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TOWARDS APPROPRIATE STANDARDS FOR RURAL ROADS: DISCUSSION DOCUMENT

Prepared on behalf of the SOUTH-AFRICAN ROADS BOARD

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SINOPSIS:

Die padgebruiker in Suid Afrika ervaar tans 'n groot gaping in die diensbaarheid en standaarde wat toegepas word op gruis- en teerpaaie. Weens die hoë koste wat die huidige teerpadstandaarde meebring kan die voorsiening of opgradering van veral lae volume paaie nie maklik geregverdig word nie.

Hierdie dokument dien as 'n voorloper tot die voorsiening van praktiese riglyne vir die keuse van toepaslike en meer bekostigbare standaarde oor die volle spektrum van plattelandse paaie. Bestaande en nuwe gedagterigtings rakende toepaslike standaarde vir verskillende omstandighede word derhalwe bespreek en aanbevelings word gegee.

SYNOPSIS:

The road user in South Africa currently experiences a large gap between gravel and paved roads in terms of the serviceability and standards which are applied. Because of the high costs associated with current paved road standards, the provision and upgrading of low volume roads in particular is difficult to justify.

This document serves as a forerunner to the provision of practical guidelines for the selection of appropriate and more affordable standards over the full spectrum of rural roads. New and existing thoughts on the subject are discussed and recommendations made to this end.

TREFWOORDE:
KEYWORDS: Low volume roads, appropriate standards, socio-economic, geometrics, materials, pavement design, roadside furniture, safety, drainage, construction, maintenance, rehabilitation, environmental aspects.

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DISCLAIMER

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Road authorities

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1 INTRODUCTION

When extending a road network the cost or standard of the facility needs to be matched to the benefits expected of such an extension. However, the spectrum of standards of road facilities seems to be discontinuous because of a perceived gap between gravel roads and high quality paved roads, and the costs and benefits can therefore not always be matched. This mismatching is a problem since paved roads with low traffic volumes do not derive the total road user benefits which they ought for that standard of facility, and gravel roads with high volumes incur excess road user costs far higher than those which would have accrued on a road of a higher standard.

Thus there exists a need for guidelines for providing roads embracing a continuous spectrum of standards, thereby enabling a better matching of a facilities standards to its anticipated traffic volumes in order that return on investment in the road be optimised. Much experience in this field is available for high volume roads, typically through routes, but for shorter roads such as local access routes with lower volumes of traffic, this experience is lacking.

In the 1950's many such roads were built with the sole intention of providing a passable route where none existed. For these roads there was therefore no need for high standards. Over the last decade road authorities and private road owners have embarked on similar projects, and in deference to escalating costs have attempted to apply minimal standards. Many of these roads have performed well, but some have not. There is therefore little doubt that when standards are lowered judiciously, the facility can fulfil its function; the problem is therefore to know how to set appropriate standards without impairing either the acceptability of the road to the road user, or the cost effectiveness of the facility to the road authority or road owner.

This document is envisaged as a forerunner to a set of guidelines for assisting in this task. Its purpose is to assist in the closing of the knowledge gap in the following ways:

i) By stimulating discussion on the following aspects of road standards among pavement engineers and road authorities:
   - Geometric design
   - Materials
   - Pavement design and construction
   - Roadside furniture
   - Safety to the road user
   - Pavement and road maintenance and rehabilitation
   - Environmental concerns

and ultimately

ii) By imparting existing knowledge in a comprehensive format to those road authorities, road owners and pavement engineers who may require it.

It is felt that the roads for which the resulting guidelines will be appropriate will be short local access routes with maximum traffic volumes of 500 equivalent vehicle units (evu's) or approximately 400 vehicles per day.

By providing this information it is hoped that moves can be made towards making the road network in this country more cost effective and in so doing assist the very economy which it is intended to stimulate.

2 SOCIO/ECONOMIC CONSIDERATIONS IN THE SELECTION OF ROAD STANDARDS

2.1 Introduction

The change in emphasis in road funding in South Africa in the 1990s will mean that fewer rural roads will be built and many desirable projects will not be implemented. It is therefore necessary to reassess planning criteria and, in particular, to bring standards for roads into line with funding expectations. Such standards must be based on sound economic principles. They must also take into account special societal needs associated with the purpose or function of the road.

Since it is not possible to deal with these principles explicitly for all types of standards presented in this document, this chapter presents a protocol for assessing projects, with the emphasis on low volume roads. It also provides the framework under which the recommended standards presented in the various chapters in this document were developed.

2.2 Economic evaluation

Generally, projects should reflect an adequate rate of return on investment. In 1992 the opportunity cost of money for roads was very high, as only the most needed roads can be built. Therefore for projects to be considered they should have a rate of return in real terms in excess of 15 per cent. This implies that where roads are not built that may be economic at lower rates, the user will contribute the difference in terms of higher vehicle operating costs.

The standard cost figures and the general methodology for economic evaluation are contained in packages such as CB-Roads and SURF. These packages incorporate benefits from vehicle operating costs, accident costs and time savings obtained from road improvements. Increasingly however, consideration of time savings may not be affordable, and only vehicle operating costs and accident costs would be considered. The approach has been to not take time savings into account explicitly but to specify minimum design speeds for the different classes of roads. Any loss of benefits obtained from this approach will once again have to be borne by the road user. Where substantial time savings are anticipated the sensitivity of this approach can nevertheless be tested, particularly in terms of the effect of alternative standards on total trip time.

When deciding on an appropriate standard, it is usual to compare the economic consequences of a higher standard with that of a lower standard. For the higher standard to be acceptable the marginal internal rate of return of the additional investment must be higher than the accepted opportunity cost of capital (15 per cent in real terms; i.e. with no inflation component).

High marginal rates imply reduced consideration of future costs and benefits. This often causes concern that low standards are built into the network to the long-term disadvantage of future traffic. Traffic growth rates are expected to be generally low in the 1990s. In cases where there is good reason to expect high growth rates the probable need for future upgrading should be taken into account and standards chosen that are compatible with a stage construction philosophy.

It is necessary to be mindful of our existing inability to upgrade those roads which require it at the moment, however, and to note that roads built as part of a phased construction may often have subsequent phases postponed indefinitely.

2.3 Societal considerations

Apart from the economic parameters referred to above, there are other factors that, though not easily quantifiable, need to be taken into account explicitly in standards for certain low volume roads. Examples are:

- passibility (periods in which the road, must be or may not necessarily be passable, i.e. usable);
- situations for which unsurfaced roads are unacceptable;
- minimum safety requirements.

Choices in standards in this regard depend greatly on the function of the road, for example whether the road provides access to work or to educational centres, or whether it serves irregular traffic from farm to market.

2.4 Approach

The above considerations make it difficult to prescribe a methodology for formally determining appropriate standards for all possible situations. Moreover, for roads with lower traffic volumes the cost of formal economic analysis would usually not be justified.

The approach adopted in this document has, therefore, been to analyze standards for various situations taking economic criteria into account as well as addressing the function of the road in terms of the needs of the community.

The results are presented in the form of a series of tables where appropriate standards are suggested for particular situations. However, for the upgrading of roads carrying higher traffic volumes and for all primary and secondary roads formal cost benefit analysis is recommended.

It must be appreciated that although this is a framework for setting standards, there is sometimes insufficient information to determine standards on this basis in a formal way. In many cases therefore in this document standards have been prescribed based on experience but subject to an overall appreciation of the above philosophy. Note also that the choice of geometric standards usually relates to the level of service provided by the facility and has to take into account the needs of the road user.

2.5 Prioritization

This document does not address the issues of project selection and project ranking. However, some general comments are of interest in the context of this document.

In the first place there is a need to consider both the need and the urgency of particular roads within the context of the overall regional development plan. Moreover, the physical development plan must be compatible with the expected funding programme.

In setting up such a regional development plan it is necessary to take into account the needs of the community, where necessary by active community participation. In this process the community may consider trade-offs, for example, between fewer high standard roads or a greater number of lower standard roads.

2.6 Future budget implications

Socio-economic considerations or the satisfaction of such needs, could hold important implications for future budgets. For example, the extensive paving of low volume roads would require recurring maintenance in the form of resealing every 5 to 8 years. This long term scenario should be carefully evaluated as well as the short term issues mentioned above. This is important as the neglect of special maintenance could result in a total collapse of the low volume network.

3 GEOMETRIC DESIGN

3.1 Introduction

The philosophy adopted in the design of a road should embrace all those factors which in any way affect the design of a road. It should determine the environment in which decisions can be taken from a macro level, such as the influence of the political, social and economic realities, to the micro level where specific guidance is provided on engineering issues such as geometric elements. It should provide guidance even where specific directives are not or cannot be prescribed.

It is very important for the designer to consider the likely speed environment in which the road will operate. A long straight road in relatively easy terrain could have a high operating speed, and surprises in the form of sharp curves, or inadequate sight distances at critical points, will create accident black spots. This is undesirable and consideration should be given to reducing the design speed in situations where problems such as sharp curves will occur. A road with long straight alignment sections will lead to higher operating speeds, whereas a road with a curving alignment which follows the contours will lead to a lower speed environment. It is desirable to reduce the design speed incrementally and geometric design should always be done in such a way that the speed environment to be expected is conveyed to the user.

It can be shown through economic analyses that a shorter road with higher standards could often be justified, but such justification often relies on the savings in travel time. In the interest of providing as great a length of appropriate roads for a given amount of funds, the greater adoption of curving alignments, both horizontal and vertical, is recommended. The emphasis should be lower construction costs, even if this would imply a longer and more curving alignment.

The direct application of these design guidelines will not necessarily result in the most appropriate design. This is especially true in rolling terrain where an alignment closely hugging the existing topography may be acceptable for a tertiary road providing basic access, but not for a freeway, although such a design might be the most economical from a construction cost point of view. Again, in undulating terrain, the most economic design might be quite unacceptable from an aesthetic viewpoint. The function and classification of the road both have a bearing on what is considered appropriate. When undertaking a design, especially the vertical and horizontal alignment of the road, harmony must be maintained between the road and the surrounding environment. The importance of the aesthetics of a road is also dependent on its location. For roads traversing ecologically sensitive areas costly measures may have to be taken to minimise the impact on the environment.

The design engineer should ensure that a balanced design is attained and should ascertain that the proposed design can, under prevailing circumstances, considered to be appropriate and acceptable for the present traffic and the expected future traffic.

Finally, it should always be borne in mind that the roads in question have the provision of access as their prime function. The horizontal and vertical alignments of a road should therefore be such that the necessary access is provided.

3.2 Other standards and guidelines

The major geometric standards and guidelines for rural roads are addressed in this document. Guidelines on other geometric elements and procedures can be found in:

- TRH17: **Geometric design of rural roads. (1988)**
- Draft UTG1: **Guidelines for the geometric design of urban arterial roads. (1986)**
- AASHTO: **A policy on geometric design of highways and streets. (1990)**
- The geometric standards of the road authority

3.3 Traffic classes

The traffic volumes a road will carry as well as the composition of the traffic stream are important determinants of geometric elements. In general, the traffic parameter is taken to be the expected average daily traffic, expressed in equivalent vehicle units (evu's) per day, that the road will carry in the design year. Table 3-1 shows the vehicle equivalency factors to be used in determining the traffic class of a given road.

The design year is found by adding the design period of the road to the year during which the road is expected to be opened to traffic. The design period is related to the class of road as shown in Table 3-2.

Table 3-1: VEHICLE EQUIVALENCY FACTORS

GENERAL TERRAIN TYPE	EVU's PER HEAVY VEHICLE
FLAT	3
ROLLING	5
MOUNTAINOUS	10

Table 3-2: DESIGN PERIOD FOR VARIOUS ROAD CLASSES

ROAD CLASS(1)	DESIGN PERIOD
PRIMARY	15 - 30 YEARS
SECONDARY	10 - 20 YEARS
TERTIARY	10 - 15 YEARS

It is recommended that traffic be divided into the six classes shown in Table 3-3.

Table 3-3: TRAFFIC CLASSES

TRAFFIC CLASS	EVUs PER DAY IN DESIGN YEAR
HA	2000 and more
HB	1000 - 1999
HC	500 - 999
LA	150 - 499
LB	60 - 149
LC(2)	1 - 59

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(1) *Primary roads* provide mobility in a national context. Traffic on these roads usually has long travel distances and the design of the roads makes provision for relatively high speeds and minimum interference of through traffic.

Secondary roads provide mobility in a regional context. Shorter travel distances are experienced on these roads and more moderate speeds are consequently acceptable. This group of roads often forms the link between the towns not situated on the primary road network.

Tertiary roads provide local mobility. Their main function is to serve the land uses next to the road network.

(2) Comment by TPA: Branch Roads

Class LC should be subdivided into 1-24 and 25-49

3.4 Design speed

The design speed is one of the main determinants of geometric design. In accordance with modern practice (AASHTO, 1990, page 123) this document assumes that the design speed relates to a high percentile value of the operating speed[(3)]. (Note - This is in contrast with TRH17 (1988) which uses the concept of the running speed, which is less than design speed).

A design speed should be selected for each section of a road taking the following factors into account:

- Traffic volume and composition
- Topography
- Classification of the road
- Roadside environment
  - Pedestrians
  - Accesses to the road
  - Land uses next to the road
- Function of the road
  - Local traffic
  - Through traffic

Care should be taken that the design speed of the various sections of a road is in balance and harmony with the operating environment. Sudden reductions in design speed should be avoided. Reductions of design speed in steps of 15 km/h, suitably signposted, to guide the road user past a particularly low speed element is recommended.

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(3) Comment by TPA: Branch Roads

The major comment on Chapter 3: Geometric Design relates to the philosophy adopted namely where it is proposed that the operating speed approaches the design speed against the existing practice where there is a marked difference between the two speeds with the operating speed being the lower speed. This approach will have a major impact on construction costs especially in undulating or mountainous areas. If at all, this approach should only be applied to primary roads or higher volume roads i.e. more than 2000 evu per day, especially in the current economic climate.

3.5 Width of lanes and shoulders

3.5.1 Paved two-lane roads

Lane widths depend on many factors. The most important of these are the amount of traffic expected to use the road, and the dimensions of the vehicles likely to use the road. An investigation into the economics of geometric standards for paved roads by the Department of Transport (Jordaan, 1992a) has determined traffic thresholds for various lane widths. These thresholds were the main determinant of lane widths for the various cross-sections proposed in this document.

South African research (Jordaan, 1992b) has shown that heavy vehicles use either the edge of the paving or the yellow line as positioning reference. The advantages of a yellow edge line are so numerous that it is recommended that all paved roads carrying more than 600 evu's per day be provided with at least 250 mm additional paved width on either side to allow for the yellow line, as shown in Figure 3-1.

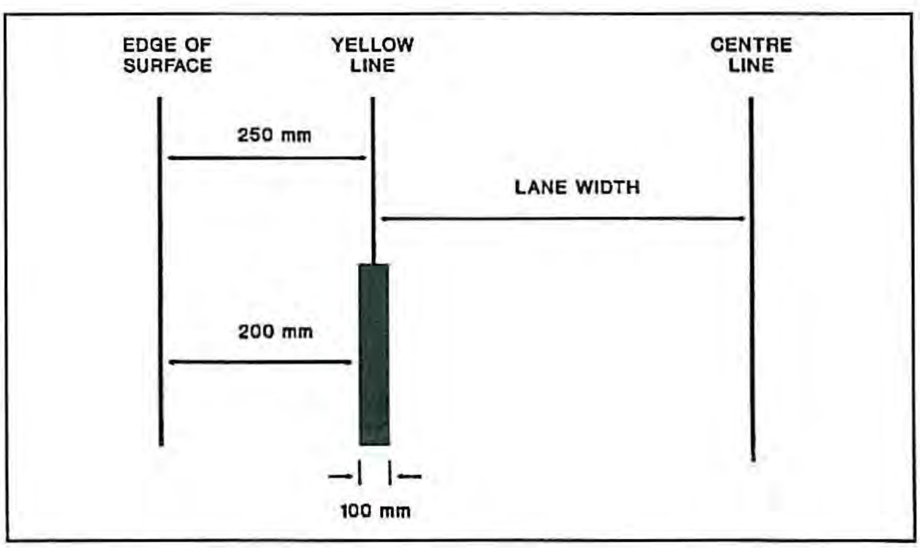

Figure 3-1: POSITION OF THE YELLOW LINE

The 200 mm outside the yellow line should be sufficient for most applications from a geometric perspective. Additional width to control the ingress of moisture into the pavement layers may be desirable and a paved shoulder of 1,0 metres is recommended for moisture control (Emery 1992).

The shoulders of a road provide a number of functions such as:

- improvement of the capacity of road, encouragement of a uniform speed, improvement of road safety and driver comfort
- provision of space for use of stationary vehicles, emergency manoeuvres and for maintenance operations
- lateral structural support, protection and control of the ingress of moisture into the adjacent pavement layers which is most effective with paved shoulders

To provide fully for all these functions is only justifiable at high traffic volumes. As traffic decreases, less and less of these functions can be accommodated.

The Road Traffic Act, 1989 (Act 29 of 1989), requires that a stopped vehicle be at least 1,0 m clear of the traffic lane. At the maximum legal vehicle width of 2,6 m, this requires an area at least 3,6 m wide outside the centre of the yellow line. Wherever possible, such as in easy terrain, or at the transition between cut and fill, a clear area outside the shoulder to accommodate such stopped vehicles is desirable.

Figure 3-2 shows the recommended cross-sections. (A full discussion on these has been prepared (Jordaan, 1992c)). Cross-section types 1, 2, 3 and 4, as shown on Figure 3-2 together with the associated traffic classes, are the recommended road cross-sections for use if the design year traffic exceeds 150 evu's per day[(4)].

Cross-section type 5 is the minimum that should be provided. Narrower paved widths will result in excessive maintenance of the edges of the paving. If the road has a very curving alignment with relatively sharp curves, and carries an appreciable amount of large heavy vehicles and/or busses, cross-section type 5 should be used with care, as the amount of widening required around curves could be extensive, and cross-section type 4 may in the end be more economical.

Sealing of shoulders

The sealing of shoulders improves the accident situation on the road, provides better support and moisture protection for the pavement layers, reduces erosion of the shoulders (especially on steep gradients),

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(4) Comment by CPA: Branch Roads and Traffic Administration

For a design speed greater than 80 km/h the lane width should be 3,7 m. For low design speeds 3,4 m is suitable, which can be reduced for lower design speeds.

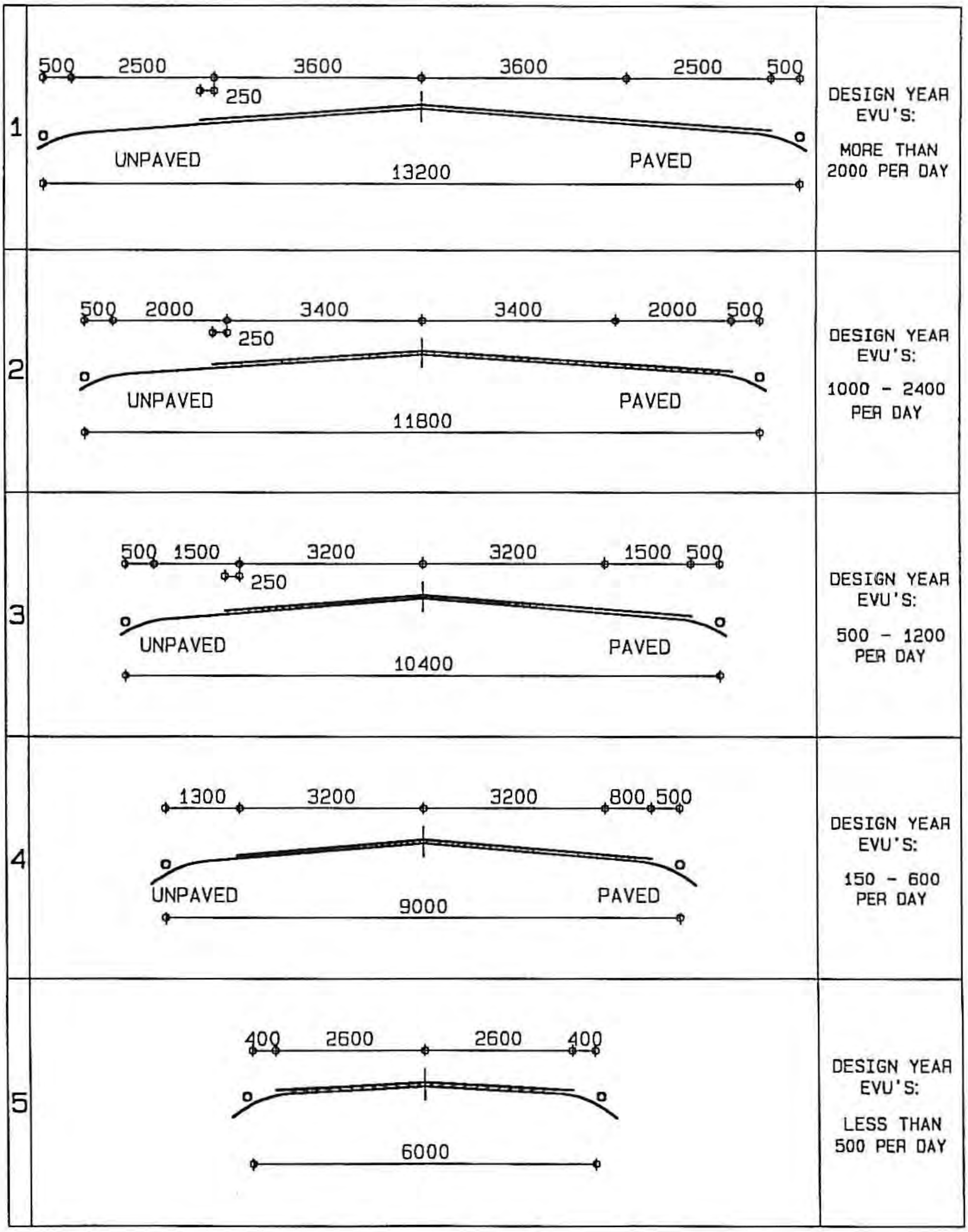


FIGURE 3-2: SUGGESTED CROSS-SECTIONS (continued overleaf)

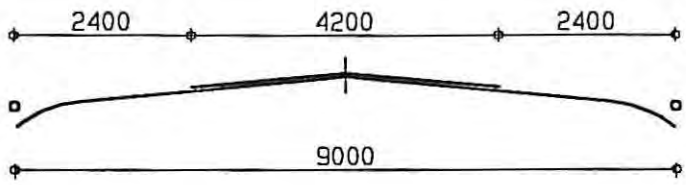
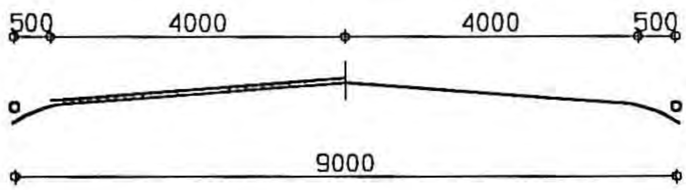
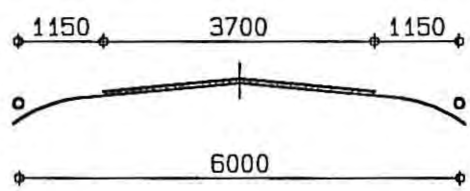
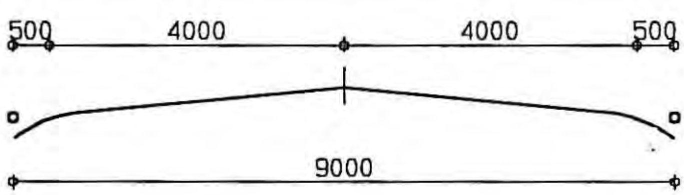
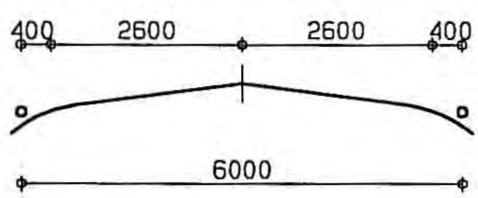
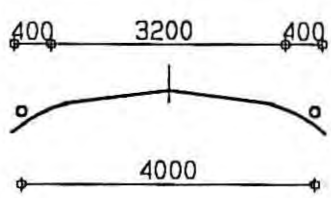
6a		DESIGN YEAR EVU'S: LESS THAN 150 PER DAY
6b		DESIGN YEAR EVU'S: LESS THAN 150 PER DAY
6c		DESIGN YEAR EVU'S: LESS THAN 150 PER DAY
7		DESIGN YEAR EVU'S: MORE THAN 150 PER DAY
8		DESIGN YEAR EVU'S: 50 - 149 PER DAY
9		DESIGN YEAR EVU'S: LESS THAN 60 PER DAY

FIGURE 3-2: SUGGESTED CROSS-SECTIONS (continued)

provides for heavy vehicles to give way to the left to create additional passing opportunities, and also reduces maintenance costs. Shoulders should be sealed if warranted by such special conditions and if it is economically justifiable. If sealed for moisture control, a minimum width of 1,0 metres is recommended (Emery, 1992).

Widening around curves

When heavy vehicles and busses traverse a curve they occupy more space than when travelling on a straight alignment. *In accordance with AASHTO (1990, p217) widening is only applied if more than an additional 0,6 m is required.* The width of widening is then increased in increments of 0,15 m. Table 3-3 shows the widening required on roads having cross-section types 3 or 4 that carries an appreciable amount of large heavy vehicles and/or busses.

Table 3-3: WIDENING REQUIRED FOR A TWO LANE ROAD WITH LANE WIDTHS OF 3,2 m

Radius of curve (m)	Widening required (m) at the shown operating speed			
	50 km/h	60 km/h	80 km/h	100 km/h
1750 and larger	nil	nil	nil	nil
875 -1750	0,30	0,30	0,30	0,45
580 - 875	0,45	0,45	0,45	0,60
435 - 579	0,45	0,45	0,60	0,60
350 - 434	0,45	0,60	0,60	0,90
290 - 349	0,60	0,60	0,75	na
250 - 289	0,60	0,60	0,75	na
220 - 249	0,60	0,75	0,90	na
195 - 219	0,75	0,75	0,90	na
175 - 194	0,75	0,90	na	na
150 - 174	0,75	0,90	na	na
120 - 149	0,90	1,05	na	na
95 - 119	1,05	1,20	na	na
80 - 94	1,20	na	na	na
70 - 79	1,35	na	na	na
less than 70	na	na	na	na

Source: Adapted from Table III-23, p217, AASHTO (1990)

Should widening be required on roads having cross-section type 1 with 3,6 m lane widths, subtract 0,40 m from the values of Table 3-3. Should widening be required on roads having cross-section type 2 with 3,4 m lane widths, subtract 0,20 m from the values of Table 3-3. Should widening be required on roads having cross-section type 5, add 0,60 m to the values for widening given in Table 3-3.

3.5.2 Paved single lane roads

Cross-sections type 6a, b and c provide for a single paved lane. Care should be taken in choosing these particular cross-sections as investigations have shown that in some instances there is only a small cost differential between these and cross-section type 4. The reason for this is that on a single lane road all heavy vehicles travel in the same wheel paths, necessitating a stronger pavement design, which could be as expensive as the wider seal.

The implementation of this type of road would usually be limited to situations where:

- the design year traffic is less than 150 EVU's per day (at this volume vehicles would meet, on average, every two kilometres). (Jordaan, 1993).

- the terrain is open, providing good visibility to the road user

- the maintainability of the shoulders, especially with respect to "drop-offs" can be assured.

Cross-section type 6a as shown on Figure 3-2 is based on the Australian standard. Passing of vehicles takes place with each vehicle moving to the left, leaving only one wheel on the sealed surface.

Cross-section type 6b is an alternative configuration for a single lane paved road. This configuration has the advantage that maintenance is required on one side of the road only. It also requires that traffic on one side have to give way and move completely onto the gravel surface.

Cross-section type 6c is the very minimum that should be provided. Special allowance for the passing of vehicles moving in opposite directions should be made.

3.5.3 Gravel roads

Three recommended cross-sections[5] for gravel roads, types 7, 8 and 9, developed as part of research project 92/271 (Jordaan, 1993), are shown on Figure 3-2 with the associated traffic classes. Type 7 could be upgraded to type 4[6], type 6a or type 6b, by improving the in-situ layers and/or adding more layers and the provision of a bituminous seal. Type 8 could similarly be upgraded to either type 5 or type 6c. Type 9 is a single-lane road and adequate sight distance must be provided. (See par. 3.6.2).

3.6 Sight distance

The sight distance is the length of roadway ahead visible to the driver. This design parameter determines safe stopping and passing distances and usually has a major effect on construction costs and safety. It is often the limiting factor in the design of vertical curves. The sight distance to be provided should always be as great as is practicable and not be less than the distance required for certain selected manoeuvres.

The sight distance experienced on gravel roads under dust conditions could be almost nil, and for gravel roads special care has to be taken to provide adequate sight distance.

Sight distance is considered for three general cases:

- stopping sight distance on two lane roads
- meeting sight distance on single lane roads
- passing sight distance

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(5) Comments by TPA: Branch Roads
- Cross-sections for gravel roads are not practical
- If evu's > 150 formation width = 10 m
- If evu's < 150 formation width = 8 m
- If evu's between 10 and 30 formation width = 6 m
- Cross-section 6c can cause maintenance problems

(6) Comments by CPA: Branch Roads and Traffic Administration
- Gravel roads are seldom built with a later upgrading to surfaced standard in mind
- Generally widths of 8.6 m and 9.8 m would be suitable for divisional and main roads, and widths should not exceed 10 m
- Where topographical constraints are a factor, road widths of about 5 m are not uncommon. 4 m and less should only appear on minor roads

3.6.1 Stopping sight distance on two lane roads

The minimum sight distance available on a roadway should be sufficiently long to enable a vehicle travelling at or near the design speed to stop before reaching a stationary object in its path. Although a greater length is desirable, sight distance at every point along the highway should be at least that required for the driver to stop the vehicle in this distance.

Stopping sight distance is made up of the following:

- distance travelled during perception/reaction time; and
- distance required to stop the vehicle from the moment of brake application.

This is expressed as follows:

$$S = 0{,}278 t_r V + \left(\frac{V^2}{254f}\right)$$

where:	D	=	Distance travelled in stopping (m)
	V	=	speed (km/h)
	f	=	longitudinal friction demand
	t_r	=	reaction time (s) (assumed as 2,50 s.)

Table 3-5 was calculated using these relationships. It should be noted that although the longitudinal friction demand for gravel roads is slightly less than for paved roads, the resulting stopping distances (Jordaan, 1993) are within 15 m of the distances for paved roads. The stopping distance on gravel roads can, therefore, for practical purposes be taken as equal to that of paved roads.

Table 3-5 applies for stopping distances on level roads and on flat vertical curves. Stopping sight distance is affected by gradient and additional sight distance has to be provided. The stopping distance on a grade is given by

$$S = 0{,}278 t_r V + \left(\frac{V^2}{254(f + 0{,}01G)}\right)$$

Where: G = Percent of grade

Table 3-6 shows stopping distances on various downgrades (the critical design situation on two lane roads) for various design speeds.

Table 3-5: MINIMUM STOPPING SIGHT DISTANCES ON TWO LANE LEVEL ROADS

DESIGN SPEED (km/h)	PAVED ROADS		
	f		STOPPING SIGHT DISTANCE (m)
	paved*	gravel**	
30	0,40	0,29	30
40	0,37	0,28	45
50	0,35	0,27	65
60	0,32	0,27	85
70	0,31	0,26	110
80(7)	0,30	0,25	140
100	0,29	na	205
120	0,28	na	285

* TRH17 (1988)

** Jordaan (1993)

Table 3-6: STOPPING DISTANCES ON DOWNGRADES

DESIGN SPEED (km/h)	GRADIENT		
	-3%	-6%	-9%
30	30	30	35
40	45	50	50
50	65	70	75
60	90	95	105
70	120	125	135
80	150	160	175
100	220	240	265
120	310	340	380

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(7) Comment by TPA: Branch Roads

The TPA does not agree with a design speed standard of 80 km/h for the horizontal alignment for a gravel road. The TPA's approach is that a gravel road may in future be upgraded to a surfaced road and to obviate a re-alignment and having to impose again on property owners, the horizontal alignment should if at all possible and cost-effective, have the same as for a surfaced road i.e. 100 km/h.

3.6.2 Meeting sight distance on single lane roads with cross-section type 9

Meeting sight distance is the distance required to enable the drivers of two vehicles travelling in opposite directions at the design speed, on a single-lane road without passing opportunity, to bring their vehicles to a safe stop before colliding. This distance is twice the stopping sight distance and is shown in Table 3-7. In the case of single lane roads with cross-section type 9, the available sight distance used in conjunction with Table 3-7, would determine the speed limit.

Table 3-7: MINIMUM MEETING SIGHT DISTANCE ON SINGLE LANE ROADS

DESIGN SPEED (km/h)	MEETING SIGHT DISTANCE (m)
30	60
40	90
50	130
60	170

3.6.3 Passing sight distance

If passing is to be accomplished with safety, the driver should be able to see a sufficient distance ahead, clear of traffic, to complete the passing manoeuvre without cutting off the passed vehicle, in advance of meeting an opposing vehicle, which appeared during the manoeuvre. Table 3-8 shows the accepted South African standard. (TRH17, 1988, Table 2.5.4)

Table 3-8: PASSING SIGHT DISTANCE ON LEVEL ROADS

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

On gravel roads, the following car is in the dust of the lead car, and the driver of the following car cannot see any appreciable distance ahead. On emergence from the dust during the start of the overtaking manoeuvre, the driver may be confronted with an oncoming vehicle immediately in front of him. On a road having a usable width of 8,0 m, three vehicles could pass with about 0,8 m clearance between them. On roads with a type 7 cross-section, therefore, the passing sight distances of Table 3-8 would be adequate. On gravel roads with cross-section type 8, passing would be extremely dangerous and should, unless the topography provides adequate sighting opportunities, be prohibited. Passing should be prohibited on single lane roads with cross-section type 9. To allow for vehicles meeting one another and having to pass, passing opportunities should be provided where possible, incurring as little as possible additional construction costs.

3.6.4 Shoulder sight distance

At a stop controlled intersection, the driver of the stationary vehicle must be able to see far enough on the major road to be able to enter the major road before an approaching vehicle reaches the intersection. The recommended sight distances and sight triangle are shown in Figure 3-3. The sight triangle must be free of any obstructions.

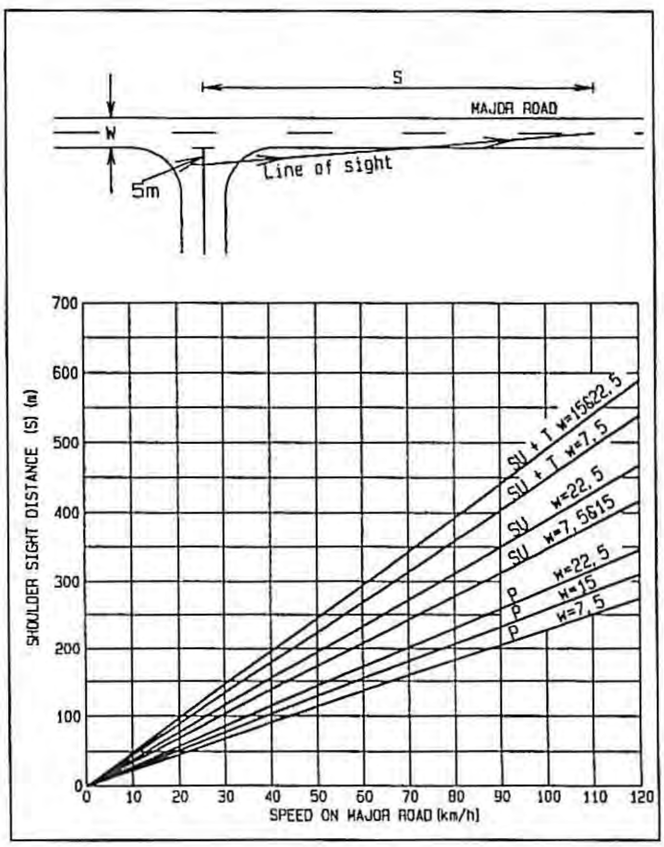

Figure 3-3: SHOULDER SIGHT DISTANCE FOR STOP CONDITION

In the case where the intersection is controlled by a yield sign, the sight distances and sight triangle are shown in Figure 3-4. In this case it is assumed that the driver on the minor road will be driving at 60 km/h and preparing to stop if necessary.

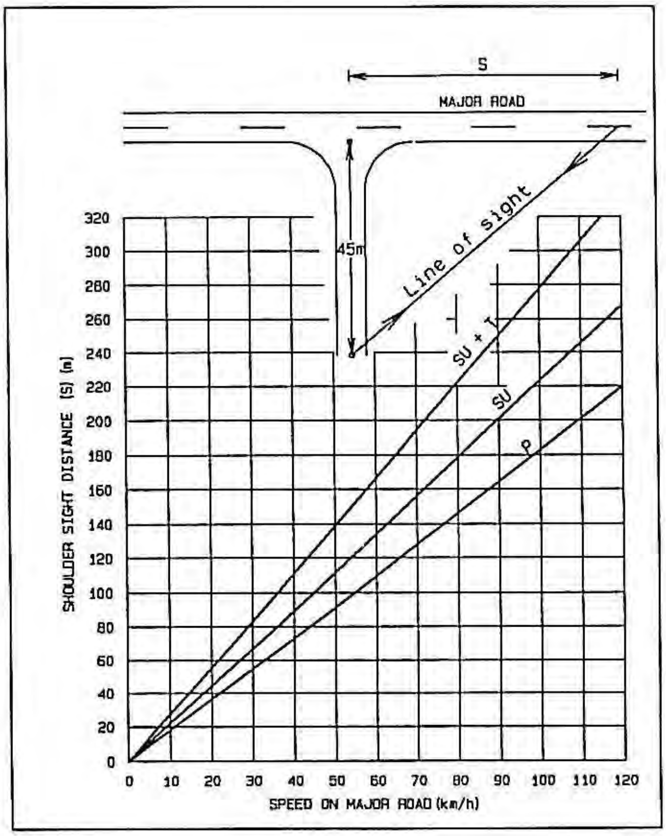

Figure 3-4: SHOULDER SIGHT DISTANCE FOR YIELD CONDITION

3.7 Vertical alignment

3.7.1 Gradients

In general the speeds of light vehicles are not greatly affected by gradient. Heavy vehicles, on the other hand, are very much affected by gradients. Table 3-9 gives general guidelines on maximum gradients.

Table 3-9: ADVISABLE MAXIMUM GRADIENTS

DESIGN SPEED (km/h)	TERRAIN TYPE		
	FLAT	ROLLING	MOUNTAINOUS
30	6%	8%*	10%*
40	6%	8%*	9%*
60	6%	7%	8%*
80	5%	6%	7%
100	4%	5%	7%
120	4%	5%	n.a.

* Care should be taken in using steep grades on gravel roads, as these may become impassable in wet weather.

In selecting gradients for a road with a design speed of 100 km/h and more, speed profiles for light and heavy vehicles should be developed, using the CB-ROADS program (DOT, 1992) and the effect of increasing or decreasing gradients be investigated for economic efficiency, within the aim of providing a least construction cost design. For roads carrying less than 500 evu's per day, significant savings in construction can be achieved by adopting grades steeper than the general guidelines in situations such as:

- comparatively short sections which, if designed at a steeper grade, would provide significant cost savings; gradients as high as 18 - 20 % could be used in such cases,
- difficult terrain where the general grades are not practical ,
- where the absolute numbers of heavy vehicles are low,
- less important roads carrying less than 500 evu's per day; (in such cases grades as steep as 12 % to 15 % could be considered).

3.7.2 Vertical curves

Vertical curves are parabolic curves, and the factor "K" of a curve indicates the horizontal length required for a 1 % change in gradient.

3.7.2.1 Crest vertical curves

It can be shown (Jordaan, 1993) that the most critical sight situation on a crest vertical curve occurs when the vehicle is on the vertical curve and comes to a stop at the end of the vertical curve on a negative gradient. Because the stopping takes place on a negative gradient, the stopping distance is longer and the "K" value higher. Table 3-10 shows the required minimum "K" values for crest vertical curves for various design speeds and gradients. The gradient selected for use in the table should be the greater of the absolute values of the two gradients meeting at the vertical intersection point.

Table 3-10: "K" VALUES FOR CREST VERTICAL CURVES ON SURFACED ROADS

DESIGN SPEED (km/h)	WITH NO ALLOWANCE FOR GRADIENT	GRADIENT AT END OF CURVE				
		-2%	-4%	-6%	-8%	-10%
40	5	5	5	6	6	6
60	18	19	21	22	24	27
80	49	52	56	62	70	79
100	105	113	126	141	160	185
120	203	223	250	284	328	n.a.

3.7.2.2 Sag vertical curves

It has been shown (Jordaan, 1993) that the most critical situation on a sag vertical curve is when the vehicle is at the start of the vertical curve and it sights an object necessitating a stop. Because the stopping takes place on a negative gradient, the stopping distance is longer and the resulting "K" value longer. Table 3-11 shows the required minimum "K" values for sag vertical curves for various design speeds and gradients. The gradient selected for use in the table should be the greater of the absolute values of the two gradients meeting at the vertical intersection point.

Table 3-11: "K" VALUES FOR SAG VERTICAL CURVES ON SURFACED ROADS

DESIGN SPEED (km/h)	WITH NO ALLOWANCE FOR GRADIENT	GRADIENT AT END OF CURVE				
		2%	4%	6%	8%	10%
40	7	7	7	8	8	8
60	17	17	18	19	20	21
80	32	32	33	35	37	40
100	50	50	52	56	59	64
120	73	73	77	82	88	n.a.

3.7.2.3 Vertical curves on single lane surfaced roads

Assuming that vertical curves which provide stopping sight distance at the operating speed are provided, two vehicles will sight each other at 1,53 times the stopping sight distance. This allows sufficient time for speed reduction and manoeuvring to pass each other, and no further allowance for sight distance has to be made.

3.7.2.4 Minimum length of vertical curves

When the algebraic difference between successive grades is less than 0,5 % the vertical curve can be omitted. For aesthetic reasons the lengths of vertical curves should not be less than shown in Table 3-12.

Table 3-12: MINIMUM LENGTHS OF VERTICAL CURVES

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

Source: TRH17 (1988, table 4.2.2)

3.8 Horizontal alignment

3.8.1 Minimum radius

The minimum radius of a circular curve is given by:

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

Where: R = radius (m)

V = design speed (km/h)

e = rate of superelevation m/m

f = side friction demand

3.8.1.1 Paved roads

The side friction demand for paved roads is related to the design speed as follows:

$$f = 0{,}19 - \frac{V}{1600}$$

The maximum rate of superelevation is taken as 0,10 m/m for rural roads. The following table shows the resulting minimum radii for various design speeds.

Table 3-13: MINIMUM RADII OF HORIZONTAL CURVES AT e = 0,10 m/m FOR PAVED ROADS

DESIGN SPEED (km/h)	f	MINIMUM RADIUS (m)
30	0,17	25
40	0,16	50
60	0,15	110
80	0,14	210
100	0,13	350
120	0,12	530

Source: TRH17 (1988)

3.8.1.2 Gravel roads

For gravel roads the maximum superelevation that should be applied is 0,10 m/m. A higher rate of superelevation could cause slow moving heavy vehicles to veer to the inside of the curve and would aggravate the problem of erosion.

The coefficient of sideways friction on a gravel road that drivers are, on average, prepared to accept has been found to be related to the radius, R, of the curve. (Jordaan, 1993). The following relationships were found:

$R < 326$ m: $f = 0{,}16 - 0{,}00034R$
$R > 326$ m: $f = 0{,}05$

Table 3-14 shows the minimum radii for gravel roads if these relationships are employed.

Table 3-14: MINIMUM RADII OF HORIZONTAL CURVES AT e = 0,10 m/m FOR GRAVEL ROADS

DESIGN SPEED (km/h)	Equivalent *f**	MINIMUM RADIUS (m)
30	0,14	30
40	0,13	55
50	0,12	90
60	0,11	135
70	0,09	205
80	0,05	340

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

3.8.2 Minimum length of curve

At small deflection angles ($\alpha°$) the length of the horizontal curve on high speed roads should be long enough to avoid the appearance of a kink in the road. The radius should be determined such that the length of curve, C_m, is as given by:

$C_m \geq 300 - 30\ \alpha°$	(m)	for $\alpha \leq 5°$
$C_m \geq 150$	(m)	for $\alpha > 5°$

3.8.3 Horizontal sight distance on curves

Care has to be taken, especially in cuttings, that sufficient lateral clearance is available to allow stopping sight distance to be attained. Figure 3-5 shows the required lateral clearance given the radius of the curve and the stopping sight distance.

3.8.4 Development of superelevation

3.8.4.1 Rate of superelevation

At radii greater than the minimum for a given design speed a rate of superelevation less than the maximum superelevation is required. Figures 3-6 and 3-7 show the rates of superelevation for given radii for paved and gravel roads respectively.

3.8.4.2 Run-off

The fully superelevated cross-section is developed from the normal cross-section over some distance. Superelevation run-off is the term generally used to denote the length needed to accomplish the change in cross-slope from a fully superelevated section to a section with the adverse camber removed. Crown run-off means the distance from this section to a normally cambered section.

The rate of rotation of the cross-section is measured by the relative slope between the carriageway edge and the axis of rotation. The slope factors given in Table 3-15 have been found in practice to give acceptable lengths of run-off.

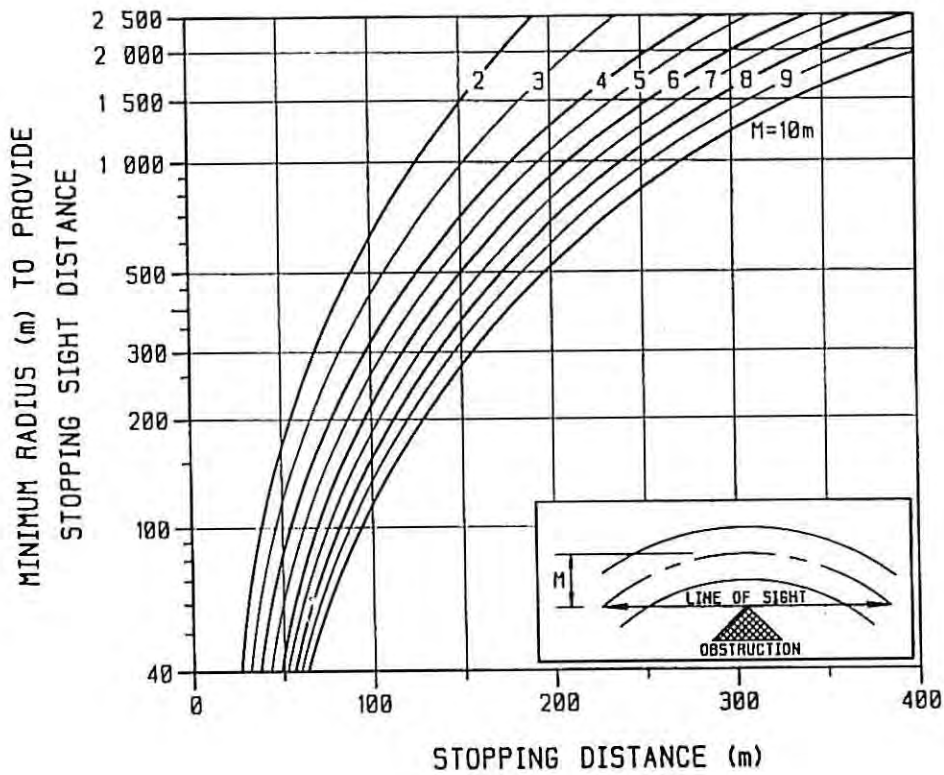

Figure 3-5: LATERAL CLEARANCES ON HORIZONTAL CURVES

Table 3-15: RELATIVE SLOPE FACTOR AND MINIMUM RUN-OFF LENGTH

DESIGN SPEED (km/h)	RELATIVE SLOPE FACTOR	MINIMUM RUN-OFF LENGTH (m)
40	140	30
60	170	40
80	200	50
100	230	60
120	260	70

* Source: TRH17 (1988)

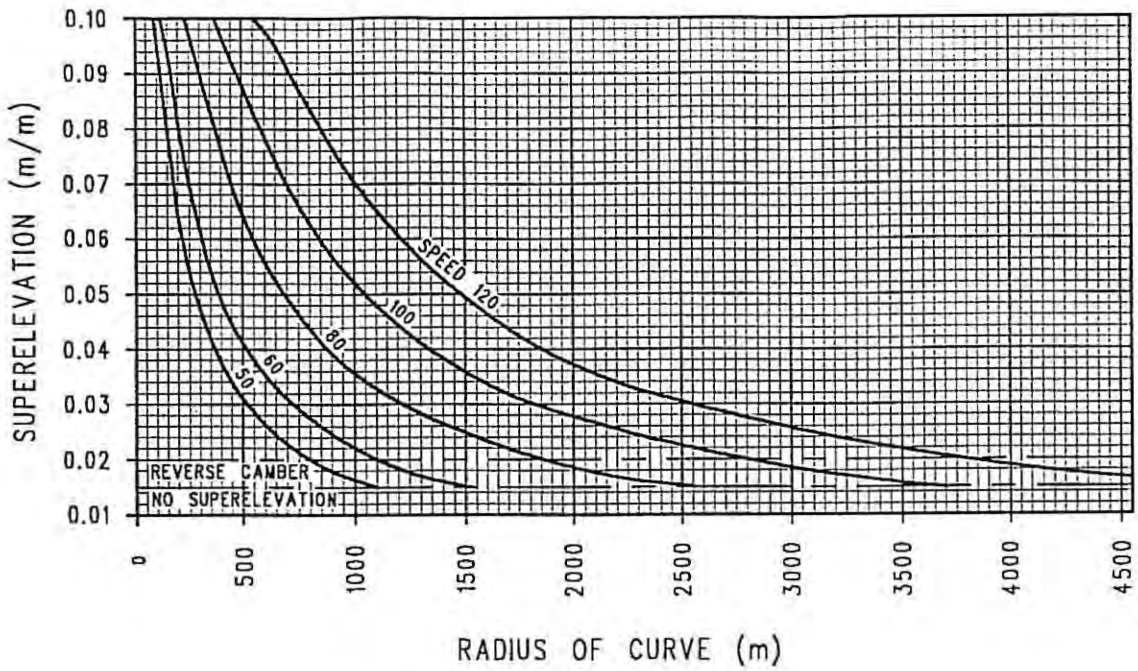

Figure 3-6: SUPERELEVATION RATES FOR PAVED ROADS

Source: Adapted from: Transvaal Provincial Administration: Typical plans for road design

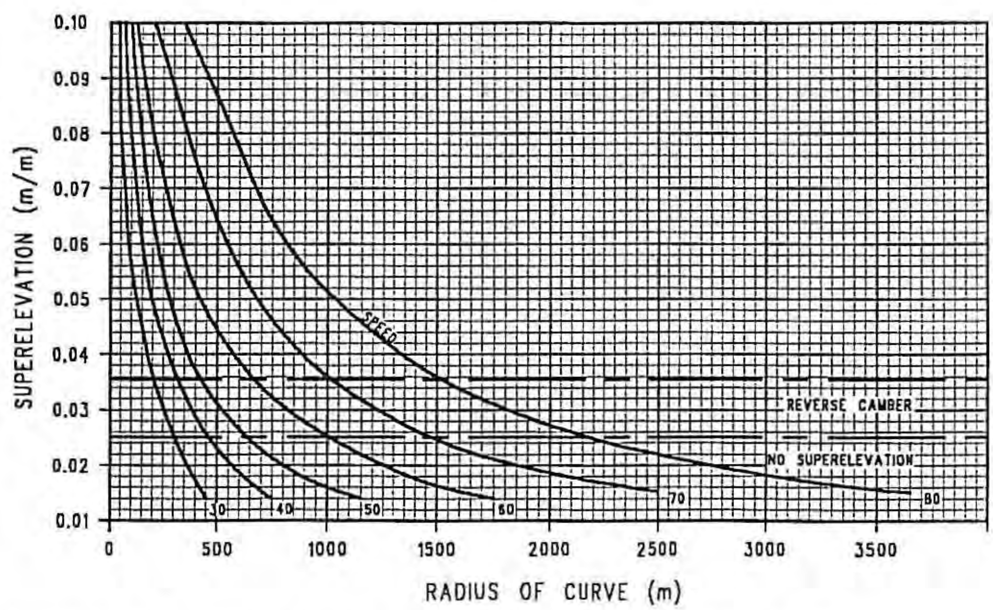

Figure 3-7: SUPERELEVATION RATES FOR GRAVEL ROADS

Source: Adapted from: Transvaal Provincial Administration: Typical plans for road design

The run-off length is calculated as the difference in height between the fully superelevated carriageway edge and the axis of rotation divided by the relative slope between them. The slope factors given in Table 3-16 are the reciprocals of the slopes, and the length of run-off is given by

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

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

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

3.9 Conclusion

The user of these guidelines should always bear the principles of sound geometric design in mind. Sound geometric design cannot be accomplished by merely adding elements together, but rather by applying the appropriate elements in a sensitive manner, taking the needs and abilities of the road user as well as the vehicle into account, in providing a design that provides a safe and comfortable driving task, compatible with the terrain through which the road passes. These guidelines are, therefore, not intended to replace the input of the experienced geometric design engineer, but are provided to supplement these inputs.

3.10 References

AASHTO (1990) *A policy on geometric design of highways and streets.* American Association of State Highway and Transportation Officials, Washington.

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Emery, S.J. *The prediction of moisture content in untreated pavement layers and an application to design in southern Africa.* Bulletin 20, DRTT, CSIR, Pretoria.

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4 MATERIALS

4.1 Introduction

The selection of materials for a pavement structure is based on a combination of structural requirements, availability, economic factors and previous experience. These factors need to be evaluated during the design in order to select the materials that are most appropriate for the prevailing conditions. The selection criteria for materials for low volume roads are essentially similar to those for high volume roads, and TRH 14 (CSRA,1985b) is the basic source document defining these. Materials are classified in here and in TRH14 on the basis of a number of properties into various categories eg G1 to G10.

For use in low volume roads it is often stated that the materials requirements should be relaxed. This is not possible without redefining the material classifications, but it is possible to relax some of the TRH4 (CSRA, 1985a) requirements in terms of the structural capacity of the pavements. The minimum "safe" design of TRH4 is for a road carrying 200 000 E80's in each direction over its design life, a road carrying less traffic than this have been designed using lower class materials and lighter structures with a reasonable degree of confidence. The main reasons for this are:

- design lives are typically shorter;
- the cumulative damaging effect of traffic on the pavement structure is relatively less;
- if the road reaches terminal condition prematurely, a continuation in service may be possible through a tacit relaxation of terminal condition standard;
- a higher level of risk can be accepted.

There has been ongoing development of materials standards in South Africa since the first drafts of TRH 4 and 14 were initially published in 1978 (followed by the updated versions in 1985), and insofar as these relate to low volume roads, they are reflected in this document. Where there is contradiction between this document and TRH 4 and 14, then the guidelines of this document will be considered to be more appropriate.

4.2 Particular requirements for low volume roads

The basecourse materials are a costly and important component of low volume pavement materials, and the most important basecourse material parameter is strength or as is commonly used for simplification, bearing capacity (Paige-Green, 1992a). This leads to the four aspects which must be satisfied with regard to the selection of materials for low volume roads:

- adequate bearing capacity under any individual applied load;
- adequate bearing capacity to resist progressive failure under repeated individual loads;
- the ability to retain that bearing capacity with time (durability); and
- the ability to retain bearing capacity under various environmental influences (which relates to material moisture content and in turn to climate, drainage, and moisture regime).

The control of moisture is the most important goal in ensuring a satisfactory performance, and in this respect it is more important than even the quality of the material (Paige-Green, 1992a). Accordingly where relaxation of material requirements is possible for low volume roads it is on condition that the drainage and moisture regime are suitable.

> *Example: Field investigations into the performance of materials for low volume roads (Paige-Green, 1992a) found sections of road with unstabilised basecourse materials with Plasticity Indices of 3 which had failed badly after 4 400 E80s, whilst others with PI's as high as 17 had performed well after 70 000 E80s. Obviously there were other differences between the two materials, but to have rejected the high PI material on the basis of PI alone would have been wrong. This example illustrates the need to use engineering judgement in selecting materials.*

Many of the standard engineering tests (grading, grading modulus, Atterbergs, laboratory soaked CBR) do not correlate well with the actual performance of the materials in pavements (Paige-Green, 1992a). However, characteristics which may make compaction or finishing difficult eg whether there are any large stones or the plasticity is high, should be considered.

It is recommended that the materials selected for low volume roads should be primarily based on:

- strength
- the strength/moisture/density relationships and
- long term strength (durability) as discussed in Section 4.6.1.

Relaxation of standards should only be done in the light of the relevant maintenance capabilities available in the area. If potholes or cracks occur and are not repaired timeously, water ingress could lead to significant failures. If the local maintenance capability is poor, it is thus recommended that as high a quality material as is locally available should be used. In areas with a very high maintenance capability, relaxations are possible. In areas where the maintenance capability is low, the choice of bituminous surfacing is even more important (as discussed in section 5.7) and in some cases a thicker bituminous surfacing using a modified binder can compensate for deficiencies in pavement materials.

4.3 Risk

The use of relaxed material standards can increase the risk of failure on any project, especially if there should be unexpected drainage deficiencies. It is common engineering practice that any design should have some degree of calculated risk. By increasing the risk of distress through relaxing material requirements it is implied that a slightly larger percentage of the project can be expected to give problems and provision needs to be made for this. It is considered more cost-effective to allow for localised repair or rehabilitation on a long project than to build conservatism into the total length of the project.

It is, however, necessary for political and public acceptance of a philosophy of increased risk of failure due to an infrequent combination of adverse circumstances (Netterberg and Paige-Green, 1988). The risks and consequences of failure need to be identified and controlled. For this, a sound degree of engineering judgement and knowledge of pavement performance and processes is necessary, and the relaxation of materials standards is only being considered here for low volume roads.

The quantification of the increased risk caused by relaxing the standards would be related to the probability that one or more of the layers within the structure was inadequate to carry the design loads. The permutation of different pavement designs, material properties and traffic loadings are such that it is not practicable to provide details in this document. This procedure is, however, fully described by Emery (1992) and can be used on an individual road taking consideration of the specific input variables.

4.4 Subgrade

Although materials are imported into pavement layers to minimise the influence of traffic loads on the subgrade, problems in the subgrade need to be addressed to prevent premature distress on the road. The subgrade material is that occurring naturally beneath the proposed pavement and thus becomes an integral part of the pavement. Southern Africa is in the fortunate position of having particularly good subgrade materials over much of the region thanks to the relatively arid recent geological history. This has, however, had the result that occasional deficiencies in the subgrade are often overlooked.

The cost of the road is integrally linked with the subgrade conditions. The poorer and more problematic the conditions, the greater the cover thickness required to support the design loads. Highly problematic or very weak materials need to be replaced or preferably improved through modification in order to minimise importation of borrow materials. It is for this reason that the layers in existing roads (either paved or unpaved) which have developed strength and density over time should be used as far as possible with minimal disruption of the acquired structure. Problem areas which may have affected these roads in the past tend to have been overcome with time, either by repeated repair or some form of remedial action, such as provision of drainage.

The term "material depth" is used to denote the depth below the finished level of the road to which soil characteristics have a significant effect on pavement behaviour. Below this depth the strength and density of the soils are assumed not to be significantly affected by applied loads. The material depth accepted for low volume roads is typically 800 mm (CSRA, 1985a), which approximates the cover necessary for a G10 material. This depth in fact depends strongly on the relative strengths of the pavement layers and mechanistic analyses of the designs in Chapter 5 show that for low volume roads only 10 per cent of the applied load is effective at a depth of 400 mm.

It is, however, imperative that the subgrade conditions for any proposed road are fully investigated by an experienced person and problem areas are identified and delineated for further investigation or testing. Those subgrade conditions which require particular attention possibly even below material depth are inter alia, soft materials, expansive clays, dispersive soils, collapsible soils and areas of potential drainage problems. Each of these is briefly discussed in the following sections.

4.4.1 Soft soils

Certain materials may be extremely soft in their natural state or become extremely soft on soaking. These occur particularly in vlei and estuarine areas. They are easy to identify either in situ during site inspections or during laboratory testing of their soaked strengths. Materials with a soaked CBR strength of less than 3 can be considered as having low shear strengths and being susceptible to high settlements under loading and special treatment is necessary. This treatment will depend on the pavement structure and design but will typically require the importation of additional layers of selected materials, with or without (preferably) the removal of the weak material, depending on the cross section profile of the pavement.

Where possible pre-loading can allow much of the settlement to occur prior to construction, this being particularly important when structures are involved.

4.4.2 Expansive clays

Soils containing an expansive clay component in adequate quantities (typically montmorillonite or smectite clays but also possibly vermiculites) may potentially result in significant volumetric changes associated with fluctuations in moisture content or stress levels. Expansive materials are most easily identified from their plasticity indices and clay-sized component using the van der Merwe (1975) plasticity chart or the Weston method (Weston, 1980). The potential heave should be calculated taking account of overburden pressure using either of the two methods described above. Should the potential heave exceed about 50 mm precautions are advisable.

One precaution often recommended is to replace the active clay to a depth of 600 mm over the pavement width with a more stable material. This is, however, costly and the acceptability of tolerating surface unevenness of the road should be investigated. If the expected differential movements within the pavement are likely to cause cracking of the surfacing, appropriate action should be taken. This includes the use of modified bitumens in the surfacing for more expansive materials. The use of a geotextile-reinforced surfacing or the possibility of treating the material in-situ with lime or a soil chemical could also be considered.

4.4.3 Dispersive soils

Dispersive soils are typically fine silty clays which contain a high percentage of exchangeable sodium or, less frequently, lithium. These materials have the ability to disperse in a moist environment and the fine particles can then be "washed" out of the soil resulting in tunnelling and the formation of cavities. Dispersive materials are often difficult to positively identify, as they require a range of chemical and physical tests. However, any field evidence of excessive erosion channelling or tunnelling should arouse suspicion and warrant additional testing or specialist advice.

Dispersive soils are difficult to treat requiring that the exchangeable sodium or lithium cations are replaced with calcium ions (from added gypsum typically). Drying out of the material and movement of moisture within the dispersive materials should be minimised. Removal of the material to a depth of 600 mm is another alternative but is costly. Without precautions, dispersive soils will invariably lead to significant distress. Dispersive soils are undesirable as fill materials for roads.

4.4.4 Collapsible soils

Collapsible soils are typically low density sandy materials which may densify under load at high moisture contents. This can result in differential movement within the road structure and general unevenness and loss of riding quality. Collapsible materials are difficult to identify without specialist laboratory testing (eg double oedometer) but an initial indication can be obtained from backfilling an excavation. A negative bulking factor is typical of collapsible soils. DCP penetration of between 55 mm and 90 mm per blow may also be indicative of collapsible soils. Should this indicate a possible collapse potential, specialist assistance should be obtained bearing in mind the risk of deterioration and the level of serviceability required.

The collapse potential of a soil can usually be reduced by high energy impact rolling, high amplitude/

high mass vibratory rolling or ripping and recompacting to an appropriate depth (600 mm recommended). For very lightly trafficked roads, the consequences of differential collapse are often tolerable, resulting in some degree of surface unevenness at worst.

It is suggested that in view of the low stresses prevailing at pavement depths greater than 400 mm (and the minimal load of the added road which is about 80 kPa, it is unlikely that more than about 20 per cent of the collapse measured in, for example, the double oedometer test (load of 200 kPa) will occur.

4.4.5 Poorly drained areas

During the site inspections and centre-line sampling, areas of potential drainage problems and high water tables should be identified. This is best done by experienced personnel who use topography, soil type and vegetation variations to identify these areas.

> *Hint: when assessing an existing sealed road, centreline failures are often indicative of a high water table and edge failures may be indicative of edge drainage problems. This knowledge can be used to identify the source of water and the possible corrective action.*

The pavement should be built up on an appropriate fill in order to minimise the effect of excessive moisture on the pavement. It may be necessary for some soils with high capillary suction potentials (fine sands and silts) to incorporate a drainage layer within the fill to ensure that the upper layers do not become too moist.

A cost-effective solution to potential drainage problems (only in semi-arid to arid areas) is to build the road using standard drainage techniques where appropriate and make allowance to install sub-surface drains where necessary in future.

4.4.6 Other potential problems

Other potential problems which should be noted and which indicate the need for specialist assistance are whether;

- the road is in dolomitic terrain (and thus possibly underlain by sinkholes);
- soluble salt problems could be likely (saline subgrades or compaction water);
- undermining in the past may result in subsidence of the area;
- biological activity (termites, moles, mole rats) may result in localised subsidence.

4.4.7 Roadbed preparation

In order to optimise the pavement design and construction procedure, the subgrade should be prepared by the removal of vegetation, roots and large boulders to as consistent a condition as possible. This will ensure that it will not be affected by any of the problems described above and will have a minimum soaked CBR of 3 over its total length. Areas which do not meet this criterion will need to be improved as discussed above or by the addition of selected material.

In certain cases cost savings can be obtained by eliminating the removal of the root layer. In these cases the roots should be less than about 20 mm in diameter.

4.5 Fill and selected layers

4.5.1 Materials and specifications

The quality of the fill and selected layers (if required) will depend on the pavement design, cross section and the subgrade conditions. The fill discussed in this document comprises the material between the roadbed and the pavement layers which is basically used as a bulk material to raise the height of the pavement to the required level. The material used for fill should have a minimum soaked CBR of 3, which is a G10 or better.

In order to provide the required structural support for the applied traffic, a number of selected layers may be necessary. The quality of these will depend on the traffic and environmental conditions and the requirements are described in Chapter 5. The standard terminology for defining pavement materials used in TRH 4 is utilised in this respect.

4.6 Pavement layers

4.6.1 Paved roads

The structural designs for low volume roads provided in the Catalogue in Chapter 5 are partly based on performance-related studies of lightly trafficked roads constructed to standards less than those in TRH 4 standards). Figure 4-1 defines the material symbols used in the Catalogue. It will be seen that this catalogue generally permits the use of poorer quality materials (ie lower classification) than is required by the catalogue of designs in TRH 4, which is partly because this catalogue applies lower traffic volumes those catered for in TRH 4. **However the materials are still classified according**

to TRH 14 Standards.

