

SATCC

Draft

Code of Practice for the Rehabilitation of Road Pavements

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PREFACE AND ACKNOWLEDGEMENT

This document is based closely on the South African guide to pavement rehabilitation, Draft TRH12: Pavement rehabilitation investigation and design, 1997. A number of detailed changes of varying significance have been introduced in order to make the guide more appropriate to the SADC region, but the basic format and content follows that of TRH12.

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1. INTRODUCTION

1.1 Background

An increase in economic activity in southern Africa during the past few decades has led to major expansion and improvement of the road network throughout the region. A wide variety of pavement types have been used for these roads, ranging from sand-sealed gravel roads through waterbound macadam pavements and well-designed natural and stabilized gravel pavements, to sophisticated crushed stone and other composite pavements for the more heavily trafficked routes.

After many years of good service, an increasing number of these roads are reaching a condition which warrants further attention and improvements in terms of riding quality and strengthening of the pavement structure. To protect the integrity of these existing roads, a shift in emphasis has taken place from the design and construction of new roads to the rehabilitation design and reconstruction of existing roads.

Experience has shown that if a deficient pavement is rehabilitated timeously, the costs involved often amount to a mere fraction of the cost of reconstruction.

This is achieved by using the remaining structural strength and behaviour of the existing pavement in the rehabilitation design for the road. Hence, rehabilitation design should include procedures that allow for the quantification and evaluation of the behaviour of the existing pavement structure. This information can then be incorporated into the rehabilitation design.

1.2 Scope

Pavement rehabilitation involves measures used to restore, improve, strengthen or salvage existing deficient pavements so that these may continue, with routine and periodic maintenance, to carry traffic with adequate speed, safety and comfort. Pavement rehabilitation is a part of road rehabilitation which also includes the rehabilitation of additional aspects such as geometrics, safety, etc.

The main categories of pavement rehabilitation are:

- Complete pavement reconstruction.
- Partial reconstruction involving the strengthening of existing pavement layers, with or without stabilization, before resurfacing.
- Asphaltic or granular overlays.
- Provision of drainage, and/or improvements to existing drainage facilities.

Any combination of the above mentioned rehabilitation activities could be applicable to a specific pavement. Furthermore, several available options exist within each of the main rehabilitation categories from which a selection could be made. With all these alternatives available, the project level rehabilitation investigation procedure must identify the most suitable option from both a structural and economical point of view.

The project level rehabilitation investigation procedure should aim to fully utilize all available information on the existing pavements. This includes information on the design of the pavement, materials used, previous maintenance on the road, history of traffic loading and information available from the pavement management system of the Road Authority.

The purpose of this document is to provide guidance for project level rehabilitation investigations of flexible pavements. In this document only surfaced pavements are considered. Although design methods are recommended, no details thereof are included. Full references to applicable rehabilitation design documents are included in Section 3 (Rehabilitation design approach and options) of this document. Main characteristics of some pavement rehabilitation design methods in use in southern Africa are also included in Appendix B.

1.3 Managing Pavement Rehabilitation Design

1.3.1 General

A pavement is usually identified for rehabilitation by the various road authorities on the basis of information received from their regional offices as part of a pavement management system (PMS). Usually this information is related to the general visual condition and riding quality of a pavement. Depending on various factors such as the type of pavement structure and the severity of the problem, several rehabilitation options may be applicable.

Since not all roads within a network can be expected to be at the same level of deterioration, or even deteriorate at the same rate, the management of rehabilitation is usually divided into network and project level activities. Only project level activities are covered in this document. Monitoring the condition of the pavement at network level is assumed to take place within a PMS, and would be aimed at identifying specific roads that may require structural rehabilitation.

1.3.2 Project Level Investigations: Design Considerations

A wrong decision on the required rehabilitation for a project could have substantial economic consequences. Therefore, more detailed pavement condition tests are usually warranted in a project level study to improve confidence in the design and selection of an appropriate rehabilitation option. Many of the design variables which have to be assumed or estimated for a new pavement can be determined with reasonable accuracy on existing pavements. These include traffic conditions and the in situ strength parameters of the pavement components. The aim of rehabilitation is to modify the behaviour of a pavement in order to carry the design traffic loading at an acceptable level of service. The more fully this behaviour is evaluated, the more accurate and therefore the more economical the rehabilitation should be.

All relevant factors which could contribute to distress must be considered during the evaluation of the pavement. These include:

- Traffic loading.
- Drainage problems.
- Non-traffic induced cracks.
- The action of pumping under traffic.
- Inadequate in-situ properties of pavement materials.
- Expansive subgrades.

Although in some formal procedures these factors may be explicitly or tacitly recognized, they are frequently not satisfactorily incorporated by the practising engineer. It is often difficult to identify whether such factors have contributed to the cause and mechanism of distress, or to determine the way in which they should be dealt with in the rehabilitation design.

To determine the best rehabilitation alternative it is essential to recognize that the future behaviour of a pavement cannot be predicted with certainty due to the variability of pavement materials and inadequate knowledge of their properties. Such uncertainty can be countered to some extent through the use of extensive testing. However, it is necessary to be **very selective about the types and number of tests to keep the investigation within acceptable logistic and economic limits**. The designer achieves this by progressively determining the contribution of additional testing until the degree of confidence necessary to determine the appropriate rehabilitation options has been obtained.

The ability of models based on pavement condition tests to predict the behaviour of pavements is very limited for reasons such as:

- The varied nature of materials.
- Differences between specified values and those actually achieved.
- The simplistic nature of models in the face of many factors affecting the behaviour of materials in pavements.
- Uncertainty of traffic prediction.

The value of the additional information to be gained by further tests can be calculated explicitly by statistical analysis when the problem is well-defined and has a simple structure. However, in pavement rehabilitation this is not always possible because of the complexity of the pavement, and the value of such information can often be assessed well enough for practical purposes by considering the cost of obtaining additional information in relation to the:

- Consequences of not making the optimal decision (in terms of cost and performance of rehabilitation measure).
- Probability of not making the optimal decision without additional information.
- Probability of not making the optimal decision in spite of having additional information.

Experience and engineering judgement can be used to assess these factors and decide whether further tests are justified.

All tests should be aimed at improving the understanding of the behaviour of the pavement, rather than providing absolute information. **Moreover, no type of test or analysis is precluded, provided that it is appropriate and that it can be justified by the value of the information that will add to the understanding of the problem.**

Based on the above principles and design considerations, the project level pavement rehabilitation approach outlined in this document will enable suitable rehabilitation options to be determined through a systematic process of testing and analysis. This involves an assessment of the pavement condition, determination of existing structural capacity, identification of the cause and mechanism of distress, use of suitable rehabilitation design methods and finally, the comparison of applicable options and strategies.

1.4 Recommended Approach

1.4.1 General

This guide does not give rigid procedures that should be followed at all times: pavement rehabilitation projects differ considerably, which makes it impossible to give a recipe approach encompassing all possibilities. However, the principles contained in this document generally represent accepted practice in southern Africa.

Normally, a particular length of road is identified for possible rehabilitation because of an excessive need for routine maintenance, poor riding quality, increasing traffic-induced distress, or a combination of these factors. However, it is unlikely that such a length of road is uniform in respect of its rehabilitation needs. Some sections within a length of road may be distressed due to localized factors, others may be exhibiting a problem limited to the surfacing, and sound sections may well exist between the distressed sections.

The objectives of the project level rehabilitation design procedure are, firstly, to divide the pavement into distinct lengths requiring different rehabilitation measures, and then to determine the most suitable measure for each length.

Since the tests that will be needed to establish the appropriate rehabilitation measures for each different section of the road will not be known beforehand, the analysis should be carried out in iterative, increasingly detailed steps. These entail:

- | | |
|--|-------------------------|
| (i) Pavement condition assessment | (detailed in Section 2) |
| (initial assessment followed by more detailed analysis), | |
| (ii) Rehabilitation design | (detailed in Section 3) |
| (iii) Economic analysis | (detailed in Section 5) |

Technically, the objectives of a project level rehabilitation investigation will be fully met by following the three stages of the investigation as recommended. However, in practice, decisions and the process of investigations are continuously influenced by managerial and practical aspects. These aspects are also discussed in Sections 1 and 4 respectively.

1.4.2 Pavement Condition Assessment

The pavement condition assessment consists of an initial assessment, covering the whole of the project length, followed by more detailed investigations of sections exhibiting structural deficiencies. The aim is to concentrate more detailed and expensive testing on those pavement sections requiring structural strengthening.

The initial assessment will normally entail a preliminary site visit and an examination of available records, which will determine the need for tests such as deflection and profile measurements and a detailed visual inspection. Guidance for undertaking a detailed visual inspection is provided in Section 2 of this document.

The objectives of the initial assessments are:

- (i) To identify sections where significant problems exist and the nature of these problems. This will establish sections:

- Where there is no significant problem.
 - Where the distress is obviously limited to the surfacing.
 - Where localized factors are the cause of the distress.
 - Which may be structurally inadequate.
- (ii) To recommend:
- Appropriate rehabilitation options for the surface distress,.
 - Remedial action for the localized distress.
 - Further tests for the sections where structural improvements may be required.
- (iii) To provide:
- A convenient record of initial measurements taken to describe the condition of the pavement.
 - Input into the further analysis of sections in need of structural improvements.

Distress is considered a significant problem if it cannot be adequately dealt with through routine maintenance as practised by the road authority concerned. In some cases the cause of distress is limited to the properties of the surfacing and is unrelated to the structural capacity of the pavement. Rehabilitation options, in such cases, can range from surface treatments to bituminous overlays, or even replacement of the surfacing, and could include recycling or application of surface rejuvenators. Usually the best option can only be determined after further detailed testing of the distressed layer.

Deficient drainage is often the main cause of isolated distress. However, this type of distress may also be due to design inadequacies, poor materials, or poor construction of a localized section. It is important to establish the exact cause of localized problems before attempting to select the most economical remedial measure. This will prevent expensive rehabilitation of the whole pavement length when only specific distressed areas need rehabilitation.

The initial assessment will have identified lengths of pavement in probable need of structural improvement. These lengths are examined further in order to ascertain the probable cause and mechanism of distress and the remaining structural capacity of the pavement. The distress may originate in any component of the pavement. For example, the deformation of the surface may have been caused by the deformation of the subgrade or base, or cracking may have started in the base and be reflected through the surfacing. Moreover, in each case there may be a number of reasons for the distress. The main objective of the detailed assessment is to accurately define for each uniform section the pavement situation which is determined by:

- Pavement design variables, e.g. type and thickness of the pavement layers.
- Traffic loading (past and future).
- Environment (temperature and moisture).
- Pavement performance.
- Present condition (visual as well as pavement tests).
- Cause and mechanism of distress.

The pavement situation of distress determines the type of rehabilitation which should be used on a pavement and is also used to assess the applicability of rehabilitation design methods for use on the pavement. Hence, enough testing should be done at this stage of the investigation to confidently determine and identify the pavement situation which describes in detail each uniform pavement section.

1.4.3 Rehabilitation Design

The pavement situation is used to rate rehabilitation design methods for the required structural strengthening of each uniform pavement section. More detailed testing may be required at this stage of an investigation.

The fundamental reasons for each factor causing distress should be considered in determining the possible rehabilitation options. For subgrade deformation as discussed above, the appropriate options could well range from a levelling course overlay to a substantial strengthening of the pavement structure, depending on the particular reasons for the subgrade deformation.

1.4.4 The Economic Analysis

A choice is made among the viable rehabilitation strategies and options by examining their cost and the consequences of the use of the option in terms of the expected future behaviour of the pavement. After determining the expected costs of the project, as shown in Appendix D, a cost-benefit analysis may be warranted to verify the viability of the project. The economic analysis includes:

Selection of an Option

Normally, a number of rehabilitation options could be applicable for use on a particular pavement. Because of the variable nature of pavements, the effect of any treatment on the future behaviour of the pavement cannot be determined absolutely. The costs of each rehabilitation option and the likely consequences of the treatment have to be weighed against the probability with which these consequences would occur. Normally the option that results in the lowest expected present worth of costs (PWOC) is selected for further analysis. Traffic delay and other road user costs should also be taken into account in determining the PWOC of all options.

Selection of a Strategy

Several strategies may be followed using a specific rehabilitation option. The difference in cost will often not be decisive and the selection of a specific strategy could depend on local circumstances and management considerations. Only after all factors have been considered can the most appropriate strategy be determined.

1.5 Management Considerations

1.5.1 Interaction of Systems

The PMS, which is used to identify pavements requiring rehabilitation, is mainly a tool assisting management in taking meaningful and rational decisions. The PMS provides input for, and is influenced by, functions such as planning, budgeting, resource allocation, etc.

The various inputs affecting pavement rehabilitation design are illustrated in Figure 1.1. It is seen that pavement rehabilitation is influenced by pavement engineering inputs, pavement-related engineering inputs, management inputs and logistical inputs.

All these inputs should be noted at the earliest possible opportunity because of their possible overriding influence on an investigation. The main elements of each of these inputs are discussed in detail below.

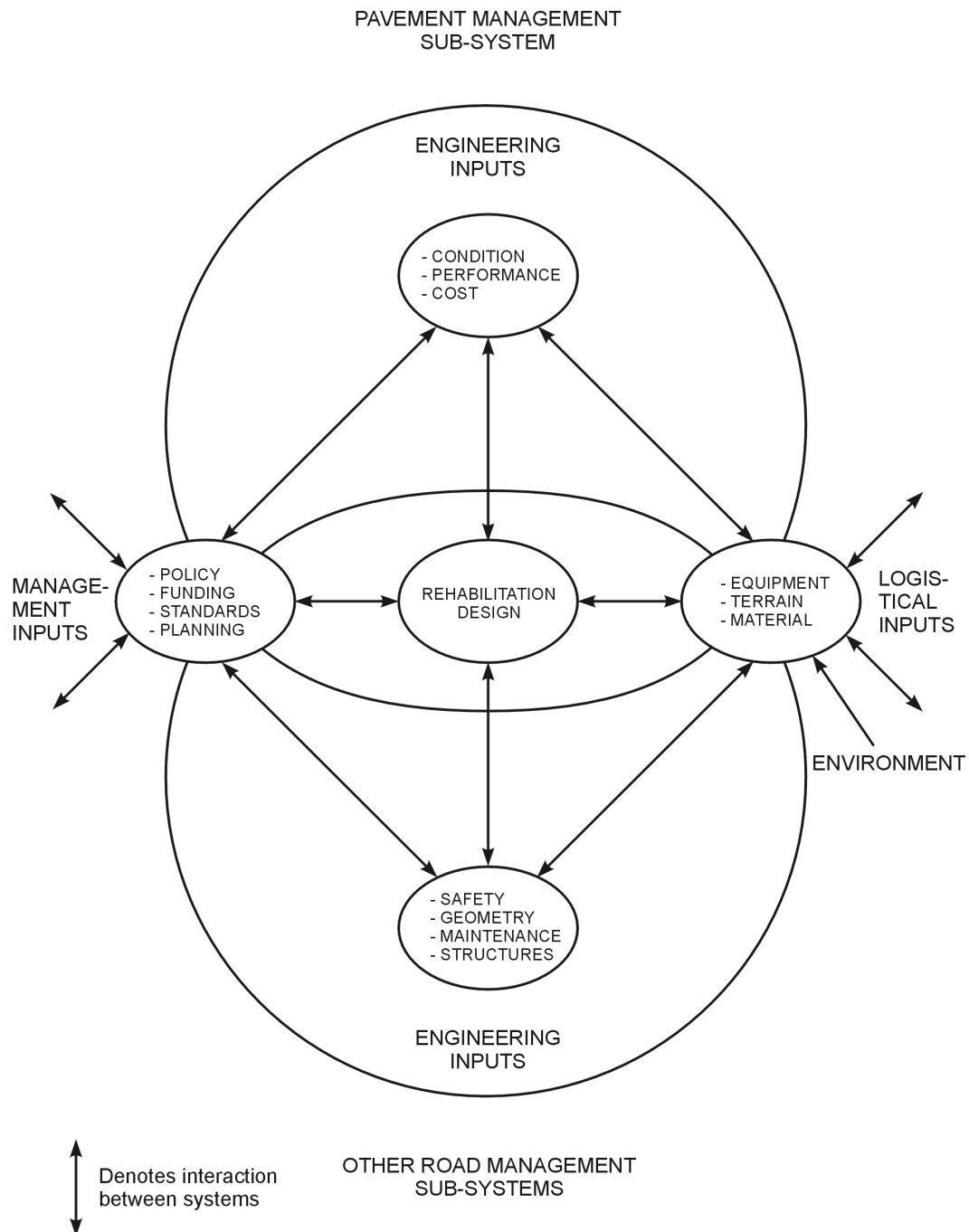


Figure 1.1: Main elements of the various inputs influencing rehabilitation design

1.5.2 Management Inputs

General

Management inputs include all the non-engineering and non-logistical aspects that could influence a rehabilitation project. These include available funds, the importance (economical, political, strategic) of a project as well as the future planning for that specific road and for other roads in the area.

Equally important is the policy of the road authority with regard to standards, including safety and liaison with other road authorities, taking into account their specific future planning in cases when a route could be influenced by their actions.

As shown in Figure 1.1, management inputs are also influenced by the specifics of any project. Hence, it is important to keep management informed and to discuss the consequences of technical as well as financial decisions. Close co-operation between management and the designer should also ensure that management is supplied with the information it needs to make timeous decisions if and when the scope of a project needs to be changed.

Funding

In the past, road funding was provided according to the demands of the various government agencies. This system worked well during periods when funding kept pace with the needs as identified by the various road authorities. However, this is no longer the case and road authorities are finding it increasingly difficult to meet their commitments and to convince funding authorities of the need for funds to upgrade and maintain their networks.

Hence, financial planning should form an integral part of road network planning as well as of road maintenance and rehabilitation planning.

For example, limited funds may make it impractical to investigate in detail relatively expensive long-term solutions. In such a case the management input of "limited funds" should guide the designer to concentrate his investigation on relatively less expensive short-term solutions.

Proper financial planning should identify needs, develop managerial strategies, make the best use of limited resources, reduce uncertainty and help educate the public and public officials. Up-to-date information on the condition of the road network, future transportation needs and consequences of funding levels must form part of financial planning, thus enabling road authorities to negotiate funds in competition with other government agencies and departments.

The identification and selection of rehabilitation projects should therefore be based on sound economic principles. For this purpose, the anticipated costs and benefits of projects must be determined to enable the calculation of feasibility indicators such as the cost-benefit ratio, the Economic Internal Rate of Return (EIRR), Present Worth of Costs (PWOC) and Nett Present Value (NPV).

Policy

The policy of a road authority is normally based on many years of practical experience. This may concern practical aspects such as the use of certain materials, equipment and/or procedures. Certain policy aspects may evolve or change with time due to new developments such as improved techniques, materials, equipment, better information, or because of forced changes related to government policy, such as lower levels of funding, or strategic development of certain areas or industries.

Hence, policy is an integral part of the planning of the transport system. Changes in input could, in a well organized structure, lead to changes in policy to accommodate and perhaps counter these influences. Policy aspects should be considered as inputs into a project-level rehabilitation study and the designer should keep informed of the policies of specific road authorities.

However, because of the uniqueness of every rehabilitation investigation and the technical expertise required from the designers involved in rehabilitation projects, it is not advisable to formulate a rigid policy regarding technical prescriptions for pavement rehabilitation projects. Nevertheless, policy can be made regarding:

- The extent to which the various levels of an investigation should be carried out and the general approach the investigator should follow.
- The standards applicable to specific roads (including design life) and the extent to which road-related aspects such as safety should be investigated.
- The appointment and method of payment of the designer (rehabilitation projects often require more expertise and input than new projects, and the policies regarding compensation of the designer should take this into account).

Standards

Road authorities may have different policies regarding standards for pavement rehabilitation projects. These standards would normally relate to the pavement category, taking into account pavement structure, traffic loading and the riding quality of the road. The standards should be a function of the class of pavement and relate to the importance of the road and the traffic (both volume and load) that the pavement carries.

The duty of the road authority is to set specific standards according to which roads should be evaluated, and to prescribe standards to which a road should be rehabilitated. Understandably, policy changes and changes in level of funding could influence standards. However, the lowering of standards to "balance the books" is not advisable due to the possibility of some "sub-standard" rehabilitated road sections in a network. Road authorities should aim to maintain appropriate standards, taking cost implications into account.

Standards for the rehabilitation of pavements should not only refer to aspects of the structure of the pavement, but also to related technical matters such as safety, geometrics and capacity.

Road authorities should therefore prescribe appropriate standards for the assessment and improvement of these aspects during rehabilitation projects.

Planning

The above-mentioned aspects related to funding, policy and standards all form an integral part of the planning of a road authority. In addition, many other aspects should be taken into account during the planning of rehabilitation projects. These also include non-technical aspects not necessarily concerning the designer, but important to management. These include:

- Programmes of road authorities, neighbouring states and provinces: projects should not be decided upon in isolation. The planning of road authorities "across the border", concerning one or several other roads in the same area which could influence the project, should be taken into account.
- Strategic roads: the need for the improvement of a road could also depend on non-structural and non-traffic associated considerations such as strategic planning. In this regard state departments should liaise with each other and road authorities should have insight into the transportation requirements as influenced by government policy such as strategic roads, accessibility to distant markets and decentralization.

1.5.3 Logistical Inputs

Equipment

The investigation should take cognisance of the availability of special equipment needed for investigation and the construction of rehabilitation. Some rehabilitation options, such as the in-place hot-mix recycling of asphaltic layers, may require highly sophisticated and/or expensive equipment which may not be readily available.

In cases where equipment is not available, the cost to the contractor, and thus to the project, of manufacturing, importing or buying such equipment should be taken into account in the evaluation of specific options.

Only a limited number of pieces of specialized equipment, such as the Deflectograph and Falling Weight Deflectometer (FWD), used for the measuring of pavement condition may be available in the country. Thus, rehabilitation investigations should be planned around the availability of such equipment. Good communication between the respective road authorities and their designers should ensure that projects are planned taking into account the availability of equipment.

Terrain

Terrain conditions and associated restrictions could limit the rehabilitation options which can be considered for a specific project. The early identification of such terrain-associated problems could save considerable time in the design and consideration of possible rehabilitation options. These problems include:

- *Traffic accommodation problems:* the characteristics of a project could be such that traffic cannot be accommodated at a reasonable cost other than on the existing road. In these cases the designer should concentrate on options which will cause minimum disruption to the traffic using the route. Aspects that could contribute to such a situation include:
 - Congested areas in cities where traffic can only be accommodated with great difficulty.
 - Mountainous areas where alternatives for the accommodation of traffic can be very expensive.
- *Bridge or obstacle clearance:* the rehabilitation design must ensure that adequate clearance under any obstacle such as a bridge is maintained. Specifications of road authorities should be adhered to and possible problems should be identified early in the investigation to allow for these cases to be accommodated in the design.
- *Roadside furniture:* the presence of, and height of, roadside furniture such as curbs could influence a decision on the applicability of a rehabilitation option, especially in an urban situation. The placement of asphalt overlays on roads with curbs could lead to the pavement being considerably higher than the curb. In such cases the removal or recycling of the asphalt layer could be considered. Similar problems could occur on rural roads with concrete side drains adjacent to the surfaced area.
- *Services:* the presence of services crossing or alongside the surfaced road should be noted and allowed for in the design of pavement rehabilitation options.

Materials

During the pavement condition assessment a survey of road-building materials available in the vicinity of the road should be made. The non-availability of good quality materials could have serious economic implications for a specific project. The early identification of material resources could save considerable effort during the design phase of an investigation.

1.5.4 Engineering Inputs From Other Sub-systems

Traditionally, rehabilitation design projects are mainly identified as a result of problems associated with the pavement structure. Although road authorities often require an evaluation of the geometric aspects of a road during a rehabilitation investigation, safety and associated aspects are sometimes neglected. Input from a related sub-system such as geometrics could make it unnecessary to investigate the pavement structure over certain lengths of the road which will be influenced by geometric changes.

Many roads considered for rehabilitation will not originally have been designed to modern standards. The opportunity presented by a rehabilitation investigation should also be used to make a full appraisal of the safety, capacity and geometric aspects of the road. However, experience has shown that many "required" improvements are not cost-effective and, if accommodated, may change the cost-benefit ratio of a project and render it uneconomic.

Improvements such as substantial re-alignment, geometric improvements, drainage and perhaps even the widening of the pavement may fall into this category. However, many opportunities for low-cost and hence cost-effective safety improvements may exist for these older roads.

The responsibility rests with the authority and the designer to ensure that the road adheres to reasonable safety standards, although these may not necessarily be the same as those applicable to new roads. Many aspects such as improved signboards, removal of obstacles close to the road, improvement of shoulders, etc, could be addressed at low additional cost during pavement rehabilitation. These improvements could be based on appropriate standards for pavement rehabilitation projects.

Alternative solutions for the improvement of the safety of roads may include methods and guidelines to improve aspects related to:

- Lane widths
- Shoulder widths
- Horizontal alignment and super-elevations
- Vertical alignment and sight distance
- Bridge widths
- Sideslopes and clear zones
- Pavement shoulder condition, including edge drops and drainage
- Intersections
- Pavement surface condition.
- Climbing lanes.

Applicable standards for the consideration of improvements related to the above could depend on:

- Changes in accident rates, user time and vehicle operating costs that can be expected with the improvement of a specific geometric aspect.

- Increases in accident rates that can be expected if the riding surface is improved without addressing safety aspects.
- Safety benefits of low-cost alternatives, such as traffic signals and markings, compared to more expensive improvements.

In many cases the traffic volume expected on the road may warrant upgrading of the road in terms of widening or surfacing of shoulders. Again, the cost implications of such improvements should be carefully considered and the assessment of improvements required in terms of capacity should be carried out in conjunction with related traffic engineering aspects such as geometry and safety.

2. PAVEMENT CONDITION ASSESSMENT

2.1 General

The condition assessment forms the basis of the investigation and design procedure recommended in this document, and the primary aims of the condition assessment are to:

- Identify uniform pavement sections and to establish general structural and/or functional needs through an initial assessment of the road.
- Determine the cause and mechanism of distress and the pavement situation through a more detailed assessment.

The nature and level of detail of the condition assessment should take into account the:

- Class of road.
- Resources available.
- Quantity and nature of the data already available concerning the pavement.

Depending on the requirements of the client, a preliminary report may be requested at the end of the condition assessment. Such a report may require the identification of preliminary rehabilitation needs and their economic implications. Section 3 (Rehabilitation design) and Section 5 (Economic analysis) of this document provide guidance on these aspects.

The initial and detailed assessments are often interwoven and therefore grouped together as the condition assessment. However, for the purpose of clarity, the objectives and scope of each are discussed separately.

2.2 Initial Assessment

2.2.1 Objectives and Scope

The objectives of the initial assessment are accomplished by:

- Recording and processing of data in a form that can be readily used in subsequent detailed investigations.
- Dividing the pavement length under examination into sections requiring different measures.
- Recommending appropriate measures for sections that obviously exhibit only surfacing distress or isolated distress.
- Identifying appropriate tests required on specific sections of the pavement for determining, with confidence, the cause and mechanism of distress and the pavement situation.

In achieving the above objectives the initial assessment is divided into:

- Gathering of information.
- Data processing.
- Pavement evaluation and initial structural capacity analysis of uniform sections.

The recommended approach for the initial assessment is outlined in Figure 2.1. From this figure it is clear that the gathering of information and the processing and evaluation of data are integrated to allow for the optimal use of available data before embarking on any further testing of the pavement.

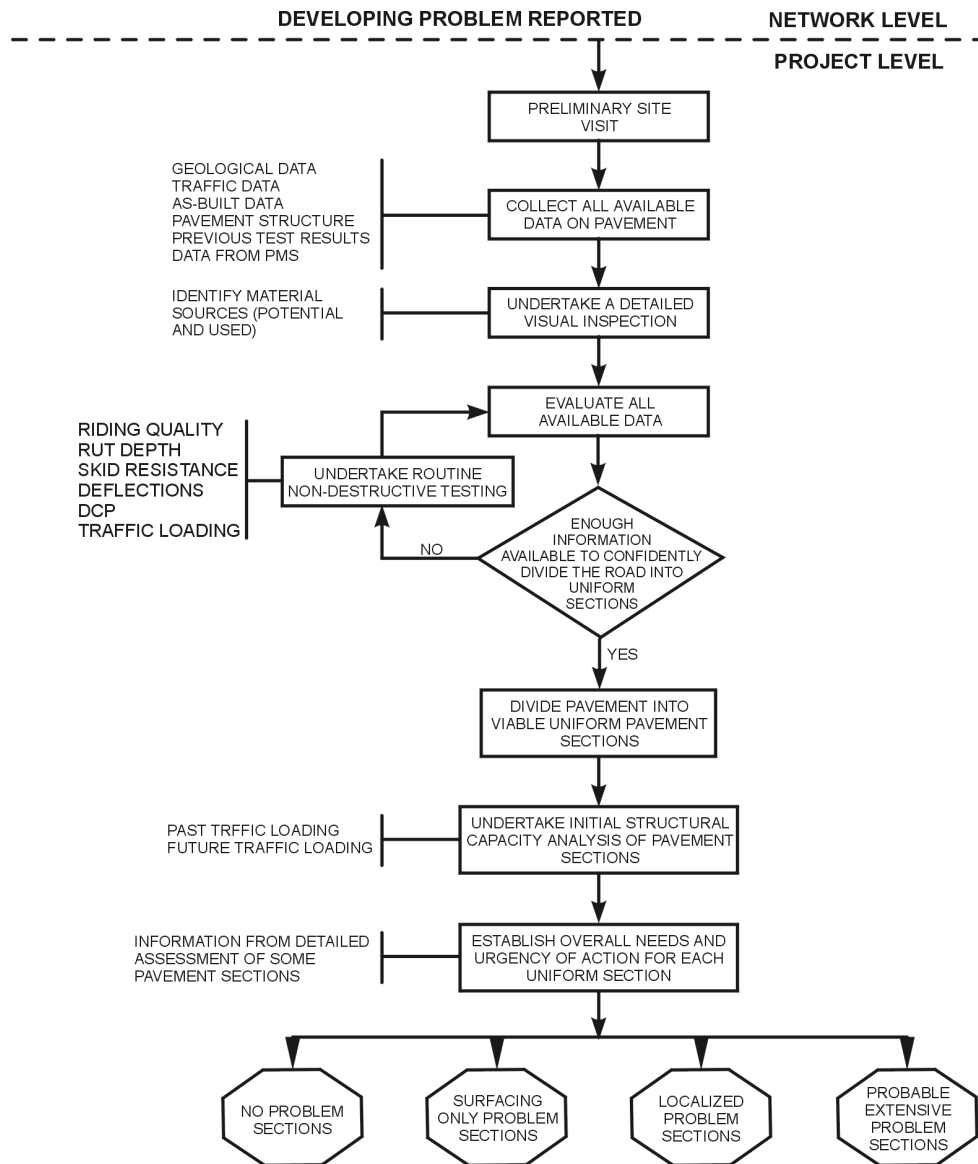


Figure 2.1: Flow diagram of the pavement initial assessment

The division between network level and project level tasks is not, and should not, be clearly defined. With Pavement Management Systems becoming more advanced, some of the tasks shown in Figure 2.1 could in time be incorporated into network level assessments.

2.2.2 Gathering of Information

Preliminary Investigation

This investigation entails the collection of all available information such as as-built data, pavement structure data and traffic loadings on the pavement, as well as any results from tests previously done on the road (such as information about the history of the pavement condition). Existing PMSs are usually an excellent source for obtaining this information.

A preliminary site inspection is essential especially if the assessor is not familiar with the pavement under investigation.

Detailed Visual Inspection

a) General

Information from visual inspections contained in PMSs is generally not detailed enough for use in project level rehabilitation investigations. Hence a detailed visual inspection is considered an essential part of a condition assessment.

Distress visible on the pavement surface is recorded in great detail in a way compatible with the eventual evaluation of the data. Of particular importance is the recording of visual clues to the cause and mechanism of distress. These include details such as construction aspects (e.g. cuttings, high embankments, etc), drainage facilities and obvious topographical and geological features.

b) Basic requirements

To permit observation of the required detail, it is recommended that the visual survey be carried out by walking to ensure quality results in relation to the type of distress and length of the project.

It is recommended that the inspection be carried out by two persons who both have a comprehensive knowledge of pavement distress and its causes. At the same time that this investigation is carried out, it is often practical and economical, using additional inspectors, to record all pertinent detail in the road reserve from fence to fence such as the location and condition of all drainage structures, road signs, lay-bys, guardrails, fencing, signs of erosion and flooding, intersections and other road furniture. It is normal during a rehabilitation contract to allow for repairs and improvements within the road reserve and this information is essential.

Different modes and types of distress are discussed in detail in TRH6¹ and TMH9². These documents, together with the applicable documentation issued by the various road authorities, should be studied before the visual inspection.

The visual inspection should be carefully planned. A form, an example of which is shown in Appendix A, must be prepared for recording all the relevant information. However, it is important that the detail and the number of variables considered are kept within manageable proportions.

Knowledge of the local geology and of the pavement structure will assist in the interpretation of field observations.

c) Recording of distress in pavements

Visible signs of distress as well as possible clues as to the cause of the distress must be recorded. For this purpose, the following information is relevant:

- Location of distress
- Mode and type of distress
- Degree and extent of distress
- Position and spacing of distress
- Pertinent construction details and deficiencies
- Topographical, geological and vegetational clues to the cause of distress
- Drainage structures/facilities.

i. *Location of distress*

The inspection must provide an accurate record of the location of each particular mode and type of distress that is evident. The location is given in relation to the length of the road, usually based on km posts, but pre-marking may be necessary to provide suitable reference.

ii. *Mode and type of distress*

Four main modes of visible distress are used:

- Deformation or unevenness
- Cracking (surfacing or structural problems)
- Disintegration (surfacing or structural problems)
- Smoothing of the surface texture (skid resistance).

These modes of distress are manifested in several typical ways and Table 2.1 gives details, together with the codes generally used to identify the types on an inspection form (see Appendix A for typical forms).

Different types of distress within the same mode are generally brought about by different causes. It is therefore important that individual types be recorded separately where practicable. However, a differentiation into all the types listed above will not always be relevant for an assessment, and therefore not essential for every visual survey.

Table 2.1: Modes and types of distress and their typical codes

Mode of Distress*	Type of distress	Code
Deformation	Depressions	DE
	Mounds	M
	Ruts	RU
	Ridges	RI
	Displacements	DI
	Corrugations	CO
	Undulations	U

Cracking	Transverse cracks	T
	Longitudinal cracks	L
	Block cracks	B
	Map cracks	MA
	Crocodile cracks	C
	Parabolic cracks	P
	Star cracks	S
	Meandering cracks	ME
	Multiple cracks	MU
Disintegration of surfacing	Ravelling	R
	Potholes	PH
	Edge breaks	EB
	Patches	PA
Smoothing of surface texture	Bleeding	BL
	Polishing	PO
* where possible the origin of the distress within the pavement should be identified during the detailed visual inspection		

iii. *Degree and extent of distress*

The degree of distress is an indication of the seriousness of the problem. Explanations of the degree into which each type of distress is classified are given in Table 2.2. The extent of distress can be given as a proportion of either the length or area of the pavement affected.

Table 2.2: Classification of degrees of distress

Degree	Severity	Description
0	-	No distress visible.
1	Slight	Distress difficult to discern. Only slight signs of distress visible.
2	Between slight and warning	Easily discernible distress but of little immediate consequence.
3	Warning	Distress is notable with respect to possible consequences. Start of secondary defects; maintenance is already possible or needed e.g. cracks can be sealed.
4	Between warning and severe	Distress is serious with respect to possible consequences. Secondary defects have developed (noticeable secondary defects) and/or primary defect is serious.
5	Severe	Secondary defects have developed (noticeable secondary defects) and/or extreme degree of primary defect.

iv. *Position and spacing of distress*

The position of distress is given in relation to the width of a traffic lane, e.g. on the shoulder, in the wheelpath, or near the centre line.

Spacing indicates the distance between the occurrences of a similar type of distress. For example, for transverse cracks the spacing indicates the average distance between the cracks.

v. *Pertinent construction details and deficiencies*

Visible construction details in the areas of distress, such as the occurrence of distress in a cut or fill, can be important in the assessment of the pavement and must be recorded.

