BITUMINOUS PAVEMENT REHABILITATION DESIGN

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SUMMARY

South Africa is a country fortunate in possessing a well developed network of paved roads that provides a basic and often overlooked facility for the social and economic welfare of the country. However, an accute shortage of funds available for pavement rehabilitation is endangering the integrity of this network, making continuous research for improved and more economical rehabilitation procedures necessary. I believe that this thesis, provides in a need for improved techniques, as a uniform approach is advocated, incorporating the best available procedures applicable throughout the country.

In compiling the document, specific attention was given to current South African practice, ensuring that the recommended procedures are based on the use of readily available equipment and techniques familiar to engineers in this country. Guidelines to all the main stages of rehabilitation investigations are contained in a logical and systematic procedure of investigation, evaluation, analysis, rehabilitation design and finally, economic appraisal of applicable options. In the process full use is made of past pavement behaviour and pavement condition, thereby making possible an early assessment of additional information needed. Although evaluation and analysis procedures are suggested for use, no formal method in this regard is excluded, provided the method is relevant to the case and information obtainable will benefit the analysis in terms of the probable influence on the best rehabilitation strategy to follow. By assessment of the value of information obtainable, much emphasis is placed on the optimum utilization of available resources in designing the best applicable remedy to an existing problem.

This procedure is demonstrated by the inclusion of a study that I undertook in co-ordination with the Transvaal Roads Department for the NITRR. This case study demonstrates the applicability of the various evaluation, analyses and rehabilitation design methods, and the use of decision trees in an economic analysis to select the best applicable rehabilitation option.

In conclusion the consequences of implementation to date are investigated. Recommendations for further research and possible improvements for future revisions of the recommended procedure are finally looked at.

SAMEVATTING

Suid-Afrika beskik oor 'n goed ontwikkelde geplaveide padnetwerk wat 'n belangrike bydra lewer tot die sosiale en ekonomiese welvaart van die land. Die huidige tekort aan beskikbare fondse vir plaveiselrehabilitasie bedreig egter die integriteit van hierdie netwerk. Gevolglik geniet navorsing met die klem op die daarstelling van verbeterde en meer ekonomiese rehabilitasie prosedures, hoë prioriteit. In die tesis word geskikte metodes saamgevoeg in 'n uniforme benadering tot plaveiselrehabilitasie in 'n poging om in dié bestaande behoefte te voorsien.

Met die samestelling van die tesis is spesifieke aandag geskenk aan huidige Suid-Afrikaanse praktyk. Hierdeur is verseker dat die aanbevole prosedures gebaseer is op die gebruik van beskikbare toerusting en benutting van tegnieke wat aan die plaaslike ingenieur bekend is. Riglyne vir alle fases in die ondersoek oor plaveiselrehabilitasie is saamgevoeg in 'n sistematiese proses van evaluering, analise, ontwerp sowel as in 'n ekonomiese analise van geskikte alternatiewe. Voorsiening word gemaak vir die volle benutting van die gedragsgeskiedenis van die plaveisel, om sodoende vroegtydig te bepaal welke addisionele inligting benodig sal word in die ondersoek. Die gebruik van geen formele analisemetode word uitgeskakel nie, solank die metodes geskik is vir die spesifieke doel, en die inligting daardeur bekom 'n wesenlike bydra sal lewer tot die keuse van 'n geskikte rehabilitasiemetode. Deur die waarde van bekombare inligting te ontleed word die optimale benutting van beskikbare middele verseker.

Die toepassing van die voorgestelde prosedure word geïllustreer deur die insluiting van 'n studie oor plaveiselrehabilitasie wat vir die NIVPN in samewerking van die Transvaalse Paaiedepartement onderneem is. Met die studie en ander ondersoeke as agtergrond, word die implementering van die voorgestelde metode ondersoek. Ten slotte word aanbevelings vir toekomstige navorsing gemaak met die oog op moontlike verbeterings in die voorgestelde prosedure.

AAN :

MY OUERS :
BOB EN DEVINA

wat in my die trots en wil om te bekwaam gevestig het

dankie

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In particular, my immediate supervisor Mr V P Servas, Assisting Group Head of the Maintenance and Construction Group of the NITRR for his encouragement, ideas and guidance.

My family for their support and patience.

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PREFACE

Following my graduation from the University of Pretoria and compulsory military service, I joined Malcolm Grant's* section, pavement rehabilitation research, in the Maintenance and Construction (M&C) group of the NITRR in January 1980. In April 1981 I was seconded to Hawkins, Hawkins and Osborn for a period of one year to gain site experience on a pavement construction project.

On my return to the NITRR, I joined Vladis Servas's* section in the M&C group and became involved in the writing of a manual in the Technical Recommendations for Highway series, i.e. draft TRH12 - Bituminous pavement rehabilitation design[1A]. This document was based on:

- existing documentation, including :
  - a number of previous draft documents by Malcolm Grant; these documents never met with the approval of the Highway Materials Committee (HMC) and were consequently not published;
  - a paper by M C Grant and P C Curtayne*[1], containing the philosophy on which the draft TRH12 is based;
  - an NITRR Technical Report by V P Servas and P C Curtayne[2], setting out the proposed approach and structure of the document;
  - an NITRR Technical Report by P C Curtayne and V P Servas[3] on the economic considerations involved in the design of pavement rehabilitation;
- extensive literature survey;
- series of discussions and exchange of ideas with members of the sub-committee on the draft TRH12 and with colleagues at the NITRR; and

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* Senior Chief Research Officers of the NITRR

- research into the applicability of various proposed sections of the document and into the consequeces of its implementation.

Much original thought went into structuring the ideas and approaches into a procedure covering the various aspects of pavement rehabilitation. During the preparation of this document I was involved in the writing of several other documents (see references 4-12). The work done during this period forms the bulk of this thesis.

