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STRUCTURAL DESIGN OF URBAN ROADS

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PREFACE

URBAN TRANSPORT GUIDELINES (UTG) is a series of documents written for practising transportation engineers which describes current recommended practice in selected aspects of urban transportation. They are based on South African experience and research and have the full support and approval of the Committee of Urban Transport Authorities.

To confirm their validity in practice, UTGs are circulated in draft form for a two-year period before receiving the final approval of CUTA. During this period, suggestions for improvement may be sent to:

The Secretary
Committee of Urban Transport Authorities
c/o Division of Roads and Transport Technology
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After final approval by CUTA, the revised document will be issued as a full UTG in both official languages.

SYNOPSIS

This document deals with the structural design of paved and gravel urban roads. It covers the selection of design strategies, the estimation of design traffic, materials, environmental and practical considerations, structural design and cost analysis.

SINOPSIS

Hierdie dokument handel oor die struktuurontwerp van stedelike geplaveide en gruispaaie. Dit dek die keuse en ontwerpstrategieë, die skatting van ontwerpverkeer, materiale, praktiese en omgewingsoorwegings, struktuurontwerp en kosteontleding.

KEYWORDS

Structural design, urban roads, paved roads, gravel roads, catalogue of designs, design traffic materials, environment, cost analysis

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1 INTRODUCTION: THE SCOPE AND PHILOSOPHY OF THE REPORT

The procedures for the structural design of road pavements presented in this document are applicable to urban roads and surfaced residential roads in southern Africa. A special section on gravel roads has also been included. The procedures are based on a combination of existing methods, experience and fundamental theory on the behaviour of structures and materials. It must be kept in mind that the proposed procedures do not necessarily exclude other design methods.

It is important to attend to layout planning and drainage design before the structural design of a road is addressed. The structural design of a pavement is aimed at the protection of the subgrade through the provision of pavement layers. With rehabilitation a chosen level of service has to be achieved over the analysis period as cheaply as possible. Structural design encompasses factors such as time, traffic, pavement materials, subgrade soils, environmental conditions, construction details and economics. The procedures cover a range of pavement types and materials currently used in local practice.

1.1 GENERAL OBJECTIVE

When a road pavement reaches the end of its serviceable life, it is usually rehabilitated in some way to provide a further period of service. If the designer has to make a fair comparison of proposed new pavement designs, he requires a common basis, usually cost.

A comparison can be made by taking into account both the structural capacity of the initial pavement and estimates of the rehabilitation measures that will probably be necessary to maintain the pavement in a servicable condition over a realistic analysis period. The design philosophy may be summarized as follows:

> The aim of design is to produce a structurally balanced pavement which, at minimum present worth of cost, will carry the traffic for the structural design period in the prevailing environment at an acceptable service level without major structural distress. If necessary, the pavement should be capable of being strengthened by means of various rehabilitation measures to carry the traffic over the full analysis period.

1.2 THE DESIGN PROCESS

A flow diagram of the design process discussed in this document is shown in Figure 1. The flow diagram has eight sections. Each section will be treated separately but all sections have to be considered as a whole before a design can be produced.

The first five sections represent the basic inputs to pavement design, namely road category, design strategy, design traffic, materials available and environment. With these as inputs the design will use the catalogue of various pavement types to obtain possible pavement structures, to include future maintenance measures and some construction considerations, and eventually to compare the different pavement structures on the basis of cost.

A simplified design flow diagram for the structural design of residential roads (Category UC and UD) is suggested in Figure 2.

The catalogue is based on experience of pavement construction and pavement behaviour throughout southern Africa. Each design has been checked by means of mechanistic pavement design techniques.

The catalogue should be adequate to provide the basic design required. However, special conditions may require a more detailed analysis by means of other methods.

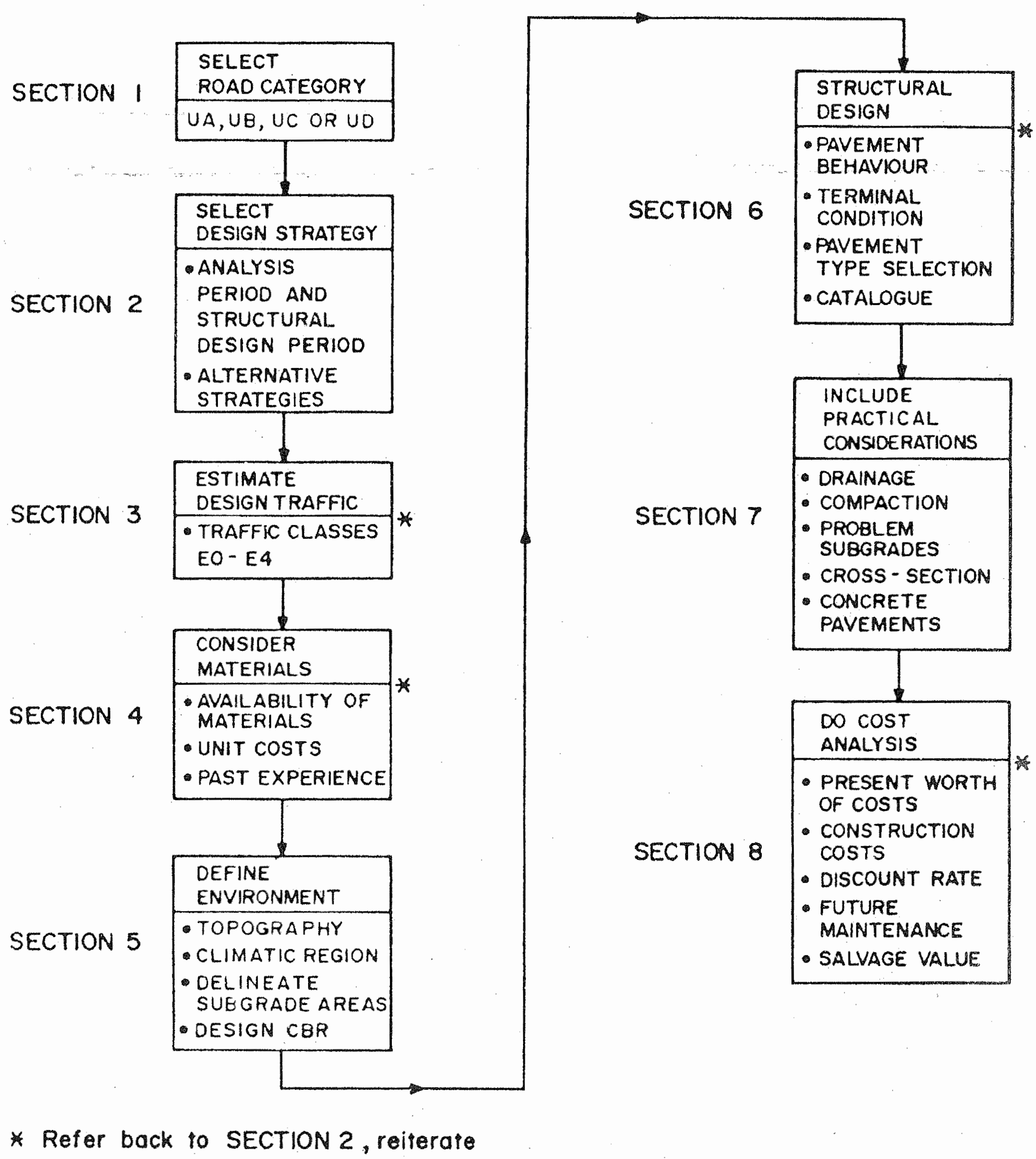


FIGURE 1

Structural design flow diagram
(Mainly for category UA and UB roads)

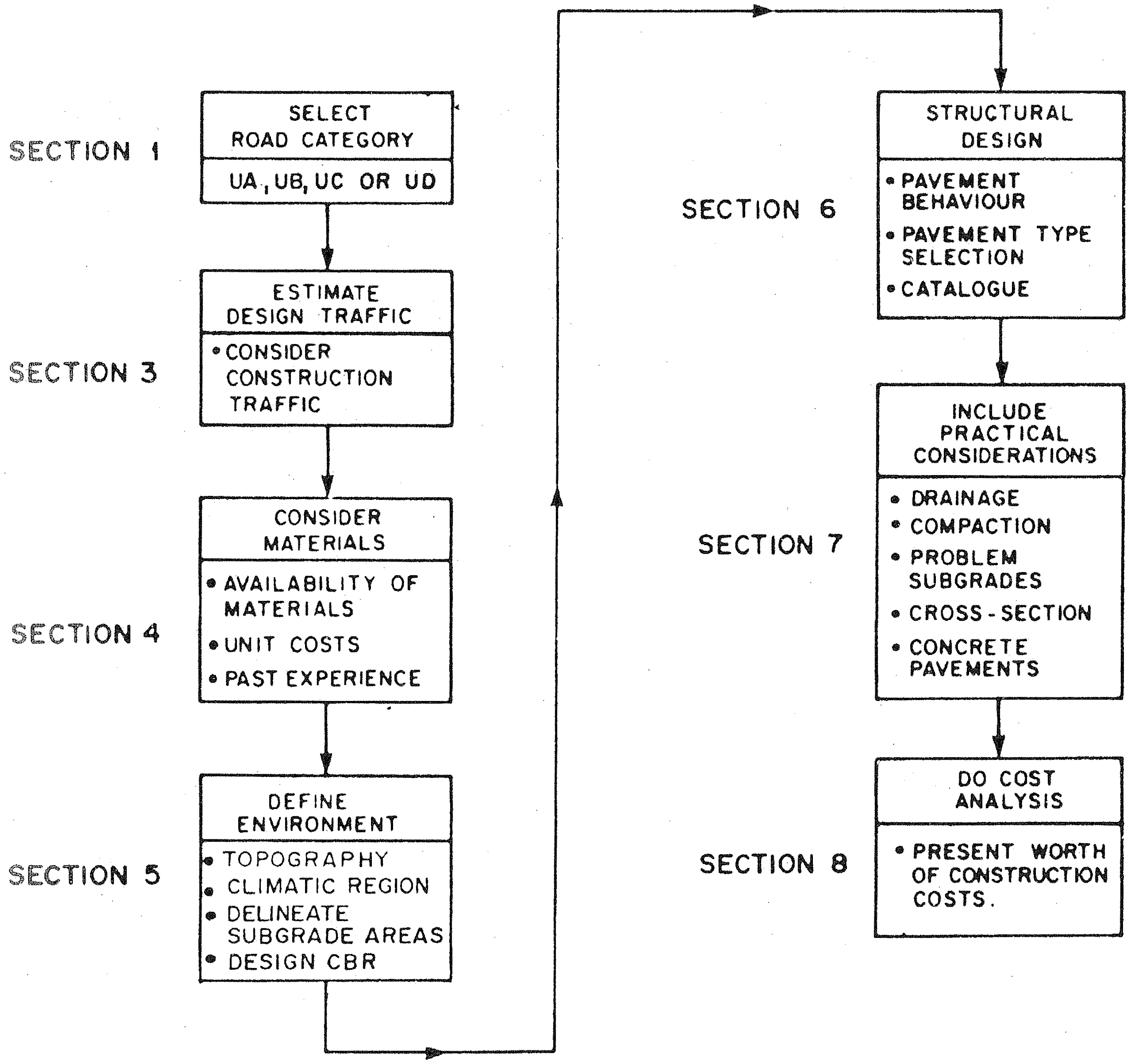


FIGURE 2

Simplified design flow diagram for residential roads (Category UC or UD)

2 ROAD CATEGORY

2.1 DEFINITION OF ROAD CATEGORIES

For the purpose of this document, four different road categories, namely UA, UB, UC and UD are considered (Figure 3). A road category is defined by a combination of parameters (Table 1) such as importance, service level, traffic and constructed standard. The four road categories mentioned above cover the range from every important arterial road with a very high level of service and very high volume of traffic, to less important, lightly trafficked residential roads and culs-de-sac with a moderate to low level of service. Figure 3 gives a schematic illustration of the different road categories.

2.2 IMPORTANCE, SERVICE LEVEL, TRAFFIC AND ROAD STANDARD

The designer should ascertain whether the traffic volume and other factors comply with Table 1 and whether they are acceptable to the controlling authority.

The level of service that a user expects from a road is related to the function of the road, to the general standard of the facility and partly to the volume of traffic carried. For example, the user will expect a better riding quality on a dual-carriageway arterial road than on a minor residential road. The design traffic is expressed in terms of the total number of equivalent standard axles (E80s) over the structural design period (refer to Section 4).

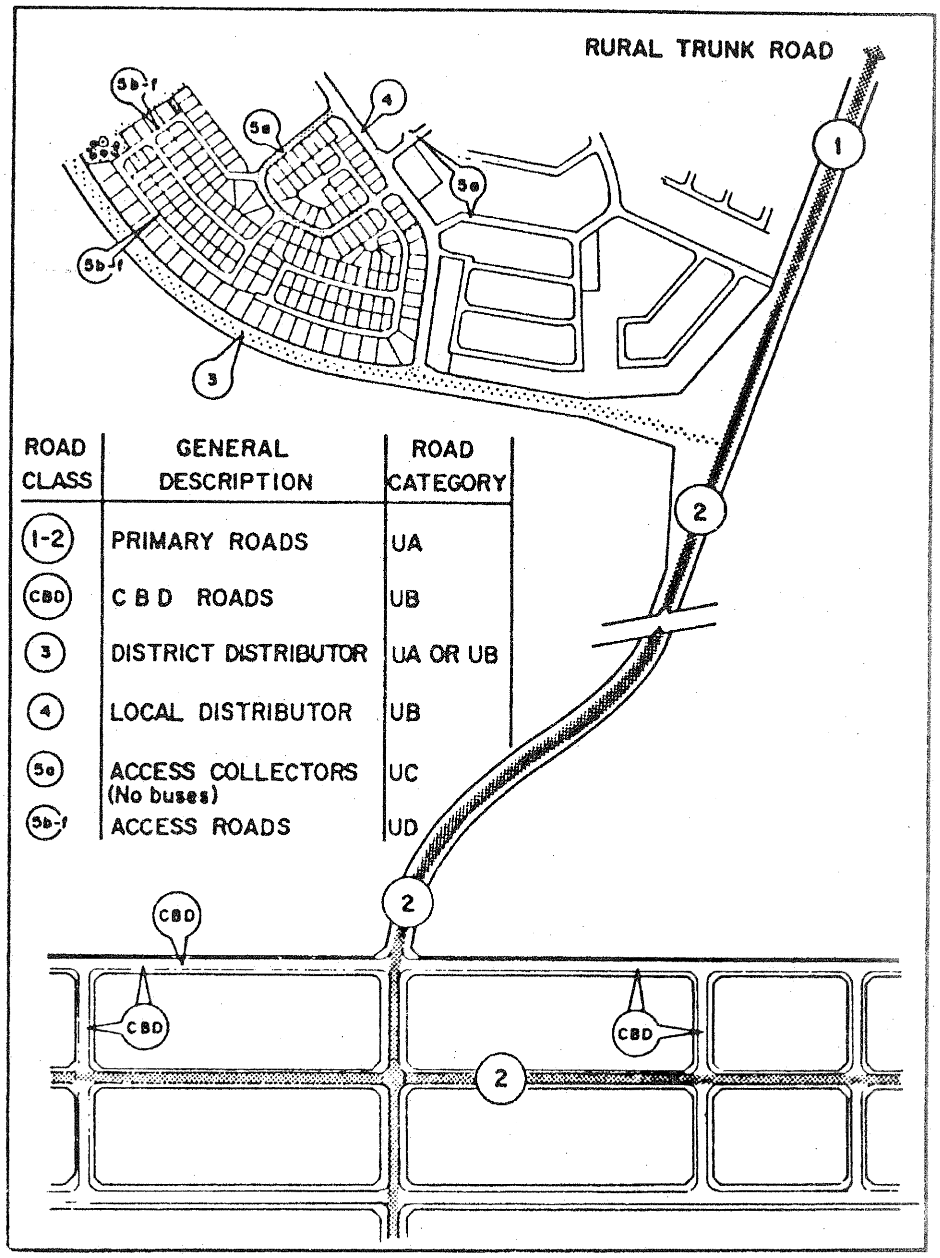


FIGURE 3

Illustration of different road categories

TABLE 1

Definition of the four road categories

	Road Category			
	UA	UB	UC	UD
General description	Primary and distributor roads		Local access roads	
Road class	1 and 2	3 and 4	5(a) and 5(b)	5(b) to 5(f)
Detailed description and function	Trunk roads and primary distributors (SAICE: Freeway and major arterials). Also by-passes and certain rural main roads.	District and local distributors (SAICE: Minor arterials and collectors). Also industrial roads, CBD roads, goods loading areas and bus routes.	Residential access collectors. Also car parks. No bus routes.	Local access roads – loops, access ways, access courts, access strips and culs-de-sac.
Importance and service level	Very important. High level of service.	Important. Moderate level of service	Less important. Moderate to low level of service.	Unimportant. Low level of service.
Total traffic over structural design period:				
(a) if road carries contruction traffic	0,8 – 50 × 10^6 E80s/lane	0,05 – 3 × 10^6 E80s/lane	<0,2 × 10^6 E80s/lane	
(b) if road does not carry construction traffic	0,8 – 50 × 10^6 E80s/lane	0,05 – 3 × 10^6 E80s/lane	<0,05 × 10^6 E80s/lane	

3 DESIGN STRATEGY

3.1 ANALYSIS PERIOD, STRUCTURAL DESIGN PERIOD AND DESIGN STRATEGY

The design strategy could influence the total cost of a pavement structure. Normally a design strategy is applicable only to Category UA and UB roads. For Category UC and UD roads, the design periods are fixed.

The analysis period is a convenient planning period during which complete reconstruction of the pavement is undesirable. The structural design period is defined as the period for which it is predicted with a high degree of confidence that no structural maintenance will be required. In order to fulfil the design objective of selecting the optimum pavement in terms of present worth of cost, it is necessary to consider how the pavement is expected to perform over the analysis period. The manner in which a design strategy can be presented is demonstrated schematically in Figure 4 which shows the generalized trends of riding quality decreasing with time and traffic for two different pavement structures, namely:

- Design 1, which requires resurfacing to maintain the surface in a good condition, and later some structural rehabilitation such as an overlay (Figure 4(a)), and
- Design 2, which is structurally adequate for the whole of the analysis period and requires only three resurfacings (Figure 4(b)).

It is important to note that any design procedure can only estimate the timing and nature of the maintenance measures that may be needed. Naturally such estimates are only approximate but they provide a valuable guide for a design strategy. The actual maintenance should be determined by way of a proper maintenance procedure. The accuracy of the prediction could be improved by having a feedback system.