Pavement distress can arise from visible construction deficiencies which should be rectified as part of any rehabilitation strategy. Examples of such deficiencies are: insufficient crossfall, blocked drains, inadequate side drains, etc.

vi. *Topography, geology and vegetation*

Topographical and geological observations and noting of vegetation may provide useful indications of insufficient drainage. Examples of such indications are given below:

- *Topography*: Pavement layers intersect the geological strata and thus obstruct the normal flow of water in the ground. This problem, if present, is usually found in cuts or at transition from cut to fill.
- *Geology*: Strata of varying permeability can be a source of drainage problems. Intrusions or a relatively impervious substratum may prevent the free passage of water, causing it to enter the pavement layers. This commonly occurs where the road elevation is close to ground level with a shallow depth of transported or residual soil over a hard and relatively impervious substratum.
- *Vegetation*: Drainage problem areas are usually associated with lush vegetation. Specific types of grass or reeds, signs of seepage or erosion and rotting vegetation are positive indications of drainage problems.

vii. *Drainage structures/facilities*

Drainage problems such as blocked or ineffective facilities could be a major cause of distress. Particular attention should be paid to the condition of the existing facilities.

d) *Inspection Form*

An example of a typical form used for the recording of information during a visual inspection is given in Appendix A. Different road authorities may have varying requirements of terms of the forms to be used that should be taken into account. For practical reasons it is suggested that codes such as those given in Table 1.1, be used to identify the different types of distress. On this form the position and extent of distress are shown graphically and space is provided for this purpose. Allowance should also be made for the aspects related to the recording of pavement performance such as drainage condition and lush vegetation.

Pavement Surveillance Measurements

The need to obtain any additional information such as riding quality or deflection bowl measurements through routine, non-destructive surveys of the pavement is determined by the detail and volume of information uncovered by the preliminary investigation. Enough information must be collected to allow for the confident division of the pavement into uniform sections taking into account both its functional and structural parameters.

Riding quality, rut depth and skid resistance measurements are used for assessing the serviceability (functional aspect) of a pavement. Deflection, deflection bowl parameters, Dynamic Cone Penetrometer (DCP) and rut depth measurements are used to assess the structural capacity of a pavement. Data representative of both the main groups (functional and structural) should be used in the condition assessment of the pavement. The test frequency depends on a number of factors such as the:

- Type of test
- The variation in the pavement property to be measured
- Road category which will determine the level of service and the accuracy required
- Statistical properties (e.g. distribution) of the measurements
- The length of the road sections to be evaluated.

The number of tests should be sufficient to ensure that conclusions are made with confidence.

2.2.3 Data Processing

General

For the effective evaluation of the condition of the pavement the collected data must be judged against set criteria and presented in a form that facilitates analytical comparison. It is therefore necessary to establish and use appropriate performance criteria for each type of measurement.

Much experience has been gained over the years in the actual measurement and recording of some of these parameters and this has resulted in the establishment of empirical relationships. The criteria recommended in this document are based on studies of these relationships and others established overseas, as well as on limits accepted in general practice. Wherever possible, however, the criteria should be modified to suit local conditions.

Performance Criteria

a) General

Depending on the parameter under investigation, performance criteria could depend on the category of road (importance and traffic loading), the pavement structure and the drainage (moisture regime) of the pavement. Criteria have been established for three condition classifications, i.e. sound, warning and severe, the definition of which depends on the parameter under investigation.

For functional aspects, the criteria refer to acceptability of the pavement in terms of existing distress (e.g. cracking, deformation, etc.) as recorded during the detailed visual inspection:

- Sound condition: adequate condition
- Warning condition: uncertainty exists about the adequacy of the condition
- Severe condition: inadequate condition.

For structural aspects, such as measured pavement or pavement layer response, the criteria refer to the expected future performance of the pavement:

- Sound condition: the measured or recorded parameter is of such a magnitude that the pavement should be able to carry the design traffic for the specific category of road without deteriorating to a warning state (state considered most economical for rehabilitation).
- Warning condition: the measured or recorded parameter is of such magnitude that the pavement should be able to carry the minimum design traffic, but not the maximum design traffic for the specific category of road, without reaching a critical state (the pavement is expected to deteriorate to a level between a warning and severe state).
- Severe condition: the measured or recorded parameter is of such magnitude that the pavement is expected to deteriorate beyond a critical state before the minimum design traffic loading for a specific category of road had been reached.

The following aspects are taken into account in the establishment of appropriate performance criteria:

i) *Category of road*

Table 2.2 gives a nominal four category road classification, for which evaluation criteria are established for A, B and C category roads. Depending on the importance of D category roads, the criteria for either B or C category roads could be used in the analysis of these roads.

Table 2.3: Nominal road categories adopted for pavement rehabilitation design

ROAD CATEGORY ¹				
	A	B	C	D
Typical description	Major inter-urban freeways and major rural roads	Inter-urban collectors and rural roads	Lightly trafficked rural roads, strategic roads	Light pavement structure, 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/ usually low level of service
TYPICAL PAVEMENT CHARACTERISTICS				
RISK	Very low	Low	Medium	High
Approximate design reliability (%)	95	90	80	50
Typical design traffic loading (MESA/lane)*	3 - 30+ over 20 years	0.3-10	< 3	< 0.7
Typical pavement design class**	T5 to T8	T2 to T6	T1 to T4	T1 to T2
Typical daily traffic: (v.p.d.)***	> 3 000	> 500	<1 000	< 300
Typical riding quality:				

- at construction - terminal condition (PSI)***	4.0 - 4.5 2.5	3.5 - 4.5 2.0	3.0 - 4.0 1.8	2.5 - 3.5 1.5
- at construction - terminal condition (IRI m/km; mm/m)***	1.2 - 2.2 4.0	1.5 - 3.0 4.5	2.0 - 3.5 5.0	2.5 - 4.0 5.5
¹ Tabulated values are given only as guidance to help select the nominal pavement classification and therefore the appropriate level for rehabilitation evaluation criteria. * MESA/lane: million Equivalent Standard Axles/lane ** SATCC pavement design catalogue traffic classes *** v.p.d.: vehicles per day **** PSI = Present Serviceability Index, scale 0-5; IRI= International Roughness Index, in mm/m or m/km.				

ii) *Pavement structures*

The following pavement structures are identified in terms of the materials used in the base of the pavement:

- Cemented materials
- Bituminous-treated materials
- Lightly cemented materials
- Natural gravel or untreated materials on:
 - An untreated subbase
 - A treated subbase (TSB).

iii) *Moisture regime*

For DCP measurements the moisture regime or drainage condition is taken into account. Four conditions are identified, namely:

- A dry moisture regime or good drainage condition (M1)
- An optimum moisture regime or average drainage condition (M2)
- A wet moisture regime or poor drainage condition (M3)
- A soaked moisture regime (M4).

b) *Recommended Criteria*

Performance criteria for the evaluation of the data collected on the pavement, visually as well as with the aid of instruments, are discussed in detail in Appendix A.

Of particular importance are the recommendations regarding the confidence levels at which pavements should be evaluated. It is considered acceptable for a small percentage of a length of road to perform unsatisfactorily at the end of the rehabilitation design period. This percentage depends on the category of road. The percentile levels recommended are given in Table 2.4.

Table 2.4: Percentile levels recommended for data processing

Category of Road	Length of Road Allowed to Perform Unsatisfactorily At the End of its Design Life (%)	Percentile Levels Recommended for Data Processing
A	5	95
B	10	90
C	20	80
D	50	50

Traffic Loading

a) General

The life of a pavement to a certain level of distress is usually expressed in terms of traffic loading. For pavement rehabilitation design purposes, the effect of the various elements which contribute to traffic loading is combined in three variables which adequately describe the load applied to the pavement. These variables are the equivalent traffic loading, the rate of accumulation and the frequency of the application. These enable the cumulative number of equivalent Standard Axle loads (ESAs) to be estimated for any given period.

The Standard Axle load is normally defined as either an 8160kg or an 80kN single axle load (also sometimes as an 18000lb or 18 kip axle load), and these can be considered identical for practical purposes. Other common nomenclatures used elsewhere for equivalent Standard Axle loads are ESALs (Equivalent Standard Axle Loads) or E80s (equivalent 80kN Standard Axle Loads). The simplification of the load variables to a single load element (cumulative ESAs) is found to produce satisfactory results in practice.

It should be noted that the Standard Axle is simply a standard unit of measurement, invariably used for determining and characterising pavement performance and behaviour characteristics under trafficking, which has no relation to any legal axle load limits.

b) Traffic load estimates using information from detailed surveys

Refer to the SATCC pavement design guide³ or TRH16⁴

Where detailed information about the axle loads of vehicles using a pavement is available, such information should be fully utilized. In such cases it is recommended that the differences in the reaction of different materials to loading be taken into account.

The formula used for the determination of equivalent traffic is:

$$\begin{aligned} \text{Either a) } F_n &= \left(\frac{P}{80} \right)^n, \text{ for kN loads} \\ \text{else b) } F_n &= \left(\frac{P}{8160} \right)^n, \text{ for kg loads} \end{aligned} \quad \dots \text{Equation 1}$$

where:

F_n is the load equivalency factor in ESAs, for exponent n , and

P = axle load (in kg or kN)

n = relative damage exponent

The value of 4 for the exponent n is often used, in line with early findings and the commonly cited "fourth-power damage effects" of heavy axle loads. It is now clear that the value is influenced by various factors, with the most significant being the pavement configuration.

Table 2.5 indicates recommended n values to be used where pavement structures are known, otherwise a value of 4 should be adopted.

Table 2.5: Recommended relative damage exponents, n^5

Pavement base/subbase	Recommended n
Granular/granular	4
Granular/cemented	3
Cemented/cemented	4.5
Bituminous/granular	4
Bituminous/cemented	4

Equation 1, with appropriate " n " values, can be used with available data to calculate the ESAs at any time in the past. These data can then effectively be used to calculate N_p and N_f where:

N_p = past cumulative traffic loading

$$= E_0 \left(\frac{(1+i)^{n+1} - 1}{i(1+i)^n} \right) \quad \dots \text{Equation 2}$$

and

N_f = expected future cumulative traffic loading

$$= E_0 \left(\frac{(1+j)[(1+j)^x - 1]}{j} \right) \quad \dots \text{Equation 3}$$

where:

E_0 = annual ESAs in the year of investigation
(assume the road will be opened the following year)

i = mean historical ESA growth rate during the past
(per cent/100)

n = age of the pavement (years)

j = expected ESA growth rate during the rehabilitation design period
(per cent/100)

x = rehabilitation design period (years)

- c) Traffic load estimates using traffic counts
Refer to the SATCC pavement design guide³ or TRH16⁴

In cases where detailed axle load data are not available, estimations of traffic loading will have to be based on traffic count data. This will often be based on the traffic composition by vehicle type, or simply the total number of vehicles and the percentage of heavy vehicles.

In such cases various assumptions need to be made to estimate N_f and N_p and care should be taken to allow for seasonal variations in traffic when using such data. These calculations are based on the number of heavy vehicles only, since the damaging effects of cars and light vehicles is negligible for practical purposes.

Table 2.6 can be used to estimate the ESAs per heavy vehicle configuration, where specific local information is not available, but every effort should be made to develop local factors from traffic and axle load surveys. The likelihood of significant error in using such generic data is high and this must be recognised by the Engineer.

Table 2.6: Estimation of average ESAs per heavy vehicle class

Vehicle class		Normal* conditions	Abnormal* conditions	Typical range**
Bus	2-axle large (>35 seats)	0.7	1.2	0.4 - 1.8
Trucks (including trailers)	2-axle***	0.7	1.5	0.3 - 2.0
	3-axle	1.7	2.0	0.5 - 3.0
	4-axle	1.8	2.5	0.7 - 3.5
	5-axle	2.2	3.2	1.0 - 4.5
	6-axle	3.5	4.7	1.2 - 6.0
	7 or more axles	4.4	6.0	2.0 - 8.0
* Normal: control of overloading; values based on South African experience ⁴ Abnormal: little formal control of overloading; indicator values ** Higher values outside this range are commonly reported for overloads *** This class in particular can show very great variation				

The table suggests values appropriate in normal conditions (where overloading is minimal, say less than 5 per cent of heavy vehicles) and for abnormal conditions (where vehicle overloading is a recognised problem). The Engineer is advised to consider both in order to make a best estimate where any doubt exists about extent of overloading. An even more simplified estimate based only on nominal ESAs per heavy vehicle (ie, regardless of configuration) is not recommended unless sufficient local data is available to minimise errors, which can be very high otherwise.

The growth rate in ESAs is dependent on the growth rate in traffic volume, the growth rate in heavy vehicles as a percentage of the total traffic volume and the growth rate in ESAs per heavy vehicle.

The expected growth rate in ESAs for the specific road can be determined from Equation 4:

$$ESA \text{ growth rate} = \left[\left(1 + \frac{h}{100} \right) \left(1 + \frac{v}{100} \right) - 1 \right] 100 \quad \dots \text{Equation 4}$$

where:

h = heavy vehicle growth rate

v = ESA/vehicle growth rate

The heavy vehicle traffic growth rate, h, can be calculated from Equation 5:

$$h = \left(\left(\frac{f}{p} \right)^{\frac{1}{n}} \left(1 + \frac{t}{100} \right) - 1 \right) 100 \quad \dots \text{Equation 5}$$

where:

f = future percentage heavy vehicles

p = present percentage heavy vehicles

n = time period

t = total traffic growth rate

Taking these factors into account, it is recommended that a low, average and high estimate be calculated by combining the low, probable and high estimates of the total traffic growth rate, the change in the percentage heavy vehicle and the ESA/vehicle growth rate. These permutations will give the range of ESA growth rates that is possible for a specific road. The cumulative past and future ESA ranges can be determined as previously shown in Equations 2 and 3.

2.2.4 Pavement Evaluation and Initial Structural Capacity Analysis of Uniform Sections

Pavement Evaluation

All information gathered on the road is taken into account when dividing it into uniform pavement sections. It follows that the information needs to be combined in an easily understood and manageable format. Examples of such a procedure are given in Appendix A. The processed data for a number of kilometres of road should be summarised to facilitate easy interpretation for the identification of uniform pavement sections.

The identified uniform pavement sections need to be divided according to their overall need into the following categories:

- Requiring no action
- With only surfacing problems
- With localized problems
- Requiring probable structural strengthening in terms of life in ESAs.

Based on this classification, and using the traffic loading estimates (of what has been carried, and what is expected over the rehabilitation design period), a rating system for the urgency of treatment is recommended. This can assist in prioritisation when a number of potential rehabilitation projects have been identified. Such a prioritisation rating system must, however, be established locally to suit specific prevailing conditions and budgetary constraints.

a) Sections with No Significant Problems

Sections with no significant problems are those which can be economically kept within acceptable levels of serviceability by routine maintenance for the medium term, typically 8-10 years. Such sections are often found between the distressed sections which have prompted the investigation, and can be excluded from further analysis.

However, because of the variable nature of distress, care should be taken to determine that the sections do in fact differ significantly from the others and that distress in them is not just being delayed for some reason (the results of the structural capacity analysis are of particular importance).

b) Sections with Obvious Surfacing-only Problems

These are sections where the distress obviously occurs only in the surfacing, and is in no way caused or aggravated by inadequacies in the underlying pavement structure or subgrade.

Distress solely related to the properties of the surfacing can take the form of:

- Deformation
- Disintegration or ravelling
- Cracking
- Loss of skid resistance through bleeding or polishing.

In the absence of other forms of distress, areas exhibiting ravelling, polishing or bleeding, can be classified as having surfacing-only problems.

In the case of cracking and/or deformation, however, further detailed testing and analyses are invariably necessary before one can be confident that these are in fact surfacing-only problems.

c) Sections Showing Localized Distress

Sections with localized distress are short (normally less than 200 m) but in clearly poorer condition than the adjacent sections. The identification of sections where localized factors are the cause of distress is important for two main reasons:

- To prevent an erroneous under-assessment of the overall structural capacity of the road, which will lead to unnecessary or excessive rehabilitation.
- To prevent the application of rehabilitation measures which do not deal with the causes of localized distress and will result in a recurrence of the distress.

Almost invariably, the main cause of localized distress on rural roads is inadequate surface and subsurface drainage. Poor drainage allows water to accumulate in the subgrade and pavement layers, which ultimately results in a reduction of the strength and load-spreading ability of the layers affected.

Specific attention must be given during the condition assessment to the identification of these sections, which can usually be verified from the:

- Condition of surface drains
- Topography and geology of the surrounding area
- Vegetation in the immediate area

which in turn can confirm the likelihood of excess moisture as the cause of distress.

During the visual inspection, or as a result of such an inspection, any additional information or clues about the cause and mechanism of localized distress must nevertheless be recorded. Further detailed investigation may still be required to establish the exact cause and mechanism of distress for any locally affected area.

d) **Sections Where Structural Improvement May Be Required**

These sections are those which, after the initial assessment, do not fit into any of the previous categories or about which there is still uncertainty as to the exact nature of the distress. They may or not be structurally inadequate but will require further analyses. These sections are dealt with in the detailed assessment phase of the condition assessment.

During the initial assessment, however, an attempt should be made to indicate the nature and urgency of the problem and the nature of more detailed and/or additional testing that should be done on these sections during the next phase of an investigation.

Naturally, further detailed testing may well identify some of the sections as exhibiting only localized distress or surfacing-only problems.

Initial Structural Capacity Analysis

a) **General**

The objective of the structural capacity analysis is to determine the remaining life (if any) of the existing pavement, i.e. the number of ESAs it will still be able to carry before it reaches a critical level of distress. This is particularly relevant where non-strengthening rehabilitation measures appear appropriate, or when a high increase in traffic loading is anticipated.

The capacity analysis will determine the possible distress of the pavement through normal traffic-induced forces during the rehabilitation design period. Where failure is likely during this period, the pavement section should be identified as needing structural strengthening.

Various pavement analysis methods may be used to determine the pavement's structural capacity. At the condition assessment phase of a rehabilitation investigation it is recommended that the structural capacity of the pavement be determined using a simple, empirically-derived relationship. A more detailed and/or sophisticated analysis of the pavement sections will be done during the rehabilitation design phase of the investigation.

Although any applicable method may be used to determine the structural capacity of the uniform pavement sections, three methods based on different pavement measurements are summarized. These are based on the following parameters:

- Surface deflection
- Dynamic Cone Penetrometer (DCP) measurements
- California Bearing Ratio (CBR)

These methods are empirically-based and have limitations in their applicability. Before using any of these methods, users are referred to Appendix B, where the assumptions, limitations and advantages of these methods are discussed in more detail.

b) Methods Based on Surface Deflection Measurements

The use of deflection measurements to assess future pavement behaviour has been well established throughout the world, largely due to the availability of deflection measuring devices in most countries. The deflection method recommended for use is the method developed at the Transport Research Laboratory (TRL) in England⁶. This method takes into account the type of base material in the prediction of remaining life, but is applicable only to a maximum of 10 million ESAs.

This method is based on the standard measurement of deflections using the Benkelman Beam (as described in the CSIR Manual K16⁷). **Measurements taken with any other instrument or using any other procedure should be converted to standard deflections measured with the Benkelman Beam.** Care should be taken with such conversions since relationships between instruments could depend on variables such as the type of pavement.

The design graphs in Figures 2.2 - 2.5 are used to predict the remaining life in term of ESAs to a rut depth of 10 mm respectively for pavements containing:

- Non-cemented granular bases
- Aggregates exhibiting a natural cementing action
- Bitumen-bound bases
- Cement-bound bases

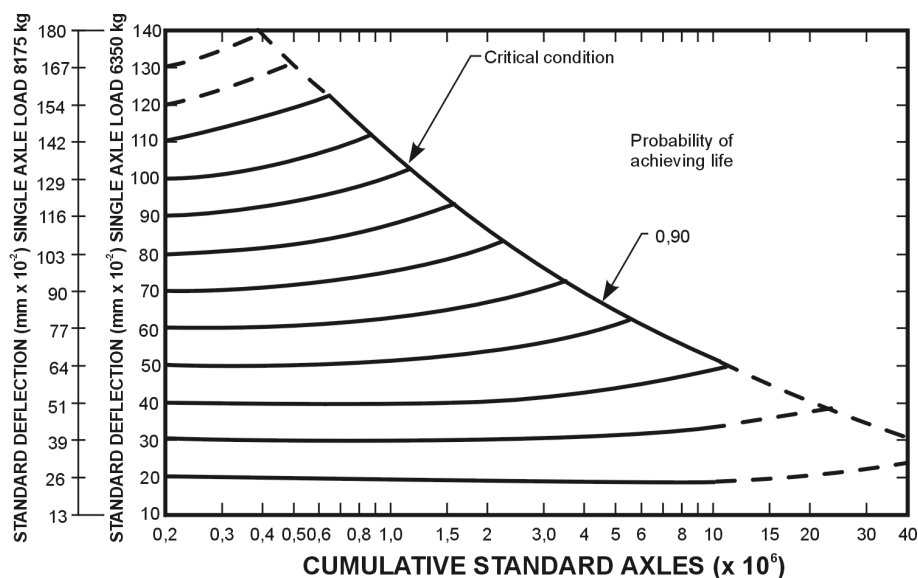


Figure 2.2: Relationship between standard deflection and life: granular road bases

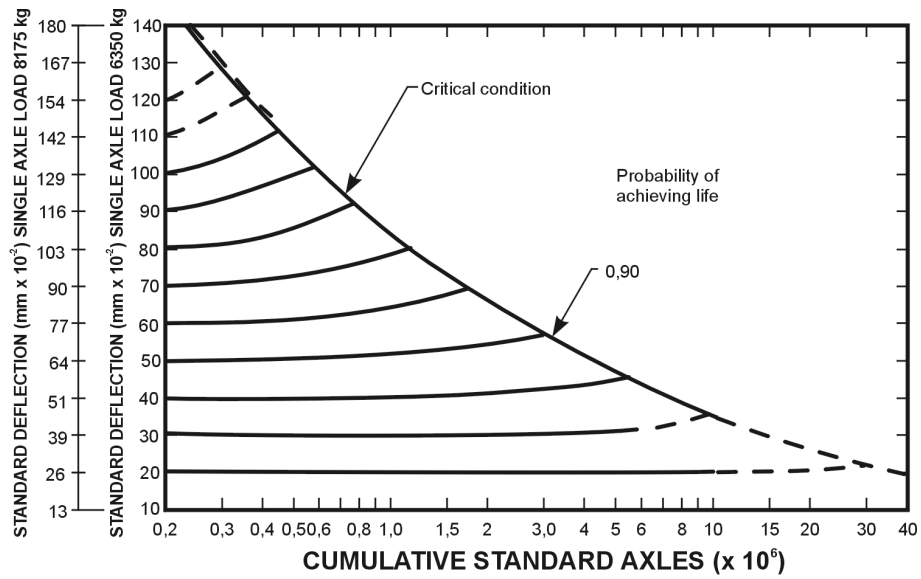


Figure 2.3: Relationship between standard deflection and life: granular road bases with natural cementing action

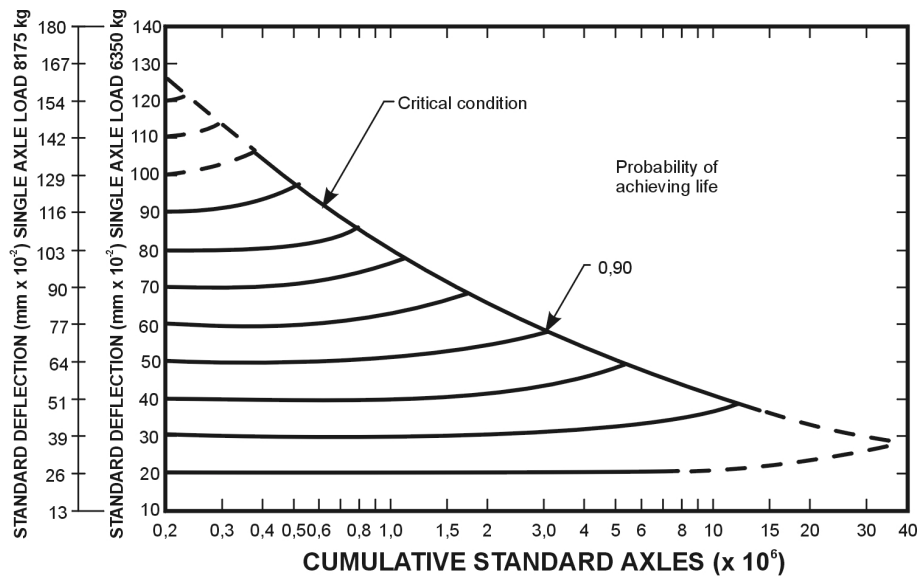


Figure 2.4: Relationship between standard deflection and life: bituminous road bases

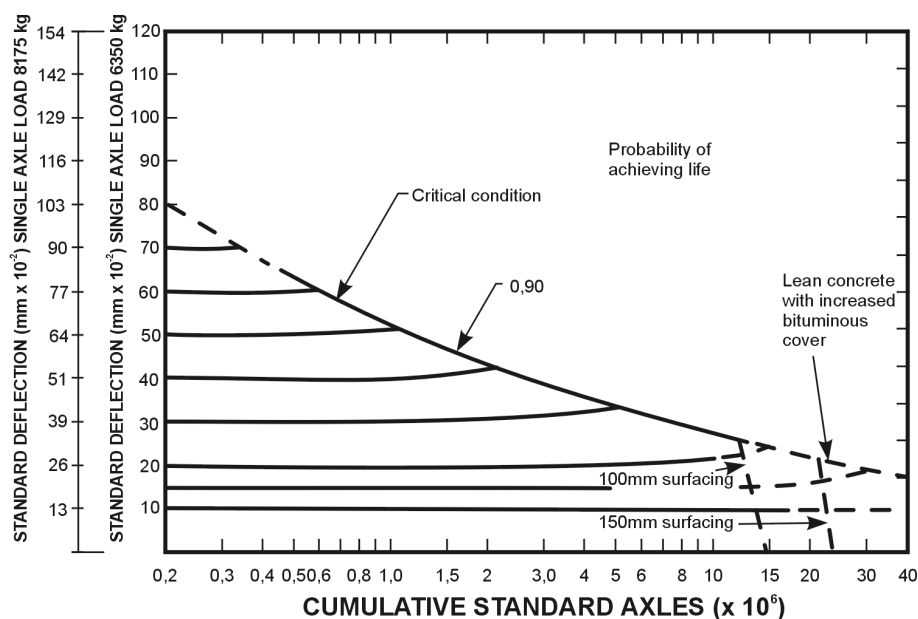


Figure 2.5: Relationship between standard deflection and life: cement-bound road bases

Selection of the appropriate chart takes place on the basis of the type of material used in the pavement structure. A pavement is classified as having a cement-bound base if more than 100 mm of such material is present in the pavement structure, even if located beneath a considerable thickness of bituminous surfacing and roadbase material.

Similarly, a pavement is classified as having a bitumen-bound base when more than 150 mm of bituminous material is present in the pavement structure (provided that there is not more than 100 mm of cement bound material present).

The bituminous layers may be separated by layers of granular material. A pavement is classified as having a granular base with natural cementing action when more than 150 mm of this material is present in the pavement structure (provided that any cement-bound layer is less than 100 mm thick and that less than 150 mm of bituminous-bound layers is present in the pavement). If none of the above applies, the pavement is classified as having a granular road base.

Some restrictions apply to the use of the chart for pavements with more than 100 mm of cement-bound materials. The performance of pavements with cemented layers could depend on deterioration associated with shrinkage cracking reflected from the cemented layers. For this reason the normal critical curve for cumulative loading (in excess of 10 million ESA) only applies to pavements with a bituminous cover of more than 175 mm. As shown by the broken line in Figure 2.5, the expected life (in terms of traffic loading) of these pavements with thinner surfacings is much reduced.

Maximum surface deflections will not give a good indication of expected life of pavements with cement-bound layers under the following conditions:

- i. When a structural weakness is present in the top cemented layer (in this case deflections are kept relatively low by the presence of an undamaged lower cement layer), and
- ii. With weak cemented materials where the aggregate grading has little mechanical stability (low deflections may be present in early life but cracking leads to early disintegration).

With the surface deflection, the past cumulative ESAs and the type of pavement known, the charts in Figures 2.2 to 2.5 are used to estimate the residual life of the pavement, as illustrated in Figure 2.6 with the following data:

Type of pavement: granular base with aggregates exhibiting a natural cemented action

Past traffic load : 3 million ESA
 Standard deflection : 0,38 mm (80 kN single axle load).

From Figure 2.6, the expected residual life
 = 11 million - 3 million ESA = 8 million ESA

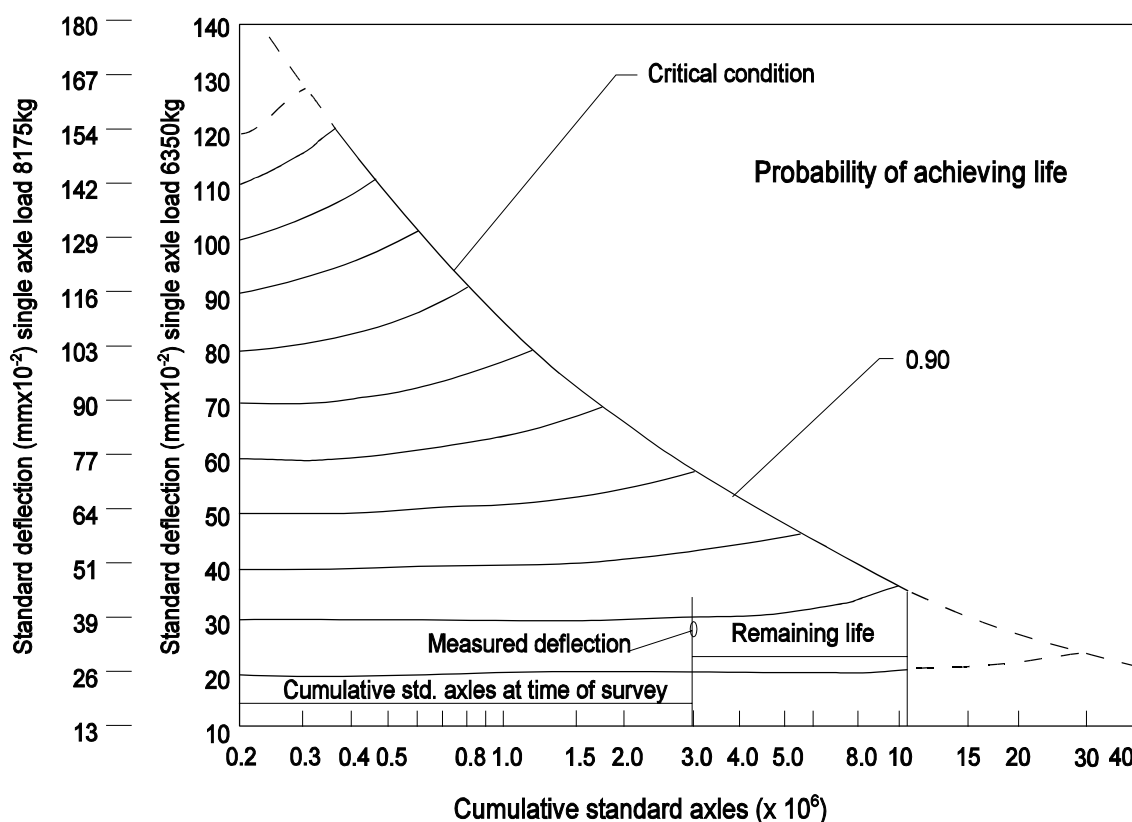


Figure 2.6: Design example for pavements with granular road bases whose aggregates exhibit a natural cementing action (from Figure 2.3)

Deflection basin parameters calculated from Falling Weight Deflectometer (FWD) measurements as discussed in Appendix A, can also be used to obtain an indication of the structural capacity of uniform pavement sections.

c) Method Based on DCP Measurements

The Dynamic Cone Penetrometer (DCP) has been used for a number of years in southern Africa as a non-destructive testing (NDT) device to measure the in situ bearing capacity of pavements. The method used to analyse the bearing capacity of pavements using DCP measurements as described in this document, was developed in South Africa at the then Transvaal Roads Department⁸.

The DCP measures the penetration per blow into a pavement through all the different pavement layers. This penetration is a function of the in-situ shear strength of the material. The penetration-depth profile gives an indication of the in-situ properties of the materials in all the pavement layers up to the depth of penetration which is normally 800 mm for road pavements.

The total number of blows required to penetrate the pavement layers to a depth of 800 mm (DSN_{800}), for a reasonably balanced pavement, is used in Figure 2.7 to determine the bearing capacity of the pavement to a rut depth of 20 mm.

The existing rut depth on the pavement section and the past cumulative traffic loading that has used the road since construction should be taken into account in calculating the remaining "life" of the pavement.

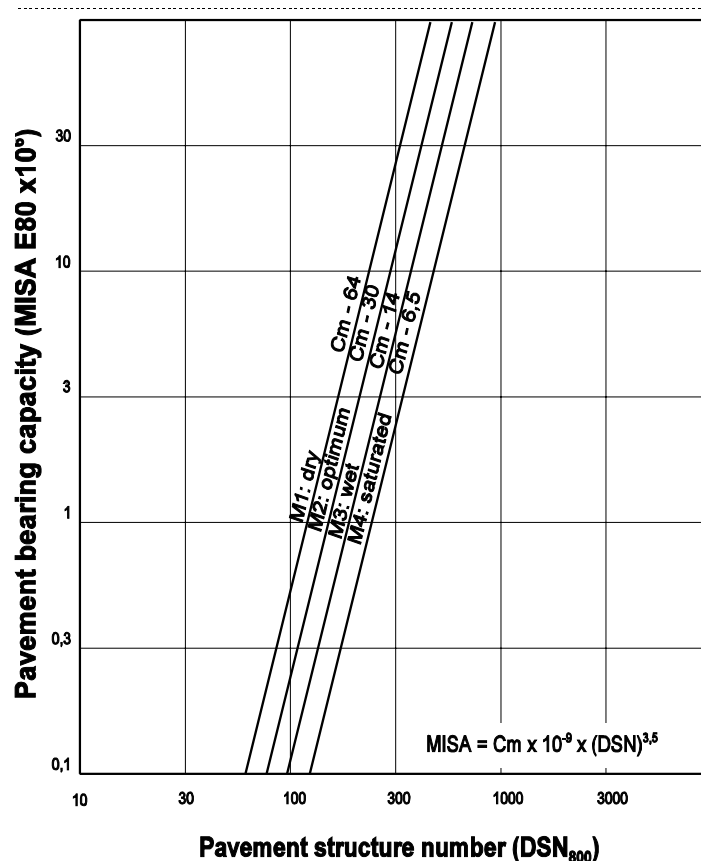


Figure 2.7: Relationship between pavement bearing capacity and DSN_{800}

d) Methods based on CBR measurements

The analysis is based on the measurement of the soaked CBR of the subgrade⁹. It is recommended that at least one CBR test be done every 500 m. The Design Subgrade Strength (DSS) value is defined as the subgrade strength value that is equal to or less than approximately 90 per cent of all test values (minimum of five tests recommended) in a section.

The representative effective thickness (T_e) of the pavement layers above the subgrade is then determined, as outlined below, to estimate pavement bearing capacity.

The effective thickness of the existing pavement is defined as the equivalent thickness of full-depth asphalt that would have the same strength as the total of the pavement layers above the subgrade of the existing pavement.

T_e is calculated by taking into account the type of material and condition of each of the pavement layers. For the different materials the following information is important:

-
- Granular material: thickness of material and classification as base, subbase or improved subgrade quality material
 - Asphalt material: thickness, type and condition of layer
 - Cemented materials: thickness, condition and support of the layer.

The representative effective thickness (T_e) is then determined from Equation 6:

$$T_e = \sum_{i=1}^n f_i t_i \quad \dots \text{Equation 6}$$

where

- t_i = thickness of the i th layer
- f_i = conversion factor of the i th layer (from Table 2.7)
- n = number of pavement layers above the subgrade

The evaluation of the condition of the pavement layers is done subjectively. Conversion factors (f) for each of the pavement layers above the subgrade are given in Table 2.7.

The DSS, together with T_e , is used in Figure 2.8 to get an indication of the remaining life of the pavement.

