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GUIDELINES FOR UPGRADING OF LOW VOLUME ROADS

Prepared on behalf of the SOUTH AFRICAN ROADS BOARD

COMPILED BY:

Division of Roads and Transport
Technology, CSIR
and
Jordaan & Joubert Inc.

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PREFACE

The purpose of this manual is to provide guidelines on the upgrading of gravel low volume access roads to roads and maintenance personnel of road authorities of all sizes . Low volume is, for the purposes of this document, defined to be less than 500 equivalent vehicle units (evu's) per day which will constitute a daily flow of approximately 400 vehicles.

The document is not intended as a fully fledged and detailed design manual; rather, it highlights pertinent aspects of road design in order to ensure that the upgrading can be carried out effectively and allows the reader to judge the full extent of upgrading required.

DISCLAIMER

The views and opinions expressed in this report are those of the authors and do not represent South African Roads Board Policy.

SOUTH AFRICAN ROADS BOARD RESOLUTION

- This report has been approved for general distribution by the South African Roads Board.

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Project Committee: Chairman: Mr J.A. du Plessis

Specialist advisors:
Mr A.O. Bergh (Mackintosh, Bergh and Sturgess)
Prof A.T. Visser (VIAED)

Project team: Division of Roads and Transport Technology

Project leader: Mr G.D. van Zyl

Dr H.E. Bofinger
the late Dr P.C. Curtayne
Ms E. de Beer
Prof. S.J. Emery
Mr D.J. Jones
Dr P. Paige-Green
Dr H. Ribbens
Mr M.C. Shackleton
Dr H. Wolff

Project team: Jordaan & Joubert Inc.

Project leader: Prof. P.W. Jordaan

Mr P.A. Pienaar

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Road authorities

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Other organisations

BKS Inc.
Development Bank of South Africa
South African Bitumen and Tar Association (SABITA)

CONTENTS

1 INTRODUCTION

Low volume gravel roads are often upgraded to levels where they could be effectively surfaced at low cost. In the majority of cases the alignment of these roads will not need to undergo major changes and provision will only be made for reducing unsafe situations, providing adequate drainage and providing an adequate pavement structure for carrying expected loads over its design life. The emphasis is therefore on the optimum use of existing road prism and in situ strength.

Although guidelines are given on appropriate standards, these should be discussed with the local communities to ensure acceptance. Because funds for road maintenance are already limited, care should be taken that the surfacing option will not increase the burden of maintenance.

It is recommended that the upgrading of existing gravel roads to lower paved standards be confined to short local roads and not carried out on long roads carrying high volumes of through traffic. In this regard, care should be taken that the upgraded road does not attract high volumes of traffic as a result of the upgrading.

This document seeks to provide such guidelines for personnel involved with this type of activity. It covers topics such as the investigation of the existing road, geometric and drainage considerations, design of the upgraded pavement and choosing surfacings for the road. It also deals with construction, maintenance and environmental aspects and provides warrants for surfacing a gravel road.

2 INVESTIGATION OF THE EXISTING ROAD

2.1 Introduction

As a precursor to the upgrading of a road, an investigation is required into the existing road to establish its condition and the extent of upgrading required. This chapter deals with that investigation, primarily in terms of upgrading an existing gravel road to a paved road, which should cover the following areas (Table 2-1):

- general assessment of road and surrounding environment,
- visual assessment of the road,
- structural assessment of pavement.

Table 2-1: EXTENT OF INVESTIGATION

GENERAL ASSESSMENT	EXTENT OF INVESTIGATION
History of the road	limited
Geology/terrain/climate	limited; look for potentially troublesome areas
Traffic	limited; check for substantial attracted traffic if the road is to be paved
Drainage	in detail; this is more important for a paved road than for a gravel road
Geometry and alignment	evaluate the extent of hazardous locations and improvements needed
Source of materials	basecourse and surfacing
VISUAL ASSESSMENT	limited: check for weak areas that may need structural upgrading
STRUCTURAL ASSESSMENT	typically with DCP and some test pits

2.2 General assessment

The general assessment is made by considering the following aspects of the road and its surrounding environment:

i) History of the road

The history of the existing gravel road can be useful in determining the extent of structural and geometric upgrading required when the road is paved. If the road has been in use for many years, performs well structurally, and accident black spots have been progressively eliminated, then it is likely to perform well as a paved road. If the road has given problems and is in the first few years of service, then the investigation should be aimed at uncovering the causes such as drainage problems, lack of compaction, poor alignment, etc.

ii) Geology, terrain and climate

A limited geological investigation will establish the subgrade conditions, e.g. does the road traverse decomposed basic rock (black turf), or more stable subgrade, or both? Is the overburden deep or shallow? Climate should also be noted.

iii) Traffic

The requirements for traffic data are given in Chapter 5. The possibility of industrial traffic using the road (quarries and cement works etc) should be considered.

iv) Drainage

The drainage structure of the road to a very large extent affects the life of the road and a detailed investigation should be made regarding the adequacy of:

(a) Surface drainage (standing pools due to rutting etc.),

(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 well 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.

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 paved road. The geometric aspects are given in Chapter 3. Careful consideration should be given to the cost implications of geometric upgrading. As a general rule, geometric improvements are not justified simply because the road is being paved. The justification for geometric improvements is given in section 3.1.

The analysis should consider:

- the existing cross-section of road,
- intersections - position and safety - merging lanes,
- passing lanes - depending on traffic,
- necessity for surfacing part of the wearing course to act as 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.

The involvement of the community served by the road in the selection of appropriate geometric standards is very important.

vi) Source of materials

The source and availability of surfacing and basecourse 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.

2.3 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 surfacing type, and to assist in dividing the road into uniform sections. The following parameters are usually noted:

- rutting,
- failures - type of failure, e.g. wearing course, subbase, subgrade. Cause of the failures. Extent of the failures and percentage failures of sections of the road,
- riding quality of the road.

2.4 Pavement structural assessment

The pavement structural assessment is used to measure the structure of the existing gravel road and its materials (as an input into the design phase). This is covered in chapter 5.

accessibility rather than mobility

↓ CC, even longer & ↑ curving alignment

3 GEOMETRIC ASPECTS

↳ this document need more road safety

3.1 Introduction

Generally speaking, the upgrading of the geometric elements of a road is very costly. Wherever the geometrics of a road can be upgraded at minimal cost, this should of course be done. In the main, however, the geometrics of the existing road should be accepted, and the road user informed of the possible dangers on the road and safe operating speeds by means of the relevant road signs. The following situations, however, may require improvements to the geometry of a road:

- where obvious reductions in construction costs can be achieved
- where minor and low cost improvements may have significant safety benefits
- where the road alignment obviously does not serve the adjacent land uses optimally
- where hidden situations requiring a low operating speed exist.

① Base year traffic = present + attracted

② Base year EVU's = Base year light vehicles + base year

③

3.2 Design year traffic

Traffic volumes are measured in equivalent vehicle units (evu's) per day. Heavy vehicles are converted to equivalent vehicles by using Table 3-1.

access traffic not through traffic

Table 3-1: VEHICLE EQUIVALENCY FACTORS

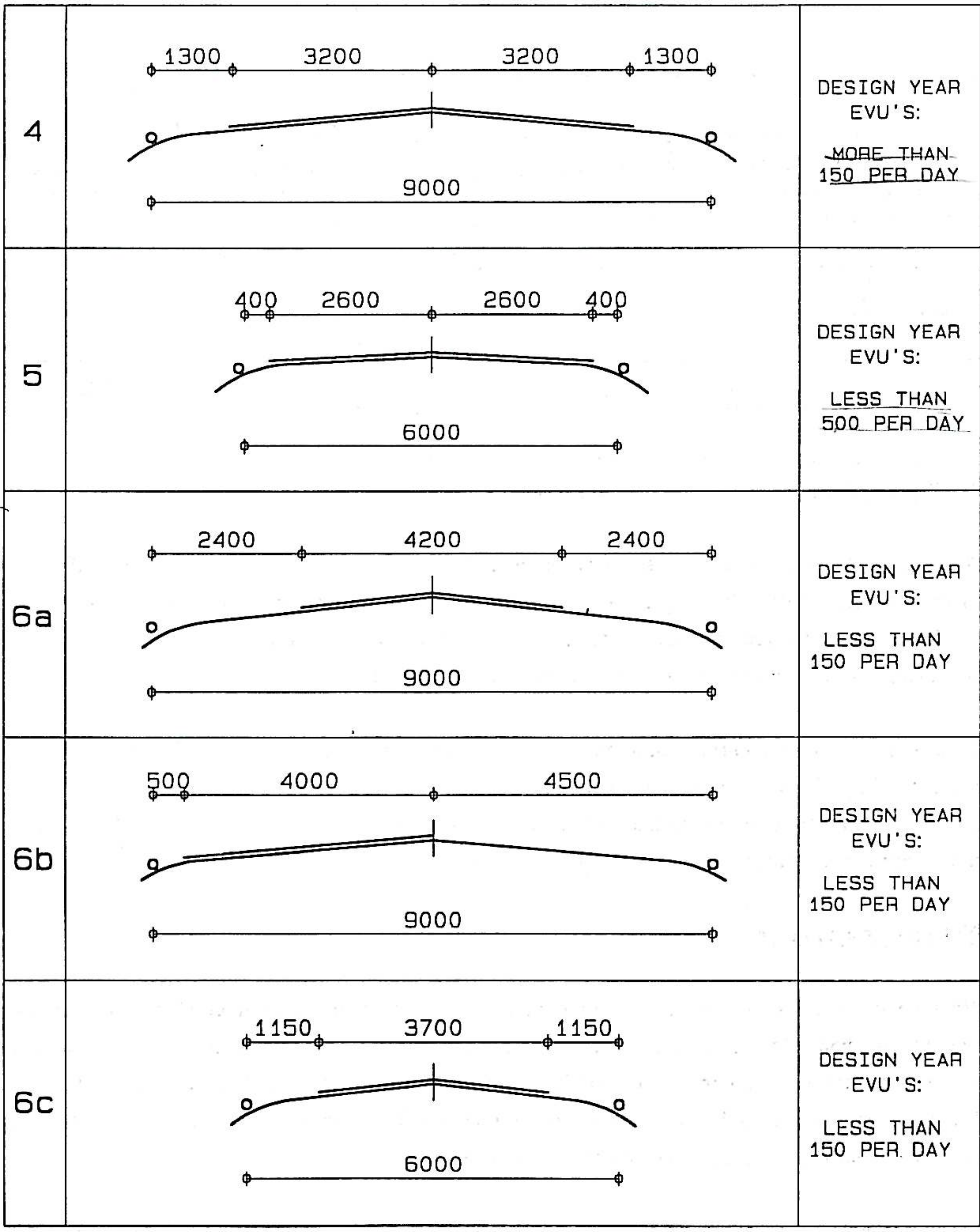
GENERAL TERRAIN TYPE	EVU's PER HEAVY VEHICLE
FLAT	3
ROLLING	5
MOUNTAINOUS	10

The design year traffic, ten years from the present, is found by multiplying the present daily traffic by the applicable growth factor from Table 3-2. *Where relevant, the traffic that may be attracted to the upgraded road should be estimated and added to the present daily traffic before multiplying with the growth factor.*

3.3 Road cross-section

Figure 3-1 shows several cross-sections, with an indication of the design year traffic volume warrants.

(4) 150 500 (5) (6)



highest

single carraige road

half width surfacing

very narrow

use with caution

Figure 3-1: RECOMMENDED PAVEMENT CROSS-SECTIONS

more in thick document

traffic classes (thick document)
4A >2000
4B
4C
LA
LB
LC 1-59

Table 3-2: 10 YEAR PERIOD GROWTH FACTORS

TRAFFIC GROWTH RATE (% p.a.)	TRAFFIC GROWTH FACTOR	TRAFFIC GROWTH RATE (% p.a.)	TRAFFIC GROWTH FACTOR
0	0,00	4 *	1,48
1	1,10	5 **	1,63
2	1,22	6 **	1,79
3 *	1,34	8	2,16

* Typical growth rates in developed areas

** Typical growth rates in developing areas

advisable max gradient
design speed / terrain type
rolling / maint

The width of the existing road is the most important factor in selecting a cross-section type. For any appreciable amount of traffic, say 150 evu's per day or more, cross-section type 5, with a paved width of 5,2 m, is the minimum that should be provided. At smaller paved widths the wear on the edge of the paving will be very severe, requiring excessive maintenance.

