TECHNICAL RECOMMENDATIONS
FOR HIGHWAYS

DRAFT TRH4 : 1996

STRUCTURAL DESIGN OF FLEXIBLE PAVEMENTS FOR INTERURBAN AND RURAL ROADS

1996

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PREFACE

TECHNICAL RECOMMENDATIONS FOR HIGHWAYS (TRH) have traditionally been aimed at informing the practising engineer about current, recommended practice in selected aspects of highway engineering, based on proven South African experience.

It is suggested that reference also be made to the **UTG3** and **UTG7** document series when in doubt as to whether the road in question will operate under rural or urban conditions.

Companion TRH documents to **TRH4** are given on the next page.

This document was produced by a subcommittee of the Road Materials Committee and, to confirm its validity in practice, is circulated in draft form for a trial period.

Any comments on this document can be addressed to the Director General: Transport, Chief Director: Roads, P O Box 415, Pretoria, 0001, Republic of South Africa.

TRH 1 Prime coat and bituminous curing membranes	**TRH 12** Bituminous pavement rehabilitation design
TRH 3 Surfacing seals for rural and urban roads	**TRH 13** Cementitious materials in road construction
TRH 4 **Structural design of flexible pavements for interurban and rural roads**	**TRH 14** Guidelines for road construction materials
TRH 5 Statistical concepts of quality control and their application in road construction	**TRH 15** Subsurface drainage for roads
TRH 6 Nomenclature and methods for describing the condition of asphalt pavements	**TRH 16** Traffic loading for pavement and rehabilitation design
TRH 7 Use of bitumen emulsions in the construction and maintenance of roads	**TRH 17** Geometric design of rural roads
TRH 8 Design and use of hot-mix asphalt in pavements	**TRH 18** The investigation, design, construction and maintenance of road cuttings
TRH 9 Construction of road embankments	**TRH 19** Standard nomenclature and methods for describing the condition of jointed concrete pavements
TRH 10 Design of road embankments	**TRH 20** The structural design, construction and maintenance of unpaved roads
TRH 11 Guidelines for the conveyance of abnormal loads	**TRH 21** Hot-mix recycling
	TRH 22 Pavement Management Systems

Note: TRH 2 not available anymore

LIST OF TRH DOCUMENTS USEFUL FOR PAVEMENT DESIGN

SYNOPSIS

Procedures for the selection and structural design of flexible and semi-flexible (or semi-rigid) pavements for interurban and rural roads are presented. A choice of four different road categories, differentiated on the basis of the service objective for the road taking into consideration traffic type, riding quality and safety standards, is provided.

The procedures cover a range of proven or best practices for materials and pavement types currently found in local practice. Factors such as service objective, road category, life cycle strategy, traffic spectrum, pavement materials, pavement behaviour and performance, environmental conditions and construction recommendations are considered. The final selection of pavement design is based on the behaviour of the pavement type and the postulated life cycle strategy, taking into account the present worth of initial and future costs as well as the results of the Heavy Vehicle Simulator (HVS) tests over the past 15 years. Possible pavement types and pavement structures are presented in a Pavement Design Catalogue.

SINOPSIS

Prosedures vir die strukturele ontwerp van buigbare en semi-buigbare (of semi-star) plaveisels vir tussen-stedelike en plattelandse paaie, word aangebied. 'n Keuse van vier verskillende padkategorieë, waar onderskeid gemaak word op die basis van die diensdoelwit vir die plaveisel word gegee. Verder word tipe verkeer, rygehalte en veiligheidsstandaarde aangespreek.

Die prosedure omvat 'n reeks van beproefde of beste praktyk vir materiale en plaveiseltipes wat tans in die plaaslike mark gebruik word. Faktore soos diensdoelwit, padkategorie, lewensiklus strategie, verkeerspektrum, plaveiselmateriale, plaveiselgedrag en prestasie, omgewingstoestande en aspekte van konstruksie word in ag geneem. Die finale keuse van plaveiselontwerp is gebaseer op die gedrag van die plaveiseltipe asook die gepostuleerde Lewensiklus Strategie, en die ontleding van huidige waarde en aanvanklike en toekomstige koste, asook die resultate van die Swaarvoertuignabootser (SVN) oor die laaste 15 jaar. Moontlike plaveiseltipes en plaveiselstrukture word in die Katalogus van plaveiselontwerpe gegee.

KEYWORDS

Structural design, road pavements, service objective, road category, design traffic, pavement materials, cost analysis, design catalogue.

CONTENTS PAGE

LIST OF FIGURES

LIST OF TABLES

1. DESIGN PHILOSOPHY AND PROCESS

SCOPE

The procedures for the structural design of road pavements presented in this document are applicable to heavily trafficked interurban roads and freeways as well as surfaced, lightly trafficked rural and rural access roads in southern Africa. They are based on a combination of existing methods, experience and fundamental theory of structural and material behaviour developed since the 1970s. The proposed procedures do not necessarily exclude other design methods.

The structural design of pavements aims to protect the subgrade from traffic loads by providing pavement layers which will achieve a chosen level of service, with maintenance and rehabilitation during the analysis period, as cost effectively as possible. It encompasses factors of time, traffic, pavement materials, subgrade soils, environmental conditions, construction details and economics. The procedures cover a range of flexible and semi-flexible (or semi-rigid) pavement types, environmental conditions and materials currently used in local practice.

The basic process of road pavement design is illustrated in Figure 1. The information and process given in the figure are by no means an exhaustive list of functions and responsibilities, but rather represent a basic approach to the holistic process of pavement design. The figure indicates five functions: ***Management, Planning, Design, Construction*** and ***Maintenance and Rehabilitation.*** For each function the main responsibilities areas are given and all form part of the holistic and integrated pavement design process. The figure also shows the appropriate section(s) relevant to this document. Many other sub-systems (and usually a data management system too) are also involved as well as the various organisations' different operational details, which are not shown here.

Therefore, the holistic approach of pavement design calls for integrated liaison and input between the personnel charged with the various functions in an organisation so that they may perform their individual tasks successfully.

FUNCTION	RESPONSIBILITIES
MANAGEMENT SECTION 1 (See Figure 2)	• Needs Identification/Vision/Mission • Philosophy/Policy • Setting of Service Objective (SO) • Functional Service Level (FSL) • Managerial Directive/Perspective
PLANNING SECTION 1 and Inputs from: Government and Non-Govermental Organisations; Communities, Road Network Plan and PMS,* etc.	• Co-ordinate actions • Assess needs • Prioritise needs • Incorporate Service Objective • Determine Road Category
DESIGN SECTIONS 3 to 9	• Final Pavement Type Selection (See Figure 2)
CONSTRUCTION SECTIONS 6 and 7	• Construction Programme • Construction Cost • Construction Methods
MAINTENANCE AND REHABILITATION SECTIONS 1 and 9, and TRH12	• Prioritise existing road maintenance/rehabilitation needs based on Life Cycle Strategy (LCS) of Selected Pavement Design and later, PMS information

* PMS: Pavement Management System (TRH22, 1994)

FIGURE 1: PROCESS OF BASIC ROAD PAVEMENT PROVISION AS DISCUSSED IN TRH4

1.1 SERVICE OBJECTIVE (SO)

Initially, when the need for accessibility or traffic capacity improvement in a certain area has been identified by the responsible authority, two basic decisions need to be taken by management in order to provide the necessary directive and inputs for the design process, namely:

- the functional service level of the road or facility improvement; and
- the analysis period over which the service is anticipated.

These inputs and directives are called the Service Objective (SO) of the project and are postulated against the background of the aim and condition of the existing infrastructure network, taking into account such aspects as the importance of the road link, riding quality, safety, traffic capacity, funding, etc. The SO largely determines the standard of the geometrical and structural designs for that particular road link. Note that the SO may change during the life of the road to accommodate a change in need. This, in turn, will influence the future maintenance considerations and priorities of the road. Therefore, a holistic view has to be maintained during all phases of pavement design.

1.2 FUNCTIONAL SERVICE LEVEL (FSL)

A distinction is made between the *functional requirements* and the *structural requirements* of a road link. The functional requirements relate to the *functional service* which the road has to deliver in order to fulfill the need as defined by the Service Objective (SO). The structural requirements, however, relate to the *structural support* (i.e. bearing capacity, see Section 3.1 later) necessary to guarantee the functional service at a given design reliability.

The functional service level (FSL) is the qualitative measure for operating conditions on a given portion of a road and is related to the perceptions of motorists of those conditions. It is basically determined by factors such as speed, travelling time, delays, freedom to change position in the traffic stream, safety and driving comfort. No specific guidelines for service level are given here, since this should be defined by the responsible road authority. However, the description of road category as given in Section 2 relates strongly to the service objective.

1.3 ANALYSIS PERIOD (AP)

The Analysis Period (AP) is usually equal to the functional period for which the road will have to deliver its functional service as determined by management through the service objective. This is sometimes put forward as a general policy to further qualify the various road categories (based on the service objective) as discussed in Section 2. The analysis period may be made up of one or more **Structural Design Period (SDP)**, each with its own **Life Cycle Strategy (LCS)**.

1.4 STRUCTURAL DESIGN PERIOD (SDP)

The Structural Design Period (SDP) is defined as the period during which it is predicted that no structural improvements will be required, linked to a specified design reliability. These periods may well be dictated by performance data from pavement management systems, such as the rate and quality of maintenance, rehabilitation and reconstruction. To select the "optimum pavement" in terms of present worth of cost, it is necessary to evaluate the life cycle strategy of the different pavement structures, as mentioned above.

1.5 LIFE CYCLE STRATEGY (LCS)

The Life Cycle Strategy (LCS) for any pavement design incorporates the predicted maintenance and rehabilitation programme for that pavement based on its *anticipated* behaviour under the prevailing conditions. It normally includes the *funding needs* programme for the specific road.

It is considered that the design process *is the first step* in the life cycle of a pavement. Not only will the final pavement design have an influence on the behaviour of the pavement but it also lays the foundation for the maintainability (i.e. frequency and type) and salvage value of the pavement when it has to be rehabilitated or reconstructed. Thus the LCS considers the overall performance of the pavement both structurally and economically over its structural design period and analysis period.

It is considered imperative that all the road pavements within the pavement management system should be managed according to a life cycle strategy, *initially drawn up during the design phase*. This strategy should then be revised as indicated by regular pavement monitoring and maintenance to ensure its cost effective performance. Such procedure will also ensure that positive proof of pavement behaviour and performance is available to assist with future design strategies and research work. See also Section 3.3 later.

1.6 STRUCTURAL OBJECTIVE (STO)

When a road pavement is designed initially, or re-designed at a later stage, the design should agree with the current service objective and with due regard for the life cycle strategy for that section of road.

Furthermore, to enable the designer to make a fair comparison between alternative new designs and appropriate rehabilitation measures, a common basis, usually cost, is required. The process takes into account both the structural capacity of the in situ material (subgrade) or existing pavement to be rehabilitated. Estimates of the appropriate rehabilitation measures that will be necessary to maintain the original pavement in a serviceable condition over the (new) analysis period (as per life cycle strategy) will also be needed.

The aim of the basic structural objective (STO) may be summarised as follows:

To produce a structurally balanced pavement structure of sufficient bearing capacity under the prevailing environmental conditions in order to fulfill the functional need as defined by the functional service level (FSL). This includes the design and maintenance predicted in the Life Cycle Strategy (LCS) that it will be able to carry the traffic cost effectively over the Structural Design Period (SDP) in accordance with the Service Objective (SO).

While the pavement may be maintained or upgraded in accordance with the life cycle strategy in order to uphold the functional serviceability commensurate with the service objective over the analysis period, it should not exhibit signs of major distress requiring structural rehabilitation. This means that the present worth of cost (PWOC) of alternative designs should be calculated during the life cycle strategy analysis in order to determine the most economical pavement structure integrated with the in situ conditions.

1.7 THE PAVEMENT DESIGN PROCESS

The detailed design process used in this document is illustrated by the flow diagram in Figure 2. There are nine sections indicated in this figure. Each section is treated separately, but all sections have to be considered before a design can be selected from the Catalogue[1]. The first six sections represent the inputs to pavement design, i.e. the service objective, road category, design strategy, design traffic, materials available and environment. With these inputs the designer will use the Catalogue for various pavement types to select candidate pavement structures. This process includes estimation of future maintenance measures and some construction considerations. Eventually the different pavement structures should be compared on the basis of the present worth of the life cycle cost.

2. ROAD CATEGORY

2.1 DEFINITION OF ROAD CATEGORIES

Generally, a road authority may have a number of road categories to suit the different levels of service the system has to deliver based on the associated service objectives. Each of these road categories will necessitate certain geometrical and structural standards to ensure that the service objective(s) of the road can be met and maintained throughout its analysis period. The more important a road, the higher its level of service and thus its physical properties and standards, hence these roads have a reduced risk of failure (i.e. higher design reliability) over the structural design period. In this document four typical road categories, are considered, as defined in Table 1.

2.2 CONSIDERATIONS REGARDING THE ROAD CATEGORIES

Having decided on the service objective and functional service level of a particular road, the road authority has largely pre-determined the road category and supporting functional and structural qualities (as mentioned in Section 1.1), and may indeed wish to have a standard policy document in this regard as a guide to personnel. Table 1 may serve such a purpose and show conditions and characteristics which may typically be expected or actually exist, but is not considered as *determinants* of the road category.

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1 Pavement Design Catalogue referred to in this document as the Catalogue

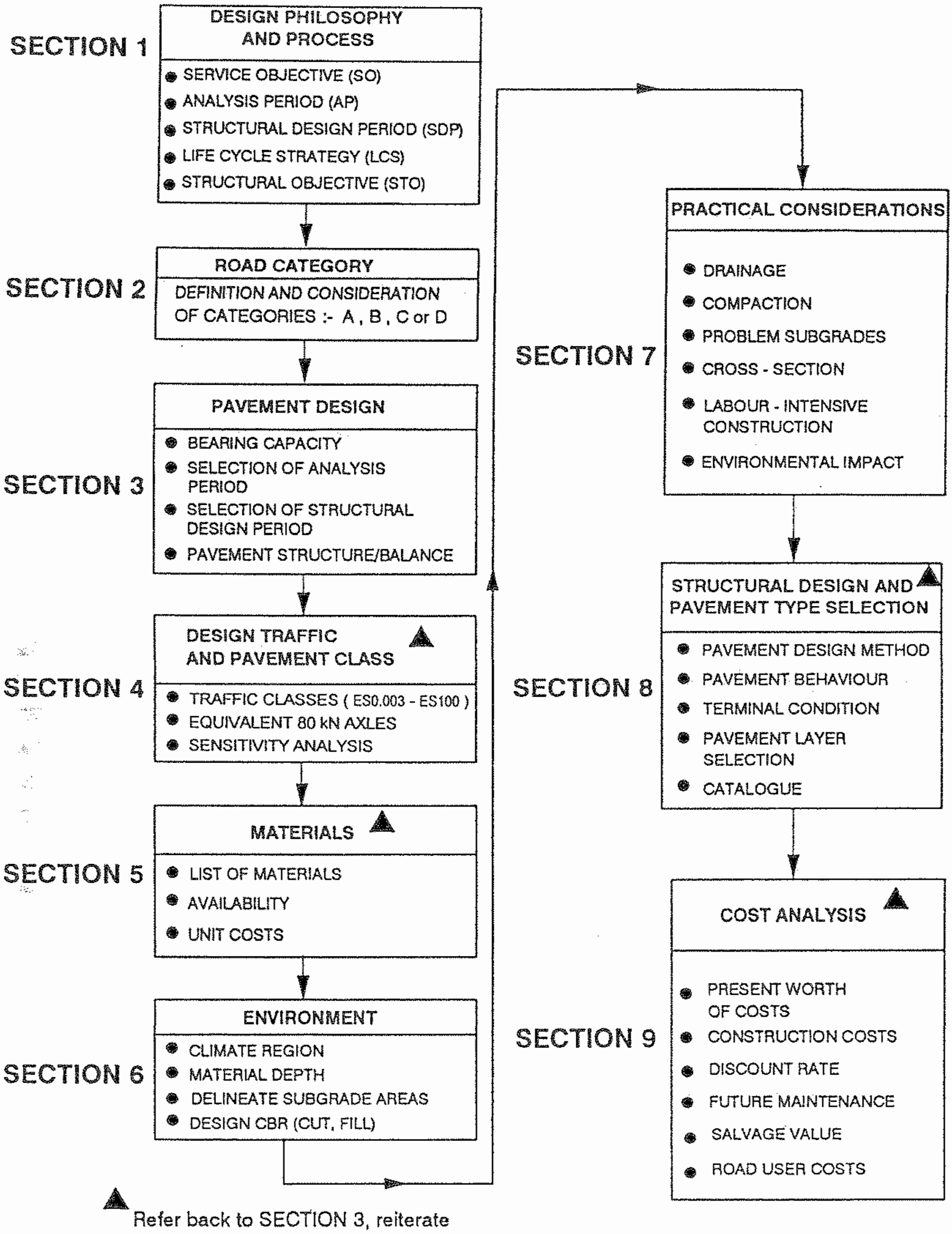

FIGURE 2: DETAILED ROAD PAVEMENT DESIGN PROCESS

TABLE 1

Definition of the road categories

ROAD CATEGORY				
	A	B	C	D
Description	Major interurban freeways and major rural roads	Interurban collectors and rural roads	Lightly trafficked rural roads, strategic roads	Rural access roads
Importance Service level	Very important Very high level of service	Important High level of service	Less important Moderate level of service	Less important Moderate to low level of service
TYPICAL PAVEMENT CHARACTERISTICS				
RISK	**Very low**	**Low**	**Medium**	**High**
Approximate Design Reliability (%) *	95	90	80	50
Total Equivalent Traffic Loading (E80/lane) **	3 - 100 x 10^6 over 20 years	0,3 - 10 x 10^6 Depending on design strategy	< 3 x 10^6 Depending on design strategy	< 1 x 10^6 Depending on design strategy
Typical Pavement Class ***	ES10 - ES100	ES1 - ES10	ES0.003 - ES3	ES0.003 - ES1
Daily Traffic: (e.v.u) ****	> 4000	600 - 10 000	< 600	< 500
Constructed Riding Quality: PSI ***** HRI (mm/m or m/km) ***Terminal Riding Quality:*** PSI HRI (mm/m or m/km)	 3,5 - 4,5 1,5 - 1,0 2,5 2,7	 3,0 - 4,5 2,0 - 1,0 2,0 3,5	 2,5 - 3,5 2,7 - 1,5 1,8 3,9	 2,0 - 3,5 3,5 - 1,5 1,5 4,5
Warning Rut Level (mm) Terminal Rut Level (mm)	10 20	10 20	10 20	10 20
Area / length of road exceeding terminal conditions (%)	5	10	20	50

* Reliability, based on percentile levels originally defined in TRH12 (1983) for A, B and C roads.

** See Section 4.

*** ES: Equivalent Standard Axle (80 kN) Class. See Table 4.

**** Approximate daily traffic in e.v.u : Equivalent vehicle unit (1,25 vehicle = 1 e.v.u) (DOT, RR 92/466/2, 1993).

***** PSI = Present Serviceability Index, scale 0 to 5 (TRH6, 1985). HRI = Half - car Roughness Index of a single averaged longitudinal profile (left & right wheel track) in mm/m or m/km. HRI = 8,470 - 3,112(PSI) + 0,324$(PSI)^2$ (See Kannemeyer, 1997).

The pavement for a Category A road will normally be constructed and maintained to higher functional standards (safety, riding quality, comfort, etc.) than pavements for Categories B, C and D roads. Table 1 shows the usual range of expected constructed riding qualities. For example, the user will expect a better riding quality and higher safety standards on a dual-carriageway freeway than on a minor rural road. This expectation is eventually also reflected in the structural design reliability and strategy. However, management may decide that the structural design reliability should be higher for a specific road than is normally associated with the functional standard for such a service objective, or vice versa.

The terminal riding qualities for the various road categories, which are also listed in Table 1, are also intended only as a guide, since in certain cases there may be valid reasons for deviating from the classification given in Table 1.

Note that the *structural design reliability* given here is related to the *length of road that will not exceed the specified terminal functional performance criteria at the end of the design period.*

The structural performance criteria used to develop the different catalogue designs given in this document are based on the South African Mechanistic Design Method (SAMDM) (Theyse et al., 1995), and were subsequently calibrated by practical considerations (i.e. theoretical layer thicknesses obtained from the SAMDM were *adapted* to conform with practical layer thicknesses).

2.3 INNOVATIVE DESIGNS

The TRH4 Catalogue designs do not preclude the designer from proposing innovative or alternative pavement designs which may be appropriate because of special circumstances in a region. Special designs may be used, for example, to accommodate cost saving or labour intensive methods (see Section 7.6), or to make use of waste materials locally available. If the client agrees, the designer can produce innovative pavements which could fall within any of the available A, B, C or D road categories. Pavements with innovative designs should be monitored over their lifetime so that, once proven in practice, these designs may be included as part of this catalogue.

3. PAVEMENT DESIGN

3.1 THE BEARING CAPACITY OF A PAVEMENT

A pavement, like any other engineering structure, is designed to withstand certain loads. In this case the primary load that needs to be considered in the design is the *traffic spectrum* that will be carried by the road. In South Africa, as in many other countries, the standard axle load is 80 kN. However, the legally permissible axle load[2] is 88 kN (South Africa, 1996).

In this document a pavement is designed to have a specific bearing capacity which is expressed in terms of the number of Standard (80 kN) Axle (SA) load repetitions that will result in a certain condition of deterioration. This condition is normally considered to be the *terminal condition*, indicating that the pavement has structurally "failed", and can no longer support the *functional service* set by the service objective.

