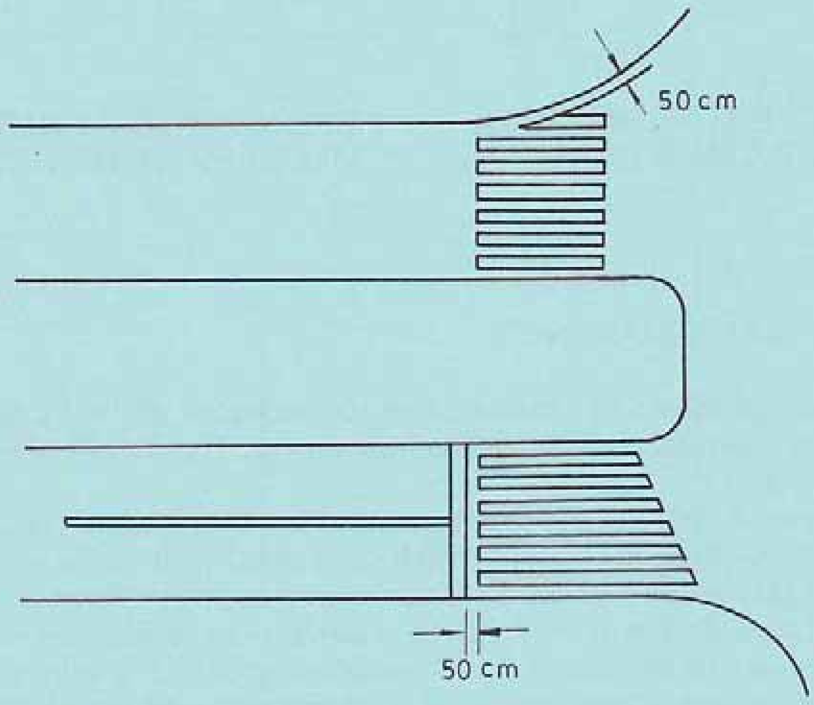
2 ROAD MARKING

FIGURE V–20 Marking for Pedestrian Crossings



$V \leqslant 60$ km/h $C \geqslant 2.5$ m
$V > 60$ km/h $C \geqslant 4.0$ m

Marking of Standard Pedestrian Crossing



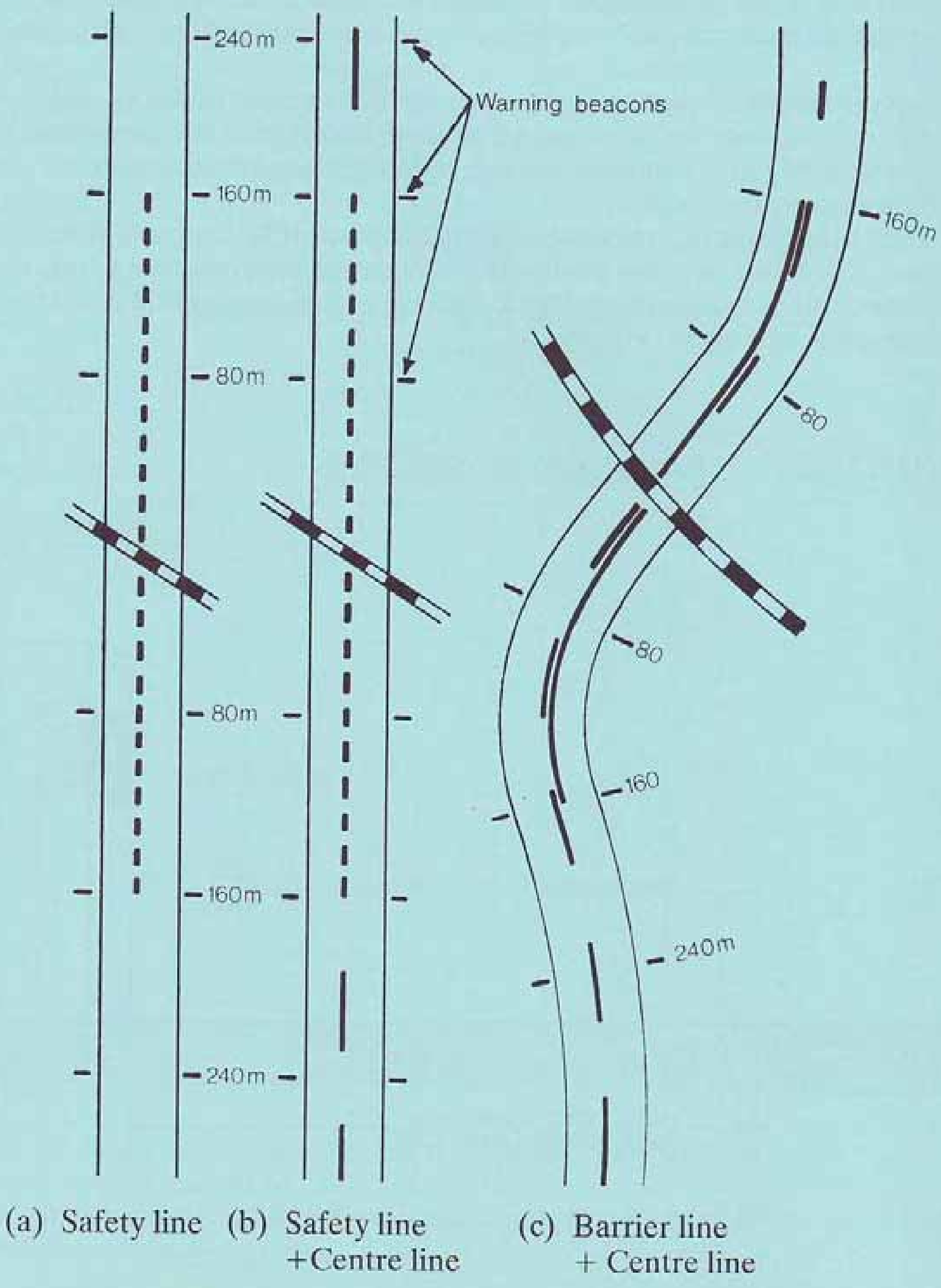
Pedestrian Crossing with Stop Line

2 ROAD MARKING

2.11 APPROACHES TO RAILWAY CROSSINGS

At level crossings where rail traffic has priority over road traffic, a warning line (1.5 m stripe, 1.5 m gap) should be marked in the centre of the carriageway, over a length of about 160 m from the 2nd bar warning beacon.
If in special cases overtaking is to be prohibited at railway crossings, a prohibitory line shall be provided on the right side next to the centre line departing from the 2nd bar beacon. If no centre line is provided over the whole road section, a centre line should begin at the 3rd bar beacon. These markings are shown in Fig V–21.
These markings are not sufficient protection by themselves and must always be used in conjunction with signs and other devices.

