Contractor Report 94

Transport and Road Research Laboratory

Department of Transport

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A REVIEW OF GEOMETRIC DESIGN AND STANDARDS FOR RURAL ROADS IN DEVELOPING COUNTRIES

by

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A. BACKGROUND

1. INTRODUCTION

1.1 BACKGROUND TO THE STUDY

A study to develop appropriate geometric design standards for use in developing countries has been undertaken by the Overseas Unit of the Transport and Road Research Laboratory (TRRL). The first part of this study was conducted in collaboration with the University of Birmingham and consisted of a review of the recently revised American (AASHTO 1984), Australian (NAASRA 1980) and British (Department of Transport 1981a) standards. The report on this work (Kosasih et al 1987) discussed the potential for applying these industrialised country standards to developing countries.

The second part of this study is reported here. It has been carried out by Roughton & Partners, Consulting Engineers for TRRL, and builds on the earlier study by reviewing all other relevant information. It describes the basis on which standards have been recommended by TRRL. It is aimed at research workers and practitioners who are knowledgeable in the field of geometric design and related standards.

A guide to geometric design for developing countries has been produced as a separate document, Overseas Road Note 6.

1.2 CONSIDERATIONS FOR DEVELOPING COUNTRIES

McLean (1978d) has noted that, in the light of experience from industrialised countries, there would seem to be three distinct stages in the development of a road network.

1. Initially, it is necessary to establish a road network to at least provide a basic means of communication between centres of population. At this stage, little attention is paid to geometric standards as it is much more important to consider whether a road link exists at all or, if it does, whether it is 'passable' at all times.

2. The next stage is to build capacity into the road network. Geometric standards probably have little to contribute to this except in the areas of road width and gradient. Much more important factors are whether or not a road is paved, or whether it has sufficient structural strength to carry the traffic wishing to use it.

3. The final stage is to consider operational efficiency of the traffic using the network and it is at this time that geometric standards become really important.

Developing countries, by their very nature, will not usually be at Stage 3 of this sequence; indeed most will still be at the first stage. However, design standards currently in use were generally developed for countries at Stage 3 and they were developed for roads carrying relatively large volumes of traffic. For convenience, these same standards traditionally have been applied to low-volume roads, as was shown by Cron (1978). Although the use of established geometric standards leads to economic and safe designs for high-volume roads, for low-cost, low-volume roads, it has been argued that the use of the same standards leads to designs which are uneconomic and technically inappropriate (Oglesby and Altenhofen 1969, McLean 1978d).

It has been noted that the operational needs of road users in developing countries are often very different from those in the industrialised countries (Hills et al 1984). Also, it has been shown by TRRL that accident rates in developing countries are generally substantially higher than those in developed countries, and that the accidents have markedly different characteristics (Jacobs 1976, Jacobs and Sayer 1983). From the point of view of safety, it therefore appears that geometric standards used in industrialised countries may not be appropriate to the developing world, and more high quality accident data is required as a basis for the identification of remedial measures. Safety should none-the-less be a fundamental consideration in the design of all roads.

When developing appropriate geometric design standards for use in a developing country, the first step should normally be to identify the objective of the road project. It is convenient to define the objective in terms of the three levels of development of a road network as described above. Thus, the objectives will be:

for level 1: to provide access;

for level 2: to provide additional capacity;

for level 3: to increase operational efficiency.

It is then possible to consider standards in the context of these three differing objectives.

For roads whose objective is to provide fundamental access (Level 1), absolute minimum standards can be used to provide an engineered road. The choice of standards will be governed only by such issues as traction requirements, turning circles and any requirement for the road to be 'all weather'.

If the object of the project is to provide additional capacity for the road (Level 2), then decisions will need to be taken on whether or not it should be paved and on what is an appropriate structural strength. Road width will normally be governed only by the requirement that vehicles should be able to pass each other.

It is only when the objective of the road is to increase the operational efficiency of a route (Level 3), that standards such as those developed in Australia, Britain and the United States become relevant. It is not really practicable to apply standards such as these to roads at Levels 1 or 2.

These points are elaborated upon in the report on the first part of the TRRL study (Kosasih et al 1987) which concluded that no standards exist at the moment which are appropriate for developing countries, but that there was some scope for adapting standards from industrialised countries.

1.3 SCOPE OF THIS REPORT

This report discusses the derivation of appropriate standards for developing countries under the headings of:-

Cross-section
Sight distance
Alignment
Operational considerations

It reviews both research that has been carried out in the field of geometric design, and standards that are currently in use. Finally, summaries are provided of recommendations made.

B. CROSS-SECTION

2. WIDTH

2.1 BASIC CONCEPTS

2.1.1 Construction cost

Road width has a profound effect on the cost of construction, and Choudhury (1980) and other authors have identified the need to adopt minimum possible standards which will not seriously affect travel comfort and safety.

The absolute minimum width of a road will depend to some extent on road alignment and the volume and type of vehicles which will use the road. At the lowest level, a road may be one-lane and of a width just sufficient for the passage of the design vehicle. In developing countries, this is likely to be a truck with a maximum width of approximately 2.5 metres (SATCC 1986).

With anything above minimum flows, it is essential that there is provision for vehicles to pass. This may be achieved on single lane roads by the provision of passing places or building shoulders on which vehicles can travel when passing.

2.1.2 Operational factors

Kosasih et al (1987) noted that clearances between vehicles passing at high speed need to be greater than between slow moving vehicles, and roads carrying large volumes of traffic require wider pavements than those carrying small volumes. They also noted that heavy traffic on a road results in frequent passing and overtaking manoeuvres and vehicles driving further from the centreline.

Normal steering deviations and tracking errors, particularly of heavy vehicles, reduce clearances between passing vehicles. The path width of vehicles is larger on horizontal curves than on the straight due to tracking, the effect being particularly significant for trucks. When a commercial vehicle is turning at low speeds, the rear wheels travel in a smaller arc than the front wheels (Brock 1973). The distance between the outer and inner arcs measured normal to the tangent gives the width of the track taken by the vehicle. Pavement widening may be necessary on both one-lane and two-lane roads in order to allow for this.

2.2 RESEARCH FINDINGS

2.2.1 Safety

Shannon and Stanley (1978) found that terrain had a stronger linear relationship with accident rate than did width, although width had some effect. Simpson and Kerman (1982) found that additional width up to 10 metres on single carriageways improved safety as well as operating cost and driver comfort, although this was for UK conditions in which the carriageway was usually delineated by raised kerbs. Zeeger et al (1981) found that very little additional benefit was realised by widening a lane to more than 3.4 metres, and overall, for paved roads, the decrease in accident rate was greater for increases in lane width than for equivalent increase in shoulder width. McLean (1978a,b and c) also found that it was beneficial to reduce the width of shoulders to obtain wider carriageways.

Road width has the greatest effect on accidents involving vehicles running off the road and head-on collisions as indicated in Table 2.1 (Zeeger et al 1981). Increases in width were found to have little effect on other types of accident.

Table 2.1 Dependence of accidents on road width

Lane Width(m)		Reduction in single-vehicle accidents and head-on collisions (%)
Before Widening	After Widening	
2.1	2.4	10
2.1	2.7	23
2.1	3.0	29
2.1	3.4	39
2.4	2.7	16
2.4	3.0	23
2.4	3.4	36
2.7	3.0	10
2.7	3.4	29
3.0	3.4	23

McLean (1985) found that the accident rate in the USA increased by about 45 per cent for a reduction in width (carriageway plus shoulder) from 13.2 metres to 8 metres, although accident rates in other countries did not match this pattern.

Oglesby and Altenhofen (1969) found accidents to be rare events on low-volume rural roads. Furthermore, there was almost no evidence to indicate that higher standards of formation width or surface type would reduce the already small number, although they found that highway engineers appeared to believe the opposite. From an economic standpoint, accident costs were found to be of a lower order of magnitude than construction and operating costs.

Jacobs (1976) found a clear relationship between width and accident rates for data from Jamaica, but not for Kenya, although in Kenya only a limited range of widths was available. None of the roads had surfaced shoulders.

Silyanov (1973) derived a relationship between accident rate and width based on data from USSR, UK, France, Hungary and West Germany. This showed the accident rate increasing by about 50 per cent as a result of a width reduction from 8 metres to 5 metres. There was considerable scatter in the data and the roads were generally without shoulders.

2.2.2 Passing and overtaking

Oglesby and Hewes (1966) concluded from empirical studies that the effect of narrow carriageways would cause vehicles to slow down and accelerate back to their original speeds on meeting or overtaking. On 5.5 metre wide carriageways, cars were found to pass oncoming trucks at clearances averaging 0.8 metres and on 6.1 metre carriageways, average clearances were found to be 1.1 metres, although in the case of a truck meeting an oncoming truck, clearances were less. Trucks overtaking other trucks remained centred in their lanes only when lanes were 3.7 metres wide or wider. Clearance for cars overtaking other cars were only 0.7 metres on 5.5 metre wide carriageways, and 1.5 metres on 7.3 metre wide carriageways.

Based on this study, Oglesby and Altenhofen (1969) determined speed reductions for meeting as shown below in Table 2.2. On the basis of these, for flows of 100 vehicles per day, all combinations of annual costs increased as carriageways were widened beyond 4.9 metres (16 feet), with the rate of increase being somewhat greater for roads sufficiently wide that passing or meeting vehicle interaction ceased. It was suggested that, at 400 vehicles per day, there might be a minimum total transport cost point at the width where interaction ceased (i.e. 24 feet), with increases in cost at greater widths. Thus, no economic reason was identified for making carriageways for low volume rural roads as wide as those called for by AASHTO standards. However, the researchers found some evidence which indicated that vehicles did not slow down on 5.5 metres (18 feet) wide pavements, further supporting the case for narrower roads.

Table 2.2: Estimated speed reduction for different road widths

Road width	(feet)	16	18	20	22	24
Speed reduction	(mph)	16	12	8	4	0

Troutbeck (1984), found from studies of two roads that a reduction in pavement width from 7.4 metres to 6.0 metres resulted in a lower level of service, as drivers required an increased overtaking time on the narrower road. Safety margins were found to be unaffected.

2.3 STANDARDS IN USE

2.3.1 Basic lane and carriageway widths

In the UK (Department of Transport 1981a), the following standard widths are used for two-lane, three-lane and four-lane roads (Table 2.3).

Table 2.3 Standard road widths in U.K.

Lanes	Standard width (m)
2	7.3
3	11.0
4	14.6

NAASRA (1980) do not recommend widths greater than 7.5 metres for two lane roads because they have found that little advantage is gained from the extra width and, where traffic volumes are heavy, occasional three-lane operation may develop.

In their recommendations for the design of single carriageways, NAASRA (1980) recommended that width should be at least 3.5 metres as a lower width would result in excessive shoulder wear. Choudhury (1980) recommended a carriageway width of between 3.0 and 3.5 metres for low cost rural feeder roads built on a formation width of 5.5 metres. For very low flows, where the possibility of vehicles meeting was considered to be negligible, a carriageway width of 3.0 metres with no shoulders was suggested as being sufficient.

The Department of Works (1985) in Papua New Guinea recommended a desirable lane width on rural roads of 3.25 metres, as this was considered sufficient to allow large vehicles to pass without either vehicle having to move sideways towards the edge of the carriageway. However, lane widths as low as 2.75 metres were considered acceptable on economic grounds. Roads with pavements of less than 5.5 metres were regarded as single-lane. No local empirical evidence was available to support these standards.

On narrow single-lane roads, one or both vehicles must have the outer wheels on the shoulder while passing. It was therefore considered desirable that the shoulders of single-lane roads should be capable of carrying traffic in all weather conditions (NAASRA 1980).

Most countries with a rural access roads programme have built roads that are sufficiently wide for two vehicles to pass safely (Hills et al 1984). Thus, roads were recommended to be at least 5 metres wide, although for unpaved roads this could include the shoulder. However, it was recognised that even at this width vehicles would still interact when passing and overtaking, with the amount of interaction depending on the clearance between vehicles.

2.3.2 Capacity

There appears to have been very little work on measuring road capacities in developing countries. However, to some extent, capacity is less important than operating conditions at moderate flows below capacity. All of the capacities quoted are estimates or assumptions.

Nielsen (1978) assumed the capacity of a 4 metre gravel road to be about 600 passenger car units (pcu) per hour increasing to about 1000 pcu per hour for a 6 metre gravel road. A 6 metre paved road would be able to carry about 1900 pcu per hour and a 7 metre paved road about 2400 pcu per hour. The gravel road capacities were estimated from the paved road values on the assumption that dust clouds would result in gaps between vehicles of 100-150 metres.

Paisley (1968) estimated the reduction in capacity for two-lane roads of less than standard width to be as shown in Table 2.4. Also, drivers were considered to steer away from obstructions at the edge of the carriageway, thus reducing the effective width as shown in Table 2.5.

Table 2.4 Estimated reduction in capacity for different road widths

Carriageway width (feet)	24	22	20	18
Percentage of capacity of 24 foot width	100	86	77	70

Table 2.5 Estimated reduction in effective width of carriageway

Clearance from pavement edge to obstruction (feet)	Effective width of 2 lanes (feet)					
	Obstruction one side			Obstruction both sides		
	24	22	20	24	22	20
6	24.0	22.0	20.0	24.0	22.0	20.0
4	23.5	21.5	19.6	23.0	21.0	19.2
2	22.5	20.6	18.8	21.0	19.3	17.5
0	21.0	19.3	17.5	18.0	16.5	15.0

It was for this reason that SATCC (1986) recommended that the minimum horizontal clearance between the carriageway edge and the face of an abutment or pier should generally be 1.5 metres.

Slow moving vehicles can also affect the effective width of the carriageway. In India a flow rate of about 100 bicycles per hour in each direction was found to produce a strip effect which reduced the effective width of the carriageway (CRRI 1982).

2.3.3 Curve widening

Hills *et al* (1984) considered that, on roads that are predominantly single-lane, sufficient width must be provided for two vehicles to be able to pass where the sight distance is restricted to less than that required for stopping or meeting. Thus, the widths on some bends and summits should be increased to at least 5.0 metres.

Oglesby and Altenhofen (1969) proposed that, for roads carrying less than 400 vehicles per day, the requirement of the constant cross-section for the full length of the road segment should be abandoned. Narrower carriageways could be considered for straights than on curves or over crests where sight distance is limited.

Widening may also be needed on the curves of two-lane roads to allow for vehicle tracking. The difference between the arcs of the front and rear wheels on the same side of a vehicle in a turn is referred to as 'cut in'. This depends on the overall wheelbase of the vehicle and Brock (1973) found that for a very long vehicle, cut in could be greater than 4 metres for a 90 degree turn and nearly 7 metres for a 180 degree turn on a 10 metre radius curve.

'Cut out' is the difference of radii of turn between the outer front wheel and the foremost outer corner of the vehicle (Brock 1973). It is only important where the front of the vehicle extends well forward of the front axle or, in the case of an articulated vehicle, where the trailer front extends well forward of the king pin. In the worst case, cut out may be approximately 1 metre on a 10 metre radius turn.

Maximum turning width = vehicle width + cut in + cut out.

In addition to cut out, allowance may be necessary for body side overhang, driver tolerance, and the additional tyre slip which could occur at higher speeds and on wet surfaces (Brock 1973).

When traversing curves vehicles do not always follow the centre of the lane. On low radius curves with small deviations, drivers tend to cut the corner, reducing the maximum lateral acceleration (Johnston 1983, McLean 1983a). On large radius curves, vehicle paths were found to be such that, at some point on the curve, vehicle path curvature was more severe than that of the curve, and this discrepancy between minimum path radius and curve radius increased with increasing curve radius. Thus, the lateral force experienced by a vehicle traversing a curve will generally be greater than that indicated by the standard equation and this discrepancy will tend to be greater for large radius or higher speed curves (McLean 1983a, Glennon and Weaver 1972). The critical point on the curve is the point of maximum friction demand, which generally coincides with either the point of maximum speed or the point of minimum path radius, or both.

Drivers also have difficulty in holding their vehicles in the centre of the lane on curves (AASHO 1965).

Widening is generally more important on narrow roads. The extra width required depends on radius, superelevation and wheelbase. AASHTO (1984) recommended, for practical reasons, that road widening should be in multiples of 6 inches with a minimum of 2 feet.

Recommended increased widths on curves in Malawi are shown in Table 2.6 (Ministry of Works and Supplies 1978).

Table 2.6 Increased width on curves in Malawi

Curve radius (m)	Increase in width (m)
less than 150	1.00
150 - 300	0.75
300 - 400	0.50

In the United Kingdom for carriageways of standard widths, an increase of 0.3 metres per lane is allowed when the radius is less than 150 metres. For carriageways of less than standard width, the widening allowance is 0.6 metres per lane where the radius is less than 150 metres, subject to maximum carriageway widths of 7.9 metres, 11.9 metres and 15.8 metres for two, three and four-lane carriageways respectively. Where the radius is between 150 metres and 300 metres, widening is 0.5 metres per lane subject to a maximum width not greater than the standard. Between radii of 300 metres and 400 metres, the widening allowance is 0.3 metres per lane subject to the same maximum.

In Papua New Guinea (Department of Transport and Civil Aviation 1983) the minimum carriageway width on curves is stated to be determined according to the curve radius, the design vehicle and the traffic flow, as shown in Table 2.7.

Table 2.7 Minimum carriageway width in Papua New Guinea

Design vehicle	ADT	Radius (m)			
		30-60	61-90	91-120	121-150
Small commercial	<300	5.8	5.7	5.6	5.5
	>300	6.5	6.4	6.3	6.2
Medium commercial	<300	6.3	6.1	5.9	5.7
	>300	7.0	6.8	6.6	6.4
Heavy commercial	<300	6.8	6.5	6.2	5.9
	>300	7.5	7.2	6.9	6.6

Widening is generally recommended to be on the inside of the curve and should normally be applied uniformly along the transition curve (AASHTO 1984, Department of Transport 1981a). It has been suggested, probably on the basis of subjective judgement, that failure to provide widening on curves will lead to vehicles, particularly heavy vehicles, straying onto the shoulder. (Ministry of Works and Supplies 1978).

AASHTO (1984) note that widening is costly and very little is gained from a small amount.

2.3.4 Maintenance

AASHTO (1984) have found that the extra cost of 3.7 metres (12 feet) lanes over 3.0 metres (10 feet) lanes is offset to a large extent by reductions in the costs of shoulder and surface maintenance due to lessened wheel concentrations at the edges of pavements. Having a wider carriageway also allows greater flexibility for traffic movement if part of the carriageway is closed for road maintenance purposes.

2.4 DISCUSSION AND RECOMMENDATIONS

Width has a large effect on construction cost and should be minimized subject to minimum operational and safety criteria.

For minimum standards, a carriageway width of 3.0 metres with no shoulders may be used in situations where there is only a small chance of vehicles meeting.

At anything above the very lowest flows, provision must be made for oncoming vehicles to pass. This will require a formation width of 5.0 metres which may include the shoulders. The continuous provision of the 5.0 metre width is not essential and, in difficult terrain, the width may be reduced as long as 5.0 metres is available periodically. The level of provision should be appropriate to the probability of vehicles meeting, sight distances and difficulty of manoeuvering.

McLean (1978d,e), for Australian conditions, found no economic justification for a road wider than 5.0 metres for traffic volumes in the region of 100 vehicles per day. Oglesby and Altenhofen (1969) found similar results for the USA. Their results also suggested that 5 metres (approximately 16 feet) would be sufficient for up to 400 vehicles per day where items such as the cost of capital were high, as would probably be the case in developing countries. It is therefore recommended that a basic carriageway width of 5.0 metres would be adequate for traffic flows of up to 400 vehicles per day. With this width, clear roadmarkings will be necessary on paved roads to enable drivers to determine clearances for oncoming vehicles and provision will be necessary for the outer wheels of trucks occasionally running onto the shoulder.

At higher flows, meeting and overtaking will become more common and a narrow carriageway will result in excessive shoulder wear. Carriageway widths of 5.5 metres will allow two commercial vehicles to pass completely within the carriageway although there will be movement towards the edge (Department of Works 1985). A carriageway width of 6.5 metres is necessary to allow commercial vehicles to pass each other while remaining centred in their lanes. The increased pavement width should reduce the amount of maintenance required to the shoulders and to the pavement edge. The literature does not support a critical value of flow for transferring from a 5.5 to a 6.5 metre carriageway. There is also little evidence to support the need for widths in excess of this although studies in the UK showed safety and operational benefits by increasing widths up to 10 metres. However, shoulders are uncommon in the UK, and the application of such a finding to developing countries is dubious.

Widening will be essential on curves on narrow single-lane roads to ensure that vehicles can negotiate the curve within the carriageway width. Also, widening to two-lanes is essential wherever sight distance is so restricted that meeting vehicles would not be able to stop before colliding.

Widening on curves is highly desirable on two-lane roads, particularly at the lower widths recommended above. Of the increases in width reviewed, the ones which appear to be most appropriate are those of the Ministry of Works and Supplies (1978) which were shown in Table 2.6.

The right of way must be adequate for the construction of the road and subsequent maintenance. Any buildings erected after construction of the road should be positioned to allow for possible road widening (Odier et al 1971).

In some areas, provision may need to be made for non-motorised transport and/or the driving of livestock, although there is little factual evidence to support particular recommendations. If there are large numbers of animal drawn carts or bicycles, it will be advisable to segregate them from the motorised traffic. The best way of providing this segregation would be to build a shoulder to the road. A shoulder width of 1.0 metre would be just sufficient for bicycles and would considerably reduce the obstruction caused by animal drawn carts. However, the shoulder may need to be of the same surface standard as the carriageway to ensure its use by non-motorized vehicles.

Most capacity estimation is based on developed country measurements. Its estimation in developing countries is uncertain and very dependent on the mix and performance of road users.

3. SHOULDERS

3.1 BASIC CONCEPTS

Shoulders can allow a certain amount of flexibility of traffic operation (Armour and McLean 1983). They provide space for overtaking; they allow moving vehicles to pass vehicles disabled in the traffic lane or vehicles waiting to turn across the oppposing flow; and they allow for traffic diversion at roadworks (Armour 1984b). In all these cases, a shoulder is used as a traffic lane, and must, therefore, be adequate structurally and have adequate horizontal and vertical clearances (Mathews and MacLean 1977). Wide shoulders also create a sense of openness and hence add to driver comfort as well as improving sight distances on horizontal curves (NAASRA 1980).

From a structural point of view, shoulders provide lateral support for the carriageway. A paved surface which is unsupported at the edge will deteriorate, the amount of deterioration depending on the traffic level and the closeness of the wheel loading to the edge. Provision of a paved shoulder increases the distance between traffic and the edge of the structure and hence reduces the amount of edge deterioration. A further requirement of shoulders is to drain the pavement and, for this purpose, it is necessary to maintain a rate of flow of water across the shoulder that matches the flow across the pavement (NITRR 1984).

