Chapter 7

Roads: Geometric design and layout planning

7

TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

SCOPE

In this chapter the major objectives are to

- stress the importance of geometric design, giving expression to planning concepts;
- emphasise the revised approach to the road hierarchy;
- suggest the importance of satisfying the needs of all road users, both vehicular and non-vehicular; and
- provide guidelines for detailed geometric design that will result in a safe, efficient, affordable and convenient road and street system.

INTRODUCTION

Reference to planning

The ultimate objective in the creation of an urban place is that it should be such that people would wish to live, work and play there. This can only be achieved by the closest co-operation between the planner and the geometric designer, because the ultimate layout of the street system effectively defines the urban area in terms of its functionality and, hence, its attractiveness to the inhabitants.

These two disciplines must also interact closely with the other disciplines involved in the provision of services to the inhabitants. As streets also form sets of conduits along which essential services such as water supply, sewerage and power are conducted, they should be so located that they do not unnecessarily constrain the provision of these services.

This chapter of the guidelines cannot, therefore, be read in isolation.

Both the planner and the designer are required to adopt a more holistic approach to determination of the street network than has previously been the case. They may find it useful to consider the total width of the cross-section as being hard open space, only part of which is dedicated to the movement function. This part of the cross-section is roughly equivalent to the road reserve as previously understood and is still required to address a range of trip purposes, trip components and modes of travel.

With regard to modes of travel, the design of the street network was historically predicated almost exclusively on the passenger car. Many trip makers will, however, always be reliant on walking or public transport as the only modes available to them. Furthermore, it is not possible to endlessly upgrade the street network in terms of a growing population of passenger cars.

Road design should, therefore, not only accommodate public transport but actively seek to encourage its use. Adding embayed bus stops to a route essentially designed for passenger cars does not constitute support for the promotion of public transport.

Obviously, bus routes should be designed with the bus as the design vehicle. This selection of design vehicle impacts, inter alia, on decisions concerning maximum gradients, lane widths and provision for bus stops. A designated bus route should have a horizontal alignment planned to enhance the attractiveness of the route to would-be passengers and also be highly accessible to pedestrians by ensuring that walking distances to the nearest bus stop are minimised. Dedicated bus lanes should be provided in areas where the volume of bus or other high-occupancy vehicles warrants their use. The dedicated bus lane implies that the street will essentially be a shared facility serving other modes of transport as well. The high volumes of bus traffic that are typically achieved when a number of bus routes converge on the CBD or some other transport hub may suggest that dedicated bus routes, as opposed to bus lanes, become a practical option.

The distinction drawn between *geometric planning and geometric design* is not always clear. In this document, geometric planning is described in Chapter 5: Planning Guidelines. A brief exposition of the difference between planning and design is offered below.

Geometric planning

Planning addresses the broad concepts in terms of which the functions of the various links in the street network are defined. These concepts address the sum of human activity, whether economic (which can be formal or informal), recreational or social - the latter including educational, health care and worship activities. In this context, it is pointed out that movement is a *derived* activity or demand.

Previously, both planning and design tended to focus on areas being dedicated to single land-use, thus forcing a need for movement between the living area and any other. Current planning philosophies favour the abandonment of single use in favour of mixed use. Mixed use suggests that people can both live and work in one area. Not only will this have the practical effect of reducing the demand for movement over long distances but, where movement is still necessary, it will support change of the mode of movement because, over short distances, walking and cycling are practical options. This will further reduce the areal extent required to be dedicated to movement. Conceptual planning leads to the definition of corridors intended to support some or other activity, one of which is movement.

Corridors, in association with their intended functions, will ultimately define the horizontal alignment of the streets located in them. A need for high traffic speeds will suggest high values of horizontal radius, whereas reductions in radius could be applied to force speeds down to match activities involving a mix of vehicular and non-vehicular traffic. Reduction of the lengths of tangents between curves or intersections could serve the same purpose.

The dominant function of the corridor defines the vertical alignment in terms of maximum and minimum acceptable gradients, vertical curvature and length of grade. For example, streets with a predominantly pedestrian function should ideally be flat, whereas - if movement includes provision for a bus route - modest gradients are allowable. Routes intended principally for the movement of vehicles other than buses may be steeper although, where very high volumes are anticipated, the adverse effect of steep gradients must be borne in mind.

Function is defined in terms of two prime components, namely the nature and the extent of demand. As such, it is the major informant of the design of the cross-section. The demand may be for high-speed, high-volume traffic flows, in which case the cross-section would comprise more than one moving lane in each direction, possibly with a median between the opposing flows and shoulders as opposed to sidewalks. On the other hand, if the demand is for predominantly pedestrian/ commercial activity, very wide sidewalks (i.e. wider than would be required merely to accommodate a volume of moving pedestrians) would be necessary to allow for sidewalk cafes, roadside vending and browsing or window shopping. While vehicles would not necessarily be excluded, their presence would not be encouraged and speeds would be forced down by having few and narrow lanes and very short tangent lengths.

Geometric design

Design is principally concerned with converting to physical dimensions the constraints introduced by planning concepts. Ongoing reference to the chapter on Planning is necessary to ensure that the road as ultimately designed matches the intentions regarding its function. It is important to realise that the function of the road reserve is broader than merely the accommodation of moving traffic which may be either vehicular or pedestrian. Although geometric design tends to focus on movement, the other functions must be accommodated. If the designer does not adopt this wider perspective, the most likely consequence would be that the original intention - “the creation of an urban place that should be such that people would wish to live, work and play there” - would be severely compromised.

The goals of transportation as propounded by the Driessen Commission are the economic, safe and convenient movement of people and goods with a minimum of side-effects. These goals are unchanged. It must be understood that, in arriving at an acceptable street design, these goals apply equally to all modes of travel. In this respect, it will invariably be necessary to seek compromises between the various modes. The one goal seen as being non-negotiable is safety. For example, the safety of pedestrians cannot be compromised in pursuit of convenience of vehicular travel.

Classification of the road and street system

The traditional five-level hierarchy of streets has effectively been abandoned, principally because it placed an over-emphasis on the vehicular movement function of the street system. The concept of a hierarchy also implicitly carried with it the notion of one part of the network being more important than another. The network comprises a system of interlinking streets serving different functions, and often serving these different functions differently.

Over-emphasis of the importance of one link at the cost of another does not only constitute poor design; it can place the network as a whole in jeopardy. In fact, all parts of the network require equal consideration. To assist designers in developing some understanding of the new classification system, Table 7.1, offering a comparison between the previous five-tier system, the Urban Transport Guideline (UTG) series, and that currently employed, is shown below.

It should, however, be clearly understood that there is not a one-to-one relationship between the current and the other classifications. The five-tier and the UTG classification systems are limited to addressing movement in terms of a spectrum of accessibility versus mobility, whereas the current classification addresses all functions of roads and streets. Reference should be made to Chapter 5.1: Movement Networks, in which the classification system is comprehensively described.

Mixed routes forming part of the “movement network” classification can be subdivided into “higher-order”, “middle-order” and “lower-order” routes. Higher-order routes would carry higher volumes of traffic *and/or accommodate higher levels of economic activity*, whereas lower-order routes would principally address local and access-seeking traffic and *accommodate higher levels of recreational activity.* Middle-order routes serve primarily as links between higher- and lower-order routes. It would thus be unwise to regard the appellation of “higher-order” as an invitation to reach for, say, UTG 5.

The five-tier system subdivided Class 5 streets into a further six sub-classes, two of which could be loosely

Table 7.1: Comparison between classification systems

MOVEMENT NETWORK	FIVE-TIER SYSTEM	URBAN TRANSPORT GUIDELINES (UTG SERIES)
Vehicle-only route	1 Regional distributor	(Freeways not included)
	2 Primary distributor	Major arterial (UTG1)
Mixed pedestrian and Vehicle route	3 District distributor	Minor arterial (UTG1)
	4 Local distributor	Collector (UTG15)
	5 Access street	Local street (UTG7 &10)
Pedestrian-only route		(not applicable)

classified as being pedestrian-only. The UTG series does not address either freeways or pedestrian-only routes. UTG 10 addresses local commercial and industrial streets which fall outside the ambit of this document.

Measures of effectiveness (MOE)

Geometric design is primarily concerned with the assemblage of a group of components leading to the creation of an operating system. As a case in point, the cross-section is not whole and indivisible: it is, in fact, heavily disaggregated. In the preparation of a design, it is thus necessary not only to be aware of the various functions that the geometric design is intended to serve but also to be able to measure the extent to which the often conflicting functions are served. Invariably, the local authority will specify that the facility provided should meet some or other specified level of utility. Measures of effectiveness (MOE) are thus required.

With regard to movement, MOEs can relate either to management of the operation of infrastructure, or to the provision of infrastructure.

In the former case, reference is to Transportation System Management (TSM) and its strategies and processes, all of which have MOEs associated with them. Reference should be made to *Guidelines for the transportation system management process* (1991), Pretoria: Committee of Urban Transport Authorities (Draft Urban Transport Guidelines: UTG 9). This document is available from the National Department of Transport.

Management is aimed at enhancing the productivity of the system as provided. A well- managed system could, with a lesser extent of infrastructure, conceivably accommodate the same demand for movement as an unmanaged system. By the same token, a design that is sensitive to the possibilities contained in good management would result in a highly efficient use of space.

A lesser extent of infrastructure implies that more land is available for other applications and the capital outlay involved in infrastructure provision per erf is lessened. This benefit applies directly to all aspects of community life. Residential properties are rendered more affordable because the costs incurred in servicing them inevitably impact on property prices. The financial return from commercial properties is enhanced because the lower land price has an effect on the business overheads of the occupier. In short, commercial activities can become more competitive.

MOEs that refer to the provision of infrastructure invariably refer to the Transportation Research Board Special Report 209: *Highway Capacity Manual (HCM)*, the most recent edition of which was issued in 1994. This manual is in general use in South Africa. The manual comprehensively addresses the entire spectrum of modes of movement, and all aspects of the road or street network, from freeways to residential streets to intersections. The HCM propounds a philosophy of Levels of Service (LOS) and, in general, refers to five levels ranging from A to E with LOS A being the highest level and LOS E corresponding to capacity. The higher levels are typically related to LOS E by utilising a volume-to-capacity or v/c ratio. The actual MOEs on which the levels of service are based vary between the various types of facility being analysed. Those applying to a freeway do not apply to a residential street or to a signalised intersection so that, although in all cases reference could be made to LOS A, what is intended is not comparable between them.

It is recommended that analysis be based on the Highway Capacity Manual, with due regard being paid to the practical benefits to be derived from the application of TSM measures.

Traffic calming

Traffic calming refers to measures usually designed to reduce either the volume of moving vehicles or their speed. The intentions behind the application of traffic calming can, however, be many and various. In addition to reductions in speed or volumes of traffic, they may include:

- noise reduction;
- reduction of air pollution; and
- provision of safe areas for pedestrians or other non-vehicular road users.

Planning aimed at achieving these intentions reduces the need for the introduction of traffic calming measures which are essentially artificial devices such as speed humps, chicanes, street narrowing devices, road closures and changes in surfacing colour or texture.

Planning measures could include the selection of link lengths as a means of reducing variations in vehicle speed. As a rule of thumb, a link length in metres that is about ten times the desired speed in km/h would ensure that a vehicle entering the link at less than the desired speed is not likely to accelerate beyond it. Alternating priority control at the intersections along a street would also serve as a restraint on excessive speed, as would bounding the various links by three-legged or T- intersections.

Small-radius horizontal curvature would effectively cause a reduction in traffic speeds. However, this application is not recommended for general practice. Highly curvilinear alignments impact adversely on the costs of provision of all the services normally located within the road reserve. Furthermore, pedestrians would be subjected to unnecessarily long travel paths between origin and destination.

Should traffic calming measures be necessary, an area-wide approach is preferable to isolated measures. The effect is to impose the desired conditions over a wide area so that low speeds or volumes become part of the drivers' expectations in respect of the area being traversed. An isolated speed hump located where a driver does not expect it can result in loss of control or damage to the vehicle itself.

Traffic calming devices are discussed in detail in Schermers G and Theyse H (1996), *National guidelines for traffic calming.*

BASIC DESIGN PARAMETERS

The design vehicle

The design vehicle is a composite rather than a single vehicle. It thus represents a combination of the critical design features of all the vehicles within a specific class weighted by the number of each make and model vehicle found in the South African vehicle population.

The dimensions offered in Table 7.2 were determined by Wolhuter and Skutil (1990). The values quoted in the table are 95 percentile values.

Because of its application in the determination of passing sight distance, the fifth percentile value of height is selected. The height of passenger cars is thus taken as 1,3 m. A height of 2,6 m is adopted for all other vehicles.

Two vehicles are recommended for use in the design of urban roads. The passenger car should be used for speed-related standards and the bus for standards relating to manoeuvrability, typically at intersections. The bus also dictates the maximum permissible gradient. Designs must, however, be checked to ensure that larger vehicles, such as articulated vehicles, can be accommodated within the total width of the travelled way, even though they may encroach on adjacent or even opposing lanes. Should these larger vehicles comprise more than 10% of the traffic stream, it will be necessary to use them as the design vehicle.

In constricted situations where templates for turning movements are not appropriate, the capabilities of the design vehicle become critical. Ninety-five percentile values of minimum turning radii for the outer side of the vehicle are given in Table 7.3. It is stressed that these radii are appropriate only to crawl speeds.

Truck speeds on various grades have been the subject of much study under southern African conditions. Bus speeds are similar and it has been found that

Table 7.2: Dimensions of design vehicles (m) (after Wolhuter and Skutil 1990)

VEHICLE	WHEEL BASE	FRONT OVERHANG	REAR OVERHANG	WIDTH
Passenger car (P)	3,1	0,7	1,0	1,8
Single unit (SU)	6,1	1,2	1,8	2,5
Single unit + trailer (SU +T)	6,7+3,4*+6,1	1,2	1,8	2,5
Single unit bus (BUS)	7,6	2,1	2,6	2,6
Semi-trailer (WB-15)	6,1+9,4	0,9	0,6	2,5

* Distance between SU rear wheels and trailer front wheels

Table 7.3: Minimum turning radii

VEHICLE	MIN RADIUS (m)
Passenger car (P)	6,2
Single unit (SU)	12,8
Single unit + trailer (SU+T)	14,0
Single unit bus (BUS)	13,1
Semi-trailer (WB-15)	13,7

performance is not significantly affected by height above sea-level. Performance can therefore be represented by a single family of curves calculated on the basis of the 95 percentile mass/power ratio of 275 kg/kW, and as shown in Figure 7.1.