A pavement could have a *bearing capacity* of 1 million Standard Axle repetitions (1×10^6 SAs), indicating that the pavement will be able to carry a *traffic spectrum* to the *Equivalent of 1 million Standard Axle loads* (1×10^6 ESA). The *traffic spectrum* therefore has to be converted to *Equivalent Standard Axle loads* (ESA) for structural pavement design purposes. The ESA is then used to calculate how long a certain pavement will be able to support a specific *traffic spectrum* before its predefined terminal functional service level is reached. However, the *same traffic spectrum* may be converted to *different* equivalent standard axle load scenarios for design purpose (i.e. different design ESAs), depending on the type and specific composition of the pavement structure.

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2 The legally permissible axle loads are given below (South Africa, 1996) :
(See also Glossary of terms)

Type of axle	No of tyres per axle	Mass (kg)	Load per Axle (kN)*
Single axle (steering)	2 or 3	7 700 (7 700)	76
Single axle (non-steering)	2 or 3	8 000 (7 700)	78
Single axle	4 or more	9 000 (8 200)	88
Tandem axle	4 or more	18 000 (16 400)	88
Tridem axle	4 or more	24 000 (21 000)	78,3

( ) Previous legal load limits

* $g = 9{,}8 \text{ m/s}^2$

Thus:

- **Pavement Bearing Capacity** is expressed in: **Standard (80 kN) Axle repetitions (SAs or 80s)**, and

- **Traffic Load Spectrum** (i.e. traffic demand) is expressed in: **Equivalent Standard Axle repetitions (ESAs or E80s)**, through the use of the concept of *relative damage*, discussed later in Section 4.4.

3.2 SELECTION OF THE ANALYSIS PERIOD

As was discussed in Section 1.2, from the designer may be required to suggest an appropriate Analysis Period (AP). There may be a difference between the *analysis period* and the *total period* over which a facility may be used. The analysis period is often related to the geometric life of the road. If the road alignment is fixed, a period of 30 years should be used. In the case of an uncertain or shorter geometric life, in a changing traffic situation, a shorter analysis period may be used. A shorter analysis period will also be used when the proposed pavement has a limited life (e.g. a road to a mine). Table 2 shows the possible ranges and recommended analysis periods for the various road categories. These values should be used for the economic analysis, unless more detailed information is available.

TABLE 2

Typical analysis periods for various road categories

Road category	Analysis period (years)		
	Range	Recommended period	
		High certainty*	Low certainty
A	20 - 40	30	—
B	15 - 30	30	25
C	10 - 30	30	20
D	10 - 20	20	10 - 15

* Fixed road geometric alignment.

3.3 SELECTION OF THE STRUCTURAL DESIGN PERIOD (SDP)

With reference to the design philosophy and process discussed in Section 1, selection of the appropriate Structural Design Period (SDP) is a matter of balance between the practical feasibility of the Life Cycle Strategy (LCS) and the resultant life cycle funding requirements for a particular pavement design. This, of course, depends heavily on the quality, quantity and availability of resources as well as political implications (Kleyn et al., 1986). For example, one could decide on a life cycle strategy comprising a relatively low standard (inexpensive) initial construction phase, followed by an intensive and costly longer term maintenance programme. The other extreme is a life cycle strategy comprising high initial standards followed by very low (or unsophisticated) maintenance needs. Here it is also important to differentiate between *preventative* vs *reactive* maintenance, as is discussed later in Section 8.2.1.

The manner in which the Life Cycle Strategy (LCS) may be presented is demonstrated schematically in Figure 3, which shows the generalised trends of riding quality decreasing with time and cumulative traffic loading for two different pavement structures, i.e.:

(a) Design 1, which requires resurfacing to maintain the surface in good condition, and later some structural rehabilitation such as an overlay (Figure 3(a)); and

(b) Design 2, which is structurally adequate for the whole of the analysis period and requires only three resurfacings (Figure 3(b)).

These figures are for illustrative purposes only and should not be taken as representative of the performance curves for typical South African pavements. Normally, well designed and timely maintained pavements exhibit much "flatter" performance curves (i.e. a relatively low rate of deterioration). Note the difference between a standard "appropriate" design and a "low cost" design: a standard appropriate design will ensure optimal cost-effective performance in accordance with the service objective, whereas a "low cost" design deliberately "lowers" standards and quality resulting in impaired reliability, as was indicated previously in Table 1.

3.3.1 Category A (Design Reliability[3] = 95 %)

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

(a) these are normally the heaviest trafficked roads in the country;

(b) road user cost are high and the cost of disrupting the traffic will probably cancel any pavement cost savings resulting from the choice of relatively short structural design periods;

(c) the road alignment is normally fixed with a high certainty of not changing; and

(d) it is usually not acceptable to the public for road authorities to carry out heavy rehabilitation on recently constructed pavements.

The structural design period adopted in this document is 25 years for Category A pavements, as shown in Table 3.

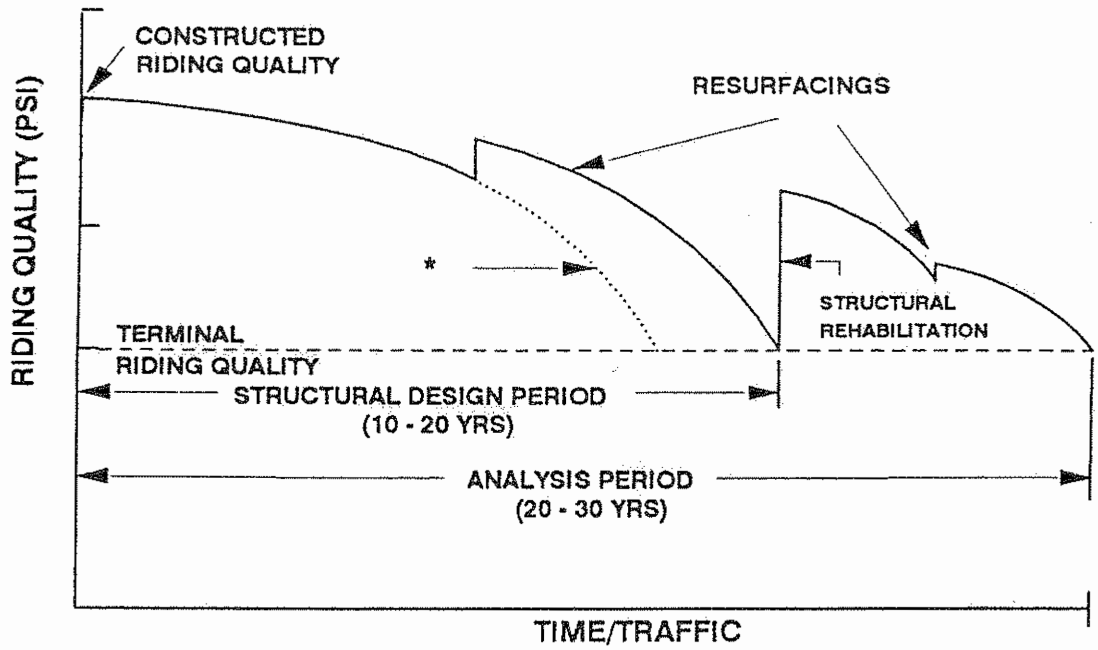
TABLE 3

Typical structural design periods for various road categories

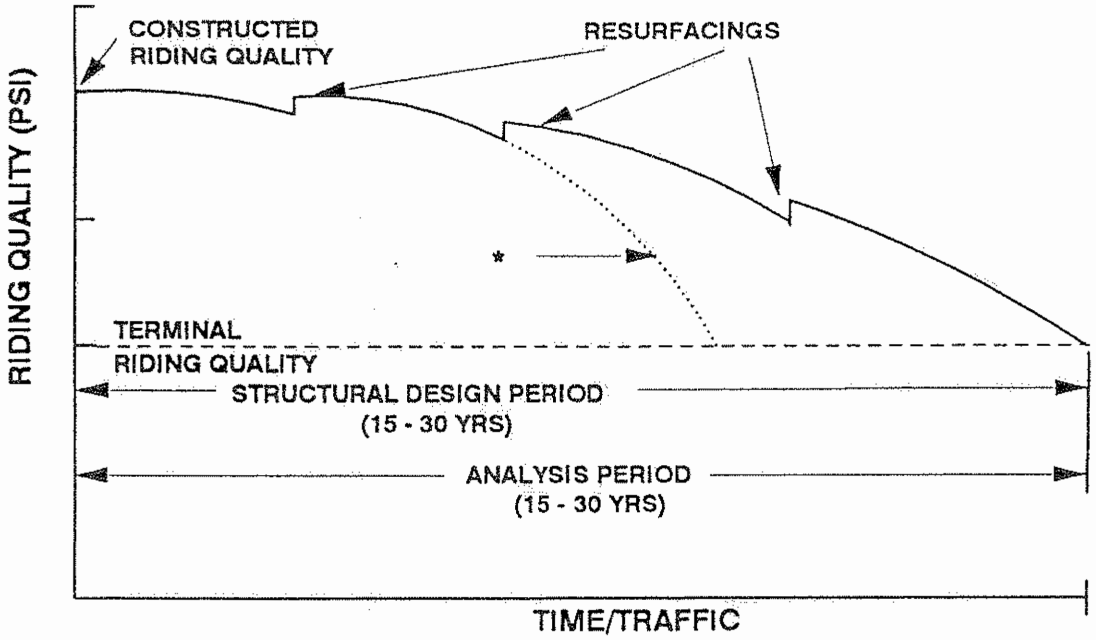
Road category	Structural design period (years)*	
	Range	Recommended
A	15 - 30	25
B	15 - 25	20
C	10 - 20	15
D	7 - 15	10

* The actual structural design period should be obtained from the relevant road authority.

3 The ***design reliability*** defined here relates to the ***"length of road that will not exceed the specified terminal functional performance criteria at the end of the Structural Design Period (SDP)"***, as was given in Table 1.



(a) DESIGN 1 REQUIRES TWO RESURFACINGS AND ONE STRUCTURAL REHABILITATION DURING THE ANALYSIS PERIOD



(b) DESIGN 2 REQUIRES THREE RESURFACINGS AND NO STRUCTURAL REHABILITATION DURING THE ANALYSIS PERIOD

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

FIGURE 3: ILLUSTRATION OF CONCEPTUAL DESIGN PERIODS AND ALTERNATIVE DESIGN STRATEGIES

3.3.2 Category B (Design Reliability = 90 %)

For Category B roads, the structural design period may vary depending on the circumstances. Long structural design periods (25 years) will be selected when circumstances are the same as for Category A roads. Factors which encourage the selection of shorter structural design periods include

(a) a short geometric life for a facility in a changing traffic situation;

(b) a lack of funds; and

(c) a lack of confidence in design assumptions, especially the design traffic.

Structural design periods may range from 15 to 25 years. When shorter structural design periods are selected, the design should be capable of accommodating pragmatic stage construction. Normally a structural design period of 20 years will be used, as shown in Table 3.

3.3.3 Category C (Design Reliability = 80 %)

A relatively short structural design period of 10 years is often selected for Category C roads, because of financial constraints. If it is difficult or impractical to carry out structural rehabilitation, a longer period of 20 years may be selected, as shown in Table 3.

3.3.4 Category D (Design Reliability = 50 %)

Category D roads are roads where expected traffic growth will be rapid or unpredictable, i.e. typical of rural access roads. A relatively short structural period (7 years) will enable maintenance and/or upgrading strategies to be adapted to circumstances without incurring a high initial capital outlay. The designs are initially more economical but carry a relatively high risk of failure.

A typical structural design period for these roads is normally 10 years, as shown in Table 3.

3.4 PAVEMENT STRUCTURAL BALANCE

As mentioned under Section 1.5 (Structural Objective), the aim is to produce a *"...structurally balanced pavement structure....".* Since a pavement is a composite system, it should be designed so that the various pavement layers react in unison. Optimally, each layer is to be stressed to the same level of its maximum bearing capacity. (i.e. without overstressing). The bearing capacity of a structurally well balanced pavement structure increases *evenly* with increasing depth (cover) over the subgrade material. This "cover" is to be increased until the target bearing capacity of the pavement is achieved.

The build-up of the bearing capacity of a pavement may occur at varying stages or levels. The more the final bearing capacity is derived from the upper pavement layers (base and subbase) *relative* to the lower layers, the "shallower" the pavement structure. The more the deeper layers contribute to the final bearing capacity *relative* to the upper layers, the "deeper" the pavement structure. Therefore, the "shallowness" or "deepness" of a pavement is a relative concept (Kleyn, 1982, De Beer et al., 1989, De Beer, 1990, 1991). This is also the basic reason for the inherent load sensitivity of a pavement as discussed later in Section 4.4.

The relative damage exponent "n" normally increases with an increase in "shallowness" of the pavement. The pavement structures suggested in this document are based on proven or best practice in South Africa which relates to pavements which tend to be mainly relatively deep, resulting in "n" being around "4" for most of its structural design life. See also Section 4.4 later.

4. DESIGN TRAFFIC

4.1 INTRODUCTION

The cumulative damaging effect of all individual axle loads (i.e. traffic spectrum) is expressed as (or converted to) the number of equivalent 80 kN single-axle loads (ESAs or E80s). It is therefore assumed that this is the number of equivalent 80 kN single-axle loads that would cause the *same* damage to the pavement as the actual traffic spectrum of all the axle loads. For structural design, an estimate of the *cumulative equivalent traffic* over the structural design and analysis period is required to produce practical pavement designs compatible with the long-term strategy. A detailed explanation of traffic estimation is given in TRH16 (1991).

The cumulative E80 is calculated by *converting* the expected traffic spectrum of all axle loads to equivalent standard axle loads per lane per day (i.e. E80/lane/day), and then summing these E80s over the design period. The sum represents the design ESAs or E80s.

4.2 CLASSIFICATION OF PAVEMENTS AND TRAFFIC FOR STRUCTURAL DESIGN PURPOSES

For the purposes of the Pavement Design Catalogue, the pavements are divided into ten different classes, namely ES0.003 to ES100, covering *extremely light traffic* to *extremely heavy traffic*. The classification is summarised in Table 4. For each of the ten pavement classes given, the design bearing capacity in terms of million standard 80 kN axles/lane (million SAs/lane) is also given[4]. The volume of traffic for each pavement class is given separately. For pavement classes ES0.003 to ES3, the volume of traffic is based on *vehicles per day per lane (v.p.d./lane)*, and for classes ES10 to ES100 the volume of traffic is based on *vehicles per day per direction (v.p.d./direction)*. An example of the conversion of the v.p.d. to equivalent standard axles for classification purposes is given at the bottom of Table 4. In addition, a description of the type of traffic for the various classes is also given in Table 4.

Pavement classes ES0.003 to ES0.3 usually provide for very light to extremely light traffic, and may include pavements in the "transition" from gravel to paved roads. This relatively finely divided group of pavements may incorporate semi-permanent and/or all weather surfacings, like gravel bonding agents, and is usually more weather sensitive than the group ES1 to ES100. The boundaries between these pavement classes (ES0.003 to ES0.3) should not be considered too strictly and good engineering judgement is needed to select the appropriate pavement class in this group.

Pavement classes ES1 to ES100 provide for the lightly trafficked roads to very high volume and/or high proportion of fully laden heavy vehicles. These roads always incorporate an all weather good quality surfacing.

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4 The pavement classification given here is based on an approximate 300 % increase in bearing capacity, starting at 3000 SAs up to 100 million SAs.

4.3 DETERMINING AVERAGE E80/LANE/DAY FROM DIFFERENT SOURCES OF TRAFFIC LOADING INFORMATION

The correct estimation of design traffic is crucial to the selection of an appropriate pavement from the Catalogue. The design traffic loading is the number of E80s on the *most heavily trafficked lane* during the structural design period.

To calculate the total design traffic per lane that a pavement will carry over its structural design life, an estimation of the present traffic loading, converted to Average Daily E80 (ADE), is necessary. The ADE is the E80/lane/day averaged over the survey period during which the axle load survey was conducted.

ADE can be calculated from the following sources of traffic loading information:

(a) Published results of surveys

Extrapolation of published ADE results from roads in the vicinity or with similar loading can be used to estimate ADE. Cognisance should be taken of potential errors which may occur when using extrapolation. Published results include documents produced by the Division of Roads and Transport Technology of the CSIR, the system of Comprehensive Traffic Observations (CTO) carried out by Department of Transport (DOT) (see also DOT, CTO Yearbook, 1995) and data collected by the road authorities.

For all data sources used, care should be taken with the vehicle classification system used, as well as the method by which the information was generated. Some E80 figures are calculated indirectly by applying factors/vehicle while others are based on actual weighed measurements.

(b) Transportation planning models

Transportation planning models applicable to South Africa are becoming accessible to planners and designers (Jordaan et al., 1989; Van Zyl, 1986). These packages are based on the standard urban modelling procedure which divides the country into a number of zones. The total number of trips generated and attracted to each zone is determined using the gross geographical product of the zones. These trips are divided between each origin and destination zone and between the different transport modes.

TABLE 4

Classification of pavements and traffic for structural design purposes

Pavement class*	Pavement design bearing capacity (million 80 kN axles/lane)	Volume and type of traffic**	
		Approximate v.p.d. per lane***	Description
ES0.003	**< 0,003**	**< 3**	Very lightly trafficked roads; very few heavy vehicles. These roads could include the transition from gravel to paved roads and may incorporate semi-permanent and / or all weather surfacings.
ES0.01	**0,003 - 0,01**	**3 - 10**	
ES0.03	**0,01 - 0,03**	**10 - 20**	
ES0.1	**0,03 - 0,10**	**20 - 75**	
ES0.3	**0,10 - 0,30**	**75 - 220**	
ES1	**0,3 - 1**	**220 - 700**	Lightly trafficked roads, mainly cars, light delivery and agriculture vehicles; very few heavy vehicles.
ES3	**1 - 3**	**> 700**	Medium volume of traffic; few heavy vehicles.
ES10	**3 - 10**	**> 700****	High volume of traffic and / or many heavy vehicles.
ES30	**10 - 30**	**> 2200****	Very high volume of traffic and / or a high proportion of fully laden heavy vehicles.
ES100	**30 - 100**	**> 6500****	

* ES = Equivalent Standard Axle (80 kN) Class.

** Traffic demand in this document converted to *Equivalent* 80 kN axles (see Section 4.4).

*** v.p.d. = vehicles per day. The approximate v.p.d. per lane for **ES0.003 to ES3** equals to the design bearing capacity, and hence pavement class based on the following: 10 % heavy vehicles from the v.p.d. per lane count, 1,2 E80 per heavy vehicle (from Table 5), 4 % growth rate in E80s (from Table 10) over a design period of 20 years (from Table 3) and traffic growth factor (f_y) from Table 12.

**** For **ES10 to ES100** the v.p.d. is *total per direction* with 20 % heavy, at 2 E80s per heavy vehicle.

EXAMPLE:
Number of vehicles per day per lane (v.p.d./lane) = 200.
Equivalent design traffic = 200 x 0,10 x 1,2 x 11 303 = 271 272 E80s, therefore the needed ***bearing capacity of the pavement*** is between 0,1 and 0,3 million standard axles, SAs (80 kN).
Therefore the pavement class for the above traffic demand is **ES0.3.**

Finally the resulting trips are assigned to each link of the network to obtain the traffic flows. These packages serve as a powerful aid for judging the future traffic on existing links and should be used in conjunction with axle load surveys.

(c) Tabulated average E80 values

The simplest and least accurate way to determine ADE is to use tabulated E80 factors representing average conditions for a particular vehicle classification system.

The first set of tabulated values is based on heavy vehicle volumetric capacity or road category. A rough estimate of the type of heavy vehicle which uses different road categories is made and a factor from Table 5 is selected. The number of heavy vehicles per lane in each group is multiplied by the relevant factor and summed to obtain the ADE. This method is approximate and must be used with circumspection (TRH16, 1991).

TABLE 5

*Determination of E80s per heavy vehicle**

Loading of heavy vehicles	E80/heavy vehicle
Mostly unladen	0,6
50 % laden, 50 % unladen	1,2
> 70 % fully laden	2,0

* Axle load > 4 000 kg, gross vehicle mass > 7 000 kg, carrying capacity > 3 000 kg (TRH16, 1991).

NOTE: These values, however, were obtained before an increase in legal axle loading in 1996.

(d) Weighing methods

Two types of axle mass surveys are usually used in South Africa, namely static and dynamic weighing. Draft TMH3 (1988) contains a detailed methodology for conducting both these survey types.

A more reliable set of tabulated values is based on different heavy vehicle configurations as shown in Table 6. The number of vehicles in each category is multiplied by the average E80 and then summed to determine the ADE.

Static weighing requires vehicles to be stationary and is thus limited to a sample of vehicles travelling during the day. Care has to be taken to select a random sample of heavy vehicles and not only those that are laden. Since the intention is to obtain a realistic idea about the axle weights on the road, weighing done for law enforcement purposes may be biased and should not be used.

The information collected is usually presented as the number of axles in each of the predefined axle mass groups. This information is then used to calculate ADE as described in the next section.

Dynamic vehicle weighing *is used at sites such as multi-lane highways or* where the terrain and traffic flow does not allow for the static weighing of all vehicles or for a representative sample to be obtained. Typically, a seven-day survey is carried out at a particular site. The results give the number of axles in each of the defined axle mass categories. In the next section the conversion of this data to ADE will be discussed. With the ready availability of portable dynamic weighing equipment in South Africa, this type of measurement would normally be taken.

TABLE 6

Average E80s for different heavy vehicle configurations (TRH16, 1991)

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

* Based on n = 4,2 (see paragraph 4.4 later).

** 2,78 E80s per bus (for a fully legally loaded bus, n = 4,2).

NOTE: These values, however, were obtained before an increase in legal axle loading in 1996.