CONTENTS

LIST OF FIGURES

LIST OF TABLES

SECTION 1

Introduction

1. INTRODUCTION

1.1 Background

The South African road network consists of more than 226 000 km of roads (rural and municipal) of which more than 77 000 km are paved. The total replacement value was estimated in 1981[13] at about R22 000 million. The importance of the road network is often forgotten and only attracts attention when an inadequate service is provided, or when sections of the network deteriorate to such an extent that the riding surface becomes unacceptable. However, when the network is allowed to deteriorates to a sub-critical level it requires much higher sums to rectify, as illustrated in Figure 1. Sub-critical conditions on South African roads have up to now been kept in check because of the commendable efforts of road authorities despite restricted funds.

However, 36 per cent of the paved rural network was built before 1960 and as it is approaching the end of its design life it will require major rehabilitation in the near future. A study by the South African Road Federation (SARF)[13] showed that the annual amount needed to preserve the integrity of the network was R520 million (1981 prices). In actual fact, the total amount spent on rehabilitation in 1981 was only R191 million, or 32 per cent of the SARF's estimated need. It is foreseen that this imballance will prevail or even worsten in future as more demands are made on available capital, e.g. defence, housing and education. Because of the critical shortage of funds for pavement rehabilitation, efforts should be concentrated on using the available funds more effectively.

Until recently the different road authorities in South Africa had no uniform strategy for the identification of roads in need of rehabilitation, or rehabilitation procedure to follow. This position improved with the development and implementation of pavement management systems (PMS) by some authorities, thereby improving the ability to identify roads which are in an inferior

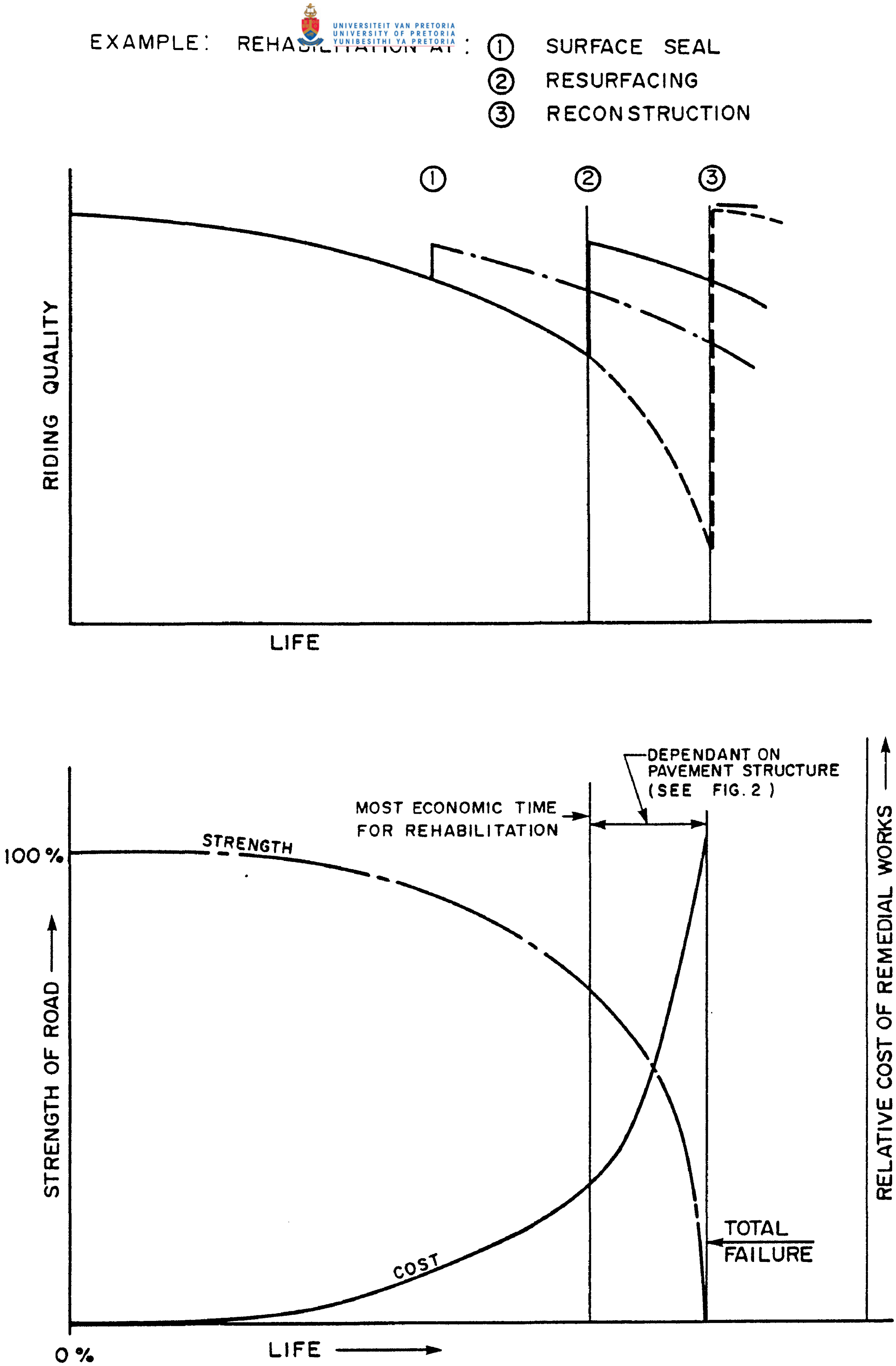

FIGURE 1 (14) (15)

ILLUSTRATION OF THE CRITICAL EFFECT OF THE TIME OF REHABILITATION ON THE COST OF REHABILITATION

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condition. However, rehabilitation procedures followed on roads identified for rehabilitation are still based on internal policies. Each road authority applies its own criteria, resulting very often in the adoption of a "blanket approach" (i.e. following a predetermined policy rather than providing for a specific need), resulting in over providing for most sections on a road. With limited funds available, this is hardly a system South Africa can afford.