Pedestrians are also considered to be "design vehicles" in the sense that they are self-propelled, occupy space and have a measurable speed of movement. When bunched with others, while awaiting an opportunity to cross a street for example, the individual pedestrian occupies a circular space of about 700 mm in diameter. Pedestrians on the move will, however, prefer to be one metre or more apart. Walking speed on average is 1,5 m/s but in certain areas, such as in the vicinity of old-age homes, hospitals and schools, allowance should be made for lower speeds.

High volumes of pedestrians also invariably force lower walking speeds. Under these circumstances, design should be predicated on a walking speed of 1,0 m/s.

The design driver

Research (Pretorius 1976, Brafman Bahar 1983) has indicated that 95% of passenger car drivers have an eye height of 1,05 m or more, and 95% of bus or truck drivers an eye height of 1,8 m or more. These values have accordingly been adopted for use in these guidelines.

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 5,7-10,0 seconds for more complex multiple-choice situations. These extended times make provision for the case where more than one external circumstance must be evaluated, and the most appropriate response selected and initiated.

The road surface

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

Skid resistance has been the subject of research worldwide, and it has been locally established by Mkhacane (1992) and Lea (1996) that the derived values of brake force coefficient are appropriate to the southern African environment. Lea established that a limiting value of 0,4 is appropriate to gravel surfaces for all speeds. This suggests that, for design purposes, a value not greater than 0,2 should be adopted for these roads. With regard to surfaced roads, there is a considerable range of values. At 50 km/h the skid resistance of a worn tyre on a smooth surface is half that of a new tyre on a rough surface, and at 100 km/h it is five times lower. Skid resistance also depends on speed, and reduces as speed increases.

The speed used in the calculation of guideline values is the operating speed, generally 80-85% of design speed.

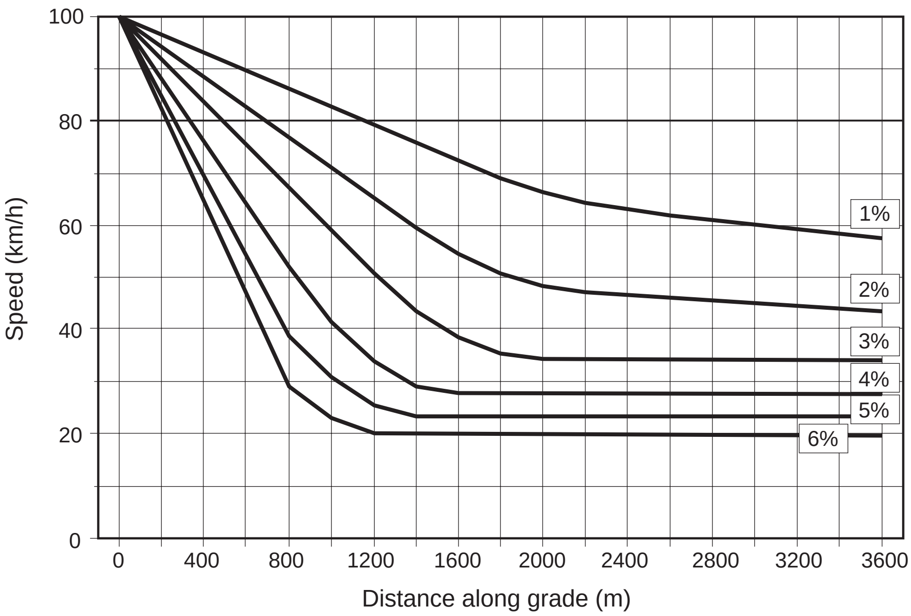


Figure 7.1: Truck speed on grades

Brake-force coefficients are given in Table 7.4. 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”.

Table 7.4: Brake force coefficients

SPEED (km/h)	COEFFICIENTS
20	0,47
40	0,37
60	0,32
80	0,30
100	0,29
120	0,28

THE ELEMENTS OF DESIGN

Design speed

Traffic speeds are measured and quoted in kilometres per hour. The Highway Capacity Manual (Transportation Research Board 1994) lists definitions of ten different speeds, such as spot speed, time mean speed, space mean speed, overall travel speed, running speed, etc. In this document, reference is principally to design speed and operating speed.

The design speed is a speed selected for the purposes of the design and correlation of those features of a road (such as horizontal curvature, vertical curvature, sight distance and superelevation) upon which the safe operation of vehicles depends. The design speed should thus be regarded more in the nature of a grouping of various design standards rather than as a speed per se.

The operating speed is the highest running speed at which a driver can travel on a given road under favourable weather and prevailing traffic conditions without, at any time, exceeding the design speed. Implicit in this definition of operating speed is the idea that the design speed is also the maximum safe speed that can be maintained on a given section of road *when traffic conditions are so favourable that the design features of the road govern the driver’s selection of speed.*

Sight should not be lost of the fact that a degree of arbitrariness attaches to the concept of maximum safe speed. The absolute maximum speed at which an individual driver is safe depends as much on the driver’s skill and reaction time, the quality and condition of the vehicle and its tyres, the weather conditions and the time of day (insofar as this affects visibility) as on the design features of the road.

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

Ceiling speed

In the urban situation, the need to vary the design speed because of physically constraining features is not likely to arise with any frequency. However, situations in which it is desirable to reduce operating speeds are common. Cases in point are areas where localised high concentrations of pedestrian traffic prevail. Examples include in the vicinity of schools (with particular reference to primary and nursery schools), old-age homes, modal transfer points and hospitals. Activity streets, where mixed usage may prevail, may require low operating speeds over substantial distances.

It would be extremely unwise to reduce the design speed in these areas, since a reduction in the design speed carries with it a reduction of sight distance. With the greater number of potential hazards that need to be observed and responded to, the driver should be afforded as much sight distance, and hence reaction time, as possible. In such areas, the design speed should be increased rather than reduced. An increase in the design speed by a factor less than 1,2 is not likely to produce any significant difference in operating conditions as perceived by the driver.

Clearly, however, the higher design speed should not serve as an inducement to increase operating speed and the concept of traffic calming would have to be brought into play.

Table 7.5 offers design and ceiling speeds appropriate to various classes of roads and streets. It should be noted that, ideally, shopping precincts such as malls should be so designed that vehicular access to them is not necessary. Should this not be possible to achieve in practice, access should be permitted only outside normal business hours. Parking areas serving shopping precincts should be designed to minimise vehicle-pedestrian conflicts.

Design hour

In the same way that a design speed is not a speed, the design hour is not an hour in the normal sense of the word. It is, in fact, a shorthand description of the conditions being designed for - specifically the projected traffic conditions.

These conditions include

- traffic volume, measured in vehicles per hour;
- traffic density, measured in vehicles per kilometre;

Table 7.5: Recommended design and ceiling speed

CLASS OF ROAD	DESIGN SPEED (km/h)	CEILING SPEED (km/h)
Vehicles only (freeways)	100 - 120	Not applicable
Vehicles only (other)	70 - 100	Not applicable
Mixed (higher order)	60 - 80	50 - 60
Mixed (middle order)	40 - 60	30 - 50
Mixed (lower order)	40 - 60	30 - 50
Pedestrian	30	20 - 30
Shopping precincts	30	<20

- traffic composition (i.e. the proportion of passenger cars, buses, rigid-chassis trucks and articulated vehicles comprising the traffic stream and usually expressed in percentage form); and
- directional split which, in an urban peak hour, readily achieves values of 80:20 or worse. Tidal flow implies that the 80:20 split in the evening peak would be in the opposite direction to that experienced in the morning peak *and both have to be designed for.*

The design hour is thus a combination of two distinctly different sets of circumstances, i.e. the morning and the afternoon peak in the case of commuter routes. Other routes may have different characteristics defining peak flows. Furthermore, the peak period may have a duration that is longer (or shorter) than 60 minutes and contain within itself a shorter period (typically 15 minutes) with very intense traffic flows.

A design life of 20 years is often assumed as a basis for design. This period may be altered subject to the planning of the authority concerned, and the evaluation of the economic consequences of departure from the suggested time span. For example, a road carrying low traffic volumes with few buses or trucks in the traffic stream may justify a shorter design life because of the savings accruing from the smaller number of axle-load repetitions in the shorter period. These savings arise from a reduction in the thickness of the design layers of the pavement and possibly even from a reduction in the quality of the materials required for road construction. A road carrying high volumes of bus or truck traffic in very hilly terrain may require a longer design life to achieve a reasonable return on the initial cost of construction.

Traffic volumes are usually expressed in terms of average daily traffic (ADT) measured in vehicles per day, with the ADT referring to an extended period, typically of the order of a year. Reference is made to Annual Average Daily Traffic (AADT) only if traffic counts are available for the period 1 January to 31 December. The ADT does not reflect monthly or daily fluctuations in traffic volume unless the month or day is explicitly specified.

The design hourly volume is frequently assumed to be the 30th highest hourly volume of the future year chosen for design, i.e. the hourly volume exceeded during only 29 hours of that year. The design hourly volume is expressed as a percentage of the ADT and typically varies from 12 to 18%. A value of 15% is thus normally assumed unless actual traffic counts suggest another percentage. Major urban links subject to commuter flows have a relatively low variation in flow when flows are ordered from highest to lowest across the number of hours in the year and, very often, the 100th highest hourly flow (at about 10 to 12% of the ADT) is an adequate basis for design.

Assessment of the total daily volume and hence hourly flows to be accommodated is a matter of some complexity. In the rural situation, naive modelling (i.e. applying a simple growth factor to present-day traffic counts) is adequate because changes in the nature and intensity of land use are slow if, in fact, they occur at all. In the urban situation, however, these changes are both significant and rapid. Furthermore, alternative routes are available. More sophisticated forms of modelling are necessary and application of naive modelling is not recommended.

The road network is really intended to support passenger trips or freight trips, with vehicle trips being almost incidental. Where would-be trip makers have a choice of mode, namely a convenient, safe, economical public transport service (which may be bus, rail bus or light rail) that really competes with the passenger car, traffic flows could be substantially lower than otherwise anticipated.

Density is a function of flow and speed, as illustrated by the units of measurement involved.

$$\text{Density (veh/km)} = \frac{\text{Flow (veh/h)}}{\text{Speed (km/h)}}$$

This function is not quite as direct as the equation implies because the basis of measurement differs. Density is measured across a considerable length of street at a single point in time, whereas flow is measured at a single point in space over an extended period of time. For this reason, the speed referred to in the above relationship is space mean speed as opposed to the more generally understood time mean speed.

It is the relationship between these measures which defines the Level of Service to be provided by the street being designed and, hence, the number of moving lanes to be provided in the cross-section.

SIGHT DISTANCE

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

Stopping sight distance (SSD)

Stopping distance involves the ability of the driver to bring the 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 has two components:

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

The stopping distance is expressed as :

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

where: s = total distance travelled (m)
v = speed (km/h)
f = brake force coefficient

Stopping sight distances are based on operating speeds. The brake-force coefficients quoted in Table 7.4 have been adopted for design, and the calculated stopping sight distances are given in Table 7.6.

Stopping sight distance is measured from an eye height of 1,05 m to an object height of 0,15 m in the case of the higher-order roads. This object height is used because an obstacle of a lower height would not normally represent a significant hazard. In residential areas, the object height can be increased to 0,6 m. This greater height provides a practical design with an adequate margin of safety for the protection of children, pets and other obstacles typically encountered on this class of street.

Object height is also taken into account because, if the sight distance were measured to the road surface, the length of the vertical curve required would be substantially increased.

This could result in streets being significantly above or below natural ground level. In the urban environment where there is a need for access to adjacent properties at relatively short intervals, this is not acceptable.

The gradient has a marked effect on the stopping-distance requirements. Figure 7.2 is an expansion of Table 7.6, demonstrating this effect.

Table 7.6: Stopping sight distance on level roads

DESIGN SPEED (km/h)	STOPPING SIGHT DISTANCE (m)
30	30
40	50
50	65
60	80
70	95
80	115
90	135
100	155
110	180
120	210

Stopping sight distance can also be affected by a visual obstruction such as a garden wall or shrubbery next to the lane on the inside of a horizontal curve, as shown in Figure 7.3.

Barrier sight distance (BSD)

Barrier sight distance is the limit below which overtaking is legally prohibited. Two opposing vehicles travelling in the same lane should be able to come to a standstill before impact. A logical basis for the determination of the barrier sight distance is therefore that it should equal twice the stopping distance, plus a further distance of 10 m to allow an additional safety margin. The values given in Table 7.7 reflect this approach.