4.4 CONVERTING TRAFFIC LOADING INFORMATION TO DESIGN E80s

Information from static and dynamic surveys is recorded as the number of repetitions of a given axle load (P). The load equivalency factor (F) 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 and state, material types, definition of terminal conditions, failure modes and road rideability. Table 7 gives average equivalency factors for a range of relative damage exponents based on the load equivalency formulae:

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

where n = relative damage exponent
F = load equivalency factor
P = axle load, in kN

TABLE 7

*80 kN single-axle load equivalency factors**

Single-axle load, P (kN)	80 kN axle equivalency factor, F for different relative damage coefficients, n			
	n = 3	n = 4	n = 5	n = 6
Less than 15	0,00	0,00	0,00	0,00
15 - 24	0,02	0,00	0,00	0,00
25 - 34	0,05	0,02	0,02	0,00
35 - 44	0,13	0,06	0,03	0,02
45 - 54	0,24	0,15	0,10	0,06
55 - 64	0,42	0,32	0,24	0,18
65 - 74	0,66	0,58	0,52	0,46
75 - 84	0,99	1,00	1,00	1,01
85 - 94	1,41	1,59	1,80	2,04
95 - 104	1,94	2,42	3,04	3,82
105 - 114	2,58	3,55	4,89	6,74
115 - 124	3,35	5,02	7,54	11,35
125 - 134	4,26	6,92	11,25	18,32
135 - 144	5,32	9,30	16,29	28,55
145 - 154	6,54	12,26	23,00	43,17
155 - 164	7,94	15,88	31,75	63,56
165 - 174	9,53	20,24	43,00	94,42
175 - 184	11,32	25,44	57,23	128,80
185 - 194	13,31	31,59	75,00	178,10
195 - 204	15,53	38,79	96,93	242,30

* Average is based on $F = ((P_{lower\ limit}/80)^n + (P_{upper\ limit}/80)^n)/2$

Traditionally, the relative damage exponent was taken as 4, but pavements that are sensitive to overloading, such as "shallow" pavements with thin cemented bases, may have n-values exceeding 4, whereas less sensitive (i.e. "deep") pavements may have n-values of less than 4.

Note that the relative damage exponent (n) is dependent on both the *type of pavement*, its *failure mechanism* and its *state*. Table 8 indicates recommended n-values related to pavement type for the purpose of equivalent traffic estimation, based on research with the Heavy Vehicle Simulator (HVS) (Van Zyl et al., 1984).

The Average Daily E80 (ADE) 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), such as shown in Table 7.

By summation, the average daily E80 is :

$$ADE = \Sigma t_j \times F_j \qquad \text{. . . . . 4.2}$$

The designer should preferably do a sensitivity analysis on the spectrum of axle loads with n-values as indicated in Table 8, especially when dealing with abnormal load spectra.

TABLE 8

Recommended relative damage exponents (n) for the purpose of equivalent traffic estimation (modified from Van Zyl et al., 1984)

Pavement type *	Range **	Recommended
Granular / Granular	3 - 6	4
Granular / Cemented	2 - 4	3
Cemented / Granular : pre - cracked *** post - cracked	4 - 10 3 - 6	5
Cemented / Cemented : pre - cracked *** post - cracked	3 - 6 2 - 5	4,5
Hot-mix base / Cemented	2 - 5	4

* Pavement type defined here as "type of base over type of subbase" material.

** The higher values of the range usually refers to a fatigue failure mode in the upper regions of the pavement, while the lower values more towards a rutting failure mode.

*** Pre-cracked phase of a lightly cemented layer is normally less than 10 % of structural life (De Beer, 1990).

4.5 COMPUTING THE ANNUAL AVERAGE DAILY E80

4.5.1 Adjusting abnormal ADE figures

Ideally the data used to calculate ADE should be collected over long enough periods to ensure that extraneous influences are minimal. Unfortunately this is not always possible. The Annual Average Daily E80 (AADE) is used to indicate the average yearly traffic flow, taking corrections for variations in the Average Daily E80 into account. AADE is the total E80s in one traffic direction applied during one year divided by 365 days.

In converting information collected over a short period, whether by a new survey or previously collected information, to an Annual Average Daily E80, the following correction factors need to be taken into account:

- variations between weekdays and weekends;
- variations between exceptional circumstances (which may be holidays); and normal days; and
- variations between in-season and out-of-season periods.

Correction for these variations is standard procedure in the traffic counting process detailed in Draft TMH8 (1987). If it is suspected that the data being used requires correction, refer to TRH16 (1991) for the relevant formulae.

4.5.2 Estimating the lane distribution of traffic

Cumulative traffic is calculated from the lane carrying the highest E80s. For multi-lane carriageways, the traffic in one direction 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, climbing lanes or interchange ramps. Suggested design factors for total traffic (B) and equivalent traffic (B_e) are given in Table 9. As far as possible, these factors incorporate the change of lane distribution over the geometric life of a facility. The factors should be regarded as maxima. Decreases in these factors may be justified based on user experiences.

TABLE 9

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

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

* Lane 1 is the outer or slow lane.

** These factors may change owing to traffic on the shoulder.

*** e.v.u. = equivalent vehicle units; one commercial vehicle = 3 e.v.u.

The Annual Average Daily E80 (AADE) can be calculated by multiplying the equivalent traffic by a lane distribution factor (B_e) :

$$AADE = ADE \times B_e \qquad \qquad 4.3$$

where ADE = ADE after adjustment (see Section 4.5.1)
B_e = Lane distribution factor from Table 9

4.6 DETERMINATION OF FUTURE TRAFFIC LOADING (TRH16, 1991)

The determination of traffic loading after the date on which the information was collected is done by projecting the initial AADE using an appropriate growth rate. The E80 growth rate comprises two components:

- the increase in heavy vehicle traffic volume. This may be considered to consist of the overall traffic growth rate and the increase of heavy vehicles as a %age of total traffic; and
- the increase in the loading of heavy vehicles.

The E80 growth rate can be calculated in a number of different ways.

4.6.1 Using historical growth rates

Historical E80 growth rates on a given route may provide a start for evaluating potential future growth if the review suggests that heavy vehicle loading will be relatively stable. Formulae for the determination of E80 growth rates from different historical data are available (TRH16, 1991).

4.6.2 Using the South African Rural Traffic Model (SARTM)

The SARTM simulates the average annual daily traffic on an extensive network of South Africa's rural roads. Results of the model may be obtained from the Traffic Demand Modelling Working Group of the Department of Transport. The model can provide the growth rate of the heavy vehicle traffic on all the major corridors at future times. It would also be possible to evaluate the effect of new routes in terms of attracted traffic and growth rate (TRH16, 1991).

4.6.3 Subjective adjustment of the E80 growth rate

The designer should always critically evaluate growth rate figures that are obtained from whatever source and consider whether the figures are realistic in the light of knowledge about local conditions. The following should be considered:

- Will facilities in the area generate additional heavy vehicle journeys and, if so, for how long?
- What economic growth is expected for the area?
- Are alternative modes of transport possible or might they be constructed?
- How could future government legislation affect heavy vehicle growth, e.g. deregulation or axle load limits?
- How much traffic will be diverted to the planned new route initially?
- Could the growth rate be negative?
- Sensitivity analysis with different growth rates in E80s: 2, 4, 6 and 8 %.

Table 10 shows current typical ranges of the E80 growth rate for various road categories in South Africa.

TABLE 10

Current typical ranges of total E80 growth rates for different road categories (modified from TRH16, 1991)

Road Category	A	B	C	D
Range of growth rates (%)	2 - 12	2 - 12	2 - 10	2 - 15
Typical growth rates (%)	4	4	4	—*

* Greater uncertainty in growth rate.

4.7 PROJECTION OF TRAFFIC LOADING OVER THE STRUCTURAL DESIGN PERIOD

4.7.1 Projection to initial base year

The AADE obtained at a time earlier than the start of the design period may be projected to the initial design year by multiplying by a compound growth factor as follows:

$$g_x = (1 + 0{,}01.i)^x \qquad \ldots\ldots\ldots \qquad 4.4$$

where g_x = growth factor from Table 11
i = growth rate in % of E80s (per annum)
x = time in years between determination of axle load data and start of design period

then

$$AADE_{initial} = AADE \times g_x \qquad \ldots\ldots\ldots \qquad 4.5$$

where $AADE$ = Annual Average Daily E80 obtained from previous data

TABLE 11

Traffic growth factor (g_x) for calculation of future or initial traffic from historical traffic data

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

* $g_x = (1 + 0{,}01.I)^x$

4.7.2 Computation of cumulative E80s

The cumulative E80s over a period (e.g. the structural design period) may be calculated from the AADE in the initial design year and the E80 growth rate over the period.

The cumulative E80 per lane may be calculated from:

$$E80_{total} = AADE_{initial} \times f_y \qquad \ldots\ldots\ldots \quad 4.6$$

where f_y = cumulative factor from Table 12
y = structural design period

Note that different growth rates may apply to different parts of the analysis period. It is likely that growth rates will decrease as the road capacity is reached. For varying E80 growth rates, the AADE at the start of each growth period is calculated using the growth factor (g_x). The cumulative E80 over each growth period is then calculated using the cumulative growth factor (f_y). Finally, the cumulative E80s for each period is summed to obtain total cumulative E80s over the design period.

If the AADE at the end of a historic period ($AADE_{end}$) and the growth rate are known, the total cumulative E80s can be determined from:

$$E80_{total} = \frac{AADE_{end}}{g_x} \times f_y \qquad \ldots\ldots\ldots \quad 4.7$$

where g_x = growth factor given in Table 11
f_y = cumulative factor given in Table 12

Once the cumulative traffic demand has been determined in terms of E80s (or ESAs), the designer should go back to Table 4 to determine the design traffic class (ES0.003 to ES100). Interpolation of the pavement designs is possible (described in Section 8.4.3) if the pavement structure is to be designed to greater accuracy with regard to traffic.

TABLE 12

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

Prediction period, y (years)	* f_y for traffic increase, I (% per annum)									
	2	4	6	8	10	12	14	16	18	20
4	1 534	1 611	1 692	1 776	1 863	1 953	2 047	2 145	2 246	2 351
5	1 937	2 056	2 180	2 312	2 451	2 597	2 750	2 911	3 081	3 259
6	2 348	2 517	2 698	2 891	3 097	3 317	3 551	3 801	4 066	4 349
7	2 767	2 998	3 247	3 517	3 809	4 124	4 464	4 832	5 229	5 657
8	3 195	3 497	3 829	4 192	4 591	5 028	5 506	6 029	6 601	7 226
9	3 631	4 017	4 445	4 922	5 452	6 040	6 693	7 417	8 220	9 109
10	4 076	4 557	5 099	5 710	6 398	7 173	8 046	9 027	10 130	11 369
11	4 530	5 119	5 792	6 561	7 440	8 443	9 588	10 895	12 384	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	27 958	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	558 416	999 544	1 793 095	3 216 609

* $f_y = 365.(1 + 0,01.i).[(1 + 0,01.i)^y - 1]/(0.01.i)$

4.7.3 Geometric Capacity

To check that the cumulative E80s do not exceed the geometric capacity of the road, the total daily traffic (N) towards the end of the structural design period can be calculated from :

$$N = HVV_e(1+0{,}01.r_1)^n + LVV(1+0{,}01.r_2)^n \qquad \ldots\ldots\ldots \quad 4.8$$

where HVV_e = equivalent heavy vehicle volume at the start of the analysis period, i.e. heavy vehicle volume x 3, to convert to light vehicles
LVV = light vehicle volume at start of analysis period
r_1 = heavy vehicle volume growth rate
r_2 = light vehicle volume growth rate
n = analysis period in years

If the total traffic volume (N) exceeds the capacity laid down for the road category in TRH17 (1988), then either the road needs geometric upgrading or the assumptions need to be reviewed.

4.8 SENSITIVITY ANALYSIS OF DETERMINED TRAFFIC LOADING

Since the information used to calculate cumulative E80s is not exact, it is necessary to perform a sensitivity analysis to consider the influence changes in the basic data have on the final decision (see Section 4.6.3).

The subjective factors mentioned earlier and others which the designer considers relevant can be taken into account to determine the appropriate range of values possible for each variable. The cumulative E80s can then be determined for a range of values for each variable, using the given formulae to determine the effect of these changes.

5. MATERIALS

5.1 GENERAL LIST OF PAVEMENT MATERIALS AND ABBREVIATED SPECIFICATIONS

The selection of materials for a road pavement design is based on a combination of availability of suitable materials, environmental considerations, method of construction, economics and previous experience. These factors need to be evaluated during the design in consideration of the Life Cycle Strategy (LCS), in order to select the materials that best suit the conditions. In addition, information on material specifications for low volume roads is discussed in more detail in the discussion document: "Towards appropriate standards for rural roads" (DOT, RR 92/466/1, 1993).

The catalogue design procedure discussed here, generally uses the standard material specifications defined in TRH14 (1985). The classification of these materials is given in Table 13. The material codes listed in this table are used extensively in the Catalogue of designs. Only abbreviated specifications are given here and TRH14 (1985) should be used for more details. Waste materials (e.g. blast-furnace slags) and pedogenic materials have not been classified because of their varying quality. If these materials are used they should be classified under the appropriate material codes (TRH14, 1985). The materials are classified, according to their fundamental behaviour, into various categories with different classes according to their strength characteristics. Measured properties should be statistically assessed according to methods in TRH5 (1987). For detailed mechanistic design purposes (as opposed to the Catalogue given here) effective elastic moduli for the different materials under different conditions are given elsewhere (Theyse et al., 1996).

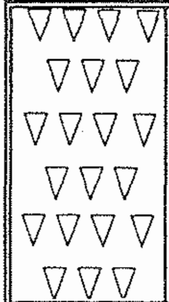
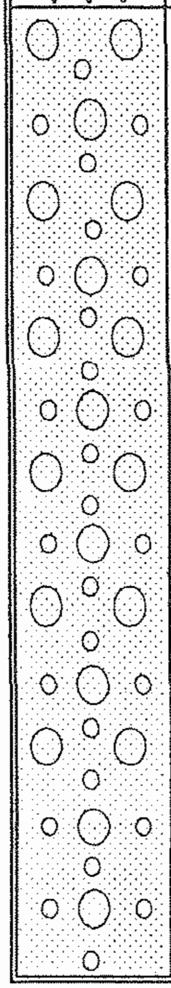
Material selection suitable for labour-intensive construction should be based on the general guidelines, as discussed later in Section 7.6 and Table 18.

5.2 DESCRIPTION OF MAJOR MATERIAL TYPES

This subsection describes the pavement construction materials in Table 13 and their major characteristics. The behaviour of different pavement types consisting of combinations of these materials is described in Section 8.2.

TABLE 13

Material symbols and abbreviated specifications used in the Catalogue designs

SYMBOL	CODE	MATERIAL	ABBREVIATED SPECIFICATIONS
	G1	Graded crushed stone	Dense - graded unweathered crushed stone; Maximum size 37,5 mm; 86 - 88 % apparent relative density; Soil fines PI < 4
	G2	Graded crushed stone	Dense - graded crushed stone; Maximum size 37,5 mm;100 - 102 % Mod. AASHTO or 85 % bulk relative density; Soil fines PI < 6
	G3	Graded crushed stone	Dense - graded stone and soil binder; Maximum size 37,5 mm; 98 - 100 % Mod. AASHTO ; Soil fines PI < 6
	G4	Crushed or natural gravel	Minimum CBR = 80 % @ 98 % Mod. AASHTO; Maximum size 37,5 mm; 98 - 100 % Mod. AASHTO; PI < 6; Maximum Swell 0,2 % @ 100 % Mod. AASHTO. For calcrete PI ≤ 8
	G5	Natural gravel	Minimum CBR = 45 % @ 95 % Mod. AASHTO; Maximum size 63 mm or 2/3 of layer thickness; Density as per prescribed layer usage; PI < 10; Maximum swell 0,5 % @ 100 % Mod. AASHTO *
	G6	Natural gravel	Minimum CBR = 25 % @ 95 % Mod. AASHTO; Maximum size 63 mm or 2/3 of layer thickness; Density as per prescribed layer usage; PI < 12; Maximum swell 1,0 % @ 100 % Mod. AASHTO *
	G7	Gravel / Soil	Minimum CBR = 15 % @ 93 % Mod. AASHTO; Maximum size 2/3 of layer thickness; Density as per prescribed layer usage; PI < 12 or 3GM** + 10; Maximum swell 1,5 % @ 100 % Mod. AASHTO ***
	G8	Gravel / Soil	Minimum CBR = 10 % @ 93 % Mod. AASHTO; Maximum size 2/3 of layer thickness; Density as per prescribed layer usage; PI < 12 or 3GM** + 10; Maximum swell 1,5 % @ 100 % Mod. AASHTO ***
	G9	Gravel / Soil	Minimum CBR = 7 % @ 93 % Mod. AASHTO; Maximum size 2/3 of layer thickness; Density as per prescribed layer usage; PI < 12 or 3GM** + 10; Maximum swell 1,5 % @ 100 % Mod. AASHTO ***
	G10	Gravel / Soil	Minimum CBR = 3 % @ 93 % Mod. AASHTO; Maximum size 2/3 of layer thickness; Density as per prescribed layer usage; or 90% Mod. AASHTO

* For calcrete PI ≤ 15 on condition that the Linear Shrinkage (LS) does not exceed 6 %.

** GM = Grading Modulus (TRH14, 1985) $= \dfrac{300 - [P_{2,00mm} + P_{0,425mm} + P_{0,075mm}]}{100}$

where $p_{2,00}$ etc., denote the percentage passing through the sieve size .

*** For calcrete PI ≤ 17 on condition that the Linear Shrinkage (LS) does not exceed 7 %.

TABLE 13 (Continued)

Material symbols and abbreviated specifications used in the Catalogue designs

SYMBOL	CODE	MATERIAL	ABBREVIATED SPECIFICATIONS
	C1	Cemented crushed stone or gravel	UCS**** : 6,0 to 12,0 MPa at 100 % Mod. AASHTO; Specification at least G2 before treatment; Dense - graded ; Maximum aggregate 37,5 mm
	C2	Cemented crushed stone or gravel	UCS : 3,5 to 6,0 MPa at 100 % Mod. AASHTO; Minimum ITS ***** = 400 kPa at 95 - 97 % Mod. AASHTO compaction; Specification at least G2 or G4 before treatment; Dense - graded; Max. aggregate 37,5 mm; Max. fines loss = 5 %******
	C3	Cemented natural gravel	UCS : 1,5 to 3,5 MPa at 100 % Mod. AASHTO; Minimum ITS***** = 250 kPa at 95 - 97 % Mod. AASHTO compaction; Maximum aggregate 63 mm; 5 % Maximum PI = 6 after stabilization; Max. fines loss = 20 %
	C4	Cemented natural gravel	UCS : 0,75 to 1,5 MPa at 100 % Mod. AASHTO; Minimum ITS***** = 200 kPa at 95 - 97 % Mod. AASHTO compaction; Maximum aggregate 63 mm; 5 % Maximum PI = 6 after stabilization; Max. fines loss = 30 %
	BEM	Bitumen emulsion Modified gravel	Residual bitumen: 0,6 - 1,5 % (SABITA, manual 14, 1993); Minimum CBR = 45 and Minimum UCS = 500 kPa @ 95 % Mod. AASHTO. Compaction: 100 - 102 % Mod. AASHTO
	BES	Bitumen emulsion Stabilized gravel	Residual bitumen 1,5 - 5,0 % (SABITA, manual 14, 1993); Minimum ITS***** = 100 kPa; Minimum resilient modulus 1000 kPa. Compaction: 100 - 102% Mod. AASHTO
	BC1	Hot - mix asphalt	LAMBS; Max. size 53 mm (SABITA, manual 13, 1993)
	BC2	Hot - mix asphalt	Continuously graded; Max. size 37,5 mm
	BC3	Hot - mix asphalt	Continuously graded; Max. size 26,5 mm
	BS	Hot - mix asphalt	Semi - gap graded; Max. size 37,5 mm
	AG	Asphalt surfacing	Gap graded (TRH 8, 1987)
	AC	Asphalt surfacing	Continuously graded (TRH 8, 1987)
	AS	Asphalt surfacing	Semi - gap graded (TRH 8, 1987)
	AO	Asphalt surfacing	Open graded (TRH 8, 1987)
	AP	Asphalt surfacing	Porous (Drainage) asphalt (SABITA, manual 17, 1994)
	S1	Surface treatment	Single seal (TRH 3, 1996)
	S2	Surface treatment	Multiple seal (TRH 3, 1996)
	S3	Sand seal	See TRH 3, 1996
	S4	Cape seal	See TRH 3, 1996
	S5	Slurry	Fine grading
	S6	Slurry	Medium grading
	S7	Slurry	Coarse grading
	S8	Surface renewal	Rejuvenator
	S9	Surface renewal	Diluted emulsion
	WM1	Waterbound macadam	Max. size 75 mm; Max.PI of fines = 6; 88 - 90 % apparent relative density
	WM2	Waterbound macadam	Max. size 75 mm; Max.PI of fines = 6; 86 - 88 % apparent relative density
	PM	Penetration macadam	Coarse stone + keystone + bitumen
	DR	Dumprock	Upgraded waste rock, maximum size 2/3 layer thickness

**** UCS: Unconfined Compressive Strength (TMH 1, 1979, Method A14)

***** ITS: Indirect Tensile Strength (SABITA Manual 14, 1993)

****** Durability (TMH 1, 1979, Method A19)

5.2.1 Granular materials and soils (G1 to G10)

Generally, these materials show stress-dependent behaviour and, under repeated stresses, deformation can occur through shear and/or densification. However, by controlling the applied stresses (i.e. traffic loading), the rate of this deformation can be controlled.