3.2 SELECTION OF ANALYSIS PERIOD AND STRUCTURAL DESIGN PERIOD

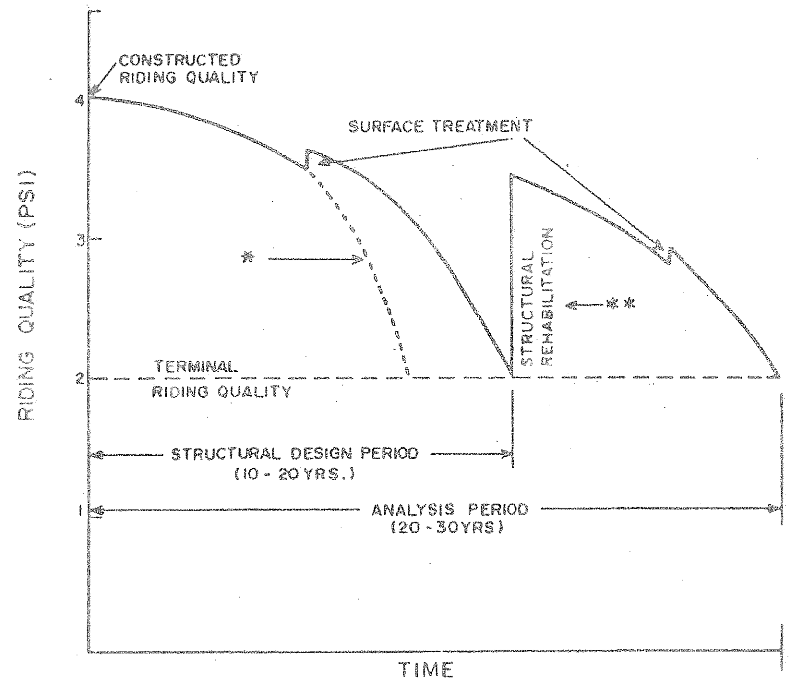
3.2.1 Selection of analysis period

The analysis period is a realistic cost period. There may be a difference between the analysis period and the total period over which a facility will be used. The analysis period is often related to the geometric life. If the road alignment is fixed, a period of 30 years should be used. In the case of a short geometric life in a changing traffic situation, a short analysis period will be used. The analysis period will influence the salvage value referred to in Section 9.

3.2.2 Selection of structural design period

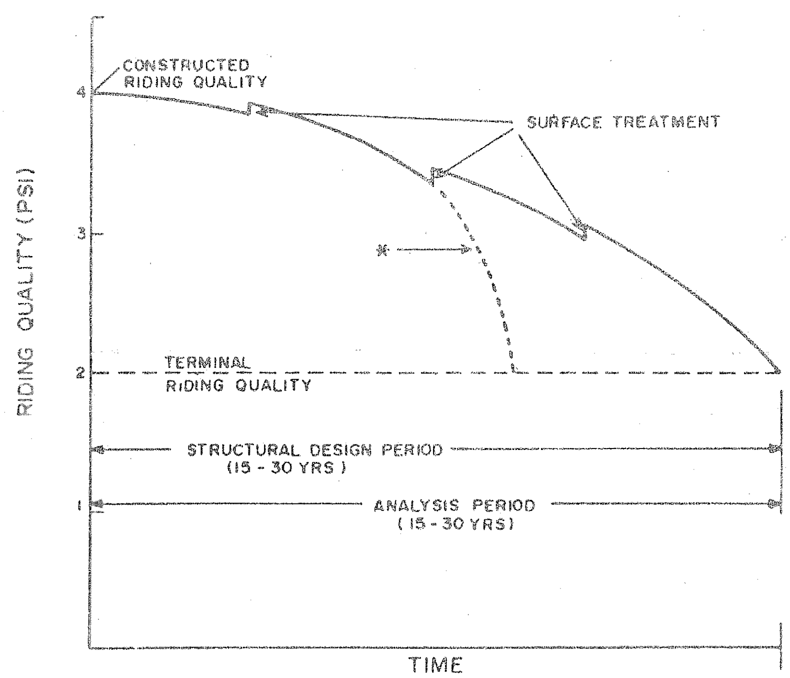
(a) Category UA roads

For Category UA roads, the structural design period should be reasonably long because:



(a)

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



(b)

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

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

** STRUCTURAL REHABILITATION USUALLY OCCURS AT A LATER STAGE

FIGURE 4

Illustration of design periods and alternative design strategies

- It is usually not politically acceptable for road authorities to carry out heavy rehabilitation on recently constructed pavements.
- Road user costs are high and the cost of the disruption of traffic will probably cancel out any pavement cost savings resulting from the choice of short structural design periods.
- The road alignment is normally fixed.

As shown in Table 2, the structural design period adopted in this document is 20 years, for Category UA roads.

(b) Category UB roads
For Category UB roads, the structural design period may vary depending on the circumstances. Long structural design periods (20 years) will be selected when circumstances are the same as for Category UA roads. Factors that encourage the selection of short structural design periods are:

- a short geometric life for a facility in a changing traffic situation;
- a lack of short-term funds, and
- a lack of confidence in design assumptions, especially the design traffic.

Structural design periods may range from 10 to 25 years. Normally a period of 20 years will be used (Table 2).

(c) Category UC roads
For Category UC roads (residential roads) a fixed structural design period of 20 years is recommended (Table 2).

TABLE 2

Structural design periods for various road categories

Road category	Structural design period* (years)	
	Range	Recommended
UA	15-25	20
UB	10-25	20
UC	10-30	20

* The analysis period for Category UA and UB roads is 30 years.

(d) Category UD roads
The traffic volume is so limited that no structural design period is applicable.

4 DESIGN TRAFFIC

4.1 TRAFFIC CLASSES FOR STRUCTURAL DESIGN PURPOSES

The cumulative damaging effect of all individual axle loads is expressed as the number of equivalent 80 kN single-axle loads (E80s). This is the number of 80 kN single-axle loads that would cause the same damage to the pavement as the actual spectrum of axle loads. For structural design, an estimate of the cumulative equivalent traffic over the structural design period is required. This cumulative equivalent traffic can be determined in two different ways:

- by estimation from tabulated traffic classes, and
- through detailed computation from initial and mean daily traffic, growth rates and lane distribution factors.

The estimation of the cumulative equivalent traffic over the structural design period from tabulated values is recommended, unless more specific information is available.

The cumulative equivalent traffic (total E80s over the design period) is grouped into six traffic classes, varying from ER for residential roads to E4 for very heavily trafficked roads. The class of traffic is a major factor in the selection of the actual pavement structure from the catalogue of designs. The traffic classes

TABLE 3

Classification of traffic for structural design purposes

Traffic class	Cumulative equivalent traffic, E80s/lane	Description
ER	($<0,05 \times 10^6$)	Residential roads.
EO	$0,05 - 0,2 \times 10^6$	Lightly trafficked collector roads, very few heavy vehicles.
E1	$0,2 - 0,8 \times 10^6$	Collector roads and lightly trafficked bus routes, mainly cars and light delivery vehicles.
E2	$0,8 - 3,0 \times 10^6$	Medium volume of traffic, bus routes and arterial roads.
E3	$3,0 - 12 \times 10^6$	High volume of traffic and/or many heavy vehicles. Major arterial routes.
E4	$12 - 50 \times 10^6$	Very high volume of traffic and/or a high proportion of fully laden heavy vehicles. Major arterial routes.

are defined in Table 3. A detailed computation of the cumulative equivalent traffic would be applicable only when the design traffic class is bound to be higher than EO (ie E1 to E4). For Category UC and UD roads, computations of traffic are normally not necessary. However, if residential roads carry construction traffic, a calculation may be necessary to show that the design traffic class changes from ER to EO.

For certain lightly trafficked roads a change in the service level may be considered because of anticipated lower vehicle speeds. In these cases a lower road category should be selected rather than changing the designs in the catalogue.

4.2 DETAILED COMPUTATION OF EQUIVALENT TRAFFIC

The detailed computation of the cumulative equivalent traffic involves:

- load equivalency of traffic;
- surveys of traffic conditions;
- projecting the traffic data over the structural design period, and
- estimating the lane distribution.

Load equivalency of traffic

The number of E80s is termed the equivalent traffic. The load equivalency factor relates the number of repetitions of a given axle load to the equivalent number of E80s. This equivalency factor is a function of pavement composition, material types, definition of terminal conditions and road rideability. Table 4 gives average equivalency factors based on:

$$F = \left(\frac{P}{80}\right)^n \qquad \text{4.1}$$

where n = 4,2
F = load equivalency factor
P = axle load

Pavements that are sensitive to overloading, such as shallow-structured pavements with thin cemented bases, may have n values of more than 4,2 whereas less sensitive, deep-structured pavements may have n values of less than 4,2. The designer can carry out a sensitivity analysis over the spectrum of axle loads with n values ranging from 2 to 6. This may be useful especially in the case of abnormal axle load spectra.

The equivalent traffic can be determined by multiplying the number of axle loads (t_j) in each load group of the entire load spectrum by the relevant equivalency factor (F_j) determined from Table 4.

TABLE 4

80kN single-axle equivalency factors, derived from

$$F = \left(\frac{P}{80} \right)^{4.2}$$

Single-axle load, P (kN)	80 kN axle equivalency factor, F	Single-axle load, P (kN)	80 kN axle equivalency factor, F
Less than 15	0,000	115 – 124	5,100
15 – 24	0,004	125 – 134	7,000
25 – 34	0,019	135 – 144	9,400
35 – 44	0,062	145 – 154	12,000
45 – 54	0,150	155 – 164	16,000
55 – 64	0,320	165 – 174	20,000
65 – 74	0,590	175 – 184	26,000
75 – 84	1,000	185 – 194	32,000
85 – 94	1,600	195 – 204	39,000
95 – 104	2,400	More than 205	50,000
105 – 114	3,600		

By summation the equivalent daily traffic is

$E = \Sigma t_j \, . \, F_j$ 4.2

4.2.2 Surveys of traffic conditions[4,1,8]

The present average daily traffic is the amount of daily traffic in a single direction, averaged over the present year. This traffic can be estimated from traffic surveys carried out at some time before the initial year. Such a survey may include:

- static weighing of a sample of vehicles;
- dynamic weighing of all axles for a sample period (eg a traffic axle weight classifier (TAWC) survey), and
- estimation procedures based on visual observations.

(a) Static weighing procedures

The static axle loads of a representative sample of vehicles can be determined by means of permanent weighbridges or portable scales. These axles loads can be classified into convenient axle load groups. A visual survey of all passing traffic should be carried out at the same time. The visual survey is used to divide the traffic into various appropriate categories. Using the axle loads determined by static weighing, an axle load is attributed to each category of traffic determined during the visual count.

This gives an estimate of the number of repetitions of a given axle load. More than 90 % of all equivalent traffic is due to laden commercial vehicles (ie more than 60 % full). Greater emphasis should therefore be placed on these vehicles. A commercial vehicle is defined as a vehicle with a carrying capacity of more than 2700 kg.

(b) Dynamic weighing procedures
The traffic data logger (TDL) can be used to determine the dynamic load of moving axles. The TDL sensor installed in the traffic lane determines the load of every axle moving over it. The load of the axle is then classified into one of 11 load groups, in increments of 2 000 kg. If an equivalency factor is then attributed to each load group, the equivalent traffic can be calculated as described in 4.2.1.

(c) Estimation procedures based on visual observations
These procedures are used when costly traffic determinations are not justified or when the characteristics of the road make dynamic weighing difficult. The methods are based on information gained from countrywide TAWC surveys and visual observations. The observer should categorize the vehicles into three groups, depending on whether the commercial vehicles are:

- mostly unladen;
- 50 % laden, or
- mostly fully laden.

The E80s per commercial vehicle should be determined according to the values given in Table 5. The total number of commercial vehicles should be multiplied by the appropriate factor to establish the equivalent traffic for the

TABLE 5

Determination of E80s per commercial vehicle

Loading of commercial vehicles (or type of road)	Number of E80s/ commercial vehicle
Mostly unladen (Category UC, residential and collector roads)	0,6
50 % laden, 50 % unladen (Category UA or UB, arterial roads and bus routes)	1,7
> 70 % laden (Category UA or UB, main arterials or major industrial routes)	2,6
Fully laden bus	3,0

period over which the visual estimation was done. Note that, over the last 10 years, the number of E80s per commercial vehicle has increased at a rate of about 6 % per annum. The values in Table 5 could change in the future, especially if the legal axle load limit changes. Table 5 also gives an indication of the number of E80s per commercial vehicle on various types of road.

Other more detailed visual estimation procedures can be used[5]. In using these procedures more attention is given to the actual loading conditions of the commercial vehicles on the specific route.

4.2.3 Projection of the traffic data over the structural design period

(a) Projection to 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 growth rate:

$$g_x = (1 + 0{,}01 \,.\, i)^x \quad \text{......} \quad 4.3$$

where
- g = growth factor
- i = growth rate
- x = time between determination of axle load data and opening of road in years

The traffic growth factor (g) is given in Table 6.

(b) Computation of cumulative equivalent traffic

The cumulative equivalent traffic (total E80s) over the structural design period may be calculated from the equivalent traffic in the initial design year and the growth rate for the design period. Where possible, the growth rate should be based on specific information. More than one growth rate may apply over the design period. There may also be a difference between the growth rates for total and equivalent traffic. These rates will normally vary between 2 % and 10 % and a value of 6 % is recommended.

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

$$E_{initial} = E.g_x \quad \text{......} \quad 4.4$$

The cumulative equivalent traffic may be calculated from:

$$N_e = E_{initial} \cdot f_y \quad \text{......} \quad 4.5$$

where
- f_y = cumulative growth factor, based on
- $f_y = 365\,(1 + 0{,}01.i)\,|(1 + 0{,}01.i)^y - 1|/(0{,}01.i)$

(y = structural design period – see Section 3)

The cumulative growth factor (f_y) is given in Table 7.

TABLE 6

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 (years)	*g for traffic increase, i (% pa)								
	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
6	1,13	1,19	1,27	1,34	1,42	1,50	1,59	1,68	1,77
7	1,15	1,23	1,32	1,41	1,50	1,61	1,71	1,83	1,95
8	1,17	1,27	1,37	1,48	1,59	1,72	1,85	1,99	2,14
9	1,20	1,30	1,42	1,55	1,69	1,84	2,00	2,17	2,36
10	1,22	1,34	1,48	1,63	1,79	1,97	2,16	2,37	2,59
11	1,24	1,38	1,54	1,71	1,90	2,10	2,33	2,58	2,85
12	1,27	1,43	1,60	1,80	2,01	2,25	2,52	2,81	3,14
13	1,29	1,47	1,67	1,89	2,13	2,41	2,72	3,07	3,45
14	1,32	1,51	1,73	1,98	2,26	2,58	2,94	3,34	3,80
15	1,35	1,56	1,80	2,08	2,40	2,76	3,17	3,64	4,18
16	1,37	1,60	1,87	2,18	2,54	2,95	3,43	3,97	4,59
17	1,40	1,65	1,95	2,29	2,69	3,16	3,70	4,33	5,05
18	1,43	1,70	2,03	2,41	2,85	3,38	4,00	4,72	5,56
19	1,46	1,75	2,11	2,53	3,03	3,62	4,32	5,14	6,12
20	1,49	1,81	2,19	2,65	3,21	3,87	4,66	5,60	6,73
21	1,52	1,86	2,28	2,79	3,40	4,14	5,03	6,11	7,40
22	1,55	1,92	2,37	2,93	3,60	4,43	5,44	6,66	8,14
23	1,58	1,97	2,46	3,07	3,82	4,74	5,87	7,26	8,95
24	1,61	2,03	2,56	3,23	4,05	5,07	6,34	7,91	9,85
25	1,64	2,09	2,67	3,39	4,29	5,43	6,85	8,62	10,83
26	1,67	2,16	2,77	3,56	4,55	5,81	7,40	9,40	11,92
27	1,71	2,22	2,88	3,73	4,82	6,21	7,99	10,25	13,11
28	1,74	2,29	3,00	3,92	5,11	6,65	8,63	11,17	14,42
29	1,78	2,36	3,12	4,12	5,42	7,11	9,32	12,17	15,86
30	1,81	2,43	3,24	4,32	5,74	7,61	10,06	13,27	17,45

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

TABLE 7

Traffic growth factor (f_y) for calculation of cumulative traffic over prediction period from initial (daily) traffic

Prediction period, y (years)	Compound growth rate, i (% pa)									
	2	4	6	8	10	12	14	16	18	20
4	1 534	1 611	1 692	1 776	1 863	1 953	2 047	2 145	2 246	2 351
5	1 937	2 056	2 180	2 312	2 451	2 597	2 750	2 911	3 081	3 259
6	2 348	2 517	2 698	2 891	3 097	3 317	3 551	3 801	4 066	4 349
7	2 767	2 998	3 247	3 517	3 809	4 124	4 464	4 832	5 229	5 657
8	3 195	3 497	3 829	4 192	4 591	5 028	5 506	6 029	6 601	7 226
9	3 631	4 017	4 445	4 922	5 452	6 040	6 693	7 417	8 220	9 109
10	4 076	4 557	5 099	5 710	6 398	7 173	8 046	9 027	10 130	11 369
11	4 530	5 119	5 792	6 561	7 440	8 443	9 588	19 895	12 384	14 081
12	4 993	5 703	6 526	7 480	8 585	9 865	11 347	13 061	15 044	17 336
13	5 465	6 311	7 305	8 473	9 845	11 458	13 352	15 575	18 183	21 241
14	5 947	6 943	8 130	9 545	11 231	13 242	15 637	18 490	21 887	25 927
15	6 438	7 600	9 005	10 703	12 756	15 239	18 242	21 872	26 257	31 551
16	6 939	8 284	9 932	11 953	14 433	17 477	21 212	25 795	31 414	38 299
17	7 450	8 995	10 915	13 304	16 278	19 983	24 598	30 346	37 500	46 397
18	7 971	9 734	11 957	14 762	18 308	22 790	28 458	35 625	44 680	56 115
19	8 503	10 503	13 061	16 338	20 540	25 934	32 859	41 748	53 154	67 776
20	9 045	11 303	14 232	18 039	22 995	29 455	37 875	48 851	63 152	81 769
25	11 924	15 808	21 227	28 818	39 486	54 506	75 676	105 517	147 559	206 727
30	15 103	21 289	30 587	44 656	66 044	98 656	148 459	224 533	340 661	517 664
35	18 612	27 858	43 114	67 927	108 816	176 464	288 595	474 509	782 431	1 291 373
40	22 487	36 071	59 877	102 120	177 700	313 586	588 416	999 544	1 793 095	3 216 609

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

4.2.4 Estimating the lane distribution of traffic

On multi-lane carriageways, the traffic will be distributed among the lanes. Note that the distribution of total traffic and equivalent traffic will not be the same. The distribution will also change along a length of road, depending on geometric factors such as climbing lanes or interchange ramps. Suggested design factors for total traffic (B) and equivalent traffic (B_e) are given in Table 8. As far as possible, these factors incorporate the change in lane distribution over the geometric life of a facility. The factors should be regarded as maxima and decreases may be justified.

