URBAN TRANSPORT GUIDELINES

Draft UTG 1 GUIDELINES FOR THE GEOMETRIC DESIGN OF URBAN ARTERIAL ROADS

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PREFACE

Urban Transport Guidelines (UTG) is a series of documents written for practising transportation engineers describing current recommended practice in selected aspects of urban transportation. They are based on South African experience and research and have the full support and approval of the Committee of Urban Transport Authorities.

To confirm their validity in practice, UTGs are circulated in draft form for a twoyear period before receiving the final approval of CUTA. During this period, suggestions for improvement may be sent to:

The Secretary Committee of Urban Transport Authorities c/o NITRR P O Box 395 0001 PRETORIA.

After final approval by CUTA, the revised document will be issued as a full UTG in both official languages.

SYNOPSIS

This document deals with the geometric design of urban arterials. It forms part of a series on freeways, collectors and arterials.

Aspects covered in the document relate to basic design concepts and criteria. From these, guidelines in respect of horizontal and vertical alignment and crosssection are derived. The location and design of intersections is also discussed.

SINOPSIS

Hierdie dokument handel oor die geometriese ontwerp van stedelike verkeersare. Dit vorm deel van 'n reeks wat handel oor deurpaaie en versamelare.

Aspekte wat deur die dokument gedek word het betrekking op basiese ontwerpkonsepte en kriteria. Hieruit word riglyne rakende horisontale en vertikale belyning en dwarssnit, afgelei. Die ligging en ontwerp van kruisings word ook bespreek.

KEYWORDS

Geometric design, urban arterials, horizontal alignment, vertical alignment, cross-section, intersections.

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1 INTRODUCTION

1.1 PURPOSE OF THE GUIDELINES

The Committee of Urban Transport Authorities (CUTA) was formed in 1982 to provide a forum for discussion to promote coordination and, where appropriate, uniformity on technical standards for, and approaches to, the road and transport systems of urban areas in South Africa.

The various agencies responsible for the design of urban roads have been concerned for some time about the wide range of geometric design standards and policies relating to the design of urban roads, not only between, but even within the various metropolitan areas. Particular problems can arise when a metropolitan route passes through several local authorities. Cases have arisen where a route passing through adjacent local authority areas has had a conspicuously different cross-section on either side of the boundary between the local authorities.

At the first meeting of CUTA in August 1982, it was decided to establish an Ad Hoc Technical Committee on Geometrics (AHTCG). This committee decided to produce guidelines for the geometric design of urban roads with the following objectives:

- (1) to promote a uniform approach to the adoption of geometric design standards for urban roads;
- (2) to recommend dimensions for geometric design elements to provide adequate standards of safety and convenience on urban roads under South African conditions; and
- (3) recognizing, that in the upgrading of urban roads and in the construction of new roads within built-up urban areas, restrictions in space so often prevent the provision of geometric design elements to ideal dimensions, to provide guidelines for the adoption of reduced dimensions which would under the prevailing circumstances still provide reasonable levels of safety and convenience within economic, environmental, social and political constraints under South African conditions.

For ease of reference, a separate guidelines document will be produced for each class of urban road.

The most urgent need was for information on arterial roads. Accordingly this document, which is the first, has been confined to arterial roads.

1.2 USE OF THE GUIDELINES

These guidelines are intended to complement design expertise and as such cannot be used by client bodies singularly as a specification for design standards. They should, however, be useful as aids towards the preparation of such specifications. For each geometric design element, a recommended dimension or value has been given. The practice of presenting "desirable" and "minimum" values has deliberately not been used, as this practice usually results in the "minimum" becoming the "standard". Instead, these guidelines have recommended the dimensions and values that should be used for an urban arterial. However, it was recognized that much of the design of urban roads will take place under conditions of restricted space that will make it unfeasible to provide the dimensions that should ideally be used on an arterial road. The selection of appropriate dimensions under such circumstances is difficult and in the past, little guidance has been available to the designer. These guidelines attempt to rectify this situation.

The first goal of the guidelines is the achievement of the recommended dimensions. Where these cannot be attained, the next goal is to achieve dimensions, which although not ideal, will provide a reasonable facility.

1.3 ASSOCIATED DESIGN FEATURES

Geometric features of road design are highly dependent on other road features. Specifically, drainage, lighting, location of utilities, surface treatment, signing, safety barriers and traffic control devices, all of which, if properly considered collectively, will lead to safe designs of roads with effective relationships to the urban environment. Currently these associated features of arterial road design have not been addressed directly in this document. The experienced designer of roads will be aware of the problems in bringing into the design process these associated features. References 1, 2, 5, 7, 10, 12, 13, 14, 19, 20, 21, 22, 24, 25, 29, 30 and 31 in Section 11 of this report are recommended in this regard. Also other documents will be developed by other CUTA committees which will augment this document on geometric design.

The subject of private driveways was given considerable thought by the Ad Hoc Technical Committee and a draft chapter on the subject was referred to the next document dealing with collector roads. It was felt that driveways *per sé* would not usually be permitted on arterial roads. The type of access normally permitted would be major and considered as a roadway intersecting with arterial for geometric design purposes. Chapter 10 describes geometric design of intersections on arterial roads.

1.4 DESIGN REFERENCES

It should be appreciated that this document is not a substitute for road design expertise.

The guidelines have been kept brief deliberately so that the recommended dimensions can be readily found by experienced designers. The guidelines were not meant to be a textbook on the art of geometric design. Nor can they, in this brief form, provide an exhaustive record of the information that designers need. It is therefore important that the guidelines be supplemented by various standard texts. The following texts were principal sources of information for these guidelines and are regarded as suitable references for design of urban arterial roads:

AMERICAN ASSOCIATION OF STATE HIGHWAY AND TRANSPORTATION OF-FICIALS. *A Policy on Geometric Design of Highways and Streets.* Washington, DC, 1984, which is an updated combination of AMERICAN ASSOCIATION OF STATE HIGHWAY OFFICIALS. *A Policy on Geometric Design of Rural Highways.* Washington DC, 1965 and AMERICAN ASSOCIATION OF STATE HIGHWAY AND TRANSPORTATION OFFICIALS, *A Policy on Design of Urban Highways and Arterial Streets,* Washington, DC, 1973.

NATIONAL INSTITUTE FOR TRANSPORT AND ROAD RESEARCH. *Geometric Design of Rural Roads.* Draft TRH 17, Pretoria, CSIR, 1985.

MATSON, T M, SMITH, W S and HURD, F W. *Traffic Engineering*. New York, McGraw-Hill, 1955.

PROVINCE OF THE CAPE OF GOOD HOPE. *Geometric Design Manual*, Updated Reprint. Cape Town, Department of Roads, May 1983.

NATAL PROVINCIAL ADMINISTRATION. *Geometric Design Manual*, Updated Reprint. Pietermaritzburg, Natal Roads Department, 1975.

ORANGE FREE STATE PROVINCIAL ADMINISTRATION. *Geometriese Ontwerp Handleiding.* Bloemfontein, Orange Free State Roads Branch, 1976.

TRANSVAAL PROVINCIAL ADMINISTRATION. *Typical Plans for Road Design*. Pretoria, Transvaal Roads Department, 1981.

NATIONAL TRANSPORT COMMISSION. *Geometric Planning Manual G2*, Pretoria, The Director General: Transport, Directorate: Land Transport, February 1984.

SOUTH AFRICAN INSTITUTE OF CIVIL ENGINEERS. *Guidelines on the Planning and Design of Township Roads and Stormwater Drainage*. 2nd edition, Johannesburg, August 1981.

COUNCIL FOR SCIENTIFIC AND INDUSTRIAL RESEARCH. *The South African Road Traffic Signs Manual.* 2nd edition, CSIR Manual K55, Pretoria, CSIR, 1982.

The reader is reminded that individual local authorities and other road authorities or owners may have their own standards and/or specific requirements.

1.5 TRAFFIC CAPACITY

The traffic carrying capacity of arterial roads has not been dealt with in this report. Whereas it is recognized that the geometry of a road will affect capacity it is noted that the analysis of traffic capacity can be extensive and intricate. The reader is referred to the 1965 *Highway Capacity Manual,* Ref. 1 and its 1985 updated version, Ref. 2.

2 DESIGN CONCEPT

2.1 FUNCTIONS AND CHARACTERISTICS OF URBAN ROADS

The road system can be classified into distinct classes of road based on the functions they have to perform. The main differences between classes of road relate to the extent to which they have to cater for movement or for access.

The classes of road can be listed in descending order of hierarchy as:

- Freeways
- Arterials (including expressways)
- Collectors
- Locals

At the top end of the scale, freeways are dedicated to movement and have access limited to interchanges with grade separation of conflicting movements. At the bottom of the scale the main function of local streets is to provide access to properties.