3.2 RESEARCH FINDINGS

3.2.1 Safety

Zeeger et al (1981) found that accidents between vehicles travelling in opposing directions decreased as shoulder width increased. This empirical finding may be interpreted as suggesting that the provision of a shoulder, or widening of an existing shoulder, increases the effective width of the running surface and encourages drivers to travel closer to the edge of the pavement, thereby increasing the lateral clearance between opposing vehicles.

The provision of a shoulder may also be considered to provide a recovery area for errant vehicles (Armour and McLean 1983). Thus, it would be expected that the number of vehicles which run off the road would decrease as shoulder width increased, and this has been found to be the case by Zeeger et al (1981). Armour (1984a) also found that paved shoulders were considerably better than gravel shoulders from the point of view of the driver being able to regain control.

Other potential safety problems, such as overtaking on wide paved shoulders or parked vehicles on narrow shoulders, did not appear to add significantly to the accident risk (Armour 1984a). Zeeger et al (1981) found that the relationship between total reduction in accidents and increases in shoulder width was as shown in Table 3.1.

Table 3.1: Shoulder width and accidents

Shoulder width (m)		Reduction in accidents (%)
Before	After	
None	0.3-0.9	6
None	1.2-1.8	15
None	2.1-2.7	21
0.3-0.9	1.2-1.8	10
0.3-0.9	2.1-2.7	16
1.2-1.8	2.1-2.7	8

Armour (1984a) found that roads with paved shoulders had a lower accident rate than roads with unpaved shoulders even when the effects of gradient and curvature were removed. However, as roads with paved shoulders will be in a generally better condition than roads with unpaved shoulders, it may be expected that they would have lower accident rates for reasons other than shoulder construction.

From an economic viewpoint, accident costs on low volume rural roads have been found to be much lower than construction and operating costs (Oglesby and Altenhofen 1969). The authors found, for example, that if all non-intersection accidents are prevented on a mile of straight road carrying 400 vehicles per day, the reduction in direct accident costs might be about $330 per year. Providing wide shoulders offers an annual accident cost reduction of only $10.

Middleton (1976) found that unintentional shoulder incursions and the damage they produce may be reduced by edge lining the pavement.

3.2.2 Structural performance

Armour and McLean (1983) found that paved shoulders on paved roads offered the best structural solution. They also determined that the use of gravel shoulders minimised construction costs, but increased maintenance costs. Gravel shoulders were also discovered to cause problems at the seal edge and moisture entry through permeable shoulder material weakened the pavement structure.

3.3 STANDARDS IN USE

3.3.1 Traffic Operations

Traffic operations in developing countries are considerably different to those found in developed countries. In developing countries, pedestrian and bicycle use of rural roads often represents a significant proportion of total traffic and even of the goods being moved (Robinson 1981). In these circumstances, the shoulder serves a similar function to a footpath in urban areas, enabling spatial separation between road user types (McLean 1978a). As cyclists and drivers of other slow vehicles would rather travel on a smooth running surface than on an uneven shoulder, it has been suggested that shoulders are built to the same surface standard as the carriageway (CRRI 1982).

In most cases, vehicles will be forced to move onto the shoulder only occasionally, although shoulder incursions will be more common on single lane roads, where, on passing, at least one vehicle must use a shoulder. Thus, on single lane roads it has been recommended that shoulders have a width of at least 1.5 metres (NAASRA 1980), and be capable of carrying traffic in all weather conditions.

Wide shoulders enable a stopped vehicle to stand clear of the traffic lanes (NAASRA 1980). A stopped vehicle can be adequately accommodated by a shoulder which is 3.0 metres wide and there is no merit in adopting a shoulder width greater than this (NITRR 1984). The need for shoulders to act as 'stand clear' areas for emergency stops is small. Most vehicle stops are discretionary and it may not be necessary to provide continuous wide shoulders to meet the needs of these vehicles (Armour and McLean 1983, Cheeseman and Voss 1967). NAASRA (1980) also recommended edge lines for fully or partially paved shoulders, since, otherwise, the problem of edge damage is merely transferred to the new edge.

On the outside of the shoulder, there will be a verge, an embankment or a drainage facility. This will generally be constructed of soil and should ensure that the shoulder is adequately drained. However, the slopes should not be so steep as to increase unacceptably the danger of vehicles leaving the road. (Department of Works 1985, AASHO 1965).

3.3.2 Stopping places

On roads without shoulders or with narrow shoulders, it is necessary to provide periodic lay-bys. Since most stops are voluntary (see above), it is not necessary to have a continuous stopping place. The recommended space between lay-bys varies quite considerably according to road type and country. Cheeseman and Voss (1967) recommend stopping/resting areas every 55 kilometres (35 miles).

NAASRA (1980) adopt a flexible approach. If minimum shoulder widths are used, full width stopping places should be provided at intervals where their provision can be carried out inexpensively. Paisley (1968) recommended that, on rural and important primary distributor roads, lay-bys should be placed at regular intervals on each side of the road. The spacing should be frequent, about two per mile (each side) on heavily trafficked roads reducing to 3 to 5 miles apart (each side) on roads expected to carry under 6000 pcu per day. The Ministry of Transport et al (1968) recommended two lengths of lay-by per mile, each about 100 yards long, on each side of three-lane and dual carriageway roads.

3.4 DISCUSSION AND RECOMMENDATIONS

Shoulders are essential on single lane roads where there is a regular possibility of vehicles meeting. Their width should be sufficient to ensure an adequate total width for the passing of vehicles, and 1.5 metres has been recommended. The levels of traffic generally will not be sufficient for paved roads or shoulders to be economic. At very low flows, passing places will be sufficient to service the requirement for meeting vehicles.

On two lane roads, the rate of accidents between opposing vehicles decreases with increasing shoulder width, and is lower when the shoulders are paved. Overtaking on wide paved shoulders has not been found to be a significant cause of accidents, although the effects will be related to carriageway width, flow and design operating speed. As paved shoulders are also advantageous for structural and maintenance reasons, it is recommended that paved roads should have paved shoulders. For lower levels of flow, it is considered acceptable for the running surfaces to be narrow, allowing occasional incursions onto the shoulder for some passing manouevres. To maintain effective operational performance, the boundary between carriageway and shoulder should be made obvious by either using a different coloured surfacing for the shoulder, or by edge lining.

Shoulders should be at least 1.0 metre wide as this is sufficient to give structural support and produce most of the benefits to traffic operation by increasing the effective width of the road.

In developing countries, rural shoulders often serve a similar function to footpaths in urban areas, enabling spatial separation between road user types. It is recommended that shoulders should normally be paved to provide a smooth surface and to be as attractive to users as the carriageway. The width of shoulder should reflect the types and amount of traffic. Provided there is adequate edge delineation, the occasional use of part of the shoulder for overtaking is considered acceptable.

Stopping places should be provided periodically to enable stopped vehicles to park off the road. Their frequency and location should be related to cost and demand.

4. CROSSFALL

4.1 BASIC CONCEPTS

The purpose of crossfall is to shed water from the carriageway and allow it to drain into side ditches. This helps to prevent water from entering and weakening the pavement, whether or not the surface is paved.

4.2 RESEARCH FINDINGS

There is little published research relating to crossfall, but unpublished work from TRRL suggests that, for unpaved roads, crossfalls in excess of 4 per cent give the best road performance. Performance of the road improves further with steeper crossfalls, but erosion can start to be a problem with some surfacing materials when values exceed about 6 per cent.

4.3 STANDARDS IN USE

4.3.1 Carriageway Crossfall

There are two main considerations when designing the crossfall or camber of a road. Firstly, the slope must be adequate to provide rapid run-off for water falling on the surface. Secondly, the slope must not be so excessive as to become a hazard to traffic (International Bank for Reconstruction and Development 1978). The minimum crossfall required is a function of surface texture and the accuracy to which the surface can be constructed. For a given crossfall, the smoother the surface the more efficient it is in shedding water (Department of Transport 1984).

NITRR (1984) have reported that crossfalls steeper than 3 per cent introduce operational problems, both in driving and in increasing wear of vehicle components. However, the majority of other countries consider this limit too low and Choudhury (1980) found no serious operational problems until the transverse slope exceeded 7 per cent.

A selection of values used or recommended by various countries or individuals are given below in Table 4.1.

4.3.2 Shoulder Crossfall

The crossfall of the shoulder depends to a large extent on the crossfall of the carriageway, and must be sufficient to ensure a rate of flow that at least matches the flow across the carriageway (NITRR 1984). This drainage requirement is reflected in the international values recommended in Table 4.1.

Table 4.1: Comparison of crossfall values

Country	Crossfall % (Shoulder Crossfall %) Paved	Gravel	Earth	Reference
Australia	2.0-3.0 (3.0-4.0)	4.0 (4.0-5.0)	5.0 (6.0)	(NAASRA 1980)
Japan	1.5-2.0	3.0-5.0	-	(Ministry of Construction 1981)
Malawi	3.0 (6.0)	4.0 (6.0)	-	(Ministry of Works and Supplies 1978)
Nigeria	1.5-3.0 (4.0)	2.5-3.0 (4.0)	-	(Federal Ministry of Works and Highways 1972)
Papua New Guinea	3.0	4.0	5.0	(Department of Works 1985)
South Africa	2.0-3.0 (4.0-5.0)		-	(NITRR 1984)
UK	2.5	-	-	(Department of Transport 1981a)
UK	3.0	4.0	5.0	(Department of Transport 1984)
USA	1.5-3.0 (2.0-6.0)	2.0-6.0 (4.0-6.0)	- (8.0)	(AASHTO 1984)
Developing countries	2.0-3.0 (4.0-5.0)	-	-	(Cron 1978)
Developing countries	3.0	4.0-6.0	-	(Hills *et al* 1984)

4.3.3 Adverse crossfall

With normal crossfall the road is crowned in the middle. A vehicle travelling along a straight section of road with a normal 2.5 per cent crossfall to the outside channel is subject to a lateral acceleration of 0.025 g by virtue of the crossfall (Department of Transport 1984). To counteract this, drivers have to steer towards the centre of the road. With progressive curvature against the crossfall, the resulting sideways force to be obtained by road tyre friction will increase rapidly.

When curvature is sufficient to render driving conditions uncomfortable, the adverse crossfall should be changed to the opposite, favourable sense. The critical radii of curvature for the elimination of adverse crossfall in the UK are given in Table 4.2 (Department of Transport 1981a). On sections of road with radii greater than those shown, the crossfall should be 2.5 per cent from the centre of the single carriageway. For horizontal curves, adverse crossfall should be replaced by favourable crossfall of 2.5 per cent, when the radius is less than that shown.

On reverse crossfall, outer shoulders should be sloped upward at about the same or a lesser rate than the slope of the running surface. Any shoulder that is sloped towards a paved surface should also be paved to prevent loose material being washed over the surface (AASHTO 1984).

Table 4.2: Radius and adverse crossfall

Design speed (km/h)	Minimum radius without elimination of adverse crossfall (m)
120	2880
100	2040
85	1440
70	1020
60	720
50	510

4.4 DISCUSSION AND RECOMMENDATIONS

Values of crossfall recommended by various authorities range from 1.5 to 3.0 per cent for paved roads. It is recommended that the normal crossfall on a paved road should be 3 per cent, as the higher value conforms to the most frequent recommendation for developing countries. Such a crossfall will be more likely to satisfy high intensity rainfall, and will allow for drainage even with some surface deformation. For gravel roads, a higher crossfall is necessary and 4 per cent would seem to provide a satisfactory minimum for working practice. 5 per cent is used by most countries for earth roads, although exact values will be dependent on local materials and conditions. The crossfall of the shoulder should be sufficient to drain the paved surface. Shoulders constructed of the same material as the running surface may have the same cross slope, whilst gravel or earth shoulders on a paved road should be 2 per cent steeper.

C. SIGHT DISTANCE

5. STOPPING SIGHT DISTANCE

5.1 BASIC CONCEPTS

5.1.1 Definitions

In order to be able to travel safely along a road, a driver must be able to see a sufficient distance ahead. The required sight distance will depend on the speed of the vehicle and the manoeuvre being undertaken. If safety is to be built into the road, then sufficient sight distance must always be available to enable a vehicle travelling at the design speed to stop before reaching an object in its path (AASHTO 1984).

Stopping sight distance is the sum of two distances: the distance traversed by a vehicle from the instant the driver sights an object requiring him to stop to the instant the brakes are applied, and the distance required to stop after braking begins (AASHTO 1984). The distance depends on the assumed height above the road of both the driver's eye and the object which necessitates the stop.

5.1.2 Effect of different vehicle types

Trucks require a greater distance to stop from a specific speed (AASHO 1965). However, truck drivers are approximately one metre higher above the road than car drivers and can thus see further. Also, trucks travel slower than cars in most situations. Specific stopping sight distances for trucks are not required except, perhaps, with horizontal sight distance restrictions at the end of a long downgrade (Cleveland et al 1985).

The occurrence of very slow moving vehicles in the traffic stream increases the incidence of unexpected stops (or near stops) for other vehicles (McLean 1978e). This is not usually a problem on crests due to the height of heavy vehicles. It does become a problem where sight distance is limited by horizontal curvature. The provision of horizontal stopping sight distance for the likely operating speed of the faster vehicle becomes important in such circumstances.

5.1.3 Manoeuvre sight distance

In many cases, a driver is more likely to avoid a hazard by lateral manoeuvring than by bringing his vehicle to a halt (NAASRA 1980). It may be more appropriate to design for a sight distance which allows a driver to manoeuvre around a hazard (McLean 1978a,b,c and d). Therefore, an alternative to stopping sight distance in difficult cases is to ensure:

a) That the carriageway is wide enough to provide a reasonable space for evasive action;

b) That the driver can perceive a hazard in sufficient time to take evasive action.

5.1.4 Meeting sight distance

Meeting sight distance ensures that two vehicles approaching each other at the design speed in the same traffic lane can stop before they collide (Valentine 1978). This is most important in the design of single-lane roads.

5.1.5 Design considerations

Stopping sight distance should be available at every point on a road. Suitable sight distance may be achieved by lengthening horizontal and vertical curves, widening verges and benching to allow visibility outside the road width.

Stopping sight distance can only occur naturally within the highway boundary at very low design speeds. Verge widening may need to be carried out to increase the sight distance artificially beyond that available within the natural boundary of the road (Simpson and Kerman 1982).

The most critical sight distance situation for a curve is when both the vehicle and the object are located on the inside lane of the curve (International Bank for Reconstruction and Development 1978). If sight distance requirements are satisfied in this case, there will always be adequate visibility for a vehicle on the outside of the curve.

5.2 RESEARCH FINDINGS

5.2.1 Object height

Ideally, the driver should be able to see the road surface but this usually leads to high cost because of the length of vertical curves. The vertical curve length required has been found to diminish very rapidly as the height of object is increased from 0 to 6 inches (AASHO 1965). For greater heights the reduction in length of vertical curve is progressively less significant.

Going from 0 to 0.15 metres	reduces length of vertical curve by	47 per cent
" " 0 to 0.30 metres	" " " " " "	57 per cent
" " 0 to 0.45 metres	" " " " " "	63 per cent

Substantial economy in construction is therefore obtained by using a 0.15 metre object instead of the zero value, yet the ability to see or appraise a hazardous situation on the road is often not materially altered.

There is little additional economy from using 0.30-0.45 metres, but this significantly decreases the driver's ability to see objects on the road.

5.2.2 Eye height

AASHO (1965) used an eye height of 1.143 metres (3.75 feet), but by 1984 this had been reduced to 1.067 metres (3.5 feet) (AASHTO 1984) and it has been recommended by Cleveland et al (1985) that this be reduced further to 1.016 metres (40 inches). This decrease in eye height is the result of an increased use of smaller cars in the USA (Pilkington 1977). Decreased eye height will result in reduced sight distances.

Haslegrave (1979), in the UK, found mean eye height to be 1.141 metres. Eye heights ranged from a minimum of 0.872 metres to a maximum of 1.283 metres. There had been a considerable reduction in eye heights since a previous study in 1962 when mean eye height was 1.239 metres.

5.2.3 Stopping requirements

The stopping period comprises a reaction time and a braking time. Most field studies have shown that driver perception and reaction times vary from 0.5 to 1.7 seconds, although at high speeds, the values were less than those at low speeds. This may be because fast drivers are usually more alert (NAASRA 1980).

5.2.4 Effect of sight distance on speed and accidents

The possibility of curtailed sight distances concealing a hazard is perceived as remote, so drivers do not generally adjust their speed to a level commensurate with sight distance restrictions (McLean 1979). Available sight distance appears to have only a marginal effect on speed (McLean 1978e, 1981). Increasing horizontal curve radius to improve sight distance may merely serve to increase operating speeds, so that the sight distance remains inadequate for the speeds that prevail. While this applies to all forms of sight distance restriction, it is particularly true for sight distances restricted by crest vertical curves located on tangent sections (McLean 1979).

Oglesby and Altenhofen (1969) have confirmed that driver behaviour, at least where speeds are below 80 km/h, does not conform to safe practices as measured by the design standards for sight distance. They also found that, although drivers did not slown down when vertical sight distance was impaired, few accidents were involved. Curvature has a significant influence on driver speed, but restrictions on sight distance have far less effect. Thus, restrictions on horizontal sight distance have more effect on drivers than restrictions on vertical sight distance.

Evidence is strong that accidents occur where sharp curves appear occasionally in relatively straight alignments, offering an additional argument for increasing curve radius (Oglesby and Altenhofen 1969). Similar advantages do not seem to occur where vertical alignment is improved, since drivers do not slow down nor are there many accidents where vertical sight distance is impaired. Evidence from the UK demonstrates that departures from stopping sight distance standards within one or two design speed steps do not have a significant impact on safety (Simpson and Kerman 1982).

5.3 STANDARDS IN USE

5.3.1 Object height

A number of different object heights are used in different countries (Table 5.1).

Table 5.1: Object Heights

Country	Object Height (metres)	Source
United States	0.15 (6 inches)	(AASHTO 1984)
Australia	0 - 0.2	(NAASRA 1980)
U.K.	0.26 - 2.0	(Department of Transport 1981a)
Southern African States	0.1	(SATCC 1986)

On causeways and floodways where, due to scouring, holes may occur in the road pavement, object heights of zero are adopted in Papua New Guinea by the Department of Works (1985).

5.3.2 Eye height

Eye heights used for design purposes in various countries are very similar and the trend has been for these to reduce over time. The 5th percentile value is normally used for design purposes.

In the UK, design eye heights of 1.05 metres and 2.00 metres are used to produce visibility envelopes (Department of Transport 1981a). As noted earlier, AASHTO (1984) use a value of 1.067 metres (3.5 feet). In Australia (NAASRA 1980), a design eye height of 1.15 metres is used for car drivers, while 1.8 metres is used for commercial vehicle drivers. In the Southern African States (SATCC 1986), a driver eye height of 1.10 metres is used, and an eye height of 1.05 metres is used in South Africa (NITRR 1984).

5.3.3 Stopping requirements

For safety, AASHTO (1984) recommend that a reaction time should be used which is sufficient for most operators. With an average reaction time of 0.5 second, 1.0 second has been recommended for design. The perception time used should also be sufficiently large to cover nearly all drivers under most highway conditions and a value of 1.5 seconds has been taken. The resulting total perception and brake reaction time as a basis of design is recommended by AASHTO (1984) to be 2.5 seconds.

NAASRA (1980) assumed a similar equivalent standard reaction time of 2.5 seconds, but considered that it could be reduced to 2.0 seconds or 1.5 seconds in constrained situations in which the driver may be assumed to be more alert. The Department of Transport (1981a) uses a value of 2.0 seconds which is based on a field study and includes a limited safety margin. Jones (1961) assumed a higher perception and reaction time of 3.0 seconds.

The time spent and distance travelled during actual braking depends on the tyres, road surface, and gradient, and the initial speed of the vehicle (AASHTO 1984, NAASRA 1980, Department of Transport 1981a).

In general, the friction available between the road and tyres decreases as the speed of the vehicle increases, with the available friction changing throughout the braking manoeuvre. Thus, the average rate of deceleration will be greater from low speeds than from high speeds. The coefficient of friction also depends on whether the road is wet or dry. Those values recommended for design are as shown in Table 5.2.

Table 5.2: Longitudinal friction values for design

AASHTO[1]		NAASRA[2]		Department of Transport[3]	
Design Speed (mph)	f	Design Speed (km/h)	f	Design Speed (km/h)	f
20	0.40	50	0.65	50	0.24
25	0.38	60	0.60	60	0.24
30	0.35	70	0.55	70	0.24
35	0.34	80	0.50	85	0.24
40	0.32	90	0.45	100	0.24
45	0.31	100	0.40	120	0.24
50	0.30	110	0.37		
55	0.30	120	0.35		
60	0.29	130	0.33		
65	0.29				
70	0.28				

1. AASHTO (1984) values are generally conservative.
2. NAASRA (1980) are values based on tests.
3. Department of Transport (1981a) values are 0.24 for desirable minimum and 0.36 for absolute minimum stopping sight distance.

The braking distance on a flat road is determined by the formula (AASHTO 1984): $d = \frac{V^2}{30f}$

Where: d = braking distance (feet).
V = initial speed (mph)
f = coefficient of friction between tyres and road surface

Normally, design criteria are based on wet pavements as this is the most critical case. Minimum stopping sight distance requirements used in different countries are shown in Tables 5.3 to 5.8.

Table 5.3: Stopping sight distances used in USA (AASHTO 1984)

Design Speed (mph)	Assumed for conditions (mph)	Perception & brake reactn. time distance (sec)	(feet)	Braking distance on level (feet)	Stopping sight distance Computed (feet)	Rounded for design (feet)
30	28	2.5	103	75	177	200
40	36	2.5	132	135	267	275
50	44	2.5	161	215	376	400
60	52	2.5	191	310	501	525
65	55	2.5	202	347	549	550
70	58	2.5	213	401	613	625

These values should encompass nearly all significant pavement types and worn tyres as well as new tyres, and all types of treads and tyre composition.