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

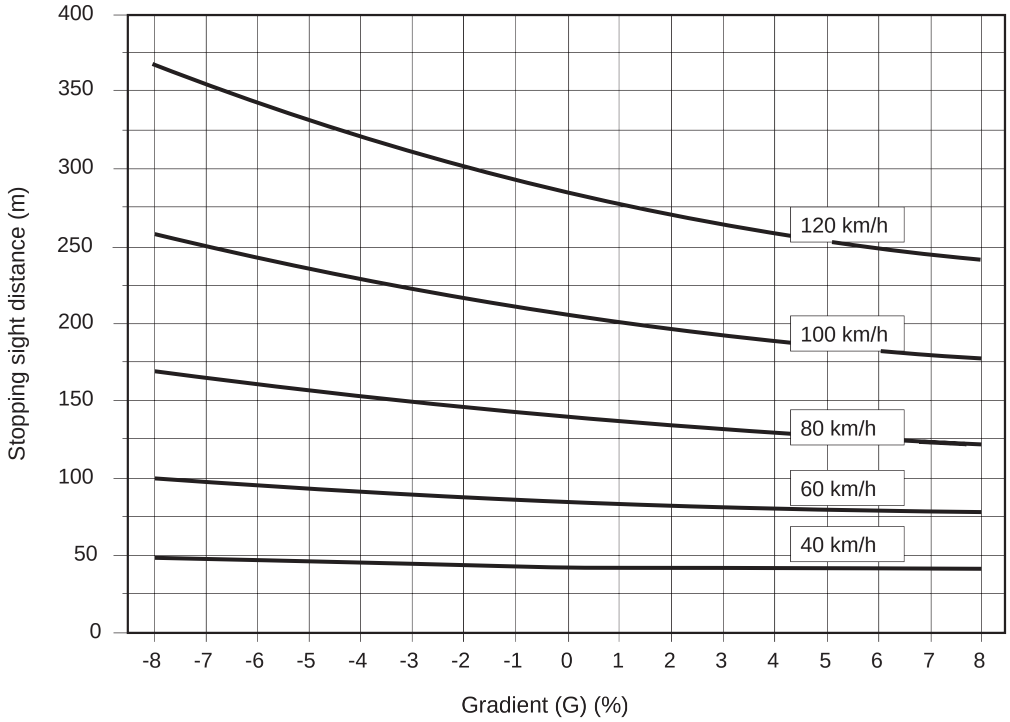


Figure 7.2: Stopping sight distance on gradients

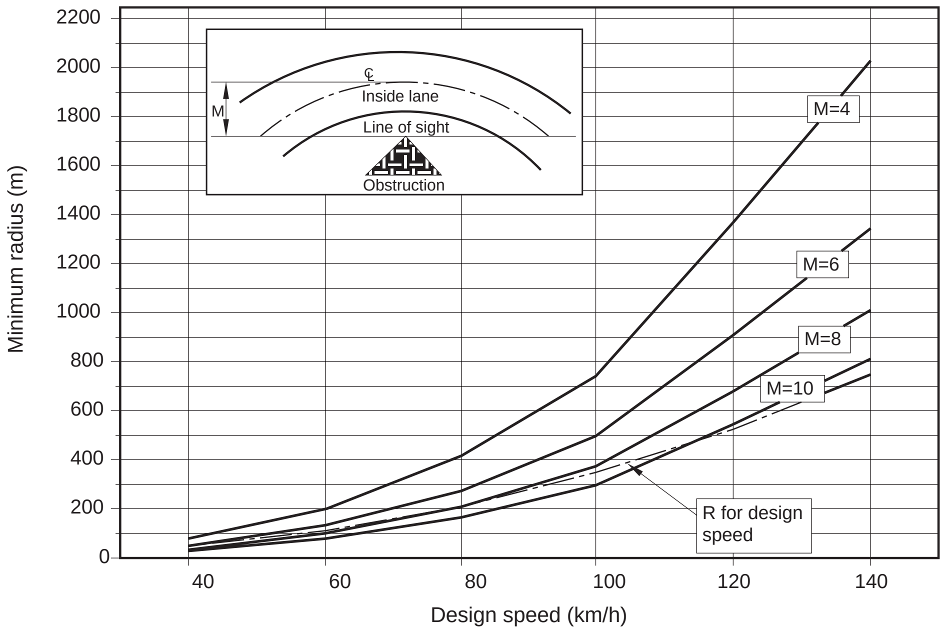


Figure 7.3: Minimum horizontal radius for stopping sight distance

Table 7.7: Barrier sight distance	
DESIGN SPEED(km/h)	BARRIER SIGHT DISTANCE(m)
40	110
60	170
80	240
100	320
120	430

Hidden dip alignments are commonly considered to be poor design practice. They typically mislead drivers into believing that there is more sight distance available than actually exists. In checking the alignment in terms of barrier sight distance, the designer should pay detailed attention to areas where this form of alignment occurs, to ensure that drivers are made aware of any inadequacies of design.

Because of the low speeds involved and the typically short lengths of lower-order mixed-usage streets, the passing operation is of little significance so that barrier markings are seldom, if ever, employed on these streets.

Decision sight distance (DSD)

The best visual cue to the driver is the roadway ahead. For this reason it is necessary in certain circumstances for the road surface itself to be visible to the driver for a given distance ahead. This is to allow sufficient time for the assimilation of a message and the safe initiation of any action required. An example is the marking that allocates specific lanes at an intersection to turning movements. Warning of this must be given sufficiently far in advance of the intersection to permit a lane change that does not detrimentally affect the operation of the intersection itself.

Decision sight distance, as given in Table 7.8, is related to the reaction time involved in a complex driving task. The reaction time selected for this purpose is 7,5 seconds, which is roughly the mean of values quoted in American practice. The calculated values in Table 7.8 are based on stopping sight distance to allow for the condition where the decision is to bring the vehicle to rest.

Table 7.8: Decision sight distance on level roads

DESIGN SPEED (km/h)	STOPPING SIGHT DISTANCE (m)
40	130
60	190
80	240
100	300
120	350

This has the effect of increasing the normal reaction time of 2,5 seconds by a further five seconds of travel at the design speed of the road. Decision sight distance is measured from an eye height of 1,05 m to the road surface, i.e. to an object height of 0 m.

Passing sight distance (PSD)

In the case of vehicles-only and higher-order mixed usage streets, passing sight distance is an important criterion indicative of the quality of service provided by the road. The initial design is required to provide stopping sight distance over the full length of the road, with passing sight distance being checked thereafter. A heavily trafficked road requires a higher proportion of passing sight distance than a lightly trafficked road to provide the same level of service. Insufficient passing sight distance over a vertical curve can be remedied, for example, either by lengthening the vertical curve to provide passing sight distance within the length of the curve itself, or by shortening the curve to extend the passing opportunities on either side. Horizontal curves can similarly be lengthened or shortened. A further possibility is the provision of a passing lane.

Passing sight distance can be calculated on one of two bases, being either the sight distance required for a successful overtaking manoeuvre or that required for an aborted manoeuvre. The former could be described as being a desirable standard and the latter as the minimum. Values quoted for the successful manoeuvre are taken from AASHTO (1994) and for the aborted manoeuvre from Harwood and Glennon (1989), who base these distances on the vehicles involved being a passenger car passing a bus or a truck.

Table 7.9 lists passing sight distances in respect of both successful and aborted manoeuvres.

Table 7.9: Passing sight distance on level roads

DESIGN SPEED (km/h)	PASSING SIGHT DISTANCE (m)	
	SUCCESSFUL MANOEUVRE	ABORTED MANOEUVRE
40	290	-
60	410	226
80	540	312
100	670	395
120	800	471

Passing sight distance in respect of a successful manoeuvre allows adequately (according to Harwood and Glennon 1989) for an aborted manoeuvre in the case of a bus or truck attempting to pass another.

As in the case of barrier sight distance, passing sight distance is not a consideration in the design of lower-order mixed-usage streets.

Intersection sight distance (ISD)

At a stop-controlled intersection, the driver of a stationary vehicle must be able to see enough of the through-road or street to be able to carry out one of three operations before an approaching vehicle reaches the intersection, even if this vehicle comes into view just as the stopped vehicle starts to move. These three operations are to:

- turn to the left in advance of a vehicle approaching from the right;
- turn to the right, crossing the path of a vehicle approaching from the right and in advance of a vehicle approaching from the left;
- to move across the major highway in advance of a vehicle approaching from the left.

In the first case, the assumption is that the turning vehicle will accelerate to 85% of the design speed of the through-road and a vehicle approaching on the through-road will decelerate from the design speed also to 85% of the design speed, leaving a two-second headway between them at the end of the manoeuvre.

According to AASHTO, the intersection sight distance required for the right turn is only about one metre less than that required for the left turn, given the same assumptions as made in the first case.

In the case of the vehicle crossing the through- road, the distance the crossing vehicle must travel is the sum of:

- the distance from the stop line to the edge of the through carriageway;
- the width of the road being crossed; and
- the length of the crossing vehicle.

This manoeuvre must be completed in the time it takes the approaching vehicle to reach the intersection, assuming that the approaching vehicle is travelling at the design speed of the through-road. For safety, the time available should also include allowance for the time it takes for the crossing driver to establish that it is safe to cross, engage gear and set his or her vehicle in motion: a period of about two seconds is normally used.

Intersection sight distances recommended in accordance with the principles outlined above are given in Figures 7.4 and 7.5. Before a lower value is adopted in a specific case, the implications of departing from the recommended values should be considered.

The line of sight is taken from a point on the centre line of the crossing road and 2,4 m back from the edge of the through-road, to a point on the centre line of the through-road, as shown in Figures 7.4 and 7.5. The setback is intended to allow for a pedestrian or cycle track crossing beyond the Stop line.

The object height is 1,3 m. The eye height is 1,05 m for a passenger car and 1,8 m for buses and all other design vehicles. There should not be any obstruction to the view in the sight triangle, which is defined as the area enclosed by the sight line and the centre lines of the intersecting roads.

Where an intersection is subject to yield control, the unobstructed sight triangle must be larger. If it is assumed that the vehicle approaching the intersection on the minor leg will be travelling at 30 km/h, a distance of 30 m would be required to stop the vehicle. If the driver is already preparing to stop, allowance for reaction time is no longer necessary and a distance of 10 m is required to bring the vehicle to a standstill. If the approach speed is 60 km/h, the required distance is 45 m.

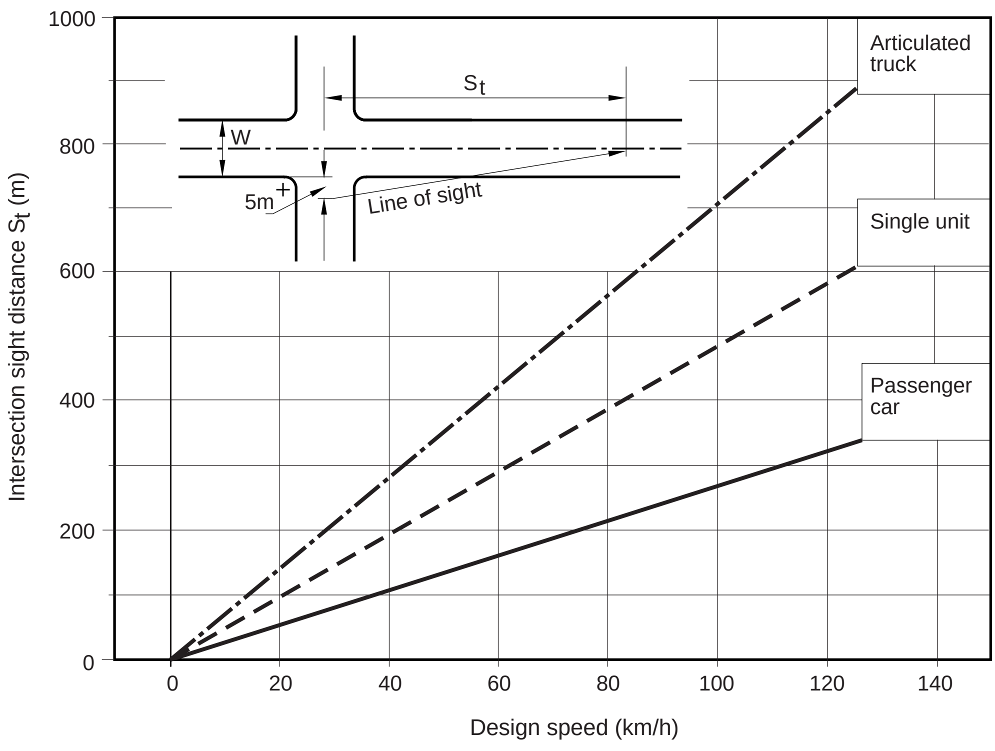


Figure 7.4: Intersection sight distance for turning manoeuvre

The sight triangles required for yield control based on an approach speed of 60 km/h are so large that the probability of their being found in an urban area is remote.

If the driver does not stop but turns to travel in the same direction as a vehicle approaching at the design speed of the through-road, the driver of the latter vehicle will be forced to slow down to match speeds at a safe following distance.

The intersection sight distance for this manoeuvre is shown in Figure 7.6

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

Intersections are, typically, the points at which pedestrians would want to cross the through-road or

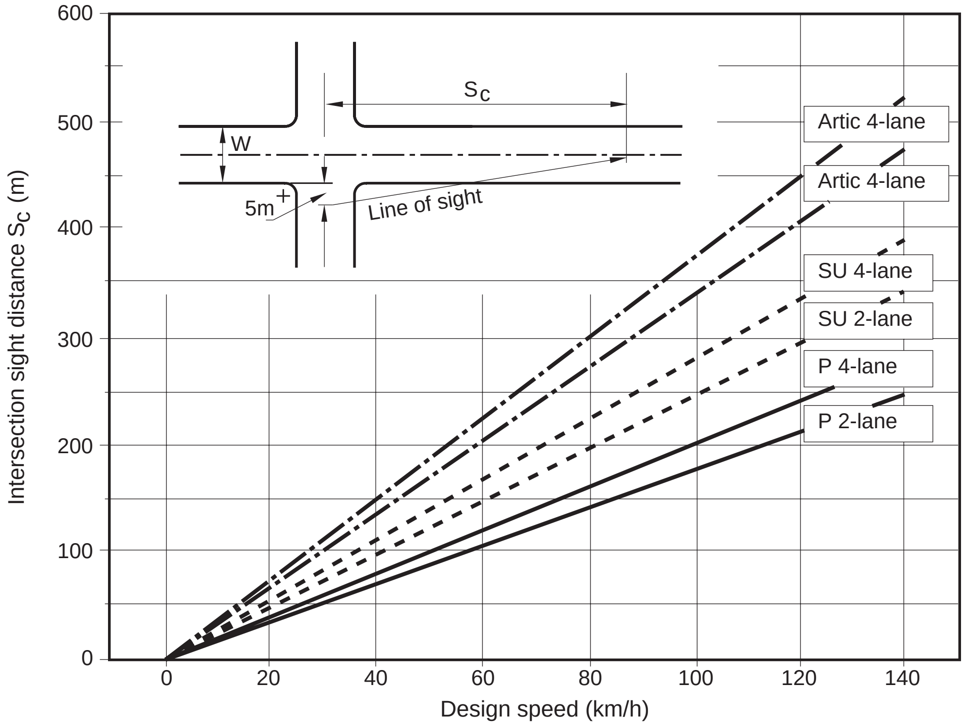


Figure 7.5: Intersection sight distance for crossing manoeuvre

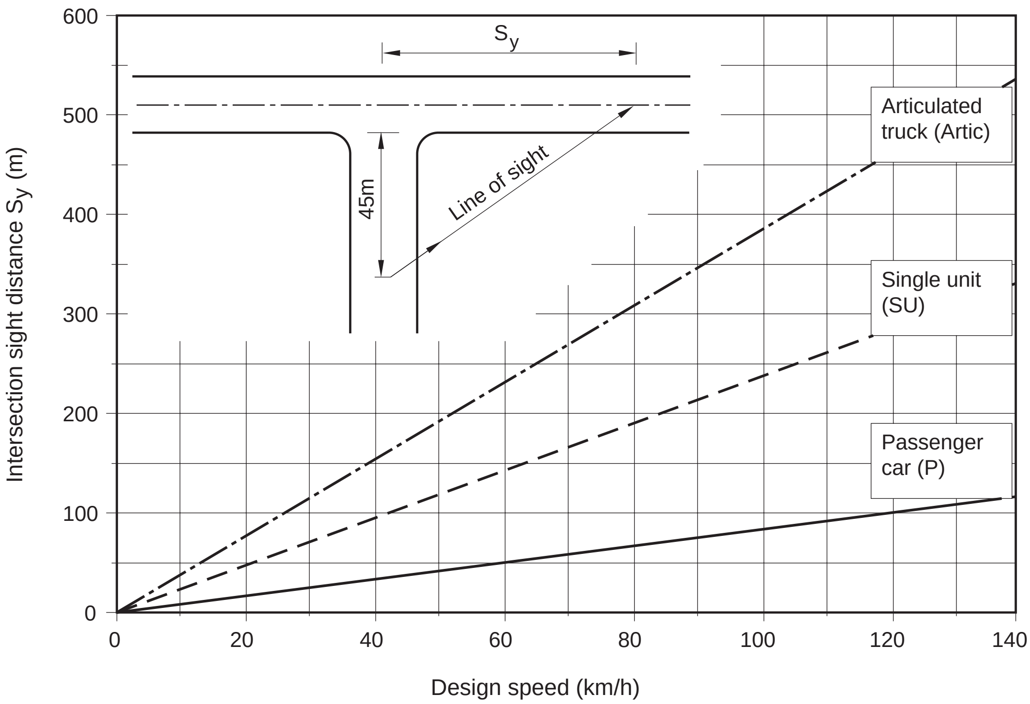


Figure 7.6: Intersection sight distance for yield condition

street. Pedestrians must therefore be provided with adequate sight distance to ensure that they can cross the through-street in safety. This case is precisely analogous to that of the vehicle at the intersection, because the principle involved is that the sight distance provided is directed towards what the pedestrian must be able to see rather than the sight distance available to drivers of vehicles on the through-road.