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. stopping sight distance / safe speed

Widening around curves min meeting sight distance
" " " on level roads

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 a road having cross-section type 4 that carries an appreciable amount of large heavy vehicles and/or busses.

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.

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)

Surfaced single lane roads

The implementation of this type of road would usually be limited to situations where the design year traffic is less that 150 evu's per day and the terrain provides good visibility to the road user.

Small cost diff. between 6 & 4,5

Cross section types 6a and 6b provide a single paved lane and allow for the passing of opposing vehicles. Care should be taken in selecting these particular road types as investigations have shown that in some instances there is only a small cost differential between these and types 4 or 5. The reason for this is that on a single lane road all heavy vehicles travel in the same wheelpath, necessitating a stronger pavement design, which could be as expensive as the wider seal.

Passing of vehicles on roads with cross-section type 6a takes place with each vehicle moving to the left, leaving only one wheel on the sealed surface. At 150 evu's per day vehicles will meet, on average, every two kilometres. At 60 evu's per day vehicles will meet, on average, every five kilometres.

Cross-section type 6b is an alternative configuration for a road with a single paved lane. This configuration has the advantage that maintenance is required on one side of the road only. It also requires that traffic on the one side must 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.

Cross-sections types 6a, b and c should only be used where the maintainability of the shoulders, especially with respect to "drop-offs" can be assured.

Paving of shoulders

In the case of cross-section type 4 the shoulders should be paved only if warranted by special circumstances, e.g. suitable gravel not available, high erosion potential, low maintenance capability, etc.

3.4 Vertical alignment

Using the available stopping sight distance, measured from an eye height of 1,05 m to an object height of 0,15 m, the safe operating speed can be determined from Figure 3-2. If the safe operating speed is less than 100 km/h, speed restriction signs showing the allowable speed should be erected on either side of the vertical curve. (Note: There is no "blind rise" warning sign and the "Advisory speed information plate" may not be displayed independently).

The dimensions of cross-section type 6c are such that vehicles travelling in opposite directions can only pass at very low speeds, say less than 15 km/h. *For roads using this cross-section twice the sight distance required by Figure 3-2 are required for a given operating speed.*

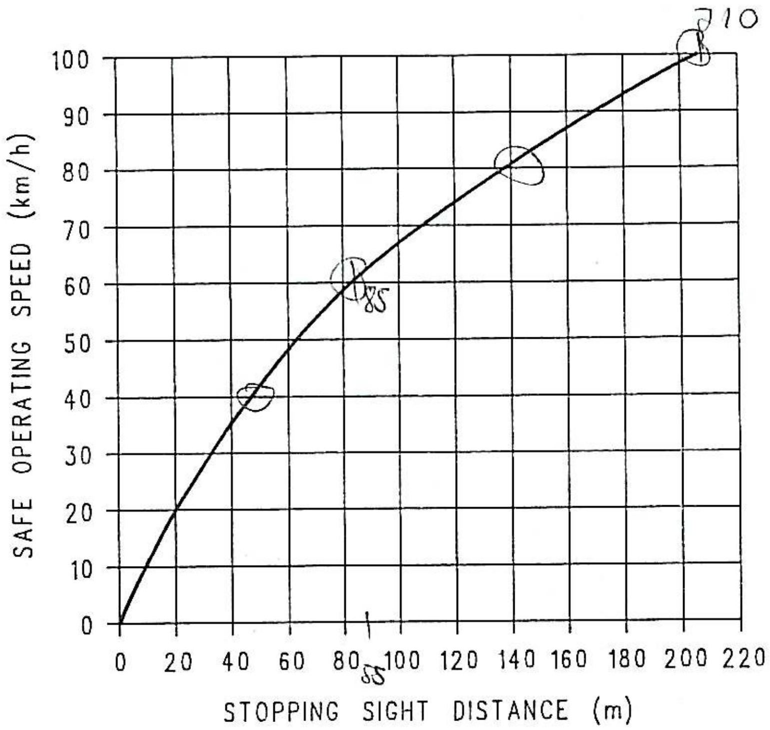


Figure 3-2: Stopping distance and operating speed

3.5 Horizontal alignment

Using the radius of the horizontal curve and the available superelevation rate, the operational speed can be found from Figure 3.3. The ~~horizontal~~ sight distance should also be measured, from an eye height of 1,05 m to an object height of 0,15 m, and an operating speed determined from Figure 3.2. The *lower* of these two operating speeds is the safe operational speed. This safe operating speed should be posted if it is less than 100 km/h. For this purpose one of the following warning signs, together with the *"Advisory speed"* supplementary plate, should be used:

- If the safe operating speed is between 10 % and 33 % lower than the operating speed on the straight alignment preceding the curve, use either the *"Gentle curve"* sign or the *"Winding road"* sign, as the situation may require.

- If the safe operating speed is reduced by more than 33 %, use either the "Sharp curve" sign or the *"Combined curves"*, sign or the *"Hairpin bend"* sign as the situation may require.

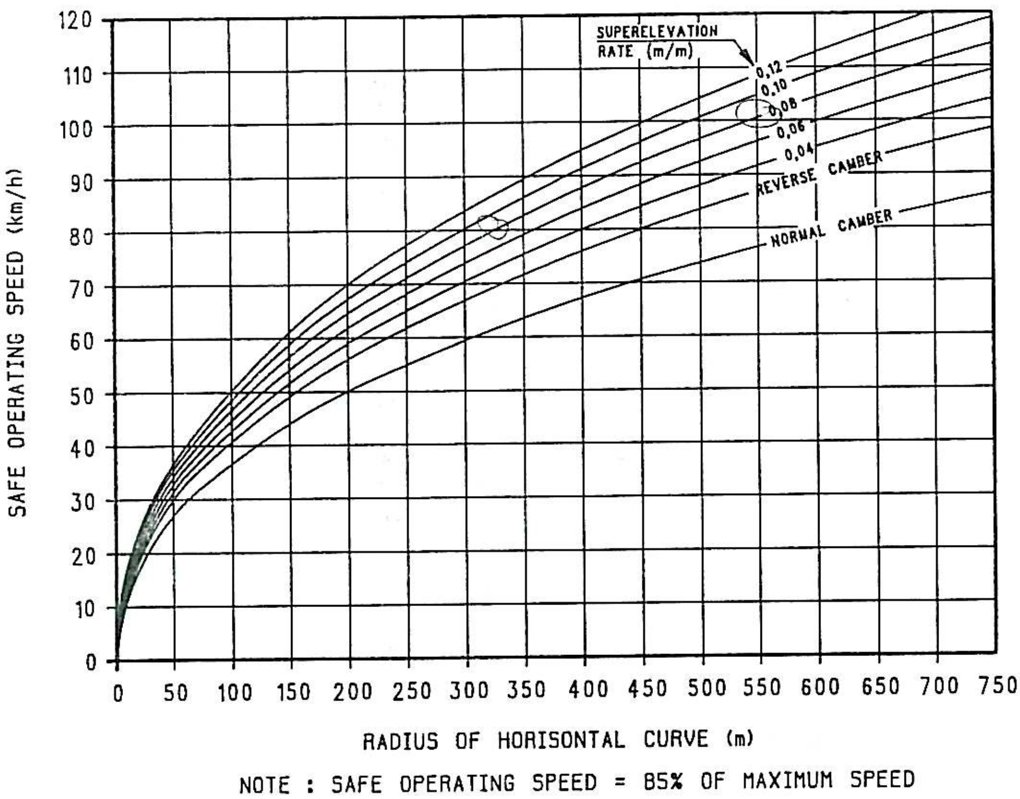


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Figure 3-3: Operating speeds and horizontal radii

→ shoulder sight distance for stop control

→

3.6 Road signs

The approach to be followed is one of providing a reasonable facility at minimum construction cost. The road user is expected to adapt his operating speed to suit the facility. In doing so, the road user should, however, be advised, through adequate and well-maintained road signs, of the safe speed on the various segments of the road. *Care should , however, be taken not to surprise the driver with a dangerous situation which is totally unexpected, as it could happen that a road sign may be obscured, stolen, or otherwise missing.* Table 3-4 shows the distance a warning sign should be placed before the limiting geometric feature.

Table 3-4: WARNING DISTANCES

OPERATING SPEED BEFORE LIMITING FEATURE (km/h)	OPERATING SPEED AT LIMITING FEATURE (km/h)	WARNING DISTANCE (m)
100	90	125
	80	125
	70	125
	60	165
80	70	75
	60	75
	50	125
	40	140
60	50	60
	40	85
	30	110
	20	125

Source: Figure 3-1, Vol, 1 South African Road Traffic Signs Manual, April 1991.

4 DRAINAGE

4.1 Introduction

4.1.1 Scope

When a road is upgraded it is important to ensure that the drainage system is also functioning well. As the upgrading of major items of drainage infrastructure (such as bridges) is generally very expensive, existing infrastructure should be utilized as far as is possible. Where required, however, the necessary drainage infrastructure should be provided to an appropriate level. It should be remembered that adequate drainage provision is more important for a paved road than an unpaved road.

The purpose of this chapter is to provide a framework to assist the designer in evaluating the adequacy of existing infrastructure and the need for new infrastructure when low volume roads are upgraded.

4.1.2 Other guidelines

It is not possible to provide comprehensive drainage design guidelines here. The reader is referred to the parent document: *Towards appropriate standards for rural roads: discussion document*, as well as 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.

4.1.3 Approach

The approach adopted is to make maximum use of existing drainage facilities, even if it does not fully comply with the required standards. It is, however, necessary to evaluate the adequacy of existing infrastructure in order to ensure that the risks associated with reducing standards are at acceptable levels.

4.2 Hydrological analysis

4.2.1 Introduction

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

Analysis procedures are not addressed in this document - the reader is referred to the guidelines available on hydrological analysis.

4.2.2 Return period

Recommended basic flood frequencies for drainage channels, culverts, low level structures and high level bridges on low volume roads are provided in Table 4-1. The design of low level structures is to be based on a fraction of the 1:2 year return period, as is discussed in Section 4.9.2.

The return period of the existing infrastructure should be determined and should then be compared to the values recommended in Table 4-1. Previous experience with regard to the adequacy of the facilities as experienced by both road users and the road authority should be taken into account in this process.

Table 4-1: 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, structures

** Refer to section 4.9.2

A decision should then be taken for each facility, i.e. whether:

- The facility meets the requirements
- The facility does not meet the requirements, but is acceptable in terms of previous experience and the risk associated with the design flood being exceeded.
- The facility is not adequate and needs to be upgraded or replaced.

4.2.3 Methods of design flood determination

Various methods for the determination of the design flood can be used. The reader is referred to the guidelines available on design flood determination.

4.3 Hydraulic calculations

Hydraulic calculations are necessary to determine or evaluate 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.

4.4 Structural loading

4.4.1 Existing structures

When "as built" drawings are available, the designer should establish whether the existing structure can carry the expected loads.

In many cases "as built" drawings are not available and the following should be done:

- The structure should be thoroughly inspected visually to establish whether there are any signs of distress or failure, and if there are, to evaluate the implications thereof.

- If the condition of the structure appears to be sound and no significant increase in traffic volumes or changes in the vehicle composition using the road are expected, the structure may be used as it is.

- If the condition of the structure appears to be sound, but a significant increase in traffic volumes or a change in the traffic composition is expected, slab thicknesses, reinforcement details and span lengths should be determined and an analysis should be done to establish whether the existing structure is adequate to carry the expected loads. The structure should be strengthened or replaced only if necessary.

- If the structure is not in a good condition, i.e. when signs of distress and failure are observed, the implications of these must be evaluated in the light of the expected loading conditions in order to establish whether the existing structure is adequate, or whether repairs, upgrading or replacement is necessary.