Table 2.7: Converting thickness of existing pavement components to Effective Thickness⁹

Classification of material	Description of material	Conversion factors
I	Native subgrade in all cases.	0
II	a. Improved subgrade. Predominantly granular materials - may contain some silt and clay but have PI of 10 or less b. Lime modified subgrade constructed from high-plasticity soils - PI greater than 10.	0.0 - 0.2
III	a. Granular subbase or base - Reasonably well-graded, hard aggregate with some plastic fines and CBR not less than 20, use upper part of range if PI is more than 6. b. Cement modified subbase and bases constructed from low plasticity soils - PI of 10 or less	0.1 - 0.3
IV	a. Granular base - Non-plastic granular material complying with established standards for high quality aggregate base. Use upper part of range. b. Asphalt surface mixtures having large well defined crack patterns, spalling along the cracks, exhibit appreciable deformation in the wheel paths showing some evidence of instability. c. Portland cement concrete pavement that has been broken into small pieces, two feet or less in maximum dimension, prior to overlay construction. Use upper part of range when subbase is present; lower part of range when slab is on subgrade. d. Soil-cement bases that have developed extensive pattern cracking, as shown by reflected surface cracks, may exhibit pumping, and pavement shows minor evidence of instability.	0.3 - 0.5
V	a. Asphalt surfaces and underlying asphalt bases** that exhibit appreciable cracking and crack patterns, but little or no spalling along the cracks, and while exhibiting some wheel path deformation, remain essentially stable. b. Appreciably cracked and faulted Portland cement concrete pavement that cannot be effectively undersealed. Slab fragments, ranging in size from approximately one to four square yards, are well seated on the subgrade by heavy pneumatic rolling. c. Soil-cement bases that exhibit little cracking, as shown by reflected surface crack patterns, and that are under stable surfaces.	0.5 - 0.7
VI	a. Asphalt concrete surfaces that exhibit some cracking, small intermittent cracking patterns and slight deformation in the wheel paths but remain stable. b. Liquid asphalt mixtures that are stable, generally uncracked, show no bleeding, and exhibit little deformation in the wheel paths. c. Asphalt treated base, other than asphalt concrete.** d. Portland Cement concrete pavement that is stable and undersealed has some cracking but contains no pieces smaller than about one square m.	0.7 - 0.9
VII	a. Asphalt concrete, including asphalt concrete base generally uncracked, and with little deformation in the wheel paths. b. Portland cement concrete pavement that is stable, undersealed and generally uncracked. 0.9 - 1.0 c. Portland cement concrete base, under asphalt surface that is stable, non-pumping and exhibits little reflected surface cracking.	0.9 - 1.0
<p>* Values and ranges of Conversion Factors are multiplying factors for conversion of thickness of existing structural layers to equivalent thickness of asphalt concrete.</p> <p>** Asphalt concrete base, asphalt macadam base, plant-mix base, asphalt mixed-in-place base.</p>		

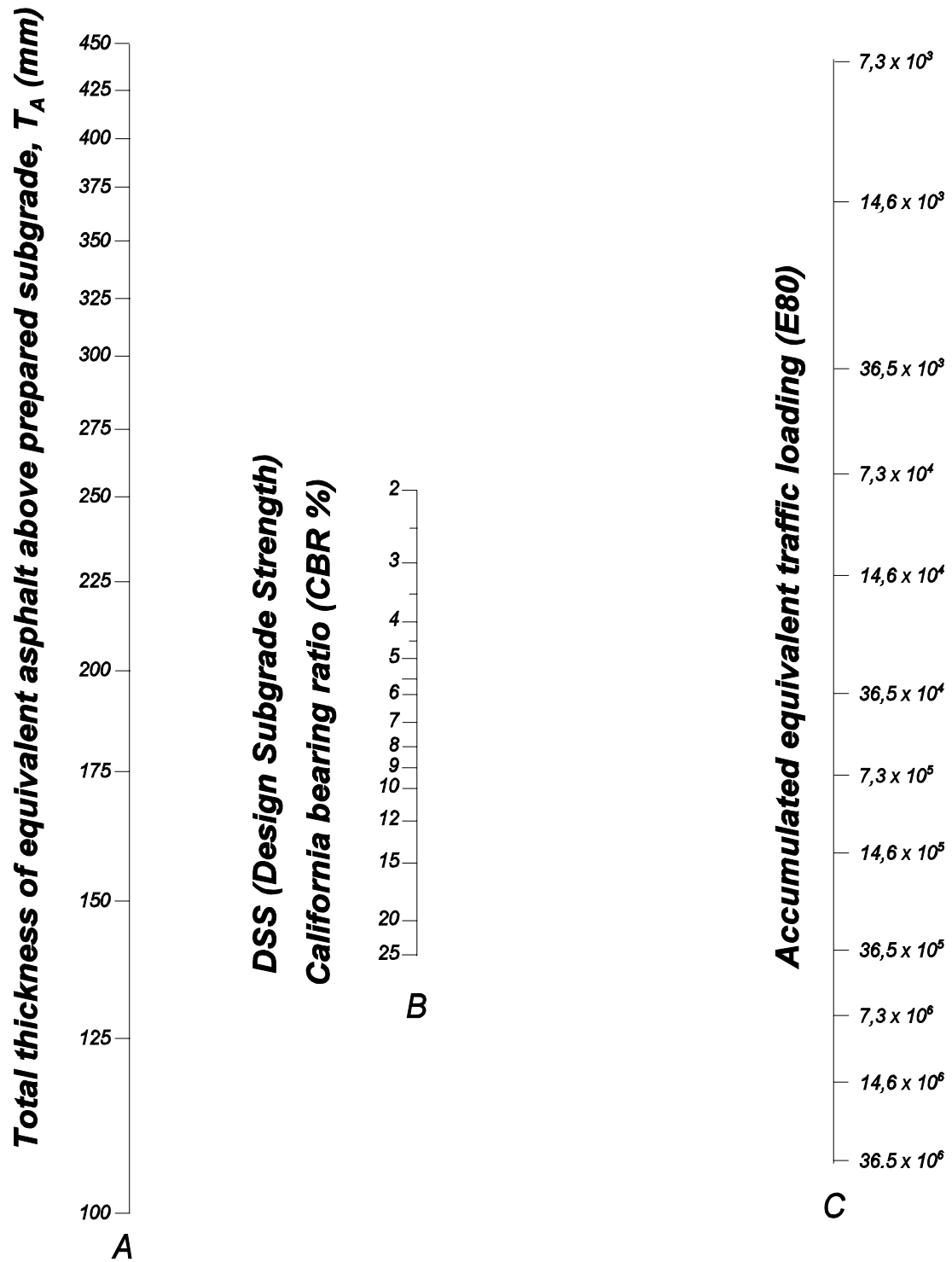


Figure 2.8: Thickness requirements for asphalt pavement structures using subgrade soil CBR⁹

2.3 Detailed Assessment

2.3.1 Objectives and Scope

The detailed assessment deals with lengths of road that have been identified as probably requiring structural improvement. The aim is to obtain sufficient knowledge about the length of pavement to enable a confident decision to be taken on the rehabilitation strategy to be followed. This is accomplished by:

- The determination of the cause and mechanism of distress in each uniform pavement section.
- A description of the pavement situation as represented by each uniform pavement section.

The cause and mechanism of distress of a pavement is an important aspect of the pavement situation which may not be known at this stage of the investigation. To identify, with confidence, the cause and mechanism of distress of each uniform section, full use should be made of experience of pavement behaviour and distress manifestations and of all available results of tests and information gained during the initial assessment, before embarking on any further investigation.

The accurate description of the pavement situation which includes the pavement material, loading, environmental and distress conditions, will assist greatly in the correct analysis and rehabilitation design of each pavement section.

The investigation procedure for the detailed part of the condition assessment is outlined in Figure 2.9.

2.3.2 Cause and Mechanism of Distress

General

The understanding of the behaviour of a pavement is the key to the identification of the origin and, hence, the cause and mechanism of distress. Where distress is manifested in a expected manner and this has been observed during the initial assessment, the cause and mechanism of distress and hence the pavement situation may be easy to verify. In this case no further testing may be required before embarking on the selection of applicable rehabilitation design methods and the design of the structural strengthening needed.

However, when doubt exists about the exact cause and mechanism of distress, analytical procedures and tests should be used to confidently determine the cause and mechanism of distress. Information gained during the initial assessment (e.g. pavement type and observed distress) is used to identify the possible causes of distress. Empirical manipulations of data, such as DCP penetrations, deflections, rut depth and traffic count, for example, give an indication of the most probable cause of distress. Confidence to proceed with the rehabilitation investigation is gained by verifying the probable cause and mechanism of distress with appropriate tests.

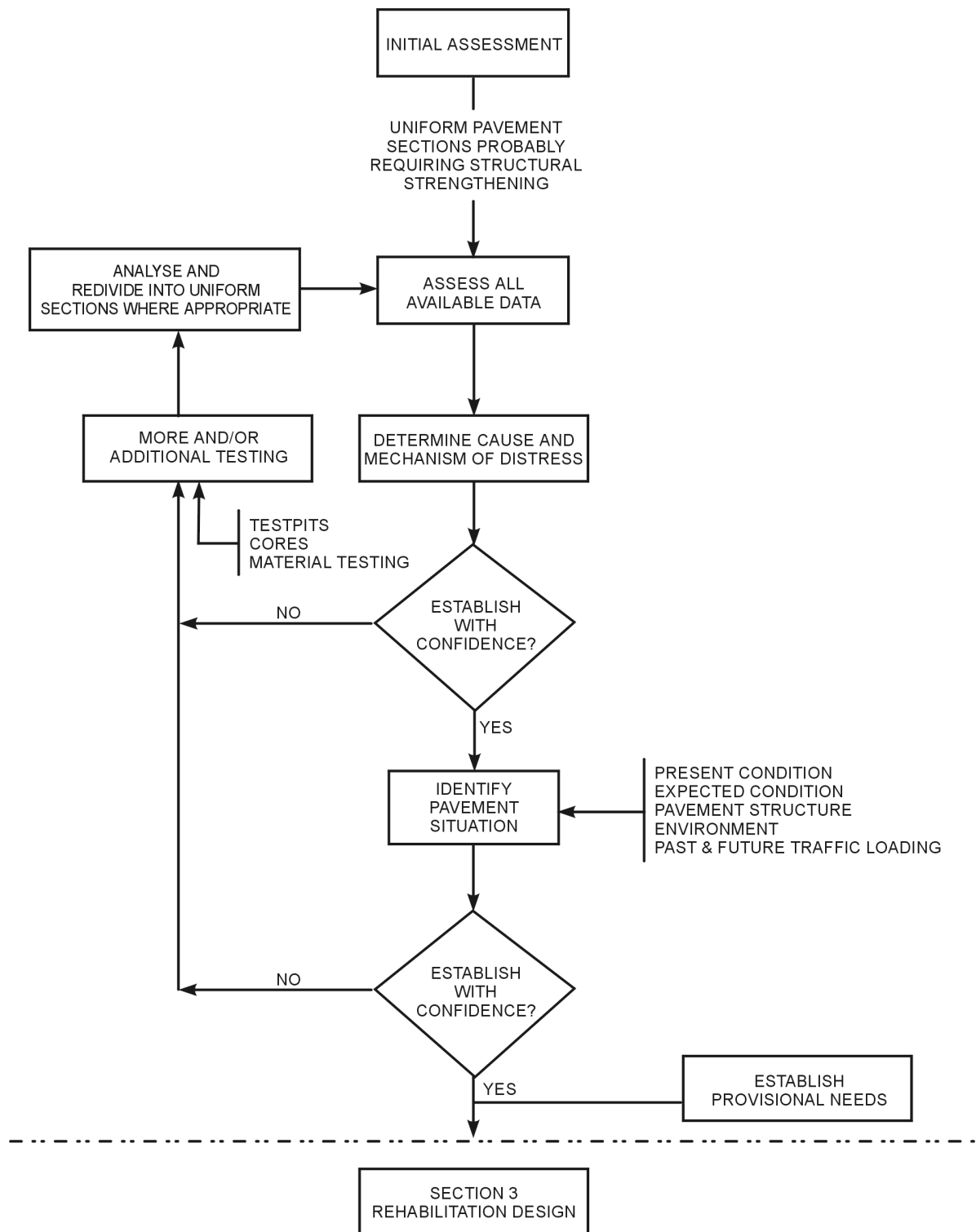


Figure 2.9: Flow diagram of pavement detailed assessment as part of the condition assessment

If the tests do not confirm what was expected, the process must be repeated, but only if more information is likely to change the decision on the rehabilitation strategy to be adopted.

More tests may be needed and this cycle is repeated (as shown in Figure 2.9) until the cause of distress can be positively identified, or until it becomes apparent that more information will have little effect on the decision. With the cause of distress confidently determined, the pavement situation can be defined for each uniform section and used to select applicable rehabilitation design methods for the next phase of the rehabilitation investigation.

Various pavement tests can be used during the detailed assessment phase of an investigation for the structural evaluation of a pavement, and these are summarised in Table 2.8. For completeness, the Table includes several methods well established in South Africa but not currently widely used yet in other SATCC States.

Pavement Behaviour

a) General

Pavement behaviour is a function of the initial as-built composition of the pavement, the load carried by the pavement and the environment in which it operates. The category of the pavement, as shown in Table 2.3, which depends on the importance of the road in terms of the traffic loading, road function, etc., can give a good initial indication of the original strength of the pavement.

In addition to the original strength, pavement behaviour is controlled by the behaviour of the materials in the various pavement layers. In this regard, the type and behaviour of the material in the base course of the pavement is of particular importance, because it:

- Is close to the surface of the road and distress in the base is often reflected on the surface of the road.
- Has little protection in terms of materials covering the layer and hence, a relatively high quality material with good load-bearing characteristics is usually required from the base layers.

Consequently pavements are often described in terms of the materials contained in the base of the pavement, which is the primary distinction used later in evaluating pavement behaviour.

Under the action of traffic and the environment, the materials in the various pavement layers may change with time. Usually by the time of the rehabilitation investigation, for example, treated layers could have broken down and should rather be classified and analysed as granular layers (termed equivalent granular layers).

b) Pavement Balance

Pavement balance may be described as the relative relationship between the load bearing properties of adjacent pavement layers. When the characteristics of the layers are such that a gradual change in these properties throughout the pavement structure is present, resulting in relatively low stress or strain concentrations, the pavement is considered to be in balance. However, when adjacent layers have vastly different stiffness characteristics, a sudden change in the load bearing properties of the layer in the pavement is present, resulting in high stress or strain concentrations. Such a pavement is considered as poorly-balanced, with the implication that the behaviour may be less predictable since the load bearing properties of an overstressed layer may change significantly.

Table 2.8: Summary of measurements most commonly used during a detailed assessment

[illegible]

Poorly balanced pavements usually contain layers which are relatively stronger or weaker in terms of the rest of the pavement. If these layers can be identified, from a DCP survey say, the potential influence of such layers on the overall behaviour of the pavement can then be assessed. Poor balance is almost invariably associated with the use of cemented material, especially for the subbase, and the stress concentrations in this relatively stiff layer can lead to breakdown (equivalent granular condition).

The concept of pavement depth is also useful in assessing likely behaviour. Pavements identified as shallow structures have most of their relative strength concentrated in the top of the pavement structure and usually consist of one or two thin, strong and relatively rigid top layers and supporting layers of which the strength support declines sharply with depth. In contrast, deep pavement structures consist of a number of layers of similar strength with depth. This can be of importance in confirming the mode of failure in a pavement.

c) **Pavement State**

Pavements may go through various phases describing changes in behaviour and hence pavement condition, and usually a pavement will change from a stiff to a more flexible type of pavement. Surface deflections (or deflection bowl parameters, as discussed in Appendix A) provide the basis for this classification as given in Table 2.9.

Table 2.9: Nominal pavement state from deflection

Behaviour	Deflection(mm)*
Very stiff	< 0.2
Stiff	0.2-0.4
Flexible	0.4-0.6
Very flexible	> 0.6
* Maximum Benkelman beam deflection under standard wheel load	

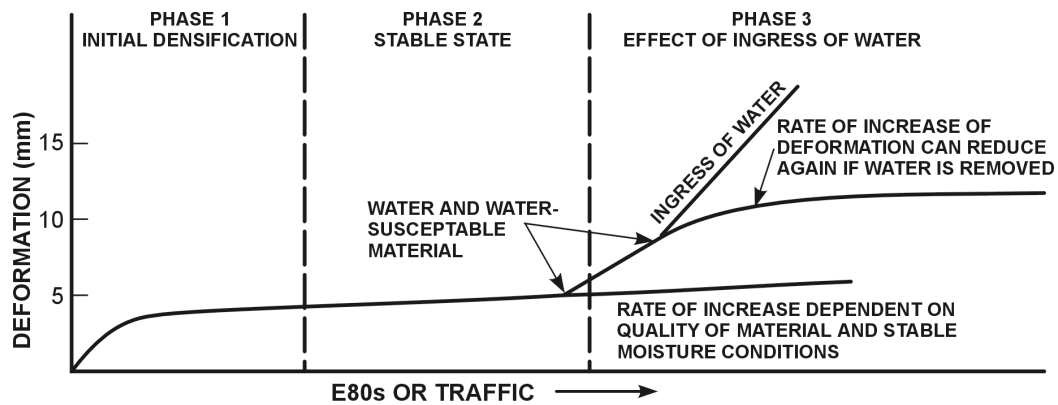
d) **Material state**

Pavement behaviour is controlled by the behaviour of the materials in the various layers. In considering general trends in the behaviour of pavements containing different pavement materials, it must be remembered that the state of the materials change with time.

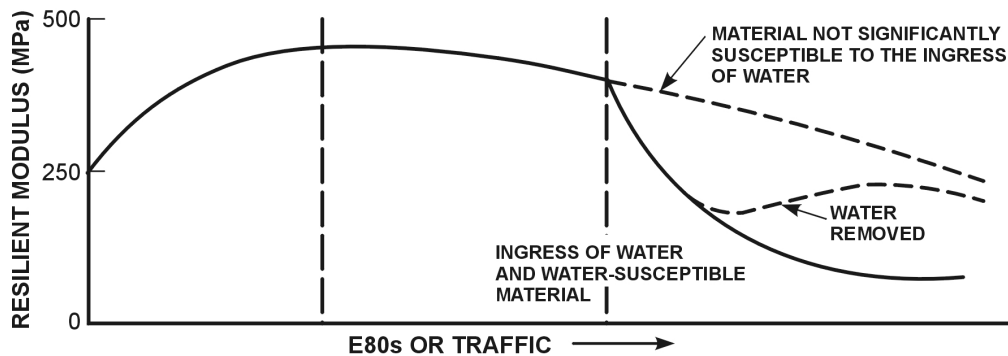
Consequently, the general trends in behaviour which are discussed refer to the original, as-built state of the material. The state of materials in a pavement under investigation may differ considerably from the original as-built state, with obvious examples being cracked or wet material.

i) **Granular pavement layers**

The general trends in behaviour of granular layers in terms of deformation and the effective dynamic modulus of the layer, are illustrated in Figure 2.10. It is seen that in the initial phase of the behaviour some deformation occurs in the wheel tracks.



CHANGE IN PERMANENT DEFORMATION



CHANGE IN EFFECTIVE DYNAMIC MODULUS

Figure 2.10: Nominal behaviour of pavement layers constructed with granular materials

This deformation is usually referred to as post-construction deformation, during which the layer further densifies under the action of traffic. It follows that the effective strength or bearing capacity of the layer may improve during this phase and an increase in the effective dynamic modulus of the layer may occur.

Bearing capacity is, inter alia, a function of density, shear strength, moisture, etc. The amount of post-construction compaction depends on the bearing capacity (density, strength) of the layer achieved during the construction of the layer, and the quality of the layer. The higher the specified level of compaction, the lower the expected initial densification.

Following a phase of initial densification (traffic moulding), the layer usually enters a stable phase during which little deformation occurs (depending on the bearing capacity).

The rate of increase in deformation during this phase depends on the initial quality of the material. The effective elastic modulus may show some change as a result of an adjustment in the balance of the pavement which may occur under the action of traffic loading. Layers with a relatively high strength within the pavement structure may de-densify, while relatively weak layers may show some increase in strength, due to densification (traffic moulding).

Untreated sub-layers usually fail when the shear strength of a layer is exceeded. Where the initial quality of the material is poor with a resultant low bearing capacity, high traffic loadings may result in quick shear failure of the layer and Phase 2 in the behaviour of the layer, as illustrated in Figure 2.10, may be very short or even non-existent.

In Phase 3, the layer shows an increase in the rate of deformation and a relatively quick decrease in its effective elastic modulus. This may be caused by an increase in moisture content (cracking of surface and ingress of water through the open cracks) resulting in a sudden decrease in bearing capacity and/or shear failure of the material.

ii) Light cementitious pavement layers

Pavement layers consisting of lightly cementitious materials may behave in two distinctly different ways. When most of the strength of the pavement is concentrated in the cement-treated layers (shallow pavement), the layer usually fails in tension. In such cases, the behaviour of the layer is similar to the behaviour of relatively strong cement-treated layers.

In cases where the strength of the pavement is distributed in depth through the pavement (deep structure), the lightly cementitious layer may fail due to crushing of the top of the layer. The general trends in behaviour of such layers in terms of pavement deformation and the effective elastic properties of the layers, are illustrated in Figure 2.11.

During the initial phase of behaviour the layer is still intact and shows little deformation and a relatively high effective elastic modulus. However, cracking may develop relatively early and the first phase of the behaviour as illustrated in Figure 2.11, may be very short, or even non-existent.

With the development of cracks at the top of the layer, the compressive strength of the layer is exceeded and crushing of the layer takes place. This phase of behaviour is characterised by a rapid reduction in the effective elastic modulus of the layer and an increase in the rate of deformation as shown in Figure 2.11.

The crumbling of the layer continues until the layer is completely broken down. The layer is then in an equivalent granular state and behaves as a granular layer.

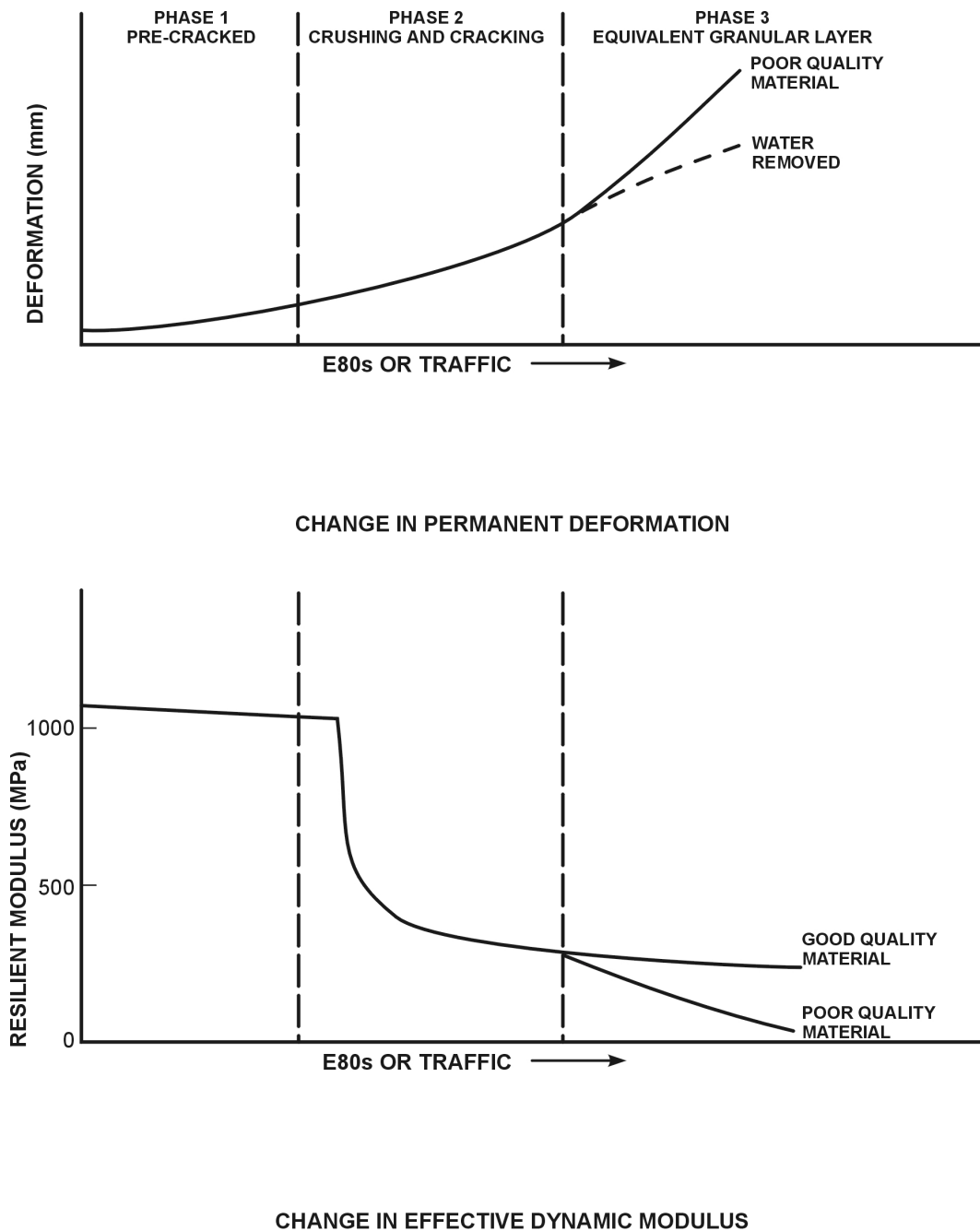


Figure 2.11: Nominal behaviour of lightly cemented pavement layers in a deep structure

lii) Cement-treated Pavement Layers

General trends of behaviour of relatively strong, cement-treated layers in terms of permanent deformation and effective elastic modulus are shown in Figure 2.10. Most of the strength of a pavement with cement-treated layers is usually concentrated in these layers. As a result, these layers usually fail in tension (fatigue).

Initially, the cement-treated layer will show virtually no increase in rut depth and the layer will have a relatively high effective elastic modulus. Typical block cracking (spacing 4-5 m), due to shrinkage, may develop early in the life of the

pavement and will most probably reflect through to the surface of the pavement if the layer is not isolated from the surface by granular layers.

In some cases, even this may not prevent the cracks from reflecting through to the surface of the road. If water is prevented from entering through these cracks, the shrinkage cracking will have no or little effect on the future behaviour of the pavement and the pavement will still be in a state of behaviour similar to the pre-cracked phase as shown in Figure 2.12.

However, as a result of fatigue from trafficking and the relative weakness of cement in tension, many cement-treated layers soon develop micro-cracks. These cracks result in a reduction of the effective elastic modulus of the layer. The layer will still appear intact (the large blocks caused by shrinkage cracking) and the rate of deformation (rutting) will still be low.

The development of micro-cracks will continue up to a point where the layer breaks down into a granular state. The breaking down of the layer usually happens relatively quickly. The layer now has little resemblance to the initial cement-treated layer. The behaviour of the layer is now similar to that of a granular layer as discussed previously. The quality of this equivalent granular layer is usually somewhat better than the quality of the virgin material used to construct the cement-treated layer.

Some variations in the behaviour of cement-treated materials may occur. Depending on the pavement composition, the point of maximum tension may not be situated at the bottom of the layer. In such cases, a crack may develop within the layer and part of the layer (usually the upper part) may break up long before the rest of the layer. In this case the top part of the cement-treated layer will form a weaker equivalent granular inter-layer.

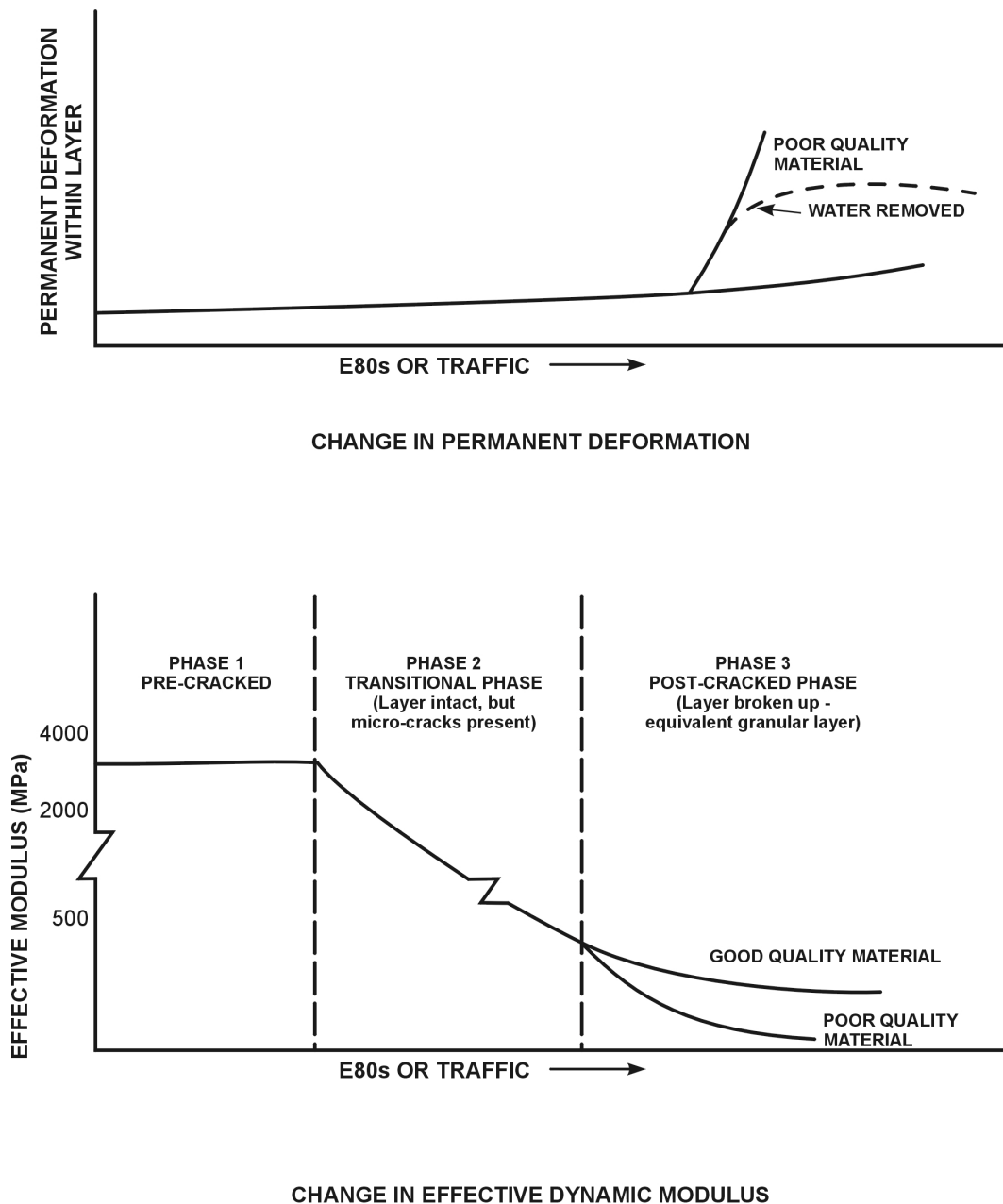


Figure 2.12: Nominal behaviour of pavement layers containing cement-treated materials

iv) Bitumen-treated pavement layers

Bitumen-treated layers are usually found at or near the surface of the pavement. Such a layer may be very thin and only acts as a protection of the lower structural layers, or it may also act as an additional structural layer of considerable thickness. Some of the layers in pavements may also be treated with bitumen and show similar characteristics to that of asphalt surfacings.

Pavement layers consisting of bitumen-treated materials (BTM) are more water resistant than layers containing cement-treated materials. Although BTM layers usually fail in tension, they are more flexible than CTMs and can, as a rule, accommodate higher deflections for the same traffic loading. However, BTM

layers are visco-elastic in behaviour and hence temperature susceptible, and may deform under high temperatures and high wheel loads.

General trends in the behaviour of bitumen-treated pavement layers in terms of permanent deformation and the effective elastic modulus of the layer are shown in Figure 2.13.

Bitumen-treated layers usually show a general increase in deformation under the action of traffic from the time of construction. The rate of deformation of the BTM layer is strongly dependent on the properties of the mix, especially the grading of the aggregate. The rate of deformation may decrease with time because of an increase in the stiffness of the binder in the layer due to ageing. However, an increase in the stiffness of the mix will make the mix more prone to fatigue and, thus, cracking.

Cracking usually starts at the bottom of the layer. However, the point of maximum strain does not always occur at the bottom of the layer and cracking may also start at a point within the layer or at the top of the layer. Low temperatures, resulting in a relatively high modulus at the top of the asphalt layer, together with the surface effect of ageing, may result in the quick development of cracking at the top of the BTM layer. Cracking will result in a decrease in the general effective modulus of the layer.

At the end of the fatigue life of the BTM layer, it will break down to a granular state which has little resemblance to the original layer. At this stage, behaviour of the layer below the surfacing is similar to that of a granular layer. The quality of the equivalent layer strongly depends on the quality of the original BTM. Similar to granular materials an initial densification will also take place before the rate of deformation reduces.

The addition of low percentages (1 to 3 per cent) of bitumen-emulsion, with or without the addition of other stabilising agents, forms the basis for the construction of emulsion-treated layers. These layers are usually constructed as sub-layers and have similar qualities to BTM layers, including:

- High resistance to water
- Highly flexible
- Tolerance towards high deflections under conditions of relatively high traffic loading without showing excessive deformation.

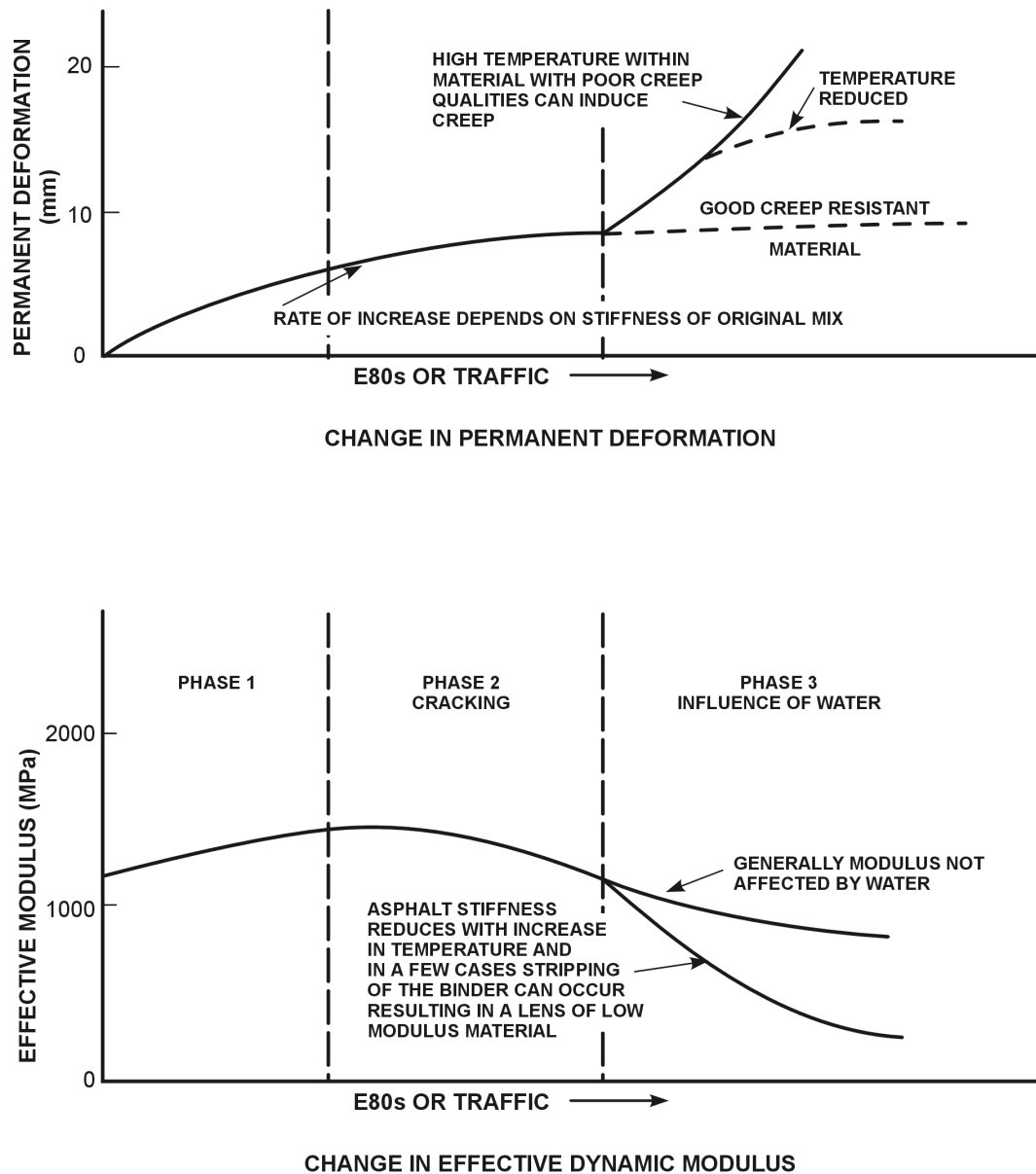


Figure 2.13: Nominal behaviour of pavement layers with bitumen-treated materials

Distress Manifestations

a) General

The mode (cracking and/or deformation) and type of distress can often indicate the need for structural strengthening. Cracking and/or deformation can originate in any of the pavement layers or subgrade.