1.2 Objectives and approach

The objectives of this thesis are :

- to present a systematic and logical rehabilitation design procedure applicable to South Africa;
- to present an example of the successful implementation of this pavement rehabilitation procedure;
- to examine the consequences of the implementation;
- to identify the need and potential for improvements;
- to recommend areas for improvement and for future research.

This thesis gives specific attention to current South African practice in an effort to produce a uniform rehabilitation practice. The procedures recommended are based on the use of equipment and techniques familiar to engineers in this country. Consequently, the recommendations should be readily applicable here, demonstrated by the inclusion of an example of implimentation of the methods.

To achieve these objectives, the rest of the thesis is divided into the following sections:

- the philosophy and approach in which pavement rehabilitation is defined and the recommended procedure outlined;

- the initial assessment, in which the "first", investigation step in pavement rehabilitation is discussed;
- detailed assessment, the objective of which is to identify the cause and mechanism of distress and to determine the structural capacity of the pavement;
- the rehabilitation design, in which various approaches are discussed, based on the identified cause and mechanism of distress;
- the economic analysis,through which the best rehabilitation procedure is selected, taking into account the economic consequences of each procedure;
- the implementation of recommended approach, in which a complete case study is given;
- the epilogue, in which the consequences of the implementation are discussed before final conclusions are reached.

In structuring the thesis, I found it necessary to include several appendices, each dealing with important aspects of the design procedure. These are :

- an appendix on the detailed visual inspection[4]- this is an essential part of the initial assessment and provides for the detailed recording of any visible distress and of clues to the cause and mechanism of distress;
- an appendix on criteria and procedures for pavement evaluation[8]- this appendix recommends criteria for describing the condition of pavements in South Africa, taking into account factors such as the category of road and the pavement structure;
- an appendix containing a rehabilitation design catalogue;
- an appendix on the principles involved in taking decisions; these principles provide the basis for the rational decision-making required in the application of the rehabilitation design procedure;
- an appendix on statistical concepts in which the necessary statistical formulae needed in the application of the procedures are given.

SECTION 2

Philosophy and approach

2.1 Rehabilitation defined.

2.2 Pavement behavior.

2.3 Design consederations.

2.4 Recommended approach.

2.4.1 The initial assessment.
2.4.2 The detailed assessment.
2.4.3 Rehabilitation desing procedurers.
2.4.4 The economic analysis.

2. PHILOSOPHY AND APPROACH

2.1 Rehabilitation defined.

Servas and Curtayne[2], defined pavement rehabilitation as involving "measures used to improve, strengthen or salvage existing deficient pavements so that these may continue, with routine maintenance, to carry traffic with adequate speed, safety and comfort." Maintenance in this regard will entail routine activities, carried out by road authorities' own labour force, on the surface and shoulders of pavements.

A pavement is usually identified for rehabilitation by the different road authorities on the basis of information received from their regional office's as part of a pavement manegement system (PMS). Usually this information is related to the general visual condition and rideability 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. The main categories of rehabilitation measures, identified by Servas and Curtayne[2], include :

(i) complete pavement reconstruction;

(ii) partial reconstruction involving the strengthening of existing pavement layers, with or without stabilization, before resurfacing;

(iii) asphaltic or granular overlays;

(iv) surfacing rehabilitation;

(v) provision of drainage, and/or improvements to existing drainage facilities.

Furthermore, they indicated that within each category several options may exist. With all these alternatives available, the use of a rehabilitation procedure must enable the identification of the most applicable option from both a structural and economical point of view.

2.2 Pavement behaviour.

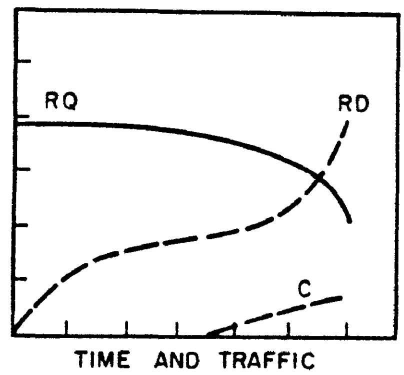
Pavements deteriorate with time due to climatic and traffic conditions. However, distinct differences in behaviour occur, largely due to differences in material characteristics. In South Africa three main structural types (identified in TRH4) are of importance in bituminous pavement rehabilitation investigations, each with its distinct manivestations of distress. External factors such as poor drainage facilities, may have a severe infuence on these generalized characteristics and should be taken into account in the process of cause identification. These types of pavement structures are :

Untreated granular base pavements : With untreated sub-layers this type of pavement generally, with time, exhibits deformation due to shear failture or dencification of materials. With stabilized sub-layers where there is little likelyhood of rutting or reflective cracking, aging or fatigue of the surfacing is the most common distress manifestation.

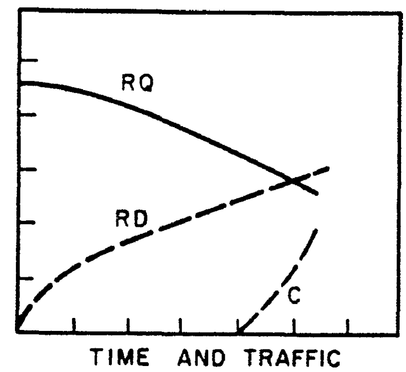
Timely rehabilitation of the cracked surface is often critical as water ingress can result in the rapid deterioration of the pavement as is clear from Figure 2(a).

