TRRL Research Report 114

Transport and Road Research Laboratory

Department of Transport

A review of some recent geometric road standards and their application to developing countries

by D Kosasih, R Robinson and J Snell

Digest of Research Report 114

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Digest RR114 — 1987

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COMPARISON OF STANDARDS

Since 1980, Australia, Britain and the United States have all made major modifications to their recommendations for geometric design standards for rural roads (NAASRA 1980, Department of Transport 1981, AASHTO 1984). This report reviews the research that formed the basis of the current standards under the headings of design speed, sight distance, horizontal and vertical alignment, and cross-section.

The three standards are all based on the concept of design speed, but the application of this differs considerably between the standards. The AASHTO method of determining design speed is based on a qualitative assessment of traffic volume and terrain conditions. It has the objective of achieving consistency of standards commensurate with the function of the road, and a balance between construction and operating costs. NAASRA introduces the concept of 'speed environment' related to terrain and range of horizontal curvature along an alignment. The design speed of individual geometric elements is related to the speed environment and, on successive elements, should not differ by more than 10 km/h. The British design speed standard is based on overall 'alignment constraints' and 'roadside friction' values. Relaxation of standards is allowed on cost grounds, but these still provide acceptable levels of safety and operating conditions.

The key design chart for the NAASRA standard is shown in Figure 6 from the report and Figure 7 shows the key chart for the British TD 9/81 standard.

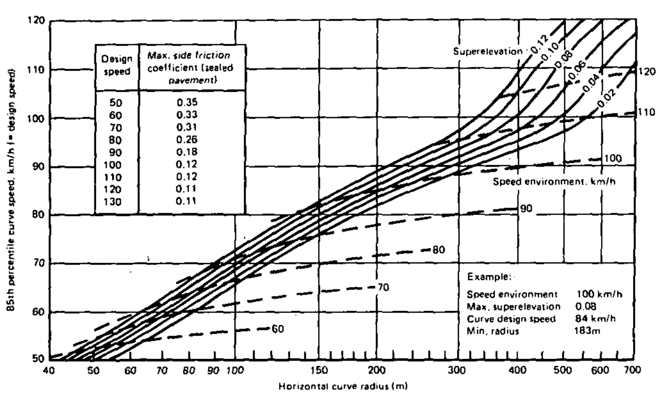

Fig. 6 NAASRA relationships for minimum curve radius

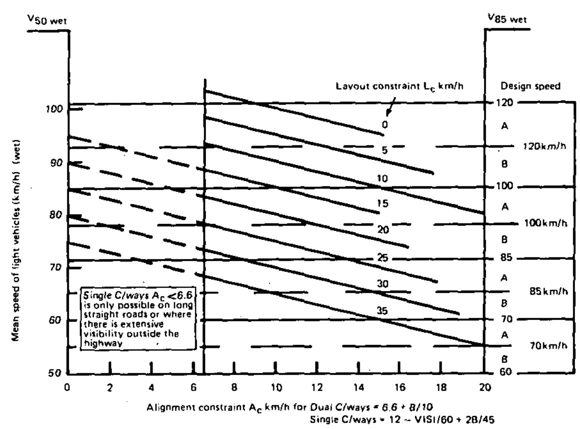

Fig. 7 TD9/81 Design chart

DEVELOPING COUNTRIES

Standards that have traditionally been applied in developing countries are also discussed. It is noted that traffic requirements, road safety and network considerations are different in developing countries and that, in order to develop local standards, it is convenient to define the objectives of road projects in terms of three levels of development. These are:

Level 1: to provide access;
Level 2: to provide additional capacity;
Level 3: to increase operational efficiency.

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 may be appropriate to design a variable width road where the cross-section is narrow on straights, but is increased on bends or where other restrictions on sight distance apply.

It is only when the objective of a road is to increase the operational efficiency of a route (Level 3) that standards such as those developed by AASHTO, NAASRA or the UK Department of Transport become relevant. It is not normally practicable to apply standards such as these to roads at Levels 1 or 2. Because the requirements of roads in developing countries are different to those in the industrialised countries where these standards were developed, the three standards should only be applied with caution in developing countries, even to Level 3 roads.

APPLICATION OF STANDARDS

Before American, Australian or British standards are applied to Level 3 roads in developing countries, it is necessary to review the assumptions on which the standards have been based to determine where they are appropriate for conditions found in individual countries. To assist with this task, this report reviews the principal assumptions in the three standards to determine which aspects of each might be appropriate in developing countries.

Guidance is thereby given on how to adapt standards from the industrialised countries for use until such time as specific standards have been developed that are appropriate for use in developing countries.

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AASHTO, 1984. A policy on geometric design of highways and streets. Washington DC: American Association of State Highway and Transportation Officials.

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NAASRA, 1980. Interim guide to the geometric design of rural roads. Sydney: National Association of Australian State Road Authorities.

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TRANSPORT AND ROAD RESEARCH LABORATORY
Department of Transport

RESEARCH REPORT 114

A REVIEW OF SOME RECENT GEOMETRIC ROAD STANDARDS AND THEIR APPLICATION TO DEVELOPING COUNTRIES

by

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Overseas Unit
Transport and Road Research Laboratory
Crowthorne, Berkshire
United Kingdom
1987

ISSN 0266-5247

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A REVIEW OF SOME RECENT GEOMETRIC ROAD STANDARDS AND THEIR APPLICATION TO DEVELOPING COUNTRIES

ABSTRACT

Since 1980, Australia, Britain and the United States have all made major modifications to their recommendations for geometric design standards for rural roads. This report reviews the research that formed the basis of the current standards under the headings of design speed, sight distance, horizontal and vertical alignment, and cross-section. Standards that have traditionally been applied in developing countries are also discussed.

It is noted that traffic requirements, road safety and network considerations are different in developing countries and that, in order to develop local standards, it is convenient to define the objectives of road projects in terms of three levels of development of the road network. These are:

Level 1: to provide access;

Level 2: to provide additional capacity;

Level 3: to increase operational efficiency.

It is only when the objectives of the road are at Level 3 that standards such as those developed in Australia, Britain and the United States are relevant and the principal assumptions in these standards are reviewed to assist in their adaptation to roads in developing countries.

1 INTRODUCTION

Geometric design is the process whereby the layout of the road in the terrain is designed to meet the needs of the road users. The principal geometric features are the horizontal alignment, vertical alignment and road cross-section. The use of geometric design standards fulfills three objectives. Firstly, the standards ensure minimum levels of safety and comfort for drivers by the provision of adequate sight distances, coefficients of friction and road space for vehicle manoeuvres; secondly, they ensure that the road is designed economically; and, thirdly, they ensure uniformity of the alignment. The design standards adopted must take into account the environmental conditions of the road, traffic characteristics and driver behaviour. The interdependence between these factors and the geometric features is summarised in Table 1.

Since 1980, Australia, Britain and the United States have all made major modifications to their recommendations for geometric design standards for rural roads. This report reviews the current standards under the headings of design speed, sight distance, horizontal and vertical alignment, and cross-section. The Australian standards were published by NAASRA (1980) as an interim guide, the British code was produced as Departmental Standard TD 9/81 (Department of Transport 1981) with subsequent background information (Department of Transport 1984) and amendments, and the American standards were published as a policy document by AASHTO (1984).

A study to develop appropriate geometric design standards for use in developing countries is being undertaken by the Overseas Unit of TRRL. As a first step in this work, a comparison between the recent American, Australian and British standards has been carried out. This Report describes the findings of this preliminary study and discusses the potential for applying these industrialised country standards in developing countries.

2 DESIGN SPEED

2.1 DRIVER BEHAVIOUR AND EXPECTATION

In design guides, a design speed for a particular road classification is usually selected according to the terrain and traffic volume. To provide consistency of the design elements, general controls for the horizontal alignment, the vertical alignment and the combination between them are given. It is also recommended that the design speed chosen should be consistent with the speed a driver is likely to expect. It is this issue which causes difficulty when applying the standards since, except for reference to typical speed distributions of a general nature on similar facilities already built, designers generally have insufficient information available to them to take account of the actual behaviour and speed expectations of drivers on the different alignment elements along a section of road.

Designing according to the design elements permitted by a specified design speed does not necessarily ensure alignment standards consistent with driver behaviour. This is because drivers tend to vary their speeds along the road especially when negotiating different horizontal curves.

TABLE 1

Driver, vehicle and road characteristics in geometric design standards

Geometric design standard	Driver characteristics considered	Vehicle characteristics considered	Road characteristics considered
Minimum safe stopping distance	Perception—reaction time	Layout of controls, braking systems, tyre condition, tread pattern	Skid resistance of road surface, design speed
Minimum safe passing distance	Judgement of gap availability and vehicle capability	Acceleration capability	Design speed
Driver eye height	Physiology	Dimensions	—
Object height	—	Dimensions for passing	—
	Horizontal geometry		
Superelevation (e_{max})	Consistency of steering effort on successive curves	—	Urban/rural environment, climatic conditions, open highway/intersection, degree of curvature
Coefficient of friction (f_{max})	Comfort	—	Skid resistance of road surface, open highway/ intersection
Radius (R_{min})	—	—	Design speed, open highway/intersection
Transition curves	Behaviour on entering curves, comfort	—	Appearance of carriageway edges, design speed
Phasing	Response to visual defects and hazards	—	Appearance, creation of visual defects and hazards, design speed
	Vertical geometry		
Crest curves	Speeds during night-time compared with day-time comfort	Headlight height, proportion of stopping distance illuminated by headlights	Drainage, appearance of road, design speed
Sag curves	Comfort	Headlight height, beam divergance, distance illuminated by headlights	Drainage, appearance of road, design speed
Gradients	Behaviour on approach to gradients	Passenger car and truck performance, power/weight ratio of design vehicle, dimensions	Crawler lanes provide overtaking opportunity, design speed

TABLE 1—continued

Geometric design standard	Driver characteristics considered	Vehicle characteristics considered	Road characteristics considered
		Cross-section	
Number of lanes	Comfort, ability to manoeuvre in traffic stream and maintain desired speed	—	Urban/rural environment, design speed
Lane width	Sensitivity to restricted width	Dimensions of design vehicle	—
Lateral clearance	Sense of restriction	—	Nature of lateral obstruction
Shoulder width	Sense of restriction	Dimensions of design vehicle	Urban/rural environment, type of facility
Median width	Sense of well-being	Vehicle/barrier collision	Type of facility, terrain, urban/rural environment, appearance of carriageway edges
Crossfall	—	—	Drainage, type of facility
Vertical clearance	Sense of restriction	Dimensions of design vehicle	Future resurfacing

2.2 AASHTO

AASHTO continues to use the conventional definition of design speed: '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'. Since the standard caters for freeways, rural and urban arterial roads, collector roads and streets, and local roads and streets, a range of design speeds is used. Design speeds recommended for local rural roads range from 20 to 50 mph whilst, for rural collectors, the range is 20 to 60 mph, both dependent on terrain and traffic volume. Rural arterials should have design speeds of 50, 60 or 70 mph in mountainous, rolling or level terrain respectively. For rural freeways the normal design speed is 70 mph which may be reduced to 60 or 50 mph in difficult terrain, this being consistent with driver expectancy.

AASHTO recommends that a design speed of 70 mph should be used on main roads to ensure an adequate design in the future should the current 55 mph speed limit in US be removed.

In recommending the above values, the standard makes the following points:

(i) Speed is governed by the traffic volume and physical limitations of the road, not the importance of the road.

(ii) Higher traffic volumes may justify higher standards, since savings in operating costs can offset the increased construction costs.

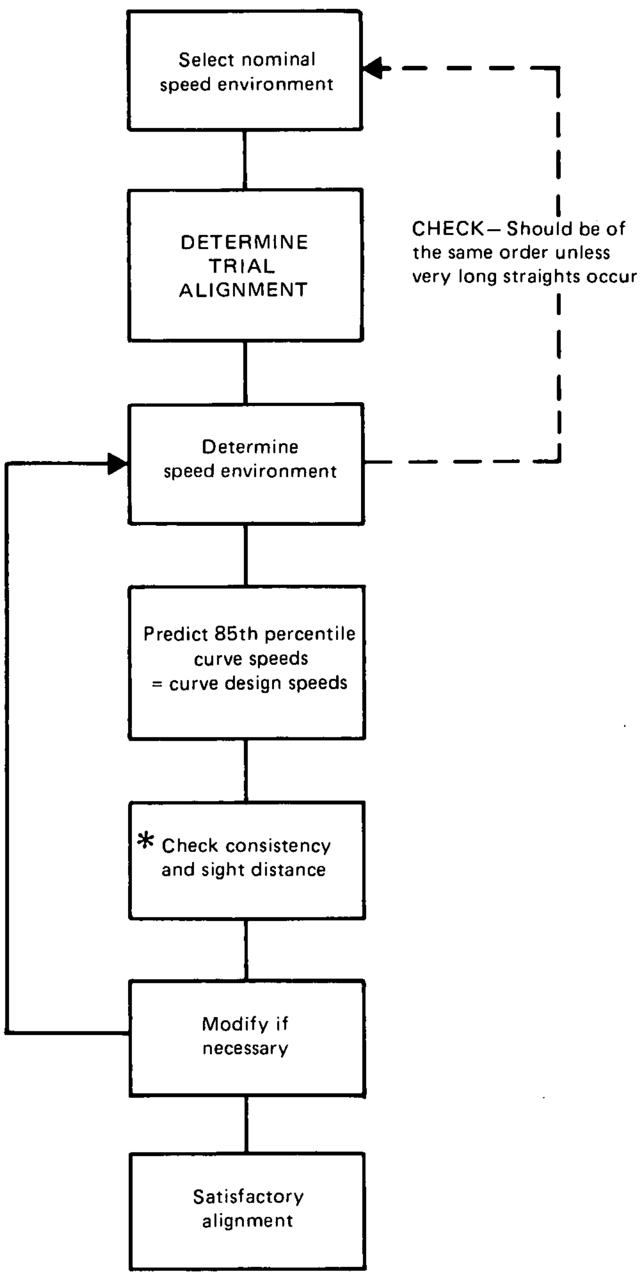
(iii) Design speed establishes minimum standards for safe operation, but there should be no restriction on the use of more generous designs if they are justified economically.

(iv) A relevant consideration in selecting design speeds is the average trip length. Standards provided on long lengths of a highway for longer trip lengths, should be as consistent as possible throughout and provide a good level-of-service.

2.3 NAASRA

2.3.1 Speed environment

To some extent, these standards are based on a field study of speeds on curves. The study was directed at investigating the relationship between vehicle speeds and the geometric properties of horizontal curves on two-lane rural roads (McLean and Chin Lenn 1977, McLean 1978 a, b, c, 1979). In order to take driver behaviour into consideration in the standards, two different speeds were recognised; namely speed environment and design speed. Speed environment is the desired speed of the 85th percentile driver and, as such, is the 85th percentile speed on the longer straights or large radius curves of a section of road where the speed is unconstrained by traffic or alignment elements. Design speed is defined as the 85th percentile speed on a particular geometric element, which is used for example to correlate curve radius, superelevation, friction demand, etc. Design speed varies along the road depending on the speed environment, the horizontal curve radius and, to some extent, on the gradients.



* The more consideration given to consistency at the trial alignment stage, the fewer will be the modifications required later

Fig. 1 NAASRA alignment selection procedure

NAASRA introduced an iterative process in the geometric design as shown in the flow chart in Figure 1. The most important part of this process is the consistency checks which ensure that the design speeds of successive geometric elements should not differ by more than about 10 km/h. This agrees with the recommendation by Leisch and Leisch (1977) that the change should not be more than 10 mph (15 km/h). On two way roads, consistency is checked for travel in both directions.

2.3.2 Selection of speed environment

The speed at which a driver will choose to travel a section of road is generally a compromise between the maximum speed at which he would be prepared to travel to reach his destination and the perceived level of risk which is seen to increase with increased speed. On straight open roads, road features will present little risk and the choice of speed will be determined largely by drivers preference and vehicle capabilities. The presence of features which a driver perceives as contributing to risk tends to restrict the speed of travel chosen. Such restrictions might arise from horizontal curvature, gradient, pavement width and condition, and the volume and nature of other traffic. Desired speeds of travel, and hence speed environment, therefore, whilst being defined in terms of unconstrained geometric elements, will be affected by overall standards of geometry and the terrain through which the road passes. Speed environments recommended by NAASRA for single carriageway roads are given in Table 2. These reflect the lower speed environment values associated with more difficult terrain resulting in higher values of bendiness on sections of road.

TABLE 2

NAASRA speed environment value as a function of overall geometric standards and terrain type for single carriageway rural roads for use when geometry is constrained.

Approximate range of horizontal curve radii (metres)	Speed environment (km/h)			
	Terrain type			
	Flat	Undulating	Hilly	Mountainous
Less than 75			75	70
75–300		90	85	
150–500		100	95	
Over 300–500	115	110		
Over 600–700	120			

2.3.3 Speeds on curves

The NAASRA standards are based on the following research results. From field observations, a good correlation was found between curve speeds and approach speeds of individual vehicles. Curve speed is defined as the speed at the mid point of the curve, whereas approach speed is the speed measured 100 to 400 metres before the entry tangent point. In general, the vehicles approaching at high speeds showed a greater speed reduction on curves compared to the vehicles approaching at low speeds. There are at least two reasons for this. Firstly, drivers adopt much higher speeds on tangent sections than the design speed; and secondly, drivers are not confident of negotiating curves at high speeds. A typical relationship between curve and approach speeds of individual cars is given in Figure 2 (McLean 1978 a).

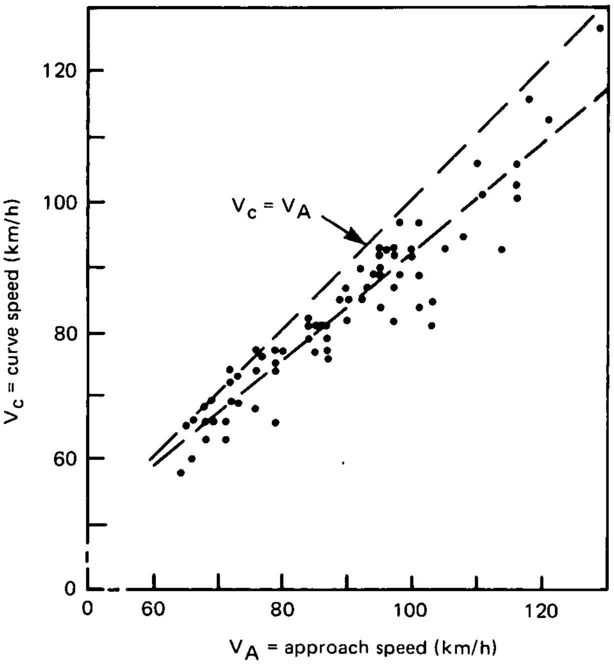


Fig. 2 Typical relationship from Australia between curve speed and approach speed for cars

For curves with speed standards (as defined below) greater than about 90 km/h, the 85th percentile car operating speeds on curves tended to be less than the curve speed standards. For curves with lower speed standards, the 85th percentile car operating speeds tended to be in excess of the curve speed standards. This is shown in Figure 3 (McLean 1978 a). The curve speed standard is defined as the maximum speed (V_d) at which vehicles can negotiate the curve without exceeding the earlier NAASRA (1970) side friction factors according to:

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

where e = superelevation
f = side friction factor
V_d = maximum (design) speed km/h
R = curve radius, metres.