Table 5.4: Stopping sight distances used in UK (Department of Transport 1981a)

Design speed (km/h)	50	60	70	85	100	120
Desirable minimum SSD (m)	70	90	120	160	215	295
Absolute minimum SSD (m)	50	70	90	120	160	215

Table 5.5: Stopping sight distances used in Malawi (Ministry of Works and Supplies 1978)

Design speed (mph)	30	40	50	60
Safe stopping sight distance(m)	60	85	105	145

Table 5.6: Stopping sight distances used in Japan (Ministry of Construction 1981)

Design speed (km/h)	Minimum sight distance (m)
20	20
30	30
40	40
50	55
60	75
80	110
100	160
120	210

Table 5.7: Stopping sight distances used in South Africa (NITRR 1984) (level roads)

Design speed (km/h)	Stopping sight distance (m)
40	50
50	65
60	80
70	95
80	115
90	135
100	155
110	180
120	210
130	230
140	255

Table 5.8: Stopping sight distances used in Sweden (Swedish National Road Administration 1982)

Design speed (km/h)	30	50	70	90	110
Stopping Sight Distance (m)	35	70	120	165	195
Exceptional Stopping Sight Distance (m)	25	50	85	135	195

Several countries recommend different stopping sight distances on gradients than on level roads as vehicles take a greater distance to stop on downgrades than on upgrades. The SATCC (1986) countries use the values shown in Table 5.9.

Table 5.9: Stopping sight distance and gradient recommended by SATCC

Design speed (km/h)	Assumed speed for upgrades (km/h)	Minimum stopping sight distance (m) Percentage gradient				
		+6%	+3%	0	-3%	-6%
40	35	44	44	44	44	47
60	55	74	76	79	83	86
80	70	116	122	126	133	140
100	85	173	178	185	197	210
120	100	242	250	260	279	301

The United States estimate gradient effects as shown in Table 5.10 (AASHTO 1984).

Table 5.10: Stopping sight distance and gradient (USA)

Increase for downgrades				Decrease for upgrades			
Design speed (mph)	Correction in stopping distance (ft)			Assumed speed for condition (mph)	Correction in stopping distance (ft)		
	3%	6%	9%		3%	6%	9%
30	10	20	30	28	-	10	20
45	20	40	70	36	10	20	30
50	30	70	-	44	20	30	-
60	50	110	-	52	30	50	-
65	60	130	-	55	30	60	-
70	70	160	-	58	40	70	-

The AASHTO (1984) stopping distances are lower for all gradients than the SATCC (1986) stopping distances, although the changes in stopping distance with gradient are similar.

Drivers seemingly have difficulty in judging the additional distance required for stopping on gradients, and it has been suggested as a safety measure that intersections should not be located on gradients steeper than 3 per cent (NITRR 1984). If it is not possible to align all the legs of an intersection to a gradient of 3 per cent or less, the through road could have a steeper gradient, because vehicles on the intersecting road have to yield or stop, whereas through vehicles would only have to do so occasionally.

The required stopping sight distance may also depend on the road surface. A paved road may allow greater friction to be developed between the tyre and the road than on an unpaved road and hence a lower braking distance as indicated in the standards of Papua New Guinea (Department of Works 1985) (Table 5.11).

Table 5.11: Stopping sight distance and surface type

Design speed (km/h)	Stopping sight distance	
	Gravel	Paved
25	25	-
30	30	30
40	45	40
50	60	55
60	80	70
70	105	90
80	135	115
100	-	170

5.3.4 Manoeuvre sight distance

Values of restricted stopping sight distance and manoeuvre sight distance used by NAASRA (1980) for design are given below:-

Table 5.12: Manouevre sight distance

Design speed of horizontal alignment (km/h)	Reaction time (sec)	Restricted stopping sight distance (m) (a)	Derived manoeuvre time (s)	Manoeuvre sight distance (m) (b)
50	2.0	45	3.2	45
	1.5	35	2.5	
60	2.0	60	3.6	60
	1.5	50	3.0	
70	2.0	75	3.9	75
	1.5	65	3.3	
80	2.0	95	4.3	95
90	2.0	120	4.8	120
100	2.0	(155)	5.6	155

a) Use where normal stopping sight distance is difficult or costly to achieve on consistent alignment sections below 100 km/h design speed when driver can be assumed to be alert.
b) Use in similar situations on consistent alignments or isolated features up to 100 km/h when carriageway includes sufficient manoeuvre width (shoulders not less than 1.5 metres and preferably 2.5 metres).

5.3.5 Meeting sight distance

SATCC (1986) recommend that meeting sight distance should generally be provided for all roads with carriageway widths of less than 5 metres. In addition, Hills et al (1984) considered that a 5 metre width would be adequate when meeting sight distance was not available.

On single-lane roads carrying more than 50 vehicles per day, Papua New Guinea Department of Works (1985) recommend that the minimum sight distance (measured between two points 1.15 metres above the road surface) should not be less than twice the stopping distance of a vehicle travelling at the design speed. Where sight distance is less than twice the stopping distance or where horizontal curvature is severe, they recommend that the width of formation should be increased to 7.0 metres, ie a two-lane pavement construction.

Valentine (1978) estimated meeting sight distances to be as shown in Table 5.13.

Table 5.13: Meeting sight distance

Design speed (mph)	Assumed speed (mph)	Single vehicle stopping distance(ft)	Two vehicle stopping distance(ft)	Stopping distance (Rounded for design)(ft)
20	20	108	216	225
25	25	140	280	300
30	28	176	352	375
35	33	228	456	475
40	36	262	524	550
45	40	313	626	625
50	44	369	738	750
55	48	432	864	850
60	52	492	984	975

Another situation in which meeting sight distance is recommended (Choudhury 1980), is when drivers attempt to overtake when the overtaking sight distance is unavailable. If the meeting sight distance is available, the overtaking manoeuvre can be attempted with reasonable safety.

Often a barrier sight distance is adopted. This is the limit below which overtaking is legally prohibited, and at which two opposing vehicles travelling in the same lane should be able to come to a standstill before impact. It should equal twice the stopping distance (NITRR 1984) as given in Table 5.14.

Table 5.14: Barrier sight distance

Design Speed (km/h)	Barrier Sight Distance (m)
50	150
60	180
80	250
100	300
120	400

5.4 DISCUSSION AND RECOMMENDATIONS

Stopping sight distance depends on object height, eye height, driver reaction time, speed and available friction between the tyre and road surface. It has been found that drivers do not adjust their speeds according to available sight distance.

Very small objects on the road surface are unlikely to be a significant hazard except where surface water may be present. A range of heights have been selected for objects on the highway for use within different standards. An object height of 0.2 metres is recommended as being within this range, as well as representing the height of an object which would be likely to cause significant hazard.

An eye height of 1.05 metres is recommended. This is representative of 95th percentile values found in practice. Truck drivers have eye height of just over 2.0 metres and the extra vision thus provided counteracts any additional stopping sight distance as a result of poorer braking performance in most circumstances.

A reaction time of 2.0 seconds is recommended. This is the value recommended by the Department of Transport (1981a), although it is slightly lower than the standard values used by NAASRA (1980) and AASHTO (1984). All have considerable in-built safety margins for most drivers and conditions.

The coefficients of longitudinal friction recommended lie between those used by NAASRA (1980) and Department of Transport (1981a) as given in Table 5.15. They include considerable safety margins, even in wet conditions. The coefficients of friction on unpaved roads may be slightly lower than on paved roads, but this difference reduces at lower speeds and it was not considered necessary to develop separate stopping sight distances. The values of stopping sight distance shown in Table 5.15 are slightly lower than those recommended for Papua New Guinea, although higher than those from developed countries. They are similar to those for Japan. In some countries, specific adjustments are made according to gradient.

On single-lane roads, the sight distance must be sufficient for two vehicles approaching each other at the design speed to stop before they collide, i.e. twice the standard stopping sight distance. However, in this case, the object is an approaching vehicle with an assumed height of 1.05 metres rather than 0.2 metres.

Table 5.15: Coefficients of longitudinal friction and sight distances

Design speed km/h	Coefficient of longitudinal friction (F)	Stopping sight distance (m)	Stopping sight distance (m) (single lane roads)
30	0.60	23	46
40	0.55	34	68
50	0.50	48	96
60	0.47	64	128
70	0.43	84	N/A
85	0.40	119	N/A
100	0.37	162	N/A
120	0.35	229	N/A

The Australian standards have introduced a manoeuvre sight distance within which a driver may take evasive action, rather than come to a stop. This has not been recommended for use in developing countries, as there are often restricted running widths, as well as pedestrian activities which may govern the practical opportunities to manoeuvre.

6. **OVERTAKING SIGHT DISTANCE**

6.1 BASIC CONCEPTS

6.1.1 Overtaking and congestion

Drivers in a traffic stream are each attempting to travel at their free speeds. The extent to which this can be achieved will depend on a series of factors which include level of flow, traffic characteristics and highway geometry. Speed will be particularly affected by the number of safe overtaking opportunities. As traffic flows increase, the economic effects of congestion will become greater. However, at all levels of flow, sufficient and adequate overtaking opportunities are required if following drivers are not to make injudicious judgements with an associated accident risk. The positive advice given by signs and road markings can play an important role.

The longer the available sight distance, and the bigger the gaps in the opposing flow, the greater the proportion of the vehicle population that will be able to overtake. However, not all overtaking manoeuvres are completed in the same time or distance, and even those on the same stretch of road will show considerable variations. Time and distance required to complete a manoeuvre depends on a number of factors including the speed and length of the overtaken vehicle, and the speed and acceleration capabilities of the overtaking vehicle. In places where overtaking opportunities are restricted, the traffic flow will be characterised by bunches of vehicles with stretches of clear road between (Hoban 1984c). The amount of bunching increases very substantially with an increase in traffic volume on two-lane rural roads.

6.1.2 Design considerations

There will normally be some sections of road, such as on bends and summit curves, where there is insufficient sight distance for safe overtaking; these may be termed "non-overtaking" sections. There will be other sections where sight distances will be sufficiently long to allow virtually all vehicles the opportunity to overtake; these may be termed "overtaking" sections. There are also sections of intermediate sight distance which will allow some drivers to overtake (Department of Transport 1984). The longer the sight distance, the greater the proportion of the vehicle population that will be able to overtake and long sight distances are therefore very desirable (Department of Transport 1984). However, to provide sight distances long enough for overtaking on the entire length of the road would be uneconomic, and it is usually better to designate some lengths as "no-overtaking" sections which are designed solely for stopping sight distance (eg crest curves). Wherever possible, "no-overtaking" sections should be followed by sections where there is sufficient sight distance to allow overtaking.

The use of shorter curves will allow intermediate straights to be longer; this will increase the available sight distances for overtaking (Lyby 1977, Hills et al 1984).

In flat terrain, the use of a curvilinear alignment may be possible. Curves could be large and hence not restrict overtaking, but produce some of the advantages of reduced headlight glare and recognition of the speed of approach of opposing vehicles (NAASRA 1980).

With increasing traffic flow, overtaking becomes more difficult, and the expense involved in ensuring long passing sight distances is no longer justified (Trapp 1977). It is more economic and effective to increase overtaking opportunities by other means. One of the most economical ways of doing this is to provide an auxiliary lane for the exclusive use of traffic in one direction (Department of Transport 1985). This may take the form of a dual carriageway, a climbing lane, or an overtaking lane in level terrain. This solution ensures that sight distance restrictions and the opposing flow do not inhibit overtaking manoeuvres.

Troutbeck (1980) identified two overtaking sight distances as being of interest:

1) Overtaking establishment sight distance;
   This is the sight distance required for a driver to initiate an overtaking manoeuvre, and

2) Overtaking continuation sight distance;
This is the sight distance required for a driver to be able safely to complete or abort an overtaking manoeuvre assuming that an oncoming vehicle becomes apparent at the most critical time, the point of no return.

6.2 RESEARCH FINDINGS

6.2.1 Overtaking speeds and times

Studies in the UK (Simpson and Kerman 1982, Department of Transport 1984) have shown that most overtaking manoeuvres take between 3 and 15 seconds to be completed and that:-

50 per cent of overtaking manoeuvres take less than 7 seconds
85 per cent of overtaking manoeuvres take less than 10 seconds
99 per cent of overtaking manoeuvres take less than 14 seconds.

Troutbeck (1981) found that an increase in length of the overtaken vehicle from 5 to 16 metres resulted in an increase in overtaking time of about 18 per cent. An increase in length from 16 to 20 metres resulted in a further increase in overtaking time of about 5 per cent. Further, overtaking times were found to increase by about 14 per cent for each 16 km/h (10 mph) increase in the speed of the overtaken vehicle.

Earlier work by Troutbeck (1980) found that vehicles travelling at slower speeds were 'caught up' more frequently than vehicles travelling faster. Eighty five percent of overtakings were of vehicles travelling at speeds of less than $V + 0.24\sigma$, where V is the mean speed and σ is the standard deviation. Troutbeck also found that 5 metre long cars were overtaken in about the same time as 16 metre long trucks, providing the trucks were travelling between 16 and 20 km/h slower than the 5 metre car. Speed distributions indicate that this is usually the case. Flying overtakings, which occur immediately after a short sharp vertical curve, were recorded as taking about the same time to execute as accelerative overtakings.

6.2.2 Commercial vehicles

Troutbeck (1981) found that increases in accelerative overtaking times of commercial vehicles with increased speed were not significant. Although it would normally be expected that overtaking times would increase as speed increased, the commercial vehicle population able to overtake at 80 km/h would be very different from the population which could overtake at 60 km/h. Hence, while overtaking times would increase with speed for an individual commercial vehicle driver, the mean overtaking times did not.

On average, heavy commercial vehicles take about four seconds longer to overtake than cars (Kosasih et al 1987). However, it is not usual for passing sight distance to be based on commercial vehicle needs, except where the proportion of trucks in the traffic stream is very high.

6.2.3 Bunching

Hoban (1983c) found that bunching of traffic in one direction was affected by that of the opposing traffic stream. If stream A was highly bunched, the interbunch gaps presented improved overtaking opportunities to the opposing stream B. Stream B then tended to have fewer bunches and so presented fewer large gaps for overtaking by stream A vehicles. This process reinforced the high bunching for stream A and low bunching for stream B.

6.2.4 Overtaking lanes

Harwood et al (1985) found no significant differences between the rates or severities of accidents between overtaking lane sections with opposing overtaking permitted and those with opposing overtaking prohibited.

6.3 STANDARDS IN USE

6.3.1 The overtaking manoeuvre

The procedure adopted by a driver as he overtakes another vehicle will determine the time and distance required for a successful overtaking manoeuvre. AASHTO (1984) made the following assumptions about the overtaking procedure:

1. The overtaken vehicle travels at a uniform speed.
2. The overtaking vehicle trails the overtaken vehicle as it enters the overtaking section.
3. When the overtaking section is reached, the driver requires a short perception time and reaction time.
4. Overtaking is accomplished under what might be termed a delayed start and a hurried return in the face of oncoming traffic. Overtaking vehicle accelerates during manoeuvre and its average speed is 10 mph faster than that of the overtaken vehicle.
5. When the overtaking vehicle returns to its own lane there is a suitable clearance length between itself and an oncoming vehicle.

The procedural assumptions appear to be reasonable, at least in the case of developed countries, although the speed differential in assumption 4 appears to be arbitrary. Using these assumptions, AASHTO have calculated the minimum overtaking sight distance as the sum of four values:-

1. Distance travelled during perception and reaction and during initial acceleration in own lane, d_1.

2. Distance travelled while the overtaking vehicle occupies the opposing lane, d_2.

3. Distance between the overtaking vehicle at end of manoeuvre and the opposing vehicle, d_3.

4. Distance travelled by an opposing vehicle for 2/3rd of the time the overtaking vehicle occupies the opposing lane, d_4.

$$d_1 = 1.47\, t_1 \left(v - m + \frac{at_1}{2}\right)$$

Where:

t_1 = time of initial manoeuvre (sec)
a = average acceleration (mph/sec)
v = average speed of overtaking vehicle (mph)
m = difference in speed of overtaken vehicle and overtaking vehicle

$$d_2 = 1.47\, vt_2$$

t_2 = time overtaking vehicle occupies opposing lane

d_3 varies from 100 to 300 feet.

If the opposing vehicle is assumed to be travelling at same speed as overtaking vehicle.

$$d_4 = \frac{2d_2}{3}$$

(During the first phase of the overtaking manoeuvre the overtaking vehicle has not come abreast of the overtaken vehicle and can return to its own lane if an opposing vehicle comes into view).

AASHTO estimated these times (seconds) and distances (feet) to be as shown in Table 6.1.

Table 6.1: Overtaking manoeuvres

Speed Group (mph)		30-40	40-50	50-60	60-70
Average overtaking speed V (mph)		34.9	43.8	52.6	62.0
Initial manoeuvre	a	1.40	1.43	1.47	1.50
	t_1	3.6	4.0	4.3	4.5
	d_1	145	215	290	370
Occupation of opposing lane	t_2	9.3	10.0	10.7	11.3
	d_2	475	640	825	1030
Clearance length	d_3	100	180	250	300
Opposing vehicle	d_4	315	425	550	680
Total distance		1035	1460	1915	2380

6.3.2 Sight distance

A number of countries have used either theoretical analyses, or direct measurements on the road, or a combination of the two, to relate overtaking sight distance requirements to the design speed of the road. These requirements are usually based on the 85th percentile driver (i.e. 85 per cent of drivers would be able to overtake if presented with this sight distance clear of opposing vehicles). Some of the values used in design are shown in the tables below.

Table 6.2: Sight distance requirements for overtaking in the UK (Department of Transport 1981a)

Design speed (km/h)	Full overtaking sight distance (m)		
	99th percentile	85th percentile	50th percentile
50	410	290	205
60	490	345	245
70	580	410	290
85	690	490	345
100	820	580	410
120	980	690	490

Table 6.3: Sight distance requirements for overtaking in the USA (AASHTO 1984)

Design speed (mph)	(km/h)	Minimum overtaking sight distance (feet)	(m)
20	32	800	230
30	48	1100	330
40	64	1500	450
50	80	1800	560
60	97	2100	650
65	105	2300	705
70	113	2500	760

Table 6.4: Overtaking sight distance requirements in Papua New Guinea (Department of Works 1985)

Design speed km/h	Overtaking sight distance (m)
25	75
30	90
40	160
50	200
60	300
70	350
80	480
100	800

The NAASRA (1980) overtaking sight distance figures are based on the work of Troutbeck (1980) as shown in Table 6.5.

Table 6.5: Overtaking sight distances (Australia)

	Overtaking sight distance			
	Establishment		Continuation	
Design speed (km/h)	time (seconds)	distance (m)	time (seconds)	distance (m)
50	13.6	350	4.5	165
60	14.6	450	5.0	205
70	15.7	570	5.4	245
80	16.9	700	6.4	320
90	18.2	840	7.6	410
100	19.7	1010	8.3	490
110	21.3	1210	9.1	580
120	23.1	1430	10.0	680
130	25.2	1690	11.0	800

Papua New Guinea (Department of Transport and Civil Aviation 1983) considered that it was inappropriate to design for overtaking or intermediate sight distances, but they considered it necessary to warn drivers against leaving their lane when there was insufficient sight distance. The appropriate value for this warning level would be the meeting sight distance.

It can be seen that the different sight distance requirements for various design speeds show considerable variation between the four countries. The standards for Papua New Guinea appear to be based on those of developed countries, but those of the other three countries are based on research in those countries.

6.3.3 Eye and object heights

For safe overtaking, the object which an overtaking driver must perceive is an oncoming vehicle (the most visible part is usually the roof). The eye and object heights used for design purposes are shown in Table 6.6, the eye heights being the same as those used for safe stopping.

Table 6.6: Eye and object heights for overtaking

Standard	Eye height	Object height
SATCC (1986)	1.10m	1.10m
AASHTO (1984)	3.5 Feet (1.07m)	4.25 Feet (1.30m)
Department of Transport (1984)	1.05 - 2.0m	1.05 - 2.0m
NAASRA (1980)	1.15 - 1.8m	1.15m
NITRR (1984)	1.05m	1.30m

6.3.4 Overtaking provision

It is necessary to determine how often the required sight distance is available. The effects on design capacity and speed of the lack of adequate provision of overtaking sight distance were estimated by the Ministry of Transport et al (1968) to be as shown in Table 6.7.

Table 6.7: Effect of substandard sight distance on design capacity and speed in the UK

Percentage of road with sub-standard sight distance	0	20	40	60	80	100
Percentage of standard design capacity	100	90	80	65	50	30
Estimated reduction in average speed (mph) of two-lane road carrying 900 pcu/hour	0	4	8	12	16	20

6.4 DISCUSSION AND RECOMMENDATIONS

Different countries have significantly different recommendations for overtaking sight distance requirements. There are a number of explanations for this:

1. The research has been undertaken in countries which may well have vehicle and driver populations with different characteristics, and hence display different overtaking performances.

2. The recommended values are based on different assumptions about the speed of the overtaken vehicle, speed of the overtaking vehicle, safety margin and decision times. These assumptions have a considerable effect on the distance required to effect an overtaking manoeuvre and hence the required sight distance.

3. Research for these values has been undertaken over many years and vehicle and driver characteristics may have changed in such a manner as to alter the overtaking sight distance requirements.

In the UK (Department of Transport 1981a, 1984), the design standards are such that the mean speed for one design step (50th percentile speed) is equivalent to the design speed for one step lower (85th percentile speed). The 85th percentile values shown in Table 6.2 are broadly supported by the empirical studies of Rumar and Berggrund (1973) and may thus be taken to be acceptable desirable values. They are slightly lower than those recommended in the USA and Australia, but all are intended to enable a vehicle which starts to overtake to complete the manoeuvre.

Although the above values are desirable, a more relevant absolute minimum sight distance is that which is equivalent to twice the stopping sight distance, which would allow two opposing vehicles to come to a standstill before impact. With longer sight distances, a driver may safely abort an overtaking manoeuvre if an opposing vehicle comes into view. The minimum sight distances thus derived are shown in Table 6.8 and are reasonably consistent with 50th percentile values in the UK (Table 6.2) and continuation sight distance values in Australia (Table 6.5).

Table 6.8: Recommended minimum overtaking sight distances

Design speed (km/h)	Minimum overtaking sight distance (m)
50	140
60	180
70	240
85	320
100	430
120	590

D. ALIGNMENT

7. HORIZONTAL CURVATURE

7.1 BASIC CONCEPTS

The minimum turning radius of a vehicle sets an absolute minimum to road curve radius. This is most important at junctions but may also be a factor on very low standard roads.

7.1.1 Motion on a curve

A vehicle traversing a curve holds its position on the road because of friction between the tyres and the road surface, and if the friction is insufficient, the vehicle will slide sideways (Department of Transport 1984).

The side friction developed on a curve is proportional to the square of the speed and the inverse of the curve radius (AASHTO 1984). On a level road, the tendency will be for vehicles to slide to the outside of curves and this tendency will be greater for vehicles with higher speeds and on smaller radius curves (Harris 1980). To counteract this effect it is normal to superelevate the road by sloping it towards the centre of the curve. This reduces the need for friction to hold the vehicle's position on the road and either makes driving safer or allows the use of higher speeds (Ministry of Transport et al 1968). The equation governing this relationship is as follows (NAASRA 1980):

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

Where, e = Rate of superelevation (m/m)
f = Side friction factor
V = Vehicle speed (km/h)
R = Radius (m)

In practice, it is impossible to provide an exact balance by superelevation because, for any given radius, a certain superelevation rate is correct for only one operating speed round the curve (Federal Ministry of Works and Highways 1972). At all other speeds, there will be a tendency for the vehicle to move outwards or inwards relative to the curve centre.