Bituminous base pavements : From Figure 2(b) it is clear that deterioration in the wheeltracks (rutting) is the most common distress observed. This may originate in either the bituminous or untreated materials. In event of tar in the base course, fatigue cracking of the base could prove the critical factor.

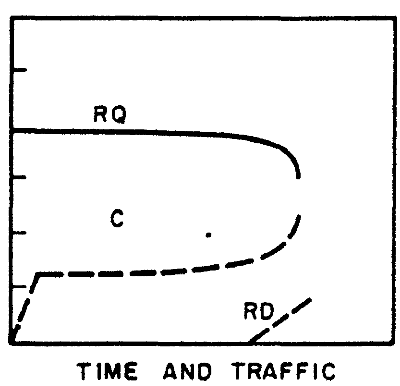
Cemented base pavements : The layer usually fails in tention as the material is subject to fatigue. Typical reflective block cracking may appear early in the pavement due to the mechanism of shrinkage and thermal stresses in the cemented layer, as is clear in Figure 2(c). Traffic will, with time, result in secondary block cracking and the ingress of water through surface cracks may result in pumping and the general deterioration of the pavement.



a) GRANULAR BASE



b) BITUMINOUS BASE



c) CEMENTED BASE

RQ = RIDING QUALITY (PSI)
RD = RUT DEPTH
C = CRACKING (DEGREE + EXTENT.)

NOTE: THESE CHARACTERISTICS HAVE BEEN VALIDATED BY HEAVY VEHICLE SIMULATOR TEST RESULTS

FIGURE 2

GENERALIZED PERFORMANCE CHARACTERISTICS EXPECTED FOR THREE BITUMINOUS PAVEMENT TYPES

940-4-4 307/1 BS (KR)

2.3 Design considerations.

Grant and Curtayne[1] identified the basic design considerations for establishing a rehabilitation procedure. The importance of the formal incorporation of the past pavement behaviour into a future design method was recognized as they declared that rehabilitation design has a major advantage over new pavement design, as many of the design variables, assumed in the case of new pavements, can be measured on the existing pavement and calibrated against actual pavement behaviour. In many rehabilitation procedures this advantage is often not fully exploited.

The behaviour of pavements may differ radically from that generally accepted, due to material or local conditions. These, identified by Grant and Curtayne[1],include :

- drainage problems;
- non-traffic-induced cracks and the action of pumping under traffic;
- inadequate in-situ properties of pavement materials;
- expansive subgrades.

The use of formal procedures in evaluating pavements by using elastic property measurements (e.g. deflections) against empirical criteria (often found in overlay design) do not provide for the identification of these "unexpected" causes of distress. Although some procedures may provide for the recognition of these causes, past behaviour is often neglected.

The aim of pavement rehabilitation, according to Grant and Curtayne[1], can be regarded as to modify the behaviour of a pavement. They further indicated that the more fully this behaviour is evaluated, the more accurate and therefore the more economical the rehabilitation should be. Because of the consequences of an incorrect or incomplete assessment, it is important for the evaluation process to be a formal part of the rehabilitation procedures.

Many types of tests are available for use in determining the condition of the existing pavement. However, it is necessary to be very selective in the types and numbers of tests in order to keep the analysis within acceptable logistic and economic limits. Some uncertainties will remain, even after extensive testing and it is necessary to realize that the future behaviour of a pavement cannot be predicted with certainty, due to factors such as the followiing, identified by Grant and Curtayne[1] :

- 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.

In selecting appropriate pavement material tests the designer should progressively consider the contribution of the expected additional information. The value of the information can be weighed up with the help of decision trees and Bayesian analysis (see Appendix 4). These statistical concepts are easy to apply 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, according to Grant and Curtayne[1], often be assessd well enough for practical purposes by considering the cost of obtaining additional information in relation to :

- the consequence of not making the optimum decision (in terms of cost and performance of rehabilitation measure);
- the probability of not making the optimum decision without additional information;
- the probability of not making the optimum decision in spite of having additional information.

Experience and engineering judgement can be used to assess these factors and accordingly decide whether or not further tests are justified.

In complement to the above considerations Servas and Curtayne[2] realized that in practice, it cannot be assumed that the pavement under consideration for rehabilitation is uniform in respect to rehabilitation need. Where possible a pavement should be divided into seperate viable sections requiring different remedial treatments. They identified the following sections that may possibly exist within a lenght of road :

- sections distressed due to localized problems;
- sections exhibiting problems limited to the surfacing;
- sections with no problems;
- sections with severe problems (i.e. extending to below the surfacing).

In the absence of a procedure allowing for the functional division of the pavement as suggested, an authority is likely to adopt a blanket approach. This will usually provide for the worst conditions on the road.

Taking the above into account, Servas and Curtayne[2] suggested a rehabilitation design method represented in Figure 3. They foresaw three detailed steps in the proposed method, i.e.

- the initial assessment;
- successive detailed assessments; and
- the economic analysis.

The above formed the basis for the development of the recommended procedure contained in this document. In this work I introduced several important changes. The main alterations involved :

- the detailed structuring of the initial assessment;
- the splitting of the proposed detailed assessment into two sections, e.g. the detailed assessment and the rehabilitation design procedures;
- the restructuring of the detailed assessment.

These changes will become clear with the discussion of the recommended approach.