Pedestrian sight distance is measured from an eye height of 1,0 m to an object height of 1,3 m. It is assumed that the pedestrian is located on the left side of the intersecting street with the oncoming vehicle approaching also from the left. This represents the longest crossing distance before a situation which is at all safe is achieved, because the further assumption is that the pedestrian will not be required to pause on the centreline of the through-road. The distances offered in Table 7.10 would be adequate for crossing a two-lane road.

Table 7.10: Pedestrian sight distance

DESIGN SPEED (km/h)	SIGHT DISTANCE(m)
30	45
40	55
50	70
60	85
70	100

If adequate sight distance is not available, it may be necessary to provide a signalised cross-walk, thus forcing through vehicles to stop. Furthermore, if an adequate gap in the through-traffic does not present itself at intervals not exceeding one minute, a signalised pedestrian crossing should also be considered.

HORIZONTAL ALIGNMENT

The horizontal alignment of a road or street is the combination of curves and straights (or tangents) presented on a plan view. Curves are usually circular, although spirals and other higher-order polynomials can be used under highly specific circumstances, which are seldom found in residential environments.

Determination of the horizontal alignment of an urban street is a planning rather than a detailed design function, and is highly iterative in nature. Iteration is not only between the three dimensions of design, e.g. where restraints in the vertical dimension may force a shift in horizontal alignment, but also involves continuous revisiting of the intentions originally formulated with regard to settlement making.

Design of the horizontal alignment must also give effect to the proposed function of the road or street. For example, the horizontal alignment of a freeway is typified by long tangents and gentle curves, whereas a residential street should be designed to discourage operating speeds higher than 40 to 50 km/h.

General principles to be observed in the determination of the horizontal alignment of a road or street are the following:

- No vehicle can instantaneously change from traversing a curve in one direction to traversing one in the reverse direction. Short lengths of tangent should thus be used between reverse curves.

- Broken-back curves (where two curves in the same direction are separated by a short tangent) should not be used as they are contrary to drivers' expectations. In the residential environment, this is difficult to avoid as cadastral boundaries are straight lines. Fitting smooth curves within a reserve comprising a series of chords of a circle is not always possible.

- Large- and small-radius curves should not be mixed. Successive curves to the left and the right should generally have similar radii and the 1:1,5 rule is a useful guide in their selection.

- In residential areas, the deviation angle of short-radius curves should not exceed 90° as, at higher values of deviation, encroachment by large vehicles on opposing lanes becomes pronounced and, furthermore, the splay that has to be provided to permit adequate sight distance becomes excessive.

- For small-deflection angles, curves should be sufficiently long to avoid the impression of a kink.

- Alignment should be sensitive to the topography to minimise the need for cuts and fills and the restriction that these place on access to erven from the street. Streets at right angles to the contours can create problems in terms of construction, maintenance, drainage, scour (in the case of gravelled surfaces) and also constitute a traffic hazard. During heavy rainstorms, water flowing down a steep street can flow across the intersecting street.

In addition, the various utilities, such as sewerage, power and water reticulation, are typically located within the road reserve. The planning of the road network must therefore also take cognisance of the limitations to which these services are subject. For example, a street located in such a fashion that its vertical alignment tends to be undulating would present significant difficulties in the location of sewer runs.

Tangents

As horizontal curves are circular, the straights connecting them are usually referred to as tangents. While the selection of radius of horizontal curvature dictates the operating speed selected by the driver, long tangents can cause speeds to increase to unacceptable levels, followed by deceleration as the next curve is encountered. It has been found that limiting the length of tangents (in metres) to about ten times the design speed (in km/h) will cause speeds to stay fairly constant. A design speed of 40 km/h would thus suggest that tangents should not be more than about 400 m in length.

In the situation of an urban grid of streets, a 400-metre-long tangent bounded at both ends by T-intersections would also tend to limit speeds to of the order of 40 km/h.

Curvature and superelevation

Acceptable rates of superelevation are offered in Table 7.11.

Table 7.11: Maximum superelevation for various classes of road

CLASS OF ROAD OR STREET	MAXIMUM SUPERELEVATION (%)
Vehicle-only (freeway)	10
Vehicle-only (other)	6 - 8
Mixed-usage (higher-order)	4 - 6
Mixed-usage (middle-order)	2 - 4
Mixed-usage (lower-order)	2 - 4

In Table 7.12, the minimum radii of horizontal curves for various design speeds and maximum rates of superelevation are calculated from the relationship

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

where: R = radius (m)
v = speed (km/h)
e = superelevation rate(m/m)
f = side friction factor

Unlike the rural situation where a tight radius curve can be matched by a high value of superelevation, the large variations in vehicle speeds encountered in the urban environment cause high values of superelevation to be inappropriate. Furthermore, there is a distinct likelihood that there would not be sufficient distance available to accommodate the development of superelevation. Property access in the immediate vicinity of the curve probably would not allow a cross-section where one road edge is a metre or more above the other.

In Table 7.12, all values have been rounded up to the nearest five metres. A camber of 2 to 3% suggests that rates of superelevation of -0,02 to +0,02 are usually normal camber situations. There is no known application for a 0% super-elevation and it should be avoided as, in the absence of a longitudinal gradient, it will cause drainage problems and ponding on the road surface.

Superelevation runoff

Streets normally have a camber with the high point on their centreline and a fall, typically of the order of 2 to 3% as suggested above, to either edge. Superelevation is developed or run off by rotating the outer lane around the centreline until a crossfall across the full width of the street, equal to the original camber, is achieved. From this point, both lanes are further rotated around the centreline until the full extent of superelevation has been achieved.

This further rotation need not necessarily be about the centreline. Special circumstances may demand a different point of rotation. A constraint on the level of one or other of the road edges may require that the constrained edge becomes the axis of rotation. The need to secure an adequate, but not too steep, fall to a drop inlet on the inside of a curve may require that the axis of rotation be shifted to a point slightly

Table 7.12: Minimum radii for horizontal curves (m)

DESIGN SPEED (km/h)	SIDE FRICTION FACTOR (f)	MINIMUM RADII FOR MAXIMUM RATES OF SUPERELEVATION (e) OF:					
		-0,02	0	+0,02	0,04	+0,06	+0,08
30	0,19	45	40	35	30	30	30
40	0,18	80	70	65	60	55	50
50	0,17	135	115	105	95	85	80
60	0,16	205	180	160	145	130	120
70	0,15	300	260	230	205	185	170
80	0,14	420	360	315	280	255	230
100	0,13	-	-	525	465	415	375
120	0,11	-	-	875	760	670	600

removed from the inner edge.

Rotation over too short a distance will create the impression of an unsightly kink in the road surface and, if the distance is too long, drainage problems are likely to occur in the area where the camber is less than about 0,5%. The rate of rotation is measured by the relative slope between the roadway edge and the axis of rotation. Relative slopes that have been found in practice to give acceptable lengths of runoff are quoted in Table 7.13. Where space does not permit the use of these rates, minimum lengths for superelevation runoff for two-lane roads may have to be adopted. These are also quoted in Table 7.13. These lengths are based on relative slopes that are generally 50% higher than those recommended for normal use.

Table 7.13: Rates and minimum lengths of superelevation runoff

DESIGN SPEED	RELATIVE SLOPE (%)	MINIMUM LENGTH (m)
40	0,7	35
60	0,6	40
80	0,5	50
100	0,4	60
120	0,4	70

Where a circular arc is preceded by a transition curve, the full superelevation is developed across the length of the transition. Transition curves, however, are only used on the tightest radius curves applied to roads with high design speeds. In all other cases, the superelevation runoff must be distributed between the tangent and the curve because full superelevation at the end of the tangent is as undesirable as no superelevation at the start of the curve. Drivers tend to follow a transition path in entering a curve and this path typically has two-thirds of its length on the tangent with the remaining third being on the curve itself. Superelevation runoff is similarly distributed to match the actual path of the vehicle.

VERTICAL ALIGNMENT

Vertical alignment is the combination of parabolic vertical curves and straight sections joining them. Straight sections are referred to as grades, and the value of their slope is the gradient, usually expressed in percentage form, e.g. a 5% grade climbs through 5 metres over a horizontal distance of 100 metres.

With the whole-life economy of the road in mind, vertical alignment should always be designed to as high a standard as is consistent with the topography. Passenger car speeds are dictated by the standard of horizontal alignment rather than by the vertical alignment, whereas the speeds of buses and other heavy vehicles are constrained more by the vertical alignment. The design speed applied to the vertical alignment should therefore match that applied to the horizontal alignment and it could be argued that a higher vertical design speed is preferable.

As in the case of the horizontal alignment, the vertical alignment should be designed to be aesthetically pleasing. In this regard due recognition should also be given to the interrelationship between horizontal and vertical curvature. A vertical curve that coincides with a horizontal curve should, if possible, be contained within the horizontal curve and, ideally, have approximately the same length.

Where a vertical curve falls within a horizontal curve, the superelevation generated by the horizontal curvature improves the availability of sight distance beyond that suggested by the value of vertical curvature. This enables the edge profiles to have a curvature sharper than the minima suggested in Table 7.14. The proviso, however, is that the driver's line of sight is contained within the width of the roadway. When the line of sight goes beyond the roadway edge, the effect on sight distance of lateral obstructions such as boundary walls or high vegetation must be checked.

A smooth grade line with gradual changes appropriate to the class of road and the character of the topography is preferable to an alignment with numerous short lengths of grade and vertical curves. The “roller coaster” or “hidden dip” type of profile should be avoided. This profile is particularly misleading in terms of availability of sight distance and, where it cannot be avoided, sight distance greater than suggested in Table 7.6 may be required in terms of accident experience. For aesthetic reasons, a broken-back alignment is not desirable in sags where a full view of the profile is possible. On crests the broken-back curve adversely affects passing opportunity.

Curvature

The horizontal circular curve provides a constant rate of change of bearing. Analogous to this is the vertical parabola which provides a constant rate of change of gradient. Academic niceties apart, there is little to choose between the application of the parabola or the circular curve, the differences between them being virtually unplottable and, in any event, within the levels of accuracy to which the pavement typically is constructed.

From the general form of a parabolic function

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

it follows that the rate of change of grade, d^2y/dx^2, equals 2a. The reciprocal of 2a, K, is thus the distance required to effect a unit change of grade. Vertical curves are specified in terms of this factor, K, and their horizontal length as shown in the relationship

$$L = A.K$$

Minimum rates of curvature

The minimum rate of curvature is determined by sight distance as well as by considerations of comfort of operation and aesthetics. The sight distance most frequently employed is the stopping sight distance which, as stated earlier, is measured from an eye height of 1,05 m to an object height of 0,15 m although, in the case of residential streets, an object height of 0,6 m could be used.

In the case of sag curves, the sight distance is replaced by a headlight illumination distance of the same magnitude, assuming a headlight height of 0,6 m and a divergence angle of 1° above the longitudinal axis of the headlights. Where adequate street lighting is available, the headlight criterion does not apply and comfort is the only criterion that limits values.

Special circumstances may dictate the use of decision sight distance or even passing sight distance. Where a sight distance other than that for stopping has to be employed, the relationship offered below can be used to calculate the required curve length and, thereafter, the K-value of vertical curvature.

Where the sight distance, S, is less than the curve length, L,

$$L = \frac{AS^2}{100\,(\sqrt{2h_1} + \sqrt{2h_2})}$$

and, where S is greater than L,

$$L = 2S - \frac{200\,(\sqrt{h_1} + \sqrt{h_2})^2}{A}$$

where: L = length of vertical curve (m)
S = sight distance (m)
A = algebraic difference in grades (%)
h_1 = height of eye above road surface (m)
h_2 = height of object above road surface (m)

Values of K, based on stopping sight distance in the case of crest curves, and on headlight illumination distance in the case of sag curves, are given in Table 7.14.

Minimum lengths of vertical curves

Where the algebraic difference between successive grades is small, the intervening minimum vertical curve becomes very short, and, particularly where the adjacent tangents are long, the impression of a kink in the grade line is created. Where the difference in grade is less than 0,5%, the vertical curve is often omitted. In Table 7.15, a minimum length of curve for algebraic differences in grade greater than 0,5% is suggested for purely aesthetic reasons.

Where a crest curve and a succeeding sag curve have a terminal point in common, the visual effect created is that the road has suddenly dropped away. In the reverse case, the illusion of a hump is created. Either effect is removed by inserting a

Table 7.14: Minimum values of K for vertical curves

DESIGN SPEED (km/h)	CREST CURVES FOR OBJECT OF HEIGHT		SAG CURVES	
	0,15 m	0,60 m	Without street lighting	With street lighting
40	6	2	8	4
50	11	6	12	6
60	16	10	16	8
70	23	Not applicable	20	Not applicable
80	33		25	
90	46		31	
100	60		36	
110	81		43	
120	110		52	

short length of straight grade between the two curves and, typically, 60 m to 100 m is adequate for this purpose.