7.1.2 Transition curves

The characteristic of a transition curve is that it has a constantly changing radius. Transition curves may be inserted between tangents and circular curves to reduce the abrupt introduction of the lateral acceleration. In some circumstances, they may also be used to directly link tangents or two circular curves (SATCC 1986).

In practice, drivers employ their own transition on entry to a circular curve and transition curves usually contribute to the comfort of the driver in only a limited number of situations. However, they provide convenient sections over which superelevation or pavement widening may be applied, and can improve the appearance of the road by avoiding sharp discontinuities in alignment at the beginning and end of circular curves. For large radius curves, the rate of change of lateral acceleration is small and transition curves are not normally required (AASHTO 1984, NAASRA 1980, Department of Transport 1981a).

Transition curves are usually based on the equation for the Euler spiral or clothoid.

7.2 RESEARCH FINDINGS

7.2.1 Minimum turning radius

A number of countries have measured the minimum turning radii of design vehicles and typical values for developing countries are those from NITRR (1984) and Papua New Guinea (Department of

Transport and Civil Aviation 1983), which are shown in Table 7.1.

Table 7.1: Minimum turning radii

Vehicle Type	Minimum Turning Radius (m)
NITRR	
Passenger Car	6.20
Single unit truck	12.81
Single unit and trailer	14.00
Single unit bus	13.10
Articulated bus	11.59
Semi trailer	13.72
Papua New Guinea	
Four wheel drive	8
Small commerical	10
Medium or heavy commercial	15
Timber jinkers	20

7.2.2 Superelevation

Ritchie et al (1968) found a strong inverse relationship between curve speed and lateral acceleration for all groups of drivers. Lateral accelerations for the 30-40 km/h range were roughly twice those for the 90-95 km/h range, whilst on the same curves, faster drivers used higher lateral accelerations. On substandard curves, the side friction factors used by the 85th percentile driver have been found to be considerably in excess of those considered safe by AASHTO (1984).

Lateral accelerations normally achieved by drivers from public road measurements in the UK were found to increase considerably with reduction in curve radius (Kerman et al 1982). Also, drivers did not appear to respond to superelevation when selecting the speed at which they traversed a curve (Department of Transport 1984). McLean (1979) found that road curvature appeared to be the dominant factor affecting speed. In Papua New Guinea (Department of Works 1985), driver approach speed was found not to be influenced by the superelevation as much as the curvature, although no empirical data were quoted. The evidence from the studies indicates that the side friction factor available has such a large safety margin that it does not affect the speeds adopted by drivers.

Cleveland et al (1985) considered the interaction between the lateral and longitudinal frictional requirements for a vehicle which brakes when cornering. The relationship derived was:-

$$F^2 = Ft^2 - (v^2/15R - Fe^2)$$

Where F = friction available for deceleration
Ft = total friction available
v = design speed (mph)
R = radius (ft)
Fe = friction developed by the circular motion.

The implication of this analysis is that the risk of having to make an emergency stop should be reduced when high side friction factors are needed, i.e. sight distances should be greater on tight radius curves.

All empirical studies have shown a decrease in friction factor (f) for an increase in speed.

7.2.3 Speed Effects

A number of studies, such as those by CRRI (1982) and Duncan (1974), have used regression analysis to determine relationships between speed and curvature. In each case they have shown that reductions in speed are associated with increased bendiness or reduced curve radius. The formula derived by Duncan (1974) for a two-lane road implied a reduction in free speed of 10 km/h for every 75 degree/km increase in bendiness.

McLean (1978b,c) reported that curve speeds were primarily influenced by the desired speed pertaining to the section of road and by the curve radius. Later he produced relationships between these variables as shown in Table 7.2 (McLean 1981).

Table 7.2: Curve speed prediction relations

Desired Speed (km/h)	Predicted Speed (km/h)
60	60-380/R
70	69-715/R
80	77-1050/R
90	85-1410/R
100	95-1960/R
110	105-2920/R
120	115-3940/R

R = curve radius (m) and Speed = 85th percentile value.

Sight distance, road width and traffic flow have been found to have only a marginal effect on vehicle speeds and lateral accelerations (Kerman et al 1982). Cars with fast approach speeds slow down relatively more on curves than cars with slow approach speeds (McLean 1978b). This causes the coefficient of variation of the speed distribution to decrease with reduced values of mean speed.

On roads with very high design speeds, where the 85th percentile driver can theoretically expect to maintain his desired speed of travel, speed studies show that drivers will still reduce speeds slightly for curves (McLean 1981). Simpson and Kerman (1982) report that, for any approach speed to a curve, there is only a minor decrease in speed adopted by drivers even on very tight bends and that the design speed of a curve is not an adequate description of vehicle speeds on curves, actual speeds being considerably higher.

7.2.4 Accidents

On an exposure basis, horizontal curves have been found to be greatly over-represented in rural highway accidents (McLean 1983a).

In the UK, Shrewsbury and Sumner (1980) found that accident rates increased with reducing curve radius, and more rapidly below a value of about 400 metres. Simpson and Kerman (1982) also found a continually increasing accident rate with reducing radius. However, as low radius curves result in much shorter curve lengths, the overall accident implications are not considerable.

The Road Research Laboratory (1965) showed that inconsistencies in the horizontal alignment of a road significantly increased accident rates, which were affected not only by individual curve radius and average horizontal curvature, but also by the combination of the two. A sharp curve on an otherwise straight alignment caused a higher accident rate than that on an alignment with a high degree of bendiness. (Table 7.3).

Table 7.3: Road curvature and accident rates

Average curvature (deg/km)	Injury accidents / 10^6 veh-km				
	Curve Radius (m)				All sections
	> 1520m (straight)	610-1520	305-610	<305	
0-25	0.7	0.7	0.6	5.3	0.8
25-50	0.6	0.6	0.6	0.9	0.6
50-75	0.4	0.3	0.6	1.0	0.5
>75	0.2	0.3	0.6	0.8	0.4

Oglesby and Altenhofen (1969) found that, when curvature is associated with other hazards, the effects can be more severe. For example, the effects of gradient and curvature appear to be compounded, so that combinations of steep gradients and low radius curves have been found to have particularly high accident rates (McLean 1983a).

Kerman et al (1982) examined the speed reduction of the 85th percentile vehicle on sub-standard curves. The results show that curvature does not have as great an effect on vehicle speeds as might be imagined, although there was a large increase in the available lateral acceleration used at the lower radii. Safety factors may be lower at the lower end of this range but studies do not show a significant erosion of safety (Simpson and Kerman 1982).

7.3 STANDARDS IN USE

7.3.1 Curve radius

The minimum acceptable radius of a horizontal curve will depend on friction availability, superelevation, and vehicle speed.

In Papua New Guinea (Department of Transport and Civil Aviation 1983), maximum speed has been tabulated against radius with assumed variable values of lateral friction coefficient at a maximum superelevation of 10 per cent (Table 7.4).

Table 7.4: Maximum speeds for horizontal curve radii in Papua New Guinea

Radius (m)	Friction coefficient (f)	Maximum speed (km/h)
20	0.44	35
30	0.41	42
40	0.38	48
50	0.36	54
60	0.33	58
70	0.30	62
80	0.27	65
100	0.22	72

A large number of countries and authors recommend the design speed method of design, whereby the geometric features of a highway are related to the speed of an assumed driver of one of the faster vehicles, usually the 85th percentile. A selection of minimum radii as related to design speed for various organisations are given below in Tables 7.5 to 7.9.

Table 7.5: Minimum radii recommended in Papua New Guinea by the Department of Works (1985) (superelevation = 10 per cent)

a) Desirable

Speed (km/h)	Desirable minimum radius where topography is not a factor (m)
50	300
60	400
80	800
100	1200

B) Absolute

Speed (km/h)	Sealed pavement (f)	Sealed pavement radius (m)	Gravel pavement (f)	Gravel pavement radius (m)
25	-	-	0.18	18
30	0.20	25	0.16	27
40	0.19	45	0.14	55
50	0.17	75	0.13	85
60	0.15	115	0.12	130
70	0.14	155	0.11	175
80	0.14	210	0.10	250
100	0.13	340	-	-

Table 7.6: Minimum radii recommended by Choudhury (1980)

Design Speed (km/h)	40	50	60	70
Minimum Curve Radius (m)	60	90	130	180

Table 7.7: Minimum radii used in the UK (Department of Transport 1981a)

Design Speed (km/h)	50	60	70	85	100	120	V^2/R
Design condition	Horizontal Curvature (m)						
Minimum radius* without elimination of adverse camber and transitions	510	720	1020	1440	2040	2880	5
Minimum radius* with superelevation of 2.5%	360	510	720	1020	1440	2040	7.07
Minimum radius* with superelevation of 3.5%	255	360	510	720	1020	1440	10
Desirable Minimum radius* with superelevation of 5%	180	255	360	510	720	1020	14.14
Absolute Minimum radius* with superelevation of 7%	127	180	255	360	510	720	20
Limiting radius with superelevation of 7% at sites with special difficulty	90	127	180	255	360	510	28.28

* not recommended for use in the design of single carriageways.

Table 7.8: Minimum radii used by Japan (Ministry of Construction 1981)

Design Speed (km/h)	Radius of curve (m)	
	Desirable Minimum	Absolute Minimum
20	15	-
30	30	-
40	60	50
50	100	80
60	150	120
80	280	230
100	460	380
120	710	570

Table 7.9: Minimum radii recommended by NITRR (1984)

Design Speed (km/h)	Minimum Radius (m)
50	80
60	110
70	160
80	210
90	270
100	350
110	430
120	530
130	640
140	760

It can be seen that there are several standards of minimum radius used, with the UK Department of Transport (1981a) having four different levels.

In Australia, the speed standard of a curve is regarded as the maximum speed at which a vehicle can negotiate the curve without exceeding the NAASRA side friction factor criteria. This will often be in excess of the nominal design speed which may be governed by sight distance (McLean 1981). For curves with speed standards of 100 km/h or more, 85th percentile free speeds are generally less that the curve speed standard, while for curves of lower speed standard, the reverse applies. For curves with speed standard between 40 and 75 km/h, the 85th percentile speeds tend to be about 12 km/h above the speed standard. It is difficult to remedy this on lower speed alignments, as increases in overall alignment standard will serve to increase the desired speed of travel, which will in turn increase the operating speed on individual alignment features. The reality of driver behaviour is such that, on low speed alignments, many drivers operate with much smaller safety margins than those traditionally assumed for design purposes.

Kerman et al (1982) recommended that the following values be assumed for speed reductions on sub-standard curves.

Table 7.10: Speed reduction on sub-standard curves

Radius	V^2/R	Design Speed (km/h)					Speed reduction 85th percentile	Gross Lateral Acceleration (85th percentile drivers)
		60	70	85	100	120		
Des Min	14	255	360	510	720	1020	3.5%	0.103g
Abs Min	20	180	255	360	510	720	5%	0.142g
Limiting	28	127	180	255	360	510	7%	0.192g
1Dep	40	90	127	180	255	360	10%	0.255g
2Dep	56	63	90	127	180	255	14%	0.328g
3Dep	80	45	63	90	127	180	20%	0.403g

For the lowest standard roads, radii can be based on minimum turning circle criteria. However, in general, it is desirable for operational reasons to have minimum radii considerably in excess of that which is just negotiable.

7.3.2 Superelevation

Superelevation reduces the element of the sideways force which needs to be developed by the lateral friction between the tyres and the road. The coefficient of friction (f) at which sideways skidding is imminent depends upon (AASHTO 1984):

(i) Speed of vehicle.
(ii) Type and construction of roadway surface.
(iii) Type and condition of tyres.

On good dry pavements, the coefficients were estimated to be f = 0.5 for low speeds, and f = 0.35 for high speeds. For design purposes, a safe side friction factor is normally used and that recommended by Cron (1978) was 0.16 at 50 km/h, reducing linearly to 0.11 at 120 km/h.

The amount of superelevation should not be less than the road crossfall for practical drainage reasons. Equally, the superelevation should not be so excessive as to cause a stationary vehicle to slide down the cross slope. In areas where there is no snow or ice, superelevations up to 12 per cent may be used (International Bank for Reconstruction and Development 1978). Papua New Guinea Department of Transport and Civil Aviation (1983) and Malawi Ministry of Works and Supplies (1978) both adopt a maximum allowable superelevation of 10 per cent whilst the UK Department of Transport (1981a) uses a maximum superelevation of 7 per cent. Where snow and ice are factors, about 8 per cent is a logical maximum to minimise slipping across the highway when stopped or when attempting to start from a stationary position (AASHTO 1984). Excessive superelevation can also lead to difficulties for animal drawn and other slow moving vehicles and cyclists.

The recommended maximum levels of superelevation adopted by different countries show substantial variations depending on the acceptable level of friction assumed as indicated below.

Table 7.11: Curve radius and superelevation in USA (AASHTO 1984)

Design speed (mph)	Maximum super-elevation (e)	Maximum Friction factor (f)	Total (e+f)	Minimum radius (ft)
30	.06	.16	.22	273
40	.06	.15	.21	509
50	.06	.14	.20	849
60	.06	.12	.18	1348
65	.06	.11	.17	1637
70	.06	.10	.16	2083
30	.08	.16	.24	252
40	.08	.15	.23	468
50	.08	.14	.22	764
60	.08	.12	.20	1206
65	.08	.11	.19	1528
70	.08	.10	.18	1910
30	.10	.16	.26	231
40	.10	.15	.25	432
50	.10	.14	.24	694
60	.10	.12	.22	1091
65	.10	.11	.21	1348
70	.10	.10	.20	1637

Table 7.12: Curve radius and superelevation in Nigeria (Federal Ministry of Works and Highways 1972)

Curve radius (m)	Superelevation (m/m)
77-191	0.12
191-259	0.11
259-335	0.10
336-411	0.09
412-488	0.08
488-579	0.07
579-671	0.06

Table 7.13 Curve radius and superelevation in Malawi (Ministry of Works and Supplies 1978)

radius (m)	superelevation (%) at different design speeds (km/h)			
	48	64	80	97
75	10.00			
90	9.60			
100	9.30			
110	8.70		RC - Reverse crossfall	
125	8.20		NC - Normal crossfall	
135	7.80			
145	7.40	10.00		
160	7.00	9.60		
175	6.50	9.30		
195	6.10	8.90		
220	5.50	8.40	10.00	
250	5.00	7.60	9.70	
290	4.40	6.80	9.30	
350	3.80	6.00	8.30	10.00
440	3.00	5.00	7.00	9.00
500	RC	4.50	6.30	8.10
580	RC	3.80	5.40	7.00
700	RC	3.30	4.50	5.90
875	RC	RC	3.60	4.60
1160	RC	RC	RC	3.40
1750	NC	RC	RC	RC
3000	NC	NC	RC	RC
Note: Where superelevation shown is flatter than the normal crossfall, then a reverse crossfall section will be used.				

Table 7.14: Curve radius and superelevation (Jones 1961)

Design speed (km/h)	Side friction factor (f)	Minimum design radius (m)					
		Superelevation					
		0.02	0.04	0.06	0.08	0.10	0.12
32	0.25	30.5	27.4	25.9	24.4	24.4	21.3
48	0.20	83.8	76.2	70.1	65.2	70.0	57.9
64	0.18	163.1	147.8	135.6	125.0	115.8	108.2
80	0.16	281.9	254.5	231.6	211.8	195.1	181.4
97	0.14	457.2	406.9	365.8	332.2	304.8	281.9
113	0.12	713.2	621.8	553.2	498.3	452.6	414.5

The Department of Transport (1981a) states that superelevation shall be provided such that:

$$S = \frac{V^2}{2.828\ R}$$

Where V = Design speed (km/h)
R = Radius of curvature (m)
S = Superelevation (up to a maximum of 7 per cent)

Table 7.15: Curve radius and superelevation in the UK (Department of Transport 1981a)

Superelevation (%)	Minimum radius (m)					
	Design speed (km/h)					
	50	60	70	85	100	120
2.5	360	510	720	1020	1440	2040
3.5	255	360	510	720	1020	1440
5	180	255	360	510	720	1020
7	127	180	255	360	510	720
Limiting Radius at 7%	90	127	180	255	360	510

An alternative design procedure is to determine the maximum superelevation for a given terrain and design speed, and to relate the minimum radius of curvature to these. A number of countries have done this including Australia (McLean 1983a, NAASRA 1980) as given in Table 7.16.

Table 7.16: Curve radius and superelevation in Australia

Design speed (km/h)	Minimum radius(m)		
	Mountainous terrain e = 0.12	General Maximum e = 0.10	Flat terrain e = 0.06
50	40	45	50
60	65	65	75
70	90	95	105
80	135	140	155
90	215	230	265
100	330	360	440
110	400	435	530
120	495	540	670
130	-	-	785

On straight sections of road, normal crossfall exists with a crown in the middle. On curves, superelevation exists which is a plane for the whole width of the carriageway (Federal Ministry of Works and Highways 1972). The transition stage between these two is called the superelevation run-off (AASHO 1965). The application of superelevation run-off depends to a large extent on the nature of the curve. On curves with transitions, the superelevation can be progressively applied along the length of the transition (Department of Transport 1981a). On curves without transitions, part of the superelevation run-off is applied on the tangent and part on the curve (AASHO 1965). Superelevation run-off should be introduced uniformly over a length adequate for the likely travel speeds, as shown in Table 7.17 (AASHO 1965).

Table 7.17: Superelevation run-off and design speed

Design speed (mph)	Design speed (km/h)	Maximum relative slope between profiles of edge and centre line of 2-lane road (%)
30	48	0.66
40	64	0.58
50	80	0.50
60	96	0.45
65	104	0.41
70	112	0.40
75	120	0.38
80	128	0.36

The maximum relative slope between profiles of the edges of a 2-lane running surface are double those shown. Length of run-off on this basis is directly proportional to the total superelevation, and is the product of the lane width and the superelevation rate.

NAASRA (1980) state that, when introducing superelevation, the rate of rotation of the pavement should not generally exceed 0.025 radians/second at the design speed, with an absolute maximum rate of 0.035 radians/second.

The rate of change of superelevation, i.e the rate of change of the difference in gradient between the carriageway edge and the rotation axis expressed as a percentage, may be constrained to lie between a maximum and a minimum value as indicated by SATCC (1986) in Table 7.18.

Table 7.18: Rate of change of superelevation (SATCC)

Design speed (km/h)	Rate of change of superelevation	
	Maximum (%)	Minimum (%)
40	1.50	0.30
50	1.25	0.30
60	1.00	0.30
70	0.75	0.30
>80	0.50	0.30

Whilst superelevation will alleviate some problems, it will create others, particularly where there is no transition curve between the tangent and the start of the curve (Glennon and Weaver 1972). The higher the superelevation, the greater the problem of driving along the length of superelevation run-off for curves that bend to the left (when vehicles drive on the right) and have no transition. As a vehicle approaches the curve, it will be presented first with an area where the cross slope is less than 1 per cent where the pavement does not drain well, thus creating a potential hydroplane section. In the second area, drivers may experience some steering difficulty because, while still on the tangent, the superelevated cross slope requires them to steer against the direction of the approaching curve. When the vehicle reaches the tangent point, the driver must reverse his steering to follow the highway curve. In this third problem area, attempts to steer the highway curve path precisely will result in the lateral friction demand exceeding the design values at the design speed because this area lacks full superelevation. Superelevation at the tangent points of curves may be only 50-80 per cent of full superelevation, although this varies with design practice.

7.3.3 Transition curves

Several methods exist for the calculation of transition curve length, including Shortts, the superelevation run-off, and the rate of pavement rotation methods. These have been described by Kosasih et al (1987), who recommended the rate of pavement rotation method, although all are acceptable in most circumstances.

The rate of change of pavement rotation is defined as the change in crossfall divided by the time taken to travel along the length of transition at the design speed.

$$Ls = \frac{e.V}{3.6n}$$

and $$Le = Ls + \frac{en.V}{3.6n} = \frac{V}{3.6n}(e+en)$$

Where
Ls = Length of transition curve (m)
Le = Superelevation run-off (m)
e = Superelevation of the curve (m/m)
en = Normal crossfall of the pavement (m/m)
V = Design speed (km/h)
n = Rate of pavement rotation (radians/s)

The length of transition curve, Ls, is used to apply the superelevation, with the adverse crossfall removed on the preceding section of tangent. The change from a normal cross section to full superelevation at the start of the circular curve is achieved over the superelevation run-off distance, Le. It is recommended that the rate of pavement rotation, n, should not exceed 0.025 radians per second of travel time for design speeds greater than 80 km/h and 0.035 radians per second of travel time for lower design speeds.

7.3.4 Safety considerations

In Papua New Guinea the Department of Works (1985) state that minimum radius horizontal curves are undesirable on crest vertical curves. The crest may mask the sharpness of the horizontal curve and benching below road level may be necessary to achieve a given sight distance. Also, at crests where the horizontal curve is sharp, it is preferable that the vertical curve be contained within the horizontal curve resulting in the horizontal curve becoming visible ahead of the crest.

AASHTO (1984) recommend that higher values than the minimum sight distance standards should be used where there is a combination of steep downhill grade and horizontal curvature. This will allow for the longer stopping distances of trucks.

For safe design, a consistency of standard has been considered to be as important as the standard itself (McLean 1977, Ministry of Works and Supplies 1978). Thus, along any given length of road, the radii of curves should be comparable and, where a sharp curve is necessary, it should be approached by larger radius curves, each successively sharper than the one before. Sharp curves at the end of long straights are particularly hazardous, and larger radius curves should be introduced between the straight and the curve where possible. Compound circular curves should be avoided where possible, and the larger radius should not be more than 50 per cent greater than the smaller radius. It is also considered good design practice to avoid the use of short tangents between two curves in the same direction. Reverse curves are also undesirable, unless there can be adequate provision for the proper reversal of superelevation between the two geometric elements.

The lengths of horizontal curves should be kept as short as safety allows so as to give the maximum lengths of straights between the curves as this increases overtaking opportunities (Lyby 1977, Hills et al 1984). For small deflection angles, curves should be long enough to avoid the appearance of a kink and NITRR (1984) suggest a minimum length of 300 metres.

7.3.5 Other design considerations

AASHTO (1984) recommend that, where possible, an alignment should follow the shortest route with no sharp curves, and with consistent levels of curvature being offered to drivers. This type of alignment has many advantages, but can result in few overtaking opportunities.