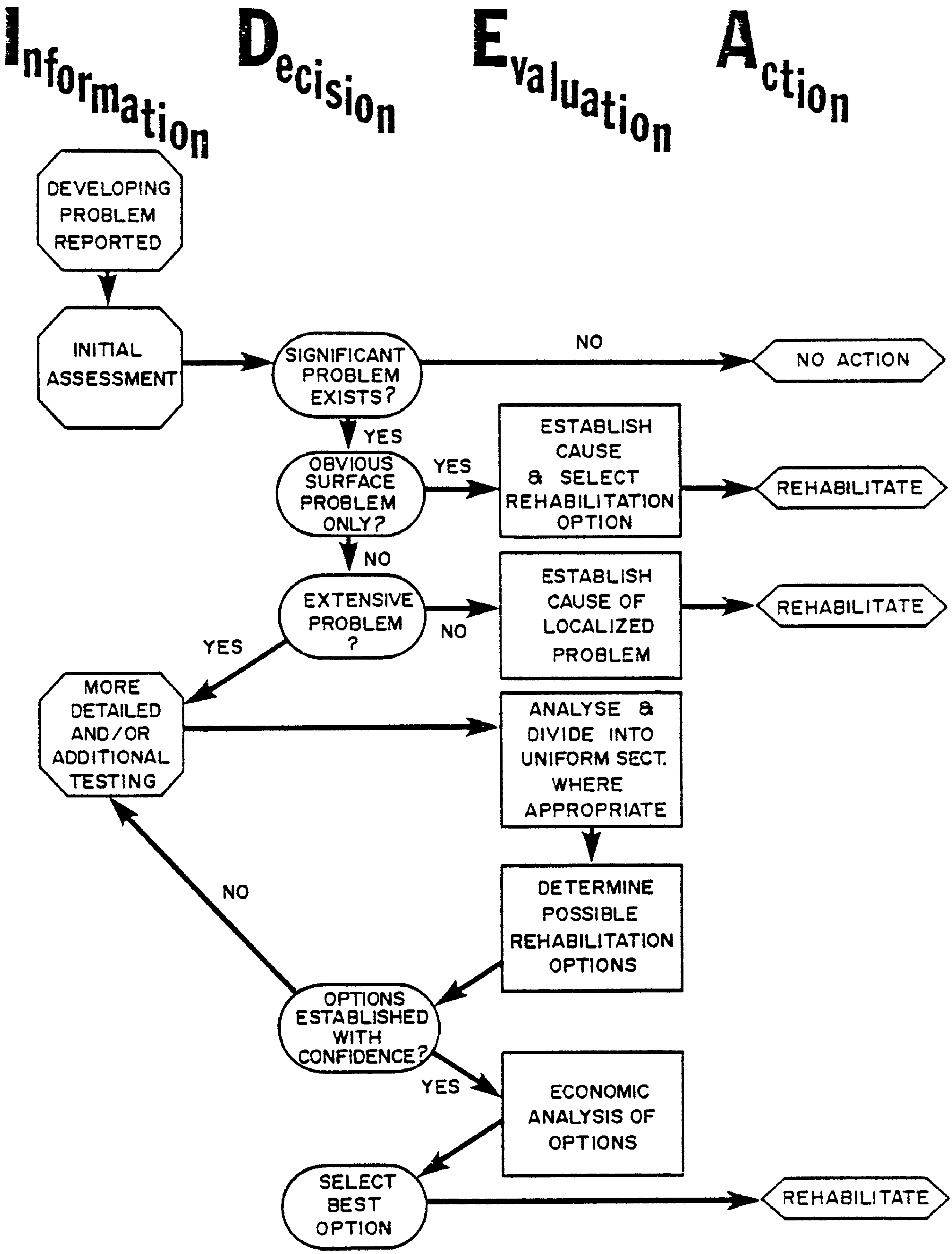


FIGURE 3

FLOW DIAGRAM OF REHABILITATION DESIGN PROCEDURE
(SERVAS & CURTAYNE)

2.4 Recommended approach.

The objectives of the design procedure recommended in this document are :

- to divide the pavement into distinct lenghts requiring different rehabilitation measures;
- to determine the cause and mechanism of distress of each uniform length of pavement;
- to design applicable rehabilitation alternatives for each uniform length of pavement;
- to determine the best rehabilitation design based on economical considerations.

These objectives are met by a systematic and logical procedure, entailing the evaluation and analysis of the pavement in increasing detailed steps before embarking on the actual rehabilitation design. This process involves the following:

(1) the initial assessment;
(2) the detailed assessment;
(3) rehabilitation design procedures; and
(4) the economic analysis.

The recommended design procedure is outlined in Figure 4.

2.4.1 The initial assessment

The objectives of the initial assessment are : (based on Servas and Curtayne[2])

(a) to provide :

- a complete record of the behaviour and condition of the pavement;
- input to any further investigations required on sections identified for further testing.