Considering the findings above, for curve speed standards less than 90 km/h, drivers tended to travel at speeds which are much faster than the design speed on the tangent sections and still above the design speed on the curves. For curve speed standards greater than 90 km/h, drivers might travel at about the design speed on the tangent sections, but they reduced their speeds below the design speed when entering the curves. This suggested that, for design speeds greater than 90 km/h, driver behaviour tended to be more conservative relative to the design assumptions. Hence the earlier NAASRA curve standards were retained for design speeds in excess of 90 km/h in order to provide a high level of safety and comfort for drivers.

2.3.4 Side friction factor

It could be deduced from Figure 3 that, for the 85th percentile curve speeds less than about 90 km/h, the corresponding side friction factors were in excess of the values assumed previously for design purposes; whereas for speeds higher than 90 km/h, the side friction factors were less than assumed for design, as shown in Figure 4 (McLean 1978 a).

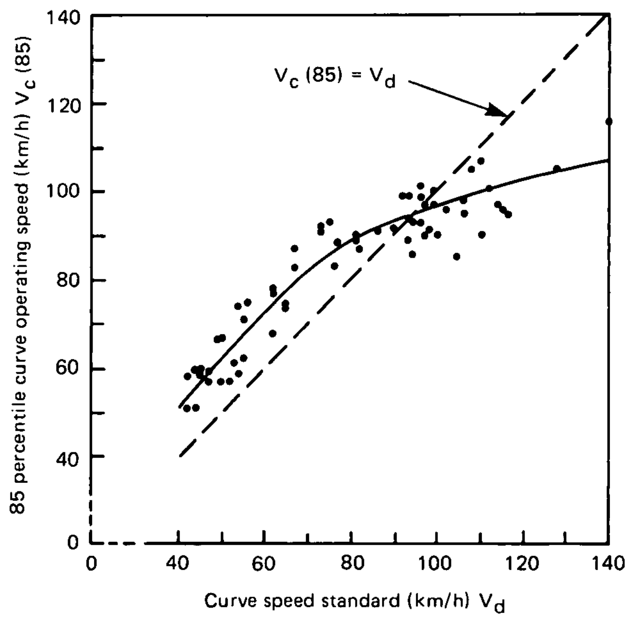


Fig. 3 Relationship between observed 85th percentile curve speed and curve speed standard for cars in Australia

The proposed design values for side friction factors derived from Figure 4 are discussed in more detail in Section 4.3

2.3.5 Curve design speed

The variation in observed 85th percentile speed on curves was explained by the following regression equation:

$$V_{85} = 53.8 + 0.464\ V_E - 3.26C + 0.0848C^2$$

where V_{85} = 85th percentile curve speed km/h
V_E = speed environment km/h
C = curvature (1000/radius, R) (metres)$^{-1}$

However for curves of radius below 70 metres (C> approximately 14), this equation was not a satisfactory representation of the observed relationship. An improved representation was provided by using four separate linear regression equations of speed on curvature for the data grouped according to four speed environment ranges. These four equations were then used as a basis to derive the family of curves relating 85th percentile car speeds to speed environment and curvature as shown in Figure 5 (McLean 1978 b). This family of curves can be combined with superelevation rates and maximum side friction factors to give values of 85th percentile curve speed for specified speed environments and curve radii shown in Figure 6.

2.4 TD9/81

2.4.1 Background to the standard

This standard is based on a speed-flow-geometry study which led to the development of both speed distribution curves and a relationship between mean operating speed and geometric features. The concept of design speed is still used, but in a more flexible way than previously.

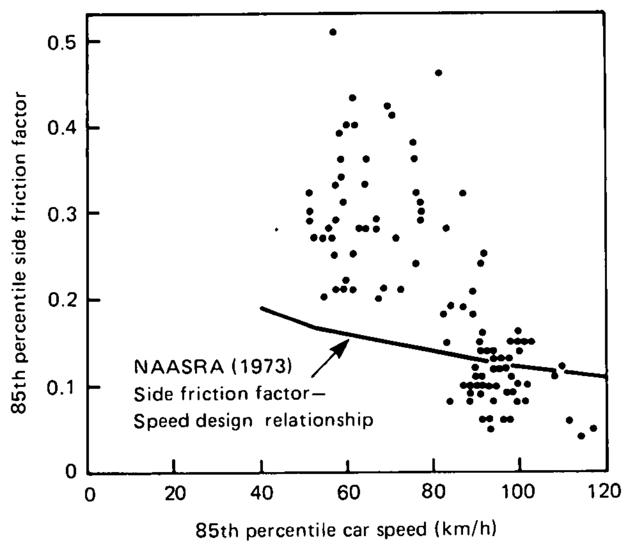


Fig. 4 Relationship between f_{85} and 85th percentile speed used as basis for NAASRA design criteria

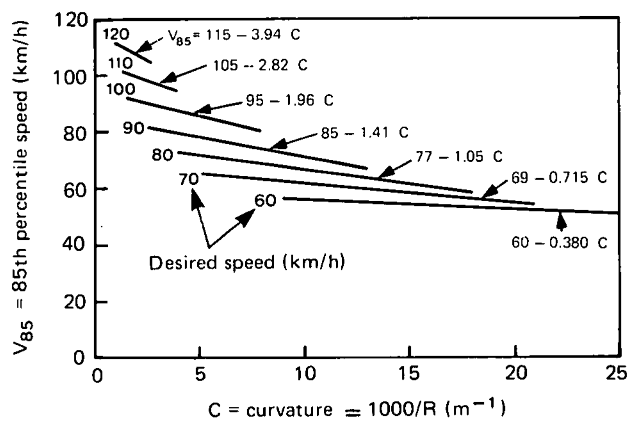
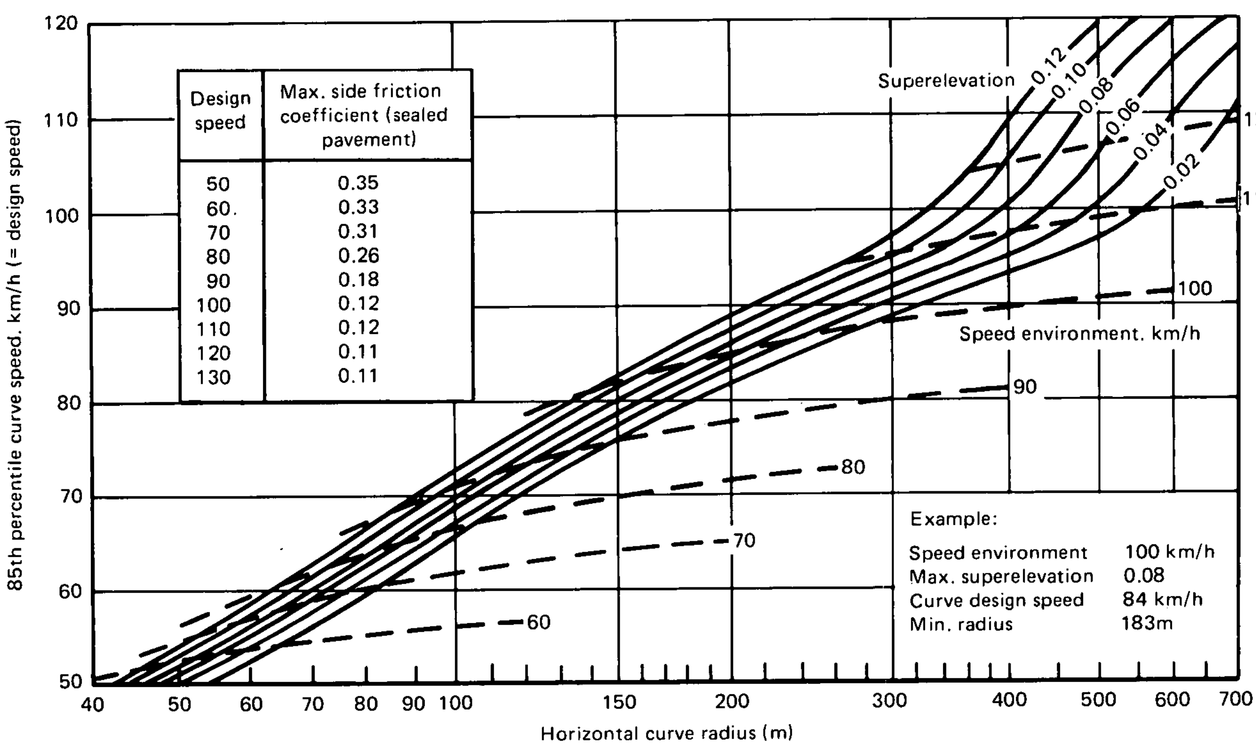


Fig. 5 Relationships used for predicting curve speeds in Australia



Design speed	Max. side friction coefficient (sealed pavement)
50	0.35
60.	0.33
70	0.31
80	0.26
90	0.18
100	0.12
110	0.12
120	0.11
130	0.11

Fig. 6 NAASRA relationships for minimum curve radius

Observations suggested that mean operating speed was a function of traffic volume and geometric features. In order to derive geometric design standards, speeds of light vehicles at the nominal traffic volume of 100 vehicles per hour were used.

These mean free operating speeds on dual and single carriageways were expressed by the following equations:

$$V_L(D-wet) = 103.4^* - \frac{B}{10} + \frac{HF}{4}$$

$$V_L(S-wet) = 73.69 + 1.1CW + \frac{VISI}{60} - 5\left\{\frac{2}{VW+1} + \frac{I}{CW}\right\} - \frac{2B}{45} - \left\{\frac{5H}{45} + \frac{2.5NG}{45}\right\}$$

where $V_L(D-wet)$ = mean free operating speed of light vehicles on dual carriageways in wet conditions (km/h)
$V_L(S-wet)$ = mean free operating speed of light vehicles on single carriageways in wet conditions (km/h)
I = number of intersections, lay-bys and non-residential accesses (total for both sides) per km
B = bendiness (degrees/km)
CW = carriageway width (metres)
VW = verge width (metres, including metre strips)
VISI = harmonic mean visibility
HF = sum of the falls (metres/km)
HR = sum of the rises (metres/km)
H = total hilliness (HR + HF) (metres/km)
NG = net gradient (HR − HF) (metres/km)

These equations were rationalised to the following single equation:

$$V_{L50}(wet) = 110 - A_C - L_C$$

where $V_{L50}(wet)$ = mean free operating speed in wet conditions.
A_C = alignment constraint (see 2.4.2)
L_C = layout constraint (see 2.4.2)

The effect of hilliness is excluded from initial assessments of design speed and specific adjustments can be applied during the detailed design of the road.

In the standard, this equation is presented in the form of a chart as shown in Figure 7. From the speed distribution curves in Figure 8 (Kerman 1980), it was found that the ratios 99th/85th, 85th/50th percentile speeds were approximately constant at the value of about $\sqrt[4]{2}$ for each road type. Nominal design speeds were then arranged on the basis of this ratio and the values of 120, 100, 85, 70, etc, km/h were adopted. Since the 85th percentile speed is normally adopted as the design speed, an increase or decrease of one design speed step means that the design is based on the 99th or 50th percentile speed respectively. For example, on a rural single carriageway road with a nominal 85th percentile design speed of 85 km/h, provision for 100 km/h geometrics would cater for the 99th percentile speed, whilst provision for 70 km/h geometrics would cater for only the 50th percentile speed. Hence the implication of raising or lowering design speed for a particular geometric element is clear.

* For dual 3 lane motorways; lower base speeds are applicable for dual 2 lane motorways and for dual 2 and 3 lane all purpose roads.

2.4.2 Determining the design speed

TD 9/81 adopts an iterative approach to design. The first step is to design a trial alignment for an assumed design speed. For this design the alignment constraint (A_C) is determined from:

For dual carriageways: $A_C = 6.6 + \frac{B}{10}$

For single carriageways: $A_C = 12 - \frac{VISI}{60} + \frac{2B}{45}$

The layout constraint, L_C, is then determined from Table 3.

Mean free operating speed, and hence design speed (ie the 50th and 85th percentile speeds under wet road conditions), are determined by entering these values of A_C and L_C into Figure 7. There are two categories A and B, for each design speed representing upper and lower bands. Whilst relaxation of standards for a given design speed is permitted for both categories on individual elements of the design, there are restrictions on the relaxations in category A because of the lower values of alignment and layout constraints and hence higher 85th percentile speeds.

The trial design speed and that determined from Figure 7 are then compared to identify locations where elements of the initial design may be relaxed to achieve cost or environmental savings, or conversely where the design should be upgraded to match the calculated design speed.

The design speed is then used to determine the design standards from Table 4, which shows desirable and absolute minimum values for each of the main elements.

2.4.3 Relaxation of standards

Desirable minimum values generally cater for vehicles at the 85th percentile speed at the normally accepted high levels of safety and driver comfort, whilst

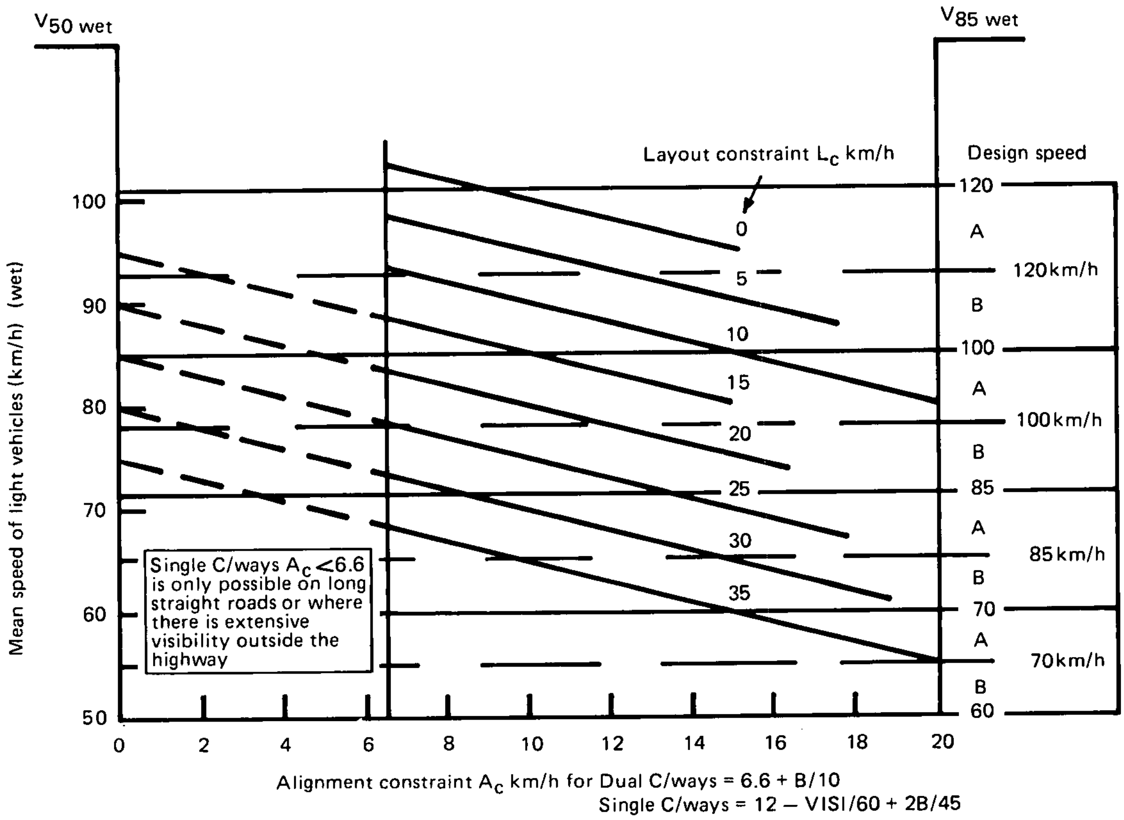


Fig. 7 TD9/81 Design chart

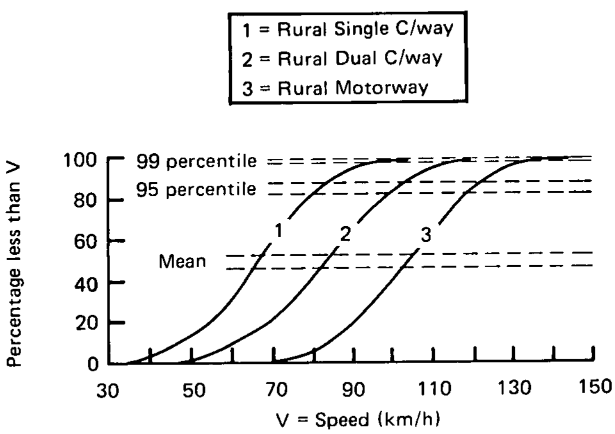


Fig. 8 Distributions of car journey speeds in UK

absolute minimum values for a particular design speed are identical to desirable minimum values for the next lower design speed step.

For the higher vehicle speeds, relaxation of standards to absolute minimum levels can imply design levels of safety and comfort below what has normally been accepted for design in the past. However the research leading to TD9/81 has shown that these high design levels of safety can be lowered to a limited extent without affecting accident rates. Further departures below absolute minimum levels may be allowed in exceptional circumstances. These further departures require detailed consideration of safety implications since, whilst they will not create hazards, the margin between what is considered safe and hazardous will, in these cases, be significant.

2.5 COMMENTS ON DESIGN SPEED

AASHTO recommends ranges of design speeds for the different road classifications, depending mainly on terrain and traffic volume, and that the standards used should be consistent on long lengths of road. Consideration should be given to the economic trade-offs between the increased construction costs of higher standards and the savings in operating costs which result. Most of these savings in operating costs will be savings in travel time from higher speeds of travel.

In general, time and vehicle operating costs can be represented by the following equation:

$$C = a + \frac{(b+d)}{V} + cV^2$$

where C = unit operating cost per km
V = operating (travel) speed, km/h
a, b, c = coefficients of vehicle operating costs
d = coefficient representing the value of time per vehicle

The above equation can be used to determine a minimum operating cost speed and will give very different values of this speed depending on whether time is valued in the road appraisal. It would seem logical to provide standards which encourage free-flow speeds in the vicinity of these minimum operating cost speeds.