Whilst the Department of Transport (1984) does not recommend sharp curves, the design philosophy leads towards significant tangent sections with relatively low radius curves so as to maximize overtaking opportunities. 'Medium radius curves are not recommended as they are considered to reduce safe overtaking opportunities and/or require large areas on the inside of curves to be cleared to provide the necessary sight distances. However, in flat inland areas of Australia, large radius curves do not restrict overtaking but produce the advantage of reduced headlight glare.

Drivers have been found to have difficulty in judging the speeds of oncoming vehicles, and hence overtaking opportunities, on long straight sections of road (Kosasih et al 1987). This may be overcome by introducing large radius curves and deflecting the alignment to the left and right alternatively. An appropriate deflection has been recommended to be 4 degrees (Odier et al 1971). An added advantage with this type of alignment is that it breaks the monotony for drivers.

The following points for consideration in horizontal design have been identified from several design standards:-

(i) Horizontal and vertical alignment features should be coordinated so as to avoid the creation of hazards and visual defects.

(ii) Compound circular curves and abrupt reversals of alignment should be avoided.

(iii) Short tangent sections between adjacent curves in the same direction should be avoided.

(iv) Large radius curves should be introduced with small deflection angles to avoid the appearance of a 'kink'.

(v) Curves should not end on structures such as bridges as they add cost and complications to design and construction. (If curvature is unavoidable, a simple curve of as large a radius as possible should be used).

(vi) Superelevation should remain a plane for the full width of the carriageway and may be obtained by revolving the pavement about the profiles of:-

  (i) The centreline
  (ii) The inside edge
  (iii) The outside edge.

(vii) Superelevation or removal of adverse crossfall should be achieved over or within the length of the transition curve. (Where no transition curve exists, it is common practice to apply two thirds of the superelevation prior to the start of the circular curve).

(viii) On superelevated curves, the outer shoulders should be sloped upward at about the same, or at a lesser rate, than the superelevation of the carriageway. Any shoulder which is sloped towards a paved carriageway should also be paved to prevent loose material washing over the surface.

(ix) For small changes in alignment, it is preferable, and usually more economical to adopt a circular curve of large radius with adverse crossfall, than a short curve with superelevation.

(x) The minimum superelevation applied to a road should be the reverse crossfall. (For unpaved roads this is also the maximum superelevation).

7.4 DISCUSSION AND RECOMMENDATION

Minimum turning radii are desirable in order that typical design vehicles are able to traverse the road without reversing. These radii will vary with length of curve and effective road width, and be governed by swept path considerations.

Several researchers have found clear relationships between curve speed, approach speed and curvature. Speeds tend to be normally distributed and 85th percentile values are usually used for design purposes. It was found in the UK that there was only a minor decrease in speed on curves of two design steps below the standard.

As standards increase, it is necessary for minimum radii to reflect the higher speeds and the critical design factor becomes the maximum lateral coefficient of friction between the tyre and road surface. The required lateral friction may be reduced by superelevating the road. Available side friction (f) decreases as speed increases and a considerable safety margin is usually incorporated in design standards. Recommended design values are shown in Figure 7.1.

Using the following equation, values of minimum radii may be calculated as given in Table 7.19.

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

Table 7.19: Side friction factor and minimum radii

Design Speed (km/h)	Side Friction Factor (f)	Minimum Horizontal Radius (m) 10% Superelevation (paved)	Zero Superelevation (unpaved)
30	0.33	16.5	21.5
40	0.30	31.5	42.0
50	0.25	56.2	78.7
60	0.23	85.9	123.3
70	0.20	128.6	192.9
85	0.18	203.2	-
100	0.15	315.0	-
120	0.15	453.5	-

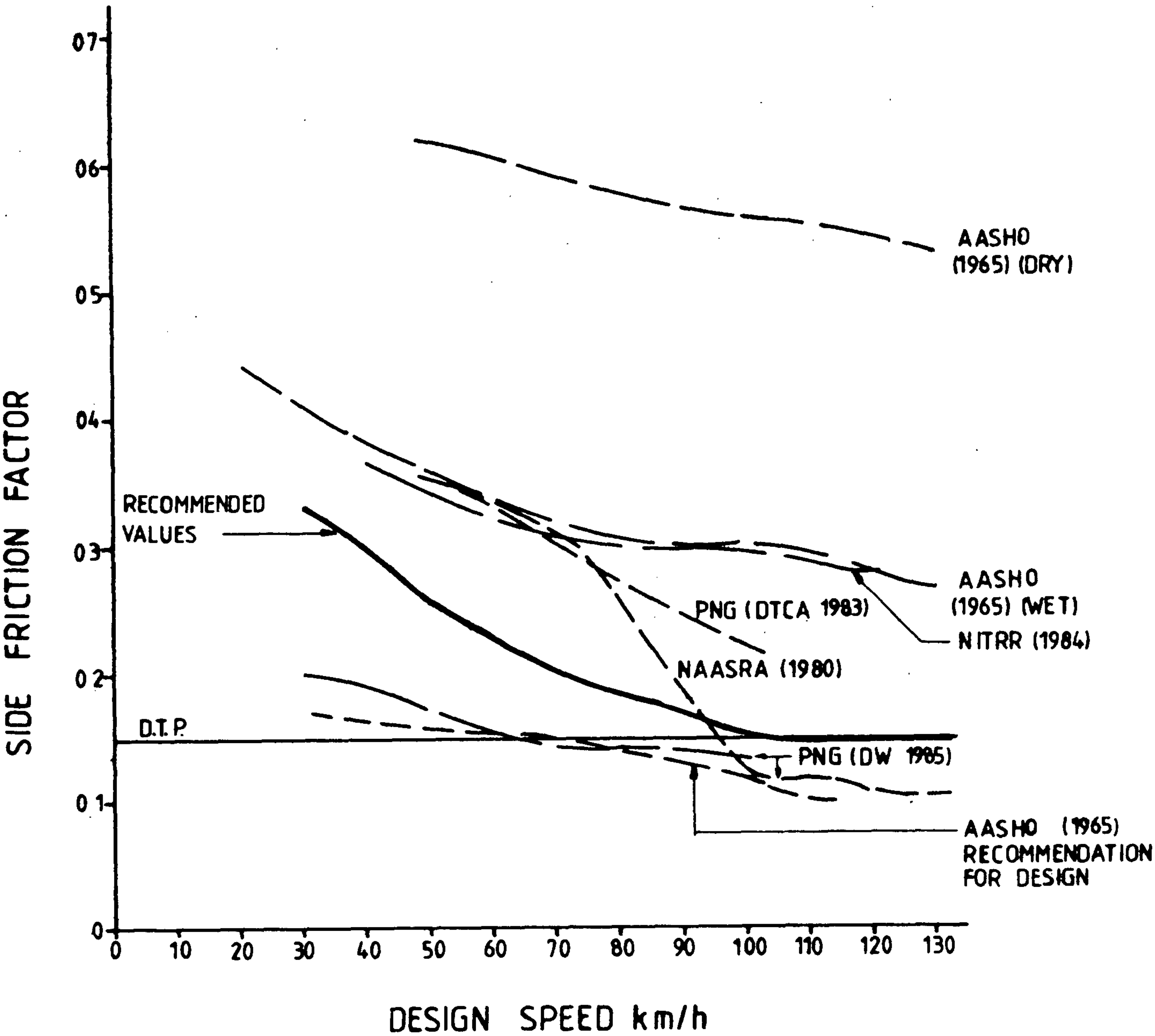
The superelevation required on a curve depends on the radius of curvature and vehicle speed. AASHTO (1984) recommends a maximum superelevation of 10 per cent, whilst NAASRA (1980) has a maximum superelevation of 12 per cent. However, excessive superelevation can cause slow-moving vehicles to slide to the inside of the curve, and the increased likelihood of slow-moving vehicles in developing countries may mean that the maximum allowable superelevation should be less than that in developed countries. In Malawi, the Ministry of Works and Supplies (1978) and in Papua New Guinea, the Department of Transport and Civil Aviation (1983) both recommend a maximum of 10 per cent and it is considered that this value would be generally suitable for developing countries. In places where snow and ice are factors, the superelevation should not exceed 8 per cent.

Superelevation may be applied in a series of ways, usually along a transition curve. The method of application is related to the transition curve design and varies between standards. The method adopted by NAASRA (1980) has been identified as being most suitable, although all are acceptable.

It has been found that inconsistencies in standard result in higher accident rates and that consistency is as important a factor as the standard itself. Increased curvature increases accident risk and this is further exacerbated when combined with other extreme geometric features.

FIGURE 7.1:
SIDE FRICTION FACTOR & DESIGN SPEED

(MEASURED & DESIGN VALUES FROM LITERATURE REVIEW & VALUES RECOMMENDED)



8. **VERTICAL CURVATURE**

8.1 BASIC CONCEPTS

The vertical profile of a road is normally composed of a series of tangents connected by parabolas. There are two types of vertical curve; crest curves and sag curves. The design of these curves is based on either a comfort criterion or a visibility criterion.

The length of a crest curve may be determined by a stopping or overtaking sight distance, or a comfort criterion. For minimum overtaking sight distance the required lengths of crest vertical curves are substantially larger than those for stopping sight distance.

The length of sag curves may be determined by visibility criteria or comfort criteria. During daylight, sag curves create no visibility problems, but at night the length of road illuminated by a vehicle's headlights may be critical.

The minimum length of vertical curve is equal to the algebraic difference in gradient times a constant, K. This constant K is often quoted as defining the radius characteristics of a vertical curve. Where the sight distance is greater that the length of vertical curve - an uncommon occurrence - this direct proportionality does not hold true.

8.2 RESEARCH FINDINGS

There is a lack of quantitative research evidence relating specifically to the design of vertical curves, but much of the research on sight distance, descibed in Part C, is relevant.

8.3 STANDARDS IN USE

8.3.1 Crest Curves

As noted above, the required crest curve lengths for safe overtaking are considerably longer than those required for safe stopping. This is demonstrated by the values used in the USA (AASHO 1965) as shown in Table 8.1.

Table 8.1: Sight Distance Requirements (AASHO)

Design Speed (km/h)	Stopping Sight Distance K value (m)	Overtaking Sight Distance K Value (m)
48	9	111
64	16	209
80	27	300
97	49	408
105	66	489
113	78	578
121	99	625
129	123	674

Values of K for crest curves are also quoted by NITRR (1984) as shown in Table 8.2.

Table 8.2: Crest Curves (NITRR)

Design speed (km/h)	K value (m)
40	6
50	11
60	16
70	23
80	33
90	46
100	60
110	81
120	110
130	133
140	163

These values are slightly higher than those used by AASHO.

The Department of Transport (1984) used two criteria to determine the desirable length of crest curves:

(i) Visibility criterion $$L = \frac{S^2A}{200\ (a+b+2ab)} = KA$$

(For sight distance greater than curve length)

(ii) Comfort criterion $$L = \frac{V^2A}{389}$$

Where L = Curve length, metres
S = Stopping sight distance, metres
A = Algebraic difference in grades, per cent
a = Eye height above road surface, metres
b = Object height above road surface, metres
V = Design speed, km/h

With a = 1.05 metres and b = 0.26 metres, the comfort criterion will govern for speeds of less than 60 km/h. Above 60 km/h the visibility criterion will govern.

In the UK (Department of Transport 1981a), crests may be designed for either overtaking or stopping sight distance as shown in Table 8.3.

Table 8.3: Vertical Curvature (UK)

Design speed (km/h)	Overtaking K Value (m)	Minimum K Value (m)	
		Desirable	Absolute
50	100	10	6.5
60	142	17	10
70	200	30	17
85	285	55	30
100	400	100	55
120	-	182	100

At a crest between two gradients in excess of 4 per cent, it is acceptable to make a further relaxation below absolute minimum crest K value, due to the reduced speeds up the gradient.

It is often impractical to design crest vertical curves for overtaking sight distance because of the high costs involved. If crest curves are not going to be designed for overtaking, the lengths of crest curves should be kept as short as possible to reduce the length of road where overtaking is not possible (Hills et al 1984).

On crest vertical curves, an object beyond the crest which would be visible under daytime conditions is shadowed by the crest at night (Cleveland et al 1985). The effective driver eye height for this condition is headlight height (Hills 1977). The problem is compounded if drivers are forced to use dipped headlights for much of the time. On straight and level roads, vehicles using dipped headlights have a visibility distance often considerably less than the minimum stopping sight distance. Using standard calculations, the maximum safe operating speed at night on crest curves would be about 50 km/h.

8.3.2 Sag curves

A number of countries have used a headlight criterion for the design of sag curves. South Africa (NITRR 1984) assume a headlight height of 0.6 metres and a divergence angle of 1 per cent above the horizontal axis of the headlights. This leads to the K values for sag curves shown in Table 8.4.

Table 8.4: Sag Curves and Headlight Criteria (NITRR)

Design speed (km/h)	K value (m)
40	8
50	12
60	16
70	20
80	25
90	31
100	36
110	43
120	52
130	57
140	64

The USA also use a headlight criterion (AASHTO 1984):

$$L = \frac{AS^2}{400 + 3.5\,S} \qquad \text{when S is less than L}$$

$$L = 2S - \frac{400 + 3.5\,S}{A} \qquad \text{when S is greater than L}$$

where S = Light beam distance, feet
L = Length of sag vertical curve, feet
A = Algebraic difference in gradient, per cent

For sag curves, the sight distance is measured to a point on the road surface illuminated by a headlamp beam with an upward divergence of 1 degree. This model is only useful when the object has retro-reflective properties, because the headlamp illumination above the vehicle axis is too weak for the driver to see any other object at these distances (Cleveland et al 1985).

However, not all countries are in favour of headlight sight distance. In 1981, the UK used the headlight sight distance (Department of Transport 1981a), but by 1984 the basis for design had changed (Department of Transport 1984).

The most common obstruction on a normal rural road is another vehicle which may or may not be stopped (NAASRA 1980). Even if it is not carrying lights, it will often have retro-reflective material at strategic locations, situated higher than the object cut off height used in the stopping sight distance calculations. As far as small non-illuminated objects are concerned, research has shown that:

a) Only larger, light coloured objects can be perceived at speeds above 80 km/h at the stopping distances used.
b) Significant improvement is unlikely, as a five fold light increase is necessary for a 15 km/h increase in speed, and a ten fold increase for a 50 per cent reduction in object size.
c) In any case, the joint requirements of driving vision, and minimising glare to oncoming traffic, set limits to beam intensity.

A general limit of 120 to 150 metres sight distance is all that safely can be assumed for visibility of an object on a bitumen road. This corresponds to a satisfactory stopping distance for 80 km/h to 90 km/h. Beyond this, it is only large or light coloured objects that will be perceived in time for reasonable evasive action to be taken on unlit roads.

The most liberal assumption of headlight beam illumination is that some illumination can be expected up to the level of the horizontal plane. The length of sag curve required to give stopping sight distance from a headlight height of 750 millimetres to zero is considerably more than required to achieve reasonable riding comfort. In addition, increasing the length of sag curve to produce a theoretical sight distance may not give the desired result. If there is a horizontal curve in addition to the sag, the headlights shine tangentially to the horizontal curve and off the pavement. It can be assumed that the beam of light spreads about 3 degrees left and right horizontally. As well as vertical control for sight distance, it is necessary to ensure that it is available in the horizontal plane. In Papua New Guinea (Department of Works 1985), design based on headlight sight distance is only used on the most heavily trafficked roads.

To minimise discomfort when passing through sag curves, it is usual to limit the vertical acceleration generated to a value of less than 0.05 g (NAASRA 1980). On low standard roads and at intersections, a limit of 0.10 g may be used which results in the K values given in Table 8.5.

Table 8.5: Vertical Acceleration and Sag Curves

Design speed (km/h)	K value (m)	
	a = 0.05 g	a = 0.10 g
40	3	1.5
60	6	3
80	10	5
100	16	8
120	23	12

In the USA (AASHTO 1984) the comfort criterion is given by:

$$L = \frac{AV^2}{46.5}$$

where L = Curve length (feet)
A = Algebraic difference in grades (per cent)
V = Design speed (mph)

In the UK (Department of Transport 1981a), sag curves are designed for an absolute minimum comfort criteria of 0.3 m/s^2 vertical acceleration:

Table 8.6: Vertical curves and comfort (UK)

Design speed (km/h)	Absolute minimum sag K value (m)
50	13.5
60	20
70	20
85	20
100	26
120	37

8.3.3 Other design considerations

Minimum radius horizontal curves are undesirable at crests as the crest may mask the sharpness of the horizontal curve (Department of Works 1985). When vertical and horizontal curves occur in combination or in close proximity to each other, it is desirable that the vertical curve should be either wholly within or wholly outside the horizontal curve (Odier et al 1971). With the vertical curve within the horizontal curve, the horizontal curve becomes visible ahead of the crest.

8.4 DISCUSSION AND RECOMMENDATIONS

Vertical curves are usually designed as parabolas. They are designed to meet either minimum sight distance or comfort criteria. Sight distance for safe stopping is the more stringent requirement.

For crest curves, safe stopping and safe overtaking values are usually calculated on the basis of object and eye height criteria discussed earlier. Safe stopping sight distance is usually an essential limitation although the NAASRA concept of safe manoeuvring sight distance may also be applied. The cost of crest curve sight distances for safe overtaking is often prohibitive and drivers are not necessarily aware of its availability since the subtleties of variations in road marking to indicate safe overtaking are often not fully understood, even in developed countries. Overtaking can be maximized by using small vertical curves allowing longer tangential gradient sections. This principle is similar to that for the design of horizontal curvature.

Since designing crest vertical curves for overtaking sight distance is expensive and is not justified in most cases, crest curves should be designed to give no more than the absolute minimum stopping sight distance. Sag curves offer adequate visibility during daylight. However, it is neither practical nor reliable to design them for visibility at night. Sag curves should therefore be designed according to comfort criteria. On major roads a vertical acceleration of 0.05 g would be an appropriate maximum, whilst on other roads, the maximum is often relaxed to 0.10 g.

For sag curves, the effective range of headlamp beams have a general limit of 120-150 metres for an unlit object. This is normally insufficient to form the basis of curve design. Sag curves are therefore generally based on comfort criteria, i.e. vertical acceleration, which falls within the broad range of 0.03-0.10 g.

9. GRADIENT

9.1 BASIC CONCEPTS

There are two aspects to a design standard for gradients; the gradient itself, and the length of the gradient (AASHTO 1984). Many vehicles can tolerate a steep gradient for a short distance, but cannot sustain speed if the gradient is long.

For low volume access roads, where operational efficiency is not the principal design objective, appropriate maximum gradients should be based upon traction ability, which will be different for 2-wheel drive, 4-wheel drive and animal drawn vehicles.

A minimum slope is necessary to drain a pavement adequately. Although part of the drainage requirement is provided by the crossfall, a longitudinal grade is desirable to assist drainage, particularly for kerbed pavements.

9.2 RESEARCH FINDINGS

McLean (1978e) found that cars can negotiate moderate inclines without an appreciable loss of speed, but speeds will decrease on steeper upgrades. Gradient has a more significant effect on truck speeds. Duncan (1974) has derived a formula for free speeds of cars on two lane roads:

$$Vo = 75 - \frac{P-15}{10} - \frac{H}{7.5}\,\frac{(185+P)}{200} - \frac{B}{7.5}\,\frac{(215-P)}{200}$$

where:

Vo = Free speed (km/h)
P = Percentage heavy vehicles
H = Average hilliness (m/km)
B = Average bendiness (degrees/km)

The forumula derived for the slope of the speed/flow relationship on a two-lane single carriageway was:

$$S = -\tfrac{1}{2}\,(Vo-55) - \frac{P}{5}\;\frac{H}{7.5}$$

where:

S = Speed/flow slope (km/h per 1000 vehicles/hour/standard lane)

It can be seen from the equations that hilliness was found to have a much lower effect on car free speeds and speed/flow slope than did bendiness. As a result, car traffic has been considered likely to gain the greatest benefit from improvements to horizontal alignment, with truck traffic benefiting most from improvements to vertical alignment (McLean 1980a).

Steepening of uphill gradients was not found from the literature to have an adverse effect on accidents. However, accident rate was found to increase with steepening downgrades (Shrewsbury and Sumner 1980).

McLean (1983a) found accident rates increased with increasing vertical gradient, particularly for gradients greater than about 4 per cent. This effect applied predominantly for traffic travelling in the downhill direction. The effects of gradient and curvature appeared to be compounded so that combinations of steep gradient and low radius curves exhibited particularly high accident rates.

Oglesby and Altenhofen (1969) found little evidence that gradients alone seriously affected accident rates within the common limits of steepness. However, they too found evidence that accidents increased when grades steeper than 5 per cent were combined with sharp curves and narrow shoulders.

In a study of shoulder design, Armour (1984a) found that the accident rate on gradient sections was 3.75 times as high as on flat sections.

9.3 STANDARDS IN USE

9.3.1 Maximum gradient

A number of countries specify maximum design gradients according to terrain type, road type and design speed. In general steeper gradients will be accepted on less important roads, with lower flows at lower design speeds and in more severe terrain. This is demonstrated by the maximum gradients recommended by AASHTO (1984) in Table 9.1. It was also recommended that the maximum design gradient should be used infrequently, rather than as a value to be used in most cases. For short grades of less than 170 metres (500 feet), grades may be 1 per cent steeper, whilst for low volume rural highways, grades may be 2 per cent steeper.

Table 9.1: Gradient and type of road

Type of road and terrain	Maximum gradient (%)					
	mph 20 (km/h)(32)	30 (48)	40 (64)	50 (80)	60 (97)	70 (113)
Local roads and streets						
Level	8	7	7	6	5	-
Rolling	11	10	9	8	6	-
Mountainous	16	14	12	10	-	-
Rural Collectors						
Level	7	7	7	6	5	4
Rolling	10	9	8	7	6	5
Mountainous	12	10	10	9	8	6
Rural Arterials						
Level	-	-	-	4	3	3
Rolling	-	-	-	5	4	4
Mountainous	-	-	-	7	6	5

The general maximum grades recommended by NAASRA (1980) are as shown in Table 9.2.

Table 9.2 Gradient and Terrain (NAASRA)

Design speed (km/h)	Maximum gradient (%)		
	Terrain		
	Flat	Rolling	Mountainous
60	6-8	7-9	9-11
80	4-6	5-7	7-9
100	3-5	4-6	6-8
120	3-5	4-6	-

These values are similar to those used by AASHTO. The values used by the SATCC (1986) countries are in general lower than those used by AASHTO as shown in Table 9.3.

Table 9.3: Gradient and terrain (SATCC)

Design speed (km/h)	Maximum gradient (%)		
	Terrain		
	Flat	Rolling	Mountainous
40	-	-	10
50	-	7	9
60	-	6	8
70	5	5.5	7
80	4	5	6
90	3.5	4.5	-
100	3	4	-
110	3	-	-
120	3	-	-

In the UK (Department of Transport 1981a), desirable maximum gradients are expressed by road types as shown in Table 9.4. However, on all-purpose roads, gradients up to 8 per cent can be considered. Steeper gradients than this are departures from standard. Simpson and Kerman (1982) considered that the overall accident implications in the use of gradients of up to 8 per cent were not severe.