A G1 is a dense-graded, unweathered, crushed stone material compacted to 86 to 88 % of apparent relative density (CSRA, 1987). The range of density given here allows for contract pricing at different target densities. An unsuitable gradation may be adjusted only by the addition of crusher sand or other stone fractions obtained from crushing of *parent rock*. A G2 material is crushed stone material compacted to 100-102 % mod. AASHTO density or 85 % of bulk relative density. G2 and G3 may be a blend of crushed stone and other fine aggregate used to adjust the grading.

G4 to G10 materials cover the range of relatively high quality gravels (CBR 80 % @ 98 % mod. AASHTO to 25 % @ 95 % mod. AASHTO) to relatively lower quality materials (CBR 15 % at 93 % mod. AASHTO to CBR 3 % @ 93 % mod. AASHTO) used in pavement layers. The durability of these materials should be checked against current specifications especially when using marginal materials on low volume roads (DOT, RR 92/466/1,1993; RR 88/032:1102, 1990).

5.2.2 Modified materials

Natural gravels may be modified with relatively low quantities of cementitious or bituminous stabilizers (2 - 3 %). This practice has value in so far as it improves material workability and moisture sensitivity, even though the quantity may be too low to prevent detrimental carbonation in the cemented materials. Further details on material durability is given elsewhere (DOT, RR 92/466/1, 1993).

Further compaction as well as quality control testing of this material is done approximately 24 hours after mixing. Bitumen emulsion treated materials for modification are discussed under Section 5.2.4.

5.2.3 Cemented materials (C1 to C4)

Cemented materials, similar to concrete, are initially elastic and have limited tensile strength and usually crack under repeated flexure. These materials also crack because of shrinkage during drying. By the application of an upper limit to the strength specification, wide shrinkage cracks can be avoided. In addition, an unbound layer covering (overlay) the cemented layers can be used to prevent penetration of reflective shrinkage cracks from the cemented layers to the surface. A C2 material may be used when a non-pumping, erosion-resistant layer is required (as for a subbase of a concrete pavement, see also DOT, manual 10, 1991).

Both C3 and C4 materials can be used in place of natural gravel layers in bases and subbases (De Beer, 1985). They can be either cement treated or lime treated, depending on the properties of the natural materials. The longer term durability (DOT, RR 88/032, 1990) and resistance to erosion (DOT, RR 91/167, 1993) of these materials should, however, be carefully assessed.

5.2.4 Bitumen emulsion treated materials (BEM and BES)

Bitumen emulsion treated materials, also known as Granular Emulsion Mixes (GEMS) (SABITA, manual 14, 1993), are granular materials which have been modified with emulsion containing 0,6 % - 1,5 % residual bitumen, i.e. Bitumen Emulsion Modification (BEM) or stabilised with emulsion containing 1,5 % - 5,0 % residual bitumen, i.e. Bitumen Emulsion Stabilisation (BES).

Although no separate GEM pavement designs are given in the Catalogue at this stage, the practice of emulsion treated materials is relatively well understood and widely used (SABITA, manual 14, 1993; De Beer et al., 1994). Although most of the pavements in the Catalogue containing natural gravel bases may benefit from this practice in terms of workability and potential enhanced performance, there are no proven mechanistic design guidelines. However, this should not prevent designers from using this technique in an innovative manner (De Beer et al., 1994; Van Wyk & Louw, 1995).

5.2.5 Hot-mix asphalt materials (BC1 to BS)

Hot-mix asphalt materials are visco-elastic and may be temperature dependent. Under repeated stresses the asphalt layer may either crack or deform or both. At low temperatures asphalt cracks and at high temperatures it deforms, usually by rutting. This behaviour, however, also depends on the type and quality of the supporting layers in the pavement structure.

The hot-mix asphalt material class has been divided into BC1, BC2, BC3 based on the maximum aggregate size, and includes Large Aggregate Mixes for Bases (LAMBS) (SABITA, manual 13, 1993).

5.2.6 Surfacings (AG to AP; 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 (TRH8, 1987). They also include porous asphalt surfacing layers (AP) (SABITA, manual 17, 1994).

5.2.7 Macadams (WM to PM)

Waterbound Macadams (WM) consist of a coarse, single-sized aggregate together with fine material which is added during construction to fill the voids. A wet or dry method of construction can be used. WM1 is compacted to 88 to 90 % of apparent relative density and WM2 to 86 to 88 % of apparent relative density. These materials show stress-dependent behaviour similar to that of other granular materials. Under repeated stresses, deformation can occur through shear and/or densification. Waterbound macadams are highly resistant to the influence of water and are most likely to be used for pavements carrying heavy traffic in wet regions. They further have the potential to create significantly more employment opportunities than crushed stone bases. Recent research indicated that Waterbound Macadam base courses are capable of withstanding significantly higher shear stresses than crushed stone base courses (Phillips, 1994). The designer should, however, satisfy himself that waterbound macadam can be constructed with the available materials and construction methods.

Penetration Macadam (PM) has a coarse single-sized aggregate and the voids are partially filled with hot bitumen.

5.2.8 Segmental concrete paving blocks

Segmented concrete block pavements, however, are not covered in the Pavement Design Catalogue discussed here, as it is felt that they are classified to fall under the innovative pavement design category. More information on the structural design of these pavements can be obtained elsewhere (UTG2, 1987; DBSA, Number 8, 1993; BS 7533, 1992).

5.3 AVAILABILITY, EXPERIENCE AND CURRENT UNIT COSTS

The designer should complete the checklist in Table 14 in order to ensure that the type of material is currently available. Past experience with a particular material should be taken into consideration together with the applicable traffic and environmental factors.

The unit costs of the available materials should be determined to calculate the construction costs of pavement alternatives (see Section 9) from a survey of recent unit costs. The CSRA database of tendered rates (CSRA, 1995) is a useful source of information in this regard but it should be borne in mind that tendered rates are specific to project circumstances.

6. ENVIRONMENT

6.1 GENERAL

The climatic conditions (moisture and temperature) under which the road will function, as well as the underlying subgrade conditions, define the environment. The environment must be taken into account in the design of pavement structures.

Usually, the lighter the pavement (and traffic) the more pronounced the relative effect of the environment will be.

6.2 CLIMATIC REGIONS AND THE DESIGN OF PAVEMENTS

The moisture conditions will largely determine the weathering of natural rocks, the durability of weathered natural road building and also, depending on drainage conditions, the stability of untreated materials in the pavement. The climate may also influence the equilibrium moisture content. Ambient pavement temperatures may affect surfacing stability (DOT, PR 91/014, 1992). The designer should always consider the climatic conditions and avoid using excessively water susceptible or temperature sensitive materials in adverse conditions.

Southern Africa can be divided into three climatic regions:

(a) a relatively large dry region;

TABLE 14

Checklist of material availability and unit cost

SYMBOL	CODE	MATERIAL	AVAILABILITY	UNIT COST
	G1	GRADED CRUSHED STONE		
	G2	GRADED CRUSHED STONE		
	G3	GRADED CRUSHED STONE		
	G4	NATURAL GRAVEL		
	G5	NATURAL GRAVEL		$/m^3$
	G6	NATURAL GRAVEL		
	G7	GRAVEL/SOIL		
	G8	GRAVEL/SOIL		
	G9	GRAVEL/SOIL		
	G10	GRAVEL/SOIL		
	C1	CEMENTED CRUSHED STONE OR GRAVEL		
	C2	CEMENTED CRUSHED STONE OR GRAVEL		$/m^3$
	C3	CEMENTED NATURAL GRAVEL		
	C4	CEMENTED NATURAL GRAVEL		
	BEM	BITUMEN EMULSION MODIFIED GRAVEL		$/m^3$
	BES	BITUMEN EMULSION STABILISED GRAVEL		
	BC1	HOT - MIX ASPHALT		
	BC2	HOT - MIX ASPHALT		$/m^3$
	BC3	HOT - MIX ASPHALT		
	BS	HOT - MIX ASPHALT		
	AG	ASPHALT SURFACING		
	AC	ASPHALT SURFACING		$/m^3$
	AS	ASPHALT SURFACING		
	AO	ASPHALT SURFACING		
	AP	ASPHALT SURFACING		
	S1	SURFACE TREATMENT		
	S2	SURFACE TREATMENT		
	S3	SAND SEAL		$/m^2$
	S4	CAPE SEAL		
	S5	SLURRY		
	S6	SLURRY		
	S7	SLURRY		
	S8	SURFACE RENEWAL		/litre
	S9	SURFACE RENEWAL		
	WM1	WATERBOUND MACADAM		
	WM2	WATERBOUND MACADAM		$/m^3$
	PM	PENETRATION MACADAM		
	DR	DUMPROCK		

(b) a moderate region; and
(c) a few small wet regions.

Figure 4 shows a map of southern Africa which indicates the different climatic regions. These are macro climates and it should be noted that different micro climates, where local areas may have high moisture conditions, could occur within these regions. It is up to the designer to identify problems that may be caused by local micro climates.

6.3 CLIMATE AND SUBGRADE CBR

The classification of the subgrade material is based on the soaked California Bearing Ratio (CBR) at a representative density (TRH14, 1985). For structural purposes, when a material is classified according to the CBR, it is implied that no 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 their use in dry regions may be over-conservative (Haupt, 1980). A procedure has been developed (Emery, 1992) to adjust soaked CBR values to unsoaked CBR values under certain conditions so that lower quality materials can be used by assuming an increased risk determined in a probabilistic manner. However, the CBR intended to be used for structural design should be done at the highest anticipated moisture content when compacted to the specified field density (TRL, 1993). It is also advisable to do a sensitivity analysis on the required CBR of the subgrade in order to quantify its effect on the final pavement design.

6.4 MATERIAL DEPTH

The concept of "material depth" is used here to denote the depth below the finished level of the road to which soil characteristics have a significant effect on pavement behaviour. In addition, the moisture regime needs to be carefully controlled by, for example, the provision of adequate subsurface drainage and/or surface drainage (the former being a function of the micro climatic conditions as well as the depth of surface drain provided). 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 required for a soil of one to two % soaked CBR. However, this depth may be insufficient in certain special cases listed in Section 7.3.

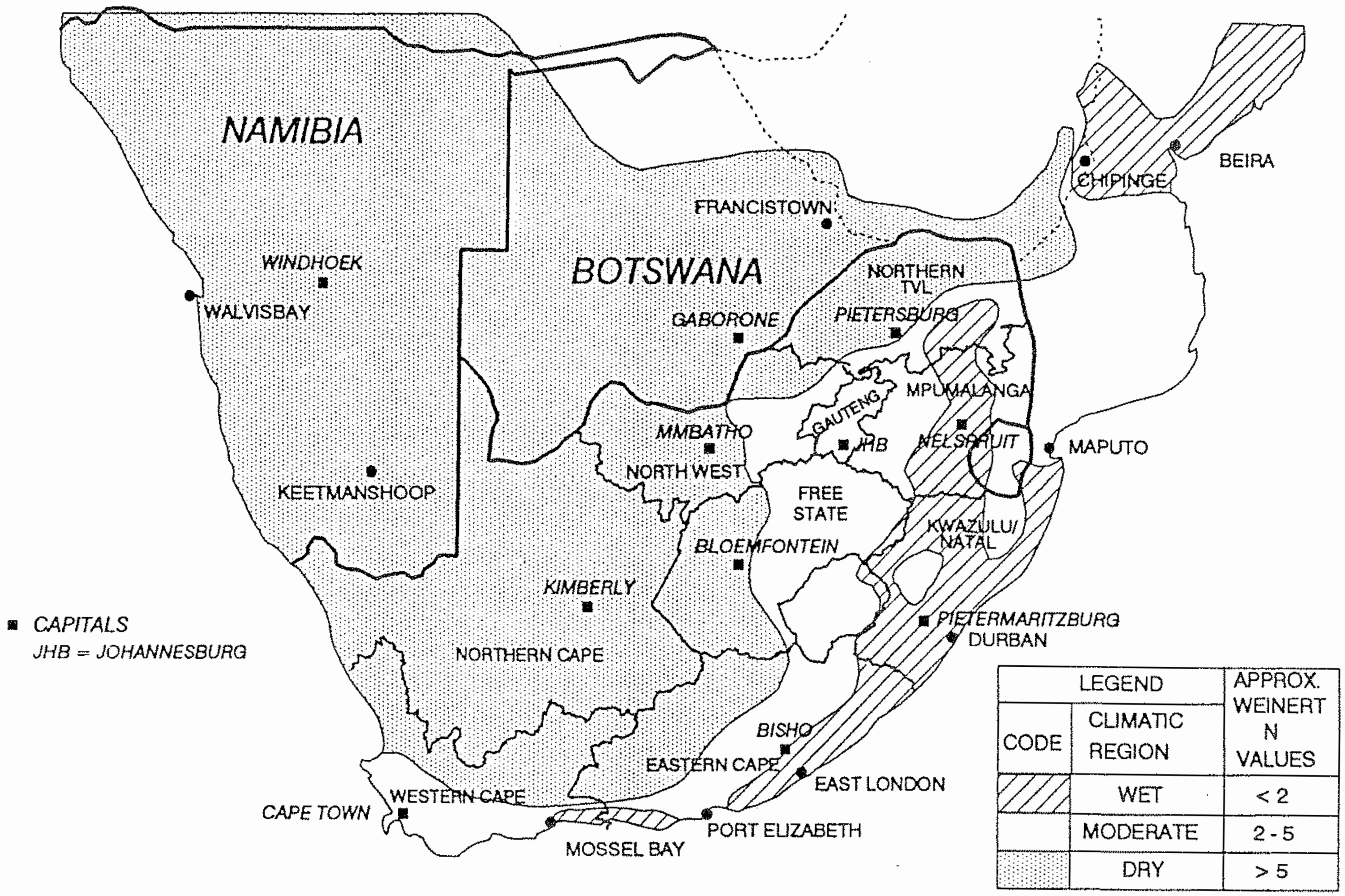


FIGURE 4: MACRO CLIMATIC REGIONS OF SOUTHERN AFRICA
(Adapted from Weinert, 1980)

Table 15 specifies typical material depths used for determining the design CBR of the subgrade for the different road categories.

6.5 DELINEATION OF IN SITU SUBGRADE AREAS

Any road project should be subdivided into similar subgrade areas. However, too fine a delineation could lead to confusion during construction. The preliminary soil survey should delineate the in situ subgrade (and/or roadbed, see Figure 9) into design units on the basis of geology, pedology, topography and drainage conditions at the site, or major soil boundaries, so that an appropriate design CBR is defined for each in situ subgrade unit.

TABLE 15

Typical material depths to be used for determining the design CBR of the in situ subgrade

Road Category	Material depth (mm)*
A	1 000 - 1 200
B	800 - 1 000
C	800
D	700

* Total thickness of pavement above the roadbed (see Figure 9). (The material depth is based on CBR cover curve approximation).

The designer should differentiate between localised appropriate or poor in situ soils and more general in situ subgrade areas. Localised soils should be treated separately from the rest of the pavement design. Normally, localised poor soils will be removed and replaced with suitable material.

6.6 DESIGN CBR OF IN SITU SUBGRADE

The aim of the design process is to protect, use and improve the bearing capacity of the in situ subgrade material so that the pavement will be able to fulfill the service objective over the analysis period (as explained previously under Section 3.4). The bearing capacity and quality of the subgrade/ roadbed/fill is of prime importance in the selection of the appropriate pavement type and hence the overall life cycle strategy.

The bearing capacity of the subgrade is improved by overlaying it with only the necessary (and appropriate) layers of material to achieve an integrated structurally balanced pavement system. The final pavement should then have the necessary design bearing capacity (and quality) to ensure fulfilment of the functional serve (performance) over the structural design period.

For practical reasons the design subgrade bearing capacity (CBR), given in this document, is limited to four classes, as shown in Table 16.

TABLE 16

Subgrade CBR classification for structural design

Class	Subgrade CBR (%)
SG1	> 15
SG2	7 to 15
SG3	3 to 7
SG4	< 3*

* Special treatment required.

The CBR is normally determined after specimens have been soaked for four days and may be adjusted according to Section 6.3. Special measures are necessary if a material is identified as Class SG4 (CBR < 3 %) within the material depth (Table 15). These include stabilisation (chemical or mechanical), modification (chemical) and the removal or addition of extra cover. After the special treatment, the material will be classified under one of the remaining three subgrade classes (SG1, SG2 or SG3).

6.7 DESIGN CBR ON FILL

When the road is constructed on fill, the designer must use 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, as is given in Table 15. TRH9 (1982) should be consulted when a material with CBR < 3 % is used in the fill.

6.8 DESIGN CBR IN CUT

The design CBR of the roadbed in a cut should be taken as the lowest realistic CBR encountered within the material depth.

7. PRACTICAL CONSIDERATIONS

7.1 DRAINAGE

Experience has shown that inadequate drainage is probably responsible for more pavement distress in southern Africa than inadequate structural or material design. Both the discharge of surface run-off and the control of subsurface water need to be considered.

Drainage design is discussed in detail in draft TRH15 (1995). The basic philosophy is to provide effective drainage to at least material depth so that the pavement structure is prevented from becoming saturated. Effective drainage is essential for proper pavement performance, since it is assumed in the structural design procedure.

Surface run-off is generally more readily controlled but attention needs to be paid to subsurface drainage. Certain in situ materials that are highly permeable, e.g. some Kalahari and Cape Flat sands, are free draining and require little extra drainage except where they are associated with high water table conditions.

Impermeable materials (or zones) may trap moisture and cause the pavement layers to become saturated, with an associated loss in strength. These will require deeper subsurface drainage.

Particular attention needs to be paid to cuttings, perched water tables, vleis, etc..

7.2 COMPACTION

Pavement layer compaction during construction has a major effect on the structural bearing capacity of a pavement. The higher the construction density of a layer, the higher the strength, (i.e. bearing capacity), and hence, the resistance to permanent (plastic) deformation. In addition, potential ingress of water into the pavement layer is also restricted as a direct result of higher compaction densities. *Effective compaction* is therefore one of the most cost-effective methods to *improve the structural capacity* of pavements. Ultimately, effective compaction may lead to achieving the highest "dry" density (i.e. "refusal" density) that can be obtained during construction by a normal compaction effort.

Design procedures assume that the material properties specified *are achieved* in the field. During the process of constructing any of the foundation layers of the road, the aspect of *uniformity of materials*, the *uniform application of water*, the *uniformity of mixing* and the *uniformity of compaction* at *optimum moisture content* (which should be carefully monitored) is *essential* for attaining the required specified densities. Different materials, however, react differently to different items of plant (or combinations of plant) (e.g. sheepfoot grid roller, heavy pneumatic roller, vibratory roller, etc.).

Granular materials, which are *well graded* (e.g. gradation curve falling on the maximum density line) are easier to compact than poorly graded materials and it may be more economical to get the gradation right before wasting time and energy with excessive rolling.

It is, however, also important to note that the *layer(s) below* the one being compacted should be of *sufficient density and strength* to *facilitate* effective compaction of the upper layers(s). Adequate compaction of asphalt layers is also crucial to satisfactory performance (SABITA, manual 5, 1992).

Table 17 gives the *minimum compaction requirements* for the various layers in the pavement structure.

7.3 SUBGRADE BELOW MATERIAL DEPTH

Certain special problems may arise in the subgrade which require individual treatment (refer to 6.5). The design procedure assumes that these have been taken into account separately.

The main problems which need to be considered are:

(a) the excessive volume changes that occur in some soils as a result of moisture change, e.g., expansive soils and soils with collapsible structures;
(b) flaws in structural support, e.g., sinkholes, mining subsidence and slope stability;
(c) the non-uniform support that results from wide variations in soil types or states;
(d) the presence of soluble salts which, under favourable conditions, may migrate upwards and cause cracking, blistering or loss of bond of the

surfacing, the disintegration of cemented bases and loss of density of untreated bases;

(e) the excessive deflection and rebound of highly resilient soils during and after the passage of a load, e.g., ash, micaceous and diatomaceous soils; and

(f) the impact of moles, ants and other burrowing animals. A cemented subbase or top selected layer can be used to bridge cavities and maintain an adequate riding quality in this case.

The techniques available for terrain evaluation and soil mapping are given in TRH2 (1978). The design of embankments should be done in accordance with TRH10 (1987).

TRH5 (1987) describes statistical methods for evaluating field test results and performing construction quality control of pavements.

7.4 PAVEMENT CROSS-SECTION

7.4.1 Variable 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, e.g., climbing lanes, the pavement structure may be varied over the cross-section of the carriageway. Provided that it is economical and practical under these circum-circumstances, the actual traffic predicted for each lane should be used for determining the design traffic. In addition, climbing and passing lanes should be wide enough to prevent channelization of traffic loading, as narrow lanes will increase the risk of failure (rutting, cracking, etc.).

The cross-section can be varied by having steps in the layer thicknesses. Under no circumstances should the steps be located in such a way that water can be trapped in them.

For low volume rural access roads the reader should consult "Guidelines for upgrading of low volume roads" (DOT, RR 92/466/2, 1993).

TABLE 17

Nominal field compaction requirements for construction of pavement layers

Pavement layer	Material or layer	Target density (Relative compaction)
Base	Hot-mix asphalt	97 % minus design voids-in-mix*
	Crushed stone:	
	G1	86 - 88 %** apparent relative density
	G2	100 - 102 % mod AASHTO or 85 % bulk relative density
	G3	98 - 100 %** mod AASHTO
	Gravel G4	98 - 100 %** mod AASHTO
	Waterbound macadam:	
	WM1	88 - 90 % apparent relative density
	WM2	86 - 88 % apparent relative density
	Cemented (C3 / C4)	97 - 98 % mod AASHTO
Subbase	Gravel (G4 / G5):	
	upper	95 - 97 %** mod AASHTO
	lower	95 % mod AASHTO
	Cemented (C3 / C4):	
	upper	96 % mod AASHTO
	lower	95 % mod AASHTO
Selected layers	Upper	93 - 95 %** mod AASHTO
	Lower	90 - 93 % mod AASHTO
Fill (within material depth)	Gravel	90 %*** mod AASHTO
	Sand	100 % mod AASHTO
Roadbed (within material depth)	Gravel	90 % mod AASHTO***
	Sand	100 % mod AASHTO
Shoulder	Gravel	93 % mod AASHTO

* Dependant on road authority.