The new approaches in both NAASRA and TD 9/81 involve a check of initial designs against an overall measure of speed to achieve consistency and reflect

TABLE 3

TD9/81 Layout constraint—Lc (km/h)

Road type	S2				WS2		D2AP		D3AP	D2M	D3M
Carriageway width (excl. metre strips)	6 m		7.3 m		10 m		dual 7.3 m		dual 11 m	dual 7.3 m*	dual 11 m*
Degree of access and junctions	H	M	M	L	M	L	M	L	L	L	L
Standard verge width	29	26	23	21	19	17	10	9	6	4	0
1.5 m verge	31	28	25	23							
0.5 m verge	33	30									

Notes:
L = Low access numbering 2 to 5 per km
M = Medium access numbering 6 to 8 per km
H = High access numbering 9 to 12 per km
* = Hard shoulder is recommended
S2 = Two-lane single carriageway
WS2 = Two-lane wide single carriageway
D2AP = Two-lane all purpose dual carriageway
D3AP = Three-lane all purpose dual carriageway
D2M = Two-lane motorway dual carriageway
D3M = Three-lane motorway dual carriageway

TABLE 4

TD9/81 Design Standards

Design speed km/h	120	100	85	70	60	50	V^2/R
STOPPING SIGHT DISTANCE m							
A1 Desirable Minimum	295	215	160	120	90	70	
A2 Absolute Minimum	215	160	120	90	70	50	—
HORIZONTAL CURVATURE m							
B1 Minimum R * without elimination of Adverse Camber and Transitions	2 880	2 040	1 440	1 020	720	510	5
B2 Minimum R * with Superelevation of 2.5%	2 040	1 440	1 020	720	510	360	7.07
B3 Minimum R * with Superelevation of 3.5%	1 440	1 020	720	510	360	255	10
B4 Desirable Minimum R with Superelevation of 5%	1 020	720	510	360	255	180	14.14
B5 Absolute Minimum R with Superelevation of 7%	720	510	360	255	180	127	20
B6 Limiting Radius with Superelevation of 7% at sites of special difficulty (Category B Design Speeds only)	510	360	255	180	127	90	28.28
VERTICAL CURVATURE							
C1 FOSD Overtaking Crest K Value	*	400	285	200	142	100	
C2 Desirable Minimum * Crest K Value	182	100	55	30	17	10	—
C3 Absolute Minimum Crest K Value	100	55	30	17	10	6.5	
C4 Absolute Minimum Sag K Value	37	26	20	20	13	9	
OVERTAKING SIGHT DISTANCE							
D1 Full Overtaking Sight Distance FOSD m	*	580	490	410	345	290	—

* Not recommended for use in the design of single carriageways

driver expectations along a section of road. This then allows some variation in design speed or standards for the individual elements to achieve cost effective designs, at the same time taking into account observed driver behaviour on the individual elements.

In the NAASRA standards, speed consistency is provided by the concept of a speed environment related to the terrain and range of horizontal curvature along the road. A family of relationships has been developed between speed environment and design speed for horizontal curves which, together with the criterion that design speed on successive elements should not differ by more than 10 km/h, ensures overall consistency with regard to driver expectations and safe efficient design of individual elements.

Similarly, in TD 9/81, an overall design speed is determined from the alignment and layout frictions along a road and variation in provision of standards on individual elements is allowed to a prescribed extent. By developing a unique relationship between design speed steps and speed distributions, and from studies of accident rates within the margin that exists between the traditional interpretations of safe and unsafe design, relaxations of standards are now possible which are acceptable in both safety and level-of-service terms.

3 SIGHT DISTANCE

3.1 BASIC CONSIDERATIONS

The driver's ability to see ahead contributes to safe and efficient operation of the road. Ideally, geometric design should ensure that at all times, any object on the pavement surface is visible to the driver within normal eye-sight distance. However, this is not usually feasible because of topographical and other constraints, so it is necessary to design roads on a basis of lower, but safe, sight distances.

There are two principal sight distances which are of particular interest in geometric design.

Stopping sight distance: If safety is to be built into the road, then sufficient sight distance should be available for drivers to stop their vehicles prior to colliding with an unexpected object on the pavement.

Passing sight distance: If operational efficiency is to be built into the road, for higher traffic volumes, then lengths of road with sufficient sight distance may have to be provided for drivers to overtake slower vehicles safely.

3.2 STOPPING SIGHT DISTANCE

3.2.1 Recommended values

It is important that, on all engineered roads, sufficient forward visibility is provided for safe stopping on vertical and horizontal curves throughout the length of the road. The derivation of stopping sight distance is based on assumed values for total driver reaction time and rate of deceleration, the latter expressed in terms of the coefficient of longitudinal friction.

$$D_s = \frac{R_T.V}{3.6} = \frac{V^2}{254.f}$$

where D_s = stopping sight distance, metres
R_T = total driver reaction time, seconds
V = design speed, km/h
f = coefficient of longitudinal friction.

Tables 5, 6, and 7 show the minimum stopping sight distances recommended by AASHTO, TD 9/81 and NAASRA respectively.

The three standards employ different stopping sight distances depending on the values of total driver reaction time, coefficient of longitudinal friction and vehicle speed assumed. These values are usually determined from experimental studies related to criteria such as safety, comfort and economics.

3.2.2 Driver reaction time

Driver reaction time consists of two components: perception time and brake reaction time. Perception time is the time required for the driver to perceive the hazard ahead and come to the realisation that the brake must be applied. This depends on the distance to the hazard, the physical and mental characteristics of the driver, atmospheric visibility, types and condition of the road and colour, size and shape of the hazard.

Brake reaction time is the time taken by the driver to actuate the brake after the decision to brake. This depends on the physical and mental characteristics of the driver, the driver position and layout of the vehicle controls.

Johansen (1977) made a detailed study of driver reaction time. He defined total driver reaction time as the time which elapses from the moment a signal is perceived until the moment the driver initiates preventative action. He described the psychological and physiological processes involved as illustrated in Figure 9. However, quantitatively, whilst it is relatively easy to carry out controlled experiments under alert laboratory conditions to measure driver reaction time, the relationship between this time and that which would obtain under non-alerted road conditions, where the perception of hazards on the road ahead is but one of a number of driver tasks, is difficult to determine. Also, it is easier to observe total reaction time rather than to measure separately its component processes.

Most of the limited number of field studies have shown that total driver reaction time varies from about 0.5 to 1.7 seconds. At high speeds, the values

TABLE 5

Stopping sight distances recommended by AASHTO

Design Speed (mph)	Assumed Speed for Condition (mph)	Brake Reaction		Coefficient of Friction f	Stopping Sight Distance for Design (ft)
		Time (sec)	Distance (ft)		
20	20–20	2.5	73.3– 73.3	0.40	125–125
25	24–25	2.5	88.0– 91.7	0.38	150–150
30	28–30	2.5	102.7–110.0	0.35	200–200
35	32–35	2.5	117.3–128.3	0.34	225–250
40	36–40	2.5	132.0–146.7	0.32	275–325
45	40–45	2.5	146.7–165.0	0.31	325–400
50	44–50	2.5	161.3–183.3	0.30	400–475
55	48–55	2.5	176.0–201.7	0.30	450–550
60	52–60	2.5	190.7–220.0	0.29	525–650
65	55–65	2.5	201.7–238.3	0.29	550–725
70	58–70	2.5	212.7–256.7	0.28	625–850
Driver eye height (feet)				3.5	
Object height (feet)				0.5	

TABLE 6

Stopping sight distances recommended by TD 9/81

design speed (km/h)	total driver reaction time (seconds)	coefficient of friction f_{wet}	stopping sight distance	
			desirable min.* (metres)	absolute min.† (metres)
50	2.0	0.25	70	50
60	2.0	0.25	95	70
70	2.0	0.25	120	95
85	2.0	0.25	160	120
100	2.0	0.25	215	160
120	2.0	0.25	295	215
driver eye height (metres)			1.05–2.00	1.05–2.00
object height (metres)			0.26–2.00	0.26–2.00

Notes:
* Based on 85th percentile speeds (design speeds)
†Based on 50th percentile speeds (one step down from the given design speeds) or based on a coefficient of friction of 0.375

of this time are less than those at low speeds. This is because fast drivers are usually more alert. It is also expected that drivers will be more alert on roads in difficult terrain and so the reaction time in this situation is likely to be less than that in rolling or level terrain. Johansen suggested a total driver reaction time of about 0.5 seconds in situations where drivers are keenly attentive and a time of 1.5 seconds for normal driving.

The AASHTO standard for total driver reaction time is 2.5 seconds which represents the time used by nearly all drivers under the majority of road conditions. Total driver reaction time recommended in TD 9/81 standards is 2.0 seconds which provides a limited margin of safety over the field study figure. NAASRA recommends total driver reaction time of 2.5 seconds as a standard value and 1.5 seconds as an absolute minimum value. The latter value should

TABLE 7

Stopping sight distances recommended by NAASRA

design speed (km/h)	coefficient of friction f_{wet}	stopping sight distance (metres)			
		normal design		constrained situations	
		$R_t = 2.5$ sec		$R_t = 2$ sec.	$R_t = 1.5$ sec.
		D_s	1.4 D_s		
50	0.65	50	70	45	35
60	0.60	65	90	60	50
70	0.55	85	120	75	65
80	0.50	105	150	95	
90	0.45	140	200	120	
100	0.40	170	240		
110	0.37	210	290		
120	0.35	250	350		
		(1)	(2)	(3)	(4)
driver eye height (m)		1.15	1.15	1.15	1.15
object height (m)		0.20	0	0.20	0.20

Notes:
(1) = Standard values for stopping sight distance
(2) = Values used in less constrained budget situations or in easier terrain
(3) = Adopted as manoeuvre sight distance
(4) = Absolute minimum values for stopping sight distance

<table>
<tr><td>Visible signal</td><td>Psychological process</td><td colspan="2">Physiological process</td><td colspan="3">Psychological process</td><td colspan="2">Physiological process</td></tr>
<tr><td>x</td><td>y</td><td>a</td><td>b</td><td>c</td><td>d</td><td>e</td><td>f</td><td>g</td></tr>
<tr><td></td><td>Attention</td><td colspan="2">Sensation</td><td colspan="3">Perception and decision</td><td>Movement</td><td>Initiation</td></tr>
<tr><td colspan="7"></td><td colspan="2">Total driver reaction time commonly defined</td></tr>
<tr><td colspan="9">Total driver reaction time better defined</td></tr>
</table>

x : Stimulus (signal, obstacle) visible (audible, etc) to a normal driver
y : Driver attention to the stimulus
a : Stimulation of the sense organ
b : Transmission of the sensation by the senory nerve and the initiation of the brain processes
c : Identification of the obstacle
d : Interpretation of the obstacle
e : Decision-making to avoid the obstacle
f : Transmission of brain impulses by the motor nerves
g : Stimulation of the muscles and the initiation of movements

Fig. 9 Total driver reaction time

be used only if drivers are expected to be driving in conditions which lead to alertness and the carriageway is sufficiently wide to provide reasonable space for evasive action.

NAASRA also recommends the use of 'manoeuvre site distance' to achieve cost effective designs of vertical curves in difficult situations. This distance ensures that the driver can perceive a hazard on the road ahead in sufficient time to take evasive action through lateral manoeuvring, rather than stopping the vehicle. Reasonable manoeuvre times from observation vary from about 3 seconds at a horizontal alignment design speed of 50 km/h to 5 seconds at 100 km/h. The resulting manoeuvre site distances, which are shown in Table 8, are close to values for stopping sight distances based on a driver reaction time of 2.0 seconds.

TABLE 8

Manoeuvre sight distances recommended by NAASRA

design speed (km/h)	derived manoeuvre time (seconds)	manoeuvre sight distance (metres)
50	3.2	45
60	3.6	60
70	3.9	75
80	4.3	95
90	4.8	120
100	5.6	155

The NAASRA manoeuvre site distance can be contrasted with the AASHTO 'decision sight distance' which has been introduced to allow for situations where the normal stopping sight distances are inadequate. This may be appropriate in situations when complex or instantaneous decisions, unexpected or unusual manoeuvres are required, and when information is difficult to perceive. Such locations might be at interchanges, intersections, changes in cross-section, or where drivers in heavy traffic need to perceive information from a variety of competing sources. The provision of the longer sight distance at these critical locations will ensure that drivers can safely

(a) detect and recognise these hazards or information sources,

(b) decide and initiate an appropriate response and

(c) manoeuvre his vehicle accordingly.

Times for these components of decision sight distances range from (a) 1.5 to 3.0 seconds. (b) 4.2 to 7.0 seconds. (c) 4.0 to 4.5 seconds, resulting in decision sight distances ranging from 10.2 to 14.5 seconds depending on design speed. These distances are at least twice the normal stopping sight distances.

3.2.3 Coefficient of longitudinal friction

The determination of design values of longitudinal friction (f) is complicated because of the many factors involved. It is, however, known that f values are a decreasing function of vehicle speed, except under the most favourable road surface textures. Coefficient of longitudinal friction is measured using either the sideways-force machine (SCRIM) or the portable skid pendulum. The main factors affecting the friction between the tyres and the road surface are:

(i) Road surface macrotexture: rough macrotexture is required to maintain skidding resistance at higher speeds.

(ii) Road surface microtexture: harsh microtextures of surfacing materials are important to provide good skid resistance as they will puncture and disperse the thin film of water remaining after removal of the bulk water by the macrotexture and tyre tread.

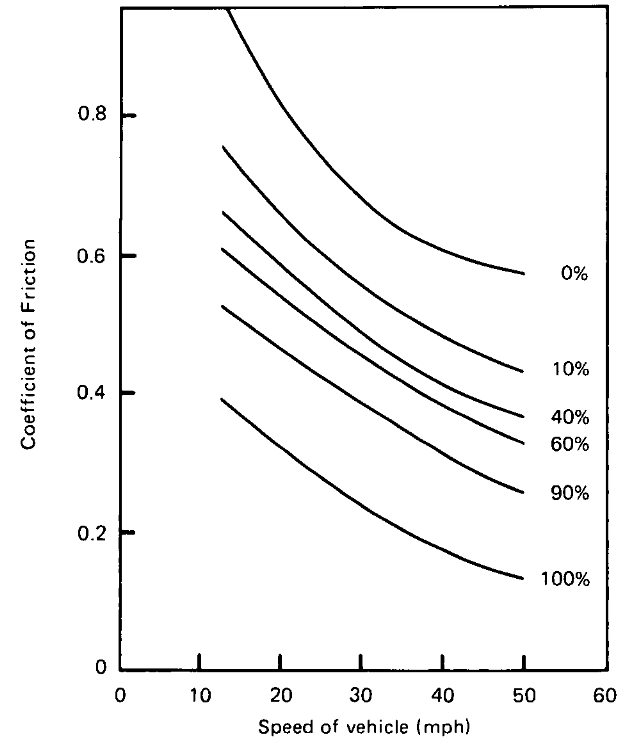
(iii) Road surface condition: wet pavements are assumed when deriving values for design purposes.

(iv) Tyres: a good tread pattern provides escape channels for bulk water and a radial ply increases contact area; tyre stiffness is also a factor.

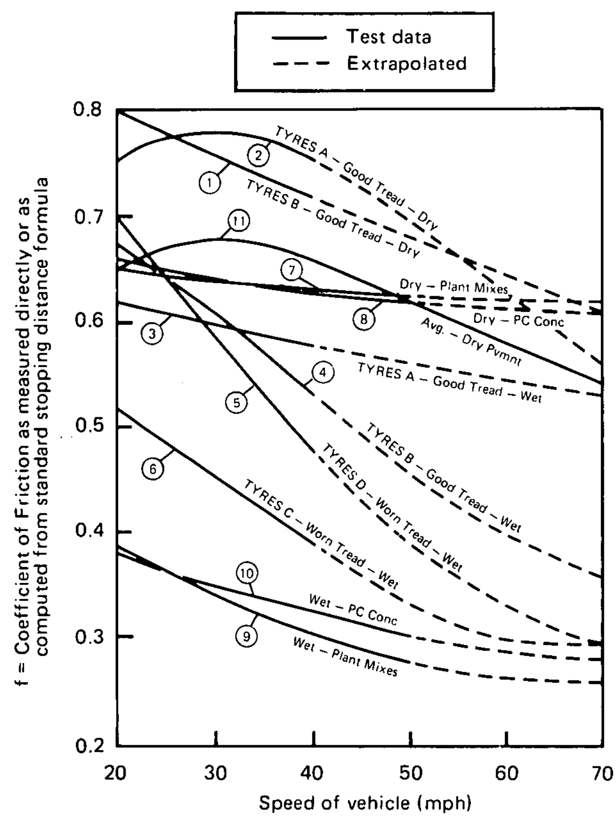
If comfort for vehicle occupants is considered to be the sole criterion, f values greater than about 0.5 should not be used as decelerations (f.g) of 0.5g result in unrestrained passengers sliding from their seats. In normal driving, such values of f would only be generated in emergency braking. For design purposes, it is important that no loss of control of the vehicle occurs during stopping and lower f values are therefore desirable.

The design values of f used by AASHTO, shown in Table 5, are generally conservative since they include most of the curves shown in Figure 10(b). The range of speeds assumed for design in Table 5 are based on average running speeds for low traffic volume conditions (AASHO 1965) at the lower extreme, and design speed at the higher extreme. This reflects current observations that many drivers travel as fast on wet pavements as on dry.

Constant f values were adopted in the TD 9/81 standards with a value of 0.25 for desirable minimum and 0.375 for absolute minimum stopping sight distances for the 85th percentile vehicle speed. The f value of 0.25 is slightly less than the minimum target value of pavement skidding resistance of 0.30 for straight sections and large radius curves proposed by TRRL (Salt and Szatkowski 1973) and shown in Table 9, whilst the f value of 0.375 is considered acceptable for retaining vehicle control in stopping on wet normally textured surfaces.



(a) Skid resistance of pavement
(based on German values)



(b) Skid resistance for various tyre and pavement conditions

Fig. 10 Variation in coefficient of friction with vehicle speed in United States

NAASRA adopted relatively high f values as shown in Table 7. These were based on the tests conducted by the Australian Road Research Board (McLean 1978 c), ranging from 0.65 at 50 km/h design speed down to 0.35 at 120 km/h, due consideration having been given to road surface polishing, the reduction in wet skidding resistance with increasing speed, and the need for vehicle control in stopping.

3.2.4 Effect of gradient

Shorter braking distances are required on uphill grades and longer distances on downhill grades as follows:

$$\text{Braking distance} = \frac{V^2}{254(f \pm G)} \text{ metres}$$

where V = design speed, km/h
f = coefficient of longitudinal friction
G = gradient, %, positive if uphill
negative if downhill

For two-lane roads, sight distances are longer on many downgrades than on upgrades, so that the above correction is provided automatically.

3.2.5 Effect of trucks

Trucks generally require longer distances to stop for a given speed than cars, but this is offset by the higher eye height of truck drivers and hence their better visibility and earlier perception of potential hazards. Truck speeds on crest curves are also generally lower than the speeds of cars. No adjustment of the stopping sight distance standards is normally considered for trucks. However, where there is a combination of steep downhill grade and horizontal curvature, higher values than the minimum standards should be used.