Table 9.4: Gradient and Road Type (UK)

Road type	Desirable maximum gradient (%)
All purpose single carriageway	6
All purpose dual carriageway	4
Motorway	3

9.3.2 Absolute maximum gradients

In Papua New Guinea (Department of Transport and Civil Aviation 1983) the maximum gradients traversable by different types of vehicles were found to be:-

4-wheel drive vehicle	20 per cent
Small commercial vehicle	18 per cent
Medium or heavy commercial vehicle	16 per cent
Timber jinkers	12 per cent

Hills et al (1984) stated that, if possible, all gradients should be limited to 15 per cent and preferably not more than 10 per cent. Vance (1978) recommended that gradients of over 10 per cent should only be used in temperate or tropical climates, where icing does not occur, and that such gradients should be paved to provide adequate traction. Mills (1980) found travel on gravel surfaces became difficult when the gradient exceeded 8 per cent, and recommended that all gradients exceeding that value should be paved, together with short connecting sections where necessary to ensure continuity of surface. On earth roads, particular soil types may give rise to slippery conditions in the wet and anything steeper than about 10 per cent can be very difficult to negotiate. On some soils, 5 per cent may be the practical limit in the wet (Hills et al 1984).

9.3.3 Critical length of gradient

The critical length of gradient vary between countries because of differences in the definition of critical length. Critical length may be defined as the point at which a truck reaches a certain speed or the point at which it has lost a certain amount of speed. These values are not standard between countries.

Critical length of gradient has been considered in the USA to be the maximum length of a designated upgrade upon which a loaded truck can operate without an unreasonable reduction in speed (AASHTO 1984). For gradients longer than the critical length, changes in location or addition of extra lanes should be made to maintain freedom of operation. Critical length of gradient is, to some extent, dependent on the gradient of approach; a downhill approach will allow vehicles to gain momentum and increase the critical length.

Papua New Guinea (Department of Works 1985) specifies its standards by a gradient and a maximum length of that gradient as shown in Table 9.5.

Table 9.5: Length of Gradient and Design Speed (PNG)

Design speed (km/h)	Gradient			
	General maximum		Absolute maximum	
	(%)	length (m)	(%)	length (m)
25	12	300	14	150
30	10	500	12	250
40	10	500	12	250
50	8	600	10	300
60	7	1100	9	400
70	6	-	8	700
80	6	-	8	1000
100	5	-	7	750

In general, the critical length of gradient decreases, as gradient increases. The values from NITRR (1984) show this effect (Table 9.6).

Table 9.6: Length and severity of gradient

Gradient (%)	Critical length of gradient (m)
3	400
4	300
5	240
6	200
7	170
8	150

Where it is necessary to exceed the critical length of gradient on heavily trafficked roads, it is desirable to provide either safe passing distances on the rise, or a climbing lane for heavy vehicles. On more lightly trafficked roads, it may be necessary to prohibit overtaking on such lengths (NAASRA 1980).

9.3.4 Minimum gradient

AASHTO (1984) suggested that kerbed pavements should have minimum longitudinal grades of 0.5 per cent. On sag curves, a minimum gradient of 0.3 per cent should be provided within 50 feet of the bottom of the sag curve.

9.3.5 Other design considerations

Although speeds of cars will be lower on steep upgrades, large differences between speeds of light vehicles and heavy vehicles will occur and truck speeds will be quite slow. NAASRA (1980) considered it important, therefore, to provide sufficiently adequate horizontal sight distance to enable faster vehicle operators to recognise when they are catching up to a slow vehicle and to adjust their speed accordingly.

The use of steeper gradients results in much shorter lengths of incline than the use of flatter gradients. However, there are operational disbenefits of steep gradients such as greater use of fuel, particularly for heavy vehicles, and additional delays caused by slow speeds (Simpson and Kerman 1982).

9.4 DISCUSSION AND RECOMMENDATIONS

Both gradient and length of gradient are important considerations in the design process, although it has been suggested that gradient is one of the standards that may be relaxed to give greatest economy with least loss of operational performance (McLean 1978e).

At the basic access level, gradients must be negotiable by the design vehicle, albeit at a very low speed. If only 4-wheel drive vehicles use the road, the maximum gradient may be up to 20 per cent, although 2-wheel drive commercial vehicles would require gradients of no more than about 15 per cent. For traction requirements and maintenance reasons, it is recommended that all gradients over 10 per cent should be paved.

As traffic flows increase, the economic benefits of reduced severity and length of gradient will become larger because of the increased number of vehicles and associated operational effects. Lower construction costs in flatter terrain will generally result in the economic justification of higher standards of gradient under those conditions. Thus, in most developed countries, maximum allowable gradients, which vary with both design speed and terrain, range from 3 per cent on high flow/high speed roads in level terrain to 16 per cent on low flow/low speed roads in mountainous terrain. In general 4-8 per cent is considered a desirable maximum on roads with higher design speeds.

Values recommended on the basis of available standards are as shown in Table 9.7.

Table 9.7: Recommended maximum gradients

Design Speed (km/h)	Maximum Gradient (%)	
	Desirable	Absolute Maximum
30	10	20
40	8	15
50	8	15
60	7	10
70	6	10
85	5	10
100	4	8
120	4	8

Vehicles will reduce speed as they climb a gradient. This speed reduction will be greatest for trucks and may be exacerbated in developing countries because of poor mechanical condition and overloading. The critical length of gradient, i.e. the maximum recommended for design, will depend largely on the number and type of vehicles using the road, and the level of service which is deemed acceptable. Values of operating cost, travel time saving and accident rates are particularly relevant. As these differ between countries, there is considerable variation as to appropriate length of critical gradient. This is discussed further in Appendix A, where a simulation model has been used to look at operational performance and to make recommendations about climbing lanes.

Accidents have been found to increase with steepening downgrade, and this is compounded by the presence of low radius curves and narrow shoulders. Care must be taken to ensure that adequate sight distances are available in such circumstances and that geometric elements are not combined in a dangerous way.

A minimum slope is desirable to drain the pavement, particularly for kerbed pavements. A suitable minimum value is 0.5 per cent.

E. OPERATIONAL CONSIDERATIONS

10. SPEED

10.1 BASIC CONCEPTS

Speed occupies a unique position in geometric design; it may be both the resultant and the determinant of design standards. Road geometry, in terms of gradient and curvature, will influence the speed adopted by vehicle drivers. However, the concept of design speed influences the design standards for geometry.

10.2 EFFECT OF GEOMETRY ON SPEED

A number of studies have shown that speed is influenced by the geometric characteristics of the road, principally hilliness and bendiness. These have been discussed in Section D.

10.3 DESIGN SPEED

Most countries design their roads according to a design speed, although the definition of design speed can vary. It is important that roads are designed for the safe operation of the majority of vehicles, and for this reason, the 85th percentile vehicle is generally used for design purposes (Department of Transport 1981a, Kerman 1980).

AASHTO (1984) defines design speed as the maximum safe speed that can be maintained over a specified section of highway when conditions are so favourable that the design features of the highway govern. The adopted design speed should be a logical one with respect to the topography, the adjacent land use and the type of highway. When designing a substantial length of highway, it is desirable to adopt a constant design speed. Changes in terrain and other physical controls may dictate a change in design speed on certain sections; if so, the lower design speed should not be introduced abruptly, but over sufficient distance to permit drivers to change speed gradually before reaching the section of highway with the lower design speed. (AASHTO 1984).

Nearly all design elements of a highway vary with a change in design speed (AASHTO 1984). In general the design speed will set minimum standards which must be observed, although there is normally no objection to exceeding these standards if it is justified economically. The geometric parameters most directly affected by design speed are curvature and sight distance requirements. These were discussed in Sections C and D.

Roads frequently display a variation in apparent design standard, as the design speed concept only fully applies in the presence of physical geometric characteristics which limit the safe speed of travel (McLean 1977). This is not the case for level tangent sections. A road can be designed to a constant design speed but because it is constrained to minimum values at only certain locations the road can appear to the driver to have a wide variation in design standard.

When a change in topographical character or some other constraint necessitates a change in alignment standard, the change should be made clear to the driver. This is best achieved by a sequence of horizontal curves, each having a predicted speed consistent with the design criterion. When going from a high to a low standard, the predicted speed on sequential curves should not differ by more than 10 km/h, and the speed environment relevant to each curve can be taken as the predicted speed of the previous curve (McLean 1979).

10.4 SPEED/FLOW

Two items are needed to produce a significant speed reduction for light vehicles (White 1963):

(i) A number of slow moving vehicles
(ii) Restricted overtaking oppportunities.

Both these situations occur where heavy vehicles operate in rolling and mountainous terrain with restricted sight distances. Table 10.1 shows the interaction between hilliness, bendiness and the proportion of heavy vehicles (Duncan 1974):

Table 10.1: Speed reduction and terrain

Heavy Vehicles (%)	5			15			40		
Hilliness (m/km)	0	15	30	0	15	30	0	15	30
Bendiness degree/km	Speed reduction (km/h)								
0	10.5	11.5	12.6	10	15	20	8.8	23.6	38.5
75	5.3	6.3	7.3	5	10	15	4.4	19.3	34.1
150	0	1.1	2.1	0	5	10	0	14.9	29.7

The greatest speed reduction in Table 10.1 is associated with a high free speed and low bendiness, a high proportion of heavy vehicles, steep gradients and two lane roads.

White (1963) found that the relationship between flow and speed varied with the width of the carriageway; on narrow carriageways, an increase in flow produced a proportionally greater reduction in speed than the corresponding reduction on a wide carriageway. Overtaking was also considered to be more difficult on a narrow road because an overtaking vehicle occupied the opposing traffic lane for a longer period when the lanes were narrow than when they were wide. Safe overtaking therefore required longer gaps in the opposing flow. This result was also confirmed by Troutbeck (1984) who found that mean overtaking time was increased from 8.6 seconds to 9.3 seconds when the road width was reduced from 7.4 metres to 6.0 metres. This effect results in a reduced level of service, which becomes more apparent as flows increase and the number of larger gaps in the opposing traffic stream decreases.

COBA (Department of Transport 1981b) employs a speed/flow relationship which takes into account the proportion of heavy vehicles, the difference in free speed between light vehicles and heavy vehicles, and the road width (TRRL 1980b).

The speed/flow slope for light vehicles is given by:

$$sl = \frac{21 + \frac{DP}{100}}{1200}$$

and for heavy vehicles

$$sh = \frac{21 - \frac{D(100-P)}{100}}{1200}$$

Where sl = Speed/flow slope for light vehicles.
sh = Speed/flow slope for heavy vehicles.
P = Proportion of heavy vehicles (%).
D = Difference of free speeds between light and heavy vehicles (km/h)

The width effect is taken into account by expressing the flow rate, Q, in vehicles per hour per standard lane (3.65 metres). The COBA relationships do not incorporate an interaction effect between gradient and the flow of heavy vehicles.

10.5 DISCUSSION AND RECOMMENDATIONS

Speed on a road is determined by the drivers' intentions, vehicle performance characteristics, road geometry, and the presence of other vehicles.

Relationships have been derived by TRRL for developing countries (Hide et al 1975, Morosiuk and Abaynayaka 1982) from which speeds may be estimated from geometric parameters. However, an initial knowledge is required of the mean free speed of vehicles on a level and straight section of road. This leads to some uncertainty in speed estimation, although the estimates of the relative effects of varying geometry will be more accurate. Within this uncertainty, the relationships may be used to estimate vehicle speeds for some design purposes.

Vehicle-to-vehicle interactions will increase with higher traffic volumes. The resulting drop in operational efficiency can be countered by improving standards as discussed in the previous sections. Changes in standards should be based on economic considerations, where possible, rather than notional levels of service.

11. **AUXILIARY LANES**

11.1 BASIC CONCEPTS

On two-lane roads, the provision of auxiliary lanes increases operational efficiency by enabling faster vehicles to more easily overtake those that are moving more slowly such as heavily laden trucks or non-motorised vehicles. On single-lane roads, passing places allow opposing vehicles room to manoeuvre around each other where there would otherwise be insufficient width.

11.2 RESEARCH FINDINGS

Hoban (1981) found that, where traffic problems are experienced due to excessive bunching and limited opportunities to overtake, short lengths of overtaking lane were more cost-effective than full duplication. There are several reasons for this:

(i) Overtaking lanes may be positioned at a least-cost site.

(ii) The highest rate of overtaking on an overtaking lane occurs over the first 500 to 1000 metres.

(iii) A large part of the benefit of an overtaking lane in terms of higher speeds occurs on the downstream section beyond the overtaking lane.

(iv) There may be a reduction in accidents by removing some overtaking vehicles away from the opposing lane.

Harwood et al (1985) found that the effectiveness of overtaking lanes increased as the traffic entering the overtaking lane became more congested. Overtaking lanes provided much higher overtaking rates than were possible on a two-lane highway. The overtaking rate in the direction without the overtaking lane was substantially less than that in the other direction, but was higher than that for a conventional two-lane highway.

In a Californian study, (Hoban 1981) the provision of an overtaking lane appeared to reduce accidents by about 25 per cent over the length of the additional lane. This could well be an under-estimate of the total safety benefit as it does not take account of the effect of removing overtaking manoeuvres from more hazardous locations.

Harwood et al (1985) found indications that the accident rates in overtaking lanes were slightly higher in the treated direction than in the untreated direction, but that overtaking lanes had slightly lower accident rates than untreated two-lane highways. They found no indication that accidents were different between lane transition areas. A slightly greater proportion of accidents occured in the transition areas than would be expected from their relative lengths alone, but the differences were not large. They concluded that there was no indication of any marked safety problem in the lane addition and lane-drop transition areas of overtaking lanes.

Overtaking lanes were found to reduce the percentage of vehicles forming platoons, both within the overtaking lane and, to a lesser extent, downstream of the overtaking lane (Harwood et al 1985). Mean speeds were found to be higher within an overtaking lane than upstream of the lane. Downstream of the lane speeds were lower than within the overtaking lane but higher than they were upstream of the lane. The persistence of operational benefits from an overtaking lane appears to be highly dependent on the geometrics and traffic flow conditions in the downstream area. The effectiveness of an overtaking lane increases as the traffic entering the overtaking lane becomes more congested, as the demand for overtaking is higher (Hoban 1982).

Overtaking lanes were highly utilised compared to a continuous extra lane at the same traffic volume (Hoban 1982). This is because traffic entering the overtaking lane was more bunched and demand for overtaking was higher. Rates of return were found to reduce with each additional increment of overtaking lane provided. In economic terms, overtaking lanes appear to offer good value for money. Overtaking lanes yield quite high rates of return based on travel time savings and improved safety (Hoban 1983a). Short overtaking lanes generally yield considerably higher returns than longer, more expensive climbing lanes. It would appear that breaking up bunches is a more important function of overtaking lanes than reducing delays caused by trucks on individual gradients. Hoban (1982) found overtaking lanes could be justified at flows as low as 950 vehicles per day. In some cases overtaking lanes can be a part way stage to full duplication for lower cost.

The result of a microscopic simulation model ran to evaluate the speed effects of introducing a climbing lane are given in Appendix A. The model estimates the extent of speed increases brought about by the addition of a climbing lane under various conditions of gradient, flow and composition.

11.3 STANDARDS IN USE

11.3.1 Overtaking lanes

Siting of overtaking section, i.e. a length where vehicles can overtake safely, is important. If faster vehicles are constrained to follow slower ones over a particular section of road, then it is desirable to follow this with an overtaking section, otherwise increased driver frustration results and they will attempt to overtake at increased risk on more dubious alignments. If an overtaking section cannot be provided within the standard geometry, an overtaking lane may be an appropriate solution.

The termination of an overtaking lane should be at a point where there is sufficient sight distance for drivers in the overtaking lane to complete or abandon an overtaking manoeuvre (Morrall and Hoban 1985). Desirably, the termination point should be on a downgrade to minimise the speed differential between faster and slower vehicles and on a straight to give drivers better visual appreciation of the approaching merge (NAASRA 1980). Operational experience has indicated that a taper length of approximately 200 metres is required for the merge on high volume two-lane carriageways. Taper lengths where the lane starts can vary from 70 metres to 200 metres.

The following problems may occur with overtaking lanes:

(i) There may be some minor front to rear accidents where the overtaking lane merges back into the two-lane section.

(ii) There may be extra delay and capacity constraints at the merge with very heavy traffic volumes.

An auxiliary lane must be clearly allocated to one direction of travel (NITRR 1984). If there is not a priority direction on an auxiliary lane, the effect is to concentrate the faster vehicles of the two opposing traffic streams into the common centre lane, with no distinction as to which of the two opposing vehicles has right of way. A lack of clarity of right of way can cause three-lane roads to be unsafe.

It is unwise to have a number of short lengths of dual carriageway in proximity as this can cause confusion to drivers. When an isolated length of dual carriageway is provided, such as over a particularly hilly section, or as an expedient to relieve traffic queuing, it is undesirable to have a length less than about 3 kilometres (NAASRA 1980).

11.3.2 Non-motorized transport

Where non-motorised vehicles are involved, the following considerations should be taken into account. Pedestrians, bicycles, animal-drawn carts, etc, travel very much slower than cars and trucks. This difference in speed can lead to safety problems and delays to motorised traffic.

If the amount of non-motorised traffic is small, it may be satisfactory to allow it to use the pavement or the shoulder. At higher flows, it may be necessary to provide a separate track. Nigeria prohibits animal transport from multilane highways and provides a separate track (Federal Ministry of Works and Highways 1972). Odier et al (1971) suggest that, wherever possible, consideration should be given to separate roadways for animal-drawn vehicles since these may be a major highway hazard in some developing countries.

In developed countries, consideration is often given to bicycles. At low volumes these will stay at the side of the road and not interfere greatly with other traffic. Where the volume of cycle traffic exceeds about 1000 per day, cycle tracks should be considered on safety grounds (Paisley 1968, Ministry of Transport et al 1968). Cycle tracks are usually designed for one-way traffic and should have a minimum width of 2 metres. If the peak flow is above 500 per hour, the track should be widened by 1 metre for each additional 500 cycles per hour.

It is essential that tracks for slow moving traffic have a well drained, good riding surface, otherwise this traffic will continue to use the carriageway.

11.3.3 Passing places

NAASRA (1980) recommend that, where single-lane roads have shoulders, there is sufficient space for passing but, on roads without shoulders, it is necessary to widen the road periodically to provide passing places.

For low standard single-lane roads, Japan (Ministry of Construction 1981) specifies that wherever possible passing places are to be provided such that:

(i) The distance between adjacent passing places is 300 metres or less.
(ii) The major parts of the roads between passing places have unobstructed sight distance from the passing place.
(iii) The length of passing place is 20 metres or more and the width of the roadway of these sections is 5 metres or more.

The Papua New Guinea Department of Works (1985) specifiy a minimum of two passing bays per kilometre.

11.4 DISCUSSION AND RECOMMENDATIONS

Auxiliary lanes can allow opportunities for faster vehicles to overtake. The benefits from the lanes will increase with greater speed differentials and in circumstances where flows or terrain otherwise restrain overtaking. Such lanes ease congestion and reduce accidents, and may be very worthwhile economically.

Very slow and vulnerable traffic should be segregated from higher speed motorized flows where possible.

On single lane roads passing places should be provided at locations of lowest cost where possible, and should be sufficiently frequent that vehicles at adjacent passing places can see each other, to avoid unnecessary reversing.

12. SPEED-RESTRICTING DEVICES

12.1 ROAD HUMPS

The object of road humps is to reduce vehicle speeds to a level, below which travel should be comfortable but, above which, it should be increasingly uncomfortable. However, road humps must neither damage vehicles nor cause drivers to lose control, no matter what the speed.

There are two basic classes of humps: those which are short enough to be straddled by the wheels of all normal vehicles, and longer humps which cannot be straddled except by a minority of large vehicles (Watts 1973). Short humps administer a sharp jolt to the vehicle suspension except at very low speeds when the crossing time is long enough for the vehicle body to be deflected upwards as each axle passes over the hump. At higher crossing speeds the vehicle's tyres and suspension tend to deflect more and there is less deflection of the body. There can be considerable impact loads caused by short humps and the maximum usable height is limited by the ground clearance of low-slung vehicles. Long humps provide a less severe ramp effect and a longer crossing time; a greater height may be used without fear of grounding low-slung vehicles. The main effect of long humps is to cause a vehicle body deflection rather than a rapid deflection of tyres and suspension.

Watts (1973) found that the smallest humps, up to 100 millimetres in length and 13 millimetres in height, were not effective in alerting the drivers of all vehicles. The main difficulty with short humps is that they are generally less noticeable and more comfortable when taken at higher speeds. Increasing the height would make the humps more noticeable but could lead to greater hazards. Increasing the length of a hump tended to reduce the hazard and produce the desired characteristics. Longer humps, even if they were higher, reduced the stresses imposed on a vehicle's suspension, and they were more comfortable when traversed at lower speeds. A hump 3.66 metres long and 0.10 metres high (12ft x 4in) produced an uncomfortable ride in most vehicles at speeds in excess of 32 km/h. At 8 km/h drivers of all types of vehicles could cross this hump with comfort.

Watts (1973) recommended that a hump should be a segment of a circle 3.66 metres long and 0.10 metres high. This would be appropriate for restricting the speed on residential roads in the UK to below 48 km/h. It is possible to vary the height of the hump to influence the speed of vehicles to the desired level.

The other important design parameter for humps is the spacing between them. When traversing a series of humps, the lowest speed occurs at the humps. Between the humps vehicles accelerate for about 60 per cent of the distance and then decelerate to negotiate the next hump. Thus the maximum speed is determined by the speed over the hump and the distance between humps, the hump crossing speed had a relatively small effect on the maximum speed attained between the humps, when the crossing speed was less than 24 km/h.

Sumner and Baguley (1979) and Baguley (1981) found road humps to be beneficial in reducing speed. They also found a significant reduction in accidents. However, they recommended that humps should not normally be located in an area where a vehicle can enter at high speed, unless there is adequate warning. Humps also cause delays to emergency vehicles and scheduled bus services and can result in drivers choosing alternative routes.

12.2 RUMBLE STRIPS

Rumble strips are patches of rough or coarse road surface which are designed to produce vibration and noise inside vehicles with the intention of alerting drivers and causing them to slow down. In order to be sufficiently noticable, the rumble strip should produce a noise increase of at least 4dB(A). Watts (1977) found a surfacing of 19 millimetre stone in epoxy resin to be suitable. This produced a durable installation and a noticable increase in noise of 9dB(A) in cars and light commercial vehicles. Three years was found to be the practical limit before extensive repair or relaying was necessary (Sumner and Shippey 1977).