** Optimum compaction for the particular material is specified.
Allows for variable contract pricing at different target densities.

*** To reduce longitudinal cracking due to differential settlement subgrade within material depth to be compacted to 93 % when widening takes place.

7.4.2 Shoulder design for flexible pavements

The shoulders are normally surfaced for Category A and B roads, and often for Category C roads as well. Figures 7 and 8 (see Section 8.4) can be used to determine the effect shoulder surfacing will have on pavement moisture condition and consequently on material and pavement selection. The design traffic aspects concerning lane distribution, as given in Table 9, must also take the shoulder into consideration.

7.5 CONSIDERATIONS FOR MANUAL LABOUR-INTENSIVE ROAD PAVEMENT CONSTRUCTION

7.5.1 Introduction

Labour-intensive projects affect the selection of the road category, pavement layer types as well as some practical considerations. Some pavement layer and material types are more suited to labour-intensive construction. It is critical that the *design* and *maintainability* should be selected with *constructibility* in mind. However, alternative designs (and methods) should be compared based on the life cycle strategy approach discussed in Section 1.5. A further important consideration is the social consequences (e.g. number of employment opportunities) created for which methods such as the cost of technique analysis (COTA) (Phillips, 1994) can be used.

Labour-intensive construction involves the use of manual labour rather than machines, where technically and economically feasible. The economic limits to using labour rather than machines varies according to a number of factors, including, among others, the productivity of labour, the cost of labour, the quantities of work involved, socio-economic considerations and time and safety restrictions.

Generally speaking, it is economically more feasible to use highly labour-intensive construction methods on lightly trafficked roads which do not require large quantities of earthworks to meet geometrical design specifications.

However, there is always a range of possible machine-manual combinations, the manual labour intensity can also be increased significantly on more heavily trafficked roads, albeit less so than on lightly trafficked roads.

All pavement layers have a good potential for manual labour, but only WM, PM and emulsion treated layers have a very good potential for manual labour (see Table 18). However, appropriate equipment should be used in conjunction with the potential job creation benefit.

The potential for manual labour or the employment creation potential, should not be the only consideration in the selection of a type of pavement. The potential for the involvement of emerging contractors should also be paramount.

Good engineering practice is equally important for labour-intensive construction. The following are some important factors to be considered:

- The appropriate material tests (i.e. gradation, PI, etc.) to be carried out on all materials before using them with confidence.

- The differentiation between new construction on virgin soils (which may need more manual labour) and upgrading of an existing road profile where some traffic compaction ("bedding in") has taken place.

- By virtue of the slow nature of manual labour on pavement layer construction, the completed layers are usually unprotected. If the unfinished base layer is used by traffic or pedestrians in this period, the surface may degrade, which will impede effective final surfacing (bitumen). Therefore, the final base layer should preferably be "primed" on a daily basis to service as protection.

- Cement and lime stabilised natural gravel layers can be very effectively used if mixed in a concrete mixer.

It is, however, important to note that the potential for manual labour assumes *appropriate equipment and material* for labour-intensive construction.

In Table 18 possible design options for low traffic volume roads and their potential for labour-intensive construction are given.

TABLE 18

Possible design options for low traffic roads and their potential for labour-intensive construction (adapted from DBSA, Number 2, 1993)

PAVEMENT MATERIAL	POTENTIAL FOR MANUAL LABOUR
Subbase	**Subbase**
In situ soil	Good
Imported soil *	Good
Base *	**Base** *
In situ soil	Good
Gravel (natural)	Good
Gravel (cement stabilised)	Good (use with concrete mixer)
Gravel (lime stabilised)	Good (use with concrete mixer)
Stone (crushed stone)	Good
Waterbound macadam (WM)	Very good
Concrete (plain and reinforced)	Good
Hot-mix asphalt (in 40 mm layers)	Practical (if mixing plant readily available)
Penetration macadam (PM)	Very good
Bitumen emulsion treated materials (BEM and BES)	Very good (no "prime" needed)
Surfacing **	**Surfacing** **
Sand seal	Good
Slurry	Good
Double seal	Fair to good ***
Single seal	Fair
Cape seal	Good (slurry application by hand)
Hot-mix asphalt	Practical (if mixing plant readily available)

* Suitability will depend on haul distance (trucks or tractor-trailers are usually used for long hauls).

** Consult DBSA, Number 2, 1993; SABITA, manual 10, 1992; SABITA, manual 11, 1994 for more information on asphalt surfacing.

*** If motorised hand sprayer is used for spray application – aggregate and rolling by hand.

7.6 INTEGRATED ENVIRONMENTAL MANAGEMENT OF ROAD CONSTRUCTION

Road construction impacts significantly on both the bio-physical and socio-economic environment. Integrated Environmental Management (IEM) is a process designed to minimise the negative environmental consequences of development. IEM identifies possible impacts during and after construction, quantifies these through various methodologies and then devises strategies for minimising their effects. It is important that this is done during the planning stage while alternatives can still be considered.

The designer should list any social or environmental impacts which the project will have. If no impacts are identified (unlikely unless the project is very small), the project can proceed without an initial impact assessment. However, one should be constantly aware of impacts that might arise during the course of the development.

If impacts are uncertain or significant, a preliminary or full assessment should be undertaken (DOT, RR 92/466/1, 1993).

8. PAVEMENT TYPE SELECTION AND STRUCTURAL DESIGN

8.1 PAVEMENT DESIGN METHODS

The designer may use a number of existing design procedures. Whatever the method used, factors such as service objective, road category, design strategy, traffic, available materials and environment must be taken into account. Some estimation of future maintenance and rehabilitation measures is necessary before a comparison of different structures can be made on the basis of present worth of whole life cycle costs. Special construction considerations that might influence either pavement structure or pavement costs are discussed in Sections 7 & 9.

This document, however, is mainly aimed at the use of a Catalogue of Designs. The Catalogue given here introduces of the concept of design reliability. The best results will probably be obtained if the Catalogue is used as a guideline ***together*** with some of the other existing design methods. Some locally used design methods are referenced below. For the application of these design methods the designer should rely on his personal experience and understanding in consultation of the detail of each method, given in the appropriate references.

8.1.1 South African Mechanistic Design Method (SAMDM)

The South African Mechanistic Design Method (SAMDM) uses linear elastic theory to determine theoretical stresses and strains at different positions in the pavement layers (Theyse et al., 1995; TRH4; Freeme et al., 1982; Walker et al., 1977). The stresses and strains which are most likely to initiate failure in a particular material type have been related to traffic load, via appropriate transfer functions. Some of these transfer functions were calibrated with Heavy Vehicle Simulator (HVS) testing and are given elsewhere (Theyse et al., 1995; TRH4).

The method is applicable to a wide range of materials and pavement types. Most of the current designs given in the Catalogue are based on this method and incorporate an appropriate design reliability[5] for each design given.

8.1.2 The Dynamic Cone Penetrometer (DCP) method

The Dynamic Cone Penetrometer (DCP) method is an empirical method developed locally during the 1970s by using the in situ measured bearing capacity of existing pavements and correlating it with the Heavy Vehicle Simulator (HVS) tests on similar material and pavement types (Kleyn, 1982; De Beer et al., 1989; De Beer, 1991).

This method is based on the structural balance concept of the pavement (explained in previous Section 3.4) and supplies the designer with the minimum in situ layer bearing capacity for a specified pavement balance. The designer, however, is still obliged to obtain and apply all material qualities as with the other design methods.

8.1.3 The Elasto-Plastic Design Method (S-N method)

The basic Elasto-Plastic (EP) design method (also referred to as the SN method (Stress-Number of load repetitions) for granular layers was developed in 1992 (Wolff, 1992). It uses the bulk stress calculated from non-linear elastic theory to determine the number of load applications an unbound pavement layer will carry until a predefined rutting criteria (plastic deformation) is reached. The strength and thickness of each layer is adjusted until the load applications for each layer are approximately similar, and the total plastic deformation (rutting) on the surface of the pavements is acceptable. The EP method is applicable to most granular layers and has successfully been applied for the design of lightly trafficked access roads (DOT, RR 92/312, 1993). This design method also incorporates the concept of *design reliability*.

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[5] For definition of design reliability, see Footnote 3 in Section 3.3.1 on Page 13.

8.1.4 California Bearing Ratio (CBR) based design methods

The California Bearing Ratio (CBR) design method was developed in the 1950s and uses empirical design charts to determine the material thickness required to protect a lower layer with a specified CBR strength (Yoder et al., 1975; Maree et al., 1984). Basically the CBR test method is used for the evaluation of unbound gravel *material quality* (see Table 13). In addition to material quality, CBR cover curves (CBR based pavement thickness design curves) were developed during 1960s, based on the CBR of the subgrade. These design curves were used to determine the cover (thickness) of unbound material (layers) required on the subgrade to produce the final *pavement design*. It is, however, important to note that when using the relevant material, environmental conditions and traffic loading, cognisance must be taken of the applicability of the CBR cover curve method to South African conditions, including road embankments (TRH9, 1982).

The CBR cover curve design method is, however, not applicable to pavement structures incorporating bound layers (i.e. stabilised or thick asphaltic materials).

8.1.5 AASHTO Guide for Design of Pavement Structures (AASHTO, 1993)

The AASHTO Guide for Design of Pavement Structures (AASHTO, 1993) provides the designer of a comprehensive set of procedures which can be used for the design and rehabilitation of pavements. This guide serves as a good background to pavement design in general.

Owing to the empirical nature of some of the guidelines given, careful consideration should be given to its application within the southern African context.

8.2 BEHAVIOUR OF DIFFERENT PAVEMENT TYPES

8.2.1 Behaviour of road pavements

For the effective design of pavements, based on the life cycle strategy, an understanding of the general behaviour of the various pavement compositions is vital.

The behaviour and performance of a pavement may be expressed and illustrated in a number of ways. One such illustration is shown in Figure 5 where the general trend of pavement "distress" is depicted against time/traffic. Note that the aspect of "distress" may be composed of one or a number of parameters, such as patching, cracking, deformation, etc..

Figure 5 is intended to illustrate a number of basic concepts, namely:

- ***The Initial Phase:*** Initially, or after reconstruction, when a pavement is open to traffic a notable increase in "bedding in" could be observed. The length and severity of this phase is an indication of the inherent ability of the pavement to withstand the traffic loads as well as an indication of the construction and material quality. Normally for a well designed pavement, this phase is relatively short and the effect only noticeable to the trained eye. However, an under-designed or poorly constructed pavement may fail prematurely, as indicated in Figure 5.

- ***The Primary Phase:*** After the Initial Phase a pavement will normally exhibit a much diminished deterioration rate, a period that may be termed the "stable" or "working" phase, during which period the service is reliable. The aim is to conserve this phase for as long as possible through timely preventative maintenance and occasional programmed rehabilitation actions.

- ***The Accelerated Distress Phase:*** When a pavement reaches the end of its structural design period (Section 3) or if the preventative maintenance is neglected, inappropriate or deferred, a third phase will set in, namely the "accelerated distress" phase. During this phase the deterioration of the pavement increases rapidly and a "terminal condition" (F1 in Figure 5) is reached. If this phase is allowed to continue the pavement will reach a predetermined terminal level and the condition of the pavement is considered as "failed". However, if appropriate reactive maintenance and/or rehabilitation measures are taken during this phase, this terminal trend may be converted into a secondary stable or working phase, as shown in Figure 5. However, the rate of deterioration may be higher in this phase than before, unless special measures have been taken to strengthen or upgrade the bearing capacity of the pavement and/or improve the capacity of the pavement to accommodate the pavement environment appropriately.

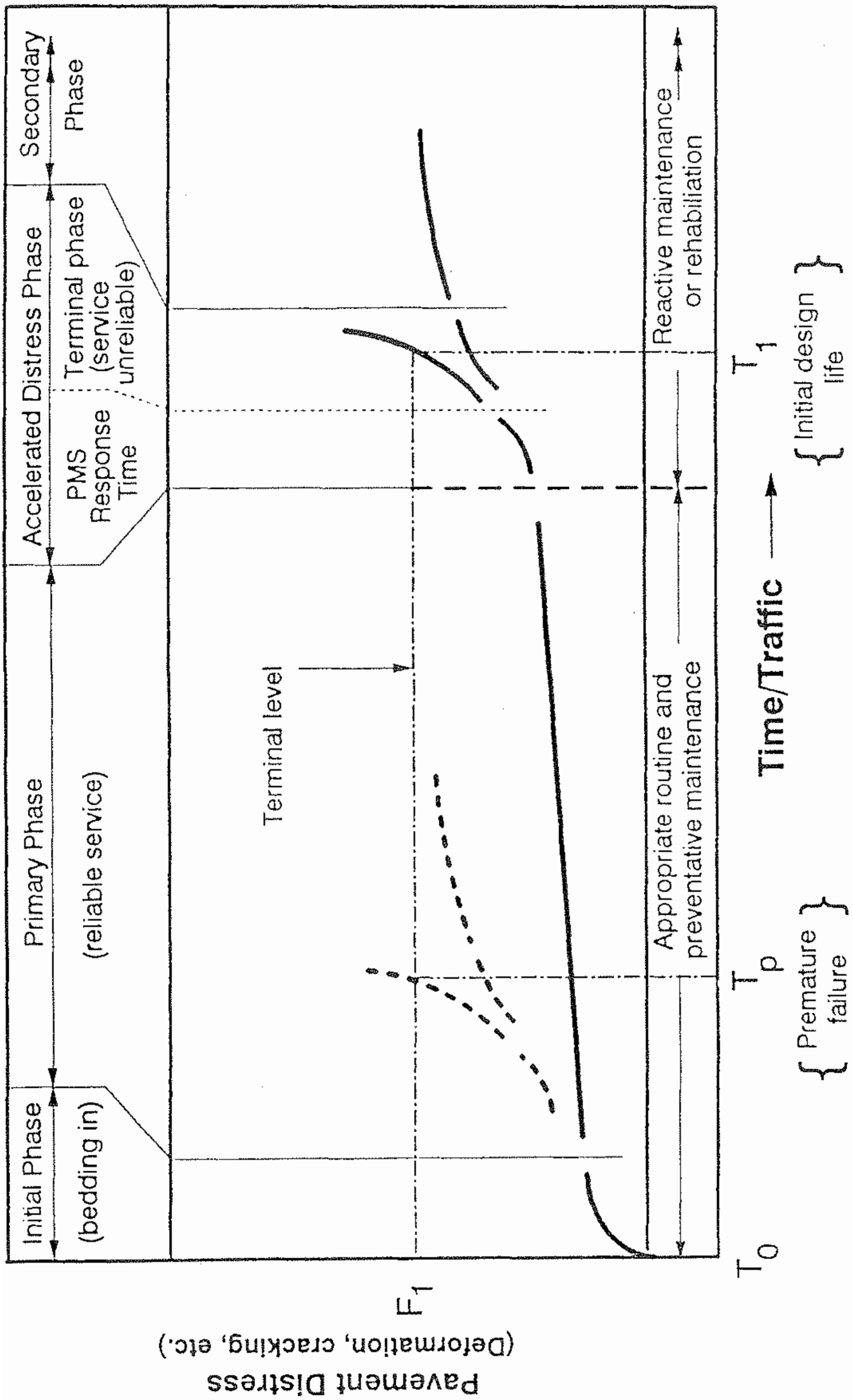


FIGURE 5: GENERALISED BEHAVIOUR OF FLEXIBLE ROAD PAVEMENTS

Various failure criteria and functional thresholds may be set for a pavement. For example, as shown in this case, when the percentage of "distress" for this section of road reaches the terminal failure criteria, F1 in Figure 5, this section of road is deemed to have "failed" by definition of the failure criteria. This does not mean that the pavement is necessarily "untrafficable", but it means that the level of functional service rendered by the road has fallen below the service objective for the road (Section 1). The in situ pavement will certainly have some "residual structural value" which can and should be harnessed as a base for the rehabilitation action.

The behaviour of a pavement and the critical levels of the different types of distress, can be *pre-set and pre-determined* to a large extent during the design and life cycle phases. However, this will largely depend on the understanding, experience and foresight of the designer in selecting and combining the most appropriate pavement design and life cycle strategy options.

Basically there are four major pavement composition types used in southern Africa, viz., granular, cemented, hot-mix asphalt and concrete base pavements. Each of these pavement types differs sufficiently in behaviour under different conditions to warrant different levels of maintenance and rehabilitation strategies (see Section 9.5), which should be evaluated against the service objective for the road. A brief description of the behaviour of each basic pavement type is given below. The design of surfacings, however, is given elsewhere (TRH3, 1996).

8.2.2 Granular base pavements

This type of pavement comprises a base of untreated gravel or crushed stone on a granular or cemented gravel subbase with a subgrade of various soils or gravels. The mode of distress usually exhibited by these pavements is largely controlled by the type of subbase.

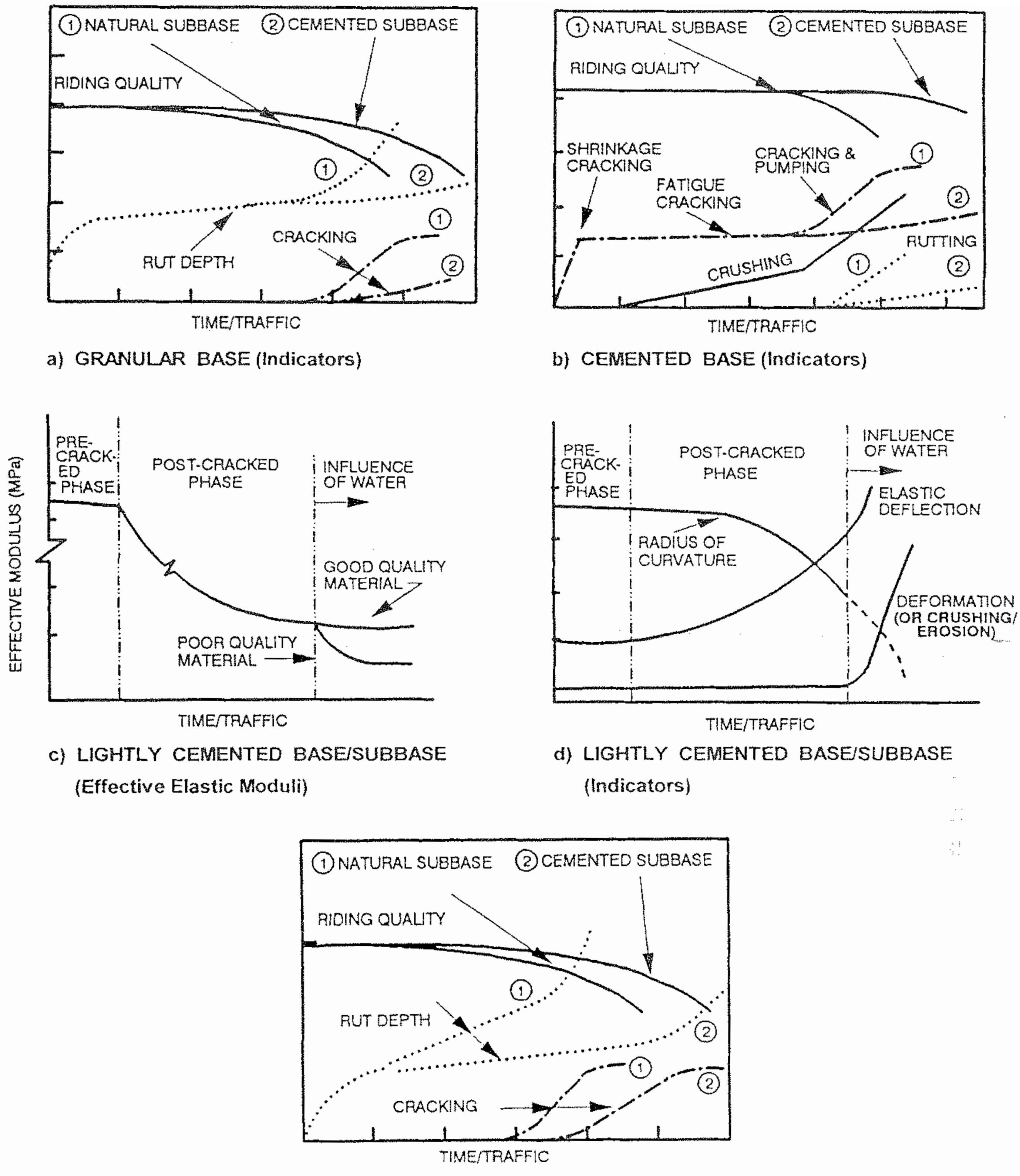
With a granular subbase it is usually permanent (plastic) deformation arising from densification and/or shear of the untreated material. This deformation may manifest itself either as rutting or as surface roughness, as illustrated by the indicators of behaviour given in Figure 6(a). It is important to ensure that these layers will have adequate bearing capacity under the operating conditions, especially the anticipated moisture regime. For this reason road authorities also tend to specify relatively high density standards (95 % to 98 % of mod. AASHTO) for upper pavement layers consisting of unbound materials. The recently developed Elasto-Plastic design method (Wolff, 1992; DOT, RR 92/312, 1993) may be used to evaluate the rutting potential of granular (unbound) layers in more detail.