3.3 PASSING SIGHT DISTANCE

3.3.1 Critical factors

Factors affecting passing sight distance are the judgement of overtaking drivers, the speed and size of overtaken vehicles, the acceleration capabilities of overtaking vehicles, and the speed of oncoming vehicles. Driver judgement and behaviour are important factors which vary considerably among drivers. For design purposes, the passing sight distance selected should be adequate for the majority of drivers.

Passing sight distances are determined empirically and are usually based on passenger car requirements. On average, heavy commercial vehicles take about four seconds longer than cars to complete the overtaking manoeuvre. Nevertheless, it is unusual for passing sight distance to be based on commercial vehicle needs, except when the proportion of trucks in the traffic stream is very high. Apart from the

TABLE 9

Minimum values* of side frictional coefficient (SFC) for different sites, proposed by TRRL for roads in the United Kingdom

SITE	DEFINITION	SFC (at 50 km/h) Risk Rating† 1	2	3	4	5	6	7	8	9	10
A1 (very difficult)	(i) Approaches to traffic signals on roads with a speed limit greater than 40 mile/h (64 km/h) (ii) Approaches to traffic signals, pedestrian crossings and similar hazards on main urban roads						0.55	0.60	0.65	0.70	0.75
A2 (difficult)	(i) Approaches to major junctions on roads carrying more than 250 commercial vehicles per-lane per day (ii) Roundabouts and their approaches (iii) Bends with radius less than 150 m on roads with a speed limit greater than 40 mile/h (64 km/h) (iv) Gradients of 5% or steeper, longer than 100 m				0.45	0.50	0.55	0.60	0.65		
B (average)	Generally straight sections of and large radius curves on: (i) Motorways (ii) Trunk and principal roads (iii) Other roads carrying more than 250 commercial vehicles per lane per day	0.30	0.35	0.40	0.45	0.50	0.55				
C (easy)	(i) Generally straight sections of lightly trafficked roads (ii) Other roads where wet accidents are unlikely to be a problem	0.30	0.35	0.40	0.45						

Notes:
* = Minimum value is defined as the mean summer SFC (average of three readings taken during the months May–September) in a year of normal weather conditions.
† = Risk rating is relative classification based on accident rates.

extra expense that this would involve, commercial vehicle drivers have greater visibility ahead because of their higher eye height and hence are able to judge sooner and better whether a gap is suitable or not for overtaking, thus partially offsetting any additional overtaking length that might be required.

3.3.2 Recommended values

The minimum passing sight distances recommended by AASHTO, TD 9/81 and NAASRA are given in Tables 10, 11, 12 respectively.

The AASHTO standard is based on four components of the overtaking manoeuvre:

(i) The distance travelled during perception and reaction time (in judging the acceptability of an overtaking opportunity) and, during the initial acceleration, to the point of encroachment on the centre line of the road.

(ii) The distance travelled while the passing vehicle occupies the opposing lane.

(iii) The distance between the passing vehicle at completion of the overtaking manoeuvre and an oncoming vehicle.

(iv) The distance travelled by an oncoming vehicle during the time from when the passing vehicle is abreast of the overtaken vehicle to completion of the overtaking manoeuvre (approximately two thirds of the time the passing vehicle occupies the opposing lane).

To determine safe passing sight distances, AASHTO assumes that the speed of the overtaken vehicle is equal to the average running speed at intermediate volumes of traffic where overtaking occurrences are most likely. The speeds of the overtaking and oncoming vehicles are considered to be 10 mph (16 km/h) faster than that of the overtaken vehicle.

The TD 9/81 standard for passing sight distance (full overtaking sight distance FOSD) was based on a study carried out by the Transport and Road Research Laboratory (Simpson and Kerman 1982), the results of which are summarised in Figure 11. This shows the distribution of overtaking durations for a typical road at an approximate design speed of 85 km/h. It can be seen that the time taken for most overtaking manoeuvres to be completed was between 3 and 15 seconds, with 85 per cent of drivers overtaking in less than 10 seconds. The 10 second value was therefore adopted for design purposes. The standard assumes that the overtaking vehicle starts to overtake at a speed two design speed steps below the nominal design speed and accelerates to the design speed over the duration of the overtaking manoeuvre, whilst the oncoming vehicle travels at the design speed.

In determining lengths of road available for safe passing, TD 9/81 assumes that such lengths terminate when only an equivalent 'Abort Sight Distance' (equal to FOSD/2) is available. This distance is that required for an overtaking driver to complete a manoeuvre in the face of oncoming vehicles from when it is abreast of the overtaken vehicle.

TABLE 10

Passing sight distances recommended by AASHTO

Design Speed (mph)	Assumed Speeds: Passed Vehicle (mph)	Assumed Speeds: Passing Vehicle (mph)	Minimum Passing Sight Distance (ft) (Rounded)
20	20	30	800
30	26	36	1,100
40	34	44	1,500
50	41	51	1,800
60	47	57	2,100
65	50	60	2,300
70	54	64	2,500
driver eye height (feet)			3.50
object height (feet)			4.25

TABLE 11

Passing sight distances recommended by TD 9/81

Design Speed km/h	Overtaking Manoeuvre Time (seconds)	Minimum Passing Sight Distance (metres)
50	10.0	290
60	10.0	345
70	10.0	410
85	10.0	490
100	10.0	580
driver eye height (metres)		1.05–2.00
object height (metres)		1.05–2.00

TABLE 12

Passing sight distances recommended by NAASRA

Design Speed (km/h)	Overtaken Vehicle Speed (km/h)	Passing Sight Distance: Time gap (sec)	Passing Sight Distance: Sight distance (metres)	Continuation Distance: Time gap (sec)	Continuation Distance: Sight distance (metres)
50	43	13.6	350	4.5	165
60	51	14.6	450	5.0	205
70	60	15.7	570	5.4	245
80	69	16.9	700	6.4	320
90	77	18.2	840	7.6	410
100	86	19.7	1010	8.3	490
110	94	21.3	1210	9.1	580
120	103	23.1	1430	10.0	680
130	111	25.2	1690	11.0	800
driver eye height (m)			1.15	—	1.15
object height (m)			1.15		1.15

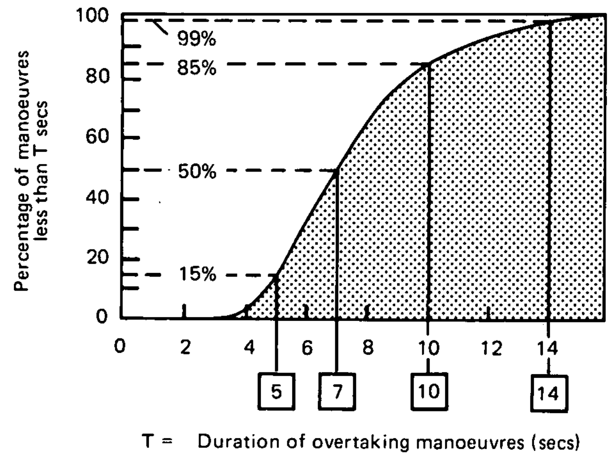


Fig. 11 Distribution of overtaking duration in United Kingdom

Passing sight distances recommended by NAASRA were based on studies carried out by Troutbeck (1981), which used a gap-acceptance approach. The passing manoeuvre was assumed to be in three phases.

Phase 1 is the distance travelled from the point at which the opposing lane is entered to when the vehicle is alongside that being overtaken.

Phase 2 is the distance from the end of Phase 1 to when the vehicle, still in the opposing lane, is clear of that being overtaken.

Phase 3 is the distance from the end of Phase 2 to the point at which the vehicle is entirely back in its own lane.

The minimum distance which is adequate to encourage a given proportion of drivers to commence an overtaking manoeuvre is known as the 'establishment sight distance' and is the sum of Phases 1 to 3. The sum of Phases 2 and 3 was considered as the Continuation distance which would enable an overtaking driver to either complete safely or abort a manoeuvre already underway.

3.4 EYE AND OBJECT HEIGHTS

The higher the driver eye height and object height, the longer will be the sight distance available over a vertical crest curve. On sag curves, obstructions can occur where an overbridge crosses the alignment. Visibility on horizontal curves depends on whether the sight line falls outside the right-of-way limits. For sight lines on horizontal curves within the right-of-way, eye and object heights are not generally significant, except where cutting slopes, bridge parapets, etc, obstruct the line of sight. Sight distances outside the right-of-way are much more dependent on eye and object heights. For example, in determining the value of Alignment Constraint A_c (see para 2.4.2),TD 9/81 requires estimation of the harmonic mean visibility between eye and object, both with assumed heights of 1.05 m.

For geometric design purposes, driver eye and object heights reflect conditions found in practice. Driver eye height depends largely on vehicle characteristics and to some extent on driver posture. It is generally accepted that provision of visibility to the road surface, ie an object height of zero, for a distance equal to that required for safe stopping is not cost-effective. Selection of a higher object height for design is thus a compromise between possible reduced safety and savings in construction costs. Object heights must also be related to whether the vehicle is stopping or passing.

Driver eye and object heights proposed by the three standards are given in Tables 5, 6 and 7. The AASHTO driver eye height of 3.50 ft reflects the observed reduction that was taken place in the last twenty-five years in average passenger car and driver eye heights. The AASHTO object height for stopping of 6" is derived from economic considerations as above. This height was the lowest which could be considered a hazard and perceived by the driver as requiring him to stop. The AASHTO object height for passing of 4.25 ft represents the current average passenger car height.

TD 9/81 proposes an envelope of clear visibility for stopping involving driver eye heights between 1.05 and 2.00 m. The lower bound represents the height exceeded by 95 per cent of driver heights in UK, whilst the upper bound is a typical eye height for heavy goods vehicle drivers. For object height, the lower bound of the visibility envelope is 0.26 m, with an upper bound of 2.00 m. For passing, an envelope of clear visibility between points 1.05 and 2.00 m above the road surface over the full passing sight distance is required.

Studies of driver eye heights in Australia have resulted in the NAASRA recommendation of 1.15 m and 1.8 m for car and heavy vehicle driver eye heights. NAASRA adopts an object of 0.2 m for stopping under normal design, but allows an object height of zero on the approaches to causeways and floodways subject to flood water residues or washouts. For passing, an object height of 1.15 m is used.

3.5 COMMENTS ON SIGHT DISTANCE

Design standards for stopping sight distance are largely dependent on assumed values of total driver reaction time and longitudinal coefficient of friction. There is consistency between the three standards in choice of total driver reaction time, with normal values in the range 2.0 to 2.5 seconds representing most drivers and road conditions. However, it should be recognised that a simple criterion is being used, based on limited field studies, and which is assumed to represent the whole population of drivers. The NAASRA standards allow a lower reaction time of 1.5 seconds where drivers are alert but, whilst this may be attractive in order to achieve construction cost savings, further investigation is needed and a more detailed specification of such situations would be helpful.

There is some variation between the three standards in assumed design values of wet coefficient of longitudinal friction. However, other important factors to be considered are those related to questions of vehicle maintenance (tyre and brake condition) and whether the skid resistance of the road surface is adequate to provide the likely required deceleration, in addition to the need to maintain vehicle control during stopping in wet conditions.

The studies for TD 9/81 have indicated increasing accident rates with reducing sight distances, particularly where the latter are sub-standard. Nevertheless, the standard allows consideration of departures from design values in difficult situations on road sections without accesses or junctions.

The NAASRA manoeuvre site distance standard will achieve cost effective designs in terms of construction costs and would appear attractive for low volume roads, given the assumed driver behaviour of lateral manoeuvring instead of stopping.

The AASHTO decision sight distances provide more generous standards where the decision and initiation of appropriate reponses to perceived information is more complex. Whilst this may be desirable in identified locations, it has been introduced here as an example of where the search for cost effective standards based on studies of driver behaviour, and the safety implications of altering standards, can result in a recommendation for higher standards than otherwise.

There are considerable differences in the three standards for passing sight distance due to different assumptions about the component distances of the standard, different assumed speeds for the manoeuvre and, to some extent, driver behaviour. Nevertheless, the standards are based on studies of driver overtaking behaviour in all three countries. An additional important practical consideration would be the siting of overtaking sections. If faster vehicles are constrained to follow slower ones over a particular subsection of road, then it is desirable to follow this with an overtaking subsection. Otherwise, increased driver frustration results and drivers will attempt to overtake at increased risk on more dubious overtaking alignments. The principle in TD 9/81 of using sharper non-overtaking bends and longer straights where overtaking is safe is a welcome move in this regard, and moves away from the provision of longer and larger radii curves in 'flowing alignments' where overtaking can only be carried out at risk.

Driver eye heights in the three standards are broadly similar, all reflecting trends that have occurred in vehicle design and, to some extent, driver position. The concept of an envelope of clear visibility used in TD 9/81 is useful in ensuring safe design for all vehicle types using the road.

Choice of object height is a compromise between safety and savings in construction cost. The use of a minimum object height for stopping of 0.26 m in TD 9/81 has brought UK standards broadly in line with those in US and Australia. Design object heights must be related to the likely occurrence of hazards on the pavement surface, which may be related to the problems of routine maintenance of roads.

4 HORIZONTAL ALIGNMENT

4.1 ALIGNMENT, USER COSTS AND ACCIDENTS

Horizontal alignment usually consists of a series of intersecting tangents and circular curves, with or without transition curves. The alignment should be designed to be as direct as possible in order to reduce road user costs, but will be constrained by topography in hilly terrain, land use, availability of road materials and crossing points. The environmental effects of roads and traffic have become increasingly important considerations in many developed countries, where the non-quantifiable costs and benefits of road schemes in addition to construction, operating and maintenance costs are taken into account. Many of these environmental effects are related to choice of alignment.

The layout of the horizontal alignment significantly affects the total cost of the road. Mean operating speed is a decreasing function of overall horizontal curvature, so that the road user costs of fuel and time, which are functions of vehicle speed, will be affected by the horizontal curvature. Construction cost normally increases with increasing horizontal radius, especially in hilly terrain.

The effects of horizontal curve radius on accident rates have been studied in the United Kingdom (Shrewsbury and Sumner 1980) and are shown in Figure 12. This study showed that accident rates increased with reducing horizontal curve radius, but more rapidly below a value of about 400 m. A more comprehensive study was carried out earlier in the United Kingdom (Road Research Laboratory 1965), the results of which are summarised in Table 13. The study showed 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 radius on an otherwise straight alignment would cause a higher accident rate than that on an alignment with a high degree of bendiness.

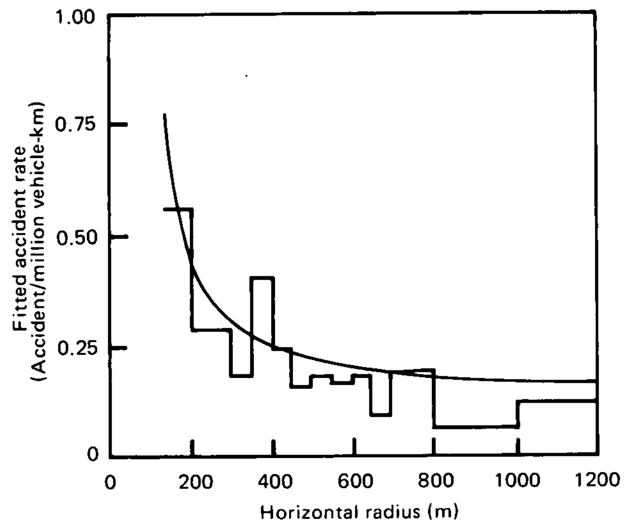


Fig. 12 Relationship between accident rate and curve radius in United Kingdom

TABLE 13

UK non-intersection injury accidents on straights and curves on a 9 metre roadway with different levels of average curvature

Average Curvature (deg/km)	Injury Accidents/10^6 veh-km				
	Straights and Radius > 1520 m	Curve Radius (m)			Total
		610 to 1520	305 to 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	0.7	0.4

4.2 VEHICLE MOVEMENT ON A CIRCULAR CURVE

When a vehicle traverses a superelevated circular curve, it is subject to a lateral force, acting in the plane of the road surface, which is counteracted by the component of the vehicle weight along the plane of the road surface, and by side friction generated between the tyres and road surface. This side friction is equal to the coefficient of friction(f) between the tyres and the road surface multiplied by the normal reaction at the tyre/road contact areas; the latter, in turn, is equal to the component of the vehicle weight normal to the plane of the road surface. For the small values of superelevation generally used on highways, the following equation can be derived from the above considerations:

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

where e = superelevation
f = coefficient of side friction, (side friction factor)
V = design speed, km/h
R = curve radius, metres.

For design purposes, a constant design speed is usually assumed and, for a given curve radius, the required superelevation can be determined which provides an acceptable level of coefficient of friction and driver comfort. The relationships between design speed, curve radius and superelevation recommended by AASHTO, TD 9/81 and NAASRA are shown in Figures 13, 14 and 6 respectively.

The relationship between superelevation and curve radius for each design speed adopted by AASHTO was based on a parabolic curve over the range of curvatures from D_o (zero degree of curve) to D_{max} (maximum degree of curve, or minimum radius). The corresponding curves for side friction factors (f) are smooth curvilinear relationships with f values gradually increasing to the maximum design value at D_{max}. This relationship ensures that, on the different horizontal curves along a section of road, for vehicles at or above average running speed, some consistency in the steering effort required to generate the side friction on successive curves is achieved, ie the driver always has to turn his wheel towards the centre of curvature at these speeds.

The TD 9/81 standards are based on the following relationship:

$$e=0.45\times\frac{V_{85}^2}{127R}=\frac{V_{85}^2}{282R}$$

where V_{85} is the normal design speed. This ensures that a vehicle at the 99th percentile speed on a curve of absolute minimum radius will experience a gross lateral acceleration of not more than 0.22 g and a nett lateral acceleration, to be balanced by friction, of not more than 0.15 g. The equation also implies that the 'hands-off' condition (nett lateral acceleration = 0) is approximately the 15th percentile speed. Hence consistency of steering effort on successive curves will be maintained for 85 per cent of drivers.

The standard does not recommend the use of curves whose radius is in band C in Figure 15. This avoids sections of road with dubious overtaking conditions for traffic in the left hand curve direction. It is therefore a principle of the standard that design should concentrate only on bands A and B for clear overtaking sections, and band D for clear non-overtaking sections.

A low degree of curvature or large curve radius is usually introduced on long straight sections by gently deflecting the alignment approximately 4° to the left and right alternatively. Its function is to break the monotony for drivers and to avoid glare from vehicle headlights or the setting sun. Drivers are also found to have difficulty in judging oncoming vehicle speeds, and hence overtaking opportunities, on long straight sections.