4.6.1.1 Untreated materials

The catalogue takes into account the difference in bearing strength between field (unique moisture content and density) and the laboratory conditions (predetermined density and 4 days soaked).

> *Example: A G6 material with a laboratory soaked CBR of 25, will perform well under dry conditions in the field and could have a significantly higher field CBR (35 or higher) depending on the in situ moisture content and density. In developing the catalogue, the field performance was noted, but the material parameters were adjusted from field conditions to the equivalent laboratory soaked conditions, in order to classify the material according to TRH 14.*

For low volume roads, there is some slight latitude permitted in classifying materials according to the TRH 14 classification, which, with the permitted relaxations, are discussed below. Broadly speaking, if a material meets the bearing strength (CBR) requirements but is marginally outside the other requirements then it will still be acceptable. In dry moisture environments, this can be relaxed even further in relation to the ability to maintain adequately.

(a) **Grading**

The particle size distributions of the materials should follow as close as possible the recommended envelopes provided in Table 4-1. These envelopes are based on Fuller-type curves and theoretically result in the maximum densities for the relative maximum sizes defined. This of course assumes that the fraction retained on each sieve size has a constant specific gravity. Certain mine wastes, alluvial and beach sands may have humps on the curves due to high specific gravities in certain size-ranges. Blending of materials may be necessary to improve the gradings in an attempt to get them closer to the envelopes.

SYMBOL	CODE	MATERIAL	ABBREVIATED SPECIFICATIONS
	G1	Graded crushed stone	Dense-graded unweathered crushed stone, max size 37,5 mm 86 - 88 % of apparent density, fines PI < 4
	G2	Graded crushed stone	Dense-graded unweathered crushed stone, max size 37,5 mm 100 - 102 % mod. AASHTO, fines PI < 6
	G3	Graded crushed stone	Dense-graded stone + soil binder, max size 37,5 mm, Minimum 98% mod. AASHTO, fines PI 6
	G4	Natural gravel	CBR $\nless$ 80; PI $\ngtr$ 6
	G5	Natural gravel	CBR $\nless$ 45; PI $\ngtr$ 10, max size 63 mm
	G6	Natural gravel	CBR $\nless$ 25, max. size $\ngtr \frac{2}{3}$ layer thickness
	G7	Gravel - soil	CBR $\nless$ 15; max. size $\ngtr \frac{2}{3}$ layer thickness
	G8	Gravel - soil	CBR $\nless$ 10; at in-situ density
	G9	Gravel - soil	CBR $\nless$ 7; at in-situ density
	G10	Gravel - soil	CBR $\nless$ 3; at in-situ density
	BC	Bitumen hot-mix	Continuously-graded; max. size 26,5 mm
	BS	Bitumen hot-mix	Semi-gap-graded; max. size 37,5 mm
	TC	Tar hot-mix	As for BC (continuously graded)
	TS	Tar hot-mix	As for BS (semi-gap-graded)
	PCC	Portland cement concrete	Mod. rupture $\nless$ 3,8 MPa; max. size $\ngtr$ 75 mm
	C1	Cemented crushed stone or gravel	UCS 6 to 12 MPa at 100% mod. AASHTO; spec. at least G2 before treatment; dense-graded
	C2	Cemented crushed stone or gravel	UCS 3 to 6 MPa at 100 % mod. AASHTO; spec. generally G2 or G4 before treatment; dense-graded
	C3	Cemented natural gravel	UCS 1,5 to 3,0 MPa at 100% mod. AASHTO; max. size 63 mm
	C4	Cemented natural gravel	UCS 0,75 to 1,5 MPa at 100% mod. AASHTO; max. size 63 mm
	AG	Asphalt surfacing	Ref. TRH8[6] gap-graded
	AC	Asphalt surfacing	Ref. TRH8[6] continuously graded
	AS	Asphalt surfacing	Ref. TRH8[6] semi-gap-graded
	AO	Asphalt surfacing	Ref. TRH8[6] open-graded
	S1	Surface treatment	Ref. TRH3[7] single seal
	S2	Surface treatment	Ref. TRH3[7] multiple seal
	S3	Sand seal	Ref. TRH3[7]
	S4	Cape seal	Ref. TRH3[7]
	S5	Slurry	Fine grading
	S6	Slurry	Coarse grading
	S7	Surface renewal	Rejuvenator
	S8	Surface renewal	Diluted emulsion
	WM1	Waterbound macadam	Max size 75 mm, PI of fines $\ngtr$ 6, 88-90% of apparent density
	WM2	Waterbound macadam	Max size 75 mm, PI of fines $\ngtr$ 6, 86-88% of apparent density
	PM	Penetration macadam	Coarse stone + keystone + bitumen or tar
	DR	Dumprock	Ungraded waste rock, max size $\frac{2}{3}$ layer thickness
	CB	Concrete paving blocks	Wet crushing strength $\nless$ 30 MPa interlocking shapes

Figure 4-1: MATERIAL DESCRIPTIONS AND SPECIFICATIONS (CSRA, 1985a)

Table 4-1: **PARTICLE SIZE DISTRIBUTION OF GRADED CRUSHED STONE AND SOIL (G2), (G3), AND NATURAL GRAVEL (G4)**

Sieve size mm	Percentage passing by mass		
	G2 and G3[a] nominal maximum size of aggregate (mm)		G4
	37,5	26,5	
53,0	100	100	100
37,5	100	100	85-100
26,5	84-94	100	-
19,0	71-84	85-95	60-90
13,2	59-75	71-84	-
4,75	36-53	42-60	30-65
2,00	23-40	27-45	20-50
0,425	11-24	13-27	10-30
0,075	4-12	5-12	5-15

Note:(a) G2 and G3 materials: 50 % by mass of individual fractions retained on each standard sieve > 4,75 mm should have at least one fractured face.

Natural gravel (G5 and G6) should have a maximum size of two-thirds of the layer thickness with respective minimum grading moduli (see equation 4.2) of 1,5 and 1,2. Some relaxation is possible here (down to 1,0 provided that the strength requirements are met).

G7 materials should have a maximum size of two-thirds of the layer thickness if used in the subbase. Other than this there are no grading requirements for G7, G8, G9 or G10 materials.

(b) **Atterberg limits**

The Atterberg limits given in Table 4-2 apply to soil fines (<0,425mm) of graded crushed stone and natural gravels (G4 and G5) after modification, if required.

Table 4-2: ATTERBERG LIMITS FOR GRADED CRUSHED STONE (G2, G3) AND NATURAL GRAVEL(G4, G5)

Property	Material type	
	G2, G3 and G4	G5
Liquid Limit (max)	25	30
Plasticity Index (max)	6	10
Linear shrinkage (%) (max)	3	5
Linear shrinkage x (%) passing 0,425mm sieve (max)[a]	170 (G4)	170

Note: a: Only applicable to nodular calcrete (Netterberg, 1971)

In the case of gravel and gravel-soil (G6 and G7) the Plasticity Index (PI) of the material should desirably not exceed 12. In the case of G6 and G7 material with a large coarse fraction, a higher PI of the soil fines may be acceptable. As a guide the maximum PI may be calculated from the following equation:

$$\text{Maximum PI} = 3GM + 10 \quad \ldots \quad 4.1$$

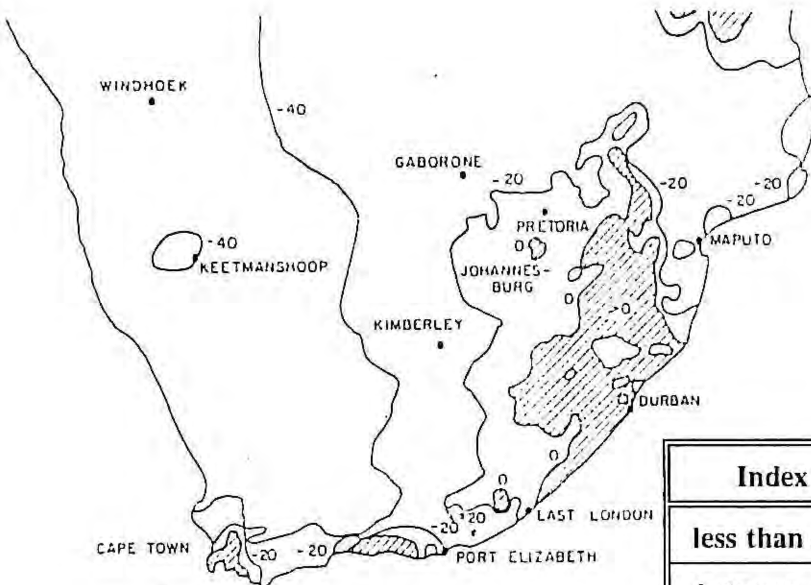
where the grading modulus, GM, is given by

$$GM = \frac{(P_{2,00mm} + P_{0,425mm} + P_{0,075mm})}{100} \quad \ldots \quad 4.2$$

where $P_{2,00mm}$, etc., denote the percentage retained on the indicated sieve size.

Relaxation for low volume roads

Relaxation of Atterberg limits is permitted for low volume roads in drier moisture conditions, provided that the material meets the appropriate bearing strength and durability requirements. No relaxation is permitted in the wet moisture environment unless the soaked CBR exceeds the specified limits by at least 10 per cent, the material has low moisture sensitivity and the pavement is well drained (Figure 4-2 shows the moisture regions of southern Africa; I_m<-20 is dry, I_m > 0 is wet). Relaxation of the PI up to 15 per cent for calcretes (Netterberg, 1971) and ferricretes is permitted.



Index	Classification
less than -40	Arid
between -40 an -20	Semi arid
between -20 and 0	Dry subhumid
greater than 0	Moist subhumid

Figure 4-2: MOISTURE REGIONS OF SOUTHERN AFRICA

(c) **Crushing strength**

The following aggregate strength requirements are recommended for graded crushed stone (G2 and G3) in Table 4-3. The durability should meet the requirements of Section 4.9.

Table 4-3: **AGGREGATE CRUSHING STRENGTH (G2 and G3)**

Property	Specification
10 per cent FACT, or	$\geq$ 110kN[a]
ACV	$\leq$ 29 per cent

Note: (a) Where calcrete is used the minimum 10 per cent FACT is 80kN.

(d) **Flakiness Index (G2)**

The weighted average Flakiness Index determined on the [-26,5mm +19,0mm] and [-19,0mm +13,2mm] fractions should not exceed 35 per cent. However, provided the crushing strength requirements are met and the necessary compaction can be achieved, the Flakiness Index can be allowed to exceed 35 per cent.

(e) **Bearing strength and swell**

Materials G2, G3 and G4 should have a CBR after soaking of not less than 80 per cent at 98 per cent Mod. AASHTO density and a maximum swell of 0,2 per cent at 100 per cent Mod. AASHTO density.

Material G5 should have a CBR after soaking of not less than 45% at 95% Mod.AASHTO density and a maximum swell of 0,5 per cent at 100 per cent Mod. AASHTO density.

Materials G6, G7, G8 and G10 should have the CBR and swell properties given in Table 4-4.

Table 4-4: **CBR AND SWELL REQUIREMENTS FOR GRAVEL AND GRAVEL-SOILS (G6, G7, G8, G9, G10)**

Requirements: soaked CBR test	**Material type**				
	G6	**G7**	**G8**	**G9**	**G10**
Minimum CBR at 93% Mod.AASHTO density (%)	25	15			
Minimum CBR at in-situ density (%)			10	7	3
Maximum swell at 100% Mod.AASHTO density (%)	1,0	1,5	1,5	1,5	1,5

(f) **Group Index**

The Group Index for gravels should be as given in Table 4-5.

Table 4-5: **GROUP INDICES FOR NATURAL GRAVEL AND GRAVEL-SOIL**

Property	**Material type**	
	G5	**G6 and G7**
Maximum Group Index	0	1
Maximum Group Index for calcretes	1	2

(g) **Field compaction**

The normal field compaction requirements for untreated layered materials are given in Table 4-6. It should be noted that the higher the density obtained the stronger the compacted material will be and the lower the potential rut formation due to densification in service.

Table 4-6: NORMAL FIELD COMPACTION REQUIREMENTS FOR UNTREATED LAYERED MATERIALS

Pavement layer	Minimum relative compaction [a]	
Basecourse	Crushed stone (G2) Crushed stone (G3), Natural gravels (G4, G5, G6)	100 % Mod.AASHTO 98% Mod.AASHTO[b] 97% Mod AASHTO
Subbase	Natural gravels (G4, G5, G6), gravel-soil (G7, G8)	95% Mod.AASHTO[b]
Selected layers	Natural gravel or gravel-soil	93% Mod.AASHTO[b]
Subgrade	Natural gravel or gravel-soil	90% Mod.AASHTO

Notes a: Compact at optimum moisture content. In the case of cohesionless single-sized sand, 100 per cent AASHTO density; cohesionless single-sized sand must contain no foreign matter and be closely graded with no cohesion in the dry state.

b: To achieve this compaction requires adequate support from the underlying layers. If it cannot be achieved in the field, then it is possible that the underlying layers have not been compacted properly. Otherwise, the basecourse should be removed, the underlying layer should be dried out to about 75% of OMC to increase its field strength, and the basecourse re-laid.

(h) **Deleterious minerals (G2,G3,G4,G5,G6,G7)**

Sulphide minerals

Crushed rock which is known to contain sulphide minerals (pyrite, marcasite, chalcopyrite, pyrrhotite), should be further investigated for deleterious effects.

Soluble salts

There are a rather large number of salts whose solubility may be deleterious to surfaced roads. Their degree of solubility is not the same, and consequently the level at which they become deleterious changes according to the type of salt. An easily applicable guide is the electrical conductivity of the fines: the conductivity of saturated paste should not be more than 1,5 mS/cm at 25 °C on the dry-screened -6,7 mm fraction, determined according to Method A21T described in TMH1 (D.o.T, 1986).

Mica

This material, especially muscovite, if it occurs freely in quantities which can easily be seen, may have a disrupting effect on a compacted base. Other micas, biotite in particular, are extremely common but not nearly as deleterious and caution should be taken not to confuse the effects. Should any doubt exist about the influence of micas on compaction, laboratory testing should be carried out to investigate whether the required densities can be obtained and whether there is any rebound or whether the CBR is decreased by the mica content.

4.6.1.2 Bound materials

If no suitable materials are available locally for base or subbase layers, stabilisation with lime, cement, lime/slag or any other pozzolannic stabilisers or combinations may be used to improve local materials. Cemented natural gravel (C4) is a selected natural material equivalent to G5 or G6 material with the addition of stabiliser which meets the density and strength requirements given below. (Only C4 materials are discussed in this document as it is considered unnecessary to utilise stronger materials (C3 or better) in low volume roads [See Table 5.11]).

(a) **Maximum size of material (C4)**

The only particle size descriptor necessary is the maximum size. After compaction in place, this should not exceed two-thirds of the compacted thickness of the layer or 63 mm, whichever is the smaller. When used in the subbase only the two thirds requirement is necessary.

(b) **Atterberg limits (C4)**

After treatment the material should have a Plasticity Index not greater than 6.

(c) **Design strength**

In regard to crushing strength requirements for cemented materials, the laboratory design strength of the cemented material should be in accordance with the values given in Table 4-7. This strength should be obtained with not more than 5% by mass of stabiliser at the specified density and at optimum moisture content. In cases where lime is used for stabilisation, the above criteria may be satisfied by accelerated curing of samples prepared in accordance with the prescribed method. The Initial consumption of lime (ICL) must be satisfied.

Table 4-7: DESIGN STRENGTH OF BOUND MATERIALS (C4)

Property	Cemented Material C4	
	Minimum	Maximum[a]
Laboratory design, unconfined compressive strength at 7 days (MPa) 100% Mod.AASHTO density	0,75	1,5
Laboratory design, unconfined compressive strength at 7 days (MPa) 97% Mod.AASHTO density	0,5	1

Note a: These maximum strength requirements are shown only as a guide.

(d) **Field compaction**

The normal field compaction requirements for treated layered materials are given in Table 4-8. When Mod.AASHTO density for cemented material is determined, the specified procedure given in TMH1(DoT, 1986) should be followed.

Table 4-8: NORMAL FIELD REQUIREMENTS FOR CEMENTED LAYER MATERIALS

Pavement layer	Compaction
Base	97% Mod.AASHTO
Subbase	95% Mod.AASHTO

(e) **Durability**

The durability of stabilised materials is critical and is discussed in detail in section 4.9.

4.6.2 Unpaved roads

Unpaved roads may be classified as **earth** or **gravel**. Earth tracks consist of the in-situ material and have not been engineered but have been produced by the passage of vehicles over time. Earth roads on the other hand are typically engineered and consist of some shaping and possibly compaction. They too make use of the in-situ material and their performance thus depends mostly on the natural properties of the local soil.

Gravel roads consist of one or more imported wearing course layers, usually of selected material, shaped and formed into an engineered structure. It is thus possible to control the performance of gravel roads by selecting the optimum material for the situation.

In order to assist with the material selection, performance-related specifications have been developed for southern African conditions (Paige-Green, 1989). Although the material durability in unpaved roads was found not to be important, mudrocks in certain areas may be subject to rapid disintegration and should be investigated by the 5-cycle wet-dry test (Venter, 1989). Other tests such as the Los Angeles Abrasion may be useful as indicators of excessively soft or hard material which may break down under traffic or will not break down under a grid-roller, respectively.

The recommended specifications for materials for unpaved rural roads are given in Table 4-9.

Table 4-9: RECOMMENDED WEARING COURSE MATERIAL SPECIFICATIONS FOR UNPAVED RURAL ROADS

Material property	Specification
Maximum size	37,5mm
Oversize index (I_o)[a]:	≤5 per cent
Shrinkage product (S_p)[b]	100-365 (< 240 preferable)
Grading coefficient (G_c)[c]	16-34
Soaked CBR	≥ 12 at ≥95 per cent Mod. AASHTO compaction

Notes: a I_o = Oversize index (per cent retained on 37,5 mm sieve)
b S_p = Linear shrinkage x per cent passing 0,425 mm sieve
c G_c= (Per cent passing 26,5 mm - per cent passing 2,0 mm) x per cent passing 4,75 mm)/100

NB. All gradings are normalised for a maximum size of 37,5 mm.

The specifications for shrinkage product and grading coefficient are shown schematically in Figure 4-3.

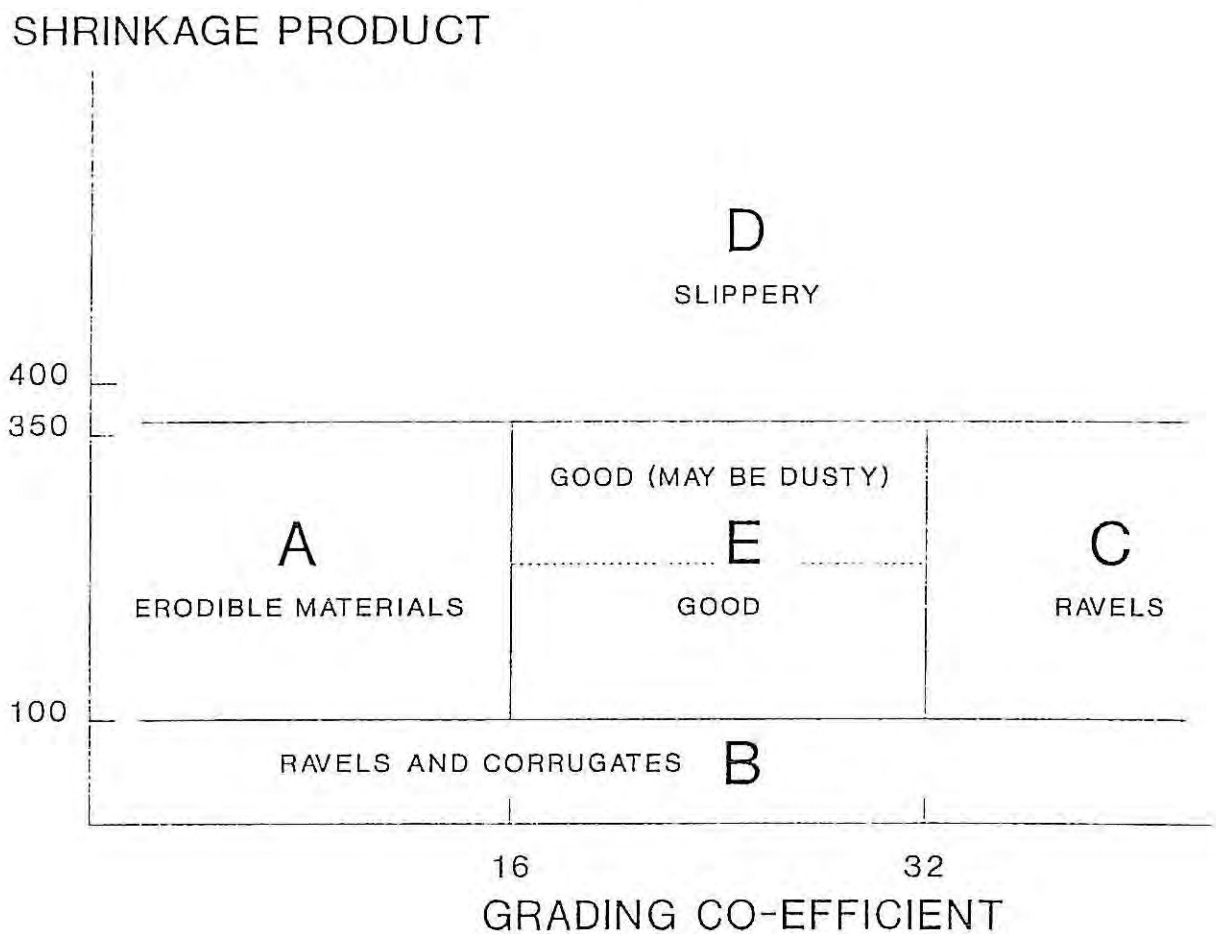


Figure 4-3: RELATIONSHIP BETWEEN SHRINKAGE PRODUCT, GRADING COEFFICIENT AND PERFORMANCE OF UNPAVED WEARING COURSE GRAVELS

The following conclusions can be drawn about each zone as defined in Figure 4-3:

A Materials in this area generally perform satisfactorily but are finely graded and particularly prone to erosion by water: they should be avoided if possible, especially on steep grades and sections with steep cambers and superelevations. Most roads constructed from these materials perform satisfactorily but may require periodic labour-intensive maintenance over short lengths and have high gravel losses due to water erosion.

B These materials generally lack cohesion and are highly susceptible to the formation of loose material (ravelling) and corrugations. Regular maintenance is necessary if these materials are used and the roughness is to be restricted to reasonable levels.

C Materials in this zone generally comprise fine, gap-graded gravels lacking adequate cohesion, resulting in ravelling and the production of loose material.

D Materials with a shrinkage product in excess of 365 tend to be slippery when wet.

E Materials in this zone perform well in general, provided the oversize material is restricted to the recommended limits.

The recommended specification (Zone E) permits a number of materials which are likely to be unacceptably dusty, but many materials which perform well would be eliminated by lowering the shrinkage product to 240. This was considered unnecessarily harsh for rural roads. Attempts should be made, however, to locate materials with a shrinkage product of less than 240 as far as possible.

By plotting the shrinkage and grading properties of a potential unpaved wearing course gravel on Figure 4-3, an indication of the suitability and any potential problems will be obtained. However, engineering judgement should also be used. In flat, dry areas, materials falling into zones A and D may be acceptable if the site-specific potential to erode or become slippery is not excessive. Similarly if there is low traffic and a high maintenance capability, the use of materials prone to corrugation may be acceptable.

4.7 Surface stabilisers

As the first step towards improvement for gravel roads, surface stabilisers can be considered. These typically involve the treatment of in situ or gravel wearing course materials with various proprietary products in order to reduce dust, lower the erosion potential, reduce gravel loss and deterioration rates or generally enhance the performance of the road. As a penultimate stage before full surface treatment, there are a number of bituminous dust palliatives and surface sealers. Discussion of the former products follows whilst the latter are discussed in Section 4.8 under surfacings. The use of surface stabilisers is only justified in some situations, and each product should be evaluated on a total life-cycle cost basis for each material in order to warrant its use on an economic basis.

The products can be divided into a number of generic groups:

- Sulphonated Petroleum Products (variously marketed as compaction aids, ionic stabilisers, soil modifiers and/or soil chemicals and others);
- Deliquescent materials (such as calcium chloride)
- Ligno sulphonates;
- Others.

4.7.1 Sulphonated Petroleum Products

These materials are surfactants and can generally be classified as sulphonated petroleum products consisting of a hydrophobic (water-repelling) hydrocarbon chain which has been sulphonated. This results in a strongly acidic product which can participate in a cation exchange reaction with clay soils and consequently modify their electro-chemical properties and render them water resistant. The surfactant (or detergent) properties also make them a good lubricant and potentially useful compaction aids. A number of these products are available locally.

Recent work has shown that the performance of these products is highly material dependent (Paige-Green, 1992b). Certain materials will not react with them whilst other materials react to different degrees with different products. It is thus imperative that all materials are tested for their potential reactivity with the various products in order to ensure that the product is not being wasted and that the best product is used for any material. This testing requires that the cation exchange capacity is evaluated both before and after treatment with the products, and the changes in the material are evaluated in terms of the pH characteristics of the untreated material.

In order for a cation exchange reaction to occur with the products it is necessary that a suitable clay component is present in the material. This component should be a 2:1 clay (eg smectite, vermiculite, chlorite) and should be present in sufficient quantities to react adequately. As an interim measure it is recommended that the materials meet the criteria of Table 4-10 and Figure 4.4.

Table 4-10: REQUIREMENTS FOR PAVEMENT MATERIALS SUITABLE FOR TREATMENT BY IONIC SOIL STABILISERS

Pavement material property	Minimum value	
Plasticity Index	$\geq 8^a$	
Percent passing 0,075 mm sieve	≥ 17	
Initial Cation Exchange Capacity[b]	$\geq$15 me/100mg	
Reduction in Cation Exchange Capacity after treatment (Figure 4-4)	at 15 me/100mg	$\geq$ 66%
	at 50 me/100mg	$\geq$ 40%
Bearing strength	no standard has been set[c]	

Notes a: Some materials, however, with plasticities as high as 16 or 20 will not react as the clay mineral type is kaolin or illite which has an inherently low cation exchange capacity.

b: Determined at a pH of 8,2 after removal of organic matter

c: There is not enough evidence yet to quantify accurately the potential increase in strength caused by the cation exchange reaction. However, based on limited available data it would appear that the material increases in quality by at least one TRH4 classification group (ie G10 to G9).

Cation Exchange Capacity testing can be performed only at specialist soils laboratories. It should be noted that Si and Al can be either cations or anions depending on the pH values. This sometimes results in spurious results from the cation exchange capacity testing after treatment with the acid products.

Many cases have been reported where the in situ strengths of non-plastic or very low plasticity materials treated with sulphonated petroleum products (SPPs) are very high. In these cases the products appear to work as a compaction aid resulting in high densities and thus high capillary suctions on desiccation. It is suggested that the particle size distributions relative to standard Fuller-type curves may be an indication of the propensity for successful use of the products as a compaction aid. This can be investigated in the laboratory by evaluating the effects of the products in comparative compaction testing.

SPP treated materials may be used in both paved and unpaved roads. For unpaved roads the material should initially comply as far as possible with the recommendations provided in Section 4.6.2. In areas where good wearing course gravels are scarce it is often prudent to apply a bituminous surfacing to SPP treated wearing courses in order to conserve the gravel. In paved roads they may be used as any layer appropriate to their initial properties and classification and the assumed strength increase eg G7 to G6.

4.7.2 Deliquescent materials

The deliquescent materials (mostly calcium and magnesium chloride) have the property of being able to absorb moisture from the atmosphere and thus exist in a slightly moist form. This allows the soil particles which have been treated (particularly the silt and clay which form dust) to adhere to each other resulting in an essentially dust free road under traffic. The products can be mixed-in or sprayed-on to the wearing course material.

These products will generally not improve a material which is unsuitable for wearing course to the extent that it will become suitable. It is thus recommended that the standard specification for wearing course materials (Section 4.6.2) is adhered to and the product is used only to palliate dust. This is especially useful in agricultural areas, adjacent to hospitals and schools and where settlements are adjacent to particularly dusty roads.

Caution should be exercised when treating materials with high shrinkage products. Soaking results in their becoming extremely slippery (both treated and untreated) but the treated materials take considerably longer to dry out resulting in poor performance for a longer period.

Deliquescent materials are highly soluble in water and it is thus important to ensure that the surface drainage is well designed and rainwater is removed as rapidly as possible. When the product is mixed into the layer capillary action results in replenishment of the product leached out of the upper portion of the road. However, in the longer term, periodic rejuvenation is necessary. No deleterious environmental effect has been recorded from these products.

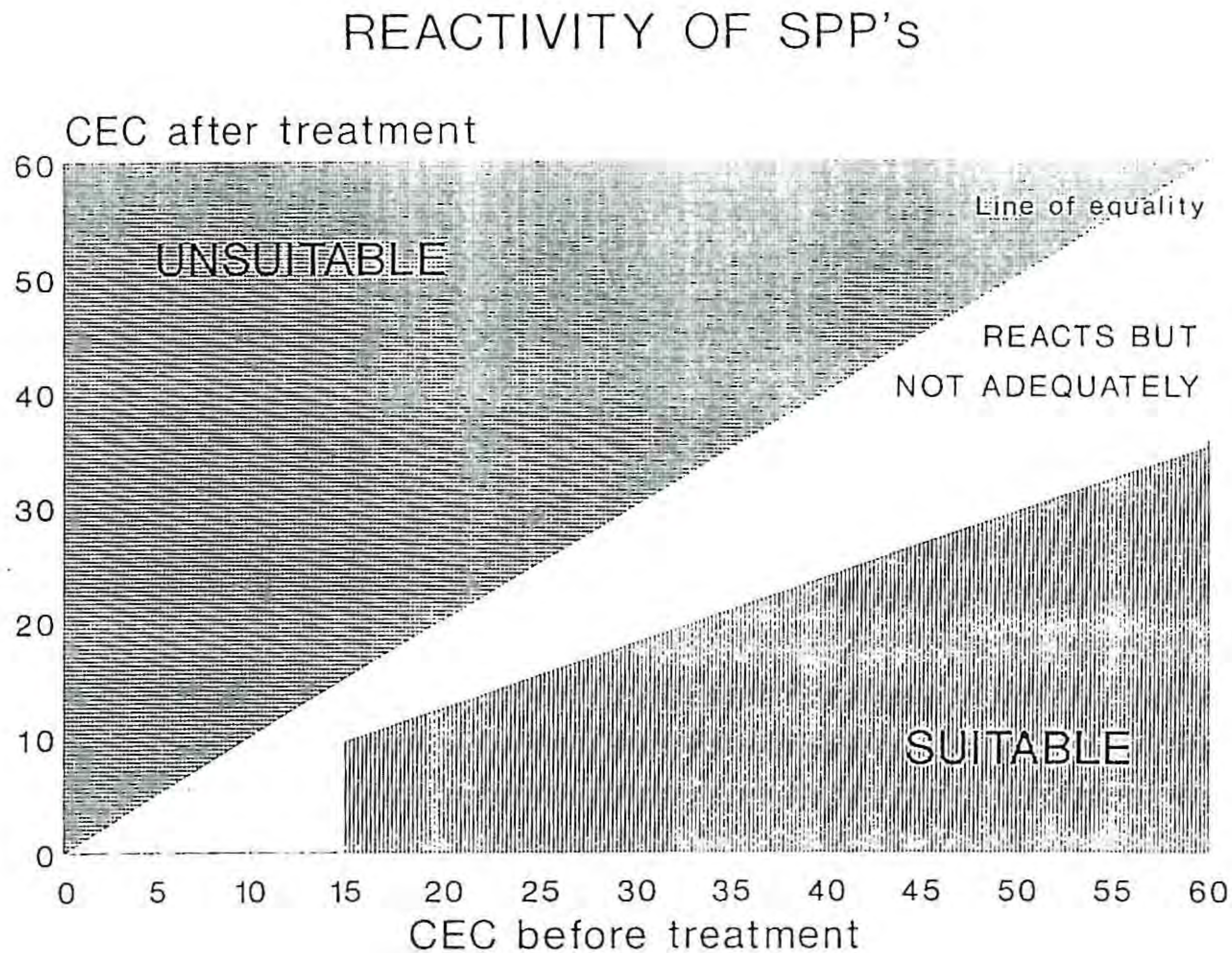


Figure 4-4: REACTIVITY OF SULPHONATED PETROLEUM PRODUCTS

4.7.3 Lignosulphonates

Lignosulphonates are a by-product of the bi-sulphite process used in pulp production. The material is a glutinous residue of the dissolved cellulose and has the property of gluing soil particles together and reducing the dust from treated unpaved roads. The products have been shown to be highly effective in reducing dust and may be mixed into the surface layer or sprayed-on, depending on the requirements and costs.

Like the deliquescent materials, these products are highly soluble in water and need periodic rejuvenation, usually at the beginning of the dry season. When treated roads are wetted, there can be problems with slipperiness. No environmentally unacceptable consequences have been reported from these materials with respect to the concentrations used in roads.

4.7.4 Others

A number of other products are available such as consisting of organic polymers (PVC and PVA), latexes, tannic acids, molasses and various other inorganic and organic products are available. Little research has been carried out on these products and specifications for their use cannot be recommended until more information is available. These products would need to be evaluated in controlled experimental sections for their use be found to be economical in terms of total life cycle cost analyses.

4.8 Surface seals

4.8.1 Introduction

Materials used for surface seals consist mainly of a bituminous (or tar) binder and aggregate (sand and/or crushed stone). The choice of surfacing type is discussed in Chapter 5 on pavement design.

4.8.2 Prime

A prime is recommended for all roads which are to have a bitumen surface, unless good quality control and maintenance capabilities exist. It protects the pavement materials against rain during construction, and, depending on the type of prime, assists in reducing carbonation of bound materials. For roads with thin bituminous surfacings (single seal and sand seal), intermittent nozzle blockages can cause areas of underspray, and the prime will compensate for this and make a significant improvement in the performance of the surfacing.

The adhesion of the surfacing to the base on weak and fine grained base materials can also be a problem. Many of the finer grained materials have layers of loose material beneath the surfacing along which the surfacing can slide and move. A non-viscous deep-penetrating prime (cutback bitumen, tar, quickdrying tar) can be used on finer basecourse materials to strengthen the upper portion of the base. Rolling 19 to 26mm chips into the top of weak bases (armouring) can assist with a good seal to base bond

4.8.3 Bitumen

Conventional bitumen

Conventional bitumen, either as road (penetration) grade, cutback or emulsion, is the most common binder used on low volume roads. Specifications for conventional binders have been developed in South Africa over many years in order to ensure good surfacing performance. These specifications are well described in CSRA, as SABS specifications and in TRH 3.

Modified bitumen

The use of modified binders in South Africa is occurring with greater frequency as more knowledge is gained about the improved properties and road performance of these products (Davidson, 1991). Specifications for modified binders are under development (Marais, 1991). The major modifiers used in bitumens in South Africa are the following:

Crumb rubber
Styrene-butadene rubber latex (SBR)
Styrene-butadene-Styrene (SBS)
Natural rubber latex
Ethylene vinyl acetate (EVA)
Styrene ester

Crumb rubber and SBR are at this stage the most commonly used modifiers and are accepted by road authorities as being more cost effective than the conventional binders for many applications. During a study on the field performance of bitumen-rubber reseals, evidence was found of extended life in excess of 70 per cent over that afforded by conventional seals on rapidly deteriorating pavements. (Renshaw et al. 1991)

Dust palliatives

Bituminous dust palliatives are amongst least expensive bitumen surfacings. They typically consist of a cutback bitumen applied to the surface of a base course with a sand cover, which acts as both a prime and a surfacing. They can be effective as a short term surfacing. However the maintenance costs rise steeply with time, and they require timeous retreatment or resealing to be cost-effective. On temporary works such as deviations, or as a form of stage construction these materials may, however, have significant benefits.

4.8.4 Aggregate for bituminous surfacings

The standards for aggregates and sands to be used in surfacing seals and slurries for high traffic volumes are well known (SABS 1083, 1976; CSRA, 1980, 1988, 1985b). TRH 3 specifies that the quality of aggregates shall conform to the recommendations given in Section 2 of TRH 14 for grading, crushing strength, Flakiness Index, polished stone value, fines and dust content, adhesion and sand equivalent values. The aggregate for surface treatments (S1 and S2), sand seals (S3), Cape Seals (S4) and rolled-on chippings consists of single-sized stone, natural sand and/or crusher sand. The quality required for these materials is of necessity high since the aggregate is exposed to severe handling, environmental and in-service conditions, especially during rolling before it is finally embedded in the bituminous binder.

For low volume roads, there is limited potential for the use of marginal surfacing materials in order to reduce the cost of bituminous surfacings. The breakdown of surfacing cost components (Wright et al., 1990) shows that aggregate cost makes up about 20% of the total surfacing cost. In South Africa, by using marginal materials, this cost could perhaps have been cut by a third, which is only 7% of the total cost of the surfacing. This reduction in cost should be weighed against the risk of premature failure. Limited relaxation of specifications from the high traffic volume specifications is given here. No relaxation in materials specifications for sands for asphalt is proposed, except that new specifications for sands for sand-asphalt are being researched at present.

The selection of surfacing aggregate for low volume roads shall be done by considering all of grading, crushing strength, polished stone value, fines and sand equivalent, and bitumen/stone adhesion.

<u>Grading</u>
The recommended grading envelopes for the various nominal sizes of crushed stone for surface treatment (S1 and S2) and Cape Seal (S4) (first layer of stone only) and rolled-on chippings are unchanged from TRH 14 and are given in Table 4-11.

Table 4-11: GRADING OF SINGLE SIZED CRUSHED STONE

Sieve size (mm)	Nominal size (mm) (percentage passing by mass)				
	19,0	16,0	13,2	9,5	6,7
26,5	100				
19,0	85-100	100	100		
16,0	n.s.	85-100	n.s.		
13,2	0-30	0-30	85-100	100	
9,5	0-5	0-5	0-30	85-100	100
6,7			0-5	0-30	85-100
4,75				0-5	0-30
2,36					0-5

The grading of sand for sand seals or stone/sand double seals can be relaxed and the relaxed grading is presented in Table 4-12 (from Paige-Green and Savage, 1990).

Table 4-12: GRADING OF NATURAL SANDS

Sieve size (mm)	Percentage passing by mass		
	Ideal	**Average**	**Fine**
9,5	100	100	
4,75	85-100	85-100	100
2,36	n.s.	n.s.	85-100
1,18	25-50	50-85	n.s.
0,600	0-20	20-50	50-80
0,300	0-5	0-10	n.s.
0,075	0-2	0-2	0-2

The suggested grading of crusher dust for dust palliatives is given in Table 4-13 (from Paige-Green and Savage, 1990).

Table 4-13: GRADING OF CRUSHER DUST FOR DUST PALLIATIVES

Sieve size (mm)	Percentage passing by mass	
	10mm nominal	7mm nominal
13,2	100	
9,5	85-100	100
6,7	0-40	85-100
4,75	0-10	n.s.
3,35	n.s.	0-30
2,36	0-1	0-10
0,600	n.s.	0-2

The grading of aggregate for slurries should conform to Table 4-14 (from Petrocol, undated) although there are other specifications which may also be suitable (CSRA, 1988). The recommendations for slurries have not been relaxed from those given by the bitumen industry. Whilst the design of slurries with different gradings is possible, the performance of new gradings in the long term is not known. The existing design procedures are based on materials adhering to the specifications and which have proven good performance. It is therefore not recommended that specifications in this regard are relaxed without proper testing on the available materials.

Table 4-14 GRADING OF SLURRY AGGREGATES

Sieve size (mm)	Percentage passing by mass		
	Fine	Medium	Coarse
9,5			100
6,7		100	n.s.
4,75		85-100	70-90
2,36	100	65-90	45-70
1,18	65-90	45-70	28-50
0,600	40-60	30-50	19-34
0,300	25-42	18-30	15-25
0,150	15-30	10-21	7-18
0,075	10-20	5-15	5-15

The suggested grading of aggregate for graded sand (Otta) seals is given in Table 4-15 (from Botswana Roads Department, 1990).

Table 4-15: GRADING FOR OTTA SEAL

Sieve size	Percent Passing by mass		
(mm)	Coarse	Fine	Wide
19	100	100	100
16	85-100	85-100	85-100
13,2	60-80	80-100	60-100
9,5	36-56	56-96	36-96
6,7	20-40	40-80	20-80
4,75	10-30	30-70	10-70
2,36	2-16	16-50	2-50
1,18	0-10	10-38	0-38
0,425	0-5	5-25	0-25
0,075	0-2	2-10	0-10

Notes: Coarse grading < 100 vpd; fine grading > 100 vpd; Grading curve should fall smoothly within envelope.

Crushing Strength

The crushing strength requirement of TRH 14 can be relaxed in the light of the adequate performance of lower crushing strength materials on low volume roads, provided that construction rolling with a steel wheeled roller is restricted. The experiences of other authorities are shown in Table 4-16 (from Netterberg and Paige-Green, 1988).

Table 4-16: CRUSHING STRENGTH

Source	10% FACT (min)	ACV (max)
TRH 4	210	21
Zimbabwe	120[ab]	30
Australia	80[ab]	30
Otta seal-sand	n.s.	40[c]

Notes a: limited application

b: soaked must be > 75% of dry value, do not use steel wheeled roller in construction

c: demonstrated in Botswana with traffic less than 100 vpd.

The absolute minimum value for crushing strength should be 120 kN (ACV 30) provided only pneumatic tyred rollers are used during construction, and if possible a higher value should be used. This may need to be revised upwards as the new super-single tyres with pressures up to 1000 kPa are introduced. The lower limits for 10% FACT may result in crushing under the high contact pressures.

Polished stone value

Polishing of stone in the surfacing lowers the skid resistance of the surfacing under wet conditions and is of particular importance when the texture depth of the surfacing is shallow. The rate of polishing, apart from the stone properties, is mainly dependent on the traffic volume. Thus for volumes less than 500 vpd polishing of stone would take considerably longer and be significantly less for traffic volumes of 5 000 vpd. For low volume roads, polishing is rarely a problem and the PSV requirement can therefore be dropped for stone used for single and multiple seals and Cape Seals.

Fines and sand equivalent

Although there should be no material smaller than a specified size according to the grading, the presence of fines and dust cannot be prevented entirely, but should be limited as specified in TRH 14. No relaxation is proposed because of its influence on the bitumen adhesion and the importance of quality in constructing these seals (Table 4-17).

Table 4-17: PERMISSIBLE FINES AND DUST CONTENTS FOR STONE FOR SURFACING (CSRA, 1985b)

Nominal size (mm)	Fines (% dust)			
	Fines content (passing 0,425mm sieve)		Dust content (passing 0.075mm sieve)	
	Stone Grade N	Grade S	Stone Grade N	Grade S
19,0 to 9,5	0.5	2,0	-	1,5
6,7	0,5	3,0	-	1,5

To limit the degree of contamination of the sand by clayey material, the sand equivalent value should be not less than that specified in Table 4-18

Table 4-18: SAND EQUIVALENT VALUE FOR SURFACING STONE

Application	Sand equivalent (min)
Sand seal	30
Otta seal	25
Slurry seal	30
Dust palliative	30

Stone-bitumen adhesion

Not all types of rock adhere equally well to bitumen and in particular "acid" rocks can be a problem. Although there is experience of the different adhesive properties of "acid" and "basic" rocks, it is safer to be guided by the surface texture of the crushed stone. However, when tar or cationic bitumen emulsions are used, such problems do not usually occur.

The crushing of medium to fine grained rocks produces a rough textured face which usually resists stripping of the binder. Such rocks are almost all carbonate rocks and calcrete, many acid or basic crystalline rocks, tillites, and any arenaceous rocks which posses a strong siliceous cementing matrix.

Coarse grained rocks possess numerous flat and smooth faces of large minerals, especially feldspars, and are noticeably more inclined to strip than the medium to fine grained types. Such stone is produced in particular from certain acid crystalline rocks, eg granites and gneisses.

Loss of stone is generally less of a problem on low volume roads than on high volume roads, but it is still a problem at intersections. Because of the effect of stripping on surfacing performance, no relaxation of the limits in TRH 14 is proposed. The stone-bitumen adhesion should be tested according to Method B11 of TMH1(DoT, 1986). This test is known as the Riedel and Weber test. Tentative limits have been suggested and are as follows:

<1 unsatisfactory	3 good
1 borderline	4 very good
2 acceptable	5 excellent

Risk of stripping can be minimised by applying sand as a second layer or by using a Cape seal.

4.8.5 Portland Cement Concrete (PCC)

The quality of material used in concrete pavement construction should comply with the requirements set out in TRH 14. The following considerations are presented there:

Table 4-19: PORTLAND CEMENT CONCRETE MATERIAL CONSIDERATIONS

Material	Consideration to be checked
Coarse Aggregate	Grading Crushing strength Flakiness Index Other
Fine Aggregate	Grading Dust content Fineness Modulus Chloride Content Deleterious materials Siliceous particle content Other
Cement, water, admixtures	Various
Strength	28 day flexural strength ≥ 3.8 MPa

4.9 Durability

4.9.1 Unbound materials

Durability is an issue for unbound crushed stone materials used in the basecourse and for surfacing aggregate (Sampson and Netterberg, 1988). The problems are whether rocks will alter chemically (decompose), or whether existing alteration products (formed by natural weathering processes through geological time) are mobilised and freed (degradation). The end result is that the performance of the material during the life of the road is reduced, and the life of the road is also reduced. This can occur within as short a time as a couple of years. If the affected material is in the basecourse, the mode of failure is shear in the base leading to rutting, crocodile cracking and potholing. This can be due to:

- disintegration of the top 10-15mm in the base forming excessive fines and loss of adhesion of the surface seal;
- breakdown of the basecourse aggregate in service from the repeated action of traffic in the presence of excess moisture with the resultant generation of plastic fines (generally expansive smectite clays) which change the nature of the materials and significantly reduce the bearing capacity.

The basic igneous (dolerite, basalt) and acid igneous (granite) classes of rocks (discussed in TRH 14 and in Weinert, 1980) are considered the most likely to decompose in a wetter environment, which can be either climatic or due to excess moisture in the pavement. The disintegrating rocks are the high silica (quartzites, quartz gravels and sandstones), arenaceous (mainly Karoo sandstones), argillaceous (mudrocks, shales) rocks and carbonates. Durability is rarely a problem in the layers beneath the basecourse.

The durability of a material can be tested by any of several tests:
- Durability Mill test,
- 10% Fine Aggregate Crushing test (10% FACT),
- Aggregate Impact Value (AIV) test (also performed as a modified AIV),
- Pick and Click, or
- Secondary Mineral Count.

While no single test has proved adequate for defining the durability of all different material types, the state of knowledge at the time of writing was that either the Durability Mill test or the modified AIV test provides the most suitable assessment. Accordingly the specifications for durability are given here in terms of these test results. Testing should be performed on all new material sources for which there is no history of performance, and on all suspect materials.

The Durability Mill test shows the durability of a material in terms of the fineness product (FP) which is the product of plasticity index (PI) and percentage material passing the 0,425mm sieve (P425) (Sampson, 1988). The recommended limits and specification are given in Table 4-20 (Sampson, 1992a). Current research by Paige-Green (1992a) indicates that these limits may be able to be further relaxed for the lighter traffic roads (E80s < 200 000), but no specifications are yet available. As an interim guide, for roads carrying less than 100 000 E80's in both directions, the proposed limits can be increased by 20 per cent. It is, however, recommended that the maximum plasticity index in any of the Durability Mill treatments should not exceed 15.

Table 4-20: MATERIAL RELATED DURABILITY LIMITS FOR UNBOUND MATERIALS

Material type	**Modified AIV**		**Durability Mill Test**	
	dry	wet/dry ratio	Max. Durability Mill index	Max. P425
Basic crystalline	< 39	< 114%	125	35
Acid crystalline			420	35
High silica[a]			420	35
Arenaceous	< 31		125	35
Argillaceous	< 24	(wet < 26)	125	35
Carbonates[b]	< 39	< 114%	125	35
Diamictites	< 22	< 115%	125	35
Metalliferous[b]	not required			
Pedogenic[c]	≤ 39	≤ 120% for calcrete ≤ 114% for silcrete	480	55

Notes a: applicable if the clay mineral present is kaolinite

b: applicable if soil binder is added to create fines

c: these are tentative limits

4.9.2 Bound materials

Durability is an important issue for lime or cement stabilised layers. Research into low volume roads (Paige-Green, 1992a) has found that many bound layers have carbonated (in which the cementation strength is lost due to interaction with CO_2 in the atmosphere and the soil), and thus have little residual stabilisation. No research is available yet to determine if durability is an issue for chemical surface stabilisers such as the SPPs although the theoretical reactions should be permanent.

As partial or complete carbonation of the treated layer occurs, this can lead to a large decrease in strength. If the traffic volume is light relative to the structural design and the moisture regime is dry or optimum, this may not lead to significant problems. However if the traffic volume is at the limit for the particular design or the pavement moisture regime is wet, then severe rutting, cracking and shearing can occur. Durability testing should be conducted for all bound materials used where the moisture regime is expected to be wet at any time, or where the strength of the bound layer is likely to be critical for the performance of the road. Three tests have been developed to measure the durability:

- gravel initial consumption of lime or cement test (ICL or ICC);
- wet/dry brushing test; and
- unconfined compressive strength test on cycled or carbonated specimen.

The test limits are shown in Table 4-21 (Sampson, 1992b)

Table 4-21: MATERIAL RELATED DURABILITY LIMITS FOR BOUND MATERIALS

Test method	Specification
Wet/dry brushing test	Hand test: stabilised base < 25% loss after 12 cycles; stabilised subbase < 40% loss after 12 cycles
	Mechanical test Stabilised base: < 8% loss after 12 cycles Stabilised subbase <13% loss after 12 cycles
Gravel ICL or ICC	Stabiliser content 1% higher than ICL or ICC[a]
Vacuum carbonated UCS	same TRH 14 limits as for normal UCS test

Note: (a) Only if carbonated UCS values are not sufficient

The tests may show the need to increase the stabiliser content. However there are both economic and engineering limits to the amount of stabiliser that should be added, and for practical purposes this is about 4 - 5%. Any more than this and cracking of the bound layer can reflect through the surface.

At the lower end, if only (2 to 3 per cent) stabilizer is added, it is often such that it is more likely to "modify" the material than stabilise it. This practice has value however in improving material workability, even though the quantity is too low to prevent carbonation.

Due to practical constraints with mixing in of the stabilizer minimum limits are usually set at 2 - 2,5 % of stabilizer.

In many cases this can result in very high layer strengths (in excess of that which is required), with the result that even after carbonation sufficient strength exists in the layer.

Example: Required strength for base: CBR 80% UCS 750 kPa

Available material:	CBR 65%
Uncarbonated:	UCS 1200 kPa (2,5% cement)
Carbonated:	UCS 800 kPa
ICC	3%
Uncarbonated:	UCS 1800 kPa (4 % cement)
Carbonated:	UCS 1500 kPa

Although the ICC is not satisfied, stabilisation with 2,5 % cement is therefore acceptable.

4.10 Environmental and conservation aspects

An increasingly important aspect regarding road construction is the environmental impact of the structure and the need for conservation of resources which are rapidly becoming scarce. The roads and borrow pits should be located on alignments and in areas where the impacts are minimal and rehabilitation of areas affected by construction is considered as part of the project and this aspect is dealt with in more detail in Chapter 12 of this document.

Material conservation is an extremely important aspect to consider when designing and maintaining low volume roads. In areas where good wearing course gravels are scarce or involve haulage over significant distances and traffic is heavy, the periodic replacement of the gravel lost under traffic is both costly and unsightly. The need for conservation may sometimes justify the surfacing of a road which would otherwise be left as gravel. There are sophisticated computer programmes (such as HDM 3 from the World Bank) which can model an entire network and allow the effect of resource constraints to be analyzed and acted on, but these are complex to use.

4.11 References

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5 PAVEMENT DESIGN

5.1 Introduction

This document is aimed at roads which access rural areas and have a traffic volumes of less than 500 evu's or 400 vehicles per day. For purposes of pavement design, the volume of heavy vehicle axles which the road has to carry is far more important than the total traffic volume. The traffic distribution and loading on roads varies and is dependent on the purpose of the road. Therefore it is possible that a strong pavement will be required even though the actual number of vehicles carried is low. Given typical vehicle distributions and growth in axle loads and volumes on rural roads, the design traffic is not expected to be greater than 800 000 E80s. The low volume roads (LVRs) referred to in this chapter have either concrete bases or bituminous surfacings to keep water out and protect the underlying layers from the disruptive effects of traffic and provide an all weather, dust-free riding surface. The roads are all considered to be short local access roads.

5.2 Design philosophy

The pavement structure of LVR's usually consists of one or two layers of natural gravel surfaced with a bituminous seal, although the use of cemented layers is not uncommon. Modes in which these pavements typically fail are cracking of the surface layer, crocodile cracking with formation of potholes and permanent deformation of the pavement system which manifests as rutting on the pavement surface. The pavement structures proposed in this chapter were designed against permanent deformation.

Designs are based on the material being at equilibrium moisture content, which further underlines the importance of timeous crack sealing or renewal of cracked bituminous surface layers. Special pavement structures are proposed for environments where maintenance of roads is expected to be infrequent.

Cracking of bituminous surfacing layers is a failure mode caused by oxidation of the binder and by the elastic behaviour of the pavement structure. It is the intention that this failure mode (cracking of bituminous surfacing layers) be identified and rectified through maintenance action. A failure criterion of 20 mm permanent deformation on the surface of the pavement was used.

The pavement structures proposed in this chapter are further based on the following conditions:

- Adequate provision of surface and sub-surface drainage as the poorer quality materials are more water sensitive.

- Adequate construction control.
- Few overloaded vehicles.

Designs are based on a 50 per cent confidence level. It is possible that a deformation of 20 mm on low volume roads may be too strict a failure criterion, particularly in the drier region of the country. The ease with which a 20 mm rut can be removed during resealing operations with a coarse slurry enables the road authority to extend the life of pavements remarkably. Therefor pavements designed according to the recommended procedure can be acceptable to the road user for much longer than anticipated, given adequate maintenance capability and favourable levels of other distress modes.

5.3 Design strategy

5.3.1 Analysis period and structural design period

The analysis period is a convenient planning period during which full reconstruction of the pavement is undesirable. The structural design period is defined as the period during which it is predicted with a reasonable degree of confidence that no structural maintenance will be required. In order to fulfil the design objective of selecting the optimum pavement in terms of present worth of cost, it is necessary to consider the way the pavement is expected to perform over the analysis period. The manner in which a design strategy can be presented is demonstrated schematically in Figure 5-1. The figure shows the generalized trends of riding quality decreasing with time and traffic for two different pavement structures, that is:

(a) Design 1, which requires resurfacing to maintain the structure in good condition, and later some structural rehabilitation (Figure 5-1(a)) and;

(b) Design 2, which is structurally adequate for the whole analysis period and only requires three resurfacings (Figure 5-1(b)).

It is important to note that any design procedure can only make an estimate of the timing and nature of maintenance measures that may be needed. Such estimates are only approximate, but they provide a valuable guide for a design strategy. The actual maintenance should be determined by a proper maintenance procedure. Maintenance and rehabilitation are described in more detail in Chapters 10 and 11.

5.3.2 Selection of analysis period and structural design period

5.3.2.1 Selection of analysis period

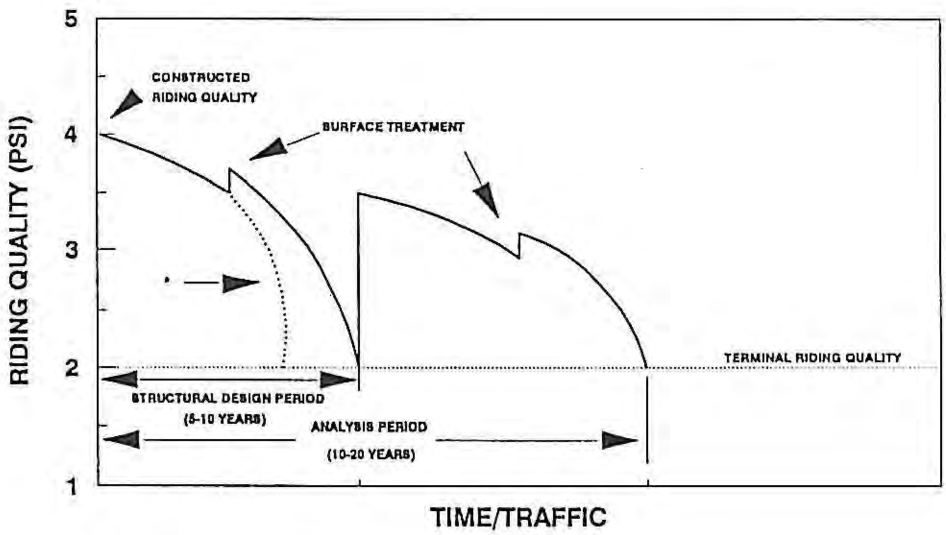
The analysis period is a realistic cost period. There may be a difference between the analysis period and the total period over which the facility will be used. The analysis period is often related to geometric life. If the road alignment is fixed, a period of 30 years should be used. In the case of a short geometric life and in a changing traffic situation, a short analysis period will be used. A short analysis period will also be used when the proposed pavement has a limited life. The analysis period will influence the salvage value introduced in Section 5.8. Table 5-1 shows the possible ranges and recommended analysis periods for low volume roads. These values should be used for the economic analysis, unless more detailed information is available.

Table 5-1: ANALYSIS PERIODS FOR LOW VOLUME ROADS.

Analysis period (years)		
Range	Recommended period	
	Fixed alignment	Uncertain conditions
10 - 20	20	10 - 15

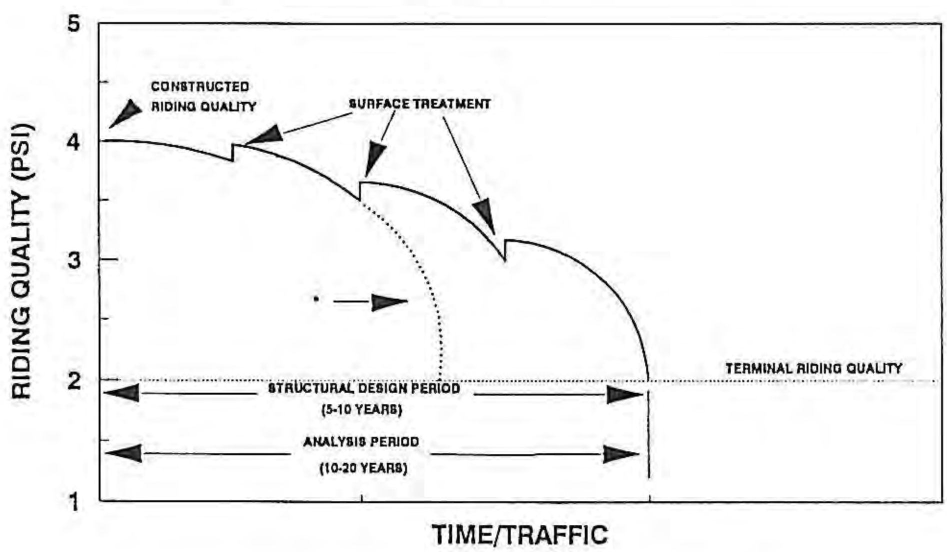
5.3.2.2 Selection of structural design period

For low volume roads it is usually more economical to select a short structural design period of say, 10 years. However, in cases where it will be difficult or impractical to carry out structural rehabilitation, for example in difficult terrain or because of financial constraints, a longer period of 15 to 20 years can be selected.



(a)

DESIGN 1 REQUIRES TWO RESURFACINGS AND ONE STRUCTURAL REHABILITATION DURING THE ANALYSIS PERIOD.



(b)

DESIGN 2 REQUIRES THREE RESURFACINGS AND NO STRENGTHENING DURING THE ANALYSIS PERIOD.

* IF SURFACING IS NOT MAINTAINED AND IF WATER-SUSCEPTIBLE MATERIALS ARE USED IN THE PAVEMENT.

Figure 5-1: ILLUSTRATION OF DESIGN PERIOD AND ALTERNATIVE DESIGN STRATEGIES (CSRA, 1985)

5.4 Behaviour of different pavement types

The behaviour of a pavement and the type of distress which will become the most critical, vary with the type of pavement. There are four major pavement types namely granular, cemented, bituminous and concrete base pavements. The behaviour of these different pavement types will determine the type of maintenance normally required and may also influence the selection of pavement type. Granular, cemented, emulsion treated and concrete base pavements are considered in this document for use on low volume road pavements. A brief description of the behaviour of granular and cemented base pavement types are given below.

5.4.1 Untreated granular base pavements

This type of pavement comprises a thin bituminous surfacing, a base of untreated gravel or crushed stone, a granular or cemented subbase and a subgrade of various soils or gravels. The mode of distress in a pavement with an untreated base is usually deformation arising initially from densification and later from small shear deformations in the untreated materials subjected to repeated traffic loading. The deformation may manifest itself either as rutting or longitudinal roughness. This is illustrated in Figure 5-2(a).

In pavements with cemented subbases, the subbase improves the elastic behaviour of the pavement in that the cemented subbase will lead to smaller elastic deflections on the surface of the pavement. The relatively strong cemented subbase will protect the underlying layers (smaller stresses will be induced by the load in the underlying layers because the load is spread over a larger area) and less rutting will result in the underlying layers. This implies that the rutting will be confined to the base layer in pavement structures with cemented subbases, whereas rutting is distributed through the whole pavement structure in structures with granular subbase layers.

Apart from shrinkage cracking, the cement treated subbase will crack under traffic loading. The cracking may propagate until eventually the layer exhibits properties similar to those of a natural granular material. It is unlikely that cracking will reflect to the surface. However, this is dependent on the pavement depth. The cement treated subbase thus moves from a pre-cracked to a post-cracked phase. The pre-cracked phase is relatively short for the cement treated materials and support normally encountered in low volume road pavements. The post-cracked phase is long when compared to the pre-cracked phase and is divided into an effective fatigue and equivalent granular phase (see Figure 5-2(b)). The granular phase of a cement treated subbase under a granular base adds substantially to the useful life of the pavement. However, especially in the lower traffic classes (E0), rutting in the untreated layers of the pavement may reach the terminal value of 20 mm before the cement treated subbase reached the end of its life in the effective fatigue phase. This is because relatively poor materials are used as base and subgrade layers for the lower traffic classes.

The effective modulus of the cement treated subbase in the pre-cracked phase is high (approximately 2 000 MPa). This relatively rigid subbase fatigues under traffic and assumes a lower effective modulus. This change in modulus does not result in a marked increase in deformation, but changes in resilient deflection and radius of curvature as shown in Figure 5-2(c) are observed.

The eventual modulus of the cemented subbase will depend on the quality of the material originally stabilized, percentage cementing agent, effectiveness of mixing process, absolute density achieved and the degree of cracking. The ingress of moisture can significantly affect the modulus in the post-cracked phase. In some cases the layer may behave like a good quality granular material with a modulus in the order of 200 to 300 MPa, while in other cases the modulus will reduce to the order of 50 MPa. This change is also shown diagrammatically in Figure 5-2(b). The net result is that the modulus of the cemented subbase decreases to very low values causing extensive rutting in the subgrade. Generally cracks on the surface will develop and with ingress of water, pumping from the subbase may occur.

The surfacing may also crack due to ageing of the binder or load associated fatigue cracking. Granular materials are often susceptible to water and excessive deformation may occur when water ingresses through the surface cracks. The water susceptibility depends on factors such as grading, PI of fines and compaction. Timely resurfacing of the surfacing is often critical for retaining the inherent strength of the pavement.

5.4.2 Cemented base

In these pavements, most of the traffic stresses are absorbed by the cemented layers and little by the subgrade. It is likely that some block cracking will be evident early in the life of cemented bases which is caused by the mechanisms of drying shrinkage and thermal stresses in the cement treated layers. Traffic-induced cracking will cause the blocks to break up into smaller ones. These cracks propagate through the surface. Ingress of water through the surface cracks may cause the blocks to rock, resulting in the pumping of fines from the lower layers. Rutting or roughness will generally be low up to this stage, but is likely to accelerate as the extent of cracking increases (see Figure 5-2(d)).