	Function	Locals	Collectors	Arterials	Freeways
1.	Traffic				
	movement	Secondary to function of access	Equal to function of property access	Primary	Primary
	Flow conditions	Interrupted flow	Interrupted flow	Uninterrupted flow except at intersec- tions and mid-block pedestrian crossings	Free-flow
	Running speeds normal (km/h)	20-40	30-70	40-90	70-120

TABLE 2.1

Functions and characteristics for road classes

	Vehicle types	Primarily passenger cars	All types including buses	All types including buses	Motor vehicles including express buses
2.	Property access	Primary considera- tion	Equal to function of traffic movement	Preferably excluded*	No access
3.	Connections	Collectors, locals	Arterials, collectors, locals	Freeways, arterials, collectors	Freeways, arterials
4.	Parking	Accepted	Accepted	Preferably excluded	Excluded

Source: Adapted from Ref. 3, Table A.5b, p.A.13.

* See Sections 1.3, 2.2, 2.5.

Table 2.1 summarizes the functions and characteristics of the various classes of urban roads.

Figure 2.1 illustrates the changing emphasis on movement and access for the various classes of road. The range covered in these guidelines is shown shaded. Ref. 4, Ref. 5.

In *Guidelines for the Provision of Engineering Services in Residential Townships* a useful table is provided which sets out the various terms used by different authorities in South Africa, United Kingdom, United States and Australia to describe the various classes of roads. Ref. 6, pp B113-114.

2.2 ARTERIAL ROADS

The prime function of arterial roads is the movement of traffic. More specifically the arterial road should cater for longer distance movements in the urban system.

To perform its function satisfactorily an arterial road requires the following special provisions:

- No access to the road from adjacent properties
- Intersection spacing of 350 m or more
- Intersection spacing to aid traffic signal coordination
- Design speed of 70 to 90 km/h
- Adequate lane width to accommodate all types of vehicles including trucks and buses

An arterial road would normally be a divided road with two or three lanes in each direction.

Provision of the properties mentioned above would optimize the safety and traffic service of the arterial road system. Departures from these properties would reduce safety and increase driver tension. Such departures should thus be made only when it is not physically or economically feasible to provide the required characteristics to perform the arterial road function.

2.3 DESIGN SPEED

The concept of design speed developed by the American Association of State Highway and Transportation Officials (AASHTO) is used by many designers to achieve a balanced design for a given roadway or roadway network. This is particularly true for rural roads or for roads through lightly developed areas.

AASHTO define speed as

the maximum safe speed that can be maintained over a specified section of highway where conditions are so favourable that the design features of the highway govern. Ref. 7, p.60.

AASHTO suggest that all pertinent features of a roadway should be related to a selected design speed in order to achieve a balanced design. They note that features such as curvature, superelevation and sight distance are directly related to, and vary appreciably with, design speed. Other features, such as widths of pavements and shoulders and clearances to walls and rails, are not considered to be directly related to design speed, but as they affect vehicle speed, higher standards are recommended for these features for higher design speeds. In these regards the AASHTO concept assumes that when change is made in design speed, nearly all elements of the highway are subject to change. They go on to recommend that the design speed chosen should be consistent with the speed a driver is likely to expect. Ref. 7, pp. 60-61.

The need for a balanced design relates to the expectations of drivers and therefore can vary considerably depending upon the functions of the road, the environment through which it passes and the posted or legal speed limit. Obviously, the need for a balanced design diminishes as the movement function and speed decrease and the access function and density of development in the surrounding area increase. Thus there has been a tendency to establish design speeds according to road type, that is, freeway, arterial and collector, and to try and balance the design throughout the length of roadway. However, in the past each type of road tended to have one design speed assigned to it regardless of environment, the restrictions caused by traffic control devices and the expected running speeds.

Figure 2.1 illustrates the varying degrees to which various types of road have to accommodate movement and access. Ideally an arterial road should cater for



FIGURE 2.1 Movement-Access Functions

access only to the extent of permitting intersections with other roads. In practice, in urban areas, prevailing conditions often force a greater access function than the ideal and the range of conditions shown in Figure 2.1 for urban arterials results.

To employ appropriate geometric design standards, it is necessary to recognize that the urban arterials for which designs will be required will not be restricted to the ideal. The range of conditions applying to urban arterials can be identified as shown in Table 2.2.

2.4 RUNNING SPEED

The term "running speed" refers to the actual speed of a vehicle over a section of road. "Average running speed" is the arithmetic mean of the individual running speeds. Ref. 7, p. 68.

AASHTO refer to running speeds being observed close to design speed for low design speed curves and being well below design speed for high design speed curves. Ref. 7, p. 69. Observations in South Africa have indicated that vehicles travel at or above design speed, even under wet weather conditions on curves with high design speeds. Ref. 8, p. 9.

2.5 PRACTICAL DESIGN SPEEDS

The recommended design speed for an arterial road with no property access and intersections spaced at 350 m or more is in the range of 79 to 90 km/h.

The overall design concept, given practical considerations, is highly dependent on selection and understanding of the speed for which a road is to be designed. The design speed is influenced by three factors:

- Function of the road
- Cost of achieving geometric standards
- Environment through which the road passes

Actual standards employed may have to depart from the desired standards for the road's function if the cost of achieving them would be prohibitive. Urban designers are often faced with traffic operating under unsafe and congested conditions. If the cost of providing a relieving facility to ideal standards is unaffordable and nothing is done, traffic will continue to operate under the unsafe conditions. However, if compromise standards, which are better than existing, but lower than ideal, can be employed within the financial constraints then traffic and safety conditions can be improved.

When roads are designed for built-up areas, the speed desired in terms of the function may be inappropriate for the environment through which it passes. For example, in a built-up area, where access cannot be denied, where intersections are closely spaced and where there is considerable pedestrian activity, high speed would be unsafe, inappropriate and undesirable.

Thus, practical considerations may modify the desired design speed and geometric standards to be employed. An identification of the effect of prevailing conditions on expected or appropriate design speeds is given in Table 2.2 for arterial roads.

	Conditions prevailing	Design speed (km/h)
1.	Expressway type. No property access. At-grade intersections spacing \ge 500 m	80-100
2.	No property access. At-grade intersections spacing \ge 500 m No grade separated intersections	70-90
3.	Property access unavoidable but limited to low density residential land use or infrequently from commercial developments. Intersection spacing \ge 100 m	60-70
4.	Property access unavoidable from residential or commercial land use. Intersection spacing \ge 100 m	50-60*
5.	Central area arterial street. Close intersection spacing with traffic signal control. Pedestrian activity.	50-60*

TABLE 2.2

Design speeds for arterials

Source: Adapted from Ref. 8, p. 14.

* The higher design speed should be used for preference.

Urban road authorities are faced with three distinct sets of conditions which affect the feasibility of achieving high design speeds and high design standards:

- New construction in undeveloped areas
- New construction within existing built-up areas
- Improvements to old roads in built-up areas

When road networks and land use are being planned in undeveloped areas, there is usually scope for standards approaching ideal to be employed. Routes can often be located to provide geometric standards for desirable speeds at reasonable cost. Also the land use can be planned, along with access and intersection spacing, to be appropriate to the speeds expected on the road network.

In existing built-up areas, route location is usually affected by financial, social, political and physical constraints. The environment is already there and the road must be designed to suit the environment, as the alternative of altering the environment is often unfeasible.

Improvement in geometrics of existing roads to achieve higher speeds is often even more restrained as the adjacent land use will have developed over the years in relation to the original design.

Practical considerations in urban areas therefore require adaptation of "ideal" standards to "practical" standards. Lowering of standards inevitably results in lower operating conditions in terms of comfort, convenience, safety and speed. As in all design considerations the trade-offs between the benefits and costs have to be assessed. Also, the implications of "doing nothing" rather than the ideal should be weighed against "doing something" even if not ideal.

3 BASIC CRITERIA

From their study of human factors in highway design and operations, Lunenfeld and Alexander concluded "Because drivers read the road and its information, and tend to believe what it appears to be telling them, a road that is substandard may not operate properly" and further that "Properly designed and operated facilities that take human factors into account generally operate safely and efficiently". Ref. 9, p.157.

In southern Africa, with population groups from the First World and the Third World, designers have to recognize the variety of skills at "reading" the road among the road users. Consistent design standards are thus all the more important.

The basic criteria for road design are common for all types of roads as they relate to typical characteristics of drivers and the performance of vehicles. The balance of this chapter is taken from Chapter 2, Basic Criteria, in *TRH 17, Geometric Design of Rural Roads.* Ref. 10. The sections on passing sight distance and decision sight distance were expanded with material from *Geometric Standards for Canadian Roads and Streets.* Ref. 11, pp B16 – B19.

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 of circumstance will be in a position to calculate appropriate values.

3.1 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.

3.1.1 Dimensions

The dimensions adopted for the various design vehicles are given in Table 3.1.