FIGURE V–21 Marking of Railway Crossings

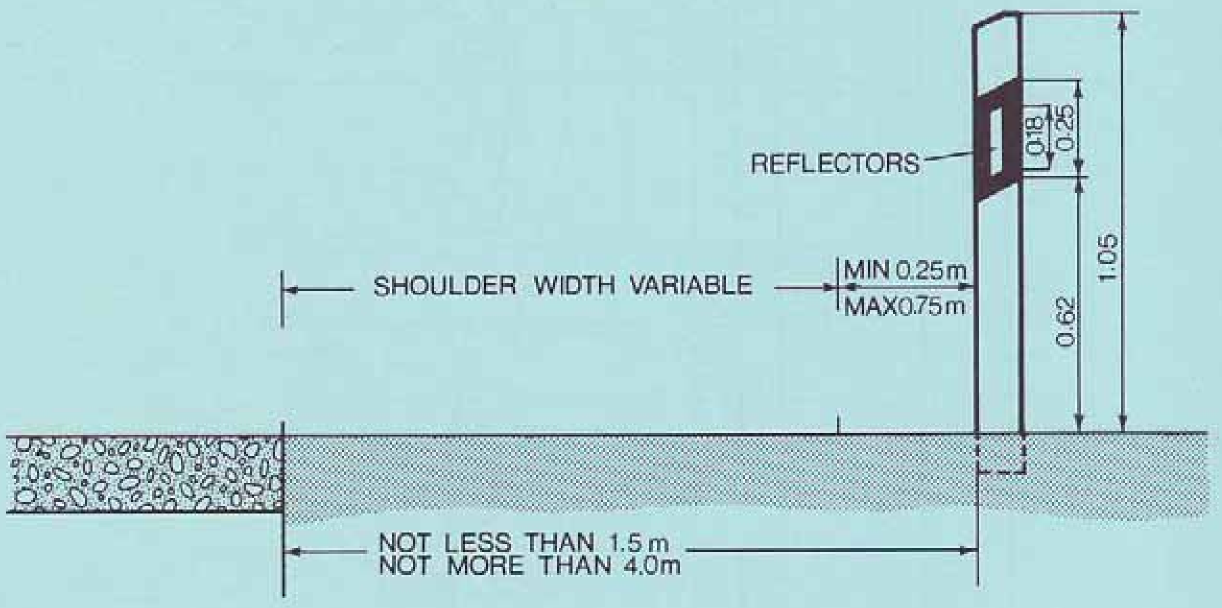


3 HAZARD AND MARKER POSTS

3.1 MARKER POSTS

Road edge delineation marker posts are effective aids for night driving.
Marker posts are to be considered as guide markings rather than warning devices and should never be substituted by a proper warning sign. They may be used on long, continuous highway sections or short sections where there are changes in horizontal alignment.
The standard size of the marker post is shown in Fig. V–22. The units should be clearly visible under clear atmospheric conditions from a distance of 300 m, when illuminated by the upper beam of standard automobile headlamps.
When used on undivided highways, bidirectional marker posts (i.e. visible from both directions) shall be installed on both sides of the road. When on dual carriageway, monodirectional marker posts shall be installed on both sides of the road.
Marker posts should be placed approximately 0.75 m clear of the shoulder but in no case more than 4 m nor less than 1.5 m from the edge of the carriageway.
In any case the distance from the carriageway edge should be consistent throughout the posted section.
On straight road sections, marker posts shall be spaced 50 m apart. Where driveways, side roads or other obstructions interrupt this regular spacing, the marker concerned shall be moved by a distance not exceeding one quarter of the normal spacing, or omitted entirely.

FIGURE V–22 Road Edge Marker Posts



3 HAZARD AND MARKER POSTS

On the approaches to and throughout horizontal curves the spacing should be such as to make five posts always visible to the right of the directional dividing line of a two-lane or the right lane of a multi-lane highway. The recommended spacing for highway marker posts on horizontal curves is tabulated in Table V–2.
Marker posts shall be placed at 30 m intervals along acceleration and deceleration lanes and along the outside or both sides of tangent portions of interchange ramps.
Where marker posts are used only on one side of an interchange ramp, and the ramp is curved, they shall be placed on the outside of the curves as set forth in the table.
On ramps where the curves reverse direction, marker posts should be used on the outside of each curve and should overlap in the area of the direction change.
Red reflectors may be used on the reverse side of any post so as to warn a motorist travelling in the wrong direction on that particular roadway or ramp.
Marker posts may be used to indicate the narrowing of the pavement and in this case the spacing should be 10 m. They should be optional on road sections between interchanges where fixed source lighting is in operation.
The type shown in Fig V–23 should be used to indicate a sharp deviation in route.
Black and white stripes should be used for permanent deviations.
Red and white stripes should be used at or near roadworks or other temporary obstructions.
The type shown in Fig V–24 should be used to indicate that the traffic lane or carriageway is closed to traffic.

Maximum Spacing of Marker Posts on Horizontal Curves

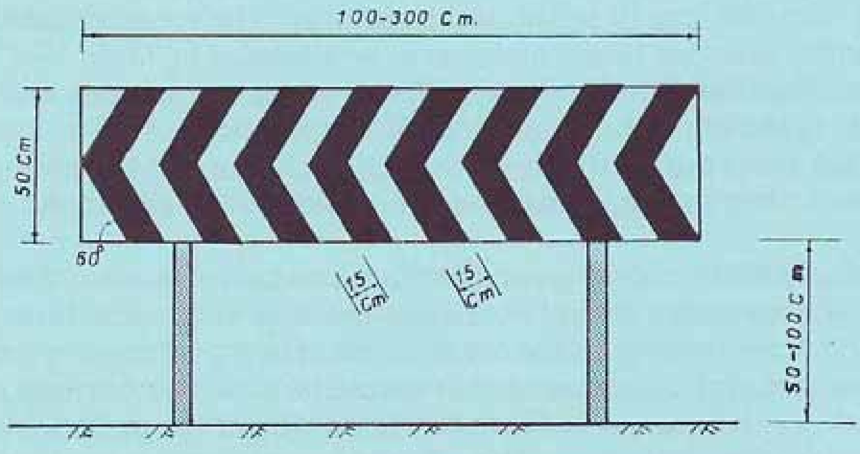
N=Normal roads, D=Dual Carriageways, E=Expressways TABLE V–2