The NAASRA standard does not specify a method for determining superelevation and side friction factors for the range of intermediate curve radii. Figure 6 is in terms of maximum friction values and superelevation rates from which a minimum curve

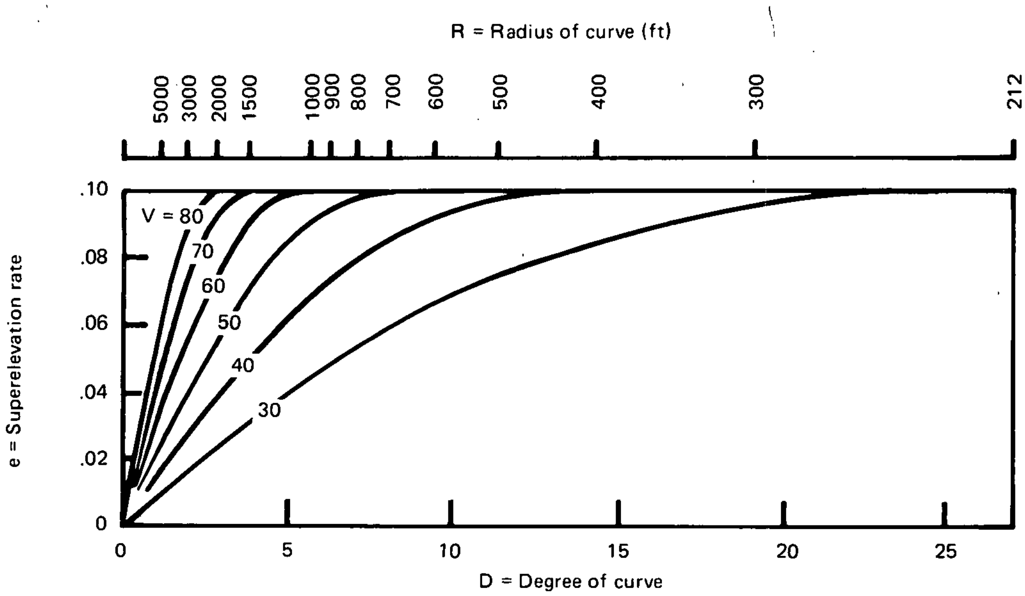


Fig. 13 Design superelevation rates for e $_{max}$ = 0.10 recommended by AASHTO

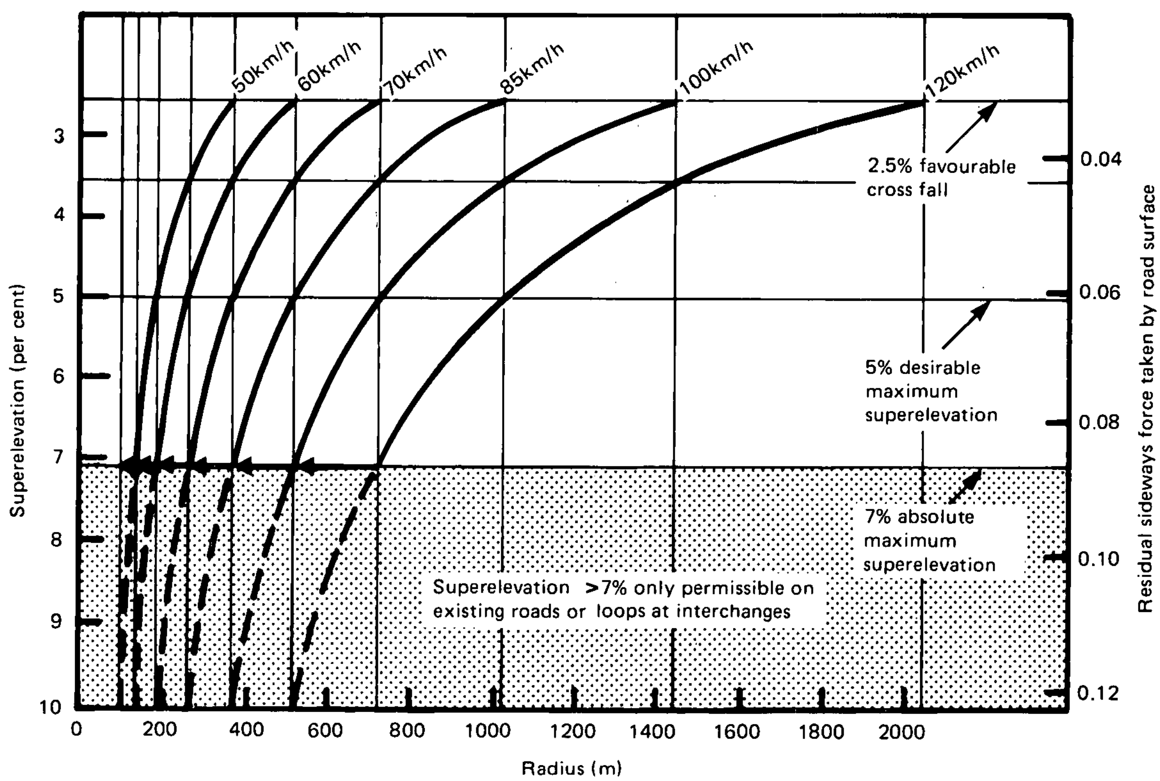


Fig. 14 Superelevation of curves recommended by TD9/81

TABLE 14

Maximum degree of curvature and minimum radius determined for limiting values of e and f by AASHTO

Design Speed (mph)	Maximum e	Maximum f	Total (e + f)	Maximum Degree of Curve	Rounded Maximum Degree of Curve	Minimum Radius (ft)
20	.04	.17	.21	44.97	45.0	127
30	.04	.16	.20	19.04	19.0	302
40	.04	.15	.19	10.17	10.0	573
50	.04	.14	.18	6.17	6.0	955
60	.04	.12	.16	3.81	3.75	1,528
20	.06	.17	.23	49.25	49.25	116
30	.06	.16	.22	20.94	21.0	273
40	.06	.15	.21	11.24	11.25	509
50	.06	.14	.20	6.85	6.75	849
60	.06	.12	.18	4.28	4.25	1,348
65	.06	.11	.17	3.45	3.5	1,637
70	.06	.10	.16	2.80	2.75	2,083
20	.08	.17	.25	53.54	53.5	107
30	.08	.16	.24	22.84	22.75	252
40	.08	.15	.23	12.31	12.25	468
50	.08	.14	.22	7.54	7.5	764
60	.08	.12	.20	4.76	4.75	1,206
65	.08	.11	.19	3.85	3.75	1,528
70	.08	.10	.18	3.15	3.0	1,910
20	.10	.17	.27	57.82	58.0	99
30	.10	.16	.26	24.75	24.75	231
40	.10	.15	.25	13.38	13.25	432
50	.10	.14	.24	8.22	8.25	694
60	.10	.12	.22	5.23	5.25	1,091
65	.10	.11	.21	4.26	4.25	1,348
70	.10	.10	.20	3.50	3.5	1,637

NOTE: In recognition of safety considerations, use of $e_{max} = 0.04$ should be limited to urban conditions

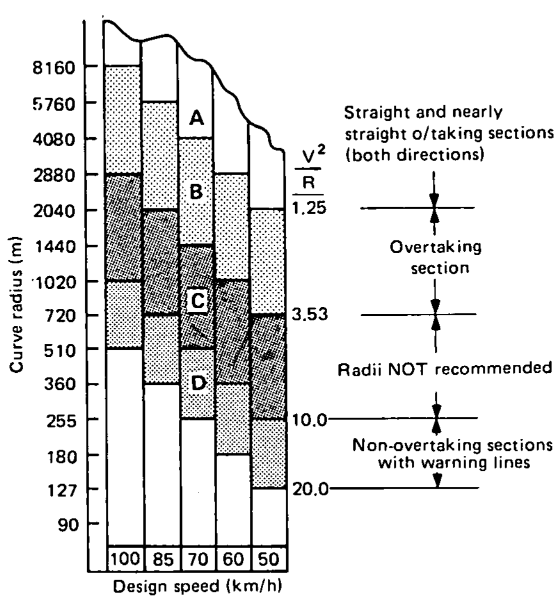


Fig. 15 Horizontal curve design recommended by TD9/81

radius is determined for a given curve design speed and speed environment. However the standard notes that radii greater than minimum, together with superelevation and friction less than maximum values, are usually adopted; and that curves are designed so that, over the range of speeds likely to occur, drivers will be required to turn their steering wheels towards the centre of curvature to generate the necessary side friction.

4.3 MINIMUM CURVE RADIUS

4.3.1 Fundamental relationship

For a given design speed, the minimum curve radius can be calculated from the equation $e + f = V^2/127.R$ using maximum values of e and f.

A maximum e value of 0.10 has generally been accepted for rural roads where ice and snow problems do not occur. A superelevation rate of 0.12 is sometimes used in very hilly terrain, but other factors should also be taken into account, such as the proportion of slow vehicles, the stability of high laden commercial vehicles, appearance of the road and the need to match levels at junctions and entrances.

The minimum curve radii recommended by AASHTO, TD 9/81 and NAASRA are given in Table 14, Table 4 and Figure 6 respectively.

Maximum f values depend on a number of factors such as driver comfort, vehicle speed, types and condition of the road surface, types and condition of tyres and expected weather. Various studies have shown a decrease in friction values for an increase in vehicle speeds.

4.3.2 AASHTO

Maximum f values recommended by AASHTO are based on comfort and safety criteria. The comfort criterion was determined by limiting the residual sideways force on the vehicle, which is related to the side friction factor. The safety criterion was satisfied by adopting smaller values than those observed from various experimental studies, as shown in Figure 16, varying from 0.17 at 20 mph to 0.10 at 70 mph.

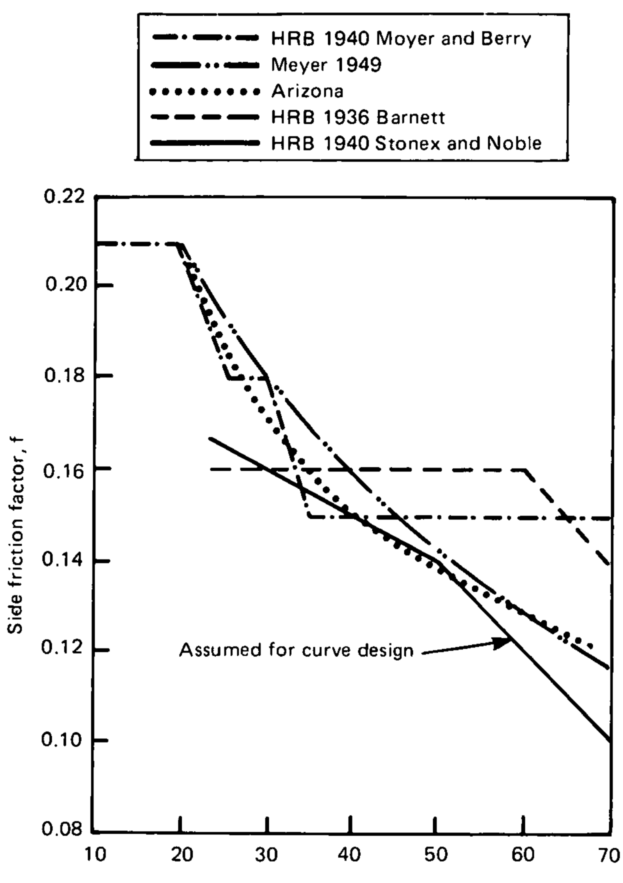


Fig. 16 AASHTO maximum safe side friction factors

4.3.3 TD 9/81

As mentioned in Section 4.2, TD 9/81 maximum values of f were based on the need to limit gross lateral acceleration to 0.22 g, a level established some fifty years ago from safety and comfort considerations. A study of driver discomfort due to lateral forces on curves (Leeming 1944, Leeming and Black 1950) confirmed this by suggesting that, when maximum superelevation ($e = 0.07$) is allowed for, the maximum design value of f should be 0.15. By requiring the 99th percentile speed vehicle to generate this f value on curves of absolute minimum radius, the corresponding 85th percentile speed gross lateral acceleration of 0.16 g on a curve of absolute minimum radius will require an f value of about 0.09 to be generated. Desirable minimum values of radius have been established using a limit on the gross lateral acceleration at design speed of 0.11 g, ie half the maximum value. With the same proportions of gross lateral acceleration taken by superelevation (45%) and by friction (55%), this results in a desirable f value of about 0.06, and a desirable maximum superelevation rate of 0.05 (5%).

Studies for TD 9/81 have shown that drivers use speeds that are reduced more on curves of lower radii compared with their approach speeds. However, the resulting calculated values of gross lateral acceleration that would be used by drivers on these curves were greater than the maximum value of 0.22 g used for design, indicating that a relaxation of standards below absolute minimum radius could be considered at very difficult sites. In these cases, values of limiting radius have been established with a maximum superelevation rate of 0.07, requiring f values of 0.15 to be generated by vehicles at design speed.

4.3.4 NAASRA

The proposed NAASRA design values for side friction factors were introduced in Section 2.3.4. The curve in Figure 17 was derived from the following considerations:

(i) For design speeds up to about 50 km/h, the curve recommended by Kummer and Meyer (1967) was adopted. This curve was based on the minimum recommended skid numbers for American roads which were a function of mean operating speed. Skid number can be regarded as being approximately equal to the wet side friction factor multiplied by 100. Although this curve was not adjusted to the 85th percentile speed (design speed), the curve was of more than two standard deviations below the mean side friction factors on horizontal curves measured in Australia and was therefore adopted as a lower bound estimate of the minimum pavement friction likely to be encountered and, as such, was regarded as the upper limit for plausible design f values.

(ii) For design speeds between 50 and 90 km/h, the data in Figure 4 was used to derive two sets of points based on:

(a) the upper 85th percentile confidence band of a linear regression on the data (Method 1, Figure 17) and

(b) grouping of the data in 10 km/h ranges to form the cumulative distributions shown in Figure 18 and then taking the 85th percentile value of each curve. (Method 2, Figure 17).

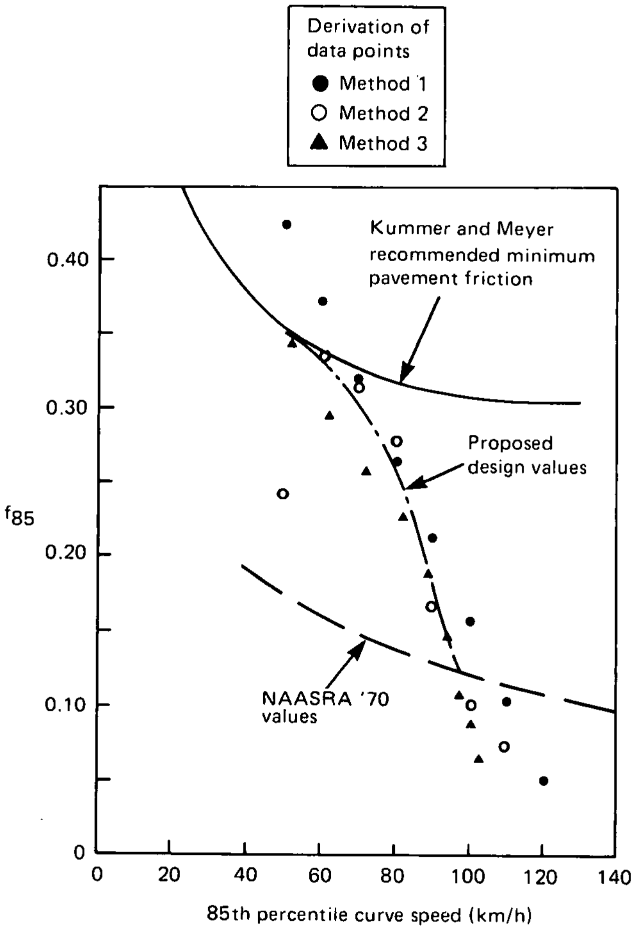


Fig. 17 Relationship between 85th percentile car side friction factor and 85th percentile car speed for Australia

A third set of points (Method 3) was calculated for 85th percentile side friction factors from the NAASRA 1973 curve speeds and side friction factors using the relationship between curve speed standards and 85th percentile curve speeds shown in Figure 3.

From the three sets of points proposed design values were derived using the best fitting curve to the sets of points and giving a smooth transition to the recommended values below 50 km/h and above 90 km/h.

(iii) For design speeds in excess of 90 km/h, the values recommended by NAASRA (1970) were retained. These values were higher than those utilised by drivers, but were adopted to provide a high level of safety and comfort.

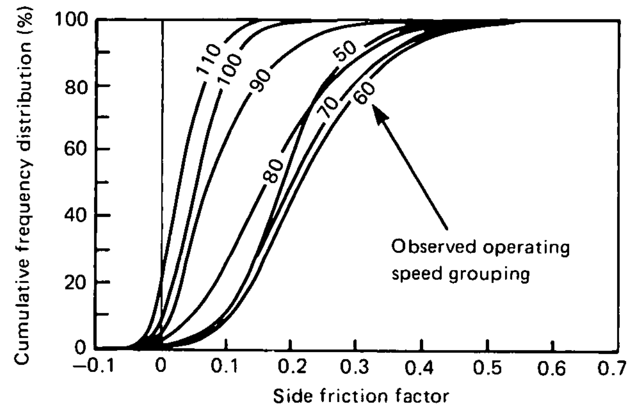


Fig. 18 Distribution of side friction factors computed from observed car speeds grouped by 85th percentile speed for Australia

The NAASRA curve/speed study found that the paths of vehicles transversing curves varied. On small radius curves, drivers tended to utilise the available lane width such that the vehicle path radius increased and the f value utilised was below that implied by the assumed circular path. For longer, large radius curves, however, drivers tended to decrease the radius of the vehicle path, utilising an f value greater than assumed. This study also suggested that there were many drivers who were prepared to tolerate a high degree of discomfort on horizontal curves. In addition, the higher f values utilised by drivers could also be due to improvements in road surfaces, tyres and vehicle performance since the earlier studies were carried out. Compared with the AASHTO standards, the NAASRA maximum values for f are higher, particularly for the lower design speeds, ranging from 0.35 at 50 km/h to 0.11 at 130 km/h.

4.4 TRANSITION CURVES

Transition curves are inserted between tangents and circular curves, or between circular curves of substantially different radius for the following reasons:

(i) to provide a gradual increase or decrease in the radial acceleration when a vehicle enters or leaves a circular curve.

(ii) to provide a length over which the superelevation can be applied.

(iii) to facilitate pavement widening on curves.

(iv) to improve the appearance of the road by avoiding sharp discontinuities in alignment at the beginning and end of circular curves.

The type of transition curve which is normally used in practice is the euler spiral, or clothoid. This spiral is defined by the degree of curvature at any point on the spiral being directly proportional to the distance along the spiral. There are several methods of determining the length of transition curves.

4.4.1 Shortt's method

This method was derived for the gradual increase in radial acceleration on railway curves. The equation used is

$$L_s = V^3/3.6^3 \, C \, R$$

where L_s = length of transition curve, metres
V = design speed, km/h
R = circular curve radius, metres
C = rate of increase of radial acceleration, metres/second^3

TD 9/81 adopts this equation to derive lengths of transition curves. It was recommended that the value of C should not normally exceed 0.3 m/sec^3 although, in difficult cases, it could be increased up to 0.6 m/sec^3.