TMH7: Code of practice for the design of highway bridges and culverts in South Africa (CSRA, 1985 and 1989) should be used for the evaluation of the structural loading of bridges and culverts. The NC loading may generally be omitted on low volume roads, but all road bridges should be designed for both the NA load and at least the NB 24 load.

4.4.2 New structures

All new structures should be designed for both the NA load and at least the NB 24 load. The design engineer must ascertain whether this loading will be adequate for a particular structure.

4.5 Formation and earthworks

4.5.1 Formation

In general it is advisable to build the formation to heights of 400 to 500 mm over flat or rolling areas. Depending on the existing formation and the pavement design, however, it may be warranted for low volume 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 acceptable to omit some or all of the culverts and to allow water to flow over the road, especially if there are no/few existing culverts. Side drains are then 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. It may also in some instances be appropriate to provide berms in the road, similar to speed humps, to prevent water from flowing in the longitudinal direction. The necessary warning signs must be provided.

4.5.2 Deep cuttings and high embankments

Existing deep cuttings and high embankments require special attention with regard to drainage to ensure that they will be stable when the road is upgraded.

The adequacy of the drainage of deep cuttings should be investigated. Where necessary, measures such as cut-off drains up slope and behind the cut face must be provided. Larger cuts may require lined drains to remove water from the cut-off drains directly down the face of the slope into foot drains. The application of other techniques such as reducing the batter should only be used where absolutely necessary. The erosion of the faces of slopes should also be addressed where necessary.

The stability of fills and the role of stormwater should also be investigated in order to evaluate the need for kerbs, down chutes, etc. Road embankments that are designed to be overtopped in a flood should also be protected against scour, if this is deemed necessary.

- Mitre drains and - banks: Used to drain water from the side drain into the veld.
- Down chutes: Used on fills and cuttings to accommodate concentrated waterflow down the fill or cutting in order to prevent erosion.

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.

4.7.2 Approach

Existing drainage channels should be evaluated in terms of the above in order to determine whether they need to be upgraded. The investigation should also address the need for other drainage channels which do not exist, but which are required.

4.7.3 Energy dissipators

Energy dissipators usually consist of still basins, protruding blocks and check dams. The erosion potential should be quantified, taking into account previous experience, in order to quantify the need for energy dissipators.

4.8 Culverts

4.8.1 Return period

Where culverts already exist, their capacity should be determined. If less than the 1:2 year flood, they must be upgraded to accommodate the 1:5 year flood (Table 4-1). If more than the 1:2 flood, but less than the 1:5 year flood, the designer must decide in terms of the characteristics of the area and previous experience whether it is justified to upgrade the capacity. In the case where the 1:5 year flood can be accommodated, existing culverts should be used as they are.

4.8.2 Culvert types

Where culverts are required, purchase and transport costs play a major role in selecting the culvert type for a particular application. The advantages of corrugated metal nestable 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 may occur in coastal areas or in mining areas. The use of 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 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 expensive soil conditions culverts must be watertight. If not, the in situ soil 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.

4.8.3 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 evaluation of existing culverts and the design of new culverts.

4.8.4 Scour

Existing and new culverts should be checked for scour, especially at the outlets. Where warranted, scour protection or energy dissipation must be accomplished.

4.8.5 Design

The spacing of culverts should be determined by taking into account the specific conditions encountered along the road. Culverts should be designed using guidelines such as the Road Drainage Manual referred to in Section 4.1.2.

If a detailed investigation and design of pipe culvert cross drainage cannot be carried out, 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

4.8.6 Selection of drainage structures

Where new drainage structures are required, 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 km^2) 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 restrictions, the unacceptability 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.

4.9 Low level structures

4.9.1 Selection of bridge structures

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 upgrading road infrastructure.

At each waterway on a low volume road justifying a bridge, or where a low level structure exists, a decision has to be made whether a low level structure is adequate, or whether a high level bridge is required. 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.

4.9.2 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.

Three design levels are defined, as shown in Table 4-3. 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 4-3, 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 4-3 describes the implications of the three design levels suggested in more detail.

Table 4-3: LEVELS OF DESIGN FOR LOW LEVEL STRUCTURES

DESIGN LEVEL	f_i	AVERAGE NO OF TIMES EXCEEDED PER YR PER GAUGING STATION			AVERAGE DURATION 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 exceeds 50 km.

- Should there be no alternative route available, or should the road be of strategic importance, the designer must choose the design level in terms of the implications described in Table 4-3.

Once the design level is known, the design flood is determined as follows:

$$Q_{design} = f_i \ x \ Q_2$$

where: Q_{design} = the design flood

f_i = a dimensionless factor related to the design level and shown in Table 4-3

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 invented structures, e.g. concrete slabs. Part of the design flood may be accommodated over the structure, provided that it is still safe for a vehicle to pass over the structure.

The structure 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 depths less than the maximum acceptable.

Q_u = the flow capacity under the structure.

As far as flow depth is concerned, it may be assumed that a vehicle 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:

- Super-critical flow: maximum depth of 100 mm
- Sub-critical flow: maximum depth of 150 mm.

4.9.3 Gradients

It is acceptable to reduce geometric standards on the approaches to low level structures. The criteria suggested are shown in Table 4-4. Gradients in excess of 10 per cent should only be provided over lengths shorter than 40 m.

Table 4-4: MAXIMUM GRADIENTS FOR LOW LEVEL STRUCTURES

DESCRIPTION	DESIRABLE MAXIMUM GRADE (%)	ABSOLUTE MAXIMUM GRADE (%)
Paved roads	12	15
Unpaved roads	10	12

The gradients of approaches at existing structures should be evaluated in terms of the above in order to decide whether the gradient should be improved.

4.9.4 Width

Recommended widths for new low level structures are provided in Table 4-5. The widths of existing structures should also be evaluated in terms of this table.

Table 4-5: RECOMMENDED WIDTH BETWEEN GUIDE BLOCKS FOR LOW LEVEL STRUCTURES (LLS)

CROSS-SECTION TYPE (Refer to Figure 3-1)	TWO LANE STRUCTURE (m)	SINGLE LANE STRUCTURE (m)
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

A decision on single or double lane width for new structures, or whether a single lane structure should be upgraded, 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 and 6c the use of single lane structures 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.
- Pedestrian volumes are low, typically less than 100 pedestrians in the peak hour.

4.9.5 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.

4.9.6 Traffic control measures

Driver safety and convenience are major factors in the evaluation or design of low level structures. Human life must never be endangered. This can inter alia be accomplished by providing guide blocks which indicate both the limits of the drift and the depth of the water.

The following road signs should also 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, e.g. sharp curve ahead, etc.

4.10 High level bridges

Where used on low volume roads, high level bridges can be either single lane or double lane structures. However, in general, 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.

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 low volume roads high level double lane bridges should only be provided on a route with traffic volumes in excess of 500 vehicles per day and where the road's importance warrants it. Poor alignment may also dictate the provision of a double lane bridge for safety reasons.

High level single lane bridges would be provided on a route where the traffic volume is less than 500 vehicles per day and where a low level structure is undesirable. As the single lane structure may be a traffic hazard with high traffic volumes, it is not recommended where the traffic volume is likely to increase significantly. 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.

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.

Recommended widths for high level bridges are provided in Table 4-6. 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.

Table 4-6: RECOMMENDED WIDTHS FOR HIGH LEVEL BRIDGES (HLB)

CROSS-SECTION TYPE (Refer to Figure 3-1)	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**
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

* 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

4.11 Subsurface drainage

Adequate stormwater drainage may in some cases alleviate the need for expensive subsurface drainage. Subsurface drains need only be installed when seepage or high watertables are encountered and not as a general policy in all cuts.

The reader is referred to *TRH 15: Subsurface drainage for roads (draft)*(1984) for guidelines on the design of subsurface drainage.

5 PAVEMENT DESIGN AND MATERIALS

5.1 Design philosophy

The low volume roads (LVR) in this document are those which serve rural areas and have a traffic volumes of less than 500 evu's or 400 vehicles per day. The upgrading referred to here is primarily aimed at providing a bituminous surfacing 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.

The approach followed and discussed can be summarised as follows:

- Determine the pavement strength required to ensure good performance and therefore low maintenance
- Test the existing gravel road structure and determine if any strengthening is required.

The designs are based on the material under the surfacing being at equilibrium moisture content which has been proved to be the case for surfaced roads in South Africa and which underlines the importance of adequate provision of drainage, construction control and resealing. Special pavement structures are proposed for environments where maintenance of roads is expected to be infrequent.

5.2 Design strategy

5.2.1 Selection of economic analysis period

The economic analysis period is a realistic cost period, which is used to compare the cost of alternative upgrading options. 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

5.2.2 Selection of structural design period

For low volume roads it is common 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. The cost differential between 10 and 20 years can sometimes be surprisingly low.

5.3 Design traffic

5.3.1 Traffic loading

For the structural design process, an estimate of the traffic loading (expressed as cumulative equivalent 80kN axle loads over the structural design period) is required. At its simplest, this is found from the average daily traffic in both directions and the percentage of heavy vehicles. Since heavy vehicles (trucks and buses) weigh so much more than cars, for all practical purposes it is sufficient just to consider the loading from the heavy vehicles and ignore the cars. This is done by estimating the weight of the heavy vehicles, and then estimating their growth rate over the structural design period. Heavy vehicle weights are expressed in terms of standard 80 kN axle loads (or E80s).

The simplest form of estimating the loading of heavy vehicles is to use 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 5-2 or Table 5-3. This factor is then multiplied by the number of heavy vehicles in both directions to obtain the average daily E80s.

Table 5-2: **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 5-3: AVERAGE E80S FOR DIFFERENT HEAVY VEHICLE CONFIGURATIONS (TRH 16, 1991).

Vehicle type	Average E80s per vehicle	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 fully laden

5.3.2 E80 Growth rate

The growth rate of heavy vehicle loading (E80s) can be different from the growth rate of heavy vehicles. This can be due to 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.**

5.3.3 Initial design traffic (E80s)

The computation of initial design traffic in terms of E80s involves the projection of current traffic 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:

The E80 growth factor g_x is tabulated in Table 5-4 for different values of traffic growth rate i and years between determination of axle load data and opening of road, x.

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

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

$$E_{initial} = E.g_x + A \quad (5.1)$$

where $E_{initial}$ = daily equivalent E80s in the initial year
E = daily equivalent traffic at time of design
g_x = E80 growth factor (see Table 5-4).
A = attracted traffic

Table 5-4: 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

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

5.3.4 Design traffic (cumulative equivalent traffic)

The design traffic (cumulative equivalent traffic expressed in terms of 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 design traffic in both directions (cumulative equivalent E80s) is calculated from:

$$N_e = E_{initial} \cdot f_y \quad \ldots\ldots\ldots\ldots (5.2)$$

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 5-5 for different E80 growth rates i and prediction period y in years. This design traffic needs to be adjusted for the number of lanes, as discussed below.

5.3.5 Distribution per lane

LVR's typically consist of two lanes, that is one lane per direction. **If the road has only one lane per direction the cumulative equivalent E80s constitute the design traffic.** 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 should not be multiplied by 0,5 before determining the design traffic class.

Table 5-5: 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
4	1 534	1 611	1 692	1 776	1 863	1 953
5	1 937	2 056	2 180	2 312	2 451	2 597
6	2 348	2 517	2 698	2 891	3 097	3 317
7	2 767	2 998	3 247	3 517	3 809	4 124
8	3 195	3 497	3 829	4 192	4 591	5 028
9	3 631	4 017	4 445	4 922	5 452	6 040
10	4 076	4 557	5 099	5 710	6 398	7 173
11	4 530	5 119	5 792	6 561	7 440	8 443
12	4 993	5 703	6 526	7 480	8 585	9 865
13	5 465	6 311	7 305	8 473	9 845	11 458
14	5 947	6 943	8 130	9 545	11 231	13 242
15	6 438	7 600	9 005	10 703	12 756	15 239
16	6 939	8 284	9 932	11 953	14 433	17 477
17	7 450	8 995	10 915	13 304	16 278	19 983
18	7 971	9 734	11 957	14 762	18 308	22 790
19	8 503	10 503	13 061	16 338	20 540	25 934
20	9 045	11 303	14 232	18 039	22 995	29 455

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

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

An important design step is to carry out a sensitivity analysis. This will consider factors such as the growth rate, E80 per vehicle or E80 per axle, initial E80 per day and structural design period. 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. By analysing minimum and maximum scenarios, the range in design E80s can be obtained.