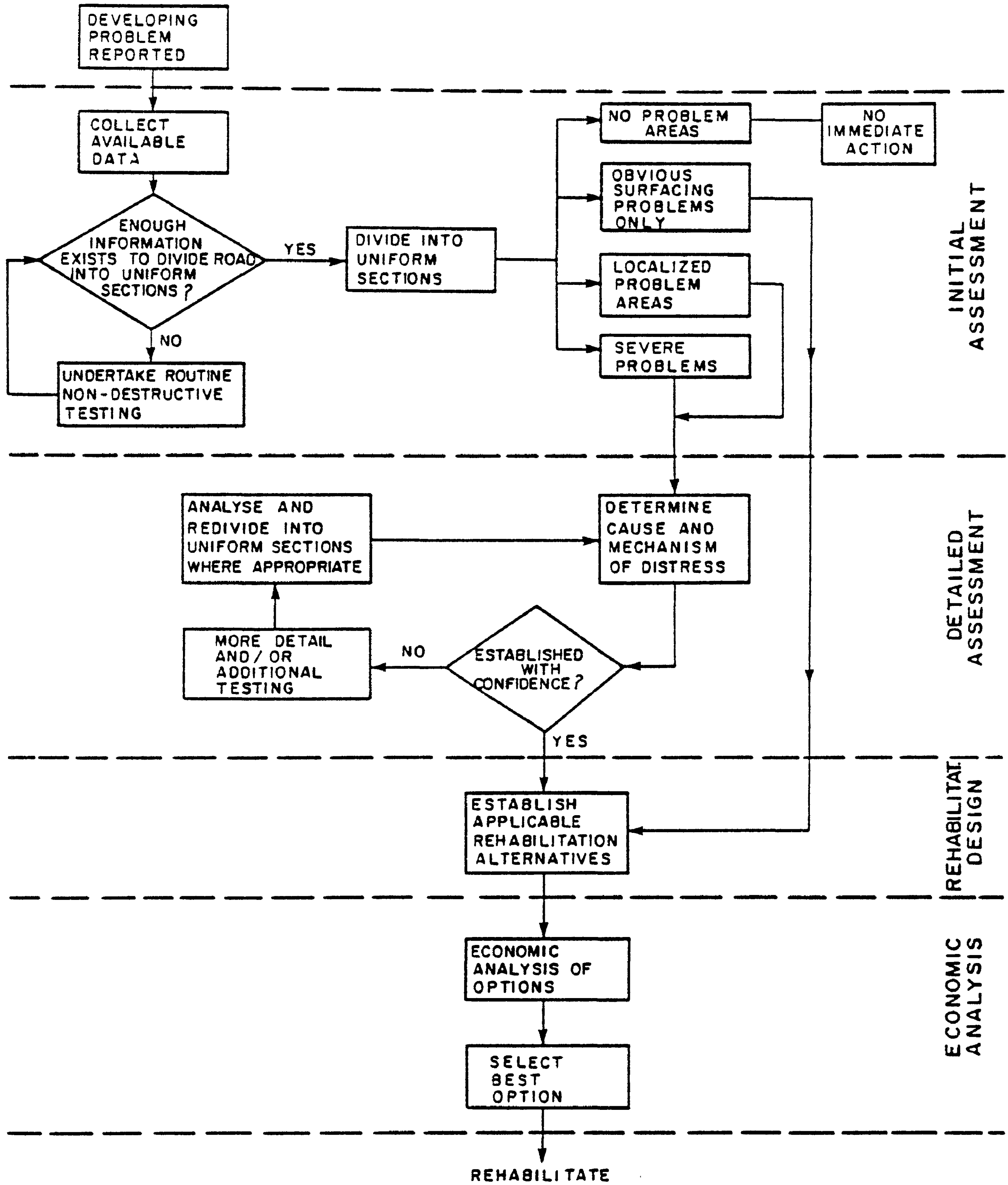


FIGURE 4

FLOW DIAGRAM OF THE RECOMMENDED REHABILITATION DESIGN PROCEDURE

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(b) to identify :

- sections requiring no action within the next 5 year period;
- sections exhibiting an obvious surfacing only problem;
- sections with localized problems;
- sections with probable structural inadequacies identified for further detailed investigations (including sections that may require attention within the next 5 year period).

(c) to suggest and/or recommend :

- possibilities as to the cause(s) and mechanism(s) of distress;
- appropriate furher testing as verification of the possible cause(s) and mechanism(s) of distress;
- where possible and/or applicable, appropriate rehabilitation design procedures.

The initial assessment will normally entail a preliminary site visit and an examination of available records, which will determine the need for obtaining any further information through routine non-destructive surveys of the pavement as well as a detailed visual inspection (Appendix 1). If a good PMS is in operation, enough information is likely to be available and no further surveys will be necessary.

2.4.2 The detailed assessment

The initial assessment will have identified lengths of pavement in probable need of structural improvement. With relation to these sections,the objectives of the detailed assessment are :

- to identify the cause(s) and mechanism(s) of distress;
- to determine the structural capacity.

Distress may originate in any component of a pavement. For example, the deformation of the surface may have been caused by deformation of the subgrade or any of the pavement layers.

Moreover, in each case there may be a number of reasons for the distress. For effective rehabilitation design the origin and reason for distress should be detemined with reasonable confidence.

2.4.3 Rehabilitation design

The objective of this section is :

- to establish all appropriate and applicable rehabilitation alternatives for each uniform pavement length.

The cause(s) and mechanism(s) of distress, will determine the applicabillity of a design. Each contributing distress factor should be considered in determining possible rehabilitation options.

2.4.4 The economic analysis

The object of this section is :

- to determine the best applicable rehabilitation method for each uniform pavement length.

The best rehabilitation option is selected by examining the cost and consequences of all the available options on a present worth of cost basis, taking into account factors such as initial cost, maintenance costs, road user costs and the salvage value of the pavement.

SECTION 3

Initial assessment

3.1 Objectives and scope

3.2 Gathering of information

3.2.1 The preliminary investigation
3.2.2 The detailed visual inspection
3.2.3 Pavement surveilance

3.3 Processing and evaluation of data

3.3.1 Sections with no significant problems
3.3.2 Sections with obvious surfacing problems
3.3.3 Sections showing localized distress
3.3.4 Sections where structural improvements may be required

3. INITIAL ASSESSMENT

3.1 Objectives and scope.

The initial assessment forms the foundation of the design procedure recommended in this document. Its primary function is to reduce the amount of further detailed assessment needed. This is accomplished by:

- recording and processing 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 the sections that obviously exhibit only surfacing distress or isolated distress; and
- suggesting appropriate tests with which to begin the detailed assessment of the sections requiring possible structural improvements.

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

- the gathering of information; and
- the data processing and evaluation.