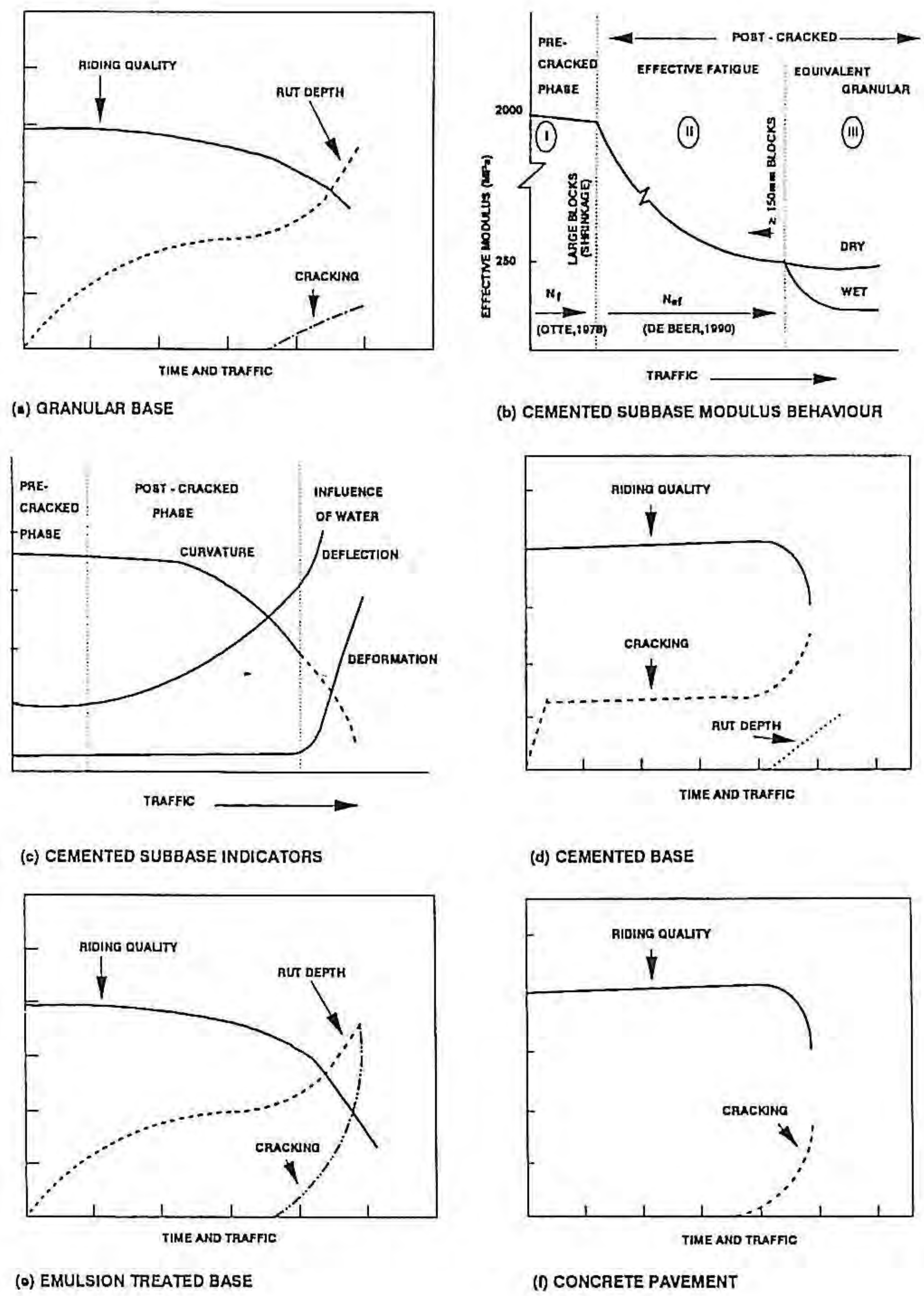


Figure 5-2: GENERALIZED PAVEMENT BEHAVIOUR CHARACTERISTICS (CSRA, 1985)

Pavements consisting of cemented bases on granular subbases are sensitive to overloading and to the ingress of moisture through the cracks. The initial cracks may be rehabilitated by sealing. When both base and subbase are cemented, the pavement will be less sensitive to overloading and moisture. Pumping problems can be minimised by the elimination of weak lenses by good construction quality, reduction of the subbases's moisture sensitivity through material modification and timeous reseal, preferably with modified binders.

Crushing of the cemented base may also occur under extreme conditions of loading (high tyre pressures).

5.4.3 Emulsion treated bases

In emulsion treated base pavements, both permanent deformation and fatigue cracking are possible. Two types of subbase can be used, namely untreated granular or weakly cemented stabilized. Rutting may originate in either the bituminous or the untreated layers or both. This is shown in Figure 5-2(e). If the subbase is cemented, there is a possibility that shrinkage or thermal cracking will reflect to the surface, especially if the base is less than 150 mm thick or if the subbase is excessively stabilised. Maintenance usually consists of crack sealing.

Recent research has shown that emulsion treated bases develop strength with time and that this strength gain needs to be incorporated during design. Lime is often added one day prior to the addition of emulsion.

5.4.4 Concrete or roller compacted concrete pavements

In concrete pavements, most of the traffic loading is carried by the concrete slab and little stress is transferred to the subgrade. For low volume roads, this implies that concrete or roller compacted concrete is ideally suited to weak subgrade conditions. Distress of the pavement usually appears first as spalling near joints and may progress to cracking in the wheel paths. Once distress becomes evident, deterioration is usually rapid.

5.5 Design traffic

5.5.1 Traffic classes for structural design purposes

The equivalent standard axle concept is used to express a range of axle loads as applied by mixed traffic, in terms of a common denominator. In South Africa, an 80 kN axle is used as the standard axle load, and the damage caused by any other axle load relative to the standard axle is defined as the equivalent standard axle (E80). For mixed traffic, the total E80s is the number of 80 kN axle loads which causes the same damage that the actual spectrum of imposed axle loads causes.

For structural design, an estimate of the cumulative equivalent traffic over the structural design period is required. The cumulative equivalent traffic (total E80s over the design period) is grouped into five traffic classes, varying from E0 for light traffic to E4 for heavy traffic (refer to TRH 4). The traffic class is a major factor in the selection of the actual pavement structure obtained from the catalogue of suitable pavement structures. The traffic classes are defined in Table 5-2. This document is concerned only with the E0 and E1 traffic classes. These traffic classes were further subdivided as shown in Table 5-3 and are related to practical differences in pavement layer thicknesses, material type and estimation of design traffic.

5.5.2 Traffic loading information

Traffic loading information may be obtained from the following sources:

(a) tabulated average E80 values
(b) published results of surveys
(c) transportation planning models
(d) estimation procedures based on visual observations
(e) weighing methods.

Table 5-2: CLASSIFICATION OF TRAFFIC FOR STRUCTURAL DESIGN PURPOSES (TRH 4, 1985).

Traffic class	Cumulative equivalent traffic (E80/lane)	Description
E0	$<0,2 \times 10^6$	Very lightly trafficked roads; very few heavy vehicles
E1	$0,2 - 0,8 \times 10^6$	Lightly trafficked roads, mainly cars, light delivery and agricultural vehicles, very few heavy vehicles
E2	$0,8 - 3 \times 10^6$	Medium volume of traffic; few heavy vehicles
E3	$3 - 12 \times 10^6$	High volume of traffic and/or many heavy vehicles
E4	$12 - 50 \times 10^6$	Very high volume of traffic and/or a high proportion of fully laden heavy vehicles

Table 5-3: SUBDIVISION OF E0 AND E1 TRAFFIC CLASSES FOR THE DESIGN OF LOW VOLUME ROAD PAVEMENT STRUCTURES.

Traffic class	Cumulative equivalent traffic (E80/lane)	Description
E0-1	$< 5 \times 10^3$	Very lightly trafficked roads; very few heavy vehicles
E0-2	$5 - 30 \times 10^3$	
E0-3	$30 - 100 \times 10^3$	
E0-4	$100 - 200 \times 10^3$	
E1-1	$200 - 400 \times 10^3$	Lightly trafficked roads, mainly cars, light delivery and agricultural vehicles; very few heavy vehicles
E1-2	$400 - 800 \times 10^3$	

The first four methods are recommended for use in the pavement design of new and rehabilitated roads where the expected design traffic volume is smaller than E1. Use of these methods ((a), (b), (c) and (d)) in determining the design traffic volume (E80s) are discussed in the following paragraphs. Use of method (e) is not discussed as it is considered too sophisticated for application to LVR's.

5.5.2.1 Tabulated factors

The simplest form of information is tabulated values representing average conditions. A rough estimate is made of the type of heavy traffic and **average E80s per heavy vehicle** are then read from a table such as Table 5-4. This factor is then multiplied by the number of heavy vehicles to obtain the average daily E80s. This method is approximate and can result in serious errors. More reliable estimates are shown in Table 5-5.

5.5.2.2 Estimation of equivalent traffic from published results

Extrapolation of the findings published for other roads in the vicinity of the road under consideration is often used. Depending on the purpose for which the data are required and the level of accuracy required, such an approach is permissible. However, cognisance should be taken of potential errors which may occur by such extrapolation. Published results include the system of Comprehensive Traffic Observations (CTO) (Department of Transport, 1989b) and data collected by road authorities.

Table 5-4: AVERAGE E80S PER HEAVY VEHICLE (TRH 16, 1991).

LOADING OF HEAVY VEHICLES	E80/HEAVY VEHICLE
Mostly unladen	0,6
50 % of heavy vehicles laden and 50 % unladen	1,2
> 70 % of the heavy vehicles fully laden	2,0

Table 5-5: AVERAGE E80S FOR DIFFERENT HEAVY VEHICLE CONFIGURATIONS (TRH 16, 1991).

Vehicle type	Average E80s	Range in average E80s per vehicle found at different sites
2-axle truck	0,70	0,30 - 1,10
2-axle bus[a]	0,73	0,41 - 1,52
3-axle truck	1,70	0,80 - 2,60
4-axle truck	1,80	0,80 - 3,00
5-axle truck	2,20	1,00 - 3,00
6-axle truck	3,50	1,60 - 5,20
7-axle truck	4,40	3,80 - 5,00

Note: (a) 2,77 when laden

Where other data sources are used, care should be taken with the vehicle classification system used, as well as the method by which the information was generated.

5.5.2.3 Estimation of procedures based on visual observations

This is a technique where an observer categorizes vehicles into different groups and uses an average E80 per axle to derive the average daily traffic (ADT). It is a simple, inexpensive technique which can give acceptable results if used with circumspection. This procedure is used when axle load determinations are not justified nor a high level of accuracy required.

A visual classification method was developed in South Africa (Lomas, 1976). In this method an estimation of the load carried by heavy vehicles is made by deciding on the extent to which the **volumetric** capacity is fulfilled. Note that this method is applied to all vehicles with an axle mass greater than 2 000 kg. These can usually be identified as those vehicles which are fitted with dual wheels. It would consequently exclude pick-ups and minibuses.

The method is explained in more detail in TRH 16 (CSRA, 1991). Suitable forms and examples of the required calculations are also given in TRH 16. Table 5-6 gives typical axle load factors for various loading conditions.

Table 5-6: AVERAGE AXLE LOAD FACTORS CORRESPONDING TO VARIOUS VEHICLE LOADING CONDITIONS TO ESTIMATE E80S FROM VISUAL SURVEYS (n=4).

DISTRIBUTION OF VEHICLE LOADING (MEDIUM-HEAVY AND HEAVY VEHICLES)	AXLE LOAD FACTORS (E80s/AXLE)
Predominantly lightly laden heavy vehicles (< 35 % of heavy vehicles fully laden; > 45 % empty)	0,3
Fully laden, partially laden and empty vehicles (40-45 % of heavy vehicles fully laden; 35-45 % empty or partially laden)	0,5
Fully laden and partially laden heavy vehicles (60-75 % of heavy vehicles full; < 30 % empty or partially laden)	0,7
Predominantly fully laden heavy vehicles (> 75 % of heavy vehicles full). Occurs only in exceptional cases.	0,9

5.5.3 Calculation of equivalent E80s (design traffic)

The computation of equivalent design traffic in terms of E80s involves the following:

(a) load equivalence of traffic

(b) projecting the traffic data over the structural design period.

This is covered in greater detail in TRH 4.

5.5.3.1 Load equivalence of traffic

The load equivalency factor relates the application of any given axle load to the damage caused relative to the standard axle, which is taken as 80 kN. Although the equivalency factor is a function of pavement composition, material types, vehicle speed, the definition of the terminal pavement condition and the pavement state, an average value is usually computed from the following relation:

$$F = (P/80)^n \quad (5.1)$$

where F = load equivalency factor

P = axle load

n = load equivalency exponent or damage factor, usually taken as 4.

Pavements that are sensitive to overloading, such as shallow-structured pavements with thin cemented bases, may have n-values of more than 4 whereas less load sensitive deep structured pavements may have n-values of less than 4. An evaluation of the sensitivity of the exponent is discussed in Section 5.5.4.

The equivalent traffic can be determined by multiplying the number of axle loads in each load group (t_j) of the entire load spectrum by the relevant equivalency factor (F_j) determined from Table 5-7.

By summation, the equivalent daily traffic is:

$$E = \Sigma t_j.F_j \quad (5.2)$$

where E = equivalent daily traffic (E80s)

t_j = number of axles in the j th load group

F_j = equivalency factor for j th load group (see Table 5-7).

5.5.3.2 Projection to the initial design year

The present average daily equivalent traffic (daily E80s) can be projected to the initial design year by multiplying by a growth factor determined from the E80 growth rate:

$$g_x = (1 + 0{,}01.i)^x \quad (5.3)$$

where g_x = E80 growth factor

i = E80 growth rate

x = time between determination of axle load data and opening of road in years.

The E80 growth factor g_x is tabulated in Table 5-8 for different values of traffic growth rate i and years between determination of axle load data and opening of road, x.

Table 5-7: 80 kN SINGLE-AXLE EQUIVALENCY FACTORS, DERIVED FROM $F = (P/80)^4$ (TRH 4, 1985).

Single-axle load, P (kN)	80 kN axle equivalency factor, F
Less than 15	0,000
15 - 24	0,004
25 - 34	0,019
35 - 44	0,062
45 - 54	0,15
55 - 64	0,32
65 - 74	0,59
75 - 84	1,00
85 - 94	1,6
95 - 104	2,4
105 - 114	3,6
115 - 124	5,1
125 - 134	7,0
135 - 144	9,4
145 - 154	12
155 - 164	16
165 - 174	20
175 - 184	26
185 - 194	32
195 - 204	39
More than 205	50

The growth rate of E80s can be different from the growth rate of heavy vehicles. This is because of the growth rate of the number of axles per vehicle and the extent to which the vehicles are loaded on average. An increase in the permissible axle load will also lead to an increase in E80 growth rate. The E80 growth rate will also depend on whether the facility is used for tourism, farming or industrialisation. Future developments will also have an influence on the E80 growth rate. Therefore, where possible, the growth rate should be based on specific information. It will normally fall between 2 and 10 per cent. A value of 4 per cent is recommended.

Apart from the increase occurring before the initial year, it is accepted that additional traffic will be attracted to a newly surfaced road.

The daily equivalent traffic in the initial year is given by:

$$E_{initial} = E.g_x + A \qquad (5.4)$$

where $E_{initial}$ = daily equivalent E80s in the initial year

E = daily equivalent traffic at time of determination of axle load data

g_x = E80 growth factor (see Table 5-8).

A = attracted traffic

5.5.3.3 Computation of cumulative equivalent traffic

The cumulative equivalent traffic (total E80s) over the structural design period may be calculated from the daily equivalent E80s in the initial design year and the E80 growth rate for the design period.

The cumulative equivalent E80s may be calculated from:

$$N_e = E_{initial}.f_y \quad (5.5)$$

where N_e = cumulative equivalent E80s for the design period

$E_{initial}$ = daily equivalent E80s in the initial year

f_y = cumulative E80 growth factor

= $365(1+0,01.i)[(1+0,01.i)^y-1]/(0,01.i)$

where i = E80 growth rate

y = structural design period in years.

The cumulative E80 growth factor f_y is tabulated in Table 5-9 for different E80 growth rates i and prediction period y in years.

5.5.3.4 Distribution per lane

LVR's typically consist of two lanes, that is one lane per direction. The cumulative equivalent E80s are therefore normally multiplied by 0,5 to obtain the cumulative equivalent E80s per lane which should be used to determine the design traffic class. In some cases one lane may be carrying loaded vehicles whereas the other is carrying empty vehicles. A multiplication factor larger than 0,5 should be considered for the critical lane in such cases.

If the road consists of only one lane, the cumulative equivalent E80s constitute the design traffic. Dependent on the width of the lane and the effect of traffic wandering, it might however not be necessary to design for the total expected number of E80s.

The traffic class can now be determined by comparing the cumulative equivalent E80s per lane for the design period (total E80s) with the ranges for the different classes given in Table 5-3.

Table 5-8: TRAFFIC GROWTH FACTOR (g) FOR CALCULATION OF FUTURE OR INITIAL TRAFFIC FROM PRESENT TRAFFIC.

Time between determination of axle load data and opening of road, x (yrs)	*g for Traffic Increase, i (% per annum)								
	2	3	4	5	6	7	8	9	10
1	1,02	1,03	1,04	1,05	1,06	1,07	1,08	1,09	1,10
2	1,04	1,06	1,08	1,10	1,12	1,14	1,17	1,19	1,21
3	1,06	1,09	1,12	1,16	1,19	1,23	1,26	1,30	1,33
4	1,08	1,13	1,17	1,22	1,26	1,31	1,36	1,41	1,46
5	1,10	1,16	1,22	1,28	1,34	1,40	1,47	1,54	1,61

$*g = (1 + 0{,}01i)^x$

5.5.3.5 Use of nomograms

The cumulative equivalent E80s for the design period (or design traffic class) may be calculated from the **initial daily equivalent E80s** and an E80 growth rate with the nomogram presented in Figure 5-3. E80 growth rates as discussed in Section 5.5.3.2 should be considered.

The cumulative equivalent E80s for the design period (or design traffic class) may be calculated from the **average daily equivalent E80s** (on an average day, near the middle of the structural design period) with the nomogram presented in Figure 5-4. The structural design period then becomes the major factor determining the design traffic class.

5.5.4 Sensitivity of traffic class to growth rate, loading and other factors

An important design step is to consider the influence of changes to the basic data used, that is growth rate, E80 per vehicle or E80 per axle, initial E80 per day, structural design period and conversion factors. Certain factors may be more uncertain or may have a larger influence than others for a specific design. Typically, however, traffic loading information and growth rate would require evaluation. A graphical plot as shown in Figure 5-5 may be used for this evaluation. Plotted on this figure are the cumulative E80s for a selected design period and initial traffic, but varying axle load and traffic growth factors. The most likely position is also shown on this graph. For the data in this figure it may be seen that changes in the growth rate and/or axle load factor (E80s per axle) would not affect the traffic class of the projected situation. However, had the axle load factor been 0,2 and the growth rate 4 per cent, then the traffic class would have been on the boundary between E0-4 and E1-1 - a fact which may not have been evident from a simple calculation. Any small change in either the axle load factor or growth rate would then place the traffic into the next higher traffic class.

Table 5-9: **TRAFFIC GROWTH FACTOR (f) FOR CALCULATION OF CUMULATIVE TRAFFIC OVER PREDICTION PERIOD FROM INITIAL (DAILY) TRAFFIC.**

Prediction period (years)	Compound growth rate (% per annum)									
	2	4	6	8	10	12	14	16	18	20
4	1 534	1 611	1 692	1 776	1 863	1 953	2 047	2 145	2 246	2 351
5	1 937	2 056	2 180	2 312	2 451	2 597	2 750	2 911	3 081	3 259
6	2 348	2 517	2 698	2 891	3 097	3 317	3 551	3 801	4 066	4 349
7	2 767	2 998	3 247	3 517	3 809	4 124	4 464	4 832	5 229	5 657
8	3 195	3 497	3 829	4 192	4 591	5 028	5 506	6 029	6 601	7 226
9	3 631	4 017	4 445	4 922	5 452	6 040	6 693	7 417	8 220	9 109
10	4 076	4 557	5 099	5 710	6 398	7 173	8 046	9 027	10 130	11 369
11	4 530	5 119	5 792	6 561	7 440	8 443	9 588	10 895	12 348	14 081
12	4 993	5 703	6 526	7 480	8 585	9 865	11 347	13 061	15 044	17 336
13	5 465	6 311	7 305	8 473	9 845	11 458	13 352	15 575	18 183	21 241
14	5 947	6 943	8 130	9 545	11 231	13 242	15 673	18 490	21 887	25 927
15	6 438	7 600	9 005	10 703	12 756	15 239	18 242	21 872	26 257	31 551
16	6 939	8 284	9 932	11 953	14 433	17 477	21 212	25 795	31 414	38 299
17	7 450	8 995	10 915	13 304	16 278	19 983	24 598	30 346	37 500	46 397
18	7 971	9 734	11 957	14 762	18 308	22 790	28 458	35 625	44 680	56 115
19	8 503	10 503	13 061	16 338	20 540	25 934	32 859	41 748	53 154	67 776
20	9 045	11 303	14 232	18 039	22 995	29 455	37 875	48 851	63 152	81 769
25	11 924	15 808	21 227	28 818	39 486	54 506	75 676	105 517	147 559	206 727
30	15 103	21 289	30 587	44 656	66 044	98 656	148 459	224 533	340 661	517 664

Based on $f = 365.(1 + 0{,}01.i).[(1 + 0{,}01.i)^y - 1]/(0{,}01.i)$

Using the formula

$$E80\ growth\ rate = \frac{(1+\frac{TT}{100})(1+\frac{E80PV}{100})(\frac{FPHV}{100})-(\frac{FPHV}{100})}{\frac{PPHV}{100}} X100$$

where TT = total traffic growth rate (%)
E80PHV = E80 per heavy vehicle growth rate
FPHV = future percent heavy vehicles
PPHV = present percent heavy vehicles

the expected E80 growth rate on a specific road can be determined for a range of scenarios as given in Table 5-10. This type of sensitivity analysis is useful for developing a realistic range for the vertical axis as given in Figure 5-5.

Table 5-10: E80 GROWTH RATES (%) FOR VARIOUS FUTURE SCENARIOS (TRH 16, 1991).

			E80/veh growth rate (%)		Total traffic growth rate (%)		
					Low	Probable	High
					4,0	5,5	7,0
Future % heavy vehicles (present 25%)	Low	25	Low	0	4,0	5,5	7,0
			Probable	2	6,1	7,6	9,1
			High	4	8,2	9,7	11,3
	Probable	30	Low	0	5,9	7,4	9,0
			Probable	2	8,0	9,6	11,1
			High	4	10,2	11,7	13,3
	High	35	Low	0	7,6	9,1	10,7
			Probable	2	9,7	11,3	12,9
			High	4	11,9	13,3	15,1

Although the exponent in the equivalency relationship is usually taken as 4, it has been shown that if deterioration other than roughness is considered, then the exponent, called the damage exponent, could vary between 2 and 10. The impact of different damage exponents on equivalent damage of the axle load data collected on any road cannot be estimated by simple inspection. A sensitivity analysis has to be performed. This would take the form of calculating the percentage ratio between the E80 calculated with the variable damage exponent and the E80 calculated with the normal exponent of 4.

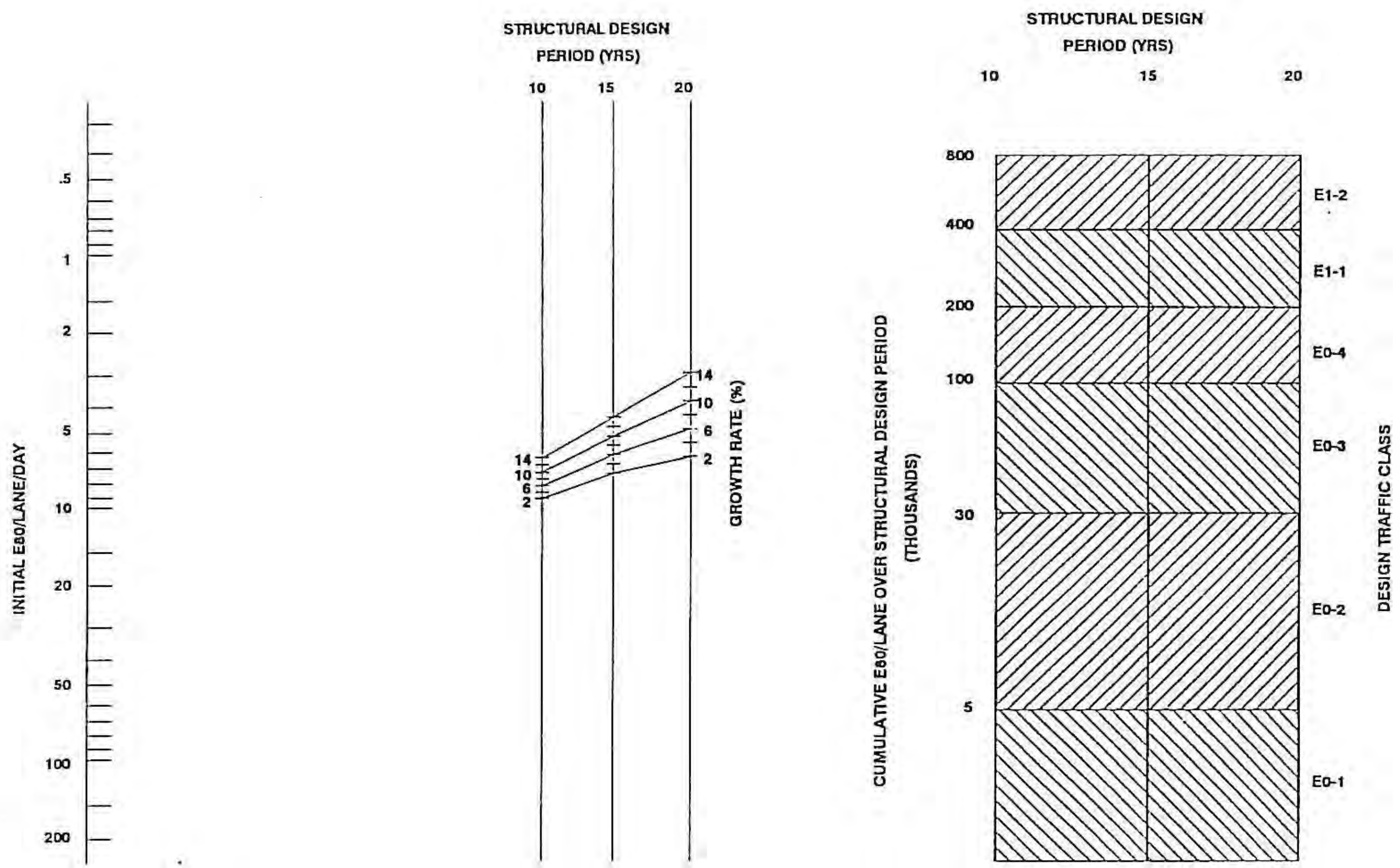


Figure 5-3: NOMOGRAM FOR DETERMINING DESIGN TRAFFIC CLASS FROM INITIAL E80/LANE/DAY, TRAFFIC GROWTH RATE AND STRUCTURAL DESIGN PERIOD (CSRA, 1985)

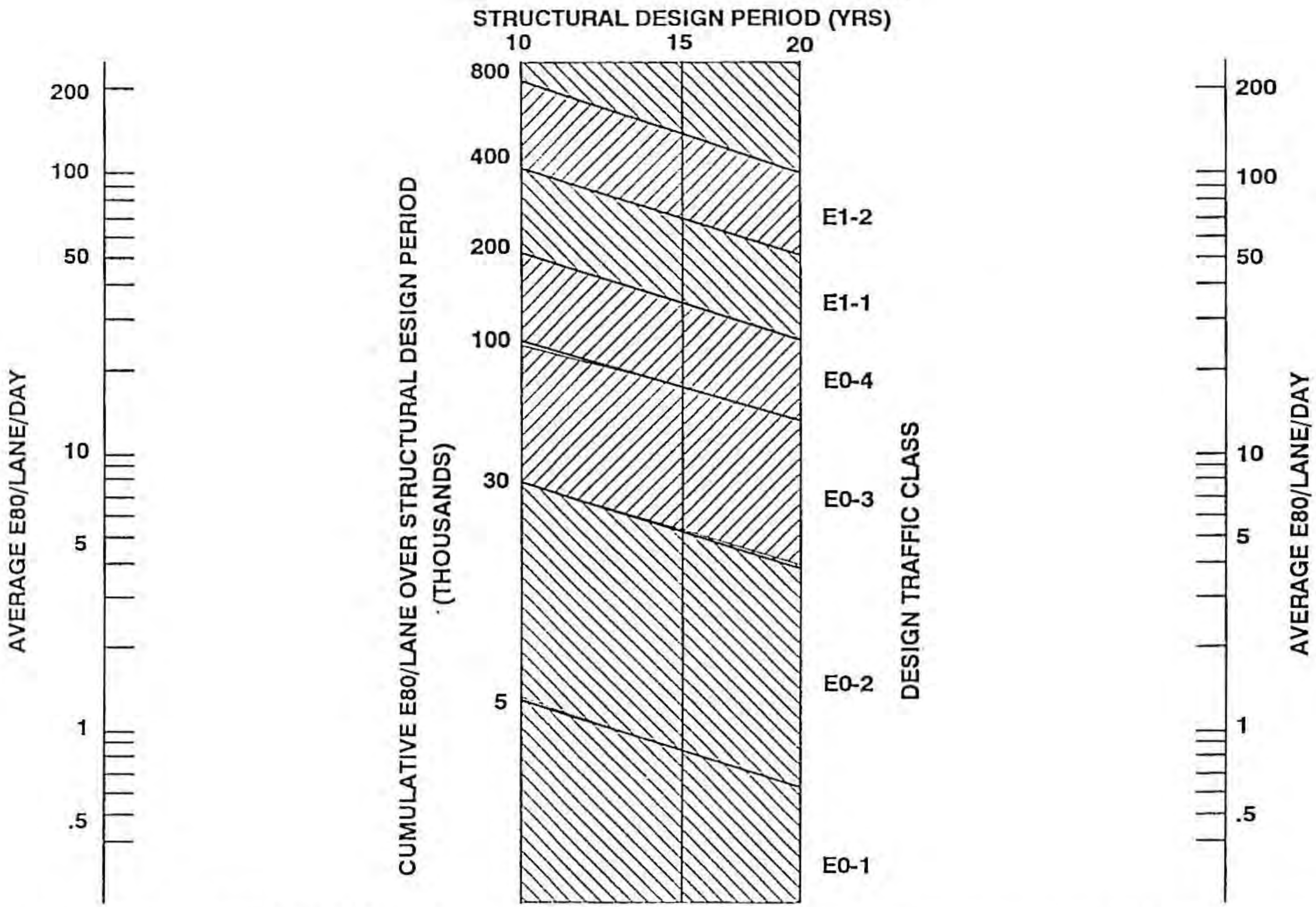


Figure 5-4: NOMOGRAM FOR DETERMINING DESIGN TRAFFIC CLASS FROM THE AVERAGE E80/LANE/DAY AND THE STRUCTURAL DESIGN PERIOD (CSRA, 1985)

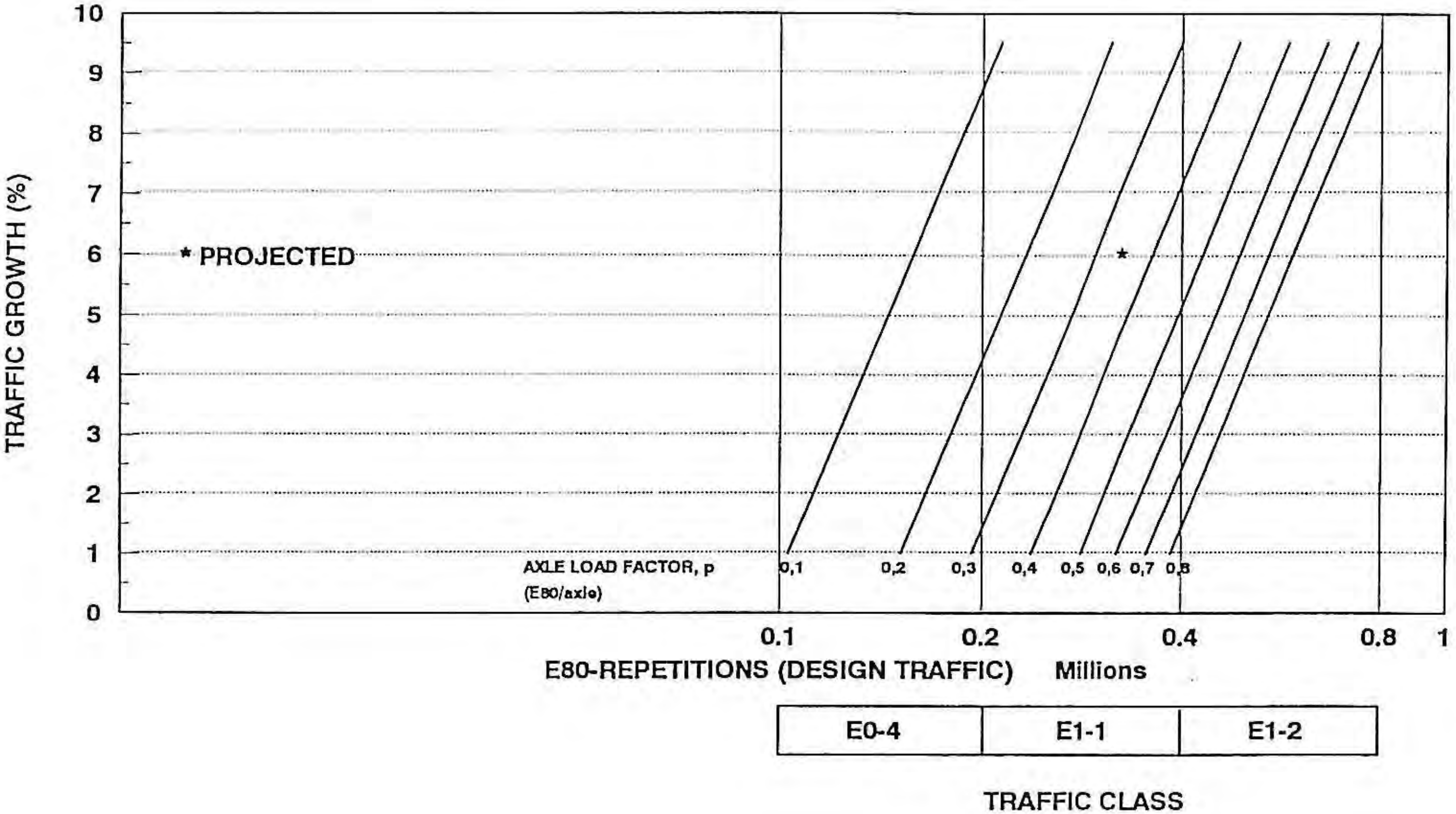


Figure 5-5: PLOT FOR EVALUATING THE SENSITIVITY OF AXLE LOAD FACTOR AND TRAFFIC GROWTH RATE (CSRA, 1991)

If the equivalent damage is insensitive to the load spectrum, then the ratio would be about 100 per cent. The percentage ratio, plotted against the selected damage coefficient d in Figure 5-6, would permit evaluation of this sensitivity.

Usually axle load survey data are graphically presented as in Figure 5-6, showing the sensitivity of the axle load distributions to changes in load equivalency exponent. From experience three types of axle load distributions have been identified based on E80 sensitivity to the exponent:

(a) Light-biased distribution consists mainly of light traffic, with most of the axle masses in the lower load categories with little overloading. Typically such roads carry light rural or urban traffic. Exponents lower than 4 have a marked increase in the total E80s, hence for an exponent of 2 the total E80s could be greater than for an exponent of 4 by 60 per cent[1];

(b) Heavy-biased distribution is found mainly on heavily trafficked roads where a high degree of overloading is likely. The effect of axle loads greater than 80 kN is accentuated by exponents greater than 4. Hence for such situations the percentage ratio could be as high as 160;

(c) Unbiased distribution is an axle load distribution which is relatively insensitive to changes in the load equivalency exponent. This distribution typically occurs on major interurban routes where a relatively low degree of overloading is found.

There are a variety of ways in which the results of sensitivity analyses can be presented. The method of representation is often in a format that the users find most comfortable. Most importantly, however, is the need to consider the influence of variations on the traffic loading figures used in the design.

5.6 The catalogue design method

The catalogue is presented in Table 5-11. A separate catalogue is given in Table 5-12 for concrete pavements from the work of Marais (1989) which makes provision for different concrete and subgrade strengths. Before any of the catalogues are used, all the factors noted in sections 5.1 to 5.5 should be considered. By making sure of the design strategy, design equivalent traffic and pavement type, the designer can choose a pavement structure. Pavement materials as described in Chapter 4 should also be considered. It should be noted that these designs are considered to be of adequate capacity to carry the total design equivalent traffic over the structural design period. Construction constraints on practical layer thicknesses and increments in thicknesses are met. It is assumed that the requirements of the material standards are met. The catalogue may not be applicable when special

1 It is believed by some experienced engineers that serious errors will occur using low n values in the power relationship between load and damage for the entire traffic spectrum, since the apparent damage caused by light vehicles in this situation is disproportionate and increases as the pavement strength (related to n) increases.

conditions arise and other design methods should then be considered. The catalogue can still be used as a guide in such cases. The catalogue does not exclude other possible pavement structures.

The catalogues provide for a range of different structures. The most appropriate structure should be selected from economic analysis based on conditions at the specific project. These conditions normally include aspects such as material availability (bitumen emulsion, cement, lime, natural gravels, crushed stone), maintenance capabilities, construction skills and established procedures. As an example, crushed stone bases normally are expensive and would seldom be used for LVR pavement structures. Cementation or modification of a natural gravel is more common.

Two procedures are given here; the first is for new roads and the second for paving of existing gravel roads (making full use of their in situ strength measured by the DCP and converted to CBR using Figure 5-7)

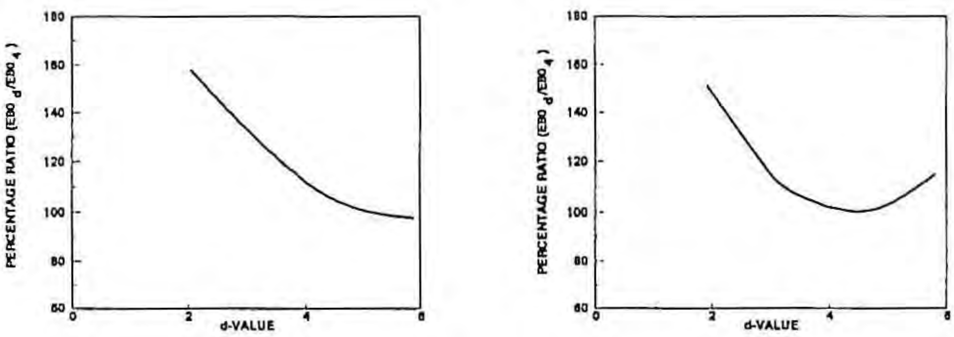
5.6.2 Stepwise procedure for using catalogue

5.6.2.1 New road

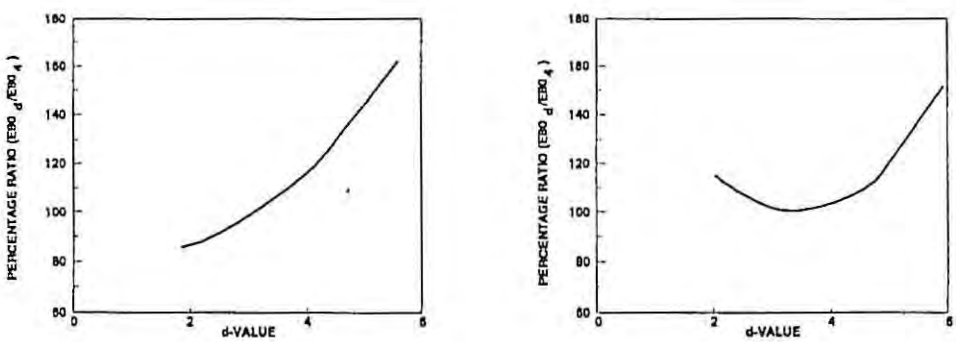
A stepwise procedure for designing a new road with the help of the design catalogue is set out below. This procedure describes a possible sequence of steps which is based on a certain set of assumptions. In practice, deviations from the described procedure are likely because of varying input parameters.

(A) Decide on design strategies

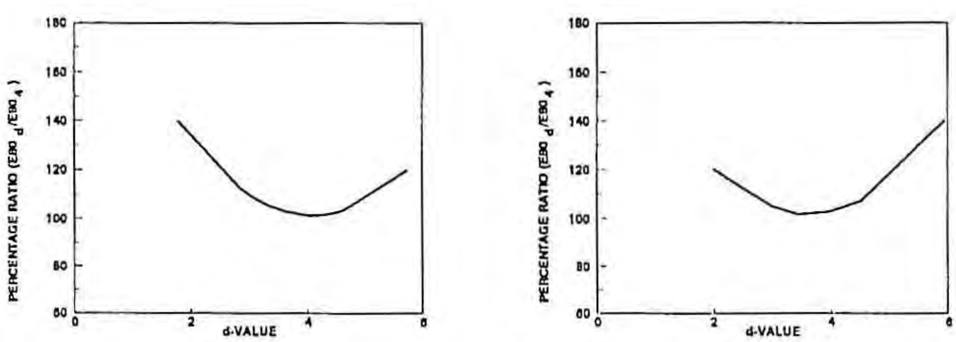
Select analysis period as described in Section 5.3.2.1.
Select structural design period as described in Section 5.3.2.2.



E80 SENSITIVITY OF TYPICAL LIGHT-BIASED AXLE LOAD DISTRIBUTIONS



E80 SENSITIVITY OF TYPICAL HEAVY-BIASED AXLE LOAD DISTRIBUTIONS



E80 SENSITIVITY OF TYPICAL UNBIASED AXLE LOAD DISTRIBUTIONS

Figure 5-6: TWO EXAMPLES OF EACH OF THREE TYPES OF E80 SENSITIVITIES OF AXLE LOAD DISTRIBUTIONS (CSRA,1991)

Table 5-11: CATALOGUE OF PAVEMENT STRUCTURES[2].

TRAFFIC CLASS	TRAFFIC (E80's)	PROPOSED PAVEMENT STRUCTURES						
		GRANULAR/GRANULAR		GRANULAR/ CEMENTED	CEMENTED/ GRANULAR	CEMENTED/ CEMENTED	EMULSION TREATED BASE§	LOW MAINTE-NANCE (Special circumstances&)
		DRY/ MODERATE	WET					
E0-1	< 5000	# 150 G6* 150 G8 150 G9 G10**	150 G5 150 G7 150 G9 G10	150 G5 125 C4 G10	100 C4*** 150 G9 G10	-	-	25 A+ 150 G6 G10
E0-2	5 000 - 30 000	150 G5 150 G7 150 G9 G10	150 G4 150 G6 150 G8 G10	-	100 C4 150 G7 G10	-	-	25 A 150 G6 150 G7 G10
E0-3	30 000 100 000	150 G4 150 G6 150 G8 G10	150 G4 150 G5 150 G6 150 G7 G10	150 G4 125 C4 150 G7 G10	125 C4 150 G5 G10	100 C4ϕ 100 C4 G10	-	25 A 150 G5 150 G9 G10
E0-4	100 000 - 200 000	150 G4 150 G5 150 G8 G10	150 G3 150 G6 150 G9 G10	150 G4 125 C4 150 G7 150 G9 G10	125 C4 150 G5 150 G7 G10	-	-	25 A 150 G4 150 G9 G10
E1-1	200 000 - 400 000	125 G4 150 G5 150 G7 150 G9 G10	150 G3 150 G6 150 G8 G10	125 G2 125 C4 150 G9 G10	125 C4 150 G4 150 G7 G10	100 C4 100 C4 150 G7 150 G9 G10	-	25 A 150 G4 150 G8 G10
E1-2	400 000 - 800 000	125 G2 150 G6 150 G9 G10	125 G2 150 G5 150 G9 G10	150 G2 125 C4 150 G9 G10	S	125 C4 125 C4 150 G7 150 G9 G10	-	25 A 150 G4 150 G5 150 G8 G10

§ Transfer functions for design purposes not yet available.

& Special circumstances refers to situations where an asphalt concrete surfacing is selected because of sever traffic movements (turning actions) or low maintenance capability. It should be noted that many experienced engineers are not in favour of using asphalt on low volume roads because of the high costs involved.

Double surface treatment assumed on all pavement structures unless otherwise indicated.

* Notation - 150 mm layer of G6 quality material.

** **Pavement assumed to be supported by in-situ material having a CBR of not less than 3 (G10) and semi-infinite depth. Layers shown in the catalogue with lower strength than the in-situ subgrade may therefore be omitted provided that adequate strength exists for the total pavement depth.**

*** C4 - cementation of G7, G8 material.

+ 25 mm asphalt (refer to materials chapter for asphalt selection).

ϕ Can be combined into one layer of 200 mm thickness.

S At present reliable calculations of life expectancy cannot be made for this type of pavement structure.

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2 The Transvaal Provincial Administration Roads Branch has developed its own catalogue of pavement designs including three categories for lighter pavement structure. Good performance of low volume roads designed according to this catalogue has been reported.

Table 5-12: THICKNESSES (mm) FOR CONCRETE PAVEMENTS (MARAIS, 1989).

TRAFFIC CLASS	TRAFFIC (E80s)	MR* = 4,4			MR = 4,1			MR = 3,8		
		SUBGRADE STRENGTH								
		< G10	G10	G8	< G10	G10	G8	< G10	G10	G8
E0-1	< 5000	140	140	130	150	140	140	160	150	150
E0-2	5 000 - 30 000	150	150	140	160	150	150	170	160	150
E0-3	30 000 - 100 000	160	150	150	170	160	150	180	170	160
E0-4	100 000 - 200 000	160	160	150	170	160	160	180	170	170
E1-1	200 000 - 400 000	170	160	150	180	170	160	190	180	170
E1-2	400 000 800 000	170	160	160	180	170	170	190	180	170

* Modulus of rupture (MPa)

(B) Calculate design traffic

Obtain the equivalent daily E80s by using one of the following methods:

(a) Published traffic surveys conducted on nearby roads:

(i) Divide the traffic load spectrum into the load categories given in Table 5-7;

(ii) Multiply the number of axles in each traffic group with the equivalency factor from Table 5-7 and sum the products to get the equivalent daily traffic (Eq. 5.2);

(iii) Where the traffic load spectrum is not available, use Tables 5-4, 5-5 or 5-6 to determine the equivalent daily traffic in terms of E80s;

(b) Conduct traffic survey on nearby roads using visual observation methods (see Section 5.5.2.3).

Determine the equivalent daily traffic in the initial design year:

(a) Estimate the E80 growth rate (Section 5.5.3.2);

(b) Project the equivalent daily traffic to the initial design year by multiplying the equivalent design traffic with the E80 growth factor from Table 5-8 to get the daily equivalent traffic in the initial year (or use Equations 5.3 and 5.4);

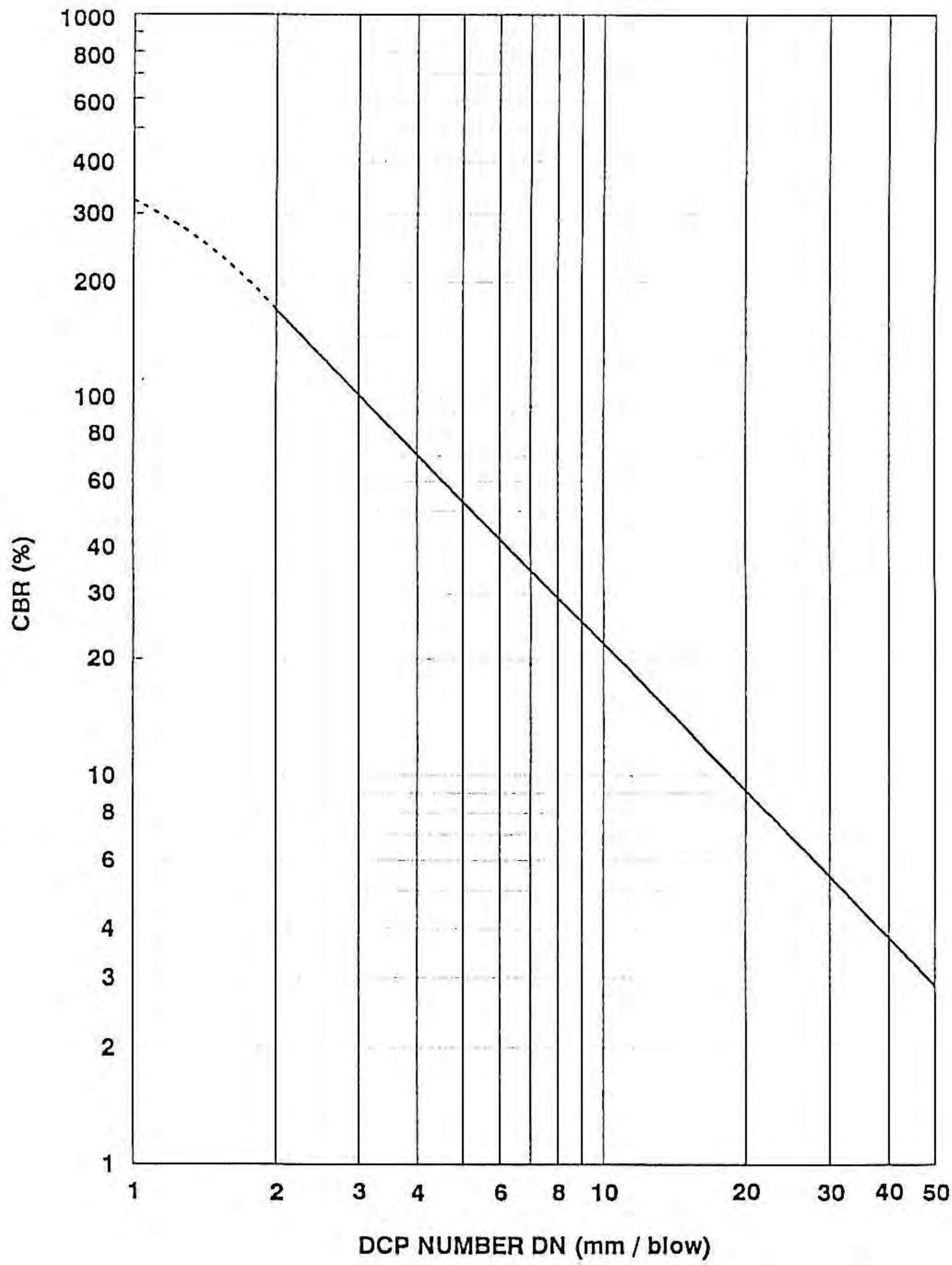

Figure 5-7: THE RELATIONSHIP BETWEEN DCP NUMBER DN AND CBR (KLEYN,1984)

Determine the cumulative equivalent E80s (design traffic):

(a) Determine the cumulative equivalent E80s (design traffic) by multiplying the daily equivalent traffic in the initial year by the cumulative E80 growth factor from Table 5-9 (or use Equation 5.5);

(b) Multiply the cumulative equivalent E80s (design traffic) by the appropriate lane distribution factor (see Section 5.5.3.4);

(c) The nomogram in Figure 5-3 can be used instead of step (a) above. The equivalent daily traffic in the initial design year must be multiplied by the lane distribution factor before the nomogram is used.

(C) Determine design traffic class according to Table 5-3.

(D) Conduct sensitivity analysis as described in Section 5.5.4. to confirm design traffic class.

(E) Consider material availability and cost

(F) Do a centre line soil survey to determine the subgrade strength.

(G) Select pavement structures from the design catalogue that are appropriate for the available materials and selected design strategy. It may be necessary to reconsider the design strategy at this stage. Remember that layers shown in the catalogue with lower strength than the in-situ subgrade strength may be omitted provided that adequate strength exists for the total pavement depth.

(H) Do an economic analysis as described in Section 5.8 to determine the most economical pavement type in terms of the present worth of cost.

5.6.2.2 Existing gravel road with optimization of in situ strength

The existing gravel road strength should be utilised in the upgrading of an existing gravel road to a paved road (just as the existing pavement strength should be utilised in the rehabilitation of an existing paved road, which is discussed in more detail in chapter 11).

This is done by using layers of the existing pavement as layers of the new pavement. For example, if an existing gravel low volume road had a wearing course of G4 material and a subbase of G5 material, then those layers could be used as the basecourse and subbase of the paved road, and they would make up probably a stronger structure than the catalogue calls for. In this example, even if the catalogue called for a new pavement of a G5 basecourse, a G8 subbase and a G10 subgrade, it would pointless to import these three layers and place them on top of the existing G4 and G5 material. One would only be building a weak pavement on top of a strong one. Indeed there are parts of South Africa where the in situ subgrade is strong enough to be classified as a G5, and so the bitumen surfacing could be laid on top of this without further layers being imported (although attention would obviously be needed to levels and smoothness).

To utilise the existing gravel road strength, the materials in the pavement layers need to be tested for their actual bearing capacity (using a DCP as discussed in Kleyn, 1984), and a comparison made between their actual and theoretical bearing capacity. For example a G5 material may have been laid without proper compaction and it will only perform like a G6 material. Alternatively a G6 material may have been so compacted with traffic and time that it can perform like a G5 material. The effect of traffic compaction on existing gravel roads is discussed in chapter 8 on construction.

Design process

In the design process, the calculation of the design E80s, catalogue design and sensitivity analysis is done in accordance with Section 5.6.2.1 above.

The materials in the catalogue are classified by their soaked bearing strength, and the existing pavement materials need to be classified in terms of soaked CBR to be related to the catalogue. The strength of a material at different moisture contents and densities can vary significantly from this, and in general the drier the compacted material, the higher the strength (Emery, 1992).

The preferable method of determining the soaked CBRs of the existing gravel road materials is to take many samples and test them in the laboratory. At the same time, field density tests of all layers should be performed to ensure that their compaction is adequate. This can involve considerable testing however, and a simpler, although less accurate, method is to use the DCP to do most of the testing in conjunction with a limited number of laboratory soaked CBR tests. Then the design can be based on soaked CBRs estimated from the relationships between field DCP-CBR and soaked CBR (Table 5-13 for roads which are presently gravel).

If the field DCP-CBRs estimated from the laboratory results are less than those actually found in the field, it is indicative that the existing gravel road has been well compacted (by traffic), and is suitable for incorporation in the design. If however the actual field DCP-CBRs are less than estimated from the laboratory, it indicates a lack of compaction and the existing gravel layer should be ripped and recompacted. Alternatively if sufficient field density tests have been performed, then these can be compared to specified Mod AASHTO densities to check the compaction.

Current research is aimed at relating field and laboratory strength. In order to ensure cost effective design of low volume pavements, the following simple procedure is provided to optimise on the in situ strength of gravel roads. This will be updated as future results become available.

USING EXISTING GRAVEL ROADS IN DESIGN

STEP 1 DO TESTING ALONG THE ROAD

DCP testing is performed along the length of road. The frequency of tests should generally be in accordance with the standards here, but the visual inspection may indicate adjustments to the frequency. If the road is very uniform the frequency can be reduced, and if it is variable then it should be increased. The basic frequency should be:

- test at the rate of 5 DCP tests per kilometre, with the tests staggered as outer wheeltrack/inner wheeltrack one side, outer wheeltrack/inner wheeltrack other side, centreline, etc;
- perform an additional test at every significant location picked up in the visual survey, such as particular failure areas;
- ensure that at least 8 DCP tests are performed per likely uniform section to provide adequate data for the statistical analysis.

Convert the DCP results to DCP-CBR values for each 150mm layer using Figure 5-7. In cases where the DCP-CBR within the layer changes significantly a weighted average DCP-CBR for the layer should be calculated:

0-110mm DCP-CBR = 75

110mm - 150mm DCP-CBR = 60

Weighted DCP-CBR = (110*75 + 40*60)/150 = 71

Take at least 2 samples per kilometre to check laboratory soaked CBR, Atterbergs and in situ moisture content of each layer. Test in situ density whenever possible.

STEP 2 DIVIDE ROAD INTO UNIFORM SECTIONS

The results of the DCP testing, together with the visual assessment will enable the length of road to be divided in relatively uniform sections for assessing the residual life and capacity.

STEP 3 CALCULATE THE REPRESENTATIVE DCP-CBR VALUE FOR EACH SECTION

The representative DCP-CBR value for each 150mm layer of the pavement in each section is calculated statistically from the individual DCP-CBR results to provide a safety margin against the variability of material within the section. A normal distribution of data is assumed and the Student's T distribution at the 80% level is used:

$$\text{Representative}_{DCP\text{-}CBR} = \text{mean}_{DCP\text{-}CBR} - .9 * (\text{standard deviation}_{DCP\text{-}CBR}) \quad \ldots\ldots\ (5.3)$$

Example The DCP results in the top 150mm of a section were as follows:

DCP-CBR: 125, 143, 120, 100, 145, 115, 140, 135

Mean (average) = 127,9 Standard deviation = 15,7

$$\text{Representative}_{DCP\text{-}CBR} = \text{mean}_{DCP\text{-}CBR} - .9 * (\text{standard deviation}_{DCP\text{-}CBR})$$
$$= 127{,}9 - .9 * 15{,}7 = 114$$

Note that equation 5.3 is using a one-tailed T-distribution for 8 samples and is reasonably robust for sample sizes from 5 to 30.

STEP 4 CONVERT FIELD DCP-CBR TO MATERIAL CLASSIFICATION

If the DCP has been used as the primary investigation tool, the field DCP-CBR results should be converted to material classification/soaked CBR using Table 5-13 and the moisture state of the gravel road at the time of DCP testing.

The soaked CBR is only one element of the materials classification, and other elements such as Atterberg limits need to be considered to assess which materials classification the in situ materials fit in. The limits (and their possible relaxations) are given further on.

Table 5-13 APPROXIMATE RELATIONSHIP BETWEEN SOAKED CBR AND FIELD DCP-CBR FOR A GRAVEL ROAD

Material classification	Soaked CBR	APPROXIMATE FIELD DCP-CBR : GRAVEL ROAD					
		Subgrade		Wearing course			
		wet climate ($Im \geq 0$)	dry climate ($Im < 0$)	dry state	moderately dry state	damp state	wet state
G4	80			318	228	164	117
G5	45			244	175	126	90
G6	25	59	65	186	134	96	69
G7	15	45	50	147	106	76	54
G8	10	38	43				
G9	7	33	37				
G10	3	20	24				

Notes 1 The inter-relationship between soaked CBR and field DCP-CBR is very approximate due to the variability of moisture contents, materials, test methods, and densities. It assumes that the density relates approximately to the field density expected for that layer. More research is needed to give confidence to this relationship.

2 The moisture contents that this table are based on are estimated moisture contents, based on various field studies and experience; they can vary in practice from the values assumed here. For the wearing course they are (expressed as the ratio of field moisture content to Mod AASHTO optimum moisture content): dry state = 0,25; moderately dry = 0,5; damp = 0,75; wet = 1,0. For the subgrade, they are: dry = 0,9; wet = 0,98.

3 This table has been developed from Table 22 and equation 36 of Emery (1992)

At the same time, the field density of the existing gravel road should be checked against specifications for a new paved road. If there are sufficient field density test results, then these can be used. Alternatively the laboratory soaked CBRs can be checked against the estimated soaked CBRs from the DCP-CBRs. If the estimated soaked CBRs from the DCP are less than the laboratory soaked CBRs, then it indicates that the field density is low, and that the layer should be ripped and recompacted or its classification reduced.

Finally any other materials problems such as oversize stone in the basecourse should be addressed.

STEP 5 COMPARE THE EXISTING GRAVEL ROAD TO THE REQUIRED CATALOGUE DESIGN

The actual pavement structure (expressed in terms of materials classification estimated in the previous step) is compared to the required catalogue design for the design traffic (which has been found above). This will indicate what new layers, if any, are required, and what layers need to be reworked or stabilised to improve their classification.

STEP 6 MATERIAL AVAILABILITY

If additional layers are required, materials which meet the requirements have to be located (desirably close by). In the case of suitable materials not being locally available, the decision to modify/stabilise local materials or to import materials is then made on economic grounds.

5.7 Choice of surfacing type

Part of the structural design of the road, and the rehabilitation, includes the choice of surfacing type. For low volume roads the surfacing is an important determinant of the performance of the road and this choice must be made carefully in response to a number of factors. In some situations a better overall pavement design will be had by using a stronger and/or thicker surfacing in conjunction with a less capable pavement structure. It should be remembered that the pavement structure must be able to carry the load and that the purpose of the surfacing is mainly to protect the pavement.

The choice of appropriate surfacing is made firstly on performance grounds, and then on financial or lifecycle cost grounds. (This section has largely been taken from SABITA Manual 10 which is based on the work by Emery et al (1991).)

5.7.1 Performance

The performance is determined by all of the environment, maintenance capability, and gradient. These are presented in Tables 5-14 to 5-16, and the restrictions on choice are progressive and sequential. Thus a restriction in any one table is sufficient to limit the choice of surfacing.

The "environment" that the road is in plays a major role in the choice of surfacing. Environment in this case includes all of climate, surroundings, topography, institutional capability,

Table 5-14: CHOICE OF BITUMINOUS SURFACING FOR RURAL ROADS BY ENVIRONMENT

ENVIRONMENT	SURFACING RECOMMENDATION
First world - high pavement standards[a]	Any[3]
First world - lower pavement standards[b]	Any: caution with thin surfacings because they need timeous maintenance; refer to maintenance table
Wet/hilly[c]	Refer to maintenance and gradient tables
Third world[d]	Refer to maintenance and gradient tables

From SABITA, 1992

Notes: a Busier roads, well constructed surfacing, good pavement structure. Typically large road authority such as Department of Transport.

b Light pavement structure, not mountainous. Typically small road authority with restricted budget, such as smaller regional or divisional councils

3 The use of dust palliatives and single seals or sand seals without prime is severely criticised. For seals on newly constructed roads it is emphasised that at least two sprays of binder should be applied.

c Mountainous, Thornthwaite's climatic moisture index $Im \geq 0$. Typically the wet areas of natal, Eastern Transvaal, and Eastern Cape.

d Generally applicable more to populated urban areas; for rural roads, can apply to roads bounded by settlement as in TBVC and SGT.

General: In this and subsequent tables, where the recommendation is 'Any', the footnote in this page must be taken into account.

Maintenance

The maintenance capability of the road authority has a major effect on the performance of the surfacing. Light seals can give good performance provided they receive adequate routine maintenance. Conversely if there is no maintenance capability, then only those surfacings which are inherently tough can survive. The maintenance capability varies widely as the institutional capabilities of the authorities vary, and maintenance must be considered as part of the stresses operating on the surfacing and the appropriate surfacing selected to cope with that. The reasons for the variation include the level of expertise in the road authority, the funds availability, security problems (risk, riots etc), and the quality of personnel.

The maintenance needed on surfacings falls into three groups. All three must be present timeously for there to be adequate maintenance:

a: major maintenance - reseal or fog spray timeously before structural deterioration occurs.

b: routine maintenance - potholes, patches, edge breaks, crack sealing.

c: cleaning (adjacent to settlements) and soil wash cleaning.

Table 5-15: CHOICE OF SURFACING FOR RURAL LOW VOLUME ROADS BY MAINTENANCE CAPABILITY

MAINTENANCE CAPABILITY	DEFINITION	SURFACING RECOMMENDATION
High	Can perform any type of maintenance	any
Medium	Routine maintenance, patching and crack sealing on a regular basis. Typically, no maintenance management system[b]	asphalt, Cape Seal, slurry[a] double seal, single seal
Low	Patching done irregularly, no committed team, no inspection system	asphalt, Cape Seal, thick slurry, double seal[c]
None	No maintenance	asphalt

Notes a: thin slurries can lead to construction problems

b: it is not essential to have a maintenance management system, but its presence indicates a certain level of capability

c: this is sensitive to construction problems

General: A properly constructed sand seal on a smooth, primed base can perform very well.

A double seal refers to any operation where at least two binder sprays and two layers of aggregate are applied. A 13,2 mm stone and sand blinding with a split application of binder is therefore considered to be a double seal.

Gradient

Gradient limits are important to limit the damage caused by water running along the surfacing, particularly for roads with kerbs such as are found in hilly areas. Gradient limits also apply to limit damage caused by shoving.

Shoving is where the bituminous surfacing slips across the basecourse, and for this reason the shoving limits are applicable only to an initial seal. It is much less common to find shoving of a reseal and in such cases there is either a built-in construction defect i.e. (lack of tack coat) or the underlying surfacing is already shoving and the reseal merely adds to the problem. The limit to guard against shoving depends partially on the basecourse: a rough basecourse is more resistant to shoving than a smooth one. A stabilized basecourse is sensitive to shoving, and this is accentuated by small radius curves and heavy vehicles. A basecourse with a thin layer of fines at the top may lead to shoving.

Water damage is the other problem with steep gradients where water flowing over the bituminous surfacing causes damage. There is a maximum water velocity for each type of surfacing before the surfacing gets damaged due to stone plucking and scour, and here gradient is used as an indication of water velocity to give limits for surfacings.

Table 5-16: CHOICE OF SURFACING RURAL LOW VOLUME ROADS FOR GRADIENT

GRADIENT	SURFACING RECOMMENDATION FOR INITIAL SURFACING
< 6%	any surfacing
6 - 8%	asphalt, Cape Seal, thick slurry[a], double seal[b], single seal[b], sand seal[b]
8 - 12%	asphalt, Cape Seal, double seal[b], single seal[ab], sand seal[ab]
12 - 16%	asphalt, Cape Seal[ab], double seal[ab]
> 16%	concrete block/concrete

Notes: a: not on stabilised basecourse

b: not if water flow is being channelled by kerbs or berms

General: geotextile-reinforced bituminous surfacing is recommended only at gradients up to 6% at present: further research is currently underway on steeper gradients

Intersections

Where the road is subject to turning vehicles (such as mine or industrial entrances, and intersections), thin seals are generally not recommended. The recommendations to limit damage by turning vehicles are given in Table 5-17.

Table 5-17: CHOICE OF SURFACING FOR LOW VOLUME RURAL ROADS FOR INTERSECTIONS AND OTHER AREAS SUBJECT TO TURNING VEHICLES

Location	Surfacing recommendation[a]
Very occasional heavy vehicles	Any
Some heavy vehicles	Cape Seal, asphalt, double stone seal with fogspray and sand blinding
Many heavy vehicles	Asphalt, concrete/concrete blocks, epoxy asphalt

Note (a): geotextile reinforced surfacings not recommended at intersections.

5.7.2 Lifecycle cost choice

Once the appropriate surfacing types have been selected from the performance viewpoint, their life cycle cost is determined to enable the most cost-effective surfacing type to be chosen. The decision process is to choose the surfacing with the lowest life cycle cost and compare its risk profile, construction cost and life with the other appropriate surfacings which are close to it in terms of lifecycle cost. The final selection is then made in accordance with all these factors.

The lifecycle cost can be found from the construction cost and the life of the surfacing. The actual values of each can be used or reference can be made to Tables 5-18 and 11-5. Since the surfacings have been chosen in accordance with section 5.10.1 and are all appropriate, it can be assumed that the maintenance costs are similar and this can be disregarded; accordingly lifecycle costs are expressed here as "comparative lifecycle costs".

Lifecycle cost

Once the likely surfacing life[4] and the construction cost of any particular surfacing have been determined, the comparative lifecycle cost can be found from Figures 5-8 and 5-9. These figures allow your actual costs to be used (up to a limit of R25/m^2) which therefore takes account of inflation and makes the figures valid for many years to come. They show, respectively, comparative lifecycle costs discounted at 8% and not-discounted (discounted at 0%). Whilst economists and funding agencies generally recommend using discounted costs, some engineers prefer to use costs which are not discounted.