Vehicle	Wheel- base (m)	Front overhang (m)	Rear overhang (m)	Width (m)
Passenger car Ref. 12, p.10	2,85	0,75	1,2	1,80
Single unit Ref. 7, p.25	6,10	1,22	1,83	2,50*
Single unit + trailer	6,7			
Ref. 13, fig. 2-404.2A	+3,4			
	+6,1			
	16,2			
Single-unit bus Ref. 14, p.53	6,00	2,50	3,50	2,60
Articulated bus Ref. 7, p.27	5,49	2,59	2,89	2,60
	+7,32			
	12,81			
Semi-trailer Ref. 7, p.30	6,10 +9,15	0,92	0,61	2,50
	15,25			

TABLE 3.1Dimensions of design vehicles

Source: Adapted from Ref. 10, Table 2.2.1 *Maximum in South Africa

3.1.2 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 con-

struction, be approximated by simple or compound curves. Figures 3.1 and 3.2 give dimensions for the construction of templates for single units plus trailers.

3.1.3 Minimum turning radius

In constructed 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 body are given in Table 3.2. It is stressed that these radii are appropriate only to crawl speeds.

TABLE 3.2

VehicleMinimum turning
radius (m)Passenger car Ref. 12, p.106,20Single unit Ref. 7, p.2513,38Single unit plus trailer Ref. 13, fig. 2-404.2A14,50*Single-unit bus Ref. 14, p.53 (updated)14,50Articulated bus Ref. 7, p.2712,80Semi-trailer Ref. 7, p.3014,08

Minimum turning radii

Source: Adapted from Ref. 10, Table 2.2.3 *Adjusted 0,50 m for body overhang

3.1.4 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 as shown in Figure 3.3. Ref. 15, p.345 Ref. 16. A mass to power ratio of 275 kg/kW has been used as representative of the 15th Percentile of South African trucks. That is, 15 per cent of trucks have a higher mass to power ratio and are not accommodated by the curves in Figure 3.3.

3.2 THE DRIVER

3.2.1 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. Ref. 17 and Ref. 18.

3.2.2 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. Ref. 7, p.137 and p.147.



FIGURE 3.1

Wheel Tracks of Rigid Chassis Vehicles



FIGURE 3.2 Wheel Tracks of Articulated Vehicles

3.3 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.

3.3.1 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. Ref. 19.

Brake-force coefficients are given in Table 3.3. 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 comfortable stop under normal conditions.

Speed	Brake-force
(km/h)	coefficient
40	0,37
60	0,32
80	0,30
100	0,29
120	0,28

TABLE 3.3

Brake-force coefficients for various speeds

Source: Ref. 10, Table 2.4.1

3.4 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 to enter the opposing lane safely while over-taking. 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.



FIGURE 3.3 Truck Speeds on Grade

3.4.1 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 resistance. 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

 $s = 0.7v + v^2/254f$

where s = total distance travelled (m)

v = speed (km/h)

f = brake-force coefficient

Stopping sight distances for a range of design speeds and appropriate brakeforce coefficients are given in Table 3.4.

Stopping sight distance on level roads		
Design speed (km/h)	v (km/h)	Stopping sight distance (m)
40	40	45
50	50	65
60	58	80
70	64	95
80	72	115
90	78	135
100	85	155
110	92	180
120	101	210

TABLE 3.4

Source: Adapted from Ref. 10, Table 2.5.1 and Ref. 7, p. 138.

Stopping sight distance is measured from an eye height of 1,05 m to an object height of 0,15 m. 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 values of stopping sight distance given in Table 3.4 are similar to the lower range of design values given in AASHTO. Ref. 7, p.138. South African practice has related to this lower range.

Values in Table 3.4 are recommended for design.



FIGURE 3.4 Stopping Sight Distance on Grades

The gradient has a marked effect on the stopping sight distance requirements. Gradient (G) modifies the stopping sight distance formula to

 $S = 0.7v + v^2/254(f \pm G)$ Ref. 7, p. 143.

where G is the per cent of grade divided by 100.

AASHTO, Ref. 7, p.143, assume v equal to design speed for downgrade conditions and v equal to a running speed which is less than design speed for upgrade conditions. Similarly, *TRH 17, Geometric Design of Rural Roads* presents values of stopping sight distance on grades with built-in assumptions concerning operating speed being less than design speed when road surfaces are wet. Ref. 10, Fig. 2.5.1(a).

Figure 3.4 is a direct graphical representation of the formula to show stopping sight distance on grades between -10 per cent and +10 per cent for running speeds v between 40 km/h and 130 km/h.

Stopping sight distance can also be affected by a visual obstruction (such as a cutslope or a wall) next to the carriageway on the inside of a horizontal curve, as shown in Figure 3.5.

3.4.2 Barrier sight distance (BSD)

Barrier sight distance is a term used in *TRH 17, Geometric Design of Rural Roads,* Ref. 10, to describe the legal limit below which overtaking is legally prohibited. In the *South African Road Traffic Signs Manual,* Ref. 20, pp. 209-210, under warrants for barrier lines, values of sight distance for various speeds are given with the distances measured between an eye height of 1,05 m and an object height of 1,30 m which represents the height of an approaching vehicle. These BSD values are reproduced in Table 3.5 for ease of reference.

Design speed (km/h)	Barrier sight distance (m)
50	150
80 80	250
100	300
120	400

TABLE 3.5Barrier sight distance

Source: Ref. 20, p.210.



FIGURE 3.5 Horizontal Radius for Stopping Sight Distance

3.4.3 Passing sight distance (PSD) (Ref. 11, pp. B16-B18)

Passing sight distance may be applicable to the design of divided arterial roads where one-half is to be used as a two-lane two-way roadway for an extended initial period.

Passing sight distance for use in design is determined on the basis of the length needed to safely complete normal passing manoeuvres. While there may be occasions to consider multiple passings, where two or more vehicles pass or are passed, it is not practical to assume such conditions in developing minimum standards. Instead, sight distance is determined for a single vehicle passing a single vehicle. Longer sight distances occur in design and these locations can accommodate an occasional multiple passing.

Standard minimum passing sight distance values are determined and given in Table 3.6 for a range of design speeds from 50 km/h to 120 km/h. It is important, for reasons of safety and service, to provide as many passing opportunities as possible in each section of road. The designer should try to ensure that there is no long stretch where passing is not possible. The amount of passing sight distance available on a section of road has considerable influence on the average speed of the traffic. This is particularly true where a road is operating near capacity. The economic effects of reduced speeds cannot be accurately determined at the present time, but there is no doubt that road users benefit considerable when able to operate at or near the design speed with minimum interference from other vehicles. The designer should consider these economic effects when setting vertical alignment.

Design speed (km/h)	Passing sight distance (m)
50	340
60	420
70	490
80	560
90	620
100	680
110	740
120	800

TABLE 3.6

Passing sight distance on level roads

Source: Ref. 11, Table B.2.4, p.B18. Ref. 10, Table 2.5.4.

Source: Ref. 11, Table B.2.4, p.B18. Ref. 10, Table 2.5.4.

3.4.4 Decision sight distance (DSD) Ref. 11, pp. B19-B19

Stopping sight distances are usually sufficient to allow reasonably competent and alert drivers to come to a hurried stop under ordinary circumstances. However, these distances are often inadequate when drivers must make complex or instantaneous decisions, when information is difficult to perceive, or when unexpected or unusual manoeuvres are required. Limiting sight distances to those provided for stopping may also preclude drivers from performing evasive manoeuvres, which are often less hazardous and otherwise preferable to stopping. Even with an appropriate complement of standard traffic control devices, stopping sight distances may not provide sufficient visibility distances for drivers to corroborate advance warnings and to perform the necessary manoeuvres. It is evident that there are many locations where it would be prudent to provide longer sight distances. In these circumstances, decision sight distance provides the greater length that drivers need.

Decision sight distance is the distance required for a driver to detect an information source or hazard which is difficult to perceive in a roadway environment that might be visually cluttered, recognize the hazard or its threat potential, select appropriate action and complete the manoeuvre safely and efficiently. Because decision sight distance gives drivers additional margin for error and affords them sufficient length to manoeuvre their vehicles at the same or reduced speed rather than to just stop, its values are substantially greater than stopping sight distance.

Drivers need decision sight distances whenever there is a likelihood for error in either information reception, decision making, or control actions. Examples of critical locations where these kinds of errors are likely to occur, and where it is desirable to provide decision sight distance are: interchanges and intersections; locations where unusual or unexpected manoeuvres are required; changes in cross-section such as toll plazas and lane drops, and areas of concentrated demand where sources of information compete, for example from roadway elements, traffic, traffic control devices, and advertising.

The decision sight distances in Table 3.7 provide values to be used by designers for approriate sight distances at critical locations and serve as criteria in evaluating the suitability of the sight lengths at these locations. Because of the additional safety and manoeuvrability these lengths yield, it is recommended that decision sight distances be provided at critical locations or that these points be relocated to locations where decision sight distance lengths are available. If it is not feasible to provide these distances because of horizontal or vertical curvature or if relocation is not possible, then special attention should be given to the use of suitable traffic control devices for providing advance warning of the conditions that are likely to be encountered.