TABLE 8

Design factors for the distribution of traffic and equivalent traffic among lanes and shoulders

Total number of traffic lanes	Design distribution factor, B_e or B				
	Surfaced slow shoulder	Lane 1*	Lane 2	Lane 3	Surfaced fast shoulder**
(a) Equivalent traffic (E80s) Factor B_e					
2	1,00	1,00	—	—	—
4	0,95	0,95	0,30	—	0,30
6	0,70	0,70	0,60	0,25	0,25
(b) Traffic (total axles or evu***) Factor B					
2	1,00	1,00	—	—	—
4	0,70	0,70	0,50	—	0,50
6	0,30	0,30	0,50	0.40	0,40

* Lane 1 is the outer or slow lane

** For dual-carriageway roads

*** evu = equivalent vehicle unit; one commercial vehicle = 3 evu

4.2.5 The design cumulative equivalent traffic

The design cumulative equivalent traffic may be calculated by multiplying the equivalent traffic by a lane distribution factor (B_e):

$$N_e = (\Sigma t_j \,.\, F_j) \,.\, g_x \,.\, f_y \,.\, B_e \quad \text{.......} \quad 4.7$$

where $\Sigma t_j \cdot F_j$ = equivalent daily traffic at time of survey

g_x = growth factor to initial year (x = period from traffic survey to initial design year)

f_y = cumulative growth factor over structural design period (y = structural design period)

B_e = lane distribution factor for equivalent traffic

The designer should now go back to Table 3 to determine the design traffic class (ER – E4). Simple interpolation techniques are available (described in Section 7) if the pavement structure is to be designed to greater accuracy with regard to traffic.

In order to check the geometric capacity of the road, the total daily traffic towards the end of the structural design period can be calculated from:

n = (initial total daily traffic) . g_x 4.8

with g_x as previously defined.

When projecting traffic over the structural design period, the designer should take into account the possibility of capacity[2] conditions being reached, which would result in no further growth in traffic for that particular lane.

Figure 5 gives a nomogram for determining the design traffic class from the initial E80s per lane per day, the growth rate and the structural design period. It is applicable only to traffic classes E1 to E4.

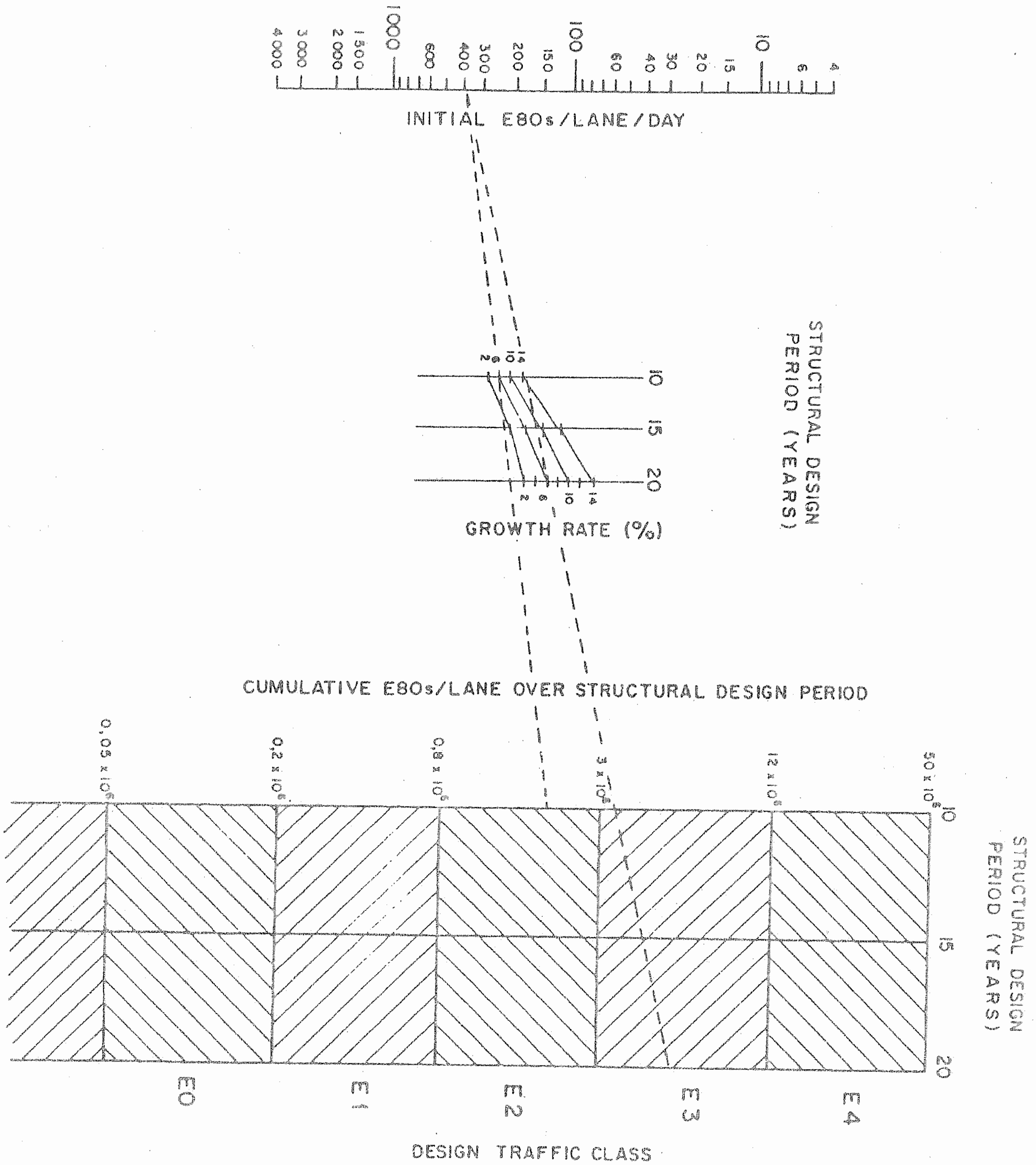


FIGURE 5[1]

Nomogram for determining design traffic class from the initial E80s/lane/day, the growth rate and structural design period

5 MATERIALS

5.1 GENERAL LIST OF PAVEMENT MATERIALS AND ABBREVIATED STANDARDS

The selection of materials for pavement design is based on a combination of availability, economic factors and previous experience. These factors have to be evaluated during the design in order to select the materials best suited to the conditions.

The design procedure generally uses the standard material specification defined in TRH14 *Guidelines for road construction materials*[10]. The classification of the materials is given in Figure 6. The material codes listed in this table are used extensively in the catalogue of designs. Only abbreviated specifications are given and TRH14[10] should be consulted for more details. Waste materials (eg blast-furnace slags) and pedogenic materials have not been classified because of their varying quality. If these materials are used they should be classified according to the appropriate material codes[10]. The materials are classified into various categories according to their fundamental behaviour and into different classes according to their strength characteristics.

5.2 DESCRIPTION OF MAJOR MATERIAL TYPES

This subsection describes the materials in Figure 6 and their major characteristics. The behaviour of different pavement types consisting of combinations of these materials is described in Subsection 6.2.

5.2.1 Granular materials and soils (G1 to G10)

These materials show stress-dependent behaviour, and under repeated stresses deformation can occur through shear and/or densification.

A G1 is a dense-graded, unweathered, crushed-stone material compacted to 86-88 % of apparent density. A faulty grading may be adjusted only by means of the addition of crusher sand or other stone fractions obtained from the crushing of the parent rock. G2 and G3 may be a blend of crushed stone and other fine aggregate to adjust the grading. If the fine aggregate is obtained from a source other than the parent rock, its use must be approved by the purchaser and the supplier must furnish the purchaser with full particulars regarding the exact amount and nature of such fine aggregate. G4 to G10 materials cover the range from high-quality gravels used in pavement layers (CBR 25 – 80) to lower-quality materials used in selected layers (CBR 3-15).

In gravel roads natural gravel materials of quality G4 to G10 are used for the wearing course.

5.2.2 Bituminous materials (BC to TS)

Bituminous materials are visco-elastic and under repeated stresses they may either crack or deform or both. Normally a BC continuously graded bitumen hot

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 $\not>$ 4
	G2	Graded crushed stone	Dense-graded unweathered crushed stone; Max. size 37,5 mm; 100 - 102 % mod. AASHTO; fines PI $\not>$ 6
	G3	Graded crushed stone	Dense-graded stone + soil binder; Max. size 37,5 mm; minimum 98 % mod. AASHTO; fines PI $\not>$ 6
	G4	Natural gravel	CBR $\not<$ 80; PI $\not>$ 6
	G5	Natural gravel	CBR $\not<$ 45; PI $\not>$ 10: Max. size 63 mm
	G6	Natural gravel	CBR $\not<$ 25; Max size $\not>$ $\frac{2}{3}$ layer thickness
	G7	Gravel - soil	CBR $\not<$ 15; Max. size $\not>$ $\frac{2}{3}$ layer thickness
	G8	Gravel - soil	CBR $\not<$ 10 at in situ density
	G9	Gravel - soil	CBR $\not<$ 7 at in situ density
	G10	Gravel - soil	CBR $\not<$ 3 at in situ density
	BC	Bitumen hot-mix asphalt	Continuously graded; Max size 26,5 mm
	BS	Bitumen hot-mix asphalt	Semi - gap - graded; Max size 37,5 mm
	TC	Tar hot-mix asphalt	As for BC (continuously graded)
	TS	Tar hot-mix asphalt	As for BS (semi - gap - graded)
	PCC	Portland cement concrete	Mod. rupture $\not<$ 3,8 MPa; Max. size $\not>$ 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 gap - graded
	AC	Asphalt surfacing	Ref. TRH8 continuously graded
	AS	Asphalt surfacing	Ref. TRH8 semi - gap - graded
	AO	Asphalt surfacing	Ref. TRH8 open-graded
——	S1	Surface treatment	Ref. TRH3 single seal
——	S2	Surface treatment	Ref TRH3 multiple seal
——	S3	Sand seal	Ref. TRH3
——	S4	Cape seal	Ref. TRH 3
——	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 $\not>$ 6 88 - 90 % of apparent density
	WM2	Waterbound macadam	Max. size 75 mm; PI of fines $\not>$ 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
	S - A	Interlocking paving blocks (type S - A)	Geometrical interlock on all vertical faces (possible to lay in herringbone bond). Wet crushing strength $\not<$ 25 MPa
	S - B	Interlocking paving blocks (type S - B)	Geometrical interlock on some vertical faces (not possible to lay in herringbone bond) Wet crushing strength $\not<$ 25 MPa
	S - C	Non-interlocking paving blocks (type S - C)	No geometrical interlock. Wet crushing strength, average $\not<$ 25 MPa, single units $\not<$ 20 MPa
	SND	Bedding sand	For use with concrete, clay or other blocks

FIGURE 6

Definition of material symbols used in catalogue of designs mix will have a higher stability and lower fatigue life than a semi-gap-graded BS material. Tar hot mixes (TC, TS) will normally have lower fatigue lives than the equivalent bitumen hot mixes. Usually the stability of a tar mix is the same as or higher than that of the equivalent bitumen mix.

5.2.3 Portland cement concrete (PCC)

Concrete is an elastic, brittle material possessing tensile strength and it may crack under excessive repeated flexure. In this document only one concrete strength is considered.

5.2.4 Cemented materials (C1 to C4)

As concrete, cemented materials are elastic, possess tensile strength and may crack under repeated flexure. These materials also crack because of shrinkage and drying. By the application of an upper limit to the strength specification, wide shrinkage cracks can be avoided. Because of the excessive shrinkage cracking of C1 materials, they are not generally used. A C2 material will be used when a non-pumping erosion-resistant layer is required (as for a concrete pavement).

C3 and C4 materials can be used as replacements for granular layers in bases and subbases. They can be either cement-treated or lime-treated, depending on the properties of the natural materials.

5.2.5 Surfacings (AG to AO; S1 to S8)

The surfacings cover the range from high-quality asphalt surfacings to surface treatments and surface maintenance measures such as rejuvenators and diluted emulsion treatments.

5.2.6 Macadams (WM to PM)

These are traditional, high-quality, but also labour-intensive, pavement materials which can be used in the place of G1 to G4 materials. However, specific knowledge of construction techniques is required. These materials are less water-susceptible than the usual granular materials and using them should be considered, especially in wet regions.

5.2.7 Paving blocks (S-A, S-B)

The use of interlocking concrete paving blocks (S - A) is limited to low-speed (<50 km/h) roads or terminal areas. Blocks with good interlocking shapes should be used.

The use of clay bricks as paving blocks (S - B) should be limited to Category UC and UD roads. The performance of clay bricks is expected to be poorer than that of concrete paving blocks.

5.3 AVAILABILITY, EXPERIENCE AND CURRENT UNIT COSTS

The designer should complete the check-list in Figure 7 in order to ensure that:

- the type of material is currently available, and
- a survey of recent unit costs of the materials has been carried out.

Past experience with a particular material as well as the relevant traffic and environmental factors should be taken into consideration.

SYMBOL	CODE	MATERIAL	AVAILABILITY	UNIT COST
	G1	Graded crushed stone		
	G2	Graded crushed stone		
	G3	Graded crushed stone		
	G4	Natural gravel		
	G5	Natural gravel		
	G6	Natural gravel		
	G7	Gravel - soil		
	G8	Gravel - soil		
	G9	Gravel - soil		
	G10	Gravel - soil		
	BC	Bitumen hot - mix		
	BS	Bitumen hot - mix		
	TC	Tar hot - mix		
	TS	Tar hot - mix		
	PCC	Portland cement concrete		
	C1	Cemented crushed stone or gravel		
	C2	Cemented crushed stone or gravel		
	C3	Cemented natural gravel		
	C4	Cemented natural gravel		
	AG	Asphalt surfacing		
	AC	Asphalt surfacing		
	AS	Asphalt surfacing		
	AO	Asphalt surfacing		
	S1	Surface treatment		
	S2	Surface treatment		
	S3	Sand seal		
	S4	Cape seal		
	S5	Slurry		
	S6	Slurry		
	S7	Surface renewal		
	S8	Surface renewal		
	WM	Waterbound macadam		
	PM	Penetration macadam		
	CB1	Concrete paving blocks		
	CB2	Clay bricks		

FIGURE 7

Check-list of material availability and unit cost

6 ENVIRONMENT

6.1 GENERAL

The environment is characterized by topography, the climatic conditions (moisture and temperature) under which the road will function, and the underlying subgrade conditions. Environmental factors must be taken into account in the design of pavement structures.

6.2 TOPOGRAPHY

Topography is dealt with in an UTG document in preparation (Urban Stormwater Management). The importance of its influence on the structural design and functional use of roads is clearly reflected in the drainage and maintenance requirements of roads in general. Macrodrainage is relevant to this discussion. In rolling or mountainous terrains there may be steep gradients which result in the erosion of gravel roads and in particular erosion of their drainage facilities, with direct implications for their safety and functional use. Roads that cross contours at an angle, or even perpendicularly, pose the most drainage problems. In such cases functional rather than structural requirements may require that a road be paved or provided with erosion protection. It is therefore important that requirements on Layout Planning and on the Stormwater Management be met before one embarks on the structural design.

6.3 CLIMATIC REGIONS AND THE DESIGN OF PAVEMENTS

The climate will largely determine the weathering of natural rocks, the durability of weathered, natural road building materials and also, depending on drainage conditions, the stability of untreated materials in the pavement. The climate may also influence the equilibrium moisture content. The designer should always consider climatic conditions and avoid using materials that are excessively water-susceptible or temperature-sensitive in adverse conditions. It is also possible to accommodate climatic conditions by either adjusting CBR values or by weighting the equivalent traffic (not both).

Southern Africa can be divided into three climatic regions:

- a large dry region;
- a moderate region, and
- a small wet region.

Figure 8 shows a map of southern Africa which indicates the different climatic regions. These are macroclimates and it should be kept in mind that different microclimates may occur within these regions.

6.4 CLIMATE AND SUBGRADE CBR

The design parameter for the subgrade is the soaked California Bearing Ratio (CBR) at a representative density. For structural design purposes, when a material is classified according to the CBR it is implied that not more than 10 % of the measured values for such a material will fall below the classification value. A proper preliminary soil survey should be conducted.

It is current practice to use soaked CBR values, but using them in dry regions may be over-conservative[4]. It is suggested that the CBR value of a material be increased if the in situ conditions are expected to be unsoaked (eg in dry regions)[1].

The dynamic cone penetrometer[4] (DCP) can be used to determine the in situ CBR and variations in in situ strengths[4]. The in situ CBRs determined by means of the DCP can be calibrated by doing laboratory soaked CBRs. If material parameters such as grading modulus (GM), plastic limit (PL) and dry density (DD) are included in the analysis, typical relations can be used to derive CBR values[15]. The relevant equation is:

$$\log_e = 1{,}1\ (\log_e \mathrm{DCP}) + 0{,}85\ (\mathrm{GM}) - 0{,}031\ (\mathrm{PL}) - 0{,}001\ (\mathrm{DD}) + 7{,}4$$

where $\log_e$ is the natural logarithm.

For the material types under consideration, the CBR is determined at 2,54 mm depth of penetration with DCP penetration mm per blow.

6.5 MATERIAL DEPTH

The term "material depth" is used to denote the depth below the finished level of the road to which soil characteristics have a significant effect on pavement behaviour. Below this depth the strength and density of the soils are assumed to have a negligible effect on the pavement. The depth approximates the cover for a soil of 1-2 % CBR. However, in certain special cases this depth may be insufficient. These cases are listed in subsection 8.3.

Table 9 specifies the material depth used for determining the design CBR of the subgrade for different road categories.