The recommended approach for the initial assessment is outlined in Figure 5. From this figure it is clear that the gathering of information and the processing and evaluation of data are integrated to allow for the optimum use of available data before embarking on any further testing of the pavement.

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

- the class of road;
- the resources available;
- the quantity and nature of the data already available concerning the pavement.

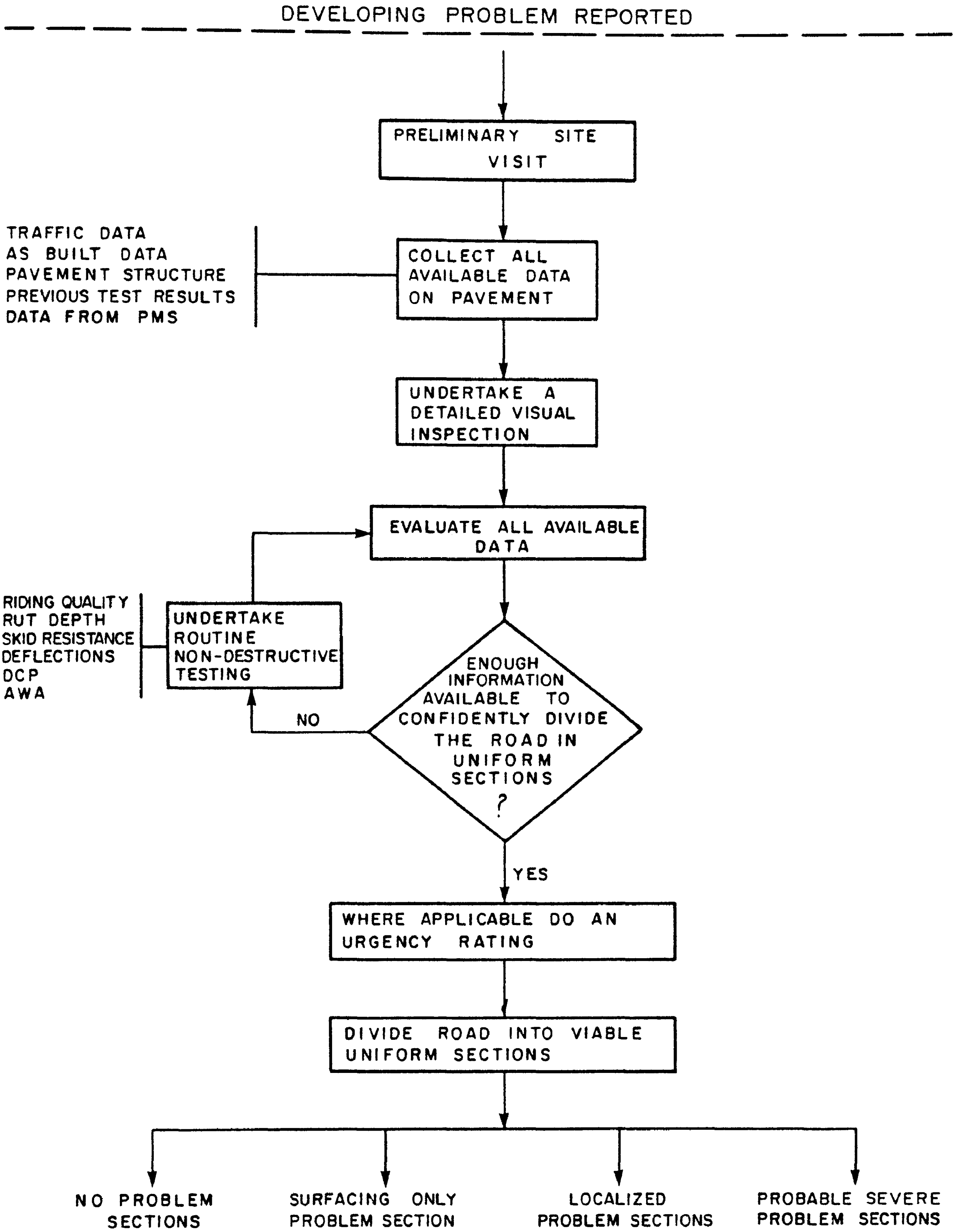

FIGURE 5

FLOW DIAGRAM OF THE INITIAL ASSESSMENT

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3.2 Gathering of information.

In the initial assessment information is usually obtained by :

- preliminary investigations;
- a detailed visual inspection;
- pavement surveilance measurements.

3.2.1 The preliminary investigation

This investigation entails obtaining all available information such as in-built data, pavement structure data and traffic loadings on the pavement, as well as any results from tests previously done on the road.

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

3.2.2 The detailed visual inspection

This inspection is considered an essential part of the initial assessment, as the information contained in PMS's is considered to be generally of inadequate detail for use in a project level study such as this. It involves the recording of visible manifestations of distress and, of particular importance, visual clues to the cause and mechanism of distress such as details of construction, topography and geology. Care should be taken to record the details in a way compatible with the eventual evaluation of the data.

Appendix 1 gives a guide for the undertaking of visual inspections.

3.2.3 Pavement surveilance

The need for obtaining any further information through routine, non-destructive surveys of the pavement by equipment such as the

PCA roadmeter (riding quality) or Lacroix deflections, is determined by the detail and volume of information uncovered by the preliminary investigation. Enough information must ultimitaly exist to allow for the confident division of the pavement into uniform sections. If a good PMS is in operation, enough information is likely to be available and no further surveys would be necessary.

3.3 Processing and evaluation of data.

In the evaluation process a decision must be taken on the need for further testing. Only routine non-destructive surveys are considered applicable in the initial assessment, but even with this restriction, several parameters can be measured. To ensure a correct decision on the type of information needed Appendix 2, in which the parameters usually measured in South Africa are discussed briefly, should be consulted.