A range of decision sight distance values that will be applicable to most situations has been developed. The range recognizes the variation in complexity that may exist at various sites. The values are based on the sum of distances travelled during premanoeuvre and manoeuvre times and validated at a number of locations.

Design speed (km/h)	Decision sight distance (m)
50	135-195
60	170-235
70	200-275
80	235-315
90	270-355
100	300-395
110	335-435
120	370-475

TABLE 3.7

Decision sight distance

Source: Ref. 11, Table B.2.5, p.B19.

For measuring decision sight distance, the height of eye of 1,05 m should be used together with an appropriate height of object depending on the anticipated prevailing conditions. In some circumstances, the driver needs to see the road surface to read pavement markings in which case the object height is zero. In other situations overhead signs over 5 m above the road surface may be the object. The lower end of the range for a given speed relates to situations in which a driver can perceive the overall picture. The higher end of the range relates to more ambiguous situations. For example, an overhead sign in advance of an intersection which indicates which lane to be in for a given movement is more ambiguous than signing at an intersection where specific movements are visible through geometric features and lane markings. Similarly, it is difficult to judge the distance to a traffic signal if only the signal head is visible. Hence in situations where a crest vertical curve obscures the approach to an intersection, the traffic signal head should be visible over the longer decision sight distance for a given design speed.

4. ROAD RESERVE, CROSS-SECTION ELEMENTS AND CLEARANCES

4.1 ELEMENTS TO BE ACCOMMODATED

Three basic components of the cross-section of an arterial have to be accommodated with the road reserve as shown in Fig 4.1. These are:

- The roadways (carriageways)
- The verges
- The median in the case of a dual roadway

These components are discussed in detail in Chapters 5, 6 and 7.

The roadway includes all the cross-section elements between the faces of the kerbs on either side. The principal variables accounting for the width of roadway are the number of lanes and the width of lane used. The actual number of lanes to be supplied depends on the projected traffic volumes. The most common roadways required for urban arterials are two or three lanes in each direction.

The verge areas on either side of the road include all the elements from the face of the kerb to the property boundaries (edges of the road reserve). An element of the verge which could have a significant effect on road reserve widths is the slope required for earthworks and this will vary from road to road.

The median separates the two roadways and is measured from kerb face to kerb face. Principal variables affecting the width of the median are:

- . Width of the right-turn lane usually included in the median
- Width of the nose alongside the right-turn lane
- Provision for future lanes



FIGURE 4.1 Road Reserve Components

4.2 DETERMINING ROAD RESERVE WIDTH

It is recommended that the road reserve width should be determined by making provision for each of the elements required. Width should be specific to the number of lanes required, the width of median, and the width of the verges including allowances for earthworks.

The practice of using arbitrary road reserve widths for urban arterials is not recommended. Arbitrary widths could either be wasteful in taking more land than necessary or alternatively lead to difficulties through not taking sufficient load, especially where significant earthworks are required.

Determination of road reserve width therefore requires the design of the crosssection elements needed as well as the vertical alignment to determine earthworks.

Nevertheless, designers should note that many Road Authorities do have policies which lay down nominal widths of road reserve required for various classes of road, including arterials.

4.3 DEVIATIONS FROM RECOMMENDED DIMENSIONS

For each of the cross-section elements, recommended dimensions have been given. These dimensions have been determined through detailed discussions and investigations by experienced highway designers. They represent what experienced members of the profession consider to be good practice.

The chapters on the various elements provide guidelines for deviations from the recommended dimensions where, for unavoidable reasons, the recommended dimensions cannot be provided. Designers should make every effort to use the recommended dimensions. Reduction in the dimensions to a certain degree, would still enable traffic to flow but would reduce the safety of traffic operation and would increase driver tension and stress.

Where insufficient road reserve to provide the recommended dimensions for each element is encountered, decisions have to be made as to which dimensions should be reduced. This involves a process of "trade-offs". For example, where high traffic volumes and large trucks are likely to operate, reduction in the verge dimension may be preferable to reduction in lane width. Conversely, where traffic volumes will not be high or there will be few trucks, but intensive pedestrian activity is expected, then it may be prudent to sacrifice lane width or median width to preserve the recommended dimensions for the verge. High right-turning volumes would indicate a preference to preserve the recommended dimensions for the median in preference to preserving other dimensions.

4.4 EARTHWORKS SLOPES

Adequate provision must be made for earthworks slopes. The width required obviously depends on the height of the earthworks and the material through

which cuts are being made or which is being compacted to form embankments.

A nominal road reserve may be used to accommodate the roadways, median and nominal verges. Local widenings of the reserve can be made to accommodate earthworks which would encroach beyond the nominal road reserve. Various local authorities have established techniques for acquiring this additional road reserve, for example, as a servitude. The property owner is thus able to use the servitude area in calculations to establish the permissible floor area that can be constructed on the property.

4.5 CLEARANCES

The standard minimum vertical clearance from any point in a roadway to an overhead structure is 5,1 m. If the structure is light such as a pedestrian overpass, then the vertical clearance required is 5,5 m or more.

Future overlays must be taken into account when determining clearances.

Many special circumstances require specific vertical clearances either above or below the road surface. These clearances have to be determined in consultation with the appropriate authority. For example, when a road passes under a high voltage line, special clearances are necessary under the Machinery and Occupational Safety Act, 1983, No 6, 1983, Ref. 21. Similarly, special clearances relate to railways and trolley bus routes and pipelines.

Overhead traffic signs require a clearance of 5,2 m. Ref. 20.

Adequate horizontal clearance should be provided. A 2 m clear verge alongside the outer kerb is recommended. Rigid structures should be placed at least 2 m clear of the roadway.

5. **ROADWAYS**

5.1 **ROADWAY ELEMENTS**

The roadway is defined as the area available for vehicle movement between the kerbs. In the case of an undivided road the kerb would separate the roadway from the verges on either side. For a divided road, on one side the kerb would separate the roadway from the verge area and on the other side, from the median.

The width of roadways is measured from the bottom of the face of the kerb to bottom of face of kerb as shown in Figure 5.1.

Elements included in the roadway are:

- Basic lanes
- Right and left-turn lanes
- Parking lanes (where permitted)
- Channels and offsets
- Shoulders (on expressways)



FIGURE 5.1

Roadway Elements

5.2 NUMBER OF LANES

The recommended maximum number of basic or through lanes is three in each direction.

The actual number of basic lanes to be provided should be based on the design-hour volume of traffic projected for the design year, which should be 20 years after the construction of the road. If the design hour used is "a typical weekday peak hour" provision should be made for Level of Service D. In practice, in many metropolitan areas, expected volumes will be so high that even Level of Service E cannot be provided for, when projecting 20 years ahead. See Ref. 1 and Ref. 2 for determination of design capacity in relation to projected demand.

Where more than six lanes are indicated, consideration should be given to developing separate parallel facilities. The maximum that can be provided is eight
basic lanes, and this should only be resorted to in exceptional circumstances. When eight lanes are used, separate right-turn phases in the traffic signals are normally required for safety reasons.

Where arterial roads are operated as one-ways, as for example in CBDs, the maximum number of lanes should be five.

Two-lane arterials are unusual as permanent facilities. However, two-lane arterials may have frequent application as the first stage of a four-lane arterial. Where an arterial road is located in a corridor, in which traffic generation requiring more than two lanes cannot be conceived, it may be appropriate to provide two lanes only. Widening should be provided at intersections to include right-turn lanes. On steep gradients an auxiliary climbing lane should be considered.

5.3 BASIC LANE WIDTHS

The recommended basic lane width for arterial roads is 3,4 m.

Lane width is measured from the centre of the lane line to the centre of the adjacent lane line for inside lanes and to the edge of the channel or to the edge of the concrete offset from the kerb in the case of a kerbside lane.

Reduction below 3,4 m would still permit traffic operation but with increased driver tension and more potential for side-swipe accidents and for collisions with roadside fixed objects.

Lane widths have to be sufficient to accommodate the widths of the design vehicles and provide clearance between vehicles and, in the case of kerbside lanes, clearance to kerbside objects. An appropriate vehicle-to-vehicle clearance for vehicles travelling in the same direction is 1,2 m. Ref. 22, p.4.

Table 5.1 shows the clearances provided by a range of practical lane widths for various combinations of design vehicles.

Lane width and clearance between vehicles				
Lane width (m)	Vehicle types	Clearance (m)		
3,0	car to car car to truck truck to truck	1,2 0,8 0,5		
3,4	car to car car to truck truck to truck	1,6 1,2 0,9		
3,7	car to car car to truck truck to truck	1,9 1,5 1,2		

TABLE 5.1

Geometric design of urban arterial roads UTG 1, Pretoria, South Africa 1986

5.4 **RIGHT AND LEFT-TURN LANES**

The recommended width of a turning lane is 3,0 m measured from the centre of the lane line to the edge of the offset or channel.