The second important design consideration is pulse length and gap between the pulses. Watts (1977) found that a regular series of 0.5 second pulses separated by 0.5 seconds was the best combination for alerting drivers. Design is based on the 85th percentile vehicle (Watts 1978). To achieve equal length pulses and gaps it is necessary for the rough area to be shorter than the smooth area. The lengths of rough and smooth area should be progressively decreased to encourage deceleration of the vehicles.

There was some evidence (Watts 1978) that rumble strips caused drivers to decelerate earlier before a junction and so reduced braking immediately before a junction. Watts (1978) found that rumble strips produced small reductions in mean speed. However, they appeared to have a relatively larger effect on faster vehicles causing a reduction in the variability of the speed distribution.

Rumble strips were noticed by virtually all drivers and more than 60 per cent took them to be an alerting device. Sumner and Shippey (1977) found that of those people who noticed the strips, 67 per cent did so by the vibration caused, 25 per cent by the noise and 8 per cent by sight. Sumner and Shippey (1977) found no beneficial effect in terms of speed reduction. There appeared to be a reduction in accidents, probably because of greater awareness.

12.3 YELLOW BARS

It is possible to manipulate the drivers sense of speed by introducing painted stripes across the road surface at an increasing frequency approaching a hazard (Burney 1977). These patterns enhance the drivers sensation of speed resulting in a reduction greater than otherwise would occur.

The criteria in the UK for the placement of the yellow bar pattern are that it must be possible to travel for at least 2 minutes at 97km/h immediately before the hazard and the negotiation of the hazard must require vehicles to slow down or stop. The main use of the yellow bar pattern has been on the approaches to roundabouts on dual carriageway roads. The pattern itself consists of 90 bars each 0.6 metres wide over a distance of about 400 metres. The bars span the full width of the carriageway. The clear distance between the bars starts at 6 or 7 metres at the upstream end and reduces exponentially to 2 or 3 metres at the downstream (hazard) end (Denton 1973).

Denton (1973) found a mean speed reduction of between 10 km/h and 17 km/h depending on the time of the day, with the greater effect during daylight. The pattern has a greater effect on fast moving vehicles. The speed reduction for the 85th percentile vehicle varied from 19 km/h to 27 km/h thus producing a narrower speed distribution.

Hellier-Symons(1981) found that the yellow bar pattern led to a substantial reduction in accidents. There also appeared to be a reduction in accident severity, probably due to the reduction in speed.

There was evidence that driver behaviour on the pattern differed according to site conditions (Burney 1977). Behaviour may also be influenced by prior encounter, however, Hellier-Symons (1981) found no evidence of a short-term learning effect. The effectiveness of the pattern was found to be sustained for at least 4 years after installation, indicating no long-term learning effect.

12.4 DISCUSSION AND RECOMMENDATIONS

Correctly designed devices can be successful in reducing vehicle speed and can therefore be recommended for use on roads where drivers may otherwise be travelling too fast. On rural roads in developing countries, the speed reduction may be useful when roads pass through villages or other speed-sensitive areas, and before sections of substandard geometry.

13. JUNCTION AND ACCESSES

13.1 BASIC CONCEPTS

Junctions and access points may affect the capacity and/or the safety of the road.

In general, the flow of the traffic through a junction will be disrupted, and this disruption will be greater if the road loses priority through the junction.

13.2 RESEARCH FINDINGS

Jacobs (1976) developed relationships linking junction spacing with accident rate in both Kenya and Jamaica. The relationship for the two countries were very different but both showed a clear increase in accident risk with an increased junction frequency. Also, much of the work on sight distance, described in Section C, is relevant.

13.3 STANDARDS IN USE

13.3.1 Capacity

For all-purpose roads in the UK, capacity is usually limited by that of the main junctions (Department of Transport 1985). On rural roads in developing countries, the roads are unlikely to be operating near capacity and most junctions will give priority to the main road, thus the capacity of the main road is not liable to be limited by the presence of junctions.

13.3.2 Location

Junctions on the inside of bends should be avoided, as sight distances from the minor road along the main road will be restricted and the junction may not be visible at all to drivers on the main road (Department of Works 1985). The provision of adequate sight distance is of prime importance if a junction is to operate safely. The driver of a vehicle stopped on the minor road must have adequate visibility along the major road, in both directions, to enable him to manoeuvre through the junction with confidence (Ministry of Transport et al 1968).

The location of junctions should be carefully selected to avoid steep profile grades, and to ensure that there is adequate approach sight distance to the junction (AASHO 1971). A junction should not be located on or just beyond a short crest vertical curve or on a sharp horizontal curve.

Drivers have been found to have difficulty in judging the additional distance required for stopping on grades (NITRR 1984). As a safety measure it is suggested in South Africa that junctions should not be located on grades steeper than 3 per cent. If it is not possible to align all of the legs of a junction to a gradient of 3 per cent or less, the through road is recommended to have a steeper gradient, because vehicles on the intersecting road will have to yield or stop, whereas through vehicles would only have to do so occasionally.

One of the consequences of a collision between two vehicles at a junction is that either or both may leave the road (NITRR 1984). It is therefore advisable to avoid locating a junction on high embankments. Also, for operational reasons, a junction should not be located on a curve with a superelevation greater than 6 per cent (NITRR 1984).

A considerable saving in accidents is likely to result by eliminating lightly trafficked side road connections onto main roads (Ministry of Transport et al 1968). For example, where two minor roads can be connected together before joining a main road, the accidents were found to reduce by about 30 per cent.

13.3.3 Sight distance

The provision of adequate sight distance is of prime importance if a junction is to operate safely. The driver of a vehicle stopped on the minor road must have adequate visibility along the major road, in both directions, to enable him to manoeuvre through the junctions with confidence (Department of Works 1985). The sight distance available to a driver approaching a junction should also be considered. He must be warned by signs that he is approaching a junction, and should have adequate forward sight distance to enable him to pull up safely at the stop line from the design speed. It is also desirable that a driver is able to perceive the layout of a junction whilst approaching it. A driver is less likely to overshoot a stop line and cause a hazardous situation if it is visible for a distance equal to the stopping sight distance of the minor road, measured from eye to road surface height.

In South Africa, a sight distance should be available, on the major road on both sides of the junction, equal to the decision sight distances. (NITRR 1984). Table 14.1 shows the decision sight distance measured from an eye height of 1.05 metres to the road surface.

Table 13.1: Design speed and decision sight distance

Design speed (km/h)	Decision sight distance (m)
40	130
50	160
60	190
70	215
80	240
90	270
100	300
110	325
120	350
130	380
140	410

13.3.4 Acceleration and deceleration lanes

Deceleration lanes are always advantageous, particularly on high speed roads, as they allow a driver of a vehicle leaving the major road to slow down away from the through traffic lanes (NITRR 1984). The length of deceleration lanes should be sufficient for vehicles to slow down from the average speed of traffic in the nearside lane to the speed necessary for negotiating the curve (Ministry of Transport et al 1968). Even if it is not practicable to provide the full length of deceleration lane, sub-standard lengths are still of great benefit.

Acceleration lanes are less useful than deceleration lanes, since entering drivers can always wait for an opportunity to merge without disrupting the flow of through traffic (NITRR 1984). They should be designed so that vehicles turning from the minor road may join the traffic flow on the major road at approximately the same speed (Ministry of Transport et al 1968). Their principal application is on high-volume roads where, in the peak hour, gaps between vehicles are infrequent and short. Acceleration and deceleration lanes will only be warranted at reasonably high flows.

Lanes may also be provided for the deceleration and storage of vehicles waiting to turn across the opposing flow. However, McCoy et al (1985) found that such lanes were not justified on approaches with volumes of less than 2500 vehicles per day on roads with unpaved shoulders. On approaches with paved shoulders, these lanes were never necessary with volumes of less than 4500 vehicles per day.

14. **BRIDGES AND UNDERPASSES**

Bridges and underpasses are expensive and it is desirable to minimise their construction cost. It is also desirable to ensure that the structure is appropriate for future needs as well as present needs (Odier et al 1971). To a certain extent these desires may be conflicting.

By carefully selecting alignments it is often possible to minimise long spans, large angles of skew, splays and tapers, small radius curvature, and deep foundations, which all increase structural costs (NAASRA 1976).

In most cases, the carriageway width under and over bridges should be sufficient to carry the full width of pavement and shoulders (NAASRA 1976). In general, it is desirable that a clearance of not less than 1.2 metres be provided between the face of a bridge structure and the edge of the adjacent traffic lane.

In Australia, single lane bridges with a width between kerbs of not less than 3.7 metres may be used where the expected traffic volume is less than 100 vehicles/day. In normal circumstances the width between kerbs is recommended to be at least 6.2 metres (NAASRA 1976).

15. SIGNING AND ROAD MARKING

15.1 SIGNS

Signs can be divided into three categories: regulatory, warning and informatory. Informatory signs allow drivers to know where they are and which route to take, but do not generally affect the way in which vehicles negotiate hazards although poorly placed informatory signs can mislead drivers and lead to dangerous situations. Regulatory and warning signs are more important in this respect, and must be positioned so that they can be seen in time for appropriate action to be taken.

It has been argued (Robinson 1981) that warning signs which inform the driver of hazardous conditions are a blatant admission of failure on the part of the road designer, but nevertheless are very useful.

15.2 CENTRE LINES

Lane markings make for more efficient and safe use of the carriageway: drivers prefer marked lanes because of the security they provide (Jones 1967). On a two-lane road, the centre-line stripe removes all doubt as to the proper positioning on the carriageway.

The marked centre-line and, desirably, any central longitudinal joint should be placed mid way between the edges of the pavement, even if the pavement has been widened to negotiate a curve (AASHTO 1984).

Research by Downing and Tahir (1986) showed that centre line markings influenced driver behaviour on curves in Pakistan. However, the influence was relatively small with the percentage of drivers crossing the centre of the road falling from 18.6 to 14.4 with the provision of a centre line. Driver training and enforcement were considered key problem areas over a wider spectrum of driver behaviour.

15.3 EDGE LINES

Edge lines mark the division between the pavement and the shoulder. They discourage traffic from travelling on the shoulder and make driving safer and more comfortable, particularly at night (Smith 1976). They also provide a continuous guide for the driver past objects which are close enough to the edge of the pavement to constitute a hazard.

Middleton (1976) found that on 5.5 metre wide pavements, shoulder damage and maintenance costs were less for pavements with edge lines. He claimed, with a 100 per cent margin for error that, on average, the savings in pavement maintenance due to edge lines would at least exceed the cost of their provision. On roads with edge lines, wheel positions showed less variation than on roads without edge lines and vehicles drove closer to the edge of the pavement.

When negotiating curves, Johnston (1983) found that the majority of the variance in vehicle position was accounted for by the nature of the road. Delineation effects were small, although wider edge lines (150 mm) had a more beneficial effect than narrower edge lines (80 mm).

Thomas (1957) found edge lines to have little effect on vehicle placement on a 24 foot (7.3 metre) carriageway and did not appear to effect the accident situation. They did, however, appear to have a psychological effect on the driver: 89 per cent of drivers were aware of them and 97 per cent of these believed that the line helped them in driving, particularly at night and during rain. He concluded that the psychological effect on a majority of drivers was the only benefit from edge lines.

Middleton (1976) found that drivers drove closer to the edge of the pavement when edge lines were present. He suggested that edge lines might reduce the possibility of a head on collision for this reason, although significant data could not be found to support this contention. Thomas (1957) found that edge lines did not appear to improve the accident situation, but also found that drivers tended to move towards the centre of edge-marked pavements.

16. **MAINTENANCE**

16.1 EFFECT OF WIDTH ON MAINTENANCE COST

Increased width produces a greater area to be maintained, thus increasing costs; but increased width also spreads the traffic loading over a greater area, thereby reducing deterioration. From a maintenance point of view, there is likely to be some optimal width.

On narrow pavements, there is a greater concentration of wheel loads and this concentration is closer to the edge of the pavement (SATCC 1986, AASHO 1965). A narrow pavement is also likely to increase the number of shoulder incursions, which will deteriorate both the pavement edge and the shoulder, particularly when the shoulders are unpaved (Oglesby and Altenhofen 1969).

AASHO (1965) state that the extra cost of 12 foot lanes over 10 foot lanes is offset to a large extent by a reduction in the cost of shoulder maintenance and a reduction in surface maintenance due to lessened wheel concentrations at the edges of pavements. Shannon and Stanley (1978) found 7.3 metre pavements to be marginally cheaper in terms of maintenance than 6.1 metre pavements. Oglesby and Altenhofen (1969) report that, for pavements wider than 24 feet, the cost of maintenance is substantially increased with increases in width. Below this width, the amount of edge patching will increase and, while there may still be a reduction in maintenance cost, the reduction will be smaller.

Australian practice has been to have paved roads with gravel shoulders to minimise construction cost. However, this practice increases maintenance costs as it causes edge problems, and moisture entry through permeable shoulder material may serve to weaken the pavement structure. (Armour and McLean 1983) concluded that shoulders should be paved, but need not be wide.

SATCC (1986) recommend that the paved width should exceed the lane width in order to reduce the cost of shoulder maintenance and lessen the wheel concentrations at the pavement edges. However, part paving of shoulders can leave a narrow strip unpaved which cannot be maintained by grader (Armour 1984b).

16.2 MAINTENANCE OPERATIONS

Maintenance operations will often require that the whole, or part, of the carriageway is closed to traffic. From an operational point of view, it is desirable that traffic can still use the carriageway during maintenance. These considerations demand a wide pavement and shoulders which are structurally capable of carrying traffic in all weather conditions (Mathews and MacLean 1977). For this reason paved shoulders provide more flexibility for traffic diversions at roadworks (Armour 1984b).

17. CONSISTENCY OF STANDARDS

On a well designed section of road in an area with generally uniform topography, a driver tends to have a speed expectancy (McLean 1979). This expectancy should be reinforced by designing curves to an approximately uniform standard (Robinson 1981). It is important that drivers are not presented with something unexpected; it is often more important that alignment standards are consistent than that they are high. Road Research Laboratory (1965) results show that inconsistency of the horizontal alignment of a road significantly increased accident rates, which were affected not only by individual curve radius and average horizontal curvature, but also by the combination of the two. A sharp curve on an otherwise straight alignment would cause a higher accident rate than that on an alignment with a high degree of bendiness (International Bank for Reconstruction and Development 1978).

McLean (1979) recommended that the predicted speed for curves occuring at the ends of long straights should desirably be not more than 10 km/h, and definitely not more than 15 km/h, below the 'speed environment' of the road section. Where this criterion cannot be met, the sharp curve should be approached by a series of curves each successively sharper than the previous one (Ministry of Work and Supplies 1978). This technique can also be used when a change in topography or some other constraint necessitates a change in alignment standard. The 'speed environment' relevant to each curve can be taken as the predicted speed of the previous curve. McLean (1979) stated that a 10 km/h variation in predicted curve speeds should be treated as the absolute maximum variation in curve standard. Indeed, NAASRA (1980) employs a consistency check in geometric design which ensures that the design speeds of successive geometric elements should not differ by more than about 10 km/h.

McLean (1983b) argues that, on low-speed alignments, many drivers operate with much smaller safety margins than those traditionally assumed for design purposes. However, through experience, drivers have learnt to operate with their own safety margins. Thus as long as nothing unexpected occurs these drivers can operate safely on roads which in theory do not conform to recognised safety standards. For this reason, McLean (1981) has recommended that standards should be set for alignment consistency rather than stipulating minimum standards for individual elements.

It is not only important that road geometry is internally consistent; it must also be 'externally consistent'. A road which is built to a design philosophy radically different to that of the existing network may prove to be unsafe, or may cause other roads in the network to become unsafe, due to changing driver expectations.

18. **RELAXATION OF DESIGN STANDARDS**

18.1 BASIC CONCEPTS

The design standards of many countries ensure that the design of the geometry of a road will permit a vehicle travelling at a design speed to negotiate all the geometric features with an adequate safety margin. On any road there will be a distribution of free vehicle speeds which usually approximates to the normal distribution. The design speed must be safe for the majority of vehicles, and is usually taken as the speed of the 85th percentile vehicle.

Design speed determines geometric charteristics such as gradient and curvature. In some cases, the requirements can be easily attained whilst, in other situations, the standard can only be achieved with a high additional cost. Adhering strictly to the standards can be prohibitively expensive, and in such circumstances the designer may be faced with the choice of either relaxing the standards or not building the road (International Bank of Reconstruction and Development 1978).

Relaxing standards will reduce construction costs, but may also produce a deterioration in traffic performance. The determination of standards and the consequences of any relaxation below these standards are key issues.

18.2 RESEARCH FINDINGS

In a case study of a road in Tasmania, McLean (1980a) found that construction costs typically increased by about 9 per cent for a 10 km/h increment in design speed. He also found that increasing the width standard added significantly to the cost.

Simpson and Kerman (1982) found that additional width on single carriageways up to 10 metres wide improved safety and driver comfort as well as reducing operating cost, although this was on roads without shoulders. The International Bank for Reconstruction and Development (1978) found that increases in pavement width offered diminishing returns once there was an adequate clearance distance between two oncoming vehicles. Successive reductions in width had a progressively more severe effect on traffic operation.

Oglesby and Altenhofen (1969) found no factual evidence to support the impression that narrow roads, or those without shoulders, were dangerous. They postulated that, because the situation seems dangerous, drivers are more alert and accidents are thereby less likely to happen.

For speeds below 80 km/h, Oglesly and Altenhofen (1969) found that driver behaviour did not conform to safe practices as determined by the design standards for sight distance. However, curvature had a significant influence on driver speed whereas restrictions on sight distance had far less effect. Drivers were found to weigh restrictions on horizontal sight distance more heavily than those on vertical sight distance. They also found strong evidence that accidents occur at occasional sharp curves in relatively straight alignments.

Simpson and Kerman (1982) found evidence that departures from stopping sight distance standards within one or two design speed steps did not have a significant impact on safety.

18.3 STANDARDS IN USE

In the UK, design standards relate to design speed. There are six design speeds which are related to each other by a multiplying factor of approximately 1.2 (Department of Transport 1981a). These design speeds are:-

Design Speed (km/h)	50	60	70	85	100	120

The free speed distributions are such that if the 85th percentile speed matches one of the design speeds, the median speed will be one design step lower, ie. if the 85th percentile speed is 100 km/h, the median speed will be 85 km/h. If the design speed of the road is 100 km/h and, because of high cost, a section is designed for 85 km/h, one relaxation will have been made from the standard. A design for a speed of 70 km/h would be two relaxations (Kerman 1980). The policy in the UK is to consider departures when the cost or environmental implications of maintaining the standards are high, although such departures are not encouraged where significant accesses or junctions occur (i.e. where there are most likely to be potential hazards).

Not all countries use the same system of design speeds, and therefore what is termed a relaxation or acceptable local reduction in standard, will vary between countries. However, the basic principle remains the same: a relaxation is when a section of road is designed to a standard lower than the design standard of the remainder of the road.

On single carriageway roads, sufficient visibility for overtaking should be provided on as much of the road as possible, especially where the traffic flows are expected to approach the design capacities (Ministry of Transport et al 1968). Where adequate overtaking sight distances are not provided throughout the length of two lane roads, the effect on capacity and speed are indicated below in Table 18.1.

Table 18.1: Effects of reduced overtaking opportunities

Percentage of road with less than standard overtaking sight distance	0	20	40	60	80	100
Percentage of standard design capacity	100	90	80	65	50	30
Estimated reduction in average speed (mph) of a two lane road carrying 900 pcu/hour	0	4	8	12	16	20

Oglesby and Altenhofen (1969) recommended that the requirement of a constant cross-section for the full length of a road segment should be abandoned for roads carrying less than 400 vehicles per day. Narrower carriageways would be considered on straights than on curves or over crests where sight distances are limited. This would substantially reduce the cost of construction. Also, when a road is to be reconstructed, this flexibility would permit spot improvements rather than complete rebuilding.

Horizontal curvature is a parameter quite closely correlated with sight distance, and design should usually aim to achieve at least desirable minimum values of radius. However, relaxations down to absolute minimum values can be made safely whenever cost or environmental savings are more than trivial (Department of Transport 1981a).

Any danger resulting from curvature is related more to the relationship of that radius to the rest of the road or the degree of relaxation, than to the absolute value of the radius. For example, a sharp curve located in level terrain or at the end of a long tangent is many times more dangerous than the same type of curve located in mountainous terrain (International Bank for Reconstruction and Development 1978).

In hilly terrain, the adoption of gradients steeper than the desirable maximum could make significant savings in environmental costs but would also result in higher user costs. On all purpose roads gradients greater than 8 per cent should be considered as departures from standard.

18.4 DISCUSSION AND RECOMMENDATIONS

Standards should not be taken to be absolute, and sensibly applied relaxations can result in substantial construction cost savings, with little additional accident risk. Such relaxations or local reductions in standards should therefore be permitted, provided that they are not introduced in such a way as to be unexpected by the driver. There is no firm evidence as to the most appropriate levels of relaxation for use in a developing country although the use of one or two design speed steps as recommended by Simpson and Kerman (1982) would seem acceptable.

In many situations the reduction in construction cost by the relaxation of design standards may be more than offset by an increase in vehicle operating and maintenance costs. The minimum total cost solution should be adopted.

19. RECOMMENDATIONS FOR FURTHER WORK

Only a small amount of research has been undertaken in developing countries into design standards. Thus, many of the recommendations in this report are based on studies in developed countries, in which only some of the findings are directly relevant. Also, even when the conditions are comparable, driver behaviour and vehicle performance may be dissimilar in developed and developing situations. Therefore, whilst the standards derived from this review are considered to be those most appropriate to the circumstances, further research is required to fully confirm their applicability and refine the recommended values.

Areas in which specific research is recommended are into safety, operational performance and economic evaluation.

In the long term, detailed accident records which may be related to geometric and flow characteristics are essential to define specific situations in which changes in standards would be likely to produce real and immediate accident savings. This process is already underway, particularly with the introduction of the TRRL Microcomputer Accident Analysis Programme (Hills and Elliott 1987). In the short term, specific studies into the effect of variations in geometric elements on driver performance may be able to give an indication of potential risk. These include:

(i) The speed reduction and lateral placement of approaching vehicles on a range of geometric elements with lower design standards. The effectiveness of centre line markings is also important.

(ii) The use/misuse of the shoulders for different standards and conditions. Trial sections with a range of treatments to indicate shoulder delineation would be advantageous. The effectiveness of shoulder treatment and width for pedestrian safety should also be considered.

(iii) The effectiveness of speed control devices to limit speeds, and their effects on safety, particularly where pedestrians are crossing.

(iv) The provision of safe facilities for pedestrians and other vulnerable road users.