Cemented subbase layers in granular base pavements generally increase the load carrying capacity of the pavement. Initially, cemented layers result in relatively high effective elastic moduli (stiffness) owing to cementation. Based on field measured deflections at various depths in these pavements, back-calculated linear elastic effective elastic moduli for pre-cracked cemented layers range between 1000 MPa and 1500 MPa (De Beer, 1985; Freeme et al., 1982). The failure mode of a cemented subbase is usually fatigue cracking and this results in a reduction of the effective elastic modulus as the layer progresses into the post-cracked state, as is illustrated in Figure 6(c). In this case an effective elastic moduli as low as 300 to 500 MPa can occur if the initial pavement structure is not well balanced and/or the original material is substandard. It is important to note that initial cracking, which causes a reduction in effective moduli, can be brought about by construction traffic. However, provided that the pavement structure is well designed and constructed, this initial reduction in effective moduli does not normally result in a marked increase in surface deformation, but changes in (resilient) deflection and radius of curvature as shown in Figure 6 (d) are generally observed.

The fatigue cracking in the cemented subbase may propagate until eventually the layer exhibits properties similar to those of a natural granular material, i.e. *an equivalent granular state*. It is unlikely that cracking will reflect to the surface and it is likely that there will be little rutting or longitudinal deformation until after the subbase has cracked extensively. However, if the subbase exhibits relatively large shrinkage or thermal cracks, these may reflect through to the surface.

In the mechanistic design method these different phases of a cemented layer have been designated as the pre-cracked and post-cracked phases. The designer accommodates these phases by re-evaluating the design with a lower subbase modulus (Theyse et al., 1995; TRH4; De Beer, 1985; Freeme et al., 1982; Walker et al., 1977).

Although the safety factor in the granular base may be reduced during the "breakdown" of the cemented subbase, it should still be within acceptable limits, depending on the strength balance of the structure. This, however, should be validated mechanistically.



a) GRANULAR BASE (Indicators)

b) CEMENTED BASE (Indicators)

c) LIGHTLY CEMENTED BASE/SUBBASE (Effective Elastic Moduli)

d) LIGHTLY CEMENTED BASE/SUBBASE (Indicators)

e) HOT - MIX ASPHALT BASE (Indicators)

FIGURE 6: GENERALISED PAVEMENT BEHAVIOUR CHARACTERISTICS AND INDICATORS

The eventual modulus of the cemented subbase will depend on the quality of the parent material originally stabilised, the cementing agent, the effectiveness of the mixing process, the density achieved and the degree of cracking. The ingress of moisture can significantly affect the modulus in the post-cracked phase. In some cases the layer may behave like a good quality granular material with a modulus in the order of 200 to 500 MPa, while in other cases the modulus will reduce to the order of 50 to 200 MPa. This change is also shown diagrammatically in Figure 6(c). The net result is that the modulus of the cemented subbase decreases to very low values and this causes fatigue and high shear stresses in the granular base, once again depending the structural balance of the pavement. Cracking of the surface will normally occur and, with the ingress of water, pumping from the subbase can also occur, which might lead to further pavement failure.

For high quality, heavily trafficked pavements it is necessary to avoid materials which will eventually deteriorate to a very low modulus. Many of these lower classes of materials have, however, proved to be adequate for lower class traffic. Erodibility and durability of cemented subbases should always be checked against existing subbase criteria (DOT, RR 91/167, 1993).

The surfacing of the pavement may also crack owing to ageing of the binder or to load-associated fatigue cracking. Granular materials are usually susceptible to water. Water ingress through surface cracks consequently results in a lowering of the bearing capacity (i.e. shear strength) of the layer and excessive deformation may occur. The water susceptibility depends on factors such as type of material, gradation, PI of the fines and density of the granular layer.

Timely preventative maintenance (periodic, routine and/or programmed) of the surfacing is critical for protecting and retaining the inherent strength of the pavement and its structural balance. In many cases asphalt patching or a thin surface treatment will be adequate, but where excessive roughness or rutting has developed on the pavement, a thin asphalt overlay (< 50 mm) may be required.

8.2.3 Cemented base

Cemented base pavements can either have uncemented or cemented subbases depending on the material quality and the life cycle strategy. Normally a cemented base pavement structure with an uncemented subbase will result in a relatively shallow structural balance, especially with moisture ingress into the subbase, and will therefore be more sensitive to overloading.

In these pavements, most of the traffic stresses are absorbed by the cemented layers and relatively little by the subgrade. It is likely that some block cracking will be evident very early in the life of the cemented layer. These cracks are caused by the mechanisms of drying shrinkage and thermal stresses and may reflect through the overlying asphaltic surfacing. Traffic-induced stresses can cause the blocks to break down into smaller ones at varying rates, depending on the degree of (over) loading. The reduction in modulus of a lightly cemented base/subbase is illustrated in Figure 6(c), and in Figure 6(d) some indicators of behaviour of this type of pavement are illustrated. When the cracks propagate through to the surface, the ingress of water may result in pumping and erosion of relatively fine material from the cemented layer as well as the underlying layer(s). This pumping is caused by the "rocking" movement of the cemented blocks under the action of traffic, resulting in an increasingly self-destructive process or moisture accelerated state of distress. This is usually quite evident after rainy weather from the accumulation of expelled fines around the surfaced cracks.

Rutting or surface roughness will generally be relatively low up to this stage, especially when the subbase is also cemented, but it is likely to increase as the extent of breakdown of the cemented blocks and pumping increases, as illustrated in Figure 6(b).

Pavements consisting of cemented bases on granular subbases are very sensitive to overloading and to the ingress of moisture through the cracks. The use of a cemented subbase in conjunction with a cemented base layer is favoured by some road authorities because of the ability of these pavements to inhibit active pumping (even though in terms of strength an uncemented subbase, may suffice). A cemented subbase also ensures a deeper pavement which is normally less sensitive to overloading.

When selecting a cemented base, it is necessary to make provision for the maintenance associated with shrinkage and thermal cracking as part of the life cycle strategy of this pavement type. The initial cracks may be rehabilitated by sealing, or by the application of a surfacing consisting of modified bitumen binders. Quantification of the crack activity (crack movement under traffic loading) may be used to determine a suitable asphaltic surfacing for these pavements (Rust et al,. 1992). Once traffic-load-associated cracking has become extensive, rehabilitation involves either reprocessing of the base or applying a substantial hot-mix asphalt or granular overlay.

Another failure mechanism of cemented bases, which can also lead to pumping and the formation of potholes, is "crushing failure" of the upper part of the base layer under repetitive stresses caused by relatively high tyre contact stresses (i.e. through high tyre inflation pressures and high tyre loading) (De Beer, 1990). When crushing leads to pumping, it usually manifests itself initially as a generalised faint "dusting" of the surfacing, after which potholes may occur as a result of traffic loading during wet periods. See also Figure 6(b).

Therefore an important consideration for thinly surfaced cemented (stabilised) base pavements is resistance to crushing failure and the design should be checked for this failure mode. This failure mechanism is usually associated with the deeper pavements with lightly cemented bases (De Beer, 1990).

Erodibility and durability of cemented materials should always be checked against existing criteria for such applications (DOT, RR 91/167, 1993).

8.2.4 Hot-mix asphalt base pavements

These pavements have an asphalt base layer generally more than 80 mm in thickness. In asphalt base pavements both deformation and fatigue cracking are possible. Two types of subbase are recommended, either an untreated granular subbase or a lightly stabilised cemented subbase. Rutting may originate in either the asphalt layer or the untreated layers or both. This is shown in Figure 6(e). If the subbase is cemented, there is a probability that shrinkage or thermal cracking will reflect to the surfacing, especially if the asphalt material is less than 150 mm thick or if the subbase is strongly cemented. If crushing failure occurs at the interface between the asphalt and the top of the cemented subbase, the horizontal tensile strain at the bottom of the asphalt base will increase, reducing the fatigue life of the asphalt base layer (De Beer, 1985; De Beer, 1992).

Although asphalt bases tends to be impermeable, certain types like dense bituminous macadam and continuously graded mixes are known to be more permeable than others. Protection should also be exerted in order to eliminate the presence of trapped-in water in the base which could result in the development of hydro-static pressure build-up. This pressure could result in stripping of the asphalt binder and premature failure of the base.

Programmed maintenance usually consists of a surface treatment to provide better skid resistance and to seal small cracks. An asphalt overlay is used where riding quality needs to be restored and when it is necessary to prolong the fatigue life of the base. Recycling of the asphalt base may also be done when further overlays are no longer adequate on the existing pavement.

The behaviour of Large Asphalt Mixes for Bases (LAMBS) (SABITA, manual 13, 1993) is similar to conventional asphalt bases but the structural properties are enhanced. LAMBS have a greater resistance to permanent deformation due to the large aggregates used and the provision of a stone skeleton (stone matrix or stone mastic asphalt (SMA)) structure.

The larger maximum aggregate used in LAMBS results in lower aggregate crushing costs and a reduced fines content, which also results in a reduction of the required binder content. However, a higher binder content may be used to increase pavement flexibility and therefore fatigue life.

For relatively thick asphalt base (>150 mm) pavements the Shell pavement design method (or any other suitable method) may be used to compare different mix designs (Shell, 1978).

It should however be noted that no LAMBS pavement designs are currently given in the Catalogue, as it is considered a relatively new technology.

8.2.5 Segmented concrete block pavements

Segmented concrete block pavement designs are becoming more popular in southern Africa, especially with the greater emphasis on labour-intensive pavement construction. In addition to its great potential for job creation, segmented concrete block pavers are suited for labour-intensive pavement construc tion.

Concrete block pavement designs and associated technology are discussed elsewhere (Draft UTG2, 1987; Phillips, 1994; UTG2, 1987; DBSA, Number 2 and Number 8, 1993, and BS 7533, 1992).

8.2.6 Concrete (rigid) pavements

Design and construction details of concrete pavements are beyond the scope of this document and the Pavement Design Catalogue. The design and construction of concrete (rigid) pavements are discussed elsewhere (DOT, manual 10, 1995).

8.3 FACTORS INFLUENCING PAVEMENT LAYER SELECTION

8.3.1 Traffic class, category and layer type

As part of the pavement design process, certain factors influencing the selection of the type of pavement need to be considered. Certain pavement types may, however, not be suitable for some road categories or traffic classes.

Table 19 shows recommended pavement types (base and subbase) for different road categories and traffic classes up to class ES30. Brief reasons why certain pavement types are ***not*** recommended are also given.

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.

Rigid pavement structures (Figure 6(b)) deteriorate more rapidly once distress is initiated (i.e. "sudden death" behaviour), whereas the more flexible pavements (Figure 6(a)) generally deteriorate slowly over time. Signs of distress are therefore more urgent on the more rigid pavements.

8.3.2 Condition at the 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 the previous section, certain kinds of distress can be expected in certain pavement types and account must be taken of such distress. Table 20 shows acceptable terminal conditions of rut depth and cracking for the various road categories and flexible pavement types.

8.3.3 Micro-climate and paved shoulders

From a geometrical requirement, the sealing of shoulders (i.e. paved shoulders) reduces the risk for accidents on the road. Structurally, it provides better support and moisture protection for the pavement layers and also reduces erosion of the shoulders (especially on steep gradients). On two lane roads paved shoulders further provide for heavy vehicles to give way to the left to create additional passing opportunities. Lastly, paved shoulders also reduce potential maintenance costs. Shoulders should be sealed if warranted by special conditions and if it is economically justifiable.

TABLE 19

Suggested flexible and semi-rigid pavement types for different road categories and traffic classes

Pavement types		Road category and traffic class								Brief reasons why listed pavement types are **not** recommended for the given road category and traffic class
		A		B			C and D			
Base	Subbase	ES100	ES3	ES10	ES3	ES1	ES3	ES1	<ES0.3	
Granular	Granular	X	✓**	✓**	✓*	✓	✓	✓	✓	Uncertain behaviour
	Cemented	✓*	✓	✓	✓	✓	✓	✓	✓	—
Hot-mix asphalt	Granular	✓	✓	✓	✓	X	✓	X	X	Cost effectiveness
	Cemented	✓	✓	✓	✓	X	✓	X	X	Cost effectiveness
Cemented	Granular	X	X	X	X	X	X	X	✓	Fatigue cracking, crushing, pumping and rocking blocks
	Cemented	X	X	✓	✓	✓	✓	✓	✓	Shrinkage cracks unacceptable

* Not recommended for wet regions without special provision for drainage.

** Only where experience has proved this to be adequate[60].

✓ = *Recommended* X = Not recommended

It is important that the layerworks of the pavement structure should be extended beyond the paved shoulder width (see Figure 9). Pavement layers consisting of water-susceptible materials are undesirable for wet regions, unless special provision is made for drainage or paved shoulders are provided. The seasonal variation in moisture content and variation in the associated CBR is shown in Figure 7. The edge of the pavement (i.e. zone where the maximum seasonal variation occurs) is of extreme importance to ultimate pavement performance with or without paved shoulders.

TABLE 20

*Possible condition at end of structural design period for the various road categories and flexible and semi-rigid pavement types**

Possible condition at end of structural design period	Road Category			
	A	B	C	D
Rut depth (mm)	20	20	20	20
Length of road exceeding stated rut depth (%)	5	10	20	50
Type of cracking/ distress:				
Granular base	Crocodile cracking, loss of surfacing, pumping of fines, deformation			
Hot-mix asphalt base	Crocodile cracking, loss of surfacing, pumping, deformation			
Cemented base	Block cracking, rocking blocks, crushing, pumping of fines, loss of surfacing, lack of durability/erodibility			

* Visual Condition Index (VCI) is also a very powerful method to describe the condition of the pavement (TRH22, 1994).

From Figure 8, the effect of shoulder width on the probability of moisture variation in the edge zone can be established. For a specified risk, the implications in terms of material quality, compaction and drainage should be compared with the additional cost of paved shoulders. Recommended paved shoulder widths for the different road categories are given in Table 21.

TABLE 21

Recommended minimum paved shoulder width for structural purposes (derived from Figure 8)*

Pavement category	Shoulder width (m)
A	1,2
B	1,0
C	0,8
D	0,3

* Actual shoulder width may increase in accordance with additional geometrical and safety considerations.

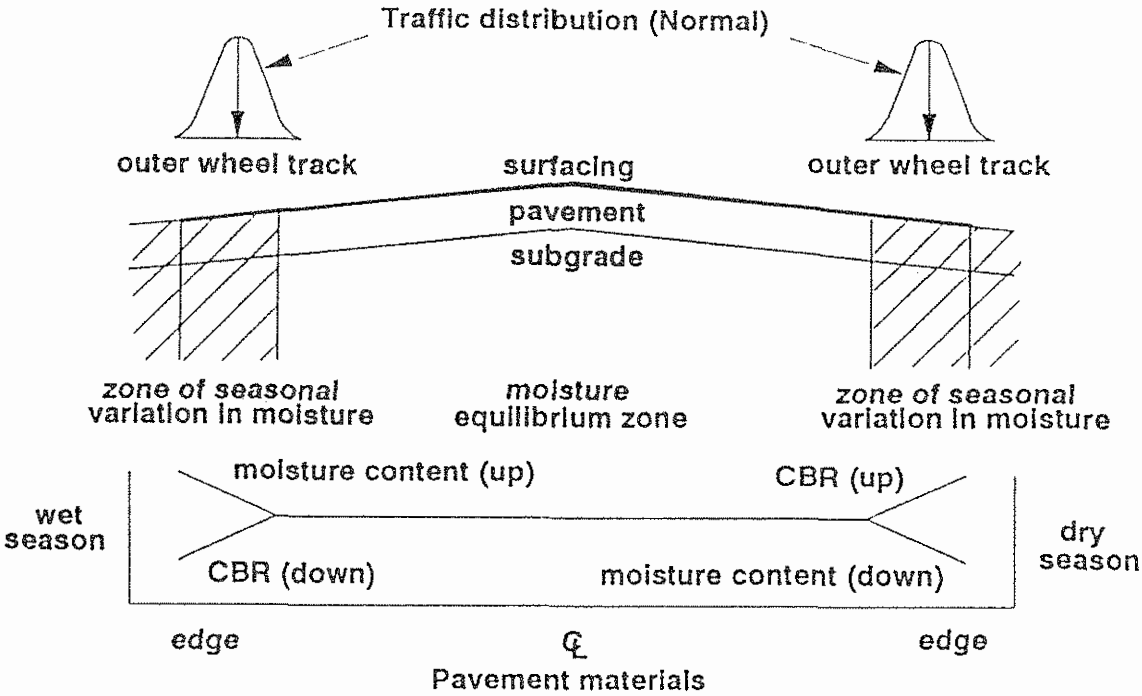


FIGURE 7: MOISTURE ZONES IN THE PAVEMENT (After Emery, 1992)

8.4 THE CATALOGUE DESIGN METHOD

8.4.1 Introduction to the Catalogue

Before the Catalogue is used, all the factors noted in Sections 1 to 6 should be considered. By making sure of the service objective, road category, design strategy, design equivalent traffic, materials availability and pavement type, the designer can choose a pavement structure. It should be noted that these pavement designs are considered to be of adequate capacity to carry the total equivalent design traffic (E80s or ESAs) over the structural design period.

Construction constraints on practical layer thicknesses and practical increments in thicknesses are *included* in the Pavement Design Catalogue given here. For these designs, it is also 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 ***can still act as a guide***. The Catalogue is not comprehensive in that designs other than those appearing innovative have been used with success. It is preferable that given designs be validated mechanistically using the SAMDM. (The appropriate pavement design transfer functions for the South African Mechanistic Design Method (SAMDM) are given elsewhere (Theyse et al., 1995).

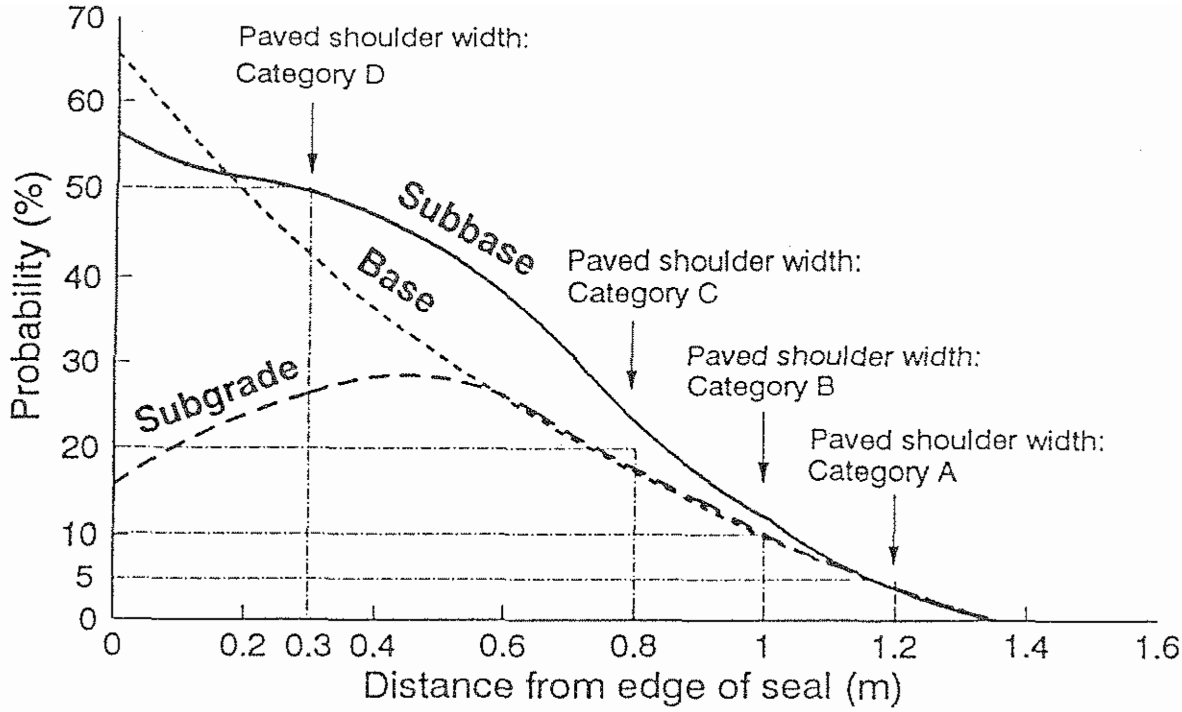


FIGURE 8: PROBABILITY DISTRIBUTION OF ZONE OF SEASONAL VARIATION AND RECOMMENDED PAVED SHOULDER WIDTH FOR THE DIFFERENT ROAD CATEGORIES (A, B, C & D)
(Modified after Emery, 1992)

8.4.2 Selected layers

Designs for Categories A, B and C in the Catalogue assume that all subgrades are brought to equal (G7) support standards, unless otherwise specified in the notes accompanying the catalogue designs. Section 6 limits the design CBR of the subgrade to four groups (Table 16). Normally, the in situ subgrade soil will be prepared through proof rolling or ripped and re-compacted 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 22 shows the preparation of the subgrade and required selected layers for the different subgrade design CBRs.

The catalogue designs for Category D are assumed to be supported by G9/G10 foundation. ***Layers shown in the Catalogue for Category D pavement designs with lower strength than the in situ subgrade may be omitted provided that adequate strength exists over the total pavement depth.***

TABLE 22

Preparation of subgrade/roadbed and required selected layers for the different subgrade design CBRs (Categories A, B, C and D)

Subgrade CBR Class	**SG4**	**SG3**	**SG2**	**SG1**
Design CBR of subgrade	**< 3**	**3 - 7**	**7 - 15**	**> 15**
Add selected layers: Upper Lower	Not applicable	150 mm G7 150 mm G9*	150 mm G7 —	— —
Treatment of in situ subgrade	Special treatment required	Rip and re-compact to 150 mm G10	Rip and re-compact to 150 mm G9	Rip and re-compact to 150 mm G7

* If the in situ subgrade is expected to be very wet, or in wet regions (Section 6), an additional 150 mm layer of G9 or a pioneer layer (CSRA, 1987) could be used.