Table 5-18: SURFACING CONSTRUCTION COST (1991 VALUES)

SURFACING	Cost R/m^2 in 1991		
	Low	Medium	High
Dust palliative	1.74	2.22	2.70
Sand seal (2-3mm thick)	1.38	1.90	2.42
Sand seal (10mm thick)	3.20	4.13	5.07
Slurry (6mm thick)	1.53	2.03	2.53
Slurry (10mm thick)	2.71	3.42	4.13
Asphalt (25mm thick)	7.88	8.70	9.52
Asphalt (30mm thick)	9.41	10.36	11.31
Single seal (10mm stone)	1.41	1.92	2.43
Double seal (13mm/6mm)	3.28	4.17	5.06
Double seal (13mm stone/sand)	2.82	3.61	4.40
Cape Seal (19mm + slurry)	4.13	5.15	6.17
add for Prime	0.80	0.94	1.09

Note: a. Prime is not included in these individual costs but is recommended for all initial surfacings except dust palliative.

[4] Refer to Chapter 11 (Table 11-6) for expected surfacing lives.

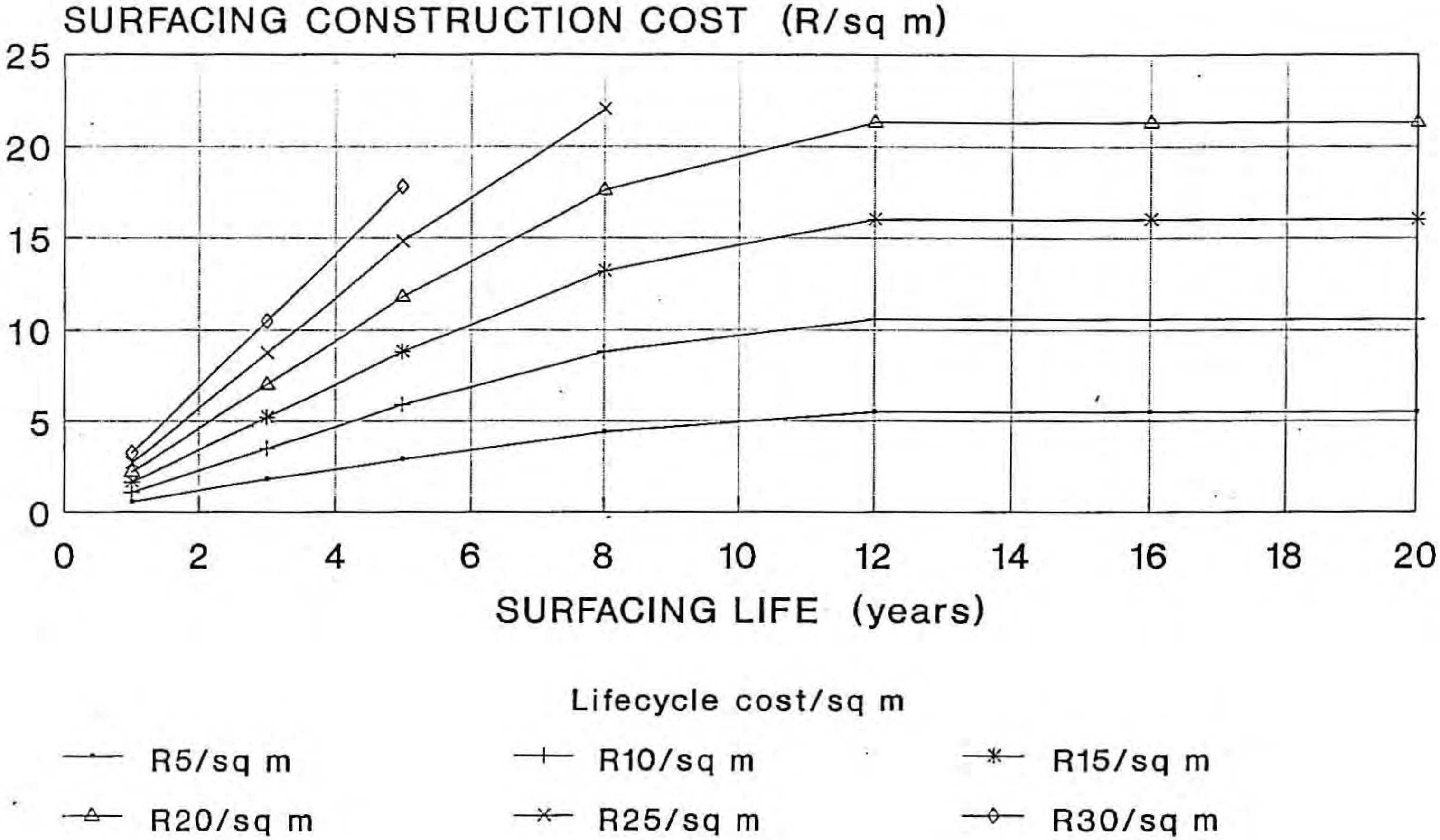

Figure 5-8: LIFE CYCLE COST - 8% DISCOUNT RATE

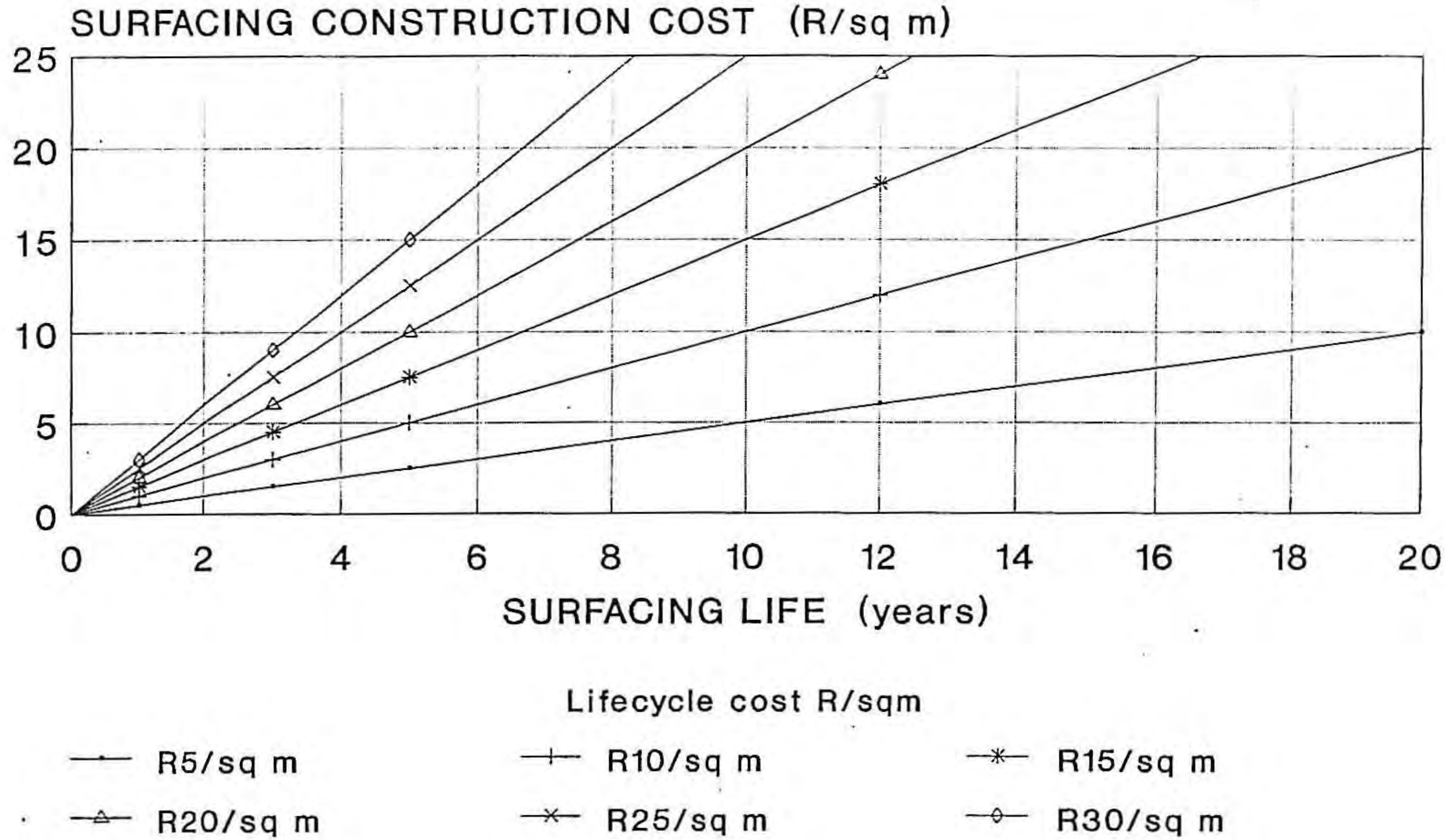

Figure 5-9: **LIFE CYCLE COST - - NO DISCOUNT**

In the figures, the comparative life cycle cost is found from the contours as rand per square metre, and is found by entering the y-axis with construction cost which is actual total cost of surfacing in Rand per square metre including prime (if applicable), engineer's fees, profit, P & Gs; and the X-axis with life of surfacing which is life before reseal is needed according to normal engineering standards.

The comparative life cycle cost given in the figures is based on a 10 year analysis period, includes salvage value at the end of the 10 years, and allows for reseals of the same surfacing type (without prime) if required. Maintenance cost is not included because it is assumed to be equal for all appropriate surfacing alternatives.

EXAMPLE

There is an existing gravel road leading to an established Transvaal township. The existing pavement structure is reasonable, but the gravel wearing course is poorly maintained. Traffic is 300 vehicles per day and growing at 5% per annum. Gradients average 3 - 7%, with occasional steeper sections. Little commercial traffic is expected, and bus traffic is also limited. It is intended to pave the road.

SOLUTION

Step 1: Appropriate surfacings

The pavement design will call for stabilising the existing wearing course, and putting a bituminous surfacing on top. Since the maintenance capability is low, the appropriate surfacings are asphalt, Cape Seal thick slurry and a double seal. (Table 5-15). A check on gradient (Table 5-16), eliminates the thick slurry because of the stabilised basecourse.

Step 2 Choose most cost effective surfacing

Determine surfacing life

No historical information is available about surfacing lives. It is a third world environment, normal pavement, reasonable quality control during construction, and although the drainage is adequate at construction the drains are expected to block up soon. From Table 11-5, it is assessed as poor conditions. The surfacing life is: asphalt 14 years, Cape Seal and double seal 8.

Determine surfacing costs

The latest price from contractors is R9,62/m^2 for 25mm asphalt; R6,07/m^2 for Cape Seal and R5,50/m^2 for a double seal.

Find comparative life cycle cost

On a discounted cost basis, From Figure 5-8:

Surfacing	*Cost*	*Life*	*Life cycle cost*
	R/m²	*years*	*disc. 8% R/m²*
Asphalt	*9,62*	*14*	*8,89*
Cape Seal	*6,07*	*7*	*7,86*
Double seal	*5,50*	*7*	*7,14*

Decision: double seal

5.8 Economic analysis

5.8.1 New roads

Alternative pavement designs should be compared with regard to life-cycle costs. The life-cycle cost analysis should be regarded as an aid to decision-making. It does not necessarily include all the factors leading to a decision and should therefore not override all other considerations. The main economic factors which determine the life-cycle cost of a facility are analysis period, structural design period, construction cost, maintenance costs, road user costs, salvage value at the end of the analysis period and real discount rate. The road user costs comprise vehicle operating, time and accident costs. These costs are assumed to be the same for the alternatives under consideration and do therefore not enter the life-cycle cost calculation. This assumption is valid as long as the pavements compared with one another are in the same road category. Road user costs are not considered further in this section.

The choice of the analysis and structural design periods will influence the cost of a road, but as was shown in Section 5.3, this is not necessarily purely an economic decision.

The construction cost should be estimated from current contract rates for similar projects. Maintenance costs should include costs of maintaining adequate surfacing integrity (e.g. resealing). The salvage value at the end of the analysis period can make a contribution towards the next pavement. However, geometric factors such as minor improvements to vertical and horizontal alignment and possible relocation of drainage facilities, make the estimation of the salvage value difficult.

The possibility of constructing a one lane facility (i.e. one lane carrying traffic in both directions), should also be investigated as part of the economic analysis for ADT's of below 150 vehicles per day. No general guidelines can be given and calculations should be done for each case.

5.8.1.1 Present worth of cost

The total life-cycle cost of a project is the construction cost plus maintenance costs, minus the salvage value, discounted where applicable. The total life-cycle cost can be expressed in a number of ways e.g. internal rate of return, net present value and present worth of costs, to name a few. The present worth of costs (PWOC) has been adopted for use in this document.

The PWOC is calculated as follows:

$$PWOC = C + M_1\,(1+r)^{-x_1} + ... + M_j\,(1+r)^{-x_j} - S\,(1+r)^{-z} \quad \ldots\ldots\ldots\ldots \quad (5.6)$$

where PWOC = present worth of costs

C	=	present cost of initial construction
M_j	=	cost of the j th maintenance measure expressed in terms of current costs
r	=	real discount rate
x_j	=	number of years from present to the j th maintenance measure within the analysis period
z	=	analysis period
S	=	salvage value of pavement at end of analysis period expressed in terms of current costs.

If the difference on PWOC between two designs is 10 per cent or less, it is assumed to be insignificant. The PWOC of the two designs are then considered to be the same.

Electronic spreadsheets are ideal for performing PWOC calculations.

5.8.1.2 Construction costs (C)

The total construction cost or the construction cost per metre squared can be calculated for different pavement options. Factors such as the availability of natural or local commercial materials, their expected trends in costs and the conservation of aggregates in certain areas, should also be considered. Practical aspects such as speed of construction and the need to foster development of alternative pavement technologies, also need consideration.

5.8.1.3 Real discount rate (r)

When a PWOC analysis is done, a real discount rate must be selected to express future expenditure in terms of present day values. This discount rate should correspond to the rate generally used in the public sector. This is currently about 10 per cent in real terms and this value is therefore recommended for general use. A sensitivity analysis could determine the importance of the value for the discount rate.

5.8.1.4 Future maintenance (M_j)

Maintenance and rehabilitation are discussed in Chapters 10 and 11 of this document. Suffice it here to say that there is a relation between the type of pavement and the maintenance that might be required during the life of the pavement. When different pavement types are compared on the basis of cost, these future maintenance costs should be included in the analysis to ensure that a reasonable comparison is made.

5.8.1.5 Salvage value (S)

It is difficult to assess the salvage value of the pavement at the end of the period under consideration. The existing pavement layers may have a salvage value if the road is to remain on the same location. However, the salvage value could be little or zero if the road is to be abandoned. The assessment of salvage value can be approached in a number of ways depending on the method employed to rehabilitate or reconstruct the pavement:

(a) Where the existing pavement is left in position and an overlay (bituminous, gravel or cemented) is constructed, the salvage value of the pavement would be the saving in the cost of constructing an overlay as opposed to constructing a new pavement to a standard equal to that of the existing pavement with the overlay. This can be termed the residual structural value.

(b) Where the material in the existing pavement is taken up and recycled for use in the construction of a new pavement, the salvage value of the recycled layers would be the difference in cost between furnishing new materials and the cost of taking up and recycling the old materials. This salvage value could be termed the recycling value.

(c) In some cases the procedure followed could be a combination of the above and the salvage value would have to be calculated accordingly.

The salvage value of individual layers of the pavement may differ considerably, from estimates as high as 70 per cent to possibly as low as 10 per cent. The residual salvage values of asphalt and gravel are generally high. The salvage value of the whole pavement would be the sum of the salvage values of the individual layers. In absence of better information, a salvage value of 30 per cent of initial construction cost is recommended.

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6 ROADSIDE FURNITURE

6.1 Introduction

Roadside furniture includes fixed objects such as barriers, utility poles, sign supports and other roadside elements usually found in the road reserve. The following elements are discussed in this chapter. It includes:

- Shy line
- Embankments and cut slopes
- Guardrails
- Bridge railings
- Drainage structures
- Trees, utility poles and other roadside obstacles
- Road signs
- Road markings
- Delineation

6.2 Clear zone concept

Single-vehicle collisions constitute more than 50 per cent of all collisions occurring on rural roads. To promote road safety an area clear of hazardous obstacles such as unyielding sign supports and utility poles, non-traversable drainage structures, and steep slopes should be provided. If an obstruction cannot be removed it should be

- placed as far away as practical from the roadway,
- redesigned to be traversable,
- shielded with a guardrail, or
- delineated to make it readily visible.

The distance from the edge of the travelled way, beyond which a roadside object will not be perceived as hazardous and cause a reduction in speed or change in position of the vehicle on the roadway, is called the shy line offset (Table 6-1). Roadside furniture should be placed beyond this offset.

To save costs site-specific alternative safety treatments should be examined to determine the most cost-effective strategy.

Table 6-1: **SHY LINE OFFSET VALUES AND FLARE RATES**

Design speed (km/h)	Shy line offset (m)	Flare rate for barrier inside Shy Line	Flare rate for barrier beyond Shy Line	
60	1,5	17:1	9:1*	11:1**
80	2,0	21:1	11:1*	14:1**
100	2,5	26:1	13:1*	17:1**

* Maximum flare rate for semi-rigid systems
** Maximum flare rate for rigid systems

6.3 Embankments and cut slopes (cross sections)

For adequate safety it is desirable to provide an unencumbered roadside recovery area that is as wide as practical. The recovery area or clear zone distance should be relatively smooth, traversable with shallow slopes. Slopes not steeper than 1:4 are required for safety. Embankments between 3:1 and 4:1 are considered non-recoverable. Slopes steeper than 3:1 are critical. Table 6-2 gives clear zone distances for design speed and design ADT.

Table 6-2: **CLEAR ZONE DISTANCE FOR DESIGN SPEED AND DESIGN ADT (IN METRE FROM SHOULDER OR EDGE OF SURFACE)**

Design Speed (km/h)	EVU's per day in design year	Fill slopes		Cut slopes		
		1:6 or flatter	1:5 to 1:4	1:3	1:4 to 1:5	1:6 or flatter
60	1 - 999	2,0	2,0	2,0	2,0	2,0
	1000 - 1999	3,0	3,7	3,0	3,0	3,0
80	1 - 999	3,0	3,7	2,5	2,5	2,5
	1000 - 1999	3,7	5,0	3,0	3,7	4,3
100	1 - 999	3,7	6,2	3,0	3,7	4,3
	1000 - 1999	6,2	8,0	3,7	4,9	6,2

6.4 Guardrails

There is no real substitute for flat slopes and clear verges. Guardrails are a compromise between the conflicting demands of construction costs and safety. A guardrail is in itself a hazard and should be designed and installed to constitute a lesser hazard than that which it is intended to protect. It should absorb the impact with as little overall severity as possible. At specific sites on low volume and/or low speed roads where the probability of a collision is low, the obstacle may be left unshielded if the option is more cost-effective than the installation of a barrier.

Studies have shown that guardrails are struck more often than any other roadside object during single-vehicle collisions. Guardrails should be placed sufficiently far from the edge of the roadway so as not to cause a hazard to vehicles on the roadway, nor reduce the effective width of the road. A guardrail should be placed beyond the shy line offset, particularly for relatively short installations. For long, continuous runs of railing, this offset distance is not so critical, especially if the barrier is first introduced beyond the shy line and gradually transitioned nearer to the roadway.

The leading and trailing ends of a guardrail installation are its most hazardous features and special attention should be paid to end treatment. To reduce the likelihood of the ends of the guardrail being struck, the ends have to be flared away from the road or buried, or both. The shy line offset values and flare rates for different design speeds are given in Table 6-1. If the guardrail installation is of insufficient length, ie less than 30m, with too few posts, it cannot develop full strength in the longitudinal direction and will fail prematurely.

Figures 6-1(a) and 6-1(b) shows when a guardrail is warranted and the application of the equal severity curve.

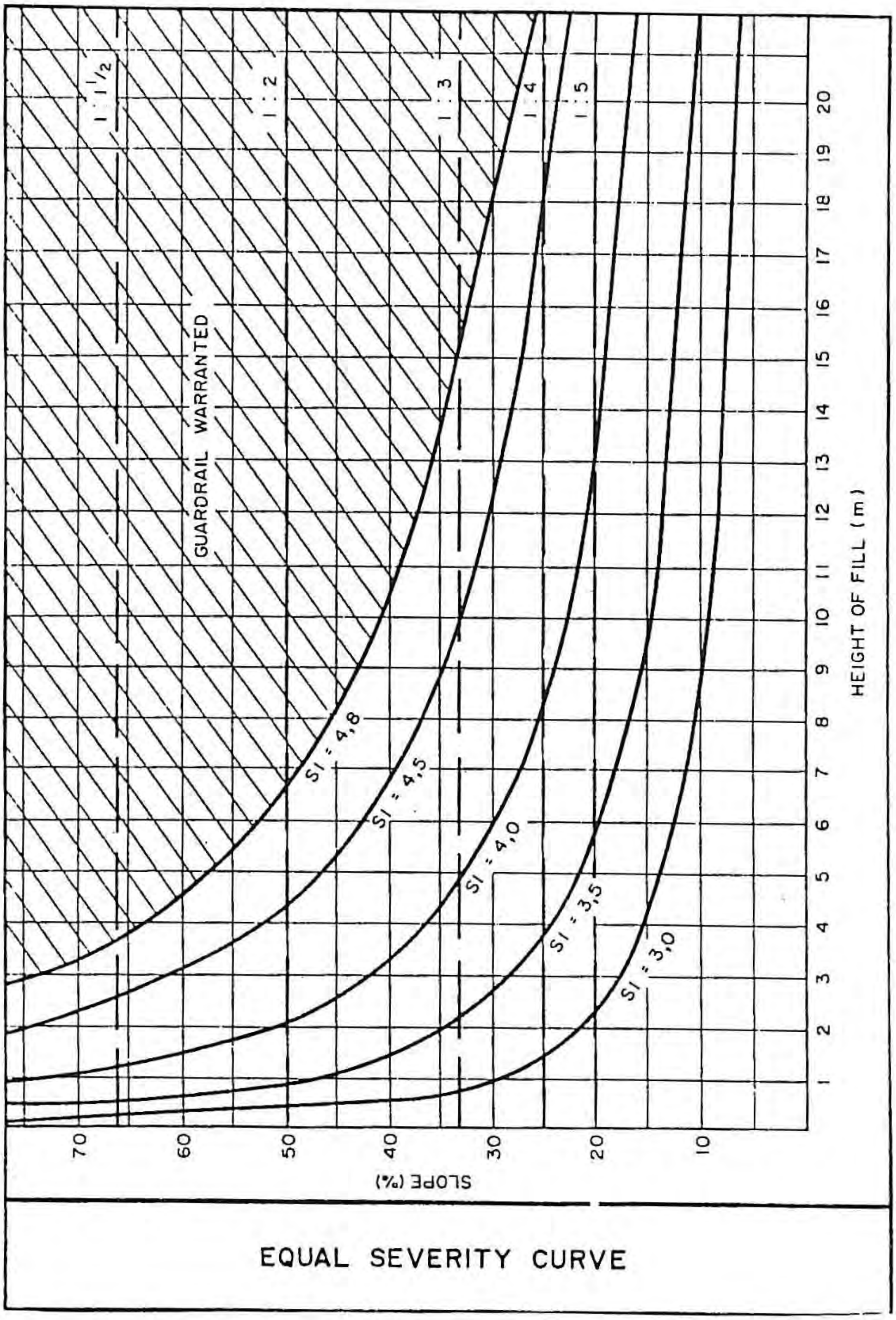


Figure 6-1: **WARRANTS FOR THE PROVISION OF GUARDRAILS**

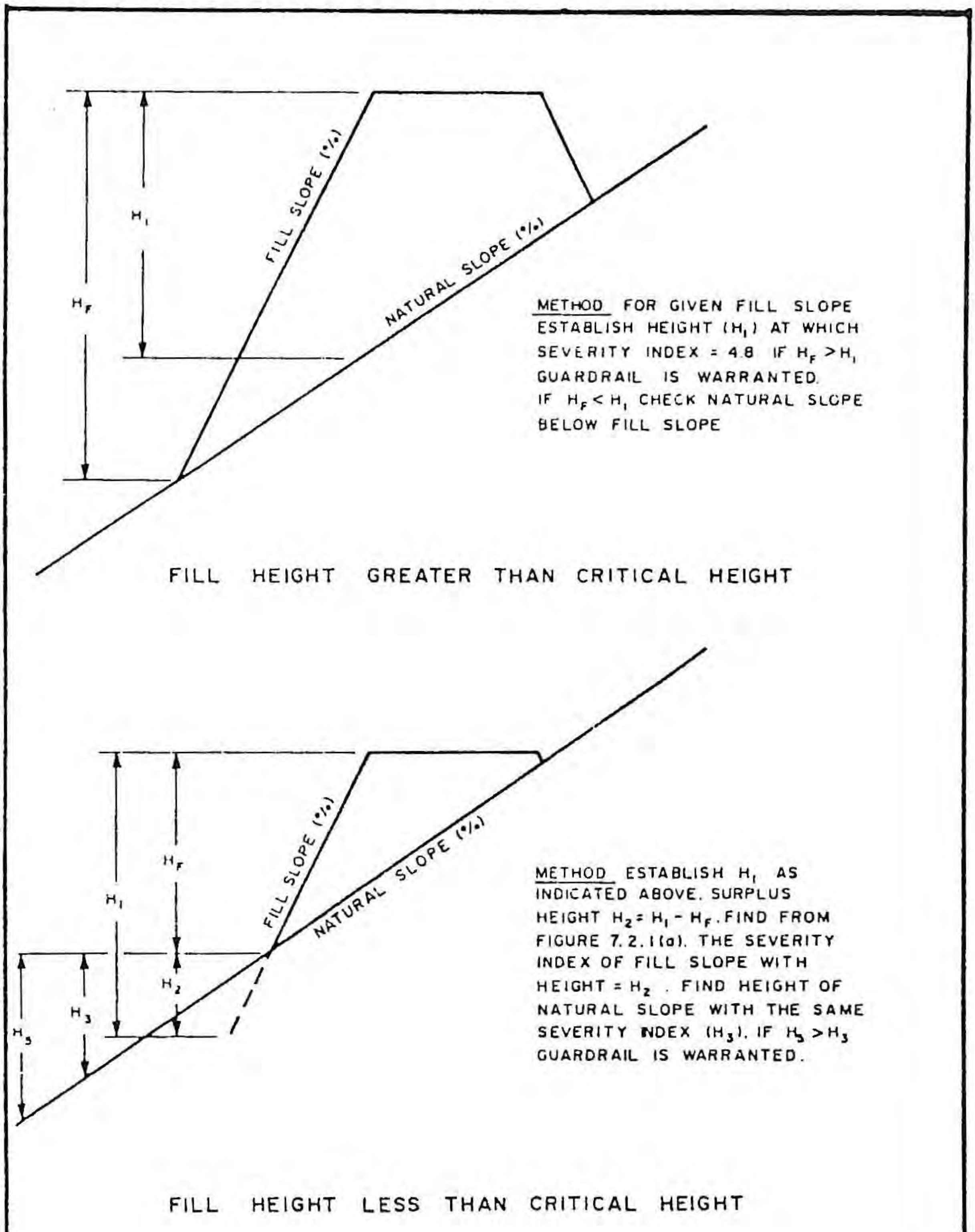


Figure 6-1(a): APPLICATION OF THE EQUAL SEVERITY CURVE

6.5 Bridge railings

Most bridge railings are an integral part of the structure and are usually designed to have virtually no deflection when struck by an errant driver. A railing should have adequate strength to redirect heavy vehicles but should also be of adequate height to prevent an impacting vehicle or its cargo from rolling over the railing.

It may not be cost-effective to provide a rigid railing with an approach guardrail and a transition section on all small structures on low volume roads. Widening the structure and leaving the edges unshielded or utilizing a less expensive, semi-rigid type railing may be more cost-effective.

Approach railings and transition sections should provide continuity and should be of adequate length and strength to prevent pocketing, snagging or penetration. Bridge railings should be delineated with danger plates.

When the bridge railing is located within the recommended shy distance, the approach railing should have the appropriate flare rate shown in Table 6-1 (Section 6.2).

Pedestrians and/or cyclists should be shielded from vehicular traffic on bridges when vehicle speed, conditions on either end of the structure and pedestrian/cycle volumes warrant separation. In slow-speed situations a curb in front of the railing may provide marginal protection for pedestrians but should be avoided if possible.

6.6 Drainage

Effective drainage should be achieved without the drainage system creating a greater hazard than the stormwater it seeks to remove.

Ditches must be designed to accommodate excessive stormwater flows with minimal damage; it should also be designed, built and maintained with road safety in mind. They are one of the commonest hazards struck during single-vehicle collisions. Slopes on the side of the ditch nearest the road should be no more than 1:3 and preferably shallower as this will minimise damage and injury. The depth and width of the channel gap on a typical U-type ditch offers no opportunity for the driver to recover if he should lose control.

The absence of pedestrian footpaths sometimes forces pedestrians to walk on the road, especially where a deep U or V-type ditch (see Figure 8-4) prevents pedestrians from walking along it. L-type and J-type drainage channels provide a safe area for pedestrians to walk during dry periods.

Drainage structures should be traversable, or extended so they are less likely to be struck, or shielded. Drop inlets should be designed and located to present a minimal obstacle to errant drivers. Openings should be treated but grates with openings as small as those used for pavement drainage are not necessary unless pedestrians are a consideration. Drainage structures should be delineated with danger plates.

6.7 Trees, utility poles and other roadside obstacles

Large trees should be removed from within the selected clear zone (see Table 6-2), or shielded if the severity of striking the tree is greater than striking the barrier. Every effort should be made to install or relocate utility poles as far from the travelled way as practical, and increasing pole spacing should be considered. Massive supports used for major electrical transmission lines should be shielded or delineated.

Visibility is important not only to the driver, but also to other road users such as pedestrians. Roadside obstacles may obstruct visibility, especially on bends, the approaches to intersections and on overtaking sections. The roadside should be clear of obstructions in areas where pedestrians or animals cross the road, so that they can be seen clearly by approaching drivers within a safe stopping sight distance. Pedestrians should also have clear views of approaching vehicles in both directions in these areas. Vegetation should be cut back regularly to ensure that sight distances are maintained.

6.8 Road signs

Road signs are extremely important tools of communication to guide and direct the driver through conflict points and hazards on the road network. They enable the driver to be given advance warning. Signs should be visible at all times, should not be obscured, should not obscure other features, and should not constitute a hazard in themselves.

Warning signs give advance notice of a potential hazard ahead or any unexpected feature of the road geometry, such as an unexpectedly sharp bend on an otherwise high speed road or the presence of a pedestrian crossing or an intersection. Warning of these unexpected features may be accompanied by a reduction in speed limit for that section.

Signs should be simple, clear and uniform, using minimal wording. Symbols should be included as they generally aid rapid understanding of the message.

The requirements given in the following sections are based on the revised South African Road Traffic Signs Manual (DoT, 1981) and Road Sign Notes.

6.8.1 Sizes of signs

Table 6-3 shows sizes of signs to be applied on low volume roads.

Table 6-3: SIZES OF REGULATORY AND WARNING SIGNS FOR DIFFERENT SPEED LIMITS (mm)

Class of sign	Type of sign	Speed limit (km/h)		
		60	80	100
Regulatory	Stop (width)	600	900	900*
	Round (diameter)	600	600	900
	Yield signs (side of triangle)	600	900	900*
	Rectangular (width x height)	450x338	600x450	900x678
Warning	Triangular (side of triangle)	900	900	1200***
	Chevron (square)	400	450	600
	Danger plates (rectangular)	600x150	600x150	600x150**

* Sizes should be increased to 1200mm at hazardous sites
** Sizes should be increased to 800mmx200mm at hazardous sites
***On gravel roads a size of 900mm may be used

6.8.2 Letter sizes and styles

Stack type signs should be used instead of map type signs. Lettering style should be DIN B. The number of destinations per sign should be kept to a minimum (Department of Transport 1981)(show main destination only). Tables 6-4 and 6-5 give letter sizes on guidance signs to be used on paved roads and gravel roads respectively.

Table 6-4: LETTER SIZES ON GUIDANCE SIGNS FOR PAVED ROADS

	Speed limit (km/h)		
	60	80	100
One destination per sign	$\frac{112}{80}$	$\frac{140}{100}$	$\frac{140}{100}$
Two destinations per sign	$\frac{112}{80}$	$\frac{140}{100}$	$\frac{175}{125}$
Three destinations per sign	$\frac{140}{100}$	$\frac{175}{125}$	$\frac{175}{125}$

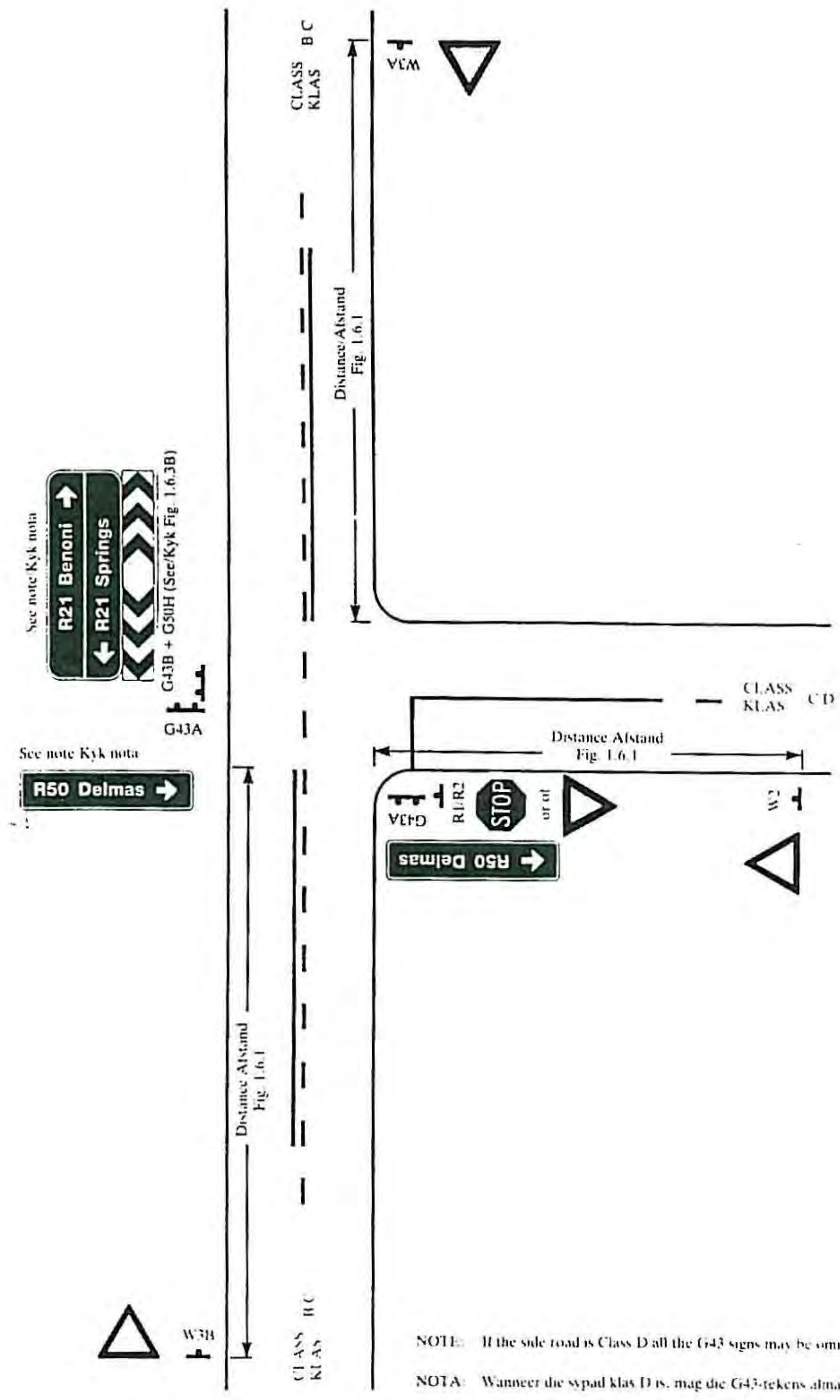
Table 6-5: LETTER SIZES ON GUIDANCE SIGNS FOR GRAVEL ROADS

	Speed limit (km/h)		
	60	80	100
One destination per sign	$\frac{112}{80}$	$\frac{112}{80}$	$\frac{140}{100}$
Two destinations per sign	$\frac{112}{80}$	$\frac{112}{80}$	$\frac{140}{100}$
Three destinations per sign	$\frac{140}{100}$	$\frac{140}{100}$	$\frac{175}{125}$

6.8.3 Signs at intersections

A typical sign layout at intersections is shown in Figure 6-2. Advance direction stack signs should only be used at problematic junctions.

TYPICAL RURAL ROAD T-JUNCTION SIGNS: CLASS B OR C MAJOR ROAD ON CLASS C OR D SIDE ROAD
TIPIESE PADTEKENS VIR T-AANSLUITINGS OP BUITESTEDELIKE PAAIE: KLAS B- OF C-GROOTPAD BY KLAS C- OF D-SYPAD



NOTE: If the side road is Class D all the G43 signs may be omitted

NOTA: Wanneer die sypad klas D is, mag die G43-tekens almal weggelaat word

Figure 6-2: SIGN LAYOUT AT INTERSECTIONS

6.8.4 Retroreflection

Retroreflective materials should be used on signs to make them clearly visible, especially at night. Table 6-6 gives the sign material to be used for different classes of signs.

Table 6-6: SIGN MATERIAL FOR DIFFERENT CLASSES OF SIGNS

Class of sign	Border	Background	Symbol
Regulatory signs	Class 1 retroreflective	Class 1 retroreflective	Semi-matt Black
Warning signs	Class 1 retroreflective	Class 1 retroreflective	Semi-matt Black
Guidance signs	Class 1 retroreflective	Semi-matt Green	Class 1 retroreflective

6.9 Road markings

Road markings can make a significant contribution to the safe and efficient operation of the road network. They should be visible in all weather conditions and should be used in conjunction with road signs. Road markings and signs are of particular use where a design element is sub-standard. The costs are minimal compared to construction costs.

Dividing or centre lines are used to indicate where overtaking is dangerous; edgelines should be used to give advance warning of changes in horizontal and vertical alignment to reduce the probability of single-vehicle collisions. A width of 100mm should be used for edgelines.

Road markings should be applied according to the South African Road Traffic Signs Manual (1981), except for the following:

- the width of the dividing or centre line should be 100mm if the speed limit is 100km/h, and 50mm if the speed limit is 80 or 60km/h
- a 12 metre module should be used with a gap-to-line ratio of 10:2 on straight sections and a ratio of 8:4 on curves:

6.10 Delineation

Chevrons and delineators should be used to break the driver's line of sight where unexpected hazards exist eg. at sharp bends, bridge heads, culverts, or other sites where sight distance problems occur. They should be positioned so that the driver will be able to see at least three delineators at any given time. At junctions the minimum standards as prescribed by the South African Road Traffic Signs Manual (1981) should be used. Painted rocks or wooden posts may be used as a low-cost solution. The sizes of chevrons and danger plates are given in Table 6-3 (Section 6.8.1).

In areas where mist often occurs the following delineation should be used:

- Road studs on paved roads
  - 24 metre spacing on straight sections
  - 12 metre spacing on intermediate curves
  - 6 metre spacing on sharp curves
  - more road studs may be used on very sharp curves

- Shoulder delineation on gravel roads
  - 36 metre spacing on straight sections
  - 24 metre spacing on intermediate curves
  - 12 metre spacing on sharp curves
  - more delineators may be used on very sharp curves

6.11 References

Department of Transport, *The South African Road Traffic Signs Manual.* Manual K55, CSIR, Pretoria, 1981 (Second Edition - currently under revision).

7 SAFETY ASPECTS

7.1 Introduction

Design standards must take into account the environmental road conditions, traffic characteristics and driver behaviour. The selection of design standards is related to road function, volume of traffic and terrain, with additional procedures for the recognition and appropriate treatment of potential hazards. A basic assumption is that drivers receive clues about the standard of the road from local surrounding features such as terrain, levels and types of flow, as well as geometric elements. Additional design consideration or special signing will only be necessary where the information available to the driver may lead to incorrect interpretation and consequent danger.

There is very little information on the effects of changes in standards on collision rates. Safety is usually assumed to be optimised by linking geometric elements to a design or operating speed, so that the resulting geometry has a consistency which reduces the likelihood of a driver being presented with an unexpected situation. The concept of driver expectation forms the basis of design standards for low volume roads.

The geometric standard of individual elements of the road will also vary with the terrain. It is necessary to identify elements of lower geometric standard to ensure that they will not result in unacceptable hazards to approaching vehicles. If the standard of a geometric element falls substantially below that on the approach section, consideration must be given to the redesign of the element or alterations to the geometry of the approach section.

Relaxation of standards may be essential in order to achieve an acceptable rate of return, but the inclusion of a short section of substandard road where achievement of the design standard would be expensive, could have serious safety implications. Potential collision risk has to be considered on a site-specific basis and may need to be reduced by providing appropriate signing or other warning measures.

In general, designers should be aware of the need to consider safety, and should make use of the opportunities which may arise at design or construction stages and which may result in substantial benefits at little additional cost.

The following features will be discussed in this chapter:

- Design and construction
  - Speed limits

- Sight distance
- Lane width
- Shoulders
- Pedestrian facilities
- Cyclists and slow-moving vehicles
- Public transport
- Fencing and animals

• Maintenance and rehabilitation
  - Roadworks signing
  - Road signs, guideposts, kilometre posts, guardrails
  - Quality of road surface and visibility
  - Roadside maintenance

• Legal aspects

7.2 Design and construction

7.2.1 Speed limits

Designs should be justified economically. In general, construction costs will be greater as the terrain becomes more difficult and higher standards will become less justifiable or achievable in mountainous terrain than for roads in flat or rolling terrain. Drivers should also expect lower standards in such conditions and therefore adjust their driving accordingly to minimise collision risk. Design speed will therefore vary with terrain and should provide an appropriate consistency between geometric elements rather than indicate actual vehicle speed at any particular location on the road section.

Speed limits, if not consistent with the nature and type of road, will not be observed by drivers. As traffic conditions and land-uses change over time, speed limits should be regularly reviewed to ensure that they relate to current circumstances. To increase effectiveness speed limits should be applied in conjunction with physical speed reduction measures and law enforcement.

• General speed limits

Common speed limits found on low volume roads are 100km/h for paved roads and 80km/h for unpaved roads. However the speed limit of each road should be assessed individually, particularly where sharp curves or short sight distances are involved. On individual sections in mountainous areas speed limits as low as 40 km/h can be posted.

Special speed limits may be appropriate at schools or other places where large number of pedestrians may be expected. In areas where a large number of pedestrians (200 in any four hours of a day) cross the road, the speed limit should be set to 70km/h and where a large number of pedestrians or pedal cyclists use the shoulder, a speed limit of 80km/h should be posted.

- Advisory speed limits

Advisory speed limits should be posted before unexpected changes in horizontal and vertical alignment eg at sharp bends with fairly long straight approaches.

7.2.2 Sight distance

It is essential for the driver to be able to perceive hazards on the road, with sufficient time in hand to initiate any necessary evasive action safely. It is also necessary to be able to enter the opposing lane safely while overtaking.

Stopping sight distance involves the capability of the driver to bring his vehicle safely to a standstill, and is based on speed, driver reaction time and skid resistance. The total distance travelled in bringing the vehicle to a stop comprises the distance covered during the reaction period and the distance required to decelerate to 0 km/h. Stopping sight distances on level roads are given in Table 3-5 (Chapter 3)

Passing sight distance is indicative of the quality of service provided by the road and distances for South African conditions are given in Table 3-8 (Chapter 3)

7.2.3 Lane width (see also section 3.5 in Chapter 3)

The narrowest width recommended for low traffic volume is 3,1m, giving a clear space of 0,3m on either side of a 2,5m wide vehicle. Lanes should be widened on the inside of curves and should be gradually introduced over the length of the transition. On roads with substantial curvature requiring local widening, it may be practical to increase width over a complete section to offer a more consistent aspect to the driver.

7.2.4 Shoulders

A partly blocked lane is acceptable under conditions of low traffic volume and low speed. In the case of low speed it would be possible for two vehicles to pass each other next to a stopped vehicle if the shoulders were not less than 1,0m wide and the narrowest recommended width of a through lane (3,1m), is assumed, giving a total cross-sectional width of 8,2m to accommodate three vehicles. For speeds higher than 60km/h a shoulder width of 1,5m is regarded as the minimum.

Surfacing of shoulders is recommended in front of guardrails, in mist belts, and wherever significant usage by pedestrians, cyclists, slow-moving vehicles or buses occurs.

7.2.5 Pedestrian facilities

Pedestrian fatalities constitute 50 per cent of the total road fatalities and 40 per cent of all pedestrian fatalities occur on rural roads. The severity of injuries is higher on rural roads; the ratio of slightly injured : seriously injured : fatally injured is 1:1:1 on rural roads compared to 6:3:1 on urban roads.

It is necessary to consider cost-effective ways of segregating non-motorised traffic at the earliest stage in the design process. The local community can be involved; they can assist in the construction of walkways in areas where they are warranted. A variety of construction materials are available such as paving blocks, asphalt, concrete, etc.

Usually no provision is made for pedestrians and pedal cyclists along rural roads - pedestrians are entirely dependent on the road shoulder when walking to a bus stop or from one place to another. Footways with a minimum width of 1m in rural areas and 1,5m in peri-urban areas should be considered. They should be situated at least 3m from the travelled way in level terrain. Footways in rolling or mountainous terrain through cuttings and fills may be situated adjacent to the roadway. Kerbstones can be used to safeguard footways under these circumstances. Provision should be made for pedestrians across bridges in the form of footways with a minimum width of 1,2m. Warrants for pedestrian footways are given in Table 7-1.

Table 7-1: WARRANTS FOR PEDESTRIAN FOOTWAYS

Footway	ADT	Pedestrian flow (per day)	
		Design speed or speed limit (60-80km/h)	Design speed or speed limit (80-100km/h)
On one side	400 to 1400 > 1400	300 200	200 120
Both sides	700 to 1400 > 1400	1000 600	600 400

When footways are not warranted but a large number of pedestrians walk alongside the road, the shoulder should be upgraded to cater for them. The minimum width of these shoulders should be 2m and they should be graded and compacted regularly. They should also be well drained. Footways could also be provided beyond the drainage facility.

In rolling or mountainous terrain with restricted sight distance, painted refuge islands of at least 1,5m wide could be considered to allow pedestrians to cross the road safely. Pedestrian crossings are usually not practicable on high-speed roads. Pedestrian guidelines, however, can be used to determine when a pedestrian crossing is warranted. Figure 7-1 illustrates the warrants for the different types of pedestrian crossings.

7.2.6 Cyclists and slow-moving vehicles

As well as reducing capacity substantially, the presence of slow-moving vehicles such as cycles and animal-drawn carts creates hazardous conditions. Other vehicles may be forced to slow down rapidly or be tempted to overtake in dangerous circumstances. They are especially hazardous at night. A wider shoulder (well-drained and well-maintained) should be provided in areas where slow-moving traffic are observed.

On roads carrying between 20 and 70 pedal cyclists travelling in one direction during any one hour of the day a cycle lane on a paved shoulder should be provided. Details on road markings and signs for pedal cyclists are given in the manual for the planning and design of bicycle facilities(DoT, 1982). If a hard gravel shoulder is provided next to the cycle lane, a cycle lane width of 1,2m would suffice, but for a soft shoulder or sloped drop-off a 1,5m wide cycle lane is recommended.

			Two-way road						One-way road					
Type of pedestrian	Speed limit (km/h)	Effective width of road (m)	A		B		C		D		E		F	
			vph	pph	vph	pph	vph	pph	vph	pph	vph	pph	vph	pph
Typical	60	7	705	385	705	95	1000	50	850	385	850	100	1200	50
		10	520	385	520	95	770	50	610	385	610	95	900	50
		14	365	385	365	90	560	50	425	385	425	90	640	50
	70	7	635	385	640	95	935	50	800	385	800	100	1150	50
		10	485	385	485	90	730	50	580	385	580	95	860	50
		14	340	385	340	90	530	50	405	385	405	90	620	50
Elderly	60	7	620	335	620	95	910	50	750	335	750	100	1100	50
		10	450	335	450	90	680	50	520	335	520	95	770	50
		14	310	335	310	90	470	50	355	335	355	90	550	50
	70	7	570	335	570	95	860	50	705	335	705	95	1030	50
		10	420	335	420	90	640	50	495	335	495	95	750	50
		14	290	335	290	90	460	50	335	335	335	90	520	50

vph = vehicles per hour
pph = pedestrians per hour

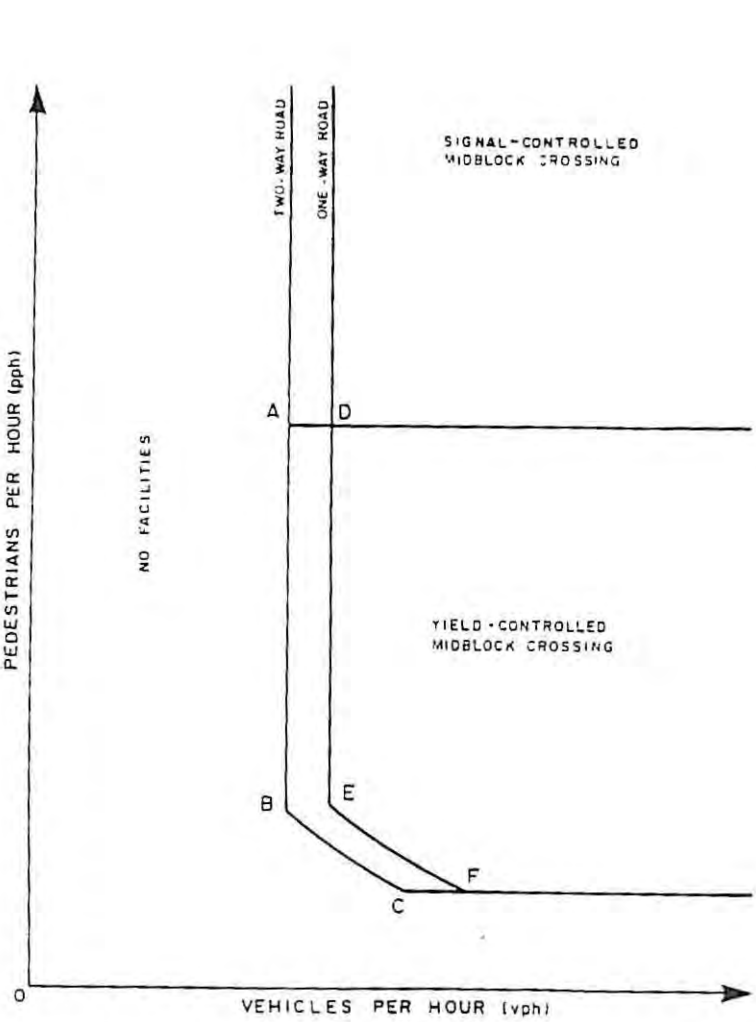


Figure 7-1: GUIDELINES FOR PEDESTRIAN CROSSING FACILITIES

7.2.7 Public transport

Bus bays, at least 3,5m wide and 10m long, should be provided and placed adjacent to the paved or gravel shoulder so that buses can stop clear of the roadway. The merging taper could be somewhat more abrupt than the deceleration taper, but should not be sharper than 3:1.

A hardened area for passengers should be provided alongside the bus bay. Bus bays and lay-bys for buses, taxis, etc should be positioned on straight, level sections; should be visible from a good distance in both directions; should have convenient and safe access; and should be well-maintained, well-drained and compacted. They should be sign-posted and advance warning should be given of the presence of pedestrians ahead.

To increase the visibility of pedestrians, bus stops should be located beyond intersections or crossings and if bus stops are provided on both sides, they should be located tail to tail as pedestrians tend to cross behind buses.

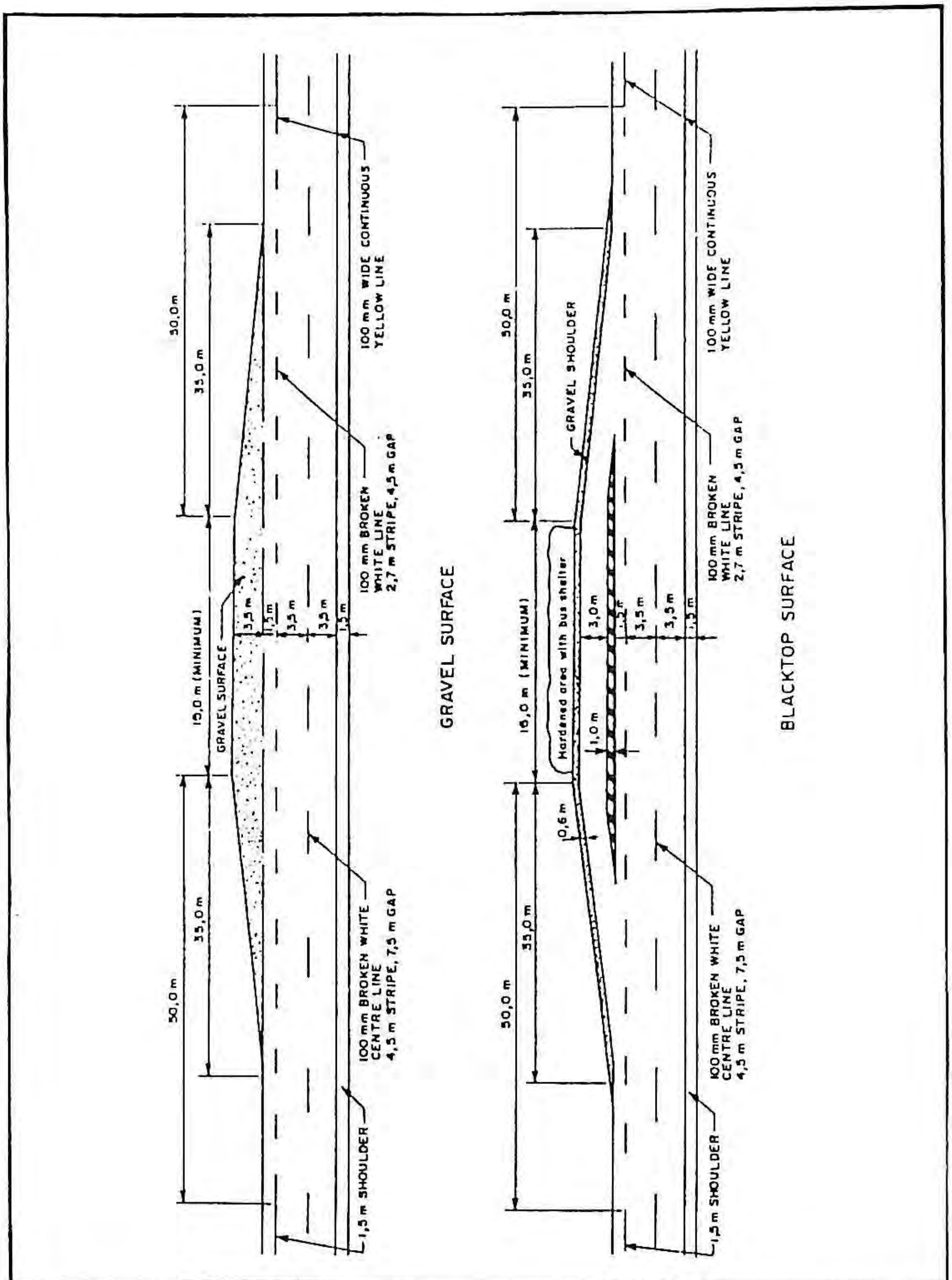
Figure 7-2 shows a typical layout of bus bays.

7.2.8 Fencing and animals

Fences should be erected to prevent collisions with stray animals on roads. The local population should be consulted on planning and design of roads. Their participation would ensure the correct positioning of pedestrian and animal gates, cattle crossings and bus stops and should minimise the destruction of fences. Education of the community is essential.

Grid gates should be provided instead of cattle gates, and only after consultation with the community. Animal warning signs should be erected where necessary.

Fences should be erected as far away as possible from the travelled way but within the road reserve. (see Table 6-2).



TYPICAL BUS BAY LAYOUTS FOR GRAVEL AND BLACKTOP SURFACES

Figure 7-2: TYPICAL LAYOUT OF BUS BAYS

7.3 Maintenance and rehabilitation

7.3.1 Roadworks signing

The temporary and continually variable nature of road construction and maintenance operations makes such sites potentially more dangerous than a permanent hazard since even a driver familiar with the route cannot rely on his previous knowledge to predict conditions. In order to clearly identify these temporary conditions from permanent ones exclusive yellow background signs are used. Where a roadway is closed, partially closed or diverted, or where an obstruction exists in the roadway, the alignment to be followed by vehicles should be delineated by delineators, cones, barricades, barriers, roadstuds or roadmarkings, or an appropriate combination of these devices.

The sizes of signs to be used during road construction are given in Table 6-3 (Section 6.8.1). Guidance signs of 1200x1600mm should only be used at very dangerous sites. Plastic delineators (danger plates) of 800x200mm should be used.

Signs have to be cleaned regularly in dusty or muddy conditions.

Road Signs Note 13 - Roadworks of the South African Road Traffic Signs Manual (DOT, 1981) gives detailed guidelines on the layout of signs during construction. A typical layout of signs for roadworks is shown in Figure 7-3.

7.3.2 Road signs, markings, guideposts, kilometre posts, guardrails

Road signs and markings should be inspected regularly for retroreflectivity during night time. Signs should be in a good condition, correctly located, properly mounted, fixed and stable. They should be visible at all times. The erection of temporary warning signs for potholes or corrugations before the next scheduled maintenance could be considered.

Damaged or faded road signs and markings should be replaced and guardrails repaired as soon as possible, especially in hazardous areas.

The introduction of a kilometre-post system, even on low-volume roads, is very important; it can be used for maintenance programs, collision reporting and the identification of hazardous locations.

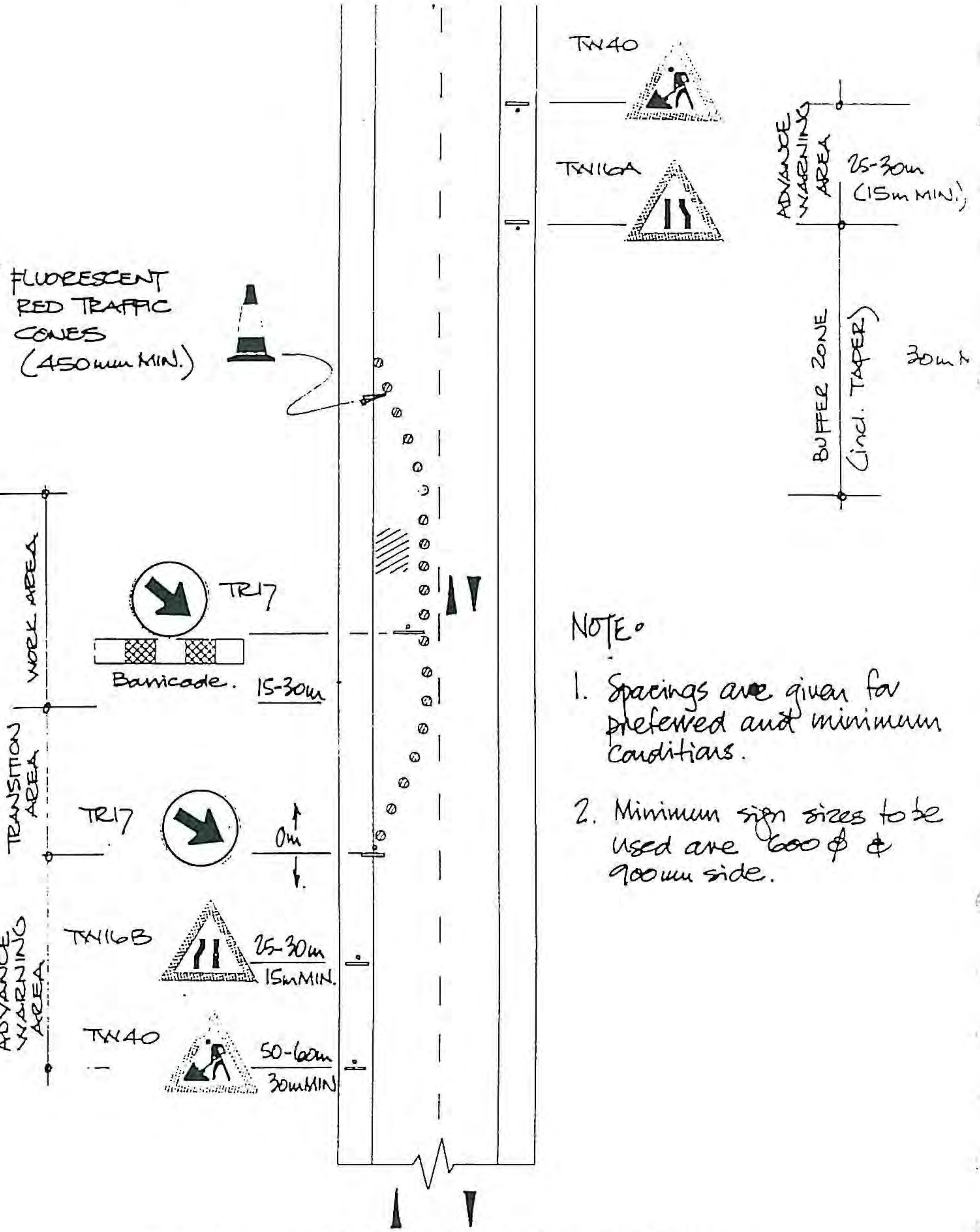


Figure 7-3: TYPICAL LAYOUT OF ROAD SIGNS FOR ROADWORKS

7.3.3 Quality of road surface and visibility

From a road safety perspective, there are a number of quality issues. Potholes should not be allowed to develop and should they do so, they should be patched immediately. Surface texture and skid resistance should be maintained; drainage ditches should remain free of obstructions and surface and ground water should be able to drain away. Shoulders should remain stable and compacted to ensure that the pavement has adequate side support. It is also important for pedestrians, cyclists, bus stops, etc that shoulders are well-maintained. Continuity between a gravel shoulder and the pavement should be maintained. Dust should be prevented or kept to a minimum on unpaved roads.

From the maintenance perspective (covered in more detail in Chapter 10)

Slopes should retain their shape and stability. Vegetation and grass in road reserve should be cut in the following cases:

- where sight distance is a problem
- where signs may be obscured
- where drainage structures may be obscured
- where pedestrians and animals may be obscured
- where veld fires could damage signs and cause smoke
- where a walkway for pedestrians is needed.

Fences should be repaired after vandalism and destruction.

7.4 Legal aspects

There are a number of legal issues relating to road safety. The position of the road authority is not always clear and is subject to interpretation by the courts. In broad terms though the road authority responsible for the construction of roads will not be liable for any damages suffered by a user of the road, unless such damage is caused by wrongful conduct of such authority. Any road user who would file a claim against the relevant authority will have to prove either intention or negligence on the part of the authority, which intention or negligence actually caused the damages suffered. In determining liability a proper investigation will have to be made in all the particular circumstances of the relevant case, to determine whether the fixing of minimum standards by the authority actually constituted wrongful conduct or were actually inadequate. The damage caused by the occasion should have been reasonably foreseeable by the authority when he fixed the standards, failing which liability is excluded.

As long as the "reasonable man"-test to determine negligence indicates that the road authority intentionally or negligently, created or allowed a dangerous situation to develop which caused harm to others, it will be held liable.

If proper and sufficient warning signs of the poor condition of the road, or dangerous situations and conditions are erected, the authority may escape liability. There will however have to be sufficient indicators of the danger. Special reasonable speed limits could also accompany warning signs at hazardous sites.

7.5 References

Department of Transport (1982), *Manual for the planning and design of bicycle facilities in urban areas.* Report No. U 12/7/4/47, Pretoria.

Department of Transport (1981), *The South African Road Traffic Signs Manual.* Manual K55, CSIR, Pretoria, 1981 (Second Edition - currently under revision).

8 DRAINAGE

8.1 Introduction

This chapter covers the principles and guidelines to be adopted in the hydrological analysis, hydraulic calculations and the design of drainage facilities. The drainage of the road prism, culverts, bridges and sub-surface drainage are addressed. Aspects covered in other design guidelines and manuals available in South Africa are not repeated here.

The reader is referred to the following documents:

- National Transport Commission (NTC). *Road drainage manual.* Chief Directorate: National Roads, Department of Transport, Pretoria, 1986.
- Alexander, WJR. *Flood hydrology for southern Africa.* South African National Committee on Large Dams, Pretoria, 1991.
- The drainage standards of the relevant road authority.

8.2 Policy on drainage design

The purpose of drainage facilities is to remove stormwater from the road surface and to allow stormwater to cross over or underneath the road in such a way that:

- Danger to vehicles and people is kept to a minimum
- Delays experienced by the public and the associated inconvenience are minimized within the constraints of the construction cost of drainage infrastructure
- Damage to roads, services and property due to flooding is kept to a minimum within the constraints of the construction cost of drainage infrastructure.

A second purpose of drainage facilities is to drain away sub-surface water which can severally damage pavement layers.

Although it is in many instances not possible, the fundamental approach to ensure low drainage infrastructure costs is to choose that road alignment which follows the natural watersheds as far as is possible. During the route determination stage of road planning this aspect should be considered in conjunction with other aspects such as the availability of land, the cost of earth- and layerworks and road user costs.

An efficient drainage system is an essential contribution to the overall design of a road. Various types of drainage facilities are employed to protect the road against damage by surface and subsurface water. Drainage facilities must be designed as simply as possible to convey the stormwater along or away from the road in the most economic, efficient and practical manner without causing unnecessary damage to the road and adjacent property.

It must be emphasised that drainage requirements vary significantly from one region to the other and also depend on the road type. While, for example, vegetation growth is encouraged in some areas to prevent erosion, it is a problem in other areas where excessive growth results in the obstruction of water flow. The resistance of in situ soils to erosion, rainfall characteristics, topography, maintenance methods, etc also vary from one region to the other. The designer must therefore give the necessary attention to the specific requirements of the region, local experience and the road authority's design guidelines.