This section refers to right and left-turn lanes which are adjacent to and not separated from basic lanes. Separate turning roadways are described in Section 10.5.

Where significant volumes of trucks are expected to be turning, the turning lane should be increased to 3,4 m or more in association with a 3,4 m adjacent lane and a 0,3 m offset from kerb. This would permit a 1,2 m vehicle-to-vehicle clear-ance between a turning truck and a through-moving car as well as a 0,6 m clear-ance to the bottom of the kerb.

The absolute minimum width of 2,7 m should be resorted to in traffic management improvements where insistence on the 3,0 m width would mean that the turning lane could not be provided.

5.5 PARKING LANES

On-street parking should not be provided on new arterial roads.

Where a new road has to be built in areas where on-street parking is present and cannot be eliminated, however undesirable, it may be necessary to provide a parking lane. In these circumstances the parking lane should have the same width as the basic through lanes, so that it can be used for moving traffic during peak periods in association with parking prohibition. Details of on-street parking are given in *Parking Standards.* Ref. 12.

5.6 CROSS-FALL

Unidirectional cross-falls of 2,0 per cent, draining away from the median are recommended for each roadway.

In areas of intense rainfall, steeper cross-falls may be necessary to facilitate drainage. The cross-fall may be increased to 2,5 per cent in such cases.

Where roadways have four basic lanes, the cross-fall should be increased by 0,5 per cent across the whole roadway or alternatively consideration may be given to a central crown (cambered) section.

In areas of steep cross-slope, it may be economical and appropriate to grade the cross-fall of the two carriageways in the same direction as the cross-slope. In such cases the cross-fall on one roadway would be towards the median. No significant discomfort is experienced by motorists on 2 or 3 per cent cross-falls towards the right-hand side instead of towards the left-hand side as is usual. Under these circumstances the cross-fall should not exceed 3 per cent.

5.7 OFFSETS AND CHANNELS

The recommended width between the bottom of the kerb face and the edge of lane is 0,3 m.

The offset is provided either by a drainage channel or, where no channel is needed, by a 0,3 m offset between the kerb and edge of lane.

5.8 SHOULDERS

For arterial roads in urban areas, the provision of outer shoulders within the roadway is not essential and the provision of inner shoulders is not recommended.

Provision for emergency stopping can be made by means of a 2 m clear verge with mountable kerbs facilitating movement onto it from the roadway.

For expressway-type facilities with some grade separations and long stretches of uninterrupted flow, the use of paved outer shoulders is recommended. If used, the recommended minimum width of these shoulders is 2,0 m.

6. VERGES

6.1 **DEFINITION AND MEASUREMENT**

The verge of an arterial road is the area between the roadway and the road reserve boundary. Its width is measured from the bottom of the face of kerb or, where no kerb exists, from the edge of hardened surface (edge of roadway) to the road reserve boundary (property line).

6.2 FUNCTIONS

The prime function of arterial roads is the movement of vehicles. The prime function of the verge of an arterial road is to provide horizontal clearance to enhance the safe and smooth flow of vehicles in the roadway. The verge is also a buffer zone between the roadway and adjacent property and therefore may include landscaping, visual screens and sound barriers. Due to the location and continuity of arterial roads in the urban framework they are often required to serve other functions and include special facilities in the verges as indicated in Table 6.1.

	New areas	Redevelopment	Central business
Driveways		X	
Footpaths	X	X	х
Overhead services	х	Х	Х
Underground services	х	X	X
Emergency parking and breakdowns	X	Х	
Bus stops	X	X	
Bicycle paths	х	Х	
Transit ways	X		

TABLE 6.1Special facilities in verges

These facilities, when required and permitted, are usually located in the verge to preserve the roadway for the prime function of the movement of vehicles.

6.3 WIDTHS

The recommended standard width of verge is 5,0 m. In new development plans the recommended standard of 5,0 m should be considered as a minimum width.

The width required for a verge may vary from road to road and along a road it may also be different on either side of the road, depending on whether the various elements are to be included or not.

In developed areas it is often impractical to set fixed widths other than a minimum to ensure that there is lateral clearance to the roadway and sight distance for safety and some flexibility to handle future unknowns. The absolute minimum verge width in these conditions is 2,0 m and this should be as a clear strip with mountable kerb. In densely developed areas such as city centres or areas of high pedestrian activity the 2,0 m may be a sidewalk and barrier kerb.

The balance of the verge should be planned according to needs with recognition that the 2,0 m clear verge may be used for certain other facilities such as underground services, parking, bus stops, driveway approaches and so forth. Typical widths of a number of elements which commonly feature in the verges of urban arterial roads are given in Table 6.2. The values given are for planning guidance only. Design of the features according to specific needs could reveal considerable differences. Also, accommodation for one feature will often satisfy the needs for other features. For example, if the road has to serve adjacent development and there is a certain number of private driveways, it would be desirable to have a 5 m wide verge to permit automobiles to turn off the roadway and stop before a private gate without obstructing through traffic. With 5 m wide verges virtually all other elements might be accommodated with the exception of major berms and earthworks.

Element	Widths
Kerb, mountable	0,3
Kerb, semi-mountable	0,15
Kerb, barrier	0,15
Drainage, inlet, manhole	1,5
Clear verge (including kerb and drainage inlet)	2,0
Footway (sidewalk)	1,5
Guardrail, barriers, walls	0,5
Electric light poles	0,3
Traffic signals	0,6-1,5
Traffic signs	0,6-2,0
Parking (parallel)	2,5
Driveway approaches	5,0
Trench width for underground service, minimum	1,0
Bus stop embayment	3,0
Bus stop passenger queue	0,7 - 1,4
Bicycle paths	3,0
Landscape strip	3,0
Berm 1,5 m high	6,0

TABLE 6.2Typical widths of verge elements

Verge elements relationships are shown in Figure 6.1.



FIGURE 6.1 Verge Elements

7. MEDIANS

7.1 DIMENSIONS

The recommended width of median is 5,0 m measured from bottom of kerb to bottom of kerb. At intersections with right-turn storage lanes the width should be 2,0 m.

The 5,0 m width of median is predicated on the basis of providing 3,0 m for a right-turn lane at intersections, together with a residual width of 2,0 m for the median at the nose and an offset of 0,3 m between the turning lane and the kerb.

The 2,0 m width alongside the right-turn lane is recommended to provide refuge for pedestrians crossing the road, one carriageway at a time. The minimum width for a kerbed median alongside the right-turn lane is 1,2 m. This width is necessary for visibility of the island, to accommodate road traffic signs, such as the "keep left" sign and to accommodate stormwater kerb-inlets in certain cases, such as on superelevated curves.

Table 7.1 below shows the functions served by various widths of median.

TABLE 7.1

Median widths

Function	Width (m)
Provision of right-turn lane and pedestrian refuge. (Recommended normal design)	5,0
Provision of right turn with painted barrier line with no pedestrian refuge. (Should be used only in exceptional circumstances)	3,0
Separation of opposing traffic. (Should be used only on long sections without intersections where sufficient width cannot be obtained for full medians)	1,5
Full median with provision for an extra lane in each direction	11,8

It is usually inadvisable to reduce the median width to 1,5 m between intersections because of the abrupt changes in alignment likely to result when widening the median to accommodate the right-turn lane and then narrowing it again after the intersection. However, where there are long sections of road between intersections and space is at a premium, consideration could be given to the narrower median provided that the transitions from the full median could be achieved with smooth, safe curves.

Where the prevailing cross slope requires the roadways on either side of the median to be at different elevations, additional width may be necessary to keep the median cross-fall to a maximum of 25 per cent.

Provision of public transport facilities such as busways or light-rail transit lines requires special treatment specific to the type of transit and the circumstances at the location. Details requiring solutions are, among others, the accommodation of the right-turn movement across the transit lanes and passenger movements as pedestrians when boarding and alighting.

7.2 MEDIAN SLOPE

The normal maximum recommended slope across a median is 10 per cent.

The absolute maximum slope should be 25 per cent and this should be used only for small differences in elevation.

At existing and proposed intersections care must be taken to ensure that vehicles crossing the arterial are not subjected to severe changes in profile.

7.3 KERBS

Semi-mountable or barrier kerbs are recommended for median islands.

Semi-mountable kerbs deter vehicles from crossing the median but do not physically prevent them. By the same token they are not likely to cause vehicles mounting them inadvertantly to go out of control.

Barrier kerbs offer more deterrence to vehicles, but also cannot physically prevent vehicles crossing. They are more likely to cause a vehicle striking them to go out of control.

At pedestrian crossings, ramps should be provided for prams and wheelchairs.

7.4 OPENINGS IN MEDIANS

Openings in medians should be restricted to intersections.

Kerbed medians reduce the number of serious head-on collisions and deter drivers from making dangerous right-turn movements into midblock driveways.