Table 7.15: Minimum length of vertical curves

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

Gradients

Maximum gradients on higher order roads

Bus and truck speeds are markedly affected by gradient. Bus routes should be designed with gradients which will not reduce the speed of these vehicles enough to cause intolerable conditions for following drivers. Glennon (1970) found that the frequency of accidents increases sharply when the speeds of heavy vehicles are reduced by more than 15 km/h.

For southern African conditions a speed reduction of 20 km/h is generally accepted as representing intolerable conditions. If gradients on which bus or 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. Wolhuter (1990) established that, on flat grades, 50 percentile bus and truck speeds are about 17 km/h lower than the equivalent passenger car speeds, so that a speed reduction of 20 km/h actually represents a total speed differential of about 37 km/h.

Maximum gradients for different design speeds and types of topography are suggested in Table 7.16. It is stressed that these are guidelines only. Optimisation of the design of a specific road, taking the whole-life economy of the road into account, may suggest some other maximum gradient.

The three terrain types described are defined by the differences between passenger-car and bus or truck speeds prevailing in them. On flat terrain, the differences between the speeds of cars and buses remain relatively constant at about 17 km/h, whereas hilly terrain causes substantial speed differentials. In mountainous terrain, buses and trucks are reduced to crawl speeds for substantial distances.

Maximum gradients on residential streets

On local streets, maximum gradient has a significant effect on the cost of township development. Where possible, road alignment should be designed to minimise the extent and cost of earthworks and to avoid problems with access and house design. It therefore has to be accepted that short sections of steep gradients may be necessary in some settlement developments.

Where a residential street is also a bus route, the gradients recommended in Table 7.16 should not be exceeded. Where this is not possible, a maximum gradient of 14% may have to be considered.

On higher-order mixed-usage streets, the recommended maximum gradient is 10% but, on sections not longer than 70 m, the gradient can be increased to 12,5%. On purely residential streets, the maximum gradient could be 12% and on sections not longer than 50 m the gradient could be increased to 16%.

Notwithstanding the values given, the following points should be taken into consideration:

- Gradients should be selected in consultation with the stormwater design engineer.
- Steep gradients on short access loops and culs-de-sac could result in properties being inundated and surface runoff washing across

Table 7.16: Maximum gradients on major roads (%)

DESIGN SPEED (km/h)	TOPOGRAPHY		
	FLAT	ROLLING	MOUNTAINOUS
40	7	8	9
60	6	7	8
80	5	6	7
100	4	5	6
120	3	4	5

intersecting streets.

- Multiple-use surfaces which serve both vehicular access and recreational purposes, including playing space for children, should be relatively flat and not provided with kerbs (they are, in fact, shared surfaces).

- Where cycling is an important mode of travel, it will be necessary to consider the effects of gradient on cycling in deciding on the road alignment.

- It is difficult to construct streets on gradients steeper than about 12% by conventional means. 12/14 ton rollers cannot climb gradients this steep. They also tend to damage the base course while attempting to stop after a downhill pass. Steeper grades should thus be constructed of concrete, brick or interlocking road stones. The last-mentioned surface is not recommended where speeds in excess of about 60 km/h are anticipated, as the partial vacuum created behind a passing tyre tends to suck out the sand from between adjacent stones and thus destroy the integrity of the surface.

- Gravel surfaces are subject to scour at water flow speeds of the order of 0,6 to 1,0 m/s. Under conditions of overland flow, this speed is achieved at slopes of the order of 7 to 8%. The slope in question is the resultant of the vectors of longitudinal slope and crossfall.

Minimum gradients

If the cross-section of the road does not include kerbing, the gradient could be 0% because the camber is continued across the adjacent shoulder, thus allowing for adequate drainage of the road surface. The verge will have to accommodate the drainage both of the road reserve and of the surrounding properties. The decision to accept a zero gradient would thus have to be informed by the stormwater drainage design. Zero gradient is not recommended as a general rule and the preferred minimum is 0,5%.

Kerbed streets should have a minimum gradient of not less than 0,5%. If the street gradient has to be less than this, it would be necessary to grade the kerbs and channels separately and to reduce the spacing between drop inlets to ensure that the height difference between the edge of the travelled way and channel is not too pronounced.

Climbing lanes

Application of climbing lanes

Climbing lanes are auxiliary lanes added outside the through-lanes. They have the effect of reducing congestion in the through-lanes by removing slower-moving vehicles from the traffic stream. As such, they are used to match the Level of Service on the rising grade to that prevailing on the level sections of the route. In the urban situation, climbing lanes may be used on vehicles-only and higher-order mixed-usage streets. They have no application on local residential streets.

Warrants for climbing lanes

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

A bus/truck speed profile should be prepared for each direction of flow. It would then be possible to identify those sections of the road where speed reductions of 20 km/h or more may warrant the provision of climbing lanes.

The traffic volume warrant is given in Table 7.17. It should be noted that the word “trucks” includes buses, rigid-chassis trucks and articulated vehicles.

A further warrant is based on matching Levels of Service (LOS) along the route. Alternatively, a form of partial economic analysis developed by Wolhuter (1990) could be used. This software - ANDOG (**AN**alysis of **D**elay **O**n **G**rades) - is available from CSIR-Transportek. It compares the cost of construction of the climbing lane to the costs of the delay incurred by not providing it.

Location of terminals

A slow-moving vehicle should be completely clear of the through-lane by the time its speed has dropped by 20 km/h, and remain clear of the through-lane until it has accelerated again to a speed which is 20 km/h less than its normal speed. The recommended taper length is 100 m so that the start taper begins 100 m in advance of the point where the full climbing lane width is required, and the end taper ends 100 m beyond the end of the climbing lane.

If there is a barrier line, owing to restricted sight distance, at the point where the speed reduction warrant falls away, the full lane should be

Table 7.17: Traffic volume warrants for climbing lanes

GRADIENT (%)	TRAFFIC VOLUME IN DESIGN HOUR (veh/h)	
	5% trucks in stream	10% trucks in stream
4	632	486
6	468	316
8	383	257
10	324	198

extended to where the marking ends, with the taper ending 100 m beyond this point.

Climbing lane width

The climbing lane should preferably have the same width as the adjacent through-lanes. On major routes, through-lanes may have widths of 3,7 m, 3,4 m or 3,1 m. It is unlikely that climbing lanes will be provided on roads where the traffic volumes are so low that a lane width of 3,1 m is adequate. Climbing lanes therefore tend to be either 3,7 m or 3,4 m wide. Even if the through-lanes are 3,7 m wide, a climbing lane 3,4 m or perhaps even 3,1 m wide may, however, be considered on the grounds of low lane occupancy and speed or some other constraining topographic circumstance. Climbing lanes on bus routes should, however, have a width of 3,7 m.

CROSS-SECTION DESIGN

The cross-section of a road provides accommodation for moving and parked vehicles, drainage, public utilities, non-motorised vehicles and pedestrians. It is also required to serve more than just movement-related activities.

Residential streets, for example, offer a neutral territory on which neighbours can meet informally. They can also serve as playgrounds for children in developments where plot sizes are too small for this purpose.

Abutting trading or light industrial activities in activity corridors may require sidewalks wider than those required purely for moving pedestrians. Pedestrians also “park”, in the sense of browsing through goods on offer (either in shop windows or by roadside vendors) or relaxing in a sidewalk café. In short, the road reserve is required to address a wide spectrum of activities. For this reason it was suggested previously that reference should be to “hard open space” with only a portion of this comprising the road reserve as previously understood.

Movement, as an activity served by the cross- section, comprises a spectrum of needs. One end of the spectrum of the movement function relates to pure mobility, as typified by the freeway and urban arterial. Vehicle movement is the sole concern and pedestrians are totally excluded from these roads. The other end of the spectrum is concerned with accessibility and the needs of the pedestrian. Vehicular movement may be necessary on these roads but it is tolerated rather than encouraged and is subject to significant restrictions. Between these two extremes, mixed usage is found with vehicular and non-vehicular activities sharing the available space. If these uses have to compete for their share of space, it can reasonably be stated that the design has failed to meet its objective.

The flexibility of the road reserve in accommodating such widely disparate needs derives from the disaggregated nature of the cross-section, as illustrated in Figure 7.7.

The cross-section may comprise all or some of the following components:

- Lanes
  - Basic
  - High Occupancy Vehicle (HOV) lanes
  - Auxiliary (turning or climbing)
  - Parking
  - Cycle

- Medians
  - Shoulders
  - Central island

- Shoulders
  - Verges
  - Sidewalks.

Lanes

Basic or through-lanes

Undivided roads may have either one lane in each direction (two-lane two-way roads) or more than one lane in each direction (multilane roads). Dual carriageway roads have two or more lanes in each direction separated by a median. Customarily, there is symmetry of through-lanes, and asymmetry

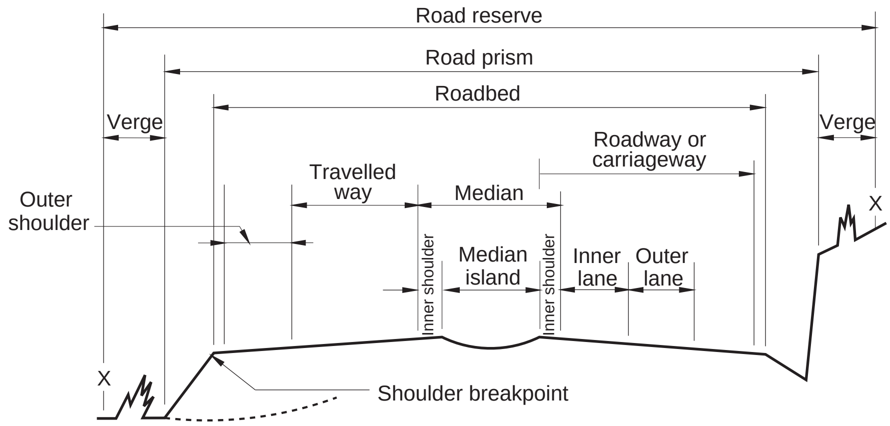


Figure 7.7: Elements of the cross-section

on a particular section of road should arise only from the addition of an auxiliary lane that is clearly allocated to one direction of travel.

The selection of lane width is based on traffic volume and vehicle type and speed. Higher volumes and speeds require wider lanes, and the greatest lane width recommended is 3,7 m. Where traffic volumes are such that a multilane cross-section or a divided cross-section is required, 3,7 m is a logical lane width to adopt.

No operational or safety benefit accrues from lane widths wider than 3,7 m, although some urban authorities allow lane widths as broad as 5,5 m. In peak hours, these wider lanes tend to carry two lanes of moving passenger cars each. They also ease the process of passenger cars overtaking buses without encroaching significantly on the opposing lane. Finally, they enable informal parking in the absence of demarcated parking bays. As such, 5,5 m lanes tend to be used only in higher-order mixed-usage streets.

The narrowest lane width recommended is 3,1 m, which gives a clear space of 0,25 m on either side of a vehicle that is 2,6 m wide i.e. a bus. This width would normally be employed only where speeds or traffic volumes are expected to be low and buses infrequent, e.g. on residential streets.

If the route is not intended ever to accommodate buses, the lane width could be reduced to as little as 2,7 m. Intermediate conditions of volume and speed can be adequately catered for by a lane width of 3,4 m.

Streets where pedestrian activities are expected to predominate may have only one lane, with provision for passing made at intervals. In this case, the lane width should not be less than 3,1 m.

Passing bays should be provided at not more than 50 m spacings. It is important that passing bays should be intervisible. If this is not achieved, motorists may enter a single lane section only to find that it is already occupied thus forcing one or other of the vehicles involved to reverse to the previous passing bay.

High occupancy vehicle (HOV) lanes

HOV lanes normally extend over considerable distances and could, therefore, be included in the category of basic lanes. These lanes are normally applied only to higher-order mixed-usage streets and are intended to serve all HOVs and not only buses. Vehicles that could be allowed to use HOV lanes thus include

- buses;
- minibus taxis; and
- car pool vehicles.

A policy decision would have to be provided by the local authority concerned in respect of the level of vehicle occupancy that would allow a vehicle to enter an HOV lane. An operational problem that immediately arises in the application of HOV lanes is their policing, to ensure that only vehicles legitimately described as HOVs use them.

As buses have an overall width of 2,6 m, the smaller basic lane widths would not be appropriate. As pointed out, a 3,1 m lane would allow only 0,25 m between the outside of the bus and the lane edge with a distinct possibility that a moving bus would not always be precisely located in the centre of the lane. At speeds higher than those encountered in residential areas, encroachment on other lanes could be expected. To avoid encroachment, a lane width of 3,7 m is the minimum that should be accepted for a HOV lane.

It is not possible to lay down hard-and-fast warrants for the provision of bus lanes. From an operational point of view, relating purely to the movement of people, it follows that the capacity of the street being analysed should be greater with the added HOV lane than without it. Where an existing lane is converted to an HOV lane, i.e. when it is no longer available for use by other vehicles, the provision of the HOV lane could lead to a decline in throughput of passengers.

The estimation of transit capacity is more complex and less precise than estimates of highway capacity as it deals with the movement of both people and vehicles whereas highway capacity restricts itself to vehicular movement. Furthermore, the variables to address in such estimates include, in addition to the normal factors applying to highway capacity

- the size of the transit vehicles, in terms of allowable passenger loadings;
- the frequency of operation of the service; and
- the interaction between passenger traffic concentrations and vehicle flow.

Reference should be made to TRB Special Report 209: Highway Capacity Manual with regard to the analysis of mass transit facilities.

The layout of bus stops is discussed under the heading "Verges".

Auxiliary lanes

Auxiliary lanes are lanes added to the normal cross-section to address a specific purpose and are normally applied only to vehicle-only or higher-order mixed-usage streets.

Typically, auxiliary lanes are added at intersections to support left and right turns so that these manoeuvres can take place at relatively low speeds without impeding the movement of the through-traffic. If a road has signalised intersections, it may be necessary to add auxiliary lanes to match the intersection capacity to that of the approach legs. These lanes are discussed in more detail below under the heading "Intersections".

Auxiliary lanes can also be provided at intersections to serve through-traffic. The intention is to match the capacity of the intersection to its upstream and downstream links. The need for such lanes and the storage length upstream and merging length downstream that they have to accommodate are a matter for analysis as described by the Highway Capacity Manual.

Climbing lanes, as auxiliary through-lanes, are discussed above under the heading "Vertical Alignment".

Parking lanes

Parking lanes are normally 2,5 m wide with a minimum width of 2,1 m, and are usually embayed. In this configuration, the parking lane is actually located in the verge area. Individual parking bays are typically 6,0 m long with each pair of parking bays being provided with an additional clear space of 1,5 m between them to allow for manoeuvring into or out of the bays. In areas where very high tidal flows are expected, it is useful to be able to use the parking lanes as moving lanes during periods of peak flow. Under these circumstances, the parking lane should have a minimum width of 3,1 m.