The following considerations are important in the economic analysis of drainage policy:

- The basic cost of construction, incorporating the use of local materials wherever possible, and of labour intensive methods where appropriate.
- The useful life and cost of replacement or extension.
- The cost to the travelling public resulting from delays or extra travel distance due to road closures.
- The cost of repairs resulting from damage due to the overtopping of the road or from the concentration of water on adjacent properties.
- The cost of drainage infrastructure maintenance.

With low volume roads the design should be aimed at reducing the cost of construction, but taking into account the benefits to be derived. In terms of benefit-cost analysis a high discount rate, eg 15 per cent, should be used in order to give more weight to present costs relative to future benefits. The main purpose with low volume roads is to provide reliable access and to "get the people out of the mud", and the lower the cost, the more infrastructure can be provided.

In some cases existing unpaved roads that are to be upgraded will only have been provided with nominal drainage, as there was often no formal drainage design done for these roads. In such cases the adequacy of existing drainage infrastructure should be evaluated in the light of previous maintenance experience and the extent of the consequences of possible overtopping. It may in many instances not be justified to upgrade drainage infrastructure to achieve the requirements associated with a specific return period.

8.3 Hydrological analysis

8.3.1 Introduction

An hydrological analysis of the area to be drained is an essential element in the design of road drainage. This analysis provides the information on runoff and stream flow characteristics which is used as a basis for the hydraulic design.

Because of the comprehensive guidelines available on hydrological analysis, for example the road Drainage Manual (NTC, 1986), analysis procedures are not repeated in this document.

8.3.2 Return period

The most important design parameter influencing the standard of drainage structures is the design return period. The return period is the average period over a large number of years during which an event (peak flow) repeats or exceeds itself. The relation between the frequency of occurrence of the peak flow (return period) and the probability of occurring during the design lifetime is given by:

$$r = \{1 - (1 - 1/T)^{L}\} \times 100$$

where:
- r = Risk probability expressed as a percentage
- T = Return period in years
- L = Design life in years

As far as the term "flood" is concerned, the following definitions apply:

From the point of view of water control, a flood is a temporary excess of water that overflows the banks of the watercourse. The most important aspect of a flood is the disturbance it causes to human activities in terms of real or economic damage and of potential risks to human lives.

From the point of view of hydrology, however, a flood is a wave that progresses along a watercourse and causes changes in water level, discharge, flow velocity and water surface slope all along the course. This second definition is applicable when reference is made to a flood, or peak flow, in this chapter.

The decision on the return period to be used for drainage infrastructure should be evaluated in terms of the cost of the structures, maintenance costs and the possible cost of damage in the case of the design flow being exceeded. The institutional capability of the responsible road authority to effect repairs should also be taken into account.

Recommended basic flood frequencies for drains, culverts, low level structures and high level bridges on low volume roads are provided in Table 8-1:

Table 8-1: RETURN PERIOD FOR DRAINAGE STRUCTURE DESIGN*

1:20 YR FLOW (m^3/s)	TYPE OF STRUCTURE			
	DRAINS	CULVERT	LOW LEVEL STRUCTURE**	HIGH LEVEL BRIDGE
0 - 10	1:2	1:5	(0,25 to 1,0) x 1:2	
10 - 20		1:5	(0,25 to 1,0) x 1:2	
20 - 150			(0,25 to 1,0) x 1:2	1:10
150 - 400			(0,25 to 1,0) x 1:2	1:20
> 400			(0,25 to 1,0) x 1:2	1:20

* Shaded cells indicate optional, as opposed to recommended, design levels
** Refer to Section 8.9.4

As far as low level structures are concerned, the reader is referred to Section 8.9.4 for an explanation of the methodology used to determine the design flood.

8.3.3 Methods of design flood determination

Various methods for the determination of the design flood can be used. One of the most commonly used methods in South Africa is the rational method. This method is based on a simplified representation for the processes and variables involved in flood run-off. Rainfall intensity is an important input in the calculations. Because an uniform area and time distribution of rainfall have to be assumed, the method is normally only recommended for catchments smaller than about 15 km^2. With caution the method can be used for catchment areas up to 100 km^2, while for catchment areas between 100 and 500 km^2 the method can be used in conjunction with other methods.

The synthetic hygrograph method is suitable for the determination of flood peaks as well as hygrographs of medium-sized catchments (15 to 5 000 km^2). The method is based mainly on regional analyses of historic data and is independent of personal judgement. The results are reliable, although some natural variability in the hydrological occurrences is lost through the broad regional divisions and the averaged form of the hygrographs.

The reader is referred to the guidelines available on design flood determination, for example the Road Drainage Manual (NTC, 1986).

8.4 Hydraulic calculations

Hydraulic calculations are necessary to determine the size and spacing of drainage structures. Other factors to be determined for the design of drainage infrastructure include flow velocities, flow depths and flow patterns. Design procedures are left to the designer, who is referred to the available guidelines, eg the Road Drainage Manual (NTC, 1986).

8.5 Formation and earthworks

8.5.1 Formation

The road formation forms an obstruction to normal water flow. It is important to ensure continuity of flow from one side of the road to the other. The road, which functions as a barrier, will cause storm runoff to flow parallel to the road embankment until a discharge point is reached, whether it be a relief culvert of a culvert at a low point.

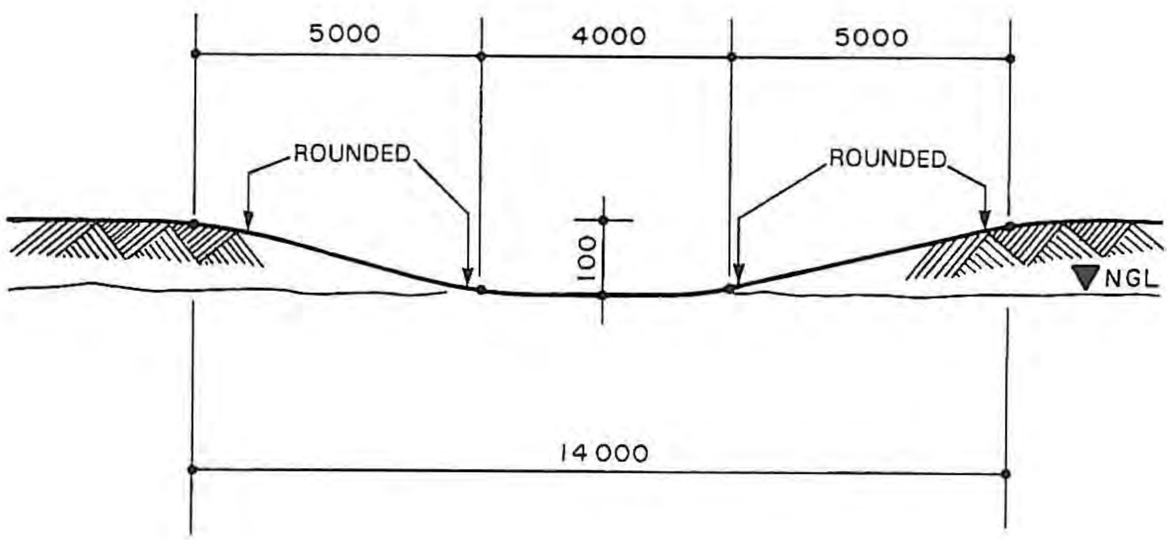
Between two watersheds there is generally one low point in the road. The distance from a watershed to this low point can be several kilometres, depending on the nature of the terrain. Provision should be made to discharge the runoff across the road at regular intervals in order to avoid a concentration of runoff at the low point and to maintain a balance of runoff. These intervals depend on the locality and type of road and could vary from 100 m to in excess of 500 m.

With high order roads drainage provision is made for stormwater to cross underneath the road. This approach requires minimum side drain depths in the case of cuttings and minimum formation heights in the case of fills (to prevent water from flowing over the road and to allow for culverts). In flat areas where embankments are constructed mainly to provide space for culverts, the optimum balance between fill costs and drainage costs should be sought.

In general it is advisable to build the formation to heights of 400 to 500 mm over flat or rolling areas.

Depending on the pavement design, however, it may be warranted for roads at the lower end of the scale (e.g. access roads) to construct the road surface very close to natural ground level, especially in flat terrain and/or in low rainfall areas (formation height less than say 150 mm). It is in such cases appropriate to consider the possibility of omitting some or all of the culverts and to allow water to flow over the road. Side drains are used to convey water to points, provided at regular spacings, where it can flow over the road. Dish drains or depressions can be provided in the pavement for this purpose - an example is shown in Figure 8-1. It may also in some instances be appropriate to provide beams in the road, similar to speed humps, to prevent water from flowing in the longitudinal direction. Typical dimensions for such a berm is shown in Figure 8-2. The necessary warning signs must be provided, e.g. W21: Uneven Roadway and W43: General warning sign: (The South African Road Traffic Sign Manual, 1982).

Although the above approach may have increased maintenance costs as a result, e.g. erosion repairs and cleaning of the road surface, the initial construction cost will be significantly lower, compared to conventional methods.



- NGL = NATURAL GROUND LEVEL
- DIMENSIONS IN MILLIMETRES
- NOT TO SCALE

Figure 8-1: TYPICAL DIMENSIONS FOR A DITCH ACROSS THE ROAD

8.5.2 Deep cuttings and high embankments

Deep cuttings and high embankments require special attention as far as drainage is concerned. Although deep cuttings and high embankments should be avoided on low volume roads, inter alia by changing the alignment or relaxing geometric standards, this may not be practically possible, especially not in mountainous terrain.

The drainage of cuttings to enhance stability should generally be restricted to simple measures such as cut-off drains unslope and behind the cut face. These are seldom lined but in potentially erodible areas it may be necessary. Larger cuts may require lined drains to remove water from the cut-off drains directly down the face of the slope into foot drains.

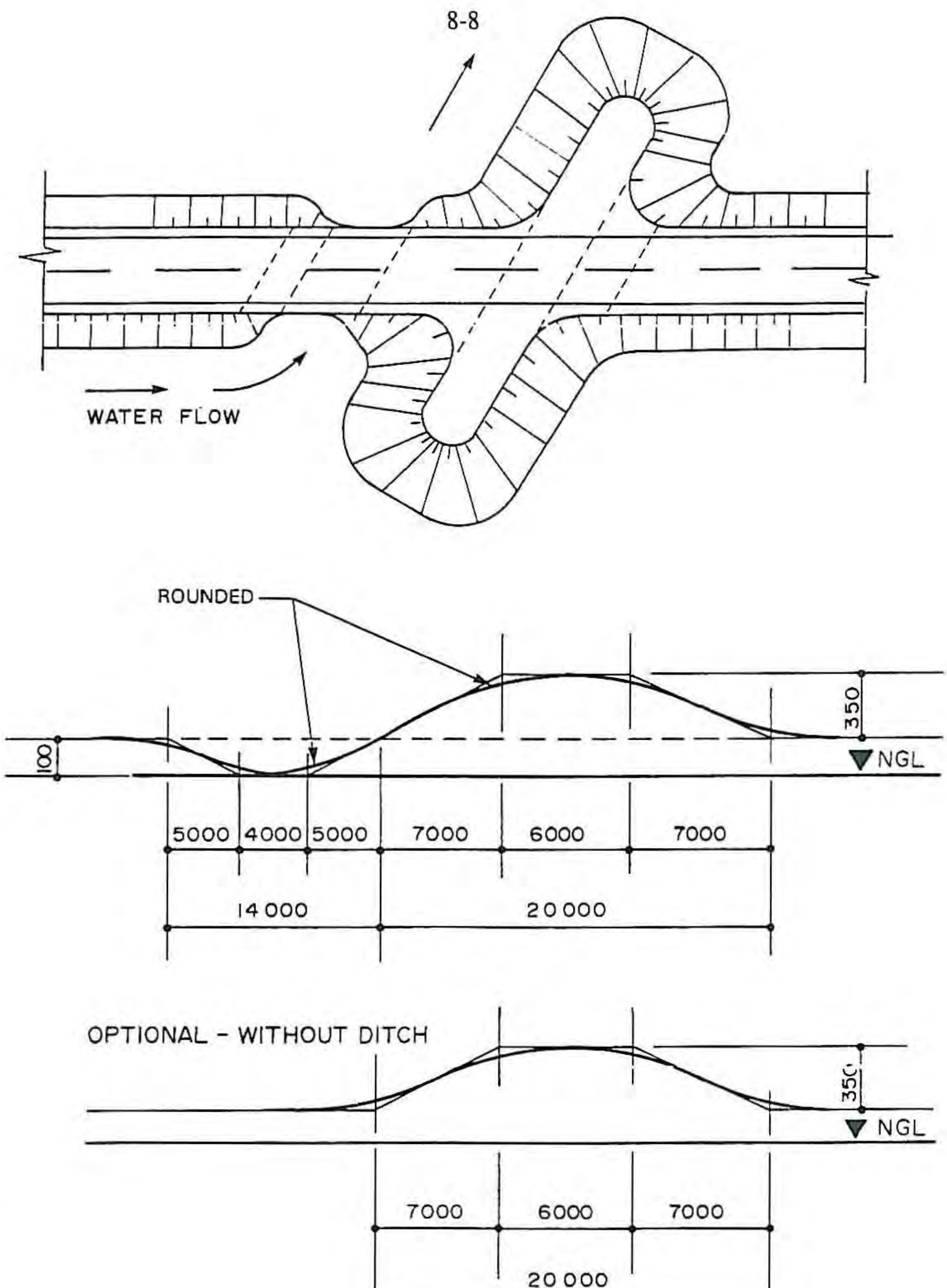


Figure 8-2: **TYPICAL DIMENSIONS FOR DRAINAGE BERM ACROSS THE ROAD**

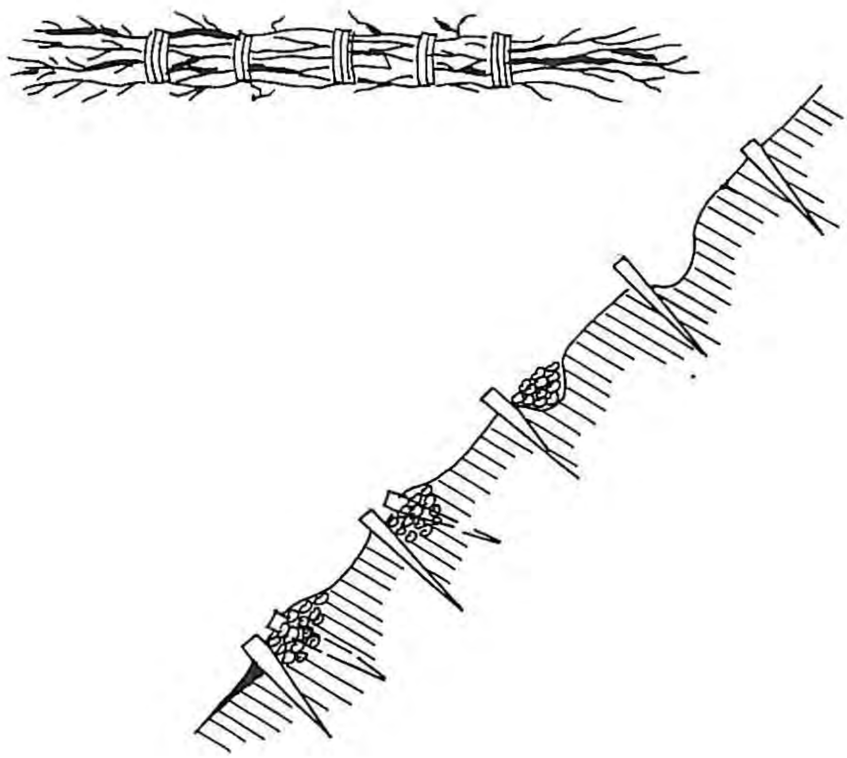


Figure 8-3: **EXAMPLE OF EROSION HEDGES TO BE PROVIDED ALONG THE CONTOURS OF CUTTINGS WHERE EROSION IS EXPECTED**

Unlike cuttings, the necessity for sophisticated drainage works to retain stability of fills is in most cases minimal, except for the provision of structures to convey surface run-off down the fill and to avoid the formation of a dam. Low cost toe drains are usually provided by simply grading a channel next to the fill. Surface drainage facilities should as a rule be minimized on low volume roads, although beams along the road edge may be necessary to control surface run-off and erosion.

The erosion of the faces of slopes is common in southern Africa, especially in the mountainous, wetter easter areas of the sub-continent. Slopes should be made as steep as is possible (and acceptable from a stability point of view) to minimize the exposure of cut and fill faces to rain and water. In some instances erosion can be prevented by establishing vegetation (for example as is shown in Figure 8-3). If the slopes are too steep (more than 1:1,5) for the effective propagation of suitable vegetation lined chutes at regular intervals should be considered.

Vegetation establishment on slopes should only be done if definitely necessary, and arrangements should then be made to ensure that vegetation is watered during the initial phase to ensure proper establishment.

In areas subject to windblown sand slopes of 1:3, or preferably 1:4, should be used on fills to minimize deposition of sand on the road surface.

8.6 Road surface drainage

It is important to ensure rapid and efficient discharge of stormwater runoff from the road surface. Where roads are graded close to the general ground level, stormwater runoff should be allowed to spill off the edge of the pavement. Road embankments that are designed to be overtopped in a flood should be protected against scour, if this is deemed desirable.

The cross-sectional shape of a road refers to the slope of the road surface. This slope determines the effectiveness with which water is removed from the road surface during rain. A distinction is made between the following terms:

- Camber: Implies two slopes away from the centre line to the shoulders. This is generally common in the design of a two-lane two-way road.
- Cross-fall: A single slope from shoulder to shoulder.

Camber or cross-slope values recommended for sections of the road which are not superelevated, are shown in Table 8-2.

Table 8-2: CROSS-SECTIONAL SLOPES[1]

ROAD TYPE	CARRIAGEWAY SLOPE (%)		SHOULDER TYPE	SHOULDER SLOPE (%)		CAMBER (Ca) OR CROSSFALL (Cr)
	TOPOGRAPHY			TOPOGRAPHY		
	FLAT	OTHER		FLAT	OTHER	
Single carriageway roads (2 lanes)						
Paved	2,5	2,0	Paved	2,5	2,0	Ca
			Gravel	3,5	3,0	Ca
Gravel	3,5	3,0	Gravel	3,5	3,0	Ca
Earth	4,5	4,0	Earth	4,5	4,0	Ca
Single lane roads						
Paved	3,5	3,0	Gravel	3,5	3,0	Cr/Ca
Gravel	3,5	3,0	Gravel	3,5	3,0	Cr/Ca
Earth	4,5	4,0	Earth	4,5	4,0	Cr/Ca

1 TPA: Roads Branch uses 2 % on paved and 3 % on gravel roads, which is considered to be sufficient. The Kwazulu Department of Works recommends 5 % on all gravel and earth roads for all topographical conditions.

From Table 8-2 it follows that two lane single carriageway roads should be built with a camber, i.e. two slopes away from the centre line to the shoulders. Single lane roads could be built with a crossfall to facilitate the construction thereof, or could have a camber, for example where labour intensive methods are used.

Refer to Chapter 3 for a discussion on the other elements of the cross-sectional shape.

8.7 Drainage channels

8.7.1 General

This section concentrates on the control of stormwater runoff from the road pavement, and on diverting stormwater away from the road. The control of stormwater from the road surface should be by means of channels or gutters while catchwater drains, mitre banks and beams are used outside the road prism.

Drainage channels include gutters formed by kerbs and asphalt beams, chutes, side drains,toe or fill drains and channels formed by catchwater beams and mitre banks. The design of a drainage channel to carry a given discharge is accomplished in two parts. The first part involves deciding on a cross-section that will carry the design discharge on a given slope.The second part of the design is the determination of the degree of protection required to prevent or minimize erosion.

Return period of the design flood

A flood return period of two years is recommended for drainage channel calculations (see Table 8-1).

Alignment and grade

The road reserve width usually allows little choice in the alignment of the grade of the channel, but abrupt changes in alignment should be avoided. A drainage channel should be graded to produce velocities that neither erode nor cause deposition in the channel. Special attention should in this regard be given to channel widths.

Table 8-3: PERMISSIBLE VELOCITIES IN EXCAVATED CHANNELS

SOIL TYPE OR LINING (NO VEGETATION)	PERMISSIBLE VELOCITY (m/s)		
	CLEAR WATER	WATER CARRYING FINE SILTS	WATER CARRYING SAND AND GRAVEL
Fine sand (non-colloidal)	0,45	0,75	0,45
Sandy loam (non-colloidal)	0,55	0,75	0,60
Silty loam (non-colloidal)	0,60	0,90	0,60
Ordinary firm loam	0,75	1,05	0,70
Volcanic ash	0,75	1,05	0,60
Fine gravel	0,75	1,50	1,15
Stiff clay (very colloidal)	1,15	1,50	0,90
Graded, loam to cobbles (non colloidal)	1,15	1,50	1,50
Graded,silt to cobbles, (non colloidal)	1,20	1,70	1,50
Alluvial silts (non-colloidal)	0,60	1,05	0,60
Alluvial silts (colloidal)	1,15	1,50	0,90
Coarse gravel (non-colloidal)	1,2	1,85	2,00
Cobbles and shingles	1,5	1,70	2,00
Shales and hard pans	1,85	1,85	1,50
Rock	Negligible scour at all velocities		

Permissible slopes

The longitudinal slope of a drain must be adequate to avoid silting and below a certain grade to avoid scouring and erosion. The minimum slope required to avoid silting is 0,6 to 0,8 per cent, and the slope should never be flatter than 0,5 per cent for an unlined drain and 0,33 per cent for a drain lined with concrete or asphalt. The maximum slope should not preferably not exceed 5 per cent for an unlined drain. In some cases circumstances many dictate higher slopes, even up to 10 per cent or more, in which case the lining of drains or other erosion protection measures should be provided.

The Road Drainage Manual (NTC, 1986) provides the minimum recommended velocities at different depths to prevent the deposition of fine sandy material. Guidelines on the maximum safe velocities for unlined drains are given in Table 8-3 for a maximum water depth of 1 m. Table 8-4 gives permissible velocities for various grass lined channels, also for a maximum water depth of 1 m.

Table 8-4: PERMISSIBLE VELOCITIES FOR CHANNELS LINED WITH VARIOUS GRASS COVERS

TYPE OF LINING	PERMISSIBLE VELOCITY (m/s)
Well established grass on any good soil	1,8
Meadow type grass, e.g. bluegrass	1,5
Bunch grasses, exposed soil between plants	1,1
Grains, stiff stemmed grasses that do not bend over under shallow flow	0,9

To reduce the cost of low volume roads, drains will normally be unlined. Where lined channels are, however, required, the following materials can be considered:

- Rip-rap and stone pitching
- Prefabricated paving blocks or grass blocks
- Gabions
- Concrete

Rip-rap and stone pitching should be used where suitable stone is readily available and where labour intensive methods are employed. Prefabricated paving blocks should only be used in exceptional cases due to the high cost of the product. The thicknesses of concrete channel linings recommended is shown in Table 8-5.

Table 8-5: THICKNESS OF CONCRETE CHANNEL LININGS

TOTAL CHANNEL DEPTH (m)	THICKNESS (mm)
0 - 0,5	60[2]
0,5 - 1,5	75
> 1,5	100

Allowable maximum side slopes are shown in Table 8-6.

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2 TPA: Roads Branch pointed out that a thickness of 60 mm of concrete may be difficult to build accurately, in which case the thickness should be increased.

Table 8-6: ALLOWABLE MAXIMUM SIDE SLOPES FOR UNLINED CHANNELS

IN SITU MATERIAL	MAXIMUM SIDE SLOPE
Rock	Almost Vertical
Stiff clay, soil with concrete lining	1:1 to 1:2
Soil with stone pitching	1:1
Large earth channels	1:1
Firm clay or small earth channels	1:1,5
Loose, sandy soil	1:2,5
Sandy clay, porous clay	1:3
Grassed channels	1:3 to 1:4

Freeboard and provision for wave action

Freeboard in drainage channels is the difference in height between the flow surface level and the top of the channel or gutter. Where there are no other restrictions, the following minimum values for freeboard are recommended for road drainage:

Channel section	$Fr < 1$	$Fr > 1$
Rectangular:	0,15 E	0,25 y
Trapezoidal:	0,20 E	0,30 y

where:

Fr = Froude number

E = specific energy = $y + v^2/2g$

y = depth of flow at deepest point

v = average velocity

Erosion protection

The amount of erosion control and maintenance can be minimised largely by the use of:

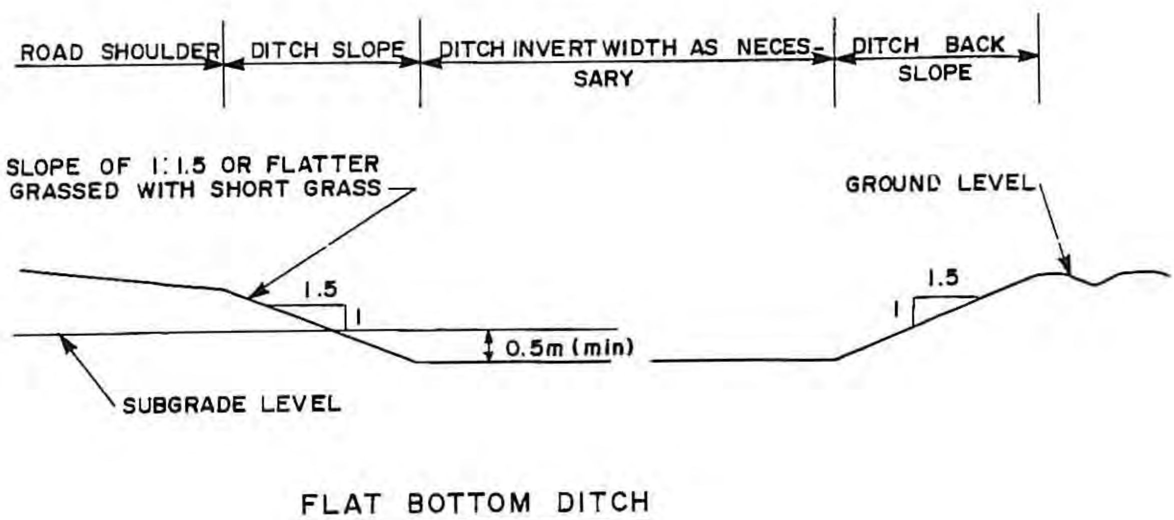
- Flat side slopes, rounded and blended with the natural terrain
- Mitre banks and beams
- Protective coverings such as concrete lining, stone pitching, grass sodding and hydroseeding.

Where these are not possible or economic, every attempt should be made to maintain a sufficiently flat channel grade. Alternatively check dams acting as weirs could be installed in the channel at regular intervals (refer to Section 8.7.9).

8.7.2 Side drains

Side drains are encountered outside the shoulder breakpoint and parallel to the road centreline. They are normally used to drain cuttings. The term is also used to describe drains constructed on road fills to prevent water from flowing down the embankment. This drain may consist of a concrete channel, or may be formed with concrete kerbs or asphalt beams. Side drains are also referred to as table drains or drainage ditches.

In general side drains should be flat bottomed and should be on the upper side of the crossfall of the formation. V-shaped side drains are prone to erosion and should only be used where it is not possible to use a flat bottomed drain. Figure 8-4 shows examples of the typical shapes, and Figure 8-5 examples of lined side drains.



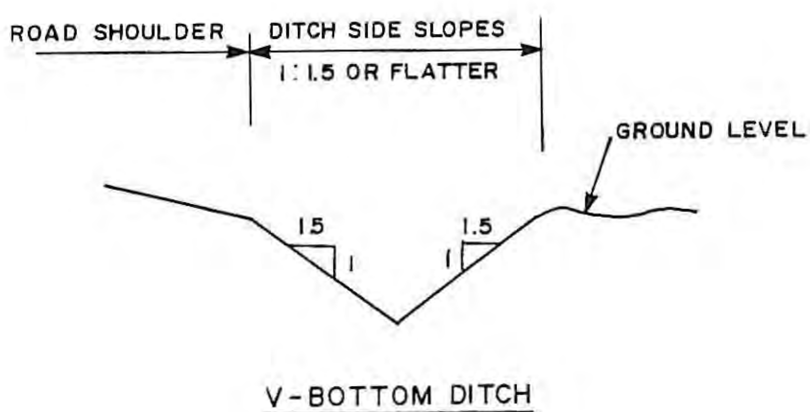
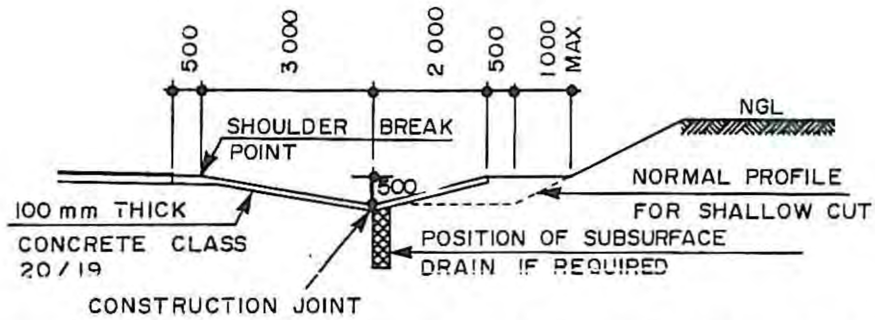
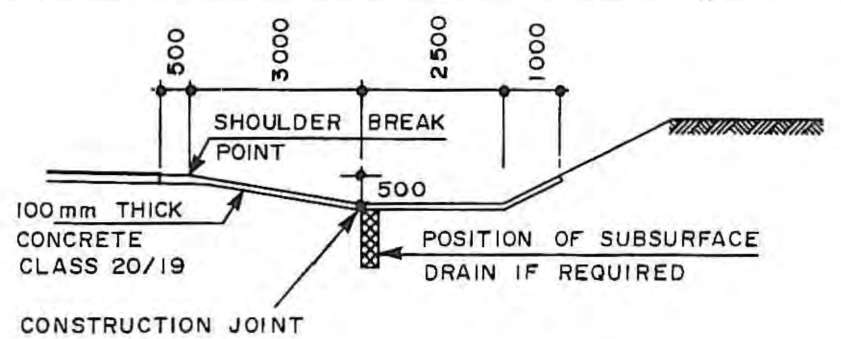


Figure 8-4: **TYPICAL SIDE DRAIN SHAPES**



CROSS SECTION OF TYPE 1 CONCRETE SIDE DRAIN



CROSS SECTION OF TYPE 2 CONCRETE SIDE DRAIN

Figure 8-5: TYPICAL LINED SIDE DRAINS

Side and mitre drains should be designed with widths and side-slopes (1:2 to 1:3) which permit ready access of a motorised grader so that maintenance can be carried out during the routine gravel surface maintenance.

The invert of side drains should be at least 0,5 m below the subgrade breakpoint to ensure that water that infiltrates the side drains does not affect the pavement layers. In active clays an extra 600 mm must be excavated and replaced be selected compacted material when concrete lining is provided. Flexible linings are, however, preferred for such conditions.

8.7.3 Berms

A berm is either a ridge placed on the top edge of a road in fill, or an earth embankment used to redirect the flow of water. In the first case the berm is used to prevent erosion caused by run-off down the side of the fill. In this case the berm is normally of a permanent nature, asphalt or concrete is used as the construction material and is only provided on paved roads. In the second case the berm serve to cut off a side drain and to direct water into a relief culvert. In this case a berm is normally constructed of soil, and may be protected with, for example, stone pitching.

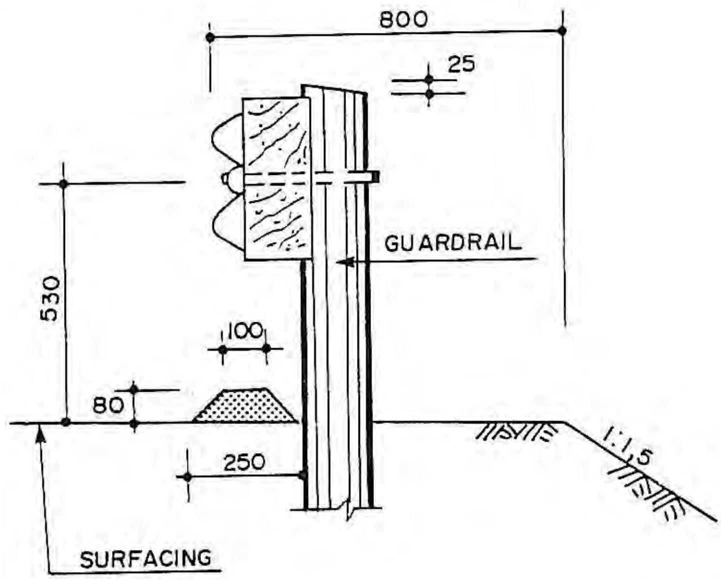


Figure 8-6: EXAMPLE OF AN ASPHALT BERM

High fills should be provided with asphalt type beams or concrete kerb and channel combinations to prevent erosion of the fill slope. This is particularly necessary where the vegetation is sparse and not lush enough to bind the soil adequately. Water collecting along such gutters should be discharged at regular intervals into chutes down the fill slope. The chute interval will depend on the gutter capacity. Capacity depends on the cross-section,grade and roughness of the gutter. The method for calculating the channel capacity can be found in the Road Drainage Manual (NTC, 1986).

As a result of traffic safety requirements, permanent berms are normally only used together with guardrails. In the case of low volume roads guard rails should, however, only be provided where absolutely necessary.

Outlets of berms and kerbs must be so placed that:

- rain water is removed from the road surface effectively
- adequate freeboard is allowed along the kerb or berm
- deep rapid flow over the road shoulder is prevented
- intermediate outlets will intercept at least 80 % of the flow occurring at their positions, and the lowest outlet will accommodate 100 %.
- the total cost of the combination of outlets plus discharge chutes is kept to a minimum.
- unnecessary concentration of water is prevented.

8.7.4 Catchwater drains and banks

Catchwater drains and banks are situated on the top edge of cuttings and are constructed to prevent water from flowing down the face of the cutting. A catchwater drain is usually placed upslope of a catchwater bank to prevent water from flowing over the cut face. The dimensions vary as a function of the amount of water that need to be conveyed. Figure 8-7 shows typical dimensions of catchwater drains and banks. In some instances the excavation of a catchwater drains is discouraged because excavation over termite workings, burrows,old tree roots, etc could lead to problems of piping and instability of the cut face. Figure 8-8 shows an example where only a catchwater bank is provided.

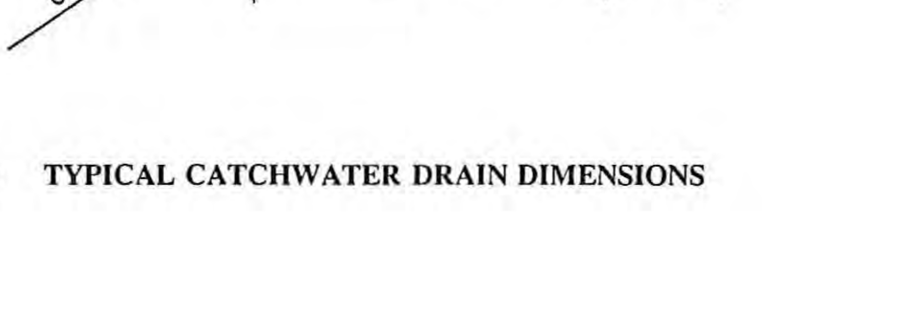


Figure 8-7: TYPICAL CATCHWATER DRAIN DIMENSIONS

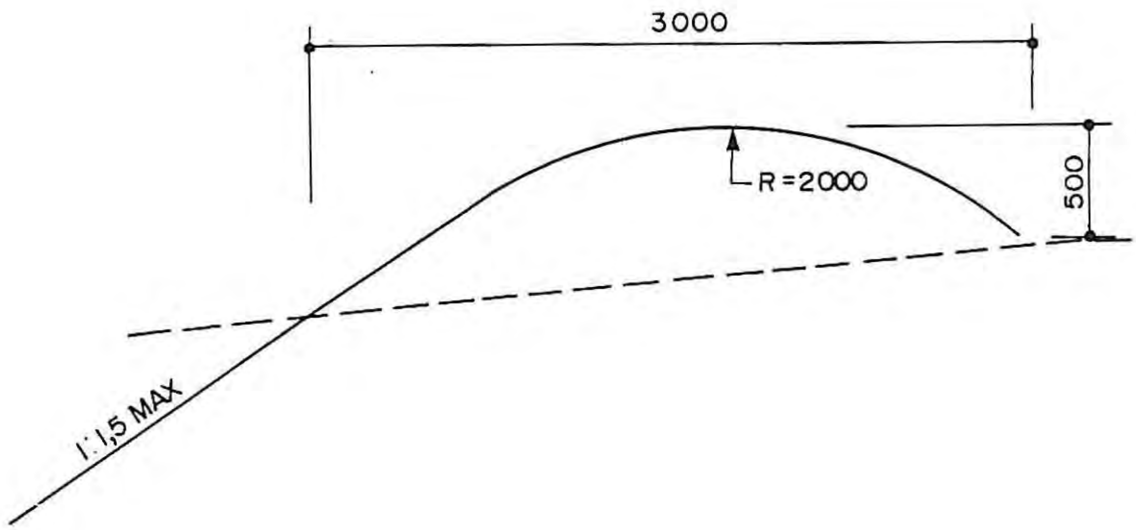


Figure 8-8: **TYPICAL CATCHWATER BANK**

The proposed layout of the catchwater drain, side drain and toe drain at the transition from cut to fill is shown in Figure 8-9.

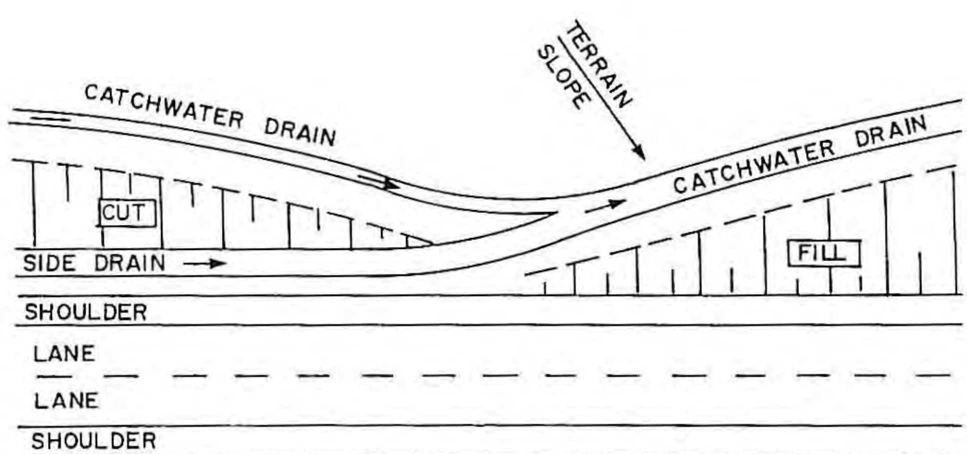


Figure 8-9: **DRAIN LAYOUT AT TRANSITION FROM CUT TO FILL**

8.7.5 Toe drains

A toe drain is used at the toe of an embankment to remove water that may tend to collect there. The dimensions vary as a function of the amount of water that need to be conveyed. Figure 8-10 shows typical dimensions of toe drains.

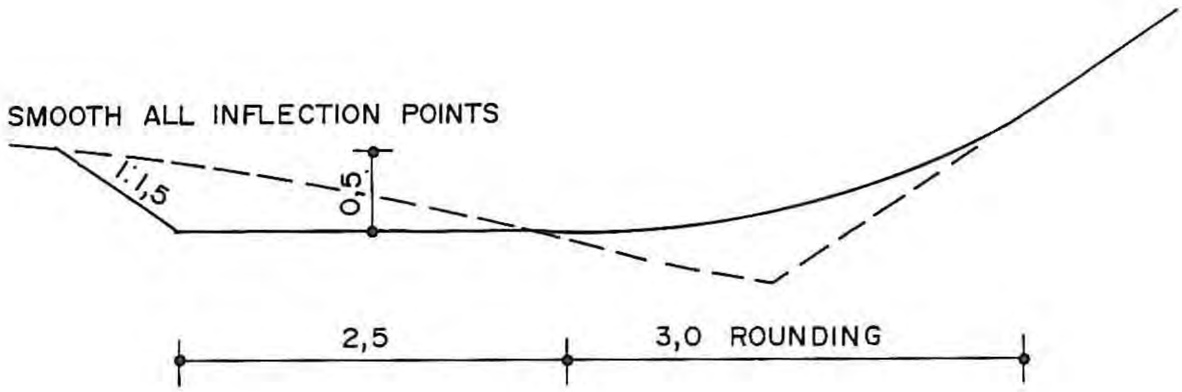


Figure 8-10: TYPICAL TOE DRAIN DIMENSIONS

Where necessary, toe protection is to be provided, as shown in Figure 8-11(a) where there is little space, or Figure 8-11(b) where there is more space available.

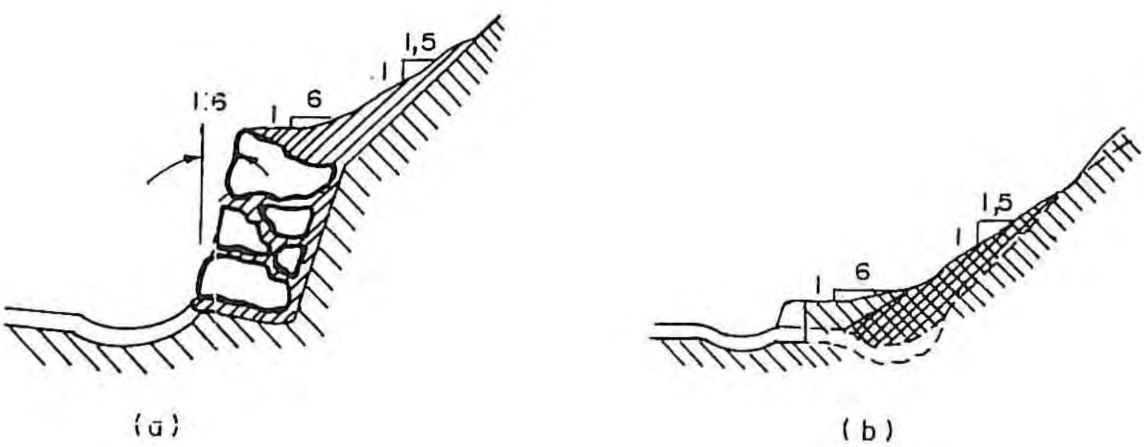


Figure 8-11: TYPICAL TOE PROTECTION MEASURES

8.7.6 Kerbs

Kerbs are normally used on bridges and at road intersections. A kerb also serves as a stormwater channel with a capacity greater than that of a concrete or asphalt berm. Due to the high cost of kerbs they should only be used on low volume roads when absolutely necessary.

8.7.7 Mitre drain and bank

A mitre drain (or discharge drain) is used to drain water from the side drain into the veld. A mitre bank is usually constructed parallel to the mitre drain, helps to change the direction of water flow and prevents water from flowing over the mitre drain.

Mitre drains must be provided at regular intervals to transport the water from the side drains or culverts to a place away from the road structure where the water can be discharged. The hydrological design of the mitre drains must be such that the water flow is always below the subgrade breakpoint. Large side drains will therefore require fewer cross drains. The design must also take into account the amount of silting up that is likely to occur between maintenance activities and which will effectively reduce the drainage capacity.

8.7.8 Down chutes

Down chutes are used on fills and cuttings to accommodate concentrated waterflow down the fill or cutting in order to prevent erosion. An example is shown in Figure 8-12. Down chutes may consist of either a pipe or an open channel[3]. In the case of a pipe corrugated metal pipes are normally used, while in the case of an open down chute, precast sections may be used, or the chute may be constructed using concrete or stone pitching. On low volume roads pipe chutes are not favoured because they are generally more expensive. On the other hand down chutes constructed by means of stone pitching is very appropriate for low volume roads because of the utilisation of natural materials. This method of construction is also suitable for labour intensive methods.

Down chutes have the same slope as the road formation. As a result of the often steep slopes special precautions are necessary at the bottom end of the chute to ensure adequate dissipation of energy. The dimensions of down chutes are determined as a function of the quantity of water that need to be accommodated.

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3 TPA: Roads Branch does not recommend the use of open chutes due to erosion problems. A 300 mm diameter Armco type closed downpipe just underneath the soil surface was found to perform better.

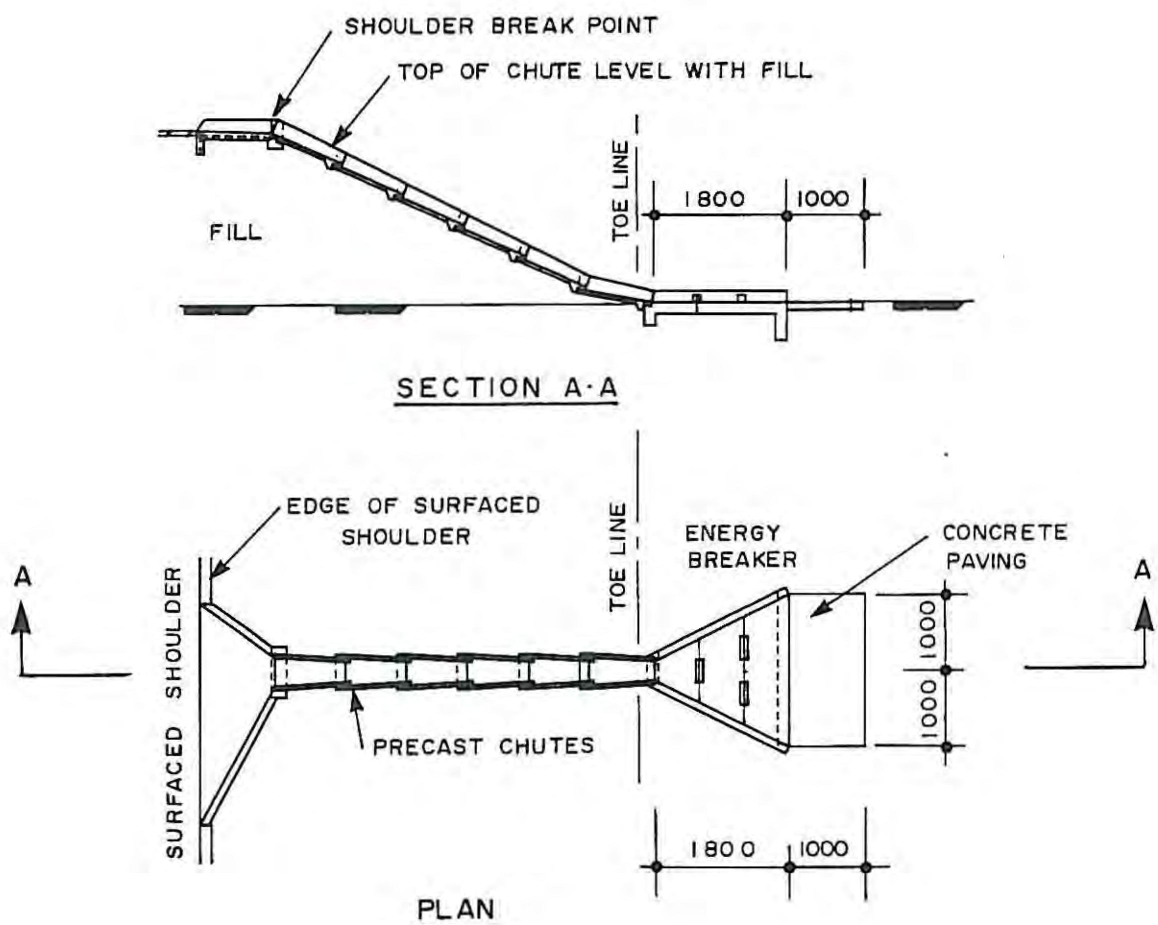
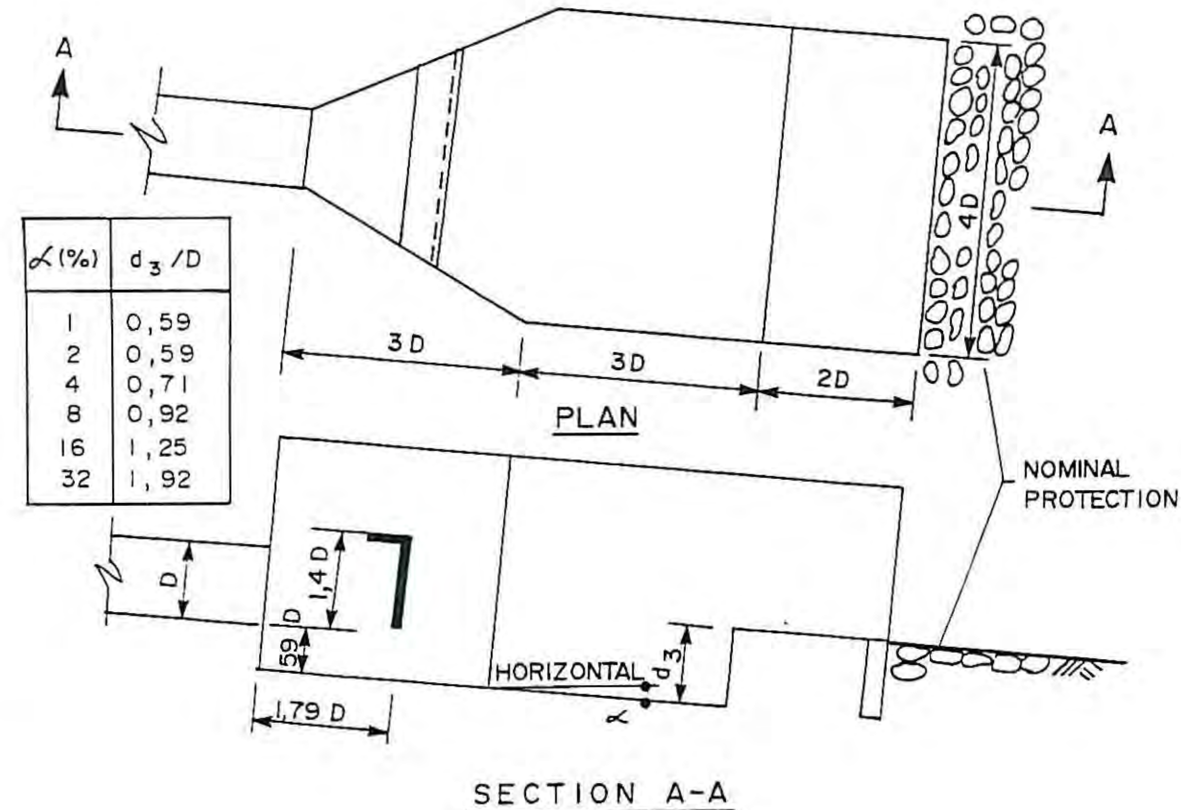


Figure 8-12: TYPICAL DETAILS OF A DOWN CHUTE

8.7.9 Energy dissipators

The function of energy dissipators is to reduce the energy of flowing water. Energy discripators are used on steep side drains (say 5 per cent and more) at culvert outlets, at the bottom end of chutes, etc.

Energy dissipators usually consist of still basins, protruding blocks or check dams. A stillbasin is used to dissipate the energy of water flowing in a channel or through a culvert. Examples of still basins are shown in Figures 8-13 and 8-14 (also referred to as a scourhole). Still basins can cause extra maintenance work and should be used sparingly. Protruding blocks are often used to dissipate energy where water is released from a channel or culvert into the veld.



∝ (%)	d_3/D
1	0,59
2	0,59
4	0,71
8	0,92
16	1,25
32	1,92

Figure 8-13: EXAMPLE OF A STILL BASIN

A check dam[4] is a wall or berm constructed across a drain at regular intervals to reduce the slope of the drain and to dissipate the energy of the water. Checkdams (also referred to as dropwalls) placed at proper intervals can go a long way to prevent erosion. Check dams should be placed such that the gradient between check dams are between 1:70 and 1:100. The check dam height should be between 0,5 and 0,7 of the side drain depth.The check dam should have a lower portion in the middle for the water to flow over in order to prevent the sides of the channel from being eroded.

4 The TPA: Roads Branch does not support check dams as it is felt that they cause flood damage. In areas such as Lebowa, however, they have been used very successfully

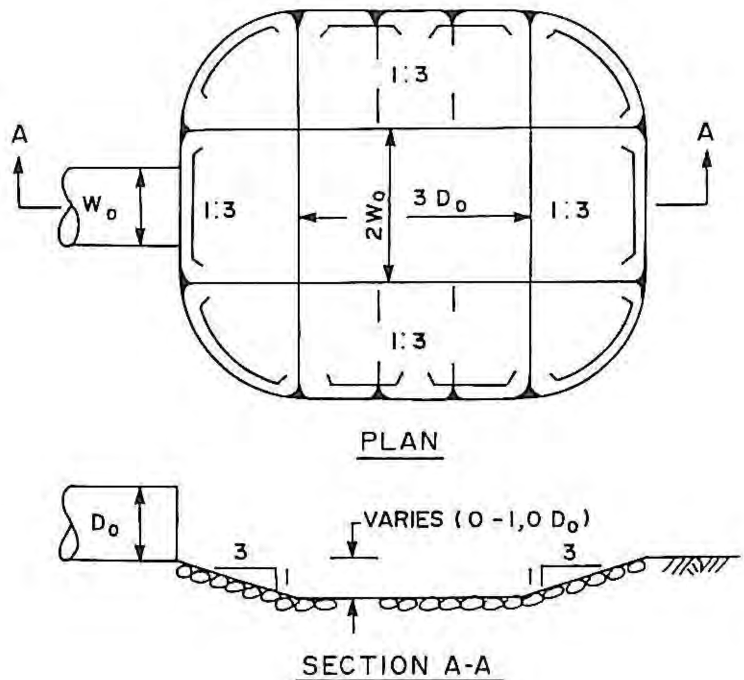


Figure 8-14: EXAMPLE OF PREFORMED SCOUR HOLE

8.8 Culverts

8.8.1 Hydraulic performance

A culvert is used to convey water from the upstream to the downstream side of a road. This water may either flow in a channel that crosses the road, or it may come from the side drain. In the latter case the culvert is referred to as a relief culvert.

Hydraulic performance is one of several factors which influence the selection of the type of culvert for a particular location. The shape and type of culvert to be used should, however, not be determined by hydraulic performance alone, as the economics and availability of different culvert types should also be considered.

8.8.2 Culvert types

Purchase and transport costs play a major role in selecting the culvert type for a particular application. The advantages of corrugated metal nettable pipe sections should be considered for small to medium sized culverts and the corrugated metal multi-plate arch for larger culverts. Metal corrosion is normally not a problem in the interior of the country but these culverts will tend to erode in coastal areas or in mining areas where the pH can be well below 6. Prefabricated concrete pipe and portal units are encouraged wherever they can be economically and practically justified. Where the above culverts are not economical, eg. due to a remote location, in situ concrete culverts may be specified if the required aggregates can be obtained. Arch culverts built of local materials are also often cheaper than prefabricated pipe or portal culverts and are labour intensive. Another option to be considered is large concrete bricks and small reinforced slabs fabricated on site for small box culverts.

In the selection of an appropriate culvert type cognizance should be taken of the in-situ soil conditions. In flat poorly drained or expansive soil conditions culverts must be watertight. If not, the in-situ soils will expand, resulting in poor riding quality on the road surface. Corrugated metal pipes are normally not watertight and should therefore only be used in well-drained areas with low plasticity in-situ materials. Rubber ring seals should be used with ogee jointed concrete pipes if watertightness is required.

8.8.3 Loading

Culverts of whatever type will be subjected to loads. The primary loads which should be considered are the following: self mass, water mass, mass of backfill, traffic loads, temporary handling and construction. These factors, individually or collectively, influence the type and class of culvert installed and must be taken into account in the design.

8.8.4 Return period

From Table 8-1 follows that culverts, which will normally only be used for a 1:20 year peak discharge not exceeding 20 m^3/s, should be designed for the 5 year flood for surfaced and unsurfaced low volume roads.

8.8.5 Design

The balance in the natural watercourse between flow and erosion processes should be disturbed as little as possible.

Flow should be concentrated as little as possible. Although the Road Drainage Manual(NTC, 1986) recommends that culverts should preferably not be further apart than 100 m, this approach may result in unnecessary high costs in the case of low volume roads. The spacing of culverts for low volume roads should therefore be determined by taking into account the specific conditions encountered along the road. If the facilities are not readily available to carry out a detailed investigation and the design of pipe culvert cross drainage, the following rule of thumb method can be used to either estimate the required or check the existing drainage system where the vertical alignment has been built to a *rolling* grade, i.e. with minimum cuts and fills:

Flat country:	One 600 mm diameter pipe every 600 to 800 m along the road centreline
Rolling country:	One 600 mm diameter pipe every 300 to 500 m along the road centreline
Hilly country:	One 600 mm diameter pipe every 200 to 300 m along the road centreline
Mountainous country:	One 600 mm diameter pipe every 200 m along the road centreline

It is important to note that when a road is under-culverted, the risk of silting-up of the table drains will occur, depending on the terrain, which will require increased maintenance. The direction of flow should also be disturbed as little as possible. The outlets should be located on the stream alignment to avoid scour.

Water velocities should be altered as little as possible. Culverts should be laid to grades that produce a non-silting or a non-erosive velocity, ideally between 1,0 and 3,5 m/s. To prevent deposition, flow velocities through culverts should not be less than 1 m/s, and the minimum slope of a culvert should accordingly not be less than one per cent. The maximum slope of a culvert is governed by the flow velocity at the outlet. If necessary this velocity must be reduced with energy dissipators to prevent scouring of the natural material downstream of the culvert.

For maintenance purposes the minimum acceptable size for a culvert is 600 mm in diameter or 750 x 450 mm rectangular.

The reader is referred to the Road Drainage Manual (NTC, 1986) for detailed guidelines on the design of culverts.

8.8.6 End structures

Inlet and outlet structures are required to prevent scouring of the roadway embankment, to provide a transition from the channel to the culvert and to improve the hydraulic performance of the culvert.

With low volume roads the emphasis in the design of end structures should be on the cost saving aspect. Unless hydraulic or other considerations require special end structures, the least expensive type of end structure should be adopted. Where possible stone pitching or masonry should therefore be used, rather than concrete, because the cost is generally lower. In the case of concrete structures consideration should be given to a standard set of adjustable steel shutters to reduce costs.

8.8.7 Scour

Unchecked erosion is a prime cause of culvert failure. The greatest scour potential is at the culvert outlet where high velocities may necessitate scour protection or energy dissipation. Scour can be eliminated by the effective use of gabion mattresses, rip-rap, stone pitching (plain or grouted), in situ concrete, concrete blocks and cutoff walls at the culvert exit. The choice of protection to be used should be determined by material availability and cost at site.

8.8.8 Selection of drainage structures

At each waterway crossing a road, a decision has to be made whether a culvert or a low level structure is more appropriate. The following guidelines are provided in this regard:

- The cost of a culvert structure should be compared to the cost of a low level structure. For small catchment areas (typically smaller than 10 km²) culverts tend to be cheaper. In the case of larger catchment areas low level structures generally offer the solution with the lowest cost. Culverts will, however, be used in larger catchment areas where it is not possible to construct low level structures, for example due to geometric limitations the unacceptably of temporary inundation, etc.

- As far as the topography is concerned, culverts tend to be more appropriate in mountainous areas where fills are required at low points due to vertical alignment parameters. In flat areas low level structures are generally more appropriate.

- Culverts are designed for a higher return period than low level structures. They will therefore be preferred on roads where the occasional disruption of traffic flow due to flooding is not acceptable.

8.9 Low level structures

8.9.1 General

A low level structure (LLS) is a submersible river crossing and is designed to experience no or limited damage when submerged. This type of structure is appropriate where inundation of the road for short periods is acceptable. The length of the structure required is often less than for a high level bridge. In general the cost of these structures are also much lower than high level bridges. LLS's are categorized as causeways and low level bridges.

Causeways

A destination is made between fords, drifts and vented causeways.

A ford consists of a retaining wall built across a stream, and a layer of material suitable to drive on, behind the wall. The purpose of the wall is to retain the material upstream of the wall. The retaining wall may consist of grouted stone or concrete. Sand or other stream bed material is compacted and a layer of gravel or small stones is constructed on top of this material. With low flow speeds water flowing over the ford causes minimal damage to the gravel layer. An example of a ford is shown in Figure 8-15.

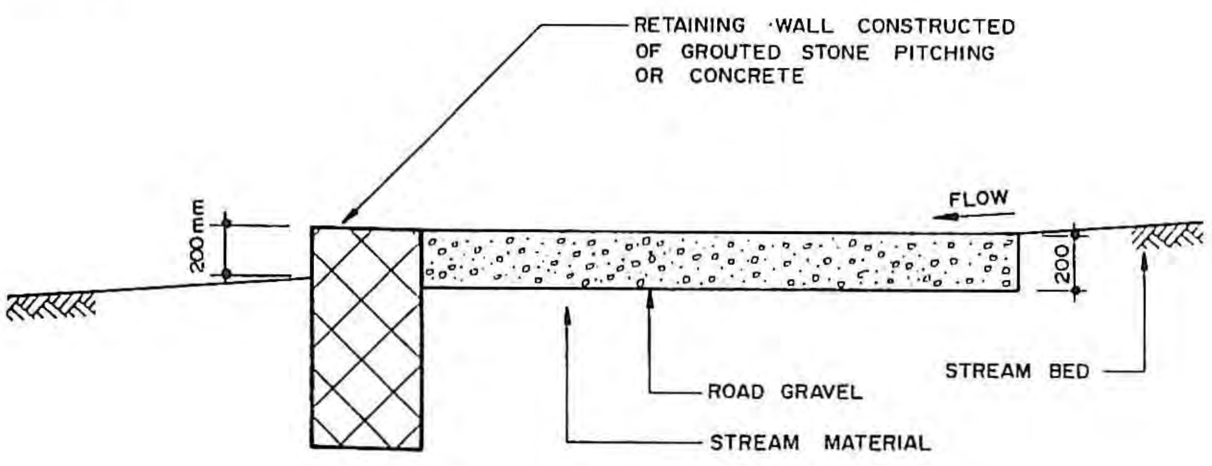


Figure 8-15: EXAMPLE OF A FORD

A drift consists of permanent surfacing through the river bed which allows water to flow over the structure. A drift is normally constructed with concrete or grouted stone. Lengthy periods of inundation may occur during the rainy season, but vehicular travel can resume immediately the flood flow subsides. Examples of drifts are shown in Figure 8-16.

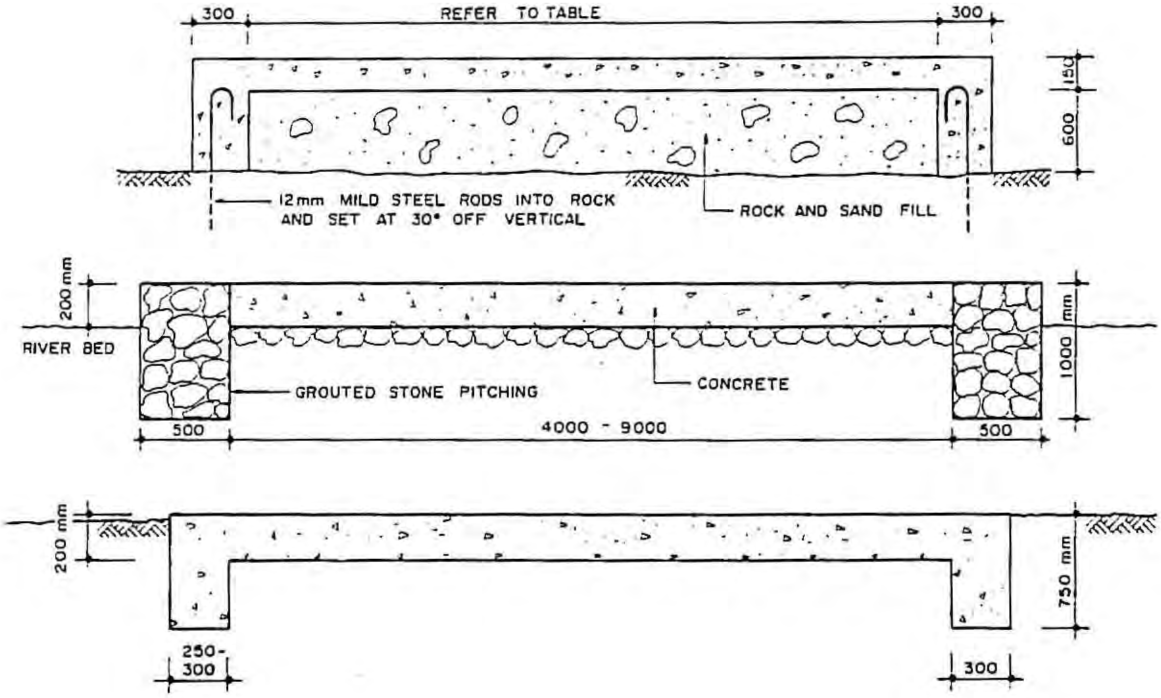


Figure 8-16: **EXAMPLES OF A DRIFT**

A vented causeway consists of a number of void-formers, protected with cut-off walls and a slab. These are normally constructed with concrete or grouted stone. Voids may be formed with pipe culverts, portal culverts or semi-circular corrugated steel arches. An example of a portal causeway is shown in Figure 8-17.