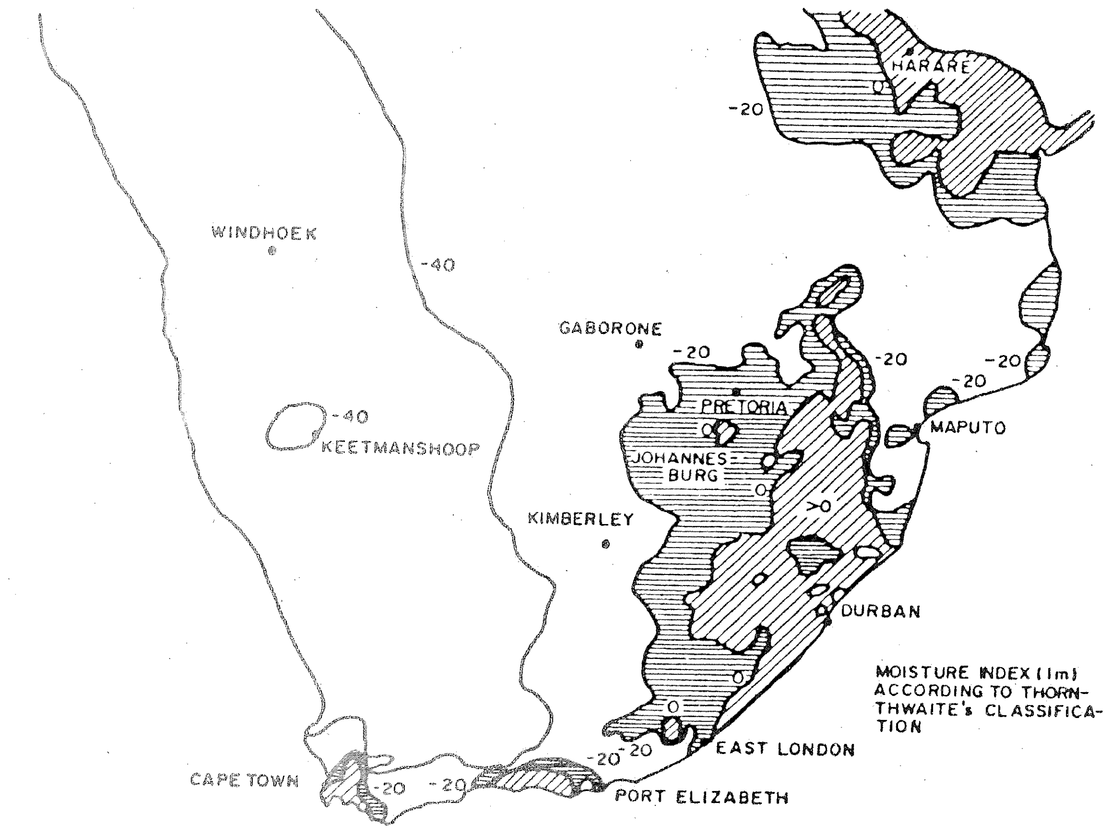
6.6 DELINEATION OF SUBGRADE AREAS

Any road development should be subdivided into significant subgrade areas. However, if the delineation is too fine it could lead to confusion during construction. The preliminary soil survey should delineate subgrade design units on the basis of geology, pedology, topography and drainage conditions – or major soil boundaries – on site so that it is appropriate to define a design CBR for each unit.

The designer should distinguish between very localized good or poor soils and more general subgrade areas. Localized soils should be treated separately from the rest of the pavement factors. Normally, localized poor soils will be removed and replaced by suitable material.

6.7 DESIGN CBR OF SUBGRADE

For construction purposes the design subgrade CBR is limited to five groups, as shown in Table 10.



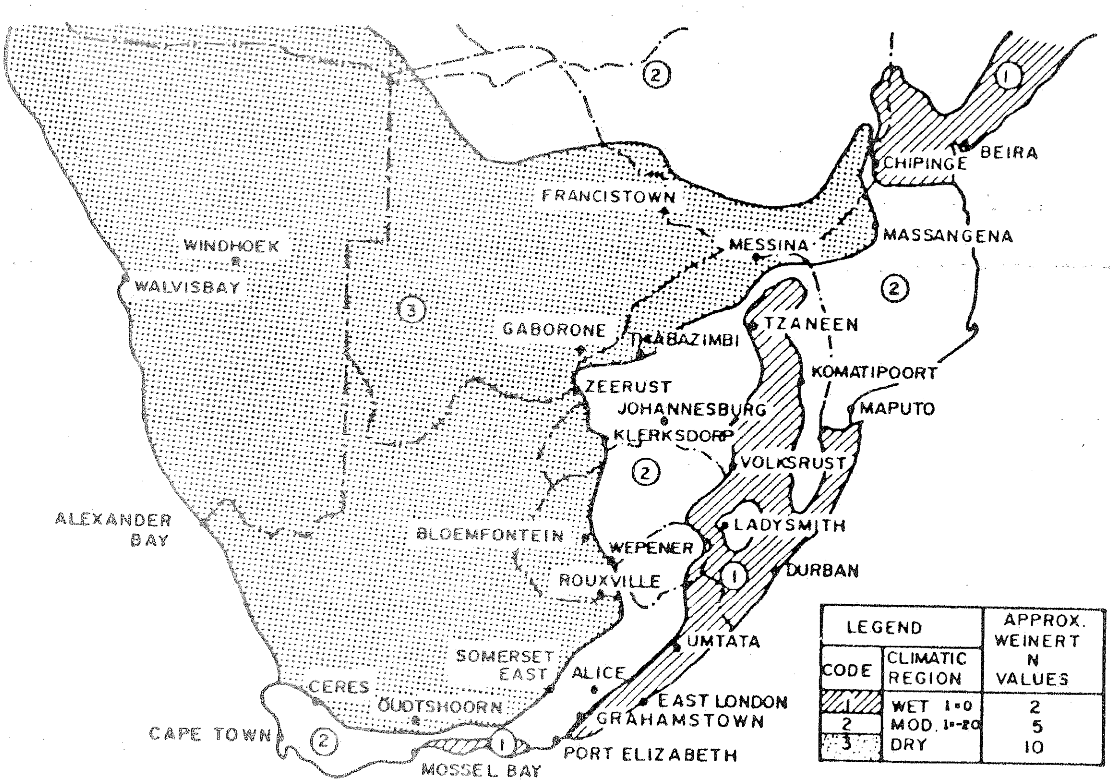


FIGURE 8

Macroclimatic regions of southern Africa

TABLE 9

Material depths to be used for determining the design CBR of the subgrade

Road category	Material depth (mm)
UA	1 000
UB	800
UC	600
UD	400

TABLE 10

Subgrade CBR groups used for structural design

Subgrade CBR (%)
< 3*
3 to 7
7 to 15
15 to 25
> 25**

* Special treatment required

** Use top 150 mm as subbase or base material, compacted to the correct standard[10]. The normal TRH14 requirements should be met.

The CBR is normally determined after samples have been soaked for four days and may be adjusted according to Subsection 6.4. Special measures are necessary if a material with a CBR less than 3 is encountered within the material depth. These measures include stabilization (chemical or mechanical), modification (chemical), and the removal or addition of extra cover. After the material has been treated, it will be classified under one of the remaining three subgrade groups.

6.8 DESIGN CBR ON FILL

When the road is on fill, the designer must avail himself of the best information available on the local materials that are likely to be used. The material should be controlled to at least the material depth. TRH9[9] should be consulted when a material with a CBR of less than 3 is used in the fill.

6.9 DESIGN CBR IN CUT

The design CBR of the subgrade in a cut should be the lowest CBR encountered within the material depth.

7 STRUCTURAL DESIGN AND PAVEMENT TYPE SELECTION

7.1 GENERAL

The designer may use a number of design procedures, such as the mechanistic design method[6], the AASHTO structural number method[14], the CBR cover curves[12] or the catalogue of designs given at the back of this document. Whatever the method used, factors such as road category, design strategy, traffic, available materials and environment must be taken into account. Some estimation of future maintenance measures is necessary before a comparison of different structures can be made on the basis of present worth of costs. Special construction considerations that might influence either the pavement structure or the pavement costs are discussed in Section 8.

This document is based on the use of a catalogue of designs. However, the best results will probably be obtained if the catalogue is used together with some other design method.

7.2 BEHAVIOUR OF DIFFERENT PAVEMENT TYPES

The behaviour of a pavement and the type of distress that will become the most critical vary with the type of pavement. There are five major pavement types, namely granular, bituminous, concrete and cemented-base pavements and pavements with paving blocks. The behaviour of these different pavement types will determine the type of maintenance normally required (Subsection 9.5) and may also influence the selection of the pavement type. A brief description of the typical behaviour of each pavement type is given below.

7.2.1 Untreated granular-base pavements

This type of pavement comprises a thin bituminous surfacing, a base of untreated gravel or crushed stone, a granular or cemented subbase and a subgrade of various soils or gravels. The mode of distress in a pavement with an untreated subbase is usually deformation arising from shear or densification in the untreated materials. The deformation may manifest itself as rutting or as longitudinal roughness. This is illustrated in Figure 9(a).

In pavements with cemented subbases, the subbase improves the load carrying capacity of the pavement, but at some stage the subbase will crack under traffic. The cracking may propagate until the layer eventually exhibits properties similar to those of a natural granular material. It is unlikely that cracking will reflect to the surface, and there is likely to be little rutting or longitudinal deformation until after the subbase has cracked extensively. However, if the subbase exhibits large shrinkage or thermal cracks, they may reflect to the surface.

Recent work has shown that the post-cracked phase of a cement-treated subbase under granular and bituminous bases adds substantially to the useful life of the pavement. Deflection measurements at various depths within the pavement have indicated that the initial effective modulus of this material is high – 3000 to 5000 MPa – as shown in Figure 9(c).

This relatively rigid subbase generally fatigues under traffic, or in some cases even under construction traffic, and assumes a lower effective modulus (500 to 1000 MPa). This change in modulus does not normally result in a marked increase in deformation, but the resilient deflection and radius of curvature do change, as shown in Figure 9(d).

In the mechanistic design[6] these phases have been termed the pre-cracked and post-cracked phases. The design accommodates the changes in modulus of the subbase, and although the safety factor in the base will be reduced, it will still be well within acceptable limits.

The eventual modulus of the cemented subbases will depend on the quality of the material originally stabilized, the cementing agent, the effectiveness of the mixing process, the absolute density achieved and the degree of cracking. The ingress of moisture can affect the modulus in the post-cracked phase significantly. In some cases the layer may behave like a good-quality granular material with a modulus of 200 to 500 MPa, whereas in other cases the modulus will be between 50 and 200 MPa. This change is shown diagrammatically in Figure 9(c).

The result is that the modulus of the cemented subbase assumes very low values and this causes fatigue and high shear stresses in the base. Generally, surface cracking will occur, and with the ingress of water, there may be pumping from the subbase.

For high-quality, heavily trafficked pavements it is necessary to avoid materials that will eventually deteriorate to a very low modulus. Many of these lower-class materials have, however, proved to be adequate for lower classes of traffic.

The surfacing may also crack owing to ageing of the binder or to load-associated fatigue cracking. Granular materials are often susceptible to water and excessive deformation may occur when water ingresses through the surface cracks. The water-susceptibility depends on factors such as grading, the PI of the fines, and density. Waterbound macadams are less susceptible to water than crushed-stone bases and are therefore preferred in wet regions.

7.2.2 Bituminous-base pavements

These pavements have a bituminous base layer of more than 80 mm thick. They can be subdivided into two major groups, namely bitumen- and tar-base pavements.

(a) Bitumen-base pavements

In bitumen-base pavements both deformation and fatigue cracking are possible. Two types of subbase are recommended, namely either an

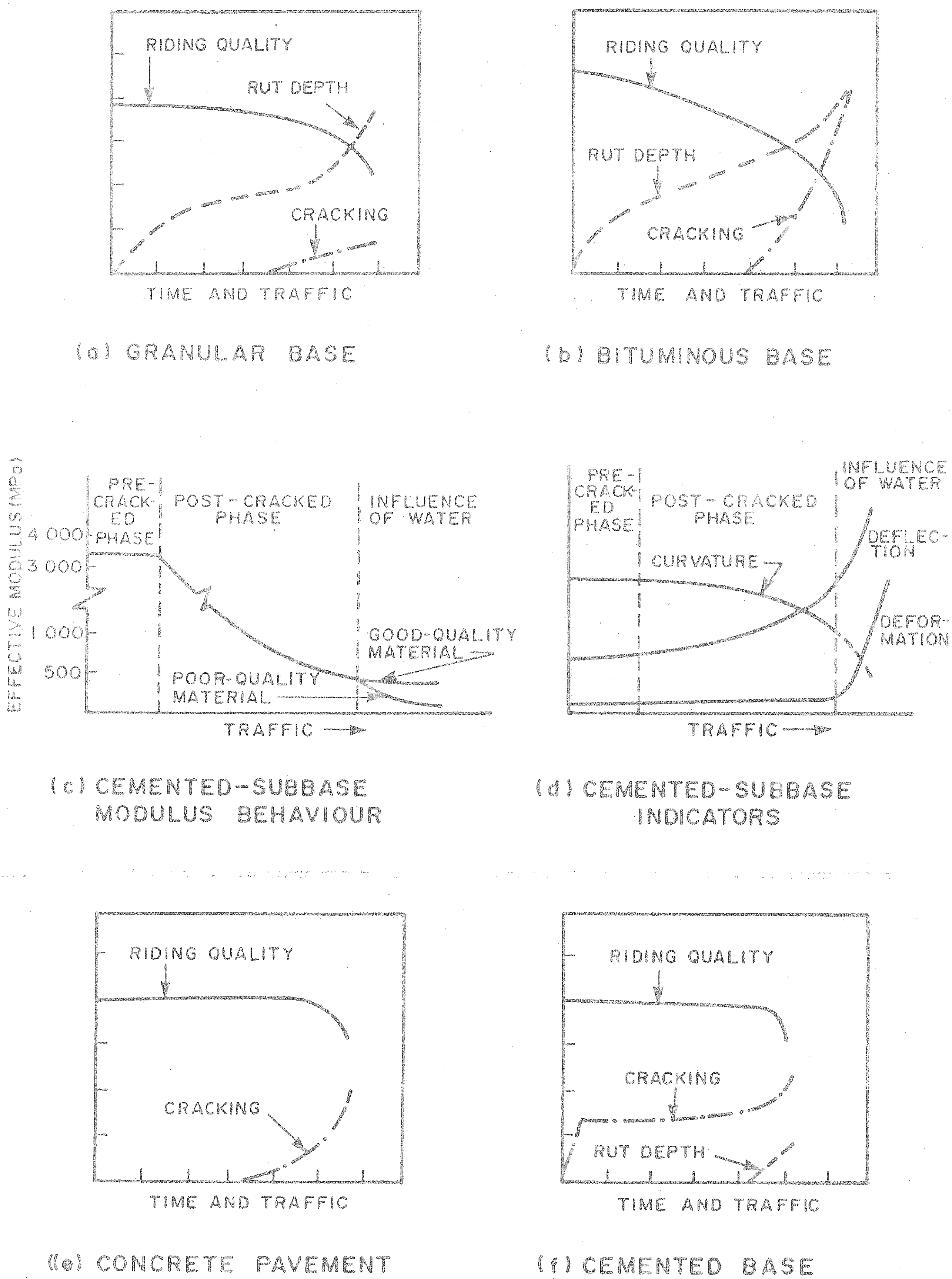


FIGURE 9

Generalized pavement behaviour characteristics untreated granular subbase or a weakly stabilized cemented subbase. Rutting may originate in either the bituminous or the untreated layers, or in both. This is illustrated in Figure 9(b). If the subbase is cemented there is a probability that shrinkage or thermal cracking will reflect to the surfacing, especially if the bituminous layer is less than 150 mm thick or if the subbase is excessively stabilized. Maintenance usually consists of a surface treatment to provide better skid resistance and to seal small cracks, an asphalt overlay in cases where riding quality needs to be restored and when it is necessary to prolong the fatigue life of the base, or recycling of the base when further overlays are no longer adequate.

(b) Tar-base pavements

The fatigue life of a tar premix is well below that of most bituminous materials. Only weakly cemented subbases are used. The main distress appears to be cracking of the cemented subbase, followed by fatigue cracking of the tar base.

Maintenance for tar bases is the same as for bituminous bases.

7.2.3 Concrete pavements

In concrete pavements, most of the traffic loading is carried by the concrete slab and little stress is transferred to the subgrade. The cemented subbase provides a uniform foundation and limits pumping of subbase and subgrade fines. Through the use of tied shoulders, most of the distress stemming from the edge of the pavement can be eliminated and the slab thickness can also be reduced. Distress of the pavement usually appears first as spalling near the joints, and may then progress to cracking in the wheel paths. Once distress becomes evident, deterioration is usually rapid. See Figure 9(e).

Maintenance consists of patching, joint repair, crack repair, under-sealing, grinding, or thin concrete or bituminous overlays. In cases of severe distress, thick concrete, bituminous or granular overlays will be used, or the concrete may be recycled.

7.2.4 Cemented-base pavements

In these pavements, most of the traffic stresses are absorbed by the cemented layers and a little by the subgrade. It is likely that some block cracking will be evident very early in the life of cemented bases; this is caused by the mechanism of drying shrinkage and by thermal stresses in the cemented layers. Traffic-induced cracking will cause the blocks to break up into smaller ones. These cracks propagate through the surface. The ingress of water through the surface cracks may cause the blocks to rock under traffic resulting in the pumping of fines from the lower layers. Rutting or roughness will generally be low up to this stage but is likely to accelerate as the extent of the cracking increases. See Figure 9(d).

Pavements consisting of cemented bases on granular subbases are very sensitive to overloading and to ingress of moisture through the cracks. When both the base and the subbase are cemented, the pavement will be less sensitive to overloading and moisture. The latter type of pavement is generally used.

The initial cracks may be rehabilitated by sealing. Once traffic-load-associated cracking has become extensive, rehabilitation involves either the reprocessing of the base or the application of a substantial bituminous or granular overlay.

7.2.5 Paving blocks

Many types of interlocking and non-interlocking segmental blocks are used in a wide variety of applications which range from footpaths and driveways to heavily loaded industrial stacking and servicing yards. The use of segmental block pavements is a recent phenomenon in South Africa. The popularity of these blocks is increasing due to a number of factors:

- the blocks are manufactured from local materials;
- they can either provide a labour-intensive operation or can be manufactured and laid by machine;
- they are aesthetically acceptable in a wide range of applications, and
- they are versatile as they have some of the advantages of both flexible and concrete pavements.

In current practice a small plate vibrator is used to bed the blocks into a sand bedding of approximately 20 mm and to compact jointing sand between individual blocks. The selection of the right type of sand for these purposes is important since a non-plastic material serves best as bedding while some plastic content is required to fill the joints.

Properly laid block pavements are adequately waterproof and ingress of large quantities of water into the foundations does not occur. The procedures for the structural design of segmental block pavements are presented in UTG2 *Structural design of segmental block pavements in southern Africa*[13] and are applicable to both industrial and road uses.

Segmental blocks are manufactured with vertically square side faces. Those that interlock are shaped so as to allow them to fit "jigsaw" fashion into a paved area. They can be made of pressed concrete, fired-clay brick or any other material. The current recommended minimum strength for structural use is given as a wet compressive strength of not less than 25 MPa.

Block pavements require the paved area to be "contained" either by kerbs or by other means of stopping lateral spread of the block. This is a requirement for both interlocking and non-interlocking shapes. Lateral movements are induced by trafficking and these movements cause breaks in the jointing sand. The associated opening-up of the block pavement makes it more susceptible to the ingress of surface water. In heavily loaded areas interlocking shapes have advantages over non-interlocking shapes, especially if vehicles with a slewing action are involved.

