

TECHNICAL RECOMMENDATIONS
FOR HIGHWAYS

TRH17

**GEOMETRIC
DESIGN OF
RURAL ROADS**

1988

ISBN 0 7988 3312 2

TRH17, Pretoria, South Africa, 1988

Published by the

Department of Transport

P O Box 415

PRETORIA

0001

Republic of South Africa

For the

Committee of State Road Authorities

REPRINTED 1991

REPRINTED 1993

Printed in the

Republic of South Africa

by V & R Printing Works (Pty) Ltd.

PRETORIA

PREFACE

TECHNICAL RECOMMENDATIONS FOR HIGHWAYS (TRH) are written for the practising engineer and describe current, recommended practice in selected aspects of highway engineering. They are based on South African experience and the results of research and have the full support and approval of the Committee of State Road Authorities (CSRA).

NOTE

This document is published by the Division of Roads and Transport Technology, CSIR, for and with the approval of the Geometric Standards Subcommittee of the Committee of State Road Authorities.

SYNOPSIS

This document deals with the geometric design of surfaced rural roads in South Africa and South-West Africa, and is based on the geometric design manuals of the rural road authorities.

Aspects covered in the document include the basic parameters of driver, vehicle and road surface. From these, guidelines relating to horizontal and vertical alignment and cross-section are derived. The location and design of intersections and interchanges are discussed, and some attention is paid to drainage elements and safety barriers.

SINOPSIS

Hierdie dokument handel oor die geometriese ontwerp van buitestedelike dek-laagpaaie in Suid-Afrika en Suidwes-Afrika en is gebaseer op die handleidings vir geometriese ontwerp van die buitestedelike padowerhede.

Aspekte wat deur die dokument gedek word sluit in die basiese parameters van die bestuurder, voertuig en padoppervlak. Hieruit word riglyne wat betrekking het op horisontale en vertikale belyning en dwarsnit, afgelei. Die ligging en ontwerp van kruisings en wisselaars word bespreek en aandag geskenk aan dreinerings-elemente en veiligheidsrelingings.

KEYWORDS

Geometric design, horizontal alignment, vertical alignment, cross-section, intersection, interchange, safety barrier, drainage.

FOREWORD

The geometric design standards of the Southern African rural road authorities were derived largely from American practice. Local experience and opinion over the years, however, have led to differences between the sets of standards used by the various authorities.

The Committee of State Road Authorities is aware that uniformity of design practice will be of value in meeting the expectations of a rapidly growing and increasingly mobile driver population. Furthermore, the Committee sees the need for practice to be more specifically oriented towards southern African conditions. It has therefore decided that the objectives of uniformity and local relevance should be more actively pursued than in the past.

TRH17 represents the achievement of the first of these objectives.

TRH17 is based on current practice and consensus among the rural road authorities, and is regarded as a pilot document. The second objective is to be attained by an on-going process of amendment of the pilot document, through consultation between the authorities, discussion with practitioners and research. The achievement of both objectives is the ultimate goal: a definitive portfolio of statements on the geometric design of rural roads.

The wide variety of topographic and climatic conditions prevailing in southern Africa precludes the provision of standards that can cover all circumstances. The rigid application of standards is, in any event, inimical to good design. TRH17 therefore offers guidelines in preference to standards. In addition, brief reasons are given for the suggested values of the guidelines. These should be useful to the designer in judging the validity of the guidelines in a specific situation, as well as the consequences of departing from the values suggested.

The Committee responsible for the preparation of this document comprised representatives of the

Department of Transport, Republic of South Africa
Transvaal Provincial Administration Roads Department
Department of Transport, South-West Africa Administration
Cape Provincial Administration Roads Department
Orange Free State Provincial Administration Roads Branch
Division of Roads and Transport Technology, CSIR.

A soundly based philosophy means far more to good design than any book of standards. It is to the development, encouragement and maintenance of this philosophy of design that TRH17 is dedicated.

T L Krüger
CHAIRMAN
Geometric Standards Subcommittee

CONTENTS

Preface
Synopsis
Foreword

CHAPTER 1 INTRODUCTION

- 1.1 Background
- 1.2 Scope of the document
- 1.3 Traffic volumes
- 1.4 Traffic speed
- 1.5 Capacity and level of service
- 1.6 Classification of rural roads

CHAPTER 2 BASIC CRITERIA

- 2.1 Introduction
- 2.2 The design vehicle
- 2.3 The driver
- 2.4 The road surface
- 2.5 Sight distance

CHAPTER 3 HORIZONTAL ALIGNMENT

- 3.1 Introduction
- 3.2 Curvature
- 3.3 Successive curves
- 3.4 Transition curves
- 3.5 Superelevation

CHAPTER 4 VERTICAL ALIGNMENT

- 4.1 Introduction
- 4.2 Curvature
- 4.3 Gradients
- 4.4 Climbing lanes
- 4.5 Passing lanes

CHAPTER 5 CROSS-SECTIONAL ELEMENTS

- 5.1 Introduction
- 5.2 Lanes
- 5.3 Shoulders
- 5.4 Medians
- 5.5 Verges
- 5.6 Slopes
- 5.7 Minor structures

CHAPTER 6 DRAINAGE

- 6.1 Introduction
- 6.2 Silting and scouring
- 6.3 Channel profiles
- 6.4 Types of drain
- 6.5 Discharge from drains

CHAPTER 7 SAFETY BARRIERS

- 7.1 Introduction
- 7.2 Guardrails
- 7.3 Median barriers

CHAPTER 8 INTERSECTIONS

- 8.1 Introduction
- 8.2 Location of intersections
- 8.3 Unchannelized intersections
- 8.4 Speed-change lanes
- 8.5 Channelized intersections
- 8.6 Median openings

CHAPTER 9 INTERCHANGES

- 9.1 Introduction
- 9.2 Ramps and their application
- 9.3 Interchanges and their application
- 9.4 Lane balance
- 9.5 Weaving

CHAPTER 10 INTERCHANGE DESIGN

- 10.1 Introduction
- 10.2 Warrants for an interchange

- 10.3 Minimum spacing of interchanges
- 10.4 Ramp design speed
- 10.5 Decision sight distance
- 10.6 Horizontal curvature on ramps
- 10.7 Superelevation on ramps
- 10.8 Crossover crown
- 10.9 Vertical alignment of ramps
- 10.10 Ramp cross-section
- 10.11 Ramp terminals

CHAPTER 11 PEDESTRIANS AND PEDAL CYCLISTS

- 11.1 Introduction
- 11.2 Footways
- 11.3 Bridges
- 11.4 Bus stops
- 11.5 Refuge island
- 11.6 Footbridges and subways
- 11.7 Lighting
- 11.8 Cycle lanes
- 11.9 Speed zoning

REFERENCES

SELECTED BIBLIOGRAPHY

LIST OF TABLES

- 1.5(a) Capacity of two-lane two-way roads
- 1.5(b) Estimated maximum average daily traffic and K factors for two-lane, two-way roads
- 1.5(c) Estimated maximum ADT and K Factors for four-lane freeways
- 2.2.1 Dimensions of design vehicles
- 2.2.3 Minimum turning radii
- 2.4.1 Brake force coefficients for various speeds
- 2.5.1 Stopping sight distance on level roads
- 2.5.2 Barrier sight distance
- 2.5.3 Decision sight distance on level roads
- 2.5.4 Passing sight distance on level roads
- 3.2.1 Minimum radii of horizontal curvature

- 3.5.3(a) Relative slope factor
- 3.5.3(b) Minimum length of superelevation run-off for two-lane roads
- 3.5.3(c) Lane factors for superelevation run-off
- 4.2.1 Minimum values of K for vertical curves
- 4.2.2 Minimum lengths of vertical curves
- 4.3.1 Maximum gradients
- 4.3.2 Critical length of grade
- 4.4.1 Traffic volume warrants for climbing lanes
- 6.2 Scour velocities for various materials
- 8.4.1 Deceleration lane length (including taper)
- 8.4.2(a) Acceleration lane length (including taper)
- 8.4.2(b) Minimum radii of horizontal curvature
- 8.5.2 Turning travelled way widths
- 10.4 Minimum design speed of semi-directional ramps
- 10.6 Minimum radii of horizontal curvature on ramps
- 10.7 Rate of superelevation development
- 10.8 Maximum change in slope across crossover crown line
- 10.9.1(a) Minimum values of K for vertical curves on ramps
- 10.9.1(b) Minimum length of vertical curves on ramps
- 11.2 Warrants for pedestrian footways
- 11.5 Minimum pedestrian sight distance in metres for different speed limits and road widths
- 11.6 Vertical and horizontal dimensions recommended for subways without artificial lighting or a break in the median

LIST OF FIGURES

- 2.2.2(a) Wheel tracks for SU design vehicle
- 2.2.2(b) Wheel tracks for SU+T design vehicle
- 2.2.4 Truck speeds on grade
- 2.5.1(a) Stopping sight distance on grade
- 2.5.1(b) Horizontal radius for stopping sight distance
- 2.5.5(a) Shoulder sight distance for stop condition
- 2.5.5(b) Shoulder sight distance for yield condition
- 3.3.1 Superelevation of reverse curves
- 3.3.2 Superelevation of broken-back curves
- 3.5.2 Superelevation rates
- 3.5.3(a) Attainment of superelevation without transition

- 3.5.3(b) Attainment of superelevation with transition
- 4.4(a) Preferred layout for a climbing lane
- 4.4(b) Alternative layout for a climbing lane
- 4.5 Layout of a typical passing lane
- 5.1 Cross-sectional elements
- 5.6.2 Suggested transition to split grading
- 6.4 Typical drain profiles
- 7.2.1(a) Equal severity curve
- 7.2.1(b) Application of equal severity curve
- 7.2.2 Mounting of guardrails
- 7.2.3 Guardrail end treatment
- 7.3.2 Median barriers
- 7.3.3 End treatment of median barriers
- 8.2 Angle of skew
- 8.3.2 Kerb types according to SABS 927-1969
- 8.5 Typical channelized intersection
- 8.5.1 Channelizing island
- 8.6 Median end treatment
- 9.2.2(a) Ramp types for left turns
- 9.2.2(b) Ramp types for right turns
- 9.3.1(a) Diamond interchanges
- 9.3.1(b) Parclo interchanges
- 9.3.2(a) Cloverleaf interchanges
- 9.3.2(b) Directional interchanges
- 9.3.3 Three-legged interchanges
- 9.4 Lane balance and continuity
- 10.11(a) Single-lane entrance
- 10.11(b) Single-lane exit
- 10.11(c) Two-lane entrance (with one lane added)
- 10.11(d) Two-lane exit (with one lane dropped)
- 11.4 Typical bus bay layouts for gravel or bitumen surfaces

1 INTRODUCTION

1.1 Background

This is the first comprehensive portfolio of geometric design guidelines for rural roads to be compiled jointly by the Departments of Transport of the Republic of South Africa and South-West Africa and the Roads Departments of the four Provinces of the Republic. It represents agreement on uniformity of geometry specifically for surfaced roads, based on current practices and standards. The document will be subject to on-going amendment as research results, tailoring adopted values to current local conditions, come to hand.

Although based on the geometric design manuals of the rural road authorities referred to, this document contains two major departures from those preceding it. Firstly, in most cases a brief explanation of the reasons for the recommended values is given. This will aid the evaluation of the applicability of these values to a specific set of circumstances, and facilitate the assessment of the consequences of departing from the guideline values.

The second major departure is that the values quoted are guidelines only. The designer is expected to apply his own judgment in the selection of design values appropriate to the project in hand. Although the need for the application of engineering judgment is stated in various of the original departmental design manuals, the phraseology of this document is intended to emphasize the point. Reference is made throughout to guideline values, and not to design standards.

1.2 Scope of the document

The scope of this document is restricted to the rural surfaced roads that are the responsibility of

- the Department of Transport, Republic of South Africa
- the Department of Transport, South-West Africa
- the Cape Provincial Administration Roads Department
- the Natal Provincial Administration Roads Department
- the Orange Free State Provincial Administration Roads Branch
- the Transvaal Provincial Administration Roads Department.

A rural road is defined as one that is not likely to acquire urban characteristics during its design life.

These departments, together with the Division of Roads and Transport Technology of the CSIR, have established the Committee of State Road Authorities, under whose aegis this document was prepared.

From time to time, individual authorities will specify guidelines appropriate to circumstances prevailing in their specific areas of responsibility. These separately published guidelines will serve to advise users of this document of differences between the requirements of the various authorities, and be the basis of on-going discussion between the authorities aimed at preserving the uniformity of practice that has been achieved. Should it be found that there are significant areas of agreement in the guidelines, the material will be incorporated into the body of this document.

1.3 Traffic volumes

The design of new routes or improvements to existing routes should be based on projected traffic volumes. A design life of 20 years is often assumed for rural roads. This period may be altered subject to the planning of the authority concerned, and the evaluation of the economic consequences of departure from the suggested timespan. For example, a relatively low-cost road carrying light traffic volumes may justify a shorter design life because of the savings accruing from the lower number of axle-load repetitions in the shorter period. A road in very hilly or mountainous terrain may require a longer design life to achieve a reasonable return on the initial cost of construction.

Projected traffic volumes should preferably not be derived only by applying a growth factor to present-day traffic counts. Where an alternative route is available, an origin-destination survey may be necessary. Where there are many possible alternatives, a full-scale transportation study may have to be considered. A useful input to the determination of projected traffic volumes is the Rural Traffic Demand Model administered by the Department of Transport.

Traffic volumes are often expressed in terms of average daily traffic (ADT) measured in vehicles per day. The ADT does not however reflect monthly, daily or hourly fluctuations in traffic volume. On rural roads the design hourly volume is frequently assumed to be the 30th highest hourly volume of the future year chosen for design, ie the hourly volume exceeded during only 29 hours of that year. The design hourly volume is expressed as a percentage of the ADT and typically varies between 12 and 18 per cent. A figure of 15 per cent is thus normally assumed unless actual counting suggests another percentage. On an annual basis, the directional split on most rural roads is approximately 50:50. However, during any specific hour the volume in one direction may be much heavier than in the other. The directional split is often in the ratio of 60:40, and the heavier flow is the design criterion.

1.4 Traffic speed

Traffic speeds are measured and quoted in kilometres per hour. The Highway Capacity Manual¹ lists definitions of ten different speeds, such as spot speed, time mean speed, overall travel speed, running speed, etc. In this document, reference is principally to design speed and operating speed.

The design speed is a speed selected for the purposes of the design and correlation of those features of a road (such as horizontal curvature, vertical curvature, sight distance and superelevation) upon which the safe operation of vehicles depends. The operating speed is the highest running speed at which a driver can travel on a given road under favourable weather and prevailing traffic conditions, without at any time exceeding the design speed. Implicit in this definition of operating speed is the idea that the design speed is also the maximum safe speed that can be maintained on a given section of road when traffic conditions are so favourable that the design features of the road govern the driver's selection of speed. One should not lose sight of the fact that a degree of arbitrariness attaches to the concept of maximum safe speed. Absolute maximum speed at which an individual driver is safe depends as much on the driver's skill and reaction time, the quality and condition of the vehicle and its tyres, the

weather conditions and the time of day, insofar as this affects visibility, as on the design features of the road.

Where it is necessary to vary the design speed along a section of road because of topographic or other limiting features, care should be taken to ensure that adequate transitions from higher to lower standards are provided.

1.5 Capacity and level of service

In the absence of a detailed inventory of traffic flow characteristics on South African rural roads, reliance is placed on the Highway Capacity Manual¹.

The capacity of a road is defined as the volume of traffic associated with Level of Service E. Level of Service B is usually selected for design purposes. Tables 1.5(a) to (c) give some useful values for capacities and service volumes from the most recent chapters of the Highway Capacity Manual¹.

1.6 Classification of rural roads

Because of differences in legal and historical backgrounds, and administrative requirements, there are differences between the classification systems adopted by the various authorities.

In spite of differences in nomenclature and application, all the classification systems are broadly functional. The road network consists of a three-level hierarchy, with each level subject to further subdivision.

Primary level

The primary level road is intended for main movement, i.e. relatively uninterrupted high-speed flow between towns. This level can be subdivided into long-distance routes connecting towns in several regions as the highest order, followed by routes connecting towns in adjacent regions, followed in turn by intra-regional routes between towns.

TABLE 1.5(a)
CAPACITY OF TWO-LANE TWO-WAY ROADS

DIRECTION AL SPLIT	TOTAL CAPACITY (passenger cars per hour)
50:50	2 800
60:40	2 640
70:30	2 480
80:20	2 320
90:10	2 160
100:0	2 000

TABLE 1.5(b)

ESTIMATED MAXIMUM AVERAGE DAILY TRAFFIC FOR SELECTED LEVELS OF SERVICE AND K FACTORS FOR FLAT, ROLLING AND MOUNTAINOUS TERRAIN FOR TWO-LANE TWO-WAY ROADS

K FACTOR	LEVEL OF SERVICE				
	A	B	C	D	E
Flat terrain					
0,10	2 200	4 900	8 800	14 500	24 600
0,11	2 000	4 500	8 000	13 200	22 300
0,12	1 800	4 100	7 400	12 100	20 500
0,13	1 700	3 800	6 800	11 100	18 900
0,14	1 600	3 500	6 300	10 300	17 500
0,15	1 500	3 300	5 900	9 700	16 400
0,20	1 100	2 500	4 400	7 300	12 300
0,25	900	2 000	3 500	5 800	9 800
Rolling terrain					
0,10	1 500	3 500	6 400	10 600	18 200
0,11	1 300	3 100	5 800	9 600	16 600
0,12	1 200	2 900	5 300	8 800	15 200
0,13	1 100	2 700	4 900	8 100	14 000
0,14	1 000	2 500	4 600	7 500	13 000
0,15	1 000	2 300	4 300	7 000	12 100
0,20	800	1 800	3 200	5 300	9 100
0,25	600	1 400	2 600	4 200	7 300
Mountainous terrain					
0,10	800	2 000	3 800	6 500	11 200
0,11	700	1 800	3 500	5 900	10 200
0,12	700	1 700	3 200	5 400	9 400
0,13	600	1 600	2 900	5 000	8 700
0,14	600	1 400	2 700	4 700	8 000
0,15	500	1 300	2 500	4 300	7 500
0,20	400	1 000	1 900	3 300	5 600
0,25	300	800	1 500	2 600	4 500

Flat terrain: $E_t = 1,5$, no-passing zones 20%

Rolling terrain: $E_t = 4,0$, no-passing zones 40%

Mountainous terrain: $E_t = 10,0$, no-passing zones 60%

For all cases a 60:40 directional split and 15% trucks are assumed.

K = Proportion of average daily traffic (ADT) in the design hour

E_t = Truck equivalency factor

TABLE 1.5(c)

ESTIMATED MAXIMUM ADT FOR SELECTED LEVELS OF SERVICE AND K FACTORS FOR FLAT, ROLLING AND MOUNTAINOUS TERRAIN FOR FOUR-LANE FREEWAYS

K FACTOR	LEVEL OF SERVICE				
	A	B	C	D	E
Flat terrain					
0,10	20 500	35 000	50 100	58 300	60 300
0,11	18 700	31 800	45 500	52 900	54 800
0,12	17 100	29 200	41 700	48 500	50 200
0,13	15 700	26 900	38 500	44 700	46 400
0,14	14 700	25 000	35 700	41 600	43 100
0,15	13 700	23 300	33 400	38 800	40 200
Rolling terrain					
0,10	15 700	26 700	38 200	44 400	46 000
0,11	14 200	24 200	34 700	40 300	41 800
0,12	13 000	22 200	31 800	37 000	38 300
0,13	12 000	20 500	29 300	34 200	35 300
0,14	11 200	19 100	27 200	31 700	32 800
0,15	10 400	17 700	25 400	29 600	30 700
Mountainous terrain					
0,10	11 100	18 800	27 000	31 400	32 500
0,11	10 100	17 200	24 600	28 500	29 600
0,12	9 200	15 700	22 500	26 200	27 100
0,13	8 500	14 500	20 700	24 200	25 000
0,14	7 900	13 500	19 200	22 400	23 200
0,15	7 300	12 600	18 000	20 900	21 700

Flat terrain: $E_t = 1,7$

Rolling terrain: $E_t = 4,0$

Mountainous terrain: $E_t = 8,0$

For all cases a 60:40 directional split and 15% trucks are assumed.

K = Proportion of ADT in design hour

E_t = Truck equivalency factor

Secondary level

The secondary level road has a collector-distributor function. While primary routes have towns as their destinations, secondary roads invariably connect local areas to the primary network. While a secondary road may connect a local area directly with the nearest town, it is more likely to have one of its terminals

on a primary road. The other terminal might be on another primary or secondary road or even at the intersection of two or more tertiary roads. Travel speeds on the secondary road network are generally not as high as those on the primary roads.

Traffic volumes very often do not warrant the surfacing of secondary roads, which are therefore divided into the two categories of unsurfaced and surfaced secondary roads.

Tertiary level

Tertiary roads are intended to provide access to properties, ie they link them to the higher-order routes in the hierarchy. It follows that traffic volumes and speeds on these roads tend to be low, so that tertiary roads are rarely surfaced. Properties are also linked directly to secondary and primary routes, but the control of access to a route is more stringently applied as the importance of the route in the hierarchy increases. In the case of a long-distance interregional freeway, direct access is prohibited, whereas on an intraregional route access may perhaps be restricted to one access per property, with virtually no restriction on access to a secondary road, except as dictated by road geometry.

Design approach

The functional classification of the various elements of a road system does not automatically lead to the selection of a design speed and cross-section for a specific link in that network, although it is an aid in the evaluation process. A short intraregional primary road may be required to serve such high volumes of traffic that a freeway is warranted, whereas a long-distance route connecting several regions may traverse such a sparsely populated area of the country that it barely warrants surfacing. In level terrain any route, regardless of its position in the general hierarchy, could be designed to a high design speed, whereas in rugged terrain the design speed of the most important link in the network would be forced down.

In general, higher-order routes tend to have higher design speeds and serve greater volumes of traffic, and hence require cross-sections with greater lane and shoulder widths, perhaps incorporating medians.

Provided that land-use patterns do not change, annual traffic growth on the lower-order routes, in terms of vehicle numbers, is likely to be low. On longer-distance primary routes urban growth will influence traffic growth on the links between urban areas, causing the growth rate to be higher on these routes. The selection of an appropriate growth factor is based on the position a road occupies in the hierarchy. Furthermore, the seasonal, weekly, daily or hourly fluctuation in traffic volumes on a given route varies with the function of the route. Functional classification of the road network is therefore useful as a guide in the selection of appropriate design parameters.

2 BASIC CRITERIA

2.1 Introduction

This chapter discusses the parameters from which these guidelines are derived. Knowledge of the design vehicle, its dimensions and performance characteristics, is necessary before climbing lanes, maximum permissible grades, intersection layout and turning roadway radii and widths can be decided on. The driver's eye height above the road surface and his reaction time are used to derive stopping and other sight distances.

When these sight distances are known, rates of vertical curvature can, in turn, be derived. The coefficient of friction of the road surface, in conjunction with the parameters relating to the driver, determines the various sight distances, and also affects superelevation rates, from which minimum horizontal radii for the various design speeds are calculated.

The derivation of the recommended values is given so that the designer dealing with some other design vehicle or circumstance will be in a position to calculate appropriate values.

2.2 The design vehicle

The only South African design vehicle for which dimensions have been established is the passenger car (P); the single unit truck (SU) is the subject of study. Dimensions have been tentatively established for the bus, although they are still subject to review. Where dimensions are not available, the dimensions of the American design vehicle have been adopted.

The vehicle most frequently employed in the design of rural roads is the single unit truck.

2.2.1 Dimensions

The dimensions adopted for the various design vehicles are given in Table 2.2.1.

TABLE 2.2.1.
DIMENSIONS OF DESIGN VEHICLES (m)

VEHICLE	WHEEL BASE	FRONT OVERHANG	REAR OVERHANG	WIDTH
Passenger car (P)	2,85	0,75	1,20	1,80
Single unit (SU)	6,10	1,22	1,83	2,60
Single unit + trailer (SU+T)	6,7 +3,4 +6,1			
Single unit bus (BUS)	6,00	2,50	3,50	2,60
Articulated bus (ABUS)	5,49 +7,32	2,44	3,05	2,60
Semi-trailer (WB-50)	6,10 +9,15	0,92	0,61	2,60

2.2.2 Templates

Templates are considered useful for establishing the layout of intersections and median openings, and their use is recommended. Once roadway edges have been established, it is further recommended that they should, for ease of construction, be approximated by simple or compound curves. Figures 2.2.2(a) and (b) give dimensions for the construction of templates for single units and single units + trailers.

2.2.3 Minimum turning radius

In constricted situations where the templates are not appropriate, the capabilities of the design vehicle become critical. Minimum turning radii for the outer side of the vehicle are given in Table 2.2.3. It is stressed that these radii are appropriate only to crawl speeds.

TABLE 2.2.3
MINIMUM TURNING RADII (m)

VEHICLE	MINIMUM TURNING RADIUS
Passenger car	6,20
Single unit	12,81
Single unit plus trailer	14,00
Single unit bus	13,10
Articulated bus	11,59
Semi-trailer	13,72

2.2.4 Performance on grade

Truck speeds on various grades have been the subject of much study under South African conditions, and it has been found that performance is not significantly affected by height above sea-level. Performance can therefore be represented by a single family of curves calculated on the basis of a mass/power ratio of 275 kg/kW and as shown in Figure 2.2.4.

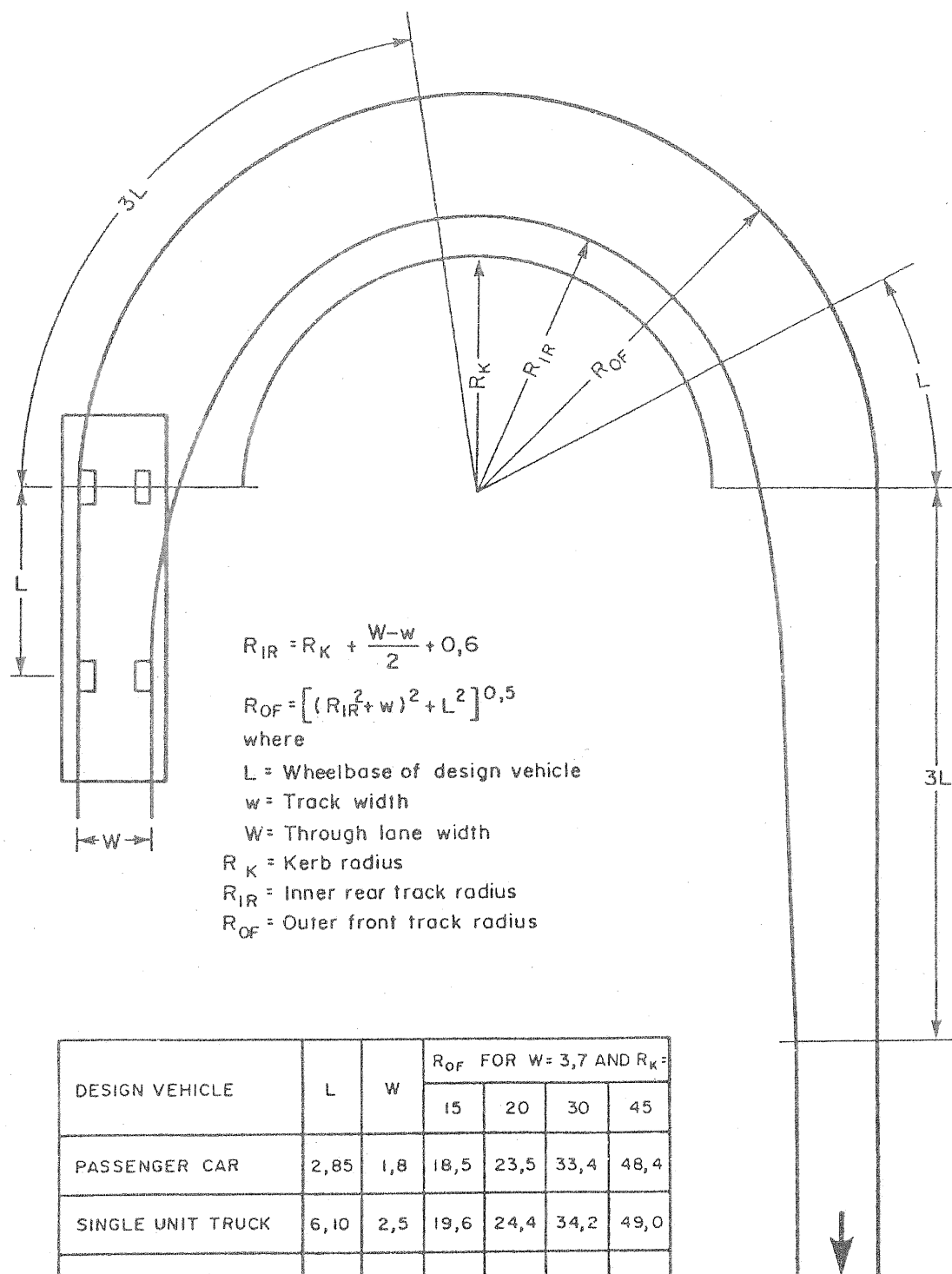
2.3 The driver

2.3.1 Eye height

Research has indicated that 95 per cent of passenger car drivers have an eye height at or above 1,05 m, and 95 per cent of truck drivers an eye height of 1,8 m or more. These values have accordingly been adopted for use in these guidelines.

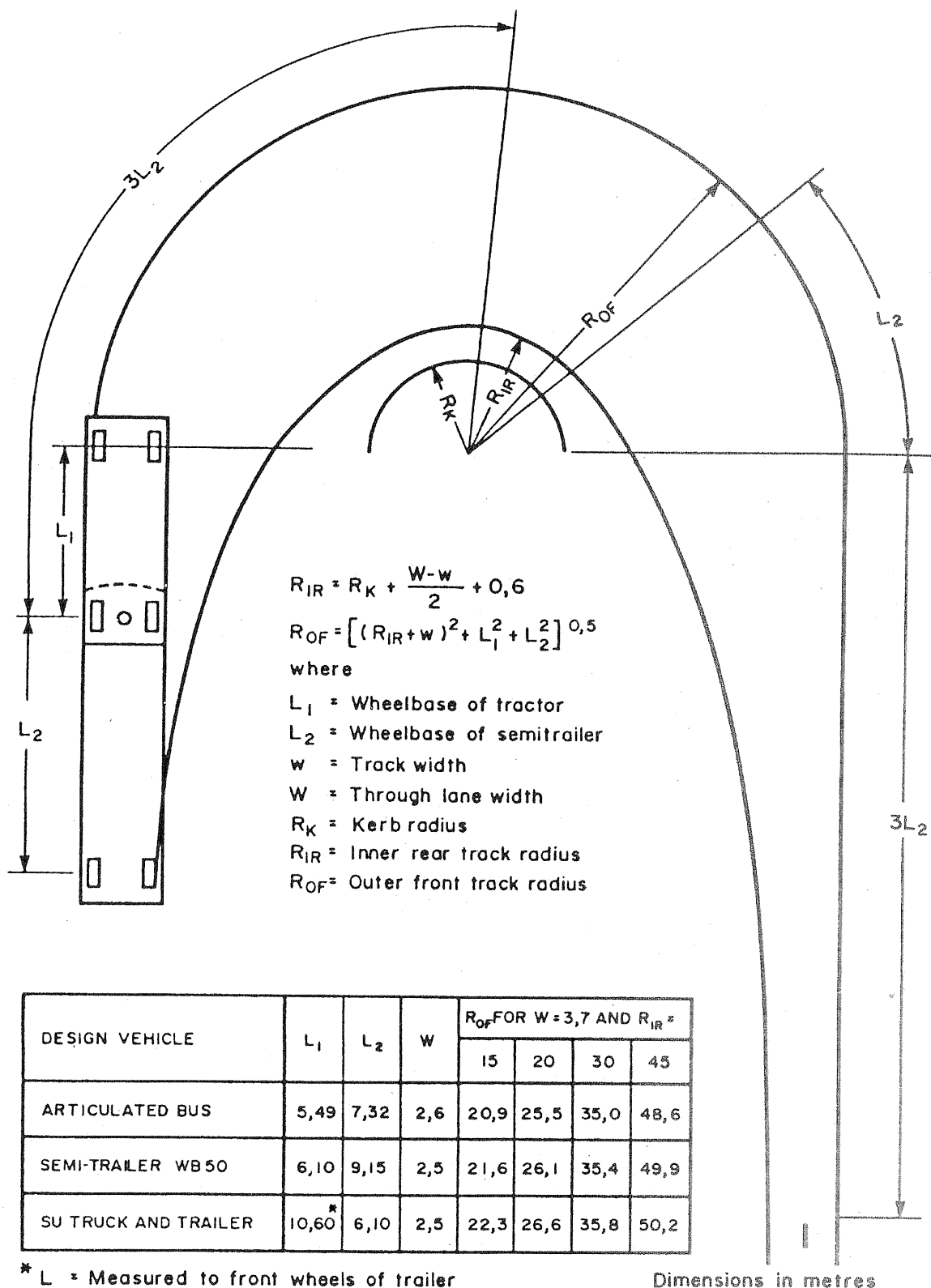
2.3.2 Reaction time

A figure of 2,5 seconds has been generally adopted for reaction time for response to a single stimulus. American practice also makes provision for a reaction time of 5,7 to 10,0 seconds for more complex multiple-choice situations, where more than one external circumstance must be evaluated, and the most appropriate response selected and initiated.

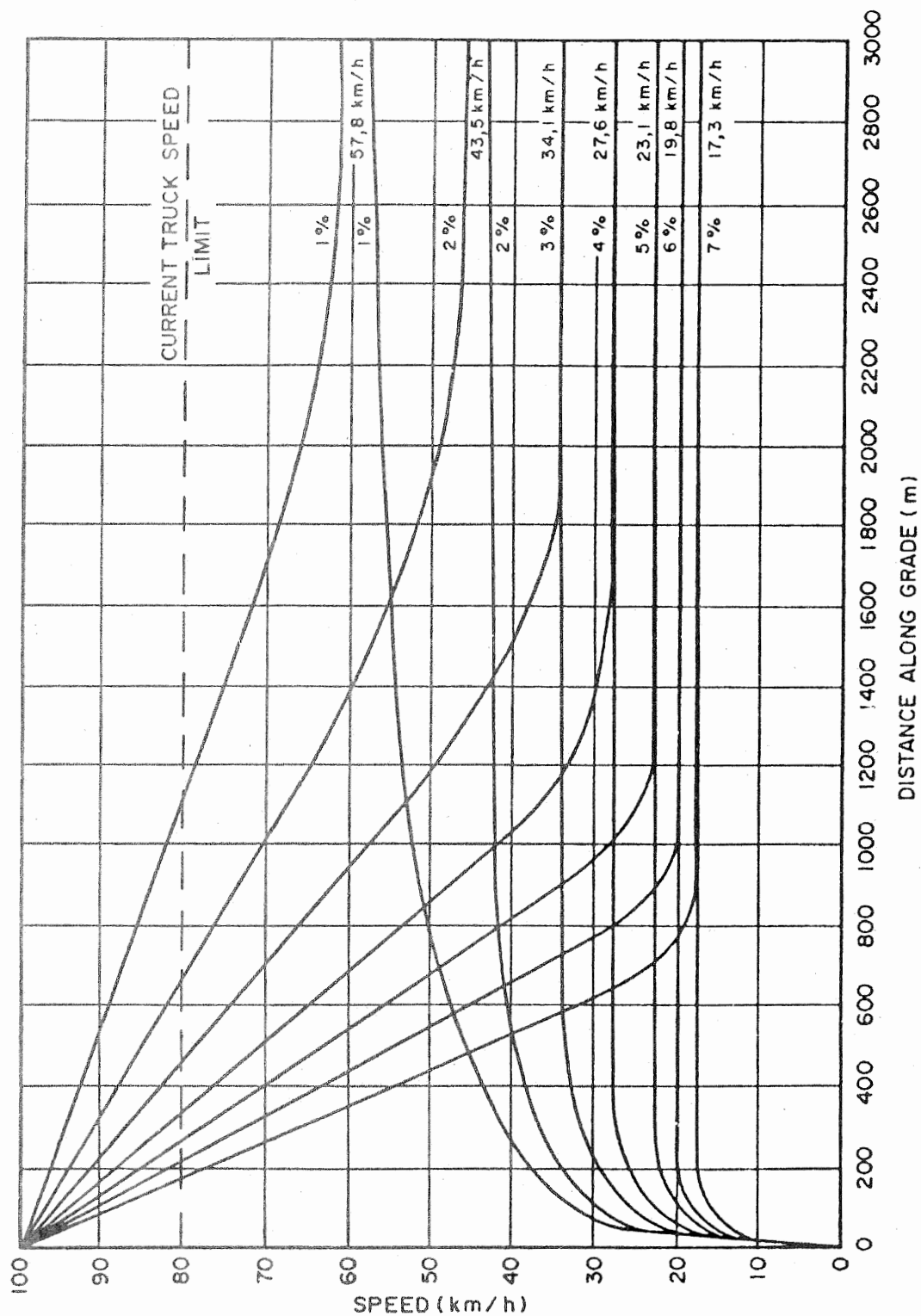


Dimensions in metres

WHEEL TRACKS OF RIGID CHASSIS VEHICLES



WHEEL TRACKS OF ARTICULATED VEHICLES



NOTE: BASED ON MASS / POWER RATIO OF 275 kg / kW

TRUCK SPEEDS ON GRADE

2.4 The road surface

The road surface has numerous qualities which can affect the driver's perception of the situation ahead of him, but skid resistance is the only one of these qualities taken into account in these guidelines.