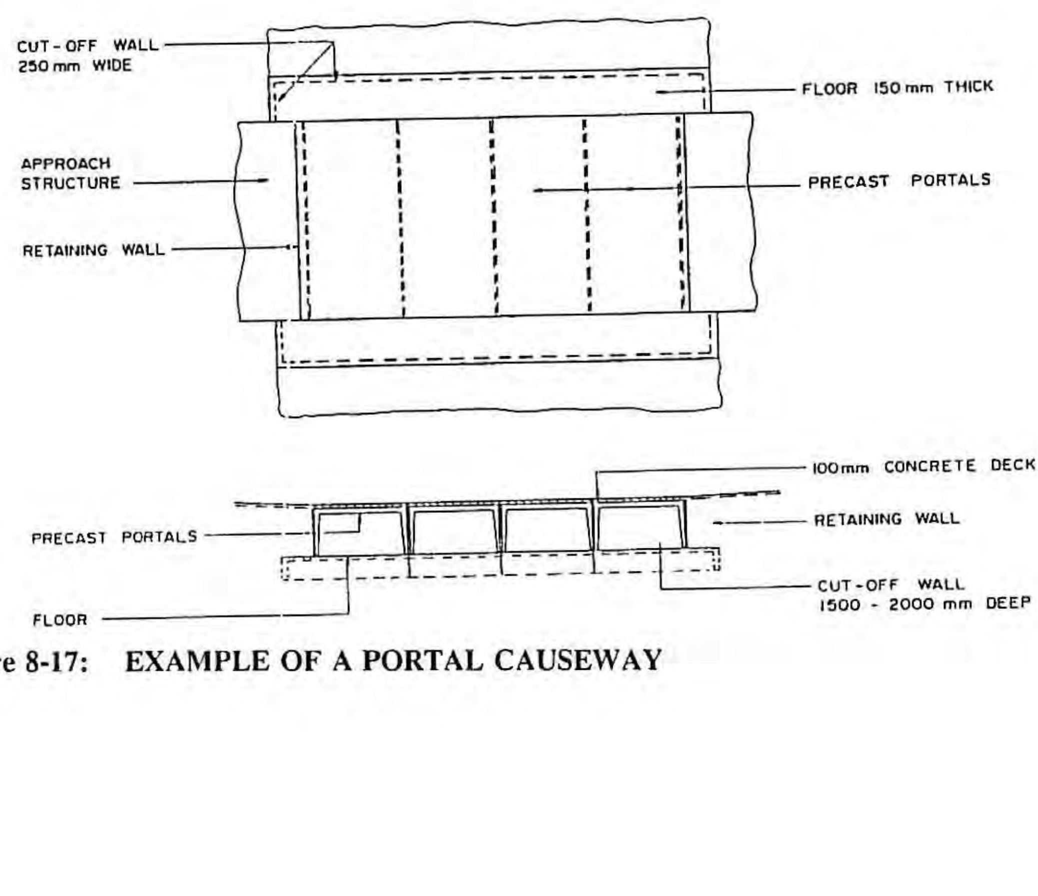


Figure 8-17: **EXAMPLE OF A PORTAL CAUSEWAY**

Arch causeways constructed of masonry are in many instances cheaper than prefabricated culverts or Armco structures of the same size and have the advantage that they are extremely suitable for labour intensive methods of construction.

Low level bridge

A low level bridge is defined as a bridge with a maximum height of 2,0 metres. The piers are normally constructed from masonry or concrete. The concrete decks are in most cases simply supported. The span typically range from 6,0 metres in the case where good founding conditions are available, to between 9 and 11 metre where piling is required.

8.9.2 Warrants for the provision of structures

There are still may cases in rural areas where low volume gravel or earth roads pass through non-perennial river beds without any structure being provided. Each time water flows in the river, a maintenance team (normally a grader) has to go out to the site to clear the river bed. Economic analysis is of great assistance in determining warrants for the provision of structures in such cases.

A model for catchment areas with predefined characteristics was developed for such an economic analysis (Pienaar, 1993). This model predicts the following:

- The total time period per year that certain flows can be expected to be exceeded. These flows are expressed as fractions of the flow with a 1:2 year return period.
- The number of times that specific flows can be expected to be exceeded per year.
- The average duration of these flows.

For the cases investigated it was found that when compared to the "Do nothing alternative", low level structures were warranted in most cases. One of the main reasons for this was the relatively high cost to send out a grader or other equipment to repair the crossing every time water has flown in the river.

8.9.3 Construction cost

The construction costs of bridges vary considerably, depending on the bridge type. In general, however, the cost of bridge structures forms a large proportion of the cost of road infrastructure. In the light of the low rainfall characteristics of South Africa there is considerable room for cost savings by rationalising bridge types in terms of the road network hierarchy. While high level bridges are in general justified on primary roads, low level structures should be provided on tertiary roads, which consist mainly of low volume roads. In the case of widespread floods most tertiary roads will then be cut off, while the primary road network will still be open to traffic. Depending on the function and location of secondary roads in the road network, they may be provided with either high level bridges or low level structures.

It is therefore clear that at each waterway on a low volume road justifying a bridge, a decision has to be made whether a low level structure or high level bridge is more appropriate. The following guidelines are provided:

- The cost of the low level structure should be compared to the cost of the high level bridge. As long as the cost of the low level structure is significantly lower, high level bridges should only be provided in cases where it is not possible to construct low level structures, for example due to geometric limitations, etc. When the additional cost of providing a high level bridge is marginal, for example when difficult founding conditions which require expensive measures are encountered, a high level bridge could be warranted.

- If it is not acceptable that disruptions of traffic flow occur, a high level bridge should be selected.

8.9.4 Design method

The design method is based on the definition of various design levels, which provides an indication of the level of service to be expected from the structure. The implications of design levels were determined by analyzing historic data for 41 hydrological gauging stations of the Department of Water Affairs. These stations are all situated in drainage regions A, B and X, which covers the largest part of the Transvaal. Data for an average period of 20 years per station was analyzed (Pienaar, 1993).

Three design levels are defined, as shown in Table 8-7. If design level 1 is used the design flow will be exceeded 1,3 times per year on average and the average flood duration will be 9 hours (as is shown in Table 8-7, these values were as high as 4,2 times per year and 30 hours per flood for some of the gauging stations.) If design level 3 is chosen, the design flow will only be exceeded 0,5 times per year on average, and the average flood duration will be 3,4 hours. Table 8-7 describes the implications of the three design levels suggested in more detail.

Table 8-7: LEVELS OF DESIGN FOR LOW LEVEL STRUCUTES

DESIGN LEVEL	f_i	AVERAGE NO OF TIMES EXCEEDED PER YR PER GAUGING STATION			AVERAGE DURIATION PER FLOOD (hrs) PER GAUGING STATION		
		Minimum value	Maximum value	Average value	Minimum value	Maximum value	Average value
1	0,25	0	4,2	1,3	0	30	9,0
2	0,50	0	2,4	0,8	0	13	5,5
3	1,00	0	1,4	0,5	0	6	3,4

The following approach is suggested for the determination of the design level:

- Design level 1 is taken as the initial choice

- The design level is increased to level 2:
  - if the traffic volume exceeds 250 vehicles per day or
  - if the additional length of alternative routes exceeds 20 km.

- The design level is increased to level 3:
  - if the traffic volume exceeds 500 vehicles per day or
  - if the additional length of alternative routes ec\xceeds 50 km.

- Should there be no alternative route available, or should the road be of stategic importance, the designer must choose the design level in terms of the implications described in Table 8-7.

Once the design level is known, the design flood is determined as follos

$$Q_{design} = f_i \, X \, Q_2$$

where: Q_{design} = the design flood

f_i = a dimensionless factor related to the design level chosen and shown in Table 8-7

Q_2 = the flood with a 1 in 2 year return period.

It is not necessary to accommodate the total design flood under the structure - such an approach would have ruled out unvented structures, e.g. concrete slabs. Part of the design flood may be accommodated over the strucutre, provided that it is still save for a vehicle to pass over the strucutre.

The strucutre should therefore be designed in such a way that:

$$Q_o + Q_u \geq Q_{design}$$

where: Q_o = the flow that can be accommodated over the structure for flow depth less than the maximum acceptalbe.

Q_u = the flow capacity under the structure.

As far as depth is concerned, it may be assumed that a vehcile should not pass over a low level structure being overtopped if the depth of flow exceeds the under-body ground clearance height of the vehicle. The flow velocity, however, also has to be taken into account.

The following design values are recommended (Pienaar, 1993):

- Super-critical flow: maximum depth of 100 mm
- Sup-critical flow: maximum depth of 150 mm

8.9.5 Gradients

It is normal to use reduced geometric standards on the approaches to low level structures.The criteria suggested are shown in Table 8-8. Gradients in excess of 10 per cent should only be provided over lengths shorter than 40 m[5].

Table 8-8: MAXIMUM GRADIENTS FOR LOW LEVEL STRUCTURES

DESCRIPTION	DESIRABLE MAXIMUM GRADE (%)	ABSOLUTE MAXIMUM GRADE (%)
Paved roads	12	15
Unpaved roads	10	12

The deck of the structure and the approach road surface should be on a continuous vertical alignment without discontinuity at the junction.

5 TPA: Roads Branch considers a grade of 10 per cent to be dangerous, even if only provided over a short distance. It is felt that the combination of the steep grade and a road surface which will be proned to erosion, will lead to collisions.

8.9.6 Dimensions

A decision on single or double lane width should be made for each individual structure, but where there are several structures in close proximity on a road the overall situation should be considered[6].

Recommended widths for low level structures are provided in Table 8-9. The width is schematically shown in Figure 8-18.

For cross-section types 4, 5, 6a, 6b, 6c, 7, and 9 (see Figure 3-2 in Chapter 3) the use of single lane causeways should be considered where the following circumstances arise:

- The approach gradients are moderate and there is no significant curvature on the immediate approaches
- The length of the low level structure is long and the savings associated with a single lane structure are considerable
- There is good visibility on the approaches to the structure and the structure occurs infrequently on an otherwise good section of road.
- The traffic volume is not expected to exceed 500[7] vehicles per day during the life of the structure.
- Pedestrian volumes are low, typically less than 100 pedestrians in the peak hour.

To prevent the openings of the structure from being blocked, the construction of inclined buttresses on the upstream side of the structure to assist in lifting the floating debris over the bridge, can be considered (Refer to Figure 8-18). Inclined buttresses on both the upstream and down stream side will also help to improve stability in flood conditions.

In general low level structures should be as low as possible in order to allow debris to pass over the strucutres and to minimize the impact of the flowing water on the strucutre.

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6 Natal: Roads Branch expressed to opinion that generally when considering low level structure sthere is not much advantage in building a double lane structure

7 Cape: Roads Branch suggested a maximum of 250 vehicles per day for singel alne strucutres. SABITA recommended 200 and TPA: Roads Branch 100 vehicles per day. A traffic flow analyis has shown that even for long bridges up to 1 000 vehicles per day can easily be accommodated under most conditions.

Table 8-8: RECOMMENDED WIDTH BETWEEN GUIDE BLOCKS FOR LOW LEVEL STRUCTURES (LLS)

CROSS-SECTION TYPE (Refer to Figure 3-2)	TWO LANE STRUCTURE (m)	SINGLE LANE STRUCTURE (m)*
1	LLS not recommended	LLS not recommended
2	LLS not recommended	LLS not recommended
3	8,0	Not recommended
4	8,0	4,0
5	6,0	4,0
6a	8,0	4,0
6b	8,0	4,0
6c	6,0	4,0
7	8,0	4,0
8	6,0	4,0
9	Not recommended	4,0

* Single lane strucutre recommnded where viable refer to text

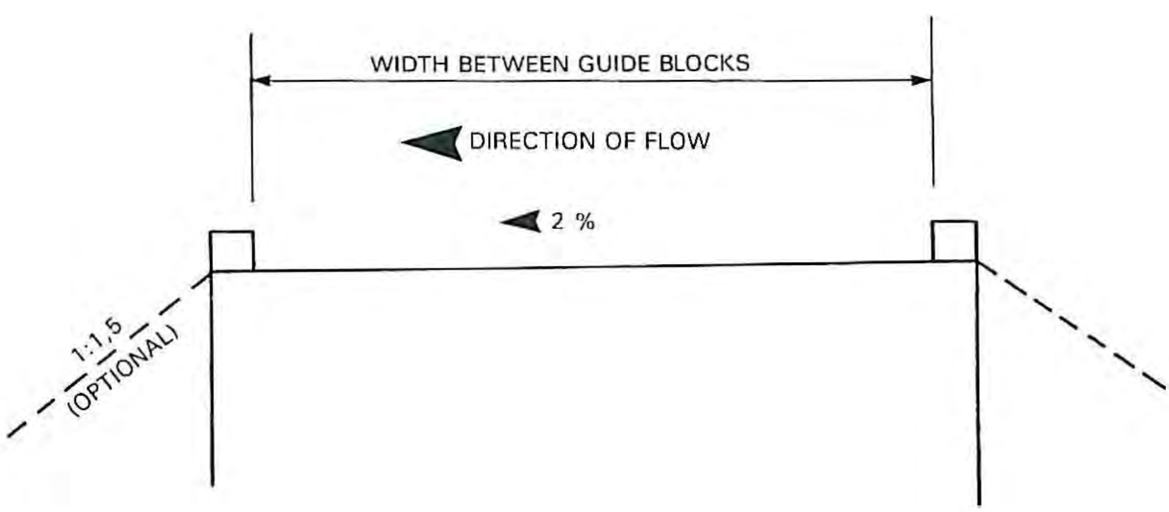

Figure 8-18: MEASUREMENT OF THE WIDTH OF LOW LEVEL STRUCUTRES

Guide blocks are to be provided on the edges of the deck to guide the road user and to assist in gauging the depth of flow over the structure. The guide blocks should be designed to withstand small debris blockages, but to allow large debris to slide across. It is recommended that the standard dimension of a guide block be taken as 250 mm cubical, and that a spacing of 2,0 m be used.

When pedestrian traffic is catered for either raised footwalks or barriers may need to be provided for safety reasons.

The size of the openings in the structure depends on the design flood chosen. However, in the design of openings it is desirable to provide as much waterway area as possible within the normal river channel in order to reduce the obstruction of the structure to river flow. For low level bridges to be successful debris from floods must pass over the structure and not cause the openings to be blocked.

All low level structures should preferably be on a zero grade to avoid varying water depths, concentration of water over the deck and obstruction of debris in the vicinity of the river banks during floods. A crossfall of 2 per cent in the direction of flow is recommended for low level structures to ensure that little/no sediment is left on the riding surface (refer to Figure 8-18).

8.9.7 Design speed

Where practical the design speed over a low level structure should be the same as for the road section.Where this is not possible the design speed may be lowered with 20 km/h (preferably), and up to 40 km/h if absolutely necessary[8].

The rates of curvature (K values) for the design of structures are obtained from Tables 3-10 and 11. In the case of a design speed of 20 km/h and lower a K value of 2 can be used for both crest and sag curves.

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8 According to TPA: Roads Branch the design speed over the structure must be the same as for the road section

8.9.8 Structural loading

TMH7: Code of practice for the design of highway bridges and culverts in South Africa (CSRA, 1985 and 1989) distinguishes between the following types of traffic loading on road bridges:

- Normal loads (NA)
- Abnormal loads (NB)
- Super loads (NC)

The Code stipulates that while the NC loading may be omitted on certain routes, all road bridges should be designed for both the NA load and at least the NB 24 load. These values are recommended for low volume rural roads, provided that the design engineer ascertains whether the loading will be adequate for a particular structure.

8.9.9 Location

Crossing a river at a skew must be avoided. The skew, coupled with the possible blocking of openings with debris, tends to channel the full force of the river towards one of the riverbanks and the chance of this approach being washed away is considerably increased. Rivers have been known to cut a completely new channel around the structure, and remain permanently realigned, necessitating the lengthening of the initial structure.

The abutments of low level structures should be keyed into the river bank. The provision of road embankments may result in the embankments being breached when overtopped by flood flow. Where the overtopping of embankments is likely, provision should be made for such overtopping by suitably cladding of the downstream faces and road surface if necessary.

In certain circumstances it may be necessary to protect both banks downstream of the structure to a level calculated on the basis of a chosen design flood, preferably the 1:5 year flood.

If a low level structure is constructed on the widest possible part of a river or stream, the volume of flow that can safely be crossed by vehicles is increased. The wider flow will also be less prone to causing damage due to lower flow velocities. On the other hand it must be kept in mind that longer structures are generally more expensive.

8.9.10 Founding conditions

Ideally, submersible structures should be founded on solid rock. It is not always possible to find suitable rock foundations for these submersible structures, but founding them on unstable or sandy material is dangerous, due to the tendency for river bed material to become liquid up to a depth in excess of 2 m in heavy flood conditions.

If it is not possible to found a low level structure on rock, a raft foundation with up to 2,5 m deep toe walls to non-erodible material (if possible) must be provided. Transverse toe walls must also be provided to form closed cells. The raft is supported on the toe walls and on the compacted in-fill material and it is imperative that the toe walls contain the in-fill under scour conditions. An apron should also be provided downstream to absorb the energy when water flows over the structure. A downstream toe wall should be placed at the edge of this apron. Such an apron may consist of mattresses, gabions, rip-rap or a concrete slab.

8.9.11 Traffic control measures

Driver safety and convenience are major factors in designing low level structures. Human life must never be endangered. This can be accomplished by providing an adequate warning system in the form of guide posts which indicate both the limits of the drift and the depth of the water.

The following road signs should be provided at low level structures:

- Drift ahead (W24 of the SA Road Traffic Sign Manual, 1982)
- Speed reduction signs as applicable
- Other signs that may be required, eg sharp curve ahead, etc.

8.10 High level bridges

8.10.1 General

A high level is bridge is a structure designed for water to pass freely underneath. This definition is preferable to the traditional distinction between bridges and culverts that often depends on the size of the structure. A high level bridge will typically be higher than 2,0 m, as compared to a low level structure.

A high level bridge structure is designed for a specific design flood, which is associated with a certain return period. In the case of the structure being submerged considerable damage and disruption can be expected.

Where used on low volume roads, high level bridges can be either single lane or double lane bridges. However, the use of high level bridges on low volume roads should be avoided as far as is possible, and low level structures should rather be used.

The single lane structure lends itself to standardisation, thereby increasing efficiency and reducing costs. (The design of double lane structures can also be standardized). The superstructure can be standardised in spans with lengths between 6 m and 18.0 m, utilising either a solid slab, spine beam or voided slab section. The abutments and piers may also be standardised for various heights of the structure.

8.10.2 Return period of the design flood

The recommended basic flood frequencies for the design of rural road bridges are given in Table 8-1.

8.10.3 Dimensions

Recommended widths for high level bridges are provided in Table 8-10. In the case of two lane structures no kerbs need to be provided. In the case of single lane structures raised kerbs should be provided to protect pedestrians and railings. Figure 8-19 shows the dimensions referred to.

Table 8-10: RECOMMENDED WIDTHS FOR HIGH LEVEL BRIDGES (HLB)

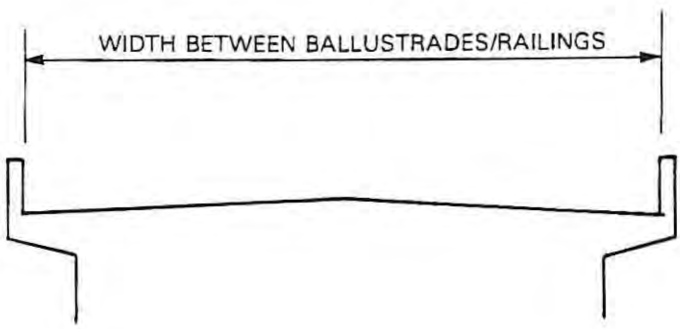
CROSS-SECTION TYPE (Refer to Figure 3-2)	TWO LANE STRUCTURES: WIDTH BETWEEN BALUSTRADES/ RAILINGS (m)	SINGLE LANE STRUCTURES: WIDTH BETWEEN KERBS (m)	SINGLE LANE STRUCTURES: DISTANCE FROM KERB TO BALUSTRADE/RAILING (m) PEDESTRIANS	
			MANY*	FEW**
1	12,2	Not recommended	LLS not recommended	
2	10,8	Not recommended	LLS not recommended	
3	9,4	Not recommended	Not recommended	
4	8,0	4,0	1,0	0,5
5***	6,0	4,0	1,0	0,5
6a***	8,0	4,0	1,0	0,5
6b***	8,0	4,0	1,0	0,5
6c***	6,0	4,0	1,0	0,5
7	8,0	4,0	1,0	0,5
8***	6,0	4,0	1,0	0,5
9	Not appropriate	4,0	1,0	0,5

* Typically more than 100 pedestrians in peak hour, but take traffic volume into account

** Typically less than 100 pedestrians in peak hour, but take traffic volume into account

*** Single lane structures recommended where viable - refer to text

1. TWO LANE STRUCTURES



2. SINGLE LANE STRUCTURES

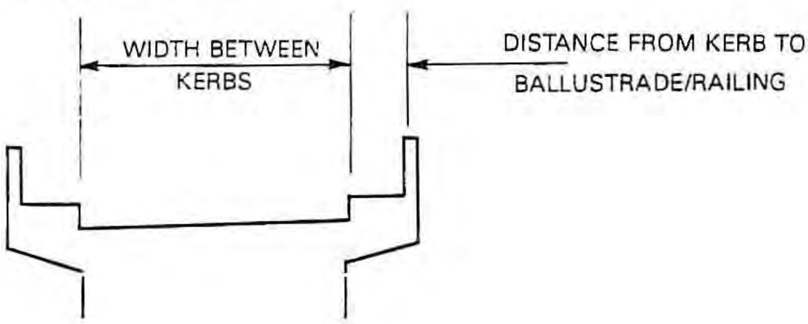


Figure 8-19: MEASUREMENT OF THE WIDTH OF HIGH LEVEL BRIDGES

A decision on single or double lane width should be made for each individual structure, but where there are several structures in close proximity on a road the overall situation should be considered.

For cross-section types 4, 5, 6a, 6b, 6c, 7, 8 and 9 the use of single lane bridges should be considered where the following circumstances arise:

- the approach gradients are moderate and there is no significant curvature on the immediate approaches
- the length of the low level structure is long and the savings associated with a single lane structure are considerable
- there is good visibility on the approaches to the structure and the structure occurs infrequently on an otherwise good section of road.
- the traffic volume is not expected to exceed 500 vehicles per day during the life of the structure.

For low volume roads high level double lane bridges should only be provided only on a route with traffic counts in excess of 500 vehicles per day and of significant priority. Poor alignment may also dictate the provision of a double lane bridge for safety reasons.

It is often expensive to widen bridges and culverts as much of the original structure may have to be removed. Therefore, serious consideration should be given to the design of the ultimate structure when the road considered is likely to be upgraded at some future time, or to design the structure in such a way that it can easily be widened at a reasonable cost if required.

When single lane structures are being planned, consideration should be given to the widths of certain types of agricultural machinery likely to use such structures.

8.10.4 Structural loading

As for low level structures - refer to Section 8.9.8.

8.10.5 Design speed

The design speed over a high level double lane bridge should be the same as that of the road. Although in the case of single lane bridges vehicles have to slow down or stop before passing over a bridge, it is also recommended that the design speed of the road be used.

8.10.6 Location

High level bridges of a permanent nature should be carefully placed on an alignment not likely to be altered for preferably 20 years[9].

8.10.7 Scour

The need for scour protection can be minimized by locating bridges on stable tangential reaches of rivers and by placing foundations on no-erodible materials. However, such a solution is not always practicable, economic or desirable from the road alignment point of view.

In such cases the designer is reminded to check for local scour at bridge sites, which is caused by macro-turbulence resulting from the concentration of energy. Potential scour around piers and abutments should also be checked and allowed for, if necessary. Where scour around the abutments

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9 Natal: Roads Branch suggested 50 years instead of 20 years

of major bridges is likely to be a serious problem, particularly where the bridge is sited in a wide flood plain, consideration should be given to the allowance of guide banks or spur dykes.

Before finally fixing the level of the pier and abutment foundation footings, consideration should also be given to the possible shifting of the river channel during a flood.

8.10.8 Freeboard to bridge soffits

Bridge freeboard is considered as the difference in level between the design high flood level and the bridge soffit level at the lowest point on the bridge soffit on the upstream side.

Table 8-11 gives the minimum freeboard for rivers discharging up to 1 000 m^3/s and carrying large debris such as trees.

Table 8-11: MINIMUM FREEBOARD REQUIREMENT FOR BRIDGES

DISCHARGE (m^3/s)	FREEBOARD (m)
100	0,3
200	0,5
300	0,6
400	0,7
600	0,8
800	0,9
1000	1,0

For floods exceeding 1 000 m^3/s and carrying large debris, the following formulae should be applied:

Freeboard = 0,6 m + d/15 (with a minimum of 1,0 m)

where d is the natural unrestricted depth of flow at the bridge crossing.

If the river carries only small debris at least 300 mm freeboard should be provided in all cases.

8.11 Subsurface drainage

Water in the structural layers of a pavement constitutes the chief cause of road failures. The purpose of subsurface drainage is to remove from the road structure, as rapidly as possible, infiltrated water occurring in damaging quantities.

Important sources of underground water are:

- Natural water table
- Irrigation, channels and dams
- Rainfall infiltration.

Such water may enter the pavement layers by infiltration, either sideways or through the road surface.

Adequate stormwater drainage may in many cases alleviate the need for expensive subsurface drainage. The ingress of water through the pavement surface should be limited by adequate cross-fall (refer to Table 8-2) and surface drainage. Furthermore, subsurface drains consisting of perforated pipes, aggregates and filter materials need only be installed when seepage or high water tables are encountered and not as a general policy in all cuts. Where subsurface water in the form of seepage from higher ground is encountered, however, subsurface drains must be installed. Subsurface drains should be installed upstream of the road and to a depth of at least 1,0 m below the subgrade.

The standard subsurface drain consists of a pipe at the bottom of a narrow trench which is backfilled around the pipe with filter material. It may be necessary for the filter material to be wrapped in a filter fabric of the filter material does not meet the grading requirements.The pipes may be perforated, slotted, porous or open jointed.

Subsurface drains can consist of any one of the following combinations:

- perforated (or slotted) pipe + granular filter
- pipe + aggregate + geotextile filter
- pipe + synthetic flow net + geotextile (fin drains).

Typical subsurface drains are shown in Figure 8-20. Woven and non-woven geotextiles can be used and for most soils the lighter grades of both types of geotextile will be suitable. Most soils will require soil/geotextile compatibility tests to select appropriate geotextiles. These include heavy clays, friable mudstones and dispersive soils.

If geotextiles are not used as filters, a granular filter must be used that is designed according to the Terzaghi criteria (TRH15, 1984). In most cases, geotextiles are cheaper than graded granular filters. Fin drains, in which synthetic materials are used both as filters and water carriers, are often the most economical type of subsurface drain and should therefore be considered.

Pipes should be of 100 or 150 mm diameter and may be perforated, slotted or woven (synthetic pipes) with solid inverts (bottom third). Unperforated pipes should be installed where dry or non seepage areas are traversed.

Where high water tables are encountered, where possible, provision should be made to elevate the road on an embankment. The bottom of the selected layers should be at least 1 m above the water table.

The reader is referred to TRH 15: Subsurface drainage for roads (draft)(1984) for guidelines on the design of subsurface drainage.

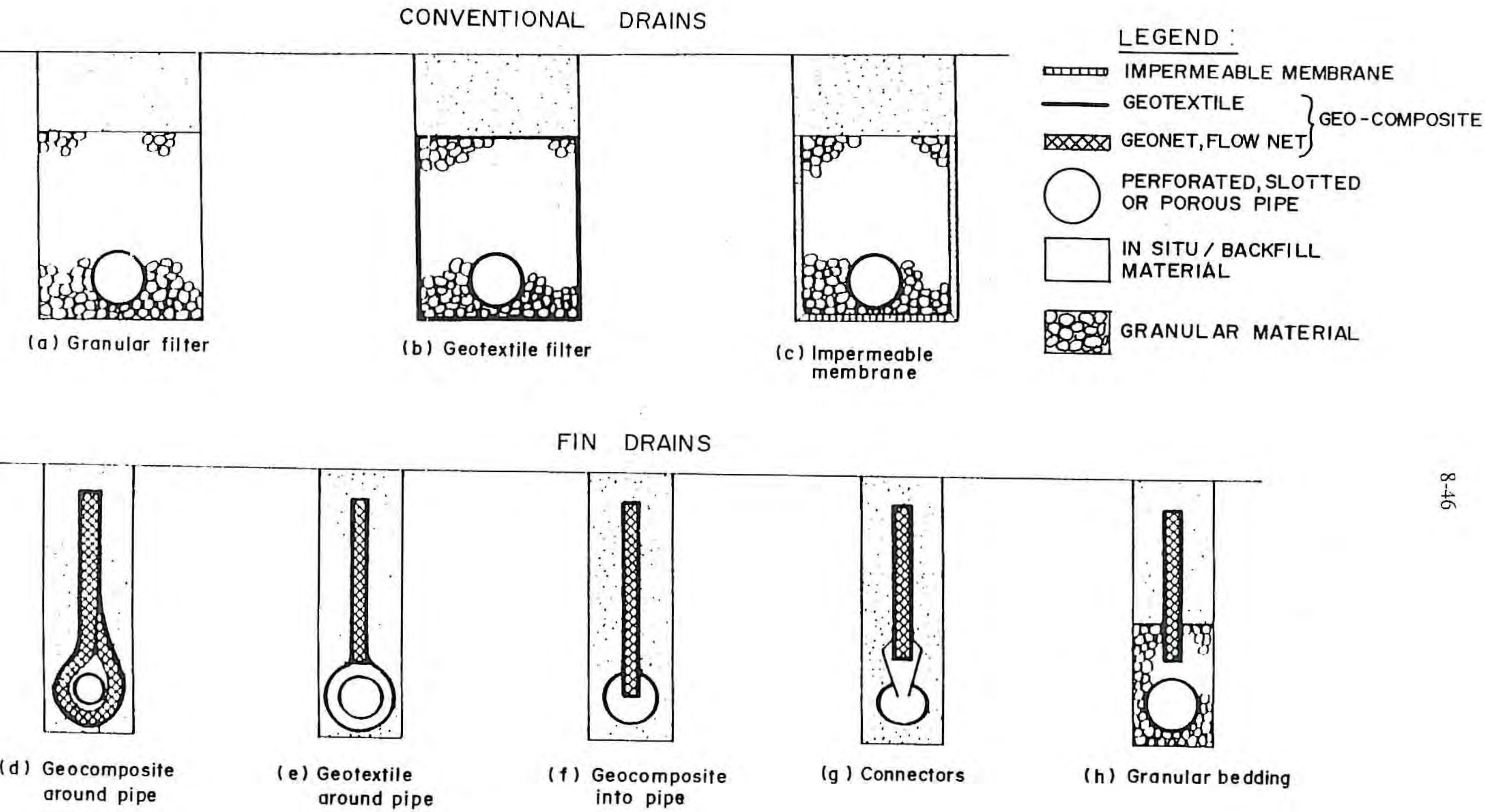

Figure 8-20: TYPICAL SUBSURFACE DRAINS

8.12 References

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9 CONSTRUCTION STANDARDS

9.1 Introduction

Standards for paved roads are very well established and detailed specifications have been drawn up to cover all aspects of their construction. Most of these standards were drawn up at a time when there were less financial restrictions on the construction of roads and some reduction in construction standards could provide economic savings on paved rural roads carrying low volumes of traffic. A study to determine if the current construction standards are appropriate is outside the scope of this project[1].

No detailed construction standards have been produced for unpaved roads and most documents give general guidance rather than specific standards which should be achieved.

Very few road projects are currently constructed by labour-enhanced techniques and construction standards for the manual components are only now beginning to receive attention.

9.2 Unpaved roads

9.2.1 General

An unpaved road can undergo a rapid and unpredicted increase in the traffic loading which may justify its upgrading, eventually to a paved road. The initial construction standard of an unpaved road should, therefore, be adequate for upgrading it at a later date to a higher standard.

General advice on some aspects of the construction of unpaved roads is included in TRH 20 (CSRA, 1990).

9.2.2 Compaction

Compaction is a relatively inexpensive operation which can have important consequences for the performance of an unpaved road[2]. It can:

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1 There is a certain amount of support for the execution of such a study.

2 Present policy in the TPA roads directorate is to do limited compaction of the wearing course using grid rollers.

- Produce a roadbed with a relatively uniform density in the upper section, thereby reducing the possibility of differential movement which can be transmitted to the pavement layers.

- Increase the density of the gravel wearing course, binding it more tightly together and reducing gravel loss.

- Prevent traffic from segregating the components of the gravel layer, causing it to perform poorly.

- Reduce the occurrence of surface defects such as ruts and potholes.

The ruts which form in a poorly compacted layer can readily be removed during routine grading operations but if water ponds in these ruts it can cause potholes in the gravel layer and problems which are more deep seated.

The increased density and binding in the surface, produced by watering and rolling it immediately after grading, can cause substantial reductions in gravel loss but the cost of these operations, particularly the cost of the water, can be very high in the dry areas of the country.

Minimum values of density for each component of a gravel pavement of an unpaved road are listed in Table 9-1. The value suggested for the gravel base has been selected as the density at which traffic should not damage the top of this layer although further compaction will occur under the action of traffic. Studies by TRRL in Kenya have shown that small and medium commercial vehicles can compact natural gravels to 104 % of Mod. AASHTO density[3]. Paige-Green (1989) states that wearing courses which have been compacted with a nominal number of passes of a grid roller can lose up to 30 % of their thickness during subsequent compaction under traffic.

It is very important that the surface be properly watered before and during compaction to ensure a tightly bound surface is produced (TRH 20). Ferry (1986) refers to this bound surface as a "crust" which has an important effect on gravel loss and riding quality.

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3 In similar studies in Brazil, a density of 95% Mod AASHTO was achieved.

Table 9-1: RECOMMENDED MINIMUM DENSITIES

Layer	Minimum density (% of Mod. AASHTO)
Subgrade	90
Subbase	93
Base and wearing course	95

The optimum moisture content should, wherever possible, be determined from a series of compaction tests in the laboratory. If a suitable laboratory is not available or if equipment for rapidly determining the moisture content is not available on site, the simple field tests described in Table 9-2 can be used to give some indication of the optimum moisture content for compaction.

Table 9-2: SIMPLE FIELD TESTS TO ESTIMATE OMC

Material	OMC for field compaction
Clean gravel Sand Crushed stone	Materials should be air-dry or saturated. Do not damage subgrade by soaking it
Clay	A piece of clay rolled into a thread starts to crumble when it is the thickness of a pencil.
Silt	Check for change in colour after the first pass of the roller. If it gets darker it is too wet and should be allowed to dry until colour change doesn't occur.
Silt-clay mixture	Use the thread rolling test.
Well-graded plastic gravel	Use thread rolling test on the fine material. Water should tend to run off surface after compaction rather than run into it.

(From Ferry, 1986)

The appropriate compaction equipment for different materials are listed in Table 9-3. If a suitable roller is not available on a particular project then engineers may be forced to produce the best compacted density with unsuitable equipment. In these conditions it is best to carry out field trials with the available rollers. Short sections of material are placed at different moisture contents and the available rollers used to determine the roller which can produce the best density and the moisture content at which the compaction should be carried out.

Where possible the density and compaction moisture content should be measured with a nuclear density meter or, alternatively, with a sand replacement test. The option of measuring the deceleration of a Clegg Hammer on the surface of a layer should not be forgotten. This instrument gives an almost instantaneous estimate of the in situ strength of the layer.

Table 9-3: APPROPRIATE COMPACTION EQUIPMENT

Type of roller	Type of material						
	Rock fill	Sand and gravel		Silt and clayey material		Clay	
		Well graded	Poorly graded	Silty sand or gravel	Clayey sand or gravel	Soft	Stiff
Steel wheeled		X	X				
Pneumatic tyred		X	X	O	O	X	
Padfoot	X		X	O	O	O	
Grid	O	O	O	O	O	O	O
Vibrating	O	O	O	O	O	O	O

(From TPA Materials Handbook, 1984)
O = suitable
X = can be used

9.2.3 Tolerances

(i) Surface level and thickness of layers

In many parts of South Africa the climatic and subgrade conditions are such that overstressing of the subgrade is not likely to be a cause of pavement failure (TRH 20). The values for the tolerances in the thicknesses of pavement layers do not, therefore, have important structural implications.

If the tolerances in thickness are too wide, roads constructed under contract are likely to have layer thicknesses substantially less than the designed thicknesses. If the tolerances are too tight the high cost of meeting these tolerances does not produce a comparable increase in the performance of the pavement.

Recommended tolerances for layer thicknesses and surface levels to use in conjunction with the Standard specifications are included in Table 9-4.

Table 9-4: RECOMMENDED TOLERANCES FOR GRAVEL ROADS

Layer	Surface level tolerances (mm)			
	H_{90}[a]	H_{100}[b]	D_{90}[c]	D_{100}[d]
Subgrade	40	50	-	-
Subbase (if used)	35	45	40	50
Wearing course	35	45	40	50

(a) 90 % of all levels meet with requirements

(b) 100 % of all levels meet with requirements

(c) 90 % of all thicknesses meet with requirements

(d) 100 % of all thicknesses meet with requirements

(ii) Crossfall

Of greater importance in the construction of gravel roads are the tolerances of crossfall (or camber). It is essential that water be prevented from ponding on a gravel road surface because the weakened zone adjacent to the water will quickly deteriorate under traffic. The crossfall must be maintained at a value great enough to prevent water ponding in normal ruts or depressions (see Table 8-2).

9.3 Paved roads

9.3.1 General

Recommended standards for paved roads have been published in the Standard Specifications (CSRA, 1987) produced by a number of road authorities and engineers should try to achieve these construction standards on rural roads carrying low volumes of traffic. Some of these standards can be reduced if they are difficult to achieve but this step should only be taken by experienced engineers who can make a sound estimate of the standards which are of low risk in particular circumstances.

Compaction is one construction operation which should not be reduced and a case can be made for increasing the minimum densities accepted in the current standard specifications on lighter structures.

9.3.2 Compaction requirements

It is considered prudent that materials used in the construction of paved roads should be prepared and spread in accordance with the CSRA Standard Specification for Road and Bridge Works, (1987). Any increase in the compaction layer thicknesses, change in the grading requirements or increase in the maximum size of aggregate allowed in a layer are likely to decrease the density that can be achieved and, consequently, the performance of the road.

9.3.2.1 Fill and insitu subgrade

The minimum density allowed in fills should be 93 % of Mod. AASHTO density. Contractors in South Africa often submit the same unit rates for achieving minimum densities of either 90 or 93 % and it is considered, therefore, that the removal of the 90 % option will not substantially increase the cost of construction but will provide a worthwhile improvement in quality.

Similarly, it is recommended that the roadbed be compacted to 90 % of Mod. AASHTO density. In dry areas where water is expensive, the option of using dry compaction techniques should be allowed for the preparation of the roadbed and compaction of the fill.

9.3.3.2 Subbase and base

The CSRA specifications for crushed stone bases has a minimum value of 100 or 102 % Mod. AASHTO for G2 material. It is recommended that the upper values be used in order that rutting can be minimised and pavement life extended. This recommendation is based on research by Shackleton (1990) who has investigated the formation of ruts in pavements with granular bases. This study identified several important features of the formation of ruts.

- When water ponds on the surface of the road, ruts form at approximately 4 times the rate at which they form when water is shed from the surface.

- Water starts to pond in ruts when they are approximately 10 mm deep. If there is a steady rut rate from a rut depth of zero to 10 mm, this point represents about 80 per cent of the life of the road.

- During the initial phase of trafficking there can be a substantially higher rate of rutting associated with the bedding in of the pavement. The rut which forms during bedding in may be as deep as 11 mm and, on that particular pavement, the traffic carried during bedding in used up 73 per cent of the life of the road.

These features are illustrated in Figure 9-1. Graph 1 shows the rutting history for a pavement in which initial traffic compaction causes an 8 mm rut. At the end of the densification phase the rate of rutting decreases until the critical rut depth of 10 mm is reached. From this point water can pond in the rut and the rut rate increases by a factor of 4.

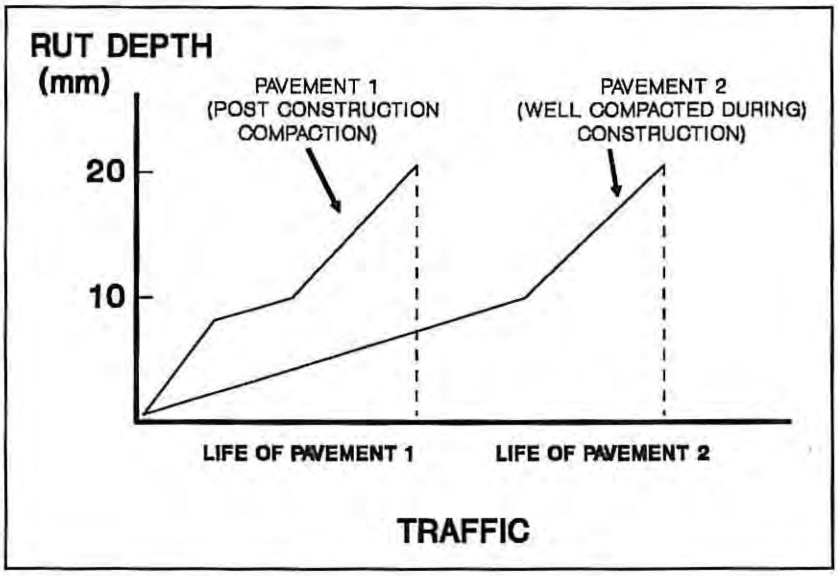

Figure 9-1: EFFECT OF THE DEGREE OF COMPACTION ON THE LIFE OF A PAVED ROAD

Graph 2 traces the formation of ruts in the same type of pavement but in this case the density of the base and sub-base were increased during construction and there is no initial traffic compaction. More than twice as much traffic is needed to form a critical rut of 10 mm and the effective life of this pavement is approximately 70 per cent greater than the previous example. It must be remembered that this increased life is achieved for very little additional cost.

With modern compaction equipment it is relatively easy to achieve 100 % Mod. AASHTO density in gravel materials. Any increase in compaction costs would be offset against the substantial increase in the life of the road together with a reduction in discounted maintenance costs. It is recommended, therefore, that gravel materials in the base should be compacted to a minimum of 100 per cent of Mod. AASHTO density, and the subbase to as high a density as possible, with a target of 100%.

If the gravels are stabilised with cement or lime, rutting is not a problem and a minimum density of 97 % of Mod. AASHTO is appropriate, particularly when increased densities are related to increased shrinkage.

The recommended compaction requirements are set out in Table 9-5.

Table 9-5: COMPACTION REQUIREMENTS[4]

Layer		Minimum Compaction requirement
Roadbed		90 % Mod. AASHTO
Fill		93 % Mod. AASHTO
Selected		93 % Mod. AASHTO
Subbase	gravel	100 % Mod. AASHTO
Base	stabilized gravel	97 % Mod. AASHTO
	gravel	100 % Mod. AASHTO
	G2	102 % Mod. AASHTO

9.4 Substitution of labour for plant

Under certain conditions construction operations may be more desirably carried out by manual methods than by traditional plant methods but the balance will be influenced by minimum wage rates, shadow wage rates, productivity and the type of operation. Although the economic cost may be cheaper, the financial cost can be substantially higher and a decision to proceed with labour-based operations can require a larger project budget.

When engineers are investigating the option of substituting labour for plant there is a series of questions which must be considered.

- What operations are practicable by manual methods?
- What standards can be achieved?
- What is the cost of achieving these standards?
- Are the standards realistic?
- Can they be modified to produce a saving in cost without reducing the performance of the road?
- Is labour available and willing to carry out the operations?

Certain operations such as the compaction of materials to densities approaching Mod. AASHTO, the haulage of materials for long distances or the production of crushed stone sizes of less than 25 mm are uneconomical by manual methods unless a particular site has some restriction.

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[4] The CPA favour a value of 97% Mod AASHTO (using statistical assessment) for gravel subbases. For stabilised subbases their minimum is 95% Mod AASHTO while a minimum of 95% is also applied to the uppermost selected layer.

Other operations such as the accurate distribution of bituminous binder or the production of a high quality surface finish on a granular pavement layer are not viable by manual methods. Concrete is one pavement material which can be laid to a good surface finish by manual methods and its use on steep gradients or areas with drainage problems may be desirable on a labour enhanced basis even on roads carrying low volumes of traffic.

In general, many construction operations can be carried out in part or completely by manual methods but they may only be financially feasible at lower daily wage rates. Projects on which labourers have been paid a daily wage that is not linked to production have invariably proved to be more expensive than those constructed with plant-based methods. The problems of negotiating a realistic daily task with unions cannot be overemphasised but if this cannot be achieved then labour-enhanced construction will be uneconomic.

Standards for projects where labour is competing with plant should be the same irrespective of the construction system which is chosen. Appropriate standards have not yet been established for labour-based projects where little plant is available. The standards applied on "self-help" projects are usually well below normal construction standards but road users are prepared to accept a lower standard of service from these roads because the alternative may be to have no access at all.

Some general comments about labour-based construction are included in a synthesis document of the Department of Transport (1989). Particular comments about the use of manual methods in the construction of bitumen surfacings are included in SABITA (1992).

9.5 Quality control and quality assurance

Good quality will only be achieved if everyone involved in a project wants a good quality product and insists on getting it. The quality has to be produced by the contractor who should carry out enough testing to ensure he controls all processes and achieves an acceptable quality. No amount of testing of a sub-standard item will make it acceptable.

The amount of control testing which should be carried out depends on the variability of each item and the difference between the target value and the minimum acceptable value which must be achieved.

Figure 9-2 includes two distributions of a particular factor and both have a similar risk of failure. The high target value of the right hand distribution is representative of many of the present construction and materials standards. The probability of premature failure has been reduced by using conservative standards. When the target value is high a substantial variability is acceptable in the factor without placing the road at risk.

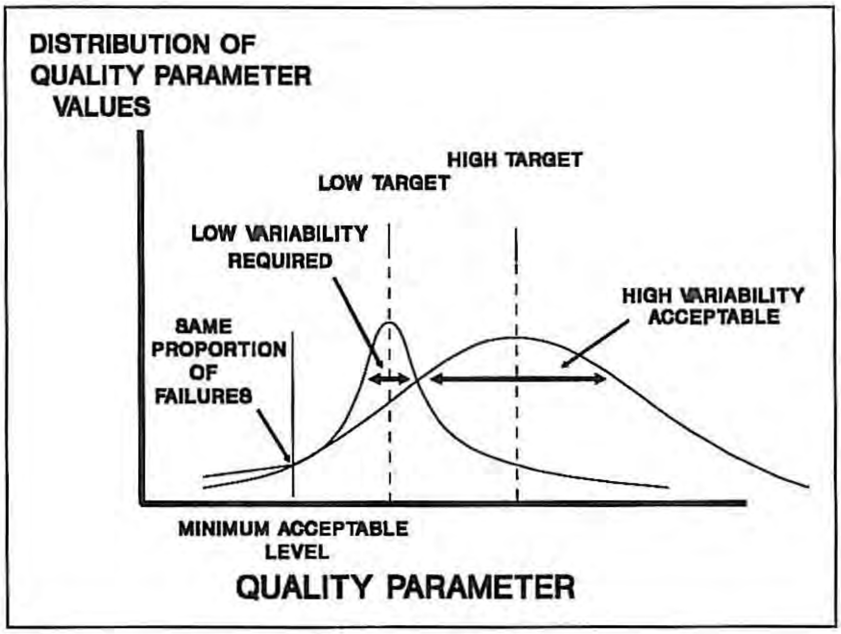

Figure 9-2: THE INTERACTION BETWEEN TARGET VALUE AND VARIABILITY

If standards are reduced then the variability must also be reduced if the risk of premature failure is not to increase. This reduction can only be achieved by more effective control of the quality of the work. In addition to increased supervision, more control testing must be carried out to identify any changes in quality.

Operations carried out by manual methods have a much greater variability than those carried out by plant and increased supervision and control testing is essential. Irrespective of the increased control, the target values for manual operations must be higher than the values used when the same operation is carried out by plant.

Quality assurance schemes are based on the interlinkage between the designer, contractor, inspector, materials supplier and client. The standards which must be achieved and the variable tables of penalties for not reaching the required standards are important parts of the scheme. Traditional standard specifications have to be rewritten before they can be included in a quality assurance scheme.

9.7 References

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10.1 Introduction

This section covers the maintenance of paved and unpaved roads, with resealing and rehabilitation being covered in Chapter 11.

10.1.1 Maintenance policy

The road authority has a general legal obligation to maintain its road network, and this will guide its maintenance policy. A clear example of this is the maintenance of safety related signs. The provision of a sign is an indication that the authority regards a situation as dangerous. The road authority has a legal obligation to provide such signs, and it also has an obligation to maintain both the signs and the adjacent areas (which would include the control of vegetation which may affect sight distance) in such a way that the signs remain clearly visible. The legal issues are less clear when the maintenance concerns items which are not as closely linked to safety. For example, if fencing is not properly maintained and an accident occurs between an animal and a vehicle, then the road authority may or may not be liable.

In line with the need to maintain its road facilities, the road authority also has an obligation to provide adequate warning of any temporary deterioration in the condition of those facilities. If there is some deterioration, such as may occur after an exceptional wet season, and there will be a backlog of maintenance, it is in the best interests of the road authority to erect the necessary regulatory and/or warning signs, and to reduce the speed limit.

Maintenance standards should be linked to the budget of the road authority, and work has been done locally to match standards with levels of affordability (NPA, 1992).

Maintenance policy must be considered, not only in the light of legal obligations, but also in terms of funding and the road authority's policies on construction, rehabilitation, and road user costs, since these are all inter-related. Issues to consider are:

- which maintenance measures have the highest pay-off under varying circumstances,
- can future maintenance requirements be reduced by building or rehabilitating roads to higher standards and when is this economical, or
- what are the consequences of deferring maintenance on part or all of the road network.

When the maintenance budget for a network is less than that required for the overall optimum, expenditures should not be reduced uniformly across all road classes (Bhandari et al, 1987). The use of selective maintenance of certain links or even sections of a link is often appropriate and economically justifiable.

10.1.2 Levels of serviceability

Maintenance requirements and costs are based almost entirely on the required level of serviceability which should be appropriate to the traffic. This is particularly important for unpaved roads where the level of serviceability is almost linearly proportional to the maintenance effort, and so tentative guidelines are given for levels of serviceability for unpaved roads in Table 10-1.

The maximum roughness is generally used to determine the grader blading frequency in maintenance management systems and can be programmed into the maintenance management system. The level of serviceability should be adapted to the primary use of the road. Important tourist routes or a farm access road over which a fragile product sensitive to extremely rough roads (such as eggs) is to be transported, may be maintained at a level of service higher than that which the traffic dictates, for obvious reasons.

The level of serviceability significantly affects vehicle operating costs, which rise steeply as the road roughness increases. The choice of level of serviceability is a balance between maintenance costs and road user costs, and will often be governed by community reaction.

Table 10-1 LEVELS OF SERVICEABILITY FOR UNPAVED ROADS

Level of service-ability	Type of road	Road roughness[a]	Required standards	
			Dustiness[b]	Impassability
1	- private road, low traffic	100 - 200 (average to bad)	5	Frequently
2	- minimum public road standard, low traffic	100 - 150 (average)	4	< 5 days/yr
3	- low to medium traffic volume - medium traffic volume, rural road - farm access road with sensitive products	80 - 120 (good to average)	4	Never
4	- medium traffic volume - important tourist route, low volume	40 - 100 (excellent to good)	3	Never
5	- high traffic volume - important tourist route, medium volume	40 - 80 (excellent to good)	3	Never

Notes a QI (Counts/km) over 10 per cent of the link
b From Paige-Green (1988)
General: Some road authorities question whether tourist roads should have a high priority

10.2 Maintenance of unpaved roads

The maintenance of the surface of unpaved roads is the major cost factor in the maintenance programme. Routine maintenance of unpaved roads comprises four levels of increasing work:

- blading by grader,
- spot regravelling (also called reconditioning),
- reworking and compaction (also called rehabilitation)[1],
- regravelling.

The long term performance of an unpaved road is closely related to the continuity of routine maintenance it receives. If an unpaved road in good condition is allowed to deteriorate through a lapse in maintenance, then it is difficult to bring its original condition back with grading alone. Some authorities have a policy of a high level of routine maintenance on all roads, which is believed to reduce overall costs. Other authorities accept a lower level of service on lower traffic roads.

For unpaved roads, economic studies by the World Bank (Bhandari et al, 1987) have shown that road roughness - and so the cost of operating vehicles - is primarily related to the frequency of blading (grading). Even at low traffic levels (25 vehicles per day), the economic returns on blading are substantial. Optimal blading is every 4 000 to 8 000 vehicle passes, but less frequent blading does not occasion serious economic loss if the road is regravelled at appropriate intervals.

Certain seasons are particularly prone to unique problems and the maintenance should be adapted to this. During the wet season erosion and potholes occur, whilst during the dry season corrugations and ravelling are the primary problems. Both the type and frequency of maintenance should thus be adapted to the conditions. For example, grass cutting and drainage teams have little to do during the dry season and could be allocated to other teams.

Dry weather grading is necessary to maintain the roughness at low levels by removing corrugations, ruts and windrows of loose material. During the wet season, however, grader maintenance should concentrate on restoring the shape of the road by skimming and if necessary applying some compaction to the moist material. The need to cut with the grader may be a sign that the level of routine blading is too low, and cutting is held by some to be an indication of undesirable practice.

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1 Some road authorities question the cost effectiveness of this action

10.2.1 Blading

A grader is run across the surface of the roads with the blade set to shape and smooth the surface. After grading, no potholes, corrugations, excessive loose material, large boulders, ruts or erosion channels should be present and straight portions of the road should have a definite crown and cross-fall while curves should have an adequate superelevation for safety. Experience has, however, shown that cross-falls and superelevations greater than about 5 per cent result in excessive erosion. A balance is required for the superelevation not to result in erosion but to be adequately safe.

In South Africa, grader blading is usually carried out at anything from a one week to six-monthly interval depending on the climate, traffic and required level of serviceability. It should be carried out during periods of average moisture when the material is most easily cut, moved and compacted. In fact, practice has shown that during the dry season the hard upper crust or "blad" should not be cut (CSRA,1990,TRH20). Blading can be classified as either light or heavy.

> Light blading consists of a light trimming of the road surface on a routine basis. During the dry season, the surface loose material should be moved towards the side of the road, while during the wet season the loose material should be graded towards the centre of the road. It must be remembered, however, that the fine material is slowly lost from the road surface in the form of dust and the repeated return of the loose surface material which is deficient in fines may lead to the formation of corrugations. "Sand blankets" are usually placed during light grading, but thick "sand blankets" or "sand duvets" should be avoided. If "sand blankets" are used, the maintenance gang should ensure that no stones larger than 25 mm are incorporated in the "sand blanket" or obscured by it.

> Heavy blading should be carried out when inspection reports indicate excessive defects, and may be an indication that there is insufficient light blading. The road surface is scarified and cut to the bottom of the deformations and reshaped. This should only be done when the material is moist and more than 75 mm of surfacing aggregate remains. Heavy grading is often necessary when "fixed corrugations" have formed. These corrugations may need initial tining or deeper cutting to break them up before being graded and recompacted.

Grading, or the wrong technique of grading, can make some roads rougher, especially those which are slightly "self-cementing" and form a crust or "blad", or those with large stones as they often tend to be torn up under the grader. Spot-regravelling is usually required to patch these. Excessive large stones cause problems with grader blading, as the stones are plucked out and dragged along the road causing long, deep gouges. They also cause excessive wear to the grader blades.

Loose material is a significant problem on unpaved roads. Many single vehicle accidents on unpaved roads are caused by windrows ("sandwalletjies") of loose material on the roads. These windrows interfere with the directional stability of the vehicles which may eventually overturn; the higher the vehicle speed, the greater the interference. It is important that these windrows be not permitted to become higher than 50 mm. In addition to the vehicle handling aspect, high windrows often conceal large stones which can cause extensive damage to the tyres and underparts of vehicles. A common problem caused by poor grading practices is damming of water on the roads by windrows left at the edge of the road. Often the material deposited at the end of the grader blade during the last run forms a bank which retains water. This should not be permitted and should either be removed by the grader, or manually after grading. Some grader operators leave a windrow like this at the edge of the prism and blade it onto the road when it is damp and will bind with the existing surface i.e. not create a loose layer. Periodic openings should be constructed in these windrows to allow the escape of surface water. Excessive grader maintenance with the production of banks often results in the level of the road being below the adjacent shoulders. Heavy grading and reshaping should be carried out in this case to avoid channelisation of the water along the road surface.

The development of ruts should be controlled during grader maintenance. Grading should occur before ruts have become deeper than about 25 mm, with the ruts being filled with loose material. Prolonged rut development results in channelling of run-off and subsequent erosion and loss of shape of the road. On excessively wide roads (more than 8 m) the vehicles tend to hollow out the centre of the road and the crown is totally destroyed. Particular care should be taken to restore and maintain this crown during grader blading.

10.2.2 Optimum blading frequency

The optimum blading frequency is strongly influenced by materials and traffic, and to a lesser extent climate and local conditions. It is possible to assess optimum regravelling and blading frequencies for individual links using the models and computer programmes in TRH 20 (1990), developed by Paige-Green (1989). The influence of material type on blading frequency determined in the models is very high, and the variability of materials is so great in practice, that no more simple method could be developed. Accordingly, while the TRH 20 models are preferred where the data are available, a less accurate and tentative guide to blading frequency is given in Table 10-2 (using results from Bhandari et al, 1987).

Table 10-2: APPROXIMATE BLADING FREQUENCY FOR UNPAVED ROADS[a]

Traffic volume (Vehicles per day)	Minimum blading frequency	Recommended blading frequency[b]
50	start and end of wet season	start, middle and end of wet season, and twice in dry season
100	start, middle and end of wet season, and twice in dry season	every month
250	every month	every two weeks
500[c]	every two weeks	every week

Notes a where accurate data are available for the road, it is recommended that the models of TRH 20 (1990) be used.

b if an authority wants to follow a policy of a high level of service, then blading must be conducted more often than this

c this may justify paving

10.2.3 Spot regravelling

Spot regravelling is carried out to replace the gravel over areas where it has become excessively thin or worn through and for filling potholes, ruts, erosion channels and even corrugations. It is normally limited to patching and repair of limited lengths of road. It is predominantly a manual operation and restricted to potholes and subgrade failures.

This should make use of the same material as the wearing course gravel. Potholes should be cleaned out, the loose material removed from the sides, moistened with water, and then back-filled with moist gravel in 50 to 100 mm layers. Each layer should be compacted (a hand rammer is adequate) until the hole is filled to about one centimetre above the surrounding road. It is useful during the regravelling process to stockpile small supplies of wearing course aggregate in the borrow pit, at the maintenance camp or along the road at strategic places for maintenance purposes.

10.2.4 Reworking and compaction

Roads which have deteriorated badly can be ripped or scarified, oversize material removed or broken and the road recompacted if regravelling is not necessary at that stage. This is common when an adequate thickness of gravel exists on a road but the roughness becomes excessive under increased traffic or for a different traffic mix. If there is an excess of oversize stone, it is worthwhile removing this by hand or processing it through a rockbuster.

10.2.5 Regravelling

Regravelling is the most expensive single maintenance procedure for unpaved roads. It is carried out every few years when the imported gravel on the road has been almost totally lost through erosion by rain and wind or abrasion by traffic, or when inappropriate material exists in the road.

In dry areas, on roads where fines are lost and coarse material causes poor riding quality, the addition of a thin layer of fine material (clay with a high PI) allows for the protection of the pavement, reduces further loss and provides a smooth ride.

a) deformation which will necessitate reconstruction; and
b) loss of the strength which has been built up in the subgrade by traffic moulding over time.

Improvements to any drainage deficiencies should be made prior to regravelling. The quality of the new gravel should comply with the required specifications discussed in Section 4. The regravelling process should follow the same procedure as the construction process with respect to the winning, hauling, spreading and compaction of the material.

The rate of gravel loss is influenced by the material properties, traffic and climate. The frequency of regravelling as a function of gravel loss can be found using the following nomograms which have been developed from TRH20 (CSRA,1990). The regravelling frequency is found by dividing the wearing course thickness with the annual gravel loss, to which should be subtracted a safety margin to protect against regravelling delays and unexpectedly high gravel losses.

When selecting materials for the wearing course, they must not be chosen only to minimize gravel loss, since their properties also affect road conditions such as slipperiness, ravelling, corrugating, dustiness and erodability. A material may have very low gravel loss properties, but may not be a good wearing course due to slipperiness.

<u>Example of the use of the nomograms for the calculation of annual gravel loss</u>

For the following road, determine the regravelling frequency:

Traffic (ADT) = 200	Weinert's N value = 3 (see figure 4-1)
Wearing course grading:	% passing 26,0 mm sieve (P26) = 75
	% passing 0,075mm sieve (P75) = 46

Plastic limit (PL) = 30 Plasticity product = P75 * PL = 46 * 30 = 1380

Step 1: determine TRLoss

The gravel loss due to traffic (TRLoss) is found from the first nomogram page. The top nomogram gives a value for R (R is a temporary value for use in the next step), from Weinert's N and % passing the 26,0mm sieve. The example shows a line drawn through the N = 3 and P26 = 75 points. The R value is 0.0221. This R value is then used in the bottom nomogram, together with traffic (ADT) to determine the TRLoss. In the example, this is 4,42mm.

Step 2: determine WLoss

The gravel loss due to weathering(WLoss) is found from the second nomogram sheet. On the top nomogram, with P26 = 75 and PP = 1380, a straight line is drawn to intersect the R line. Using the value N=3 and the intersection point with the R line a value for WLoss can be read off as 0,57 mm.

Step 3: determine Annual Gravel Loss

The values for TRLoss and WLoss are used in the bottom nomogram. A line is drawn with WLoss = 0.57 mm and TRLoss = 4,42 mm and where it intersects the Annual Gravel Loss line the value 18,2 can be read off. The annual gravel loss is therefore approximately 18mm.

When using the nomograms, it must be remembered that there are limitations due to the empirical nature of the formula, and extrapolation outside the graduation of the nomograms is not advisable.