Cycle lanes

Ideally, cycle lanes should be located in the verge area, as the speed differential between bicycles and pedestrians is likely to be less than that between bicycles and motorised vehicles. Where this is not possible and either there is significant cycle traffic or it is desired to encourage bicycles as a mode of travel, a cycle lane can be added outside those intended for motorised vehicles.

Such lanes should be of the order of 1,5 m wide and clearly demarcated as cycle lanes. If these lanes are wider than 2,0 m, passenger cars are likely to use them, possibly even for overtaking on the left, which is a manoeuvre to be actively discouraged.

Shoulders

The shoulder is defined as the usable area alongside the travelled way.

Shoulders are applied only to roads where pedestrian traffic is not specifically catered for. Their width does not, therefore, make provision for the mounting of guardrails, or for edge drains or shoulder rounding. The shoulder breakpoint is some distance beyond the edge of the usable shoulder, usually about 0,5 to 1,0 m.

A stopped vehicle can be adequately accommodated by a shoulder which is 3,0 m wide, and there is no merit in adopting a shoulder width greater than this. The shoulder should, on the other hand, not be so narrow that a stopped vehicle would cause congestion by forcing vehicles travelling in both directions into a single lane. However, a partly blocked lane is acceptable under conditions of low speed and low traffic volume. With the narrowest width of through-lane, i.e. 3,1 m, it is possible for two vehicles to pass each other next to a stopped vehicle where the shoulders are not less than 1,0 m wide, giving a total cross-sectional width of 8,2 m to accommodate three vehicles. It is stressed that this width is an irreducible minimum and appropriate only to low lane volumes and low speeds.

Hazards tend to cause a lateral shift of vehicles if located closer than 1,5 m to the lane edge, and for speeds higher than 60 km/h a shoulder width of 1,5 m should be regarded as the minimum.

Intermediate traffic volumes and higher operating speeds require a shoulder width greater than 1,0 m, and three alternative shoulder widths are suggested, namely 1,5 m, 2,0 m and 2,5 m.

The 3,0 m shoulder is appropriate for the highest operating speeds and heavy traffic volumes.

Medians and outer separators

The median is the total area between the inner edges of the inside traffic lanes of a divided road, and includes the inner shoulders and central island. The purpose of the median is to separate opposing streams of traffic and hence reduce the possibility of vehicles crossing into the path of opposing traffic. This is accomplished by the selection of an appropriate median width or by the use of a physical barrier such as a guardrail.

Median width depends not only on traffic volume but also on the function of the road and traffic composition. A median functioning as a pedestrian refuge could be narrower than one protecting a turning vehicle which could be anything up to a combination vehicle (i.e. semitrailer plus trailer). A median narrower than 3,0 m does not offer pedestrians any sense of security, particularly when buses or trucks are travelling in the immediately adjacent lanes.

A median of less than 1,5 m in width is physically dangerous to pedestrians and should not be considered wherever pedestrian traffic is likely to be encountered. However, with severe spatial limitations, it is possible to use medians this narrow. They would serve only to accommodate back-to-back guardrails to ensure vehicular separation. A median that is 5,0 m wide would be able to accommodate a right-turn lane with provision for a pedestrian refuge, but would also require guardrail protection to separate the opposing flows of traffic.

Medians are totally inappropriate in residential streets. These streets are principally directed to the function of accessibility, including vehicles turning right from the street to enter individual properties. Medians, particularly when raised and kerbed, preclude this movement. If medians are depressed, it is possible for vehicles to traverse them but, for oncoming vehicles, this is an unexpected and correspondingly dangerous manoeuvre.

The purpose of an outer separator is to separate streams of traffic flowing in the same direction but at different speeds and also to modify weaving manoeuvres. In general, the standards applied to medians are also appropriate for outer separators. The outer separator could, for example, support the situation of a relatively high-speed road traversing a local shopping area. Vehicles manoeuvring into or out of parking spaces and pedestrians are thus safeguarded from collisions with fast-moving through-traffic.

This situation would, however, constitute poor planning or be forced by a situation outside the control of the planner. This contention refers to the fact that the central high-speed lanes would be a significant barrier to pedestrians wishing to cross the street and effectively create two independent shopping areas from an otherwise integrated unit.

Verges

The verge is defined as the area between the roadway edge and the road reserve boundary.

All facilities not directly connected with the road, e.g. telephone or power lines, are normally located in the verge. In the case of the freeway, the verge is simply the clear space between the shoulder breakpoint and the reserve boundary. On the other hand, in the urban, specifically the residential, environment it is the verge that gives the street its richness and unique character. As in the case of the cross-section as a whole, where the total width is built up as the sum of various disaggregated elements, the verge width is also the sum of the various elements it is required to contain. In general, the verge should have a width of the order of about 5 metres, but, as implied by the preceding statement, this can only be regarded as a very rough rule of thumb.

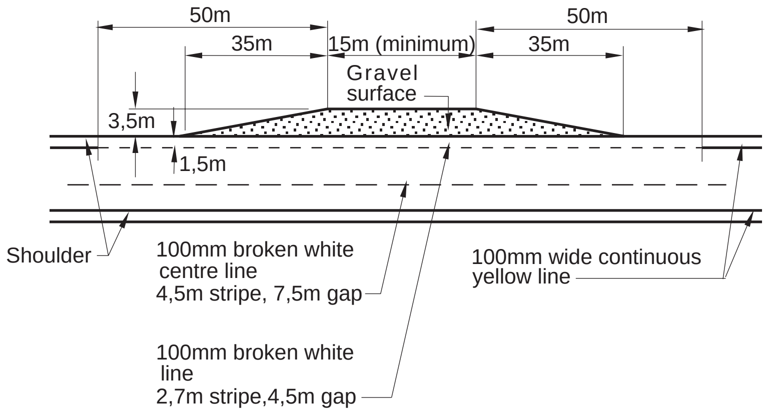
Even where HOV lanes are provided, it is desirable to locate bus stops in the verge. Typical layouts for bus stops are shown in Figure 7.8.

Various elements and their typical widths are listed in Table 7.18.

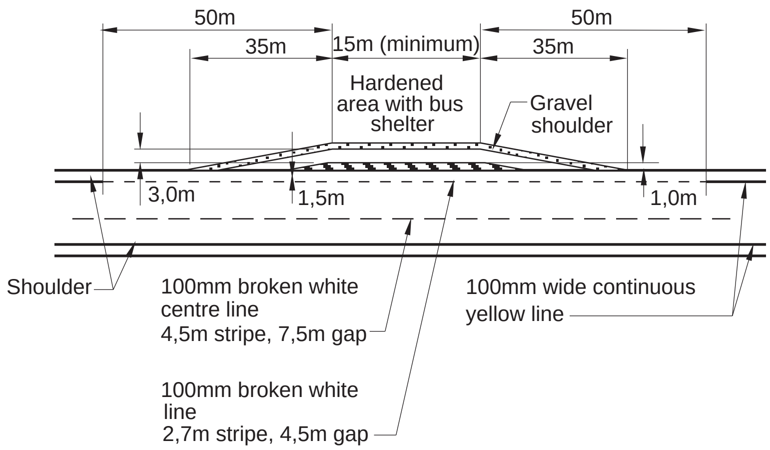
Sidewalks

Wherever there is significant usage by pedestrians, the shoulder is replaced by a sidewalk. A sidewalk is understood as comprising the entire width between the adjacent kerb face and the reserve boundary, and is generally paved over the full width of the verge. If the provision for pedestrians does not use this full width, reference is made to “footways”. The width of the sidewalk is dictated by the anticipated volume of pedestrian traffic, with an additional allowance being made for any other application intended as part of the function of the road reserve.

Reference should be made to the discussion of “hard open spaces” in Chapter 5 of this document.



(a) Gravel surface



(b) Blacktop surface

Figure 7.8: Typical bus stop layouts

Table 7.18: Typical width of verge elements

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

The topic of design for pedestrians is exhaustively described in: *Pedestrian facility guidelines: Manual to plan, design and maintain safe pedestrian facilities*. Department of Transport Report 92/126, Pretoria.

In this document, the sidewalk is defined as comprising two elements, these being the "effective" width and the "ancillary" width. The effective width is that portion of the sidewalk dedicated to movement and the ancillary width the portion of the road reserve otherwise used. The effective width is shown as being set back by 0,5 m from the adjacent kerb face.

Required effective widths of sidewalk for various LOS can be calculated by dividing the flow in pedestrians/minute by the values of pedestrians/minute/metre as shown in Table 7.19.

Table 7.19: LOS criteria for sidewalk width

LEVEL OF SERVICE	PEDESTRIANS/MINUTE/METRE
A	0,6
B	2,1
C	3,0
D	4,6
E	7,6

The total width of the sidewalk is thus the effective width as calculated plus the setback of 0,5 m plus the ancillary width determined by the other functions to be accommodated by the sidewalk.

Slopes

Camber and crossfall

Camber implies two slopes away from a central high point, as in a two-lane two-way road, where the cross-section slopes down from the centre line to the shoulders. Crossfall is a single slope from shoulder to shoulder. The slope, whether camber or crossfall, is provided to facilitate drainage of the road surface.

The steepness of slope lies in the range of 2 to 3%. In areas where heavy rainfall is common or where the most economical longitudinal gradient is 0%, the steeper slope is preferred. Cambers steeper than 3% introduce operational problems, both in driving and in increased wear of vehicle components. Where a surfaced shoulder is provided, the camber should be taken to the outer edge of the shoulder. Unsurfaced shoulders should have a crossfall of 4% to ensure a rate of flow across this rougher surface that matches the flow across the surfaced area.

In the case of very narrow reserves, such as in the case of lanes or alleys where spatial restrictions may preclude the provision of drainage outside the width of the travelled way, a negative or reverse camber, i.e. sloping towards a central low point, could be considered. In this case, the centreline of the street is the low point to create a flat V configuration. The entire surfaced width then serves as a drainage area.

Medians

Two different conditions dictate the steepness of the slope across the median: drainage and safety. As suggested earlier, the normal profile of a median would be a negative camber to facilitate drainage. The flattest slope that is recommended is 10%. Slopes flatter than this may lead to ponding and may allow water to flow from the median onto the carriageway.

Slopes steeper than 25% (or 1:4) would make control of an errant vehicle more difficult, leading to a greater possibility of cross-median accidents. If surface drainage requires a median slope steeper than 1:4, this aspect of road safety would justify replacing surface drainage with an underground drainage system.

Cut and fill batters

Gradelines that require cuts or fills so high that their batters require specific attention are alien to mixed-usage streets. The intention with these streets is that the gradeline should be as close as possible to the natural ground level and, preferably, slightly below it. This is necessary to ensure ease of access to adjacent properties and also to support the drainage of the surrounding area.

On vehicle-only roads, the slopes of the sides of the road prism are, like those of medians, dictated by two different conditions. Shallow slopes are required for safety, and a slope of 1:4 is the steepest acceptable for this purpose. The alternative is to accept a steeper slope and provide for safety by some other means, such as guardrails. In this case the steepest slope that can be used is dictated by the natural angle of repose and erodibility of the construction material.

INTERSECTIONS

Introduction

Intersections are required to accommodate the movement of both vehicles and of pedestrians. In both respects, intersections have a lower capacity than the links on either side of them. In consequence, it is the efficiency of the intersections that dictates the efficiency of the network as a whole.

Various measures of effectiveness (MOE) may be proposed. These include energy consumption, time, safety and convenience, and one should not lose sight of the fact that these measures apply as much to pedestrians as they do to vehicles. Furthermore, while they are not mutually exclusive, clashes between these measures can arise. For example, the minimisation of energy usage suggests that the major vehicular traffic should be kept moving at all times. The safety and convenience of all other road users, vehicular and pedestrian alike, would obviously be compromised as a result. Optimisation of the design of any intersection thus requires consideration of the MOEs appropriate to it and to those to whom these measures should be applied.

In the case of residential streets, the needs of pedestrians should take precedence over the needs of vehicles whereas, on higher-order roads, the needs of moving vehicles are more important. It follows that, in the former case, convenience would be a prime measure of efficiency. In the latter, energy consumption becomes significant. In both cases, safety is a major concern.

With regard to vehicular traffic, the operation of an intersection requires that opposing streams of vehicles are forced either to reduce speed or to stop. Optimisation of intersection efficiency in this case refers essentially to a reduction of delay. Delay has two components of interest. In the first, reference is to time costs. Signalisation, for example, forces a major flow periodically to be brought to a stop to allow entry by the minor flow. A signalised intersection will, therefore, always tend to show higher total delay than would a priority-controlled intersection. In the second component, reference is to energy costs because stationary vehicles with their engines idling are still consuming fuel.

Vehicles travelling in a common direction are at a low level of risk. Vehicles travelling in opposing directions are at a higher level of risk. Highest yet is the level of risk associated with vehicles travelling in crossing directions. Most accidents occurring on the road network take place at its intersections.

For reasons both of efficiency and safety it is, therefore, necessary to pay careful attention to the design of intersections. Aspects of design that have to be considered are:

- the location of intersections;
- the form of intersections;
- the type of intersection control; and
- the detailed design of individual intersection components.

These four aspects are discussed in more detail in the sections that follow.

Location of intersections

Two aspects of the location of intersections require consideration. The first of these relates to the spacing between successive intersections and the second to restraints applying to the location of individual or isolated intersections.

Successive intersections

The spacing of successive intersections is essentially a function of planning of the area being served. Minimum distances between intersections are primarily concerned with the interaction between these intersections. In the case of the major links in the movement network, access control measures are usually brought to bear to ensure the efficient functioning of the intersections on them.

Ensuring green wave progression along a route with signalised intersections would require spacings of the order of 500 m.

A driver cannot reasonably be expected to utilise the decision sight distance to an intersection effectively if an intervening intersection requires his or her attention. The sign sequence for an intersection includes signs beyond the intersection. Where an intersection is sufficiently important to warrant a sign sequence, the driver should be beyond the last of these signs before being required to give his attention to the following intersection. Under these circumstances, a minimum spacing of 500 m between successive intersections is also suggested.

Spacings of this magnitude, therefore, typically apply to vehicle-only or to higher-order mixed-usage streets.

On local streets, the goal is maximum accessibility. Any form of access control is inappropriate. One criterion for the spacing of their intersections should be that they are not so close that waiting traffic at one intersection could generate a queue extending beyond the next upstream intersection. Very closely spaced intersections would also result in a disproportionate percentage of space being dedicated to the road network.