The type of distress can provide valuable input for the identification of a problem in any uniform pavement section. Hence, the recognition of these visual indicators of problems are helpful in the condition assessment of the road.

b) Deformation

i. Deformation caused by surfacing inadequacies or problems

Premature rutting or corrugations may be caused by surfacing problems only, since under certain conditions asphalt layers may deform. In the analysis, no correlation between rutting and maximum deflection will be found and a DCP survey will show good shear strength in the base or subbase layers.

Although heavy traffic loadings, high temperatures and steep gradients associated with tight curves are contributing factors, deformation of asphalt layers is usually associated with a lack of creep resistance of the mixture.

Normally surface treatments do not deform; however, shear may take place. This is easily identified during the visual inspection. These and other unusual surfacing conditions are dealt with in TRH3¹⁰.

ii. Deformation caused by base or subbase inadequacies

Deformation of the subbase and base layers is normally associated with shear displacement or densification of the material. Displacement occurs when the shear stresses imposed by traffic exceed the shear strength of the pavement layers. Overstressing is normally attributed to layers being of inadequate quality under the prevailing conditions, and results in the heaving of the surface next to the wheel tracks. Shear or plastic deformation results in heaving of the surface adjacent to the wheel paths with associated cracking. Rutting is usually confined to the wheel path itself.

Careful observation during the visual inspection should enable base or subbase shear to be identified through these characteristic manifestations. The width of rutting gives an indication of the position of shear failure; the wider the rut, the deeper the failed layer.

A DCP survey is often useful to get an indication of the extent and position of distress in any layer(s) of the pavement, particularly if the layers are granular. For bituminous layers, creep tests may be more appropriate.

iii. Deformation caused by post-construction densification of material

Further densification of the subgrade and pavement layers can occur under the action of traffic. As in subgrade associated deformation, usually no heave occurs on the surface and rutting is quite uniform in the longitudinal direction.

The structural capacity of pavements with shear failure compared with the pavements showing signs of post-construction densification is very different. In the case of shear, the structural capacity will decrease, and deformation will increase if the pavement is not rehabilitated. In the case of post construction densification, the pavement increases its structural capacity during trafficking and tends towards a stable condition.

A DCP survey in areas between ruts may, if compared with results in the wheel path, give an indication of whether densification has occurred.

- iv. Deformation originating in the subgrade (traffic associated)
Overstressing of the subgrade by traffic loads is generally attributed to insufficient load distribution by the pavement layers because of inadequate pavement thickness and/or strength. This inadequacy should have become apparent from the preliminary structural capacity analysis.

The subgrade material deforms when its shear strength is exceeded by the stresses imposed on it, and with accumulated traffic rutting occurs in the wheel tracks. The rutting in this case is usually fairly wide, covering about half the lane width. Due to the depth of the overstressed material and the restriction of the pavement layers, heaving between the wheeltracks seldom occurs. This form of distress is often associated with cracking due to the inability of the surfacing material to deform with the rest of the pavement.

When there is substantial rutting and a general increase in rut depth (or a decrease in riding quality) associated with an increase in deflection, it indicates that the pavement structure has not adequately protected the subgrade.

- v. Deformation caused by active subgrade and collapse settlement (non-traffic associated)

Traffic loading is not the only cause of deformation of the subgrade resulting in a loss of riding quality. Deformation of the pavement layers can result from densification or collapse settlement of materials in the subgrade caused by overburden pressures and changes in moisture conditions which encourage further densification. Seasonal and long-term changes in moisture content can also cause variable shrinkage and swell. A general knowledge of the subgrade geology will prove useful and soil tests will help to identify this cause of distress.

Deformation in these cases normally takes the form of longitudinal undulations across the whole pavement width. Where environmental forces (e.g. influence of adjacent trees on subgrade moisture) are the sole cause of the distress, no rutting will be found in the wheel paths.

- vi. Deformation (other problems)
Deformation may also be caused by non-material related causes, such as moles (a problem encountered in the coastal areas of the Western Cape in South Africa and also seen in the Khalahari in Botswana), or the roots of trees.

c) Cracking as a Mode of Distress

Cracking occurs when the strain induced on a pavement layer exceeds the limiting values of the materials used. Various factors can contribute to cracking either by causing excessive strains or by lowering the maximum strain limit of the material; both aged binder and low-quality materials in the mixture can have the latter effect.

In asphalt layers the brittleness of the binder is an important factor. Binders in asphalt mixtures weather rapidly when the air-voids content in the mix exceeds four per cent (this is often the case with poorly compacted mixes). As a binder weathers, its ductility decreases and the temperature at which it becomes brittle increases. It follows that, with time, asphalts become more susceptible to cracking as their flexibility and fatigue resistance decrease. The age of the asphalt or surface treatment can therefore give an

indication of the cause of cracking. Layers with ages near the upper limit of ranges given in Table 2.10 could indicate that a brittle binder may be the cause of the cracking.

Shrinkage, temperature variations and traffic-induced stresses can cause significant cracking in treated layers. In time, these cracks reflect through the surfacing.

Table 2.10: Typical surfacing life

Base Type	Surfacing type (≤ 50 mm thickness)	Typical range of surfacing life (years)		
		Road class and typical design traffic		
		A >3 MESAs	B <10 MESAs	C, D <3 MESAs
Granular	Seals*			
	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 - 14
	Cape seal	8 - 10	10 - 12	8 - 18
	Hot-mix asphalts			
	Continuously graded asphalt	8 - 11	-	-
	Gap-graded asphalt	8 - 13	-	-
Bituminous	Seals*			
	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 - 14
	Cape seal	-	8 - 15	8 - 18
	Hot-mix asphalts			
	Continuously graded asphalt	8 - 12	8 - 12	-
	Gap-graded asphalt	8 - 14	10 - 15	-
Cemented	Seals*			
	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
	Hot-mix asphalts			
	Continuously graded asphalt	-	5 - 10	-
	Gap-graded asphalt	-	6 - 12	-
-	Surface type not normally used			
*	On top of continuous or gap-graded asphalt			

Laboratory tests on asphalt specimens are usually needed before cracking can be confidently attributed to causes such as a high voids content, poor aggregate properties in the mix, or stripping of the binder.

The most commonly observed cracking patterns are discussed below:

i. Crocodile cracking

Traffic-associated crocodile cracking normally starts in the wheel paths as short longitudinal cracks. With thin surfacings this progresses through star and map cracking to crocodile cracking. With thicker surfacings or asphalt bases, the cracks generally propagate further along the wheel path before secondary cracks form, resulting in the familiar crocodile pattern.

This cracking often occurs as a result of structural inadequacies. If the load-distributing properties of the pavement layers are inadequate for the subgrade strength, high deflections will occur under traffic loading, resulting in high tensile strains being induced in the surfacing layers. Crocodile cracking can also be caused by dry or brittle surfacing. In these cases no rutting is usually evident and cracking initially occurs between the wheel paths.

Inadequacies in the load distribution ability of the pavement structure are difficult to identify from surface inspection alone. Deflection measurements can be used to give more quantitative information on the origin of the distress. An obvious correlation between the occurrence of severe forms of crocodile cracking and high deflections is an indication of a pavement structure of inadequate thickness (care should be taken to exclude shrinkage cracking or non-traffic-associated cracking when doing this correlation). Often a DCP survey can be used to pinpoint a structurally inadequate layer.

ii. Block cracking

Block cracking is usually caused by the shrinkage of treated layers. These cracks are not confined to the wheel paths. The distress initially appears in the form of longitudinal or transverse cracks and develops into block cracking. The spacing between these block cracks largely depends on the type of material, the thickness of the pavement layer and the shrinkage characteristics of the stabilizing agent. Although traffic loading does not initiate this distress, it tends to cause secondary cracking which may eventually become crocodile cracking.

Initial cracking due to shrinkage of the stabilized layer may not affect the strength of the pavement. However, it indicates a potential lowering of the structural capacity of a pavement due to the possibility of the ingress of surface water into the structural layers and, hence, pumping and the resultant disintegration of the layer.

iii. Longitudinal cracking

Problems due to poor construction techniques are often manifested in longitudinal cracking. These include poor construction joints and segregation of materials during construction. These problems are visually easily detectable and should be identified during the detailed visual inspection. Swelling of subgrades or settlement of embankments may also cause longitudinal cracking. Such cracks

are not confined to the wheel paths and are often found near the shoulders of the pavement.

Longitudinal cracking may also be the first indication of structural problems. If this cracking occurs in or close to the wheel paths, it is likely to develop into small block cracking followed by crocodile cracks. Longitudinal cracks can thus be an indication that the pavement is approaching the end of its service life, and therefore needs further investigation.

iv. Transverse cracking

Transverse cracking is symptomatic of temperature associated distress. This cracking is thus not usually directly associated with structural problems, but it can initiate further deterioration if the ingress of surface water occurs.

Shrinkage cracking of the asphalt surfacing is indicative of an asphalt which does not respond easily to changes in temperature, i.e. mostly a stiff, dry asphalt. As stiffness tends to increase with time, older pavements are usually more susceptible to this form of cracking. The closing of these cracks in the wheel paths through the effect of trafficking is an indication that the problem may be limited to the surface layer only.

Transverse cracking as a result of the shrinkage of unbound layers is a rare phenomenon. Freezing of some unbound base and subbase materials can cause fairly high volumetric changes, resulting in transverse cracks. In time, these will be reflected through to the surface.

2.3.3 Pavement Situation Identification

Pavement rehabilitation design methods incorporate assumptions in terms of pavement materials, loading, environment and distress conditions which, in combination, are defined as the pavement situation. Hence a prerequisite for rehabilitation design is the correct identification of the pavement situation in each uniform pavement section. Aspects which influence the applicability of rehabilitation design methods are discussed in detail in Section 3.

The following is of importance in the identification of the pavement situation:

- Variables associated with pavement design:
 - The composition of the pavement in terms of the thickness of the layers and the type and properties of material in the layers
 - The current composite strength and/or component strength of the pavement structure as measured
 - Design traffic loading (past and future).
- Variables associated with pavement performance:
 - The mode of distress
 - The origin of distress within the pavement
 - The critical parameters within the pavement.

The pavement situation for each uniform section can be summarized on a form as shown in Figure 2.14.

Road number:	MR27 - SLOW LANE				
Traffic loading: past future	1.3 million ESAs				
	5 million ESAs; 10 million ESAs; 20 million ESAs				
Road category	B				
Pavement structure	Surfacing	Base	Subbase	SSG	Subgrade
Type of material	Asphalt (Cont. Graded)	Cement treated gravel	Laterite	-	In situ sand
Thickness of layers	35 mm	230mm (2x115)	200 mm	-	∞
Type of pavement	CTB - pavement				
Strength evaluation	1. Deflection		2. Radius of curvature		3. Rut depth
Instrument used	BENKELMAN BEAM		DEHLEN CURVATURE METER		STRAIGHT EDGE (2 m)
Applicable criteria: sound	< 0,3 mm		> 200 m		< 10 mm
warning	≥ 0,3 mm + < 0,6 mm		< 200 m + > 100 m		≥ 10 mm + 20 mm
severe	≥ 0,6 mm		≤ 100 m		≥ 20 mm
Measured response	Mean = 0,36 mm 90th P = 0,395 mm		Mean = 247 m 90th P = 165 m		Mean = 6,2 mm 90th P = 8,4 mm
Condition	WARNING / WARNING		SOUND / WARNING		SOUND / SOUND
Performance	Observed / Expected (Delete not applicable)				
Mode of distress	CRACKING				
Type of distress	BLOCK CRACKING + SECONDARY CRACKING IN WHEELTRACKS				
Possible origin	CTB				
Possible cause / mechanism	TENSILE STRAIN AT THE BOTTOM OF THE BASE				

Figure 2.14: Pavement situation identification

3. REHABILITATION DESIGN APPROACH AND OPTIONS

3.1 Objectives and Scope

This section deals with lengths of road that have been identified during the condition assessment as probably requiring structural improvement. The aim of this phase of the investigation is to identify appropriate rehabilitation design methods and to determine the actual rehabilitation needs using these methods.

Several pavement rehabilitation design methods are in general usage. These range from empirically-derived methods to sophisticated mechanistic methods based on multi-layer linear elasticity theory. Consequently they differ in their suitability for solving specific problems, expertise required and the cost of implementation. Furthermore, the benefits to be obtained from the use of a specific method could depend on the design traffic, type of distress, type and condition of a pavement, and the cause and mechanism of distress.

It follows, then, that rehabilitation design methods are limited in their applicability and that their indiscriminate use could seriously affect the accuracy of the rehabilitation design.

A specific rehabilitation design method may be applicable only to specific pavement material, loading, environmental and distress conditions which in combination represent specific pavement situations. The identification of possible pavement situations and the determination of the suitability of specific methods to analyse these situations could lead to improved utilisation of the methods and thus to more effective rehabilitation design. Unfortunately, the limitations of a specific method are often hidden and hence considerable experience and expertise in pavement rehabilitation is required for the effective use of the available methods.

The investigation procedure for the rehabilitation design phase is outlined in Figure 3.1. It can be seen that, depending on circumstances, further additional and/or more detailed testing may be required.

3.2 Rehabilitation Options

Before the determination of the rehabilitation needs through the use of a rehabilitation design method, the prevailing pavement situation, the pertinent managerial considerations and practical and functional aspects should be considered to identify applicable rehabilitation options for each uniform section.

As previously shown, rehabilitation may range from complete pavement reconstruction to the improvement or provision of relatively small aspects such as drainage facilities.

Any combination of the abovementioned rehabilitation activities could be applicable to a specific pavement. However, many rehabilitation design methods only provide for the design of asphalt overlays which, in many cases, may not be the most suitable option. For example, where a structural weakness is present in a pavement layer such as the base, it may be advisable to improve the quality of the layer rather than to use an overlay. Hence, before embarking on the design of the rehabilitation, applicable rehabilitation options for each uniform section should be identified.

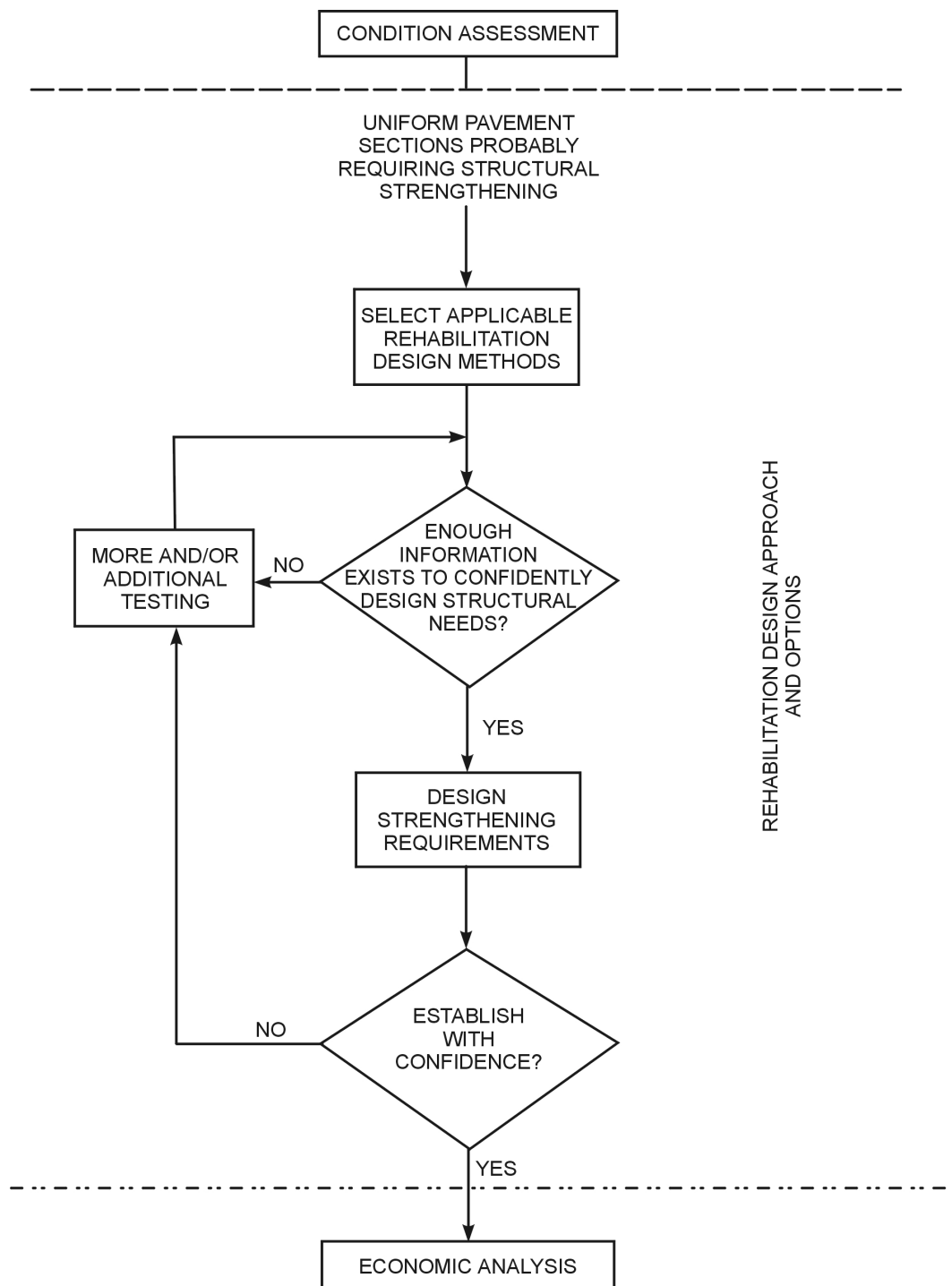


Figure 3.1: Flow diagram of the rehabilitation design phase of a project-level investigation

3.3 Method Applicability

3.3.1 General

The type and number of assumptions on which any method is based determine the limitations of its applicability. In order to select applicable methods for use on a specific pavement, the characteristics of the methods available must be compared to those of the identified pavement situation. Two tasks are important:

- The identification of the prevailing pavement situation that needs to be analysed (as described in Section 2).
- An assessment of a design method in terms of its capability to address critical aspects that may be associated with any given pavement situation.

The critical elements of the design and performance variables in a pavement are now identified as a first step towards the assessment of the applicability of methods for the analysis of a pavement situation. **If more than one design method is found to be applicable for use on a specific pavement, the methods should each be used in a multiple analysis approach for the design of the rehabilitation needs of the pavement.**

3.3.2 Design Variables

General

Design variables embody all the structural, load and environmental factors, the interaction of which primarily determine the performance of a pavement. The following elements of these variables could have an influence on the ability of a method to analyse any given pavement situation and should be used to assess the capabilities of rehabilitation design methods.

Structural Variables

Methods should be assessed for limitations with respect to the:

- Type of material in the pavement, and especially the base layers as there is significant difference in behaviour between different types (bitumen- and cement-treated, granular).
- Thickness of pavement layers.
- Strength of pavement layers.
- Strength of the pavement as a whole.

Load Variables

Methods should be assessed for limitations with respect to the design traffic loading. This is usually done in terms of the cumulative Equivalent Standard Axle loads (ESAs). Equations used in methods are derived through the assessment of the behaviour of pavements or pavement materials during loading. These observations cover a range of ESAs, defining the range of traffic loading conditions for which the method is applicable.

Environmental Variables

The ability of methods to take into account the daily and seasonal variations in climate should be noted. The daily variations in temperature are of importance during the evaluation of the pavement due to their effect on pavement test measurements. Similarly, seasonal variations in both temperature and moisture could have a marked effect on the bearing capacity of the pavement. Although these factors will not directly influence the applicability of the methods, their omission could adversely affect the results obtained through the use of the methods.

3.3.3 Performance Variables

Performance variables concern the surface condition and material behaviour elements of a pavement. These variables are usually a function of the interaction of the structural, load and environmental variables with time.

Pavement condition is described by the visible surface condition of a pavement, which is a function of any distress present. Rehabilitation design methods base their analysis of the structure and the design of rehabilitation options on limiting criteria for specific modes of

distress such as cracking and/or deformation. Critical levels of deformation and/or cracking are directly or indirectly incorporated in most rehabilitation design methods.

The modes of distress (deformation/cracking) are used in pavement rehabilitation design methods to calculate pavement life. However, deformation and/or cracking can originate in any of the pavement components, i.e. the surfacing, base, subbase, selected subgrade or subgrade. The origin of distress and the distress mechanisms are often closely associated with the material used in the various pavement layers of the structure. For example, in the mechanistic design method, the parameters shown in Table 3.1 may be important.

Table 3.1: Critical parameters and properties associated with various material types

Material Type	Distress	Critical Distress Parameters
Thin hot-mix surfacing (20 - 75 mm)	Fatigue cracking	Horizontal tensile strain at bottom of the layer (ϵ_r)
Thick asphalt bases	Fatigue cracking	Horizontal tensile strain (ϵ_r)
	Deformation	Stress state in asphalt
Granular layers	Deformation	Stress state (σ_1, σ_2) in middle of layer
Cemented layers	Shrinkage cracking	Horizontal tensile strain (ϵ_r)
	Fatigue cracking (slab state)	Horizontal tensile strain (ϵ_r)
	Deformation (in the equivalent granular state)	Stress state (σ_1, σ_2) in middle of layer
Subgrade	Deformation	Vertical compressive strain (ϵ_v) at top of layer

Rehabilitation design methods are only applicable for use on pavements where the mechanism of distress is similar to that on which the method is based. It follows that rehabilitation design methods should be assessed in terms of the following performance variables:

- The mode(s) of distress (deformation, cracking) and the respective criteria used to define a terminal pavement condition.
- The mechanism of distress and the critical distress parameters used in the analysis of the mode of distress.

Similar to the design variables, the elements of the performance variables identified above should be used to assess the capabilities of rehabilitation design methods.

3.4 Pavement Rehabilitation Design Methods

3.4.1 Available Approaches

General

Pavement rehabilitation design methods may be based on empirical or theoretical principles. In practice, methods often include both theoretical and empirical concepts and can therefore be associated with assumptions and limitations inherent in these concepts. An understanding of the general characteristics and the influence of assumptions on the prediction of pavement behaviour is fundamental to the assessment and rating of available methods.

Methods Based on Empirical Concepts

Empirically-derived methods are based on the results of observations made or experience gained in trends of behaviour of pavement structures. These observations are usually limited to specific types of pavement constructed with materials peculiar to certain areas and subject to specific traffic and environmental conditions. The experience gained under such conditions cannot confidently be applied to different pavements, materials, traffic and environmental conditions.

Methods based on empirical principles can be divided into methods based on the condition assessment approach, pavement component analysis approach and response analysis approach. The main characteristics of these approaches are summarized in Figure 3.2.

a) Condition Assessment Approach

Methods using the condition assessment approach rely on the knowledge and experience of the assessor to evaluate the condition of the pavement. Decisions about the need for and type of rehabilitation are based on subjective judgment. The assessor applies personal knowledge of the expected behaviour of the pavement, taking into account the type of pavement, materials, distress, traffic, drainage facilities, geology, topography, vegetation, climate and season within an area as the main input into his design framework. A typical flow diagram of rehabilitation design methods based on the condition assessment approach is shown in Figure 3.3.

b) Pavement Component Analysis Approach

Methods using the pavement component analysis approach are based on empirical correlations between material tests and expected pavement performance. These methods use laboratory or in situ measurement of an empirically defined material property, such as the California Bearing Ratio (CBR) or Dynamic Cone Penetrometer (DCP) value, to evaluate pavement behaviour. A typical flow diagram of rehabilitation design methods based on the component analysis approach is shown in Figure 3.4.

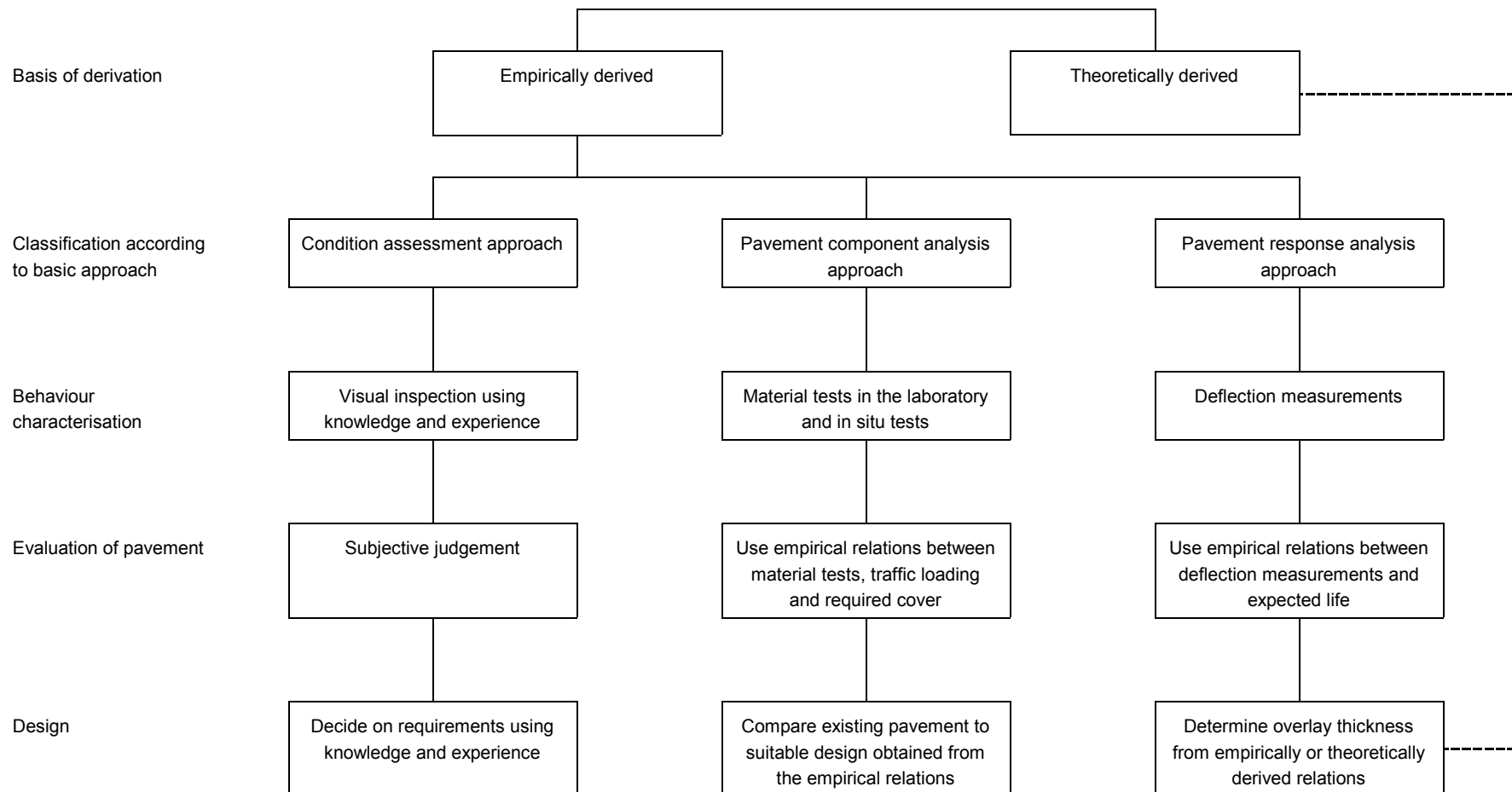


Figure 3.2: Main characteristics of the different empirically-derived approaches to rehabilitation design

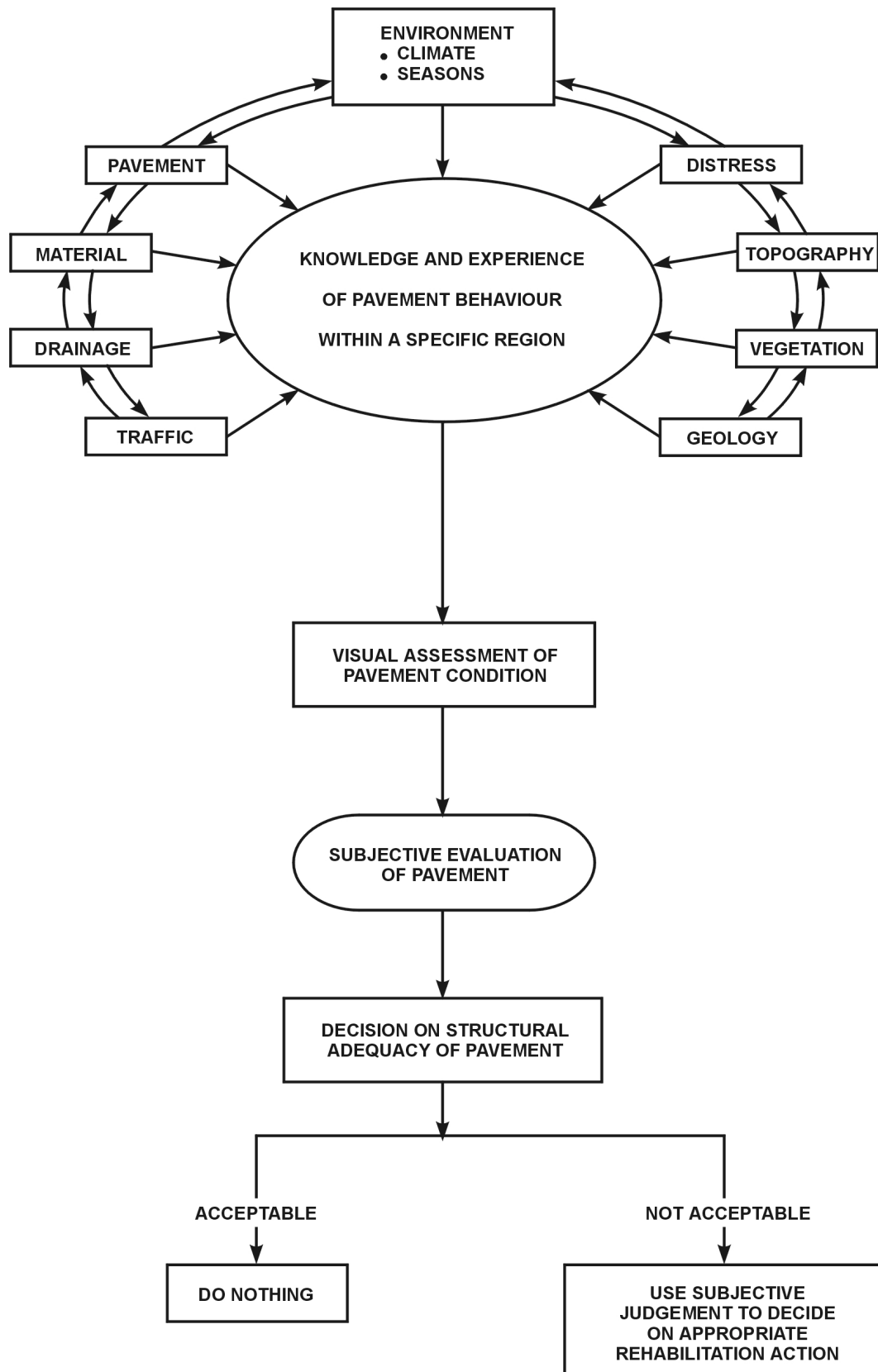


Figure 3.3: Empirical rehabilitation design methods based on condition assessment approach

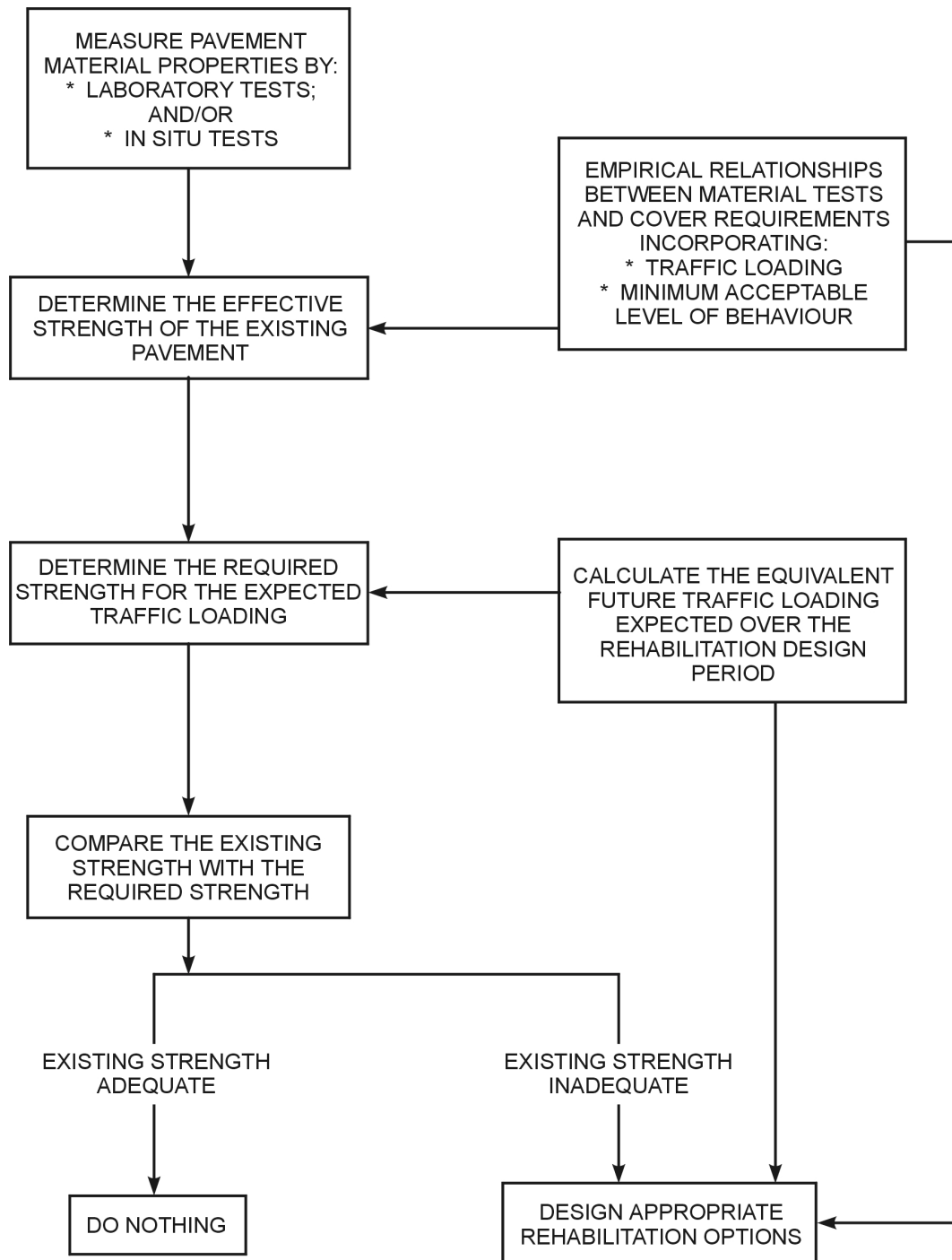


Figure 3.4: Empirical rehabilitation design methods based on component analysis approach

c) Pavement Response Analysis Approach

Methods using the response analysis approach are based on one or more empirically-derived relationships which are used to predict pavement behaviour. Most of these methods use surface deflection as a measurement of pavement response. A typical flow diagram of rehabilitation design methods based on the response analysis approach is shown in Figure 3.5.