8.4.3 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 class defined in Section 4 (Table 4). 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. This is possible because for many designs the only difference between the structures for the various classes of traffic is a change in the layer thickness. However, there is often a change in material quality as well as in layer thickness. Simple interpolation is then inadequate and the designer will have to use proper design methods (Theyse et al., 1995; Freeme et al., 1982; Walker et al., 1977; Yoder et al., 1975; Shell, 1978).

9. COST ANALYSIS

9.1 GENERAL

Alternative pavement designs should be compared on the basis of the present worth of the life cycle costs. *The cost analysis should be regarded as an aid to decision-making. It does not necessarily include all the factors leading to a decision and should therefore not override all other considerations.* The main economic factors which determine the cost of a facility are the analysis period, the structural design period, the construction cost, the maintenance costs and the real discount rate. Salvage value at the end of the analysis period and road user costs over the project life may also be considered.

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 of these differences on road user costs is not taken into account directly.

The choice of the analysis period and the structural design period will influence the cost of a road, but in Section 2 it is shown that this is not necessarily purely an economic decision.

9.2 PRESENT WORTH

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

The present worth of costs can be calculated as follows:

$$PWOC = C + (M_1(1+r)^{-x_1} + .. + Mj(1+r)^{-x_j}) - S(1+r)^{-z} \quad \dots\dots\dots \quad 9.1$$

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 (where x_j = 1 to z)
z = analysis period
S = salvage value of pavement at the end of the analysis period expressed in terms of present values.

PWOC is used to determine the ***relative*** cost difference between pavement structures. Items which, in some cases do not vary between different pavement structures, such as road user costs or salvage value, need not always be included in the comparison.

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 simple computer program can be designed for easy calculation of the present worth of costs.

9.3 CONSTRUCTION COSTS (C)

The checklist of unit costs given in Section 5 should be used to calculate the equivalent construction cost per square metre. Factors such as 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 should also be considered.

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. This is currently about 8 % (CEAS, 1989) in real terms (i.e. after compensating for the effect of inflation).

Unless the client clearly indicates that he prefers some other rate, 8 % is recommended for general use. It is highly recommended that a sensitivity analysis should be done to determine the importance of the value of the real 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 8 that there is a relationship between the type of pavement and the maintenance that might be required in the 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 reasonable comparison is made.

Figures 3 and 5 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 critically important to ensure adequate performance of these pavements. The lives of the various surfacing types will depend on the traffic and the type of base used. Table 23 gives guidance on the range of typical surfacing lives that can be expected from various surfacing types. These values may be used for a more detailed analysis of future maintenance costs.

TABLE 23

Suggested typical ranges of surfacing life periods (without rejuvenators) 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 (≤ 50 mm thickness)	Typical range of surfacing life		
		Road category and traffic*		
		A (ES3 - ES100)	B (ES1 - ES10)	C, D (ES0.003 - ES3)
Granular	Bitumen sand or slurry seal	—	—	2 - 8
	Bitumen single surface treatment	6 - 8	6 - 10	8 - 11
	Bitumen double surface treatment	6 - 10	6 - 12	8 - 13
	Cape seal	8 - 10	10 - 12	8 - 18
	Continuously graded asphalt premix	8 - 11		
	Gap-graded asphalt premix	8 - 13		
Hot- mix asphalt	Bitumen sand or slurry seal	—	—	2 - 8
	Bitumen single surface treatment	6 - 8	6 - 10	8 - 11
	Bitumen double surface treatment	6 - 10	6 - 12	8 - 13
	Cape seal	—	8 - 15	8 - 18
	Continuously graded asphalt premix	8 - 12	8 - 12	—
	Gap-graded asphalt premix	8 - 14	10 - 15	—
	Porous (drainage) asphalt premix	8 - 12	10 - 15	—
Cemented	Bitumen sand or slurry seal	**	—	—
	Bitumen single surface treatment	**	4 - 7	5 - 8
	Bitumen double surface treatment	**	5 - 8	5 - 9
	Cape seal	**	5 - 10	5 - 11
	Continuously graded asphalt premix	**	5 - 10	—
	Gap-graded asphalt premix	**	6 - 12	—

— Surface type not normally used.

* See Tables 1 and 4.

** Base type not used (refer to Section 8).

Typical maintenance measures that can be used for the purpose of cost analysis are given in Table 24. It should be noted that since the costs are discounted back 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 24.

There are two types of maintenance measures:

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

The structural design period (SDP) has been defined (Section 3) as the period during which it is predicted with a high degree of confidence that no structural maintenance will be required. Therefore, if typical 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. Table 24 makes provision for both moderate and severe distress.

The typical maintenance measures given in Table 24 should be replaced by more accurate values if specific knowledge about typical conditions is available.

9.6 SALVAGE VALUE (S)

The salvage value of the pavement at the end of the period under consideration is difficult to assess. Unless the analysis period varies considerably between pavement alternatives or there are exceptional circumstances such as land resale implications, the salvage value for cost comparison purposes should be taken as zero (DOT, RR 91/438, 1991).

9.7 ROAD USER COSTS

Road user costs are normally not considered in a total cost analysis, as the pavement designs are considered to provide "equivalent service" during the analysis period.

However, road user delay costs can be considered where they will make an impact on the comparative PWOC. The factors that determine the road user costs are:

(a) vehicle operating costs (fuel, tyres, vehicle maintenance and depreciation), depending mostly on the road alignment, but also on riding quality (PSI);

(b) accident costs, depending on road alignment, skid resistance and riding quality;

(c) delay costs, depending on the maintenance measures applied and the traffic situation on the road (this is a difficult factor to assess and may include aspects such as the provision of bypasses); and

(d) environmental and other non-tangible benefits such as noise reduction.

If the analysis warrants the calculation of road user costs, they can be estimated using data currently available (Schutte, 1984).

TABLE 24

Typical future maintenance for life cycle cost analysis

Base type	Typical maintenance measures*			
	Measures to improve the surfacing condition**		Structural maintenance	
	Original surfacing		Moderate distress	Severe distress
	Surface treatment	Asphalt		
Granular	S1 (9 yrs) S1 (18 yrs)	S1 (11 yrs) S1 (20 yrs) *or* AG (11 yrs) AG (22 yrs)	30 - 40 AG, AC	> 100 BS, BC *or* Granular overlay *or* Recycling
Hot-mix asphalt	S1 (10 yrs) S1 (20 yrs)	S1 (11 yrs) S1 (21 yrs) *or* AG (11 yrs) AG (22 yrs)	30 - 40 AG, AC	> 100 BS, BC *or* Recycling of base
Cemented	S1 (5 yrs) S1 (10 yrs) S1 (15 yrs) S1 (20 yrs)	S1 (5 yrs) S1 (10 yrs) S1 (15 yrs) S1 (20 yrs)	Further surface treatments	Thick granular overlay *or* Recycling of base

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

** Refer to previous Table 23 for typical surfacing lives.

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TRH9 see Construction of road embankments.

TRH10 see Site investigation and the design of road embankments.

TRH12 see Flexible pavement rehabilitation investigation and design.

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GLOSSARY OF TERMS (see Figure 9)

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) occurring immediately beneath the surfacing and over the subbase or, if there is no subbase, over the subgrade.

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

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 cumulative equivalent traffic on the heaviest trafficked lane predicted for the structural design period.

Distress - the visible manifestation of the deterioration of the pavement with respect to either the serviceability or the structural capacity.

Equivalent traffic - the number of equivalent 80 kN (standard) single axle loads (E80s or ESA's) which cause the same cumulative damage as the actual traffic spectrum.

Equivalent vehicle unit (e.v.u.) - the number of through-moving passenger cars to which a given vehicle is equivalent, 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.

Heavy vehicle - a vehicle with an axle load > 4 000 kg, usually with dual rear wheels.

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

Interurban road - a primary road between urban areas carrying from light to heavy traffic with a high level of service.

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.

Maximum legally permissible axle load - maximum axle load legally allowed on South African roads from 1996 (South Africa, 1996) (see below):

Type of axle	No of tyres per axle	Mass (kg)	Load per Axle (kN)*
Single axle (steering)	2 or 3	7 700 (7 700)	76
Single axle (non-steering)	2 or 3	8 000 (7 700)	78
Single axle	4 or more	9 000 (8 200)	88
Tandem axle	4 or more	18 000 (16 400)	88
Tridem axle	4 or more	24 000 (21 000)	78,3

( ) Previous legal load limits * $g = 9{,}8\ m/s^2$

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 material - a material the physical properties of which have been improved by the addition of a stabilising 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 smooth and comfortable or bumpy and thus unpleasant and perhaps dangerous.

Roadbed - The in situ material below selected layers and fill (see Figure 9)

Rural road - a surfaced secondary road serving small rural communities and carrying very light traffic with a relatively low level of service.

Selected layer - the lowest of the pavement layers, comprising controlled material, either in situ or imported (classification codes G7 to G10 - see 5.2).

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

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

Standard Axle (SA) - 80 kN single axle dual wheel configuration is the Standard Axle (SA) in South Africa. The tyre contact stress is 520 kPa. (The maximum legally permissible single axle load (4 or more tyres) is 88 kN).

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 during 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, e.g. overlay, smoothing course and surface treatment, partial reconstruction (say base and surfacing), etc.

Subbase - the layer(s) occurring beneath the base or concrete slab and over 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 - 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.

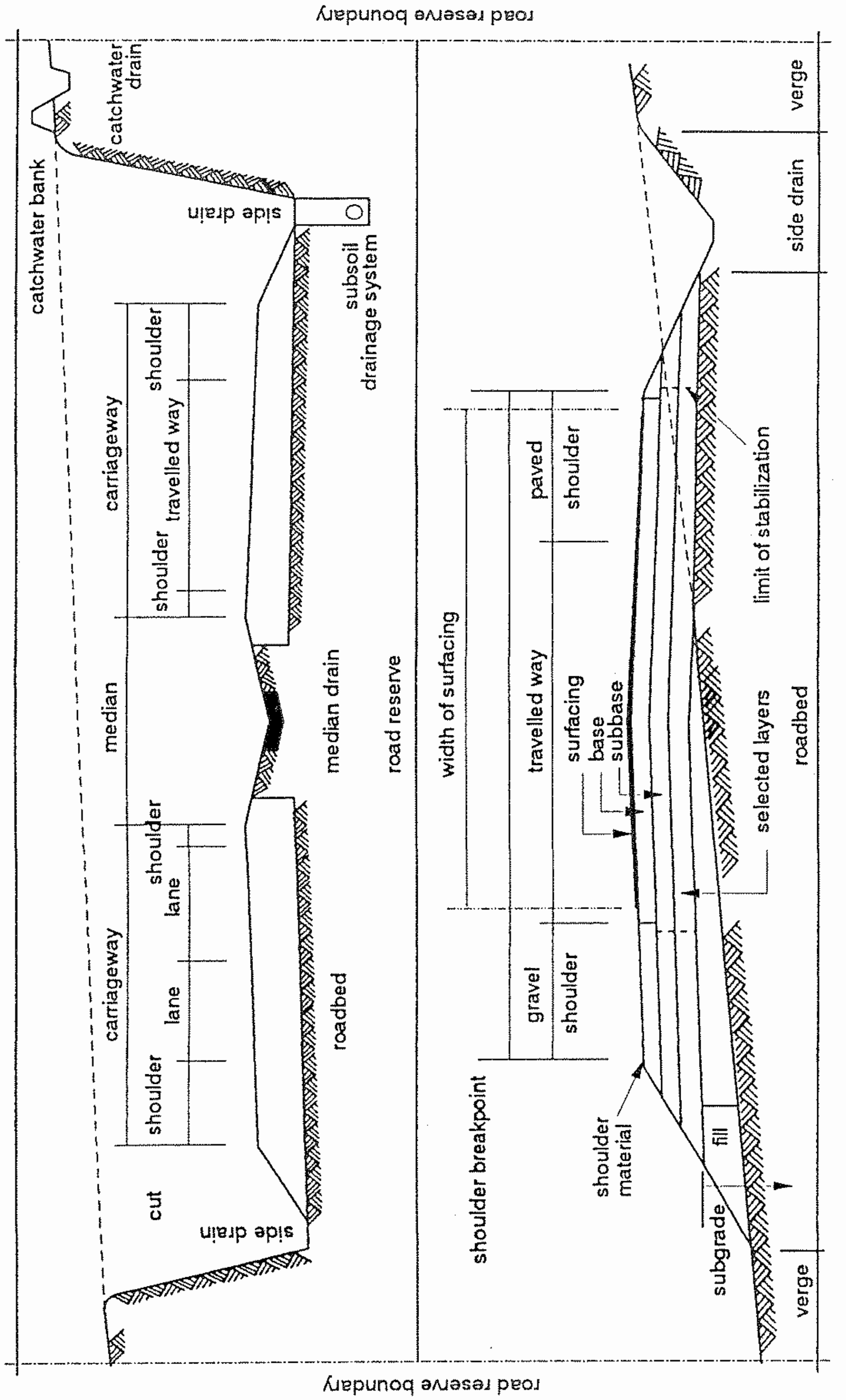


FIGURE 9: PAVEMENT STRUCTURE TERMINOLOGY

APPENDIX A

EXAMPLE OF THE STRUCTURAL DESIGN OF AN INTERURBAN ROAD PAVEMENT

A.1 MANAGERIAL/CLIENT INFORMATION ON THE NEW ROAD TO BE PROVIDED

After a need for a new road had been identified by provincial authorities, directives from the Management and Planning offices to the Design office (see Figure 1), or to the Consulting Engineer defined a service objective that a new 82 km interurban provincial collector road needs to be provided between an existing and a future urban area. The road is considered an important link with a relatively high level of functional service and the planning suggests that it will be a major two-lane road and that it should be opened to traffic in five years.

For the purposes of the example the following information is available:

> Expected annual average daily equivalent design traffic (AADE) in the area = 180 E80/lane/day. (If the traffic information is not readily available from the Planning office (or client body), then consult and follow the guidelines as set in Section 4 of this document):
>
> Moderate climatic region,
>
> Centre line subgrade CBR-values :
> 5;7;6;7;7;8;5;14;12;9;10;13;16;22;25;22;20;21;17;18;12;14;12;12;10;11;10;9;6;7;5;8;8;9

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

A.2 ROAD CATEGORY

A.2.1 Road category (Table 1)

After consultation with the client it is agreed that the road can be regarded as a Category B road. Therefore the cumulative design equivalent traffic should fall in the range of 0,3 x 10^6 to 10 x 10^6 E80/lane over the structural design period.The constructed riding quality, in PSI, should be between 3,0 and 4,0 with a terminal value of 2,0.

A.3 DESIGN STRATEGY

A.3.1 Select analysis period (Table 2)

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

A.3.2 Select structural design period (Table 3)

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

Therefore AP = 30 years; SDP = 15 years.

A.4 ESTIMATE DESIGN TRAFFIC

The current equivalent traffic (180 E80/lane/day) may be projected to the initial year using the growth factor, g_x, from Table 11. The cumulative equivalent traffic over the structural design period can be determined by multiplying the initial equivalent traffic by the cumulative growth factor, f_y, from Table 12. 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 % (higher values are unrealistic for this road). Table A1 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 ES3.

TABLE A1

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

Growth rate (%)	g_x (Table 11)	f_y (Table 12)	Cumulative equivalent traffic (E80s) over SDP	Design traffic class (Table 4)
2	1,10	6 440	$1{,}3 \times 10^6$	ES3
4	1,22	7 600	$1{,}7 \times 10^6$	ES3
6	1,34	9 010	$2{,}2 \times 10^6$	ES3
8	1,47	10 700	$2{,}8 \times 10^6$	ES3

A.5 MATERIALS

Table A2 may be compiled by filling in the checklist in Table 14. The unit prices listed are all 1995 prices. They were determined from the CSRA database of tendered rates (CSRA, 1995). These are average prices for use in this example only.

TABLE A2

Checklist of material availability and unit cost (Base year 1995)

SYMBOL	CODE	MATERIAL	AVAILABILITY	UNIT COST/m^3	
				BORROW PIT:	COMMERCIAL:
	G1	GRADED CRUSHED STONE	✓	R 86,00	R122,00
	G2	GRADED CRUSHED STONE	✓	R 86,00	R122,00
	G3	GRADED CRUSHED STONE	✓	R 78,00	R116,00
	G4	NATURAL GRAVEL	✓	R 42,59	R 57,64
	G5	NATURAL GRAVEL	✓	R 39,23	R 53,50
	G6	NATURAL GRAVEL	✓	R 35,31	R 50,36
	G7	GRAVEL/SOIL	✓	R 33,03	R 45,65
	G8	GRAVEL/SOIL	✓	R 32,31	R 44,87
	G9	GRAVEL/SOIL	✓	R 31,40	R 44,00
	G10	GRAVEL/SOIL	×	—	—
	C1	CEMENTED CRUSHED STONE OR GRAVEL	✓	R106,05	R145,00
	C2	CEMENTED CRUSHED STONE OR GRAVEL	×	R102,00	R140,00
	C3	CEMENTED NATURAL GRAVEL	✓	R 63,00	R 76,70
	C4	CEMENTED NATURAL GRAVEL	✓	R 61,70	R 71,70
	BEM	BITUMEN EMULSION MODIFIED GRAVEL	×	—	
	BES	BITUMEN EMULSION STABILISED GRAVEL	×	—	
	BC1	HOT - MIX ASPHALT	✓	—	
	BC2	HOT - MIX ASPHALT	✓	R320,00	
	BC3	HOT - MIX ASPHALT	✓	—	
	BS	HOT - MIX ASPHALT	✓	—	
	AG	ASPHALT SURFACING	✓	—	
	AC	ASPHALT SURFACING	✓	R350,00	
	AS	ASPHALT SURFACING	✓	R430,00	
	AO	ASPHALT SURFACING	×	—	
	AP	ASPHALT SURFACING	×	—	
	S1	SURFACE TREATMENT	✓	R 5,10/m^2	
	S2	SURFACE TREATMENT	✓	R 6,00/m^2	
	S3	SAND SEAL	×	R 2,35/m^2	
	S4	CAPE SEAL	×	R 6,90/m^2	
	S5	SLURRY	×	R561,00/m^3	
	S6	SLURRY	×	R590,00/m^3	
	S7	SLURRY	×	R616,00/m^3	
	S8	SURFACE RENEWAL (30%)	×	R 1,20/litre	
	S9	SURFACE RENEWAL (60%)	×	R 2,03/litre	
	WM1	WATERBOUND MACADAM	×	R150,00/m^3	
	WM2	WATERBOUND MACADAM	×	R150,00/m^3	
	PM	PENETRATION MACADAM	×	—	
	DR	DUMPROCK	×	R 30,00/m^3	

A.6 ENVIRONMENT

A.6.1 Climatic region

The road lies within a moderate climatic region.

A.6.2 Delineation of subgrade areas and design CBR of subgrade

The given CBR values are illustrated in Figure A1(a). By visual inspection of the CBR values, five subgrade areas can be delineated.

Subgrade Area 1 : CBR = 5;7;6;7;7;8;5
Subgrade Area 2 : CBR = 14;12;9;10;13;16
Subgrade Area 3 : CBR = 22;25;22;20;21;17;18
Subgrade Area 4 : CBR = 12;14;12;12;10;11;10;9
Subgrade Area 5 : CBR = 6;7;5;8;8;9

NOTE: An alternative method for delineation is given by the *1993 AASHTO Design* Guide (Appendix J), (AASHTO, 1993), which is based on cumulative differences.

In Table A3 the CBR percentile values and design CBR of the subgrade are given. Since the design is for a Category B road, the 10 percentile "lower than" CBR values should be used in the determination of the Design CBR of the subgrade. Figure A1(b) also illustrates the cumulative "smaller than" distribution histogram of the five subgrade areas indicated in Figure A1(a).

TABLE A3

Design CBR of the subgrade in example

Subgrade area	Percentile values*		CBR Class	Design CBR of subgrade
	50%	10%		
1	6,2	4,4	SG3	3 - 7
2	12,0	8,8	SG2	7 - 15
3	20,5	16,7	SG1	> 15
4	11,0	8,6	SG2	7 - 15
5	7,0	4,6	SG3	3 - 7

* Category B roads: 90% reliability (see Table 1).
Therefore the 10 percentile "less than" CBR value is selected from each subgrade area (see also Figure A1 (a) and (b)).

A.7 STRUCTURAL DESIGN AND PAVEMENT TYPE SELECTION

A.7.1 Pavement type selection

From Section 8.4 it follows 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. From Table 19 it follows that cemented subbases are recommended for all base types, although granular subbases can also be used for hot-mix asphalt and granular bases.

A.7.2 Selected layers

The selected layers necessary for the different subgrade areas are given in Table A4 (according to Table 22).

TABLE A4

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

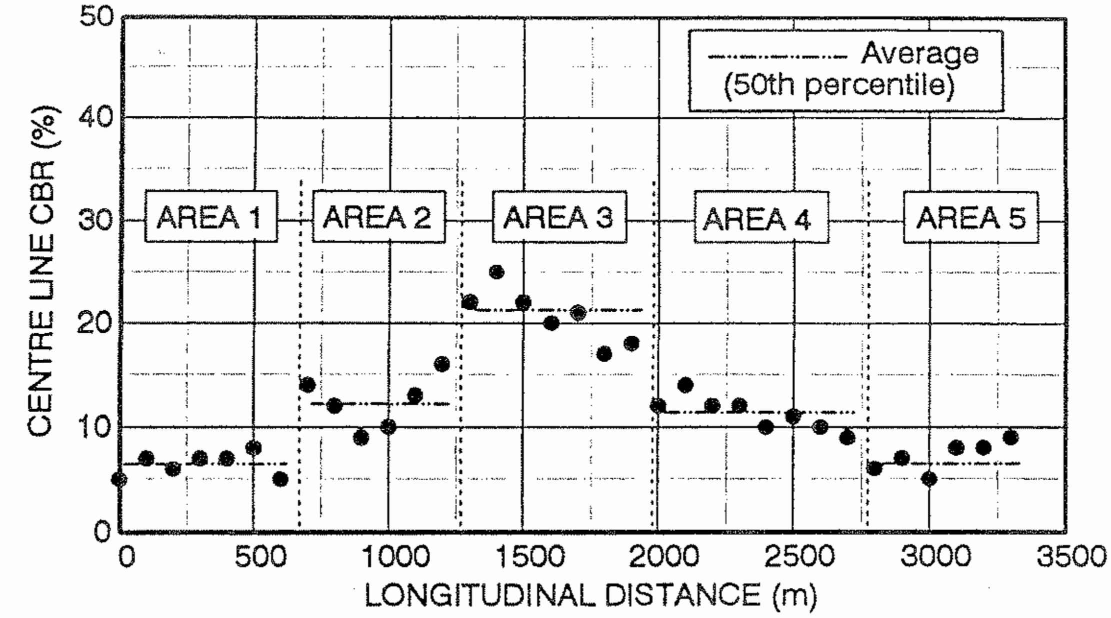
* R + R = Rip and Recompact.