7.5 PAINTED MEDIANS

Painted medians may be used instead of kerbed medians when insufficient space is available to meet the minimum sizes and widths of island. When insufficient width is available to provide a nose of 1,2 m width alongside a right-turn lane, a painted nose should be used. The markings should be made in accordance with the requirements of the *South African Road Traffic Signs Manual*. Ref. 20.

Visibility of painted medians is, however, poor in wet weather.

8. TAPERS

There are two basic types of taper, each with different geometric requirements in various circumstances:

- An "active taper" causes lateral transition of traffic.
- A "passive taper" allows lateral transition of traffic.

Active tapers are used to narrow a roadway or a lane, or to merge two lanes into one.

Passive tapers are used to widen a roadway or a lane, or to add a lane.

In general, active tapers should be long, and passive tapers may be short.

Tapers are also used in higher type intersections to lead into and out of turning roadways. Similarly, tapers are used at the ends of embayments for bus stops and for parking.

8.1 TAPERS TO DEFINE TURNING LANES

With reference to Figure 8.1, a typical right-turn lane requires a passive taper from the right edge of the through lane to the right edge of the turning lane. This taper is normally at a rate of 1 in 10. For a 3 m wide turning lane the length of taper would be 30 m. However, in urban conditions the need for storage length in the turning lane often outweighs the need for a smooth transition and the taper rate can be reduced to 1 in 2. When traffic is light and speeds are high the transition can be done in the storage lane. When traffic is heavy and speeds are low, the 1 in 2 taper will allow 4 to 5 additional cars to queue for the turn. This will lessen the chance of vehicles at the tail of the queue blocking through traffic.

If a median has to be created or widened to "shadow" the right-turn lane, then an active taper will be required on the approach. Rates of taper for this type of condition are given in Table 8.1 which is derived from Table RMC in the *South African Road Traffic Signs Manual.* Ref. 20, p.224.

The taper rates associated with painted lines in Table 8.1 are preferred for all applications. The rates associated with kerbs are minimum rates and the kerb should be clearly visible and highlighted with paint or markers.

8.2 TAPERS TO NARROW OR MERGE LANES

These tapers are active and the rates given in Table 8.1 are appropriate. Again the more gradual taper rates associated with painted line tapers are preferred in all applications.

FIGURE 8.1

Passive Tapers in Right-Turn Lane



FIGURE 8.2 Tapers in Turning Roadways

Design speed (km/h)	30	50	60	80	100
Recommended taper (1 in) (Min. for painted line taper)	20	25	35	40	45
Min. for kerbed taper (1 in)	10	15	20	25	30

TABLE 8.1Taper rates for active tapers

Source: Ref. 20, p.224.

8.3 TAPERS TO WIDEN OR ADD A LANE

These tapers are passive and can be sharper than active tapers. As noted in Section 8.1 the taper rate can be as low as 1 in 2 if the additional lane has storage behind an intersection as its prime function. Taper rates for passive tapers are given in Table 8.2.

TABLE 8.2

Taper rates for passive tapers

Design speed (km/h)	30	50	60	80	100
Taper rate (1 in)	5	10	15	20	25

8.4 TAPERS IN TURNING ROADWAYS

As shown in Figure 8.2 passive and active tapers may be used to set off a turning roadway from through lanes in channelized intersections.

The rates of taper given in Table 8.2 apply to the passive taper leading into the turning roadway. When tapers are applied to either end of the horizontal curve of a left-turning roadway the far-side taper can be shortened to the same rate of taper as the near-side passive taper. If the turning lane does not lead into an extra lane on the through cross-road it is important that the approach angle is reasonable to give the driver a good view of the traffic stream to be entered. In this regard, this taper can be deleted and the circular arc of the turning roadway can be tangent to the cross-road.

8.5 TAPERS IN BUS STOP EMBAYMENTS

Details of bus stop embayment design are given in *Bus Terminals and Bus Sta*tions Planning and Design Guidelines, Ref. 14. The passive taper leading into an embayment is at a taper rate of 1 in 4. In higher speed conditions, 80 km/h or more, this taper is increased to 1 in 6. The active taper leading out of the embayment is 1 in 6 in normal urban conditions and 1 in 12 for higher speed situations.

8.6 TAPERS IN PARKING EMBAYMENTS

Where parallel parking is provided off the travelled way of an arterial in an embayment the normal practice is to provide tapers at the start and end of the embayments, both at a taper rate of 1 in 2. In such a case demarcation for parking is desirable.

9. ALIGNMENT, CURVATURE AND GRADIENTS

9.1 MINIMUM RADIUS FOR HORIZONTAL CURVES AND SUPERELEVATION

Due to topographical and existing development constraints the designer of urban arterial roads is often confronted with tight design situations where curves have to be fitted in. In high-speed rural situations a sharp curve can be given a high rate of superelevation to offset the side friction forces. In urban areas with stop-go traffic conditions and limited space for embankments it is often not practical to use high rates for superelevation.

The maximum rate of superelevation recommended for urban arterial roads is 0,06 and possibly 0,08 on expressway-type arterials.

The recommended design practice is to use, where possible, large radius curves without superelevation. Where large radius curves are not possible, superelevation can be introduced to offset the side friction forces of small radius curves.

In Table 9.1, the minimum radius values of horizontal curves for different design speeds are derived from the formula,

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

where

R = radius of curve in metres

V = design speed in km/h

e = superelevation rate in metres per metre

f = side friction factor

127 = a constant for metric units.

Design	Side	Minimu	ım radius	for maxim	um supere	elevation ra	ates (e)
km/h	factor (f)	-0,02	0	+0,02	+0,04	+0,06	+0,08
50	0,16	140	125	110	100	90	
60	0,15	220	190	170	150	135	125
70	0,15	300	260	230	205	185	170
80	0,14	425	365	315	280	255	230
90	0,13	585	495	400	380	340	305
100	0,13	executive	610	530	465	420	380

TABLE 9.1Minimum radius for horizontal curves (m)

Figures 9.1 and 9.2 give design superelevation rates for above minimum radii under design standard conditions of maximum superelevation.

 $e_{(max)} = 0.06$ and 0.08 respectively. Ref. 7, pp.177-191.

The margin of safety in Table 9.1 is quite high as the friction factors used relate to driver comfort rather than to limiting friction between tyres and roadway. AASHTO recognize that higher friction factors can be used on low-speed urban streets and suggest for example that f could equal 0,30 instead of 0,17 for a 30 km/h design speed and could equal 0,18 instead of 0,15 for a 60 km/h design speed. Ref. 7, p.210.

It should be noted that the friction factors based on driver comfort were measured in the 1930s and 1940s. Since then there have been many innovations in vehicle suspension, steering mechanisms and tyres, all of which make driving and particularly cornering more comfortable.



FIGURE 9.1 Superelevation Rates for $e_{(max)} = 0,06$



FIGURE 9.2 Superelevation Rates for $e_{(max)} = 0,08$

Hence, while the values given in Table 9.1 are recommended minimums, a designer, if faced with costly solutions, could choose radii up to about 30 per cent less in some situations and still provide a reasonable design. In such situations, however, careful attention should be given to other elements of design such as sight distance, pavement surface, lane widths, clearances and signing.

9.2 TRANSITION CURVES

Transition curves may be used to good effect in the design of urban arterials, particularly for high-speed expressway-type arterials. However, as traffic on most urban arterials operates at 80 km/h or less, the need for transition curves is not essential, particularly if superelevation rates are 0,06 or less.

9.3 SUPERELEVATION RUN-OFF

There are a number of procedures used to achieve transition from normal crossfall to superelevation. Normally the cross gradient can be rotated about the centre line of roadway, that is dropping one edge and raising the other. This method usually requires the least length to develop the full transition. The cross gradient could also be rotated about the inside or outside edge of roadway or about the centre line of the median. For the same rate of transition this will require twice the length as rotation around the centre line. Attention must be given to potential differences in elevation across the median.

The basic principles are to achieve visually smooth transitions and to maintain proper drainage run-offs.

The achievement of visually smooth transitions is empirical and procedures described by AASHTO refer: Ref. 7, pp.197-209 and pp.212-217. In essence the gradient of an edge profile should not exceed the gradient of a centre-line profile by varying amounts depending on operating speed and distance between the two profiles. The basic relationship between edge and centre-line profiles for a two-lane roadway is given in Table 9.2.

TABLE 9.2

Relationship of speed to maximum relative profile gradients, edge and centreline two-lane roadway

Design speed km/h	Maximum relative gradients	Equivalent maximum Relative slopes
30	0,80	1 in 125
50	0,67	1 in 150
80	0,50	1 in 200
110	0,40	1 in 250

Source: Adapted from Ref. 7, Table 111-13, p.199.

If the rotation is around the centre line of four lanes instead of two lanes theoretically the length of run-in or run-off should be double. AASHTO, however, suggest empirically that the length need only be 1,5 times as much. Ref. 7, p.201.