2.4.1 Skid resistance

Skid resistance has been the subject of research worldwide, and it has been locally established that the derived values of brake force coefficient are appropriate to the South African environment. There is a considerable range of values. At 50 km/h the skid resistance of a worn tyre on a smooth surface is half that of a new tyre on a rough surface, and at 100 km/h it is five times lower. Skid resistance also depends on speed, and reduces as speed increases.

The values adopted for design in these guidelines are conservative, and the speed used in the calculation of guideline values is the operating speed, generally 80 to 85 per cent of design speed.

Brake force coefficients are given in Table 2.4.1. No allowance is made for a safety factor, as these represent actually measured values for a worn tyre on a smooth wet surface, which in engineering terms constitutes a 'worst case'. Furthermore, the coefficient of friction is lower in sliding than in rolling, so that, as long as the driver is not involved in an emergency situation, he has adequate distance for a stop under normal conditions.

TABLE 2.4.1
BRAKE FORCE COEFFICIENTS FOR VARIOUS SPEEDS

SPEED (km/h)	BRAKE FORCE COEFFICIENT
40	0,37
60	0,32
80	0,30
100	0,29
120	0,28

2.5 Sight distance

Sight distance is a fundamental criterion in the design of any road, be it urban or rural. It is essential for the driver to be able to perceive hazards on the road, with sufficient time in hand to initiate any necessary evasive action safely. On a two-lane two-way road it is also necessary for him to be able safely to enter the opposing lane safely while overtaking. In intersection design, the application of sight distance is slightly different from its application in design for the open road, but safety is always the chief consideration.

2.5.1 Stopping sight distance (SSD)

Stopping distance involves the capability of the driver to bring his vehicle safely to a standstill, and is thus based on speed, driver reaction time and skid resis-

tance. The total distance travelled in bringing the vehicle to a stop comprises two components:

- the distance covered during the driver's reaction period,
- the distance required to decelerate to 0 km/h.

The stopping distance is expressed as

$$s = 0,694 v + v^2/254 f$$

where s = total distance travelled (m)

v = speed (km/h)

f = brake force coefficient

Stopping sight distances based on operating speeds and the appropriate brake force coefficients have been adopted for design, and are given in Table 2.5.1.

TABLE 2.5.1
STOPPING SIGHT DISTANCE ON LEVEL ROADS

DESIGN SPEED (km/h)	STOPPING SIGHT DISTANCE (m)
40	50
50	65
60	80
70	95
80	115
90	135
100	155
110	180
120	210
130	230
140	255

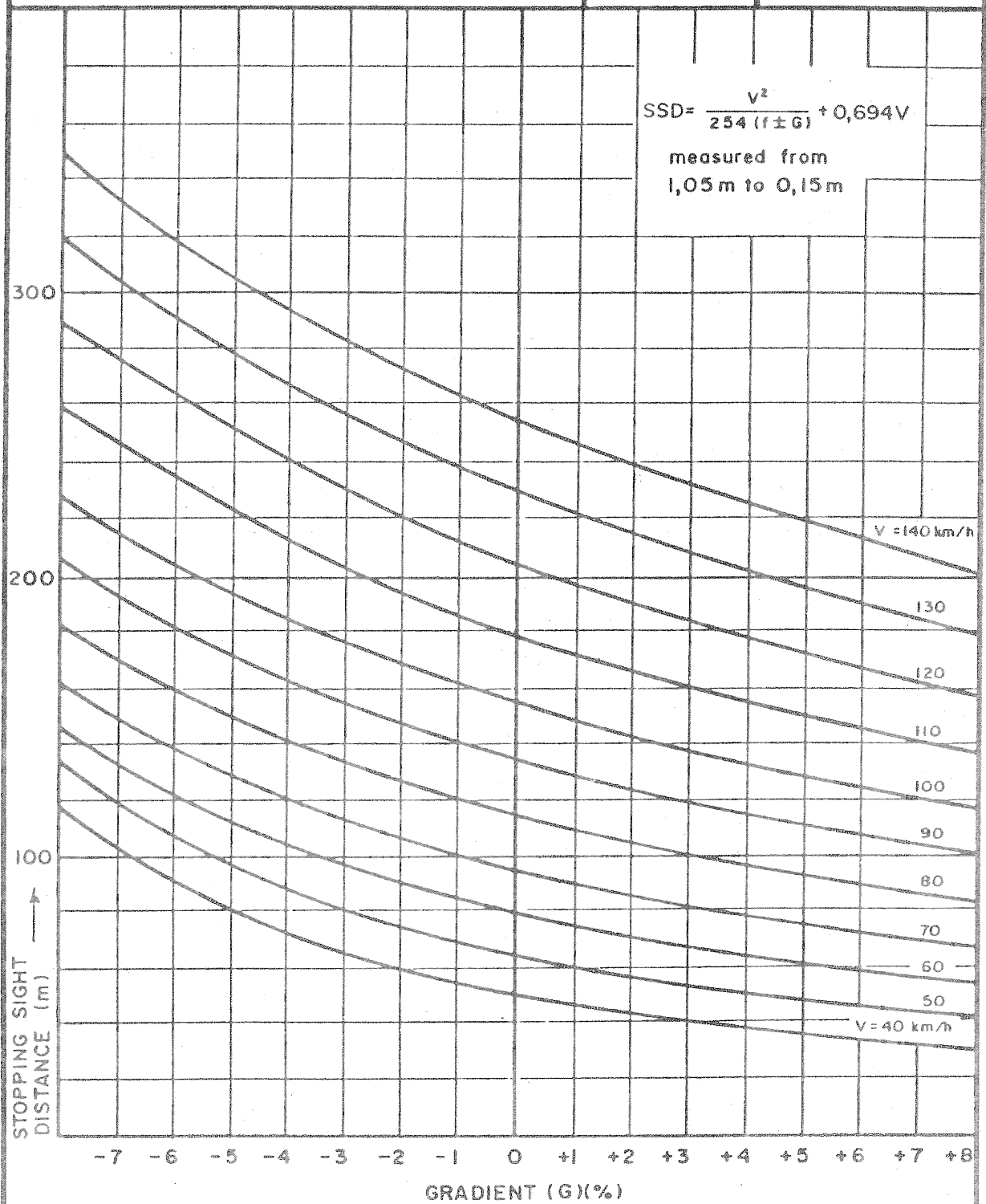
Stopping sight distance is measured from an eye height of 1,05 m to an object height of 0,15m. This object height is used because an obstacle of a lower height would not normally represent a significant hazard. Object height is taken into account because measuring the sight distance to the road surface would substantially increase the length of the vertical curve and hence the earthworks required.

The gradient has a marked effect on the stopping distance requirements, and Figure 2.5.1(a) is an expansion of Table 2.5.1 demonstrating this effect.

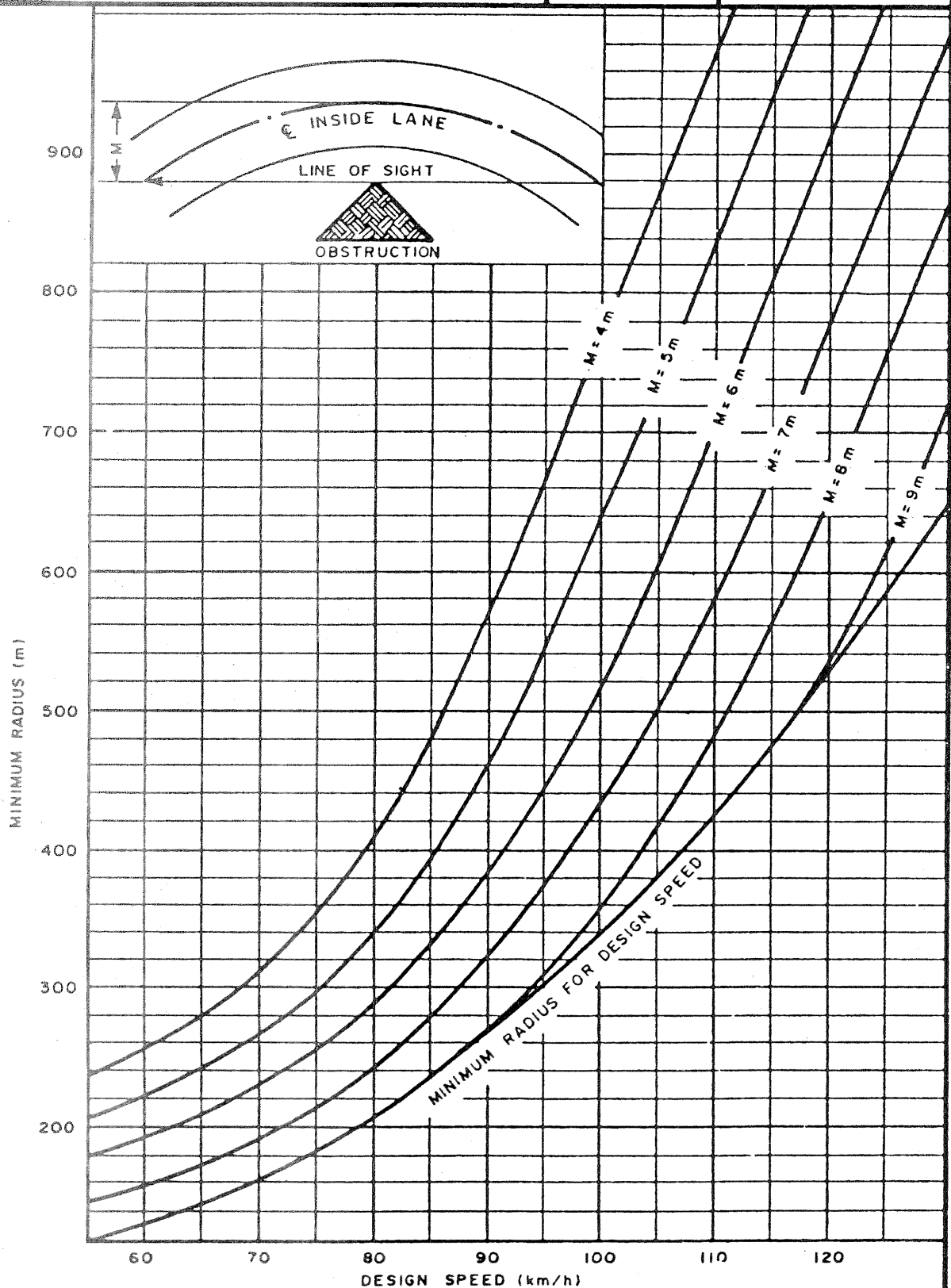
Stopping sight distance can also be affected by a visual obstruction (such as a cut slope) next to the carriageway on the inside of a horizontal curve, as shown in Figure 2.5.1(b).

2.5.2 Barrier sight distance (BSD)

Barrier sight distance is the limit below which overtaking is legally prohibited. Two opposing vehicles travelling in the same lane should be able to come to a standstill before impact. A logical basis for the determination of the barrier sight



STOPPING SIGHT DISTANCE ON GRADE



HORIZONTAL RADIUS FOR STOPPING SIGHT DISTANCE

distance is therefore that it should equal twice the stopping distance. The values given in the South African Road Traffic Signs Manual² approximate this approach, and are given in Table 2.5.2 for ease of reference.

TABLE 2.5.2
BARRIER SIGHT DISTANCE

DESIGN SPEED (km/h)	BARRIER SIGHT DISTANCE (m)
50	150
60	180
80	250
100	300
120	400

Barrier sight distance is measured to an object height of 1,3m, eye height remaining unaltered at 1,05m. The greater object height is realistic because it represents the height of a low approaching vehicle.

Hidden dip alignments are commonly accepted as poor design practice, but are still found on many rural roads. They typically mislead drivers into believing that there is more sight distance available than actually exists. In checking the alignment in terms of barrier sight distance, the designer must pay detailed attention to areas where this form of alignment occurs, to ensure that drivers are made aware of any inadequacies of design.

2.5.3 Decision sight distance (DSD)

The best visual cue to the driver is the roadway ahead. For this reason it is necessary in certain circumstances for the road surface itself to be visible to the driver for a given distance ahead, to allow sufficient time for the assimilation of a message and the safe initiation of any action required. An example is the gore markings at the nose of an off-ramp. A further example is the marking allocating specific lanes at an intersection for turning, where warning of this must be given sufficiently far in advance of the intersection to permit a lane change that does not detrimentally affect the operation of the intersection itself.

Decision sight distance, as given in Table 2.5.3, is related to the reaction time involved in a complex driving task. The reaction time selected for this purpose is 7,5 seconds, which is roughly the mean of values quoted in American practice. The calculated value in Table 2.5.3 is thus based on stopping sight distance to allow for the condition where the decision is to bring the vehicle to rest. This has the effect of increasing the normal reaction time of 2,5 seconds by a further 5 seconds of travel at the design speed of the road. Decision sight distance is measured from an eye height of 1,05 m to the road surface, ie to an object height of 0 m.

TABLE 2.5.3
DECISION SIGHT DISTANCE ON LEVEL ROADS

DESIGN SPEED (km/h)	DECISION SIGHT DISTANCE (m)
40	130
50	160
60	190
70	215
80	240
90	270
100	300
110	325
120	350
130	380
140	410

2.5.4 Passing sight distance (PSD)

Passing sight distance is seen as an important criterion indicative of the quality of service provided by the road. The initial road design would provide stopping sight distance over the full length of the road, with passing sight distance being checked afterwards. A heavily trafficked road requires a higher percentage of passing sight distance than a lightly trafficked road to provide the same level of service. Insufficient passing sight distance can be remedied, for example, either by lengthening a vertical curve to provide passing sight distance within the length of the curve itself, or by shortening the curve to extend the passing opportunities on either side of the curve, depending on the prevailing circumstances. Horizontal curves can similarly be lengthened or shortened.

Passing sight distance, as determined for South African conditions, is given in Table 2.5.4.

TABLE 2.5.4
PASSING SIGHT DISTANCE ON LEVEL ROADS

DESIGN SPEED (km/h)	PASSING SIGHT DISTANCE (m)
60	420
70	490
80	560
90	620
100	680
110	740
120	800

2.5.5 Shoulder sight distance

At a stop-controlled intersection, the driver of a stationary vehicle must be able to see enough of the major highway to be able to cross before an approaching vehicle reaches the intersection, even if this vehicle comes into view just as the stopped vehicle starts to cross.

The distance the crossing vehicle must travel is the sum of the distance from the stop line to the edge of the through carriageway, the width of the road being crossed and the length of the crossing vehicle. This manoeuvre must be completed in the time it takes the approaching vehicle to reach the intersection, assuming that the approaching vehicle is travelling at the design speed of the through road. For safety, the time available should also include allowance for the time it takes for the crossing driver to establish that it is safe to cross, engage gear and set his vehicle in motion; a period of about two seconds is normally used.

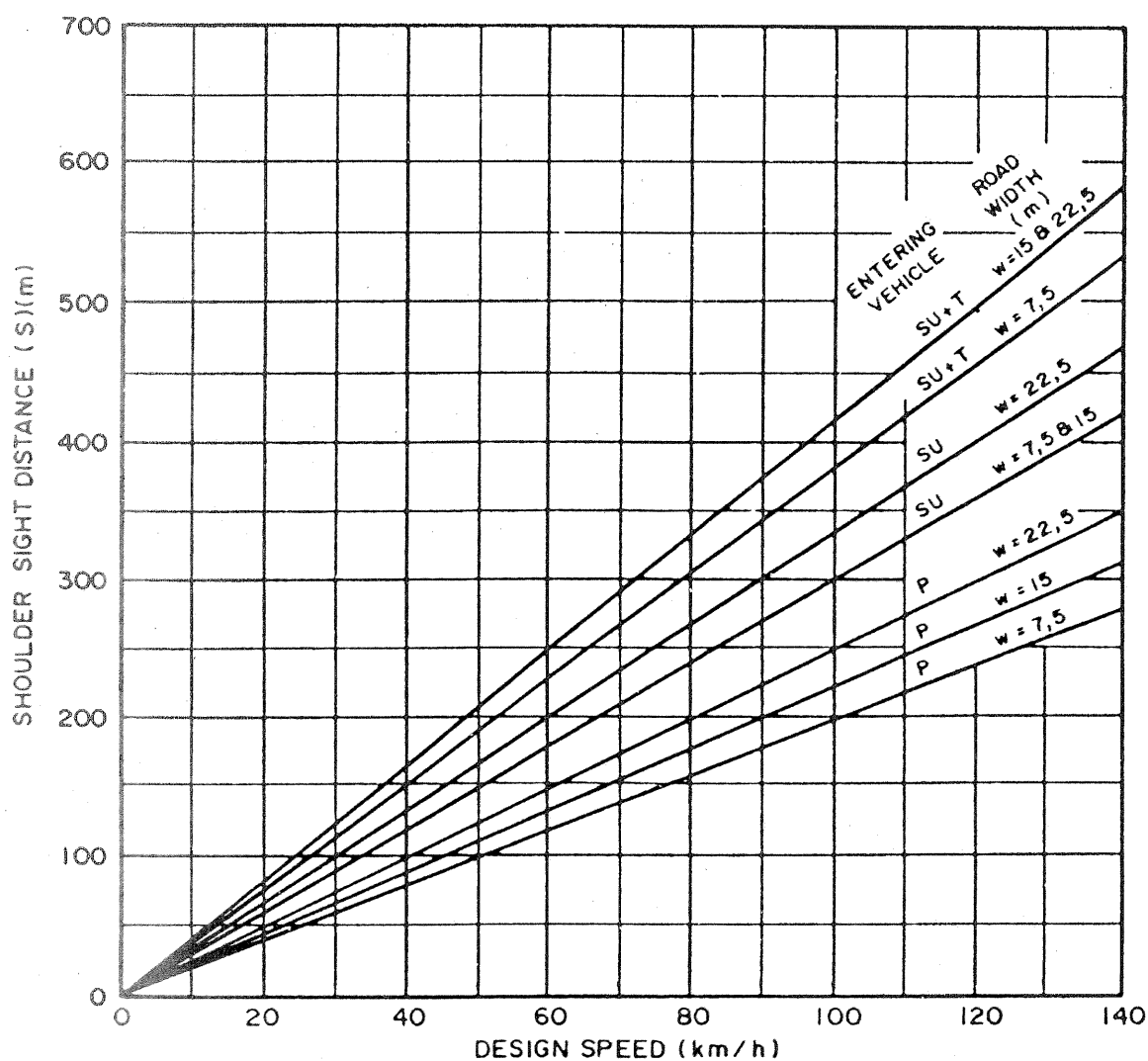
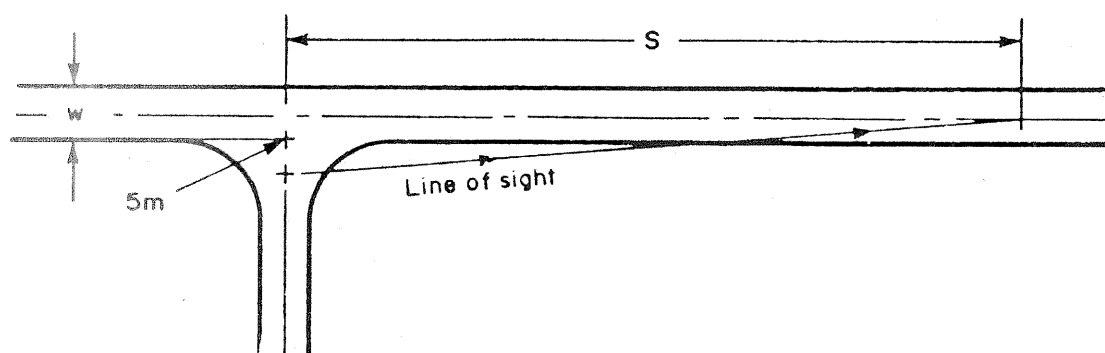
The line of sight is taken from a point on the centre line of the crossing road and 5 m back from the edge of the through road to a point on the centre line of the through road, as shown in Figure 2.5.5(a).

The object height is 1,3m. The eye height is 1,05 m for a passenger car and 1,8 m for all other design vehicles. There must be no obstruction to the view in the sight triangle, defined as the area enclosed by the sight line and the centre lines of the intersecting roads.

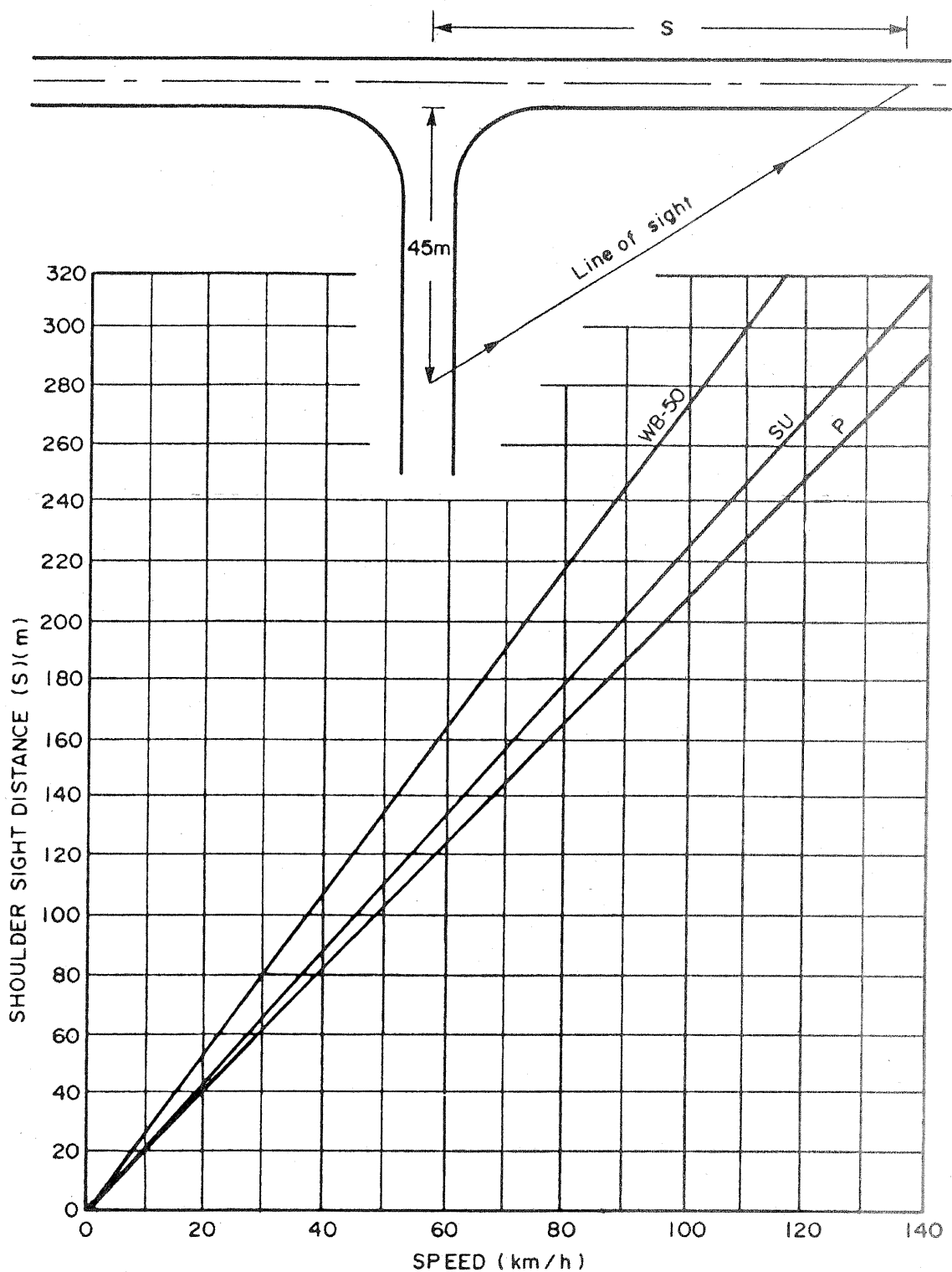
Shoulder sight distances, recommended in accordance with the principles outlined above, are also given in Figure 2.5.5(a). Before a lower value is adopted in a specific case, implications of departing from the recommended values should be studied.

Where an intersection is subject to yield control, the unobstructed sight triangle must be larger. It is assumed that the driver approaching the intersection on the minor leg will be travelling at 60 km/h and preparing to stop, in which case a distance of 45 m is required for him to bring his vehicle to a standstill. If he does not stop, but turns to travel in the same direction as a vehicle approaching at the design speed of the through road, the driver of the latter vehicle will be forced to slow down to match speeds at a safe following distance. The shoulder sight distance required for this manoeuvre is shown in Figure 2.5.5(b).

Because the driver approaching the yield sign may be required to stop, shoulder sight distance as defined and measured for the stop condition must also be available.



SHOULDER SIGHT DISTANCE FOR STOP CONDITION



SHOULDER SIGHT DISTANCE FOR YIELD CONDITION

3. HORIZONTAL ALIGNMENT

3.1 Introduction

The ease, comfort and safety of operation of a vehicle on a road are determined by consistency of design, among other things. This consistency is achieved partly by relating the magnitude of successive elements of horizontal and vertical alignment to a speed. Although these elements are subject to the laws of mechanics, it has been found in practice that the distribution of speed across the vehicle population and the variations in vehicle characteristics sometimes make it necessary to depart from theoretically calculated values. It has also been found that various combinations of elements are aesthetically unpleasing, so that the appearance of the road spoils the general environment.

This chapter details radii of horizontal curvature appropriate to various design speeds, discusses superelevation, and makes suggestions on the aesthetic influence of variations in horizontal alignment.

3.2 Curvature

Attempts have been made from time to time to introduce various forms of curvilinear alignment. These have included a process of fitting curves into the topography and then connecting them with relatively short tangent sections. It has also been suggested that circular curves could be replaced by higher order polynomials. For the purposes of these guidelines it is assumed that in the process of route location the location of a series of tangents is done first, followed by the selection of curvature.

3.2.1 Minimum radii

The minimum radius is a limiting value for a given design speed, and is determined from the maximum rate of superelevation and the maximum allowable side friction factor (see Subsection 3.5). Minimum radii for the various design speeds are given in Table 3.2.1 below.

In general, these radii should be used only under the most critical conditions. The deviation angle of each curve should be as small as the physical conditions permit, so that the road is as directional as possible. This deviation should be absorbed in the flattest possible curve so that passing opportunities will not be unduly restricted. It should be borne in mind however, that excessively long curves may generate operational problems as discussed below.

3.2.2 Minimum length of curve

For small deflection angles, curves should be long enough to avoid the appearance of a kink. A minimum length of 300 m is suggested. If space is limited, this length can be reduced to 150 m. For deflection angles of less than 5°, the minimum length of the curve should be increased 150 m by 30 m for each 1° decrease in the deflection angle.

TABLE 3.2.1
MINIMUM RADII OF HORIZONTAL CURVATURE

DESIGN SPEED (km/h)	RADIUS (m)
50	80
60	110
70	160
80	210
90	270
100	350
110	430
120	530
130	640
140	760

3.2.3 Maximum length of curve

A long curve, particularly if it is of near-minimum radius, may cause tracking problems. These are suffered principally by vehicles travelling at speeds markedly different from the design speed of the road.

The chief complication introduced by the long curve is, however, its possible effect on passing opportunities. It should be noted that on a two-lane road the principal obstruction to sight distance is, in fact, the vehicle to be overtaken. On a left-hand curve, the overtaking manoeuvre would thus have to commence at a considerable distance behind the leading vehicle. On a right-hand curve, sight distance is less of a problem than on a tangent section. Apart from having the road in view without having to move out from behind the leading vehicle, the presence of superelevation favours an increase in available sight distance. Furthermore the distance to be traversed during the overtaking manoeuvre on a left-hand curve (an outside path) is greater than on a right-hand (an inside path), apart from the need for earlier commencement of the overtaking manoeuvre.

Consequently the length of a curve should preferably not exceed 1000m.

3.3 Successive curves

The process of locating a road as a series of tangents tends to cause curves to be relatively far from each other, so that each curve is an independent feature. As the topography becomes more rugged, the intervals between curves shorten, until the stage is reached when successive curves can no longer be dealt with in isolation. Three cases must be considered. These are

- the curve followed by a curve in the opposite direction (reverse curve),
- the curve followed by a curve in the same direction (broken-back or flat-back), and
- the compound curve which, like the broken-back curve, consists of successive curves in the same direction, but does not have an intervening tangent.

3.3.1 Reverse curves

Any abrupt reversal in alignment should be avoided. Such a change makes it difficult for the driver to keep within his own lane. It is also difficult to superelevate both curves adequately, and erratic operation may result. A reversal of alignment should therefore include a length of connecting tangent, or, preferably, a section of equal length including spiral curves. The distance between circular curves is dictated by the requirements of superelevation development. Figure 3.3.1 shows a treatment of reverse curvature which is not only aesthetically pleasing, but also has the advantage that the connecting tangent can be made shorter than dictated by the inclusion of a cambered section.

3.3.2 Broken-back curves

The broken-back curve is generally considered undesirable, since drivers do not expect successive curves to be in the same direction. The preponderance of successive curves in opposite directions has developed in drivers a subconscious habit of following them.

Operational problems introduced by an unexpected camber between the curves, or drainage problems caused by flat spots on the road surface, are further undesirable aspects of the broken-back curve, apart from its unpleasing appearance.

Broken-back curves cannot always be avoided, and it is suggested that the connecting tangent should be at least 150m long. It is also suggested that the tangent should have a single crossfall, rather than reverting to a normal camber for a short distance. This treatment of the broken-back curve is illustrated in Figure 3.3.2.

The term 'broken-back' is not usually employed when the connecting tangent is 500m or more in length.

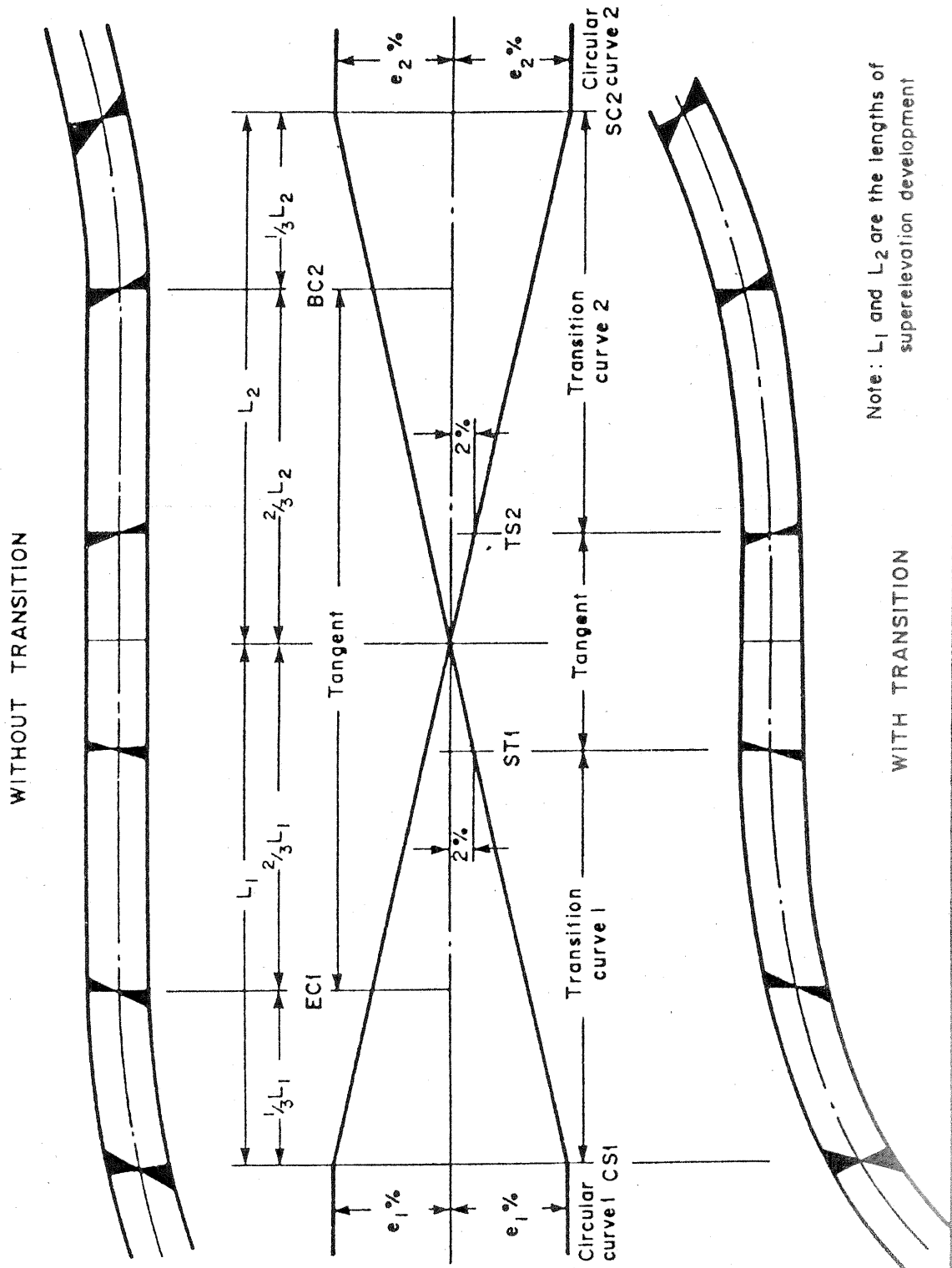
3.3.3 Compound curves

The use of compound curves affords flexibility in fitting the road to the terrain and other controls, and the simplicity with which such curves can be used may tempt the designer to use them without restraint. Caution should however be exercised in the use of compound curves, because, with the possible exception of the interchange loop, the driver does not expect to be confronted by a change in radius once he has entered a curve. Their use should also be avoided where curves are sharp.

Compound curves with large differences in curvature introduce the same problems as are found at the transition from a tangent to a small-radius curve. Where the use of compound curves cannot be avoided, the radius of the flatter circular arc should not be more than 50 per cent greater than the radius of the sharper arc, ie R_1 should not exceed $1,5 R_2$. A several-step compound arc on this basis is suitable as a form of transition from either a flat curve or a tangent to a sharper curve, although a spiral is to be preferred.

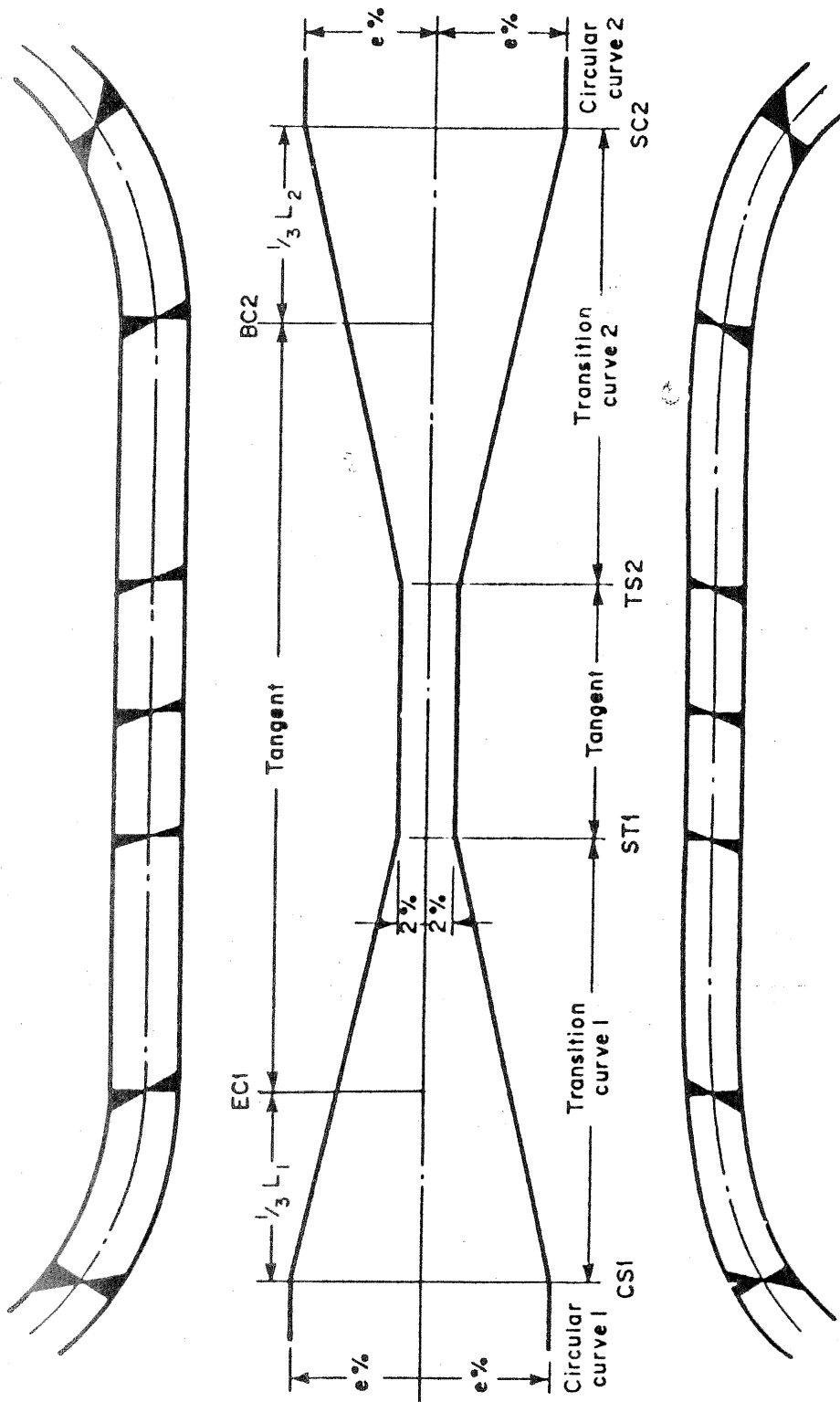
3.4 Transition curves

Any vehicle follows a transitional path as it enters or leaves a circular horizontal curve. In most cases this path is contained within the limits of normal lane width.