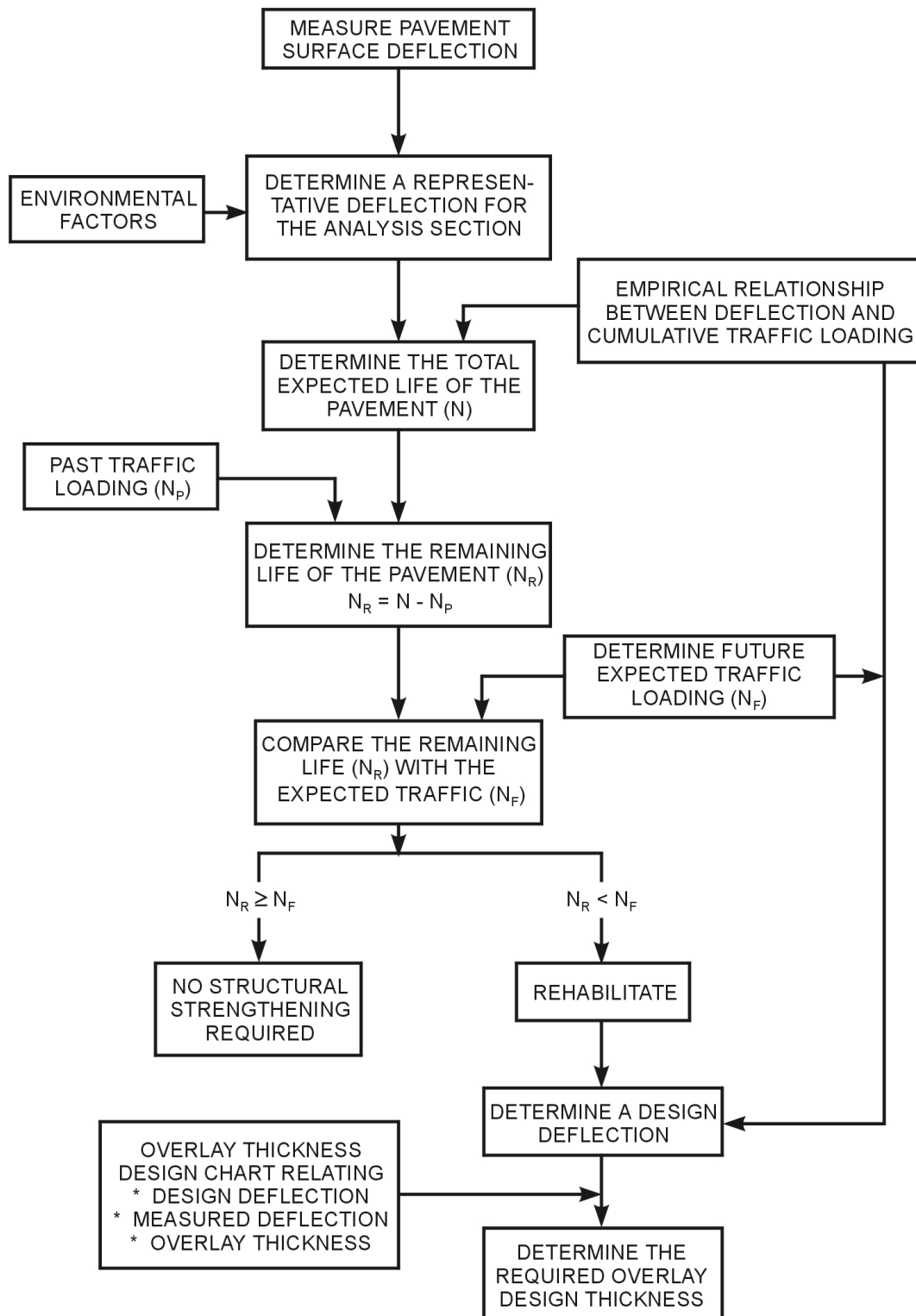


Figure 3.5: Empirical rehabilitation design methods based on the response analysis approach

Methods Based on Theoretical Concepts

Most of the theoretically-based pavement rehabilitation design methods that have been developed to a stage where practical implementation is possible, have been based on linear elasticity theory. Hence, this document will only discuss methods based on approaches using the linear elastic stress-strain theory (also termed "non-hereditary elastic response models").

A mathematical model usually gives an approximation of actual behaviour. The limitations associated with any one model depend on the type of assumptions that form its basis. Therefore, an understanding of the assumptions associated with the linear elastic model would also provide an indication of its limitations.

Rehabilitation design methods incorporating theoretical concepts are based on a number of approaches developed to simplify the use of mathematical models. The various methods are categorised according to the basic approach followed. The approaches usually give a good indication of the level of sophistication and the general applicability of the methods using the approach. Approaches identified are:

- Empirical-theoretical approach
- Behaviour catalogue approach
- Design curves approach
- Design charts approach
- Non-simplified approach.

The main characteristics of the approaches developed from the linear elasticity theory are summarized in Figure 3.6.

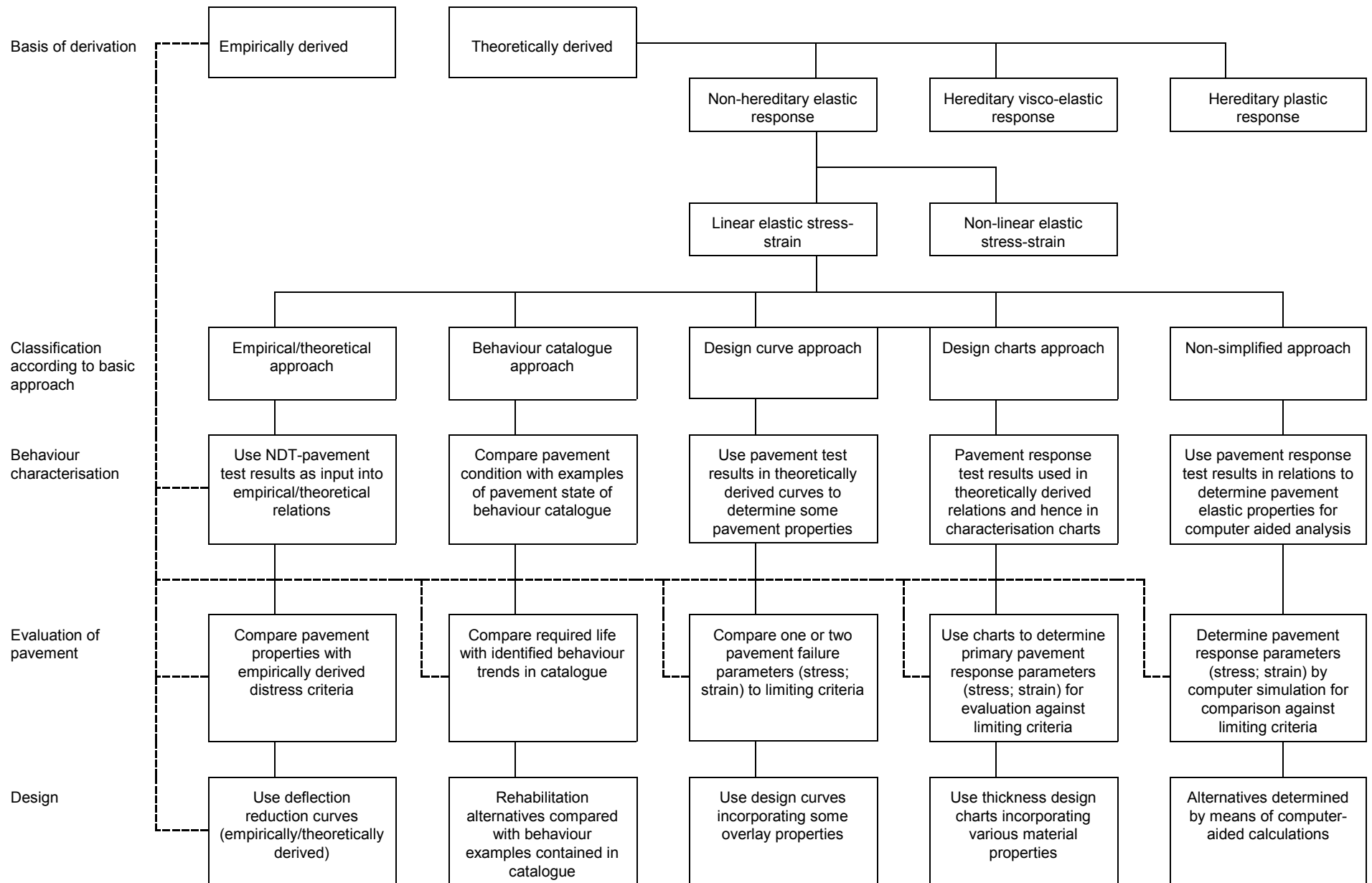


Figure 3.6: Main characteristics of some approaches to pavement rehabilitation design based on linear elastic stress-strain theory

3.4.2 Rehabilitation Design Methods Used in Southern Africa

General

The selection of methods according to the recommended approach will ensure that an applicable method is used for the rehabilitation design of each uniform pavement section. Rehabilitation design methods are not discussed in detail in this document, but the main characteristics of some of these are summarized in Appendix B. The recommended methods are the:

- Asphalt Institute method⁹
- DCP method⁸
- TRL deflection method⁶,
- Shell overlay design method¹¹
- South African mechanistic design method¹².

Various methods, including those discussed in Appendix B have been assessed^{13,14} both theoretically and practically for a limited number of pavements in southern Africa. Based on this assessment some recommendations are made with regard to the use of rehabilitation design methods in southern Africa.

Empirically-derived Methods

Some of the abovementioned methods are based on the same approach (refer to Appendix B) and the recommendations regarding the further development and use of these methods concern more than one method. Hence, these methods are discussed under the heading of the basic approach by which they are classified.

The design curves of the empirically-derived methods considered are based on deformation only and contain many other limitations (refer to Appendix B). Hence, it is recommended that the use of these methods be limited to low category pavements with low traffic volumes. Results obtained through the use of these methods on other pavements should be verified through the use of a theoretically-based method.

a) Response Analysis Approach

The Asphalt Institute and the TRL deflection methods are based on the response analysis approach. Deflection-based methods have severe limitations for the analysis of pavements containing semi-rigid cemented layers. Hence these methods are not recommended for the analysis of pavements containing cement-treated layers.

i. Predicting Remaining Life

It is important to take the type of pavement into account and hence the design curves of the TRL deflection method are recommended to determine the remaining life of the pavement. These curves should be used with the deflection measurements processed as recommended in Appendix B, taking into account the various pavement categories.

It has been shown^{7,14} that results obtained through the use of the TRL curves are adversely affected by the use of the temperature adjustment figures (micro climate), as recommended in the TRL deflection method. Similar trends were found with the use of the temperature adjustment figure recommended for use by the Asphalt Institute method.

Hence, it is recommended that the curves for the prediction of remaining life be used without the adjustments of deflection measurements to allow for the effect of temperature, until such required adjustments have been studied in more detail under southern African conditions.

Currently specific information on the adjustment of deflections to take into account seasonal variations (macro-climate) are not available for southern Africa. It is recommended that such adjustments be incorporated in the method based on the response analysis approach, after a study to determine their effect has been undertaken in southern Africa.

ii. Rehabilitation Design

The methods based on deflection measurements use overlay design curves to determine the rehabilitation needs of pavements. The cause and mechanism of distress are important when deciding on the rehabilitation needs. Hence the use of overlay design curves should be limited to pavements where the cause and mechanism of distress is associated with the subgrade.

b) Pavement component analysis approach

The Asphalt Institute and the DCP methods are based on pavement component analysis approach.

i. DCP Measurements

The DCP measuring device cannot be used on pavements with strongly cemented layers. However, limited studies have shown that the DCP method can successfully be used on pavements containing thick bituminous treated layers.

The DCP method, as analysed in Appendix B, is recommended for use on balanced or nearly-balanced pavements containing bituminous treated, lightly cemented ($UCS < 3 \text{ MPa}$) and granular layers. The effect of seasonal changes could be incorporated to improve the accuracy of long term predictions.

ii. Analysis of Pavement Layers

The Asphalt Institute method (refer to Appendix B) has been shown to give reasonably accurate predictions of remaining life, but should not be used to determine rehabilitation requirements. It is recommended that this procedure be used for the prediction of remaining life only, and that the results be compared to DCP and/or deflection based methods.

Theoretically-derived Methods

Methods developed overseas are usually based on distress criteria which are not generally applicable for use in southern Africa^{7,14}, so it follows that methods and procedures which were not developed in southern Africa should not be used before these have been verified under local conditions.

Furthermore, the assumptions incorporated in the methods based on the empirical/theoretical approach, the design curves approach and the design charts approach, make these methods unreliable for the analysis of complicated problems. Hence, it is recommended that methods based on these approaches should not be used for the analysis of pavements.

The only theoretically-based methods that can, with confidence, be recommended for the detailed analysis of pavements are those based on the non-simplified approach. The South African mechanistic design method is based on the non-simplified approach and is recommended for use.

3.4.3 Recommendations

Three methods for pavement rehabilitation design are recommended for use in southern Africa. These are a deflection method, DCP method and the South African mechanistic design method. These methods are assessed in terms of their limitations and abilities with regard to the various design and performance variables in Figures 3.7 and 3.8 respectively.

In order to assist in the selection of an appropriate rehabilitation design method, the use of the recommended three methods in terms of the main design variables are shown in Figure 3.9. Note that Figure 3.9 should be used together with Figures 3.7 and 3.8 before a final selection of an applicable method is made (see Footnote¹).

¹ The South African Department of Transport through its research programme in the Directorate: Transport Economic Analysis, has commissioned the completion of three documents for the rehabilitation design of pavements for use in Southern Africa. These are:
Department of Transport Research Report 91/243: "Pavement Rehabilitation Design Based on Maximum Surface Deflection Measurements",
Department of Transport Research Report 91/241: "Pavement Rehabilitation Design Based on Pavement Layer Component Tests (CBR and DCP)", and
Department of Transport Research Report 91/242: "The South African Mechanistic Pavement Rehabilitation Design Method".

Rehabilitation design method	Limitations in structural variables			Accumulated traffic load	Climate (incorporate functions for)	
	Type of pavement	Thickness	Component strength		Micro-climate	Macro-climate
Deflection method	Cemented base	n/a	n/a	Remaining life: 0.4 - 10 million ESAs Overlay design: 0.2 - 20 million ESAs	No	No
	Bituminous base	-	-			
	Lightly cemented base	-	-			
	Granular base	-	-			
DCP Method	Cemented base	n/a	n/a	0.2 - 10 million ESAs	No	Partial
	Bituminous base	-	-			
	Lightly cemented base	-	UCS < 3 MPa			
	Granular base	-	-			
South African mechanistic method	Cemented base	-	-	0.01 - 100 million ESAs	No	Partial
	Bituminous base	-	-			
	Lightly cemented base	-	-			
	Granular base	-	-			

- n/a Method not applicable for the analysis of pavement type
 ? Uncertainty
 - No restrictions

Figure 3.7: Applicability of rehabilitation design methods in relation to the main design variables in a pavement situation^{13,14}

Mode of distress and criteria used to define the pavement failure condition			Critical distress parameter	Pavement component										Origin, cause and mechanism of distress taken into account in design of options?				
				Material type		DEFORMATION						CRACKING						
Rehab design method	Deformation (rutting)	Cracking	Stability stress state	Surface	Pavement layers			SSG + subgrade	Composite structure	Surface		Pavement layers						
				Bituminous	Bituminous	Granular or equivalent granular		Granular	-	Bituminous	Bituminous		Cemented					
					Stress state (σ_1 σ_2)	Vertical strain (ϵ_v)	Vertical strain (ϵ_v)	Stress and or strain in layers	Bottom layer	In layer	Bottom layer	In layer	Bottom layer	In layer				
Deflection method	20 mm	X	X	X	X	X	✓	✓							No			
DCP method	20 mm	X	?	?	✓	X	X	✓							Yes			
South African mechanistic method	8 mm or 12 mm or 18 mm	✓	X	X	✓	✓	✓	X	✓	With recom	✓	With recom	✓	With recom	Yes			

-  Method not applicable for the analysis of the mode of distress
 ? Uncertainty
 ✓ Takes into account
 X Does not take into account

Figure 3.8: Applicability of rehabilitation design methods in relation to the main performance variables in a pavement situation^{13,14}

Category of Pavement	Typical Traffic Class	Type of Pavement	Method* Recommended	Verification
A	> 10 million ESAs	Cemented base	3	-
		Bituminous base	3	(1/2)
		Lightly cemented base	3	(1/2)
		Granular base	3	(1/2)
B	Up to 10 million ESAs	Cemented base	3	-
		Bituminous base	3	1/2
		Lightly cemented base	3	1/2
		Granular base	3	1/2
	Up to 3 million ESAs	Cemented base	3	-
		Bituminous base	1/2/3	2/1
		Lightly cemented base	1/2/3	2/1
		Granular base	1/2/3	2/1
C; D	Less than 3 million ESAs	Cemented base	3	-
		Bituminous base	1/2/3	1/2
		Lightly cemented base	1/2/3	1/2
		Granular base	1/2/3	1/2
() = use with reservation				
* 1 = Deflection method 2 = DCP method 3 = South African mechanistic method				

Figure 3.9: Guidelines for the use of the recommended rehabilitation design methods^{13,14}

4. PRACTICAL AND FUNCTIONAL ASPECTS

4.1 Introduction

The practical and functional aspects discussed here include the applicability, constructability, performance (structural and functional) and maintainability of methods, materials and procedures. These aspects differ considerably from project to project and the designer requires detailed knowledge of each project and experience in pavement rehabilitation design in general to effectively take these into account.

This section covers some of the more common practical and functional aspects that should be considered in pavement rehabilitation design. The omission of these aspects could easily result in an otherwise technical sound design being unsuccessful. Practical and functional aspects influencing rehabilitation design should not be seen in isolation and this section should be considered together with the rest of the document.

4.2 Applicability

4.2.1 General

Many procedures, methods and other aspects may not be applicable to specific project conditions, and the applicability should be determined before undertaking rehabilitation design. In this discussion, "applicability" could refer to the use of methods of investigation and design, material selection, modification and application, and most important, the practicality of available rehabilitation options.

4.2.2 Methods of Investigation and Design

Methods of investigation and design and their applicability to specific projects are discussed in detail in Sections 2 and 3. These sections and accompanying references should be studied to ensure that applicable tests and methods are being used on a particular project.

4.2.3 Materials

The availability of materials has been identified in Section 1 as a logistical input which could influence pavement rehabilitation design. The available materials influence rehabilitation design in that they may only be suitable for use in particular pavement layer types. The characteristics of the materials may be such that strengthening or modification may be required, or in some cases the available materials may not be suitable for modification or strengthening. Hence the rehabilitation design should ensure that the available materials can be used as specified.

4.2.4 Rehabilitation Options

Several rehabilitation options or strategies could normally be used to rectify a distressed pavement. However, it is often found that some options are more suitable than others, and the designer could save considerable effort and time by concentrating on these options. The suitability of rehabilitation options is often a function of the characteristics of the pavement, such as the cause and mechanism of distress and the pavement situation. For example, if pavement deformation has been identified as originating in the asphalt surfacing of a pavement, it could be more applicable to replace the existing surfacing with a deformation-resistant surfacing than to cover the deformed layer with another asphalt layer.

4.3 Constructability

4.3.1 General

Although many designs may be applicable, or certain alterations desirable, the actual construction thereof could present major obstacles. These could include aspects such as the accommodation of traffic during rehabilitation, the widening of the current paved surface, the availability of equipment and the influence of the rehabilitation action on the environment.

4.3.2 Accommodation of Traffic

The accommodation of traffic has been identified as a logistical input into rehabilitation design in Section 1, which is influenced by the type of terrain and area in which the road is to be constructed. Traffic accommodation could be the decisive factor in the selection of a specific rehabilitation option and requires consideration from the outset of a project as it could account for ten to twenty per cent of the total costs of the project.

4.3.3 Widening of the Paved Surface

Some sub-standard roads may technically require widening of the paved surface. However, where little extra width may be required the constructability and compatibility with the existing pavement should be kept in mind. Because of confined space, it could be difficult to work to tight specifications, making widening very expensive and hence not cost-effective. Where widening is essential, the improvement of existing pavement layers or the additional surfacing of the shoulders at little extra cost should be investigated. Widening should be planned so that the joint does not coincide with one of the wheeltracks.

Widening of the road should also take into account the drainage capacity of the existing and new pavement layers. If the additional width is not compatible with the existing pavement it could result in the trapping of water and hence, early failure.

4.3.4 Layer Thickness Constraints

Existing surface irregularities, geometry and existing layer thicknesses should be taken into consideration in the final design of layer thicknesses. It is considered essential to take cross sections of the existing road before final design documents are prepared to ensure that the required rehabilitation needs are met, allowing for existing irregularities.

4.3.5 Availability of Resources

In practice, rehabilitation investigations and design should take into account the availability of specialized equipment. This aspect has been discussed under logistical inputs to rehabilitation design in Section 1 of this document.

4.3.6 Environmental Influence

The rehabilitation of a pavement could cause considerable environmental pollution which could play an important role in the selection of a rehabilitation option. For example, dust pollution (granular materials) or smoke pollution (hot-mix recycling) could exclude some options for use in built-up or sensitive areas. On the other hand, the creation of large quantities of waste material is often not desirable, prompting the investigation of options which allow for the re-use of materials.

4.4 Performance Adequacy

4.4.1 General

The performance of the rehabilitated pavement is often associated with only the structural adequacy of the design. These structural aspects are fully covered by the approach to rehabilitation design in Sections 2 and 3 in this document. However, both the structural and functional performance of the pavement are influenced by additional aspects such as the adequacy of drainage facilities and the type of surfacing or wearing course. These aspects should also be taken into consideration during the design of rehabilitation options.

4.4.2 Drainage

General

Pavement distress can often be related to the lack of drainage facilities, their ineffectiveness or inadequacy. A detailed survey of the adequacy and performance of the existing drainage facilities and recommendations for their improvement where required, are considered an essential part of a rehabilitation investigation. Drainage improvements should receive the highest priority, and should include surface drainage, side drains, subsurface drainage and cross drainage.

Surface Drainage

The rehabilitated pavement surface should allow for the free flow of rain water from the paved surface through the provision of an adequate side slope.

Side Drains

Blocked and overgrown side drains are ineffective, often resulting in the ponding of water. Concrete side-drains should be considered in cases of erosion or obvious blockage or in cuttings. The placement and influence of concrete side drains on safety, maintenance and future pavement rehabilitation should be taken into account in their design.

Sub-surface Drainage

Sub-surface drains are relatively expensive and, if required, should be thoroughly investigated and motivated. These would usually be appropriate in cuttings where localized pavement failures have occurred.

Cross Drainage

Additional cross drains should only be considered in cases where regular flooding of the road occurs. It is important to improve existing ineffective cross-drains to ensure that they will function properly in the future.

4.4.3 Surfacing and Wearing Course Selection

Several functional and practical aspects should be considered in the selection and design of the surfacing or wearing course. The design of asphalt surfacings is covered in detail in TRH8¹⁵, where aspects such as deformation and fatigue characteristics are discussed. Similarly, surface treatments and the design thereof are covered in detail in TRH3¹⁰. These documents therefore provide good insight into the selection and design of the surfacing or wearing course where local guidance is unavailable.

In addition to the structural qualities of the surfacing or wearing course, functional aspects such as riding quality, skid resistance, traffic noise and even optical (i.e. light reflections) properties should be considered.

4.5 Maintainability

Pavements tend to have a long lifespan which could include several maintenance and rehabilitation cycles. Hence, in the selection of the type of rehabilitation of a pavement, the future maintenance or even rehabilitation of the pavement should be considered. It is clearly unwise to invest in an asset (pavement) if the future availability or quality of maintenance is at risk.

With the possible use of innovative, modified or new materials especially, the availability of material, equipment, expertise and funds for the maintenance of the pavement in the future need to be considered.

5. ECONOMIC ANALYSIS

5.1 Introduction

Economic analysis is an aid in the decision-taking process, providing an important basis for sound and objective decision-taking. Economic analysis determines the cost-benefit ratio and the Economic Internal Rate of Return (EIRR) of a project to justify its commission relative to competing capital projects. It is usually done prior to the awarding of contracts for project-level rehabilitation investigations as part of the prioritisation of rehabilitation projects, and is normally obligatory when funding from donor agencies is sought.

This type of detailed analysis should be undertaken by an experienced economist, and is outside the scope of this document, although the underlying principles are outlined here.

In the case of comparing rehabilitation options for the same project, it is not usually necessary to undertake a full analysis leading to an EIRR value. Rather, the objective of the economic analysis as part of a project-level rehabilitation investigation procedure is to determine the most economical of the appropriate remedial options.

This is a more simplified process, since a number of the parameters are the same in each case, and the basis for comparison is total Present Worth of Costs (PWOC). These are made up from agency costs and road user costs:

- i. Agency Costs:
 - Initial rehabilitation costs (including costs for traffic accommodation)
 - Maintenance costs during the analysis period (including administration costs)
 - Future capital costs (e.g. stage remedial action).
 - Salvage value at the end of the analysis period.
- ii. Road User Costs:
 - Delay costs due to rehabilitation activities.
 - Vehicle operating costs.
 - Accident costs.
 - Time costs.

The various rehabilitation options are compared by calculating the total present worth of the costs (PWOC) of the various items listed above for each option. Ideally, this comparison should incorporate for each option the probability of different outcomes and resultant actions during the analysis period to obtain the most probable cost of each option. This can be facilitated by the use of decision trees and Bayesian analysis, and this approach is demonstrated in Appendix D.

The recommended procedure for the economic analysis of rehabilitation options for a specific project is outlined in Figure 5.1.

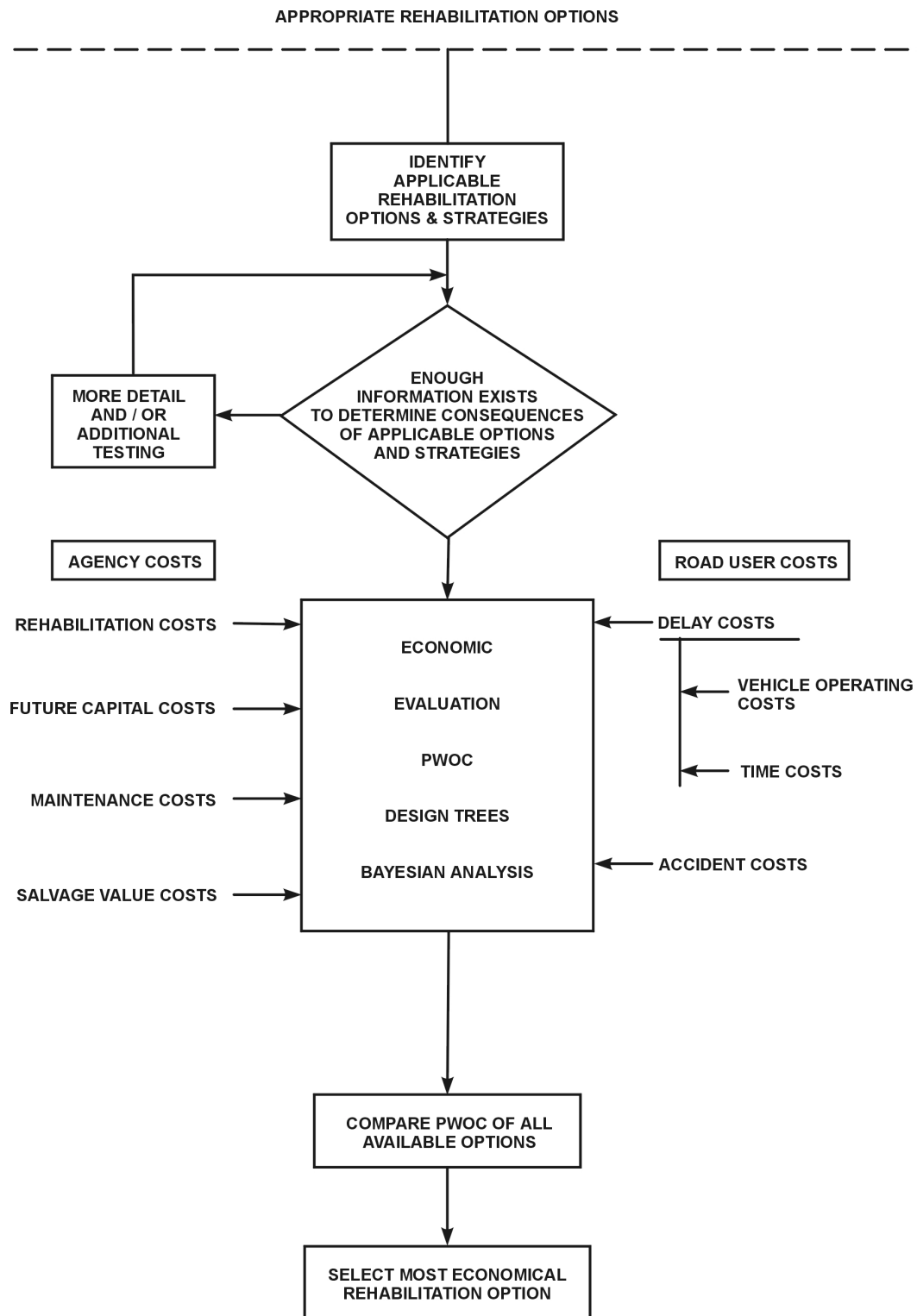


Figure 5.1: Economic analysis as part of project level pavement rehabilitation design

5.2 Agency costs

5.2.1 Rehabilitation costs

This cost item is the **initial cost** of the rehabilitation of the section of road for the option under investigation, and should reflect the probable contract cost if the project is to be let. Current unit costs, which should include all relevant capital cost items associated with the rehabilitation construction (such as the costs of traffic accommodation), must be used in the calculations.

5.2.2 Maintenance Costs

Maintenance costs expected during the design period should be included in the economic analysis. These will include costs involved in the regular inspection of the pavement, administration costs and repair costs due to natural ageing and weathering of the pavement, e.g. restoring the skid resistance and the sealing of cracks.

Many of these costs are difficult to assess since rehabilitation options often react differently with time, due to the susceptibility of certain materials to ageing, cracking or weathering. Therefore, the economic analysis should provide for the likelihood that these additional costs will occur. These costs are included as current unit costs.

Table 2.10 gives an indication of the expected life of pavement surfacings under normal conditions, after which maintenance is usually required, and this can be used where local information is unavailable to determine periodic maintenance costs (resealing/resurfacing).

5.2.3 Future Capital Costs

This cost item includes all major capital costs foreseen during the rehabilitation design period as a result of the adoption of a specific rehabilitation option. These costs occur when, for monetary or political reasons, a holding action and future interim rehabilitation actions are considered, or when stage rehabilitation construction is considered. Current unit costs are used.

This cost item also includes the cost of major rehabilitation because of distress or lack of maintenance. Provision should be made for interim rehabilitation, taking into account the category of road, the policy of the road authority and the rehabilitated pavement structure.

5.2.4 Residual Value (Salvage Value)

The value of the pavement at the end of the analysis period is dependent on various factors, including the:

- Anticipated uses of the material or pavement at the end of the analysis period, such as:
 - Recycling of the material.
 - Removing and selling of the material.
 - Rehabilitation of the pavement and hence re-use of the material as a foundation for a new road.
 - Abandoning of the road with or without restoring the original vegetation.
 - Combinations of the above.
- Volume, type, age and expected life of the material.
- Market value of the pavement or material at the end of the analysis period.

The salvage value is often difficult to assess due to the various unknown factors. However, due to the length of the analysis period and the discount rate, small differences in salvage value estimates usually have little influence on the final result.

The obvious method for calculating the salvage value is to determine the value of the material at the end of the analysis period. This may be the difference between the costs of new material and costs of using the existing material on the road. For example, the salvage value of asphalt overlay can be determined by using a straight line depreciation (Equation 7), where:

$$\text{Salvage value} = \left(1 - \frac{A}{B} \right) \text{Cost of new overlay} \quad \dots \text{Equation 7}$$

with

- A = age of the overlay
- B = expected life of the overlay.

However, the above method can be misleading if it is expected that the road will have to be rehabilitated again at the end of the analysis period. The fact that the road has already been established and is carrying traffic is worth a considerable amount and this could be a major contribution to the salvage value of a road. In these cases the difference between the cost of constructing a new road and the cost of rehabilitating and/or upgrading the existing road would best represent the salvage value of the pavement.

5.3 Road User Costs

5.3.1 General

Road user costs, when taken over the design period, will usually far exceed the agency costs. They are, however, difficult to determine with any level of confidence and hence detailed economic analyses require specialist economist input. In the case of comparing rehabilitation options for the same road, however, certain conditions are the same which simplifies the calculation.

In certain cases, differences in road user costs between different options for the same project will, in effect, be minimal and may be disregarded in a comparative study if taken over the same analysis period under the same traffic conditions.

However, road user costs cannot be disregarded completely in comparative studies since alternative remedial options may require different construction techniques and different construction periods. This will result in differences in the disruption of traffic during the actual rehabilitation of the pavement and, hence, differences in the delay costs and accident costs to the road user.

Under heavy traffic conditions these costs may contribute significantly to the total cost of some remedial options and should be taken into account. In these studies, vehicle operating costs and time costs are only of importance when calculated as part of the delay costs incurred during the actual rehabilitation of the road.

5.3.2 Delay Costs Due to Rehabilitation Activities

General

This cost item can be significant, especially when a heavily trafficked road such as an inter-urban freeway is rehabilitated. Only the differences between the costs of the various options need to be considered.

If all the available options disrupt the flow of traffic for the same period of time and to the same extent, this cost item will be the same for all the alternatives and may therefore be disregarded. However comparing, for example, an overlay and reconstruction, differences in both the degree of disruption and the rehabilitation construction time will occur. In this case, delay costs may prove to be the decisive factor in selecting a rehabilitation option.

Delay costs, as a result of longer time spent on the road, the slower speeds and the more congested traffic conditions, include time costs, vehicle operating costs and accident costs. The cumulative effect of all these contributory items should be included in a thorough study.

Vehicle Operating Costs (VOC)

Vehicle operating costs comprise the following components:

- Vehicle capital cost
- Vehicle maintenance cost
- Fuel costs
- Oil costs
- Tyre costs

Some of the components of vehicle operating costs, such as depreciation costs, are not applicable when delay costs due to rehabilitation activities are calculated. Each component therefore requires careful consideration and should not be blindly incorporated into any analysis.

Many road departments try to monitor VOC in order that full economic analyses can be undertaken on different projects but, as indicated above, the application and interpretation will normally be undertaken by an experienced economist. In such cases, differences in terrain and in deterioration of different roads, lead to significant differences in VOC.

Fortunately this cost item can usually be disregarded in the comparative economic analysis of rehabilitation works for a particular road because it is usually the same for the different options. This is because the various options tend to meet similar design requirements, e.g. the provision of an acceptable service on the same route over the same design period.

Time Costs

This cost item should reflect the economic cost of time spent by road users on the different rehabilitation options. For practical purposes in comparing rehabilitation options on the same road section, it can be reduced to the differences in time lost due to delays caused by rehabilitation activities (the cost to the economy). It is, however, by far the most difficult to assess, and even the most experienced economists tend to simplify (reduce) the time cost component in full economic analyses.

The reason for this is that the estimate, which must answer the question "How much economic (productive) time is lost due to the rehabilitation works compared with before and what is its value?", is comprised solely of vagaries, some of which include:

- Apart from estimating the additional time spent by the road users due to delays at the site, there will generally be some knock-on or ripple effect due to, for example, road users seeking alternative routes causing delays on other roads, delayed deliveries holding up production, etc.
- How is the total additional time converted into economic (productive) time? Not all additional time (time "wasted") can be considered productive, but how can the balance be determined? Does everyone affected work?
- What should be the unit cost associated with the time value? Should it be based on national per capita income? Is the sample biased because it does not reflect the overall socio- and demographic makeup of the country?

and so on. For comparative purposes, the time costs should be ignored.

5.3.3 Accident Costs Due to Rehabilitation Activities

This cost item is determined by using accident frequency and accident unit costs statistics to estimate a cost attributable to the rehabilitation work. Taken over the rehabilitation design period, these costs may be equal for the various rehabilitation options and, in such cases, can be disregarded (assuming that all options will ensure that a minimum level of service is maintained during the rehabilitation design period).

However, during the rehabilitation construction period, different options may cause differences in traffic congestion, and therefore notional differences in the number of accidents and hence in accident costs.

In such cases some estimate of the costs and, in particular, the cost differences should be made.

5.4 Principles of the Economic Analysis

5.4.1 Present Worth of Costs (PWOC)

The different cost items discussed in the preceding sections need to be meaningfully combined into a single comparative value for each rehabilitation option. The present worth of cost (PWOC) method is commonly used for this purpose.

In this method all future costs are discounted to the present worth of costs (ie, to the worth in today's currency value) by using an acceptable discount rate. The rehabilitation option which produces the lowest expected PWOC should be selected as the most economic rehabilitation option.

The present worth of costs can be calculated from the following formula (Equation 8):

$$PWOC = P + \sum_{i=1}^n A_i (1+r)^{-i} - S_n (1+r)^{-n}$$

Term 1
Term 2
Term 3

...Equation 8

with:

PWOC	=	Present worth of costs
Where P	=	Initial rehabilitation costs

A_i	=	Relevant costs occurring during the analysis period after i years
S_n	=	Salvage value at the end of the analysis period n years later
n	=	Analysis period
r	=	Discount rate

The equation consists of three basic parts:

- Term 1 - the initial cost (including the construction cost and any other costs associated with the construction work as derived from the road user costs)
- Term 2 - summation of all periodic costs arising during the life of the road (in the analysis period), such as reseals, or additional work for a stage-construction, discounted to today's values.
- Term 3 - the salvage value of the road at the end of the analysis period, subtracted from the cost assessment as it represents a benefit to the road agency.