9.4 LANE WIDENING

Lane widening for sharp curves as found in turning roadways at intersections is discussed in Section 10.5. With 3,4 m basic lane widths for arterials, there is also a requirement for lane widening in certain of the minimum radius design situations. For a radius of 100 m or less a 0,45 m widening is recommended. For a radius of 100 m to 200 m a 0,3 m widening is recommended. Such lane widening is normally developed on the inside of curve edge over the length of superelevation run. Ref. 7, pp.236-243.

9.5 SIGHT DISTANCE ON HORIZONTAL CURVES

See Subsection 3.4.1 and Figure 3.5.

9.6 GENERAL HORIZONTAL ALIGNMENT CONTROLS

In addition to the preceding suggested standards concerning horizontal alignment, the following general statements are presented as design supplements: Ref. 22, pp.12-14.

- i) On higher type arterials, alignment should be as directional as possible, but consistent with topography and with preserving developed properties and community values. On lower types, the alignment should both enhance scenic views and discourage high-speed traffic. The alignment should minimize nuisance factors to neighbourhoods, such as excessive cuts or fills, sharp break-over angles, speed-inducing sections, or cut throughs.
- ii) Sharp horizontal curvature should not be introduced at or near the top of a pronounced crest vertical curve, nor at or near the low point of a pronounced sag vertical curve.
- iii) Consistent alignment is necessary. A driver should not be surprised. Sharp curved sections after long straight sections should be avoided.
- iv) Independent horizontal alignment can be employed on divided roadways to increase safety and better fit existing physical design restraints.
- v) As a rule of thumb, the maximum number of breaks in the course of a horizontal line that a driver can see should not exceed two.
- vi) Kerb alignment should always be smooth and avoid kinks due to small deflection angles, except at beginnings of turning lanes.
- vii) Broken-back curves are not desirable.

9.7 VERTICAL ALIGNMENT Ref. 10, p.4.2

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 the 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. 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.

9.8 VERTICAL CURVATURE

The rate of vertical curvature, called K, is the distance required to effect a 1 per cent change of grade. Vertical curves are specified in terms of this factor, K.

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

where L =length of vertical curve in metres

and A = the algebraic difference between grades in percentage.

9.8.1 Minimum rates Ref. 10, p.4.2.1

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 degree above the longitudinal axis of the headlights.

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 9.3.

9.8.2 Minimum lenghts Ref. 6, p.4.2.1

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 1,0 per cent, the vertical curve is often omitted. For algebraic differences in grade greater than 1,0 per cent, minimum lenghts are suggested in Table 9.4 for purely aesthetic reasons.

TABLE 9.3

	Stopping		þ	<
Design speed	sight	Crest	Sa	ng
	distance (m)		Headlight	Comfort
40	45	6	6	4
50	65	11	11	6
60	80	16	17	8
70	95	23	24	12
80	115	33	31	16
90	135	46	49	20
100	155	60	52	25
110	180	81	55	25
120	210	110	60	30

Minimum values of K for vertical curves

* Based on a level section. Adjustments required for gradient.

Source: Adapted from Ref. 10, Table 4.2.1

TABLE 9.4

Minimum lenghts of vertical curves

Design speed	Length of curve
(km/h)	(m)
40	60
60	100
80	140
100	180
120	220

Source: Ref. 10, Table 4.2.2

9.9 GRADIENTS Ref. 10, Subsection 4.3

9.9.1 Maximum gradients

The speed of passenger cars is relatively unaffected by gradient, and the horizontal alignment will tend to govern the selection of speed.

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Truck speeds are, however, markedly affected by gradient. The design should therefore aim at gradients 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.

Maximum gradients for different design speeds and types of topography are suggested in Table 9.5. 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.

Design speed km/h		Topography	
	Flat	Rolling	Mountainous
50	8	9	11
60	7	8	10
80	6	7	9
100	4	5	6

TABLE 9.5

Maximum gradients (in per cent)

Source: Ref. 22, p.23

9.9.2 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 20 km/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.

Gradient %	Length of grade (m)
3	400
4	300
5	240
6	200
7	170
8	150

TABLE 9.6 Critical length of grade

Source: Ref. 10, Table 4.3.2

Critical lengths can be read off from Figure 3.3, and are given in Table 9.6 for convenience.

9.9.3 Minimum gradients

The recommended minimum gradient is 0,5 per cent, i.e. 1 in 200.

9.9.4 Warrants for climbing lanes

Climbing lanes are not a usual feature of urban arterial roads. In special circumstances where they may be warranted the designer is referred to Subsection 4.4 in Ref. 10.

10. INTERSECTIONS

10.1 INTERSECTION FORM

The large variety of intersection designs encountered can be classified into a limited number of basic forms, viz.:

- T-type and T-skewed
- Y-type
- Cross and cross-skewed
- Staggered (to the left and to the right), staggered-skewed
- Multi-leg

Two cardinal rules should apply to the design of intersections: Ref. 23, p.507.

- (1) No intersection should be planned for more than four two-way intersection legs.
- (2) The angle of crossing manoeuvres should be approximately a right angle for movements intended to operate at high relative speed.

The maximum departure from a right angle is recommended as 20°.

Figure 10.1 shows the intersection forms recommended. Ref. 24, Fig. 21, sheet 1. A multi-leg intersection should not be provided in new design. In road improvement schemes, existing multi-leg intersections should be converted to four-leg intersections through channelizing procedures.

Staggered intersections are acceptable when the distance between the offset legs is adequate for weaving and storage of right-turning vehicles. When this distance is adequate, design becomes that for two intersections, each of three legs. The preferred direction for the offset should be such that traffic travelling from one of the minor roads onto the other via the arterial road should be able to do so without making a right turn from the arterial road.

10.2 SPACING OF INTERSECTIONS

The recommended minimum distance between four-leg intersections on arterials with no property access is given in Table 2.2.

T-type intersections should not be closer than 100 m, with a maximum of six intersections per kilometre.

The recommended minimum distance between a freeway ramp intersection and another intersection on the arterial road is 300 m.



FIGURE 10.1

Intersection Forms

Uniform spacing promotes progressive flow of traffic through signalized intersections. With spacings between 500 m and 600 m optimum progression can be achieved. Every effort should be made for such uniform spacings although in practice, the intersection location is often governed by topography and prior development. Ref. 25, p.105.

No hard and fast rules can be laid down for the spacing of intersections. Tradeoffs have to be considered. Infrequent spacing promotes faster and smoother traffic flow with fewer conflict situations. However, traffic volumes and turning volumes in particular are then higher at the few intersections available. In addition less traffic service is given to the surrounding development.

10.3 SIGHT DISTANCE

As a matter of policy, all intersections on an arterial road should be controlled.

The minimum degree of control would be yield control. In other circumstances the minor road may have stop control, while the arterial road has priority. Intersection design should make provision for the eventual control of all four-leg intersections by signals.

The sight distance required in the design of intersections depends on the type of control at the intersection.

10.3.1 Stop control

At a stop-controlled intersection, the driver of a stationary vehicle must be able to see enough of the major road 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 roadway, the width of the road being crossed and the length of the crossing vehicle. This manoeuvre must be completed in the time it takes for the approaching vehicle to reach the intersection, assuming that the approaching vehicle is travelling at the operating speed of the through road. For safety, the time available should also include allowance for the time it takes 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 10.2.

The object height is 1,3 m. 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.



FIGURE 10.2 Shoulder Sight Distance for Stop Condition

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Shoulder sight distances, recommended in accordance with the principles outlined above, are given in Figure 10.2. Before a lower value is adopted in a specific case, the implications of departing from the recommended values should be studied.

10.3.2 Yield control

Leisch suggests that where an intersection approach is controlled by a yield sign it can be assumed that a driver on that approach will reduce his speed sufficiently to enable him either to stop or to accelerate and pass through the intersection. Ref. 26, p.4.31

The required sight line for this condition is established by the stopping sight distance for the reduced speed along the controlled approach, and by the distance that a vehicle would travel along the uncontrolled approach during the time that it would take for the vehicle on the yield approach to enter and pass through the intersection. He suggests an approach speed on the yield-controlled approach of 25 km/h for urban conditions. The distance required for the vehicle to decelerate to standstill would be 24 m (see Figure 10.3). If the vehicle on the crossing road 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 on the through road would be forced to slow down to match speeds at a safe following distance, and the distance, S, allows for this speed adjustment.

10.4 TURNING SPEED AND RADIUS

Turning manoeuvres at urban intersections are made at much lower speed than the basic operating speed of the arterial road.

Right turns involve direct crossings of opposing vehicle paths and are usually made at minimum speeds of 15 km/h or less. Ref. 26, p.4.43.

Left turns are also made at low speed but speeds would generally be higher than the right-turn speeds. Drivers do respond to the restricted speed required in turning through an intersection. It is recommended that speeds on turning roadways be considered as being in the range of 25 km/h to 35 km/h.

10.4.1 Side friction factors

Higher side friction factors are tolerable for the low speeds used in intersection turning movements. Table 10.2 below records the suggested side friction factor for various turning speeds. These friction factors vary from 0,27 for 35 km/h up to 0,45 for 10 km/h. Ref. 7, p.220.