SUPERELEVATION OF REVERSE CURVES

WITHOUT TRANSITION



Note : L_1 and L_2 are the lengths of
superelevation development

WITH TRANSITION

SUPERELEVATION OF BROKEN-BACK CURVES

With combinations of high speed and sharp curvature, the resultant longer transition may encroach on an adjoining lane. The principal advantage of a transitioned lane is that it gives the driver a natural path that is easy to follow, and at the same time provides a suitable arrangement for superelevation run-off. The appearance of the road is also enhanced by the use of spirals, since this avoids noticeable breaks at the beginning of circular curves, these breaks often being made more pronounced by the superelevation run-off.

Transition curves are recommended for use where the associated circular arc is to have a superelevation of 60 per cent or more of the maximum. The form recommended is the spiral, and the length of the spiral is equal to the length required for the development of superelevation from the point where a crossfall equal to normal camber has been achieved, ie from reverse camber (RC).

3.5 Superelevation

A vehicle following a circular path is forced outwards by centrifugal force. This force is counterbalanced by the vehicle weight component caused by superelevation, e , or the side friction, f , developed between the tyre and the road surface or a combination of the two. This combination is given in the expression

$$e + f = V^2/127 R$$

where V = design speed (km/h)
 R = radius (m)

Using the maximum rate of superelevation and side friction, the minimum radius of curvature for given speed can be determined. Minimum radii calculated on this basis and rounded off for design are given in Table 3.2.1.

3.5.1 Maximum rates of superelevation and sideways force coefficient

The maximum rate of superelevation used in the design of South African rural roads is 10 per cent.

The maximum sideways force coefficient accepted for design purposes is expressed as

$$f_{\max} = 0,19 - V/1\ 600$$

where V = design speed (km/h)

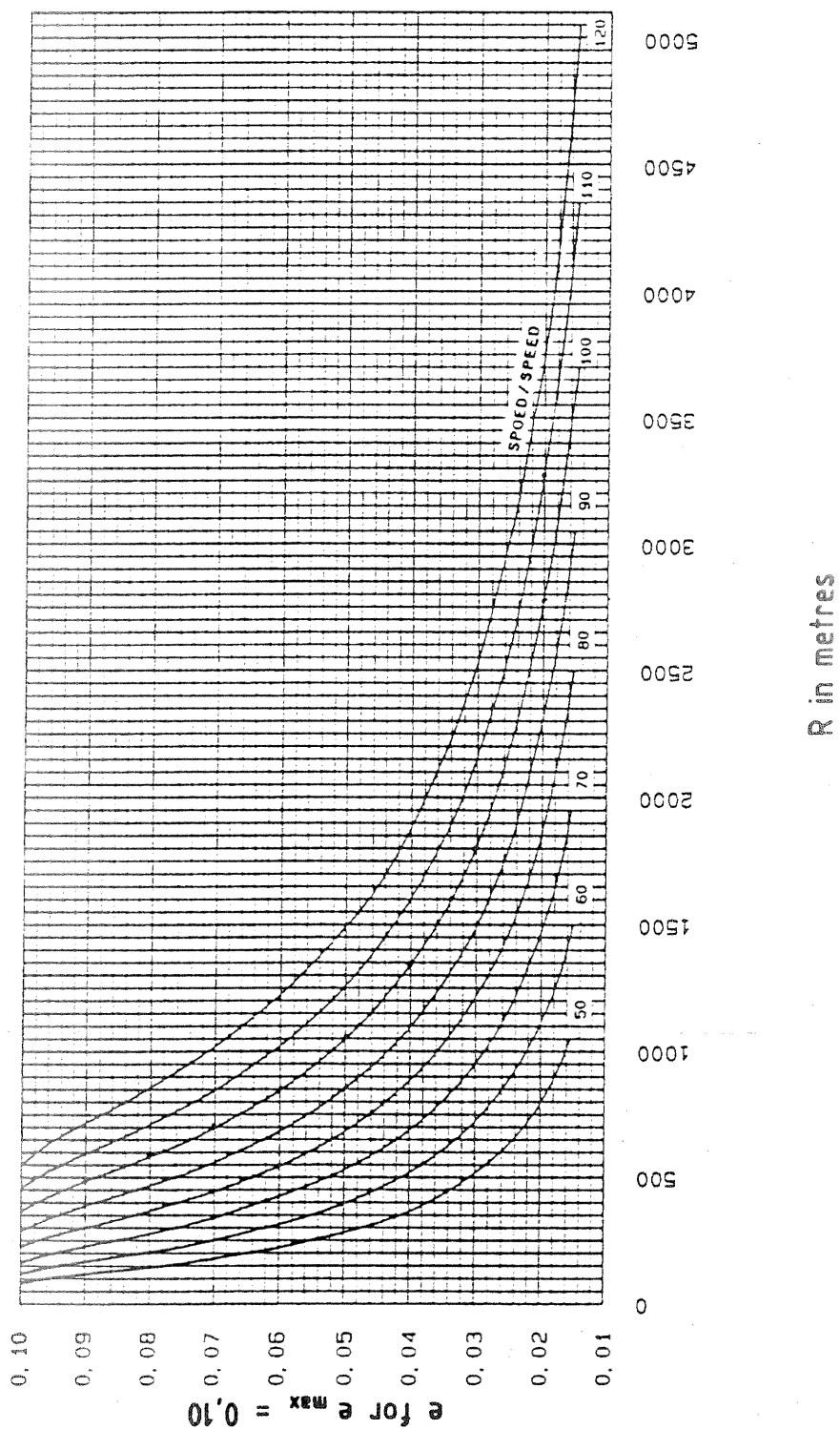
representing a safety factor of approximately 3.

3.5.2 Design superelevation rates

There are five recognized methods of distributing superelevation, e , and sideways force coefficient, f . The method adopted in South Africa is based on using superelevation to balance all the centrifugal force generated at the average running speed, with side friction balancing additional centrifugal forces generated at higher speeds. Figure 3.5.2 shows rates of superelevation appropriate to above-minimum radii of curvature for the various design speeds.

3.5.3 Run-off

Superelevation run-off is the term generally used to denote the length of road needed to accomplish the change in cross-slope from a fully superelevated section to a section with the adverse camber removed. Crown run-off means the



SUPERELEVATION RATES

distance from this section to a normally cambered section. In current design practice the appearance of the superelevation run-off largely governs its length. A too-rapid rotation gives the road surface a warped appearance and breaks the smooth three-dimensional flow of the road edge, which is probably the most notable feature of the road to the driver.

a) Rate of rotation

The rate of rotation is measured by the relative slope between the carriageway edge and the axis of rotation. The slope factors quoted in Table 3.5.3(a) have been found in practice to give acceptable lengths of run-off.

TABLE 3.5.3(a)
RELATIVE SLOPE FACTOR

DESIGN SPEED (km/h)	RELATIVE SLOPE FACTOR
40	140
60	170
80	200
100	230
120	260

b) Calculation of run-off length

The run-off length is calculated as the difference in height between the fully superelevated carriageway edge and the axis of rotation divided by the relative slope between them. The slope factors given in Table 3.5.3(a) are the reciprocals of the slopes, and the calculated length of run-off is

$$L = \frac{w.e.s.l}{100}$$

where L = length of superelevation run-off (m)
w = lane width (m)
e = superelevation
s = relative slope factor
l = lane factor

Crown run-off is calculated in the same way, with the superelevation replaced by the normal camber, usually 2 per cent.

c) Minimum run-off length

Minimum lengths for superelevation run-off are given in Table 3.5.3(b) for two-lane roads. While no maximum lengths are suggested, too long a run-off might cause drainage problems at the commencement of the run-off section.

TABLE 3.5.3(b)**MINIMUM LENGTH OF SUPERELEVATION RUN-OFF FOR TWO-LANE ROADS**

DESIGN SPEED (km/h)	RUN-OFF (m)
60	40
80	50
100	60
120	70

The length of run-off for surfaces wider than one lane is subject to the same considerations as apply to the two-lane road rotated about its centre line. On this basis, the length of run-off for four-lane would be twice that for two-lane roads, and for six-lane roads the length of run-off would be three times as long. Often, however, it is not feasible to provide lengths based on such direct ratios, although it is generally agreed that superelevation run-off lengths should be greater for roads wider than two lanes. On a purely empirical basis it is concluded that minimum design superelevation lengths for wider pavements should be calculated by use of the lane factors given in Table 3.5.3(c).

In the case of a divided road, the median is disregarded, in selection of the lane factor, if narrower than 4,6 m. If the median is wider than 12,2 m, the two carriageways are regarded as separate entities. For intermediate widths of median either the higher or the lower lane factor may be selected.

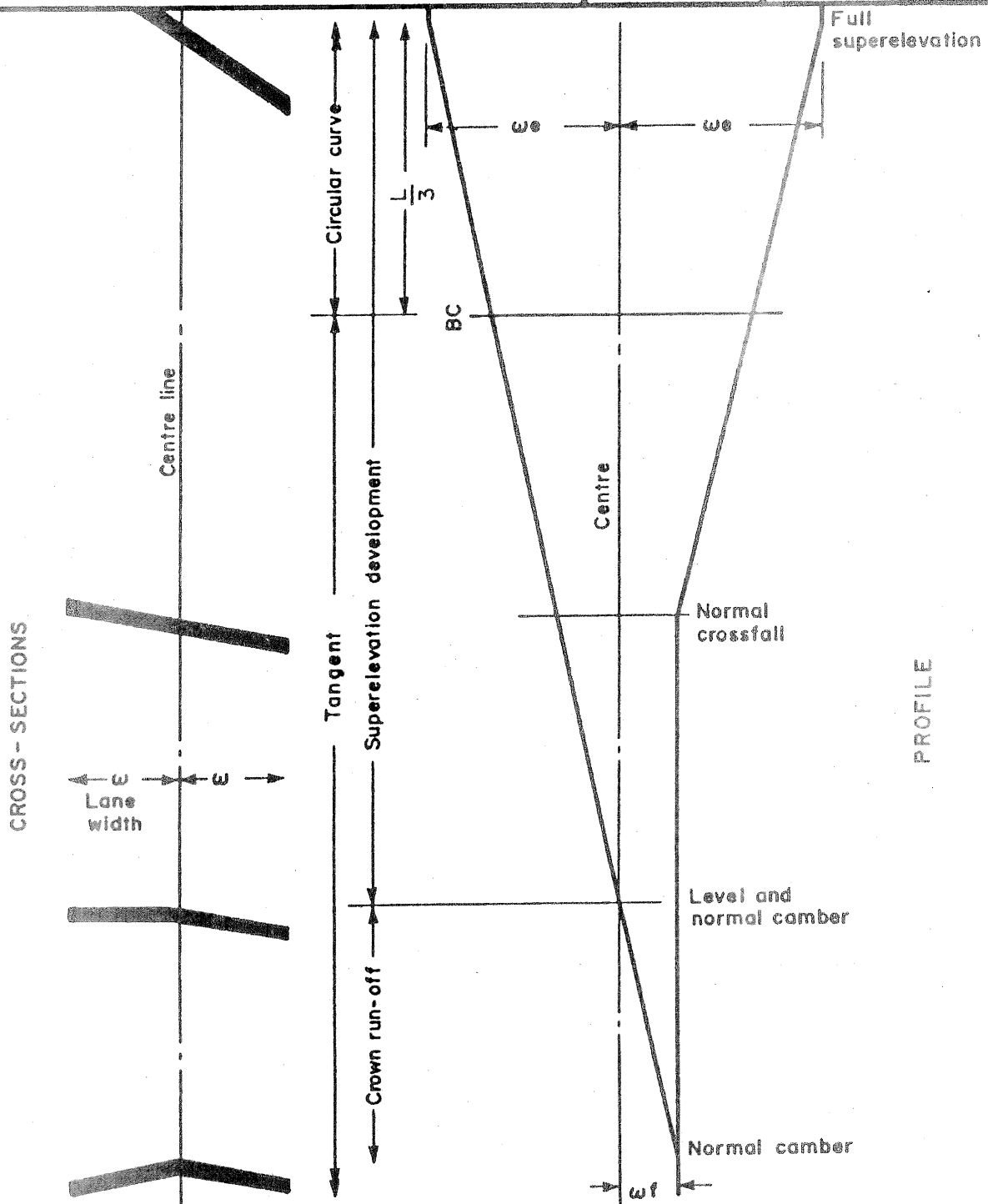
d) Location relative to curve

Where a circular arc is preceded by a transition curve, the full superelevation will be achieved at the start of the circular arc. As suggested previously, the length of the transition will be sufficient to contain the superelevation run-off from the point where the road reaches a crossfall equal to the normal camber. The rest of the superelevation run-off and the crown run-off occur on the tangent preceding the transition curve. Where no transition curve has been provided the superelevation development must be distributed between the tangent and the curve, because full superelevation at the end of a tangent is as undesirable as no superelevation at the start of a curve. The compromise generally employed is to have two-thirds of the superelevation run-off on the tangent, and one-third on the curve. The path of the vehicle, even if there is no transition curve, will be a spiral commencing before the start of the curve and ending some distance beyond it; this compromise to some extent matches the actual path of the vehicle.

The attainment of superelevation is illustrated in Figures 3.5.3(a) and 3.5.3(b).

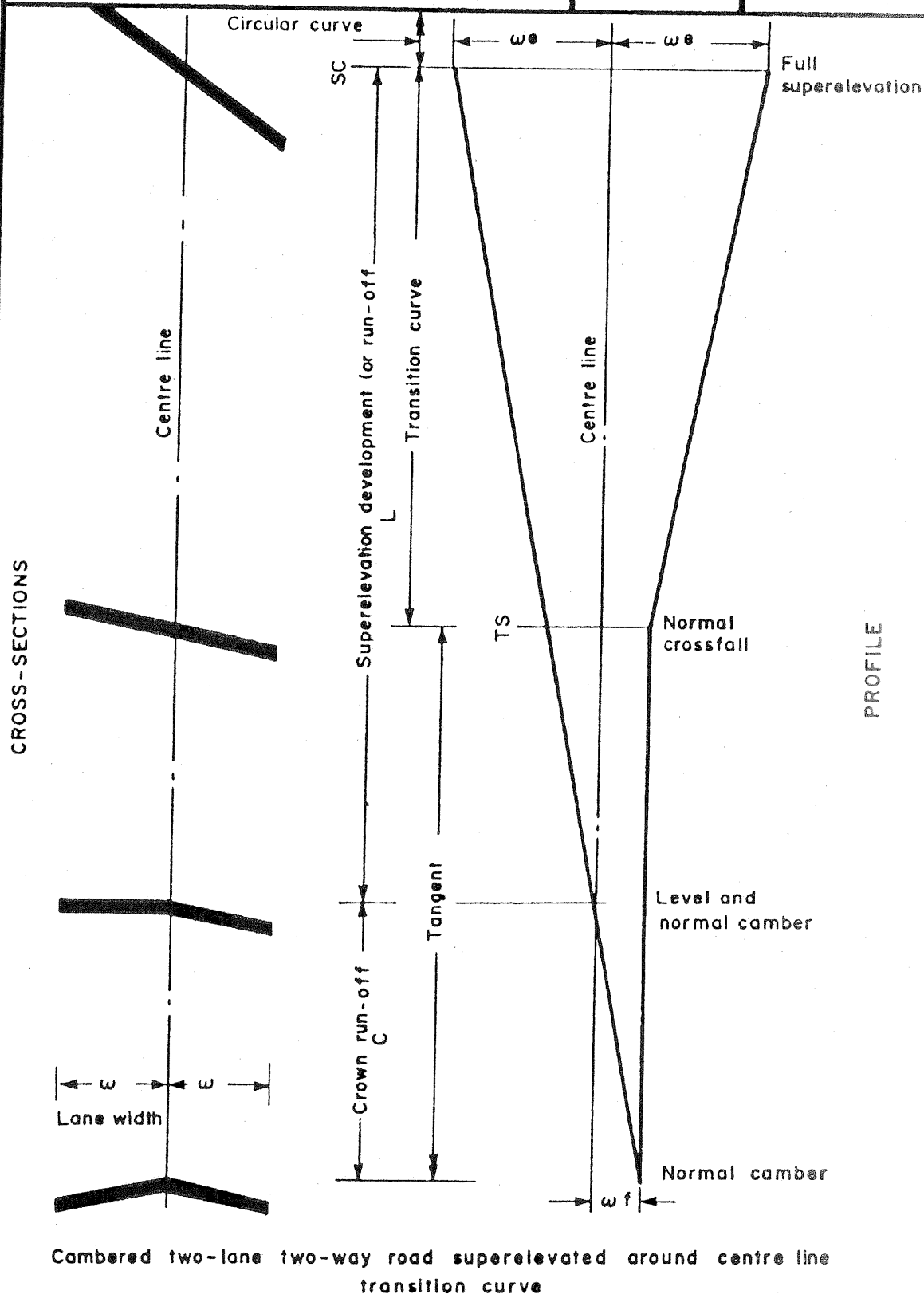
TABLE 3.5.3(c)
LANE FACTORS FOR SUPERELEVATION RUN-OFF

CROSS-SECTION	NUMBER OF LANES	LANE FACTOR	MEDIAN WIDTH (m)
Undivided	2	1,0	—
	3	1,2	—
	4	1,5	—
Divided	2	1,5	Less than 4,6 m
	3	2,0	
	2	1,0 or 1,5	Between 4,6 & 12,2 m
	3	1,2 or 2,0	
	2	1,0	Greater than 12,2 m
	3	1,2	



Cambered two-lane two-way road superelevated around centre line without transition curve

ATTAINMENT OF SUPERELEVATION WITHOUT TRANSITION



ATTAINMENT OF SUPERELEVATION WITH TRANSITION

4. VERTICAL ALIGNMENT

4.1 Introduction

Vertical alignment is the combination of parabolic vertical curves and tangent sections of a particular slope. The selection of rates of grade and lengths of vertical curves is based on assumptions about characteristics of the driver, the vehicle and the roadway. Vertical curvature may impose limitations on sight distance, particularly when combined with horizontal curvature. The slope of tangent sections introduces forces which affect vehicle speed, driver comfort and the ability to accelerate and decelerate.

With the whole-life economy of the road in mind, vertical alignment should always be designed to as high a standard as is consistent with the topography.

The vertical alignment should also be designed to be aesthetically pleasing. In this regard due recognition should be given to the inter relationship between horizontal and vertical curvature. As a general guide, a vertical curve that coincides with a horizontal curve should, if possible, be contained within the horizontal curve, and should ideally have approximately the same length.

A smooth grade line with gradual changes appropriate to the class of road and the character of the topography is preferable to an alignment with numerous short lengths of grade and vertical curves. The 'roller coaster' or 'hidden dip' type of profile should be avoided. This profile is particularly misleading in terms of availability of sight distance and, where it cannot be avoided, greater sight distance than suggested in Table 2.5.1 may be required in terms of accident experience. A broken-back alignment is not desirable on aesthetic grounds in sags where a full view of the profile is possible. On crests the broken back adversely affects passing opportunity.

As long as the driver's line of sight is contained within the width of the roadway, the superelevation generated by horizontal curvature improves the availability of sight distance, even though the edge profiles may have a curvature sharper than the minimum suggested below. When the line of sight goes beyond the roadway edge, the effect on sight distance of lateral obstructions such as cut faces or high vegetation must be checked.

4.2 Curvature

From the general form of a parabolic function

$$y = ax^2 + bx + c$$

it follows that the rate of change of grade, $\frac{d^2y}{dx^2}$, equals $2a$. The reciprocal, K , is the distance required to effect a unit change of grade. Vertical curves are specified in terms of this factor, K , and their horizontal length calculated by multiplying K by the algebraic difference in percentage between the grades on either side of the curves.

4.2.1 Minimum rates of curvature

The minimum rate of curvature is determined by sight distance as well as by considerations of comfort of operation and aesthetics. The sight distance most

frequently employed is the stopping sight distance measured from an eye height of 1,05 m to an object height of 0,15 m, although special circumstances may dictate the use of decision sight distance or even passing sight distance. In the case of sag curves, the sight distance is replaced by a headlight illumination distance of the same magnitude, assuming a headlight height of 0,6 m and a divergence angle of 1° above the longitudinal axis of the headlights.

TABLE 4.2.1
MINIMUM VALUES OF K FOR VERTICAL CURVES

DESIGN SPEED (km/h)	CREST CURVES	SAG CURVES
40	6	8
50	11	12
60	16	16
70	23	20
80	33	25
90	46	31
100	60	36
110	81	43
120	110	52
130	133	57
140	163	64

Values of K, based on stopping sight distance in the case of crest curves, and headlight illumination distance in the case of sag curves, are given in Table 4.2.1.

4.2.2 Minimum lengths of vertical curves

Where the algebraic difference between successive grades is small, the intervening minimum vertical curve becomes very short, and, particularly where the tangents are long, this can create the impression of a kink in the grade line. Where the difference in grade is less than 0,5 per cent, the vertical curve is often omitted. For algebraic differences in grade greater than 0,5 per cent, a certain minimum length is suggested for purely aesthetic reasons. For freeways a minimum length of 240 m is recommended. The lengths suggested in Table 4.2.2 below apply to all roads other than freeways.

TABLE 4.2.2
MINIMUM LENGTHS OF VERTICAL CURVES

DESIGN SPEED (km/h)	LENGTH OF CURVE (m)
40	60
60	100
80	140
100	180
120	220
140	260

4.3 Gradients

4.3.1 Maximum gradients

The speed of passenger cars is relatively unaffected by gradient, and the horizontal alignment will tend to govern the selection of speed. Truck speeds are however markedly affected by gradient. The design should therefore aim at grades which will not reduce the speed of heavy vehicles enough to cause intolerable conditions for following drivers. Overseas experience has indicated that the frequency of truck accidents increases sharply when truck speed is reduced by more than 15 km/h, and for South African conditions a speed reduction of 20 km/h has provisionally been accepted as representing intolerable conditions. If grades on which the truck speed reduction is less than 20 km/h cannot be achieved economically, it may be necessary to provide auxiliary lanes for the slower-moving vehicles. It has been established that on flat grades truck speeds are about 17 km/h lower than passenger car speeds, so that a speed reduction of 20 km/h actually represents a total difference in speed between trucks and passenger cars of about 37 km/h.

TABLE 4.3.1
MAXIMUM GRADIENTS

DESIGN SPEED (km/h)	TOPOGRAPHY		
	FLAT	ROLLING	MOUNTAINOUS
60	6%	7%	8%
80	5%	6%	7%
100	4%	5%	6%
120	3%	4%	5%

Maximum gradients for different design speeds and types of topography are suggested in Table 4.3.1. It is stressed that these are guidelines only. The optimization of the design of a specific road with the whole-life economy of the road taken into account may suggest some other maximum gradient.

4.3.2 Critical length of grade

The critical length of any given grade is defined as that length which causes the speed of the design truck to be reduced by 20km/h. The starting point of the grade can be approximated as a point halfway between the preceding vertical point of intersection and the end of the vertical curve. The critical length therefore indicates where the provision of an auxiliary lane may have to be considered.

TABLE 4.3.2
CRITICAL LENGTH OF GRADE

GRADIENT (%)	LENGTH OF GRADE (m)
3	400
4	300
5	240
6	200
7	170
8	150

Critical lengths can be read off from Figure 2.2.4, and are given in Table 4.3.2 for convenience.

4.4 Climbing lanes

A climbing lane is an auxiliary lane added outside the continuous lanes and has the effect of reducing congestion in the through lanes by removing slower-moving vehicles from the traffic stream. The climbing lane is sometimes not effectively utilized, however, especially when traffic flows are heavy, because the drivers of slower vehicles fear that they will not be allowed to merge with the faster vehicles where the climbing lane ends. The preferred layout would however force faster vehicles to merge with the slower, thus allaying this fear to some extent.

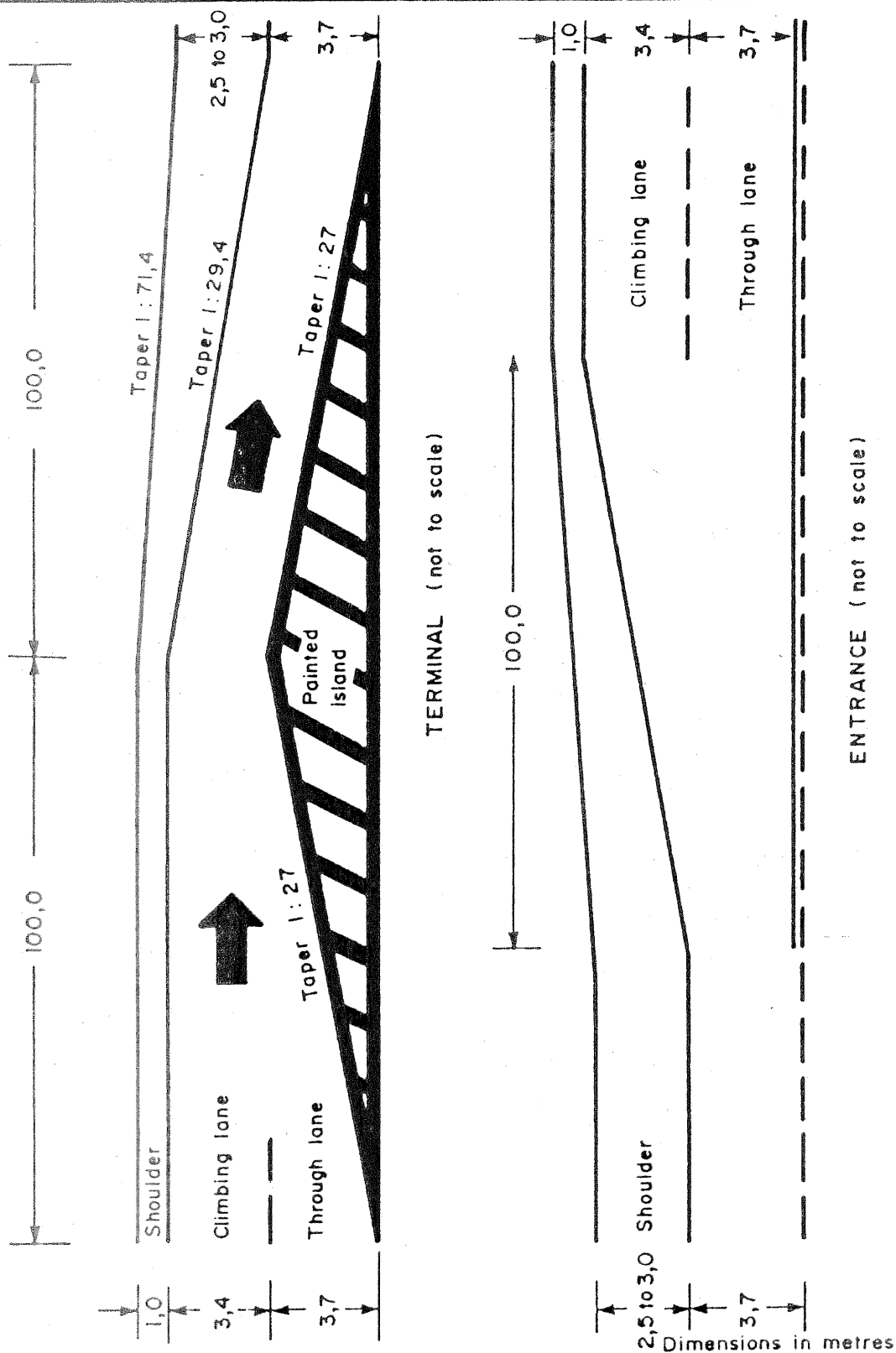
Another solution suggested is to provide an auxiliary lane between the through lanes to operate as a passing lane (see Subsection 4.5). This would also cause faster traffic to merge with the slower vehicles at the end of the auxiliary lane, an easier manoeuvre to carry out. This system is often found where a two-lane interchange ramp has a one-lane merging end.

In the rest of this section, references to climbing lanes should be regarded as including passing lanes even where passing lanes are not specifically mentioned.

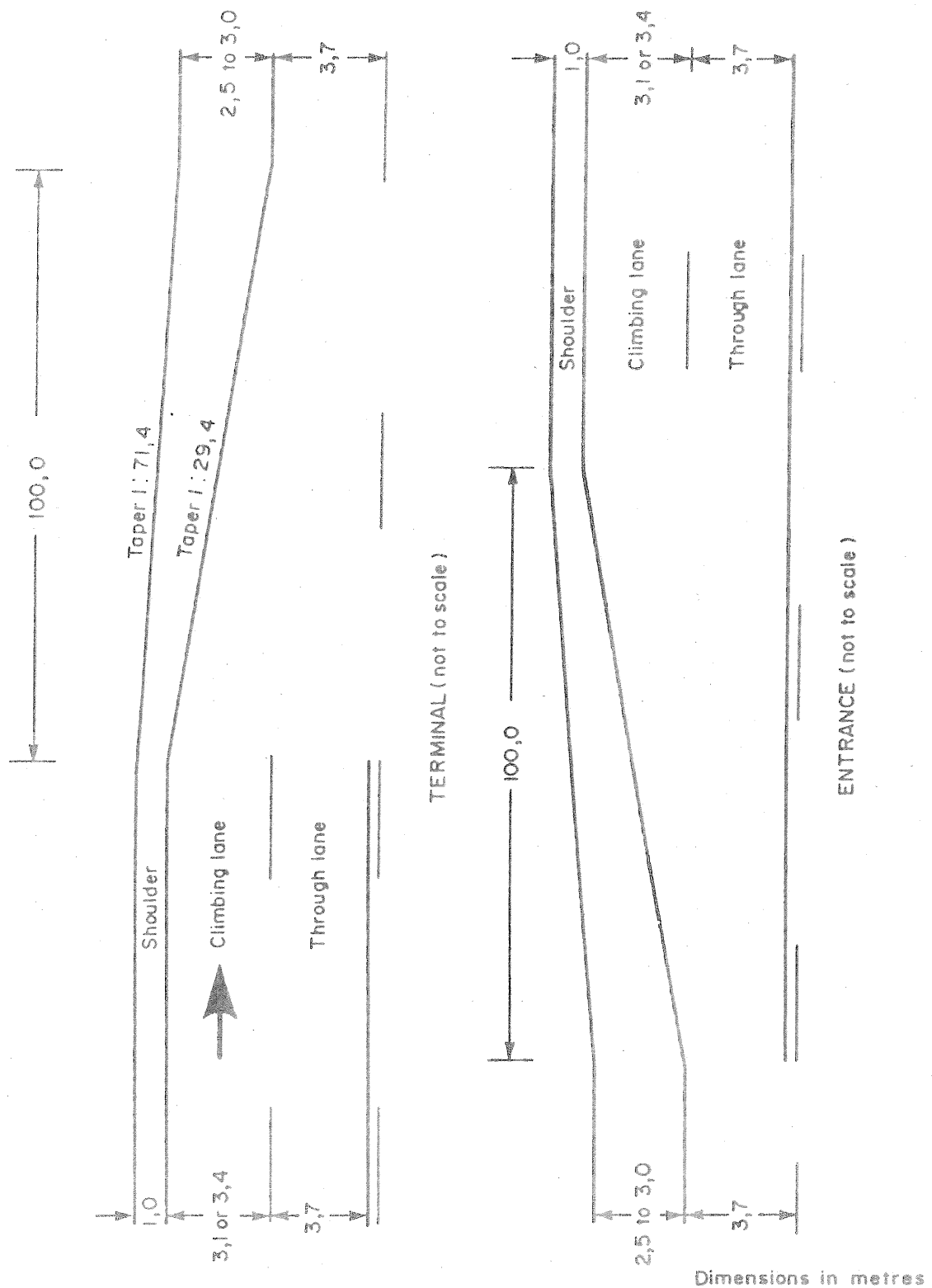
Figures 4.4(a) and 4.4(b) show two alternative layouts for climbing lanes. The layout shown as Alternative 1 is preferred and this preference is based purely on the fact that a vehicle can merge more readily with a slower- than with a faster-moving stream of traffic. This operational factor does however require a greater length of terminal to accommodate the reverse curve path required of the merging vehicle. Alternative 2 is therefore more commonly used, particularly on straight alignments.

4.4.1 Warrants for climbing lanes

As implied earlier, the maintenance of an acceptable level of service over a section of the route is one of the reasons for the provision of climbing lanes. Another reason is the enhancement of road safety by the reduction of the speed differential in the through lane. The warrants for climbing lanes are therefore based on both speed and traffic volume.



LAYOUT FOR A CLIMBING LANE
ALTERNATIVE 1



LAYOUT FOR A CLIMBING LANE
ALTERNATIVE 2

Any grade which exceeds the critical length given in Table 4.3.2 will cause truck speed to be reduced by more than 20 km/h. The effect of the preceding grade can, however, not be ignored. A truck speed profile should be prepared for each direction of flow. It will then be possible to identify those sections of the road where speed reductions of 20 km/h or more warrant the provision of climbing lanes.

The traffic volume warrant is given in Table 4.4.1, related to the percentage of trucks in the design hour traffic stream.

TABLE 4.4.1
TRAFFIC VOLUME WARRANTS FOR CLIMBING LANES

GRADIENT	TRAFFIC VOLUME IN DESIGN HOUR	
	5% trucks	10% trucks
4%	632	486
6%	468	316
8%	383	3
10%	324	198

4.4.2 Location of terminals

A slow-moving vehicle must be completely clear of the through lane by the time its speed has dropped by 20 km/h, and must remain clear of the through lane until it has accelerated again to a speed which is 20km/h less than its normal speed. The taper length has been fixed at 100 m, so that the start taper begins 100m in advance of the point where the full climbing lane width is required, and the end taper ends 100 m beyond the end of the climbing lane. If there is a barrier line, owing to restricted sight distance, at the point where the speed reduction warrant falls away, the full lane should be extended to the point where the marking ends, and the taper should end 100 m beyond this point.

4.4.3 Climbing lane cross-section

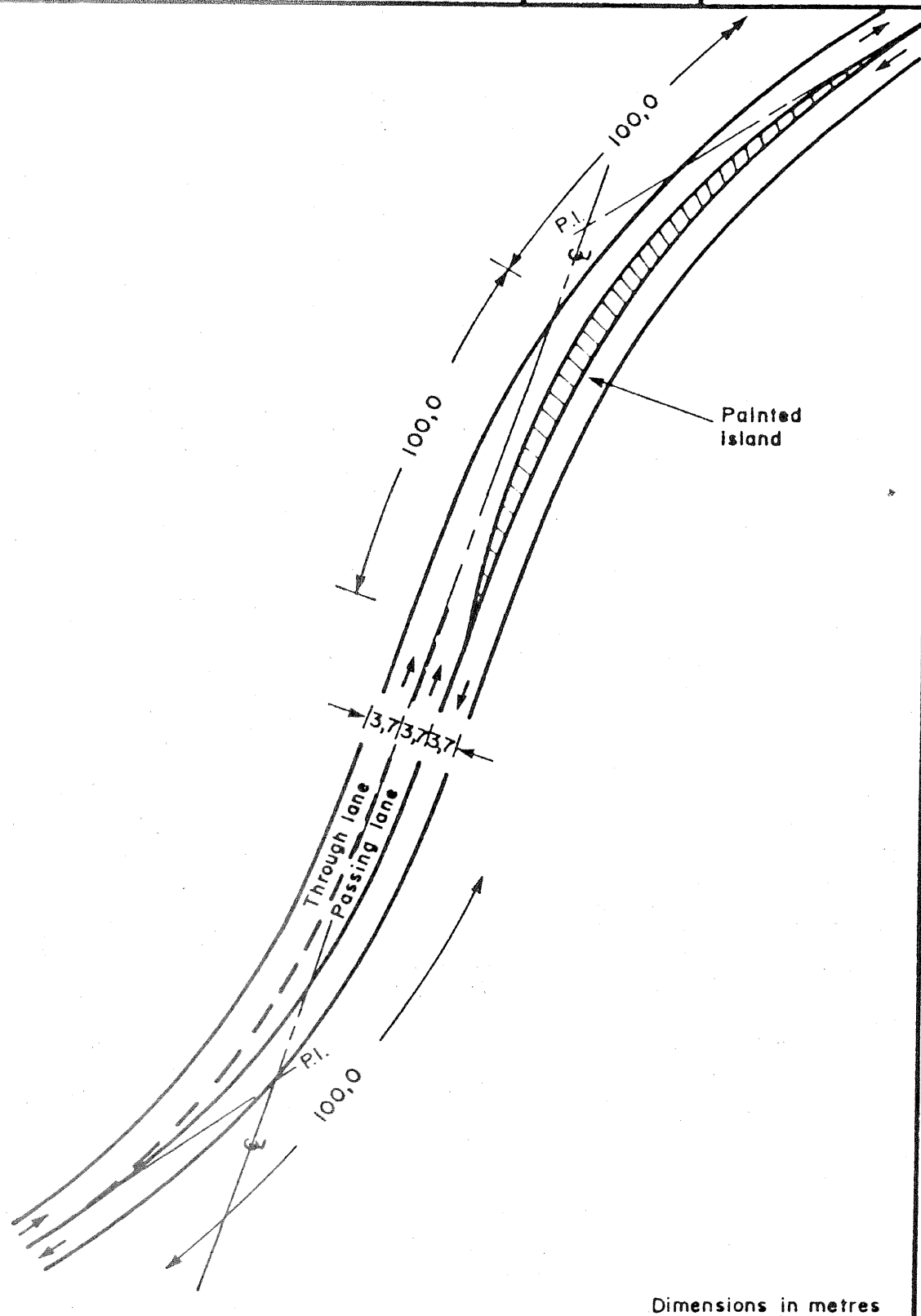
The climbing lane should have the same width as the adjacent through lanes. It will be seen in the next chapter that through lanes may have a width of 3,7 m, 3,4 m or 3,1 m. It is unlikely that climbing lanes will be provided on roads where the traffic volumes are so low that a lane width of 3,1 m is adequate. Climbing lanes therefore tend to be either 3,7 m or 3,4 m wide. Even if the through lanes are 3,7 m wide, a climbing lane 3,4 m or perhaps even 3,1m wide may be considered on the grounds of low lane occupancy and speed.