Experience has shown that joints should be 2 to 5 mm wide. Geometric design should follow practices for other pavements. Variable road widths, cures and junctions do not present problems in practice, since the blocks are small and can easily be cut and placed to suit the geometry of the pavement. In practice the minimum cross-fall for block pavements should be one per cent. For wide areas of industrial paving, special care should be taken to ensure that the cross-fall of the surface is adequate. Cambered cross-sections are also satisfactory.

Edge restraint is required along the edges of a block pavement to prevent the outward migration of blocks, which would result in the opening of joints and loss of bond between the blocks. Edge restraint can be provided by means of conventional kerbing. The joints between the blocks seal better with time due to the action of weathering and the addition of road detritus to the joints, thereby improving the total strength of the block pavement. One per cent falls (minimum) to the surface of block pavements allow water to drain across the pavement, reducing ingress by absorption through the joints, and eliminate ponding. The joints between the blocks on steep gradients may form the drainage paths for rainwater. In such cases the pattern of the blocks is an important consideration. Experience has shown that a herringbone pattern is best for use on steep gradients and for industrial paving.

An advantage of the blocks is that they can be re-used. They can be lifted if repairs have to be carried out to failed areas of subbase or if services have to be installed and can be relaid afterwards. As far as the structural design of segmental block pavements is concerned, this re-use of the blocks has no disadvantages.

Little maintenance work is normally required with segmental block paving. Maintenance involves the treatment of weeds and the correcting of levels or surfacing if the initial construction had been poor. The correction of surface levels is done by removing the area of blocks affected, levelling the subbase, compacting the subbase (often with hand hammers) and replacing the blocks.

Segmental paving provides an exciting addition to the pavement construction methods possible in southern Africa.

7.2.6 Gravel roads

The two most common causes of poor performance of gravel roads are slipperiness and potholing when wet, and excessive dust and ravelling when dry. The formation of corrugations is normally the result of inadequate compaction or low plasticity combined with traffic. Frequent maintenance (eg grading, watering and the addition of material) is therefore necessary.

7.3 POSSIBLE CONDITION AT END OF STRUCTURAL DESIGN PERIOD

There is no design method available to predict the exact condition of a length of road 10 to 20 years in the future. However, as shown in the previous paragraph, certain kinds of distress can be expected in certain pavement types and account must be taken of such distress. Table 11 shows acceptable terminal conditions of rut depth and cracking for the various road categories and pavement types. Figure 10 demonstrates that the rut depth values in Table 11 actually represent ranges of failure conditions.

TABLE 11

Possible condition at end of structural design period for various road categories and pavement types

Possible condition at end of structural design period	Road category			
	UA	UB	UC	UD
Rut depth (mm)	20	20	20	20
Length of road exceeding stated rut depth (%) (refer to Figure 10)	10	15	25	40
Type of cracking:				
Granular base	Crocodile cracking, surface loss, pumping of fines			
Bituminous base	Crocodile cracking, pumping			
Concrete pavement	Slab cracking, spalling at joints, pumping of fines			
Cemented base	Block cracking, rocking blocks, pumping of fines			
Length of road on which stated types of cracking occur (%)	10	15	25	40

7.4 SELECTION OF PAVEMENT TYPES FOR DIFFERENT ROAD CATEGORIES, TRAFFIC CLASSES AND CLIMATIC REGIONS

The behaviour of different pavement types is dealt with in Subsection 7.2. Certain pavement types may not be suitable for some road categories, traffic classes or climatic regions. Pavement structures with thin, rigid or stiff layers at the top (shallow structures) are generally more sensitive to overloading than deep structures. If many overloaded vehicles can be expected, shallow structures should be avoided.

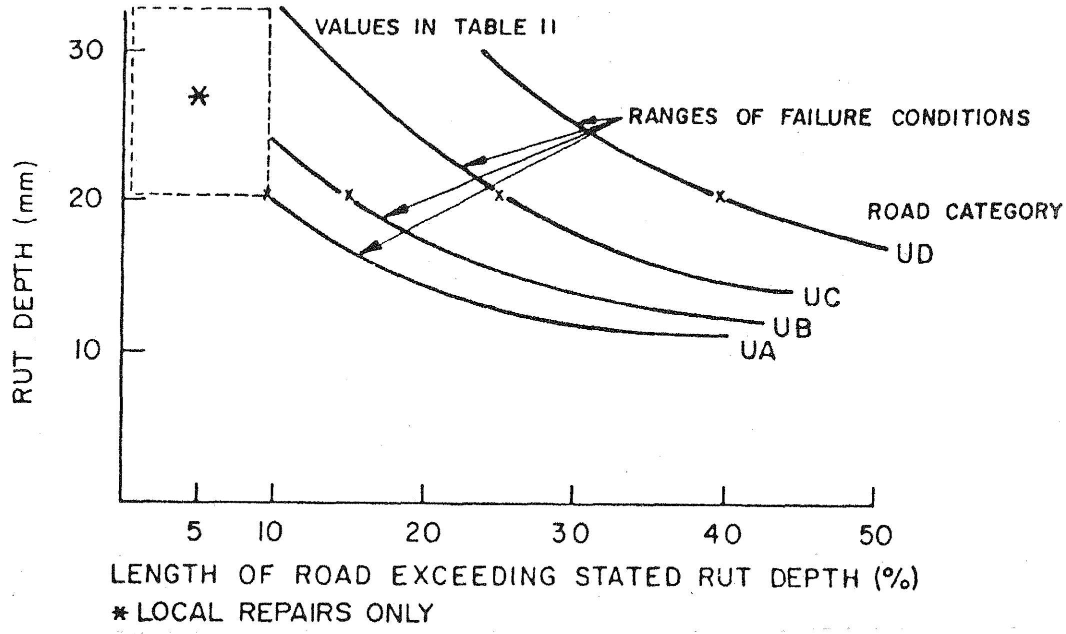


FIGURE 10

Ranges of terminal rut depth conditions for different road categories

Figure 9 indicates that the more rigid structures deteriorate rapidly once distress is shown, whereas the more flexible pavements generally deteriorate more slowly. Signs of distress are often more visible on rigid pavements.

Pavement structures consisting of water-susceptible materials may be undesirable for wet climatic regions, unless special provision is made for drainage.

Table 12 shows recommended pavement types (base and subbase) for different road categories and traffic classes. Reasons why certain pavement types are not recommended are also stated briefly.

7.5 THE CATALOGUE DESIGN METHOD

7.5.1 Introduction to the catalogue

Before the catalogue is used, all the factors noted in Sections 2 to 6 should be considered. By determining the road category, design strategy, design equivalent traffic, availability of materials and the pavement type, the designer can choose a pavement structure. It should be noted that these designs are considered adequate to carry the total design equivalent traffic over the structural design period. Construction constraints on practical layer thicknesses and increments in thicknesses are met. It is assumed that the requirements of the material standards are met. The catalogue may not be applicable when special conditions arise; other methods should then be used, but the catalogue

TABLE 12

Suggested pavement types for different road categories and traffic classes

Pavement types		Road category and traffic class									Abbreviated reasons why listed pavement types are *not* recommended for the given road category and traffic class
Base	Subbase	UA			UB			UC		UD	
		E4	E3	E2	E2	E1	EO	EO	ER	—	
Granular	Granular	X	X	√*	√*	√	√	√	√	√	Uncertain behaviour
	Cemented	√*	√	√	√	√	√	√	√	√	—
Bitumen	Granular	√	√	√	√	√	X	X	X	X	Too expensive
	Cemented	√	√	√	√	√	X	X	X	X	Too expensive
Tar	Granular	X	X	X	X	X	X	X	X	X	Fatigue life of tar premix too low, too expensive
	Cemented	X	√	√	√	√	X	X	X	X	E4: Uncertain behaviour, EO, ER too expensive
Concrete	Granular	X	X	X	X	X	X	X	X	X	Extra thickness required to prevent fatigue cracking, pumping and faulting
	Cemented	√	√	√	√	X	X	√ψ	√ψ	X	Too expensive, too difficult to trench
Cemented	Granular	X	X	X	X	X	√	√	√	√	Fatigue cracking, pumping and rocking blocks
	Cemented	X	X	√	√	√	√	√	√	√	Shrinkage cracks unacceptable
Paving blocks	Granular	X	X	X	√	√	√	√	√	√	Not recommended at high speeds
	Cemented	X	X	√	√	√	√	√	√	√	Not recommended at high speeds

√ = Recommended
x = Not recommended
* Not recommended for wet regions without special provision for drainage
ψ Only for steep gradients

can still act as a guide. The catalogue does not necessarily exclude other possible pavement structures.

7.5.2 Selected layers

The catalogue assumes that all subgrades are brought to equal support standards. Section 6 limits the design CBR of the subgrade to five groups (Table 10). Normally, the in situ subgrade soil will be prepared or ripped and recompacted to a depth of 150 mm. On top of this prepared layer, one or two selected layers will be added. The required selected subgrade layers will vary according to the design CBR of the subgrade. Table 13 shows the preparation of the subgrade and required selected layers for the different subgrade design CBRs.

7.5.3 Interpolation between traffic classes

The pavement structures in the catalogue are considered adequate to carry the total design traffic according to the upper value of the traffic classes defined in

TABLE 13

*Preparation of subgrade and required selected layers for the different subgrade design CBRs**

Design CBR of subgrade	< 3	3 – 7	7 – 15	15 – 25	> 25
Add selected layers:	Not applicable				
Upper		150 mm G7	150 mm G7	-	-
Lower		150 mm G9	-	-	-
Treatment of in situ subgrade	Special treatment required	Rip and recompact to 150 mm G10	Rip and recompact to 150 mm G9	Rip and recompact to 150 mm G7	Use as subbase or base layer**

* Not applicable to Category UD roads: for these use only one selected layer (G7) if required.

** Compacted to the appropriate density (refer to Table 10).

Section 4 (Table 3). The total design traffic may be predicted with more accuracy than is implied by the traffic classes. In such a case the designer may use a simple linear interpolation technique. In many designs the only difference between the structures for the various classes of traffic is a change in the layer thickness. In these cases the designer may use linear interpolation. However, there is often a change in material quality as well as in thickness. Simple interpolation is then inadequate and the designer will have to use other design methods[6,12].

7.5.4 Surfacings

Urban and residential roads carry both traffic and stormwater run-off. The traffic often consists of either large volumes of lightly loaded vehicles (eg on CBD roads) or virtually no trafficking (eg on residential roads and culs-de-sac). In both these cases a high-quality surfacing is required. Such a surfacing is also necessary because the road acts as a water channel.

The factors influencing the choice of surfacing are:

- local experience;
- availability;
- road category;
- design traffic class;
- environment (eg moisture, temperature and ultraviolet radiation);
- pavement type;

- turning movements, intersections, braking movements, and
- gradients.

The catalogue specifies the surfacing type, but allows a choice of surfacings for the lower categories of road. The controlling authority should select a surfacing from the catalogue that will give satisfactory performance.

If a waterbound macadam is used in the base in the place of a G1 to G4 material, the thin surfacings will be inadequate to provide an acceptable riding quality. In such cases, thicknesses of up to 50 mm of asphalt premix may be required.

7.6 GRAVEL ROADS

7.6.1 General

These unpaved roads may still be used under certain conditions. A brief description of design factors will follow. Normally, unpaved roads could be considered for use as Category UC or UD roads, although there may be special cases where they can be regarded as Category UB roads.

A gravel road may be regarded as a long-term facility or as an interim step towards a paved road. This will influence the level of the surface with regard to stormwater facilities.

7.6.2 Selected layers for gravel roads

The selected layers should be designed according to 7.5.2. Therefore, at subgrade level, there is no distinction between paved and unpaved roads. This follows from fundamental strength requirements and it will also simplify possible later changes from gravel to paved roads.

7.6.3 Design of gravel wearing course

The standards for gravel wearing courses are laid down in TRH14[10]. The quality and thickness of the wearing course may also depend on the design approach.

(a) The gravel road may be regarded as a long-term facility, in which case the most suitable wearing course will be selected for the prevailing conditions (climate, material availability and traffic). On more heavily trafficked roads, the gravel wearing course should be used as a future subbase. On urban access or residential roads it would constitute the future basecourse.

(b) The gravel wearing course may be regarded as an interim riding surface which will be overlaid or removed when the road is paved. If the gravel wearing course is going to be overlaid later, the material should comply with the subbase standards applicable to the future pavement. If the wearing course is to be removed later, consideration should be given to the inclusion of a proper subbase during construction, should such a subbase be necessary later.

(c) The gravel wearing course may be regarded as the base layer of the future paved road. This will normally be possible only for some Category UC or UD roads, but then special restrictions should be placed on the plasticity index of the fines.

The thickness of the gravel wearing course will depend on the road category and on the design approach chosen. Passability during the wet season is the best criterion to use in the design of a gravel road. It is best determined by the soaked laboratory CBR of the surfacing gravel material. Figure 11 gives a proposed limit related to the ADT[14].

When a gravel wearing course has to be provided the existing subgrade cannot support the traffic loads adequately under all climatic conditions. Therefore the gravel wearing course serves as protection to the subgrade. The thickness requirement for the gravel wearing course can be determined from empirical models. Figure 12 shows a model which gives the minimum cover thickness for adequate protection of the subgrade[14]. When gravelling, the thickness of gravel should be the minimum cover thickness plus provision for gravel loss until the road has been regravelled (usually for a six-year life). In Table 14 the thickness requirement is also given in general relation to the road category. A distinction is made in each of these functional classes between a long-term facility and an interim facility involving upgrading possibilities. These values have been selected to provide an adequate pavement, considering that some form of surface maintenance is usually applied and ruts as large as 75 mm are normally not tolerated in practice.

TABLE 14

Design thickness of gravel wearing course for different road categories

Road category	UB**		UC		UD	
Design approach	Long-term facility	Interim facility	Long-term facility	Interim facility	Long-term facility	Interim facility
Thickness of wearing course* (mm)	200	150	150	100	125	100

* The quality of the wearing course may change depending on the climate, material availability and the design approach.