FIGURE 10.3

Shoulder Sight Distance for Yield Condition

	40	4 -	05	05
Speed V (km/h)	TU	15	25	35
Side friction factor f	0,45	0,38	0,32	0,27
Assumed minimum superelevation e	0,00	0,00	0,00	0,02
Calculated safe minimum radius R (m)	2	5	15	33
Recommended design radius (m)	12	12	15	35

TABLE 10.1Speed-curvature relationships for intersections

Source: Adapted from Ref. 7, Table III-16, p.220.

10.4.2 Superelevation

Problems associated with the vertical grading of intersections frequently do not permit superelevation to be applied to turning movements. The lower speeds of 10 km/h to 25 km/h can be maintained with no superelevation and small radii. When speeds above 25 km/h are desired, a minimum of 0,02 of superelevation should be applied.

10.4.3 Safe radii

Table 10.1 shows the speed-curvature relationships for turning movements at intersections. For the lower speeds the radius is controlled more by the turning characteristics of the vehicle than the speed-curvature relationship.

10.5 TURNING-ROADWAY WIDTHS

Table 10.2 shows the recommended widths of turning roadways associated with intersection channelization. The widths are set out in terms of three different operating cases:

Case I for single-lane one-way operation without any provision for passing a stalled vehicle.

Case II is also for single-lane one-way operation, but makes provision for passing a stalled vehicle.

Case III makes provision for two-lane operation, whether it be one-way or twoway.

Each of these operating cases requires different lane widths, depending on the type of vehicle governing the design.

10.6 CHANNELIZATION AND TRAFFIC ISLANDS

Channelization can be defined as the use of islands at intersections to direct traffic along definite paths to simplify operations. Ref. 23, p.503. The purposes of channelization are:

- To separate manoeuvre areas and present drivers with one decision at a time
- To control the manoeuvre angle to obtain small angles for merging and the diverging at low relative speeds and crossing at approximate right angles at high relative speeds

	Pavement width (m) for:								
R Radius on inner edge of carriageway (m)	Case I 1-lane, 1-way operation; no provision for passing a stalled vehicle		Case II 1-lane, 1-way operation with provision for passing a stalled vehicle			Case III 2-lane, either 1-way or 2-way operation			
	Design traffic condition				onditio	n			
	A1	B2	C3	A1	B2	C3	A ¹	B2	C3
15 20 30 40 60 90 120 150 Width r Mountable curb	5,1 4,5 4,2 4,0 3,7 3,7 3,7 3,4 modific	5,1 4,8 4,5 4,5 4,5 4,2 4,2 4,2 ation re	6,5 5,4 5,1 4,8 4,5 4,5 4,5 4,2	6,5 5,9 5,7 5,4 5,4 5,1 5,1 5,1 g edge	7,1 6,5 6,2 5,9 5,7 5,7 5,7 5,7 of pave	8,2 7,6 7,1 6,8 6,5 6,2 6,2 6,2 ement t	8,8 8,2 7,9 7,6 7,6 7,4 7,4 7,4 7,4	9,9 9,3 8,8 8,5 8,2 7,9 7,9 7,9 7,9 7,9	11,9 10,5 9,9 9,3 8,8 8,5 8,2 8,2
Barrier curb: One side Two sides	Add 0,3 Add 0,6			None Add 0,3			Add 0,3 Add 0,6		
¹ Traffic condition A ² Traffic condition B	 Presin Sut 	edomina gle-unit fficient s semi-tra	ntly pas (SU) true SU vehic ailer vehic	ssenger cks. cles to g icles.	vehicle jovern c	s, but s lesign, t	some co out som	onsidera e consid	tion for
virattic condition C	 Sufficient bus and combination-types of vehicles to govern de- sign. 						ern de-		

TABLE 10.2

Design widths for separate turning roadways of various radii

Source: Ref. 7, p.232.

- To control speed by bending or funneling
- To provide a refuge between traffic flows for pedestrians
- To provide protection and storage of turning and crossing vehicles
- To eliminate excessive intersectional areas which permit drivers to perform improper manoeuvres or to travel along unpredictable paths
- To prevent illegal manoeuvres
- To provide space and protection for traffic control devices

The traffic lanes so formed should be delineated by traffic islands preferably formed by semi-mountable or barrier kerbs. Islands may be classed into three groups:

- Directional islands which direct the traffic along the correct channel or prevent illegal manoeuvres
- Divisional islands which separate opposing traffic flows
- Refuge islands to protect pedestrians crossing the roadway or to accommodate traffic control devices

10.6.1 Size of island

Small islands should be avoided as they have low visibility and are unable to safely accommodate traffic signs or other control devices. The minimum width of islands should be 1,2 m. The minimum area of island should be 5 m². Islands used for pedestrian refuge should preferably be 2 m wide with a minimum of 1,2 m together with ramped kerbs. Where wheelchair access is to be considered a minimum width of 1,5 m is needed together with ramped kerbs.

10.6.2 Approach-end treatment

The most serious problems caused by poorly designed islands are related to inadequate approach-end treatment. The points at the intersections of the side of islands should be rounded for visibility.

Island kerbs are usually introduced suddenly and should be offset from the edge of basic lanes. A mountable kerb need not be offset from the edge of a turning roadway except that the approach nose should preferably be offset an additional 0,5 to 1,0 m to reduce its vulnerability. Barrier kerbs should be offset from the edges of through and turning roadways.

The approach end of an island should be conspicuous to approaching drivers and should be definitely clear of vehicle paths, physically and visually so that drivers will not veer from the island. The approach nose should be offset an amount greater than that of the side of the island itself. Where feasible the total nose offset should be 1,0 m to 2,0 m from the normal edge of the through roadway and 0,5 m to 1,0 m from the pavement edge of the turning roadway. Ref. 7, p.759 and Ref. 3, p.D16.

Figure 10.4 shows how offset to triangular islands should be handled.



FIGURE 10.4 Island offsets

10.7 RIGHT AND LEFT-TURN LANES

Right-turn lanes are required to provide storage space for right-turning vehicles while they wait for gaps in the opposing flow of traffic to make the right turn, or while they wait for the appropriate signal indication to do so.

The minimum length of right-turn lane should provide storage for at least two cars. With more than 10 per cent truck traffic, provision should be made for at least one car and one truck.

The length of storage lane depends on the volume of right-turning traffic, the volume of opposing traffic and the cycle length. As a thumb rule for low right-turning volumes up to 200 vph, storage should be provided for three times the average number of vehicles turning right per signal cycle.

Figure 10.5 shows the length of left and right-turn lanes that should be provided at signalized intersections. Ref. 22, Figs. 9.2 and 9.3, p.35.

Figure 10.6 shows the length of right-turn storage lanes to be provided on fourlane at-grade unsignalized intersections. Ref. 22, Fig. 9.1, p.34.

10.7.1 Width of lane

Refer to Subsection 5.4.

10.7.2 Tapers to form turning lanes

Refer to Subsection 8.1.

10.8 RADII AT CORNERS

The fundamental principle to be applied in determining the radius of a left-turn movement at the intersection is that the traffic departing from the arterial road should be able to move from the arterial road onto the cross-road without encroaching onto a lane that might be occupied by oncoming traffic on the crossroad.

No hard and fast rule can be laid down for the treatment at the intersection. Treatments vary from simple curve through simple curve plus tapers to three centre curves. Table 10.3 shows typical curve radii associated with various tapers and approach-curve treatments.

Figures 10.7 and 10.13 show various treatments of left-turn design commonly used. Ref. 3, p.D38 and Ref. 24, Figs. 24 and 26, sheets 1-5.

10.9 CORNER SPLAYS

Corner splays will generally be required from the properties at the corners of the intersection. The actual shape of the splays will depend on the design of the particular intersection. As a general rule the objective should be to maintain a minimum 3,5 m width of border area around the corner.

Treatment	Minimum curve radius (m)	Approach and departure treatment
Turning roadway with channelizing island: Curve and tapers Turning roadway with channelizing island:	25 to 30	1:10 tapers
Three-centred curve	15	45 m
Three-centred curve (no turning roadway)	6	30 m curves
Simple curve and tapers	6	1:15 tapers
Simple curve	10 to 12	

TABLE 10.3Typical kerb radii at intersections

10.10 KERBS

Barrier kerbs or semi-mountable (45° slope) kerbs are preferred at intersections to improve visibility.

At pedestrian crossings, ramps should be provided for prams and wheelchairs.



FIGURE 10.5

Turning lanes: signalized intersections



FIGURE 10.6 Right-turn lanes: unsignalized



FIGURE 10.7 Typical left-turn designs



Arterial with tapers at intersections with minor roads



FIGURE 10.9

Typical major intersection channelization slip road







FIGURE 10.11 Alternative left-turn configuration

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FIGURE 10.12 Protected left-turn slip road

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FIGURE 10.13 Slip road plus additional merging lane

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