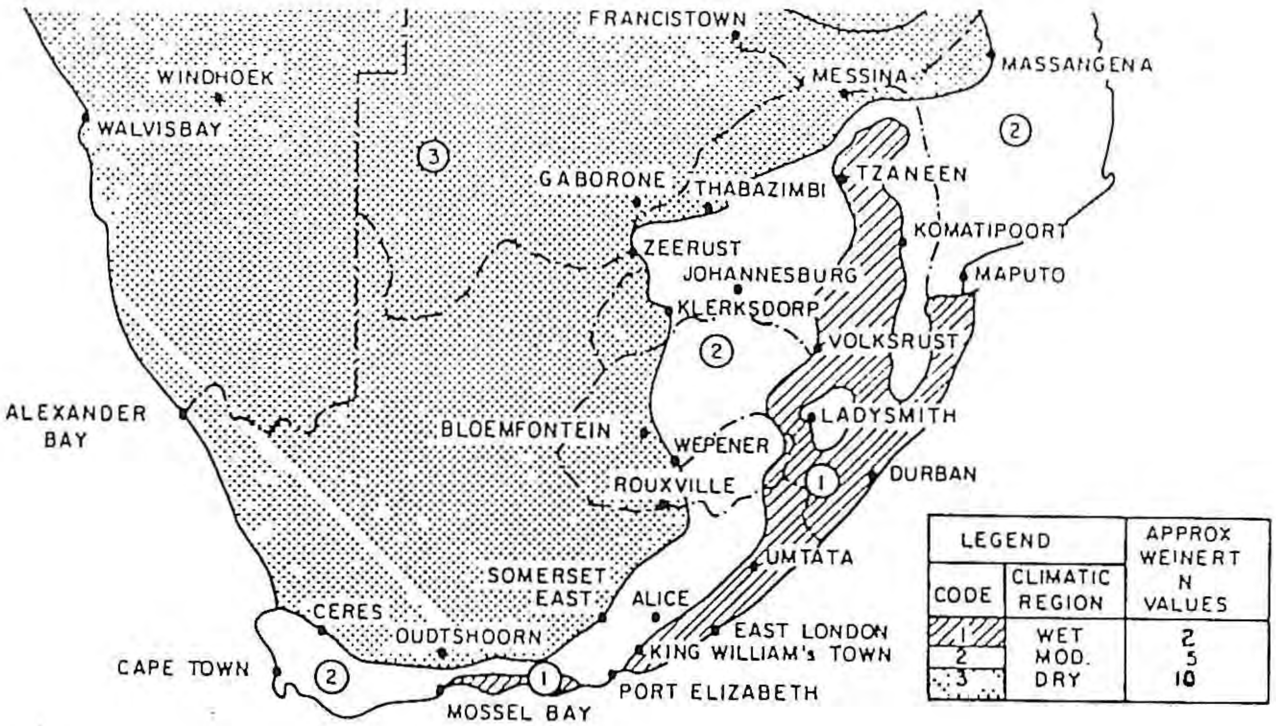

Figure 10-1: WIENERT'S N VALUES FOR SOUTHERN AFRICA

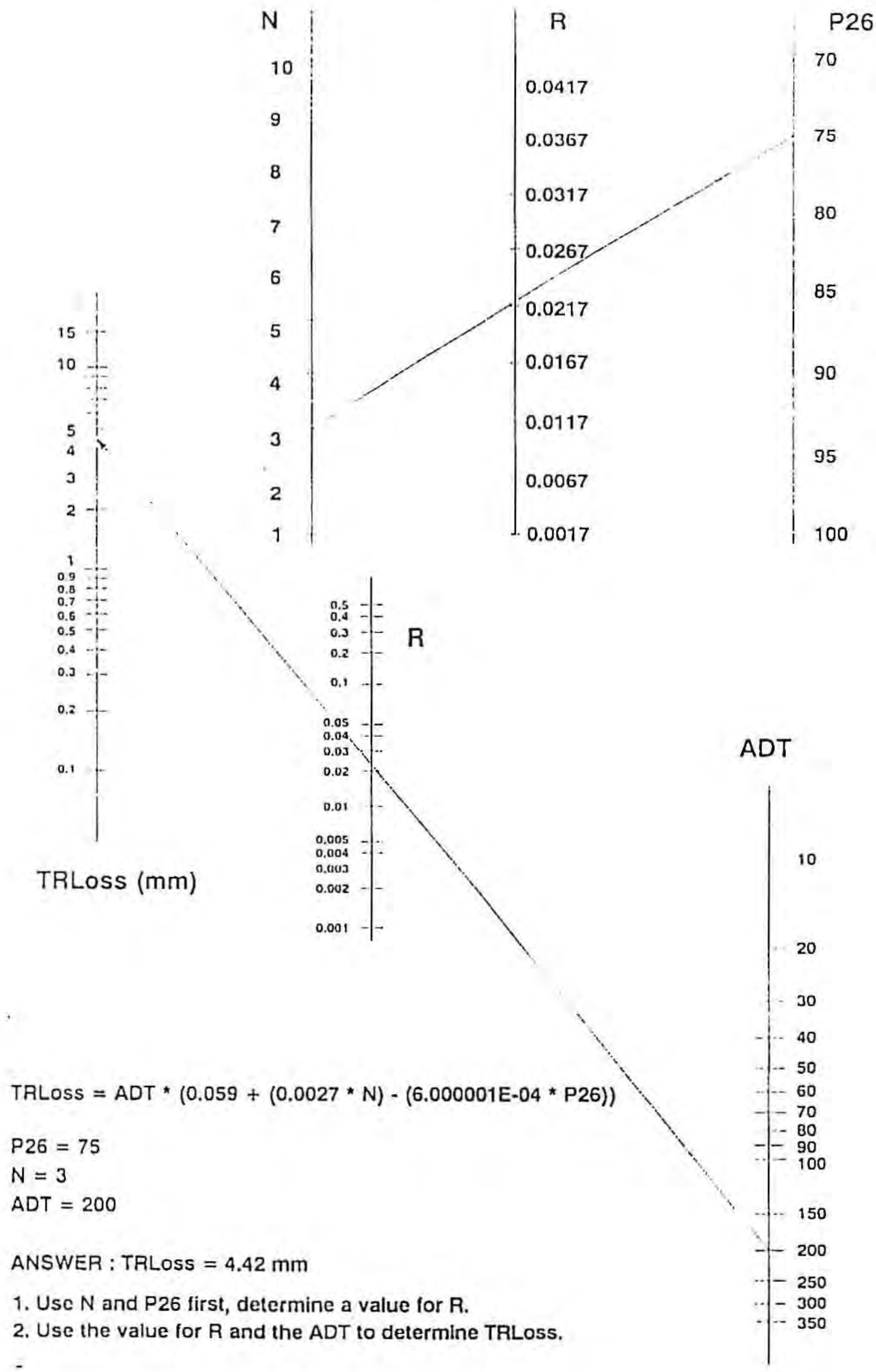

Figure 10-2(a): **NOMOGRAM FOR DETERMINING GRAVEL LOSS**

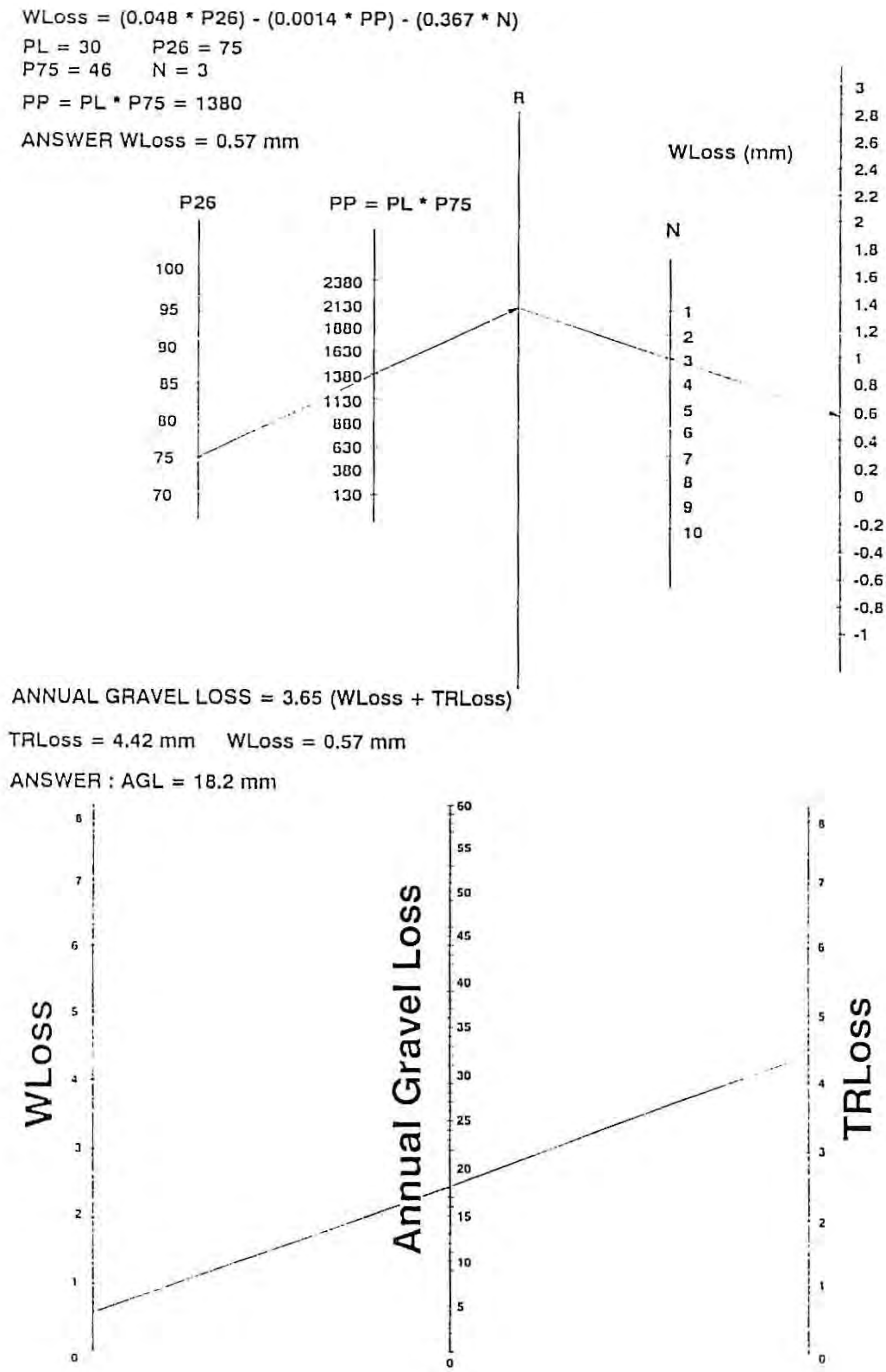

Figure 10-2(b): NOMOGRAM FOR DETERMINING GRAVEL LOSS

10.3 Maintenance of paved roads

Maintenance of paved roads involves actions to keep the surface intact and functioning properly. If the bituminous surfacing shows signs of weakening, it is essential to determine where the fault lies. Specific types of maintenance action are discussed in this section including crack and joint sealing, partial-depth patching, full-depth patching, and utility cut repairs. These will correct the most common problems which arise.

10.3.1 Special requirements for low volume roads

The maintenance of low volume roads brings several requirements which are less commonly encountered in the maintenance of higher volume roads, because low volume roads encompass a wide range of materials and environments. In some low volume roads, materials are used which are more moisture sensitive than those used in standard TRH 4 (CSRA,1985a) designs. For example G6 materials are used as the basecourse in some of the designs in Chapter 5 of this document; this is a more moisture sensitive material than say the G2 materials (CSRA, 1985b,TRH 14) or cemented bases which are found in some TRH 4 designs.

Another concern is the institutional capabilities of road authorities with low volume roads. Some low volume roads are maintained by authorities with inadequate resources, and as a result, timeous maintenance is not always available. This can allow what may have been initially a small maintenance problem (such as a pothole) to develop into a large maintenance problem. For roads in areas where the institutional capability for maintenance is low, the choice of materials and especially the choice of surfacing, must be adjusted (Emery et al, 1991). The thinner seals can give good performance only if they receive adequate routine maintenance. If there is no maintenance capability, then only those surfacings which are inherently tough can survive, which generally means the thicker surfacings. Table 5-15 gives guidance on this.

10.3.2 Ravelled Surface

Description and causes

Ravelling is a condition in which asphalt becomes rough in texture due to a loss of stone. The causes could include insufficient or aged binder, excessively open-graded mix, poor compatibility of stone and binder, or fracturing of stone.

Maintenance action to repair

Ravelled surfaces will require binder to be added. If ravelling has not developed too far the condition can be corrected by an enrichment coat (diluted emulsion). Otherwise a reseal or asphalt overlay will be necessary, an option which is covered in Chapter 11 on rehabilitation.

10.3.3 Stripped Surface (aggregate loss)

Description and causes

Stripping is a condition in which stone is lost from a seal, due to one or more of the following reasons:

- Ageing and hardening of the binder.
- Cold or wet weather before, during, or soon after spraying.
- Wet or dusty stone to which the binder has not adhered.
- Insufficient binder for the size of stone used.
- Stone of a rock type to which the binder does not readily adhere.
- Use of a penetration grade bitumen when a cutback was needed, preventing the binder from wetting the stone properly.
- Use of a cutback grade bitumen when a penetration grade bitumen was needed, making binder too soft to hold the stone under the action of traffic.
- Insufficient rolling and / or excessive brooming of the stone before allowing it to be traversed by fast traffic.

Maintenance action to repair

If stripping is occurring due to aged and hardened binder it should be treated without delay to avoid further loss of stone. If there has been loss of scattered individual stones only the condition may be corrected by an enrichment coat (diluted emulsion) with or without fine cover stone or sand. If stripping has occurred over large isolated areas, a reseal will be necessary as discussed in Chapter 11.

10.3.4 Fatty, bleeding or slick surface

Description and causes

A fatty surface is due to surplus binder on the surface. Fatty surfaces become soft in hot weather (bleeding) and slippery in wet or frosty weather. Slick surfaces are hard, smooth, and slippery. They result from:

- loss of stone from seals due to stripping;
- excessive application of binder during surfacing operations or excess binder rising from the underlying surface, bad patches, etc.;
- breakdown of stone;
- over-filled voids in asphalt;
- stone penetrating the pavement surface and sinking into the binder.

Maintenance action to repair

The successful treatment of this condition is difficult and requires careful consideration and field trials before any extensive work is carried out. Possible methods of treatment are:

- reseal (discussed in Chapter 11 on rehabilitation),
- incorporation of additional stone into the existing binder. The existing binder should be softened by spraying it with cutter in hot weather. It is essential to use only clean, angular, small sized stone, preferably heated or precoated (not sand).

10.3.5 Polished stone surface

Description and causes

Exposed stones may become polished under the action of traffic, causing slippery conditions when wet even though the surface is generally not smooth. This is rare on low volume roads, and so is not covered here.

10.3.6 Shoving

Description and causes

A fairly regular waviness (somewhat resembling corrugations in an unpaved road) may develop in thick bituminous surfaces due to movement under traffic. The deformations are usually shallow and are not likely to be confused with larger depressions resulting from weaknesses in the pavement or the subgrade.

Maintenance action to repair

If the bituminous material has been compacted by traffic to a stable condition it will be practicable to fill in the depressions with asphalt. If it remains unstable it will be necessary to remove the unsound material and replace it with stable asphalt.

10.3.7 Rutting

Description and causes

Rutting usually takes the form of depressions in the wheel tracks. If the transverse deformation is accompanied by adjacent bulging of the pavement or shoulder surface it may be a sign of excessive subgrade movement or weak pavement.

Maintenance action to repair

The treatment in this case will be as described for shoving provided there is no subgrade movement. Any faulty subgrade material must be replaced by suitable material. Stable rapid setting slurries are often used to rectify rutting problems.

10.3.8 Cracks

Description and causes

A bituminous surfacing may crack for a variety of reasons and often, in early stages, the crack pattern can indicate the cause. When the cracks have developed over a large area and become sufficiently wide and numerous to allow the entry of surface water or disturbance of the surfacing by traffic it can be very difficult to determine the original cause of the trouble. For these reasons, it is important that the initial cracking be investigated so that the appropriate remedial action is taken later. Such cracking can seldom be seen from a moving vehicle and an inspection on foot is always desirable. Even a narrow crack will allow appreciable quantities of water to penetrate the pavement.

Thin surface layers of asphalt or slurry are less flexible than seals and therefore more likely to crack; for both forms of surfacing, flexibility is reduced by the hardening of the bitumen binder as the surfacing becomes older, and the propensity to cracking increases.

Cracks should generally be treated to prevent water from entering and destroying the base course where the materials are moisture sensitive. As it is often difficult to know *a-priori* which are problem materials, crack sealing or patching is a routine maintenance activity that should be performed on a scheduled basis as defined by local experience as to its cost-effectiveness. It is most effective or warranted in wet environments, or where moisture sensitive materials have been used.

Shrinkage cracks

Shrinkage cracks are likely to be found in all types of old bituminous surfaced pavements. They usually start as small regularly spaced cracks forming at right angles to the edge of the pavement and may develop into an irregular pattern of large blocks often having a concave shape or transverse cracking. This condition is caused by the shrinkage of the bituminous layer itself and is more common in asphalt pavements and in hot arid climates.

Reflection cracks

Reflection cracks reproduce the crack pattern in the pavement underneath and may be longitudinal, transverse, diagonal, or block. They occur most frequently in overlays on stabilised base pavements. They may also occur in overlays on bituminous pavement wherever cracks in the old pavement have not yet been properly repaired. This type of crack is not usually associated with pavement distortion. In drier areas, stabilisation cracks can sometimes be left untreated because the basecourse is generally not moisture sensitive.

Edge cracks

Edge cracks are longitudinal cracks (parallel to the edge), typically about 300mm in from the pavement edge, and may have branching transverse cracks. These cracks are usually caused by expansion and contraction of the subgrade. They can be due to regular moisture movements, unusual conditions of drought or flood, or because the pavement or subgrade was not constructed at its equilibrium moisture content or because of poor shoulder maintenance. They are often associated with expansive soil, and are important to treat (and retreat). The use of geotextile/bitumen strips or patches may be indicated.

Crocodile cracks

Crocodile cracks are a series of small blocks resembling the pattern on a crocodile skin, and they are often a further development of the previous four types of cracks mentioned. In most cases crocodile cracking is caused by excessive deflection of the surface over unstable subgrade or pavement, particularly in the vehicle wheel tracks. The excessive deflection may be brought about by many repetitions of heavy loads or by few repetitions of very heavy loads. When crocodile cracking occurs in the form of fine cracks not accompanied by surface deflection, this may be due to the age and brittleness of the binder, in which case the cracking usually occurs between the wheel tracks or over the full width of the road.

Maintenance action to repair

Various methods exist to reseal cracks, from cleaning and filling to covering with impermeable elastic material. Cracks wide enough to be treated (say 3mm) should either be patched using the various commercially available patches or strips, or filled with a (preferably modified) bituminous binder such as a cutback bitumen or bitumen emulsion (possibly modified). With the patches and strips, the surface is primed and the patch or strip laid over. With the bituminous binder, it must have a viscosity low enough to enable it to be poured or worked into cracks. A squeegee is useful in assisting a binder to penetrate cracks, or there are commercially available crack sealing machines which assist in achieving this penetration. Light sanding of the surface may be necessary to prevent traffic picking up surplus binder. Large areas with cracks probably require resealing with a modified binder, or surfacing with a full width geotextile reinforced surfacing.

10.3.9 Pot Holes

Description and causes

Pot holes can develop from pavement or surfacing failures. Pot holes not accompanied by distortion of the adjacent surface are usually due to a cracked bituminous surface allowing moisture to enter the pavement. The pavement may soften slightly or water may penetrate horizontally under the bituminous surface. In either case a small area of the cracked wearing surface is likely to lift out under the action of traffic, starting a pot hole. Such pot holes usually appear after rain. They may occur where a bituminous surface has not bonded properly with the course below, due to inadequate priming, dusty patches, or incorrectly omitting the tack coat under a course of asphalt.

Maintenance action to repair

The repair of pot holes will involve the vertical trimming of edges and removal of loose material prior to reinstatement. The bituminous surfacing should be further trimmed back so that the edge does not coincide with the edge of the pothole. Gravel or fine crushed rock are usually used to fill the lower portion of the hole, preferably stabilised with bitumen, cement or lime; and it is then necessary to seal the compacted material. This is done by priming the exposed surface and then covering with asphalt, a seal, or a commercially available patch. If open-graded asphalt is used, then due to the relative ease with which moisture can penetrate it, it should have a seal or commercial patch placed on top of it. Shallow potholes such as those caused by surfacing failures can be filled directly using commercially available cold mixes or slurry.

10.3.10 Edge Failures

Description and causes

Fretting or breaking of the edge of a bituminous surface may be caused by worn shoulders (resulting in insufficient side support to the pavement), inadequate strength at the edge of the pavement, or entry of water through the shoulder.

Maintenance action to repair

There are two repair options. The first is that the failed area should be boxed out and both pavement and shoulder material replaced. After thorough compaction asphalt should be applied. The second is that the edge failure be covered with a geotextile/bitumen bandage with possibly some surface shape correction by an overlay.

10.3.11 Large Depressions

Description and causes

Large depressions occur in the pavement surface when a fill has been inadequately compacted, for instance at bridge abutments.

Maintenance action to repair

Depressions caused by failure to compact the fill may continue to increase in size and depth, and deep seated correction may be necessary. If it is deemed satisfactory to make a surface correction, asphalt would be normally used.

10.3.12 Service cuts and trenches

Description and causes

Service cuts are cuts made within or across the road surface to provide a crossing or channel for services such as cables, pipes etc. Maintenance of service cuts is a special activity which is only required when a service cut has been made and the initial patch, if any, is not performing well. Quite often, the service company that made the cut is responsible for patching the cut and returning the pavement to a serviceable condition.

Maintenance action to repair

The repair of service cuts is essentially the same as for potholes.

10.4 Maintenance management

Maintenance management systems in South Africa have been in operation for a number of years. They include the MDS (Visser, 1984), systems used by the various Provinces, and systems developed by various consulting engineers.

The foundation of any maintenance system is the road inventory, which is a database of all the roads in the network with information on the road condition and traffic. Desirably it also includes data on pavement history and structure. The road inventory usually forms part of a pavement management system (PMS), and in most road authorities, the PMS is developed with a major maintenance component built in. At a later stage, a more sophisticated maintenance management system (MMS) module is then developed which can control minor maintenance as well.

10.4.1 Pavement management systems

The basic pavement management system has been developed for paved roads. It uses a database of present pavement conditions, requires the minimum in technology and has no prediction capability for pavement deterioration. The data base is used to estimate current maintenance and rehabilitation needs, and funding requirements to meet those needs. A simple prioritising method is usually added.

The more sophisticated PMS systems include prediction models to estimate life cycle costs for various maintenance and rehabilitation strategies and to prioritise the planning and programming requirements within budget and policy constraints. This can be developed further to include network optimization, in which the maintenance and rehabilitation strategies can be found to meet the performance standards over a long period and not exceed available funding.

For any road authority, some form of basic PMS is considered very useful for its paved road network. If the capability is not available in-house to operate a PMS, this can be done commercially. For an authority with a high proportion of low volume roads and a restricted budget, a very basic PMS would be adequate.

10.4.2 Unpaved roads maintenance management systems

The maintenance management of unpaved roads differs from that of paved roads in that the maintenance effort is more intense and more difficult to predict and control. For that reason, the maintenance of unpaved roads may be controlled by a separate system outside the normal PMS. It is important to control the maintenance of unpaved roads. The annual expenditure on maintenance of unpaved roads in southern Africa in 1991 was of the same order as that of paved roads.

Maintenance management systems for unpaved roads provide answers to questions such as the following (Visser, 1984):

- What budget is required?
- How many graders and staff are required?
- How often should each road be bladed?

- What is the resultant level of serviceability?
- What volume of gravel needs to be replaced annually?
- What roads should be upgraded to bituminous standard?

Presently the management of the maintenance of unpaved roads is limited over much of southern Africa. The frequency of grader maintenance tends to be based primarily on the number of graders available with a systematic type of programme being followed to utilise each grader to maximum advantage.

There are now computer programs which optimise the unpaved road maintenance schedule on the basis of the total transport costs. The Maintenance and Design System (MDS) is one such program. The MDS can be used to optimise the grader blading frequency of the road links in a network. Those roads for which it is economically beneficial (in terms of the total cost) to maintain more frequently (because of traffic or material characteristics) are identified. An idealised regravelling frequency of the links is also obtained. An advantage of the MDS is that the annual loss of gravel on a district or network basis is estimated and the amount of gravel which needs to be replaced annually to avoid the development of a backlog of gravel replacement for the district or network as a whole is identified. This can be used beneficially in the budgeting process and avoids periods when unexpectedly large quantities of gravel are suddenly required. Aspects such as "when is it more economical to seal certain roads than maintain them" and "the optimum number of graders required" are also covered by the MDS.

The MDS only indicates the predicted maintenance requirements of a link and does not identify problem areas within the link. Often short lengths of the link may become subjected to bad erosion, potholes, excessive churning of the wearing course or corrugations. These problems can only be identified by detailed inspection and are often best remedied by labour intensive techniques, spot regravelling or low-cost drags.

10.5 Maintenance outside the pavement

10.5.1 Roadside maintenance

Outside the pavement, the roadside maintenance encompasses the full road reserve. The main activities are vegetation control, litter, drainage, and structures.

10.5.1.1 Vegetation control

The main maintenance activity affecting the roadside is vegetation control: bush clearing and grass cutting. This procedure is carried out mainly for safety reasons but also to avoid damage to vehicles from vegetation overhanging the pavement edge and to reduce the fire hazard in some areas. The frequency of this maintenance depends on the relevant level of service (Table 10-1) and should be adapted for the funding level and the area under consideration. Areas with a high rainfall and short-radius horizontal and vertical curves (i.e. short sight distances) will require considerably more vegetation control than long straight roads in arid areas. The frequency of vegetation control is recommended in Table 10-3 (Department of Transport, 1989).

10.5.1.2 Litter

Collection of litter should also be programmed periodically, especially near built-up areas and on more heavily trafficked and tourist routes. Debris from car accidents, discarded car parts (e.g. exhaust pipes and silencers) which periodically litter the road and dead animals should be removed as soon as possible.

Table 10-3: FREQUENCY OF VEGETATION CONTROL

LOCATION	RECOMMENDED FREQUENCY[a]	NOTES
Road reserve	Cut once per year	
Shoulder (2m wide strips), near signs, and where obscuring sight lines	At least once per year	Maximum grass height 300mm
Weed spray	As required	Not more than once per year
General bushing	As required	Not more than grass cutting frequency

Note a: Vegetation control is expensive and may be further reduced on very lightly trafficked roads

10.5.1.3 Drainage

Repair and prevention of erosion affecting drains, cut and fill slopes, ditches, and culvert inlets and outlets is needed occasionally. Drainage maintenance is especially important to ensure good life from paved roads and to keep unpaved roads functional and in good repair.

For unpaved roads, the first significant problem is to remove the bulk of the rain-water from the road surface without causing erosion of the gravel wearing course. For this to occur effectively the surface of the road should be well maintained with a good shape (definite crown), no potholes, no deep corrugations or ruts and an adequate cross-fall. Experience shows that a cross-fall of about 4 or 5 per cent is the optimum which allows adequate run-off without erosion. Longitudinal slopes and cross-falls steeper than about 5 per cent are prone to erosion. On the steep slopes commonly encountered in the eastern areas of southern Africa, erosion is a significant problem and no cost-effective way of avoiding this has been developed as yet.

For side and mitre drains, these should be designed with widths and side-slopes (not more than 1:2 or 1:3; and desirably shallower to improve safety for vehicles running off) which permit ready access of a motorised grader so that maintenance can be carried out during the routine pavement surface maintenance. It is important to ensure that routine grader blading does not leave windrows blocking the entrance to mitre drains. Drain maintenance should endeavour to retain the grass cover which reduces the erosion potential. This is especially necessary during manual clearing around culverts and drains. It is important that silt excavated from drains is removed as far as possible from the drains, as it is of too poor quality to use to patch or repair the road surface.

Regular inspection and maintenance of culverts is necessary to ensure that they are not affected by erosion, silting, blocking or corrosion. The pipes, headwalls, joints and adjacent fills and outlets should be carefully inspected and repaired where necessary. Failure to remedy problems in time may result in extensive damage during wet periods.

10.5.1.4 Structures

Bridges should be inspected for damage to handrails, abutments, piers, decks, woodwork and guideblocks. Scouring of the foundations and approach fills adjacent to wingwalls, should be remedied and any excessive cracking of the piers or deck reported to the structural engineer. Exposure of reinforcing material should be specifically checked for.

10.5.2 Shoulder maintenance

The surface of the shoulder may be earth, gravel, or sealed. It should have a smooth running surface, a minimum of loose materials, an adequate slope for drainage, sufficient strength to support wheel loads and a surface flush with the pavement edge. Shoulder maintenance will be needed to maintain these conditions.

For earth and gravel shoulders, maintenance involves:

- smoothing and reshaping, usually with a grader,
- adding new material to replace material which has been lost by the action of traffic or water erosion,
- boxing out and replacing material in weak and rutted sections.

For grassed shoulders, maintenance involves:

- mowing,
- correcting the accumulation of soil both from the road and the shoulder in the grass alongside the road and eventually the formation of a ridge adjacent to the pavement. Water accumulates against this ridge after rain and seeps into the pavement leading to failures of the base.

For sealed shoulders, maintenance involves:

- edge patching,
- resealing (normally done in conjunction with the reseal of the whole road; this is discussed in section 11 on rehabilitation).
- application of diluted emulsion to compensate for there being minimal traffic on the shoulder

10.5.3 Signs and markings

Signs and markings should be maintained in legible condition; this is especially important for safety related signs. Reflectorised signs after a period of time suffer considerable loss of reflective efficiency, even though they do not appear faded in daylight. Signs should be kept clean and free from dirt, diesel smut, vandalism, and other contamination. Road markings should be repainted periodically. They should never be allowed to deteriorate such that less than 10% is visible. The frequency of remarking depends on the type and life of the paint used. The recommended maintenance schedule for signs and markings is given in Table 10-4.

Maintenance of guard rails and guide posts is primarily a matter of replacing them following accidents and damage due to general wear and tear.

Table 10-4: MAINTENANCE OF SIGNS AND MARKINGS

Item	Maintenance	Frequency
Reflectorised signs	Washing	As required
	Replace	Every 5-7 years, or when damaged
Non-reflectorised signs	Paint	Every three years
Road markings	Repaint	< 300 vpd: every 4 years 300 - 2000: every 2 - 4 years
Road studs[a]	Replace	When >20% have been lost

Note a: Some authorities replace road studs less often.

10.6 References

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11 REHABILITATION

11.1 Introduction

The rehabilitation of roads covers more extensive work than maintenance. In this document, rehabilitation of paved roads would include resealing and more substantial repair. For unpaved roads, it covers regravelling and more substantial works, and includes paving an existing unpaved road.

11.1.1 Philosophy of rehabilitation

Pavements deteriorate with time due to ageing, environmental influences, and traffic load. The need for rehabilitation arises when the pavement shows some form of deterioration in its riding quality, or when the potential for such deterioration exits and some action to prevent it is economically feasible. Rehabilitation can be divided into two related phases: pavement rehabilitation and geometric rehabilitation. The pavement rehabilitation is aimed at restoring the pavement to a certain standard. Geometric rehabilitation is aimed at improving the alignment, width or other geometric parameters to current standards. It is common on high volume roads for both to be performed. The trigger is usually the need for pavement rehabilitation, but geometric rehabilitation is undertaken simultaneously. It is common for this to result in substantial projects.

For low volume roads, as a general rule, geometric rehabilitation is not justified simply because the road is undergoing pavement rehabilitation. Geometric rehabilitation is justified because:

(i) there is a high rate of accidents on the road; or

(ii) there has or will be a significant change in the traffic since the road was built or last rehabilitated.

The pavement history is the most important guide to the extent of pavement rehabilitation needed. If the pavement fails in the first few years of service, then there is likely to be a problem with the materials, design or construction, and the implication is that rehabilitation could be extensive. However when a pavement has been in service for 10 to 20 years without serious problems, then it can be assumed that there is little wrong with it, even though some parameters may not meet the standards for new pavements.

For low volume roads, as a general rule the decision on the extent of rehabilitation should be made according to the pavement history, current condition and in situ strength and not necessarily according to the standards for new pavements.

Naturally it should be recognised in the analysis that changes in structural capacity may have occurred (e.g. as a result of cracking of the surfacing and water ingress) or that changes in the mass of traffic on that pavement may have occurred or be expected, and these should be taken into account. In addition, the strength of each layer is dependent in part on the strength of the layer beneath, so that a high quality, densely compacted basecourse requires at least a subbase of reasonable quality and good compaction. However the rehabilitation design of low volume roads should seek to use as much of the existing pavement as possible, and for that reason the investigation and design should concentrate on relevant measures of the actual road performance.

Example *There are some roads which do not meet the skid resistance requirements. This can occur on low volume roads where the bitumen application rate has been increased compared to busier roads. For roads in drier areas, this requirement can be relaxed and is not in itself justification for rehabilitation. The exception is if that section is an accident blackspot due to a lack of skid resistance. However in such cases, it will be found that only a short section of road will need treatment.*

11.1.2 Timing of pavement rehabilitation

The timing of rehabilitation is a function of the terminal levels adopted and the mechanism of failure based on safety as first priority, protection of pavement as second and preventative needs as third. A guide to timing is given in Table 11-1 and a guide to terminal levels is shown in Table 11-2. The terminal levels used though vary from authority to authority, and some use a more sophisticated approach of "warning level" and "unsound level". The terminal level should be considered in the light of the circumstances for each road. If a road matched some terminal level and it had granular layers, in a wet area, with a high traffic growth and a high percentage of heavy vehicles, it would indicate a need for action. However a road carrying light traffic, few heavy vehicles, nil traffic growth, dry area could probably be allowed to exceed the terminal levels slightly, depending on the expected rate of deterioration

It should be noted that it is more cost-effective over the life of the pavement to consider timing in terms of preventative action rather than restoration of a damaged asset. For low volume roads, this generally means timeous resurfacing before the structural strength of the road is lost, and a programme of resealing at fixed intervals can have merit if no more sophisticated management system is available. Various strategies can be modelled in a sophisticated analysis to determine the optimum rehabilitation timings, but almost always, the benefit of preventative action can be shown.

One problem with the timing of rehabilitation is the lead time for project investigation, design, tendering, etc. once the problem has been noticed. Generally it can be anticipated that rehabilitation could take a year to start from the time the need is first noticed. In the interim period, if the surfacing is not waterproof the rate of deterioration of the pavement usually accelerates rapidly (Figure 11-1), and the pavement structure can be very badly affected by the time the rehabilitation starts. If the rehabilitation is intended to rebuild the road, then this is perhaps not so serious, but if the rehabilitation was limited to restoring the riding quality or a reseal, then the delay can be very serious and potentially costly.

Table 11-1: TIMING OF REHABILITATION

LEVEL OF REMEDIAL ACTION	DISTRESS MODE	RECOMMENDED TIMING
ROUTINE	POTHOLES > 150 mm	IMMEDIATELY
	POTHOLES < 150 mm	AS SOON AS POSSIBLE (ASAP)
	CRACKS > 3 mm	ASAP
	EDGE BREAKS	AS FOR POTHOLES
RESEAL	ACTIVE STONE LOSS /RAVELLING	IMMEDIATELY
	PUMPING	IMMEDIATELY
	SURFACING FAILURES	IMMEDIATELY
	SKIDDING PROBLEMS	IMMEDIATELY
	FINE CRACKING	URGENTLY
	RUTTING ONLY	URGENTLY
	DRY BINDER	ASAP
	FATTINESS	ASAP
REHABILITATION CALLED FOR WHEN:	MAINTENANCE IS INSUFFICIENT TO DEAL WITH FAILURES OR POTHOLES MORE THAN 15 % OF A 3-5 km LENGTH OF ROAD IS AFFECTED BY THESE FAILURES. RIDING QUALITY IS POOR (AVERAGE PSI < 2.2)	

Table 11-2: TERMINAL LEVEL FOR REHABILITATION

PAVEMENT CONDITION	DISTRESS	
	LEVEL	% ROAD EXCEEDING LEVEL
Rut depth	20 mm[a]	20%
Cracking crocodile longitudinal other		 15 - 25%[b] 60 - 90%[c] 30 - 50%[c]
Disintegration Patching Ravelling		 10 - 20%[d] 40 - 60%[e]
Smoothening		40 - 60%[f]

Source: from TRH12 (1991) and TRH4 (1985)

(a) For low volume roads, dependent on the risk of skid resistance related accidents, this can be 30 mm

(b) Level indicated is a percentage of the wheel path, not over the full width of the road

(c) Longitudinal and 'other' cracking usually requires crack sealing and not major rehabilitation

(d) It is usually more cost effective to rework the pavement layers if more than 10 - 15 % of the pavement is affected

(e) Ravelling should be prevented through application of diluted emulsion or reseal; it does not require major rehabilitation

(f) If the extent of smoothening is more than isolated can be a problem, but this is dependent on the risk of accidents which is in turn related to traffic, visibility and rainfall

11.2 Investigation

The full investigation of rehabilitation project should cover three areas:

- general assessment of road and surrounding environment,
- visual assessment of surface of road, and
- structural assessment of pavement.

In general the extent of investigation depends on the extent of rehabilitation required. A reseal on a road with no structural distress requires mainly a visual assessment of the road. The extent of investigation should be guided by Table 11-2. The individual items are discussed in more detail in the following pages.

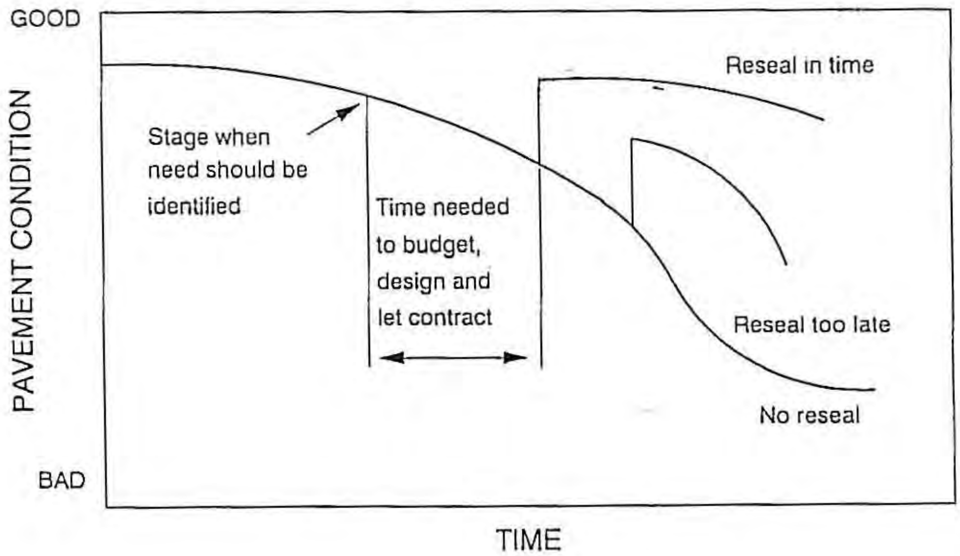


Figure 11-1: OPTIMAL TIMING OF RESEALS

Table 11-3: EXTENT OF INVESTIGATION

GENERAL ASSESSMENT	EXTENT OF INVESTIGATION	
	Structural rehabilitation unlikely (i.e. reseal)	Structural rehabilitation expected
History of the road	only reseal history	in detail
Geology/terrain/climate	limited	in detail
Traffic	limited	in detail
Drainage	in detail	
Geometry and alignment	nil	in detail
Source of materials	surfacing only	in detail
VISUAL ASSESSMENT	limited: check that structural rehabilitation not needed	in detail
STRUCTURAL ASSESSMENT	should not be required	in detail

11.2.1 General assessment

The general assessment is made by considering the following aspects of the road and its surrounding environment:

i) History of the road

In practical terms, the investigation is guided by the pavement history. If the pavement has failed in the first few years of service, then the investigation should be aimed to uncover the cause of failure in one or all of materials, design or construction. If a pavement has been in service for 10 to 20 years without serious problems, then it can be assumed that there is little wrong with the materials, design or construction, and it is rarely necessary to undertake a full soil survey. The history can be found from one of more of these sources:

- as-built drawings,
- original design specification,
- pavement management system records,
- a limited field investigation to establish what constitutes the existing construction, and/or
- life or the road to date in years, since construction.

ii) Geology, terrain and climate

A limited geological report is required to establish the subgrade conditions e.g. does the road traverse decomposed basic rock (black turf), or acid rock (stable subgrade), or both? Is the overburden deep or shallow? The climate should also be noted (e.g. rainfall, minimum and maximum temperature).

> *Hint: where the road passes over active subgrades, the riding qualities of the road surface tend to undulate and in many cases there is no actual failure of the road except that the riding qualities become poor. The time period for these undulations to take place could extend over a number of years and it is more economical in many cases to take out these undulations with levelling course asphalt than to reconstruct the whole section affected. These undulations can be taken out with ordinary hot premix if placed in the summer months and levelled off with a grader and there is no necessity to bring a premix train in to do the work.*

iii) Traffic

The requirements for traffic data are given in Chapter 5; generally the following traffic data should be collected if possible:

- total E80s (E80 is a standard 80kN axle load) that the road has carried to date,
- anticipated E80's the road must carry
- present traffic count and analysis of the traffic (as well as percentage overloading), and

- possibility of industrial traffic using the service (quarries and cement works etc).

iv) <u>Drainage</u>

The drainage structure of the road to a very large extent affects the life of the road and a detailed investigation report should be made regarding the adequacy of:

(a) Surface drainage (standing pools due to rutting etc.),
Hint: latest research shows that ruts as shallow as 10mm on a road crossfall of 2% can lead to water standing in the ruts after rain and significant increase in the deterioration of the pavement. This is important for roads in wet climates, and especially with flat crossfalls. The investigation should include the frequent use of least a stringline across the road to measure the rut depths in the wheeltracks.

(b) Table drains (longitudinal drains).
Hint: look out for and correct:
- *rock intrusions in the table drain,*
- *any sign of ponding,*
- *"V"-shape construction (note that flat bottomed drains are far superior),*
- *table drains constructed on the lower side of the road where the formation falls away from the road, catching the water that would normally flow away.*
- *erosion taking place in cuttings and on embankments which will silt up the drainage system and culverts.*

(c) Mitre drains,
Hint: ensure that there is an efficient flow of water entering the mitre. There are many cases where at the entrance of the mitre drain, the water ponds and this affects the foundation layers of the road resulting in failures or undulations on the surface of the road.

(d) Culverts:

(i) adequacy of opening (size) (eg flooding) and length of culvert (road width);
Hint: the length of the culvert, that is the road-width of the culvert, is important from a safety point of view. On many of the old roads the box culverts are less than 7,3m wide and these can be widened or extended economically with either pre-cast box sections or in-situ concrete.

(ii) inlet and outlet conditions (eg ponding, headwalls)
Hint: drop inlets can be used to allow silting to take place in the table drain before entering the culvert if it is difficult to get a reasonable slope in the pipe to carry the silt through to the lower end of the culvert.

(iii) silting

Hint: silting-up of the table drains or longitudinal drains can be an indication of too few culverts. Grassing of erodible slopes and erodible embankments is money will spent and it is labour-intensive. The grassing of the flat bottom table drain is also advisable.

(e) Erosion (caused by inadequate drainage systems, too high velocities, cuttings and embankments)

Hint: box culverts in a valley may be quite large to take the flow of the water at a fairly high velocity. This causes erosion on the downstream side of the vlei, and this can be solved by putting concrete walls across the channel to spread the flow of the water beyond the culvert and reduce the velocity.

Hint: when passing through geological areas where dispersive soils occur, severe erosion problems can be encountered, not only on the embankments but also in the table drains. These can be addressed by treating the surface with lime or gypsum and establishing a good grass coverage of the area.

(f) Cause of any pumping in foundation layers and proposed solutions (e.g. underdrains).

(g) The watertightness of the surfacing. Large cracks are easy to see, but if the road can be inspected during wet conditions, the minor cracking of the surface can also be seen.

v) <u>Geometry and Alignment</u>

The geometry and alignment of the road must be analyzed with respect to their suitability for the traffic projected for the design period of the rehabilitated pavement. The geometric standards for new low volume roads are given in Chapter 3. These should not be automatically adopted for the rehabilitation of roads, and careful consideration should be given to the cost implications of geometric upgrading. As a general rule, geometric rehabilitation is not justified simply because the road is undergoing pavement rehabilitation. Geometric rehabilitation is justified because:

(i) there is a high rate of accidents on the road; or

(ii) there has been a significant change in the traffic since the road was built or last rehabilitated.

The analysis should consider:

- the existing cross-section of road and shoulders, e.g. shape of surface and shoulders (e.g. condition of shoulders),
- intersections - position and safety - slip lanes,
- climbing lanes - depending on traffic,
- necessity for surfacing the shoulders or part of shoulders,
- minor horizontal and vertical alignment changes that may be required - as well as inadequate super-elevation,

- the existing standard of the road or sections from a safety point of view.

vi) Source of materials

The source and availability of base, subbase and shoulder material should be established as well as the cost implications (e.g. crusher run versus natural gravels)

Hint: material may be available from within the road reserve by widening and "daylighting" cuttings.

11.2.2 Visual assessment

A visual assessment of the road is essential. It is used to identify weak areas and isolated failures, to guide the selection of rehabilitation type, and to assist in dividing the road into uniform sections. The visual assessment should be performed in accordance with TRH 12 or other road authority manual. The following parameters are usually recorded:

- rutting - rut depths should be measured,
- ravelling - condition of binder, percentage stone loss,
- surface texture - variation, skid resistance, bleeding.
- failures - type of failure, e.g. surfacing, base, subbase, subgrade. Cause of the failures. Extent of the failures and percentage failure of total road surface or percentage failures of sections of the road,
- riding quality of the road - demarcated in 100 metre sections,

11.2.3 Pavement structural assessment

The pavement structural assessment is used to measure the quality of the insitu pavement and its materials (as an input into the rehabilitation design phase), and also to measure the remaining life of the existing pavement (as a measure of the extent of structural rehabilitation necessary). This can be done by a number of methods, and these can be used singly or in combination. No single method is necessarily superior to the others, and each method has its advantages and limitations.

Structural assessment measures include measuring:

- deflection of the pavement under a load (Benkelman Beam, Lacroix deflectograph, IDM falling weight deflectometer),
- deflection considered together with the deflection bowl parameters,

- Dynamic Cone Penetrometer (DCP),
- test pits and laboratory testing of materials,
- insitu plate bearing or CBR tests, and
- rut depth measurements.

The results of the structural assessment, together with the results from the other assessments, are used to divide the road into sections of uniform strength. It is seldom that the whole road fails, and it may be therefore be possible to limit the rehabilitation to sections of the road. The residual life is estimated separately for each section. If the residual life is inadequate, then rehabilitation design will be needed, which is discussed in section 11.3.2.

11.2.3.1 Estimation of residual life using the DCP method

The estimation of residual life is shown here using the DCP method (Kleyn, 1975; Kleyn and Van Heerden, 1983). For low volume roads, the DCP method has been shown to give reasonable results in many cases, and it has the advantage that it is simple to use. The limits to its application for assessing residual life are:

- it is only applicable to pavements with thin bitumen surfacings and granular or lightly cemented (UCS < 3 000 kPa) layers,
- it is limited to balanced or nearly balanced pavements (the pavement strength decreases reasonably uniformly with depth),

Note that these limitations do not apply to its use in rehabilitation design as discussed in section 11.3.2. In this method, the residual life is calculated as a functior ›f:

- the moisture regime, and
- total number of blows to penetrate the pavement layers to a depth of 800 mm (DSN_{800}), and/or
- total number of blows to penetrate the pavement layers to a depth of 200 mm (DSN_{200}), and/or
- DCP penetration rate for the upper 50mm of the pavement in blows/mm (DN_{50}).

The steps in the method are:

<u>STEP 1</u> <u>IDENTIFY THE MOISTURE REGIME</u>

The moisture regime or drainage condition is identified as one of:

- dry moisture regime or good drainage conditions (M1); found in hot, arid climates such as the northern Cape,
- optimum moisture regime or average drainage conditions (M2); this is a typical condition in Transvaal and the Orange Free State,

- wet moisture regime or poor drainage conditions (M3); this is common in Natal, coastal Cape Province, Eastern Transvaal, and mountainous areas,
- soaked moisture regime (M4); this is not common in South Africa, but could be found in very wet areas such as vleis.

STEP 2 DO DCP TESTING ALONG THE ROAD

DCP testing is performed along the length of road. The frequency of tests should generally be in accordance with the standards here, but the visual inspection may indicate adjustments to the frequency. If the road is very uniform the frequency can be reduced, and if it is variable then it should be increased. The basic frequency should be:

- test at the rate of 5 DCP tests per kilometre, with the tests staggered as centreline, outer wheeltrack one side, outer wheeltrack other side, centreline, etc;
- perform an additional test at every significant location picked up in the visual survey, such as particular failure areas;
- ensure that at least 8 DCP tests are performed per likely uniform section to provide adequate data for the statistical analysis.

STEP 3 DIVIDE ROAD INTO UNIFORM SECTIONS

The results of the DCP testing, together with the visual assessment will enable the length of road to be divided in relatively uniform sections for the purposes of residual life capacity assessment.

STEP 4 CALCULATE THE REPRESENTATIVE DCP VALUE FOR EACH SECTION

The representative DCP value (DSN_{800} or DSN_{200} or DN_{50}) for each section is calculated statistically to provide a safety margin against the variability of material within the section. A normal distribution of data is assumed and the Student's T distribution at the 80% level is used:

$$\text{representative } DSN_{800} = \text{mean } DSN_{800} - .9 * (\text{standard deviation } DSN_{800}) \quad \ldots\ldots\ldots (11.1)$$

$$\text{representative } DSN_{200} = \text{mean } DSN_{200} - .9 * (\text{standard deviation } DSN_{200}) \quad \ldots\ldots\ldots (11.2)$$

$$\text{representative } DN_{50} = \text{mean } DN_{50} + .9 * (\text{standard deviation } DN_{50}) \quad \ldots\ldots\ldots\ldots (11.3)$$

Example The DCP results in a section were as follows:

DSN_{800}: 125, 143, 120, 100, 145, 115, 140, 135

Mean (average) = 127,9 Standard deviation = 15,7

Representative DSN_{800} = mean DSN_{800} - 0,9 * (standard deviation DSN_{800}) . . . using (11.1)

= 127,9 - 0,9 * 15,7 = 114

Note that equations 11.1 to 11.3 are using a one-tailed T-distribution for 8 samples and are reasonably robust for sample sizes from 5 to 30.

STEP 5 CALCULATE THE RESIDUAL TRAFFIC CAPACITY

Total traffic capacity is the total traffic from the time of construction to the assumed terminal condition of 20mm rut depth. For low volume pavements using the DCP method, this is a reasonable assumption of terminal condition. The residual traffic capacity is calculated as the difference between the total traffic capacity and the traffic to date.

The total traffic loading that the pavement structure can carry before developing a rut depth of 20mm is found from Figure 11.2, which has been graphed from equation 11.4.

$$E80 \times 10^6 = C_m \times 10^{-9} \, (DSN_{800})^{3.5} \quad \text{..........} \quad (11.4)$$

where: C_m = 64 for moisture regime M1 (dry)
C_m = 30 for moisture regime M2 (optimum)
C_m = 14 for moisture regime M3 (wet)
C_m = 6,5 for moisture regime M4 (soaked)

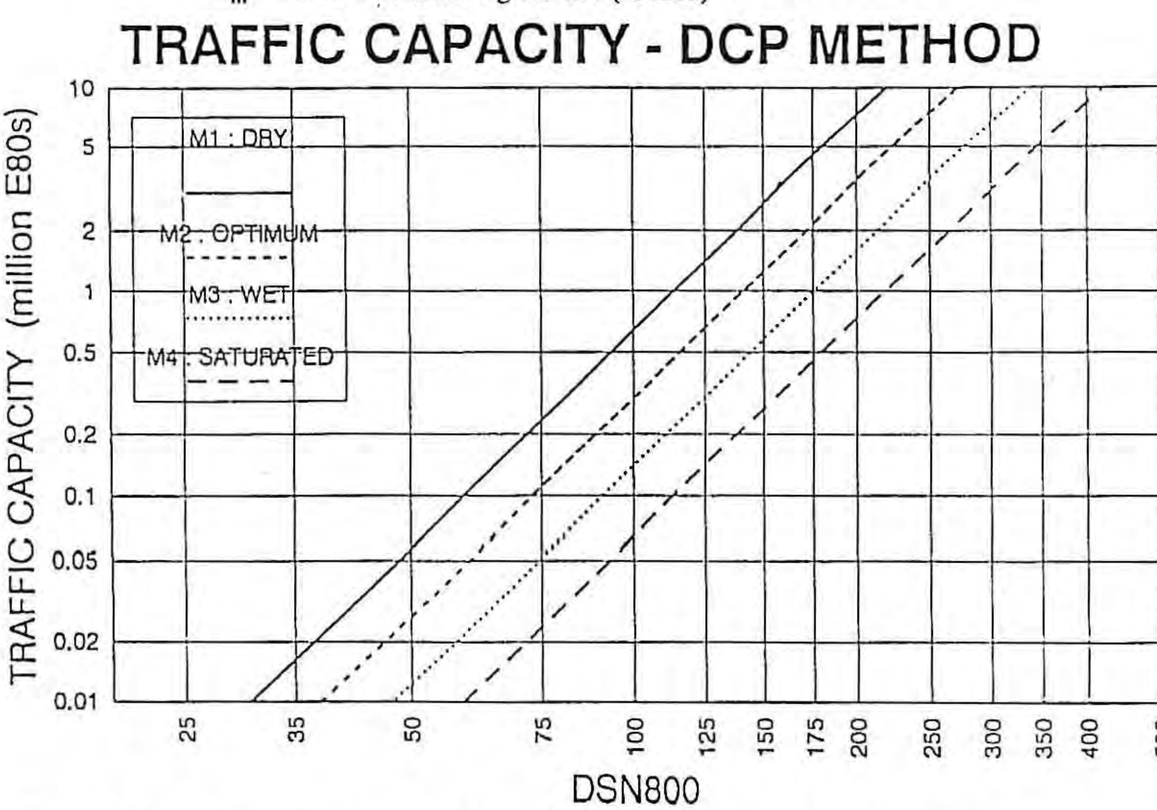


Figure 11-2: ESTIMATING TRAFFIC CAPACITY BY THE DCP METHOD

- For pavements containing only granular layers, the rut depth is assumed to increase with traffic loading as shown in Figure 11-3.

- For pavements containing any lightly cemented layers, the existing rut depth of the pavement section is used to calculate the remaining "rut depth" capacity which is translated into traffic loading by equation 11.5 (De Beer, 1990):

$$R_L = \frac{DSN_{200}}{10^{\,3,828 \; - \; \frac{DN_{50}}{1,3857}}}$$

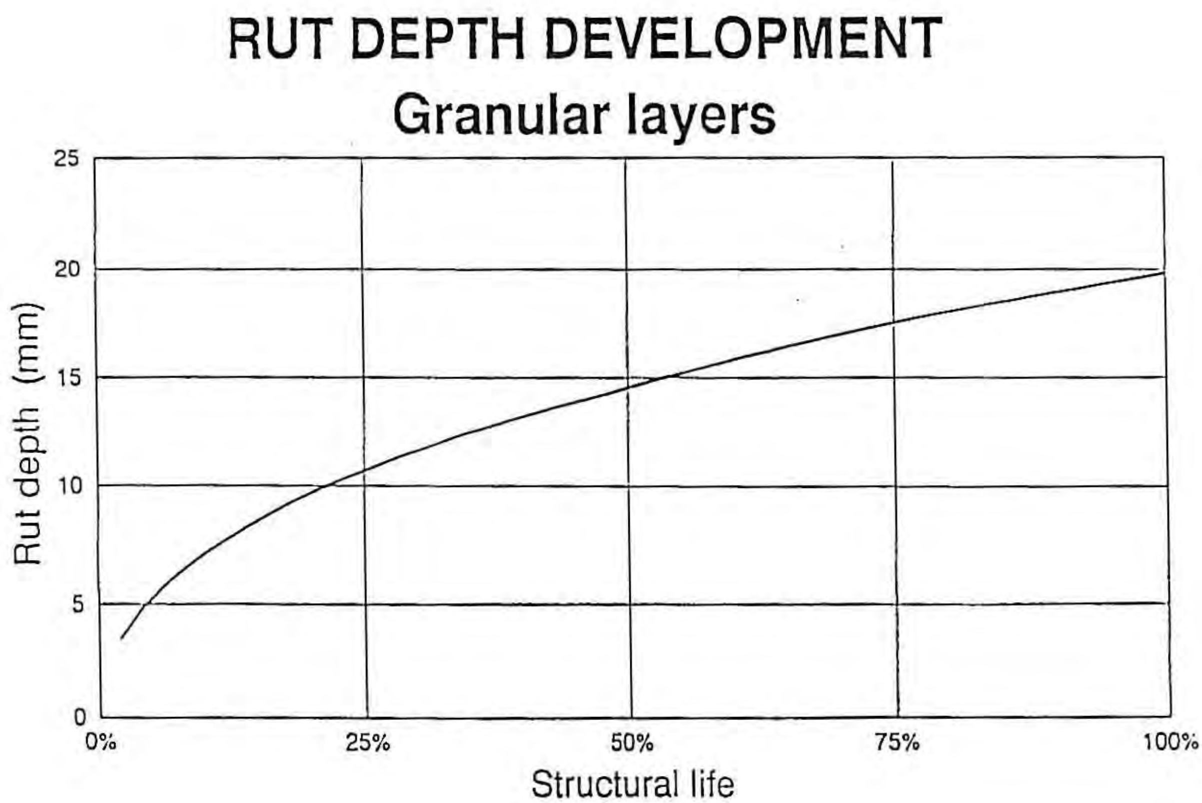


Figure 11-3: RUT DEPTH DEVELOPMENT IN GRANULAR LAYERS

11.3 Pavement rehabilitation design for paved roads

Pavement rehabilitation design on paved roads is considered here in two classes: non-structural rehabilitation (such as reseals and overlays) and structural rehabilitation (Table 11-4). This section applies to flexible pavements only, since the rehabilitation design for concrete roads is a specialised task.

Table 11-4: CLASSES OF PAVEMENT REHABILITATION

NON STRUCTURAL	STRUCTURAL
resealing	partial reconstruction (possibly reshaping, strengthening, and/or stabilisation
asphalt overlay	full reconstruction
holding action	

Any combination of these actions could be applicable to a specific pavement, and drainage improvements could be linked to any of these.

11.3.1 Non structural rehabilitation

The function of a surfacing is shown in Table 11-5. Resealing or a thin overlay is undertaken when the surfacing is no longer achieving the purpose for which it was applied, but the pavement is structurally adequate. A holding action (which could be patching and a reseal) is undertaken when the road needs structural rehabilitation but there is insufficient funding at the time.

11.3.1.1 Timing of resealing

The best time to undertake non-structural rehabilitation is when it causes the most benefits from an economic point of view. This is usually just before rapid deterioration starts taking place. This can be explained by looking at the performance curve in Figure 11-1. The application of a seal before rapid deterioration starts to take place will keep the road condition at a high service level. If the pavement is allowed to deteriorate too much, it is often found that the reseal strategy is not sufficient to restore the service level or that the effective life of the seal is very short. In these cases major rehabilitation is often the only solution. Note that effective programming for specific reseal projects cannot cover more than the following two or three years.

In theory this approach is sound. The problem is, however, that it is not always easy to forecast exactly when rapid deterioration will take place. Very often when a pavement shows rapid deterioration the damage has already been done. What also aggravates the situation is that the need must be identified well in advance in order to programme, design and let contracts.

Table 11-5: FUNCTION OF SURFACING

BASIC FUNCTIONS OF A SURFACING	MECHANISMS OR DEFECTS RESPONSIBLE FOR FAILURE	PRE-MECHANISM	ASSESSABLE ATTRIBUTES
Prevents ingress of water	Porous surfacing	Dry binder Too little binder	Dry binder Voids, stone loss
	Surfacing cracks	Dry binder Pre-cracking	Surfacing cracks Dry binder
	Structural cracks	Various	Structural cracks Pumping
Protects base from traffic wear	Surfacing failures, potholes	Surfacing cracks, dry binder, structural cracks	Surfacing failures, surfacing cracks, dry binder, structural cracks
Prevents windscreen damage	Stone loss	Too little binder or dry binder	Dry binder, too little binder

To minimise lifecycle costs, it is recommended to seal a year too early than a year too late because of the uncertainties of future pavement performance.

The simplest form of timing is to reseal roads at fixed intervals, which has merit in organisations with a limited capacity for regular inspections. While it is not possible to be prescriptive about the life of a surfacing, conservative guidelines are given in Table 11-6 to evaluate the intervals between resurfacing (Emery et al, 1991).

Table 11-6: EXPECTED SURFACING LIVES FOR RURAL LOW VOLUME ROADS

SURFACING	EXPECTED LIFE[c,d]	
	poor conditions[a]	good conditions[b]
Asphalt	10-14	15-20
Double seal, Cape seal	6-8	9-13
Single seal	4-6	5-9
Sand seal, single	1-3	2-4[e]
Sand seal, double	5-7	7-11
Dust palliative	1-3	2-4
Slurry, thin	2-4	4-6
Slurry, thick	4-7	8-10

Notes: a. Poor conditions means third world environment or problems such as weak pavement structure, poor quality control, poor drainage provision, etc.
b. Good conditions means first world environment with no problems.
c. This assumes that the surfacing is being used in an appropriate context (see Chapter 5).
d. Geotextile reinforced surfacings should have similar lives
e. Where the base is primed before the sand seal is applied, double the indicated life can be expected

11.3.1.2 Resealing

Resealing is performed when there is a need to improve the function of the surfacing and the shape, riding quality and rut depths are still acceptable. The choice of reseal type is made according to the following factors:

(i) Traffic volume

The general trend with lower traffic is to reduce the stone size (to 6,7 or 9,5 mm) with the effect of reduced binder application and thus reduced reseal costs. Other factors often contribute to a final decision of keeping to a 13,2 mm stone such as the availability and cost of smaller stone, pavement condition (more distress, more binder needed), embedment potential, and risk of sudden increase in traffic.

(ii) Heavy vehicles

The actions of heavy vehicles can create problems with gradients and stopping, and turning actions. At intersections with stopping and turning actions, or climbing lanes, problems occur with single seals. Use may be made of double stone seals such as 13/6, stone and sand, or stone and slurry.

(iii) Existing surfacing and/or pavement condition

The defects considered to have a decisive influence on the surfacing type are the following:

(a) Rutting

The decision is governed by the risk of skid resistance problems. For rutting less than 10mm on low volume roads in low rainfall areas, it is considered a low risk not to improve. In the wetter regions of the country, the situation is improved by the following:

- coarse stone seal to dissipate water,
- application of a coarse slurry (not recommended by all road authorities because of bad experiences of stripping in wet conditions).

In cases of rutting more than 10 mm with little other distress types, the measures are aimed at filling the ruts followed by a treatment of uniform texture. In cases of deep ruts and other distress types such as cracks it is considered important to add a stone seal on top of the slurry as soon as the embedment potential allows it.

Note that rutting does not necessarily indicate the need for full pavement rehabilitation. The pavement may still have residual strength, and the rutting may have been due to traffic compaction of a poorly constructed pavement. The DCP testing of pavement structural capacity will indicate the residual strength.

(b) Cracking

Different types of cracking, crack widths, secondary effects such as spalling, secondary cracking, spacing, and pumping all play a significant role in the decision to reseal and in the choice of the appropriate resurfacing type.

The severity of cracking has several dimensions. Crocodile cracking is considered the most severe type of cracking, followed by block cracking with small spacing and evidence of pumping and or secondary cracking. Other types of cracks such as transverse and longitudinal cracks are not often regarded as severe indications for reseal purposes unless pumping occurs and the spacing is small. The crack width and spalling influence the decision of when to reseal (priority) or influence the decision to rather seal the cracks individually.

As the severity and extent of cracking and patching increase, the more binder is needed to prevent reflection of the distress. Bigger stone, double or choked seals are therefore used on lower volume roads with severe pavement distress to extend the life of these pavements. The use of modified binders is increasing to deal with this type of situation. Some road authorities already believe that it is cost effective to use modified binders on low volume roads. The use of geotextile reinforced bitumen surfacings should also be considered especially if the cracking is related to expansive materials.

(c) Ravelled Surface

If ravelling has developed beyond maintenance capability or emulsion enrichment, it can be corrected by a reseal or slurry, or in severe cases resurfacing with asphalt will be necessary. Texture treatment may be desirable.

(d) Fatty, bleeding or slick surface

The successful treatment of this condition is difficult and requires careful consideration and field trials before any extensive work is carried out. Possible methods of treatment are:

- resealing with a 13,2 mm single seal with the use of modified binders if regarded as necessary,
- texture treatment with a slurry (not on a soft tacky binder and only when the

existing texture varies from smooth to coarse.

The need to rectify fatty surfaces is again dependent on skid resistance risks. If the risk of accidents is low it is not regarded important to improve.

(e) Dry and brittle surfacings

The appropriate action is dependent on the voids and texture of the surfacing and the traffic volume. If possible, a diluted emulsion is sprayed.

(f) Texture

Fine texture Single stone seals can be directly applied on this type of surface with the size of stone dependent on the traffic volume and the severity of distress types.

Medium to coarse textures Dependent on the type and severity of distress, various strategies can be followed, eg:

- application of diluted emulsion,
- small stone single seal such as a 6,7 mm,
- texture treatments with sand seals or slurry seals,
- inverted double seals in cases of severe distresses.

The application of bigger stone single seals is not recommended unless modified binders with high application rates are used. The reason for this is the risk of losing stone, especially those on top of the stones in the existing surface.

(g) Texture treatment

Texture treatment is applied to very coarse textured surfacings or in a situation where wheeltracks are fine textured or fatty with coarse textures in between. It is not considered important to reseal just because of the variation itself. However when the need arises to seal a road with varying texture, some type of texture treatment may be desirable to improve the performance of the reseal. The choice of texture treatment depends on the economics. The strategy is to prepare the surface for the next stone reseal with a texture treatment such as a sand seal or slurry. Recommendations in this regard are the following:

- sand seal with low traffic if cheaper than a fine slurry,
- fine slurry on higher traffic roads with low skid resistance risks,
- coarse slurry in cases of high skid resistance risks,
- apply an inverted double seal directly on the varying textured surface,
- a single 13,2 mm seal with modified binder.

With these factors in mind, Table 11-7 can be used for the selection of reseal type (Van Zyl, 1990).

Table 11-7: SIMPLIFIED DECISION TABLE FOR THE SELECTION OF RESEAL TYPES FOR LOW VOLUME ROADS

RUTTING	TEXTURE	CRACKING	RECOMMENDATIONS[a]
< 10 mm (acceptable)	Coarse or varying	Little	Sand seal, slurry, Otta seal, inverted double seal, asphalt
< 10 mm (acceptable)	Coarse or varying	Severe	Inverted double seal[b], double seal[b], texture treatment+single seal[b], asphalt
< 10 mm (acceptable)	Fine	Little	Single seal, sand seal, slurry, Otta seal, double seal, Cape Seal, asphalt
< 10 mm (acceptable)	Fine	Severe	Double seal[b], single seal[b], asphalt
> 10 mm (not acceptable)		Little	Inverted double seal, coarse slurry, asphalt
> 10 mm (not acceptable)		Severe[c]	Inverted double seal[b], coarse slurry+single seal[b], asphalt

Notes: a: refer to constraints in Chapter 5.7 as well
b: modified binder preferred for high crack severity
c: geotextile reinforcement may be desirable

11.3.1.3 Overlays

An overlay (or levelling course) is preferred to a reseal when the ruts are deep or when the riding quality of the road needs to be improved. However if the shape of the road is very poor, the cost of levelling courses and overlays can be more expensive than reworking the base. If the shape of the road is reasonable, the overlays can be applied to a rolling grade and not to pre-designed levels; this is more economical and the final result is quite satisfactory as far as riding quality is concerned. The choice of overlay type is given in Table 11-8 (Bergh, 1992).

Table 11-8: SIMPLIFIED DECISION TABLE FOR THE SELECTION OF OVERLAY TYPES FOR LOW VOLUME ROADS

REASON FOR OVERLAY	RECOMMENDATIONS[d]
Structural[a]	Possible levelling course, then asphalt overlay[c]
Reshape parabolic cross-section to straight line (reduce camber)[b]	Construct wedge overlay in outer one third of the lane, and asphalt overlay
Improve riding quality	Asphalt, quickset slurry
Fill ruts and improve surface drainage	Single modified binder seal, quickset slurry, asphalt
Improve riding quality and fill ruts	Asphalt, quickset slurry

Notes:
- a Unlikely for a low volume road
- b Can be expensive
- c Asphalt refers to 20-25mm continuously graded asphalt. For low volume roads this is less expensive than gap-graded asphalt or 40mm asphalt.
- d A single seal applied after 3-4 months of traffic will extend the life of the asphalt by 25-40%.

11.3.1.4 Holding action

A holding action may be considered when major rehabilitation cannot be executed immediately. This typically takes the form of a reseal, possibly with selected patching. Examples on South African roads exist where holding actions have extended the service life of pavements for more than eight years. The tendency in recent years is to make use of modified binders in this regard (Renshaw et al, 1991). For low volume roads, the option of a holding action should always be assessed. The assessment should consider the implications for different periods of the holding operation. However when reseals are not able to hold rapidly deteriorating roads together for more than two or three years, it is accepted that the only alternative left is major rehabilitation.

The economics of a holding action can be assessed by comparing the costs of the holding option with the cost of the rehabilitation, over a period of say 10 years. The results of one such analysis are shown in Figure 11-4. The rehabilitation cost was R100 000/km and it had a life of 10 years. The cost of a single seal with a modified binder was calculated over the same 10 year period assuming different lives for the holding action. For example with a life of 5 years, the holding action would have to be repeated once in the 10 year analysis period. The results have been expressed as present worth of costs in real terms with an 8% discount rate. It can be seen that if the time between holding actions is three years for this example, then the holding action is actually less expensive. The example here is a simple one, and the analysis should consider the life of the rehabilitation, the life of the holding action, the possibility of funding cuts which would not allow ongoing reseals as part of the holding action, and traffic delay costs.

Hint: repair and patching work is expensive. When 15-20% of the total road has failed through lack of maintenance or poor drainage and construction, it may be more economical to rework the base (and maybe subbase) than to embark on an expensive repair programme. Holding operations could be considered.