** Design traffic class >E1.

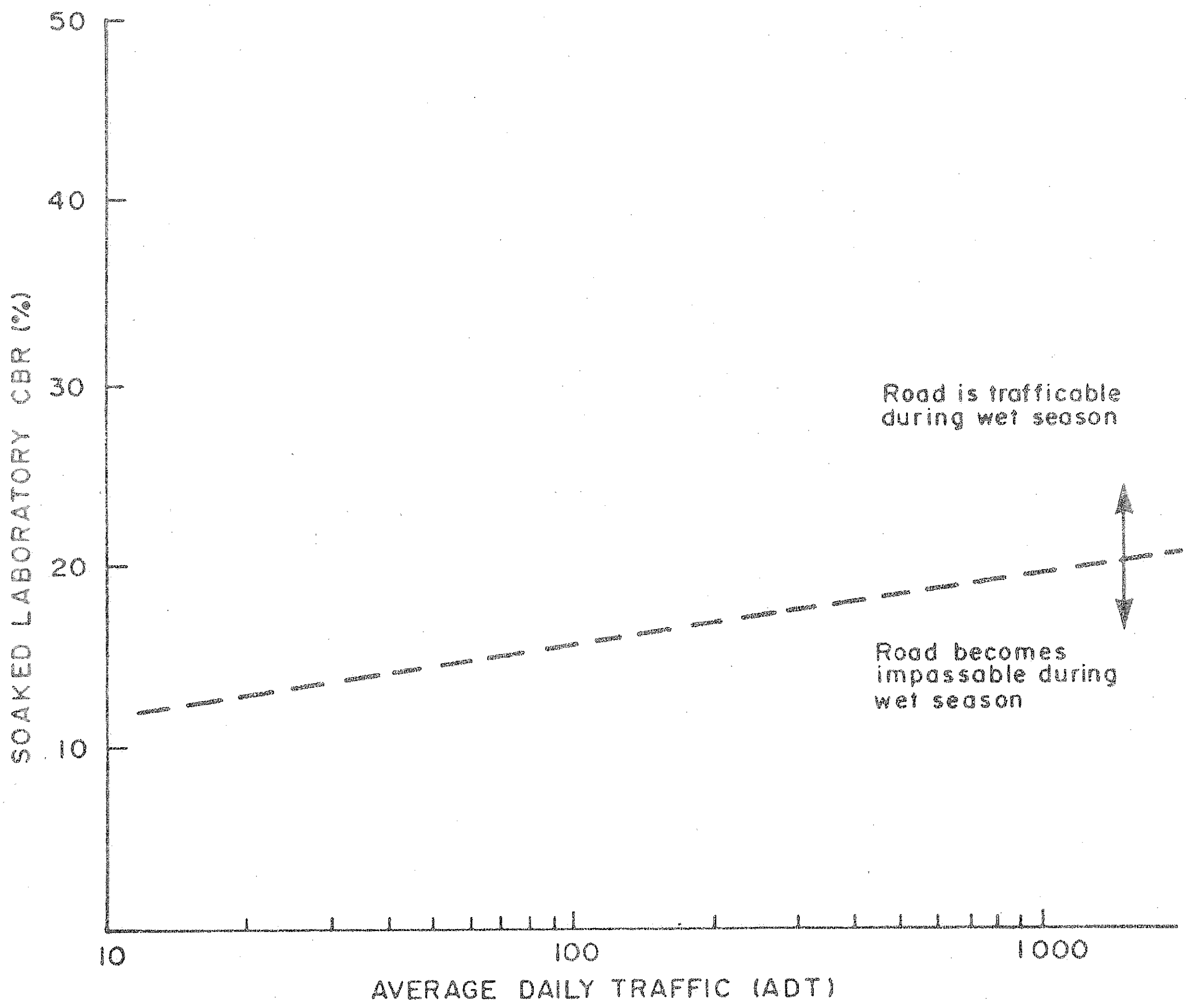


FIGURE 11

Design curve for the passability of unpaved roads

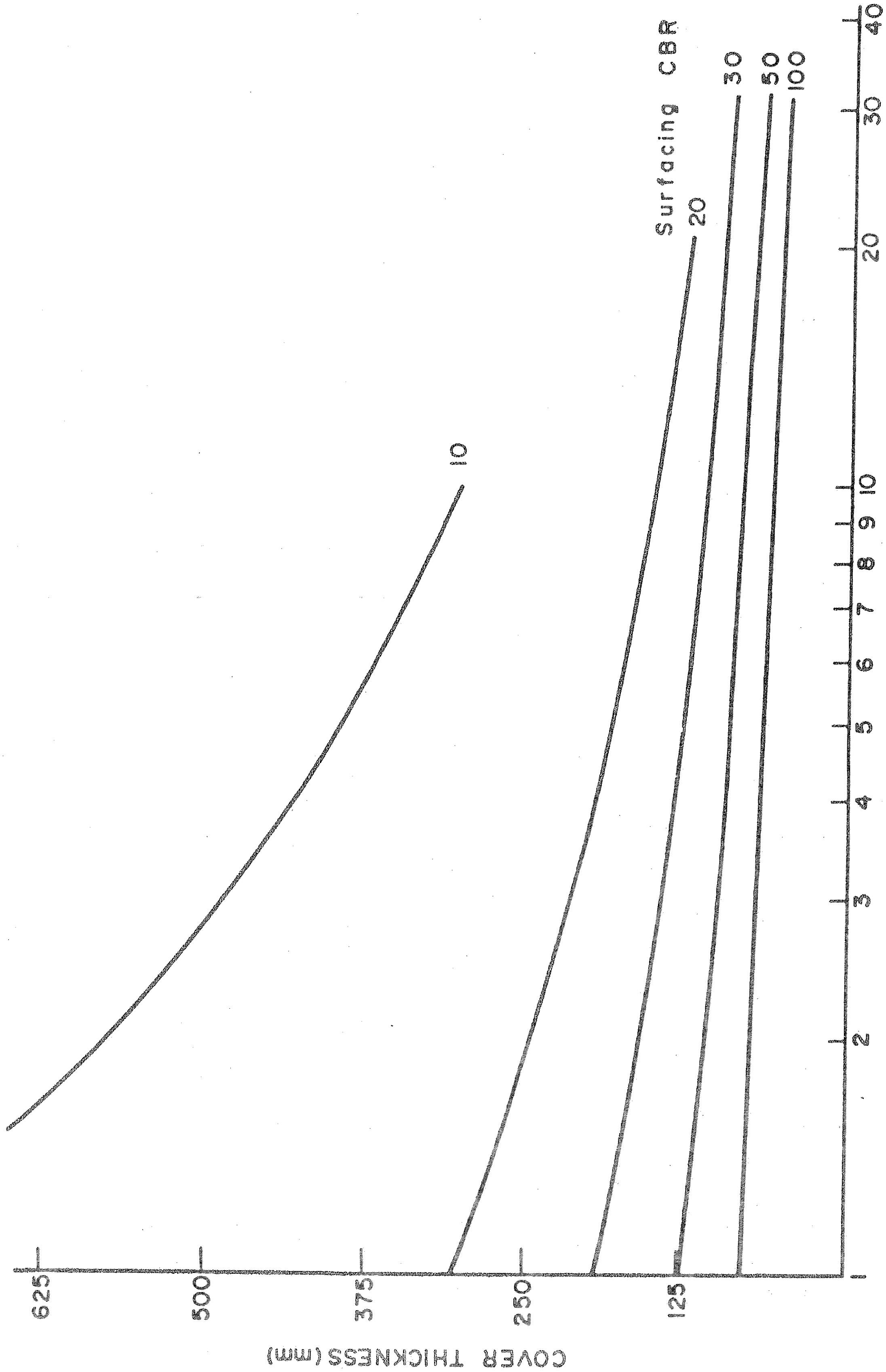


FIGURE 12

CBR design curves for cover thickness of gravel roads

8 PRACTICAL CONSIDERATIONS

8.1 DRAINAGE

Experience has shown that inadequate drainage is probably responsible for more pavement distress in southern Africa than inadequate structural or material design. Effective drainage is essential for good pavement performance, and it is assumed in the structural design procedure.

Drainage design is an extensive subject and both the discharge of surface run-off and the control of subsurface water have to be considered[16,17*]. Here the basic philosophy is to prove effective drainage to at least material depth so that the pavement structure does not become saturated.

8.2 COMPACTION

The design procedures assume that the specified material properties are met in the field.

TABLE 15

*Compaction requirements for the construction of pavement layers**
(and reinstatement of pavement layers)

Pavement layer	Compacted density	
Surfacing	Bituminous	95 % 75-blow Marshall
Base (upper and lower)	Crushed stone	G1 86 % to 88 % apparent density G2 100 % to 102 % Mod: AASHTO
	Crushed stone G3 and gravel G4	98 % Mod AASHTO
	Bituminous	95 % 75-blow Marshall 92 % theoretical max
	Cemented	97 % Mod AASHTO
Subbase (upper and lower)		95 % Mod AASHTO
Selected subgrade		93 % Mod AASHTO
Subgrade (within 200 mm of selected subgrade)		90 % Mod AASHTO
(within material depth)		85 % Mod AASHTO
Fill		90 % Mod AASHTO
(cohesionless sand)		(100 % Mod AASHTO)

* Refer to TRH14[10]

Table 15 gives the minimum compaction standards required for the various layers of the pavement structure. Note that below base level the standards are independent of the type of material used.

8.3 SUBGRADE BELOW MATERIAL DEPTH**

Special subgrade problems may arise that require individual treatment. The design procedure assumes that these have been taken into account separately. The main problems that have to be considered are:

- the extreme changes in volume that occur in some soils as a result of moisture changes, eg in expansive soils and soils with collapsible structures;
- flaws in structural support, eg sinkholes, mining subsidence and slope instability;
- the non-uniform support that results from wide variations in soil types or states;
- the presence of soluble salts which, under favourable conditions, may migrate upwards and cause cracking, blistering or loss of bond of the surfacing, disintegration of cemented bases and loss of density of untreated bases, and
- the excessive deflection and rebound of highly resilient soils during and after the passage of a load, eg in ash, micaceous and diatomaceous soils.

The techniques available for terrain evaluation and soil mapping are given in TRH2[8]. The design of embankments should be done in accordance with TRH10[9].

8.4 ROAD LEVELS

The fact that the provision of vehicular access to adjoining roads, dwellings and commercial establishments is the primary function of an urban road, means that road levels become a rather more important factor in urban areas than they are in rural or interurban road design. Urban road levels place some restrictions on rehabilitation and create special moisture/drainage conditions.

In some cases, rehabilitation in the form of an overlay may cause a problem, particularly with respect to the level of kerbs and channels, camber and overhead clearances. In these cases strong consideration should be given to bottom-heavy designs, ie designs with a cemented subbase and possibly a cemented base, which would mainly require the same maintenance as thin surfacings and little structural maintenance during the analysis period.

Urban roads are frequently used as drainage channels for surface water run-off. This is in sharp contrast with interurban and rural roads which are usually raised to shed the water to side table drains some distance from the road shoulder.

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* UTG document in preparation (Urban Stormwater Management)

** Refer to 5.4

8.5 SERVICE TRENCHES

Trenches excavated in the pavement to provide essential services (electricity, water, telephone, etc) are frequently a source of weakness. This is as a result of either inadequate compaction or saturation of the backfill material.

Compaction must achieve at least the minimum densities specified in the catalogue of designs and material standards (Table 15). These densities are readily achieved when granular materials are used, but it become much more difficult when natural materials are used, particularly in the case of excavated clays. When dealing with clay subgrades it is recommended that if it is economically feasible, a moderate-quality granular material be used as a trench backfill in preference to the excavated clay. In roads of Category UB and higher it is preferable to stabilize all the backfill material and in lower categories the provision of a stabilized "cap" over the backfill may be considered to eliminate settlement as far as possible.

Service trenches can also be the focal points of drainage problems. Settlement in the trench, giving rise to standing water and possibly to cracking of the surface, will permit the ingress of moisture. Fractured water, sewerage or stormwater pipes lead to saturation in the subgrade and possibly in the pavement layers as well.

Alternatively, a trench backfilled with granular material may even act as a subsurface drain, but then provision for discharge must be made.

8.6 PAVEMENT CROSS-SECTION

Generally it is preferable to keep the design of the whole carriageway the same with no change in layer thickness across the road. However, where there are significant differences in the traffic carried by individual lanes, eg in climbing lanes, the pavement structure may be varied over the cross-section of the carriageway, provided that this is economical and practical. Under these circumstances, the actual traffic predicted for each lane should be used in determining the design traffic.

The cross-section can be varied by means of steps in the layer thickness. Under no circumstances should the steps be located in such a way that water can be trapped in them. A typical pavement cross-section for a paved urban road is shown in Figure 14.

8.7 CONSIDERATIONS FOR CONCRETE PAVEMENTS

Construction details[11] are beyond the scope of this document. However, the following are some basic practical recommendations:

(a) The subgrade should be prepared to provide a uniform support.

(b) The subbase should be stabilised to a high quality to provide a non-pumping, erosion-resistant, homogeneous pavement support.

(c) When jointed concrete pavements are used, attention should be given to the detail of the spacing, type of joint and joint sealing of such joints.

8.8 KERBS AND CHANNELS

Kerbs and channels are important to prevent edge erosion and to confine stormwater to the road surface.

Consideration should be given to the type and method of construction of kerbs when deciding on a layer thickness for the base.

It is common practice to construct kerbs upon the (upper) subbase layer to provide edge restraint for a granular base. This restraint is beneficial for achievement of specified density and strength.

In the case of kerbing with as-fixed size (ie precast kerbing or kerbing with fixed shutters cast in situ) it may be advantageous to determine the base thickness from the kerb size, eg if the design calls for a 30 mm AG underlain by a 125 mm G4, and the gutter face is 160 mm, use a 130 mm G4.

8.9 EDGING

Instead of kerbs, edging could be used for certain low-traffic roads when the shoulder or sidewalk soil is of adequate stability. The shoulder should be shaped to the correct level and the edge may be sealed by a prime coat, a sand seal, a slurry seal or a premix. A degree of saving may be possible by utilizing trimmed grass verges where longitudinal gradients are slight and stormwater flows are low.

8.10 ACCESSIBILITY

Access to dwelling units should be provided in such a way as to provide adequate sight distances and a smooth entry, but the access ways should at the same time keep stormwater on the road from running into adjacent properties.

At pedestrian crossings special sloped openings in the kerbs should be made provision for to accommodate the handicapped and hand-pushed carts.

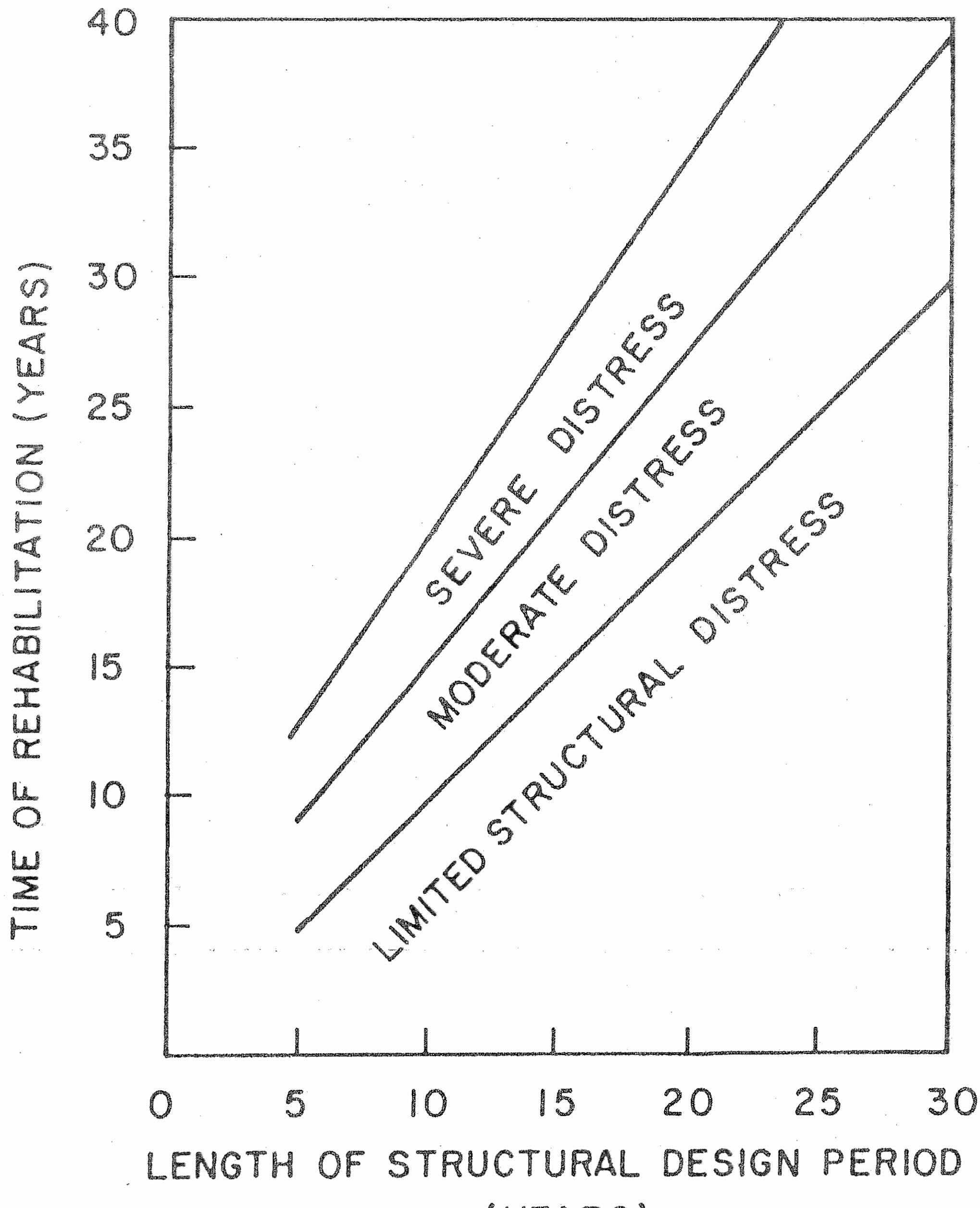
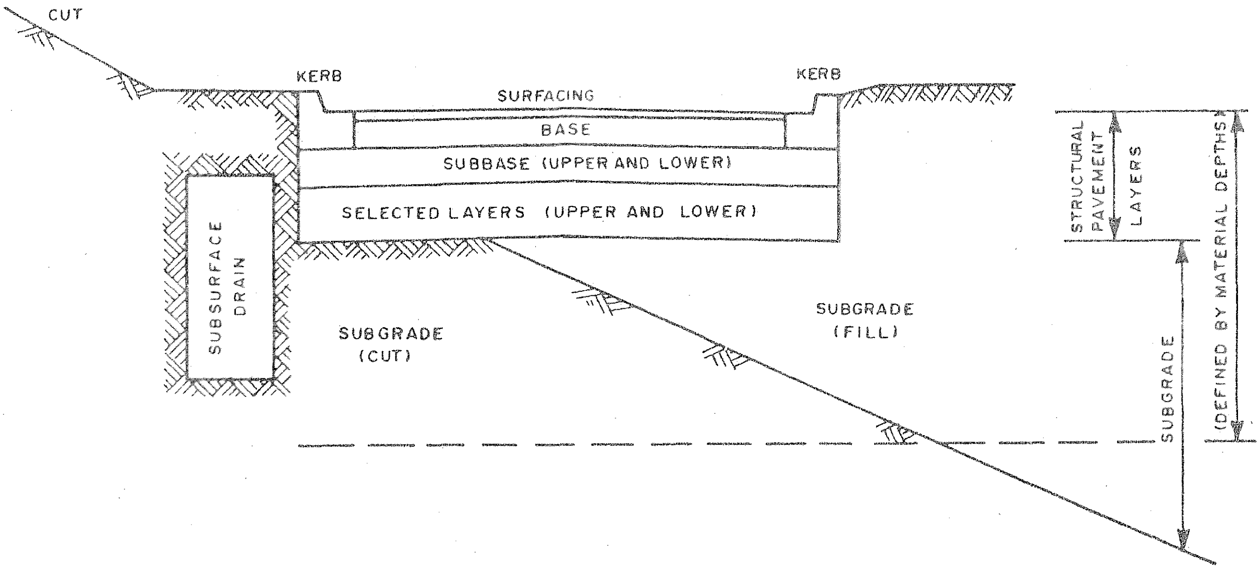


FIGURE 13

The degree of structural distress to be expected at the time of rehabilitation for different structural design periods



NOTE : THE PURPOSE OF THIS DIAGRAM IS TO ILLUSTRATE ALL THE DIFFERENT ASPECTS OF PAVEMENT STRUCTURE TERMINOLOGY. IT IS NOT NECESSARILY A REFLECTION OF NORMAL PRACTICE

FIGURE 14

Pavement structure terminology

9 COST ANALYSIS

9.1 GENERAL

Alternative pavement designs should be compared on the basis of cost. The cost analysis should be regarded as an aid to decision-making. However, a cost analysis does not necessarily include all the factors necessary to take a decision and it should therefore not override all other considerations. The main economic factors that determine the cost of a facility are the analysis period, the structural design period, the construction cost, the maintenance cost, the salvage value at the end of the analysis period and the real discount rate.

A complete cost analysis should be done for Category UA and UB roads. For Category UC and UD roads, only a comparison of the construction costs will normally suffice.

The method of cost analysis put forward in this document should only be used to compare pavement structures in the same road category. This is because roads in different categories are constructed to different standards and are expected to perform differently with different terminal levels. The effect these differences have on road user costs is not taken into account directly.

The choice of analysis period and structural design period will influence the cost of a road, but in Section 3 it was shown that the final decision is not necessarily based purely on economics.

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

9.2 PRESENT WORTH

The total cost of a project over its life is the construction cost plus maintenance costs, minus the salvage value. The total cost can be expressed in a number of different ways but for the purpose of this document, the present worth of costs (PWOC) approach has been adopted.

The present worth of costs can be calculated as follows:

$$PWOC = C + M_1\,(1 + r)^{-x_1} + \ldots M_j(1 + r)^{-x_j} + \ldots - S(1 + r)^{-z} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad 9.1$$

where PWOC = present worth of costs

C = present cost of initial construction

M_j = cost of the j^{th} maintenance measure expressed in terms of current costs

r = real discount rate

x_j = number of years from the present to the j^{th} maintenance measure, within the analysis period

z = analysis period

S = salvage value of pavement at the end of the analysis period expressed in terms of the present value.

If the difference in present worth of costs between two designs is 10 % or less, it is assumed to be insignificant, and the present worth of costs of the two designs is taken to be the same.

A computer program can be designed to facilitate the calculation of the present worth of costs.

9.3 CONSTRUCTION COSTS (C)

The check-list of unit costs given in Section 5 should be used to calculate the equivalent construction cost per square metre. Factors to be considered include the availability of natural or local commercial materials, their expected trends in costs, the conservation of aggregates in certain areas and also practical aspects, such as speed of construction and the need to foster the development of alternative pavement technologies.

The cost of excavation should be included as certain pavement types will involve more excavation than others.

9.4 REAL DISCOUNT RATE (r)

When a present worth analysis is done, a real discount rate must be selected to express future expenditure in terms of present-day values. This discount rate should correspond to the rate generally used in the public sector. In 1980 this was about 10 % in real terms (ie after compensating for the effect of inflation). Unless the client clearly indicates that he prefers some other rate, 10 % is recommended for general use. A sensitivity analysis could be carried out to determine the importance of the value of the discount rate.

9.5 FUTURE MAINTENANCE (M_j)

Maintenance management or maintenance design is beyond the scope of this document. However, it has been shown in Section 7 that there is a relation between the type of pavement and the maintenance that might be required in future. When different pavement types are compared on the basis of cost, these future maintenance costs should be included in the analysis to ensure that a sound comparison is made.