Isolated intersections

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

The location of an intersection on a horizontal curve can create problems for the drivers on both legs of the intersecting road.

Drivers on the intersecting road leg on the inside of the curve will find it difficult to see approaching traffic, because this traffic will be partly behind them. The fact that a large portion of the sight triangle could fall outside the normal width of the road reserve would also mean that both adequate decision sight distance and adequate shoulder sight distance may be lacking.

Drivers on the leg of the intersecting road on the outside of the curve seldom have any problems with sight distance because, in addition to having approaching traffic partly in front of them, they may have the added height advantage caused by the superelevation of the curve. They do, however, have to negotiate the turn onto the through-road against an adverse superelevation. In the urban situation, where values of superelevation are low, this does not constitute a serious problem.

The risk involved in sharp braking during an emergency should also be borne in mind when locating an intersection on a curve.

Generally, an intersection should not be located on a curve with a superelevation greater than 6%.

The stopping distance required on a downgrade of 6% is approximately 40% longer than that required on a level road. Drivers seemingly have difficulty in judging the additional distance required for stopping on downgrades and it is suggested, as a safety measure, that intersections should not be located on grades steeper than 3%.

If it is not possible to align all the legs of an intersection to a gradient of 3% or less, the through-road could have a steeper gradient because vehicles on the intersecting road have to stop or yield, whereas through vehicles may only have to do so occasionally.

Where steep gradients on the intersecting road are unavoidable, a local reduction in its gradient within the reserve of the through-road should be considered. The reason for this is that buses and freight vehicles have difficulty in stopping and pulling away on steep slopes. Typically, the camber of the through-road would be extended along the intersecting road for a sufficient distance to allow such vehicles to stop clear of the through-lane on the through-road and pull away with relative ease. A distance of approximately 10 m from the shoulder breakpoint is required for this. After that, reverse curves with an intervening gradient of 6% or more can be used to match the local gradeline of the intersecting road to the rest of its alignment. In the case of private accesses, steeper grades can be considered.

One of the consequences of a collision between two vehicles at an intersection is that either or both may leave the road. It is therefore advisable to avoid locating an intersection other than at approximately ground level. Lateral obstructions of sight distance should also be considered when the location of an intersection is being determined.

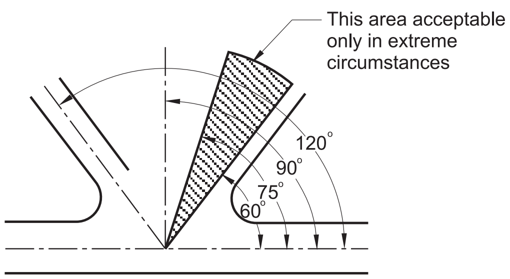
The location of an intersection may have to be modified as a result of an excessive angle of skew between the intersecting roads, i.e. the change of direction to be negotiated by a vehicle turning left off the through-road. Preferably, roads should meet at, or nearly at, right angles. Angles of skew between 60° and 120°, with 0° representing the direction of travel on the through-road, produce only a small reduction in visibility for drivers of passenger vehicles, which often does not warrant realignment of the intersecting road. However, the range of angles of skew between 60° and 75° should be avoided because a truck driver wishing to enter the through-road at an intersection with an angle of skew of 75° or less would find the view to his left obscured by his vehicle. Therefore, if the angle of skew of the intersection falls outside the range of 75° to 120°, the intersecting road should be relocated.

Figure 7.9 illustrates the acceptable angles of skew.

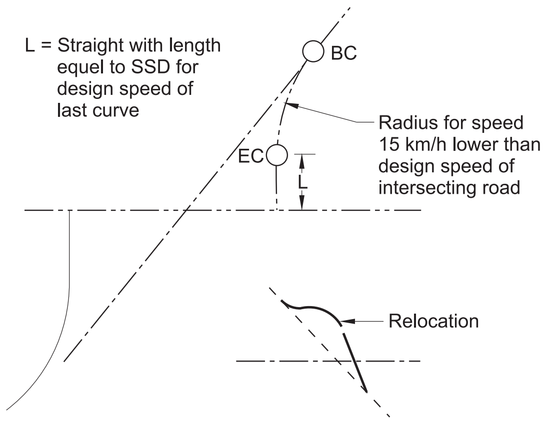
The types of intersection control

Signalisation

Signalisation applies only to higher-order roads and is, in any event, an expensive form of control. Signals should not be employed, in the first instance, as speed-reducing devices. Signals introduce an inefficiency into the system by imposing delay on the through flow to allow the intersecting flow either to cross or to join it. The objective, generally, is to keep the introduced inefficiency to a minimum through the use of proper signal progression. It is interesting to note that an arterial with good signal progression can deliver more vehicles per unit of time to the CBD street system than the latter can handle. This may be an argument in favour of deliberately



(a) Acceptable angles of skew



(b) Relocation of skew intersections

Figure 7.9: Angles of skew

interrupting the flow by a well-planned discontinuity in progression.

Signals should not be used on the highest order roads. At design speeds of 100 km/h and higher, problems with regard to the length of the amber phase and the extent of the dilemma zone manifest themselves. The dilemma zone is that length of road upstream of a signal where, if the signal changes to amber, it is possible neither to clear the intersection before the onset of the opposing green phase nor to stop in advance of the pedestrian cross-walk. If these speeds are to be maintained, freeway operation has to be considered.

Multi-way stop or yield

Reference is to the situation where every approach to the intersection is subject to stop or yield control. Some local authorities in South Africa have also instituted a variation on this form of control by applying stop control not to all but to the majority of approach legs, e.g. two out of three or three out of four. In the United States, the operation of intersections subject to this form of control grants priority to vehicles approaching from the left. The South African operation is based on the principle of “first come, first served”.

This form of control is appropriate to the situation where no clear distinction can be drawn between the intersecting roads in terms of relative importance and where traffic flows on each are more or less equal. It is typically regarded as an interim measure prior to the installation of traffic signals.

Mini roundabouts, traffic circles and gyratories

The principle difference between these three forms of control is the diameter of the central island. The gyratory can have a central island with a diameter of 50 m or more, whereas the traffic circle would typically have a central island with a diameter of the order of 10 m and the mini-roundabout a central island that could range from a painted dot to about 4 m diameter.

The gyratory and the traffic circle operate on the basis of entering vehicles being required to yield to traffic approaching from the right. The mini-roundabout, on the other hand, is controlled by a traffic circle yield sign which, as defined in the Road Traffic Regulations, “Indicates to the driver of a vehicle approaching a traffic circle that he shall yield right of way to any vehicle which will cross any yield line at such intersection before him and which, in the normal course of events, will cross the path of such driver’s vehicle.”

The distinction between behaviour at a traffic circle and that required at a mini-roundabout is not clear to many drivers. Confusion is exacerbated by the fact that a warning sign (a triangular sign with the apex uppermost) requires that right of way should be granted to vehicles approaching from the right as is the case of the traffic circle.

Priority control

Priority control implies that one of the intersecting roads always takes precedence over the other with control taking the form of either stop or yield control. This form of control applies to the situation where it is clear which is the more important of the two intersecting roads. Priority control can also be alternated between successive intersections, for example in a residential area where the layout is more-or-less a grid pattern and the intersecting streets are of equal importance. In this case, the switching of priority would partially serve as a traffic calming measure. In general, this is the most commonly used form of intersection control.

Selection of appropriate control measure

Each intersection should be considered on its own merits and hard and fast rules or a “recipe” method for the selection of the control measure to be employed at any given intersection are not recommended.

Volumes of vehicular traffic being served by the intersection are not considered to be anything other than the roughest of guides. In terms of traffic movement through an intersection, the basic goal should be to minimise delay as far as possible.

Delay has two components: geometric delay and traffic delay. Geometric delay is the delay caused by the existence of the control measure employed. The control measure requires that a driver slow down or stop at the intersection, check that the intersection is clear and then accelerate back to the previous speed. Geometric delay is the difference between the time taken to perform the required set of actions and that expended in travelling through the intersection area at undiminished speed. Traffic delay is that generated by opposing traffic with, as an example, right-turning traffic being impeded by heavy opposing flows or causing impedance to following vehicles that wish to travel straight through the intersection.

Schermers (1987) reports that, at flows of less than 300 vehs/15 mins, traffic delays with four-way stops are less than those with signalisation. In the range above 900 vehs/15 mins, signalisation demonstrated the lowest level of delay. Mini- roundabouts, on the other hand, demonstrate relatively low levels of delay, i.e. even less than four-way stops, up to a flow of about 400 vehs/15 mins. Thereafter, the extent of delay from using mini-roundabouts increases rapidly but remains less than that with signalisation until a flow of 900 vehs/15 mins is achieved. Total traffic delay at 300 veh/15 min amounts to about 3 000 veh.sec/15 mins or about 10 seconds per vehicle on average whereas, at 900 vehs/15 mins, total traffic delay is about 16 000 veh.sec/15 mins or about 18 seconds per vehicle on average.

Priority control does not lend itself readily to analyses of the above form, as the total delay is even more heavily dependent on the split between the opposing flows than in the cases discussed above. Heavy flows on the through-road result in there being relatively few gaps in the traffic stream that are acceptable to drivers wishing to enter or cross from one of the intersecting legs. This results in substantial delays being generated. On the other hand, for the same total flow but with fewer of these vehicles travelling on the through-road, adequate gaps exist for even relatively heavy flows on the intersecting road to experience little delay.

Calculation of delay using a model such as the well-known Tanner formula should be applied to these intersections.

The form of intersections

Intersection form is dictated largely by planning considerations. However, even during the planning phase, due cognisance has to be taken of the safety of vehicles or pedestrians within the area of the intersections. Safety is enhanced by, inter alia, reduction of the number of conflict points at which accidents can occur.

The number of conflict points increases exponentially with the number of legs added to the intersection. A three-legged intersection generates six vehicle-vehicle conflict points, whereas a four- legged intersection has 24 and a five-legged intersection 60. Accident history shows that this increased potential for collision at intersections is, in fact, realised.

In addition to the decrease in safety with an increasing number of approaches to an intersection, there is also a decline in operational efficiency, i.e. an increase in delay.

Multi-leg intersections, i.e. intersections with more than four legs, should not be provided in new designs and, where they occur in existing networks, every effort should be made to convert them to four- or three-legged intersections through channelisation procedures.

Staggered intersections address the problem of skewed intersections (i.e. those with angles of skew outside the limits recommended above) which can be either three- or four-legged. Skewed intersections present a variety of problems. In the first instance, angles of skew greater than those specified generate a line-of-sight problem for the driver on the intersecting road. Secondly, a vehicle required to turn through the acute angle will be moving at a very slow speed, suggesting that those entering the through-road may require a greater sight distance than would be the case for a 90° turn. Finally, the surfaced intersection area becomes excessive. Without channelisation of this movement, a driver traversing the intersection is confronted by a large surfaced area without any positive guidance on the route to be followed. Unpredictable selections of travelled path can represent a distinct hazard to other vehicles in the intersection area.

Four-legged skewed intersections should be relocated so that they form either a single crossing with an angle of skew closer to 90°, or a staggered intersection, which is a combination of two three-legged intersections in close proximity. The right-left stagger, i.e. where the driver on the intersecting road is required to turn right onto the through-road followed by a left turn off it, is preferred. In this case, the driver waits on the intersecting road for a gap in the through-traffic prior to entering the through-road, with the left turn off it being unimpeded except possibly by pedestrians. The left-right stagger may require the vehicle to stop on the through-road while awaiting a gap in the opposing flow to complete the right turn. Relocation of the intersection is to be preferred to the left-right stagger.

Mini-roundabouts can have either three or four approach legs. They comprise either a slightly raised or a painted central island, usually less than 4 metres in diameter, with the traffic lanes being deviated slightly, both to accommodate the island and to force a reduction of speed through the intersection. The discussion above relating to angles of skew applies equally to this form of intersection.

Intersection components

Auxiliary through-lanes

Auxiliary lanes for through-traffic are added outside the through-lanes to match the capacity of the intersection with that of the road between intersections. These lanes are normally only provided at signalised intersections. The length of lane to be added is a matter of calculation. It is dependent on the traffic flow to be serviced and on the length of green time available for the approach leg in question. In the case of priority control, auxiliary through-lanes would not be required on the through-road. Traffic volumes on the intersecting road would probably be too low to warrant their application. This should, however, be checked.

Auxiliary turning lanes

Turning lanes provide for traffic turning either to the left or to the right and can thus be added either outside the through-lanes or immediately adjacent to the centreline.

In the latter case, the through-lanes would have to be deviated away from the centreline if there is not a median island wide enough to accommodate the right-turn lane. Particularly at night, a wet, hence reflective, road surface causes the road markings not to be readily visible. Deviation of a through-lane should therefore be clearly demarcated by road studs to ensure that vehicles do not inadvertently stray into the right-turn lane.

The extent of deviation provided is dependent on the extent of offset provided to the right-turn lane. Good practice suggests that opposing right-turn lanes should be in line with each other to provide the turning driver with the maximum clear view of oncoming traffic that would oppose the turning movement. In this case, the deviation of the through-lane would amount to only half of the width of the turning lane.

Ideally, turning lanes should have the same width as the adjacent lanes but spatial limitations may require that a smaller width be used. The low speeds anticipated in turning lanes in combination with relatively low lane occupancy make it possible to use lesser widths, but the width of the turning lane should not be less than 3,1 m.

The length of turning lanes has three components: the deceleration length, the storage length and the entering taper.

Deceleration should, desirably, take place clear of the through-lane. The total length required is that necessary for a safe and comfortable stop from the design speed of the road. Stopping sight distance is based on a deceleration rate of 3,0 m/s^2 and a comfortable rate is taken as being half this, so that

$$s = \frac{v^2}{38{,}9}$$

where s = deceleration lane length (m)
v = design speed (km/h)

The storage length has to be sufficient for the number of vehicles likely to accumulate during a critical period. It should not be necessary for right-turning vehicles to stop in the through-lane. Furthermore, vehicles stopped in the through-lane while awaiting a change of traffic signal phase should not block the entrance to the turning lane. In the case of unsignalised intersections, the storage length should be sufficient to accommodate the number of vehicles likely to arrive in a two-minute period. At signalised intersections, the required storage length depends on the signal cycle length, the phasing arrangement and the rate of arrivals of right-turning vehicles. The last-mentioned can be modelled using the Poisson distribution which is

$$P(x=r) = \frac{e^{-m}\, m^r}{r!}$$

where e = base of natural logarithms

m = constant equal to the value of arrivals

r = the number of arrivals for which the probability is being calculated.