(v) The effects of successive reductions or increases in the design speeds of geometric elements on actual speeds should be investigated, for both safety and operational purposes. This may be best undertaken by using field studies to calibrate a simulation model such as those by Boyce and McDonald (1986), McLean (1978e), Morales and Paniati (1986) or Gynnerstedt and Johnsen (1981). Data would be required on freely moving vehicles as well as those constrained by lack of overtaking opportunities resulting from approaching vehicles and/or sight distance limitations.

A considerable amount of research is required on the operational performance of highways, particularly to assess:-

(i) The effects of slow moving vehicles in a traffic stream. This would include both slow moving trucks and buses as well as animal drawn vehicles.

(ii) The effects of length and steepness of gradient on delay and congestion.

(iii) The effects of stopped vehicles on flow. A vehicle parked on a narrow road will be likely to have a much more severe effect than if it were parked on a wide road.

(iv) The effects of restricted widths at bridges on flow and delay, as well as safety.

All of the above require empirical studies, but (i) and (ii) are paricularly amenable to understanding with the aid of simulation models.

An underlying objective of the setting of geometric design standards is to achieve a satisfactory economic construction. The present economic models of RTIM2 and HDM-III consider only free flowing conditions. Two key aspects of their development should thus be the introduction of congestion effects and research into associated vehicle operating cost relationships.

APPENDIX

THE EFFECTS OF CLIMBING LANES

A1 INTRODUCTION

Laden commercial vehicles will normally be slowed by long, steep gradients. Such speed reductions may substantially affect the speeds and journey times of other faster vehicles, particularly if opportunities to overtake are limited. The level of delay encountered may vary with length and steepness of gradient, levels and composition of traffic, and geometric alignment on the gradient and adjacent section of highway. In developing countries, the situation is often exacerbated by trucks and buses operating with very low power-to-weight ratios, and by the presence of other factors, such as poor carriageway delineation, pedestrians, animals and animal drawn vehicles.

In general, the introduction of a climbing lane will increase benefits through journey time savings. Other potential savings are reductions in vehicle operating costs and accidents. Reductions in vehicle operating costs are difficult to quantify, and the present relationships for developing countries are based on average geometric conditions which cannot be applied with confidence to individual geometric elements, nor in situations in which traffic congestion exists. Whilst the real costs of accidents in developing countries has been shown to be substantial, no predictive relationships exist whereby possible accident reductions can be related to the design of geometric elements with any certainty. Indeed, without good driver discipline, clear road markings and appropriate detailed design, it is possible that climbing lanes may increase the number or severity of accidents. Until there is clearer research evidence, the benefits from climbing lanes have to be identified solely from journey time savings.

Journey time savings from the introduction of climbing lane should be used with local unit values of time to derive levels of benefit. These can then be compared to the additional costs of construction.

In this study, a purpose built simulation model (Boyce and McDonald 1986) has been used to estimate the effects of climbing lanes in a range of situations. The advantage of simulation over other methods is that it enables the effects of changes in traffic and geometric variables to be estimated in a structured way. The main disadvantage is that, however carefully the model has been calibrated, the results may not exactly represent the particular situation being modelled. However, the lack of relevant empirical data precludes other approaches, and other similar studies have also made use of a simulation approach (Gynnerstedt and Johnsen 1981, Brodin <u>et al</u> 1979).

A2 THE SIMULATION MODEL

A2.1 Structure

The model was developed by the University of Southampton to simulate traffic movements on single and dual carriageway roads in developing countries. Consideration of the opposing flow enables a range of situations to be represented, including single carriageways and climbing lanes. A time-based simulation with a fixed time advance of one second has been adopted.

The model has been written in Fortran and consists of five sections. These are linked together by the program SIMULATE as shown in Figure A1. The five sections and the subroutines used in each are listed below:-

Program Section	Subroutines
Data Input	SECTION DATA
	FLOW DATA
	VEHICLE DATA
Vehicle Movement	SIGHT
	MODE
	FREEFLOW
	CAR FOLLOW
	UPDATE
Overtaking	START PASS
	OVERTAKE
	END PASS
	ABORT
	CAR FOLLOW 2
	REVISE FOLLOW
Entry and Exit Conditions	ENTRY
	START CLOCK
	LEAVE
	REMOVE
Opposing Flow	OPPOSITION

A2.2 Possible Simulations

The simulation program is able to model the following situations for a range of flows and compositions:-

(1) Test Section. A highway link may be composed of up to 20 adjoining sections, each of which has a constant set of geometric features. These features include gradient (positive or negative), road width (speeds decrease when width is below 5 metres), curvature (in terms of degrees/kilometre), verge width (used in the calculation of sight distance), superelevation and road roughness. 'Warm-up' and 'cool-down' sections of length 1 kilometre have been used at the beginning and end of the test section to overcome problems that 'end effects' might otherwise cause.

(2) Vehicles. The model is currently capable of handling the following different vehicle types: small cars, large cars, light goods vehicles, buses, light trucks, medium trucks, heavy trucks and aritculated trucks. Each individual vehicle has a series of characteristics associated with it and slower moving animal or man powered vehicles may also be readily included. These attributes are selected as the vehicle enters the simulated section by random sampling from input distributions which are dependent on the vehicle type.

The vehicle characteristics selected at entry include vehicle length, vehicle weight, stopped gap (the distance a driver will leave between the front of his vehicle and the rear of the preceding vehicle when stationary), deceleration (for decelerating into curves, etc), maximum acceleration from rest on a level road, overtaking safety margin (in terms of time), desired speed (separately for paved and unpaved roads), maximum used driving power, maximum used braking power, friction ratio and maximum average speed.

A2.3 Speeds

Free flow speeds of each vehicle are constant over sections of constant geometry. A freely moving vehicle starting a section at a speed other than the free flow speed will accelerate or decelerate until it reaches this speed. The free flow speed is determined as the minimum of the desired speed, the speed as restricted by driving power, braking power, curvature and roughness. Driving power restricted speed is determined by the maximum driving power, vehicle weight and section gradient, and represents the maximum speed which the vehicle can sustain on a particular gradient. Braking power restricted speed is only important on downgrades, and represents the maximum safe speed of descent, based on the maximum used braking power, vehicle weight and section gradient. Curvature restricted speed is the maximum safe speed for negotiating bendy sections of road. It is based on the friction ratio of the vehicle, and the curvature and superelevation of the section. Roughness restricted speed depends on the roughness of the section and the maximum average speed of the vehicle. It is not comfortable to drive faster than this speed.

In many instances, vehicles will not be travelling in the free flow mode. For these situations, the simulation employs a full car following model.

A2.4 Overtaking

Vehicles which are not in the free flow mode are given the opportunity to overtake. In order to start an overtaking manoeuvre, a vehicle must be able to find a gap in the simulated flow which is sufficiently large to be able to re-enter safely; it must not be being overtaken by a vehicle in the offside lane and, in the case of single carriageways, there must be a sufficiently large sight distance and gap in the opposing flow to be able to complete the manoeuvre safely.

A2.5 Output

Output from the model is in two forms, the principal of which is a table of vehicle journey times over the section. This output may be used to draw speed/flow graphs. The subsidiary form is a plot of the speed profile of selected vehicles over the simulated section. This plot can show the vehicle type, whether the vehicle is in free flow or car-following mode, the ruling speed constraint (i.e. desired, driving power, braking power, curvature or roughness) and where overtaking manoeuvres occur.

A3 CLIMBING LANE SIMULATION

Climbing lanes were simulated with lengths of 100, 300, and 600 metres and for gradients of 4, 8 and 12 per cent. A zero gradient run was also undertaken to form a basis for comparison. The test section consisted of a 500 metre level straight, followed by the gradient section, followed by a 1000 metre level straight. Mean journey times and speeds were produced for the complete test section, as the effects of the gradient could extend some distance downstream, as well as upstream.

Traffic flow levels of 200, 400, and 600 vehicles per hour were considered, with equivalent levels of opposing flow. Three vehicle types were used with cars forming a constant 20 per cent of the total flow and with percentages of heavy vehicles of 20 and 40 per cent and light goods vehicles the balance. The mean free speeds of the different vehicle types on a level straight section of road were 97 km/h for cars, 76 km/h for light goods vehicles, and 61 km/h for heavy vehicles.

The travel time results of the simulation for a composition with 40 per cent and 20 per cent heavy vehicles are shown in Tables A1 and A2 respectively. Each of the values has been derived as the mean of over 1500 simulated vehicle journeys through the test section. Whilst the results generally slow logical and consistent trends, some random variability is evident, an inherent characteristic of simulation. The results in Tables A1 and A2 were used to estimate the likely increases in speed which would occur over a one kilometre length of road containing the gradient section.

The general findings of the simulation on the effects of introducing a climbing lane were as follows:-

1) The effects were negligible for flows of less than 200 vehicles per hour.
2) The effects were negligible for gradients up to about 100 metres in length.
3) Variations in the percentage of heavy vehicles from 20 per cent to 40 per cent had little effect. (This was largely the result of the greater delay to cars with the higher proportion of heavy vehicles being offset by the shorter delays to the additional heavy vehicles).
4) Speed increases were found to increase with both steepness and length of gradient as shown in Figure A2.

5) The effects shown in Figure A2 were estimated for flows of 200, 400 and 600 vehicles per hour in each direction. Speed increases were were found to increase by about 25 per cent when the flows increased from the lower flow level to 400/600 vehicles per hour on a 300 metre long gradient. On a 600 metre gradient, the equivalent increase was about 60 per cent.

A4 DISCUSSION AND RECOMMENDATIONS

As with all simulation models, calibration and validation are most important. So far, this has only been possible using information from the UK, with limited data from elsewhere. Further overseas studies have been recommended to complete this process and give the results of the model a wider credibility to supplement existing assessment models such as RTIM2 (Parsley and Robinson 1982) and HDM III (Watanatada et al 1985).

It is recommended that Figure A3 be used to estimate speed increases with the introduction of a climbing lane.

There are two main factors which should be taken into account when considering the use of the above recommendation. The first is that the simulation has been based on a straight section of road, with overtaking possible at all locations. The level of benefit of a climbing lane is therefore likely to be greater in most real situations where there are geometric limits on overtaking opportunities. In such circumstances, the provision of a climbing lane will allow the dissipation of queues built up on the approach sections, with similar additional benefits to those which would have accrued through an 'extra' lane at any other location. Secondly, the simulation model is based on average conditions, and is unlikely to apply directly to any single situation. It is recommended that, where possible, local information is collected to support any applications.

TABLE A1: MEAN TRAVEL TIMES OVER THE TEST SECTION
(40 per cent heavy vehicle composition)

GRADIENT	MEAN TRAVEL TIMES (SECONDS) Without climbing lane (With climbing lane) Flow (vehicles per hour per direction)		
%	200	400	600
100m Long gradient			
4	81.3 (81.3)	88.0 (88.0)	94.5 (94.5)
8	81.6 (81.6)	88.5 (88.5)	95.0 (95.0)
12	81.7 (81.7)	88.7 (88.7)	95.3 (95.2)
300m Long gradient			
4	91.7 (90.6)	99.6 (97.7)	107.1 (104.6)
8	92.8 (91.3)	101.3 (98.9)	108.9 (106.5)
12	93.4 (91.8)	102.2 (99.8)	110.0 (107.9)
600m Long gradient			
4	108.0 (105.5)	117.8 (112.8)	126.7 (120.1)
8	111.5 (108.2)	123.3 (116.5)	134.1 (125.2)
12	115.5 (110.8)	130.4 (120.6)	148.4 (133.1)

TABLE A2: MEAN TRAVEL TIMES OVER THE TEST SECTION
(20 per cent heavy vehicle composition)

GRADIENT	MEAN TRAVEL TIMES (SECONDS) Without climbing lane (With climbing lane) Flow (vehicles per hour per direction)		
%	200	400	600
100m Long gradient			
4	74.5 (74.6)	80.1 (80.0)	86.4 (86.4)
8	74.8 (74.8)	80.3 (80.3)	86.4 (86.4)
12	75.0 (75.0)	80.5 (80.4)	86.9 (86.9)
300m Long gradient			
4	84.0 (83.0)	90.8 (88.7)	97.6 (96.0)
8	84.7 (83.5)	91.6 (89.3)	98.9 (97.1)
12	85.1 (83.8)	92.2 (90.0)	99.7 (98.1)
600m Long gradient			
4	98.5 (96.3)	107.0 (102.4)	115.1 (109.2)
8	100.7 (98.0)	110.5 (104.7)	120.0 (113.6)
12	103.2 (99.7)	114.0 (107.1)	126.6 (117.2)

FIGURE A1: FLOW CHART SIMULATION PROCEDURE

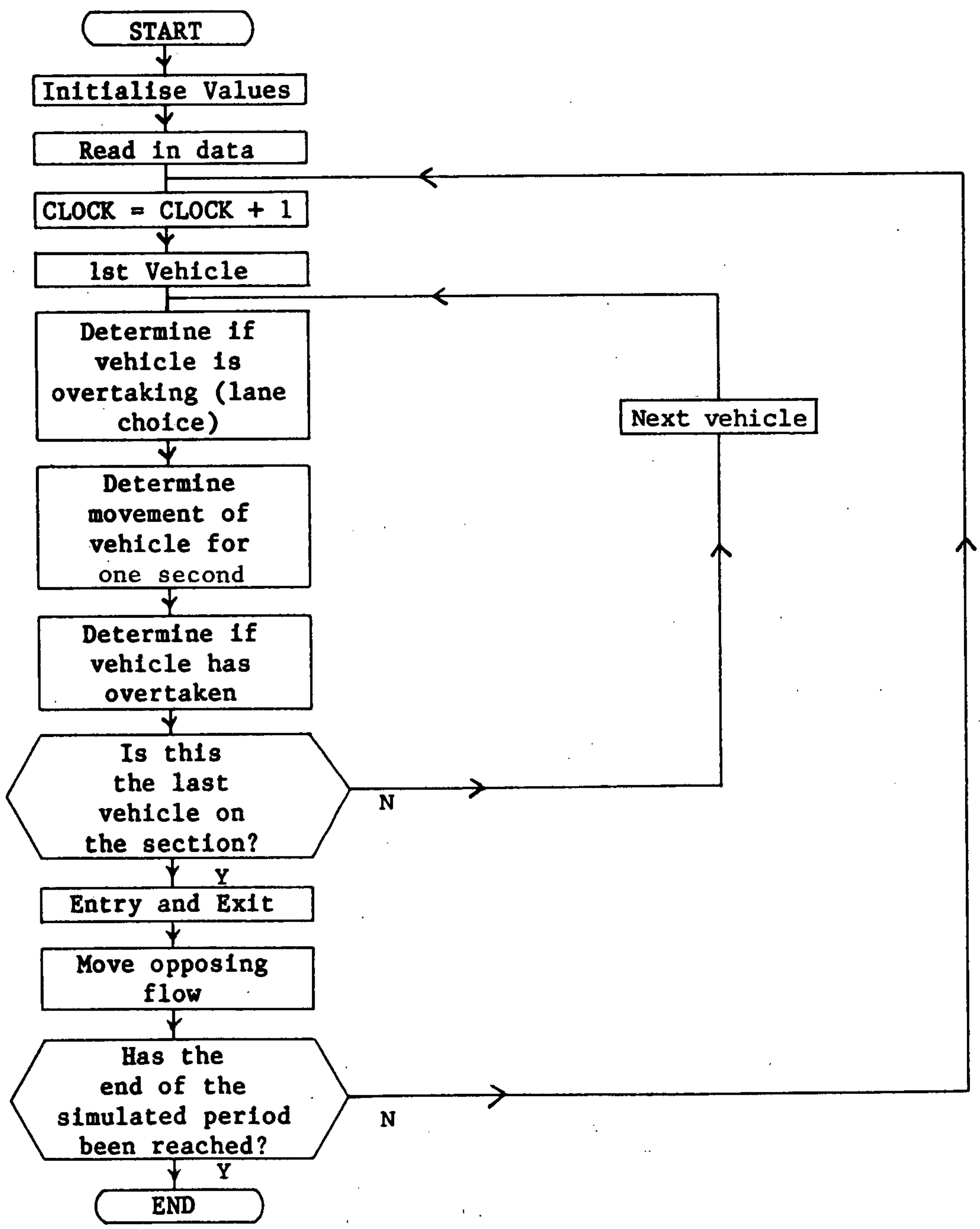
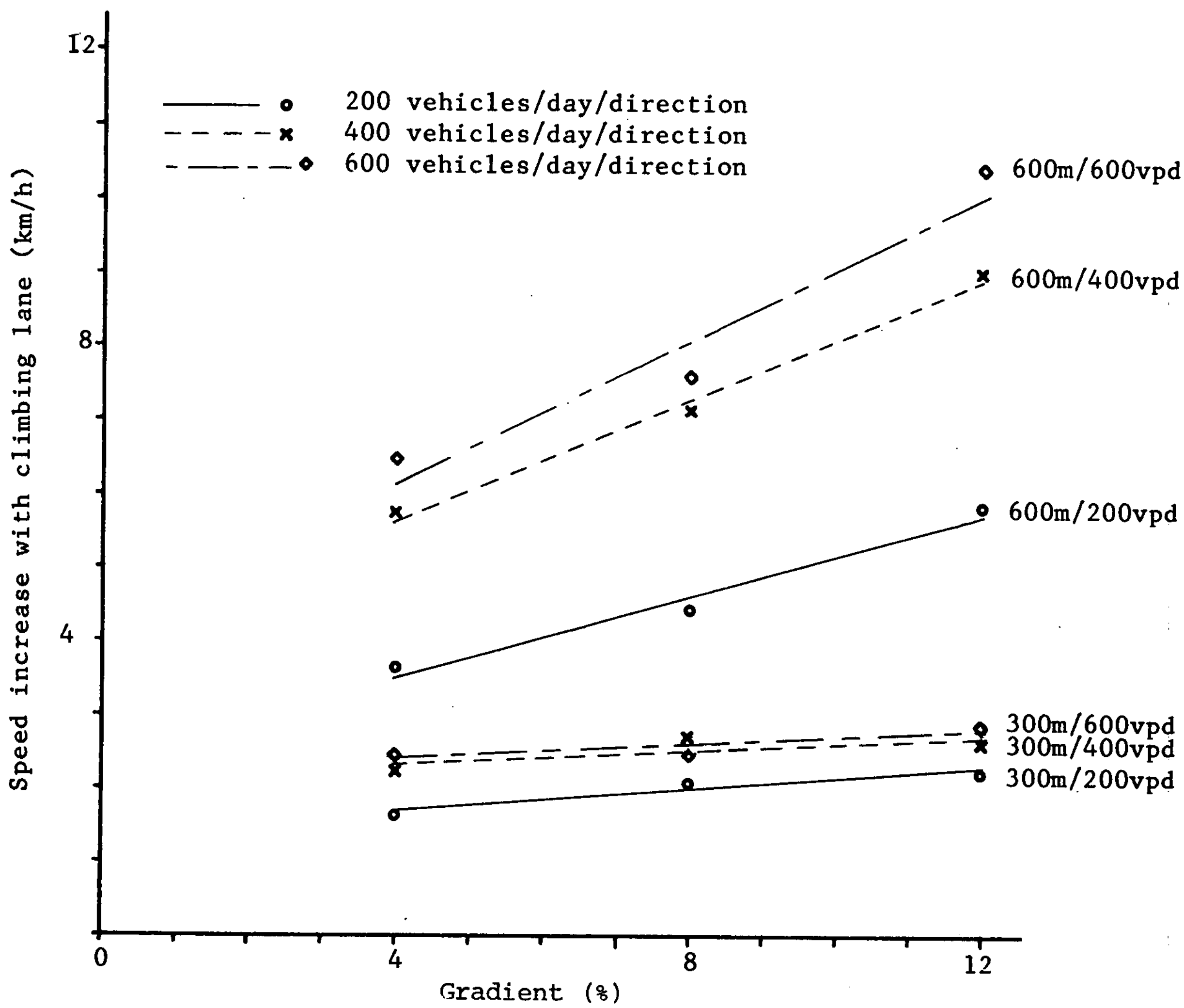


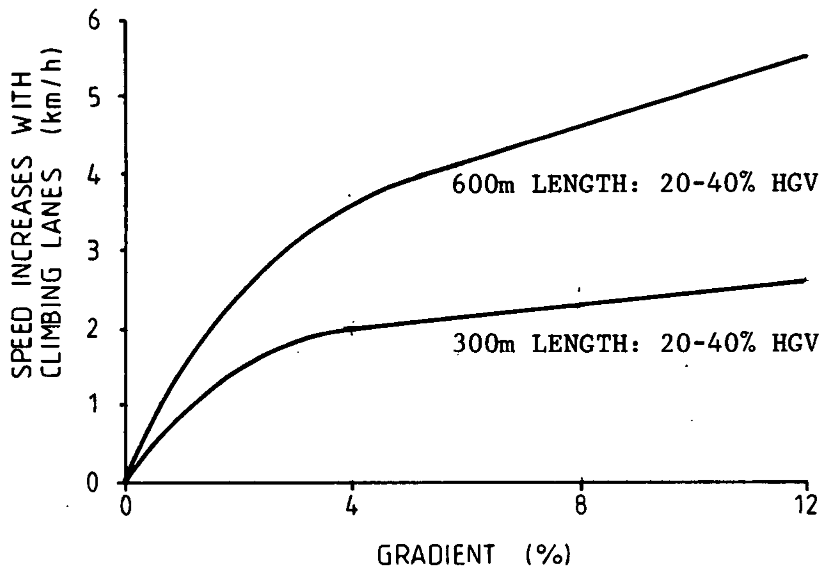
FIGURE A2:
SIMULATED SPEED INCREASES WITH CLIMBING LANES *

(40% heavy vehicles)



* The speed increases have been estimated as average values over a 1.0 kilometre section containing the gradient, but include the complete speed change effect estimated from the simulation over the test section including the 1.5 kilometres of 'run in' and 'run out'.

FIGURE A3: ESTIMATED SPEED INCREASES WITH CLIMBING LANES



Note:

1) The above results are estimations based on simulation. Vehicle performance and driver characteristics will vary from country to country and the assumptions incorporated here should be considered as coarse approximations.

2) Climbing lanes on gradients of up to 100 metres in length were shown to have little effect.

3) Varing the percentage of heavy vehicles (HGV) from 20 per cent to 40 per cent has little effect on the mean speed.

4) The above curves are based on directional flows of 200 vehicles per hour. For lower flows, the speed increases with a climbing lane were small, although for higher flows of 400 to 600 vehicles per hour, the speed increases were found to be about 25 per cent on a 300 metre gradient, and about 60 per cent on a 600 metre long gradient.

5) The speed increases shown in the above figure are values averaged over a 1.0 kilometre section of road which contains the gradient section.

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