A.7.3 Possible pavement structures

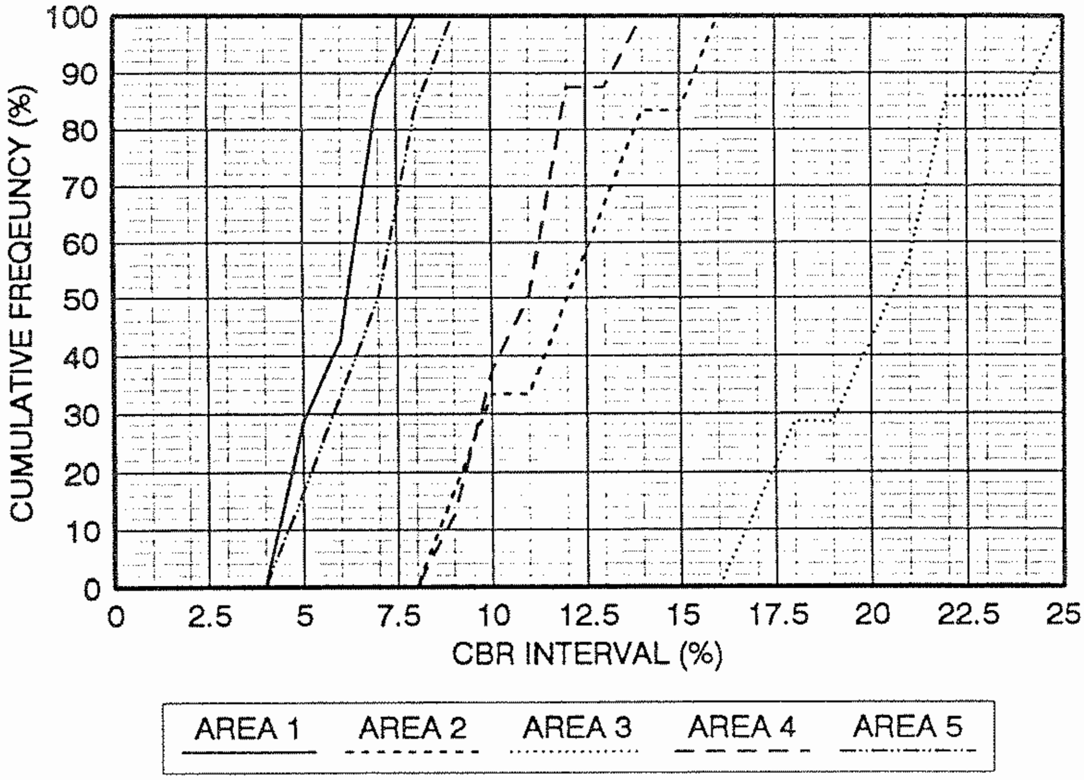
Table A5 shows possible pavement structures according to the Catalogue (Category B road, class ES3 traffic).

A.8 PRACTICAL CONSIDERATIONS

The designer should consider the consequences, if any, of the practical considerations given in Section 7 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. For this road category, the shoulders would be surfaced.



(a) In situ centre line CBR values



(b) Cumulative "less than" distribution histogram of CBR values

FIGURE A1: CENTRE LINE SUBGRADE CBR VALUES OF EXAMPLE

A.9 COST ANALYSIS

A.9.1 Construction cost

For the comparison of different pavement types, the construction cost of only the subbase, base and surfacing need be considered. These costs follow directly from the unit costs given in Table A2.

A.9.2 Future maintenance and life cycle strategy

The structural design period is 15 years and the analysis period is 30 years. The future maintenance for each pavement type can be estimated from Tables 23 and 24. 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. Table A6 shows estimated maintenance measures for the different pavement types of their life cycles.

TABLE A5

Possible B-category pavement structures (SDP = 15 yrs)

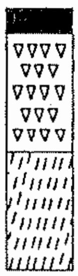
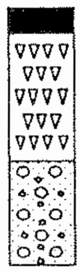
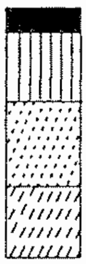
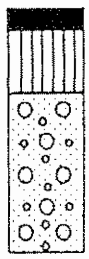
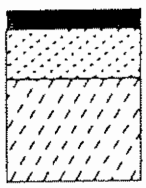
BASE	STRUCTURE
GRANULAR	ES3  S or 30A 150 G3 150 C4  S or 30A 150 G3 150 G5
HOT - MIX ASPHALT	ES3  30 AG 90 BC/BS 100 C4 100 C4  30 AG 120 BC/BS 150 G4
CEMENTED	ES3  S2 125 C3 200 C4

TABLE A6

Typical maintenance measures for the different flexible and semi-rigid pavement types over their life cycles

Pavement type		Maintenance measures	
Base	Subbase	For surfacing	Structural maintenance
Granular	Granular	30 AG (11 yrs)	40 AG (20 yrs)
	Cemented	30 AG (11 yrs)	35 AG (21 yrs)
Hot-mix asphalt	Granular	30 AG (12 yrs)	35 AG (21 yrs)
	Cemented	30 AG (12 yrs)	30 AG (21 yrs)
Cemented	Cemented	S1 (5 yrs)	S1 (20 yrs)
		S1 (10 yrs)	150 G1 + S2 (25 yrs)
		S1 (15 yrs)	

A.9.3 Discount rate

For the purposes of this example, a discount rate of 8 % is normally selected. However, it is better to do a sensitivity analysis with discount rates of 6, 8 and 10 %.

A.9.4 Salvage value and road user costs

Although the salvage value and road user costs may vary with pavement life, it is unlikely that the relative effect will influence the present worth of costs significantly. This can be confirmed with trial values.

A.9.5 Present worth of costs

The present worth of costs can be calculated from :

$$PWOC = C + (M_1(1+r)^{-A1} + .. + Mi(1+r)^{-Ai}) - S(1+r)^{-Z}$$

Table A7 shows the 1995 present worth of costs for each pavement type taking the construction cost and maintenance strategy from Table A6 into account. The sensitivity analysis shows that the discount rate can vary from 6 to 10 % without having a significant influence on the present worth of costs.

It also follows from Table A7 that the pavements with granular or cemented bases are significantly less costly than the others. Therefore, a pavement with a granular or ce-

TABLE A7

Present worth of costs (Base year 1995)

Pavement structure	Initial costs/m²	Maintenance	Initial costs/m²	Discounted maintenance costs/m²			Present worth of costs/m²		
				Discount rate			Discount rate		
				6 %	8 %	10 %	6 %	8 %	10 %
30 AG 150 G3 150 G5	10,50 11,70 5,88	30 AG (11 yrs) 40 AG (20 yrs)	10,50 14,00	5,53 4,37	4,50 3,00	3,68 2,08			
	28,08			**9,90**	**7,50**	**5,76**	**37,98**	**35,59**	**33,85**
30 AG 150 G3 150 C4	10,50 10,70 9,26	30 AG (11 yrs) 35 AG (21 yrs)	10,50 12,25	5,53 3,60	4,50 2,43	3,68 1,66			
	31,46			**9,13**	**6,94**	**5,34**	**40,59**	**38,39**	**36,79**
S2 150 G3 150 C4	6,00 11,70 9,26	S1 (10 yrs) 40 AG (19 yrs)	5,10 14,00	2,85 4,63	2,36 3,24	1,97 2,29			
	26,96			**7,47**	**5,61**	**4,26**	**34,43**	**32,56**	**31,21**
30 AG 120 BC2 150 G5	10,50 38,40 5,88	30 AG (12 yrs) 35 AG (21 yrs)	10,50 12,25	5,22 3,60	4,17 2,43	3,35 1,66			
	54,78			**8,82**	**6,60**	**5,01**	**63,61**	**61,39**	**59,79**
30 AG 90 BC2 150 C4	10,50 28,80 12,34	30 AG (12 yrs) 30 AG (21 yrs)	10,50 10,50	5,22 3,09	4,17 2,09	3,35 1,42			
	47,14			**8,31**	**6,26**	**4,76**	**59,95**	**57,90**	**56,40**
S2 125 C3 200 C4	6,00 7,88 12,34	S1 (5 yrs) S1 (10 yrs) S1 (15 yrs) S1 (20 yrs) S2 (25 yrs) 150 G1 base (25 yrs)	5,10 5,10 5,10 5,10 6,00 12,90	3,81 2,85 2,13 1,59 1,40 3,01	3,47 2,36 1,61 1,09 0,88 1,88	3,17 1,97 1,22 0,76 0,55 1,19			
	26,22			**14,78**	**11,29**	**8,86**	**41,00**	**37,51**	**35,07**

mented base will be selected. Considering the behaviour of the different pavement types given in Section 8, it is proposed that a structure with a granular base and a cemented subbase should be used. Such a pavement will probably not show the cracking and crushing problems of a pavement with a cemented base.

The selected pavement could also have a double surface treatment instead of an asphalt surfacing. This would probably result in a slightly lower riding quality. Maintenance measures could consist of a single surface treatment (S1) at 10 years, as well as 40 mm of asphalt surfacing (AG) at 19 years. The present worth of costs at a discount rate of 8 % will be R32,56/m^2, which is less than the calculated R38,39/m^2 for a similar pavement with an asphalt surfacing. The cost saving should be weighed against the lower riding quality, but the current policies of the Roads Authority may also influence the decision.

A.10 ALTERNATIVE STRATEGY

The overlay necessary for the cemented base pavement 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 ES10. Table A8 shows a suitable pavement structure for this traffic class. The future maintenance will only include surface treatments and no structural maintenance will be required over the design period. Table A9 shows the present worth of costs for the alternative strategy. The costs are slightly higher than the cost of the original strategy and the granular base pavement remains the most cost-effective structure.

TABLE A8

Pavement structure required for a longer structural design period

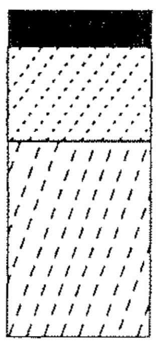
BASE	STRUCTURE
CEMENTED	ES10  S2 150 C3 300 C4

TABLE A9

Present worth of costs for a longer structural design period (alternative strategy) (Base year 1995)

Pavement structure	Initial costs/m²	Maintenance	Initial costs/m²	Discounted maintenance costs/m²			Present worth of costs/m²		
				Discount rate			Discount rate		
				6 %	8 %	10 %	6 %	8 %	10 %
S2	5,10	S1 (5 yrs)	5,10	3,81	3,47	3,17			
150 C3	9,45	S1 (10 yrs)	5,10	2,85	2,36	1,97			
300 C4	18,51	S1 (15 yrs)	5,10	2,13	1,61	1,22			
		S1 (20 yrs)	5,10	1,59	1,09	0,76			
		S2 (25 yrs)	5,10	1,19	0,74	0,47			
	33,06			**11,57**	**9,28**	**7,58**	**44,63**	**42,34**	**40,64**

CATALOGUE

DESCRIPTION OF THE CATALOGUE

The Catalogue includes most of the factors that have to be considered by the designer. Firstly, there is the service objective (Section 1), 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 or three traffic classes. A variety of pavement types (Section 8) are available, although the availability and cost of materials, and also experience with those 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 8). The Catalogue does not include other practical considerations such as drainage, compaction, shoulder design or pavement cross-section. These aspects should still be considered and are covered by Section 7.

THE USE OF THE CATALOGUE AND SPECIAL CONDITIONS

The Catalogue should not be used without considering the behaviour of the different pavement types, the 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 designs are conservative as they are designed to carry the upper traffic class with the stated approximated design reliability (see Table 1). To determine the reliability for a specific design it would be necessary to check the design with the appropriate transfer functions (Theyse et al., 1995).

The Catalogue caters for conditions normally encountered in road building. If special or unique conditions exist, the Catalogue can be used as a ***guide*** or first approximation, but need not reflect the final design.

CATALOGUE CONTENTS

GRANULAR BASES (MODERATE OR DRY REGIONS)

DATE 1996

ROAD CAT.	ES0.003 < 3000	ES0.01 $0,3\text{-}1,0\text{x}10^4$	ES0.03 $1,0\text{-}3,0\text{x}10^4$	ES0.1 $3,0\text{-}10\text{x}10^4$	ES0.3 $0,1\text{-}0,3\text{x}10^6$	ES1 $0,3\text{-}1,0\text{x}10^6$	ES3 $1,0\text{-}3,0\text{x}10^6$	ES10 $3,0\text{-}10\text{x}10^6$	ES30 $10\text{-}30\text{x}10^6$	ES100 $30\text{-}100\text{x}10^6$	Foundation
	PAVEMENT CLASS AND DESIGN BEARING CAPACITY (80 kN AXLES/LANE)										
A							40A 125 G2 150 C3 40A 150 G2 150 G5	40A 150 G2 250 C3	50A 150 G1 250 C3	50A 150 G1 300 C3	
B						S 125 G4 150 C4 S 150 G4 150 G5	S*/30A 150 G3 150 C4 S*/30A 150 G3 150 G5	40A 150 G2 200 C4 30A 150 G2 200 G5			150 G7 150 G9 G10
C				S 100 G5 125 C4 S 125 G4 125 G6	S 125 G5 125 C4 S 125 G4 150 G6	S 125 G4 125 C4 S 125 G4 150 G5	S 150 G3 150 C4 S 150 G3 150 G5				
D	S1 100 G5 100 G7	S1 100 G5 125 G7	S1 100 G4 125 G7	S1 100 G4 125 G6 S1 100 G5 100 C4	S 125 G4 125 G6 S 100 G5 125 C4	S 125 G4 150 G6 S 125 G5 150 C4					150 G9 G10

Symbol A denotes AG, AC, OR AS. A0, AP may be recommended as a surfacing measure for improved skid resistance when wet or to reduce water spray.
S denotes Double Surface Treatment (seal or combinations of seal and slurry)
S1 denotes Single Surface Treatment
* If seal is used, increase C4 and G5 subbase thickness to 200mm.

GRANULAR BASES (WET REGIONS)

DATE 1996

ROAD CAT.	PAVEMENT CLASS AND DESIGN BEARING CAPACITY (80 kN AXLES/LANE) ES0.003 < 3000	ES0.01 $0,3\text{-}1,0\times10^4$	ES0.03 $1,0\text{-}3,0\times10^4$	ES0.1 $3,0\text{-}10\times10^4$	ES0.3 $0,1\text{-}0,3\times10^6$	ES1 $0,3\text{-}1,0\times10^6$	ES3 $1,0\text{-}3,0\times10^6$	ES10 $3,0\text{-}10\times10^6$	ES30 $10\text{-}30\times10^6$	ES100 $30\text{-}100\times10^6$	Foundation
A							30A 150 G1** 200 C3	40A 150 G1 300 C3 *(250 C3)	50A 150 G1 400 C3 *(300 C3)		150 G7 150 G9 G10
B						S 150 G2 150 C4 S 150 G2 200 G5	S/30A 150 G1** 200 C4	40A 150 G1 300 C4 *(250 C4)			
C				S 100 G5 125 C4 S 125 G4 125 G6	S 125 G5 125 C4 S 150 G4 150 G6	S 125 G2 150 C4 S 150 G2 150 G5	S 150 G2** 200 C4 S 150 G2 150 G4				
D	S1 100 G5 100 G7	S1 100 G5 125 G7	S1 100 G4 125 G7	S1 100 G4 125 G6 S1 100 G5 100 C4	S 125 G4 125 G6 S 100 G5 125 C4	S 150 G4 150 G6 S 125 G5 150 C4					150 G9 G10

Symbol A denotes AG, AC, OR AS. A0, AP may be recommended as a surfacing measure for improved skid resistance when wet or to reduce water spray.

S denotes Double Surface Treatment (seal or combinations of seal and slurry)

S1 denotes Single Surface Treatment

* If water is prevented from entering the base, the subbase thickness may be reduced to the values indicated in brackets.

** Base thickness may be reduced by 25 mm if cemented subbase thickness is increased by 50 mm.

HOT-MIX ASPHALT BASES

DATE 1996

ROAD CAT.	PAVEMENT CLASS AND DESIGN BEARING CAPACITY (80 kN AXLES/LANE)										Foundation
	ES0.003 < 3000	ES0.01 $0{,}3\text{-}1{,}0\times10^4$	ES0.03 $1{,}0\text{-}3{,}0\times10^4$	ES0.1 $3{,}0\text{-}10\times10^4$	ES0.3 $0{,}1\text{-}0{,}3\times10^6$	ES1 $0{,}3\text{-}1{,}0\times10^6$	ES3 $1{,}0\text{-}3{,}0\times10^6$	ES10 $3{,}0\text{-}10\times10^6$	ES30 $10\text{-}30\times10^6$	ES100 $30\text{-}100\times10^6$	
A							40A 80 BC 250 C3	40A 90 BC 300 C3	40A 120 BC 400 C3	50A 180 BC 450 C3	
B							30A 80 BC 200 C4	30A 80 BC 300 C3			150 G7 150 G9 G10
C											
D											

Symbol A denotes AG, AC, OR AS. A0, AP may be recommended as a surfacing measure for improved skid resistance when wet or to reduce water spray.
Symbol BC does not include LAMBS (BC1 Table 13)
S denotes Double Surface Treatment (seal or combinations of seal and slurry)
S1 denotes Single Surface Treatment

CEMENTED BASES

DATE 1996

ROAD CAT.	PAVEMENT CLASS AND DESIGN BEARING CAPACITY (80 kN AXLES/LANE)										Foundation
	ES0.003 < 3000	ES0.01 $0,3\text{-}1,0 \times 10^4$	ES0.03 $1,0\text{-}3,0 \times 10^4$	ES0.1 $3,0\text{-}10 \times 10^4$	ES0.3 $0,1\text{-}0,3 \times 10^6$	ES1 $0,3\text{-}1,0 \times 10^6$	ES3 $1,0\text{-}3,0 \times 10^6$	ES10 $3,0\text{-}10 \times 10^6$	ES30 $10\text{-}30 \times 10^6$	ES100 $30\text{-}100 \times 10^6$	
A							30A 150 C3 200 C4				150 G7 150 G9 G10
B						S 125 C3 150 C4	S 125 C3 200 C4	S/30A 150 C3* 300 C4			
C				S 100 C4 100 G6	S 200 C3 S 125 C4 125 G6	S 125 C3 125 C4	S 150 C3 150 C4				
D	S1 100 C4 100 G8	S1 100 C4 125 G8	S1 125 C4 125 G7	S1 150 C4 150 G7	S 125 C4 125 G6	S 125 C4 150 G6					150 G9 G10

Symbol A denotes AG, AC, OR AS. A0, AP may be recommended as a surfacing measure for improved skid resistance when wet or to reduce water spray.

S denotes Double Surface Treatment (seal or combinations of seal and slurry)

S1 denotes Single Surface Treatment

* Crushing of cemented base may occur

WATERBOUND MACADAM BASES

DATE 1996

ROAD CAT.	ES0.003 < 3000	ES0.01 $0,3\text{-}1,0\times10^{4}$	ES0.03 $1,0\text{-}3,0\times10^{4}$	ES0.1 $3,0\text{-}10\times10^{4}$	ES0.3 $0,1\text{-}0,3\times10^{6}$	ES1 $0,3\text{-}1,0\times10^{6}$	ES3 $1,0\text{-}3,0\times10^{6}$	ES10 $3,0\text{-}10\times10^{6}$	ES30 $10\text{-}30\times10^{6}$	ES100 $30\text{-}100\times10^{6}$	Foundation
	PAVEMENT CLASS AND DESIGN BEARING CAPACITY 80kN/LANE										
A#							30 - 40A* 125 WM1 150 C3	40A* 150 WM1 125 C3 125 C4	50A* 150 WM1 150 C3 150 C3		
B					S* 100 WM2 150 G5	S* 125 WM2 150 G5	S or 30A* 125 WM2 150 C4	40A* 125 WM1 125 C4 125 C4			150 G7 150 G9 G10
C				S* 100 WM2 100 C4 S* 100 WM2 125 G5	S* 100 WM2 125 C4	S* 100 WM2 150 G5	S* 125 WM2 100 C4 S* 125 WM2 150 G5				
D											

* SYMBOL A DENOTES AG, AC, OR AS. SYMBOL S DENOTES S2 OR S4, SEE TABLE 13 FOR MATERIAL SYMBOLS PM = PENETRATION MACADAM.
FOR SELECTED LAYERS REFER TO PARAGRAPH 8.4.2: FOR FUTURE MAINTENANCE TO PARAGRAPH 9.5.
A0, AP PERMITTED AS A SURFACING MEASURE FOR SKID RESISTANCE OR REDUCTION OF WATER SPRAYING.

FOR CATEGORY A PAVEMENTS CONSTRUCTED WITHOUT PAVERS, AN ASPHALT LEVELING COURSE OF 25mm to 30mm IS NORMALLY NEEDED TO IMPROVE RIDING QUALITY.