Figures 4 and 9 show that the life of the surfacing plays an important part in the behaviour of some pavements. For this reason, planned maintenance of the surfacing is very important to ensure that these pavements perform satisfactorily. The lives of the various types of surfacing will depend on the traffic and the type of base used. Table 16 gives guidelines regarding the range of typical surfacing lives that can be expected of various surfacing types. These values may be used for a more detailed analysis of future maintenance costs.

Typical maintenance measures that can be used for the purpose of cost analysis are given in Table 17. It should be noted that since the costs are discounted to the present worth, the precise selection of the maintenance measure is not very important. Some maintenance measures are used more commonly on specific pavement types and this is reflected in Table 17. There are two types of maintenance measure:

- measures to improve the condition of the surfacing, and
- structural maintenance measures applied at the end of the structural design period.

TABLE 16

Typical ranges of surfacing life periods for various surfacing types for the different road categories and base types (if the surfacings are used as given in the catalogue)

Base type	Surfacing type	Typical range of surfacing life (years) — Road category and traffic: UA (E2-E4)	UB (EO-E2)	UC (ER)
Granular	Slurry seal	-	-	4 – 13
	PVC-tar single surface treatment	-	-	7 – 13
	Bitumen single surface treatment	-	6 – 11	10 – 16
	Bitumen double surface treatment	-	7 – 14	10 – 18
	Cape seal	-	7 – 15	10 – 18
	Open-graded asphalt*	6 – 10	7 – 12	-
	Thin continuously graded asphalt	9 – 13	10 – 15	13 – 22
	Thin gap-graded asphalt	9 – 15	11 – 16	14 – 23
Bituminous	Slurry seal	-	-	**
	PVC-tar single surface treatment	-	-	**
	Bitumen single surface treatment	-	-	**
	Bitumen double surface treatment	-	-	**
	Cape seal	-	-	**
	Open-graded asphalt*	6 – 10	7 – 12	**
	Thin continuously graded asphalt	9 – 14	10 – 16	**
	Thin gap-graded asphalt	9 – 16	11 – 17	**
Cemented	Slurry seal	**	-	4 – 13
	PVC-tar single surface treatment	**	-	7 – 13
	Bitumen single surface treatment	**	6 – 10	9 – 15
	Bitumen double surface treatment	**	7 – 11	10 – 15
	Cape seal	**	7 – 12	10 – 17
	Open-graded asphalt*	**	7 – 12	-
	Thin continuously graded asphalt	**	8 – 13	10 – 20
	Thin gap-graded asphalt **	**	9 – 14	11 – 21

* A surface type not normally used.
** On top of a continuously graded or gap-graded asphalt.

TABLE 17

Typical future maintenance for cost analysis

Base type	Typical maintenance measures*			
	Measures to improve the surfacing condition**		Structural maintenance	
	Original surfacing Surface treatment	Asphalt premix	Moderate distress	Severe distress
Granular	S1 (10-15 yrs) S1 (18-27 yrs)	S1 (12-20 yrs) S1 (21-30 yrs) or AG (13-22 yrs) AG (24-33 yrs)	30-40 AG, AC	> 100 BS, BC or Granular overlay or Recycling of base
Bituminous	—	S1 (13-17 yrs) S1 (22-28 yrs) or AG (13-17 yrs) AG (26-34 yrs)	30-40 AG, AC	> 100 BS, BC or Recycling of base
Concrete	Joints repair, surface texturing (15 yrs, 30 yrs) (equivalent to cost of 20 mm PCC)		Further joint and surface repairs	Concrete granular or Bituminous overlay or Recycling
Cemented	S1 (8 -13 yrs) S1 (16-24 yrs) S1 (23-30 yrs)	S1 (8-13 yrs) S1 (16-24 yrs) S1 (23-30 yrs)	Further surface treatments	Thick granular overlay or Recycling of base
Paving blocks	No maintenance measures		Relevelling of blocks	Rebuild base, bedding sand and blocks

* S1 (10 yrs) represents a single surface treatment at 10 years and 40 AG (20 yrs) represents a 40 mm thick bitumen surfacing at 20 years.

** Refer to Table 16 for typical surfacing lives.

The structural design period (SDP) has been defined (Section 3) as the period for which it is predicted with a high degree of confidence that no structural maintenance will be required. Therefore typical structural maintenance will on average only be necessary at a later stage. If structural maintenance is done soon after the end of the structural design period, the distress encountered will only be moderate. When structural maintenance is done much later the distress will generally be more severe. Figure 13 gives guidelines regarding the degree of distress to be expected at the time of rehabilitation for different structural design periods. Table 17 makes provision for both moderate and severe distress.

The typical maintenance measures given in Table 17 should be replaced by more accurate values if specific knowledge about typical conditions is available.

Road user delay costs should also be considered, although no proper guideline is readily available. The factors that determine the road user costs are:

- running costs (fuel, tyres, vehicle maintenance and depreciation) which depend mostly on the road alignment, but also on riding quality (PSI);
- accident costs, which depend on road alignment and riding quality, and
- delay costs, which depend on the maintenance measures applied and the traffic situation on the roads. (This is a difficult factor to assess as it may include aspects such as the provision of detours.)

9.6 SALVAGE VALUE (S)

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

(a) Where the existing pavement is left in position and an overlay is constructed, the salvage value of the pavement would be the difference between the cost of constructing an overlay and the cost of constructing a new pavement to a standard equal to that of the existing pavement with the overlay. This is termed the residual structural value.

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

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

The salvage values of individual layers of the pavement may differ considerably, from estimates as high as 75 % to possibly as low as 10 %. The residual salvage value of gravel and asphalt layers is generally high, whereas that of concrete pavements can be high or low depending on the condition of the pavement and the method of rehabilitation. The salvage value of the whole pavement would be the sum of the salvage values of the individual layers. In the absence of better information, a salvage value of 30 % of initial construction cost is recommended.

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APPENDIX 1

GLOSSARY

Analysis period – A selected period over which the present worth of construction costs, maintenance costs (including user costs) and salvage value are calculated for alternative designs and during which full reconstruction of the pavement is undesirable.

Base – the layer(s) immediately beneath the surfacing and on top of the subbase or, if there is no subbase, on top of the subgrade.

Behaviour – the function of the condition of the pavement with time.

Commercial vehicle – a vehicle with a carrying capacity exceeding 2 700 kg, usually with dual rear wheels.

Design CBR of subgrade – the representative laboratory California Bearing Ratio value for the subgrade which is used in the structural design.

Design cumulative equivalent traffic (N_e) - the equivalent traffic predicted for the structural design period.

Distress – the visible manifestation of the deterioration of the pavement in respect of either serviceability or structural capacity.

Equivalent traffic – the number of 80 kN single-axle loads (E80s) that cause the same cumulative damage as the actual traffic.

Equivalent vehicle unit (evu) – the number of through-moving passenger cars a given vehicle is equivalent to, based on its headway and delay-creating effects.

Geometric design – the design of the geometry of the road surface for traffic flow, and for the safety and convenience of the road user.

Initial equivalent traffic – the average daily equivalent traffic predicted for the first year of the structural design period.

Material depth – the depth defining the pavement, and the minimum depth within which the material CBR should be at least 3 % at in situ density.

Mechanistic analysis – analysis of a system taking into account the interaction of various structural components as a mechanism, here used to describe a design procedure based on fundamental theories of structural and material behaviour in pavements.

Modified aterial – a material of which the physical properties have been improved by the addition of a stabilizing agent but in which cementation has not occurred.

Pavement layers – the combination of material layers constructed over the subgrade in order to provide an acceptable facility on which to operate vehicles.

Performance – the measure of satisfaction given by the pavement to the road user over a period of time, quantified by a serviceability/age function.

Present worth of costs – sum of the costs of the initial construction of the pavement, the later maintenance costs and the salvage value discounted to a present monetary value.

Riding quality – the general extent to which road users experience a ride that is either smooth and comfortable or bumpy and thus unpleasant and perhaps dangerous.

Selected layer – the lowest of the pavement layers comprising controlled material, either in situ or imported (Classification codes G7-G10).

Serviceability – the measure of satisfaction given by the pavement to the road user at a certain time, quantified by factors of riding quality and rut depth.

Slab – the pavement layer of concrete which is placed over a prepared subbase and that acts as base and surfacing combined.

Structural design – the design of the pavement layers for adequate structural strength under the design conditions of traffic loading, environment and subgrade support.

Structural design period – the chosen minimum period for which the pavement is designed to carry the traffic in the prevailing environment with a reasonable degree of confidence that structural maintenance will not be required.

Structural distress – distress pertaining to the load-bearing capacity of the pavement.

Structural maintenance – measures that will strengthen, correct a structural flaw in, or improve the riding quality of an existing pavement, (eg overlay, smoothing course and surface treatment, partial reconstruction (of say the base and surfacing), etc).

Subbase – the layer(s) beneath the base or concrete slab and on top of the selected layer.

Subgrade – the completed earthworks within the road prism prior to the construction of the pavement. This comprises the in situ material of the roadbed and any fill material. In structural design only the subgrade within the material depth is considered.

Subgrade design unit – a section of subgrade with uniform properties and/or load-bearing capacity.

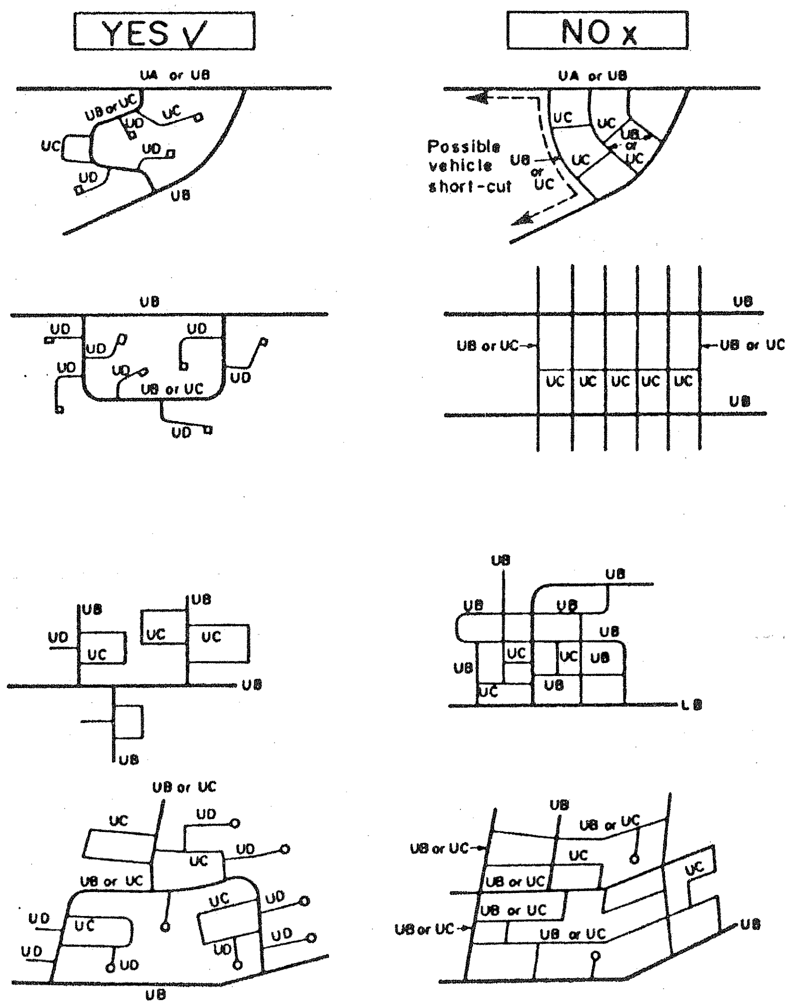
Surfacing – the uppermost pavement layer which provides the riding surface for vehicles.

Surfacing maintenance – a measures that maintain the integrity of the surface in respect of skid resistance, disintegration and permeability without necessarily increasing the structural strength of the pavement.

APPENDIX 2
THE INFLUENCE OF TOWNSHIP LAYOUT ON THE CHOICE OF ROAD CATEGORY

Township layout has a strong influence on the choice of road category, especially as regards the roads within the township. A well-designed layout will provide for bus routes (Category UB) within acceptable distances from all dwellings, but also on well-defined roads. This implies that all other roads within the township will be either Category UC or UD roads. However, a less-defined layout may imply that a number of roads could act as bus routes, so that they all have to be designed to carry buses, and these therefore have to be Category UB routes.

Figure 15 illustrates this principle.



NOTE: Layouts in the YES column maximize the use of UC and UD category roads. Those in the NO column make it impossible to use the UD category because of the difficulty of predicting traffic loading.

FIGURE 15

The influence of layout on road category

APPENDIX 3

EXAMPLE OF THE STRUCTURAL DESIGN OF A CATEGORY UB ROAD

1 OBJECTIVE

A district distributor road linking a township to the existing primary distributor system of Johannesburg has to be designed. It will be a four-lane facility and the following information is available:

(a) Centre line subgrade CBR values:
3; 5; 4; 5; 5; 6; 3; 12; 10; 7; 8; 11; 14; 20; 23; 20; 18; 19; 15; 16; 10; 12; 10; 10; 8; 9; 8; 7; 4; 5; 3; 6; 6; 7

(b) Current traffic (from a transportation study):
Equivalent traffic = 200/day/direction, consisting mostly of cars, a number of buses (70 % laden) and light as well as medium heavy commercial vehicles.

Expected growth rate = between 2 and 8 %.

(c) Road will be opened to traffic within three years.

Design pavement structures of different base types and compare these structures on the basis of costs before making a final selection.

2 ROAD CATEGORY

2.1 Road category (Table 1)

This can be regarded as a Category UB road. The cumulative design equivalent traffic should therefore be between 0,05 and 10^6 and 3 x 10^6 E80s/lane over the structural design period. A moderate level of service (in terms of riding quality) is expected.

3 DESIGN STRATEGY

3.1 Analysis period (Table 2)

The new alignment will probably not change again and therefore a period of 30 years is selected.

3.2 Select structural design period (Table 2)

A period of 15 years is selected. A longer period could have been selected, but as there is some uncertainty about the growth in traffic and future development in the area, a period of 15 years is more suitable.

Therefore AP = 30 years; SDP = 15 years.

4 DESIGN TRAFFIC

The lane distribution factor (B_e) from Table 8 is 0,95 for the slow lane. The fast lane will be designed for the same traffic as the slow lane. The current equivalent traffic may be projected to the initial year using the growth factor (g_e) from Table 6. The cumulative equivalent traffic over the structural design period can be determined by multiplying the initial equivalent traffic by the cumulative growth factor (f_e) from Table 7. The growth rate of the traffic is uncertain and it is necessary to do a sensitivity analysis with growth rates ranging from 2 % to 8 %. Table 18 shows the cumulative equivalent traffic and the applicable traffic class for a structural design period of 15 years. Regardless of the selected growth rate, the design traffic class is E2.

TABLE 18

Cumulative equivalent traffic and applicable traffic class for SDP = 15 years

Lane distribution factor	Growth rate %	g_x*	f_y**	Cumulative equivalent traffic over SDP	Design traffic class
0,95	2	1,06	6 440	$1,3 \times 10^6$	E2
	4	1,12	7 600	$1,6 \times 10^6$	E2
	6	1,19	9 010	$2,0 \times 10^6$	E2
	8	1,26	10 700	$2,6 \times 10^6$	E2

* for x = 3 years
** for y = 15 years

5 MATERIALS

The table in Figure 16 may be filled in by using the check-list in Figure 7. The unit prices listed are 1983 prices.

6 ENVIRONMENT

6.1 Climatic region

According to Figure 8, the road lies within the moderate climatic region.

6.2 Delineation of subgrade areas and design CBR of subgrade

By visual inspection of the given CBR values, five subgrade areas can be delineated:

Subgrade Area 1: CBR = 3; 5; 4; 5; 5; 6; 3
Subgrade Area 2: CBR = 12; 10; 7; 8; 11; 14
Subgrade Area 3: CBR = 20; 23; 20; 18; 19; 15; 16
Subgrade Area 4: CBR = 10; 12; 10; 10; 8; 9; 8; 7
Subgrade Area 5: CBR = 4; 5; 3; 6; 6; 7

SYMBOL	CODE	MATERIAL	AVAILABILITY	UNIT COST /m^3
	G1	Graded crushed stone	✓	R28,00
	G2	Graded crushed stone	X	—
	G3	Graded crushed stone	X	—
	G4	Natural gravel	X	—
	G5	Natural gravel	✓	R15,00
	G6	Natural gravel	X	—
	G7	Gravel-soil	✓	R8,0
	G8	Gravel-soil	X	—
	G9	Gravel-soil	✓	R3,50
	G10	Gravel-soil	✓	R3,50
	BC	Bitumen hot-mix asphalt	✓	R87,50
	BS	Bitumen hot-mix asphalt	✓	R87,50
	TC	Tar hot-mix asphalt	✓	R85,00
	TS	Tar hot-mix asphalt	✓	R85,00
	PCC	Portland cement concrete	✓	R 80,00
	C1	Cemented crushed stone or gravel	✓	R40,00
	C2	Cemented crushed stone or gravel	X	—
	C3	Cemented natural gravel	✓	R25,00
	C4	Cemented natural gravel	✓	R21, 00
	AG	Asphalt surfacing	✓	R105,00
	AC	Asphalt surfacing	✓	R105,00
	AS	Asphalt surfacing	✓	R105,00
	AO	Asphalt surfacing	✓	R95 ,00
—	S1	Surface treatment	✓	R1,40 /m^2
—	S2	Surface treatment	✓	R2,10 / m^2
—	S3	Sand seal	X	—
—	S4	Cape seal	X	—
—	S5	Slurry	X	—
—	S6	Slurry	X	—
—	S7	Surface renewal	X	—
—	S8	Surface renewal	X	—
	WM	Waterbound macadam	X	—
	PM	Penetration macadam	X	—
	CB	Concrete paving blocks	X	—

FIGURE 16

Check-list of material availability and unit cost

Table 19 shows the calculation of the lower 10th percentile values and design CBR of the subgrade areas.

TABLE 19

Design CBR of the subgrade

Subgrade area	Mean value, $\bar{x}$	Standard deviation, S	Lower 10th percentile $\bar{x} - 1{,}282$ S	Design CBR of subgrade
1	4,4	1.1	3,0	3 – 7
2	10,3	2,6	7,0	7 – 15
3	18,7	2,7	15,3	> 15
4	9,3	1,6	7,2	7 – 15
5	5,2	1,5	3,3	3 – 7

7 STRUCTURAL DESIGN AND PAVEMENT TYPE SELECTION

7.1 Pavement type selection

From Section 7.2 of the text, it appears that all the pavement types are acceptable for the given conditions. No adverse climatic conditions are expected and the traffic loading shows no abnormal trends or distributions. Table 12 shows that cemented subbases are recommended for all base types, although granular subbases can also be used for bitumen premix and granular bases.