A modified equation could also be used which takes account of the superelevation (e) on modern highways. This leads to much shorter lengths of transition:

$$L_s = \frac{V}{3.6^3 \, C} \left\{ \frac{V^2}{R} - 127e \right\}$$

This equation implies that, for a driver at the 'hands-off' speed for a particular curve radius and superelevation, then $L_s = 0$, which is theoretically correct.

Leeming (1944) observed that there was no theoretical justification for any particular length of transition curve, since driver comfort in negotiating superelevated curves was dependent on the value of lateral acceleration itself and not on its rate of increase, the latter being the basis for the above equations.

AASHTO suggests that roads do not need the same degree of precision in computing length of transition curve using either of the above equations. A more practical control was adopted known as the 'superelevation run-off' method.

4.4.2 Superelevation run-off method

Superelevation run-off is defined as the length of road required to achieve the change in superelevation from a normal cross section on a tangent to the fully superelevated cross section required on the circular curve, or vice versa. This length is determined such that the slope of the pavement edges (ie the edge of pavement profiles) over the transitional length compared with the centreline slope or profile should not exceed a maximum value. These maximum relative gradients are usually established from considerations of appearance of the road. The values recommended by AASHTO are given in Table 15.

Superelevation run-off is directly proportional to the total superelevation, which is the product of the lane width and the summation of the normal crossfall and superelevation rate:

$$L_e = \frac{b}{m} \, (e + e_n)$$

and $$L_s = \frac{b}{m} \, e$$

where L_e = superelevation run-off, metres
L_s = length of spiral, metres
b = lane width, metres
m = relative gradient
e_n = normal crossfall of pavement
e = superelevation of the curve

Table 16 shows the values recommended by NAASRA to obtain a smooth visual appearance.

TD 9/81 stipulates that the edge of carriageway profile gradients should not differ by more than 1 per cent with respect to the line of rotation to ensure satisfactory appearance.

TABLE 15

Relative gradients between pavement edge and centre-line for two-lane roads recommended by AASHTO

design speed (mph)	maximum relative grade (%)
20	0.75
30	0.67
40	0.58
50	0.50
60	0.45
65	0.41
70	0.40

TABLE 16

Relative gradients between pavement edge and centre-line for two-lane roads recommended by NAASRA

design speed (km/h)	maximum relative grade (%)
40 or under	0.90
60	0.60
80	0.50
100 or over	0.40

4.4.3 Rate of pavement rotation method

This method adopted by the NAASRA standards was based on driver comfort as well as road appearance criteria. The rate of pavement rotation (n) is defined as the change in crossfall divided by the time taken to travel along the length of superelevation transition at a given design speed. NAASRA recommends that the rate of pavement rotation 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 design speeds up to 70 km/h.

Thus

$$L_s = \frac{e\,V}{3.6\,n}$$

and $$L_e = L_s + \frac{e_n\,V}{3.6\,n}$$

where V = design speed, km/h

4.4.4 Other considerations

The superelevation run-off method in Section 4.4.2 can result, in some instances, in unacceptably short transition lengths, particularly for low superelevation rates and with higher design speeds. AASHTO therefore recommends minimum lengths, regardless of superelevation, ranging from 100 ft to 200 ft for design speeds of 30 and 70 mph respectively. These distances are approximately those travelled in two seconds at the design speed.

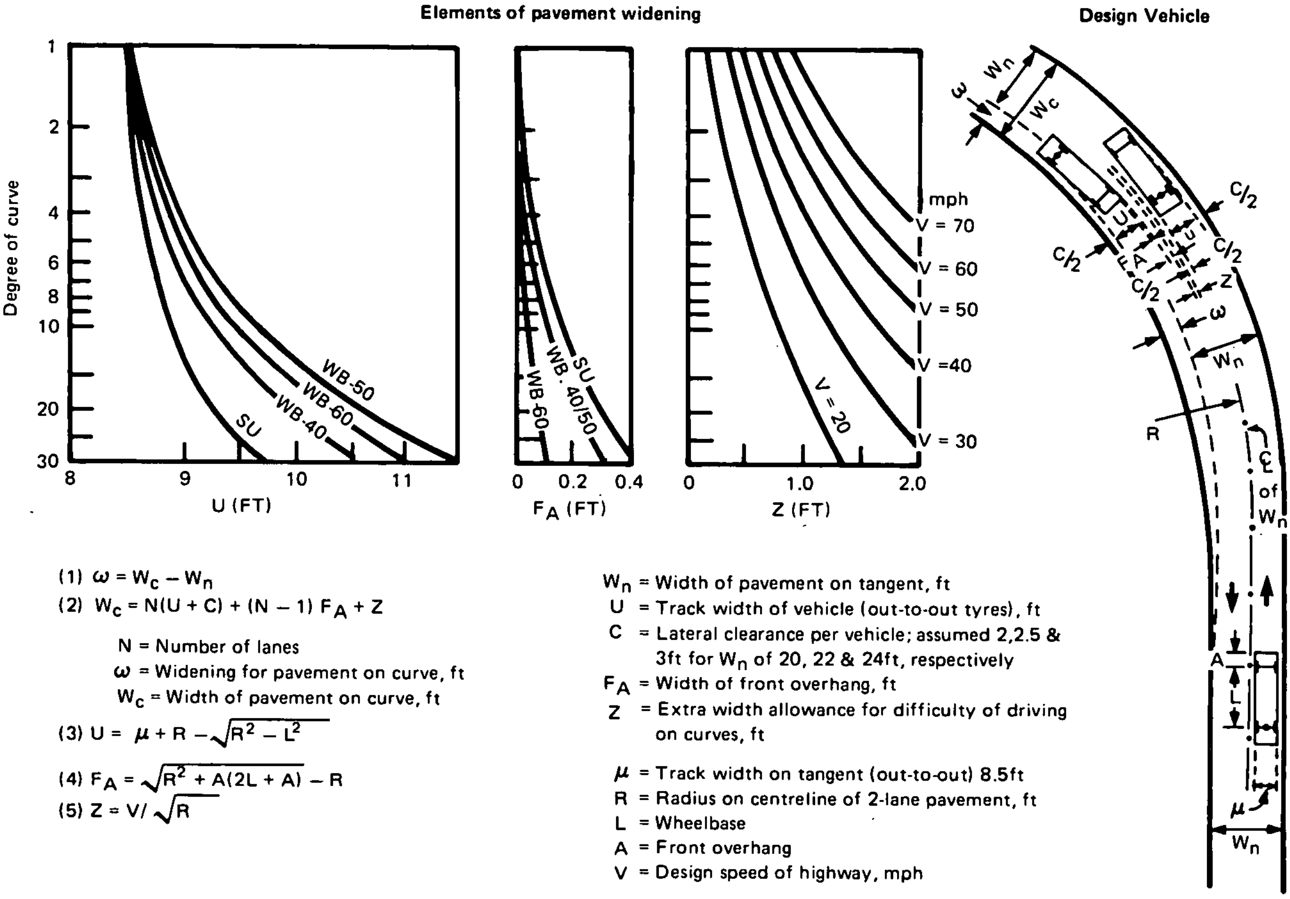
The TD 9/81 standard recommends that, on sharp curves, the length of transition should be limited such that the shift (p) of the circular curve is not greater than one metre

$$\text{ie } p = \frac{L_s^2}{24\,R} < 1 \text{ metre}$$

UK practice has traditionally preferred transition curves which consume an angle of at least 3° for aesthetic reasons. Thus, a minimum length of transition curve is given by: $L_s = R/9$.

NAASRA recomends that, for appearance purposes, length of transitions should be sufficient to provide a shift of between 0.25 and 0.5 metres. If the shift would otherwise be less than 0.25 metres, the transition may be omitted.

The AASHTO standards allow the transition to be omitted if the required superelevation is less than about 3 per cent, which would give shifts broadly consistent with the NAASRA recommendations. On



(1) $\omega = W_c - W_n$

(2) $W_c = N(U + C) + (N - 1)\,F_A + Z$

N = Number of lanes

ω = Widening for pavement on curve, ft

W_c = Width of pavement on curve, ft

(3) $U = \mu + R - \sqrt{R^2 - L^2}$

(4) $F_A = \sqrt{R^2 + A(2L + A)} - R$

(5) $Z = V/\sqrt{R}$

W_n = Width of pavement on tangent, ft

U = Track width of vehicle (out-to-out tyres), ft

C = Lateral clearance per vehicle; assumed 2,2.5 & 3ft for W_n of 20, 22 & 24ft, respectively

F_A = Width of front overhang, ft

Z = Extra width allowance for difficulty of driving on curves, ft

μ = Track width on tangent (out-to-out) 8.5ft

R = Radius on centreline of 2-lane pavement, ft

L = Wheelbase

A = Front overhang

V = Design speed of highway, mph

Fig. 19 Derivation of AASHTO criteria for widening on curves

such curves, a vehicle can follow a transitional path within its own lane without the provision of a transition curve. In addition, the effects of such curves on appearance is negligible.

4.5 PAVEMENT WIDENING ON CURVES

On horizontal curves, vehicle path width is larger since the rear wheels track inside the front wheels. In addition, there is a tendency for drivers to shy away from the edge of the carriageway as they traverse a curve. Therefore, pavements are sometimes widened on curves to provide a safe clearance between opposing vehicles.

The amount of widening required depends on

curve radius,
basic lane width on straight sections,
vehicle dimensions, and
required safe clearance (empirical value).

An empirical derivation for pavement widening on curves was developed by AASHTO and is reproduced in Figure 19. For practical reasons, and because of the empirical nature of the extra width derivation, it is recommended that design values for widening should be multiples of one half-foot and the minimum value should be two feet. Table 17 gives the values recommended by AASHTO assuming a rigid chassis design vehicle.

Neither the NAASRA nor TD 9/81 standard relate widening specifically to design speed, unlike the AASHTO standard. These standards for widening are shown in Tables 18 and 19.

4.6 COMMENTS ON HORIZONTAL ALIGNMENT

For the detailed design of horizontal curves, both safety and driver behaviour considerations have been important in establishing standards. Lower curve standards do give the designer more flexibility in difficult areas and can help reduce construction and land-take costs, but the safety and driver implications should also be considered in these cases.

TABLE 17

AASHTO calculated and design values for pavement widening on open highway curves with two-lane pavements, one-way or two-way.

Degree of curve	Widening, in feet, for 2-lane pavements on cuves for width of pavement on tangent of:														
	24 feet						22 feet					20 feet			
	Design speed, mph						Design speed, mph					Design speed, mph			
	30	40	50	60	70	80	30	40	50	60	70	30	40	50	60
1	0.0	0.0	0.0	0.0	0.0	0.0	0.5	0.5	0.5	1.0	1.0	1.5	1.5	1.5	2.0
2	0.0	0.0	0.0	0.5	0.5	0.5	1.0	1.0	1.0	1.5	1.5	2.0	2.0	2.0	2.5
3	0.0	0.0	0.5	0.5	1.0	1.0	1.0	1.0	1.5	1.5	2.0	2.0	2.0	2.5	2.5
4	0.0	0.5	0.5	1.0	1.0		1.0	1.5	1.5	2.0	2.0	2.0	2.5	2.5	3.0
5	0.5	0.5	1.0	1.0			1.5	1.5	2.0	2.0		2.5	2.5	3.0	3.0
6	0.5	1.0	1.0	1.5			1.5	2.0	2.0	2.5		2.5	3.0	3.0	3.5
7	0.5	1.0	1.5				1.5	2.0	2.5			2.5	3.0	3.5	
8	1.0	1.0	1.5				2.0	2.0	2.5			3.0	3.0	3.5	
9	1.0	1.5	2.0				2.0	2.5	3.0			3.0	3.5	4.0	
10–11	1.0	1.5					2.0	2.5				3.0	3.5		
12–14.5	1.5	2.0					2.5	3.0				3.5	4.0		
15–18	2.0						3.0					4.0			
19–21	2.5						3.5					4.5			
22–25	3.0						4.0					5.0			
26–26.5	3.5						4.5					5.5			

NOTE:
Values less than 2.0 may be disregarded.
3-lane pavements: multiply above values by 1.5.
4-lane pavements: multiply above values by 2.
Where semitrailers are significant, increase tabular values of widening by 0.5 for curves of 10 to 16 degrees, and by 1.0 for curves 17 degrees and sharper.

TABLE 18

NAASRA recommended values for curve widening for two-lane pavements based on a rigid design truck

curve radius (m)	total amount of widening (metres) where normal width of traffic lane is			
	6.0 m	6.5 m	7.0 m	7.5 m
<50	2.0	1.5	1.5	1.0
50–100	1.5	1.0	1.0	0.5
100–250	1.0	1.0	0.5	
250–750	1.0	0.5		
>750	0.5			

TABLE 19

TD 9/81 recommended values for curve widening

curve radius (metres)	pavement widening (metres) on carriagway width of	
	7.3 m	<7.3 m
<150	0.6 (7.9)	1.2 (7.9)
150–300	—	1.0 (7.3)
300–400	—	0.6 (7.3)

Note: Figures in brackets are the maximum allowable carriageway widths including widening.

Current economic assessment procedures for alternative alignments do not take the radius of individual horizontal curves into account in determining mean operating speeds and hence fuel and time costs. Mean operating speeds are related to overall horizontal curvature (bendiness).

The need to determine side friction factors, and hence minimum curve radius for design standards, means that studies of these aspects of driver behaviour on curves need to be carried out, as has been the case in all three countries.

Those studies for TD 9/81 standards have been concerned with accident rates and the relationship between approach speed and curve speed. The procedures outlined in Section 2 to determine design speed, together with a limited allowable relaxation of horizontal curve standards below desirable minimum, should ensure that some measure of consistency, and hence safety, is achieved in choice of horizontal curves in designs. Consistency of steering effort on successive curves of an alignment should also help to improve safety. The method of determining superelevation rates on curve radii above the minimum values ensures that the majority of drivers will have to turn their steering wheels towards the centre of the curve to generate the required friction to maintain equilibrium.

Detailed stūdies of speeds and friction factors on bends for the NAASRA standards have led to the adoption of side friction factors for design much more related to driver behaviour than previously.

The recommendation in TD 9/81 standards that horizontal radii should be chosen to provide clear overtaking and non-overtaking sections avoids curves giving dubious overtaking conditions. This principle would appear to have wider application provided appropriate pavement markings are used and adhered to by drivers. The use of gentle deflections of alignment on otherwise long straights is also desirable.

The use of transition curves on all but the largest radius curves is general practice in the three countries. Elsewhere, for large radius curves, the question arises as to whether shifts of vehicle paths within a lane are desirable.

Whilst there is some variation between the three countries in the methods used to determine transition curve length, perhaps the NAASRA rate of pavement rotation has most appeal as it takes into account driver comfort and quality of ride as well as the appearance of the road.

All three standards stipulate amounts of pavement widening on curves for basic narrow road widths and sharp curve radii. The AASHTO empirical derivation of pavement widening values is the most explicit and is a good example of how such standards can be determined from studies incorporating appropriate design vehicles and driver behaviour.

5 VERTICAL ALIGNMENT

5.1 GRADIENT

In current geometric design standards, maximum gradients are determined according to road class or design speed, terrain, and vehicle performance.

A chart was produced in the TD 9/81 standards to show the relationship between road user costs and gradients, and this is shown in Figure 20. It can be seen that the road user costs increase rapidly with increasing gradient. This chart allows the designer to carry out cost benefit analysis for individual gradients and particularly to investigate the economic implications of steep gradients. The standard user costs are for both single carriageways with climbing lanes, and dual carriageways. The disbenefits of steep gradients on single carriageways with insufficient traffic to justify a climbing line are insignificant and a minimum construction/ environmental cost solution is recommended in these cases.

USER COST

Dual C/ways $$\left[\text{Total UUC} \times \left(1+\frac{H}{20}\right)\right]+\left[£1.33 \text{ million} \times L\right]$$

Single C/ways $$\left[\text{Total UUC} \times \left(1+\frac{H}{20}\right)\right]+\left[£1.42 \text{ million} \times L\right]$$

Where L = Length of section in km
H = % HGVs

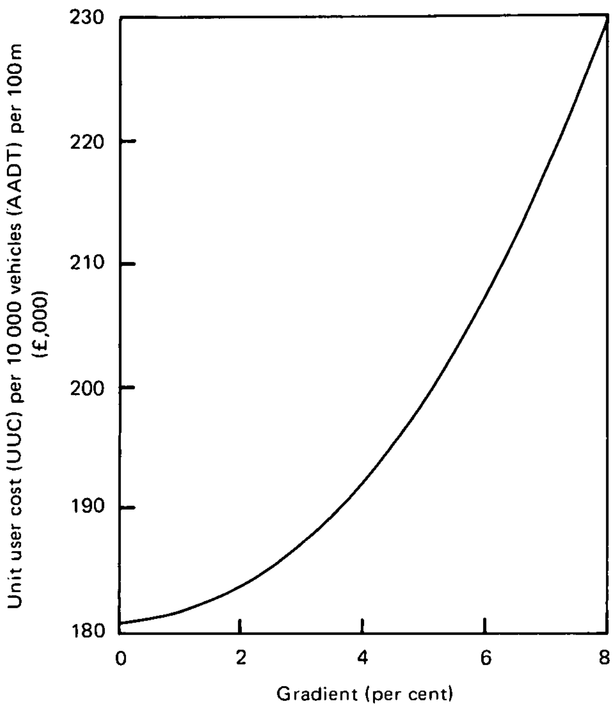


Fig. 20 Relationship between user cost and gradient from TD9/81

The AASHTO standard noted that passenger cars can readily negotiate gradients as steep as 4 or 5 per cent without appreciable loss in speed, except for cars with low power-to-weight ratios. The effect of gradients on truck speeds is much more pronounced and the maximum gradient that can be negotiated depends on the power-to-weight ratio. For the purpose of deriving geometric design standards, AASHTO described the terrain as follows:

Level terrain: That condition where sight distances, as governed by both horizontal and vertical restrictions, are generally long or could be made so without construction difficulty or major expense.

Rolling terrain: That condition where the natural slopes consistently rise above and fall below the road gradeline and where occasional steep slopes offer some restrictions to normal road alignment.

Mountainous terrain: That condition where longitudinal and transverse changes in the elevation of the ground with respect to a road are abrupt and where the road bed is obtained by frequent benching or side hill excavation.

Tables 20 and 21 give the maximum gradients recommended by the AASHTO and NAASRA standards.