As noted previously in Section 5.2.4, the estimation of salvage value can be difficult and, in fact, inappropriate if the road will be in different conditions at the end of the analysis period for different options.

Since the objective of the economic analysis as part of the rehabilitation procedure is to determine the most economical rehabilitation option, the salvage value is most conveniently included in the analysis **by considering the difference in costs at the end of the analysis period, to bring all the options to the same end condition**. In such a comparison, the costs to bring all the options to the same end condition at the end of the analysis period are added rather than subtracted as Term 3.

5.4.2 Specific Aspects

Analysis Period

The analysis period is the period over which the different remedial options are compared and during which all relevant cost items are taken into account. Usually this period is taken to be the same as the rehabilitation design period. When the PWOC method is used for the economic analysis, the analysis period must be the same for all the alternative options within a specific rehabilitation study.

Inflation

When all cost items escalate at the same rate, inflation is taken into account by calculating all future expenditures at current unit costs. The real discount rate is then used to determine the PWOC of these future costs.

However, the cost of some materials may escalate at much higher or lower rates than the average inflation rate. These differences can be taken into account by applying the approach adopted in the following example.

Example

Assume that the price of asphalt will escalate at an annual rate of 15 per cent for the next 10 years, whereas the average rate of inflation is expected to be 10 per cent per annum over the same period.

The PWOC of a 50 mm asphalt overlay placed in 10 years' time is now calculated as follows, assuming a current discount rate of 8 per cent (use the figure appropriate for the country, normally readily available through national statistical data):

Current cost (1997) of one km of 50 mm asphalt overlay = CU* 50 000 (say)

* CU = currency unit

- Project today's cost at the expected price escalation rate (15 per cent per year)
 $\therefore$ the expected 2007 cost of one km of 50 mm asphalt overlay in 2007 money
 $= \text{CU } 50\,000 (1+0,15)^{10} = \text{CU } 202\,278$

- bring this cost back to today's prices by removing the inflation effect (10 per cent per year)
 The 2007 cost of a 50 mm asphalt overlay in 1997 money
 $= \text{CU } 202\,278 (1+0,10)^{-10**} = \text{CU } 77\,987$

** NB Note the negative sign as this is effectively going back in time

This now gives today's effective cost for the asphalt overlay laid in 2007 (significantly more than today's cost, due to the real price escalation effect, more than inflation).

The PWOC (1997) of one km of 50 mm asphalt overlay placed in 2007 is then calculated using the effective cost of placement in today's prices, and the current discount rate:

$$= \text{CU } 77\,987 (1+0,08)^{-10} = \text{CU } 36\,123$$

Discount Rate

The use of the present worth of cost method requires the selection of a suitable discount rate. This rate is dependent on various factors, including the:

- Effective rate of borrowing money.
- Rate of return that money can earn if invested.

Discount rates are determined by national financial agencies, based on the various factors, and the appropriate one should be used in PWOC calculations. Typical values lie between three and 10 per cent.

5.5 Incorporating Uncertainty

The discussion on the various cost items highlighted the general uncertainty inherent in any projection of future circumstances. In order to accommodate this, it is recommended that Bayesian analysis and decision trees are used in the PWOC analysis to appraise some of the uncertainties. The principles are outlined in Appendix C.

Decision trees and the Bayesian theory are ideally suited to assess various probable outcomes within the PWOC method. This system further provides for the incorporation of personal experience of conditions and materials in a formal analysis procedure. The application of these procedures for the economic analysis of applicable rehabilitation options is demonstrated in Appendix D.

6. GLOSSARY OF TERMS

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.

Cause of distress - a set of circumstances necessary for the occurrence of the distress.

Coefficient of skid resistance - coefficients of friction obtained by instruments used to assess skid resistance under conditions similar to those experienced by skidding vehicles.

Cumulative number of equivalent Standard axle loads - the total equivalent standard axle loads (ESAs) over a design period.

Deflection (representative) - the value of surface deflection measured under a standard 40 kN dual wheel load that is used to represent the deflection of a section of road. It is that deflection below which the deflection of 95 per cent of the section length occurs.

Deflection (surface) - the recoverable vertical movements of the pavement surface caused by the application of a wheel load.

Deformation - a mode of distress, the manifestation of unevenness of the surface profile.

Degree of distress - a measure of the severity of the distress.

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 of the structural capacity.

Dynamic Cone Penetrometer (DCP) - an instrument for assessing the in-situ CBR of materials.

Equivalent traffic - the number of equivalent standard single axle loads (ESAs) 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.

Extent of distress - proportion of the pavement exhibiting this distress or the frequency of occurrence of the distress.

Fill - subgrade material placed over the roadbed.

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.

Heavy Vehicle Simulator (HVS) - a machine for the accelerated simulation of traffic loads on a section of pavement.

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

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

Length of distress - a measure of the extent of distress.

Lacroix Deflectograph - a machine for the measurement of deflection on the surface of a pavement under a standard 40 kN dual wheel load.

Maintenance - a remedial measure to improve the serviceability (not usually the geometric properties) or the structural capacity of a road.

Maintenance priorities - the selection of certain maintenance tasks deemed to be of greater economic value than others.

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

Material properties - the reaction of various road building materials to changes in stress under varying conditions of moisture and density.

Maximum annual deflection - the deflection measured under a standard 40 kN dual wheel load adjusted for seasonal variations to represent the maximum measurable deflection during a one-year period on a section of road.

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.

Mechanism of distress - the physical process by which the cause of the distress results in the distress.

Modes of distress - the six major classes of distress.

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.

Multi-depth Deflectometer (MDD) - instrument for the measurement of in-situ deflections at various depths in a pavement.

Pavement behaviour - the function of the condition of the pavement with time.

Pavement description - description of the condition of a road in terms of its ability to fulfill its functional requirements.

Pavement evaluation - the assessment of the degree to which the road fulfils its functional requirements.

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

Pavement situation - incorporates all the design (structural, traffic and environmental) variables and performance (surface and material) variables that could influence pavement behaviour.

PCA Roadmeter - instrument for assessing riding quality, developed by the Portland Cement Association.

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

Position of distress - the situation of distress with respect to the surface of the road.

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.

PSI - Present Serviceability Index, a measure of riding quality obtained by the use of instruments.

Pumping - the mechanism causing water to move soil fines from within the pavement up through the surface of the pavement. Also used as a description of the appearance of the pavement as a result of the mechanism working.

Reflection cracks - cracks in asphalt overlays or surface treatments that reflect the crack pattern of the pavement structure underneath.

Rehabilitation design period - the chosen minimum period for which a pavement rehabilitation is designed to carry the traffic in the prevailing environment, with a reasonable degree of confidence, without necessitating further pavement rehabilitation.

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.

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

SCRIM - instrument for the assessment of skid resistance.

Selected layer - the lowest of the pavement layers, comprising controlled material, either in situ or imported.

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.

Skid resistance - the general ability of a particular road surface to prevent skidding of vehicles.

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

Smoothing of surface texture - mode of distress, loss of macro-texture and/or micro-texture.

Standard Axle (SA) - (80 kN/8,160kg/18,000lb/18kip) single axle dual wheel configuration.

Structural capacity - the ability of the pavement to withstand the effects of climate and traffic.

Structural description - a list of values of properties of the pavement relevant to a mechanistic evaluation.

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 evaluation - the assessment of the structural capacity of a 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.

Structural response - the stresses and strains introduced in to the pavement by loading on the surface.

Structural strength - an assessment of the strains that can be tolerated by the pavement in relation to those induced by a standard axle load under prevailing environmental conditions.

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.

Surface distress - distress pertaining to the wearing and skid resistance of the surfacing.

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

Surfacing integrity - a measure of the condition of the surfacing as an intact and durable matrix (it includes values of porosity and texture).

Surfacing maintenance/rehabilitation - 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.

Terminal level - a minimum acceptable level of some feature of the road in terms of its serviceability.

Types of distress - the sub-classification of the various manifestations of a particular mode of distress.

Volume binder concentration - the volume of binder in a sample of asphalt expressed as a percentage of the bulk volume of the asphalt.

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APPENDIX A

CONDITION ASSESSMENT: PERFORMANCE CRITERIA FOR THE EVALUATION OF PAVEMENTS

APPENDIX A: CONDITION ASSESSMENT: PERFORMANCE CRITERIA FOR THE EVALUATION OF PAVEMENTS

A.1 General

Depending on the parameter under investigation, performance criteria could depend on the category of road (importance and traffic loading), the pavement structure and the drainage of the pavement. Criteria are established for three condition classifications, i.e. sound, warning and severe, the definition of which depends on the parameter under investigation.

In reference to the present condition of the pavement as recorded during the detailed visual inspection and the measurement of the present serviceability of the pavement, the criteria refer to acceptability of the pavement in terms of existing distress (e.g. cracking, deformation, etc.), the existence of which is usually recorded on visual inspection forms. An example of a form to be used at project level is shown in Figure A.1. In these cases the criteria are defined as follows:

- Sound condition: present condition is adequate.
- Warning condition: uncertainty exists about the adequacy of the present condition.
- Severe condition: present condition is inadequate.

In terms of the measured pavement or material response, the parameters are used to assess the structural capacity of the pavement. In these cases the pavement condition is defined as follows:

- Sound condition: the measured or recorded parameter is of such magnitude that the pavement should be able to carry the design traffic for the specific category of road without deteriorating to a critical state (stage considered most economical for rehabilitation).
- Warning condition: the measured or recorded parameter is of such magnitude that the pavement should be able to carry the minimum design traffic, but not the maximum design traffic for that specific category of road without reaching a critical state (the pavement is expected to deteriorate to a level between a critical and failed state).
- Severe condition: the measured or recorded parameter is of such magnitude that the pavement is expected to deteriorate beyond a critical state before the minimum design traffic loading for a specific category of road had been reached.

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NAME: J SMITH LANE: SLOW CHAINAGE 16.0 - 16.5
NB: READ INSTRUCTIONS BEFORE USE

CHAINAGE (km)	ROAD WIDTH	POSITION TYPE	DEGREE	SPACING (m)
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16.1		C	4	
16.2		T	3	
16.3			2	
16.4				
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A.2 Visually Recorded Distress

The criteria refer to the extent of the distress measured as a percentage of a unit length. Recommendations towards appropriate basic unit lengths to be used for the analysis of pavements for project level investigations are given in Table A.1. For visually recorded distress these are dependent on the category of road and the type of distress, as shown in Figure A.2. The criteria recommended in Table A.2 are derived from Figure A.2, based on South African experience. Only distress with a severity rating (degree) of 3 to 5 is used for determining the condition of the pavement.

The following general rules apply:

		Extent of distress (severity rating 3 to 5)	< X	Sound condition
X	≤	Extent of distress (severity rating 3 to 5)	< Y	Warning condition
Y	≤	Extent of distress (severity rating 3 to 5)		Severe condition

However, for combinations of cracking or cracking other than crocodile and longitudinal cracking the percentage of length of the different types of cracking are added together and considered as "other" cracking.

For patching:

- Replace the "extent of distress" with the equation:

$$Lp + \sqrt{A_L}$$

where

Lp = per cent of unit length covered by patching

A_L = area of patches in m^2

Table A.1: Typical unit lengths for the processing of project level data

Road classification	Length of basic evaluation sections
Major inter city freeway	50 m to 100 m
Rural freeway/Category A road	100 m to 200 m
Major rural road/Category B road	100 m to 400 m
Tertiary rural road/Category C and D roads	100 m to 1 000 m
Major city road	50 m to 100 m
Collectors	50 m to 200 m
Residential roads	50 m to 400 m
(Note: Several basic unit lengths are usually combined to form a uniform pavement section which is used to determine structural needs.)	

Table A.2: Performance criteria for visually recorded distress

Distress Parameters	Category of road					
	A		B		C and D	
	X	Y	X	Y	X	Y
Cracking:						
- Crocodile	5	15	10	20	15	25
- Longitudinal	30	60	45	75	60	90
- Other*	10	30	20	40	30	50
Deformation	5	15	10	20	15	25
Disintegration						
- Patching	10	30	20	40	30	50
- Ravelling	20	40	30	50	40	60
Smoothering	20	40	30	50	40	60
* Combinations of cracking or cracking other than crocodile or longitudinal						

Drainage problems such as blocked side drains, lack of drain facilities and standing water next to the road should be recorded as severe on the detailed visual inspection forms.

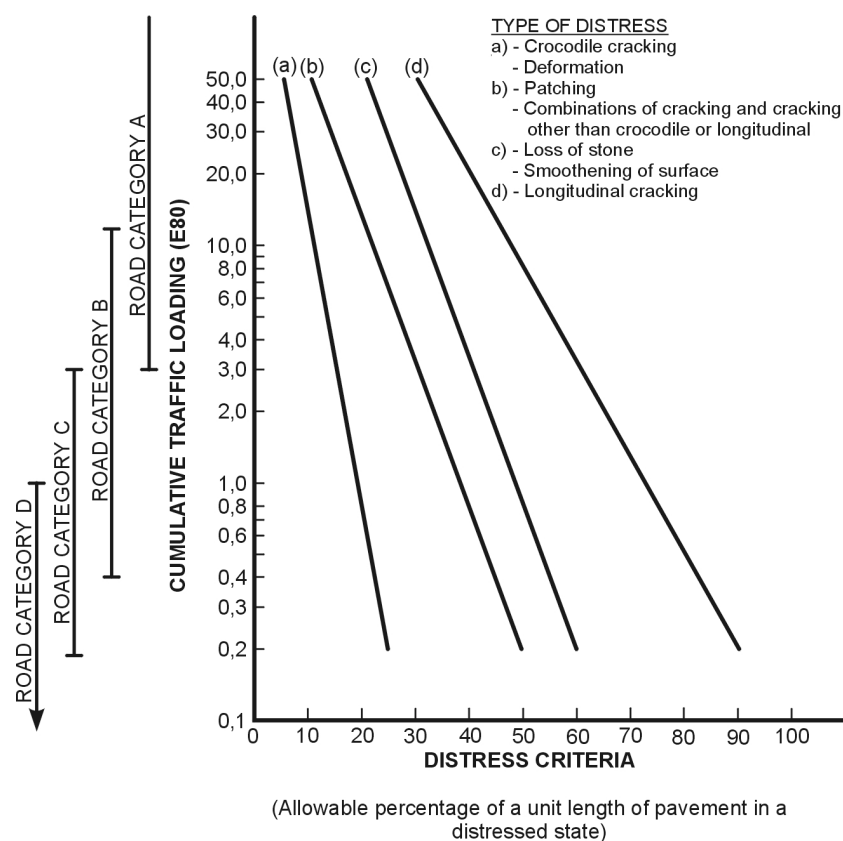


Figure A.2: Performance criteria for visually recorded distress

A.3 Mechanical Pavement Surveillance Measurements

It is considered acceptable for a small percentage of a length of road to perform unsatisfactorily at the end of the rehabilitation design period. This percentage depends on the category of road. The percentile levels recommended are given in Table A.3.

Table A.3: Percentile levels recommended for data processing

Category of road	Length of road allowed to perform unsatisfactorily at the end of its design life (%)	Percentile levels recommended for data processing
A	5	95
B	10	90
C	20	80
D	50	50

A.3.1 Assessment of serviceability

The following parameters are measured to assess serviceability:

- Riding quality in Present Serviceability Index (PSI), International Road Index (IRI) or Half-car Roughness Index (HRI).
- Rut depth in mm.
- Skid resistance in the sideways force coefficient at a speed of 50 km/h (SFC50).

Criteria recommended for riding quality and rut depth are given in Table A.4. Skid resistance criteria are risk related and thus vary, depending on logistics and other pragmatic issues relevant to particular circumstances.

Table A.4: Performance criteria recommended for the assessment

Road category	Riding quality (PSI)		Riding quality (IRI)		Riding quality (HRI)		Rut depth (mm)		Skid resistance (SFC80)	
	X	Y	X	Y	X	Y	X	Y	X	Y
A	3.0	2.5	2.9	3.5	2.0	2.7	10	20	Refer road authority	
B	2.5	2.0	3.5	4.2	2.7	3.5	10	20		
C	2.0	1.5	4.2	5.1	3.5	4.5	10	20		

A.3.1 Assessment of structural capacity

This is primarily done by making use of surface deflections (Benkelman beam), radius of curvature and DCP penetrations. Performance criteria recommended for these parameters are given in Table A.5. The criteria for deflection and radius of curvature measurements are shown in Figures A.3 and A.4.

Table A.5: Criteria recommended for the assessment of structural condition

Parameter	Basecourse material	Moisture regime	Category of road					
			A		B		C	
			X	Y	X	Y	X	Y
Deflection (Benkelman beam) (mm)	Gravel/untreated subbase	-	0.4	0.7	0.5	1.0	0.7	1.4
	Gravel/treated subbase	-	0.3	0.6	0.4	0.9	0.6	1.3
	Lightly cemented	-	0.3	0.6	0.4	0.8	0.6	1.2
	Bituminous	-	0.2	0.5	0.3	0.7	0.5	1.0
	Cemented	-	0.2	0.4	0.3	0.6	0.4	0.8
Radius of curvature (m) (Dehlen Curvature Meter)	Gravel/untreated subbase	-	100	60	80	45	60	30
	Gravel/treated subbase	-	150	80	110	60	80	40
	Lightly cemented	-	200	100	150	75	100	50
	Bituminous	-	300	130	200	90	130	65
	Cemented	-	300	150	200	100	150	80
DCP measurements (DSN ₈₀₀ or number of blows to penetrate 800mm)		Equilibrium	350	155	230	110	155	70

In recent years, more sophisticated apparatus for the evaluation of the structural capacity of pavements has become available, including various instruments for the measurement of the deflection bowl, e.g. the Falling Weight Deflectometer (FWD) and Deflectograph. In general, the deflection basin parameters as defined in Table A.6 are used to assess the structural capacity of a pavement.

The magnitude of these parameters not only depends on the characteristics of the pavement, but also on the type of instrument used to measure the deflection bowl. Hence, different criteria for the various parameters should be determined for each type of instrument and each agency should determine limits appropriate to their network.

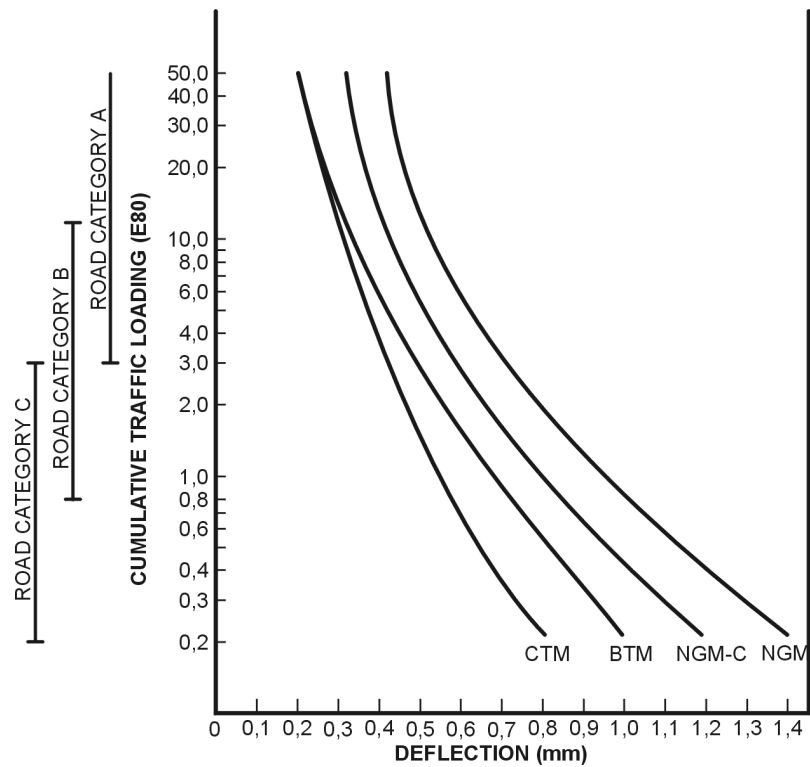


Figure A.3: Criteria for measured maximum surface deflection (Benkelman beam)

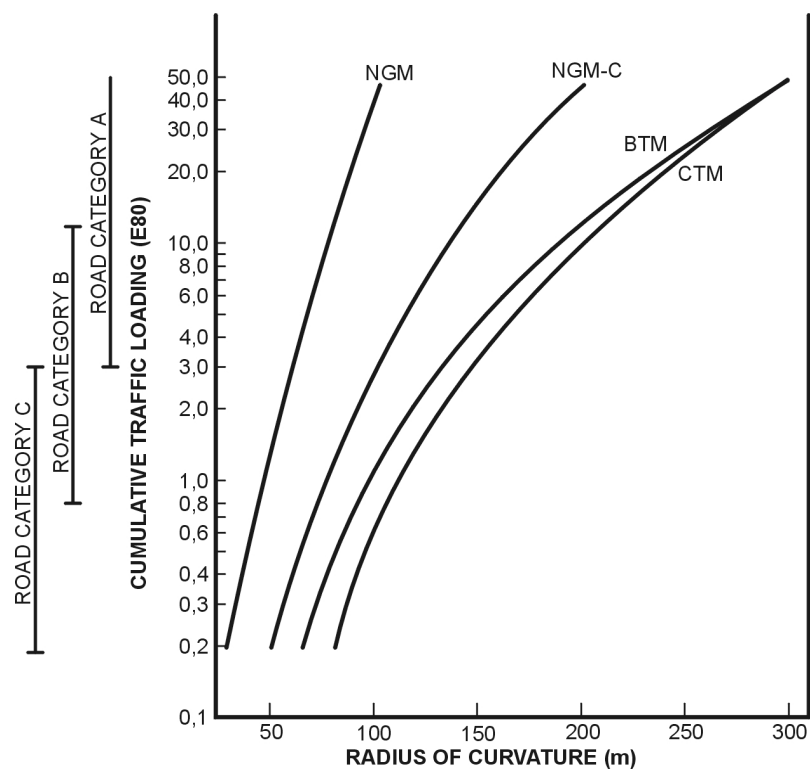


Figure A.4: Criteria for measured radius of curvature (Dehlen curvature meter)

Table A.6: Summary of the most commonly used deflection bowl parameters

Parameter	Formula	Measuring device
1. Maximum Deflection	δ_0	Benkelman beam Lacroix Deflectograph
2. Radius of Curvature	$R = \frac{r^2}{2 \delta_0 (1 - \delta_r / \delta_0)}$ $r = 127 \text{ mm}$	Curvaturemeter
3. Spreadability	$\frac{[(\delta_0 + \delta_1 + \delta_2 + \delta_3) / 5] 100}{\delta_0}$	Dynalect
4. Area	$A = 6 [1 + 2 (\delta_1 / \delta_0) + 2 * (\delta_2 / \delta_0) + \delta_3 / \delta_0]$	Falling weight deflectometer (FWD)
5. Shape Factors	$F_1 = (\delta_0 - \delta_2) / \delta_1$ $F_2 = (\delta_1 - \delta_3) / \delta_2$	FWD
6. Surface Curvature Index	$SCI = (\delta_0 - \delta_2), \text{ where}$ $r = 305 \text{ mm}$ $\text{or } r = 500 \text{ mm}$	Benkelman beam Road rater FWD
7. Base Curvature Index	$BCI = \delta_{610} - \delta_{915}$	Road rater
8. Base Damage Index	$BDI = \delta_{305} - \delta_{610}$	Road rater
9. Deflection Ratio	$Qr = \delta_r / \delta_0 \text{ where}$ $\delta_r \approx \delta_0 / 2$	FWD
10. Bending Index	$BI = \delta / a \text{ where}$ $a = \text{Deflection Basin Length}$	Benkelman beam
11. Slope of Deflection	$SD = \tan^{-1} (\delta_0 - \delta_r) / r$ $\text{where } r = 610 \text{ mm}$	Benkelman beam

A.4 Pavement Evaluation Form

All information gathered on the road is taken into account when dividing it into uniform pavement sections. It follows that the information needs to be combined in an easily understandable and manageable format. An example of a form used for this purpose is given in Figure A.4.

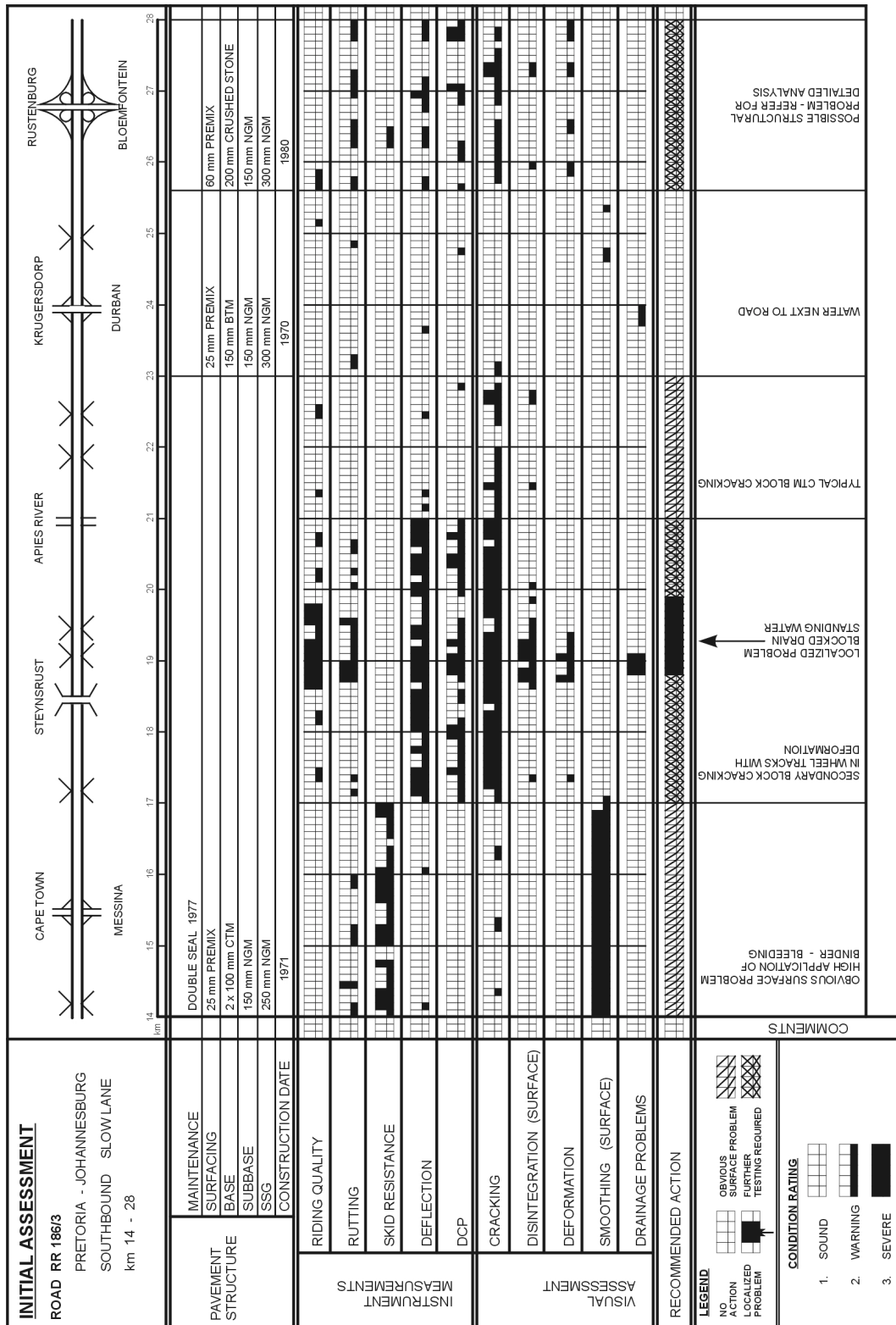


Figure A.5: Example of a plot of the processed data for the evaluation of pavements

APPENDIX B

PAVEMENT REHABILITATION DESIGNS USED IN SOUTHERN AFRICA: PRINCIPLES AND MAIN CHARACTERISTICS

APPENDIX B: PAVEMENT REHABILITATION DESIGNS USED IN SOUTHERN AFRICA: PRINCIPLES AND MAIN CHARACTERISTICS

B.1 Asphalt Institute Method

This method was published in 1969 by the Asphalt Institute⁹. The design manual covers both geometric and structural improvements of pavements in order to increase the traffic capacity, load carrying ability and the safety of the road user.

The manual identifies the following causes of a structurally inadequate pavement:

- Increase in traffic
- Change in the pavement material properties
- Inadequate design procedures

Different causes of the pavement distress problem are identified, but no distinction is made between the recommended approaches to evaluation and rehabilitation design of any pavement. Two empirically derived procedures for the evaluation of structural adequacy and overlay design are given. These procedures are presented as applicable to all flexible pavements and any cause and mechanism of distress. The recommended procedures are based on the pavement component analysis and pavement response analysis (surface deflection) approaches.

Although seemingly different, both the recommended procedures are empirically derived and aim at providing adequate protection to the subgrade of the pavement. In this case the pavement rehabilitation problem is approached in a similar way to well-known empirical methods used for the design of new pavements, such as the CBR approach. Some subjective use of the existing condition of the pavement is made in the pavement component analysis procedure.

The main characteristics of the Asphalt Institute method are summarized in Figure B.1 and in Table B.1.

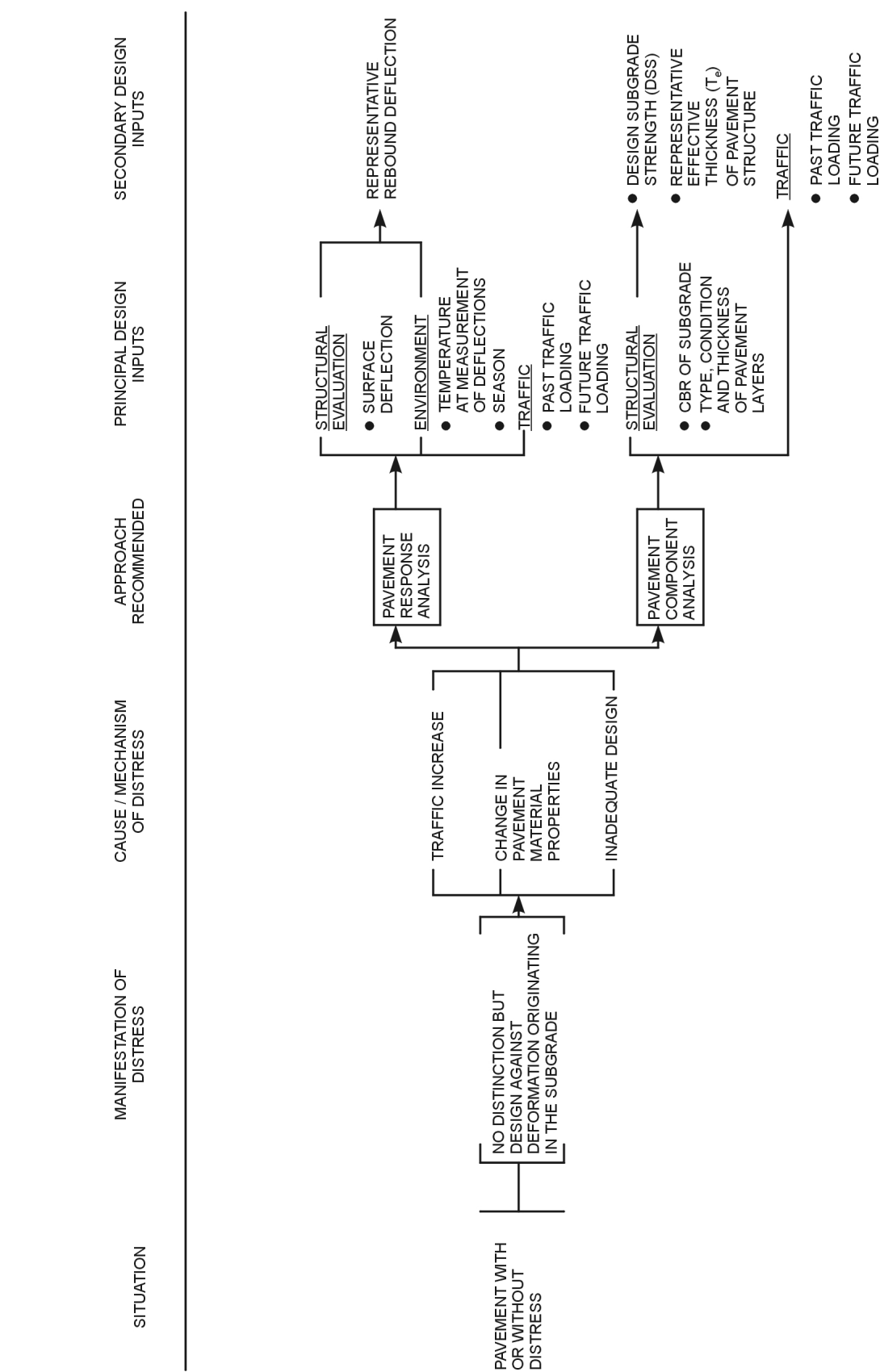


Figure B.1: Asphalt Institute method for structural design of pavement rehabilitation

Table B.1: Main characteristics of the Asphalt Institute pavement rehabilitation design method

Developed	Basis of derivation	Classification according to approach	Pavement evaluation surveys	Applicability	Input to design	Limitations (main)	Advantages (main)
Asphalt Institute in the USA	Empirical	Pavement response analysis 1 Deflection-based 2 Pavement component analysis	1 Surface deflections 2 CBR of subgrade Type, thickness and condition of pavement layers	<ul style="list-style-type: none"> - Developed with data from pavements with granular sub-layers and thin surfacing - Pavement component analysis method incorporates all types of material using conversion factors and subjective condition ratings 	<ul style="list-style-type: none"> - Surface deflection - Temperature - Past traffic loading - Future traffic loading - CBR of subgrade (DSS) - Type, condition, thickness of pavement layers - Past traffic loading - Future traffic loading 	<ul style="list-style-type: none"> - Empirically base with limitations to applicability - Design based on protection of subgrade only - No seasonal adjustment factors given - AI curve applicable to traffic loading up to $7,3 \times 10^6$ E80s - Component analysis procedure applicable up to $36,5 \times 10^6$ E80s 	<ul style="list-style-type: none"> - Based on easy-to-apply and well-known concepts
Assumptions			Material characterisation	Residual life	Basis of design life	Pavement transfer functions consideration	
<ul style="list-style-type: none"> - Those applicable to deflection-based methods - Deflection-overlay thickness relation incorporates assumptions of linear elasticity theory - Those applicable to pavement component analysis method - The cover requirement of the pavement is independent of the requirements of the individual pavement layers. - All pavement materials can be converted to an equivalent asphalt thickness. 			1 - 2 Uses subjective assessment of the condition of pavement layers	1 Use of deflection versus life curve 2 Uses nomograph relating pavement equivalent thickness to life	<ul style="list-style-type: none"> - Protection of the subgrade - Pavement deformation 	1 Deflection of structure	

B.2 The DCP Method

The DCP pavement rehabilitation design method⁸ is empirically derived and based on observations and experience with the use of the DCP mainly in the Transvaal region of South Africa. The assumptions applicable to the pavement component analysis approach are also applicable to this method. The method was basically developed for use on pavements with thin surfacing and natural gravel sublayers. However, research has shown that the method can also be used on pavements with lightly cemented layers ($UCS < 3 \text{ MPa}$). Research is continued to determine the applicability of this method to pavements other than those consisting of layers with granular materials only.

The method aims at achieving a balanced pavement design and at optimizing the utilization of the in-situ pavement material strength. This is done through the design of a pavement capable of carrying the expected future traffic and by comparing the existing pavement DCP properties with those of the design pavement. Many of the concepts used originated from practical experience with the use of the instrument and results from HVS tests were used to verify these concepts and to establish structural capacity curves.

The development of these curves was based on a rut depth of 20 mm measured under a 2.0 m straight-edge which was defined as representing a terminal pavement condition. The design curves were derived using the mean values measured and are therefore only an indication of mean expected life.

This design method can lead to various applicable design pavements as several pavement layer configurations will be capable of carrying a specified design traffic. Therefore, a trial-and-error procedure will have to be used to determine the most economical rehabilitation option, taking into account the existing pavement structure and the materials locally available. Ample opportunity is allowed for the assessor to incorporate his own experience in the design of applicable rehabilitation alternatives.

The main characteristics of the method are summarized in Figure B.2 and in Table B.2.