Like the lane width, the shoulder width along side a climbing lane should ideally match the shoulder widths in advance of and beyond the climbing lane. Climbing lanes are invariably required in areas where earthworks may be heavy, and a reduction in shoulder width would lead to significant savings in construction costs. A minimum usable shoulder width of 1,0 m would be acceptable, also on the grounds of low lane occupancy and low speeds. The shoulders of climbing lanes are normally surfaced. (See also Subsection 5.3.)

4.5 Passing lanes

Passing lanes have the same width as the through lanes, and do not cause a change in shoulder width. In spite of the advantages that the passing lane has over the climbing lane, it must be used with restraint. Incorrect design could result in opposing vehicles being in the same lane, or the downhill vehicles being forced out of their natural path to accommodate the passing lane. It is suggested that passing lanes should be so located that the tapers at either end fall on horizontal curves, allowing the roadway edges to sweep naturally up to and away from the wider cross-section. If it is not possible to have both tapers on horizontal curves, an attempt should be made to have the end taper on a horizontal curve, because it is at this point that the opposing traffic stream is most likely to encroach on the passing lane.

The layout of passing lanes is shown in Figure 4.5.



LAYOUT OF TYPICAL PASSING LANE

5. CROSS-SECTIONAL ELEMENTS

5.1 Introduction

The cross-section of a road provides accommodation for moving and parked vehicles, drainage, public utilities and, to a lesser extent in the rural areas, pedestrians. For the safety and convenience of drivers, wide lanes and shoulders and gently sloping border areas are desirable, since they forgive minor errors of judgment and promote ease of operation.

Cross-sectional dimensions are discussed in the following sections. Alternatives to the dimensions suggested may be appropriate for particular conditions. Variations should be selected to suit these conditions, and careful consideration should be given to the function of the cross-sectional element before any departure from the recommended values.

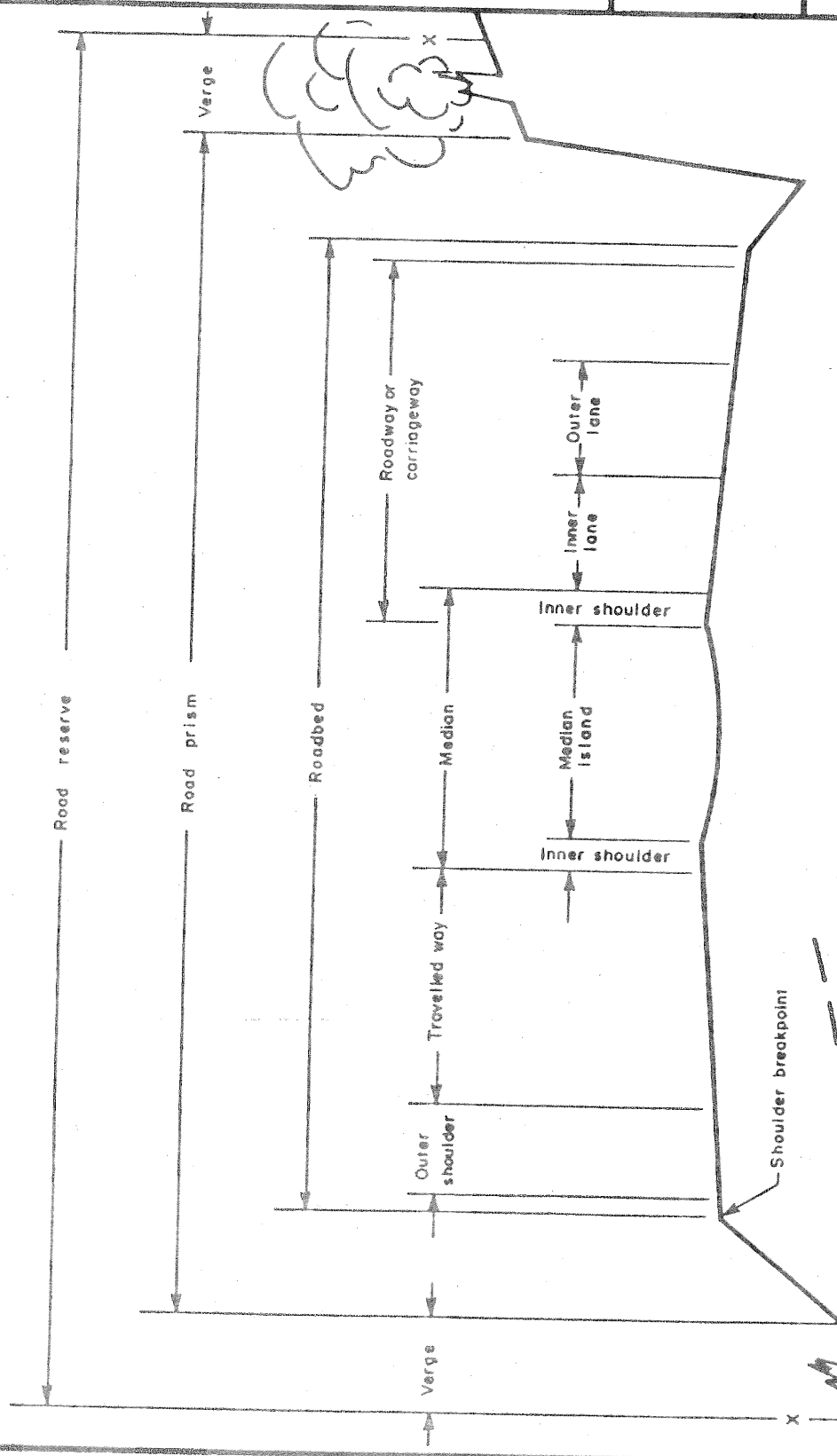
5.2 Lanes

Undivided roads may have either one lane in each direction (two-lane two-way roads) or more than one lane in each direction (multi-lane roads). Dual carriage-way roads have two or more lanes in each direction and are described in terms of the total number of lanes, e.g. as four-lane divided or six-lane divided roads.

Customarily, there is symmetry of through lanes, and assymetry on a particular section of road should arise only from the addition of an auxiliary lane that is clearly allocated to one direction of travel. Three-lane two-way roads have been built that were intended to function as two-lane two-way roads with a continuous central passing lane. These roads were found to have twice the capacity of two-lane two-way roads, but they have been abandoned, in spite of the saving in construction costs resulting from the narrower cross-section, because the practical effect of the three-lane cross-section is to concentrate the faster vehicles of the two opposing traffic streams in a common lane. This is similar to the situation found in the overtaking manoeuvre on a two-lane road, but in the latter case it is clear which of two opposing vehicles has the right of way. When three-lane roads are only marked as having three lanes with no passing restrictions, there is no clarity regarding right of way, and it is this lack of clarity that causes three-lane roads to be unsafe.

The selection of lane width is based on traffic volume and vehicle type and speed. Higher volumes and speeds require wider lanes, and the greatest lane width recommended is 3,7m. The narrowest width recommended is 3,1m, giving a clear space of 0,3 m on either side of a vehicle that is 2,5 m wide. This lane width will normally be employed only where speeds or traffic volumes are expected to be low. Intermediate conditions of volume and speed can be adequately catered for by a lane width of 3,4 m.

Where traffic volumes are such that a multi-lane cross-section or a divided cross-section is required, 3,7 m is a logical lane width to adopt. Lesser lane widths may however be warranted by abnormal circumstances.



CROSS-SECTIONAL ELEMENTS

5.3 Shoulders

The shoulder is the usable area alongside the travelled way, and the width allowed does not make provision for the mounting of guardrails, or for edge drains or shoulder rounding. The shoulder breakpoint is some distance beyond the edge of the usable shoulder, and this distance is usually about 0,5 to 1,0 m.

There are many possible uses for shoulders, including road side vending, but in South Africa only emergency stopping is considered. A stopped vehicle can be adequately accommodated by a shoulder which is 3,0 m wide, and there is no merit in adopting a shoulder width greater than this. The shoulder should not be so narrow that a stopped vehicle would cause congestion by forcing vehicles travelling in both directions into a single lane. However, a partly blocked lane is acceptable under conditions of low speed and low traffic volume. Assuming the narrowest width of through lane, ie 3,1m, it would be possible for two vehicles to pass each other next to a stopped vehicle if the shoulders were not less than 1,0 m wide, giving a total cross-sectional width of 8,2 m to accommodate three vehicles. It must be stressed that this width is an irreducible minimum and appropriate only to low lane volumes and low speeds, such as are encountered in climbing lanes. Hazards, including the edges of high fills, tend to cause a lateral shift of vehicles if located closer than 1,5 m to the lane edge, and for speeds higher than 60km/h a shoulder width of 1,5 m should be regarded as the minimum.

Intermediate traffic volumes and higher operating speeds require a shoulder width greater than 1,0m, and three alternative shoulder widths are suggested, namely 1,5m, 2,0 m and 2,5m. The 3,0 m shoulder is appropriate for the highest operating speeds and heavy traffic volumes.

Where the traffic situation dictates a dual-carriageway cross-section, the highest standard of shoulder width is called for, namely 3,0 m in the case of the outer shoulder. Only 1,0 m is required for the inner shoulder, however, where it would be possible to move a broken-down vehicle onto the median and thus clear the lane, or where the vehicle would have to be moved across one lane only to reach the safety of the outer shoulder, as would occur on a four-lane divided road. As it is generally conceded that crossing two lanes with a defective vehicle would be very difficult, a six-lane divided road should have inner shoulders 3,0 m wide. The alternative shoulder widths suggested above would not normally be used for the inner shoulders of a divided road.

The surfacing of shoulders is recommended

- for freeways;
- in front of guardrails;
- where the total gradient, ie the resultant of the longitudinal gradient and the camber (or superelevation) exceeds 5 per cent;
- where the materials of which the shoulders are constructed are readily erodible, or where the availability of materials for shoulder maintenance is restricted;
- where heavy vehicles would tend to use the shoulder as an auxiliary lane;
- in mist belts;
- wherever it is economically justified;
- wherever significant usage by pedestrians occurs (as specified in Chapter 11).

A patchwork of surfaced shoulders would be both unsafe and unsightly. Where the lengths of intervening unsurfaced shoulders would be relatively short, it is suggested that they should also be surfaced. If a warrant exists for surfacing 60% of the shoulders on a route, the balance should also be surfaced.

5.4 Medians

The median is the total area between the inner edges of the inside traffic lanes of a divided road, and includes the inner shoulders and central island. The purpose of the median is to separate opposing streams of traffic and reduce the possibility of vehicles crossing into the path of opposing traffic. This is accomplished by the selection of the width of the median or by a physical barrier such as a guardrail. Medians are also used to reduce the nuisance of headlight glare by the planting of shrubs on the central island. The shrubs should not grow so tall that sunlight could fall into the driver's eyes in bands – the stroboscopic effect encountered in avenues of trees in the early morning or late afternoon. In addition, the stems of the shrubs should not grow so thick as to become a further possible hazard to the motorist; a maximum stem thickness of 100 mm is recommended. Medians should, as far as possible not be obstructed by kerbs, guardrails, lampposts etc.

Median width depends not only on traffic volume but also on the function of the road and traffic composition. For example, a median functioning purely as a pedestrian refuge could be much narrower than one protecting a turning vehicle which could be anything up to a combination vehicle (ie semitrailer plus trailer).

A median width of 9,2 m would eliminate most cross-median accidents, and this width is recommended as a minimum where no barriers are provided between opposing traffic flows. Where a road is to be constructed in stages, the median will have to be wide enough to accommodate future lanes, without falling below the recommended width of 9,2 m in the final stage.

It is suggested that the median island should be depressed rather than raised, because a raised median, with kerbed median island, will automatically require the inner shoulder to be 3,0 m wide to allow sufficient space for emergency manoeuvres, including stopping. A depressed median also facilitates drainage, as discussed in the following chapter.

The purpose of an outer separator is most frequently to separate streams of traffic flowing in the same direction but at different speeds and also to modify weaving manoeuvres. In general, the standards applied to medians are equally appropriate to outer separators.

(For median slope, see Subsection 5.6.2; for median barriers, see Section 7.3.)

5.5 Verges

The verge is defined as the area between the longitudinal works and the road reserve boundary. The boundary of the longitudinal works is often at the top of cut or toe of fill, but the longitudinal works may also include side drains or catchwater drains and hence be wider than the width of the road prism, for example. Any services not directly connected with the road, e.g. telephone or power lines, are normally located in the verge.

The verge should be at least of 3,0 m wide.

5.6 Slopes

5.6.1 Camber and crossfall

Camber implies two slopes away from a central high point, as in a two-lane two-way road, where the cross-section slopes down from the centre line to the shoulders. Crossfall is a single slope from shoulder to shoulder. The slope, be it camber or crossfall, is provided to facilitate drainage of the road surface. The steepness of slope is almost invariably 2 per cent, although, in areas where heavy rainfall is common or where the most economical longitudinal gradient is 0 per cent, this can be increased to an absolute maximum of 3 per cent. Cambers steeper than 3 per cent introduce operational problems, both in driving and in increased wear of vehicle components. Where the shoulder is surfaced, the camber should be taken to the edge of the outer shoulder. Unsurfaced shoulders should have a crossfall of 4 per cent to ensure a rate of flow across this rougher surface that matches the flow across the surfaced area.

5.6.2 Medians

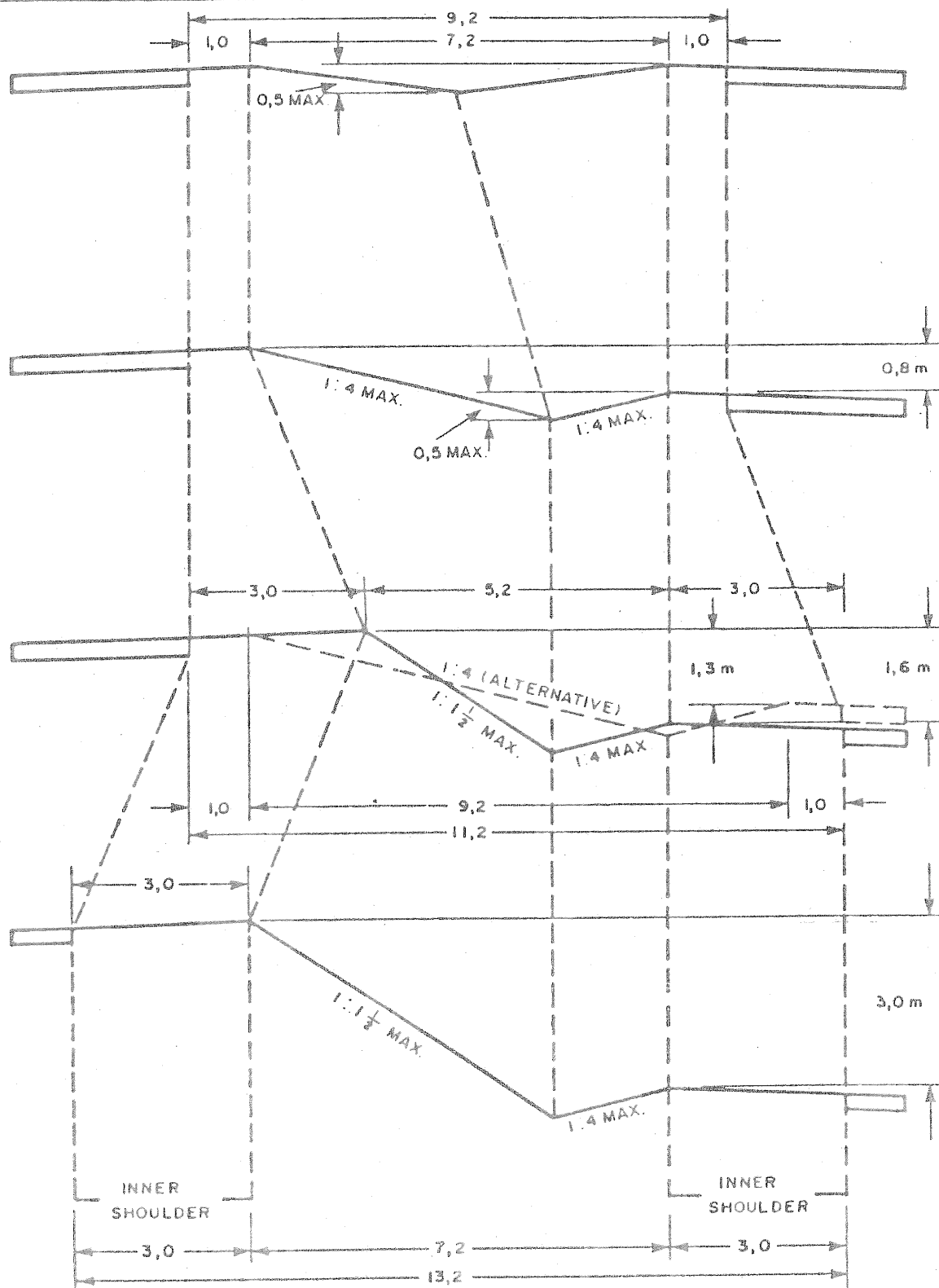
Two different conditions dictate the steepness of the slope across the median: drainage and safety. As suggested earlier (Section 5.4), the normal profile of a median would be a negative camber, ie sloping towards a central low point, to facilitate drainage. The flattest slope that is recommended is 1:10. Slopes flatter than this may lead to ponding and water flowing from the median to the carriageway.

Slopes steeper than 1:4 would make control of an errant vehicle more difficult, leading to a greater possibility of cross-median accidents. If surface drainage requires a median slope steeper than 1:4, this aspect of road safety would serve as a warrant for replacing surface drainage by an underground drainage system.

Differential, or split, grading requires the median to be sloped to absorb the height difference between the carriageways. This is achieved, in the case of small height differences, by locating the low point of the median eccentrically, retaining the maximum permissible slope. The limit is reached when the low point is adjacent to the lower carriageway and functioning as a side drain. If a steeper slope is required, the carriageways will have to be designed as completely independent roadways, with full-width shoulders, guardrails if necessary, and a sufficient distance between shoulder breakpoints, with the side slope appropriate to the in-situ material, to accommodate the height difference between carriageways. Figure 5.6.2 illustrates the transition from a single-graded divided road to a split grading with a slope of 1:1,5 across the median.

5.6.3 Cut and fill batters

The slopes of the sides of the road prism are, like those of medians, dictated by two different conditions. Shallow slopes are required for safety, and a slope of 1:4 is the steepest acceptable for this purpose. The alternative is to accept a steeper slope and provide for safety by some other means, such as guardrails. In this case the steepest slope that can be used is dictated by the natural angle of repose and erodibility of the construction material. Non-cohesive materials require a batter of 1:2, whereas cohesive soft materials can maintain a slope of 1:1,5. Cuts in firm cohesive materials such as the stiffer clays can be built to a



SUGGESTED TRANSITION TO SPLIT GRADING

slope of 1:1. Rock cuts can be constructed to a slope of 1:0,25 (4:1) provided that the material is reasonably unfissured and stable.

It is stressed that the batters suggested are only an indication of normally used values. The detailed design of a project should therefore include geotechnical analysis, which will indicate the steepest batters appropriate to the construction or in-situ material. Economic analysis will indicate the height of fill above which a slope of 1:4 should be replaced by a steeper slope and alternative provision made for safety. The transition from the flat batter slopes to slopes dictated by the materials typically occurs at a height of fill of 3m.

5.7 Minor structures

5.7.1 Agricultural underpasses

An agricultural underpass provides access across a proclaimed road for mechanical farm implements. These underpasses offer the benefits of safety to the road user and convenience to the landowner but are expensive to provide. The provision of an underpass at a particular point is therefore a matter of economic evaluation and policy, rather than of design.

The height of the underpass depends on whether it is the only access to the farmhouse from the public road, in which case a height of 5,1 m is required. If it is not the only access, the height can be reduced to 4,0 m. In both cases a width of 4,0 m is generally adequate.

5.7.2 Other minor structures

Underpasses for stock farming purposes are generally 2,4 m by 2,4 m, but for small stock, such as sheep, this dimension can be reduced to 1,8 m by 1,8 m. Although sheep can pass through a structure smaller than 1,8 m by 1,8 m, they will not do so unless driven.

Pedestrian underpasses are discussed in Chapter 11.

6 DRAINAGE

6.1 Introduction

The drainage associated with any road can be divided into two broad categories: the drainage of the catchment area traversed by the road and the drainage of the road reserve itself. It is the latter which concerns the geometric designer. He must ensure that the construction materials, particularly in the design layers, will not lose their bearing capacity by becoming saturated. He must also ensure that the road surface can drain quickly to reduce the possibility that vehicles will hydroplane or skid out of control. For the same reason, the road should not serve as a drain for other areas.

Drainage devices, such as inlets or side drains, may be very close to the path traversed by vehicles, and the designer therefore has a responsibility to ensure that effective drainage is achieved without the drainage system creating a greater hazard than the stormwater it seeks to remove. Finally, he also has a responsibility to ensure that the discharge of water from the road reserve does not create a hazard or nuisance outside the limits of the reserve.

This chapter discusses the various elements of the drainage system commonly encountered in the rural environment, and makes recommendations regarding their dimensions and location.

6.2 Silting and scouring

Both silting and scouring of a drain increase the hazard to the road user. Scour may lead to the creation of a deep channel that would be impossible to traverse with any degree of safety, and may also cause erosion of the shoulder and ultimately threaten the integrity of the travelled way itself. Silting may block the drain, so that water that should have been thus removed will be discharged onto the road surface.

The effectiveness of the drain depends on water speed, which is a function of longitudinal slope, amongst other variables. There is a range of slopes over which water speeds on in-situ materials will be so low that silting occurs, and a further range where water speeds will be high enough to cause scour. At slopes between these two ranges neither silting nor scouring will occur, and an unpaved drain will be effective.

Paving solves some of the problems caused by both silting and scouring. Paving generally has a lower coefficient of roughness than in-situ materials, so that water speeds are higher in a paved drain than in an unpaved drain with the same slope. Furthermore, it is possible to force higher speeds in the paved drain by selection of the channel cross-section. The problem of silting can be resolved, at least partially, by paving the drain.

The flow velocity below which silting is likely to occur is 0,6m/s. Flow velocities above which scour is likely to occur are given in Table 6.2.

TABLE 6.2
SCOUR VELOCITIES FOR VARIOUS MATERIALS

MATERIAL	ALLOWABLE VELOCITY (m/s)
Fine sand	0,6
Loam	0,9
Clay	1,2
Gravel	1,5
Soft shale	1,8
Hard shale	2,4
Hard rock	4,5

Conventional open-channel hydraulics will, in conjunction with Table 6.2, indicate when either silting or scouring is likely, and hence whether it is necessary to pave a drain or not.

As a rough guide to longitudinal slopes it is suggested that an unpaved drain should not be steeper than 1:50, or flatter than 1:200. A paved drain should not be flatter than 1:300. Practical experience indicates that it is difficult to construct a paved drain accurately to the tolerances demanded by a slope flatter than 1:300, and local imperfections may cause silting of an otherwise adequate drain.

Where the longitudinal slope is so flat that self-cleansing water speeds are not achieved, even with paving, it will be necessary to consider a piped drainage system.

It is possible, as an alternative to lining a material subject to scour, to reduce flow velocity by constructing weirs across an unpaved drain. The drain will then in effect become a series of stilling basins at consecutively lower levels. While this could be an economical solution in terms of construction cost, it has the disadvantage of confronting an errant vehicle with an area of deep localized erosion, immediately followed by a stone-pitched or concrete wall. If this alternative is to be considered at all, it should be restricted to roads with very low traffic volumes, and the weirs should be spaced as far apart as possible.

6.3 Channel profiles

6.3.1 Longitudinal profile

Channels that are roughly parallel in plan to the centre line of the road generally follow the vertical alignment of the centre line as well. However, the gradients on the centre line are not always within the limits suggested above for channel slopes and the designer will have to give attention to channel grading. An example is the top of a crest vertical curve where, for a vertical curve K value of 100, the centre line of the road is flatter than 1:200 for a distance of 100m. In this case the 50m of channel on either side of the highest point will have to be individually designed.

6.3.2 Transverse profile

Drains constructed through in-situ materials generally have flat inverts so that, for a given flow, the flow velocity will be reduced. The flat inverts reduce the possibility of scour, and are easy to clear if silting occurs. Paved drains, not being susceptible to scour, have a V-profile. Self-cleansing velocities are thus achieved at relatively small flows, and the need for maintenance reduced.

The sides of the drain should not be so steep as to be dangerous to the road user, and a maximum slope of 1:4 is recommended. Ideally, both sides of the drain should be designed to this slope or flatter. Where space for the provision of the drain is restricted, the slope closest to the road should remain at 1:4, and the outer slope should be steepened. This has the effect of locating the drain as far away as possible from the path of vehicles. An example is the side drain in a cut, where the outer slope of the side drain forms an extension of the cut face. These slopes, in combination with the flat invert, give the trapezoidal profile of an unpaved drain.

It is recommended that the bottom of a lined V-profile and the junctions between the sides and bottom of an unlined trapezoidal profile should be slightly rounded. The rounding will ease the path of an errant vehicle across the drain, and reduce the likelihood that the vehicle will dig its front bumper into the far side of the drain and somersault.

Because of the need to safeguard the design layers against saturation, it was previously common practice to recommend a minimum depth of drain. Emphasis has now shifted to the safety of the road user, leading to the recommendation of a maximum depth of drain. The recommended maximum depth is 500 mm. The volume of water conducted by a drain thus indicates the required width of the drain rather than its depth, since the need to keep the design layers unsaturated has not changed.

6.4 Types of drain

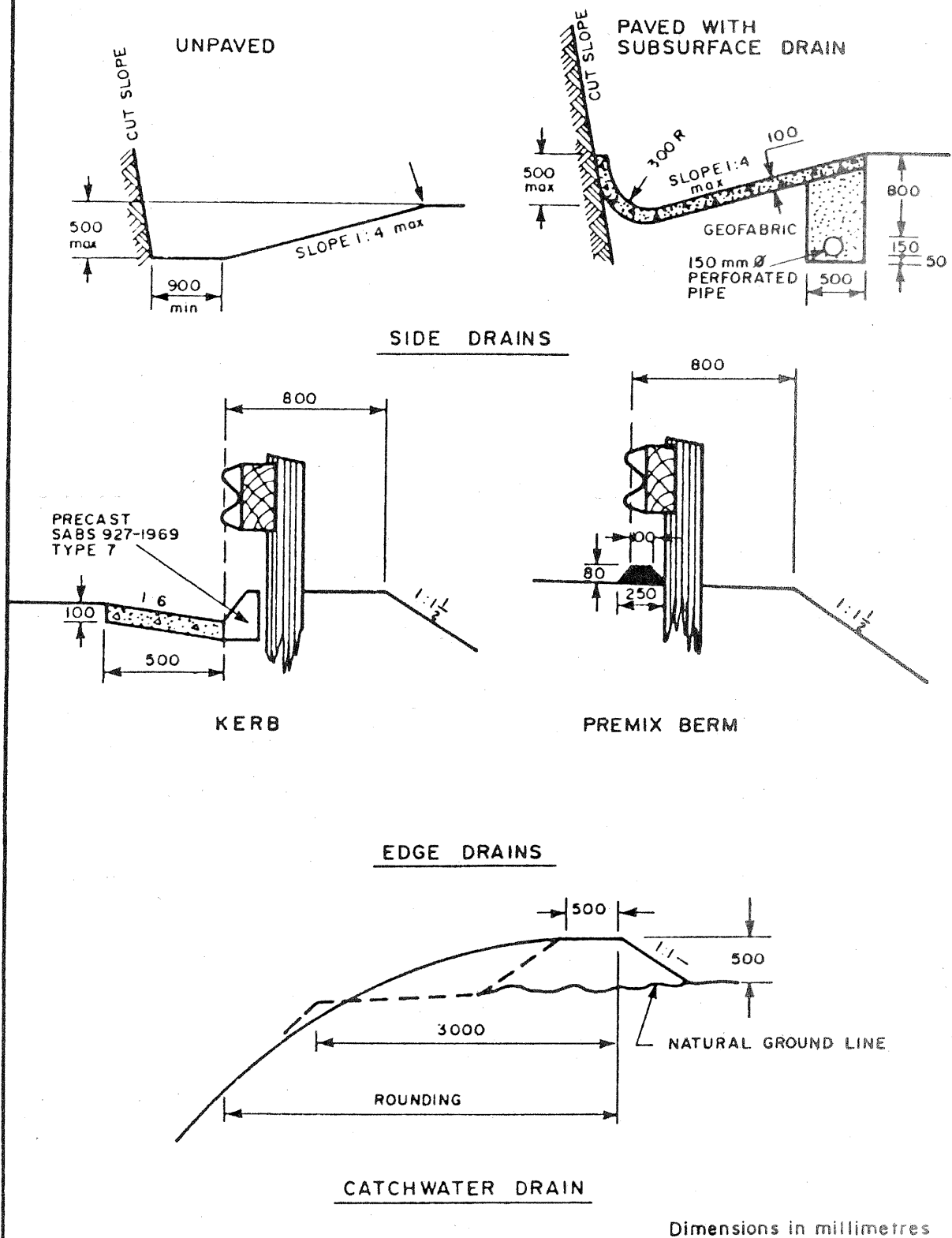
Typical drain profiles are illustrated in Figure 6.4.

6.4.1 Side drains

Side drains are located beyond the shoulder breakpoint and parallel to the centre line of the road. While usually employed in cuts, they may also be used to run water along the toe of a fill to a point where the water can conveniently be diverted, either away from the road prism or through it, by means of a culvert. Side drains are intended as collectors of water, and the area that they drain usually includes a cut face and the road surface.

6.4.2 Edge drains

Edge drains are intended to divert water from fill slopes that may otherwise erode either because of the erodibility of the material or because they are subjected to concentrations of water and high flow. Guardrail posts tend to serve as points of concentration of water, so that, as a general rule, edge drains are warranted when the fill material is erodible or when guardrails are to be installed.



TYPICAL DRAIN PROFILES

Edge drains should preferably be raised rather than depressed in profile. A depressed drain located almost under a guardrail would heighten the possibility that a vehicle wheel might snag under the guardrail.

Edge drains are constructed of either concrete or premixed bitumen. The premix berm normally has a height of 75 to 80 mm, and is trapezoidal in profile with a base width of 250 mm and a top width of 100 mm. The concrete edge drain is a normal barrier kerb and channel as specified in SABS 927-1969³. This requires a properly compacted backing for stability and is therefore less convenient to construct than the premix berm.

6.4.3 Catchwater drains

The catchwater drain, a berm located at the top of a cut, is to the cut face what the edge drain is to the fill. It is intended to deflect away from the cut overland flow from the area outside the road reserve. Even if the cut is through material which is not likely to scour, the catchwater drain serves to reduce the volume of water that would otherwise have to be removed by the side drain.

A catchwater drain is seldom, if ever, lined. It is constructed with the undisturbed topsoil of the area as its invert, and can readily be grassed as a protection against scour. Transverse weirs can also be constructed to reduce flow velocities, since the restrictions mentioned in Section 6.2 do not apply to the catchwater drain. The cut face and the profile of the drain reduce the probability of a vehicle entering the drain, but, should this happen, the speed of the vehicle will probably be low.

6.4.4 Median drains

The median drain not only drains the median, but also, in the case of a horizontal curve, prevents water from the higher carriageway from flowing in a sheet across the lower. The space available for the provision of a median drain makes it possible to recommend that the transverse slopes should be in the range of 1:4 to 1:10. If the narrowest median recommended in Section 5.4 is used, a transverse slope flatter than 1:10 may make it difficult to protect the design layers of the road. Unlike the side drain, the median drain, whether lined or not, is generally constructed with a shallow V-profile with the bottom gently rounded.

6.5 Discharge from drains

The main problem of median drainage is not the transport of the water along the median so much as the removal of water from the median. Likewise, water concentrated by an edge drain must be removed from the shoulder before it encroaches on the travelled way. Generally, discharge from drains must be considered as carefully as the drainage system itself, if this discharge is not to cause either a hazard to the road user and damage to the permanent works, or a nuisance to adjacent land-owners.

6.5.1 Underground systems

The geometric designer is not directly concerned with the underground system, except for its inlets. These must be hydraulically efficient and correctly located to ensure that water does not back up onto the road surface or saturate the de-

sign layers. To restrict the hazard to the road user, inlets that are flush with the surface drain invert are preferable to raised structures.

Underground reticulation is costly both to provide and to maintain. The designer should therefore, without violating the principles discussed above, attempt to reduce the use of underground drainage as far as possible by the discerning use of surface drainage.

6.5.2 Chutes

Chutes are intended to convey a concentration of water down a slope which, without such protection, would be subject to scour. They may vary in size from a large structure to a half-round precast concrete product, but they are all open channels. Flow velocities are high, so that stilling basins are required if downstream erosion is to be avoided. An example of the application of chutes is the discharge of water down a fill slope from an edge drain. The entrances to chutes require attention to ensure that water is deflected from the edge drain into the chute, particularly where the road is on a steep grade.

It is important that chutes be adequately spaced to remove excess water from the shoulders of the road. Furthermore, the dimensions of the chute and stilling basin should be such that these drainage elements do not represent an excessive risk to errant vehicles. Generally, they should be as shallow as is compatible with their function, and depths in excess of 150 mm should be viewed with caution.

Because of the suggested shallow depth, particular attention must be paid to the design and construction of chutes to ensure that the highly energized stream is not deflected out of the chute. This is a serious erosion hazard, and can be obviated by replacing the chute with a pipe.

6.5.3 Mitre banks

As their name implies, these banks are constructed at an angle to the centre line of the road. They are intended to remove water from a drain next to the toe of a fill, and to discharge it beyond the road reserve boundary. Several mitre banks can be constructed along the length of a drain, as the concentration of water in the drain should ideally be dispersed and its speed correspondingly reduced before discharge. Speed can be reduced not only by reducing the volume, and hence the depth of flow, but also by locating the mitre bank so that its toe is virtually parallel to the natural contours. The upstream face of a mitre bank is usually protected by stone pitching, since the volume and speed of flow of water it deflects may cause scour and ultimately lead to the breaching of the mitre bank.

7. SAFETY BARRIERS

7.1 Introduction

Road user safety, or rather its absence, has evident economic consequences in terms of property damage and loss of earnings or production resulting from physical injury, in addition to the emotional consequences of pain, suffering and death. Safety and economy are the twin foundation stones on which competent design rests. Inadequate consideration of either will automatically result in inadequate design.

Earlier chapters have discussed, with both safety and economy in mind, the selection of elements of the vertical and horizontal alignment and the cross-section of the road, as well as the provision of drainage. This chapter, however, is devoted to elements which are primarily aimed at road user safety.

7.2 Guardrails

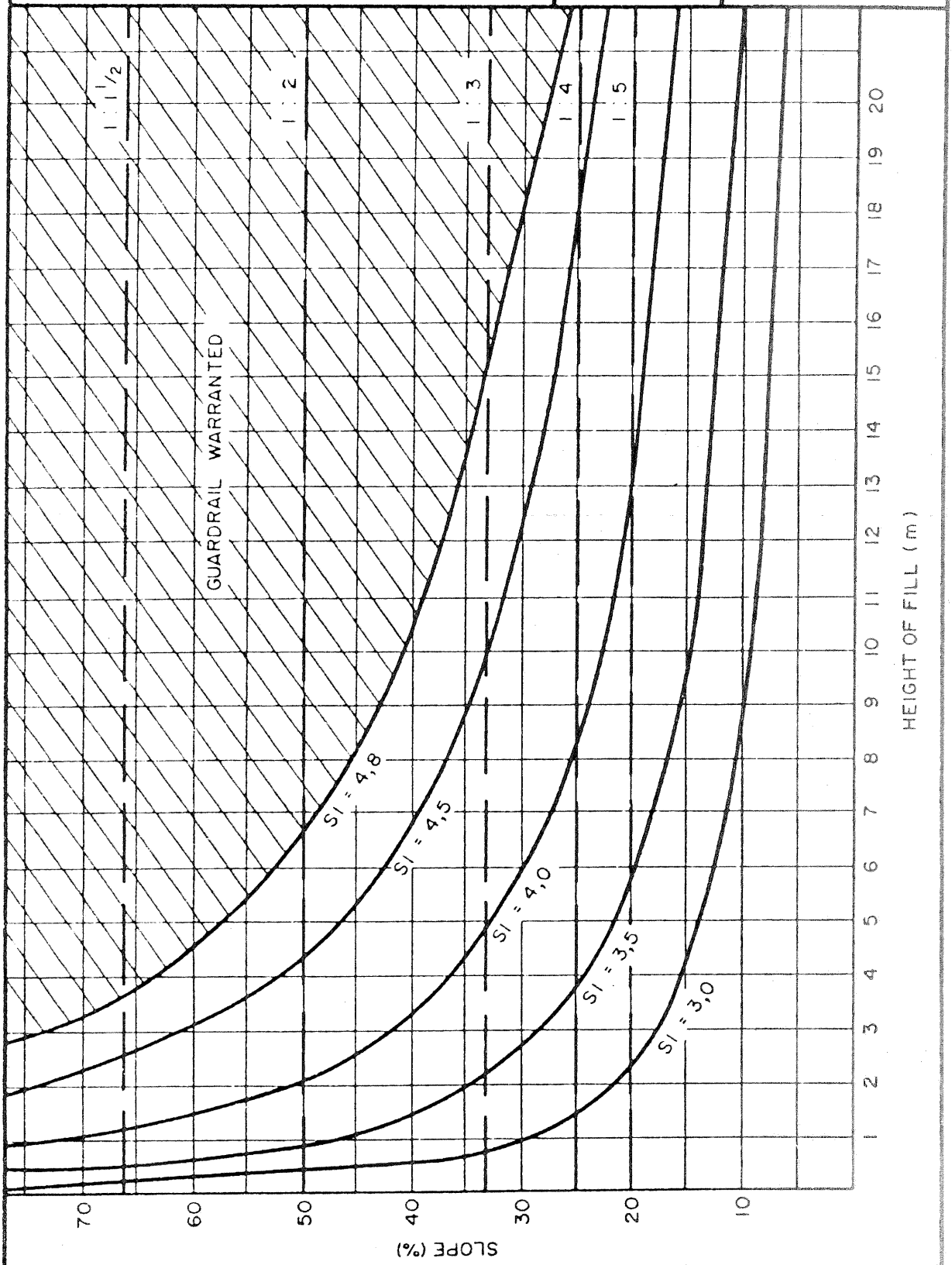
7.2.1 Warrants

To promote road safety there is no real substitute for flat slopes and clear verges. Guardrails are a compromise between the conflicting demands of construction costs and safety, and are themselves a hazard. To be warranted the guardrails should be a lesser hazard than that which they are intended to replace. On existing roads an important warrant for guardrail installation is an adverse accident history. In the case of proposed roads, it is necessary to consider whether the outcome of an accident is likely to be more serious without guardrails than with them.