5.4 Pavement structure

5.4.1 Selection from catalogue

The catalogue presented in Table 5-6 provides a range of structures appropriate to carry the relevant design E80s, but does not exclude other possible pavement 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, maintenance capabilities, construction skills and established procedures.

5.4.2 Testing the in situ strength of existing gravel road

The existing gravel road strength should be utilised in the upgrading of an existing gravel road to a paved road. This is done by using the existing pavement as part 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 a structure which is probably stronger 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 be 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. 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 evenness). Further examples are given in Figure 5-1.

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; see Figure 5-2), 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.

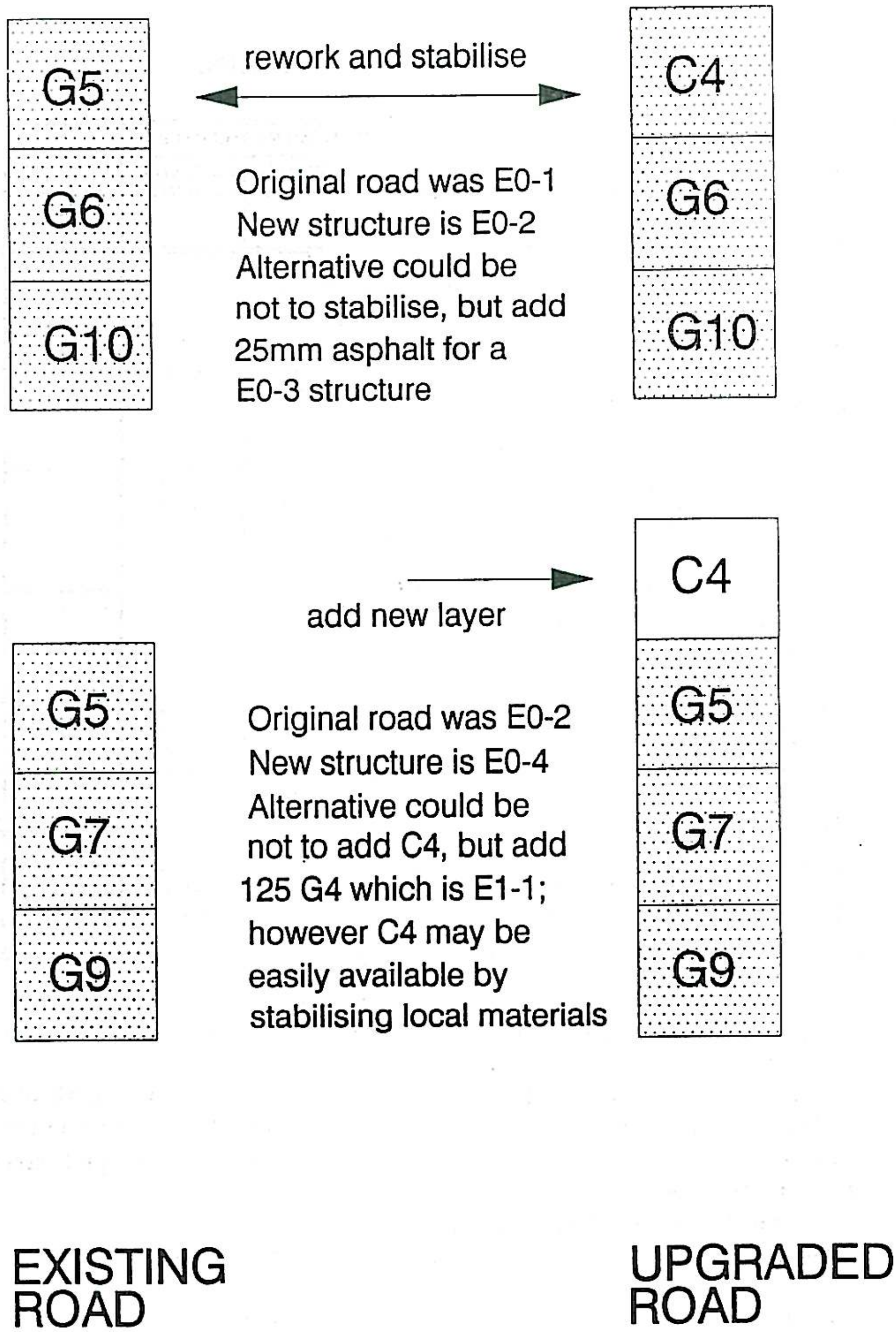

Figure 5-1 EXAMPLES OF USE OF EXISTING GRAVEL ROAD

Table 5-6: CATALOGUE OF PAVEMENT STRUCTURES.

TRAFFIC CLASS	TRAFFIC (E80's)	PROPOSED PAVEMENT STRUCTURES						
		GRANULAR/GRANULAR		GRANULAR/ CEMENTED	CEMENTED/ GRANULAR	CEMENTED/ CEMENTED	EMULSION TREATED BASE§	LOW MAINTENANCE
		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	$	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.

\# 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.

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

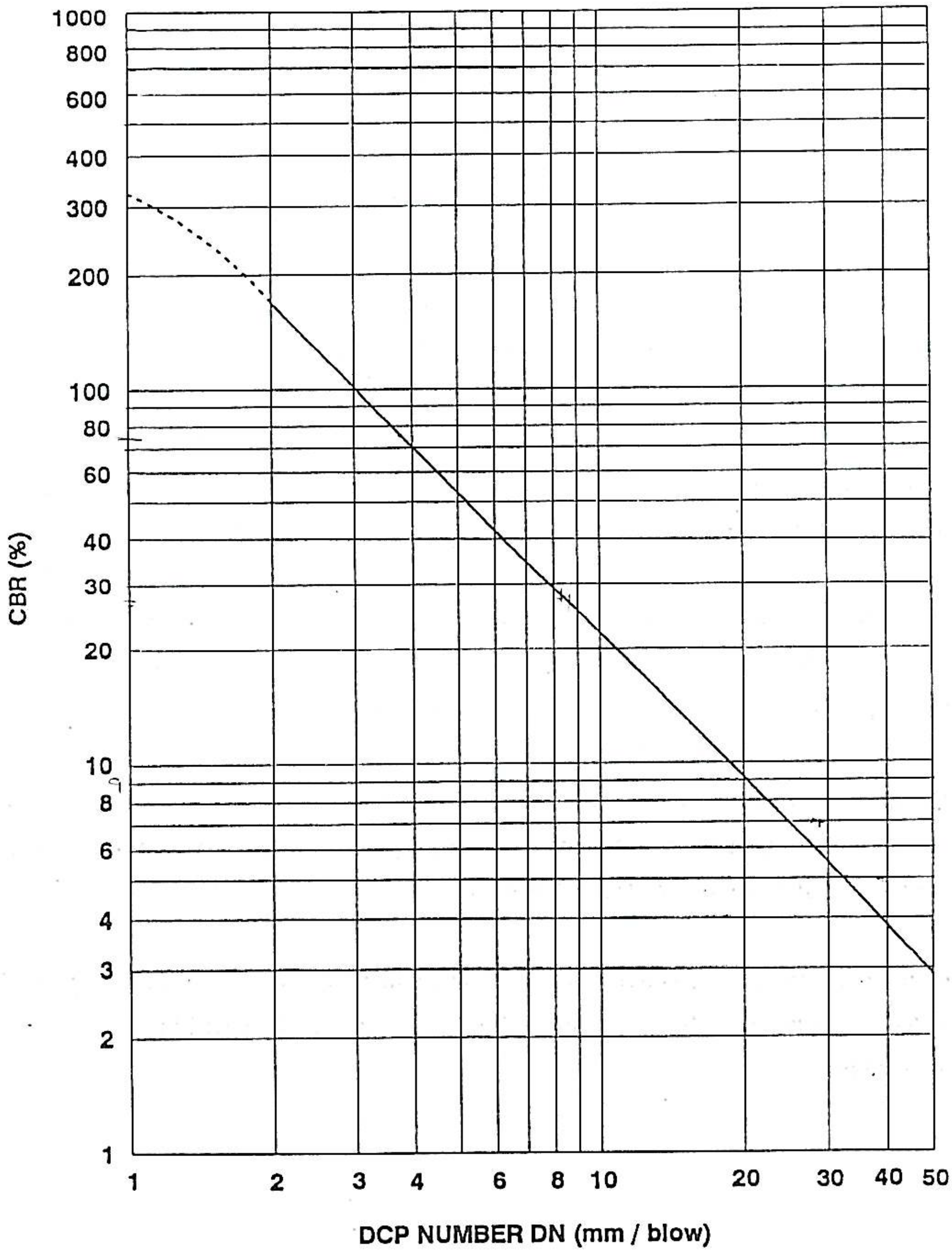

Figure 5-2: THE RELATIONSHIP BETWEEN DCP NUMBER DN AND CBR (KLEYN,1984)

The materials in the catalogue are classified by their soaked bearing strength (as is done in TRH 14), and the existing pavement materials need to be classified in terms of their soaked CBR to be related to the catalogue. However, 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). The DCP-CBR therefore needs to be adjusted to soaked CBR.

The preferable method of determining the soaked CBR 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 5-7 for roads which are presently gravel), and cross-checked with the laboratory 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.

In order to ensure the 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.

PROCEDURE FOR 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.

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.

STEP 2 DIVIDE ROAD INTO UNIFORM SECTIONS FOR REHABILITATION

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 5-3. 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 5-2. 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 . . (5.3)

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 (5.3)
= 127,9 - .9 * 15,7 = 114

Note that equation 5.3 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 5.3. This is then drawn on a layer strength diagram (such as Figure 5-3 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.

<u>STEP 4</u> <u>ESTIMATE DESIGN TRAFFIC</u>

The design traffic over the structural period is estimated in accordance with the procedures given earlier in this section. It is then classified as follows:

light $< 0,2 \times 10^6$ E80s per lane
medium $0,2 - 0,8 \times 10^6$ E80s per lane

STEP 5 COMPARE THE REPRESENTATIVE DCP WITH THE DESIGN DCP

The representative design DCP curve for each section is plotted on a copy of Figure 5-3, 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 5-3 with its clear overlay Figure 5-3A; 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 5-4. The design DCP curve has been calculated in accordance with steps 1 to 5, and has been plotted onto a copy of Figure 5-3. Assuming that the design traffic has classified as "light" according to STEP 4, then it can be seen from Figure 5-4 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 5-5 shows this. In Figure 5-5, the clear plastic overlay (Figure 5-3A) has been laid on top of Figure 5-4, 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.

5.5 Pavement materials

p. 5.19.

The selection of materials for the 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 TOWARDS GUIDELINES FOR LOW VOLUME ROADS and TRH 14 are the basic source documents defining these. Materials are classified here on the same basis as TRH14 with a number of categories ranging from G1 to G10. There is some relaxation of the classification permitted, but the main change has been the structural design for each pavement layer. In the catalogue for example, the use of G5 or G6 material is permitted in the basecourse for certain design traffic volumes, rather than the G4 material which TRH 4 requires. Figure 5-6 defines the material symbols used in the Catalogue.

5.5.1 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. 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).