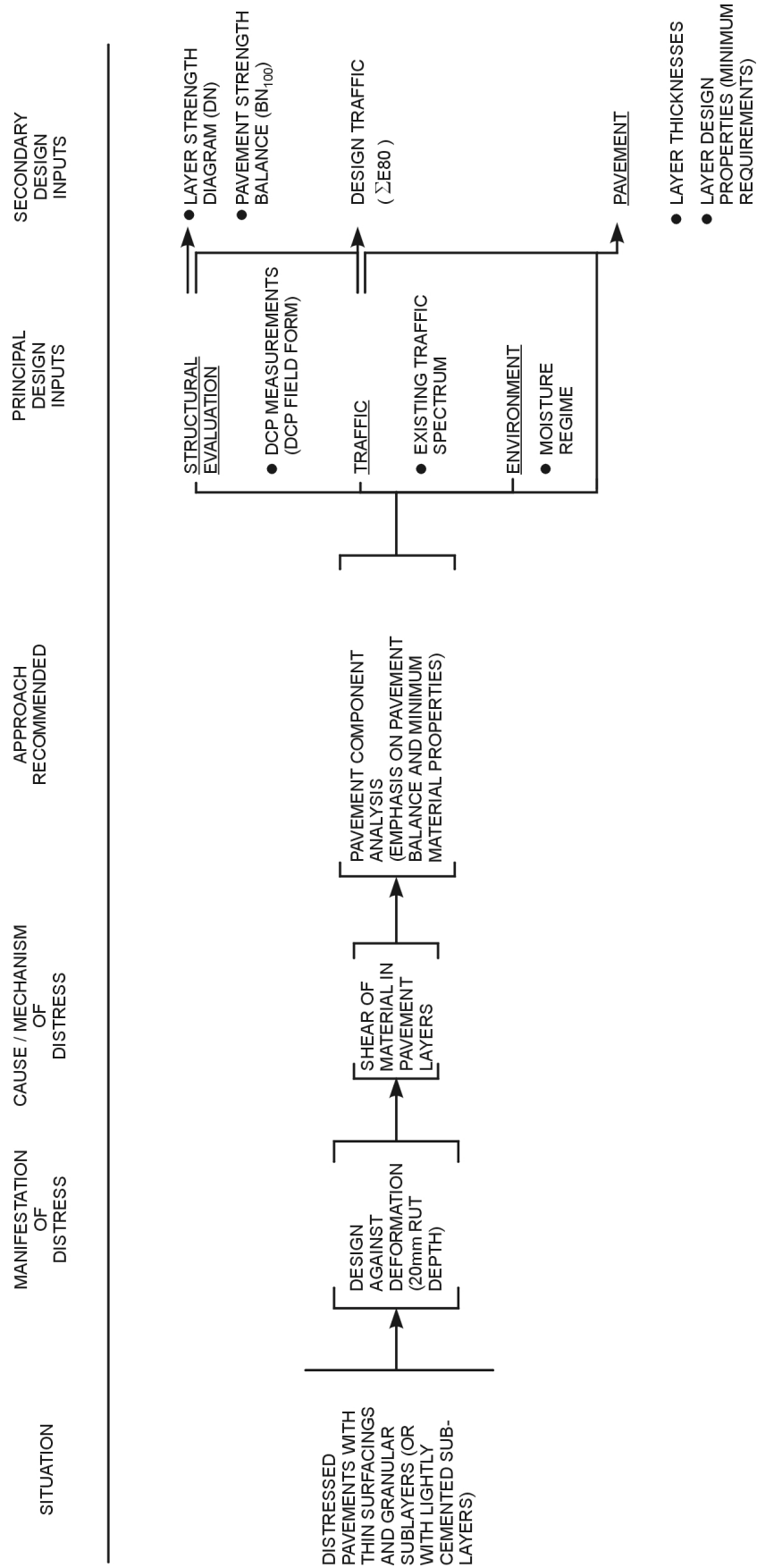


Figure B.2: Dynamic cone penetrometer method for pavement rehabilitation design

Table B.2: Main characteristics of the DCP pavement rehabilitation design method

Developed	Basis of derivation	Classification according to approach	Pavement evaluation surveys	Applicability	Input to design	Basis of design life	Pavement transfer functions considered
South Africa (Transvaal Roads Department)	Empirical	Pavement component analysis	Dynamic Cone Penetrometer (DCP) measurements	<ul style="list-style-type: none"> - Light pavements with granular sub-layers and thin surfacing - Pavements with lightly cemented layers (further research) - Developed with data from balanced pavements only 	<ul style="list-style-type: none"> - DCP curve (DN of layers) - Moisture regime - Design traffic 	<ul style="list-style-type: none"> - Prevention of excessive stress in layers through balanced design - Prevention of excessive deformation through shear of pavement layers 	
Assumptions		Materials characterisation	Residual life	Limitations (main)		Advantages (main)	
<ul style="list-style-type: none"> - Those applicable to pavement component analysis methods - The bearing capacity of the pavement is a function of the combined strength of all the pavement layers to a depth of 800 mm - Pavement requiring rehabilitation are "in balance" 		<ul style="list-style-type: none"> - DCP penetrations (DN) used to determine the CBR or UCS of materials 	<ul style="list-style-type: none"> - Not provided for but the structural capacity versus DSN800 relation can be used to estimate residual life 	<ul style="list-style-type: none"> - Empirically based with limitations associated with the component analysis approach - Does not take seasonal variations into account - Some concepts have not been properly verified in practice - For use on balanced pavements only - Design curves applicable to traffic between $0,2 \times 10^6$ and 10×10^6 E80s 		<ul style="list-style-type: none"> - Based on NDT in situ pavement tests - Allows for the assessment of individual pavement layers - Based on the optimum utilization of in situ pavement layer properties 	

B.3 The TRL Surface Deflection Method

The pavement evaluation and rehabilitation design method developed at the Transport Research Laboratory (TRL) was published in 1978⁷. The method is empirically derived and uses surface deflection as a design parameter. It was based on many years of research and experimental work on roads dating back to 1956.

The main objective of the method is to provide a system for the design of pavement-strengthening measures that will enable the engineer to:

- Predict the remaining life of a pavement before a critical condition is reached.
- Design the thickness of overlay required to extend the life of the pavement to carry a given design traffic.

The method is based on the characterization of the structural condition of the pavement through the measurement of surface deflection. Although all measurements were based on deflections under a dual wheel load of 3 175 kg moving at creep speed, this can easily be adapted to deflections measured under a Standard axle load. The design manual pays special attention to the analytical procedures and the correct measurements, adjustment and use of pavement surface deflections in the various design charts. The method also takes into account and distinguishes between deflections measured on the main types of roadbase.

The recommended method entails the following:

- Measurement of the surface deflection of the pavement.
- Adjustment of the measured deflection.
- Estimation of past, present and future expected traffic on the pavement.
- Prediction of the residual life of the pavement before reaching a critical condition.
- Design of an overlay to extend the life of the pavement to carry the design traffic.
- Matching the overlay design to variations in the structural condition of the pavement.

The main characteristics of the TRL deflection method are summarized in Figure B.3 and Table B.3.

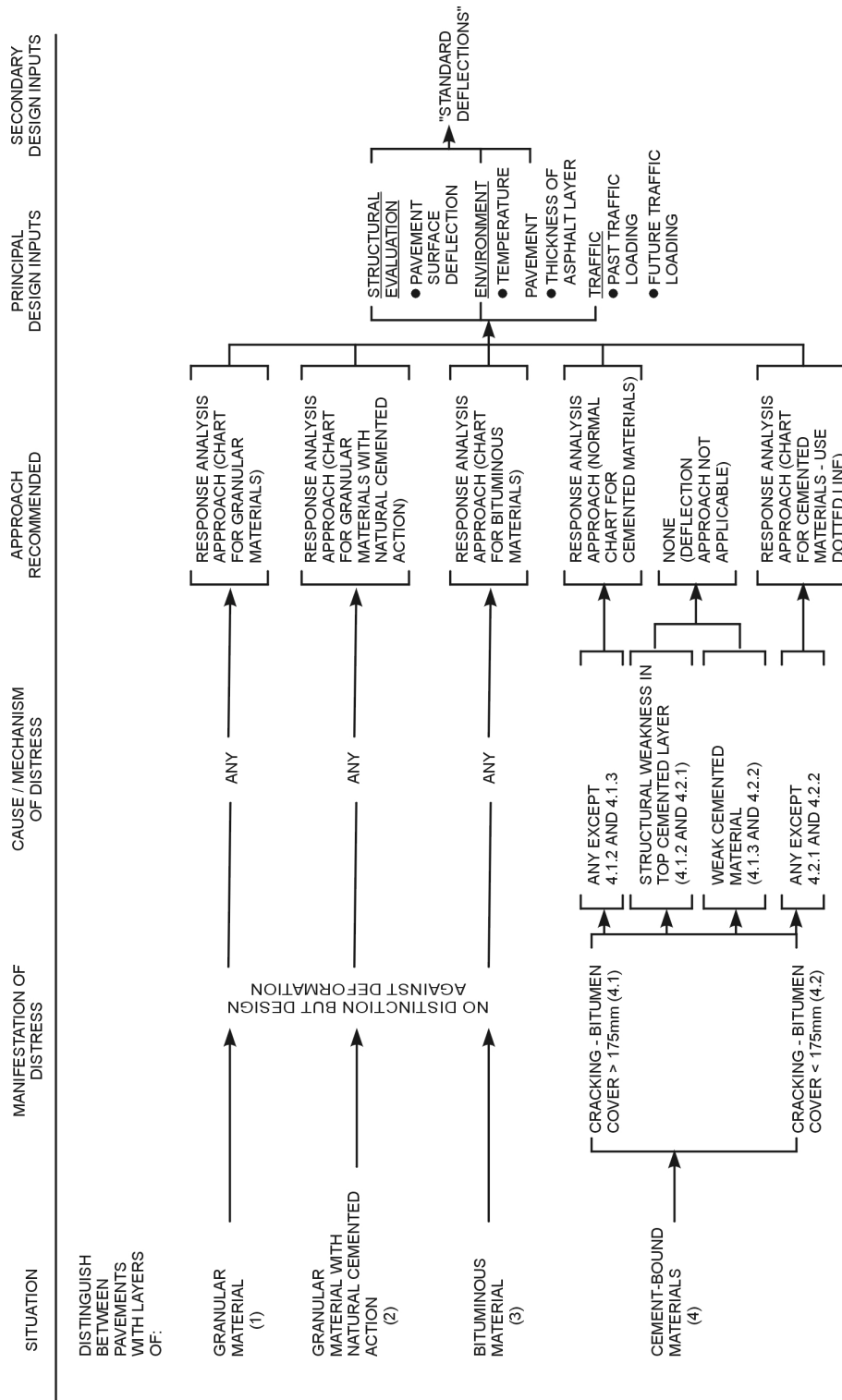


Figure B.3: TRL method for the structural design of pavement rehabilitation

Table B.3: Main characteristics of the TRL surface deflection method

Developed	Basis of derivation	Classification according to approach	Pavement evaluation surveys	Applicability	Input to design	Limitations (main)	Advantages (main)
United Kingdom (TRL)	Empirical	Pavement response analysis (deflection based)	<ul style="list-style-type: none"> - Surface deflections - Temperature 	<ul style="list-style-type: none"> - Design charts developed for main types of flexible pavement - Some limitations of use for pavements with cemented layers 	<ul style="list-style-type: none"> - Surface deflections - Temperature at time of measurements - Thickness of asphalt layer - Type of pavement - Past traffic loading - Future traffic loading 	<ul style="list-style-type: none"> - Empirically based with limitations in applicability - No seasonal adjustment given - Applicable for traffic loading up to 10×10^6 E80s - Design (except for pavement with cemented layers) based on deformation originating from the subgrade only 	<ul style="list-style-type: none"> - Base on easy NDT testing - Is easy to use - Distinguishes between the types of pavement - Identifies some limitations in the use of the method on pavements with cemented layers - Adjust deflections to take the effect of temperature variations into account
Assumptions			Materials characterisation	Residual life	Basis of design life	Pavement transfer functions considered	
<ul style="list-style-type: none"> - Those applicable to deflection-based methods - Pavement life is a function of type of pavement and surface deflection 			-	Use of deflection versus life curves	Deformation in the form of rut depth except for pavements with cemented layers where cracking is used as a criterion	Deflection of structure	

B.3 Shell Overlay Design Method

The Shell overlay design method was developed at the Koninklijke Shell Laboratorium in Amsterdam from the Shell design procedure for new pavements¹¹. The charts originally developed for the design of new pavements are also used for the design of asphalt overlays for existing pavements. Non-destructive test (NDT) methods are used to assess the properties of the existing pavement.

In the Shell method, a number of design charts are used to determine the required thickness of overlays for the rehabilitation of a pavement. The design charts were derived from the results of analyses of pavements by means of the theory of linear elasticity. In these analyses the pavement was assumed to be adequately represented by a three-layered model consisting of a top layer (surfacing) of asphaltic material, a middle layer (base) of granular or cementitious material and a bottom layer (subgrade) of semi-infinite dimensions as shown in Figure B.4. In this model the pavement materials are characterized by the following properties:

- Surfacing layer: the effective modulus of deformation (E_1 value), a Poisson's ratio (v_1) and a layer thickness (h_1).
- Base layer: the effective modulus of deformation (E_2 value), a Poisson's ratio (v_2) and a layer thickness (h_2).
- Subgrade layer: the effective modulus of deformation (E_3 value) and a Poisson's ratio (v_3) (the layer thickness is assumed to be infinite).

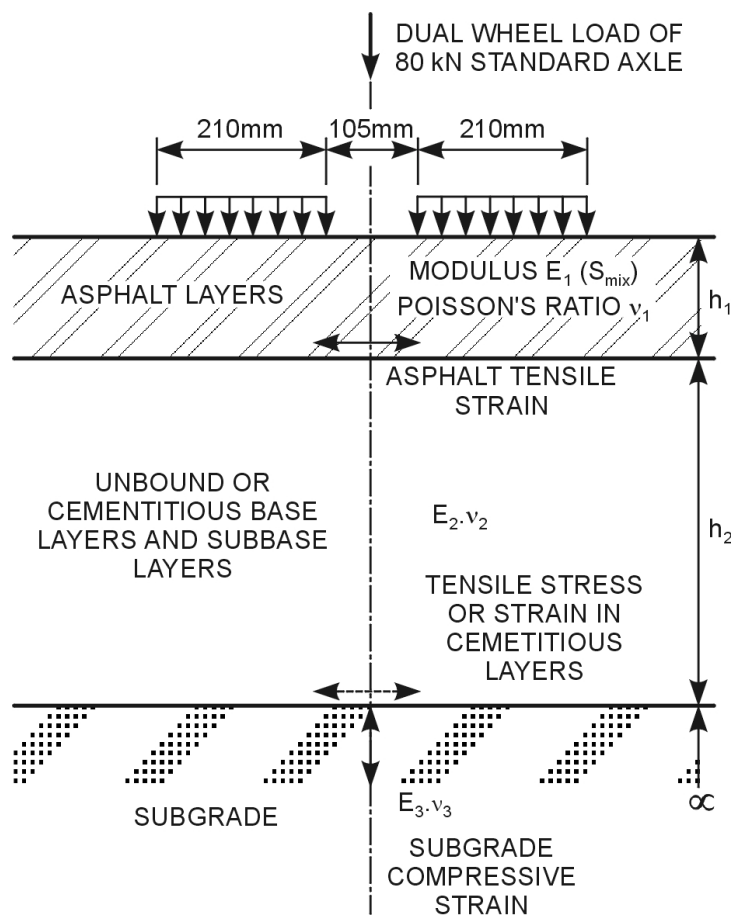


Figure B.4: Simplified pavement structure

In the development of the method, the BISAR computer program was used to calculate the stresses and strains in pavement structures. The results obtained were used to compile the design charts. The primary design criteria used for the compilation of these charts were the:

- Compressive vertical strain in the surface of the subgrade which controls deformation of the subgrade material.
- Horizontal tensile strain in the asphalt layer which controls cracking of the asphalt layer.
- Tensile stress of strain in any cementitious base layer which controls cracking of the cementitious layer.

In the evaluation of pavements using the derived charts, fixed values are assumed for the Poisson's ratios for all the layers. Falling Weight Deflectometer (FWD) measurements, material tests and available data are used to determine the other in-situ properties. The maximum deflection and the shape of the deflection bowl, as measured by the FWD, are used to determine the subgrade modulus (E_3) and the effective thickness of the asphalt layer (h_1). The deflection bowl is characterized by the ratio (Q_r) of the deflection at a distance r from the load (s_r) to the deflection under the centre of the test load (s_o). The distance r should preferably be such that $Q_r \sim 0.5$.

The design charts ensure that the strains (mentioned above) are limited to such an extent that virtually no cracking will occur in the structure and that there will be no excessive permanent deformation in the subgrade during the design life of the pavement.

Practical constraints made it impossible to develop design charts for every conceivable pavement configuration. Hence, the method will often require design charts to be interpolated for overlay design. The use of such charts eliminates the need to calculate the critical parameters of the pavement through computer simulation of the pavement response.

Although the design approach is based on the above mentioned strain criteria, the Shell method also provides for the testing of the expected permanent deformation in the asphalt layer and for checking of the maximum overlay thickness. Provision is made to include climatic variations (temperature changes) in the design of the overlay. Procedures included in the method take into account the variations in asphalt mix properties available for overlays and assess their differences in comparison with the asphalt on the existing pavement and their influence on the expected future behaviour of the rehabilitated pavement.

The main characteristics of the Shell overlay design method are summarized in Figure B.5 and Table B.4.

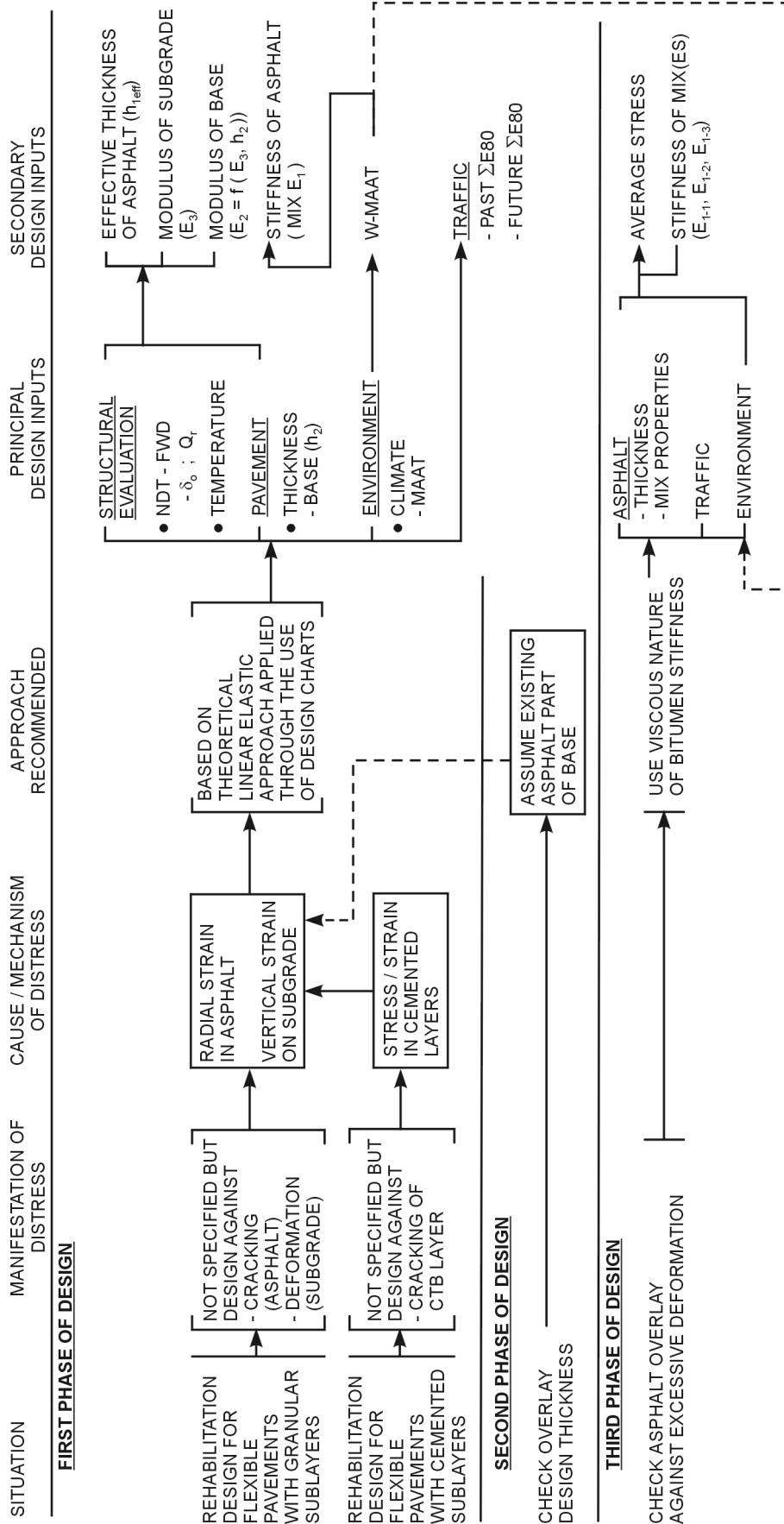


Figure B.5: Shell method for the structural design of pavement rehabilitation

Table B.4: Main characteristics of the Shell overlay design method

Developed	Basis of derivation	Classification according to approach	Pavement evaluation surveys	Applicability	Input to design	Limitations (main)	Advantages (main)
The Netherlands (Koninklijke Shell Laboratorium Amsterdam)	Theoretical (linear elasticity theory)	Application through the use of design charts	Falling weight Deflectometer (FWD)	All types of flexible pavements (special provision for cementitious layers)	<ul style="list-style-type: none"> - Maximum deflection - Deflection ratio (Qr) - Temperature 	<ul style="list-style-type: none"> - Incorporates only temperature changes - as a climate effect - Does not assess unbound pavement layers - Charts based on asphalt fatigue and subgrade deformation only 	<ul style="list-style-type: none"> - Bases on NDT tests - Tests overlay mix properties into account - Incorporates and allows for temperature gradients in the asphalt layer - Allows for the checking of deformation in asphalt layers - Incorporates different climates through w-MAAT factor
Assumptions			Materials characterisation	Residual life	Basis of design life	Pavement transfer functions considered	
<ul style="list-style-type: none"> - Those applicable to linear elasticity theory - Distress = f (asphalt strain, subgrade strain) - Modulus of base is a function of modulus of subgrade and the thickness of the base layer - Poisson's ratio for all layers = 0,35 			Use NDT (FWD) measurements <ul style="list-style-type: none"> - maximum deflections (σ) - deflection ratio (Qr) - temperature - thickness of base (h_2) 	Use FWD measurements between the wheel tracks to determine as built pavement life from design charts	<ul style="list-style-type: none"> - Tensile strain in asphalt - Compressive strain in subgrade - Viscous-nature of asphaltic material 	<ul style="list-style-type: none"> - Deflection basin - Asphalt tensile strain - Subgrade compressive stress 	

B.4 The South African Mechanistic Design Method

The South African mechanistic rehabilitation design method¹² was developed at the Division of Roads and Transport Technology, CSIR. It incorporates experience in pavement behaviour gathered through many years of accelerated pavement testing with the fleet of Heavy Vehicle Simulators (HVSs).

The method is based on the theory of linear elasticity. In practice, this theory is applied to pavement analysis through the use of a catalogue of behaviour states or, alternatively, through a computer-aided simulation of the response of the pavement under loading. The catalogue includes assessments of the behaviour and condition of a large number of pavements. Information is given on average values for several pavement indicators and on possible visual distress, recommended test methods, recommended analysis methods and possible rehabilitation options. Therefore, the catalogue can be used to do a comprehensive analysis of a pavement. The theory of linear elasticity as well as results and experience gained over many years of HVS testing have been combined to compile the catalogue.

A prerequisite for the use of the method is the identification of the class, type and state of the pavement and the state of the pavement materials as shown in Figure B.6. The material in each pavement layer is classified and material properties are assigned to each pavement layer through the use of published values selected from tables. These tables were compiled from practical experience with HVS testing and from the theoretical analysis of pavement under various conditions. (Naturally, elastic properties obtained by means of other procedures may also be used.) The state of the pavement and the material properties are used in the catalogue of behaviour states to identify and correlate the condition of the existing pavement with one of the analysed pavements from the catalogue. The assessor is advised only to do a computer-aided analysis of the pavement when no correlation can be achieved.

The method relies strongly on the correct identification of the current state and condition of the pavement. The role of specialized tests to assist in the identification of the behaviour state is discussed. These tests are not used for the direct characterization of material properties and pavement behaviour. Instead the pavement class, type and state as well as the material state are used somewhat subjectively to assign pavement properties to the different layers. Due to the emphasis on pavement behaviour, much effort is put into a detailed description of the change in behaviour with time and traffic loading. All the main flexible types of pavement (granular, cemented and bituminous) are discussed and changes in conditions, such as in the pavement layer state, are also taken into account.

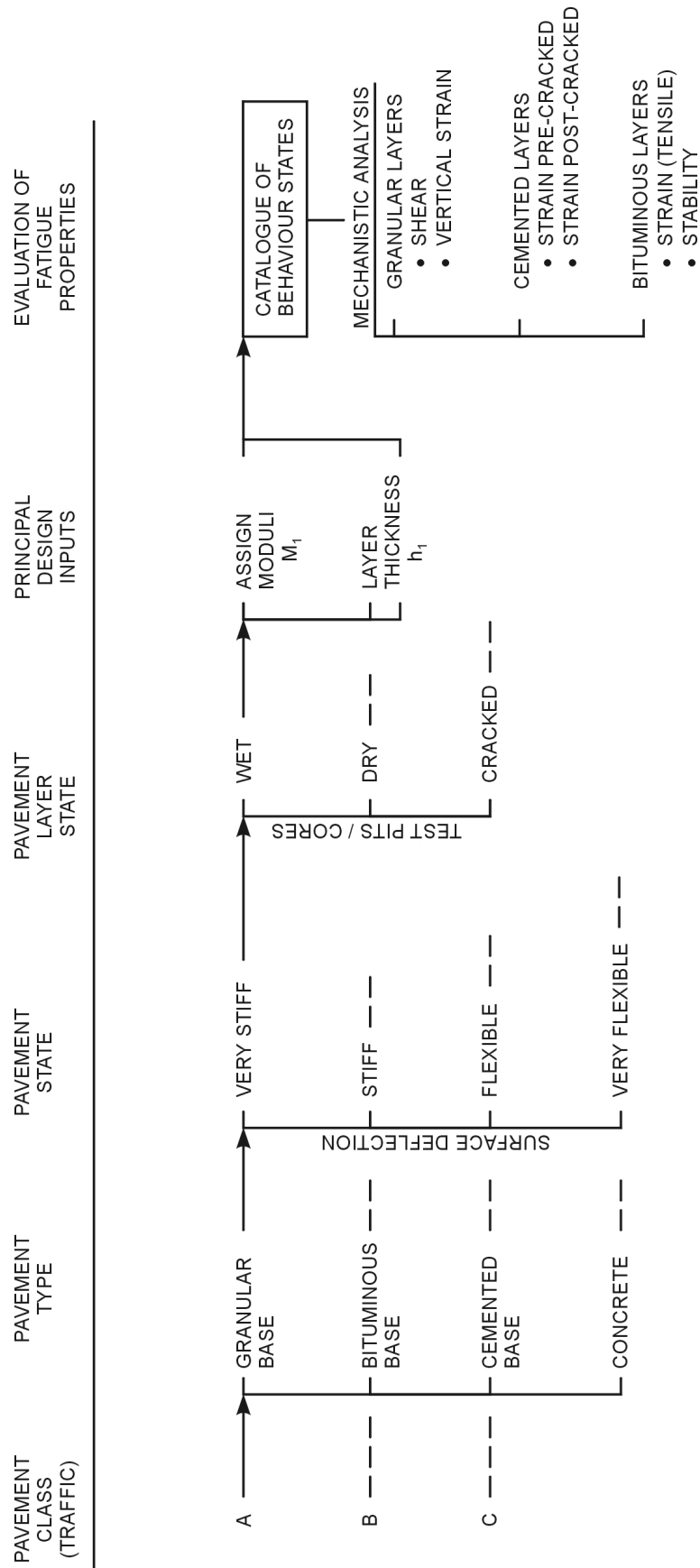


Figure B.6: Identification of the state and condition of a pavement

In a computer-aided analysis the material properties already assigned are used in computer programs to calculate the critical distress parameters. The modulus of deformation (E), Poisson's ratio (ν) and thickness (h) of each layer (subgrade considered as semi-infinite in thickness) are used in any of a number of available programs based on the theory of linear elasticity. The calculated critical parameters (stresses, strains and deformations) are weighted against limiting criteria to determine the need to strengthen the pavement. No method is given to determine directly the thickness of overlay, if applicable, or the amount and type of strengthening required. Rehabilitation options (e.g. thickness of an overlay) are determined through estimation or trial and error.

The method provides for a comprehensive theoretical analysis of various possible material conditions for the different pavement types. This method is one of the few that allows for the evaluation of all the types of pavement material in all the pavement layers, such as granular materials in a base or subbase layer. However, no provision is made for evaluation of possible permanent deformation in asphalt layers or of the pavement as a whole.

Although the method recognizes the importance of environmental changes such as temperature and moisture, no procedure is given for the inclusion of these factors in the rehabilitation design.

The main characteristics of the South African mechanistic design method are given in Figure B.7 and Table B.5.

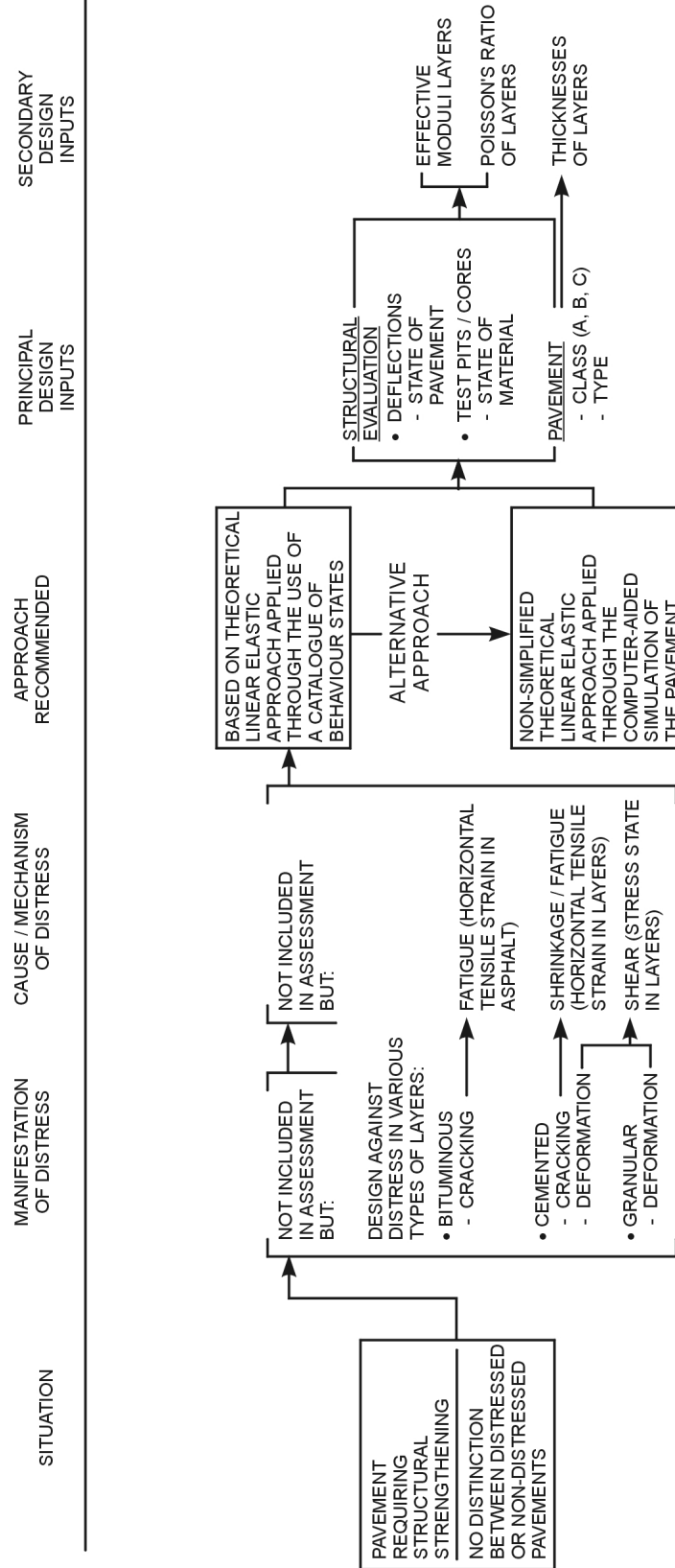


Figure B.7: SA mechanistic method for the structural design of pavement rehabilitation

Table B.5: Main characteristics of the South African mechanistic rehabilitation design method

Developed	Basis of derivation	Classification according to approach	Pavement evaluation surveys	Applicability	Input to design	Limitations (main)	Advantages (main)
South African (NITRR)	Theoretical (Linear elasticity theory)	<ol style="list-style-type: none"> 1 Application through the use of a catalogue of behaviour states or alternatively 2 Non-simplified approach using computer assisted simulation 	<ul style="list-style-type: none"> - Deflections - Cores - Test pits 	All flexible and rigid types of pavement	<ul style="list-style-type: none"> - Pavement class - Pavement type - Pavement state - Pavement layer state 	<ul style="list-style-type: none"> - Catalogue includes a limited number of analyses covering only the main trends in behaviour - No direct procedure of material characterization is given - Does not take into account variations in environmental conditions - Does not take into account past life of the pavement 	<ul style="list-style-type: none"> - Allows for analysis of all types of material in all type of layers - Allows for analysis of pavements with thin asphalt layers
Assumptions	Materials characterisation	Residual life		Basis of design life		Pavement transfer functions considered	
<ul style="list-style-type: none"> - Those applicable to the use of linear elasticity theory - State of the pavement is a function of surface deflection 	Assign moduli from tables using: <ul style="list-style-type: none"> - state of material - state of pavement - type of material 	Not discussed but: <ol style="list-style-type: none"> 1 Catalogue difficult to use for the assessment of residual life 2 Non-simplified approach is used for the assessment of various distress parameters and thus for prediction of residual life 		<ul style="list-style-type: none"> - Bituminous layers: Tensile strain at bottom of layer - Cemented layers: Tensile strain at bottom of layer - Granular layers: Stress state of layer - Subgrade: Vertical strain at the top of the layer 		<ul style="list-style-type: none"> - Stress in granular materials - Strain in bituminous and cementitious materials 	

APPENDIX C

CONCEPTS IN DECISION THEORY

APPENDIX C: CONCEPTS IN DECISION THEORY

The object of this Appendix is to give a basic introduction to the concepts of Bayesian analysis and decision trees recommended for use in the economic appraisal of alternative rehabilitation options.

In pavement rehabilitation design there is considerable doubt about the condition of the pavement after a certain period. This is due to various uncertainties such as variations in the quality of material and in construction techniques, and uncertain traffic conditions. To allow for possible deterioration in the condition of the pavement, a technique is introduced whereby the possibility of different outcomes after a certain period of time is included in the analysis.

This approach allows subjective judgment, based on experience of conditions and materials, to be incorporated in the system with observed data to obtain a balanced estimation of the expected cost of an alternative. For any given action (rehabilitation option) the engineer often has some knowledge of the possible outcome or outcomes (conditions) to be expected after a certain period of time. He may also expect one outcome to be more likely to occur than others, and therefore could subjectively assign probabilities of occurrence to each possible outcome.

For example, in Figure C.1 an illustration of a branch of a decision tree, a given action α may have three possible outcomes with values θ_1 , θ_2 and θ_3 , which, it is judged, may occur with respective probabilities of P_1 , P_2 , and P_3 .

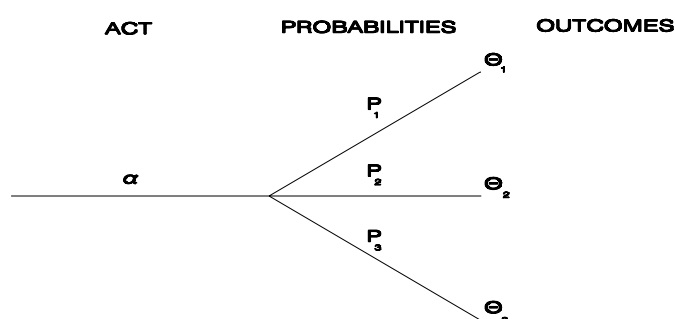


Figure C.1: Example of a given act with three possible outcomes

In this case the expected value of α is given as:

$$\begin{aligned}
 E(\alpha) &= P_1 \theta_1 + P_2 \theta_2 + P_3 \theta_3 \\
 &= \sum_i P_i \theta_i \\
 \text{where } \sum_i P_i &= 1
 \end{aligned}$$

...Equation C.1

In practice one action is generally followed by another after a certain time, for example an overlay is followed by a seal when the overlay is cracked after a time. The principle illustrated in Figure C.1 may now be extended to include a further action with its own possible outcomes, as illustrated in the decision tree in Figure C.2. The information in Figure C.2 is contained in Table C.1.