TABLE 20

Maximum gradients recommended by AASHTO

(a) Local roads and streets

Type of terrain	Design speed (mph)				
	20	30	40	50	60
Level	8	7	7	6	5
Rolling	11	10	9	8	6
Mountainous	16	14	12	10	—

(b) Rural collectors

Type of terrain	Design speed (mph)					
	20	30	40	50	60	70
Level	7	7	7	6	5	4
Rolling	10	9	8	7	6	5
Mountainous	12	10	10	9	8	6

(c) Rural arterials

Type of terrain	Design speed (mph)		
	50	60	70
Level	4	3	3
Rolling	5	4	4
Mountainous	7	6	5

TABLE 21

General maximum gradients* recommended by NAASRA

Design speed (km/h)	maximum grade (per cent)		
	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	—

Notes: * Values closer to the lower figures would be aimed for on primary highways
† Grades over 10 per cent should be used with caution

The length of steep gradient can also be limited to maintain the quality of service of the road. AASHTO produced a chart to determine, for a given percentage and length of gradient, various speed reductions that would occur. This chart is shown in Figure 21 and is derived from consideration of the performance of a typical heavy truck of 300 pounds per horse power. It was recommended that the maximum, or critical, length of gradient which causes a speed reduction of not more than 10 mph is used in the design. A longer length of gradient could be used if a climbing lane is provided on the upgrade. On existing roads, Figure 21 can be used to determine where a climbing lane should start for various assumed reductions in the speed of trucks due to the gradient.

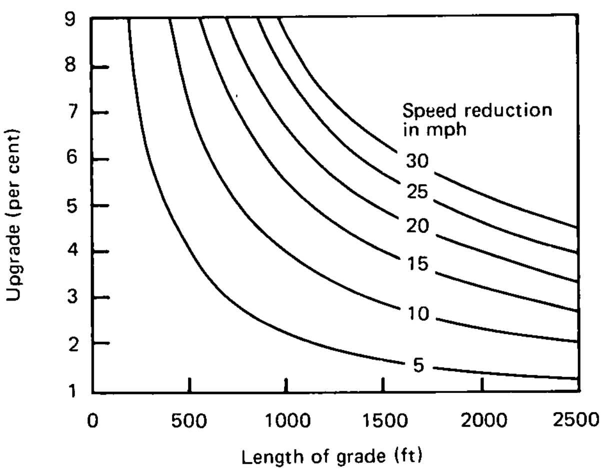


Fig. 21 AASHTO critical lengths of gradient for design (assumed typical heavy truck of 300lb/hp, entering speed = 55mph)

5.2 VERTICAL CURVES

Vertical curves are required to provide a smooth transition between consecutive gradients. For both crest and sag curves, the minimum length required may be fixed by sight distance or driver comfort cirteria. An absolute minimum length of vertical curve is usually stipulated to avoid poor appearance of the road when very short curves are used with shallow approach gradients.

The most common type of vertical curve used in practice is the simple parabola which gives a constant rate of change of curvature, and hence visibility, along its length. It is relatively easy to calculate this manually, the form being:

$$y = \frac{G.L}{200}\left\{\frac{x}{L}\right\}^2$$

where y = vertical distance from the tangent to the curve, metres
x = horizontal distance from the start of the vertical curve
G = algebraic difference in gradients, per cent
L = length of vertical curve, metres.

Minimum required lengths of crest curves are normally designed to provide sufficient sight distance during daylight conditions. Longer lengths would be needed to meet the same visibility requirements at night on unlit roads. Even on a level road, low-beam (meeting) headlight illumination may not show up small objects at the design stopping sight distances. However, higher objects and vehicle tail lights will be illuminated at the required stopping sight distances on crest curves, and it is felt that, since drivers are likely to be more alert at night, these longer lengths of curve are not justified.

Working on the parabolic properties of vertical curves it can be shown that for crest curves:

$$\text{For } S<L \quad L_{minimum} = \frac{G.S^2}{200\ (\sqrt{h_1}+\sqrt{h_2})^2}$$

$$\text{For } S>L \quad L_{minimum} = 2S - \frac{200(\sqrt{h_1}+\sqrt{h_2})^2}{G}$$

where $L_{minimum}$ = the minimum length of vertical crest curve, metres
S = sight distance required, metres
G = algebraic difference in gradients, per cent
h_1 = driver eye height, metres
h_2 = object height, metres

Note: For $S<L$, then L/G = Constant (K) for a given design speed and eye and object height.

During daylight hours, it is assumed that adequate sight distance is available on sag curves. At night, however, visiblity is limited by the distance illuminated by the headlamp beams. Working on the parabolic properties of vertical curves it can be shown that for **sag** curves:

$$\text{For } S<L: \quad L_{minimum} = \frac{G.S^2}{200(h_1 + S\tan\theta)}$$

$$\text{For } S>L: \quad L_{minimum} = 2S - \frac{200(h_1 + S\tan\theta)}{G}$$

where h_1 = headlight height (AASHTO = 2 ft, 0.6 metres)
θ = angle of upward divergence of light beam (AASHTO = 1°)

Note: For $S<L$, then L/G = Constant (K) for a given design speed.

However, on sag curves, it is doubtful if headlamps can illuminate the road where longer stopping distances at higher speeds are involved, and this is particularly the case for low meeting beams.

In these cases, it is better to base design of curve lengths on a driver comfort criterion, using a limitation on the vertical acceleration experienced when traversing vertical curves. The minimum lengths of vertical curve based on this criterion are calculated from the following equation:

$$L_{minimum} = \frac{G.V^2}{1300C}$$

where $L_{minimum}$ = the minimum length of vertical curve, metres
G = algebraic difference in gradient, per cent
V = design speed, km/h
C = vertical acceleration, metres per sec^2

Note: L/G = Constant (K) for a given design speed and vertical acceleration.

NAASRA recommended values of vertical acceleration (C) of less than 0.05 g, where g is the gravitational constant. These values of C could be increased to 0.10 g for low standard roads. The value of 0.03 g was recommended by AASHTO, and also adopted in TD 9/81.

The driver comfort criterion applies to both crest and sag curves. However, on crest curves, this criterion gives much shorter lengths of curve than the stopping distance criterion, except for low design speeds. Therefore for crest vertical curves 'rounded' K values for design are usually based on stopping.

The appearance of vertical curves should be considered if the value of G is relatively small. Table 22 shows the minimum lengths of vertical curve for satisfactory appearance recommended by NAASRA. AASHTO recommend that minimum lengths of curve (ft) should be at least three times the design speed (mph).

TABLE 22

Lengths of vertical curves based on appearance criterion as recommended by NAASRA

Design speed (km/h)	length of vertical curve for satisfactory appearance (metres)
40	20–30
60	40–50
80	60–80
100	80–100
120	100–150

When sight distance or comfort criteria are used to establish lengths of vertical curve, it has been seen that the length of curve required to achieve a one per cent change in gradient (L/G) is equal to a constant (K) for a given design speed, eye and object heights, or limiting value of vertical acceleration. The standards of all three countries makes use of rounded K values for design purposes which, when multiplied by the algebraic difference in gradient, give the required curve lengths. The AASHTO, TD 9/81 and NAASRA rounded K values for stopping and passing on crest curves and for stopping/comfort on sag curves are shown in Tables 23, 24, 25 respectively.

The TD 9/81 standard does not recommend the use of desirable minimum K values in the design of crest curves on single carriageway roads since they will create dubious overtaking opportunities. It is considered advantageous to provide absolute minimum K values with appropriate line markings to create a clear non-overtaking section of road.

TABLE 23

AASHTO rounded K values

Design speed (mph)	K values (ft per per cent change in gradient)		
	crest curves stopping	crest curves passing	sag curves stopping
20	10–10	210	20–20
25	20–20	300	30–30
30	30–30	400	40–40
35	40–50	550	50–50
40	60–80	730	60–70
45	80–120	890	70–90
50	110–160	1050	90–110
55	150–220	1230	100–130
60	190–310	1430	120–160
65	230–400	1720	130–180
70	290–540	2030	150–220

Whilst passing sight distances are about two to three times as long as stopping sight distances, required lengths of crest curves for passing can be up to twenty times as long, showing that generally it is not feasible to provide for passing sight distance visibility over crest curves.

For K values greater than 167 ft, AASHTO suggests that careful consideration must be given to drainage design in the vicinity of the apex of curves, particularly on long curves with kerbed pavements because of the flat profiles created.

5.3 COMMENTS ON VERTICAL ALIGNMENT

Whilst all three countries establish maximum gradients from general considerations of road class, terrain, design speed and vehicle performance, the use in TD 9/81 of user costs including delays, vehicle operation and accidents, to evaluate steep gradients is an extension of the cost/benefit analysis approach into the detailed assessment of individual design components, and is an example of the need to give increased consideration to the cost effectiveness of designs. This will involve assessment of both quantifiable and non quantifiable effects of the design. The latter can give rise to difficulties. For

TABLE 24

TD 9/81 rounded K values

	K values (metres per per cent change in gradient			
Design speed (km/h)	crest curves des min stopping	crest curves abs min stopping	crest curves passing	sag curves abs min
50	10	6.5	100	9
60	17	10	142	13
70	30	17	200	20
85	55	30	285	20
100	100	55	400	26
120	182	100	—	37

TABLE 25

NASSRA rounded K values

Design speed (km/h)	K comfort		K crest stopping		K passing
	C = 0.05 g	C = 0.10 g	$h_2 = 0.2$ m	$h_2 = 0$	(establishment)
50	4	2	5.4	10.8	133
60	6	3	9.2	18.4	220
70	8	4	15.7	31.4	353
80	10	5	23.9	47.6	532
90	13	6.5	42.5	85.0	767
100	16	8	62.7	125.4	1109
110	19	9.5	95.7	191.4	1591
120	23	12	135.5	271.0	2223
130	—	—	195.2	390.0	3104

Design speed (km/h)	K sag (headlight illumination) Zero upward angle of divergence headlight height = 0.75 m
50	17
60	28
70	48
80	74
90	131
100	150
110	150
120	150
130	150

instance, when considering the effects of gradients on the quality of service of the road, there will be factors which do not lend themselves readily to quantification such as driver comfort and stress.

Given the very long curve lengths that are required to design for safe passing, designers will generally find that such provision is not cost effective. The TD 9/81 recommended use of absolute minimum crest curve lengths with clear non-overtaking line markings from considerations of safety and construction cost is worthy of wider application.

Widening of the carriageway to provide overtaking opportunities on crest curves, particularly in conjunction with climbing lanes, might well be a more feasible solution if the traffic volume is sufficiently high.

Hence designers will generally be providing for adequate stopping sight distance or using a driver comfort criterion in the design of vertical curves. On unlit roads, provision of visibility for stopping at night for the higher design speeds is affected by the ability of the vehicle headlamps to illuminate objects at the longer stopping distances required. Low meeting beam illumination is inadequate in most cases. These difficulties have led to the use of a driver comfort criterion for the design of sag curves in TD 9/81 and NAASRA standards which result in shorter curves compared to those given by stopping distance criteria. NAASRA states that 'the only method of achieving full compatibility between theoretical sight distance by day and night is by roadway lighting.'

6 CROSS SECTION

6.1 ROAD WIDTH

The following factors need to be taken into account when determining road width:

(i) Classification of the road:
A road is normally classified according to its function in the road network. The higher the class of road, the higher the level of service expected and the wider the road will need to be.

(ii) Traffic:
Heavy traffic volumes on a road means that passing of oncoming vehicles and overtaking of slower vehicles are more frequent and therefore the paths of vehicles will be further from the centreline of the road and the wider should be the traffic lanes.

(iii) Vehicle dimensions:
Normal steering deviations and tracking errors particularly of heavy vehicles reduce clearances between passing vehicles. Therefore these higher traffic volumes require wider traffic lanes.

(iv) Vehicle speed:
As speeds increase, drivers have less control of the lateral position of vehicles, reducing clearances, and so wider traffic lanes are needed.

AASHTO recommends that 12 ft lane widths (3.65 m) should be provided on main roads ensuring desired clearances between commercial vehicles. For two lane rural highways less than 22 ft wide, inadequate vehicle and edge of pavement clearances were found to be undesirable features with even moderate volumes of mixed traffic. At higher volumes, 10 and 11 ft lanes result in a significant reduction in capacity, as well as affecting driver comfort and accident rates. AASHTO considers that an effective pavement width of 20 ft is only adequate for low volume roads where the proportion of heavy vehicles is low.

NAASRA recommends the use of single lane carriageways on roads where traffic is less than 150 vpd in open terrain. To avoid excessive shoulder wear, the traffic lane width should be at least 3.5 m, and the shoulder should be capable of carrying vehicles in dry and wet weather. Use of a traffic lane width in the range 4.5 to 6.0 metres is not recommended as meeting vehicles may both attempt to remain on the traffic lane. NAASRA recommendations for minimum widths of sealed traffic lanes on undivided roads are shown in Table 26.

TABLE 26

Pavement widths, shoulder widths and/or roadway widths proposed by NAASRA

design traffic volume (vpd)	1–150	150–500	500–1000	>1000
minimum lane width (metres)	one lane 3.5	two lanes 3.0	two lanes 3.0–3.5	two lanes 3.5
minimum shoulder width (metres)	1.5–2.5	1.0–1.5	1.0–2.0	1.0–3.0

TD 9/81 recommends pavement widths of 7.3 and 10.0 m for single carriageway roads, and 7.3, 11.0 and 14.6 m for 2, 3 and 4 lane dual carriageways excluding edge treatments. This standard recommends full consideration be given to a wide single two-lane carriageway (WS2, basic width 10 m) solution. This type of road provides improved overtaking opportunity and an improved level of service for higher volumes of traffic, and evidence suggests that they are safer than the conventional 6.5–7.5 m carriageways.

6.2 SHOULDER WIDTH

Wide shoulders have the advantage of enabling a vehicle to pull off the travelled lanes of the carriageway and to stand clear of moving traffic, thus avoiding creation of a hazard and maintaining the capacity of the travelled lanes. They also enable a driver to deviate to avoid collision with objects on the road and allow room for regaining control of the vehicle. Wide shoulders also create a sense of openness and hence add to driver comfort as well as improving sight distances on horizontal curves.

NAASRA recommends a minimum shoulder width of 1.0 m for two lane roads as shown in Table 26. Wider shoulders can add significantly to construction costs and if minimum widths are generally used, full width stopping places should be provided at intervals where their provision can be carried out inexpensively. A stopping place width of 3.0 m will cater for heavy vehicles. NAASRA suggests a sealing of 0.3–0.5 metre width of shoulder is desirable on low volume roads for edge wear reduction, and reports that sealing of a one metre strip is common practice in Australia.

AASHTO recommends a desirable 10 ft shoulder which allows up to a 2 ft clearance between a stopped vehicle and the edge of pavement. In difficult terrain and on low volume roads, a minimum usuable width of 2 ft is recommended with 6 to 8 ft preferable. AASHTO states that lateral obstructions placed 6 ft from the edge of pavement do not affect road capacity.

6.3 PAVEMENT CROSSFALL

The purpose of pavement crossfall is to drain the road surface. Hence the crossfall should be as flat as drainage needs permit and these, in turn, are conditioned by the type of pavement and nature of the surface. A reasonably steep lateral slope is desirable to minimise water ponding on flat longitudinal sections due to pavement imperfections or unequal settlement. On the other hand, pavements with steep crossfall can be uncomfortable for drivers because vehicles will tend to veer toward the edge of pavement.

Table 27 shows the normal crossfall for pavements recommended by AASHTO. The values for high traffic type surfaces are based on consideration of driver needs and require accuracy and smoothness of the finished surface. These values will assist in providing safe operation under high traffic volumes and speeds, particularly in overtaking where crown lines are crossed.

Greater crossfalls are desirable to drain intermediate traffic type surfaces because of the likelihood of less accuracy in construction and of unevenness of the pavement surface, though they are less desirable from a traffic operational view.

The steeper crossfalls recommended for low traffic type surfaces are intended to prevent the absorption of water into the surface due to the possible use of less impervious layers and greater surface irregularity.

Table 28 shows the normal crossfall for the different types of pavement recommended by NAASRA.

TD 9/81 standards recommended a normal crossfall of 2.5 per cent for both bituminous and concrete pavements.

6.4 SHOULDER CROSSFALL

Shoulder crossfall on straight sections of road should enable surface water to drain away easily from the pavement and the values used therefore depend on the materials with which the shoulder is constructed. If shoulder materials are more porous and weaker than pavement materials, steeper shoulders should be provided. However, construction work is simplified if shoulder crossfall is the same as that of the pavement.

AASHTO recommends shoulder slopes of 2 to 6 per cent for hard shoulders, 4 to 6 per cent for gravel or crushed rock shoulders and about 8 per cent for turf shoulders. However, it recognises that the algebraic difference in the pavement and shoulder grades should be limited to avoid too great a cross slope break for traffic operational reasons. On superelevated curves, outer shoulders should desirably be sloped upward at about the same or at a lesser rate than the superelevation of the pavement. If shoulder crossfall is flatter than the pavement superelevation, AASHTO recommends that the algebraic difference in the pavement and shoulder grades should be limited to 8 per cent. This will help reduce the hazards in drivers pulling off the carriageway over the cross slope break or being incorrectly positioned within the travelled lane.

Any shoulder which is sloped against a sealed pavement should also be sealed to prevent loose material washing over the pavement surfaces.

NAASRA advises that, on tangent sections, shoulder crossfall can be up to 2 per cent steeper than that of the adjacent traffic lanes. Hence recommended shoulder crossfall on straights are 6 per cent for earth/loam shoulders, 4 to 5 per cent for gravel shoulders and 3 to 4 per cent for sealed/stabilised shoulders. On curves shoulders can be superelevated to crossfalls not less than that of the travelled pavement.

TABLE 27

Normal crossfall of pavements recommended by AASHTO

surface type	range of normal crossfall (per cent)
high	1.5–2.0
intermediate	1.5–3.0
low	2.0–6.0

TABLE 28

Normal crossfall of pavement recommended by NAASRA

type of pavement	normal crossfall (per cent)
Earth, loam	5.0
Gravel, water-bound macadam	4.0
Bituminous seal coat	3.0–4.0
Bituminous concrete	2.5–3.0
Portland cement concrete	2.0–3.0

6.5 COMMENTS OF CROSS-SECTION

The important considerations here are width and crossfall of pavements and shoulders.

The volume of traffic and the type of vehicle in the traffic stream are the main factors affecting pavement width. Above a traffic volume of about 1000 vpd, standard lane widths of 3.5 or 3.65 m are desirable considering the reductions in capacity, driver comfort and safety associated with narrower lanes. For very low traffic volumes NAASRA advocates the use of single lane carriageways in open terrain. For traffic volumes in between and in less open terrain two lane carriageways are a minimum requirement, but narrower lane widths can be considered as providing acceptable levels of service.

The TD 9/81 recommendation that full consideration be given to wider single two-lane carriageways to provide improved levels of service and safety should perhaps be noted with caution for use elsewhere due to its driver behaviour implications.