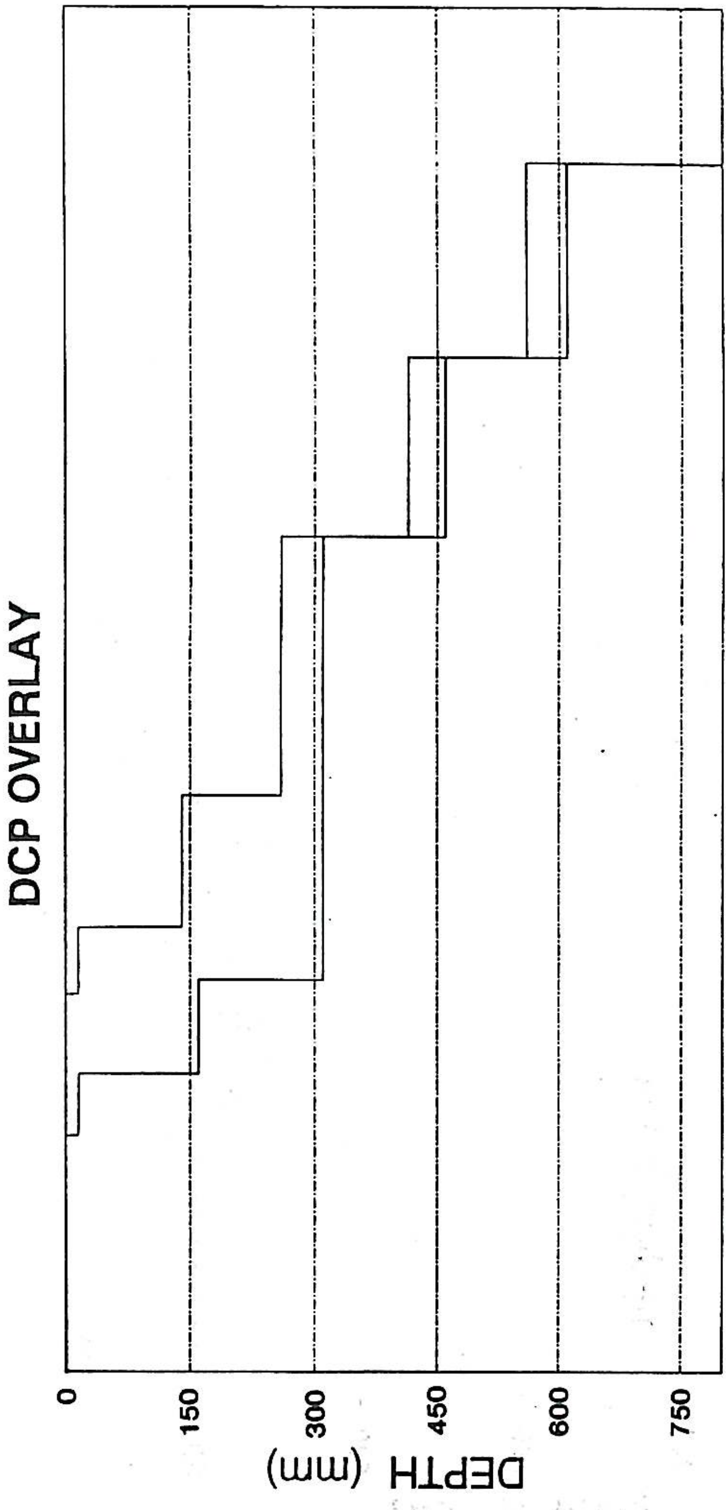

Figure 5-3A: DCP OVERLAY

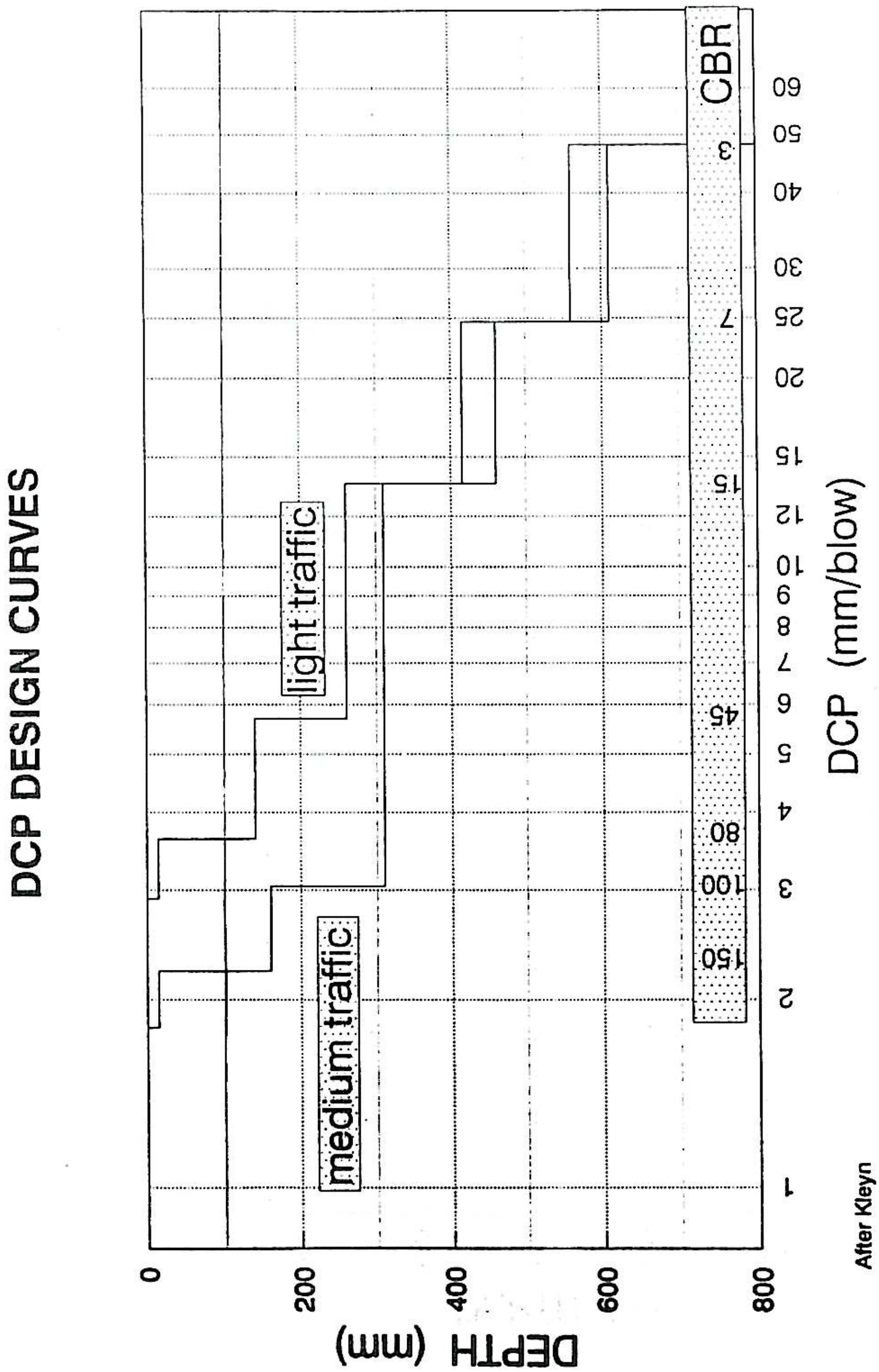

Figure 5-3: DCP DESIGN CURVES

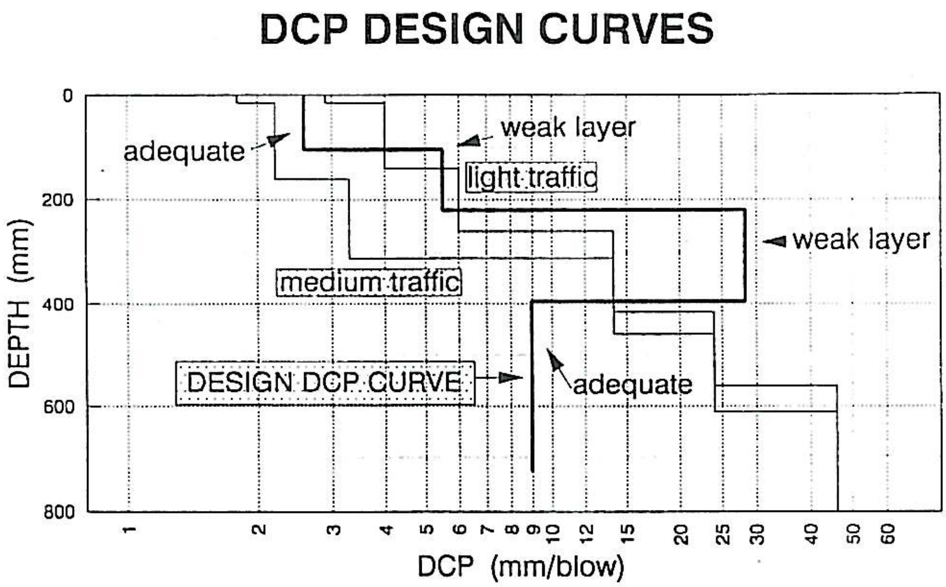

Figure 5-4: MEASURED DATA AND DESIGN CURVES

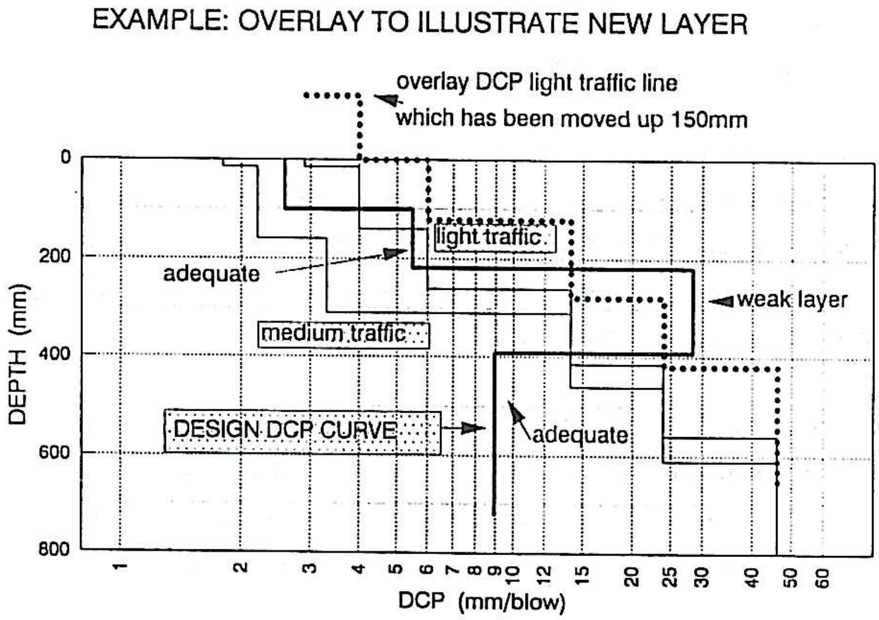

Figure 5-5: ILLUSTRATION OF THE EFFECT OF A NEW LAYER

5.5 Pavement materials

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- 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. Accordingly, where relaxation of the TRH 14 material requirements is allowed here it is on condition that the drainage and moisture regime are suitable, but the overriding factor to consider is strength and long term strength (durability).

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 n.m 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 5-3 MATERIALS CLASSIFICATION

Relaxation of standards should also 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 substantial failures. If the local maintenance capability is poor, it is thus recommended that less moisture sensitive materials or as high a quality material as is locally available should be used. In areas with a very high maintenance capability, larger relaxations are possible.

5.5.2 Subgrade

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.

<u>Soft soils</u>

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.

<u>Expansive clays</u>

Expansive materials are most easily identified from their plasticity indices and clay-sized component using the van der Merwe plasticity chart or the Weston method. 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.

Possible strategies:

Replace active clay to a depth of 600mm

Use of geotextiles

Increase overburden (pavement thickness)

Chemical treatment (lime)

Accept differential movement and cracking (provision can be made for modified binder or geotextile surfacings)

Dispersive soils

Dispersive soils are typically fine silty clays which contain a high percentage of exchangeable sodium or, less frequently, lithium. Any field evidence of excessive erosion, channelling or tunnelling should arouse suspicion and warrant additional testing or specialist advice. They can be identified by the Emmerson crumb test, in which a dry crumb is placed in a beaker of distilled water without agitation or stirring. If the water remains clear, the soil is flocculated; if it becomes turbid, the soil is dispersive.

Possible strategies

Minimise moisture movement

Remove material to depth of 600mm

Accept risks and make provision for maintenance

* Risk
- calculated
- acceptance
- sharing
- quantification

Collapsible soils

The collapse potential of a soil can usually be reduced by high energy impact 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.

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.

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 additional sub-surface drains where necessary in future.

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.

Roadbed preparation (on any new sections)

In order to optimise the pavement design and construction procedure, the roadbed on any new sections 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.