Table C.1: Resultant outcomes and actions for two time periods for any one initial action

Initial Act	Probability	Outcome	Second Act	Probability	Outcome
α_1	P_{11}	θ_{11}	b_{11}	P_{1111} P_{1112}	θ_{1111} θ_{1112}
			b_{112}	P_{1121} P_{1122}	θ_{1121} θ_{1122}
	P_{12}	θ_{12}	b_{121}	P_{1211} P_{1212}	θ_{1211} θ_{1212}
			b_{122}	P_{1221} P_{1222} P_{1232}	θ_{1221} θ_{1222} θ_{1232}
			b_{123}	P_{1231} P_{1232}	θ_{1231} θ_{1232}
	P_{13}	θ_{13}	b_{131}	P_{1311} P_{1312}	θ_{1311} θ_{1312}
			b_{132}	P_{1321} P_{1322}	θ_{1321} θ_{1322}

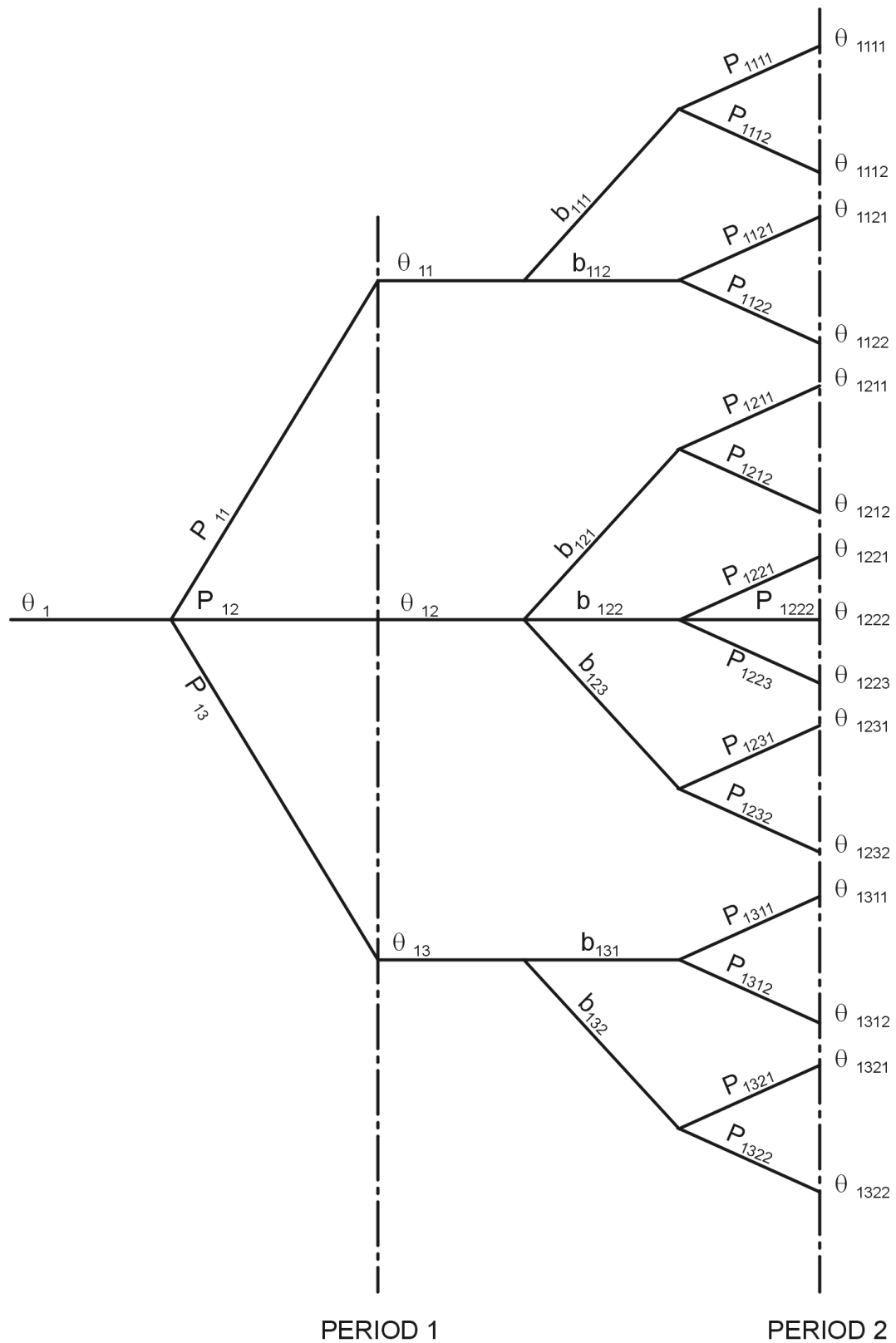


Figure C.2: Decision tree with outcomes and possible second acts for two time periods

The expected value of the example contained in Table C.1 is given as:

$$\begin{aligned}
 E(\alpha_i) = & P_{11} \max \{(P_{1111} \theta_{1111} + P_{1112} \theta_{1112}) ; (P_{1121} \theta_{1121} + P_{1122} \theta_{1122})\} \\
 & + P_{12} \max \{(P_{1211} \theta_{1211} + P_{1212} \theta_{1212}) ; (P_{1221} \theta_{1221} + P_{1222} \theta_{1222} \\
 & + P_{1223} \theta_{1223}) ; (P_{1231} \theta_{1231} + P_{1232} \theta_{1232})\} \\
 & + P_{13} \max \{(P_{1311} \theta_{1311} + P_{1312} \theta_{1312}) ; (P_{1321} \theta_{1321} + P_{1322} \theta_{1322})\}
 \end{aligned}$$

...Equation C.2

$$\begin{aligned}
 & = \sum_i P_{1i} \max \left(\sum_j P_{1i1j} \theta_{1i1j} ; \sum_j P_{1i2j} \theta_{1i2j} ; \sum_j P_{1i3j} \theta_{1i3j} \right) \\
 & \quad \text{where } \sum_i P_{1i} = 1 \\
 & \quad \sum_j P_{1i1j} = 1 ; \sum_j P_{1i2j} = 1 ; \sum_j P_{1i3j} \theta_{1i3j} = 1
 \end{aligned}$$

The example illustrated in Figure C.2 and Table C.1 represents the analysis of one rehabilitation alternative only. All appropriate options should be analysed in a similar manner. The expected values of each appropriate option, analysed over the same period, may then be compared to determine the option with the lowest expected value.

APPENDIX D

EXAMPLE: ECONOMIC ANALYSIS

APPENDIX D: EXAMPLE: ECONOMIC ANALYSIS

Example: economic analysis of applicable rehabilitation options using the concepts of decision trees and Bayesian theory.

In this hypothetical example a rehabilitation option is analysed using decision trees and the Bayesian theory to calculate the PWOC of the option. Various outcomes and appropriate actions are considered over the rehabilitation design period of 20 years.

D.1 Background

The slow lane of a pavement consisting of a 50 mm asphalt surfacing on top of two 115 mm cement treated base course (CTB) layers exhibited severe distress in the form of cracking and pumping and was identified for rehabilitation. A detailed condition assessment indicated that the distress was confined to the wearing course and the upper zone of the base.

The road was divided into uniform pavement sections according to their rehabilitation requirements. These sections were divided into those requiring:

- No rehabilitation
- Sealing of cracks
- Structural strengthening.

For this example the options applicable to the sections requiring structural strengthening and those requiring sealing of cracks are analysed to show the application of the various principles as discussed in Section 5 and Appendix C.

D.2 Sections Requiring Structural Strengthening

The subsequent rehabilitation design showed the following options to be equivalent functionally and applicable for restoring the pavement sections requiring structural strengthening:

- Option 1: Replacing the upper CTB with a virgin bitumen-treated base (BTB) and reinstating the wearing course with asphalt.
- Option 2: Replacing the upper CTB with a recycled mix, using available material reclaimed from the wearing course, and reinstating the wearing course with a virgin asphalt (40 mm) to match the existing asphalt.
- Option 3: Replacing the upper CTB with granular base material. Reinstating wearing course with asphalt (40 mm) to match.
- Option 4: Replacing the upper CTB with the milled CTB material as a granular base, treated with emulsion (+ 1,0 per cent net bitumen) to aid compaction and reduce water sensitivity. Reinstating wearing course with asphalt (40 mm) to match.

The options listed above are now economically analysed and compared (*CU*: Currency Unit):

D.2.1 Option 1

Removal of wearing course by milling, milling of upper 100 mm of CTB, replacing removed CTB with a BTB (90 mm) and replacing asphalt wearing course.

Unit costs

Milling of asphalt (30 mm)	:	CU	0.44/m ³
Milling of CTB (100 mm)	:	CU	17.68/m ³
Hauling of milled CTB to spoil (10 km)	:	CU	7.00/m ³
BTB supply & compact	:	CU	120.00/t
Asphalt wearing course	:	CU	130.00/t

Costs calculated per km for 3.7 m lane width

Milling of asphalt	:	CU	1 628.00
Milling of CTB	:	CU	6 541.60
Hauling of CTB to spoil (includes 1.3 bulk factor)	:	CU	3 174.60
BTB, supply and place	:	CU	91 908.00
Asphalt wearing course	:	CU	44 252.00
Total/lane/km		CU	147 504.20

Costs which will be common to all rehabilitation options being investigated, e.g. transportation of milled wearing course and possible seal on top of new asphalt wearing course (for uniformity of texture), have not been included in the economic comparison.

D.2.2 Option 2

Removal of wearing course by milling, milling of upper 100 mm of CTB, replacing the removed CTB with a BTB using reclaimed wearing course to constitute 25 per cent of the new mix, reinstating the asphalt wearing course.

Unit costs

Milling of asphalt (30 mm)	:	CU	0.44/m ²
Milling of CTB (100 mm)	:	CU	17.68/m ³
Hauling of milled CTB to spoil (10 km)	:	CU	7.00/m ³
Recycled asphalt (BTB)	:	CU	100.00/t
Asphalt wearing course	:	CU	130.00/t

Costs calculated per km for 3.7 m lane width

Milling of asphalt	:	CU	1 628.00
Milling of CTB	:	CU	6 541.60
Hauling of CTB to spoil (10 km)	:	CU	3 174.60
Asphalt (90 mm)	:	CU	76 590.00
Asphalt wearing course	:	CU	44 252.00
Total/lane/km		CU	132 186.20

D.2.3 Option 3

Removal of wearing course by milling, milling of upper 100 mm of CTB, replacing removed CTB with crushed stone granular base (G2) and replacing wearing course.

Unit costs

Milling of asphalt (30 mm)	:	CU	0.44/m ²
Milling of CTB (100 mm)	:	CU	17.68/m ³

Hauling of milled CTB to spoil (10 km)	:	CU	7.00/m ³
G2 crushed B/C (90 mm)	:	CU	60.00/m ³
Asphalt wearing course (40 mm)	:	CU	130.00/t

Costs calculated per km for 3.7 m lane width

Milling of asphalt	:	CU	1 628.00
Milling of CTB	:	CU	6 541.60
Hauling of CTB to spoil	:	CU	3 174.60
G2 crushed B/C	:	CU	19 980.00
Asphalt wearing course	:	CU	44 252.00
Total/lane/km		CU	75 576.20

D.2.4 Option 4

Removal of wearing course by milling, milling of upper 100 mm CTB, retaining milled material and recompacting with emulsion (+ 1,0 per cent net bitumen), replacing asphalt wearing course.

Unit costs

Milling of asphalt (10 mm)	:	CU	0.44/m ²
Milling of CTB (100 mm)	:	CU	17.68/m ³
Addition of emulsion (1.0% net, bit,ncl. spraying, mixing & compaction)	:	CU	26.00/m ³
Compaction of B/C	:	CU	6.00/m ³
Asphalt wearing course (40 mm)	:	CU	130.00/t

Costs calculated per km for 3.7 m lane width

Milling of asphalt (30 mm)	:	CU	1 628.00
Milling of CTB (100 mm)	:	CU	6 541.60
Addition of emulsion	:	CU	8 658.00
Mixing and compaction	:	CU	1 988.00
Asphalt wearing course (40 mm)	:	CU	44 252.00
Total/lane/km		CU	63 077.60

From the above it is clear that for Options (3) and (4) the initial costs are significantly lower than for the other options considered.

Past experience has shown that there should not be any significant difference in the expected life of any of the options and hence (1), (2) and (3) are rejected for economical reasons.

Option (3) has an initial cost 20 per cent greater than that of Option (4). As no real difference can be expected in the salvage value between the options, nor any difference in maintenance during the design life, the initial costs may in this case be used for comparison without a detailed economic analysis.

Option (4), is thus recommended for those sections identified for rehabilitation.

D.3 Sections Requiring Resealing

These sections are structurally sound but have open cracks allowing for the ingress of water into the pavement. Although it was considered likely that a conventional seal would prove adequate for sealing these pavement sections, other applicable options such as a bitumen rubber seal or the option to postpone treatment ("do nothing now") should also be investigated.

Unit costs assumed for the economic comparisons of the various options (given per km for 3.7 m width) are:

- Rehabilitation (according to Option (4) in the preceding section) - CU 63 000/km
- Conventional 9.5 mm reseal (CU 2.60/m²) - CU 9 600/km
- Bitumen rubber 9.5 mm reseal - CU 16 280/km
(2.7 €/m², 1.04 CU/€ sprayed, 1.60 CU/m² chipping)

D.3.1 Salvage Value Costs

The "salvage value cost" is the cost of rehabilitation required at the end of the analysis period for each option which would theoretically reinstate the road to a similar "end" condition. In this case it was decided to "reinstate" the road to a freshly rehabilitated condition, e.g. having an uncracked surface.

An accelerated test showed that even if the surfacing is kept sealed against the ingress of water, the slow lane, under its expected traffic loading, will deform to a terminal state in 20 years (requiring rehabilitation). Should the pavement be susceptible to water ingress, the pavement is expected to reach a terminal condition within five years.

D.3.2 Option 1: "do nothing"

If no remedial action is taken to seal these pavement sections, it is assumed that the pavement will require rehabilitation after an additional five years.

For this option (do nothing at present - refer to Figure D.1) two conditions are considered possible after five years, i.e.:

- Condition 1A: cracked and deformed condition with the probability of occurrence being 70 per cent (subjectively determined)
- Condition 1B: cracked condition (severe cracking and pumping) with the probability of occurrence being 30 per cent (subjectively determined).

If the pavement is cracked and deformed after five years (Option 1A), it will have to be rehabilitated (according to approach already recommended).

After a further 10 years the rehabilitated pavement may be cracked (Option 1A1, 40 per cent probability) or it may be in a sound condition (Option 1A2, 60 per cent probability).

For the cracked condition (Option 1A1) two options are considered, i.e. a conventional reseal (Option 1A1/1), which at year 20 should be sound (100 per cent), or alternatively to do nothing (Option 1A1/2), which at year 20 may be cracked (40 per cent) or cracked and deformed (60 per cent). For the sound condition (Option 1A2) no remedial action is deemed necessary and it is considered that at year 20 it may be cracked (70 per cent probability) or sound (30 per cent probability).

A similar approach of reasoning to the above is adopted for the other two options which are analysed in this example.

The analysis is done using a discount rate of 6 per cent.

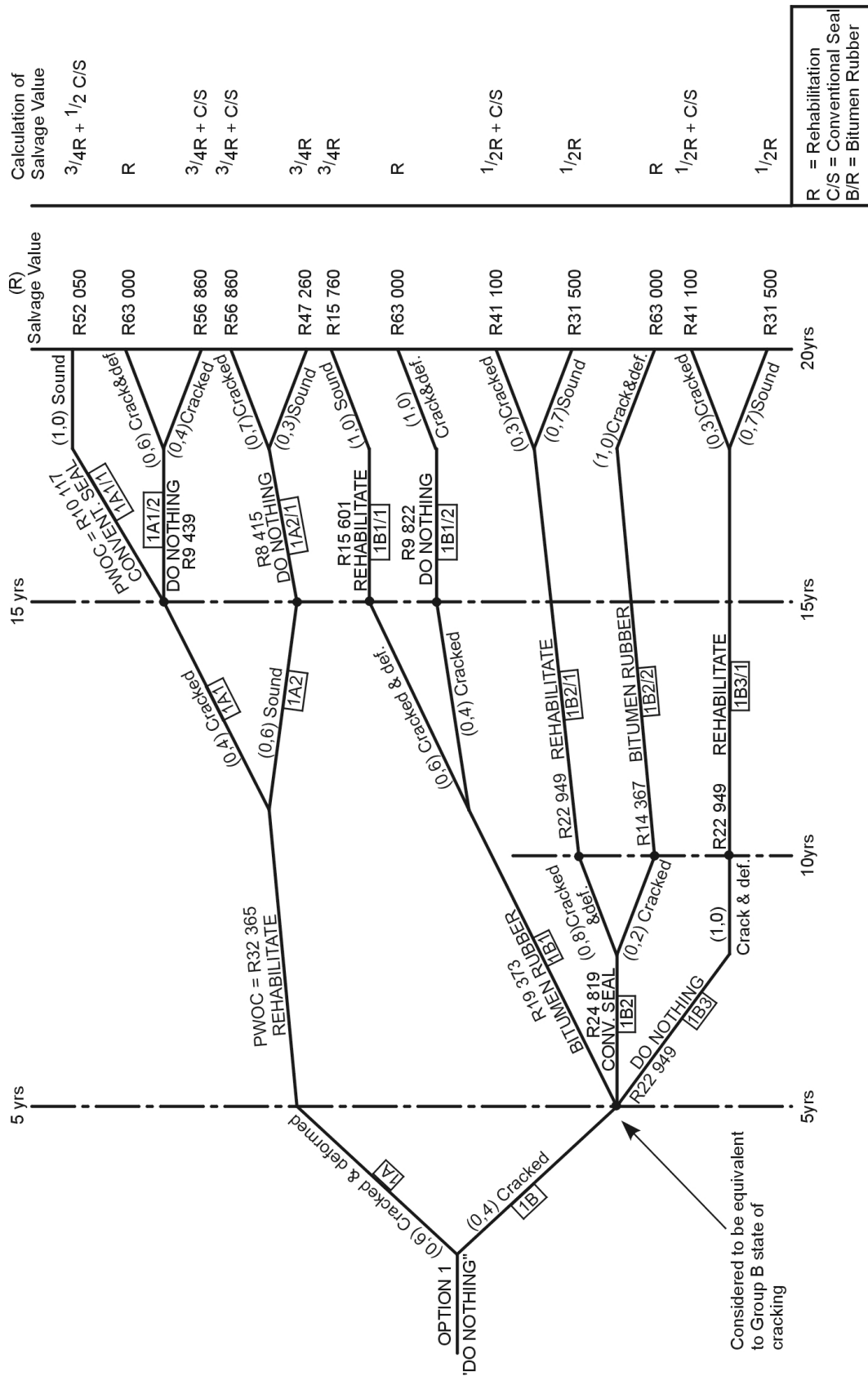


Figure D.1: Decision tree for the "do nothing" alternative

Condition 1A: Pavement Cracked and Deformed

Abbreviations used: I/C = initial cost
 B/R = bitumen rubber
 C/S = conventional seal
 R = rehabilitation

1S1/1: Initial Cost of Conventional Seal (discounted for 15 years at 6 per cent p.a.) +(E Probability of occurrence X salvage value costs discounted for 20 years).

$$\begin{aligned}
 &= \text{I/C C/S (15 yrs)} + 1.0 (\text{CU 52 050}) (20 \text{ yrs}) \\
 &= (\text{CU 9 600}) (1.06) - 15 + 1.0 (\text{CU 52 050}) (1.06) - 20 \\
 &= \text{CU 4 004} + \text{CU 16 228} \\
 &= \text{CU 20 234/km}
 \end{aligned}$$

$$\begin{aligned}
 1A1/2: & \text{I/C nil (15yrs)} + 0.6 (\text{CU 63 000})(20\text{yrs}) + (\text{CU 56 860})(20\text{yrs}) \\
 &= \text{nil} + 0.6 (\text{CU 63 000})(1.06) - 20 + (\text{CU 56 860})(1.06) - 20 \\
 &= \text{nil} + \text{CU 11 7860} + \text{CU 7 092} \\
 &= \text{CU 18 878/km}
 \end{aligned}$$

$$\begin{aligned}
 1A2/1: & \text{I/C nil (15yrs)} + 0.7 (\text{CU 56 860})(20 \text{ yrs}) + 0.3 (\text{CU 47 260})(20 \text{ yrs}) \\
 &= \text{nil} + 0.7 (\text{CU 56 860})(1.06) - 20 + 0.3 (\text{CU 47 260})(1.06) - 20 \\
 &= \text{nil} + \text{CU 12 410} + \text{CU 4 420} \\
 &= \text{CU 16 830/km}
 \end{aligned}$$

$$\begin{aligned}
 1A: & \text{I/C R (5yrs)} + 0.4 (\text{CU 18 878}) + 0.6 (\text{CU 16 830}) \\
 &= (\text{CU 63 000})(1.06) - 5 + 0.4 (\text{CU 18 878}) + 0.6 (\text{CU 16 830}) \\
 &= \text{CU 47 080} + \text{CU 7 552} + \text{CU 10 098} \\
 &= \text{CU 64 730/km}
 \end{aligned}$$

Condition 1B: Pavement Cracked

$$\begin{aligned}
 1B1/1: & \text{I/C R (15yrs)} + 1.0 (\text{CU 15 760})(20\text{yrs}) \\
 &= (\text{CU 63 000})(1.06) - 15 + 1.0 (\text{CU 15 760})(1.06) - 20 \\
 &= \text{CU 26 288} + \text{CU 4 914} \\
 &= \text{CU 31 202/km}
 \end{aligned}$$

$$\begin{aligned}
 1B1/2: & \text{I/C nil} + 1.0 (\text{CU 63 000})(20\text{yrs}) \\
 &= \text{nil} + \text{CU 19 644} \\
 &= \text{CU 19 644/km}
 \end{aligned}$$

$$\begin{aligned}
 1B1: & \text{I/C B/R (5yrs)} + 0.6 (\text{CU 31 202}) + 0.4 (\text{CU 19 644}) \\
 &= \text{CU 16 280} (1.06) - 5 + 0.6 (\text{CU 31 202}) + 0.4 (\text{CU 19 644}) \\
 &= \text{CU 38 746/km}
 \end{aligned}$$

$$\begin{aligned}
 1B2/1: & \text{I/C R (10 yrs)} + 0.3 (\text{CU 41 100}) (20 \text{ yrs}) + 0.7 (\text{CU 31 500})(20 \text{ yrs}) \\
 &= \text{CU 63 000} (1.06) - 10 + \text{CU 3 844} + \text{CU 6 876} \\
 &= \text{CU 45 898/km}
 \end{aligned}$$

$$\begin{aligned}
 1B2/2: & \text{I/C (10 yrs)} + 1.0 (\text{CU 63 000}) (20 \text{ yrs}) \\
 &= (\text{CU 16 280}) (1.06) - 10 + 1.0 (\text{CU 63 000})(1.06) - 20 \\
 &= \text{CU 28 734/km}
 \end{aligned}$$

$$\begin{aligned}
 1B2: & \text{ I/C (5 yrs) + 0.8 (CU 45 898) + 0.2 (CU 28 734)} \\
 &= (\text{CU 9 600})(1.06) - 5 + 0.8 (\text{CU 45 898}) + 0.2 (\text{CU 28 734}) \\
 &= \text{CU 7 174} + \text{CU 36 718} + \text{CU 5 746} \\
 &= \text{CU 49 638/km}
 \end{aligned}$$

$$\begin{aligned}
 1B3:/1: & \text{ I/C (10 yrs) + 0.3 (CU 41 100) (20 yrs) + 0.7 (CU 31 500) (20 yrs)} \\
 &= (\text{CU 63 000}) (1.06) - 10 + 0.3 (\text{CU 41 100}) (1.06) - 20 + 0.7 (\text{CU 31 500}) (1.06) - 20 \\
 &= \text{CU 35 178} + \text{CU 3 844} + \text{CU 6 876} \\
 &= \text{CU 45 898/km}
 \end{aligned}$$

Hence, the present worth of cost (PWOC) for Option 1 (i.e. do nothing)

$$\begin{aligned}
 &= \text{Initial Costs} + (\Sigma \text{ probability of occurrence} \times \text{PWOC of most economical remedial measure}) \\
 &= \text{nil} + 0.6 (\text{CU 64 730}) + 0.4 (\text{CU 38 746}) \\
 &= \text{CU 38 838} + \text{CU 15 498} \\
 &= \text{CU 54 336/km}
 \end{aligned}$$

D.3.3 Option 2: Conventional 9.5 mm Single Seal

A conventional single seal on a cemented base under this traffic is expected to have a life of 4 to 7 years. Since the cracking in question is not severe (sections requiring structural rehabilitation and more severely cracked sections having already been separated) the life of such a seal on these pavement sections is estimated at 7 years.

When these cracks have developed to a more severe stage, past experience indicates that a conventional reseal will retain the cracks for not more than 3 years. Bitumen rubber trails indicate that this product performs somewhat better under these circumstances.

The consequences of this option are shown in the decision tree in Figure D.2. From the decision tree the PWOC for Option 2 is calculated as follows:

$$\begin{aligned}
 2A: & \text{ I/C R (7 yrs) + 0.5 (CU 50 550)(20yrs) + 0.5 (CU 40 950)(20yrs)} \\
 &= \text{CU 56 162/km}
 \end{aligned}$$

$$\begin{aligned}
 2B1/1: & \text{ I/C R(17yrs) + 1.0 (CU 9 450)(20yrs)} \\
 &= \text{CU 26 342/km}
 \end{aligned}$$

$$\begin{aligned}
 2B1/2: & \text{ I/C nil (17yrs) + 1.0 (CU 63 000) (20 yrs)} \\
 &= \text{CU 19 644/km}
 \end{aligned}$$

$$\begin{aligned}
 2B1: & \text{ I/C B/R (7 yrs) + 0.5 (CU 26 342) + 0.5 (CU 19 644)} \\
 &= \text{CU 33 822/km}
 \end{aligned}$$

$$\begin{aligned}
 2B2/1: & \text{ I/C R (12 yrs) + 0.2 (CU 34 800) (20 yrs) + 0.8 (CU 25 200) (20 yrs)} \\
 &= \text{CU 27 734/km}
 \end{aligned}$$

$$\begin{aligned}
 2B2: & \text{ I/C C/S (7yrs) + 0.4 (CU 39 766) + 0.6 (CU 27 734)} \\
 &= \text{CU 38 930/km}
 \end{aligned}$$

$$2B: = 2B1 = \text{CU 33 822/km}$$

$$2C1: I/C \ R \ (10 \text{ yrs}) + 0.4 \ (CU \ 41 \ 100) \ (20 \text{ yrs}) \\ = CU \ 46 \ 198/\text{km}$$

$$2C2/1: I/C \ B/R \ (10\text{yrs}) + 1.0 \ (CU \ 63 \ 000) \ (20 \text{ yrs}) \\ = CU \ 23 \ 650/\text{km}$$

$$2C2/2/1: I/C \ R \ (15 \text{ yrs}) + 1.0 \ (CU \ 15 \ 750) \ (20 \text{ yrs}) \\ = CU \ 31 \ 198/\text{km}$$

$$2C2/2/2: I/C \ C/S \ (15 \text{ yrs}) + 1.0 \ (CU \ 63 \ 000) \ (20 \text{ yrs}) \\ = CU \ 31 \ 276$$

$$2C2/2: I/C \ C/S \ (10 \text{ yrs}) + 0.3 \ (CU \ 31 \ 198) + 0.7 \ (CU \ 23 \ 650) \\ = CU \ 31 \ 276/\text{km}$$

$$2C: I/C \ \text{nil} + 0.1 \ (CU \ 46 \ 198) + 0.9 \ (CU \ 29 \ 274) \\ = CU \ 30 \ 966/\text{km}$$

Hence PWOC for Option 2 (i.e. conventional seal)

$$= \text{Initial Cost} + (\text{E probability of occurrence} \times \text{PWOC of most economical remedial action})$$

$$= CU \ 9 \ 600 + 0.1 \ (CU \ 56 \ 162) + 0.7 \ (CU \ 33 \ 822) + 0.2 \ (CU \ 30 \ 966) \\ = CU \ 9 \ 600 + CU \ 5 \ 616 + CU \ 23 \ 676 + CU \ 6 \ 194 \\ = CU \ 45 \ 086/\text{km}$$



D.3.4 Option 3: Bitumen Rubber Seal

The pavement sections under consideration are structurally sound and the cracking is not severe. Hence, the bitumen rubber, if used on these sections, will not be required to provide a structural contribution.

Following the decision tree in Figure D.3 for the option, the PWOC is calculated according to the procedure used for Options 1 and 2 above.

Option 3(a) assumes that the bitumen rubber could have an extra life of 50 per cent over that of the conventional seal (i.e. 10 years as compared with 7 years) on this section with no severe cracking.

$$3A: I/C \text{ R (10 yrs)} + 0.4 (CU \text{ 41 100})(20 \text{ yrs}) + 0.6 (CU \text{ 31 500})(20 \text{ yrs})$$

$$= CU \text{ 46 198/km}$$

$$3B1: I/C \text{ R (10 yrs)} + 1.0 (CU \text{ 63 000})(20 \text{ yrs})$$

$$= CU \text{ 28 734/km}$$

$$3B/2/1: I/C \text{ R (15 yrs)} + 1.0 (CU \text{ 15 750})(20 \text{ yrs})$$

$$= CU \text{ 31 198/km}$$

$$3B2/2: I/C \text{ C/R (15 yrs)} + 1.0 (CU \text{ 63 000})(20 \text{ yrs})$$

$$= CU \text{ 23 650/km}$$

$$3B2: I/C \text{ C/S (10 yrs)} + 0.4 (CU \text{ 31 198}) + 0.6 (CU \text{ 23 650})$$

$$= CU \text{ 31 998/km}$$

$$3B: 3B1 = CU \text{ 28 734/km}$$

$$3C1/1: I/C \text{ R (15 yrs)} + 1.0 (CU \text{ 15 750})(20 \text{ yrs})$$

$$= CU \text{ 31 198/km}$$

$$3C1/2: I/C \text{ C/S (15 yrs)} + 1.0 (CU \text{ 63 000})(20 \text{ yrs})$$

$$= CU \text{ 23 648/km}$$

$$3C1: I/C \text{ nil (10 yrs)} + 0.4 (CU \text{ 31 198}) + 0.6 (CU \text{ 23 650})$$

$$= CU \text{ 26 670/km}$$

Hence PWOC for Option 3(a) i.e. sealing with bitumen rubber

$$= \text{Initial Costs} + (\text{E probability of occurrence} \times \text{PWOC of most economical remedial measure})$$

$$= I/C \text{ B/R} + 0.1 (CU \text{ 46 198}) + 0.7 (CU \text{ 28 734}) + 0.2 (CU \text{ 26 670})$$

$$= CU \text{ 16 280} + CU \text{ 4 620} + CU \text{ 20 114} + CU \text{ 5 334}$$

$$= CU \text{ 46 348/km (assuming that the bitumen rubber could have an extra life of 50 per cent over the conventional seal over not too severe cracks).}$$

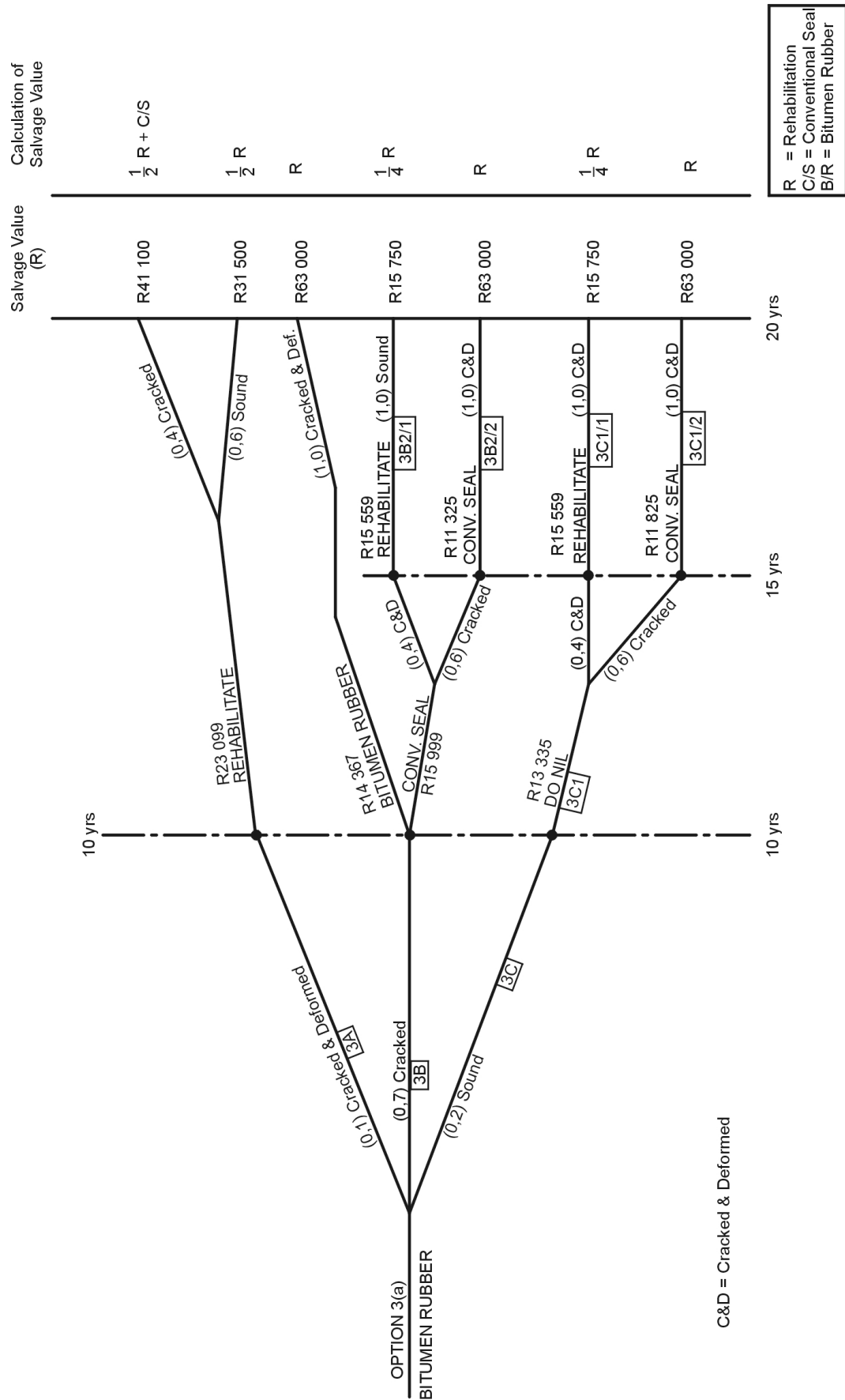


Figure D.3: Decision tree for Option 3(a) (bitumen rubber)

Option 3(b) assumes that, while the cracks are only of a lesser severity and no structural value is required from the bitumen rubber, the weathering characteristics of the bitumen rubber are the critical criterion. The assumption is that the bitumen rubber will oxidise within the same period as the conventional seal (since there is no proof to the contrary at present). When the cracking has become more severe it is suggested that the bitumen rubber will perform better than a conventional seal.

Hence, using the decision tree used for Option 2, only the initial cost changes. It follows that:

PWOC for Option 3(b) = CU 51 766/km.

Discussion of options for pavement sections exhibiting less severe cracking:

Option 1 - "do nothing now" : CU 54 336/km

Option 2 - conventional seal : CU 45 086/km

Option 3(a) bitumen rubber (assuming a 50 per cent longer life on not severe cracks) :
CU 46 348/km

Option 3(b) bitumen rubber (assuming no longer life on not severe cracks): CU 51 766/km

From the above results of the economic analysis of the various options it is clear that the "do nothing" option is not to be recommended.

The most economical option was found to be a conventional seal.

The use of bitumen rubber only becomes economically equivalent to a conventional seal if it can be assured that at least a 50 per cent increase in life will be achieved on the pavement sections exhibiting less severe cracking.