Radius of curves (m)	Spacing of Delineator (m)		Spacing in advance and beyond curves (m)					
			First Delineator		Second Delineator		Third Delineator	
	N	D/E	N	D/E	N	D/E	N	D/E
20	3	3	6	6	10	10	20	25
30	3	3	7	7	11	11	20	21
40	4	4	9	9	15	15	20	30
50	6	6	2	12	20	20	33	40
60	7	7	17	17	29	40	33	50
70	8	8	23	23	33	50	33	50
80	10	10	28	35	33	50	33	50
90	12	12	33	42	33	50	33	50
100	15	15	33	50	33	50	33	50
200	20	20	33	50	33	50	33	50
300	20	27	33	50	33	50	33	50
400	33	35	33	50	33	50	33	50
500	33	45	33	50	33	50	33	50
600	33	50	33	50	33	50	33	50
700 and more	33	50	33	50	33	50	33	50

Spacing for radii not shown may be interpolated from table,
minimum spacing should be 3m and for all kinds of roads.
Maximum spacing should be 33m for normal roads and 50m for dual carriageways, expressways and freeways.

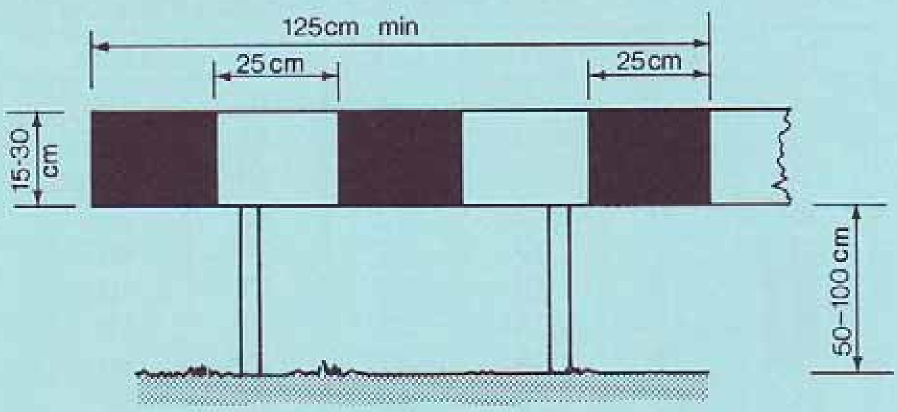
3 HAZARD AND MARKER POSTS

FIGURE V–23 Marker for Sharp Deviation of Route



Note: Alternating black and white stripes to be used for permanent deviation.
Alternating red and white stripes to be used for temporary deviation during road works and maintenance works.
All colours except black should be reflectorized.

FIGURE V–24 Marker to Indicate Traffic Lane or Carriageway Closed to Traffic



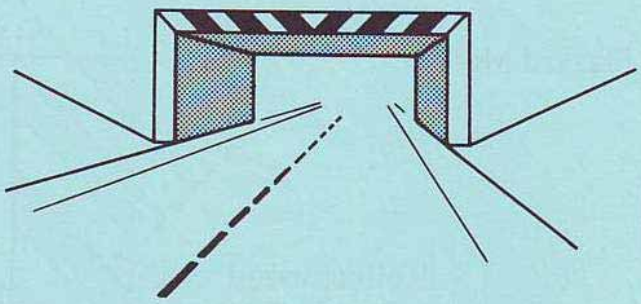
Note: Alternating red and white stripes (reflectorized)

3 HAZARD AND MARKER POSTS

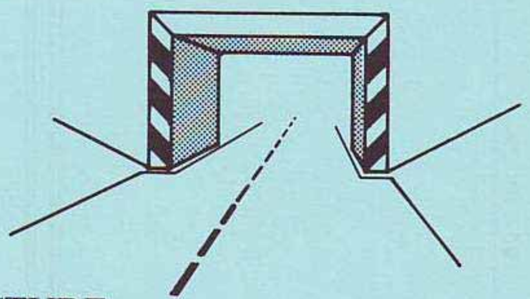
3.2 MARKING OF OBJECTS WITHIN THE ROADWAY

Bridge piers and abutments within 2 m of the pavement edge should be marked as shown in Fig V–25.
The marking can be used either for low structures, narrow structures, and low and narrow structures.