5.5.3 Fill and selected layers

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 raises 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.

5.5.4 Pavement layers - untreated materials

The catalogue takes into account the difference in bearing strength between field and laboratory conditions. 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.

MATERIAL PROPERTIES :

(a) **Grading**

The particle size distributions of the materials should follow TRH 14.

(b) **Atterberg limits**

The Atterberg limits should follow TRH 14, except that 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 and the materials have low moisture sensitivity and the pavement is well drained (Figure 5-7 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 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

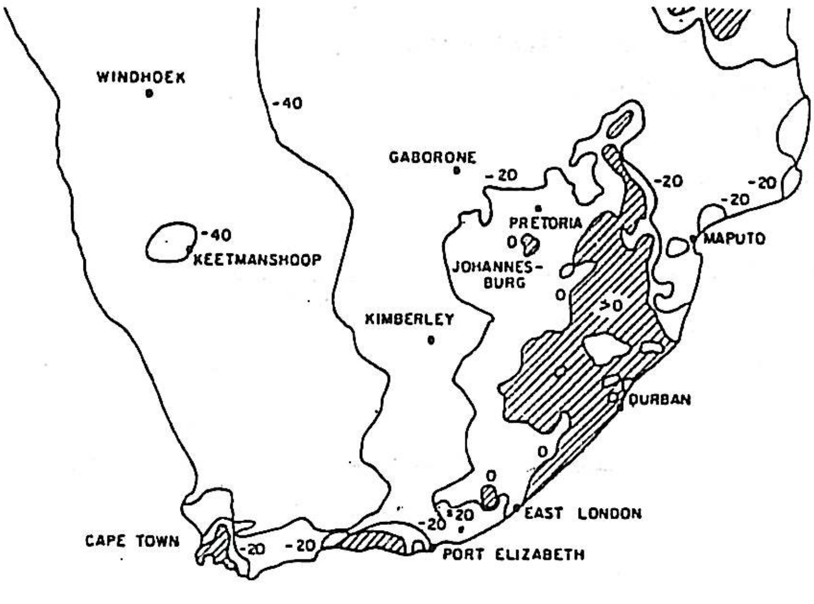


Figure 5-7 **MOISTURE REGIONS OF SOUTHERN AFRICA ACCORDING TO THORNTHWAITE**

aggregate strength

(c) **Crushing strength**

The crushing strength should follow TRH 14.

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

The Flakiness Index should follow TRH 14. 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**

The bearing strength and swell should follow TRH 14.

(f) **Field compaction**

The normal field compaction requirements for untreated layered materials apply. It is emphasized 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.

(g) **Deleterious minerals (G3,G4,G5,G6,G7)**

The limits should follow TRH 14.

(i). – Durability

5.5.5 Pavement materials - bound layers

If no suitable materials are available locally for base or subbase layers, modification or 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 following density and strength requirements: UCS (100% Mod AASHTO density) = 750 kPa to 1 500 kPa.

UCS (97% Mod AASHTO density) = 500 kPa to 1 000 kPa.

All the material requirements should follow TRH 14. However durability of stabilised materials has become an important issue since TRH 14 was published and is addressed in section 5.5.7. Note that durability is rarely a problem in the layers beneath the basecourse.

5.5.6 Durability - unbound materials

The durability of an unbound material can be tested by any of several tests, and either the Durability Mill test or the modified Aggregate Impact value (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). The recommended limits and specification are given in Table 5-8. 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 tests should not exceed 15.

grading
atterberg limits
design strength
durability

5.5.7 Durability - bound materials

Durability is an important issue for lime or cement stabilised layers. On existing low volume roads, many bound layers have carbonated (in which some or most of the cementation strength is lost due to interaction with CO_2 in the atmosphere and the soil), and thus have little residual stabilisation. 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. This has been found to explain some of the previously unexplained failures of roads with stabilised layers.

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.

UCS

cracking?

The test limits are shown in Table 5-9

Table 5-8: **MATERIAL RELATED DURABILITY LIMITS FOR UNBOUND MATERIALS**

<table>
<tr><th rowspan="2">Material type</th><th colspan="2">Modified AIV</th><th colspan="2">Durability Mill Test</th></tr>
<tr><th>dry</th><th>wet/dry ratio</th><th>Maximum fineness product</th><th>Maximum P425</th></tr>
<tr><td>Basic crystalline (dolerite, norite)</td><td rowspan="3">< 39</td><td rowspan="4">< 114%</td><td>125</td><td rowspan="7">all 35</td></tr>
<tr><td>Acid crystalline (granite, gneiss)</td><td>420</td></tr>
<tr><td>High silica[a] (chert, quartzite)</td><td>420</td></tr>
<tr><td>Arenaceous (sandstone)</td><td>< 31</td><td rowspan="4">all 125</td></tr>
<tr><td>Argillaceous (mudrock, shale)</td><td>< 24</td><td>(wet < 26)</td></tr>
<tr><td>Carbonates[b] (dolomite limestone)</td><td>< 39</td><td>< 114%</td></tr>
<tr><td>Diamictites (tillite)</td><td>< 22</td><td>< 115%</td></tr>
<tr><td>Metalliferous[b] (magnesite, magnetite)</td><td colspan="4">not required</td></tr>
<tr><td>Pedogenic[c] (calcrete, ferricrete, laterite)</td><td>≤ 39</td><td>≤ 120% for calcrete
≤ 114% for silcrete</td><td>480</td><td>55</td></tr>
</table>

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

Table 5-9: 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 to 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% stabiliser 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 resist the effects of carbonation. Due to practical constraints with mixing in of the stabiliser minimum limits are usually set at 2 to 2,5% of stabiliser.

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*
	For C4 (100% Mod AASHTO density)		*UCS 750 kPa*
	Available material:		*CBR 65*
2.5% cement:	*uncarbonated:*		*UCS 1200 kPa*
	carbonated:		*UCS 800 kPa*
4% cement:	*uncarbonated:*		*UCS 1800 kPa*
	carbonated:		*UCS 1200 kPa*
ICC		*3%*	

Although the ICC is not satisfied, stabilisation with 2,5 % cement is considered acceptable, because the carbonated UCS value is greater than 750 kPa.

5.5.8 Pavement materials - soil chemicals / ionic stabilisers / sulphonated petroleum products / compaction aids

These chemicals can generally be classified as sulphonated petroleum products and they react with some 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. Note that this class of chemicals excludes lime and cement and the various dust palliatives.

Research work at CSIR has shown that the performance of these products is highly material dependent. 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 specialist 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 be tested at a specialist laboratory and meet the criteria of Table 5-10.

Table 5-10: REQUIREMENTS FOR PAVEMENT MATERIALS SUITABLE FOR TREATMENT BY SOIL CHEMICALS / COMPACTION AIDS

Pavement material property	Minimum value	
Plasticity Index	≥ 8[a]	
Percent passing 0,075 mm sieve	≥ 17	
Initial Cation Exchange Capacity[b]	≥15 me/100mg	
Reduction in Cation Exchange Capacity after treatment	at 15 me/100mg	≥ 66%
	at 50 me/100mg	≥ 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 accurately quantify 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).**

5.6 Materials for surface seals

Materials used for surface seals consist mainly of a bituminous binder and aggregate (sand and/or crushed stone). The choice of surfacing type is discussed in Chapter 6.

5.6.1 Prime

A prime is recommended for all roads which are to have a bitumen surface, unless good quality control and maintenance capabilities exist. It makes a significant difference in performance for the lighter surfacings such as sand seals and single seals which have only a single application of bitumen. The rule-of-thumb is to have at least two sprays of bitumen for a new surfacing, because nozzle blockages and other problems in construction make a single spray of bitumen susceptible to potholes and failures.

5.6.2 Bitumen

Conventional bitumen

Conventional bitumen, either as 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. Specifications for modified binders are under development. Crumb rubber and SBR are at this stage the most commonly used modifiers and are accepted by road authorities as being cost effective binders for quite a few applications.

Dust palliatives

Bituminous dust palliatives are amongst the lowest cost bitumen surfacings. They typically consist of a special 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 (< 3 years). However the maintenance costs rise steeply with time, and they require timeous retreatment or resealing. On temporary works such as deviations, or as a form of stage construction these materials may, however, have significant benefits over the gravel wearing course option.

5.6.3 Aggregate for bituminous surfacings

The aggregates and sands to be used in surfacing seals and slurries should follow TRH 14 and SABITA publications. For low volume roads, there is limited potential for the use of marginal surfacing materials in order to reduce the cost of bituminous surfacings, and this is discussed 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.

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 absolute minimum value for crushing strength should be a 10 % FACT of 120 kN (ACV 30), and if possible a higher value should be used.

Polished stone value

Polishing of stone in the surfacing lowers the low speed 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. This implies that traffic volumes less than 500 vpd could take ten times as long to polish stone to the same extent as 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.

6 CHOICE OF SURFACING TYPE

For low volume roads the surfacing is important for good performance of the road and this choice must be made carefully. **In some situations a better pavement design will result from using a stronger and/or thicker surfacing in conjunction with a poorer quality pavement structure.**

The choice of appropriate surfacing is made firstly on performance grounds, and then on cost grounds (this chapter has largely been taken from SABITA Manual 10).

6.1 Performance

The performance is determined by the environment, maintenance capability, and gradient. These are presented in Tables 6-1 to 6-3, and the restrictions on choice are progressive and sequential. Thus a restriction in any one table is sufficient to limit the choice of surfacing.

Environment

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 6-1: CHOICE OF BITUMINOUS SURFACING FOR RURAL ROADS BY ENVIRONMENT

ENVIRONMENT	SURFACING RECOMMENDATION
First world - high pavement standards[a]	Any
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

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 c Mountainous, Thornthwaite's climatic moisture index Im ≥ 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.

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 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.

Table 6-2: 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

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.

Gradient

Gradient limits are important to limit the damage caused by water running along the surfacing (parallel to the centreline), as opposed to water running off the surfacing. Water damage is a problem with steep gradients particularly for roads with kerbs such as are found in hilly areas. The 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.

Gradient limits also apply to limit damage caused by shoving. Shoving occurs when 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 on small radius curves carrying many heavy vehicles. A basecourse with a thin layer of fines at the top may lead to shoving.

Table 6-3: 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 6-4. These only need to be met at the intersections, and not along the whole road. Where a light seal has been used on the road, it is common just to reinforce the intersection by adding something like a thin slurry to turn the seal into a Cape Seal.

Table 6-4: CHOICE OF SURFACING FOR LOW VOLUME RURAL ROADS FOR INTERSECTIONS AND OTHER AREAS SUBJECT TO TURNING VEHICLES

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

Note a: geotextile reinforced surfacings not recommended at intersections.

6.2 Choice of most cost-effective surfacing

Once the appropriate surfacing types have been selected from the performance viewpoint, their lifecycle 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 lifecycle cost and compare its construction cost and life with the other appropriate surfacings which are close to it in terms of lifecycle cost.

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 6-5 and 6-6. Once the likely surfacing life and the construction cost of any particular surfacing have been determined, the lifecycle cost can be found from Figure 6-1.

Table 6-5: SURFACING CONSTRUCTION COST (1991 VALUES)

SURFACING	Cost R/m² 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.

In Figure 6-1, the lifecycle 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 lifecycle cost given in Figure 6-1 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. It is discounted back to present value at a rate of 8% in real terms.

Table 6-6: EXPECTED SURFACING LIVES FOR RURAL LOW VOLUME ROADS

SURFACING	EXPECTED LIFE[c]	
	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
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. geotextile reinforced surfacings should have similar lives

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

It is assumed that 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 6-2). A check on gradient (Table 6-3), eliminates the thick slurry because of the stabilised basecourse.