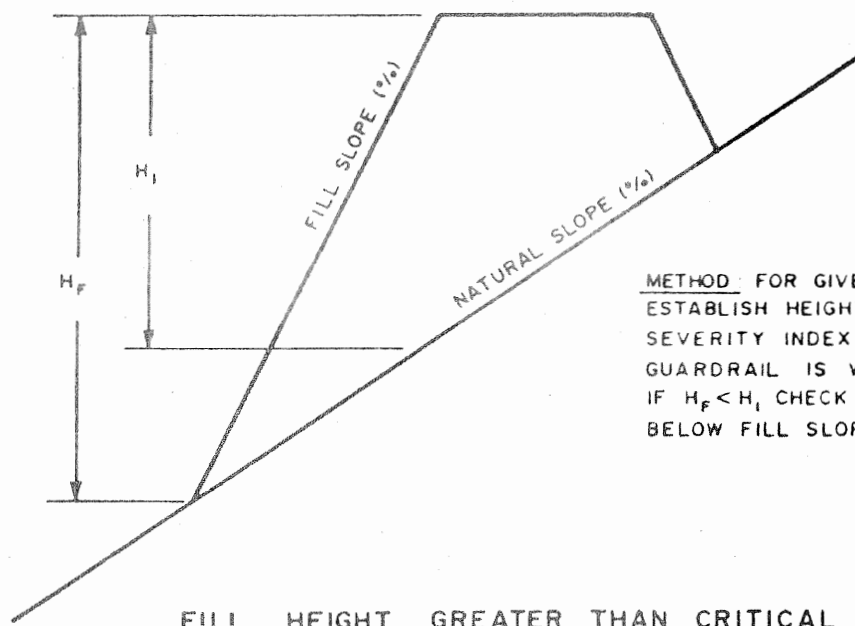
The seriousness of an accident involving a vehicle going down a slope is measured in terms of a severity index (SI). This index compares the cost of such an accident with that of a property-damage-only accident. The cost of an accident involving a vehicle striking a guardrail is also compared with the cost of a property-damage-only accident, providing a further Severity Index. Comparison of the two indices will indicate whether a guardrail should be installed. For convenience, the indices are combined in a single curve, the Equal Severity Curve, for various combinations of fill height and fill slope. The curve shown in Figure 7.2.1(a) is based on a Severity Index of 4,8. Figure 7.2.1(b) illustrates the application of this curve.

Errant vehicles seldom travel more than 10 m beyond the edge of the carriageway, and there should be no hazardous obstructions in this area. If an obstruction cannot be removed, guardrails may be warranted, depending on the extent of the hazard presented by the obstruction. These obstacles include permanent water deeper than 1 m, and drop-offs higher than 1 m. Culvert inlets and outlets, even if higher than 1 m, need not necessarily be regarded as drop-offs, because the target area is less than that presented by the length of guardrail required for effective protection.

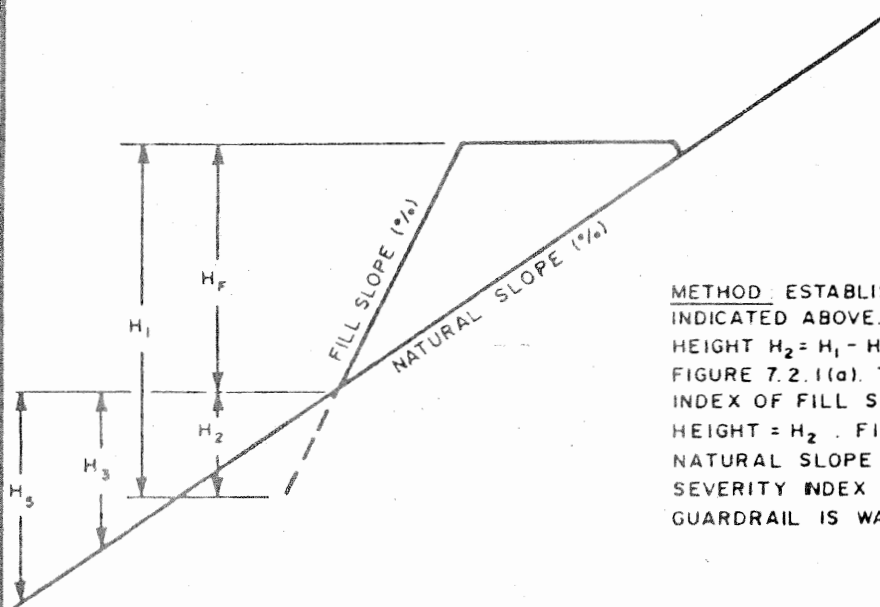
If the guardrail installation is of insufficient length, ie less than 30 m, with too few posts, they cannot develop their full strength in the longitudinal direction and will fail prematurely.



EQUAL SEVERITY CURVE



METHOD FOR GIVEN FILL SLOPE
ESTABLISH HEIGHT (H_1) AT WHICH
SEVERITY INDEX = 4.8. IF $H_F > H_1$
GUARDRAIL IS WARRANTED.
IF $H_F < H_1$ CHECK NATURAL SLOPE
BELOW FILL SLOPE



METHOD ESTABLISH H_1 AS
INDICATED ABOVE. SURPLUS
HEIGHT $H_2 = H_1 - H_F$. FIND FROM
FIGURE 7.2.1(a). THE SEVERITY
INDEX OF FILL SLOPE WITH
HEIGHT = H_2 . FIND HEIGHT OF
NATURAL SLOPE WITH THE SAME
SEVERITY INDEX (H_3). IF $H_3 > H_2$
GUARDRAIL IS WARRANTED.

APPLICATION OF EQUAL SEVERITY CURVE

Protection from a jointed rock face close to the edge of the carriageway can best be provided by a rigid barrier.

On tight radius curves, where the height of the fill does not meet SI warrants for a guardrail, it is suggested that the outside fill slope should be flattened rather than that a guardrail should be installed.

Generally, before it is accepted that a guardrail is warranted, an attempt should be made to remove the obstruction warranting the guardrail, or the possibility of using a slope sufficiently flat to obviate the need for a guardrail should be investigated.

7.2.2 Mounting of guardrails

Guardrails, which are heavily galvanized 300 mm W-beams as described in SABS 1350-1982⁴, are mounted on creosoted timber posts that are 1,8 m long and in the diameter class 175 to 200 mm with domed tops or 70° tapered tops. These posts are described in SABS 457-1982⁵. Spacer blocks, with dimensions of 360 x 150 x 100 mm, are mounted between the posts and the guardrail to ensure that a vehicle wheel that has been snagged by the guardrail will not hit the posts. As a further safety precaution, the overlap between successive rails is in the direction of travel.

The guardrail should be mounted with its face about 300 mm beyond the edge of the usable shoulder, to ensure that the full width of the shoulder is, in fact, usable.

Guardrail mounting is illustrated in Figure 7.2.2.

The centre of the guardrail is usually 530 mm above the edge of the usable shoulder, corresponding to the height of the centre of gravity of the average passenger car.

Guardrail posts should be located at 3,81 m centres to match the predrilled slots in the rails, except as discussed in Subsection 7.2.3.

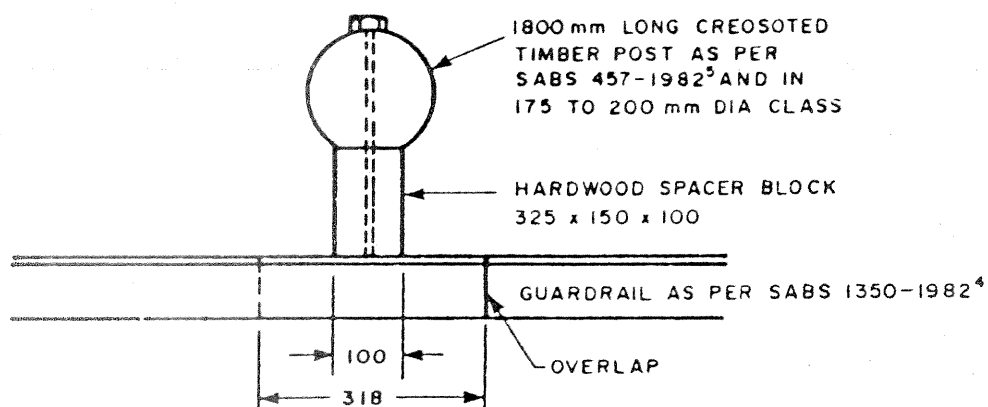
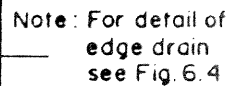
As a conspicuous element of the driver's view of the road, the guardrail should present a continuous smoothly flowing appearance, even if this entails slight departures from the mounting position suggested above.

7.2.3 End treatment

The leading and trailing ends of a guardrail installation are its most dangerous features, the former more so than the latter.

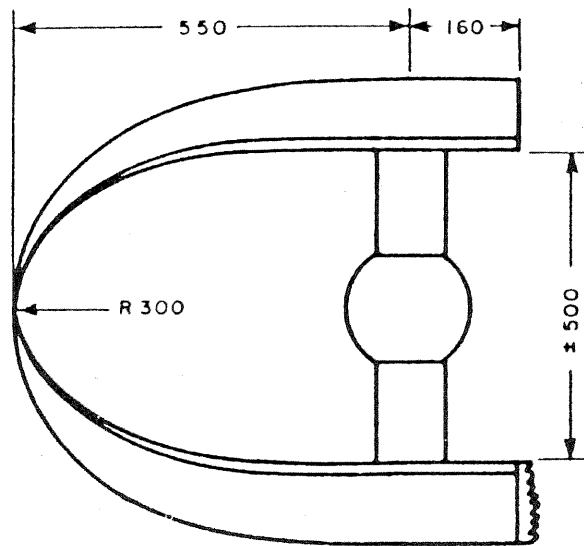
Two different end treatments have been evolved to reduce the consequences of striking a guardrail end on. The first accepts that the guardrail will be struck, and reduces the severity of the accident by impact attenuation and spreading the load across the body of the vehicle by replacing the chisel edge of the guardrail with a bull-nose of fairly wide radius. The second treatment reduces the likelihood that the end of the guardrail will be struck by flaring the end away from the road, or burying it. Often the end is both flared and buried.

With both treatments the post spacing is halved over the first three to five lengths, as shown in Figure 7.2.3.

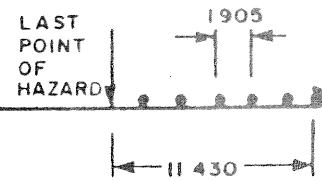
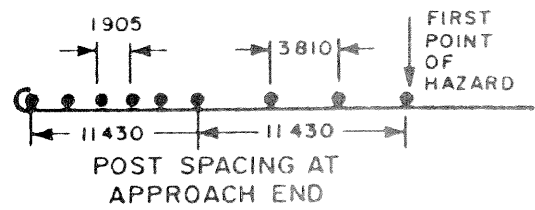


Note: Post top may be either tapered 70° or domed. Dimensions in millimetres

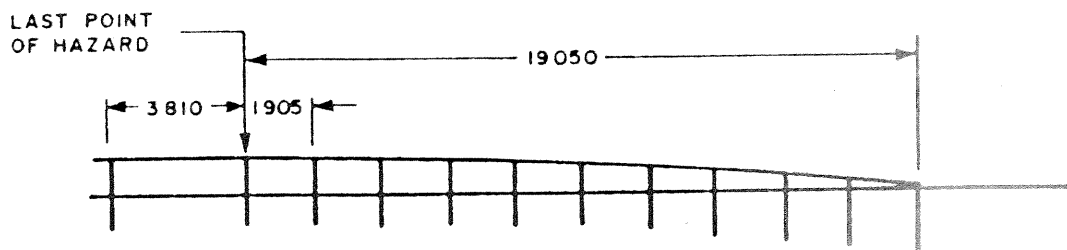
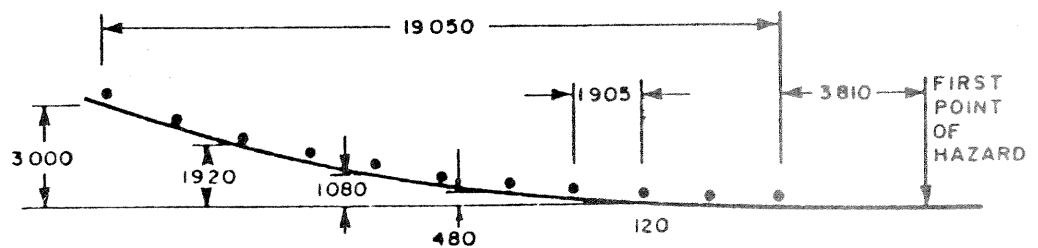
MOUNTING OF GUARDRAILS



DETAIL



BULL-NOSE



FLARED AND/OR BURIED END Dimensions in millimetres

GUARDRAIL END TREATMENT

7.3 Median barriers

Barriers are installed on medians to reduce the likelihood of cross-median accidents, as well as to offer protection against obstacles in the median. Very often the barrier used is a guardrail so that the comments in Section 7.2 also apply to this section. A hedge, in addition to providing screening from the glare of approaching headlights, can also serve as a vehicle barrier.

7.3.1 Warrants

Median barriers are not normally used on a road with a speed limit of less than 80km/h.

In the case of existing roads, the major warrant for the installation of a median barrier is once again an adverse accident history.

Median barriers should be considered for a proposed road if the width of the median is less than the 9,2 m suggested in Chapter 5. If the slope of the median is steeper than 1:4, the carriageways are normally considered to be separate roads, and the warrants discussed in subsection 7.2.1 apply. Median barriers are also warranted if the barrier will present a lesser hazard than some immovable object such as a bridge pier.

7.3.2 Type of barrier

Different types of median barrier are shown in Figure 7.3.2. Guardrails mounted back to back shown as Type M1 are used as median barriers if the width of the median is between 5 and 9,2 m. If the width is between 2 and 5 m, the guardrails are supplemented by steel channel sections, 102 x 51 x 3,8 mm in section shown as Type M2. A median less than 2 m wide would warrant consideration of a rigid or Type M3 barrier, although some authorities prefer this barrier to the guardrail regardless of the width of the median. It is recommended that the modified New Jersey profile, as shown, be used.

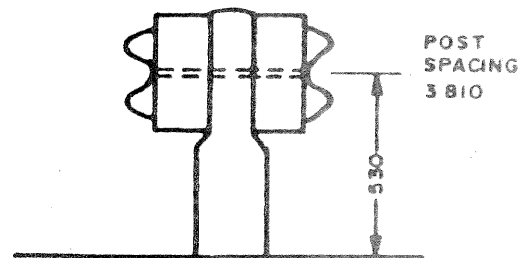
Different types of median barriers are shown in Figure 7.3.2.

7.3.3 End treatment

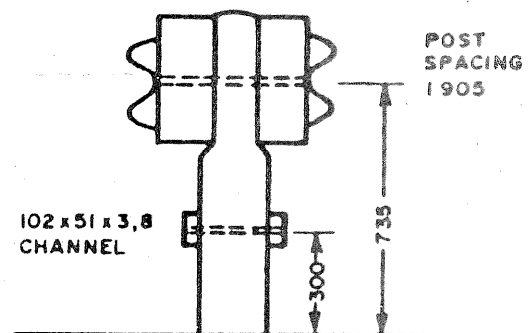
End treatments may be either bull-nosed or buried. Where the median barrier is required as protection against an obstacle in the median, the median should be wide enough to allow for flaring, although the extent of flaring would possibly be less than that normally used on the outer shoulder. If the barrier is warranted by the width of the median, it would normally be erected on the centre line of the median, which excludes the possibility of flaring.

The end treatment of median barriers is illustrated in Figure 7.3.3.

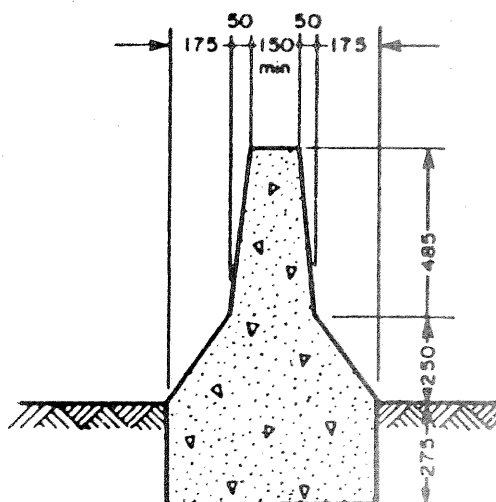
TYPE M1



TYPE M2

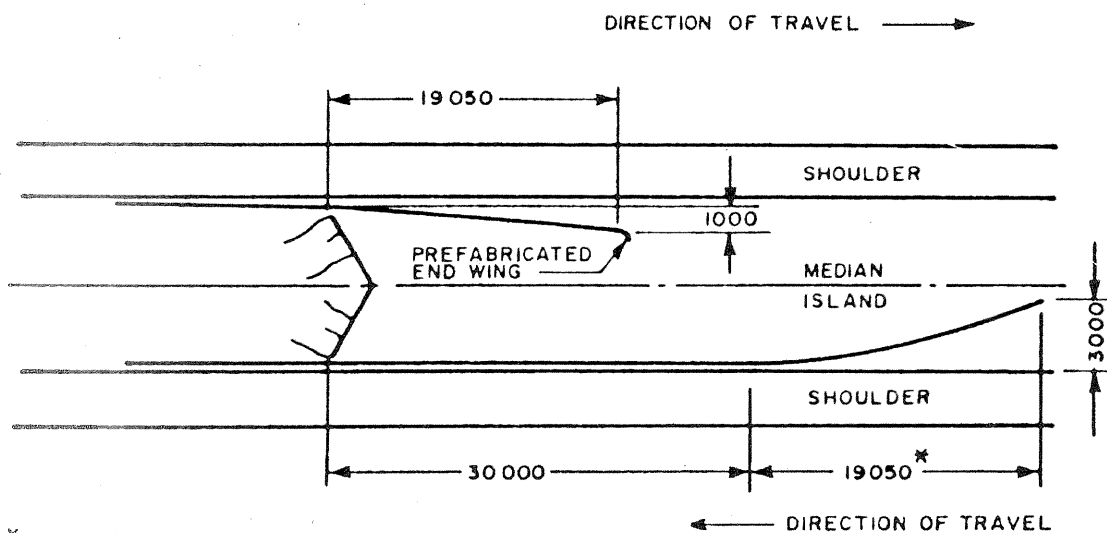


TYPE M3



Dimensions in millimetres

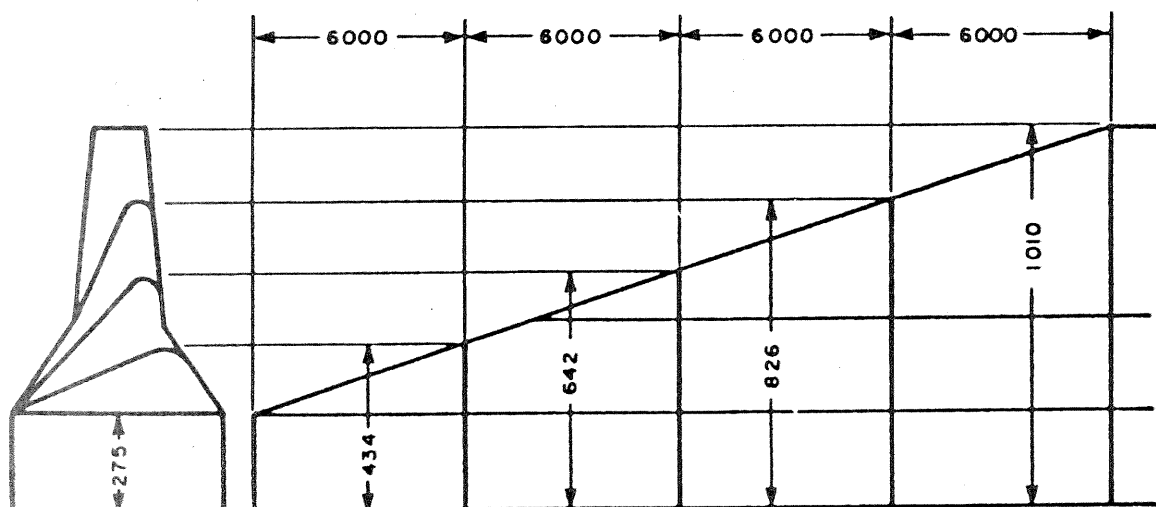
MEDIAN BARRIERS



* See Figure 7.2.3 for detail of parabolic flare

END TREATMENT AT STRUCTURE OR OBSTACLE

Note: Where barrier is warranted by median width the ends are bull-nosed or buried as shown in Figure 7.2.3



END TREATMENT OF NEW JERSEY BARRIER

Dimensions in millimetres

END TREATMENT OF MEDIAN BARRIERS

8. INTERSECTIONS

8.1 Introduction

An intersection is an important part of a road network, because the safety, speed and cost of operation of vehicles on the network are greatly influenced by the effectiveness of its intersections.

The main objectives of intersection design are to ensure effective utilization of the road network and to reduce the severity of potential conflicts between vehicles, or between vehicles and pedestrians, while facilitating the necessary manoeuvres.

A three-legged intersection generates six vehicle conflict points and twelve vehicle-pedestrian conflict points, and a four-legged intersection has twenty-four vehicle and twenty-four vehicle-pedestrian conflict points. Accident history shows that this increased potential for collision at intersections is in fact realized.

In this chapter the location and the various elements of intersections are discussed.

8.2 Location of intersections

Considerations of safety suggest various restraints on the location of intersections. The need for drivers to discern and readily perform the manoeuvres necessary to pass through an intersection safely means that decision sight distance as described in Chapter 2 should be available on the major road on both sides of the intersection. The driver on the minor road will require shoulder sight distance, also described in Chapter 2, to be able to cross the major road safely. Modification of the alignment of either the major or the minor road, or of both, may make it possible to meet these requirements for a safe intersection. If not, it will be necessary to relocate the intersection.

If heavy earthworks beyond the normal limits of the road reserve are required in order to secure the necessary sight distances, relocation of the intersection should also be considered.

The location of an intersection on a horizontal curve can create problems for the drivers on both legs of the minor road. Drivers on the minor road leg on the inside of the curve will find it difficult to see approaching traffic, because this traffic will be partly behind them. The fact that a large part of the sight triangle falls outside the normal width of the road reserve also means that both decision sight distance and shoulder sight distance may be obscured. Drivers on the leg of the minor road on the outside of the curve seldom have any problems with sight distance because, in addition to having approaching traffic partly in front of them, they have the added height advantage caused by the superelevation of the curve. They do, however, have to negotiate the turn onto the major road against an adverse superelevation. The risk involved in sharp braking during an emergency should also be borne in mind when locating an intersection on a curve. Generally, an intersection should not be located on a curve with a superelevation greater than 6 percent.

Figure 2.5.1(a) shows that stopping distance requirements increase with steepening negative gradient. The additional stopping distance required on a downgrade of 6 per cent is between 30 and 45 per cent of the stopping distance

required on a level road, whereas the additional distance required on a 3 per cent downgrade is about 15 per cent of the stopping distance required on a level road. Drivers seemingly have difficulty in judging the additional distance required for stopping on grades, and it is suggested as a safety measure that intersections should not be located on grades steeper than 3 per cent. If it is not possible to align all the legs of an intersection to a gradient of 3 per cent or less, the through road could have a steeper gradient because vehicles on the intersecting road have to stop or yield, whereas through vehicles would only have to do so occasionally.

In the case of private accesses, steeper grades can be considered.

Tractors drawing trailers or other farming equipment have difficulty in stopping and pulling away on steep slopes. It is therefore customary to extend the shoulder slope of the major road along a farm road for a sufficient distance to allow a tractor-trailer combination to stop clear of the shoulder of the major road and pull away with relative ease.

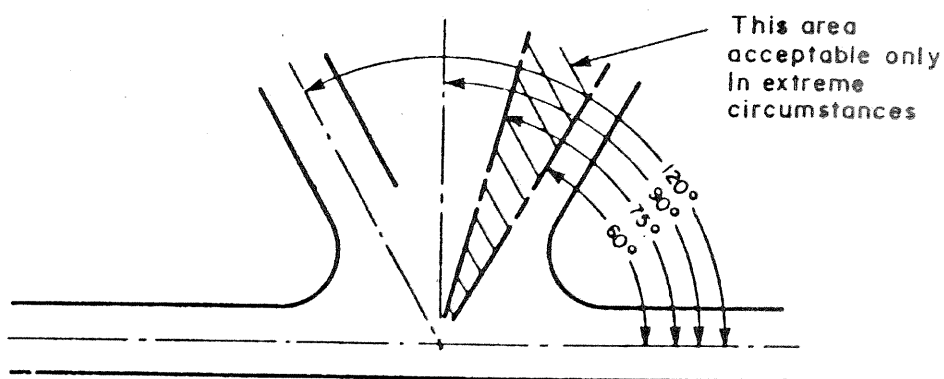
A distance of approximately 8 m is usually allowed for this. After that a gradient of 6 per cent or more can be used to bring the access road to ground level.

One of the consequences of a collision between two vehicles at an intersection is that either or both may leave the road. It is therefore advisable to avoid locating an intersection on a high fill. The obstruction of sight distance by bridge parapets should also be considered when the location of an intersection is being determined. In the case of the crossing road ramp terminal of a narrow diamond interchange, both problems may arise, ie the intersection may be both on a high fill and adjacent to a structure, so that these intersections call for careful design.

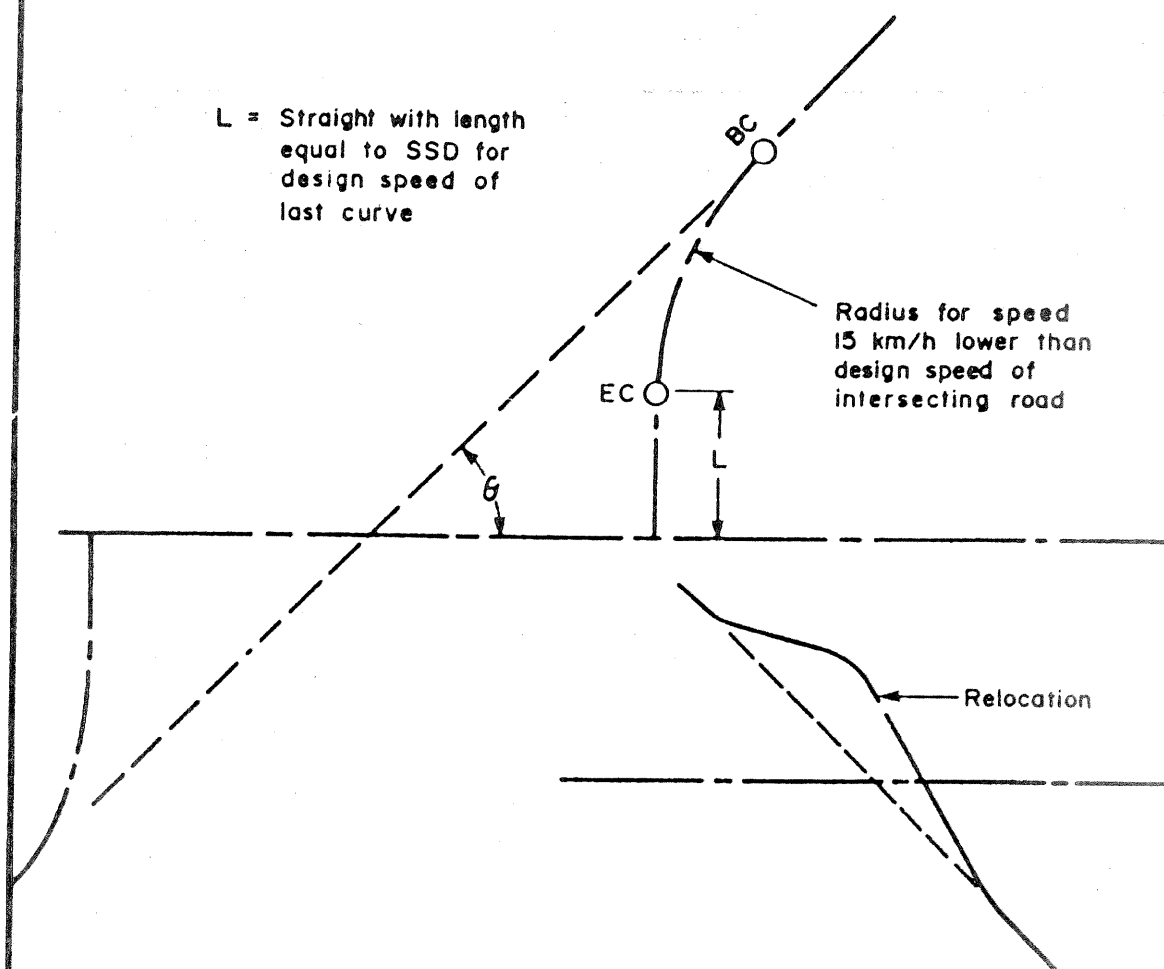
A further restriction on the location of intersections is the distance between successive intersections. A driver cannot reasonably be expected to utilize the decision sight distance to an intersection effectively if an intervening intersection requires his attention. The sign sequence for an intersection includes signs beyond the intersection, and the driver should be beyond the last of these signs before being required to give his attention to the following intersection. A minimum spacing of 500 m between successive intersections is therefore suggested.

The location of an intersection may be modified by the angle of skew between the intersecting roads, ie the change of direction to be negotiated by a vehicle turning left off the major road. Preferably, roads should meet at, or nearly at, right angles. Angles of skew between 60° and 120° produce only a small reduction in visibility for drivers of passenger vehicles, which often does not warrant realignment of the minor road. However, the range of angles of skew between 60° and 75° should be avoided because a truck driver wishing to enter the major road at an intersection with an angle of skew of 75° or less would find the view to his left obscured by his vehicle. Therefore, if the angle of skew of the intersection falls outside the range of 75° to 120° , the minor road should be relocated. Figure 8.2(a) shows acceptable angles of skew.

Two possibilities for relocation can be considered. The minor road can be relocated at a single intersection with an acceptable angle of skew, or the intersection can become a staggered junction. These relocations are illustrated in Figure 8.2(b). When the staggered junction option is selected, it should be noted that a



(a) ACCEPTABLE ANGLES OF SKEW



(b) RELOCATION OF SKEW INTERSECTIONS

ANGLE OF SKEW

right-left stagger (a vehicle crossing the major road will turn right from the minor road onto the major road and then left off the major road onto the continuation of the minor road) is preferred to a left-right stagger. The latter places the right turn on the major road, where it can present a definite hazard to other vehicles.

8.3 Unchannelized intersections

8.3.1 Bellmouth radii

At most intersections traffic needs can be catered for by the provision of an unchannelized bellmouth, and curve radii of 15 m will usually be adequate.

If the joining road is a private property access, the radii can be reduced, and radii of between 10 and 12 m are acceptable. Very often, the bellmouth is unsurfaced in the case of a private access. Radii at the shoulder breakpoint would be about 7 m.

8.3.2 Surfacing and delineation

At intersections other than private accesses the bellmouth is surfaced, principally to prevent loose material being brought onto the through road with consequent damage to the road surface and loss of skid resistance. This surfacing also protects the edge of the through road from crumbling and potholing, and should be taken at least as far as the end of the bellmouth.

A problem frequently encountered at rural intersections is the tendency of vehicles to encroach on the shoulder while turning. Where the shoulder is unsurfaced, this causes the bitumen edges to break up and the shoulder material to ravel. Adequate delineation of the carriageway edges by means of mountable or semi-mountable kerbing or channelling can, however, alleviate this problem to a large extent. Figure 8.3.2 illustrates various kerb types that can be employed. It has been found in practice that the semi-mountable kerbs shown as Nos 6 and 8 are effective delineators. No 6 on edge can also serve as a barrier kerb.

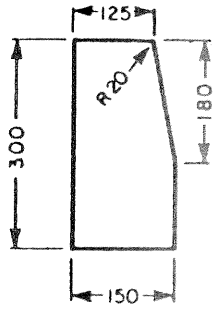
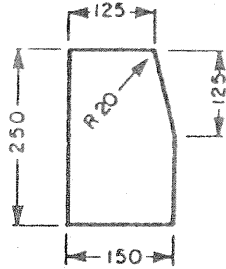
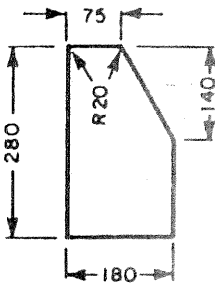
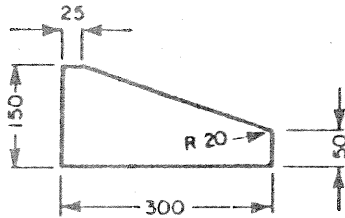
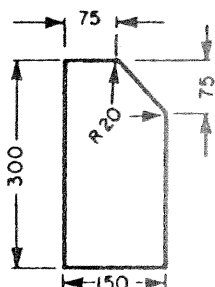
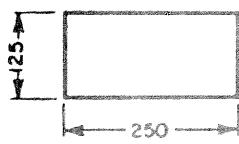
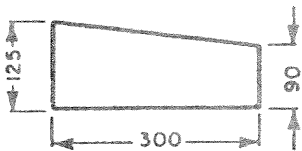
8.4 Speed-change lanes

A speed-change lane is an auxiliary lane, including tapered areas, and is intended for the acceleration and deceleration of vehicles entering or leaving the through-traffic lanes, because undue speed changes on the travelled lanes disrupt the flow of through traffic and are often hazardous. To preclude or minimize these undesirable aspects of operation at intersections, speed-change lanes are standardly provided on roads having expressway characteristics, and are frequently used on other major roads.

8.4.1 Deceleration lanes

Deceleration lanes are always advantageous, particularly on high speed roads, because the driver of a vehicle leaving the major road has no option but to slow down on the through-traffic lane if a deceleration lane is not provided. The failure of the following drivers to brake, possibly through lack of alertness, causes many rear-end collisions.

The lengths of deceleration lanes are based on a 'comfortable' deceleration rate which is approximately half that used in the calculation of stopping sight distance. The taper rates are selected on the basis of accommodating a circular

GEOMETRIC DESIGN OF SA RURAL ROADS		FIGURE 8.3.2	APRIL '88
TYPE	No.	DIMENSIONS	
BARRIER	3		
	4		
SEMI - MOUNTABLE	7		
	8		
	9		
CHANNELS	13		
	14		
KERB TYPES ACCORDING TO SABS 927 - 1969			

path with a radius appropriate to a 2% crossfall and the design speed of the major road. It is assumed that a vehicle will leave the through lane at operating speed and negotiate the taper at unaltered speed, ie zero speed differential, and decelerate on the parallel portion of the lane.

Lengths of deceleration lanes for various design speeds of through roads and turning radius are given in Table 8.4.1(a)

TABLE 8.4.1(a)
DECELERATION LANE LENGTH (INCLUDING TAPER) (m)

DESIGN SPEED OF MAJOR ROAD (km/h)	DESIGN SPEED OF TURNING ROADWAY (km/h)								TAPER RATE
	Stop	20	30	40	50	60	70	80	
60	150	140	135	125					1:15
70	180	170	165	155	135				1:17
80	200	195	190	180	165	150			1:19
90	230	225	220	210	200	180	160		1:21
100	255	245	240	230	220	205	185	165	1:23
110	285	275	265	260	250	235	215	195	1:25
120	310	300	295	285	275	255	240	220	1:27

These lengths are based on level grades. On upgrades, lengths could be decreased and, on downgrades, should be increased. Table 8.4.1(b) offers suggested ratios between lane lengths on gradients and on level grades.

TABLE 8.4.1(b)
ALLOWANCE FOR GRADIENTS

GRADIENT (%)	PROPORTIONAL INCREASE/DECREASE IN LENGTH
-6	1,3
-4	1,2
-2	1,1
0	1,0
2	0,9
4	0,9
6	0,8

8.4.2 Acceleration lanes

Acceleration lanes are less useful than deceleration lanes, since entering drivers can always wait for an opportunity to merge without disrupting the flow of through traffic. Their principal application is on high volume roads where, in the peak hour, gaps between vehicles are infrequent and short.

The length of acceleration lanes is based on acceleration at a rate of $1,5 \text{ m/s}^2$ to the operating speed of the major road, with the merging manoeuvre taking place at operating speed. Acceleration also takes place on the taper which is thus included in the overall length of the acceleration lane.