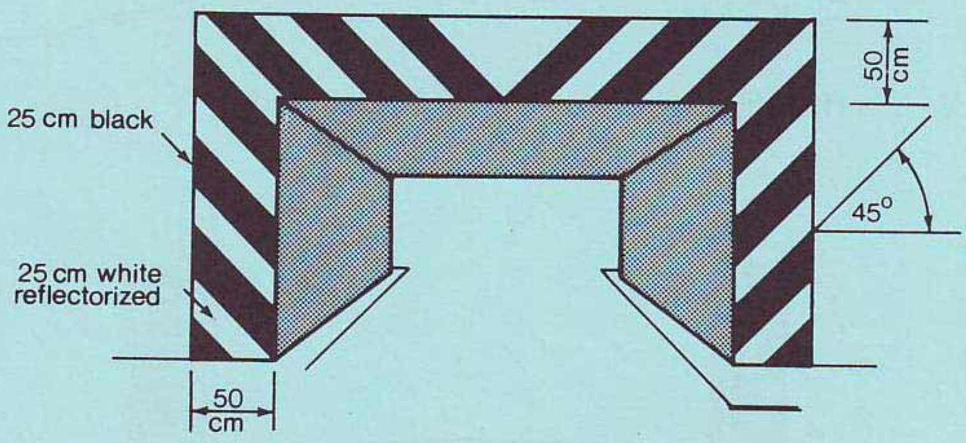
FIGURE V–25 Hazard Marking for Narrow and Low Structures



LOW STRUCTURE



NARROW STRUCTURE



NARROW AND LOW STRUCTURE

3 HAZARD AND MARKER POSTS

3.3 MARKING OF OBJECTS ADJACENT TO THE ROADWAY

The reflectorized hazard marker (Fig V–26) should be used to mark structure limits which are within 50 cm of the edge of the roadway. This marker shall be used in all cases where the narrow structure sign is erected ahead of the structure. The hazard marker should also be used to mark trees, rocks, curbs, approaching ends of sections with guard rails etc., which are close enough to the roadway to constitute a hazard to the motorist.
The markers can be attached directly to the structure or object presenting the hazard or erected on posts immediately in front of it.
Care should be taken that the inside edge of the marker is in line with the inside edge of the structure or object (Fig. V–27).

FIGURE V–26 Hazard Marker

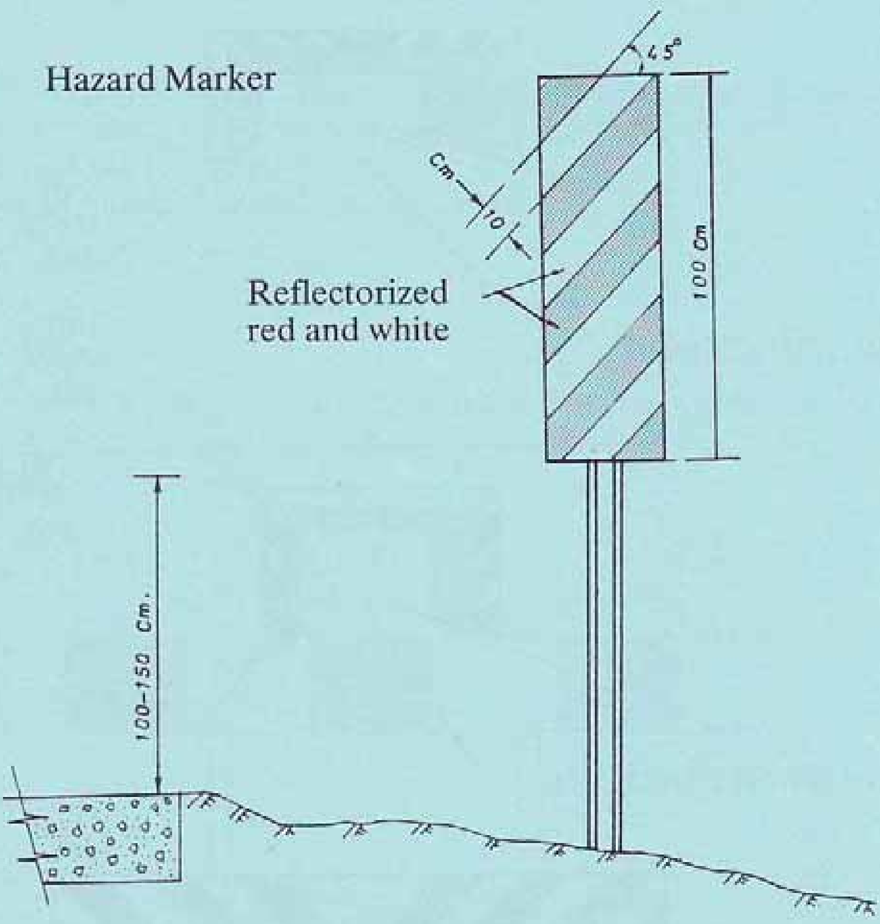


FIGURE V–27 Hazard Marker Location at Bridge End