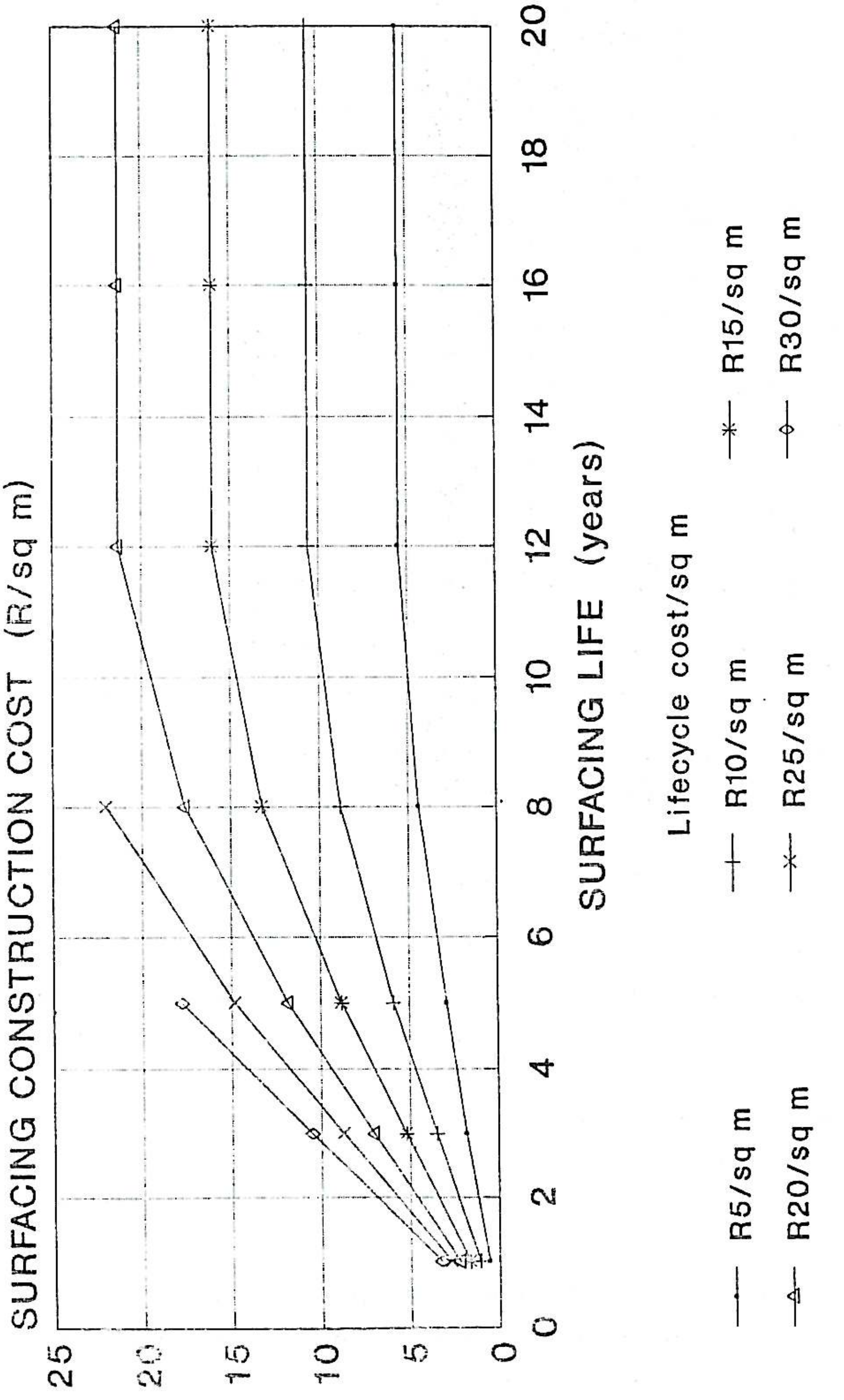


Figure 6-1: LIFE CYCLE COSTS FOR SURFACINGS DISCOUNTED AT 8%

Step 2 Choose most cost effective surfacing

<u>Determine surfacing life</u>

No historical information is available about surfacing lives. It is almost a third world environment, normal pavement, reasonable quality control during construction, and although the drainage is adequate at construction any drains are expected to block up soon. For Table 6-6, it is assessed as poor conditions. The surfacing life is: asphalt 14 years, Cape Seal and double seal 7 years (which is much less than you would expect from the same surfacings used in good conditions)

<u>Determine surfacing costs</u>

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

<u>Find comparative lifecycle cost</u>

From Figure 6-1:

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

The double seal has the lowest lifecycle cost. Its construction cost is also the lowest. If the road authority can accept a 7 year life before reseal, then choose this.

Decision: double seal

7 CONSTRUCTION, MAINTENANCE AND ENVIRONMENT

7.1 Introduction

The issues of construction, maintenance and environment for low volume roads have been well covered in the TOWARDS APPROPRIATE STANDARDS FOR RURAL ROADS discussion document. In upgrading gravel roads, these same issues apply and this chapter is intended to highlight the most important of those issues.

7.2 Construction

Standards for roads are very well established and detailed specifications have been drawn up to cover all aspects of their construction. Extensive analysis of potential areas of cost savings in upgrading gravel roads has shown that there is almost nothing to gain by a reduction in construction standards, and a lot to lose. The areas of potential cost savings lie in geometrics (especially) and the optimum use of the existing pavement structure, not in construction.

7.2.1 Gravel roads

Because a gravel road has the potential for eventually becoming a paved road, the initial construction standard should 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).

Compaction is a relatively inexpensive operation which can have important consequences for the performance of a road. Not only is it of benefit when the road is upgraded to a paved standard, but it also can:

- 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.

It is very important that the surface be properly watered before and during compaction to ensure a tightly bound surface is produced (TRH 20). This bound surface or "crust" has an important effect on gravel loss and riding quality.

7.2.2 Paved roads

Recommended standards for paved roads have been published in the many Standard Specifications (such as CSRA, 1987 or those of the road authorities) and one should try to achieve these construction standards even on rural roads carrying low volumes of traffic. Some of these standards possibly can be reduced if they are difficult to achieve but this step should only be taken by experienced people who can make a sound estimate of the standards which are of low risk in particular circumstances.

Compaction is one construction operation for which standards must not be reduced and a case can even be made for increasing the minimum densities accepted in the current standard specifications on lighter structures. 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 98 per cent of Mod. AASHTO density, and the subbase to as high a density as possible, with a minimum of 95 per cent.

If the gravels are stabilised with cement or lime, rutting is not a problem and a minimum density of 97 per cent of Mod. AASHTO is appropriate, particularly when increased densities are related to increased shrinkage.

7.2.3 Substitution of labour for plant

Under certain conditions construction operations may be successfully carried out by manual methods rather than by traditional plant methods but the balance will be influenced by minimum wage rates, productivity and the type of operation.

If manual methods are used, the product is inherently more variable and a greater degree of supervision is required to ensure that the quality is comparable to that obtained using plant.

When 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 save costs without reducing the performance of the road?
- Is labour available and willing to carry out the operations?

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. Standards for projects where labour is competing with plant should be the same irrespective of the construction system which is chosen. Some general comments about labour-based construction are included in a synthesis document of the Department of Transport, and particular comments about the use of manual methods in the construction of bitumen surfacings are included in SABITA manuals (referred to in the TOWARDS APPROPRIATE STANDARDS FOR RURAL ROADS [Department of Transport 1992]).

7.3 Maintenance

7.3.1 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.

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. The use of selective maintenance of certain links or even sections of a link is often appropriate and economically justifiable.

7.3.2 Maintenance of paved 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 designs an therefore the maintenace of surfacing and drainage are crucial. For example G6 materials are used as the basecourse in some of the designs in this document; this is a more moisture sensitive material than say the G2 materials or cemented bases which are found in some stronger 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 possible. 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. The implications of a rapidly expanding paved road network on future budgets should also be carefully evaluated, and appropriate provision for funds be made.

7.4 Environment

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. It will become increasingly advisable that before any existing roads are upgraded, the relevant authorities determine the impact of the roadway improvements on both the bio-physical and socio-economic environments. This is especially true in environmentally sensitive areas such as St Lucia.

The Integrated Environmental Management (IEM) 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 process is intended to guide, rather than impede, the development process by providing a positive interactive approach to gathering and analysing useful data.

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, one 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. 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.

- When the issues have been identified, the project can be classified as having no negative impacts (i.e. 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, one should be constantly aware of impacts that might arise during the course of the development, for example, clearing of vegetation, extraction from proposed material sources, noise from construction equipment and placement of temporary accommodation for construction crews.

If the impacts are uncertain or significant, a preliminary or full assessment should be undertaken as described in TOWARDS APPROPRIATE STANDARDS FOR RURAL ROADS.

8 WARRANTS FOR SURFACING AN EXISTING GRAVEL ROAD

8.1 Introduction

The decision to surface or pave an existing gravel road is usually made when the traffic volume exceeds a certain level, or some other factor dictates surfacing. This chapter provides the warrants for surfacing as a function mainly of traffic volume. Considerable work has been done by SABITA on warrants for surfacing an existing gravel road and this chapter has been derived from that.

The basis for the warrants for surfacing can be at one of several levels of increasing complexity:

FINANCIAL	is it less expensive to the road authority to pave the road OR keep the road as a gravel road and pay the high costs of maintenance, i.e. grading and regravelling
ECONOMIC	is it less expensive to the community to pave the road OR keep the road as a gravel road and pay the relatively higher costs of maintenance, vehicle operating costs, safety and productivity
SOCIO-POLITICAL	is the road to be paved to improve the quality of life (factors such as dust, mud, all-weather access) or to create employment opportunities or development spin-offs

Guidance for the correct level to choose is given below. Generally the more factors one considers, the lower the traffic volume at which the surfacing becomes justified. The reduced levels of funding for road construction likely to prevail in South Africa in the 1990s mean that the warrants for surfacing must become more stringent. Therefore the number of factors that can be considered in the analysis will be reduced in comparison to cost-benefit studies done in the early 1990s. The trend is for the road user to bear a proportion of the costs themselves, and so the concept of "partial economic analysis" is introduced which does not take all the economic costs into account in the analysis. Socio-political decisions cannot yet be quantified in such analyses, and the best advice is to make the decision to surface an existing gravel road on the basis of the warrants presented here, plus a good understanding of the locally important issues.

The difference between full and partial economic analysis is time cost. Time costs (or the time savings which can be achieved by the provision of a surfaced road compared to a gravel road) are not included in this economic analyses, and are borne by the road user. The choice of which analysis type to use is made using Table 8-1.

Table 8-1 ANALYSIS TO BE USED FOR ROAD SURFACING DECISION

Road / project	Appropriate basis for decision
Rural - donor funded : World Bank, Development Bank of Southern Africa, African Development Bank, etc.	Full economic analysis, including VOC, time and accident costs; and including construction and maintenance costs. NOT PROVIDED FOR IN THIS CHAPTER.
Rural - central, provincial or local government funded; Parks, forestry; Socio-political decision to upgrade.	**Partial economic analysis**, including VOC and accident costs, but not time costs; and including construction and maintenance costs.
Private roads Military	**Financial analysis**, including construction and maintenance costs only.

8.2 Cost-benefit analysis

The warrants for surfacing an existing gravel road were found in the SABITA studies by performing extensive cost-benefit analyses for a wide range of conditions and factors including construction costs, size of project, location of project, maintenance, economic region, traffic growth rate, terrain, and discount rate; these were used to provide input data to the new analysis in this document. A check was made for each calculation to ensure that the first year rate of return was greater than the discount rate at the breakeven traffic.

The costs (and thus benefits) were calculated as life cycle costs in real terms over a standard 10 year analysis period. Salvage value was included to ensure the results were valid for longer analysis periods. The costs were discounted at a real rate of 15% to give present worth of costs (PWOC). This discount rate is higher than that used in earlier cost-benefit studies, and reflects the more stringent approach to surfacing decisions in 1992/93. The benefits were the savings in road maintenance cost, vehicle operating costs, and accident costs for the paved road compared to the unpaved road.