In both signalised and unsignalised intersections, the length of storage lane should be sufficient to accommodate at least two passenger cars. If buses or trucks represent more than 10% of the turning traffic, provision should be made for storage of at least one passenger car and one bus or truck. It should be noted that the length of the design bus is 12,3 m, compared with the 9,1 m of the truck.

The shorter length could be used only if the road is not intended to serve as a bus route.

Tapers

Tapers can be either passive, allowing a lateral movement in the traffic stream, or active, forcing the lateral movement to take place. Thus, the addition of a lane to the cross-section is preceded by a passive taper, and a lane drop by an active taper. In general, an active taper should be long whereas a passive taper can be short. In the latter case, a taper can be "squared off", meaning that a full-width lane is added instantaneously and the taper demarcated by road marking as opposed to physically constructing a tapered length of road.

Taper rates are shown in Table 7.20.

The lower taper rate for active kerbed tapers is permissible because of the higher visibility of the kerbing which, for this purpose, should be highlighted with paint or reflective markings. Very often, the need for storage space at urban intersections outweighs the need for smooth transitions. In this case the passive taper rate can be reduced to 1:2.

Kerbing and kerb radii

Barrier or semi-mountable kerbing is recommended for intersections because of the more visible demarcation of the lane edge that they offer. In the presence of pedestrians, barrier kerbing is the preferred option. In either case, ramps should be provided for prams and wheelchairs. Kerbing is normally offset by 0,3 m from the lane edge.

Left-turning traffic must be able to negotiate the turn without encroaching on either the adjacent shoulder or sidewalk or on the opposing lane. The latter requirement can, however, be relaxed in the case of the occasional large vehicle on a street with low traffic volumes.

Various forms of edge treatment are described in Table 7.21.

It should be noted that the minimum outer turning radius of a passenger car is of the order of 6,2 m. A passenger car would thus, at a crawl speed, just be able to maintain position relative to a left- turning kerb with a radius of about 4,0 m.

The three-centre curve closely approximates the actual path of a vehicle negotiating the turn. This has the effect of reducing the extent of the surfaced area that has to be provided and is, thus, particularly useful where a change of direction of greater than 90° has to be accommodated. This close approximation to actual wheel paths also suggests that it offers a level of guidance to turning vehicles better than that provided by simple curves.

Corner splays

Depending on the width of the road reserve, it may be necessary to splay the reserve boundary in the intersection area to provide adequate stopping sight distance for drivers both on the major and the minor legs of the intersection. In general, a minimum width of border area around the corner should be of the order of 3,5 m.

Table 7.20: Taper rates

DESIGN SPEED (km/h)	30	40	50	60	80	100
Passive tapers						
Taper rate (1 in)	5	8	10	15	20	25
Active tapers						
Taper rate (1 in) for painted line taper	20	23	25	35	40	45
Taper rate (1 in) for kerbed taper	10	13	15	20	25	30

Table 7.21: Typical edge treatments for left turns

TREATMENT	MINIMUM KERB RADIUS (m)	APPROACH/DEPARTURE TREATMENT
Simple curve	10	Nil
Simple curve with tapers	6	1:15 tapers
Three-centred curve	6	Ratio of curvature 2:1:4
Channelised turning roadway with three-centred curve	15	Ratio of curvature 2:1:4
Channelised turning roadway with simple curve and tapers	25	1:10 tapers

Channelisation

Channelisation involves the use of islands and road markings and is usually required where traffic volumes are high. The purposes of channelisation with regard to vehicle movement are to:

- separate areas for manoeuvring and present drivers with one decision at a time;
- control the direction of movement of vehicles to obtain small angles for merging and diverging at low relative speeds or approximate right angles for crossing at high relative speeds;
- control speed by redirection or funnelling, the latter implying a steady reduction of lane width over a short distance;
- provide protection and storage for turning vehicles;
- eliminate excessive surfaced areas which permit drivers to perform improper manoeuvres or to travel along paths unpredictable to other drivers;
- prevent illegal manoeuvres, such as turns in the wrong direction into one-way streets; and
- provide space and protection for traffic control devices and other road signs.

Channelisation is also required at intersections where traffic volumes may be relatively low but pedestrian volumes high. In this case, the function of channelisation is directed towards providing refuge for pedestrians seeking to cross the various traffic flows.

Walking speeds are typically of the order of 1,5 m/s. In the vicinity of old-age homes and similar areas, accommodation should be made for a walking speed of about 1 m/s. Crossing a two-lane street would thus require a gap or a lag of about seven seconds. A gap is the difference between the times of arrival at the crossing point by two successive vehicles. The lag is the unexpired portion of a gap, i.e. the time between the pedestrian arriving at the crossing point and an opposing vehicle arriving at the same point. On multilane streets, the crossing time is correspondingly higher.

If traffic flows are such that a gap or lag equal to or greater than the crossing time is not available at about one minute intervals, pedestrians are tempted to cross by pausing on the roadmarkings between the various flows to await the gap in the next flow to be crossed.

This is not a normally recommended practice. It can be avoided by the provision of islands which have the effect of reducing the duration of the required gap, hence increasing the probability of its occurrence. Alternatively, the creation of adequate gaps can be forced by the use of demarcated or, preferably, signalised pedestrian crossings.

Islands

Islands, whether painted or kerbed, can be classed into three groups:

- directional islands, which direct traffic along the correct channels or prevent illegal manoeuvres;
- divisional islands, which separate opposing traffic flows; and
- refuge islands, to protect pedestrians crossing the roadway or turning vehicles that are required to stop while awaiting gaps in the opposing traffic, and also to provide space for traffic control devices.

Typically, islands are either long and narrow or triangular in shape. The circular central island is considered more a traffic control device than a channelisation feature. Small islands have low visibility and cannot serve safely as either accommodation for pedestrians or traffic control devices. Islands should not have an area of less than 5 m^2 or width of less than 1,2 m. Islands used as pedestrian refuges should preferably be 3,0 m wide. Painted islands do not constitute a significant refuge for pedestrians and should not be used in this application. Pedestrian refuge islands should be provided with barrier kerbing and ramps for wheelchairs and prams.

Island kerbing is usually introduced suddenly. For this reason, the approach end requires careful design. In the case of triangular islands, the point of intersection of the approach sides of the island should be rounded and painted and, possibly, also be provided with reflective markings. The approach end should also be offset from the edge of the adjacent lane, as shown in Figure 7.10.

Turning roadways

The normal track width of a vehicle is the distance between the outer faces of the rear tyres. When a curve is being negotiated, the rear wheels track inside the front wheels and the track width then becomes the radial distance between the path of the outside front wheel and the inside rear wheel. On turning roadways, it is this greater width that has to be accommodated. The turning roadway width is thus a function of:

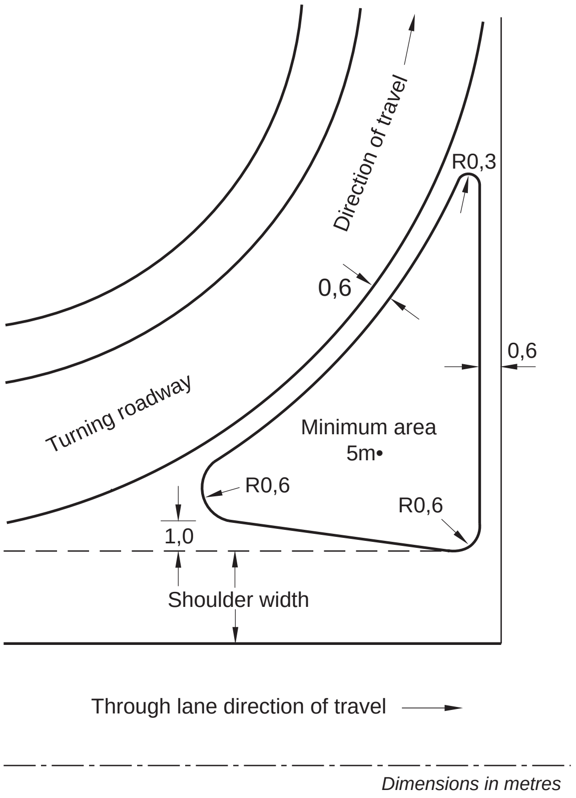


Figure 7.10: Layout of island

- the flow operation being designed for;
- the design vehicle to be accommodated; and
- the radius of curvature of the turning roadway.

With regard to the first-mentioned, three operations need to be addressed. These are:

- Case I: One-lane one-way operation, with no provision for passing a stalled vehicle.

- Case II: One-lane one-way operation, with provision for passing a stalled vehicle.

- Case III: Two-lane operation, either one-way or two-way.

Design vehicles to be accommodated are passenger cars (P), single-unit trucks or buses (SU) and articulated vehicles (WB12). The selection of design vehicle is addressed as the traffic condition being designed for. These are:

- Traffic Condition A - Predominantly P vehicles but some consideration given to SUs.

- Traffic Condition B - Sufficient SUs (approx 10%) to govern design but some consideration given to WB12s.

- Traffic Condition C - Sufficient WB12s to govern design.

Provision for passing stalled vehicles and two-lane operation require a combination of design vehicles. The combinations normally considered are shown in Table 7.22.

Table 7.22: Design traffic conditions

CASE	A	B	C
I	P	SU	WB12
II	P-P	P-SU	SU-SU
III	P-SU	SU-SU	WB12-WB12

The selected lane width either should or can be modified depending on the selected edge treatment. Three edge treatments have to be accommodated. These are mountable kerbing, barrier or semi-mountable kerbing, and the stabilised shoulder. They can occur in combination, e.g. a barrier kerb on one side of the lane and a stabilised shoulder on the other. Shoulders would only be used at intersections if the cross-section of the road upstream and downstream of the intersection includes shoulders. In the presence of pedestrians, kerbing is necessary.

Recommended turning roadway widths are listed in Table 7.23. for the various cases, conditions and edge treatments.

The crossfall prevailing on the through-lanes is simply extended across the turning roadways. Should superelevation be deemed to be necessary, it could be achieved by the use of a crossover crown line, in which case the maximum superelevation could be as high as 6%.

Mini-roundabouts

Mini-roundabouts are the one exception to the general practice of avoiding the use of intersection controls as traffic calming devices. They can be used either as intersection controls or as traffic calming devices. In the latter case, however, their application should be as part of area-wide traffic calming schemes rather than in isolation.

The mini-roundabout comprises a central circular island, deflector islands on each of the approach legs and a circular travelled way around the central island. On a three-legged or T-intersection, it is frequently necessary to apply speed humps to the approach from the left on the cross leg of the T. The reason for this is that the location of the circular island is invariably such that traffic on this leg has a straight line path through the intersection and it is necessary to force a reduction of speed.

Table 7.23: Turning roadway widths (m)

RADIUS ON INNER EDGE (m)	CASE I			CASE II			CASE III		
	DESIGN TRAFFIC CONDITION								
	A	B	C	A	B	C	A	B	C
15	5,4	5,4	6,9	6,9	7,5	8,7	9,3	10,5	12,6
25	4,8	5,1	5,7	6,3	6,9	8,1	8,7	9,9	11,1
30	4,5	4,8	5,4	6,0	6,6	7,5	8,4	9,3	10,5
50	4,2	4,8	5,1	5,7	6,3	7,2	8,1	9,0	9,9
75	3,9	4,8	4,8	5,7	6,3	6,9	8,1	8,7	9,3
100	3,9	4,5	4,8	5,4	6,0	6,6	7,8	8,4	0,0
125	3,7	4,5	4,8	5,4	6,0	6,6	7,8	8,4	8,7
150	3,7	4,5	4,5	5,4	6,0	6,6	7,8	8,4	8,7
TANGENT	3,7	4,5	4,5	5,1	5,7	6,3	7,5	8,1	8,1
WIDTH MODIFICATION APPROPRIATE TO EDGE TREATMENT									
Mountable kerb	none			none			none		
Barrier kerb one side	add 0,3 m			none			add 0,3 m		
Barrier kerb both sides	add 0,6 m			add 0,3 m			add 0,6 m		
Stabilised shoulder one or both sides	Condition B & C lane widths on tangent may be reduced to 3,7 m for 1,2 m or wider shoulder			Deduct shoulder width; minimum width as for Case I			Deduct 0,6 m where shoulder is 1,2 m or wider		

Mini-roundabouts have negative implications for cyclists and, to a lesser extent, for pedestrians. Circulation of traffic through a roundabout is not straightforward and the need for vehicles to stop at the intersection is reduced. Crossing opportunities for pedestrians are thus similarly reduced and the task of judging acceptable gaps is more difficult. The circulatory flow and reduced carriageway width offer less protected space for cyclists. In addition, motorists are seldom prepared to yield the right of way to cyclists. These two classes of road users thus require careful consideration in the design of these intersections and appropriate facilities provided for them.

Central island

The central island is typically of the order of 4,0 m in diameter. It may simply be a painted island, although the preferred option is that it should be an asphalt hump. The latter offers more specific guidance to drivers and ensures that the mini-roundabout operates as intended. The height of the hump should be in the range of 75 mm to 100 mm. This is a compromise between the height that is visible to approaching drivers and the height that long vehicles can traverse without damage. The guidance role is strengthened, and the asphalt hump simultaneously protected from damage by passing vehicles, if the central island is surrounded by a 25 mm high steel hoop, securely anchored to the road surface.

Width of travelled way

The inscribed circle diameter is dependent on the design vehicle. Britain and Australia both recommend a diameter of 28 m, suggesting thus that the travelled way has a width of 12 m, assuming that the traffic circle has a diameter of 4 m. This corresponds to use of a WB-12 as the design vehicle. South African practice, where the mini-roundabout is typically used in residential areas, is to use the passenger car or the bus as the design vehicle of choice. If the roundabout is not located on a bus route, the inscribed circle diameter could be reduced to 14 m. Bus routes would require an inscribed circle 25 m in diameter and this width is adequate for two-lane operation by passenger cars.

Deflector islands

Kerbed islands are used to guide vehicles into the intersection area. The side of the island closest to the traffic approaching the intersection thus constitutes an active taper. A taper rate of 1:10 appropriate to a design speed of 30 km/h should be employed. The side of the island downstream of the intersection is also an active taper insofar as it serves as a transition from the wider lane width around the mini-roundabout to the normal lane width applying to the rest of the street, and the same taper rate applies. In this case, however, the taper rate is relative to the direction of movement of vehicles exiting from the intersection. The approach end of the island should be rounded and offset as illustrated in Fig 7.10.

The kerbed island is normally preceded by a painted island.

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