Regarding shoulder width, the AASHTO levels of provision are the highest in the three countries. The NAASRA minimum shoulder width of 1.0 metres on two lane roads for traffic volumes less than 1000 vpd, together with frequent provision of wider stopping places, is a useful compromise in terms of construction cost and levels of service and safety. Pavement crossfalls are related to the classification of the surface quality as this affects drainage capability and driver behaviour. Higher traffic volumes require better surface quality for driver comfort and safety and therefore lower pavement crossfalls are desirable. On lower surface qualities, drainage of the road surface assumes a more important role than driver comfort and safety for the low traffic volumes on these surfaces. Shoulder crossfalls should not be designed in isolation from pavement crossfalls. It is desirable to use similar crossfalls on shoulders as on the adjacent pavement for ease of construction and to avoid cross slope breaks which, particularly on horizontal curves, can reduce safety.

7 APPLICATION OF STANDARDS IN DEVELOPING COUNTRIES

7.1 AVAILABLE STANDARDS

The American, Australian and British standards that have been described are representative of a range of standards in use in the industrialised countries. Each has been recently revised and the Australian and British codes now provide for less conservative and more cost-effective designs than their predecessors.

A variety of standards are also used in developing countries and a survey of some of these was carried out by Cron (1975). It was shown by Cron that there were many similarities among the standards from the 63 countries studied and this is illustrated in Figures 22–24 for stopping sight distance, passing sight distance and minimum radius of horizontal curvature. Cron recommended standards based on the average values found from his study. However, it is clear from looking at his results that many of the countries studied had based their own standards on the then current American standards (AASHO 1965). It is therefore not too surprising that Cron's own recommendations also approximate to the AASHO 1965 standards. It is clear that there is little in the way of rational basis to these recommendations other than an averaging approach resulting from the use of American standards in many developing countries.

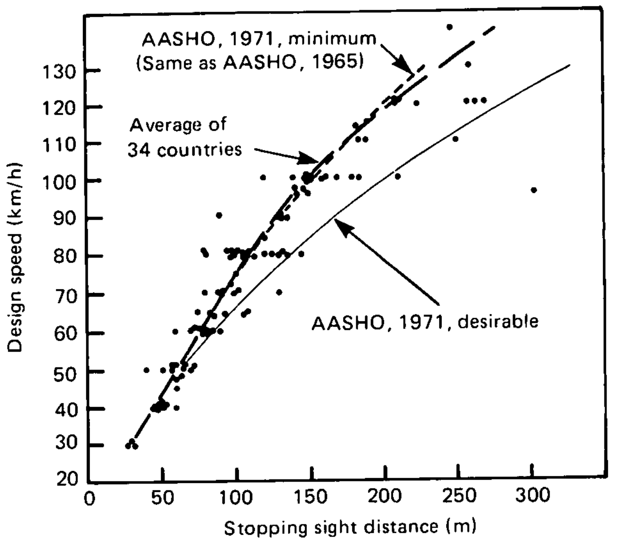


Fig. 22 Relationship between design speed and stopping sight distance in 34 countries

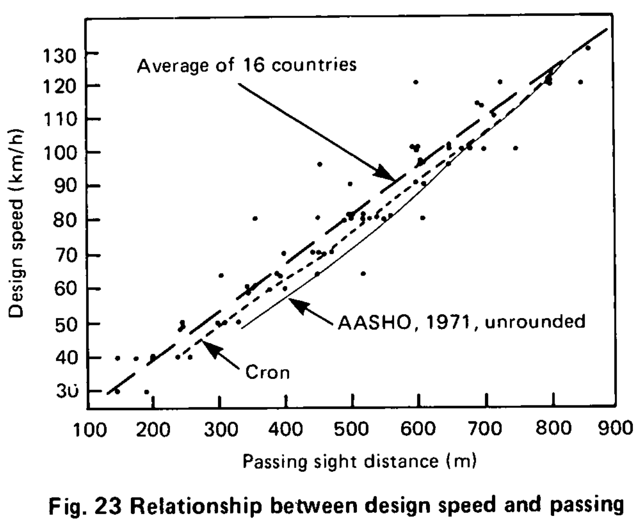


Fig. 23 Relationship between design speed and passing sight distance in 16 countries

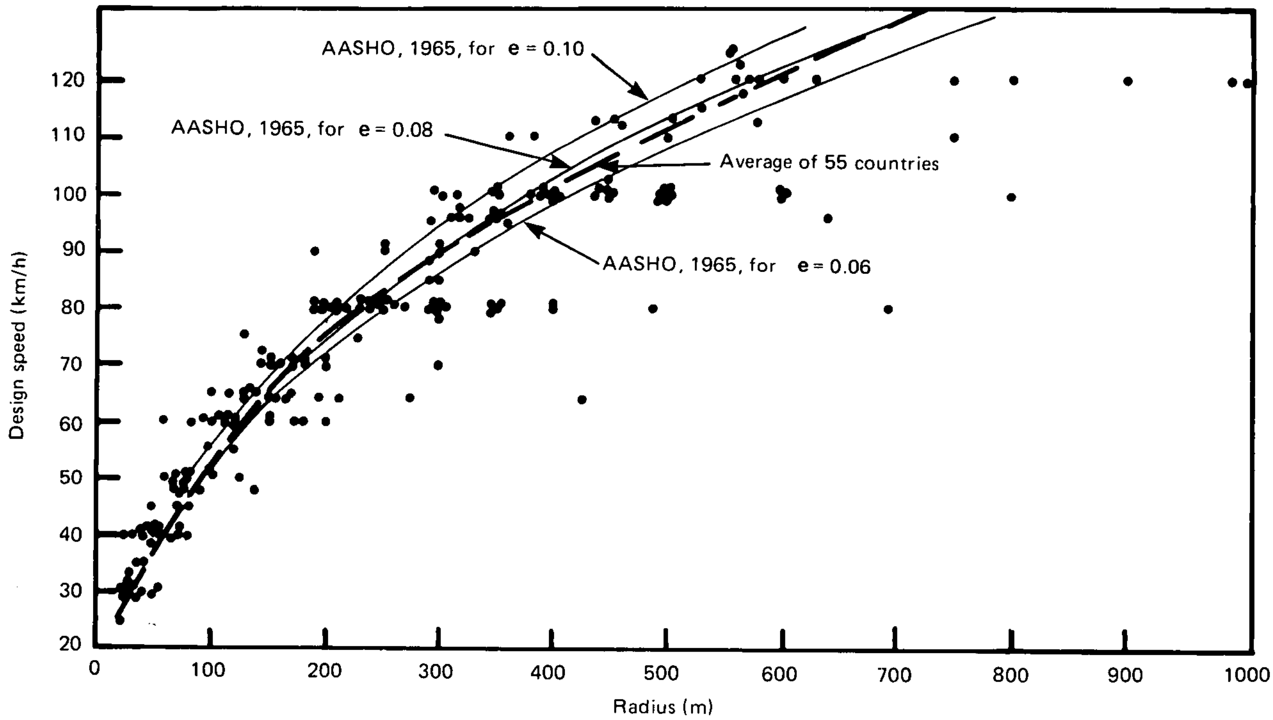


Fig. 24 Relationship between design speed and radius in 55 countries

In the book 'Low cost roads', Odier *et al* (1971) suggested standards for roads in developing countries, but the basis for these is not clear, other than being based on even earlier American standards published in 1954.

A literature search carried out at TRRL has failed to identify any developing country standards that are based on research into economics and safety carried out in that country.

7.2 CONSIDERATIONS FOR DEVELOPING COUNTRIES

7.2.1 Level of development

McLean (1978d) has noted that, in the light of experience from the developed 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 have traditionally been applied to low-volume roads, as was shown by Cron (1975). 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).

7.2.2 Traffic requirements

It has been noted (Hills *et al* 1984) that the needs of road users in developing countries are often very different from those in the industrialised countries. In developing countries, pedestrians, animal drawn carts, etc, are often important components of the traffic mix, even on major roads. Lorries and buses often represent the largest proportion of the motorised traffic. In the industrialised countries, traffic composition is dominated by the motor car. As a result, there may be less need for high speed roads in developing countries and it will often be more appropriate to provide wide and strong shoulders to allow their use by slow-moving vehicles at the cost of some reduction in the standards of alignment. Traffic volumes on most rural roads in developing countries are also relatively low. Thus, providing a road with high geometric standards may not be economic, since transport cost savings may not offset construction costs. The requirement for wide carriageways, flat gradients and full overtaking sight distance may therefore be inappropriate. Also, in countries with weak economies, design levels of comfort used in industrialised countries may well be a luxury that cannot be afforded.

7.2.3 Road safety

Little research has been carried out in developing countries on the relationships between accident rates and road geometry, but that which has been undertaken indicates that the number of junctions per kilometre appears to be the most significant factor, followed by horizontal and vertical curvature (Jacobs 1976). In most of the countries studied, there was little variation in road width but, in one country where considerable variation did occur, the data suggested that, on a range of roads carrying between 200 and 2000 vpd, an increase in width from 5 to 7 metres might reduce accident rates by up to 40 per cent.

High accident rates were observed on gravel roads. Among the possible causes of this might be poor geometry, slipperiness of the surface in wet weather and poor visibility caused by dust and high vehicle speeds. Since, in two of the countries studied, accident rates reduced with reduced road roughness, it is likely that by paving gravel roads, accident rates will be reduced.

Results of work on the relationship between accident rates and traffic flow tend to be inconclusive, even in the developed world. This uncertainty can be largely attributed to the multi-causal nature of accidents. From the limited studies carried out in developing countries, the number of accidents per unit length of road appeared to increase at the same rate as the traffic flow. Within this overall relationship, it is possible that single vehicle accident rates are decreasing whilst vehicle-vehicle accident rates are increasing.

Results so far obtained suggest that the accident rates in developing countries are considerably higher than in the developed ones for similar levels of vehicle flow and geometric design. This is probably because other factors are involved which have not been measured, such as road user behaviour and vehicle condition and maintenance.

Thus, from the point of view of safety, it appears that geometric standards used in the developed countries are not appropriate to the developing world. More research is needed to enable safety aspects to be used to provide appropriate geometric standards from first principles.

7.2.4 Network considerations

When constructing or improving the road network when economic constraints apply, the most economic solution for one road link may not necessarily be the best solution for the network as a whole. The high cost of implementing one project may consume funds which would be better spent over all of the network. In developing countries, there will usually be gaps in the basic road network in addition to poor geometric standards over the network. If individual projects are designed and built to traditional standards, this will reduce the rate at which new roads can be provided to fill the basic gaps in the network. It will generally be more appropriate to use the lowest practicable standards in order to maximise the length of road that can be constructed.

7.3 DEVELOPMENT OF LOCAL STANDARDS

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 in Section 7.2.1. Thus, the objectives will be:

for Level 1: to provide access;

for Level 2: to provide additional capacity;

and 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. Some studies (Oglesby and Altenhofen 1969, Hide *et al* 1975) have suggested that, for relatively low traffic volumes, road widths in excess of 5 metres cannot be justified in terms of accident reductions or traffic operations. It may be more appropriate to design a variable road width where the cross-section is narrow on straights, but is increased on bends or where other restrictions on sight distance apply.

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 by

AASHTO, NAASRA or TD 9/81 become relevant. It is not really practicable to apply standards such as these to roads at Levels 1 or 2. Because the requirements of roads in developing countries are different to those in the industrialised countries where these standards were developed, the AASHTO, NAASRA and TD 9/81 standards should only be applied with caution in developing countries, even to Level 3 roads. Before they are applied, it is necessary to review the assumptions on which the standards have been based to determine where they are appropriate for conditions found in individual countries. To assist with this, the principal assumptions are reviewed in the next sub-section.

7.4 REVIEW OF ASSUMPTIONS

7.4.1 Design speed

The AASHTO method of determining design speed is based on a qualitative assessment of traffic volume and terrain conditions. It has the objective of achieving consistency of standards commensurate with the function of the road, and a balance between construction and operating costs. Economic trade-offs between the extra costs of higher standards and savings in operating costs are likely to be worthwhile only at higher traffic volumes, and then only if travel time is valued.

NAASRA introduces the concept of speed environment related to terrain and range of horizontal curvature along an alignment. The design speed of individual geometric elements is related to the speed environment and, on successive elements, should not differ by more than 10 km/h.

The TD 9/81 design speed standard is based on overall alignment constraints and roadside friction values. Relaxation of standards is allowed on cost grounds, but these still provide acceptable levels of safety and operating conditions.

Both NAASRA and TD 9/81 methods involve initial assumptions of speeds and standards for the design of trial alignments. From the trial design, overall measures of speed environment or design speed are then checked against the initial assumptions, thus indicating whether the design should be upgraded or relaxed for overall consistency and cost-effectiveness.

Ideally, before any such methodologies are applied in a developing country, then fundamental studies are needed into vehicle speeds on straights, gradients, and horizontal and vertical curves. These would enable the design of individual elements to be related to observed driver behaviour, and thus expectancy, and to an overall consistency of standard. If the results of such studies are not available, then subjective judgements must be made when applying any of the industrialised country standards to local conditions.

Design speed has been the traditional method of achieving overall consistency of standards and has normally been chosen to represent the faster vehicles in the traffic stream, typically the 85th percentile speed. Given the different traffic mix in developing countries with the larger proportion of commercial vehicles, there may be some scope for designing, instead, for the average (50th percentile) vehicle. Such a relaxation in standards would be particularly appropriate in difficult terrain where more cost-effective designs would result.

Consistency of standards between geometric elements is clearly important and the standards reviewed suggest that design should ensure that speeds on successive elements do not differ by more than about 15 km/h.

7.4.2 Sight distance

All the standards are based on assumed values of driver reaction time in the range of 2.0–2.5 secs and values of the coefficient of longitudinal friction. The resulting values of stopping sight distance in the three standards are very similar. However, the NAASRA manoeuvre sight distance is an attractive concept which produces more cost-effective designs.

Sight distances also depend on assumptions about driver eye height and object height. Simple studies can be carried out in developing countries to determine appropriate values, but are unlikely to produce radically different values to those used in the three standards which are all very similar.

Differences in the assumptions about driver behaviour on overtaking result in large differences in passing sight distances in the different standards. However, driver behaviour differences in different countries may well be greater than the differences between the standards themselves. The provision of full overtaking sight distance results in very expensive designs and consideration should perhaps be given to the TD 9/81 approach of providing clearly marked non-overtaking crests and horizontal curves with minimum standards and longer straights between. The main concerns with such an approach are the different standards of driver behaviour in developing countries and the levels of maintenance required to keep line markings visible on the road.

7.4.3 Horizontal alignment

To determine horizontal alignment standards, the first step is to decide upon appropriate values of e_{max} and f_{max}. Vehicle and tyre conditions and wet skid resistance on bends need to be considered in order to determine these values. Considerations of these factors suggest that, for design, values of coefficient of friction closer to those used in the US and UK are more appropriate for developing countries than the higher values in Australia, except perhaps in dry climates.

Given satisfactory maximum design values of coefficient of friction and superelevation, minimum curve radii can then be established for a given design speed. Emphasis should then be placed on providing curves to this standard, which are as sharp and short as possible. This ensures an economic design as well as clear non-overtaking bends.

For curves above the minimum radius, it will be necessary to determine what superelevation is needed. Any of the three methods can be used, but there is some advantage in trying to achieve consistency of steering effort such as implicit in the TD 9/81 standard. Any of the methods of applying transition curves can probably be used, but the NAASRA rate of pavement rotation method is attractive because it claims to take into account driver comfort, quality of ride and road appearance.

Widening on bends may be needed on curves of small radius. Of the three standard methods available, the AASHTO approach seems to be the most comprehensive, but the choice really depends on the availability of funding and must be a local decision.

7.4.4 Vertical alignment

It has already been noted that it will probably not normally be economical to design crest curves with full passing sight distances. If overtaking is needed in these situations, it may be more appropriate to provide a wider road over crests to enable manoeuvring to take place in the event of vehicle conflict. In daylight, a minimum of stopping or manoeuvre sight distance must be provided but, at night, it will not usually be possible to provide either of these within the headlight distance.

There is also a problem of providing a sufficient length of illumination under meeting beam conditions on sag curves and these will normally need to be designed on the basis of driver comfort considerations. Sag curves should be designed primarily on economic criteria, but checking that the values of vertical acceleration (C) do not exceed 0.03 g–0.05 g.

The maximum gradients suggested in TD 9/81 seem very restrictive for use in developing countries. The AASHTO and NAASRA values are perhaps more appropriate, with the latter offering slightly more flexibility. Ideally, gradients should be designed using economic models (Parsley and Robinson 1982, Watanatada *et al* 1985) to minimise the sum of construction and road user costs, where this is possible. The need for the provision of climbing lanes is affected by many factors and it is difficult to make a firm recommendation for the use of any of the three standards being considered. Their use will depend on the desired level of service and the availability of funding, both of which can only be determined locally.

7.4.5 Cross-section

All of the standards recommend a lane width of at least 3.5 metres for traffic levels in excess of about 1000 vpd. Climbing lanes and local widening may also be appropriate to improve traffic operations, as discussed earlier.

Both NAASRA and TD 9/81 suggest 1 metre sealed shoulders, whereas AASHTO recommends a minimum width of about 2 metres, but not necessarily sealed. NAASRA consider that sealing is justified because of the saving in edge repairs. Thus, a cost-effective suggestion may be that, for paved roads, a one metre sealed shoulder should be provided and full-width stopping places should be added at intervals along the road. If large amounts of non-motorised traffic are expected on the road, consideration should be given to the provision of wider shoulders to accommodate them.

TD 9/81 gives a fixed crossfall for all road types, whereas both AASHTO and NAASRA give a range of values. For the ranges suggested, there is likely to be little effect on safety or comfort, but the NAASRA values generally give slightly better drainage characteristics and provide a greater margin for error during construction.

8 SUMMARY

The object of geometric design is to provide a basic level of safety and comfort for the road user and to ensure that design is both economic and uniform. New geometric standards have recently been introduced in America, Australia and Britain, and the basis for these has been reviewed. These standards have, in several cases, been based on empirical research.

However, it has been recognised that the behaviour and needs of road users in developing countries are often very different from those in the industrialised countries and, as a result, different standards may be appropriate. There is a need for developing countries to study the philosophy of the standards used in industrialised countries with a view to carrying out research under their own conditions in order to develop their own standards.

Until appropriate standards for use in developing countries have been developed, this Report describes the potential for adapting standards from the industrialised countries.

9 ACKNOWLEDGEMENTS

This study was carried out as a joint project between the Department of Transportation and Highway

Engineering at the University of Birmingham (Head of Department: Dr G Lees) and the Overseas Unit of the Transport and Road Research Laboratory (Head of Unit: Mr J S Yerrell). During the period of the project, Mr Kosasih was a research student at the University. The work forms part of the research programme of the Overseas Development Administration.

We are grateful to the American Association of State Highway and Transportation Officials and to the National Association of Australian State Road Authorities for permission to quote material from their design guides.

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Printed in the United Kingdom for Her Majesty's Stationery Office
(1209/87) Dd8222700 7/87 C10 G426 10170