Lengths of acceleration lanes for various design speeds of through roads and curves are given in Table 8.4.2(a)

TABLE 8.4.2(a)
ACCELERATION LANE LENGTHS INCLUDING TAPER (m)

DESIGN SPEED OF MAJOR ROAD (km/h)	DESIGN SPEED OF TURNING ROADWAY (km/h)								TAPER RATE
	Stop	20	30	40	50	60	70	80	
60	130	130	130	130					1:35
70	160	140	140	140	140				1:37
80	230	220	200	180	150	150			1:40
90	310	300	280	260	220	160	160		1:42
100	390	380	360	340	300	240	170	170	1:45
110	470	460	440	420	380	320	250	170	1:47
120	550	540	520	500	460	400	330	240	1:50

The radii of curvature for the various design speeds are given in Table 3.2.1, and, for convenience, some are repeated in Table 8.4.2(b), with the addition of design speeds below 50 km/h.

TABLE 8.4.2(b)
MINIMUM RADII OF HORIZONTAL CURVATURE

DESIGN SPEED (km/h)	RADIUS (m)
20	10
30	25
40	50
50	80
60	110
70	160
80	210

8.4.3 Right-turn lanes

Right-turning vehicles tend to lower the level of service in the intersection area. Where the level of service is already low, ie where there are high volumes of traffic, a single vehicle waiting for a gap in the opposing traffic can cause a considerable queue of following vehicles.

Where the level of service is high, ie where there are low volumes of traffic, the restrictive effect of a right-turning vehicle on traffic flow may be slight but the presence of a stationary vehicle in a high-speed traffic stream constitutes a hazard which can be avoided.

Right-turn lanes are therefore normally provided at major rural intersections to provide for deceleration and right turns from the major to the minor road. These lanes can thus be designed in accordance with the suggestions contained in Section 8.4.1.

Two possible treatments of right-turn lanes can be considered, both of which include the provision of an additional lane. Where traffic volumes are high and speeds are accordingly low, the risk of a severe accident is slight and the lane from which the right turn is made can be designated for both through and turning traffic. In the absence of turning traffic the flow of through traffic will benefit. Where speeds are high it is advisable to offer the turning vehicles the protection of an exclusive lane. The required use of the outside (left-hand) lane by through traffic can be indicated by mandatory arrows, possibly reinforced by a painted island.

The use of a painted island restricts the length of the right-turn lane. The location of the island must allow adequate storage for the number of vehicles expected to arrive in an average two-minute period in the peak hour. As a minimum requirement, space for at least two passenger cars should be provided; when over 10 per cent of the traffic consists of trucks, provision should be made for at least one car and one truck. The two-minute waiting time is arbitrary, and some other period may be used, depending largely on the opportunities for completing the right turn. This, in turn, depends on the volume of opposing traffic.

8.5 Channelized intersections

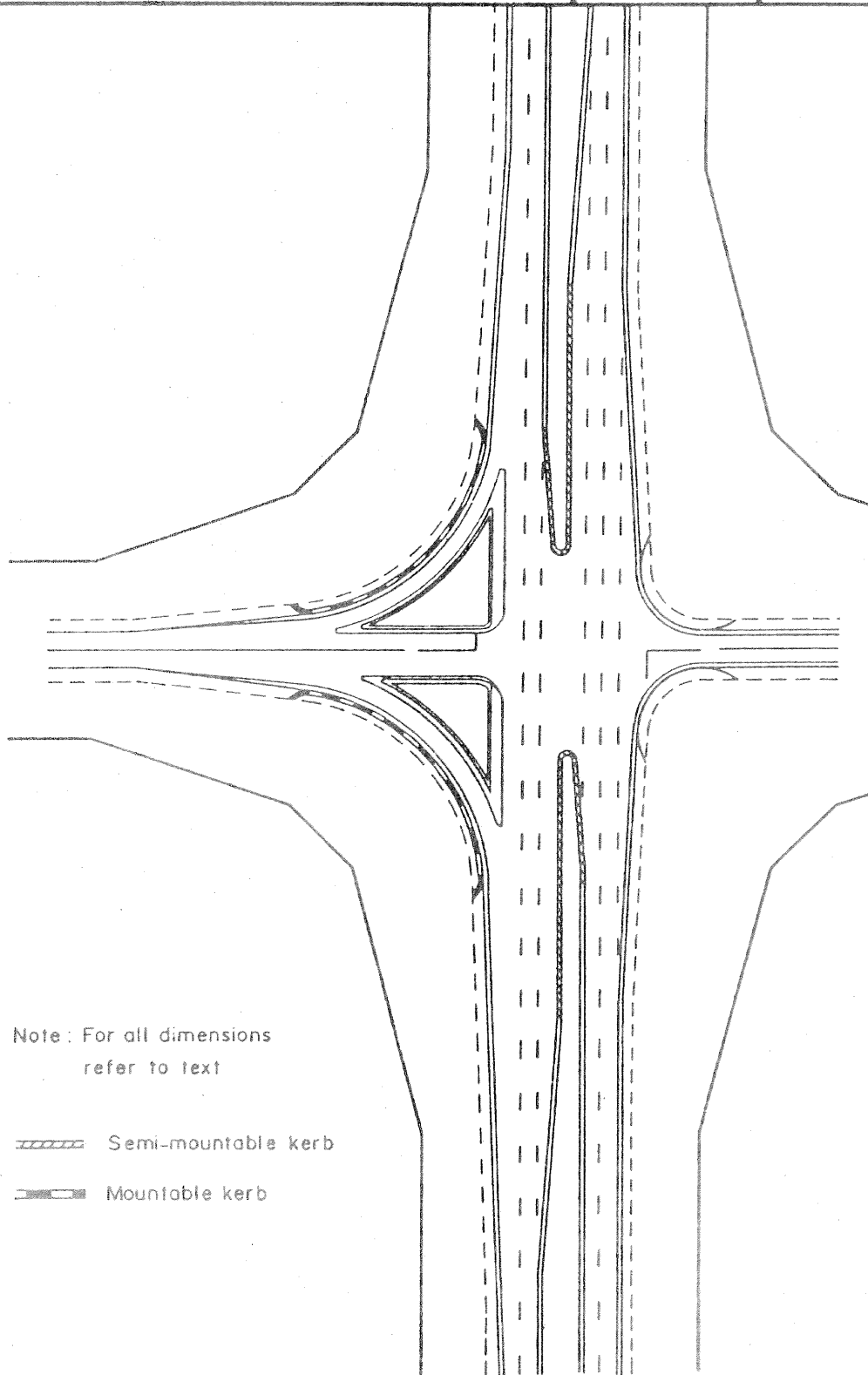
At-grade intersections with large paved areas, such as those with large corner radii and those at oblique angle crossings, permit hazardous uncontrolled vehicle movements, require long pedestrian crossings and have unused pavement area. Even at a simple intersection there may be large areas on which some vehicles can wander from natural and expected paths. Under these circumstances it is usual to resort to channelization of the intersection by the introduction of islands, bearing in mind always that islands are also hazards, and should be less hazardous than what they replace.

Figure 8.5 shows a typical channelized intersection.

8.5.1 Islands

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

- separation of conflicts
- control of angle of conflict
- reduction of excessive pavement area
- regulation of traffic and indication of proper use of intersection
- arrangement to favour a predominant turning movement



TYPICAL CHANNELIZED INTERSECTION

- protection of pedestrians
- protection and storage of turning vehicles
- location of traffic control devices.

Islands are generally either long or triangular in shape and are situated in areas not normally used as vehicle paths. Kerbed islands are sometimes difficult to see at night because of the glare of on-coming headlights. Painted, flush or depressed medians and islands are generally preferred in the rural environment, with mountable or semi-mountable kerbs as a second choice.

Intersections with multiple turning lanes may require three or more islands to channelize the various movements. There is a practical limitation on the use of multiple islands: it may cause confusion leading to inadvertent entrance of a one-way lane by opposing traffic. A few large islands are preferable to a greater number of smaller islands. It is suggested that an island, to be readily visible, should have an area of not less than 5 m².

The location of islands in relation to the adjacent carriageways is indicated in Figure 8.5.1. The edge of the island next to the through road is offset from the edge of carriageway by the width of the shoulder plus 1 m at the approach end, and by the width of the shoulder only at the other end. The offset from other road edges for an unkerbed island is 0,6 m. If the island is kerbed the offset is increased by 0,3 m to allow for the effect that kerbing has on the lateral placement of moving vehicles.

To enhance the visibility of kerbed islands, the nose at the approach end of an island should have a minimum radius of 0,6 m; for the other corners of an island a minimum radius of 0,3 m is adequate.

8.5.2 Turning roadways

Turning roadways can be designed for three possible types of operation:

- | | |
|--------|--|
| Case 1 | One-lane one-way with no provision for the passing of stalled vehicles |
| Case 2 | One-lane one-way with provision for the passing of stalled vehicles |
| Case 3 | Two-lane one-way operation |

Three traffic conditions must also be considered:

- | | |
|-------------|--|
| Condition A | Insufficient SU vehicles in the traffic stream to influence design |
| Condition B | Sufficient SU vehicles to influence design |
| Condition C | Sufficient semi-trailers in the traffic stream to influence design |

The length of turning roadways at intersections is normally short, so that design for Case 1 operation is sufficient. It is reasonable to assume, even in the absence of traffic counts, that there will be enough trucks in the traffic stream to warrant consideration, and Condition B is normally adopted for design purposes. Widths of turning travelled way for the various cases and conditions are given in Table 8.5.2. The radius in the table is the radius of the inner edge of pavement.

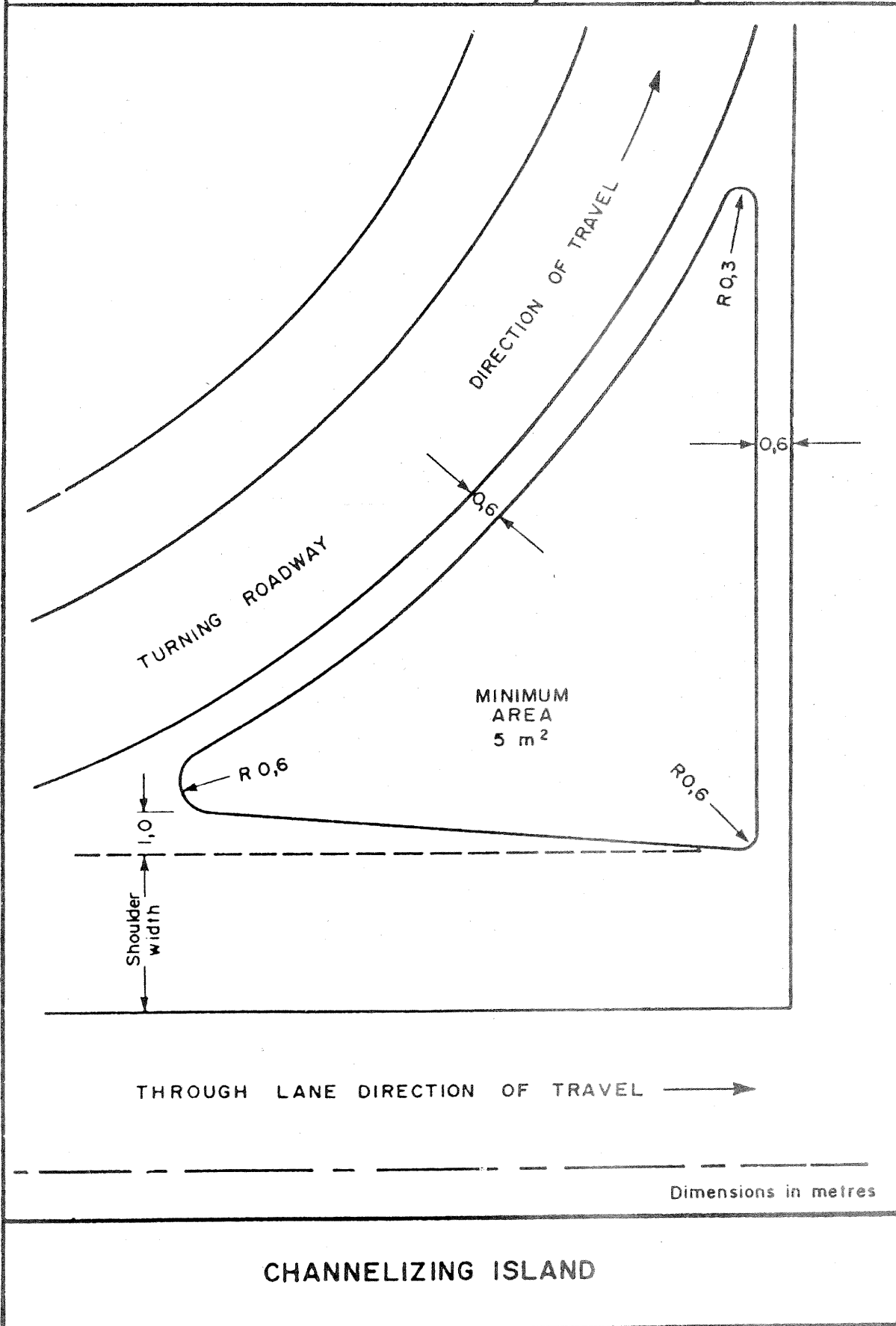


TABLE 8.5.2
TURNING TRAVELLED WAY WIDTHS (m)

INNER RADIUS (m)	CASE 1			CASE 2			CASE 3		
	CONDITION			CONDITION			CONDITION		
	A	B	C	A	B	C	A	B	C
15	4,0	5,5	7,9	6,1	8,8	13,4	7,9	10,7	15,2
20	4,0	5,2	6,7	5,8	8,2	11,0	7,6	10,1	12,8
30	4,0	4,9	6,4	5,8	7,6	10,4	7,6	9,4	12,2
40	3,7	4,9	6,4	5,5	7,3	8,8	7,3	9,1	10,7
60	3,7	4,9	5,2	5,5	7,0	8,2	7,3	8,8	10,1
80	3,7	4,6	5,2	5,5	6,7	7,6	7,3	8,6	9,4
100	3,7	4,6	4,9	5,2	6,7	7,3	7,0	8,4	9,1
150	3,7	4,3	4,6	5,2	6,7	7,3	7,0	8,2	9,1
Tan	3,7	4,0	4,3	5,2	6,4	7,0	7,0	8,0	8,2

8.5.3 Superelevation of turning roadways

The minimum radii of curvature for the various design speeds are given in Table 8.4.2(b), and the maximum rate of superelevation appropriate to these radii is 10 per cent. Turning roadways at at-grade intersections are invariably short, however, and are very often not preceded by speed-change lanes, so that it is difficult in practice to achieve more than a nominal superelevation on these roadways.

If circumstances make it possible to apply superelevation to turning roadways, the superelevation rates indicated in Figure 3.5.2 are recommended.

8.6 Median openings

The general layout of median openings at intersections is normally dictated by wheel-track templates. However, median openings should not be shorter than:

- the surfaced width of the crossing road plus its shoulders, or
- 12,4 m;

whichever is the lesser;

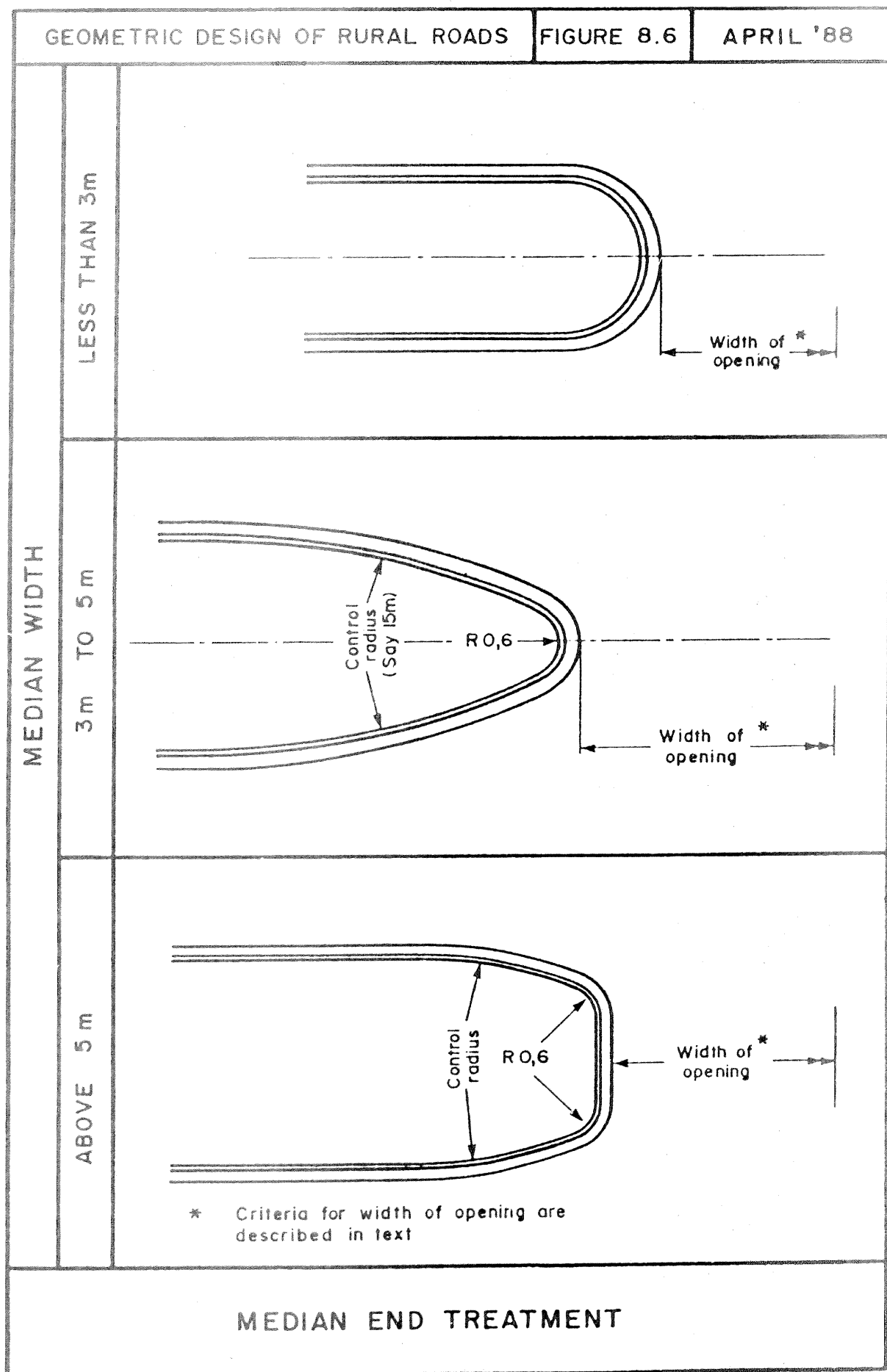
or, where there is kerbing, median openings should not be shorter than the surfaced width of the crossing road, plus 2,5 m.

A further control on the layout of the median opening is the volume and distribution of traffic passing through the intersection area. If the median is wide enough to accommodate them, it may be advisable to make provision for speed-change and storage lanes. The additional lanes will reduce the width of the median at the point where the opening is to be provided, and thus influence the median end treatment.

There are three possible shapes of median end to be considered. The simplest is a semicircle, and this is adequate for medians of up to 3 m wide. For medians wider than 3 m, a bullet-nose end treatment is preferred. The bullet-nose is

formed by two portions of control radius arc and an assumed small radius, eg 0,6 m. The bullet-nose closely follows the path of the inner rear wheel of the design vehicle and results in less intersection pavement and a shorter length of opening than the semicircular end. For wider medians, a bullet-nose end requires shorter lengths of opening, and above a width of 5 m the minimum lengths to provide for cross-traffic, as listed above, become controlling. At this stage the bullet nose end should be replaced by a flattened bullet nose, the flat end being parallel to the centre line of the crossing road.

The advantages of the bullet-nose and the flattened bullet-nose over a semi-circular end treatment are that the driver of a right-turning vehicle channelized for most of its path has a better guide for the manoeuvre, and the elongated median is better placed to serve as a refuge for pedestrians crossing the divided road. The bullet-nose curves help to position the right-turning vehicle to turn towards the centre line of the crossing road, whereas the semi-circular curve directs this vehicle into the opposing traffic lane of the crossing road.



9 INTERCHANGES

9.1 Introduction

An interchange is an intersection at which conflicts between different traffic movements are resolved by introducing a vertical separation between them. The complexity of the layout of the interchange may vary from separation of the through flows only, with turning taking place at the level of the lesser movement, to separation of all the movements.

Although certain combinations of ramp type recur so often that these combinations have acquired names, it must be remembered that each interchange represents a unique combination of through and turning movements. A more effective interchange will therefore be produced by considering the individual movements and the traffic volumes associated with each, and building up the layout, than by selecting a layout type and then forcing the various ramps to fit it. The discussion of the various types of interchange and their application forms the major part of this chapter; different ramps and their application are also discussed. The detail of geometric design elements is dealt with in Chapter 10.

9.2 Ramps and their application

9.2.1 Ramp terminals

There are two types of terminals that can be employed as part of a ramp: the free-flowing terminal and the stop-condition terminal.

The term *free-flowing* implies that the terminal is negotiated at more or less the speed prevailing on the through road. Traffic on the terminals thus diverges from or merges with traffic on the through road at very flat angles, so that the terminal consists of either a simple taper or a combination of taper and parallel lane. The parallel type forces a reverse curve path on vehicles negotiating the terminal, and is therefore not favoured. It can usefully be applied, however, where a ramp must be lengthened to allow for acceleration or deceleration, on a steep grade for example, and where this length cannot be provided by any other means.

Major forks, where two routes of roughly equal importance diverge, and branch connections, where such routes merge, are also free-flowing terminals. The principal difference between these and other free-flowing terminals is that the design of the former is based on auxiliary lanes, as required by lane balance (discussed in Section 9.4) rather than on the provision of tapers.

The discussion of at-grade intersections in Chapter 8 also applies to stop-condition terminals. The only variation to be found in these terminals is their location, which may be either remote from the grade separation structure or close to it. Individual terminals may vary from a simple bellmouth to a complex signalized channelized intersection, depending on the relationship between the turning movements at the intersection, and their magnitude. The common denominator amongst stop-terminals is that the ramps are designed for one-way operation, so that where these terminals are used (with the exception of the Parclo ramp) one leg of a three-legged intersection and two legs of a four-legged intersection are always intended for one-way flow, a characteristic not shared by all other intersections.

9.2.2 Ramp types

There are three basic ramp types, as sketched in Figures 9.2.2(a) and 9.2.2(b).

The outer connection is provided for left-turning vehicles. The diamond ramp is a modification of the outer connection, the only difference between the two being that the outer connection commences and terminates under free-flowing conditions, whereas the diamond ramp begins with a free-flowing terminal and ends with a stop-condition terminal. This makes it possible to accommodate right turns at a diamond ramp.

Two possible ramp configurations can be considered for a right turn. Where traffic conditions are light, a loop ramp is often employed. Turning vehicles are required to negotiate a 270° change of direction at relatively low speeds, but the loop has the advantage that it does not require an additional structure, although it invariably requires lengthening of the main structure. Loops commence and terminate under free-flowing conditions.

Replacement of the end terminal with a stop condition terminal makes it possible to accommodate left turns on the loop. Under these circumstances the loop ramp is often referred to as a Parclo ramp, although, strictly speaking, this is a misnomer (see Subsection 9.3.1). The Parclo ramp can be used as a substitute for a diamond ramp where topographic restraints or the development of the surrounding area would prohibit construction of a diamond ramp, or make it more expensive than the alternative.

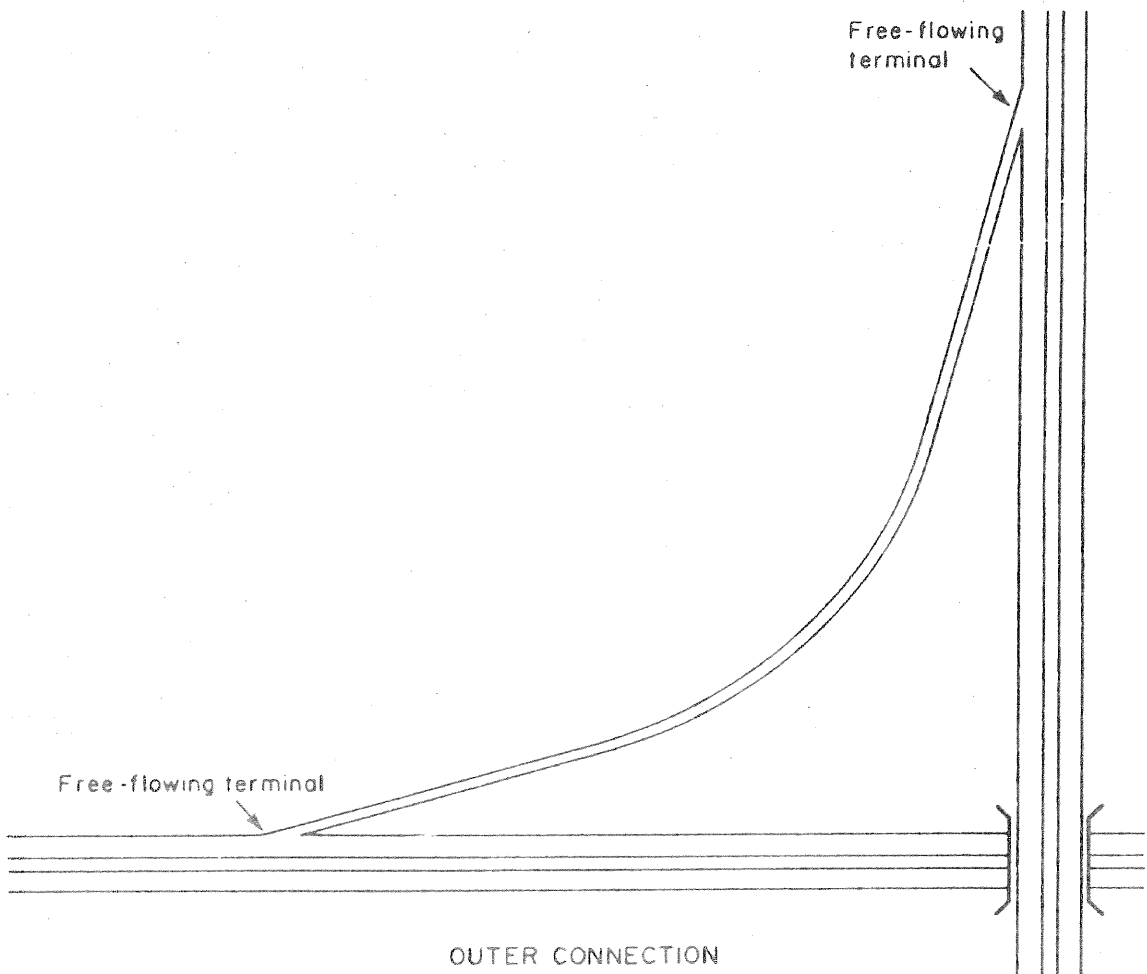
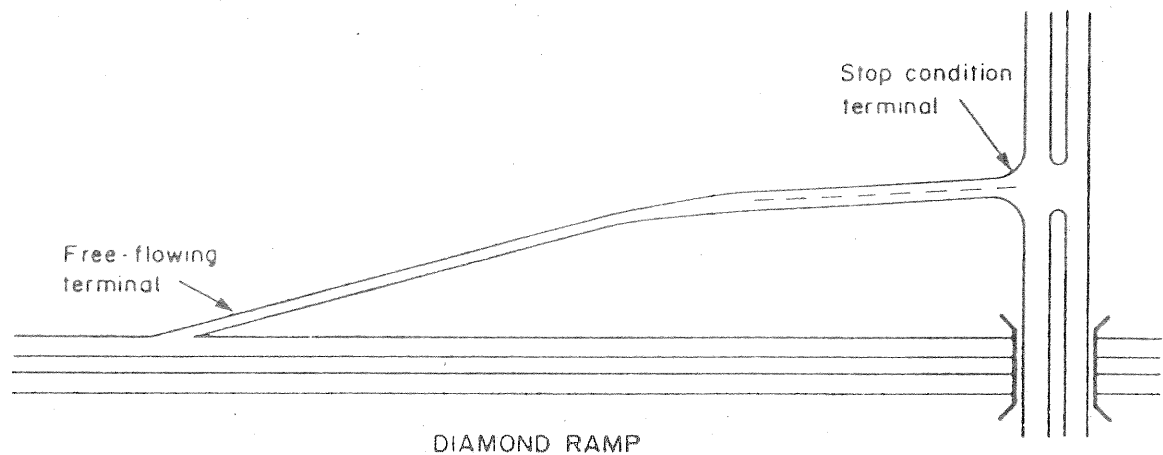
The low travel speeds on a loop cause it to have a low capacity, so that, if the right turn involves a high traffic volume, a more directional ramp can be used with advantage. Purely directional ramps have the disadvantage that turning traffic diverges from and merges with through traffic from the right, whereas these lanes are intended for fast-flowing through traffic. The semi-directional ramp, where traffic departs from and joins through traffic from the left, is preferred, because this is more consistent with drivers' expectations regarding turns at interchanges, and leads to smoother operation. Signposting of the interchange, which is a major factor in the ease of operation at interchanges, is also vastly simplified.

9.3 Interchanges and their application

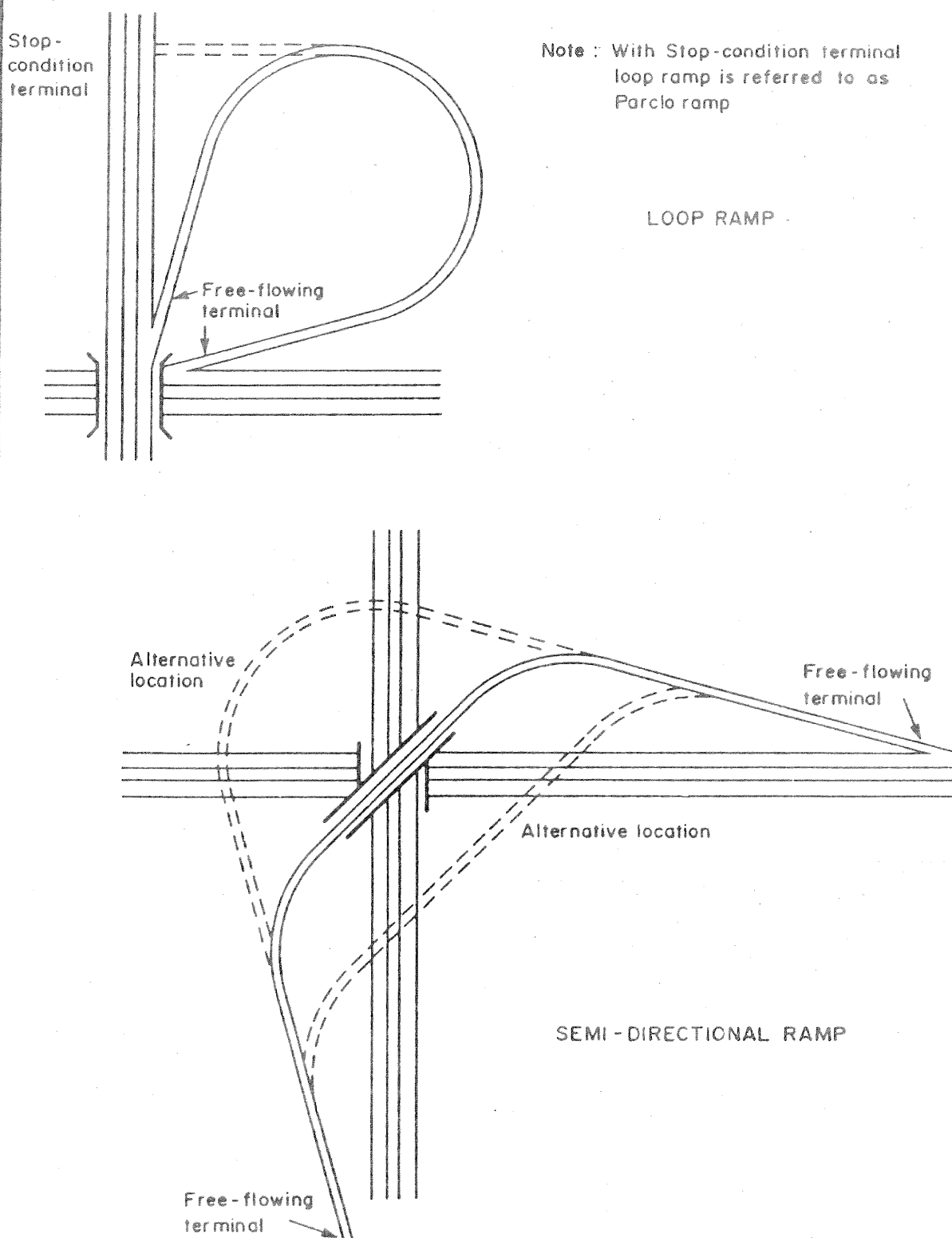
Interchanges are divided into two functional classes, referred to as access (or minor or service) interchanges, which serve local areas by providing access to freeways, and systems (or major) interchanges, which serve local areas by providing links between freeways. These two fundamentally different applications require different types of interchange layout.

Access interchanges are between freeways and roads on which at-grade intersections occur. Generally, the ramps at an access interchange can be regarded as surrogates for the major highway, with the crossing road ramp terminals functioning as major at-grade intersections on that highway. It is possible, by judicious selection of ramp type, to favour heavy turning movements to a greater extent than is possible with a normal at-grade intersection.

The systems interchange has ramps with free-flowing terminals at both ends, and the volume of turning movements is high so that there is a need for high design speeds on the ramps. All turning movements are separated, and, ideally, weaving in the interchanges is reduced to a minimum.



RAMP TYPES FOR LEFT TURNS

**RAMP TYPES FOR RIGHT TURNS**

9.3.1 Access interchanges

A frequently employed access interchange is the diamond, as shown in Figure 9.3.1(a). It has free-flowing terminals onto the freeway, and the single exit simplifies signing of the freeway. A further advantage is that it is economical to construct. The principal disadvantages of the diamond are that the right turns have an adverse effect on the capacity of the crossing road, and that there is difficulty in obtaining adequate shoulder sight distance at the crossing road ramp terminals when these are located close to the grade separation structure. Turning traffic leaving the freeway is obliged to stop at the crossing road ramp terminal, and inadequate storage can result in the queue of waiting vehicles extending back onto the freeway. Wrong-way movements occur more easily at diamond interchanges than at any other, and require these interchanges careful design.

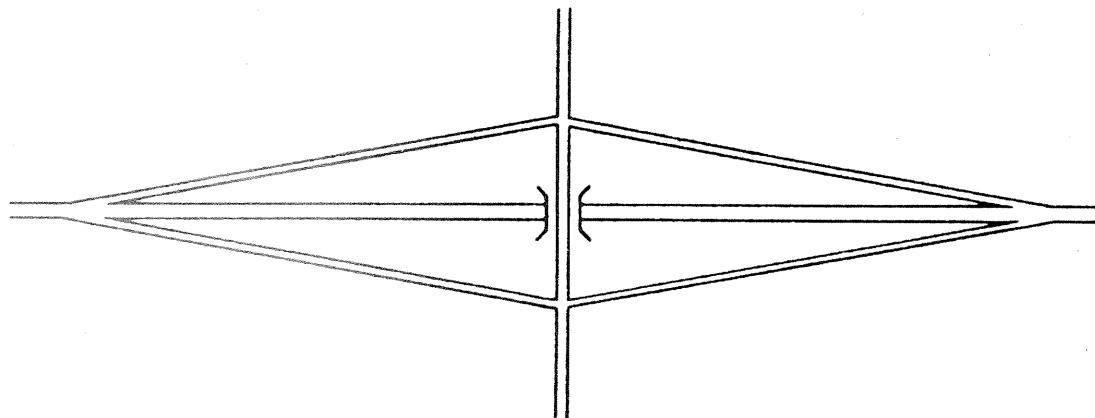
A distinction is drawn between the wide and the narrow diamond interchange.

In the wide diamond the crossing road ramp terminals are remote from the grade separation structure, whereas in the narrow diamond the crossing road ramp terminals are close to it, with the sight distance problem mentioned above. The wide diamond locates the crossing road ramp terminals more favourably in terms of sight distance and possibly also in terms of height of fill. Because of the greater area in the quadrants bounded by the ramps, the wide diamond has greater possibilities with regard to future expansion of the interchange to handle increased volumes of traffic. The greater requirement for land does, however, impose additional expropriation costs. The other disadvantage of this diamond is that it imposes a longer travel path on right-turning vehicles.

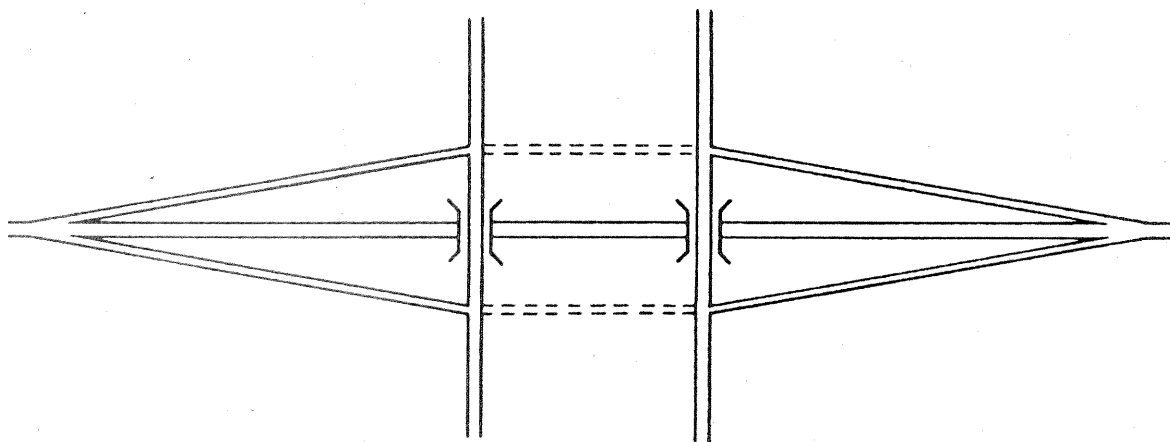
A modification of the diamond is the split diamond, which involves the use of two crossing roads. The four-legged crossing road ramp terminals of the simple diamond are replaced by three-legged intersections. Turning traffic is spread across two crossing roads, reducing the volume of turning traffic on each, and hence reducing the impedance that they present to other vehicles. The split diamond is, in essence, a combination of two incomplete interchanges. It is generally considered good design practice to have a turning movement and its reverse flow located at the same interchange, which precludes the possibility of a longitudinally split diamond. A further variation is the transposed split diamond, which generates weaving manoeuvres on the freeway, and thus has restricted application.