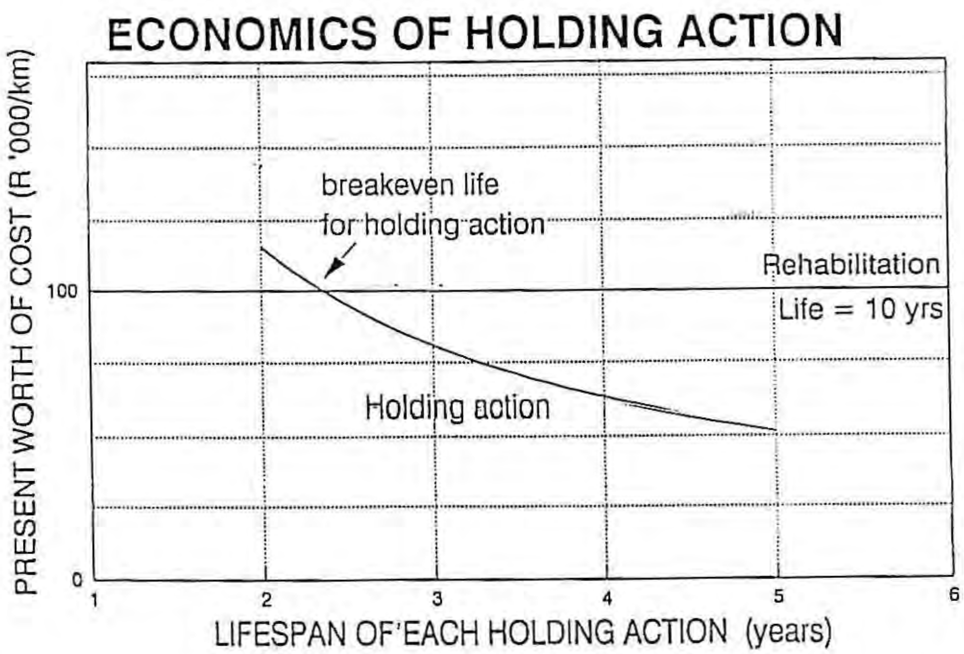


Figure 11-4: THE ECONOMICS OF HOLDING ACTIONS

11.3.2 Structural design of rehabilitation

The design of structural rehabilitation follows closely from the structural assessment in section 11.2.3. For the structural design of rehabilitation, there are numerous design methods, and each of these has its advantages and limitations. The methods can be classified into two types:

- empirical methods which are based on observations made and are limited to specific pavements, materials, traffic and climates. Examples include condition assessment, DCP, CBR, deflections, ruts, etc.
- theoretical methods based on calculating pavement response in terms of basic material properties of stress and strain. Examples include mechanistic methods.

Often several methods are used in a multiple analysis approach. The choice and use of various methods is discussed in more detail in TRH 12. Structural design of rehabilitation can be complex, and only the simplest methods suitable for smaller projects will be addressed here.

11.3.2.1 Design

For low volume roads, the DCP method of rehabilitation design has the advantages that it incorporates insitu bearing strength into the design and is easy-to-use. It is described here to illustrate the application of structural design of rehabilitation. There are two methods which can be used: a) the TPA method which is the simpler but provides designs for only a light traffic and a medium traffic road, and b) an extension to the catalogue method which enables the catalogue of section 5 to be applied and so offers a wider utilisation of existing materials. The two are very similar and vary only in STEP 5 below.

The main limit to the DCP method of rehabilitation, apart from restricting it to the less complex rehabilitation designs, is that it assumes that the pavement moisture regime will not wetten up from that at the time of testing. This assumption is not invalidated if the surfacing was cracked at the time of testing, provided that the cracking is not severe, deep, and widespread and the testing was done in the dry season. Testing of severely cracked roads, and testing of roads which are known to contain clayey materials should be confined to the wet season or the first month of the dry season. DCP CBRs can be related back to those for soaked and then back to those for the expected moisture regime (Emery 1992).

It should be noted that an adverse change to the drainage pattern can change the pavement moisture regime. This can occur where a cutting is deepened or altered, or the alignment otherwise changed.

The rehabilitation design is done by comparing the actual DCP results with the required layer-strength diagram. This will highlight the weak layers. The steps in the method are somewhat similar to the steps taken when assessing the residual life of the pavement (section 11.2.3):

<u>STEP 1</u> <u>DO AN INVESTIGATION ALONG THE ROAD</u>

An investigation is performed along the road, as generally described in sections 11.2.1 to 11.2.3, although the step of estimating residual life can be omitted. It is suggested that it is useful to take at least 2 samples per kilometre to check laboratory soaked CBR, Atterbergs and insitu moisture content of each layer. Test insitu density (optional). However this sampling may be omitted by some authorities.

<u>STEP 2</u> <u>DIVIDE ROAD INTO UNIFORM SECTIONS FOR REHABILITATION</u>

The results of the investigation, including the DCP testing and visual assessment, enable the length of road to be divided in relatively uniform sections for the purposes of rehabilitation. The minimum length of section should be 100 metres, and desirably 1000 metres. On long lengths of road with uniform conditions, it can be 10 000 metres. Note that construction of sections shorter than 500 metres is awkward. It may be that a low DCP result occurs in a spot which was identified in the visual survey as an isolated problem area; these are typical of an isolated drainage problem and consideration should be given to repair of these individually rather than taking them as representative of the section.

STEP 3 CALCULATE THE REPRESENTATIVE DCP CURVE FOR EACH SECTION

The representative DCP curve for each section is calculated as the lower 80 percentile DCP from the actual field data, and plotted on a form like Figure 11-5. The easiest is if the field DCP results show a uniform layer structure, with each layer being say 150mm thick. It is straightforward to find the actual DCP-CBR for each layer for each DCP test from Figure 5-7. The representative DCP-CBR for each layer is then found statistically to provide a safety margin against the variability of material within the section. A normal distribution of data is assumed and the Student's T distribution at the 80% level is used:

representative DCP-CBR = mean DCP-CBR - .9 * (standard deviation DCP-CBR . . . (11.5)

Example The DCP results for the top layer in a section were as follows:
DCP-CBR: 125, 143, 120, 100, 145, 115, 140, 135
Mean (average) = 127,9 Standard deviation = 15,7
Representative DCP-CBR = mean DCP-CBR - .9 * (standard deviation DCP-CBR) using (11.5)
= 127,9 - .9 * 15,7 = 114

Note that equation 11.5 uses a one-tailed T-distribution for 8 samples and is reasonably robust for sample sizes from 5 to 30.

Some authorities prefer to do their calculations in terms of "mm per blow" or "blows per layer", and then convert to DCP-CBR at the end.

It gets more difficult when the layers are not uniform i.e. results show the layer thickness on four successive holes as: 140mm, 170mm, 150mm, 130mm. Then a "best fit" must be assessed for the pavement structure. If the "best fit" is not easy to find, then the recommended approach is to divide the pavement into 50mm thick layers at each DCP testhole, and calculate the representative DCP-CBR for each 50mm thick layer for the section using equation 11.5. This is then drawn on a layer strength diagram (such as Figure 11-5) and the variation of DCP-CBR with depth can be seen.

Other authorities divide the pavement into 150mm thick layers, and calculate the representative DCP-CBR for each 150mm layer, but that gives too generalised a result and does not fully pick up thin weak layers, or alternatively overemphasises thin strong layers.

STEP 4 ESTIMATE DESIGN TRAFFIC

The design traffic over the structural period is estimated in accordance with the procedures in Section 5.

For the catalogue approach to rehabilitation design, the design traffic classified in accordance with Table

5-11. For the TPA design curve approach, the design traffic is classified as follows:

light	< 0,2 x 10^6 E80s per lane
medium	0,2 - 0,8 x 10^6 E80s per lane

STEP 5 COMPARE THE REPRESENTATIVE DCP WITH THE DESIGN DCP

5.1 Catalogue design approach

The representative DCP-CBR curve for a section is compared to the catalogue design (from section 5) to assess if the pavement is adequate for the design traffic. To do this, the representative DCP-CBR values must be converted to layer thicknesses and material types, i.e. 150mm G5.

The materials in the catalogue are classified by their soaked bearing strength (as is done in TRH 14), and the actual pavement materials need to be classified in terms of their soaked CBR to be related to the catalogue. Since the CBR of a material in the field at different moisture contents and densities can vary significantly from its soaked CBR, and in general the drier it is, the higher the field CBR (Emery, 1992), it means that the DCP-CBR results need to be adjusted from field to soaked condition, before one can state with confidence that a particular layer is in fact equivalent to a particular material type.

The preferable method of classifying the existing gravel road materials is to take many soil samples and test them in the laboratory. At the same time, field density tests of all layers should be performed to ensure that their compaction is adequate. This can involve considerable testing however, and a simpler, although less accurate, method is to use the DCP to do most of the testing in conjunction with a limited number of laboratory soaked CBR tests. Then the material type can be estimated from the relationships between field DCP-CBR and soaked CBR (Table 11-9 for roads which are presently gravel), and cross-checked with the laboratory CBRs. Some authorities even omit the step of performing laboratory soaked CBRs.

The compaction can also be checked, because if the field DCP-CBRs estimated from the laboratory soaked CBR results are less than those actually found in the field, it is indicative that the existing gravel road has been well compacted (by traffic), and is suitable for incorporation in the design. If however the actual field DCP-CBRs are less than estimated from the laboratory, it indicates a lack of compaction and the existing gravel layer should be ripped and recompacted. Alternatively compaction can be checked if sufficient field density tests have been performed, when they can be compared to specified Mod AASHTO densities.

Table 11-9 APPROXIMATE RELATIONSHIP BETWEEN SOAKED CBR AND FIELD DCP-CBR FOR A GRAVEL ROAD

Material classification	Soaked CBR	APPROXIMATE FIELD DCP-CBR : GRAVEL ROAD					
		Subgrade		Wearing course			
		wet climate (Im≥0)	dry climate (Im<0)	very dry state	dry state	moderate state	damp state
G4	80			318	228	164	117
G5	45			244	175	126	90
G6	25	59	65	186	134	96	69
G7	15	45	50	147	106	76	54
G8	10	38	43				
G9	7	33	37				
G10	3	20	24				

Notes 1 The inter-relationship between soaked CBR and field DCP-CBR is very approximate due to the variability of moisture contents, materials, test methods, and densities. It assumes that the density relates approximately to the field density expected for that layer. More research is needed to give confidence to this relationship.

2 The moisture contents that this table are based on are estimated moisture contents, based on various field studies and experience; they can vary in practice from the values assumed here. For the wearing course they are (expressed as the ratio of field moisture content to Mod AASHTO optimum moisture content): very dry state = 0,25; dry = 0,5; moderate = 0,75; damp = 1,0.

3 This table has been developed from Table 22 and equation 36 of Emery (1992)

The actual pavement structure (now expressed in terms of layer thickness and materials classification) is compared to the catalogue design for the design traffic (which has been found above). This will indicate what new layers, if any, are required, and what layers need to be reworked or stabilised to improve their classification.

If additional layers are required, materials which meet the requirements have to be located (desirably close by). In the case of suitable materials not being locally available, the decision to modify/stabilise local materials or to import materials is then made on economic grounds.

EXAMPLE:

A road in a moderate region currently has gravel wearing course and is expected to carry 25 000 E80s (traffic class E0-2) over the next 20 years. The decision to upgrade the road to a surfaced facility has been taken and a pavement design for this operation is required.

The DCP results showed that a reasonably uniform wearing course of 150mm thickness existed.

The DCP-CBRs for the first 50mm were as follows:
DCP-CBR: 91, 79, 74, 85, 69, 81, 64, 80
Mean (average) = 77,9
Standard deviation = 8,7
*Representative DCP-CBR: = mean DCP-CBR - .9 * (standard deviation DCP-CBR)*
*= 77,9 - .9 * (8,7)*
= 70

By a similar process, the representative DCP-CBR for 50-100mm and for 100-150mm were found to be approximately the same. The DCPs showed that the materials underlying the wearing course were strong.

It was found that the pavement moisture content was close to the Mod AASHTO optimum moisture content and therefore when using Table 11-9, it was assumed that the wearing course was in a 'damp' state. Table 11-9 indicates that the material in the wearing course can be classified as a G6 material.

Testing in the laboratory indicated that the material soaked CBR was 27 which is very close to the value one would expect for an adequately compacted G6 material (25 see table 5-8, soaked CBR column). Had the laboratory CBR been much higher than this value of 25, it would have indicated that although its strength was sufficient to classify it as G6, its compaction was not. In this case either the wearing course would have had to have been recompacted or the material reclassified (as G7, say).

For the climate (moderate) and traffic (E0-2) three possible structures are indicated by the catalogue:

Granular base option: Seal + 150 mm G5 base + 150 mm G7 + 150 mm G9 + G10
Stabilised base option: Seal + 100 mm C4 base + 150 mm G7 + G10
Low maintenance option: 25 mm asphalt + 150 mm G6 base + 150 mm G7 + G10

Granular base option

Since the existing material is a G6 only the base requirement needs to be met. The G6 material may be sufficiently thick to compensate for the fact that it is not as strong as required (G5), but this would have to be established by analysis. If it is inadequate, an analytical design would have to carried out, since the importing of an entire G5 basecourse would not be cost effective, particularly when the requirements for the other two options are considered.

Stabilised base option

The existing material is better than the layer required to support the stabilised base. There are then two options open to the designer; if the layer underlying the G6 in the existing road can be classified as G7 and is thicker than 150 mm, then the G6 material could be stabilised and compacted in place and then sealed. If the underlying layer is found to be less than a G7 then a stabilised base should be placed over the existing G6 material.

Low maintenance option

This option would normally be selected when the maintenance capability of the relevant road authority was such that regular maintenance was not possible. This particular design requires only the addition of a 25 mm thick asphalt layer to the existing gravel road.

5.2 TPA design curve approach

In the TPA design curve approach, the representative design DCP curve for each section is plotted on a copy of Figure 11-5, and compared to the DCP curve for the design traffic class. If there is sufficient bearing strength in the existing pavement, then no structural upgrading is required (in this case, the representative DCP line will always be to the left of the design DCP line). Note that rehabilitation may typically be then be limited to an overlay to restore shape. This is a simpler approach than above, but is more limited in its options.

If however the bearing strength at any depth is lower than the design DCP line, then structural rehabilitation is required. The most common rehabilitation measures are:

- add a new layer,
- remix and stabilise a layer (which typically improves the bearing strength to stronger than 2 mm/blow).

The effect of a new layer or two can be easily estimated by overlaying Figure 11-5 with its clear overlay Figure 11-5A; slide the overlay up by each new layer to see the effect of the extra cover obtained from the new layer. The overlay has been drawn with typical layers of 150mm drawn on to assist in its use.

An example of the design process is shown in Figure 11-6. The design DCP curve has been calculated in accordance with steps 1 to 5, and has been plotted onto a copy of Figure 11-5. Assuming that the design traffic has classified as "light" according to STEP 4, then it can be seen from Figure 11-6 that the top layer from 0 to 100mm depth is stronger than required by the light traffic line, and so this layer is adequate. However from 100 to 140mm, and again from 220 to 395mm, the actual pavement is weaker than required, and some structural rehabilitation is required.

The most common rehabilitation for these low volume roads is to add another layer, and Figure 11-7 shows this. In Figure 11-7, the clear plastic overlay (Figure 11-5A) has been laid on top of Figure 11-6, and then moved up the page equivalent to a depth of 150mm. This replicates the effect of overlaying with a 150mm layer of basecourse quality material. It can now be seen that the only weakness is between 220mm and 395mm. The designer now has the choice of:

- adding a second layer of 150mm, or
- boxing out the existing pavement to 400mm, stabilising, and recompacting, which should improve the bearing strength of the existing material without the need to add any new layers.

11.3.2.2 Materials

In rehabilitation, the standards for materials are given in section 4 of this manual. They obviously apply to new materials imported for new layers, but their application to existing materials in the road is not however straightforward. Broadly speaking the main materials requirement for the continued use of the existing materials as significant layers in the new structure is that they have adequate bearing strength. In some cases, attention may need to be given to durability if the materials are expected to degrade in the road. DCP rehabilitation design is based on insitu bearing strength and if the existing pavement materials provide this, then they should be generally accepted even if they fail to meet all the standards for new materials.

The acceptability of marginal insitu materials depends mainly on climate. In dry or moderate climates, relaxation of material standards relating to moisture sensitivity (such as Plasticity Index) can be tolerated. In wet climates, the relaxation of material standards is less acceptable. The issue is so multifarious that no simple standard can be set, and the final decision will be made on the basis of experience with the material types, history of the pavement, and cost.

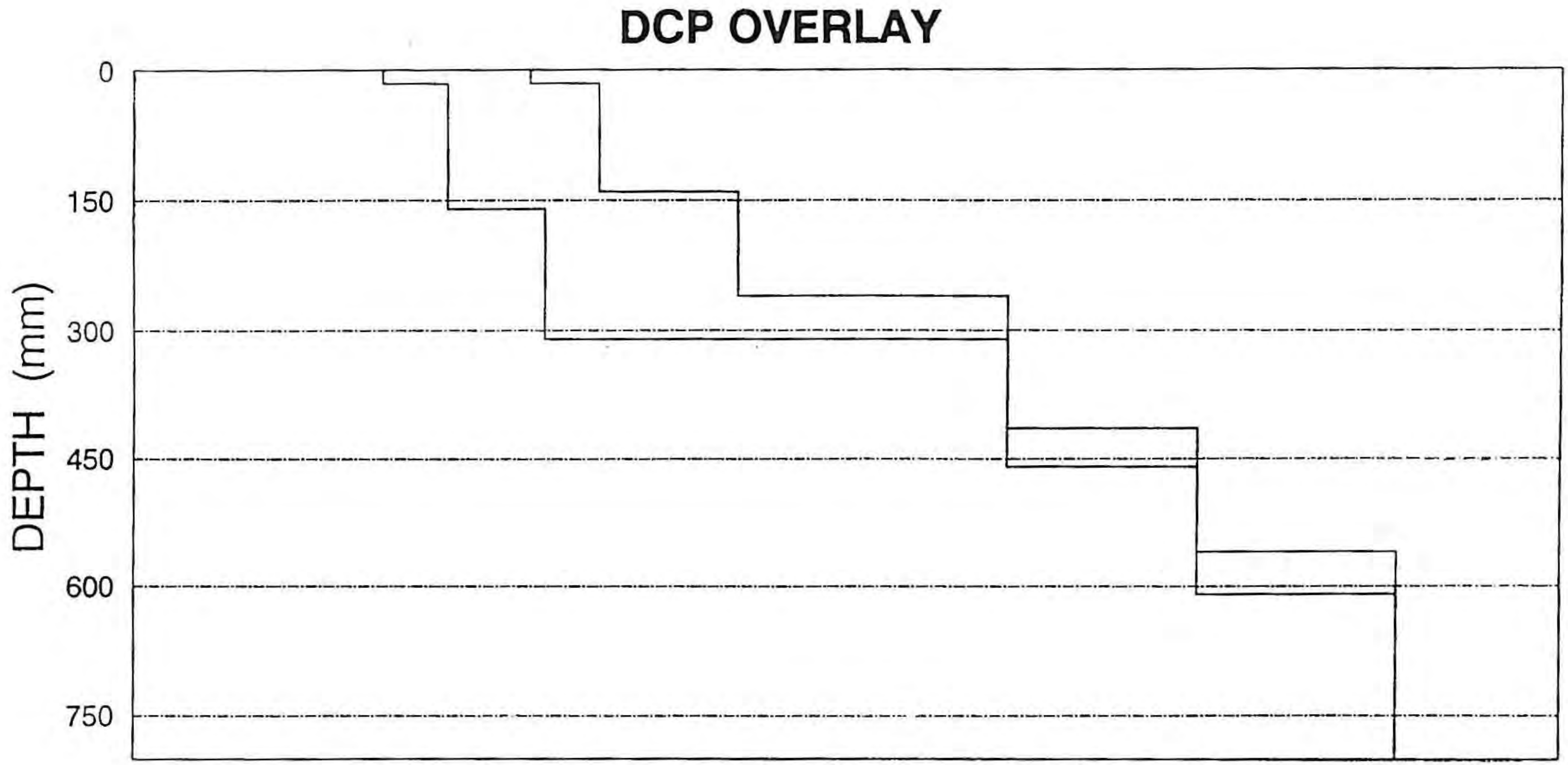
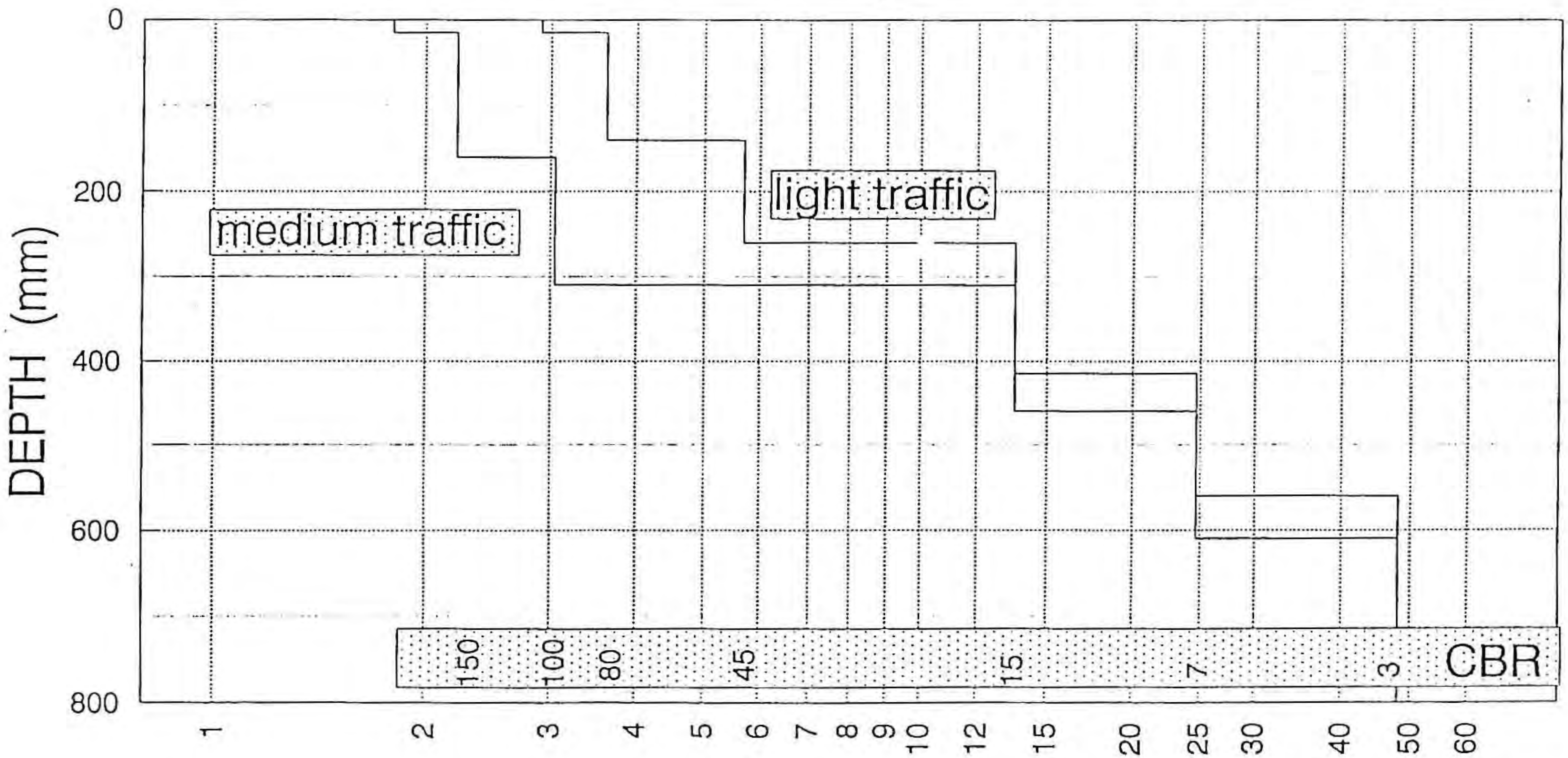


Figure 11-5A: DCP OVERLAY

DCP DESIGN CURVES



After Kleyn

Figure 11-5: DCP DESIGN CURVES

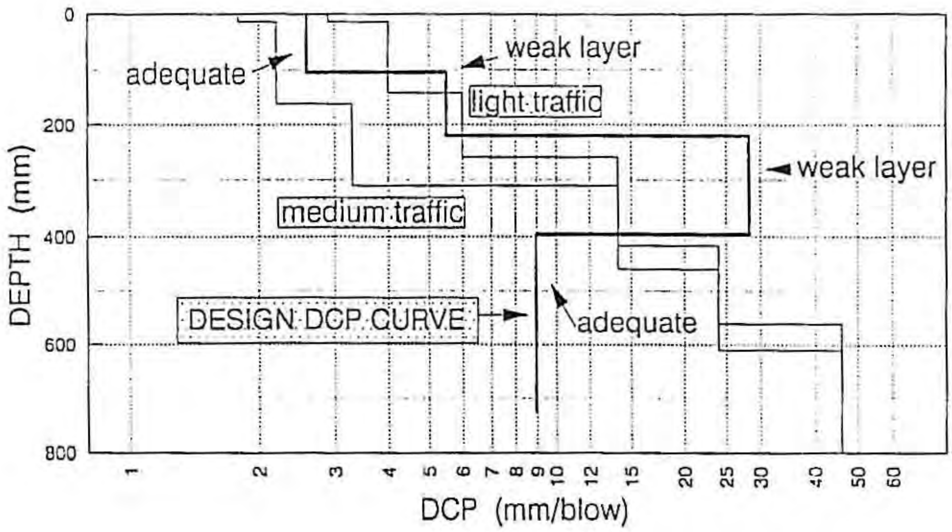


Figure 11-6: MEASURED DATA AND DESIGN CURVES

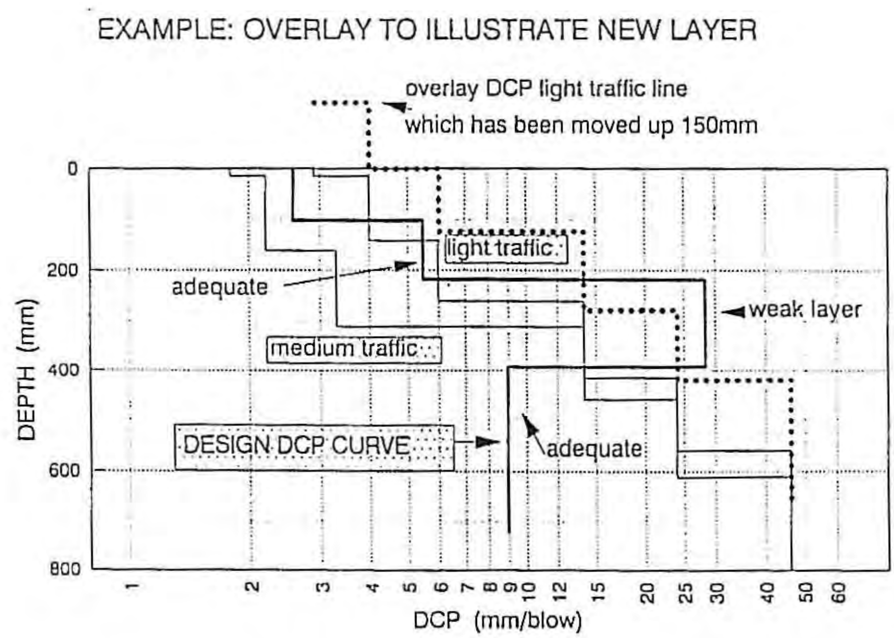


Figure 11-7: ILLUSTRATION OF THE EFFECT OF A NEW LAYER

11.3.3 Geometric rehabilitation

Geometric rehabilitation is aimed at improving the alignment, width or other geometric parameters to current standards. For low volume roads, as a general rule, geometric rehabilitation is not justified simply because the road is undergoing pavement rehabilitation. Geometric rehabilitation is justified because:

(i) there is a high rate of accidents on the road; or
(ii) there has been a significant change in the traffic since the road was built or last rehabilitated.

Geometric rehabilitation of low volume roads offers the engineer the opportunity for the greatest initiative in reducing costs. The geometric standards for new roads are given in chapter 3 as guidance, but in rehabilitation, it should be borne in mind that earthworks can increase the cost quite substantially when applying strict geometric standards.

Low cost improvements during rehabilitation can be had by directly addressing particular safety issues rather than improving the road geometrics generally. Examples of these include:

- increased provision of signs,
- removal of obstacles close to the road,
- improvement of shoulders, and
- accident black spot improvement.

11.4 References

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12 ENVIRONMENTAL CONSIDERATIONS

12.1 Introduction

One of the primary factors affecting the economic viability of any country is the presence of a reliable and adequate transport infrastructure, the main components being roads and railways. Without these routes, the population could not become economically active, agricultural and mining produce cannot reach their market places, health services become ineffective and energy supply becomes costly and unreliable. The need for environmental considerations relevant to roads must be tempered with the need to have a road network at an affordable price.

The provision of roads and their associated structures and material borrow areas often leaves significant scars on the environment. These visual impacts are particularly distinct due to their lineal nature. In addition, the traffic using the roads typically causes noise, vibration and air pollution. However, the negative impacts caused by certain roadways may often be countered by positive impacts. For instance, the construction of an alternative route or town bypass may have a negative impact on an unspoilt landscape and the local economy (reduced trade from through traffic), but may have a positive impact on the residents of the town by decreasing the traffic and hence noise and air pollution and improving safety conditions.

Environmental impact assessments are a relatively new consideration in road and transport related activities, and although a number of studies on the effects of traffic in urban areas have been undertaken (Green and Faure, 1991), very little work has been conducted locally on the impact caused by roads and the associated traffic on the rural physical environment. It is therefore advisable that, before any new roads are constructed, or existing roads are rehabilitated or upgraded, the relevant authorities determine the impact of the roadway or the improvements on both the bio-physical and socio-economic environments. Environmental impact assessments are not yet a legal requirement for all new road projects in South Africa, however, in some overseas countries impact assessments are a prerequisite for road construction and most funding agencies will not consider the granting of loans until a thorough investigation has been concluded.

This chapter discusses some of the environmental problems associated with road construction and makes some suggestions as to how the roads industry can approach the next century with a greater environmental awareness. It applies to both low and high volume roads.

12.2 The Status Quo

The location of roads, selection of construction materials and design of the pavements and associated structures are presently based, to a large extent, on traditional engineering principles embodied in a number of guides and manuals prepared by road authorities and research organisations. However, very few of these manuals include aspects concerning environmental conservation practices. Some of the individual road authorities typically have their own in-house guides to environmental assessment (Walker, 1987).

Although guides for environmental planning have been published in South Africa (EPPIC, 1980; Faure, 1990), no guides expressly for the Environmental Impact Assessment (EIA) of proposed road alignments or road rehabilitation projects are readily available to authorities, consultants or contractors at present. The guidelines on Integrated Environmental Management (IEM) published by the Council for the Environment (1989) and Department of Environment Affairs (1992) provide a tool for the engineer dealing with a road project, but are not in common use yet.

12.3 Integrated Environmental Management (IEM)

The Integrated Environmental Management (IEM) (Council for the Environment, 1989) process is designed to ensure that the environmental consequences of development proposals are understood and adequately considered at all stages of the development process. It encompasses a broad range of methodologies including terrain evaluation, ecological studies, cost benefit analysis, social impact assessment, risk assessment, technology assessment and futures research.

The purpose of IEM is to resolve or mitigate any negative impacts and to enhance positive aspects of development proposals, by identifying the most acceptable proposal or alternative for meeting the objectives of a proponent, without imposing undue environmental costs on other parties. The process is intended to guide, rather than impede, the development process by providing a positive interactive approach to gathering and analysing useful data. Irrelevant studies can thus be avoided and data collection and analysis can be made more efficient, thus saving developers and resource managers time, effort and money.

The principles underpinning IEM can be thus summarised as follows (DEA, 1992):

- informed decision making;
- accountability for information on which decisions are taken;

- accountability for decisions taken;
- a broad meaning given to the term *environment*;
- an open, participatory approach in the planning of proposals;
- consultation with interested and affected parties;
- due consideration of alternative options;
- an attempt to mitigate negative impacts and enhance positive aspects of proposals;
- an attempt to ensure that the 'social costs' of development proposals be outweighed by the 'social benefits';
- democratic regard for individual rights and obligations;
- compliance with these principles during all stages of the planning, implementation and decommissioning of proposals, and
- the opportunity for public and specialist input in the decision-making process.

The IEM procedure is summarised in Figure 12-1

12.3.1 IEM and Road Construction

The IEM procedure can be effectively applied to road construction, rehabilitation and maintenance projects. The procedure should be implemented in the planning stage of any project, when suitable alternatives can still be selected. The IEM procedure can be typically incorporated into a project in the following way.

- During the planning stage of the project, no matter how small, the engineer should list any possible impacts that may arise during the development, and identify any individuals, groups or societies that may be affected by the development or have an interest in the development. In recent times, interested and affected parties have placed considerable pressure on developers, be it for a single impact such as noise, and it is therefore advisable to identify these individuals or groups and their respective issues before construction begins so as to avoid possible legal action, unnecessary delays, work stoppages and even re-design. Issues can, for example, include visual impacts, noise and air pollution, severance of farms or whole communities and destruction of natural vegetation, natural history or cultural sites.

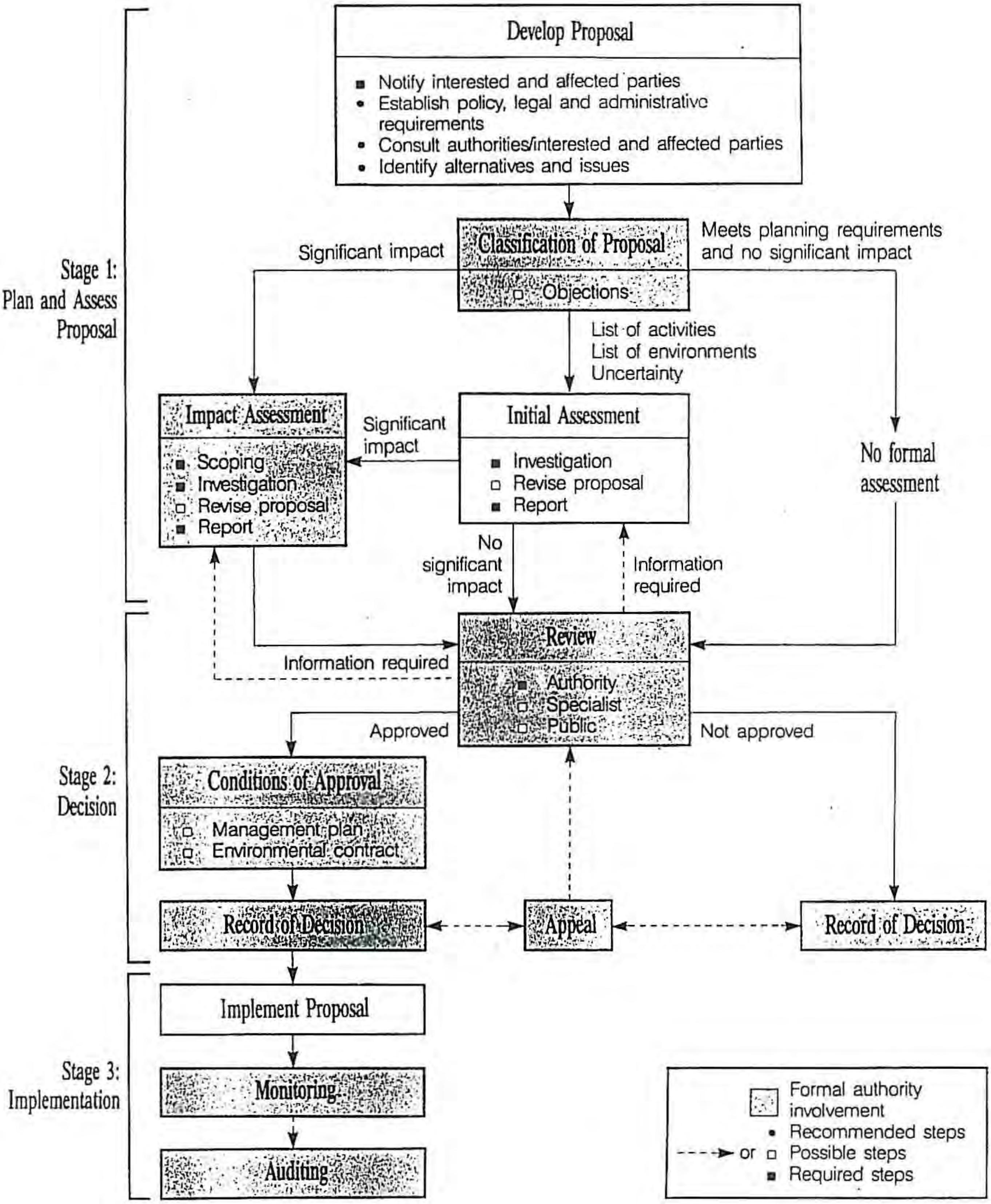

Figure 12-1: THE IEM PROCEDURE

- When the issues have been identified, the project can be classified as having no negative impacts (ie no significant issues were identified and no persons or parties are affected), uncertain negative impacts or significant negative impacts.

- If no impacts are identified (which is unlikely unless the project is very small), the project can proceed without an initial impact assessment. However, the engineer should be constantly aware of impacts that might arise during the course of the development, for example, clearing of vegetation, noise from construction equipment and placement of temporary accommodation for construction crews.

- If the impacts are uncertain, a preliminary or initial assessment should be undertaken by the engineer using (but not confined to) a checklist or matrix (an example is provided in Appendix 12-1). A decision on whether the proposed project can proceed or whether a full impact assessment is necessary can then be made.

- If significant impacts are evident from the beginning of the project or were identified during the initial assessment (for example, removal of large tracts of indigenous forest or the crossing of a wetland) a full impact assessment will have to be conducted. This would involve meeting with the interested and affected parties and identifying the issues to be investigated. Where necessary, the engineer should appoint and co-ordinate specialists with appropriate experience to assess the issues and alternatives, and recommend mitigatory actions. An environmental impact report can then be compiled and reviewed by the relative authorities and lead interested and affected parties. Alternatives and mitigatory action for the contractor will be included and on approval of the report, the development could begin. Depending on the development, environmental monitoring plans may have been drafted by the specialists during the assessment, in which case auditing of the work according to the recommendations of the monitoring plan would take place.

- If the project involved the necessity of an extensive environmental impact assessment, the engineer should seriously consider the appointment of a consultant with the relevant experience to conduct the study. The interested and affected parties may also demand that the assessment be undertaken by an independent and impartial company.

12.4 Environmental Impacts of Roads

Various aspects concerning roads and associated traffic result in impacts on the environment. In many instances, the designers and contractors may not be aware of the potential impacts that may result during and after the development. It is therefore important that the Integrated Environmental Management procedure be followed by the developer and consultant, and should the potential for significant impacts be identified, then an environmental impact assessment can be undertaken. At this point, key issues requiring evaluation will be identified.

A number of impacts associated with the roads and transport industry have been identified and each of these is discussed in turn below (Paige-Green et al, 1991). The list is not exhaustive but is considered to cover most of the major socio-economic and bio-physical considerations, and is included to familiarise the user with the impacts that they may encounter.

12.4.1 Route Location

The optimum route alignment in terms of subgrade materials and geometric constraints is often in the ridge component of the topography. In these areas, the residual soils are thin, the material in which cuttings are excavated tends to be more stable and drainage problems can be minimised. In contrast to these engineering advantages are the environmental disadvantages. The earthworks in the steeper terrain become deeper and more prominent, the roads have a greater visual impact, the threat of erosion and land instability increase and the steeper terrain makes revegetation more difficult.

The narrow zones on either side of ridge crests are often important ecological areas as a species transition related to the aspect may occur in these regions.

Conversely the construction of roads adjacent to rivers or near valley bottoms may result in the necessity to canalise the rivers in order to avoid undercutting or erosion of constructed fills. This could have serious consequences in terms of regional run-off patterns as well as aesthetic implications and harmful effects on the river ecology.

Relatively small changes in the vertical position of a road with respect to the contours and tree lines can make a difference between visually intrusive and visually acceptable route location (Moore, 1990). The use of trees and even earth mounding for screening of roads in areas with high quality landscapes has been successful in other countries (Moore, 1990).

The expropriation of high quality agricultural land can be considered as an environmental impact in socio-economic terms. Roads should be located on low quality agricultural or poor pasture land wherever possible. Economic impacts, both positive and negative are often dependant on the route location, for example, the by-passing of towns often has negative economic impacts on that community and increases in traffic after road improvements may lower the value of adjacent property.

Table 12-1: SUMMARY OF THE IMPACTS OF ROUTE LOCATION

IMPACT	MITIGATION
Visual	Select alternative with least impact - use contours and tree lines
Erosion	Minimise earthworks
Land instability	Minimise earthworks
Vegetation	Implement recommendations from terrestrial ecologist
Regional runoff	Implement recommendations from hydrologist
River ecology	Implement recommendations from freshwater ecologist
Expropriation	Select route over low yield agricultural areas
Economic	Include environmental factors in the economic analysis

12.4.2 Excavation and Dumping

The construction of roads often results in the necessity for extensive earthworks operations. This typically occurs during the construction of cuttings and embankments, the removal of unsuitable or deleterious materials from beneath the proposed road structure and the construction of bridges, culverts and other necessary drainage structures. The construction of tunnels would naturally be included under this section but as these are considered to be major structures, full environmental impact assessments tend to be carried out as a matter of course and they are therefore not considered in this document.

The excavation of cuttings is not considered to be a major environmental issue provided the cut is designed and shaped such that it merges with the surrounding natural terrain and is adequately revegetated or otherwise treated. Environmental predicaments tend to arise when geometric and topographical constraints dictate that the batters are very steep. The removal of material from the foot of a slope disturbs the natural stability which can be considered to be at the lower limit of equilibrium. Any change in moisture, loading or other environmental aspect may result in failure and the need for extensive rehabilitation requirements. This is generally both environmentally and economically unacceptable. The design of cuts therefore needs to be carried out in an environmentally aesthetic manner (Anon, 1984). Simple guidelines for the selection of batters for slopes in various materials are provided in TRH18 (CSRA, 1987).

The material removed from cuttings should, wherever possible, be used in the construction of adjacent embankments. However, this is not always feasible and disposal of the excess is therefore necessary. Topsoil should be separated and used for rehabilitation of disturbed areas. The placement of the discard material should be carefully considered in order to reduce the risk of pollution to water courses, while revegetation with suitable indigenous species should be carried out immediately so as to minimise erosion, restore the aesthetics and stabilise the material. It should be ensured that sites of conservation interest are not used for the disposal purposes.

Table 12-2: SUMMARY OF THE IMPACTS OF EXCAVATION AND DUMPING

IMPACT	MITIGATION
Tunnels	Conduct separate EIA
Visual impact	Design according to local topography - rehabilitate immediately
Land instability	Implement recommendations of engineering geologist
Fill material	Use material from cut if suitable
Material disposal	Select site carefully during EIA and revegetate immediately after dumping

12.4.3 Material Acquisition

The importation of appropriate materials for the road formation and structural layers is an essential aspect of road construction, although part of the requirement is supplied from cut/fill balancing. Suitable unpaved wearing course materials in arid areas are often restricted to the top 300 or 400 mm of the soil profile. This typically results in extensive stripping and removal of the soil (and vegetation) cover. Revegetation in these areas is very slow and unsightly scars remain for extended periods.

Borrow areas excavated for the construction of paved roads can usually be closed and rehabilitated immediately after construction. To this extent, paved roads have an advantage over unpaved roads, where repeated reworking of the borrow pit does not allow the area to be economically or effectively rehabilitated in the short term. When considering sensitive environmental areas, it may be desirable to pave certain roads, which in strictly economic terms do not warrant paving, so as to allow immediate rehabilitation of the borrow pits.

The closure and rehabilitation of borrow areas must include all access roads to the excavation and material stockpile areas. The compacted material should be ripped and shaped in order to avoid erosion and to facilitate the establishment of vegetation as quickly as possible.

The continued use or recycling of existing construction materials, perhaps in a lower layer in a pavement, is a sound environmental practice. This not only avoids having to acquire additional material but also averts having to dispose of waste material.

Table 12-3: SUMMARY OF THE IMPACTS OF MATERIAL ACQUISITION

IMPACT	MITIGATION
Borrow pit location	Include in EIA, obtain relative permits and land owners permission and submit rehabilitation plan
Visual impact	Rehabilitate according to plan
Vegetation	Implement recommendations of terrestrial ecologist
Access roads	Rehabilitate according to plan
Other	Ensure areas of natural or cultural importance are avoided

The excavation of borrow pits is now subject to legislation. The Minerals Act (Act No 50 of 1990) stipulates the following as to rehabilitation of the surface of the land (Section 38):

> "The rehabilitation of the surface of the land concerned in any prospecting or mining shall be carried out by the holder of the prospecting permit or mining authorization concerned-
>
> (a) in accordance with the rehabilitation programme approved in the terms of section 39, if any;
> (b) as an integral part of the prospecting or mining operations concerned;
> (c) simultaneously with such operations, unless determined otherwise in writing by the regional director; and
> (d) to the satisfaction of the regional director concerned."

The following sections stipulate the legal requirements of rehabilitation:

- Section 39: Layout plan and rehabilitation.
- Section 40: Removal of buildings, structures and objects.
- Section 41: Restrictions in relation to use of surface land.
- Section 42: Acquisition of purchase of certain land and payment of compensation under certain circumstances.

Details on rehabilitation are provided in Appendix 12-2.

12.4.4 Erosion

Soil erosion is a serious problem affecting much of South Africa (Huntley et al, 1989). Most of the earthworks associated with road construction and maintenance, including cuts, fills, waste dumps, borrow areas and even the construction site itself may all contribute to the soil erosion problem if preventative measures are not taken. The erosion potential of a soil is highly dependent on the properties of that soil, the grade and length of the slope and the nature of the slope surface (grassed, barren, stony, etc). It should be noted that prevention is always easier than cure with respect to soil erosion.

The covering of large areas with road pavements, drainage structures or erosion protection devices can exacerbate the erosion potential by reducing the potential infiltration of an area and consequently increasing the run-off volumes, concentrations and velocities.

Soil erosion should be considered during the planning stages of the project. The relevant structures and sites can then be located to minimise erosion and its consequences. In veld areas where the soil has been exposed or where erosion channels are visible, testing for the erosion potential and dispersivity of the material should be carried out (CSRA, 1987).

Table 12-4: SUMMARY OF THE IMPACTS OF EROSION

IMPACT	MITIGATION
Erosion potential	Implement recommendations of engineering geologist
Regional runoff	Implement recommendations of hydrologist

12.4.5 Noise

Two categories of noise originate from roads and associated traffic, namely that emitted during actual construction and secondly that transmitted by traffic once construction has been completed.

Construction procedures usually involve large earth moving machinery and the possible use of explosives. The necessity to reduce construction noise in sensitive areas should be highlighted during the initial site assessment in order to allow the contractor to modify standard construction procedures or provide for special equipment in his tender.

Traffic noise has been researched from various perspectives, the most important being the effect on human activities (Green and Faure, 1991; Liebenberg, 1991). The long term effect on fauna, due to the diversity, is difficult to quantify. Solutions to the problem in urban areas usually involve the implementation of enforced speed limits, the diversion of traffic or the erection of sound proofing (which is often aesthetically unacceptable). The type of road surfacing can also contribute significantly to reducing or amplifying road noise, however, in southern Africa, only limited research and implementation has been carried out in this regard (Visser and Walker, 1981; Von Meier and Heerkens, 1986; Emery et al, 1991).

Table 12-5: SUMMARY OF THE IMPACTS OF NOISE

IMPACT	MITIGATION
Construction noise	Include details in tender documents, educate contractors and inform local residents
Traffic noise	Enforce speed limits and select appropriate surfacing - divert heavy traffic if necessary
Traffic noise - fauna	Implement recommendations of terrestrial ecologist

12.4.6 Vibration

Construction procedures usually induce heavy ground vibration through blasting, ripping, rock breaking or even vibratory or impact rolling. These may be relatively unobtrusive to humans but the frequencies and wavelengths may have significant effects on building foundations (especially historical buildings) and on local fauna. The successful breeding of, for example, crocodiles in particular may be seriously hampered by excessive ground vibration.

Table 12-6: SUMMARY OF THE IMPACTS OF VIBRATION

IMPACT	MITIGATION
Construction vibration	Include details in tender documents, educate contractors and inform local residents
Traffic vibration	Implement recommendations of terrestrial ecologist

12.4.7 Pollution

- Air

Air pollution during and after road construction is primarily produced through exhaust emissions, dust generation and release of volatiles from construction materials, including bitumens, tars, soil stabilisers and dust palliatives. Exhaust emissions from heavy vehicles tend to increase on steep grades and at intersections.

The engineer is not able to control exhaust emissions, but it should be borne in mind that these would have been generated in the area irrespective of the presence of a new road albeit on an adjacent or alternative route.

Dust generation on busy unpaved roads is a major problem from the road safety as well as the environmental point of view. In hilly areas where winter temperature inversions are common the air quality can be detrimentally affected by dusty roads. The generation of dust is very material dependent and although dust can be controlled to a certain extent by good material selection practices (CSRA, 1990), it is often more economical to pave the road or provide some form of environmentally acceptable dust palliative.

- Water

The influence of roads on water pollution is primarily through the erosion of soil materials into water courses, the leaching of natural soluble components (salts may occur naturally in the construction materials or compaction water) or additives or stabilisers from the pavement materials.

- Aesthetic

Aesthetic pollution is typically brought about as the result of improper disposal of excess materials, waste rubble and containers (eg bitumen drums, cement pockets, etc) and litter. Old roads and bridges, if not adequately rehabilitated or demolished result in aesthetic pollution and even the construction camps with their associated litter, although temporary, may result in significant visual impacts.

Table 12-7: SUMMARY OF THE IMPACTS OF POLLUTION

IMPACT	MITIGATION
Volatiles and leaching	Include details in tender documents, educate contractors and inform local residents
Exhaust emissions	Inform local residents - select alternative route if necessary. Divert heavy traffic from steep grades in urban areas
Dust	Apply dust palliative or appropriate surfacing
Erosion	As per recommendations on erosion
Construction waste	Educate contractors, include construction site in EIA
Old roads and bridges	Rip and revegetate old roads, demolish bridges and remove all waste

12.4.8 Severance

Although severance has been covered in a number of the previous sections, it is an aspect which deserves separate discussion. The linear nature of roads may result in portions of communities, agricultural areas, wildlife sanctuaries, hydrological features or recreation areas becoming separated from the rest of the area. This should be avoided as far as possible. Severance of small farms may significantly affect their financial viability.

The severance of hydrological features without due caution can have significant consequences on sensitive areas such as wetlands. An incorrectly placed road fill could detrimentally effect the drainage of a wetland by dividing the area into two distinctly different zones; one which is relatively dry and one where ponding of the water occurs, thus resulting in the destruction of the extremely sensitive wetland ecology.

Table 12-8: SUMMARY OF THE IMPACTS OF SEVERANCE

IMPACT	MITIGATION
General severance	Select route so as to minimise severance. Consult local residents.
Wildlife sanctuaries	Consult terrestrial ecologist
Wetlands	Consult hydrologist

12.4.9 Impacts on Terrestrial, Freshwater and Marine Ecology

Ecologically sensitive areas with unique faunal or floral populations are unlikely to be widely known. In order to avoid unnecessary destruction of these habitats, the engineer should consult the relative authorities during the planning stages of the project. If necessary, a relevant specialist can be appointed to assist with the impact assessment. A list of the relative authorities appears in Appendix 12-3.

Certain fauna reside some distance from their breeding grounds or foraging areas, many of which can be separated by roadways. Recently a number of instances of providing "animal crossings" such as bridges and culverts to allow this process to continue unimpeded have been reported (Moore, 1990). It is important to timeously identify areas where such measures are necessary.

It is known that for some species of bird, if their habitat is split in two, less than half the population remains in each part (Simpson, 1991). This may be true for other animals, reptiles or insects.

Suitable fencing should be provided where heavy traffic passes through wildlife sanctuaries and the risk of animals being on the roads is high (Moore, 1990).

Table 12-9: SUMMARY OF THE IMPACTS ON TERRESTRIAL FRESHWATER AND MARINE ECOLOGY

IMPACT	MITIGATION
Planning	Consult relevant authorities
Flora and fauna	Implement findings of relevant ecologist

12.4.10 Other aspects

Other environmental aspects concerning road construction are numerous, and although they may be given less consideration than those described above, are nevertheless just as important. This category covers aspects such as the degradation of natural history (particularly those of palaeotological significance) and cultural sites (those of archaeological, burial or religious significance).

Particular caution should be exercised when designing and constructing surface and sub-surface drainage and de-watering measures. The potential effect on the regional groundwater regime should be noted in order to ensure that dams and agricultural water sources (eg boreholes) are not detrimentally affected.

Deviations or temporary alternative routes are typically required during construction, upgrading or rehabilitation projects. These need to be rehabilitated during the final stages of the project. Realignment of a road will often require that the old road is ripped up and revegetated.

It is interesting to note that the construction of toll roads is considered to have serious environmental consequences in the United Kingdom (Simpson, 1991). The additional land necessary for the construction of toll booths and the associated infrastructure can be significant. However, locally constructed toll roads appear to pay more attention to environmental aspects in the overall context.

Table 12-10: SUMMARY OF THE IMPACTS ON NATURAL AND CULTURAL HISTORY

IMPACT	MITIGATION
Palaeontology	Consult relative authorities
Archaeology	Consult relative authorities
Cultural sites	Consult relative authorities and local residents
Adjacent routes	Consider adjacent routes in the impact assessment
Drainage measures	Consult hydrologist
Deviations	Rehabilitate accordingly

12.5 Conclusion

Roads (and transport routes in general), by their nature, are environmentally obtrusive. It is important therefore to construct them such that the impact on the environment is minimised as far as possible.

This chapter suggests ways in which the environmental impacts should be addressed during the early stages of a project in order to allow uninterrupted construction and costly time delays resulting from remedial measures.

In most cases a satisfactory compromise can be achieved without the environment being excessively degraded or the road project being inordinately disadvantaged in terms of economics or construction duration.

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APPENDIX 12-1: AN EXAMPLE OF AN ENVIRONMENTAL CHECKLIST

1 ENVIRONMENTAL CHECKLIST

The following checklist is an example that can be used during the planning stage of the proposed development. The engineer can complete the checklist and decide whether the development will have no significant impacts on the environment, or whether an initial assessment should be carried out. Details of potential interested and affected parties and pressure groups can also be noted.

CLASSIFICATION OF ENVIRONMENTAL IMPACTS				
IMPACT	DETAILS	IMPACT		
		NO	?	YES
Route location	Visual			
	Erosion			
	Land instability			
	Vegetation			
	Regional runoff			
	River ecology			
	Expropriation and economic impacts			
Excavation and dumping	Visual			
	Land instability			
	Fill material			
	Material disposal			
Material acquisition	Borrow pit location			
	Visual			
	Vegetation			
	Access roads			
	Other			
	Rehabilitation			
Erosion	Erosion potential			
	Regional runoff			
Noise	Construction			
	Traffic			
	Traffic - fauna			
Vibration	Construction			
	Traffic			
Pollution	Volatiles and leaching			
	Exhaust emissions			
	Dust			
	Erosion			
	Construction waste			

CLASSIFICATION OF ENVIRONMENTAL IMPACTS				
IMPACT	DETAILS	IMPACT		
		NO	?	YES
Terrestrial ecology	Flora			
	Fauna			
Severance	General			
	Wildlife sanctuaries			
	Wetlands			
Other	Palaeontology			
	Archaeology			
	Cultural sites			
	Adjacent routes			
	Drainage			
	Deviations			
Additional				

APPENDIX 12-2: GUIDELINES FOR REHABILITATION

1 GUIDELINES FOR REHABILITATION

1.1 Introduction

Before 1990, borrow pits excavated for road construction were not subject to the requirements of mining legislation. However, with the passing of the Minerals Act (Act 50 of 1990), the Road Authorities were informed by the Department of Mineral and Energy Affairs that the excavation of borrow pits was now subject to the Act. Amongst other items, the act states that all borrow pits must be rehabilitated to the satisfaction of the Government Mining Engineer.

No fixed guidelines for the actual rehabilitation are provided as every borrow pit should be managed according to the local conditions (ie topography, geology, climate and local vegetation).

1.2 Guidelines

The following guidelines are recommended for the engineer.

- The Chief Mining Inspector of the region must be informed in writing of the proposed borrow pit location before excavation starts. The letter must include a scale map with the location of the proposed borrow pit indicated and a full description of the excavation including the following:

  (i) Details of applicant
  (ii) Location of borrow pit
  (iii) Surface area, depth and period of operation
  (iv) Proposed rehabilitation (soil, vegetation, land use and visual impact)

- In terms of the act, a manager must be appointed for each borrow pit. The manager would usually be the site engineer if consultants have been appointed, or the local roads superintendent if Provincial Authorities are used. The manager will be responsible for the control and supervision of the excavation.

- Once permission has been granted for excavation, sketches and photographs of the area and a species list of the vegetation should be made. Drainage patterns should also be carefully recorded. These will assist in the rehabilitation operation.

- The topsoil should then be removed and stockpiled on the edge of the borrow pit. Under no circumstance should the soil be removed by local residents and care must be taken to avoid erosion of the stockpile. It should not be compacted, but turned occasionally if left for long periods.

- Rehabilitation must start as soon as the required material has been removed and should follow the plan submitted to the Chief Mining Inspector. Attention should first be given to the slopes of the excavation - for pits shallower than 1.5 metres the slope should not exceed 1:3, while the slopes of deeper pits should not exceed 1:2. Slopes should also be formed so that the rehabilitated area will harmonise with the surrounding area. All discard material from the excavation and cut and fill operations should be deposited into the pit to reduce the depth. After shaping, the topsoil should be evenly replaced over the entire surface of the excavation. Retaining walls and terracing should be utilised to prevent erosion wherever necessary. Wherever possible, borrow pits should be fully rehabilitated and not left as dams which become unsightly during dry periods, are prone to erosion and attract dumping.

- All temporary structures (including foundations and concrete slabs) must be dismantled and removed. Access roads to the excavation must also be rehabilitated.

- Sufficient seed is often present in the soil and will readily germinate after spreading. If germination does not occur, suitable seed will have to be sown. The seed of similar species to those already established in the area should be used. Revegetation should be carried out early in the rain season wherever possible. Advice on species selection and establishment should be sought from qualified personnel.

- The Chief Mining Inspector must be informed as soon as the vegetation has been re-established and a representative will then inspect the rehabilitation.

- The rehabilitation must be monitored by the manager to ensure that the vegetation survives, that exotic species do not establish themselves and that erosion is controlled.

APPENDIX 12-3: CONTACTS FOR ENVIRONMENTAL INFORMATION

The following authorities and institutions can be contacted with regard to information on flora and fauna in a particular area, known ecologically sensitive areas and known fossil and archaeological sites etc.

ENVIRONMENTAL INFORMATION		
No	**INFORMATION**	**CONTACT**
1	Integrated Environmental Management	Dept Environment Affairs, Pretoria
2	Terrestrial ecology	TPA Nature Conservation, Pretoria CPA Nature Conservation, Cape Town OFS Nature Conservation, Bloemfontein Natal Parks Board, Pietermaritzburg National Parks Board, Pretoria Botanical Research Institute, Pretoria
3	Palaeontology	Geological Survey, Pretoria Bernard Price Institute, Johannesburg
4	Archaeology and cultural sites	University of Stellenbosch University of Cape Town University of the Witwatersrand University of Pretoria National Museum, Bloemfontein Natal Museum, Pietermaritzburg National Museum, Windhoek Local municipality and RSC
5	Noise	SA Bureau of Standards
6	Heritage Protection	National Monuments Council Simon van der Stel Foundation
7	Hydrology	Department of Water Affairs
8	Sociology	Human Sciences Research Council
9	General	Environmental Services, CSIR

APPENDIX 12-4: LEGAL REQUIREMENTS

Numerous acts and ordinances are applicable to the roads and transport industry and a number of them provide measures to exercise control over the negative environmental impacts of road transportation either directly or indirectly. The following Acts should be considered:

- National Roads Act, 54 of 1971
- Road Traffic Act, 29 of 1989
- South African Transport Services Act, 65 of 1981
- Minerals Act, 50 of 1991
- Land Survey Act, 9 of 1927
- Physical Planning Act, 125 of 1991
- Fencing Act, 31 of 1963
- Expropriation Act, 63 of 1975
- Subdivision of Agricultural Land Act, 70 of 1970
- Conservation of Agricultural Resources Act, 43 of 1983
- Agricultural Pests Act, 36 of 1983
- Environment Conservation Act, 100 of 1992
- Forest Act, 122 of 1984
- Mountain Catchment Areas Act, 63 of 1970
- National Parks Act, 57 of 1976
- National Monuments Act, 28 of 1969
- Atmospheric Pollution Prevention Act, 45 of 1965
- Health Act, 63 of 1977
- Internal Health Regulations Act, 28 of 1974
- Machinery and Occupational Services Act, 6 of 1983
- Water Act, 54 of 1956
- Water Research Act, 34 of 1971
- Lake Areas Development Act, 39 of 1975
- Sea shore Act, 21 of 1935
- Sea Birds and Seals Protection Act, 46 of 1973
- Sea Fisheries Act, 12 of 1988
- Dumping at Sea Control Act, 73 of 1980
- Advertising on Roads and Ribbon Development Act, 21 of 1940
- Hazardous Substances Act, 15 of 1973
- Explosives Act, 26 of 1956

The following Provincial Ordinances should be considered:

- Transvaal Nature Conservation Ordinance, 12 of 1983
- Transvaal Town Planning and Townships Ordinance, 15 of 1986
- Cape Nature and Environmental Conservation Ordinance, 19 of 1974

- Cape Land Use Planning Ordinance, 15 of 1985
- Natal Nature Conservation Ordinance, 15 of 1974
- Natal Town Planning Ordinance, 27 of 1949
- Natal Prevention of Environmental Pollution Ordinance, 21 of 1981
- Orange Free State Nature Conservation Ordinance, 8 of 1969
- Orange Free State Townships Ordinance, 9 of 1969

Pressure groups who are against certain developments will closely monitor a project to insure that the developer is abiding by all legal requirements. It is therefore imperative that the developer, consultant and contractor are fully aware of any regulations that may be applicable to the development.

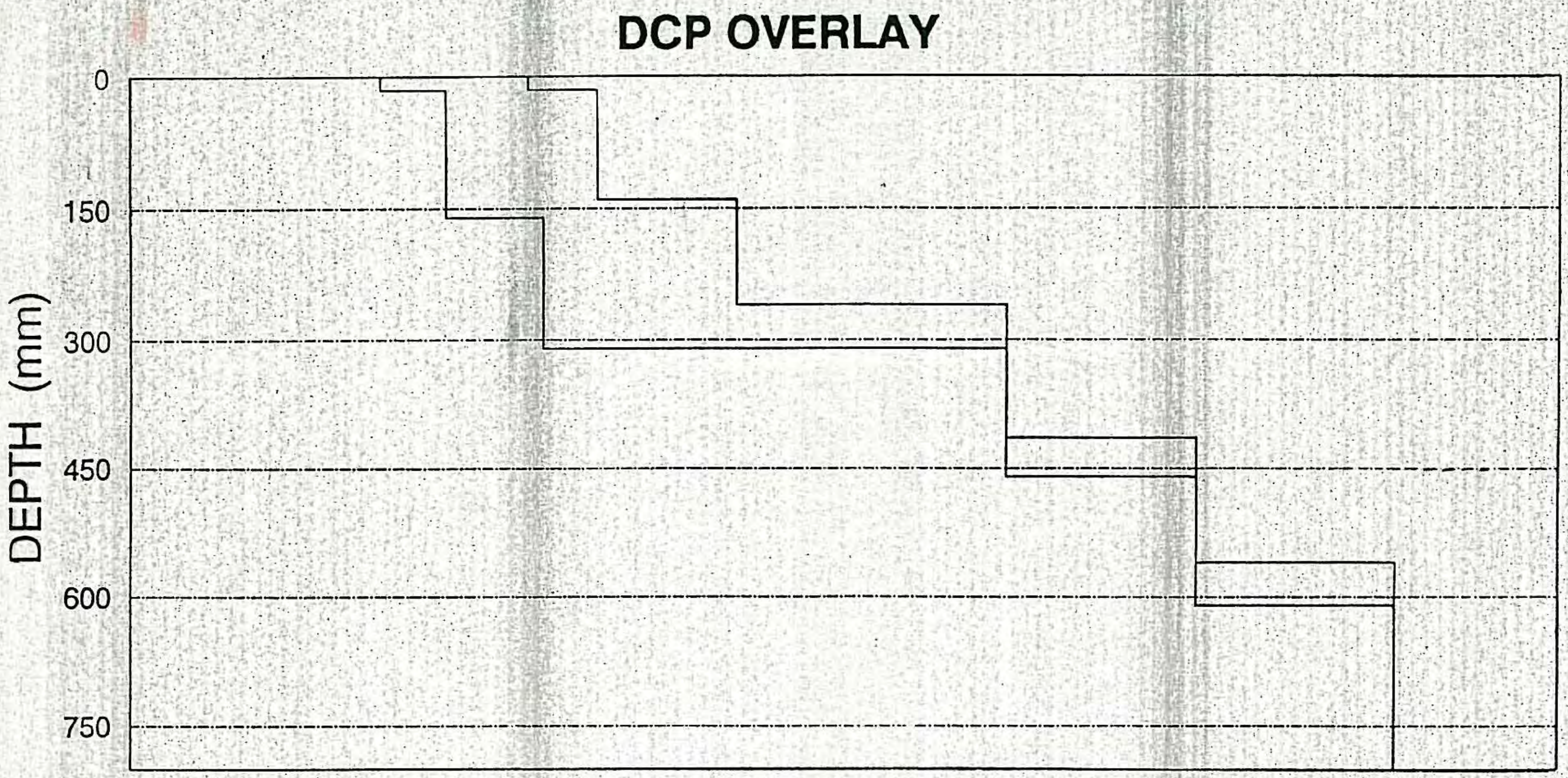

Figure 11-5A: DCP OVERLAY