7.2 Selected layers

The selected layers necessary for the different subgrade areas are given in Table 20 (based on Table 15).

TABLE 20

Selected layers for the different subgrade areas

Subgrade area	Design CBR	Lower selected layer	Upper selected layer	Treatment of in situ subgrade
1	3 – 7	150 mm G9	150 mm G7	R+R* to 150 mm G10
2	7 – 15	-	150 mm G7	R+R to 150 mm G9
3	> 15	-	-	R+R to 150 mm G7
4	7 – 15	-	150 mm G7	R+R to 150 mm G9
5	3 – 7	150 mm G9	150 mm G7	R+R to 150 mm G10

* R + R = rip and recompact.

7.3 Possible pavement structures

Figure 17 shows possible pavement structures according to the catalogue* (Category UB road, class E2 traffic).

8 PRACTICAL CONSIDERATIONS

The designer should consider the consequences, if any, of the practical considerations outlines in Section 8 for the possible pavement structures. For example, if local materials are very sensitive to water, he may specify that special care should be given to the cross profile.

9 COST ANALYSIS

9.1 Construction cost

For the comparison of different pavement types, the construction costs of only the subbase, base and surfacing need to be considered. These costs follow directly from the unit costs given in Figure 14.

9.2 Future maintenance

The structural design period is 15 years and the analysis period is 30 years. The future maintenance can be estimated from Tables 16 and 17. This road is near a regional office and one can expect timely maintenance (say between 1,0 and 1,5 x SDP). The distress will therefore only be moderate (Figure 13). Table 21 shows estimated maintenance measures for the different pavement types.

9.3 Discount rate

A discount rate of 10 % is normally selected. (A sensitivity analysis with discount rates of 8, 10 and 12 % showed that there is little difference between 8 and 12 %).

9.4 Salvage value

The salvage value will probably vary with the type of pavement. Table 22 shows typical salvage values for the different pavement types.

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* Because the catalogue of designs is updated from time to time, the designs in this example may differ. The catalogue reflects the latest designs that have been accepted by the Highway Materials Committee.

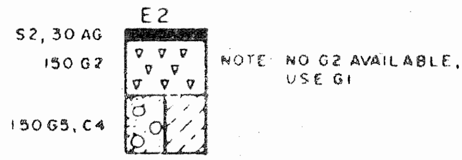
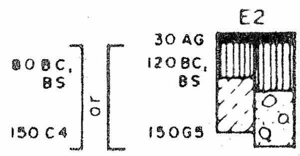
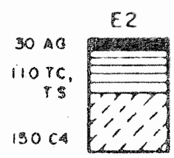
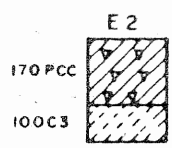
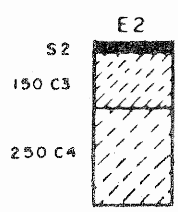
BASE	STRUCTURE
GRANULAR	
BITUMEN PREMIX	
TAR PREMIX	
CONCRETE PAVEMENT	
CEMENTED	

FIGURE 17

Possible pavement structures (SDP = 15 years)

TABLE 21

Typical maintenance measures for the different pavement types

Pavement type	Maintenance measures		
Base subbase	For surfacing	Structural maintenance	
Granular	Granular	30 AG (11 years)	40 AG (30 years)
	Cemented	30 AG (11 years)	35 AG (21 years)
Bitumen	Granular	30 AG (12 years)	35 AG (21 years)
Premix	Cemented	30 AG (12 years)	30 AG (21 years)
Tar premix	Cemented	30 AG (12 years)	30 AG (21 years)
Concrete	Cemented	Equivalent to cost of 20 PCC (10 years) 20 PCC (20 years)	150 PCC (25 years)
Cemented	Cemented	S1 (10 years) S1 (16 years) S1 (22 years)	150 G1 + S2 (25 years)

TABLE 22

Typical salvage values for different pavement types

Pavement type		Salvage value (% of initial costs)
Granular	: Granular	50 %
	Cemented	60 %
Bituminous	: Granular	50 %
	Cemented	60 %
Concrete		75 %*
Cemented		75 %*

* These values are high because of substantial overlays near the end of the analysis period.

9.5 Present worth of costs

The present worth of costs can be calculated from:

$$PWOC = C + \Sigma_1^n M_j (1 + r)^{-x\,j} - S(1 + r)^{-z} \quad \text{.......} \quad 9.1$$

Table 23 shows the present worth of costs for each pavement type.

TABLE 23

Present worth of costs

Pavement structure	Initial costs R/m²	Discounted maintenance costs: Measure	Discounted maintenance costs: Cost R/m²	Discounted salvage value R/m²	Present worth of costs R/m²
Asphalt surfacing	3,68	30 AG (12 years)	1,00		
Granular base	4,20	40 AG (23 years)	0,47		
Granular subbase	2,25				
	10,13		1,47	0,29	11,31
Asphalt surfacing	3,68	30 AG (13 years)	0,91		
Granular base	4,20	35 AG (25 years)	0,34		
Cemented subbase	3,15				
	11,03		1,25	0,38	11,90
Asphalt surfacing	3,15	30 AG (13 years)	0,91		
Bitumen premix base	10,50	35 AG (24 years)	0,37		
Granular subbase	2,25				
	15,90		1,28	0,50	16,68
Asphalt surfacing	3,15	31 AG (14 years)	0,83		
Bitumen premix base	7,00	30 AG (26 years)	0,26		
Cemented subbase	3,15				
	13,30		1,09	0,46	13,93
Asphalt surfacing	3,15				
Tar premix base	9,35	30 AG (12 years)	1,00		
Cemented subbase	3,15	30 AG (23 years)	0,35		
	15,65		1,35	0,54	16,46
Concrete pavement	14,40	20 PCC (15 years)	0,38		
Cemented subbase	2,50	180 PCC (25 years)	1,33		
	16,90		1,71	0,73	17,88*
Surface treatment	3,68	S1 (10 years)	0,54		
Cemented base	3,75	S1 (16 years)	0,30		
Cemented subbase	3,15	S1 (22 years)	0,17		
	10,58	150 GI (25 years)	0,39		
		35 AG (25 years)	0,34		
			1,74	0,45	11,87

* If concrete was designed for 20 years (say E3), this value would not be lower due to the extra initial costs.

Table 23 shows that the pavements with granular or cemented bases are significantly less expensive than the others. Therefore, a pavement with a granular or cemented base will be selected. As regards the behaviour of the different pavement types given in Section 7, it is proposed that a structure with a granular base and a cemented subbase be used. Such a pavement will probably not have the cracking problems of a pavement with a cemented base. A cemented subbase will be slightly more expensive than a granular subbase but is considered worth the extra cost.

The selected pavement could also have a double surface treatment instead of an asphalt premix surfacing. This would probably result in a slightly lower riding quality. The cost saving should be weighed against the lower riding quality, but the current policy of the Municipal Roads Department may also influence the decision.

10 ALTERNATIVE STRATEGIES

The overlays necessary for cemented-base and concrete pavements could be avoided by choosing a longer structural design period of 20 to 25 years. At a growth rate of 6 %, the design traffic class will be E3. An analysis has shown that the costs are slightly higher than the costs of the original strategy and that the granular base pavement remains the most cost-effective structure.

APPENDIX 4

EXAMPLE OF THE STRUCTURAL DESIGN OF CATEGORY UC AND UD ROADS WITHIN A TOWNSHIP

1 OBJECTIVE

The objective of this example is to describe the simplified design of Category UC or UD roads within a township – in comparison with the full design example given in Appendix 3.

2 ROAD CATEGORIES (Table 1, Figure 3)

The road categories (either UC or UD) are dependent on the township layout (as shown in Appendix 2) and on economic factors such as car ownership and use, affluence etc. If there is a high percentage of high-order roads, the designer could reconsider the township layout.

3 DESIGN STRATEGY

As explained in Figure 2 and Section 3, a fixed strategy is followed for Category UC or UD roads. A complete economic analysis over an analysis period of say 30 years is not required.

4 DESIGN TRAFFIC

For Category UC roads, the design traffic class is either ER ($< 0,05 \times 10^6$ E80s) or EO ($0,05$ to $0,20 \times 10^6$ E80s). Construction traffic is probably the most important factor influencing the decision about the design traffic class, but other factors such as car ownership and use* could influence the decision regarding the design traffic class. (Note: These roads will not carry bus traffic.) Should any of these roads act as a temporary Category UB road during a period of development, appropriate traffic studies should be done.

5 MATERIALS

As before.

6 ENVIRONMENT

As before.

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* CAMERON, J W M and DEL MISTRO, R F. *Traffic movements in residential environments: Six case studies in Pretoria.* NITRR Technical Report RT/35/81, Pretoria, CSIR, 1981.

7 STRUCTURAL DESIGN AND PAVEMENT TYPE SELECTION

As before – local experience of the behaviour of certain pavement types may exclude some designs.

8 PRACTICAL CONSIDERATIONS

As before.

9 COST ANALYSIS

A full cost analysis is not required. Only initial construction costs will be compared. However, local experience regarding certain pavement types may sometimes override decisions based on construction costs only.

APPENDIX 5

THE CATALOGUE OF DESIGNS

1 DESCRIPTION OF THE CATALOGUE

The catalogue deals with most of the factors that have to be considered by the designer. Firstly, there is the road category (Section 2) and the design equivalent traffic (Section 4) which depend on the design strategy and the structural design period (Section 3). For each road category there is usually a choice of two to three traffic classes. A variety of pavement types (Section 7) is available, although the availability and cost of materials and also experience regarding the materials have to be considered (Section 5). The subgrade has been treated separately (Section 6) and the catalogue assumes that all subgrades are brought to equal support standards (Section 7). The catalogue does not include practical considerations such as drainage, compaction or pavement cross-section. These aspects should still be considered and are covered in Section 8.

2 THE USE OF THE CATALOGUE AND SPECIAL CONDITIONS

The catalogue should not be used without considering the behaviour of the various pavement types, their possible condition at the end of the structural design period and the factors influencing the selection of pavement types for different road categories and traffic classes. The best results will probably be obtained if the catalogue is used together with some other design method. The catalogue does not necessarily exclude other design methods or other pavement structures.

The catalogue caters for conditions normally encountered in road building. Whenever special or unique conditions exist, the catalogue can be used as a guide or fist approximation, but it need not reflect the final design.

GRANULAR BASES
(MODERATE OR DRY REGIONS)

DATE 1984

ROAD CATEGORY	DESIGN TRAFFIC CLASS E80s / LANE OVER STRUCTURAL DESIGN PERIOD				
	E0 $<0,2 \times 10^6$	E1 $0,2-0,8 \times 10^6$	E2 $0,8-3 \times 10^6$	E3 $3-12 \times 10^6$	E4 $12-50 \times 10^6$
UA			30-40A† 125 G2 150 C4 30-40A† 150 G2 150 G5	40 A† 150 G1 125 C3 125 C4	50 A† 150 G1 150 C3 150 C3
UB		S† 125 G4 150 C4 S† 150 G4 150 G5	S or 30 A† 125 G2 125 C4 S or 30 A† 150 G3 150 G5	40 A† 150 G2 125 C4 125 C4	
UC	S† 100 G4 100 C4 S† 100 G4 125 G5	S† 100 G4 125 C4 S† 100 G4 150 G5	S† 125 G3 150 C4 S† 150 G3 150 G5		
UD	S1 150 G5	S1 125 G5 125 G6			

† SYMBOL A DENOTES AG, AC OR AS. SYMBOL S DENOTES S2 or S4
FOR SELECTED LAYERS REFER TO PARAGRAPH 7.5; FOR FUTURE MAINTENANCE TO PARAGRAPHS 7.2, 7.3 AND 9.5
AO PERMITTED AS A SURFACING MEASURE FOR SKID RESISTANCE OR REDUCTION OF WATER SPRAYING.

GRANULAR BASES

(WET REGIONS)

ROAD CATEGORY	DESIGN TRAFFIC CLASS E80s / LANE OVER STRUCTURAL DESIGN PERIOD				
	EO <0,2 X 10^6	E1 0,2-0,8 X 10^6	E2 0,8-3 X 10^6	E3 3-12 X 10^6	E4 12-50 X 10^6
UA			30-40A† 125 G1 150 C3	40A† 150 G1 125 C3 125 C4	50A† 150 G1 150 C3 150 C3
UB		S† 125 G2 150 C4 S† 150 G2 150 G5	30A† OR S† 125 G1 150 C4	40A† 150 G1 250 C4	
UC	S† 100 G4 100 C4 S† 100 G4 125 G5	S† 100 G2 125 C4 S† 100 G2 150 G5	S† 125 G2 150 C4 S† 150 G2 150 G5		

† SYMBOL A DENOTES AG, AC OR AS. SYMBOL S DENOTES S2 OR S4
FOR SELECTED LAYERS REFER TO PARAGRAPH 7.5; FOR FUTURE MAINTENANCE TO PARAGRAPHS 7.2, 7.3 AND 9.5.
AO PERMITTED AS A SURFACING MEASURE FOR SKID RESISTANCE OR REDUCTION OF WATER SPRAYING.

CEMENTED BASES

DATE
1984

ROAD CATEGORY	DESIGN TRAFFIC CLASS E80s / LANE OVER STRUCTURAL DESIGN PERIOD				
	E0 $< 0,2 \times 10^6$	E1 $0,2-0,8 \times 10^6$	E2 $0,8-3 \times 10^6$	E3 $3-12 \times 10^6$	E4 $12-50 \times 10^6$
UA					
UB		S2 / S4 150 C4 200 C4	S2 150 C3 250 C4	S2 / 30 AG 150 C3 300 C4	
UC	S1 100 C4 100 C4	S† 125 C4 125 C4	S† 150 C4 150 C4		
UD	S1 150 C4	S1 125 C4 125 C4			

†SYMBOL S DENOTES S2 OR S4
FOR SELECTED LAYERS REFER TO PARAGRAPH 7.5; FOR FUTURE MAINTENANCE TO PARAGRAPHS 7.2, 7.3 AND 9.5.
AO PERMITTED AS A SURFACING MEASURE FOR SKID RESISTANCE OR REDUCTION OF WATER SPRAYING.

BITUMEN HOT-MIX BASES

DATE
1984

ROAD CATEGORY	DESIGN TRAFFIC CLASS E80s / LANE OVER STRUCTURAL DESIGN PERIOD					
	< 0,2 X 10^6	0,2–0,8 X 10^6	0,8–3 X 10^6	3–12 X 10^6	12–25 X 10^6	25–50 X 10^6
UA				40 AG 90 BC/BS 125 C3 125 C4 40 AG 180 BC/BS 150 G5	40 AG 100 BC 150 C3 150 C4 40 AG 200 BC 150 G5	40 AG 120 BC 150 C3 150 C4 40 AG 225 BC 150 G5
UB			30 AG 90 BC/BS 150 C4 30 AG 120 BC/BS 150 G5	30 AG 80 BC/BS 125 C3 125 C4 30 AG 155 BC/BS 150 G5		
UC			S[†] 80 BC/BS 150 C4 S[†] 120 BC/BS 150 G5			

† SYMBOL A DENOTES AG, AC OR AS. SYMBOL S DENOTES S2 OR S4
FOR SELECTED LAYERS REFER TO PARAGRAPH 7.5; FOR FUTURE MAINTENANCE TO PARAGRAPHS 7.2, 7.3 AND 9.5.
AO PERMITTED AS A SURFACING MEASURE FOR SKID RESISTANCE OR REDUCTION OF WATER SPRAYING

TAR HOT-MIX BASES

DATE 1984

ROAD CATEGORY	DESIGN TRAFFIC CLASS E80s /LANE OVER STRUCTURAL DESIGN PERIOD				
	E0 <0,2 X 10^6	E1 0,2-0,8 X 10^6	E2 0,8-3 X 10^6	E3 3-12 X 10^6	E4 12-50 X 10^6
UA				40 AG 180 TC 150 C4 40 AG 120 TC 125 C3 125 C4	
UB			30 AG 120 TC/TS 150 C4	30 AG 155 TC 150 C4 30 AG 110 TC 125 C3 125 C4	
UC			S2 / S4 110 TC/TS 150 C4		

FOR SELECTED LAYERS REFER TO PARAGRAPH 7.5 ; FOR FUTURE MAINTENANCE TO PARAGRAPHS 7.2, 7.3 AND 9.5
AO PERMITTED AS A SURFACING MEASURE FOR SKID RESISTANCE OR REDUCTION OF WATER SPRAYING.

WATERBOUND MACADAM BASES

DATE 1984

ROAD CATEGORY	DESIGN TRAFFIC CLASS E80s / LANE OVER STRUCTURAL DESIGN PERIOD				
	E0 <0,2 X 10^6	E1 0,2–0,8 X 10^6	E2 0,8-3 X 10^6	E3 3-12 X 10^6	E4 12-50 X 10^6
UA			30 – 40 A† 125 WM1 150 C3	40 A† 150 WM1 125 C3 125 C4	50 A† 150 WM1 150 C3 150 C3
UB		S† 100 WM2 150 C4 S† 125 WM2 150 G5	S OR 30 A† 125 WM2 150 C4	40 A† 125 WM1 125 C4 125 C4	
UC	S‡ 100 WM2 100 C4 S‡ 100 WM2 125 G5	S‡ 100 WM 2 125 C4 S† 100 WM2 150 G5	S† 100 WM2 100 C4 S‡ 125 WM2 150 G5		

†SYMBOL A DENOTES AG, AC OR AS. SYMBOL S DENOTES S2 OR S4
FOR SELECTED LAYERS REFER TO PARAGRAPH 7.5; FOR FUTURE MAINTENANCE TO PARAGRAPHS 7.2, 7.3 AND 9.5
AO PERMITTED AS A SURFACING MEASURE FOR SKID RESISTANCE OR REDUCTION OF WATER SPRAYING.

CONCRETE PAVEMENTS

DATE
1984

ROAD CATEGORY	DESIGN TRAFFIC CLASS E80s / LANE OVER STRUCTURAL DESIGN PERIOD				
	E0 $< 0,2 \times 10^6$	E1 $0,2-0,8 \times 10^6$	E2 $0,8-3 \times 10^6$	E3 $3-12 \times 10^6$	E4 $12-50 \times 10^6$
UA				190 PCC 100 C2	225 PCC 100 C2
UB			170 PCC 100 C3	180 PCC 100 C2	
UC					

FOR SELECTED LAYERS REFER TO PARAGRAPH 7.5; FOR FUTURE MAINTENANCE TO PARAGRAPHS 7.2, 7.3 AND 9.5.
AO PERMITTED AS A SURFACING MEASURE FOR SKID RESISTANCE OR REDUCTION OF WATER SPRAYING.