The Parclo (PARTial CLOverleaf) interchange is a cloverleaf (see Section 9.3.2) without its full complement of loops. The remaining loops originally had free-flowing terminals at both ends, which produced a combination of free-flowing and stop-condition terminals within a relatively short distance on the crossing road. The crossing road free-flowing terminals have subsequently been replaced by stop-condition terminals. Invariably, these terminals are combined with the outer connection terminals to form conventional three-legged intersections. Although the Parclo owes its origin to the cloverleaf interchange, the modification of the loop ramp now causes it to be more in the nature of a distorted diamond ramp.

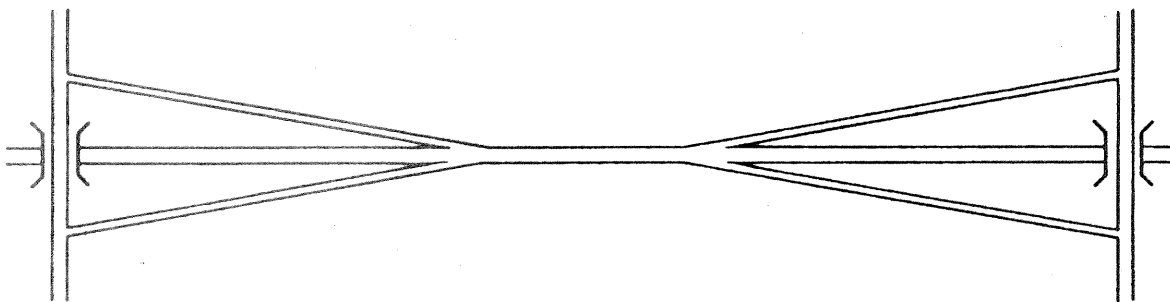
If the traffic flow on the crossing road of the interchange is heavy, and the major turning movement from the freeway to the crossing road is to the right, use of a Parclo ramp will be beneficial. The effect of the ramp is to transpose the turns so that the right turn becomes a left turn off the ramp. This generally eases the traf-



SIMPLE DIAMOND



SPLIT DIAMOND



TRANSPPOSED SPLIT DIAMOND

DIAMOND INTERCHANGES

fic flow and improves the capacity of the crossing road by either eliminating traffic signals altogether or reducing three signal phases to two.

There are three generally used configurations of Parclo interchanges as illustrated in Figure 9.3.1(b), named according to the quadrants in which the loops are located. The Parclo-A has the loops located in advance of the grade separation structure, and the Parclo-B has the loops beyond the structure. The Parclo-AB has the loops on the same side of the crossing road. A fourth configuration has the loops on the same side of the freeway, so that vehicles using the loops are forced to weave either on the freeway or on an adjacent collector-distributor road. This configuration, like the transposed split diamond, has restricted application.

9.3.2 Systems interchanges

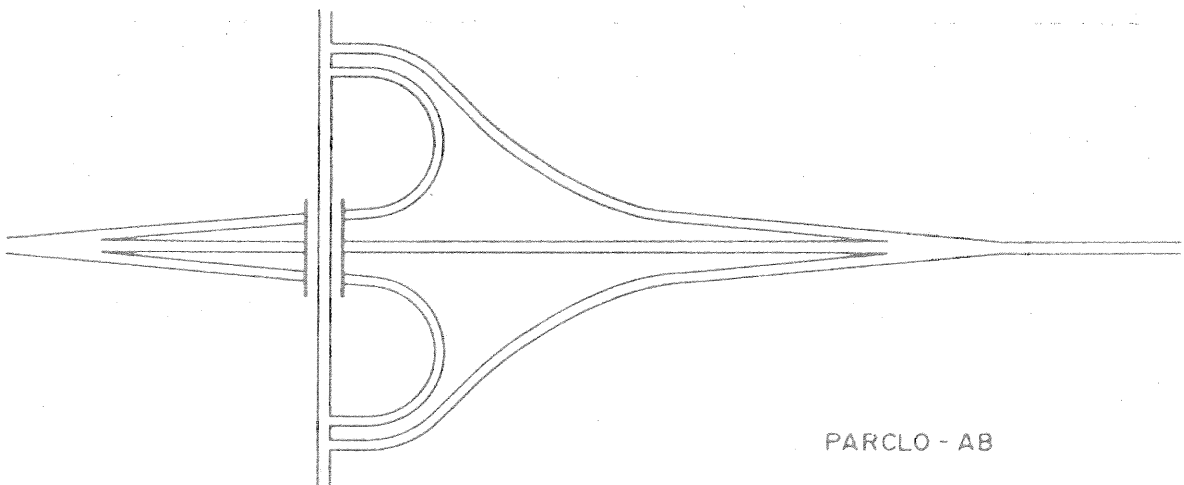
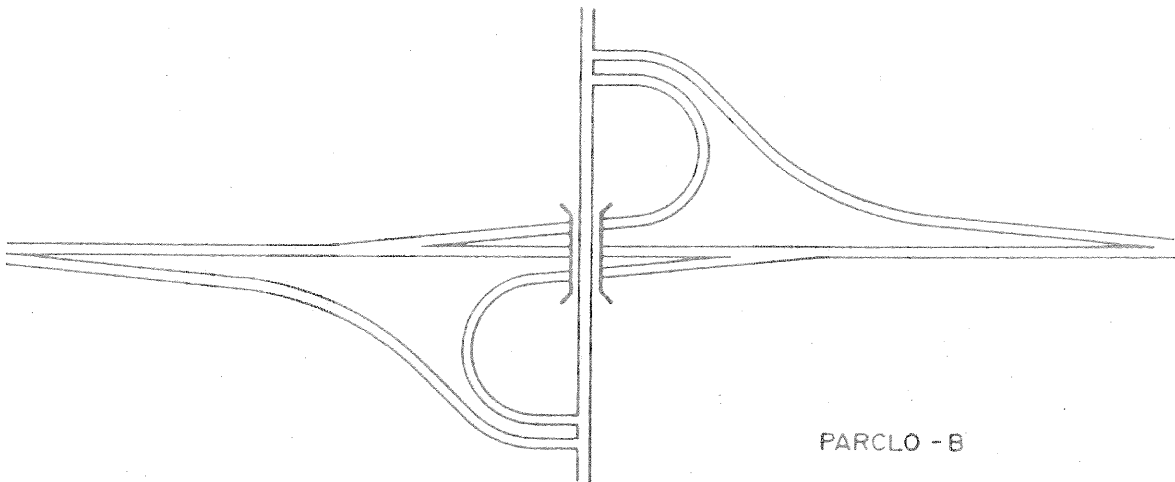
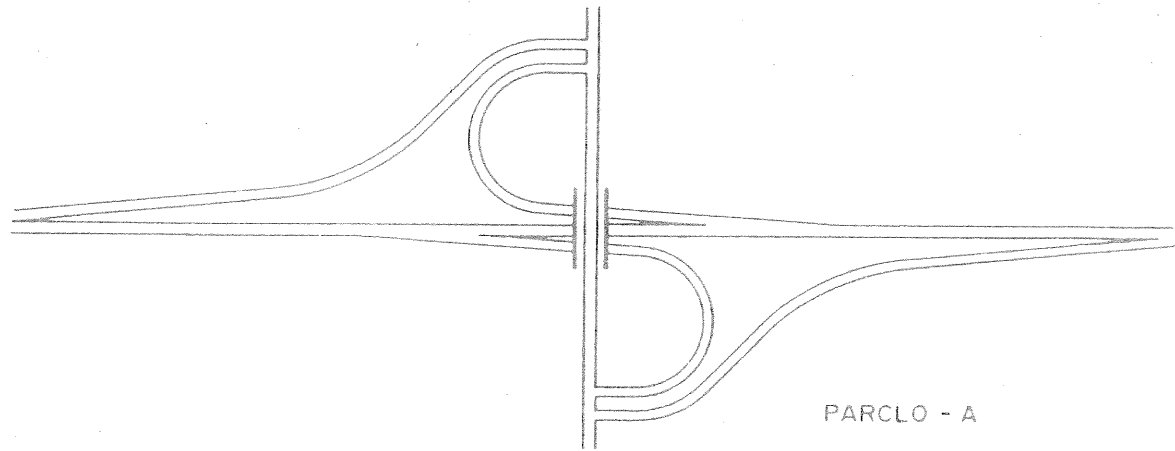
In systems interchanges, turning movements are catered for by individual ramps, all of which have free-flowing terminals at both ends. The layout of these interchanges is invariably complex, involving a substantial area and possibly more than one structure.

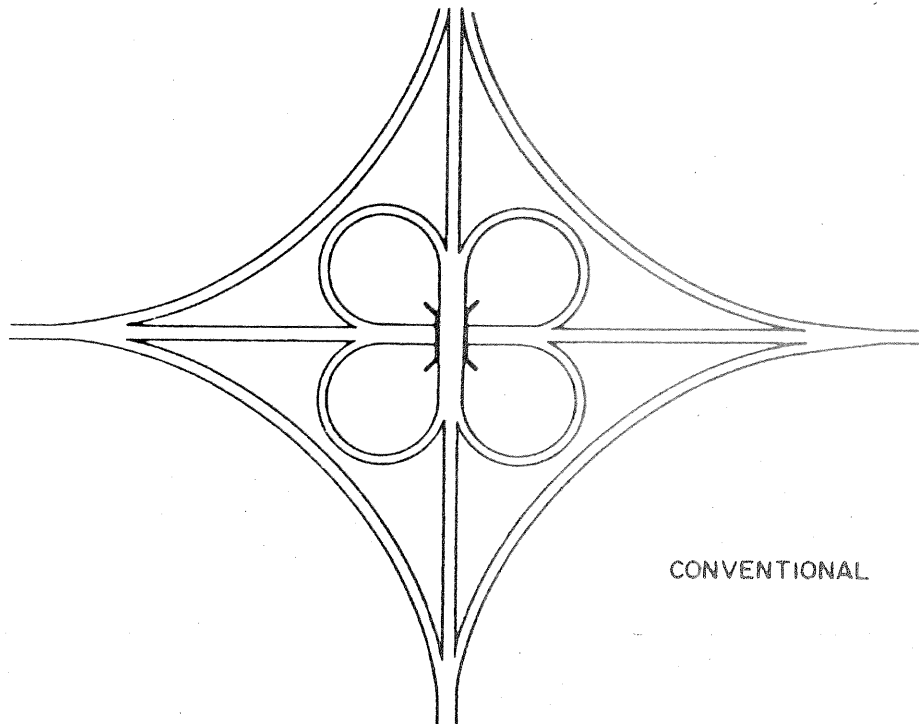
The cloverleaf owes its characteristic layout to the provision of an outer connection for each left turn and a loop for each right turn, as shown in Figure 9.3.2(a). This layout has only one structure but requires considerable space. The principal disadvantage of the cloverleaf is that all right-turning traffic is required to weave. In the conventional cloverleaf this manoeuvre takes place on the freeway, which disrupts the smooth flow of through traffic. If the loops commence and terminate on collector-distributor roads located alongside the carriageway, the situation is improved because the frictional effect on the through traffic is eliminated, and the weaving traffic can carry out this manoeuvre unhindered by through traffic on the outside lane of the freeway.

The directional interchange provides all the right turns with semi-directional ramps. These ramps can be restricted to crossing at a common point, as illustrated in the four-level interchange shown in Figure 9.3.2(b), in which four levels of grade separation and six bridge decks are involved. Removing one of the levels makes another four structures necessary, as in the three-level interchange also illustrated in Figure 9.3.2(b).

The two types discussed above can be considered extreme cases, ie all right turns on loops or all right turns on semi-directional ramps. Between these two extremes are layouts where one or more of the right turns is provided with a loop and the rest with semi-directional ramps. This has the effect of eliminating one or more structures and can thus be used with advantage when traffic on one of the right turns is relatively light.

When two loops are used in a systems interchange, one of the freeways may have both loops to one side so weaving may take place, as previously discussed. Generally loops are located in diagonally opposite quadrants. Interchanges with this layout are referred to as 'directional interchanges' with a reference to the number of loops included in the layout, eg Directional Interchange with one Loop.

**PARCLO INTERCHANGES**

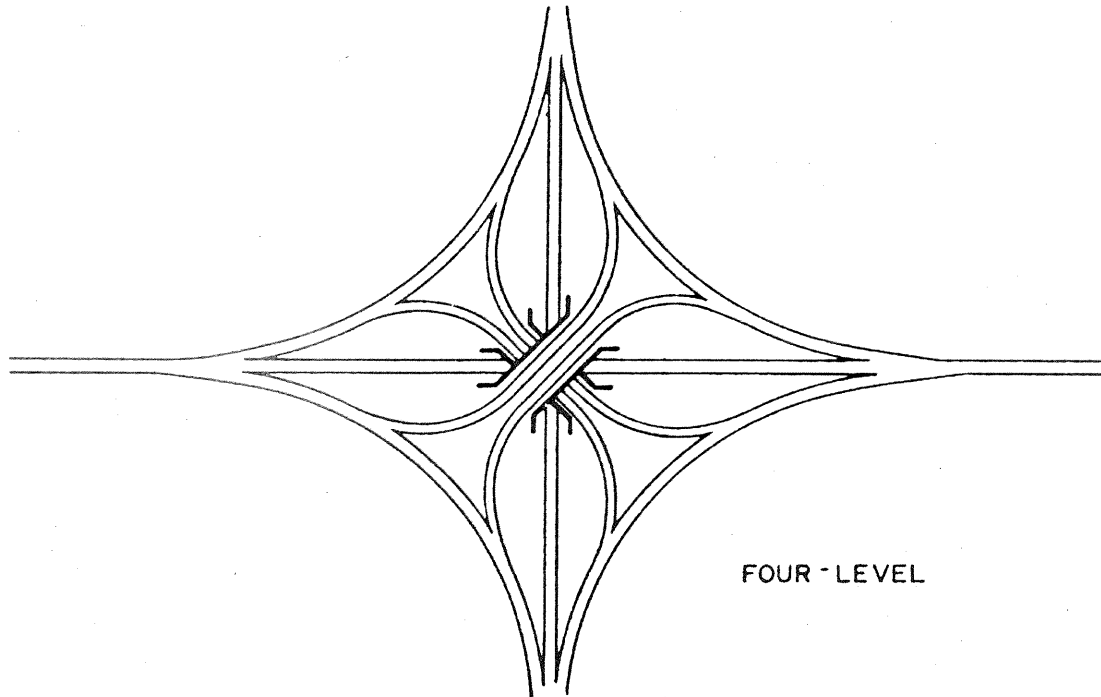


CONVENTIONAL

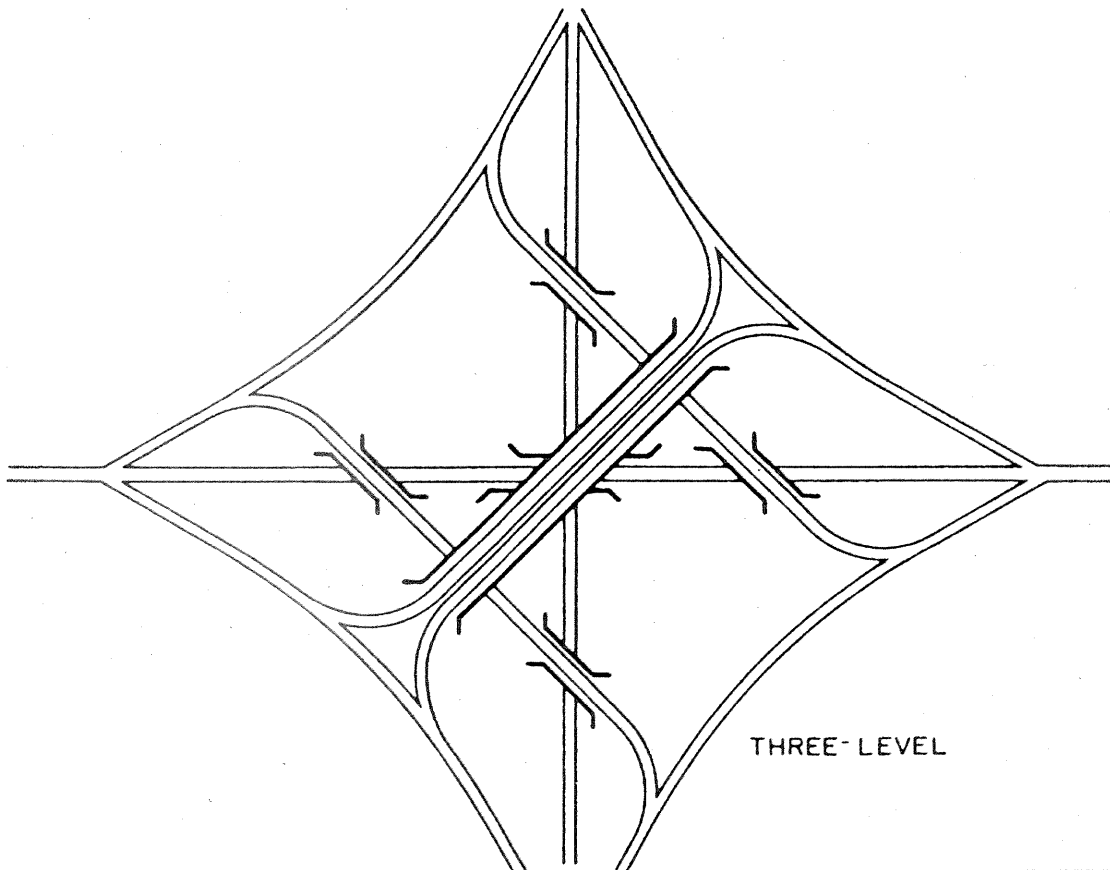


WITH COLLECTOR-DISTRIBUTOR
ROADS

CLOVERLEAF INTERCHANGES



FOUR-LEVEL



THREE-LEVEL

DIRECTIONAL INTERCHANGES

9.3.3 Three-legged interchanges

The discussion of access and systems interchanges has so far been limited to four-legged interchanges. A freeway terminating at its intersection with another would however give rise to a three-legged systems interchange. Likewise, access to a local area on one side of a freeway would require a three-legged access interchange, although, in this case, it is often advantageous to allow for development of the area on the other side of the freeway by constructing a four-legged interchange in the first instance.

Left turns take place on outer connections and right turns are provided with either loops or semi-directional ramps. It is not customary to provide both right turns with loops, as this will give rise to weaving on the through road. The combinations employed are usually one semi-directional ramp and one loop, or two semi-directional ramps.

The combination of semi-directional ramp and loop is referred to as a Trumpet (or Jughandle) interchange. The heavier of the right turns would normally be favoured by placing it on the semi-directional ramp, so that two configurations are possible: the Trumpet-A and the Trumpet-B. As in the case of the Parclo interchange, the suffix indicates whether the loop is in advance of or beyond the grade separation structure. Trumpet interchanges are illustrated in Figure 9.3.3.

If both right turns are on semi-directional ramps, the third layout shown in Figure 9.3.3 results.

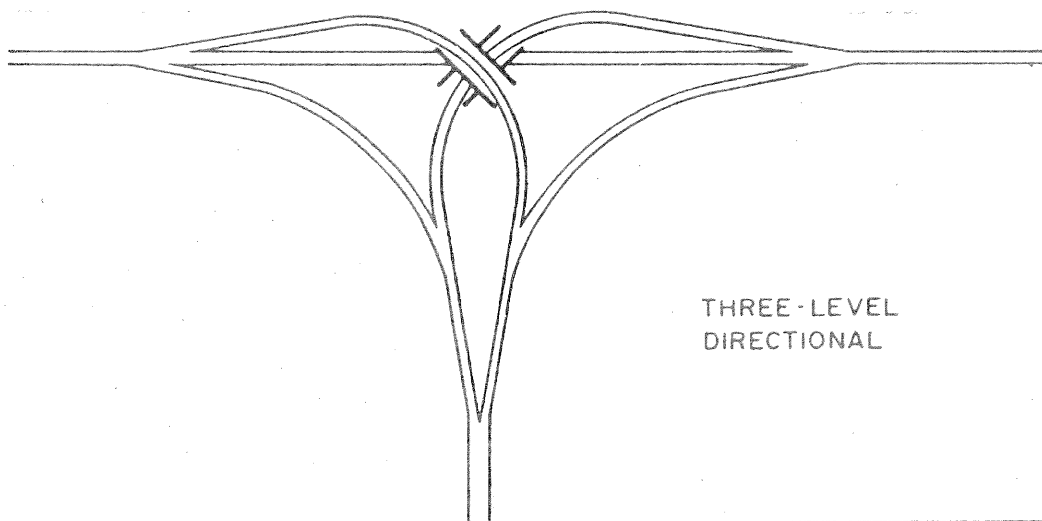
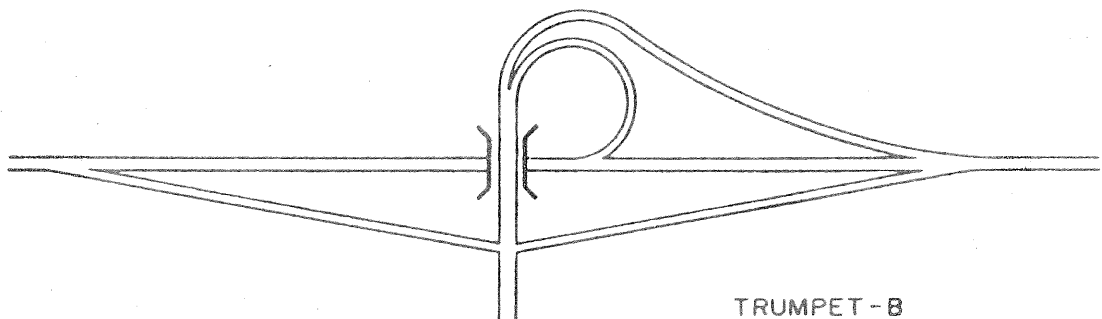
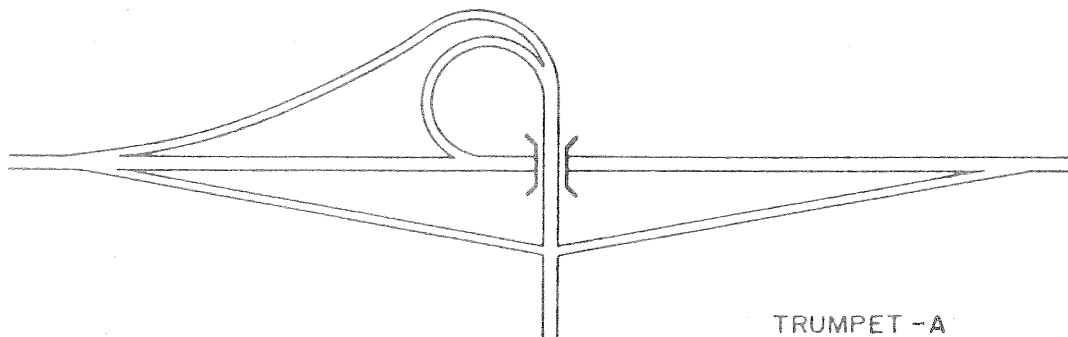
9.4 Lane balance

If a lane were to have an off-ramp as its only destination, a driver in that lane would have no choice but to exit from the freeway, whether this was his intention or not. The lanes should reflect the various options permitted at the interchange by branching off where vehicles diverge. In a two-lane exit, the outside lane has to continue along the off-ramp with the second lane branching off. Proper signing is called for to ensure that the driver is aware of the fact that the outside lane is being discontinued.

It would be easier for the entering driver not to have to merge with high-speed traffic on the freeway, but it is clearly impractical to add a further lane to the freeway at every on-ramp; the driver has no option but to merge, and this is reflected in the merging of the on-ramp with the outside lane. At a two-lane entrance the outside lane is added to the total number of lanes on the freeway, and the second lane merges with the outside lane of the freeway.

Lane balance gives the driver the option of continuing along the freeway or leaving it without having to change lanes to achieve his object.

Allied to the concept of lane balance is continuity of the basic number of lanes on the freeway. It is possible that the volume of traffic exiting from the freeway at a particular interchange may be sufficiently high to warrant discontinuing a lane beyond the off-ramp, with the volume of entering traffic at the same interchange sufficiently high to require the addition of a lane beyond the on-ramp. The short length of freeway between the ramp terminals would thus have one lane fewer than the freeway on either side of the interchange. In practice the prediction of the volumes turning to given destinations along the freeway and the assignment of this traffic to specific interchanges cannot match the degree



THREE - LEGGED INTERCHANGES

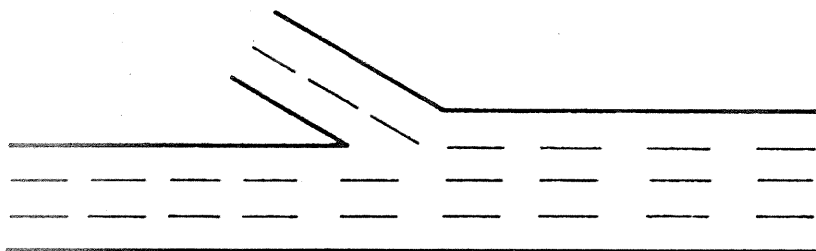
of precision required to add and drop lanes over short distances. Furthermore, this does not allow for the possibility that a given exit from the freeway may be closed by an accident, for maintenance, or because of some circumstance off the freeway. All the traffic that would normally have exited at that point would therefore have to travel to the next exit under circumstances of extreme congestion. Continuity of the basic number of lanes over a reasonable length of the freeway contributes to ease of operation on the freeway and affords the driver flexibility in the selection of routes to his chosen destination.

Figure 9.4 illustrates the application of lane balance and continuity of the basic number of lanes at interchanges.

9.5 Weaving

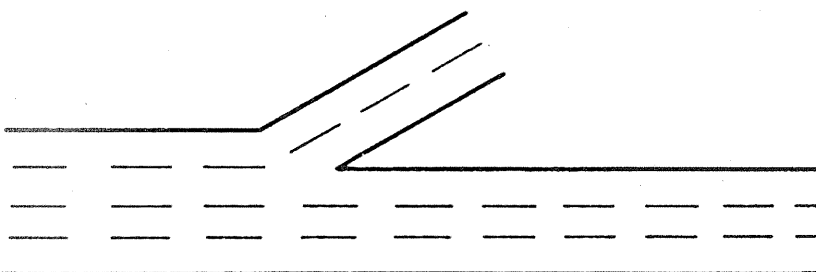
Weaving involves two flows of vehicles crossing one another at a flat angle, and the section of road on which this is accomplished is referred to as a weaving section. In its simplest form the weaving section consists of two lanes that merge to form one lane and then diverge to form two lanes again. This form of weaving section is found between the loops of a cloverleaf interchange, and the traffic flows involved are two inner or crossing flows, and one outer or through flow. More often the weaving section includes two outer flows in addition to the two inner flows. Both of these weaving sections are referred to as simple weaves, because vehicles correctly positioned to carry out the weaving manoeuvre do not change lanes, lane balance ensuring that the required operations are merging followed by diverging. Successive closely-spaced entrances and exits may, however, require a lane change to be incorporated into the weaving process. Under these circumstances the process is termed compound or multiple weaving.

In previous sections it has been suggested that weaving should be avoided if at all possible. This does not imply that weaving should be avoided at all costs. The alternative to allowing weaving is to provide the crossing traffic flows with a grade separation, but this is not economically justifiable if the flow volumes are light. If the distance required to effect a grade separation is not available, there may be no alternative to a weaving section.



THROUGH LANES UPSTREAM + RAMP LANES - 1
= THROUGH LANES DOWNSTREAM

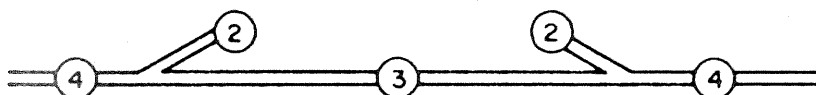
MERGE



THROUGH LANES UPSTREAM
= THROUGH LANES DOWNSTREAM + RAMP LANES - 1

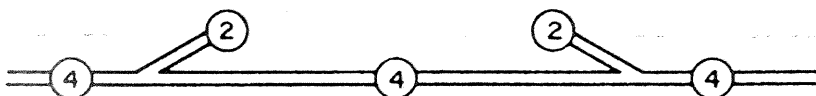
DIVERGE

1.



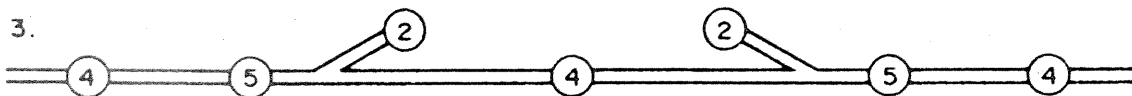
Lane balance , but no continuity of basic number of lanes

2.



Basic number of lanes but no compliance with lane balance

3.



Continuity of basic number of lanes and lane balance

CO-ORDINATION OF LANE BALANCE & BASIC NUMBER OF LANES

LANE BALANCE AND CONTINUITY

10 INTERCHANGE DESIGN

10.1 Introduction

In the previous chapter, the various components of interchanges were discussed.

This chapter discusses the warrants for the provision of an interchange. Suggestions are offered with regard to the dimensions of the components of the interchange, and guidelines proposed for the selection of the design speeds of ramps and hence their horizontal and vertical alignment.

10.2 Warrants for an interchange

Although Section 9.3 discussed interchanges chiefly in the context of freeways, interchanges do have other applications as well. These applications are discussed below.

The need to enhance ease of flow could lead to the decision to limit access to a given road, and it is this need for control of access that warrants the provision of an interchange. Even if full control of access is not applied, the flow of traffic, expressed in terms of a level of service, can still be enhanced by an interchange. Where two expressways intersect, it may be found that traffic volumes are too high to be accommodated at an at-grade intersection, regardless of the level of sophistication of the provision made for turning movements by means of channelization, signalization and auxiliary lanes for through traffic. Generally, if an intersection is likely to become a bottleneck, and all possibilities for improving its capacity have been exhausted, an interchange is warranted. In the planning of a new road under circumstances where, owing to close spacing or heavy traffic volumes, for example, the intersections control the level of service of the route, it may be necessary to consider the provision of interchanges at points where the design level of service cannot be achieved with intersections.

A further warrant for an interchange is its potential contribution to the safety of the road user. An existing intersection may be hazardous for a variety of reasons, for example, steep gradients on one or more legs of the intersection or a heavy turning movement where drivers may feel obliged to accept very small gaps. In rolling terrain the most suitable available site for an intersection may have restricted sight distance, or be located on a near-minimum-radius curve.

An interchange may thus be the solution to problems of road safety or topographic restraints.

As the provision of an interchange is inevitably costly, the designer should not accept that an interchange is the best solution to a problem without exhaustive analysis of the alternatives. Such an analysis should weigh the effect of the longer travel paths of turning vehicles against reduced delay and probably some reduction in the number of accidents.

10.3 Minimum spacing of interchanges

In the previous chapter, reference was frequently made to the adverse effect of weaving on the operation of an interchange. Weaving can have an equally adverse effect on the operation of the freeway, when traffic entering the freeway at

one interchange is required to weave with traffic wishing to exit at the next interchange. Successive interchanges should therefore not be so closely spaced that weaving causes the level of service on the freeway to drop to an unacceptable level. There are two main factors to consider in determining the minimum acceptable spacing of interchanges.

The effective use of an interchange is determined in part by the clarity of its signposting, since the driver, particularly a driver unfamiliar with the area, must be given adequate advance warning of the location of exits to specific destinations. The signing of interchanges is dealt with in the South African Road Traffic Signs Manual². In that document it is suggested that a Class I Sign Sequence should begin with a Pre-Advance Sign mounted two kilometres in advance of the exit and end with a Confirmation Sign (depending on the distance to the next interchange), mounted about 750 m beyond the entrance. The signing will be equally effective if the Confirmation Sign is replaced by a Pre-Advance Sign to the next interchange.

When locating two consecutive interchanges, the designer should consider the distance required to eliminate weaving between them, and the distance required for effective signing, taking the greater of these two as the minimum acceptable distance between the interchanges. As a very rough guide, a distance of about 5 km would normally be the minimum acceptable spacing between successive interchanges on rural freeways.

Where a rural area is more intensively developed, as in the Pretoria-Witwatersrand-Vereniging area, for example the spacing of successive interchanges may be forced to less than the 5 km suggested above. Under these circumstances a distance of 2,4 km could be considered the minimum for successive access interchanges. The minimum spacing between an access interchange and a systems interchange should not be less than 3,6 km. As these distances are measured from intersecting road to intersecting road, it is clear that weaving between interchanges and the effective signing of the freeway will both require very careful consideration.

Rest areas and their location have an impact on signing requirements. This is expressed as the minimum distance between edge-line breakpoints, these being the far ends of the tapers of on- and off-ramps of successive interchanges or accesses to rest areas.

A distinction is drawn between two access conditions, being:

- Condition 1 where the rest area has access directly to the freeway, or
- Condition 2 access to the crossing road of the interchange.

Three alternative cases are also considered. These refer to the other element that has to be considered in the sign sequence between it and the rest area in question, and these are:

- Case 1 the other element is an interchange with a major road as its crossing road;
- Case 2 the other element is an interchange with a minor road as its crossing road;
- Case 3 the other element is another rest area with direct access to the freeway.

Finally, the location of the other element, namely whether it is upstream or downstream of the rest area, also has a bearing on the sign sequence employed and the corresponding minimum spacing between it and the rest area.

The relevant minimum distances are given in Table 10.3, and it is stressed that these distances are measured between edge-line breakpoints.

TABLE 10.3
*MINIMUM SPACING BETWEEN REST AREAS AND
ADJACENT INTERCHANGES*

CONDITION	CASE			LOCATION
	1	2	3	
1	2 340	1 440	1 440	D
	1 590	1 590	1 440	U
2	2 490	1 590	1 590	D
	1 700	1 700	1 550	U

10.4 Ramp design speed

The design speed for a ramp should be related to the design speed of the through and intersecting roads, and should preferably not be less than the operating speed of the through road. Ramp design speed can, however, gradually be reduced to half that of the through road under restricted circumstances.

In general, a design speed of 40 km/h is adequate for loops, as the advantages of a higher design speed will very often be nullified by the additional distance of travel resulting from the correspondingly larger radii required. As the free-flowing ramp terminal is designed to the speed of the through road it may be necessary to achieve the minimum radius by compounding it with larger radii as discussed in Section 10.6.

A semi-directional layout is selected for a given ramp when a high volume of turning traffic is expected. Free-flowing terminals at both ends of the ramp will accommodate traffic entering and leaving the ramp at speeds close to the operating speeds of the through and intersecting roads. A low design speed on the mid-section of the ramp will clearly have a restrictive effect on the capacity of the ramp and is therefore not acceptable. The minimum design speed of a semi-directional ramp should not be less than the speed suggested in Table 10.4.

TABLE 10.4
DESIGN SPEED OF SEMI-DIRECTIONAL RAMPS

THROUGH ROAD (km/h)	RAMP (km/h)
60	60
80	70
100	80
120	90
140	100

Direct connections, such as the outer connectors of a cloverleaf interchange, should also be designed for the speeds suggested in the table.

Diamond ramps always, and Parclo ramps usually, have a free-flowing terminal at one end and a stop-condition terminal at the other. The free-flowing terminal and the section of ramp immediately following should have a design speed equivalent to the operating speed of the through road, and the design speed should not be less than 80 km/h. After that the design speed may become progressively lower, but must be at least 40 km/h at the stop-condition terminal. As in the case of the loop, the Parclo ramp may also have a minimum radius appropriate to a design speed of 40 km/h.

10.5 Decision sight distance

Subsection 2.5.3 states that under certain circumstances it is necessary for the driver to be able to see the road surface for a given distance ahead measured from an eye height of 1,05 m. Free-flowing terminals are examples of the need for decision sight distance. Stop-condition terminals often have more than one lane, even though the major portion of the ramp may be a single lane only, and these lanes may be allocated specifically to one or another of the turning movements carried out at the terminal. It is essential for the driver to be able to see the road markings indicating this lane allocation. The decision sight distance for a ramp is based on the normal stopping sight distance given in Figure 2.5.1(a), but measured to an object height of 0,0 m rather than 0,15 m.

10.6 Horizontal curvature on ramps

The minimum radii of curvature given in Table 3.2.1 and Table 8.4.2(b) are calculated from the maximum rate of superelevation and the maximum allowable side friction factor appropriate to the design speed selected. There is no reason why the location of a curve on a ramp should modify the calculation, and the radii given in the tables referred to are repeated below for convenience. The restricted circumstances prevailing on a ramp may, however, provide an insufficient distance for the development of maximum superelevation, and the designer will then have to select a curve of larger radius to match the extent of superelevation development available to him.