For the **financial analyses** the following costs were considered:

construction costs	maintenance costs

For the **partial economic analyses** the following costs were considered:

construction costs	maintenance costs
vehicle operating costs	accident costs

Three levels of construction cost were used to span the range of likely circumstances:

LOW — **no basecourse upgrade and cheap costs** (cheap: larger projects, or projects close to bitumen and stone/sand suppliers)

MEDIUM — **rip and recompact the existing wearing course and average costs** (average: this could be a large sized project in a remote location with 300km haul for bitumen, or a medium project in a rural location with 150km haul for bitumen, or a small sized project in a rural location with cheap prices)

HIGH — **import new 150mm basecourse and high costs** (high: this could be a medium sized project in a remote location with 300km haul for bitumen plus high prices)

The construction costs used here were the cost of surfacing and basecourse preparation needed for the surfacing (and including all relevant costs of P & Gs, camp, travel, engineer's fees, profit, contingency, etc). Since this document applies to existing gravel roads, no construction cost had to be considered for providing the gravel road. No allowance was made for any costs associated with geometric changes or drainage improvements which might be done at the time of surfacing.

The traffic growth rate was assumed to be 3% per year over the period, except that the first year after surfacing had a traffic growth rate of 25% (the common "jump-up" after sealing). The analysis here is only accurate between 50 and 500 v.p.d., which is the normal traffic range to consider surfacing. In the Figures, traffic volumes above 500 are shown with an arrow, and traffic volumes below 50 are shown as 50 v.p.d. The terrain used was "rolling". In mountainous areas, the traffic to justify surfacing will be slightly less than shown here. In flat areas, it will be slightly higher.

The warrants presented here are simplified, and cost-benefit analysis can be performed in more detail for individual projects using the SURF computer programme from SABITA, or CB-Roads from

Department of Transport. This would be desirable for the larger projects.

8.3 Traffic volume to justify a bitumen surfacing

The traffic volume to justify a bitumen surfacing is given in the following figures, which are for **partial economic analysis** with no time costs (Figure 8-1), and for **financial analysis** (Figure 8-2). Note that these figures indicate when a surfacing type is **not** justified.

WORKED 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. Is a surfacing justified?

STEP A

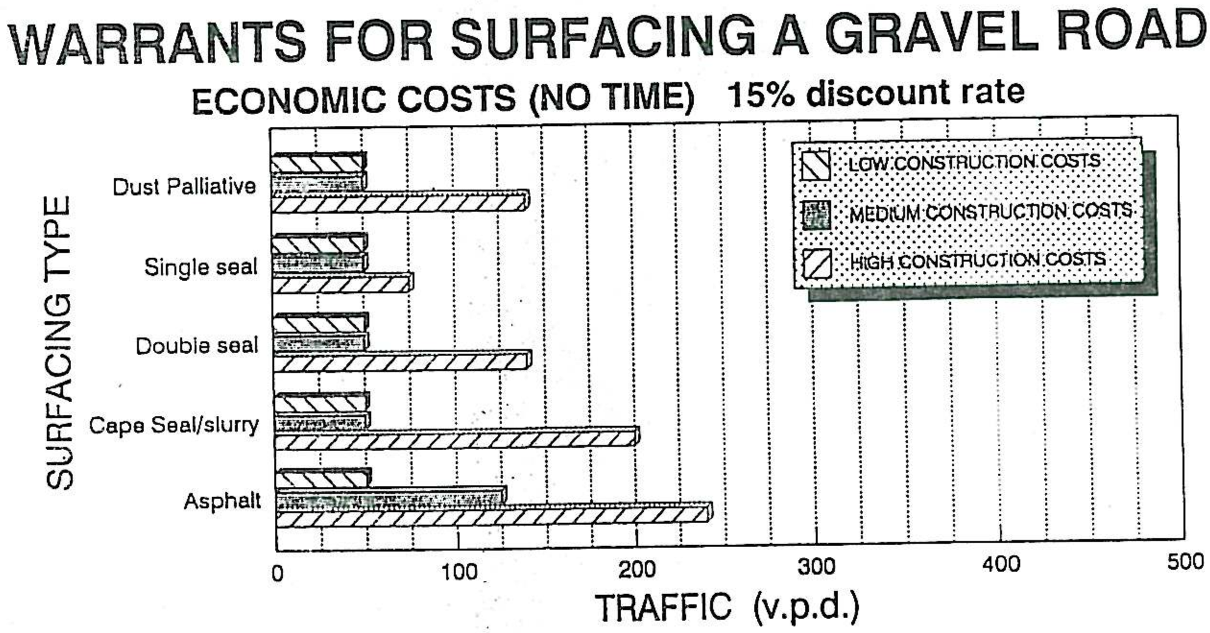
A1 Choose analysis method

This is a rural road to a township with socio-political overtones. From Table 8-1, choose partial-economic analysis.

A2 Is bitumen justified?

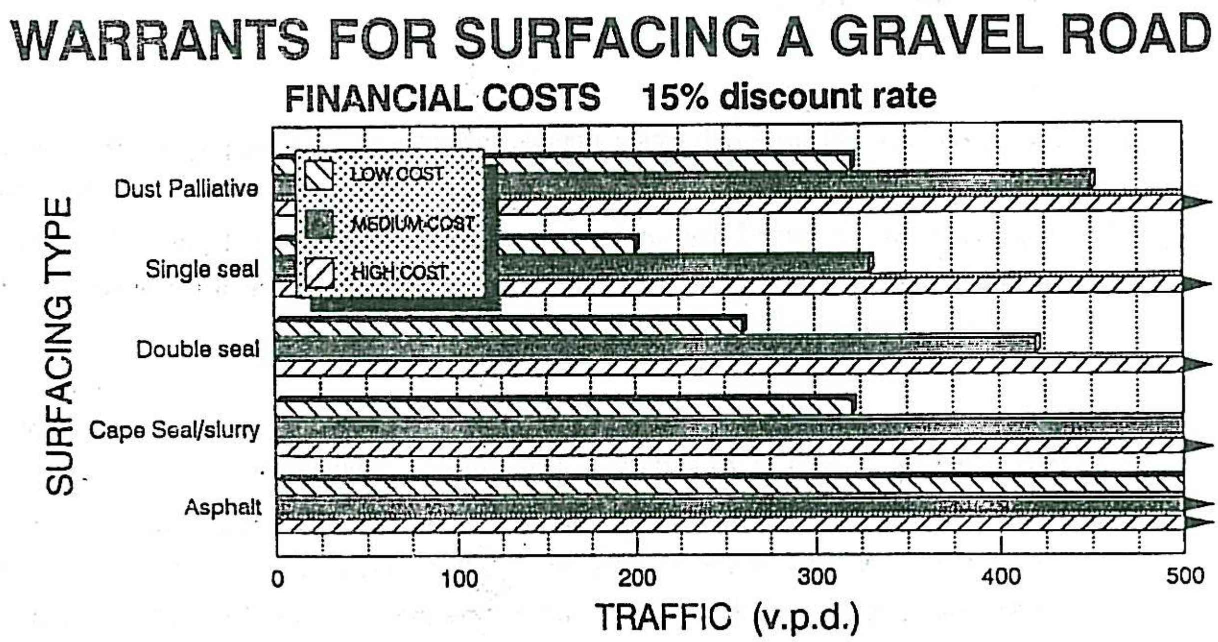
The basecourse will need only some ripping, recompacting and shaping before surfacing. Construction costs are medium because the work is in a rural area 150km from major suppliers. From Section 8.2, the construction cost is "average". From Figure 8-1, clearly any bitumen surfacing is justified at this traffic level, and the selection of surfacing type can then be made in accordance with chapter 6.

Note that if the analysis had been on a financial basis rather than a partial economic analysis, then at 300 v.p.d., none of the surfacings were justified, although the single seal is close and perhaps in a year or two as the traffic rises, the surfacing would be justified. Alternatively if there is local pressure to surface the road, then this could be enough to tip the decision to surface with a single seal. If there was a lot of local pressure, then the decision should have been made on a partial economic basis anyway (Table 8-1), and not on a financial basis.



Low: no basecourse upgrade, cheap prices
Medium: rip/recompact base; medium prices
High: add new base; high prices

Figure 8-1 ECONOMIC COSTS



Low: no basecourse upgrade, cheap prices
Medium: rip/recompact base; medium prices
High: add new base; high prices

Exceeds 500 v.p.d.

Figure 8-2 FINANCIAL COSTS

9 SELECTED REFERENCES

9.1 Main reference

This document is closely based on the following document, and it should be considered as the first source of reference for any enquiry:

DEPARTMENT OF TRANSPORT (1992). *TOWARDS APPROPRIATE STANDARDS FOR RURAL ROADS: Discussion document.* PROJECT REPORT PR 92/466/001, PRETORIA.

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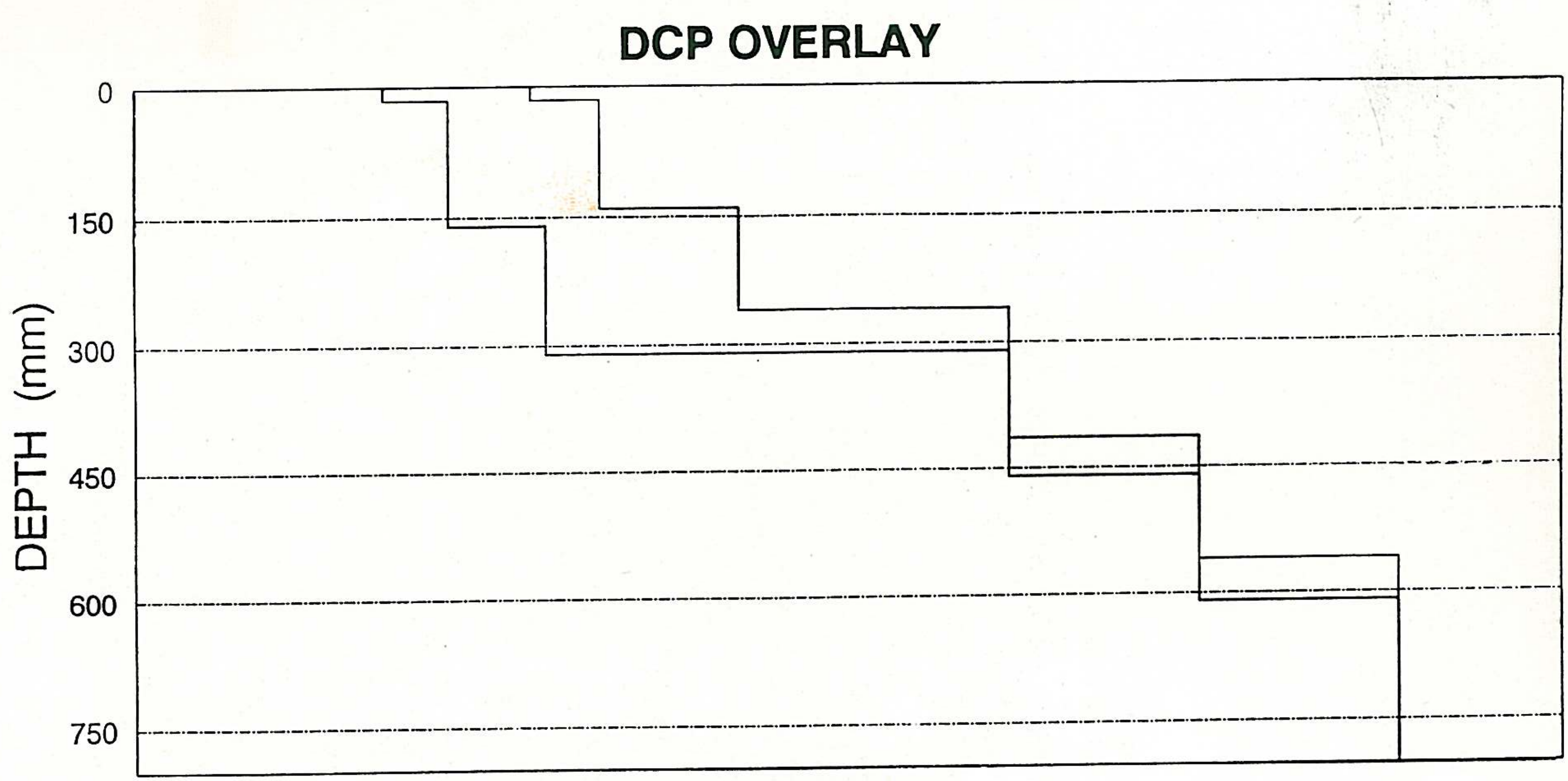

Figure 5-3A: DCP OVERLAY