TABLE 10.6
MINIMUM RADII OF HORIZONTAL CURVATURE ON RAMPS

DESIGN SPEED (km/h)	RADIUS (m)
20	10
30	25
40	50
50	80
60	110
70	160
80	210
90	270
100	350
110	430
120	530

It is generally accepted that changes in design speed should not be too marked, so that these changes should occur in increments not exceeding 10 km/h. The two lowest speeds in Table 10.6 apply to the design of stop-condition terminals, and the others to the design of the ramp. It will be noted that the ratio between succeeding radii is generally about 1:1,5. When a compound curve of above-minimum radii is being determined, this ratio can be employed to advantage.

Drivers are reluctant to brake sharply on a curve, and deceleration along a compound curve would take place under conditions of no, or at most gentle, braking. The successive curves forming the compound curve should thus each be long enough to allow the driver to match his speed to that judged appropriate to the following section of curve without sharp braking. This condition is achieved if the length of the arc is approximately a third of its radius.

10.7 Superelevation on ramps

The selection of a superelevation rate of 10 per cent as the maximum for open road conditions is based on the likelihood of there being vehicles in the traffic stream that will be travelling at speeds considerably different from the design speed. As ramp design speeds are lower than those on the through and intersecting roads of an interchange, it is reasonable to expect that vehicle speeds on ramps will more closely match the selected design speed, so that higher rates of superelevation can be adopted. Higher rates of superelevation would, however, require greater lengths for superelevation development, and because the necessary length would probably not be available, the maximum rate of 10 per cent is also applied to ramps.

The rates of superelevation applied to curves of more than minimum radius, as shown in Figure 3.5.2, are based on a maximum rate of 10 per cent, and can thus be applied to ramp curvature.

The development of superelevation on a ramp takes into account the comfort of the occupants of a vehicle traversing the ramp. Under the less restricted circumstances of the open road the length of development can be extended to enhance the appearance of the curve. It is convenient to express the rate of development in terms of the change in superelevation rate per unit length, as shown in Table 10.7. Grade line levels are usually calculated at 20 m intervals, and the table has been extended to include the rates of change over this distance.

TABLE 10.7
RATE OF SUPERELEVATION DEVELOPMENT

DESIGN SPEED (km/h)	RATE OF CHANGE PER m (%)	RATE OF CHANGE PER 20 m (%)
40	0,195	3,9
50	0,185	3,7
60	0,175	3,5
70	0,165	3,3
80	0,155	3,1
90	0,145	2,9
100	0,135	2,7

10.8 Crossover crown

A crossover crown line is a longitudinal line at which an instantaneous change of slope across the pavement occurs. The only difference between this and the normal crown of the road is that it may occur at any position across the pavement other than the centre of the road. The principal application of the crossover crown is at ramp tapers, where it can be used to begin the superelevation of the first ramp curve earlier than would otherwise be the case. The crossover crown may pose a problem to the driver, particularly of a vehicle with a high load, because the vehicle will sway as it crosses over the crown line, and, in extreme cases, may be difficult to control. For this reason, the maximum suggested algebraic differences in slope on either side of the crossover crown line are given in Table 10.8.

TABLE 10.8
MAXIMUM CHANGE IN SLOPE ACROSS CROSSOVER CROWN LINE

DESIGN SPEED (km/h)	ALGEBRAIC DIFFERENCE IN SLOPE (%)
20 and 30	5 to 8
40 and 50	5 to 6
60 and over	4 to 5

10.9 Vertical alignment of ramps

Having the crossing road of the interchange passing over the through road not only improves the target value of the interchange, but is also advantageous in terms of the vertical alignment of the ramps. The off-ramp will climb to the crossing road, reducing the distance required for stopping, and the on-ramp will drop to the through road making it easier for the driver to accelerate to the speed prevailing on the through road. In addition, the driver on the on-ramp will be better placed to observe the oncoming flow of traffic on the through road. A decision to have the crossing road over the through road will, however, have to include consideration of the restrictive effect that the bridge may impose on abnormal loads and the restraints placed on the vertical alignment of the two roads by the topography of the area and local conditions.

Under the above circumstances, a steep ramp is to be preferred provided that the gradient on the ramp does not exceed 8 per cent. In the vicinity of the stop-condition terminal, a maximum gradient of 3 per cent should be applied, as suggested in Section 8.2. If the off-ramp has a downgrade, and the on-ramp an upgrade, the gradients suggested may still be used, but allowance should then be made for the greater distances required for acceleration and deceleration.

10.9.1 Vertical curves

As stated, in Subsection 2.5.3, it is essential for the driver to be able to see the road markings on the ramp, and the suggested sight distance on ramps should be the normal stopping sight distance, but measured from an eye height of 1,05 m to the road surface. The vertical curvature required for this is given in Table 10.9.1(a).

TABLE 10.9.1(a)
MINIMUM VALUES OF K FOR VERTICAL CURVES ON RAMPS

DESIGN SPEED (km/h)	CREST CURVES	SAG CURVES
40	12	12
50	20	18
60	30	25
70	43	32
80	63	41
90	87	51
100	114	62
110	154	75
120	210	91

The minimum length of vertical curvature suggested for a ramp is based purely on aesthetics, but it will very often not be possible to achieve the minima suggested in Table 4.2.2. Lower values are proposed in Table 10.9.1(b), based on a length in metres equal to 0,6 of the design speed in kilometres per hour, rounded off to the nearest 10 m.

TABLE 10.9.1(b)
MINIMUM LENGTH OF VERTICAL CURVES ON RAMPS

DESIGN SPEED (km/h)	LENGTH OF CURVE (m)
40	30
60	40
80	50
100	60
120	80

10.10 Ramp cross-section

If a stalled vehicle blocks an off-ramp, the line of stopped vehicles will soon extend back to the freeway creating a hazardous situation and also affecting the quality of traffic flow on the freeway. The blocking of an on-ramp will lead to the blocking of the stop-condition terminal, impeding the flow of traffic along the crossing road. An overall ramp width of 8,0 m, comprising two shoulders 2,0 m wide and a lane 4,0 m wide, would be adequate for this situation and also allow for future conversion of the single lane into two narrower lanes.

The basic lane width quoted above refers to tangent sections of ramp. The widths given in Table 8.5.2 for turning roadways for Case 1, Traffic Condition B, can be used on curved sections of the ramp, on the assumption that there will be sufficient SU vehicles in the traffic stream to affect the selection of lane width. The availability of the surfaced shoulders for the passing of stopped vehicles makes it unnecessary to provide for passing within the width of the lane.

10.11 Ramp terminals

Bellmouths are used in the design of the terminal where the ramp joins the intersecting road and traffic enters the intersecting road at angles near to 90° . Tapers are used for vehicles entering or exiting from the through road at flat angles. The ramp terminal of the intersecting road should be designed in accordance with the guidelines in Chapter 8. Through road ramp terminals are discussed below.

The spacing of successive terminals should be such that the manoeuvres carried out by a driver entering at one terminal are not hampered by vehicles entering at the next terminal downstream. The distance between an entrance and the following exit should allow for weaving between the two terminals. An exit followed by another exit does not cause any driving problems, and if this were the only criterion, successive exits could be closely spaced. It is necessary however, for the driver to be able to differentiate clearly between the destinations served by two successive exits, and adequate space must be allowed for effective signing. A distance of 300 m between successive terminals is adequate for terminals located on the freeway itself. If successive terminals are on a collector-distributor road, or on the ramps of a systems, the distance between terminals can be reduced to 240 m. If the ramps on which successive terminals occur form part of an access interchange, the distance between terminals can be further reduced to 180 m. The distances suggested correspond to decision sight distance for the different design speeds that are likely to apply to the various circumstances.

Two types of free-flowing ramp terminal are discussed in Subsection 9.2.1, namely the parallel terminal and the taper. The parallel terminal involves a combination of a taper with a length of auxiliary lane, and is used when, because of steep gradients, an additional length is required for either acceleration or deceleration and the necessary distance cannot be obtained by other means. The length of the auxiliary lane would normally be 600 to 1000 m. These auxiliary lanes could also be introduced for the purpose of achieving lane balance at a terminal. The distance of 600 m corresponds to a travel time of 20 s, which is double the reaction time required for complex decisions.

Two different criteria apply to the selection of taper rate, depending on whether the ramp is an exit or an entrance.

If the ramp is an exit, the only task required of the driver is to negotiate a change of direction while not encroaching on either the adjacent lane or the shoulder. It is customary to indicate the start of the taper clearly by introducing it as an instantaneous change of direction rather than as a gentle curve. If a crossover crown is not used, the crossfall across the taper will be the same as the crossfall on the through lane, ie 2 per cent. This corresponds to the superelevation applied to a curve of radius 2700 m at an operating speed of 100 km/h. A vehicle can be contained within the width of travelled way available to it while negotiating a curve of this radius if the taper rate is in the region of 1:15. Higher design speeds and hence higher operating speeds require flatter tapers, and lower design speeds would make it possible to consider sharper tapers, as shown in Table 10.11.

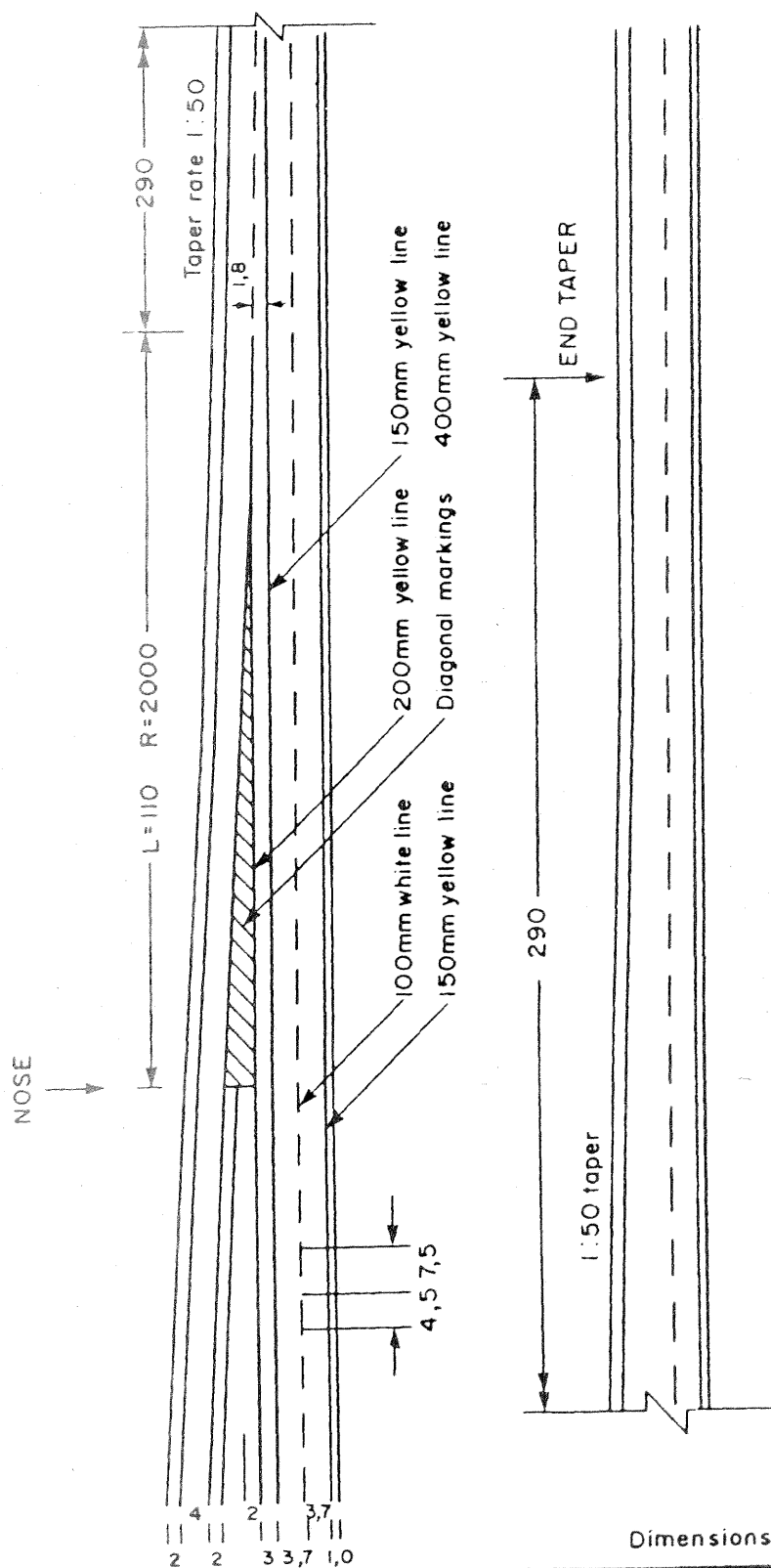
At an entrance taper the driver must, in addition to negotiating a change of direction, merge with through traffic in the outside lane of the through road. It has

been found that a rate of convergence of about 1:50 provides an adequate merging length.

Figures 10.11(a) to (d) illustrate tapers that can be used for single and two-lane entrances and exits.

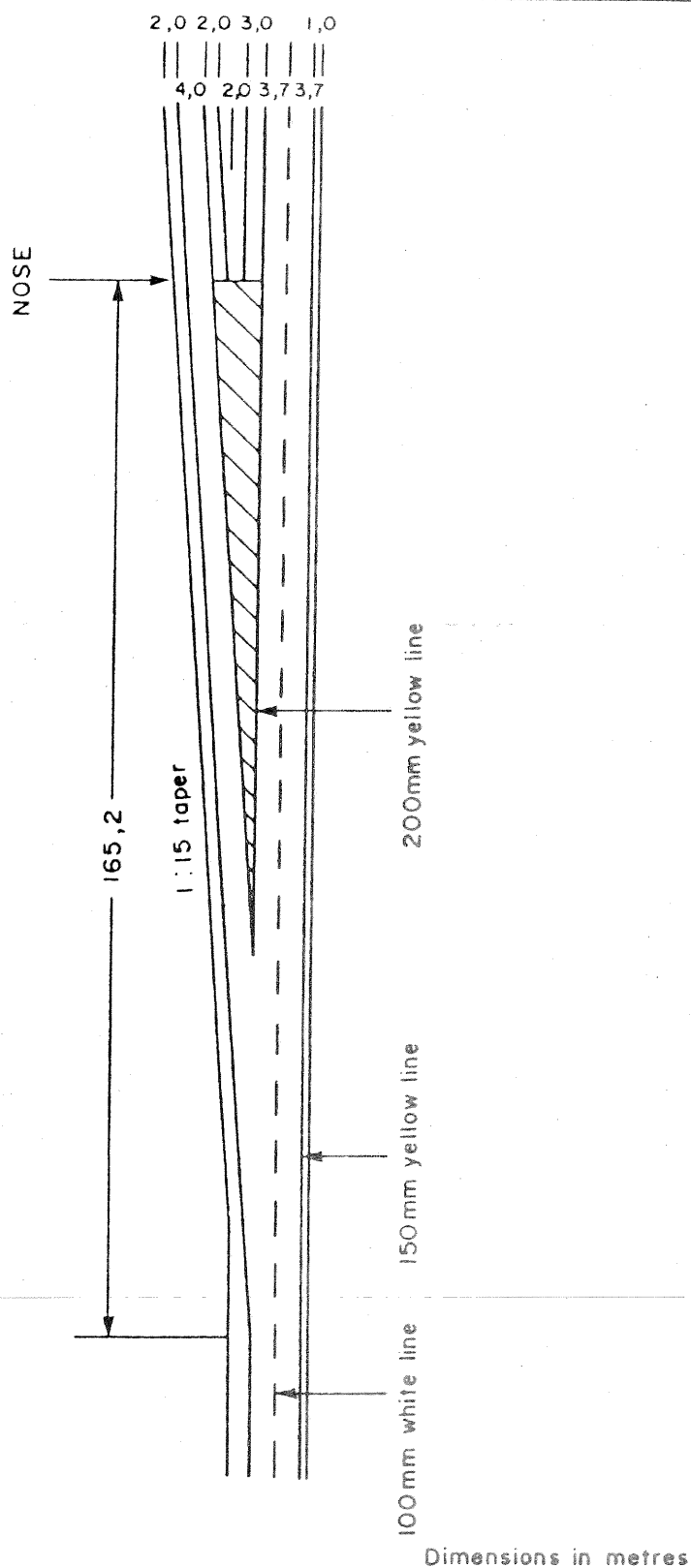
The dimensions shown on these figures are also appropriate for major forks and branches. The main difference between forks and branches and ramps is that the first two are a continuation of through roads. In the case of forks and branches, ramp speeds would not be used in design and the restriction on exiting or entering from the right does not apply.

Note: For 50m in advance of the nose, the ramp should not be more than 0,75m lower than the freeway

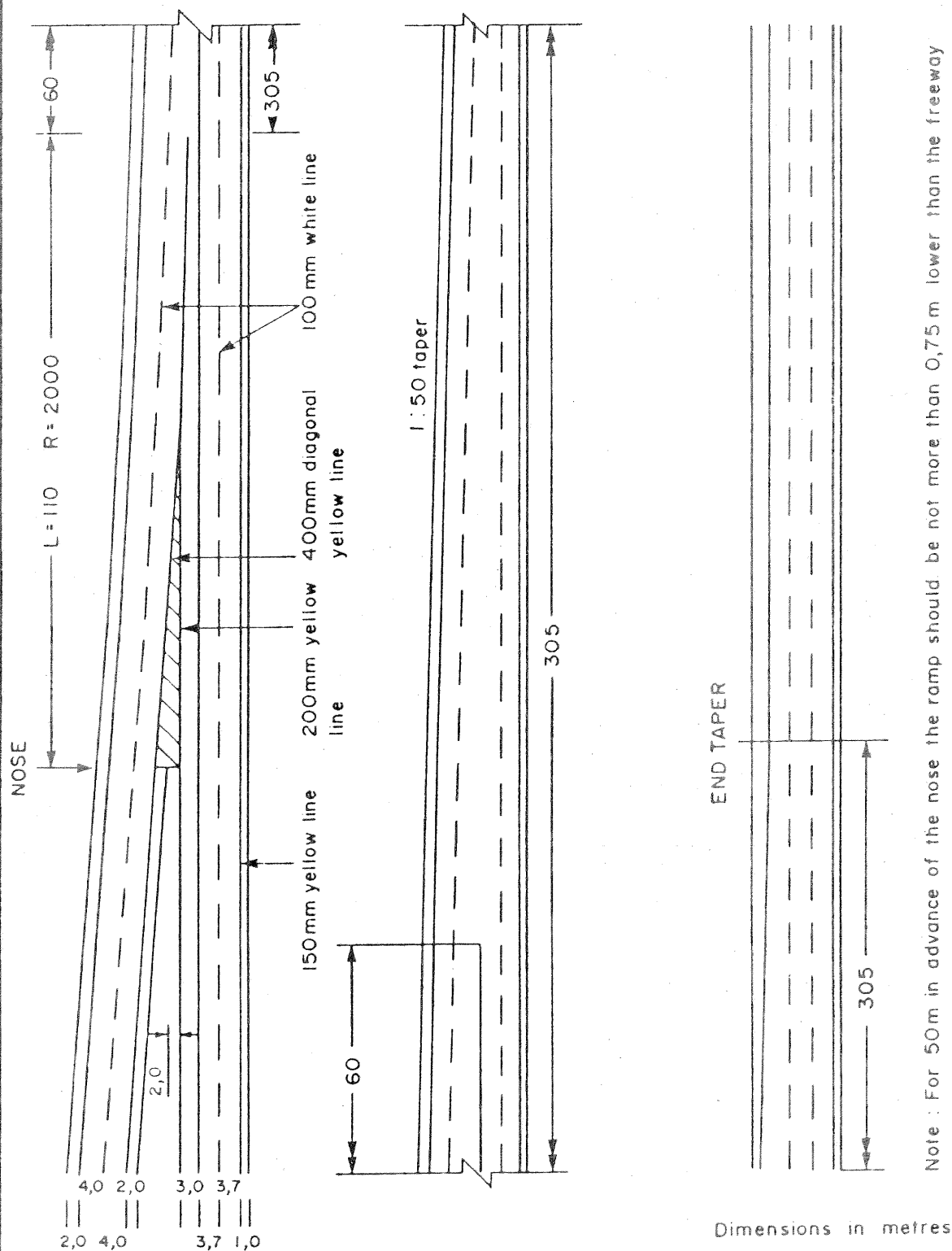


SINGLE-LANE ENTRANCE

Note: Sight distance of 300 m measured from an eye height of 1,05 to the road surface should be provided in the area in advance of the nose



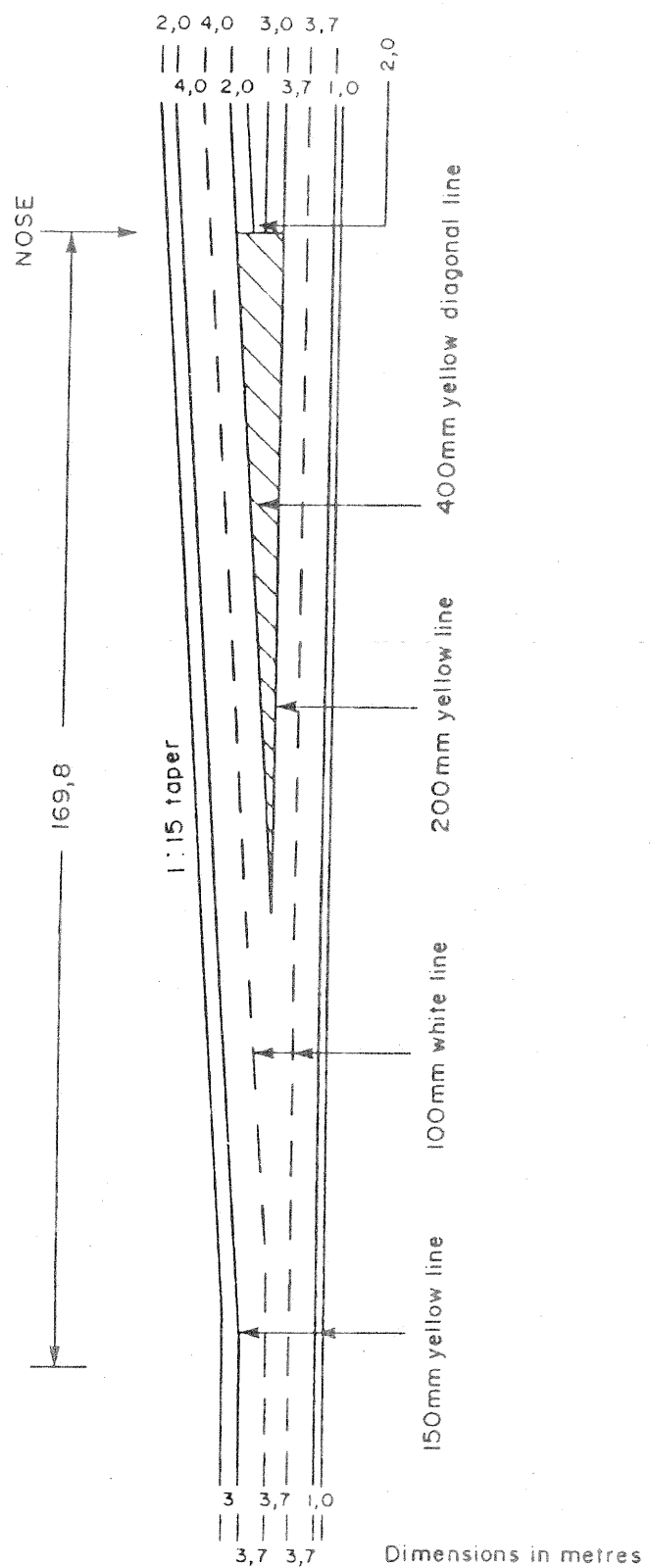
SINGLE-LANE EXIT



Note : For 50m in advance of the nose the ramp should be not more than 0,75 m lower than the freeway

TWO-LANE ENTRANCE (WITH ONE LANE ADDED)

Note: Sight distance of 300m measured from an eyehight of 1,05m to the road surface should be provided in the area in advance of the nose



TWO LANE EXIT (WITH ONE LANE DROPPED)

11. PEDESTRIANS AND PEDAL CYCLISTS

11.1 Introduction

Research has indicated that moderate to heavy pedestrian and pedal cycle traffic and concentrations of accidents are experienced along some tarred rural roads, especially in the following areas: in the proximity of towns, in densely populated rural areas (eg at schools, bus stops, shops and housing next to the roadway), at mines and mining compounds, industrial plants, agricultural depots, irrigation schemes, etc. Usually, no specific provision is made for pedestrians and pedal cyclists at these places – pedestrians are entirely dependent on the road shoulder when walking to a bus stop or from one place to another. This chapter sets out the conditions under which pedestrian and pedal cyclist facilities should be provided.

11.2 Footways

Warrants for the provision of footways depend on the vehicle-pedestrian hazard, which is governed mainly by the volumes of pedestrian and vehicular traffic, their relative timing and the speed of vehicular traffic. Paved footways are warranted once the minimum conditions specified in Table 11.2 have been met.

To ensure that they are used for the purpose intended, footways must have all-weather surfaces, otherwise pedestrians will choose to walk on the carriageway. Footways should have a minimum width of 1 m in rural areas and 1,5 m in peri-urban areas, and should be situated at least 3 m from the travelled way in level terrain. Footways along the reserve boundary are not popular with pedestrians but may in certain circumstances be preferable. When footways are to be provided in rolling or mountainous landscape through cuttings and fills, they may be situated adjacent to the roadway, but in such a case special provision should be made to protect pedestrians by means of a precast kerb or guardrails.

TABLE 11.2
WARRANTS FOR PEDESTRIAN FOOTWAYS

FOOTWAY	AVERAGE DAILY TRAFFIC (ADT)	PEDESTRIAN FLOW (per day)	
		Design speed or speed limit (60-80 km/h)	Design speed or speed limit (80-120 km/h)
On one side	400 to 1 400	300	200
	> 1 400	200	120
Both sides	700 to 1 400	1 000	600
	> 1 400	600	400

In cases where footways are not warranted but a large number of pedestrians walk alongside the road, the road shoulder should be upgraded to cater for them. The minimum width of these shoulders should be 3 m and they should be

graded and compacted regularly to provide pedestrians with a hard surface to walk on. Paved shoulders, 1,5 m wide should be provided in high rainfall areas. Road shoulders must also be well drained to prevent water from accumulating on them, thereby forcing pedestrians to walk on the carriageway. Where sections of the road shoulder are paved, special care must be taken to ensure that the water outlets are at a lower level than the road surface.

When a road is realigned or reconstructed, a part of the old road surface could be retained as a walkway for pedestrians or as a cycle lane. Where a footway is located on a sharp bend, consideration should be given to erecting guardrails between the back of the shoulder and the footway.

11.3 Bridges

On routes with footways these should be continued across any bridges; and if it is likely that footways will be provided along the route in the foreseeable future, a footway should be provided on a bridge. The minimum width of the footway on the bridge structure should be 1,2 m. Where necessary, footways must be diverted as soon as is practicable beyond the bridge approach fills or cuts from the back of the shoulder to a position of at least 3 m from the travelled way. On long bridges or on bridges that are intended to carry large numbers of school children a separate walkway should be provided, as shown in Figure 11.3.1. On existing or new bridges, light aluminium structures can also be used to provide a separate walkway when warranted.

Care must be taken that the approach footways provide safe and relatively direct access to the footway on the bridge. This may require the erection of barriers to channelize pedestrians onto the bridge. A flush roadway shoulder should never terminate in a raised footway on a bridge. Where such installations exist and their removal is not economically justifiable, the ends of the footway should be protected by a guardrail or a 20:1 transition provided between the raised footway and the shoulder.

11.4 Bus stops

Pedestrian accidents often occur at bus stops because buses stop too close to the road edge, obstructing the view of drivers of pedestrians crossing the road.

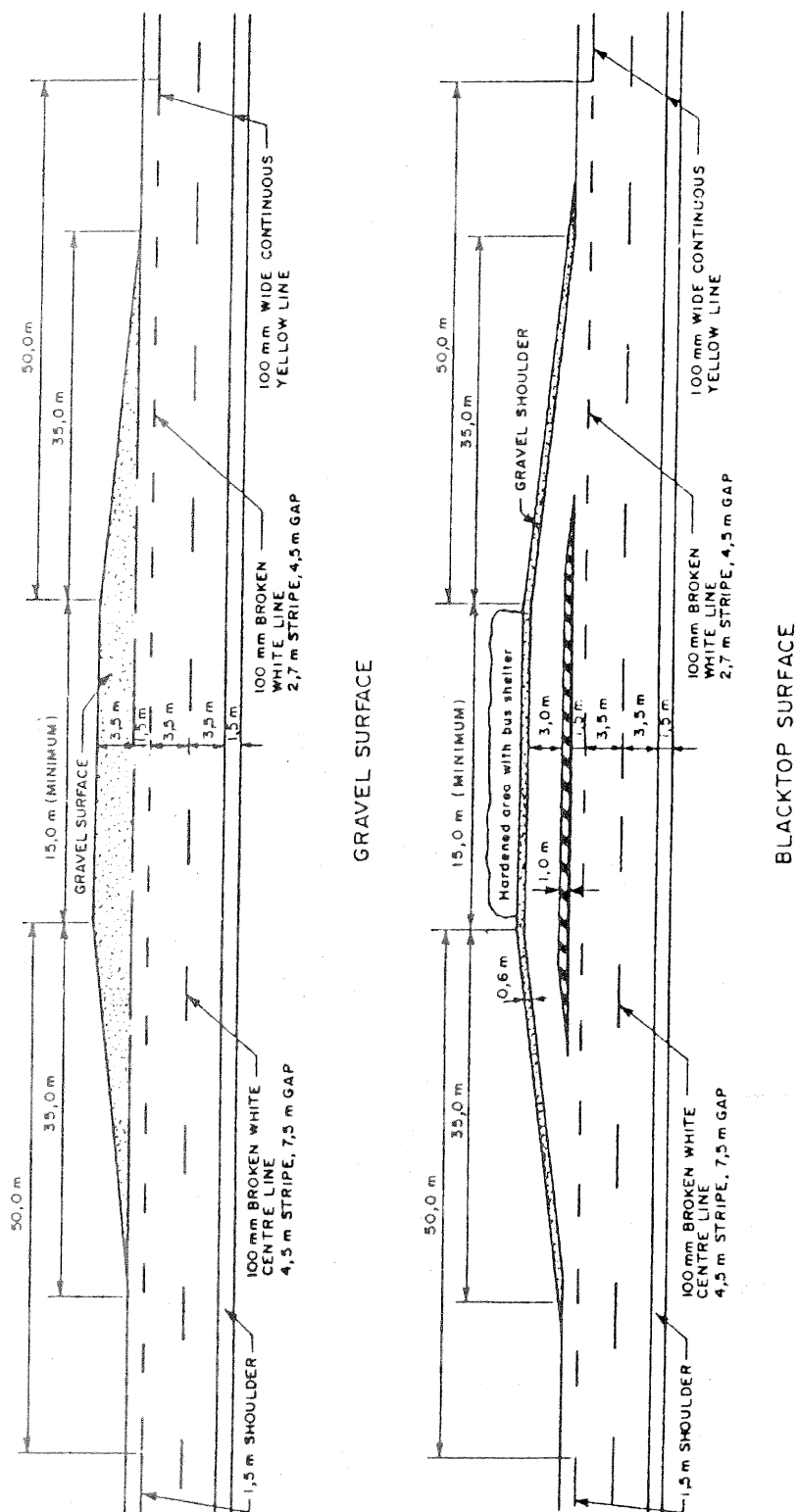
The layout of a bus stop consists of three elements:

- (1) a deceleration lane or taper to permit easy entrance to the loading area;
- (2) a bus bay with holding area far enough from the roadway edge to eliminate sight distance problems; and
- (3) a merging area to enable re-entry to the road.

Typical layouts showing the minimum requirements for gravel and blacktop bus bay layouts are shown in Figure 11.4.1. On roads with an average daily traffic exceeding 1500 vehicles, bus bays should be provided with a blacktop surface.

Deceleration lanes should be tapered at a flat angle of not less than 5:1 (preferably 10:1) and on high speed roads, 15:1 to allow drivers to pull off the through lane completely. The yellow edge line should be replaced with a broken yellow line where the bus exits or re-enters the carriageway.

Bus bays should be at least 3,5 m wide and should be placed adjacent to the paved or gravel shoulder so that buses can stop clear of the roadway. The



TYPICAL BUS BAY LAYOUTS FOR GRAVEL AND BLACKTOP SURFACES

length of a bus bay in rural areas should be not less than 10m. Where multiple bus bays are provided – such as in a peri-urban area – the length of the individual bays should not be less than 10 m. On heavily trafficked roads, a channelizing island 1 m wide may be provided along the road edge line to direct bus drivers to stop clear of the road shoulder. A hardened area for passengers should be provided alongside the bus bay in the ratio of 5 m² for every 10 persons at peak hour.

The merging taper could be somewhat more abrupt than the deceleration taper, but should not be sharper than 3:1.

11.5 Refuge island

Refuge islands can be used to help pedestrians to cross wide or busy roads. Without affecting traffic capacity, they allow pedestrians to cross one traffic stream at a time.

Pedestrian accidents often occur at bus stops, shops, schools etc. along rural roads but the provision of a pedestrian crossing on high-speed roads is not usually practicable unless special precautions are taken to safeguard pedestrian crossings by means of joggle bars and other measures. However, it is often in mountainous areas or in rolling topography that restricted sight distance does not always allow pedestrians enough time to cross the road safely. In such areas, if the minimum sight distances for pedestrians crossing rural roads (given in Table 11.5) cannot be attained, a refuge island should be provided. In problem areas, a properly designed refuge island is considered a safe alternative; where one is used, pedestrian risk is reduced by 50 per cent.

On new or reconstructed roads, refuge islands should be at least 1,5 m wide (preferably 2 m) and may take the form either of raised islands or of painted refuges. If raised, the sides should be semi-mountable. Painted refuges, however, must be safeguarded with protective concrete strips such as joggle bars or the like. In addition, the approaches to the refuge island must be tapered and clearly demarcated with the necessary road signs and markings. The road markings equipped with reflectorized road studs must channelize vehicular traffic away from the refuge island. The "Keep left" sign R17 must also be displayed prominently to safeguard drivers.

TABLE 11.5
MINIMUM PEDESTRIAN SIGHT DISTANCES IN METRES FOR DIFFERENT
SPEED LIMITS AND ROAD WIDTHS

SPEED LIMIT (km/h)	MINIMUM PEDESTRIAN SIGHT DISTANCES FOR FOR DIFFERENT ROAD WIDTHS (m)		
	Two-lane	Three-lane	Four-lane
60	85	130	170
70	100	150	200
80	115	170	230
90	130	190	255
100	140	215	285
110	155	235	310
120	170	255	340

11.6 Footbridges and subways

Guidelines on the provision, layout and siting of foot-bridges and subways on rural freeways are contained in NITRR Technical Report RV/3. The parameters to be taken into account when providing a grade-separated crossing are: the persistent tendency of pedestrians to cross the freeway at-grade at a specific point; the distance from other freeway crossing facilities; pedestrian-related accidents on freeways, and physical characteristics, eg topography, which facilitate convenient crossing facilities. Weights are allocated to each parameter and sites are ranked according to a priority rating.

The width of footbridges and subways should be in the range of 2,1 to 3 m. For long subways without artificial lighting or where natural light cannot be allowed in through a central opening (by means of a break in the median for instance) the dimensions in Table 11.6 may serve as a guide. Subways must be so designed that the pedestrian can see from end to end and therefore feel a sense of security. The minimum clearance required for foot-bridges above road surface is 5,2 m.

TABLE 11.6

VERTICAL AND HORIZONTAL DIMENSIONS RECOMMENDED FOR SUBWAYS WITHOUT ARTIFICIAL LIGHTING OR A BREAK IN THE MEDIAN

LENGTH OF SUBWAY (m)	WIDTH (m)	VERTICAL HEIGHT (m)
14 or less	2,1	2,1
14 to 24	2,4	2,4
more than 24	3,0	3,0

Whenever possible, the access to a foot-bridge or subway should be in line with the normal walking path. If the access to the crossing requires pedestrians to divert from their path, it prolongs the walking distance and lengthens the crossing time. These factors increase the probability of pedestrians being discouraged from using the crossing. Precaution must also be taken to force pedestrians by means of physical devices such as barriers to make use of refuge islands, foot-bridges or subways in cases where the facility has to be deviated from the normal walking path.

11.7 Lighting

When pedestrian casualties occur frequently at night alongside roads in peri-urban and rural areas, the provision of lighting should be considered. Recommended lighting values for different road classes are contained in the Code of Practice for Public Lighting of the SABS (Report No. 098). On the road shoulders the luminance should be at least 50 per cent of that of the carriageway surface.

11.8 Cycle lanes

On roads carrying between 20 and 70 pedal cyclists, travel in one direction during any one hour of the day, a Class 3 cycle lane (cycle lane on paved shoulder) should be provided. When the paved road shoulder is used as a cycle lane it be-

comes a traffic lane and should therefore be appropriately marked.

If a hard gravel shoulder is provided next to the cycle lane, a cycle lane width of 1,2 m would suffice, but for a soft shoulder or sloped drop-off a 1,5 m wide cycle lane is recommended.

11.9 Speed zoning

Particulars on the setting of rural speed limits are given in NITRR Technical Report RV/19. In a rural or peri-urban area where during any four hours of an average day more than 200 pedestrians cross a road with a central median within a length of 150 m, the speed limit should be set at 80 km/h; on roads without a central median the speed limit should be 70 km/h. In areas where a large number of pedestrians or pedal cyclists use the road shoulder or cycle lane, the speed limit should be set at 80 km/h.