For purposes of evaluation, available data should be processed into a quantified form. This is done by using criteria, representative of the condition of the pavement, in the processing of the individual parameters. Appendix 2 gives an extensive list of criteria applicable in South Africa. These criteria are not intended to be used mechanically but should be applied with sound engineering judgement.

Appendix 2 also gives practical guidelines to the combination and presentation of all relevant information for the evaluation of a pavement. Where applicable, for practical reasons, the evaluation should also include an urgency rating such as that given in Table 1. This will allow the road authority concerned, before embarking on any detailed investigations, to compare different rehabilitation projects with ease, to astablish priorities and to re-assess its commitments.

TABLE 1 - Urgency rating for pavement evaluation

Urgency	Required remedial action
1	No further action required in foreseeable future
2	Re-investigate in 5 years' time
3	May require action within 5 years' -(a holding action may prove appropriate)
4	Rehabilitation should not be postponed
5	Urgently in need of rehabilitation

In accomplishing the set objectives the evaluation should enable the following uniform sections to be identified on the road :

3.3.1 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 next 5 years (urgency rating of 1 and 2).

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 on them is not just being delayed for some reason.

3.3.2 Sections with obvious surfacing-only problems

Sometimes 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 related solely 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 addition, porous or permeable surfacings can aften cause distress and should therefore be recognized as a surfacing problem. Usually bleeding, polishing and ravelling can be identified as surfacing only problems on the basis of information obtained during the initial assessment. Further detailed assessments are invariably necessary before one can be confident that cracking and/or deformation are surfacing-only problems.

In the absence of other forms of distress, areas exhibiting ravelling, polishing or bleeding, can be classified as having surfacing-only problems. Further tests on the surfacing material are usually needed to establish the appropriate rehabilitation alternatives. However, these tests should not be considered as part of the subsequent detailed assessment phase.

The selection and design of maintenance seals and asphalt surfacing do not fall within the scope of this thesis - these aspects are dealt with in detail in various existing documents such as TRH3, TRH7 and TRH8.

3.3.3 Sections showing localized distress

The identification of sections where localized factors are the cause of distress is important for two main reasons.

- Firstly, to prevent an erroneous assessment of the overall structural capacity of the road and subsequent unnecessary or excessive rehabilitation.

- Secondly, to prevent the application of rehabilitation measures which do not deal with the causes of localized distress and will result in a recurrence of such distress.

In South Africa 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, whitch ultimately results in a reduction of the the strength and load-spreading ability of the layers affected. Valuable clues to drainage problems can be obtained by observing :

- the topography and geology of the surrounding area, and
- the vegetational characteristics of the immediate area.

Particular attention must be given in the initial assessment to identifying these sections. During the visual inspection, or as a result of such an inspection, any additional information or clues as to the causes and mechanism of localized distress must be recorded.

Further detailed investigation may be required to establish for any locally affected area the exact cause and mechanism of distress in a particular section, the resultant damage to the pavement structure, and the best remedial action.

The evaluation procedures discribed in the detailed assessment is of parcticular relevance in dealing with localized distress, e.g. use of the DCP to determine the extent of distress. In general these sections are reconstructed and manuals dealing with the construction of new pavements, such as TRH4, should be consulted.

3.3.4 Sections where structural improvement may be required

Lengths of pavement which, after the initial assessment, do not fit into any of the above categories, or on which further uncertenties as to the exact nature of the distress exist, may be structurally inadequate and will require further analysis. These sections are further dealt with in the detailed assessment.

However, an attempt should be made during the initial assessment to indicate the nature and urgency of the problem and what initial testing should be done on these sections during the second phase.

Naturally, further detailed assessment may well identify some of the sections as in fact exhibiting localized distress or surface-only problems.

SECTION 4

Detailed assessment

4.1 Procedure

4.1.1 Objectives and Scope

4.1.2 Approach

4.1.3 Processing of data

4.2 Analysis methods

4.2.1 The Asphalt Institute curve

4.2.2 DCP design curve

4.2.3 Deflection - rut depth relation

4.2.4 Deflection - riding quality relation

4.3 Identification of the cause and mechanism of distress

4.3.1 Deformation in combination with other distress manifestations

4.3.1.1 Caused by surfacing inadequacies

4.3.1.2 Caused by subbase or base inadequacies

4.3.1.3 Originating in the subgrade due to insufficient load distribution by the pavement layers

4.3.1.4 Caused by active subgrades and collapse settlement

4.3.1.5 Caused by post-construction compaction of material

4.3.2 Cracking as a mode of distress

4.3.2.1 Crocodile cracking

4.3.2.2 Block cracking

4.3.2.3 Longitudinal cracking

4.3.2.4 Transverse cracking

4.4 Structural capacity analysis

4.4.1 Deflection based method

4.4.2 DCP based method

4. DETAILED ASSESSMENT

4.1 Procedure

4.1.1 Objectives and Scope

This section deals with lengths of road that have been identified as probably requiring structural improvement. The aim of the detailed assessment 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 :

- assessment of available data to establish possible distress origins;
- analysis of data to establish the most probable cause(s) and mechanism(s) of distress;
- verification of the probable cause(s) and mechanism(s) of distress through tests and/or further analysis;
- determination of the remaining life (where applicable) of the pavement sections.

To facilitate the above, the detailed assessment is divided into sub-sections, with suggestions to :

- procedures for the analysis of data;
- guidelines for the identification of the cause(s) and mechanism(s) of distress;
- procedures enabling structural capacity analysis.

The recommended approach is outlined in Figure 6.

4.1.2 Approach

Full use should be made of experience, and of all available results of tests and information gained during the initial assessment, before embarking on any further investigations.

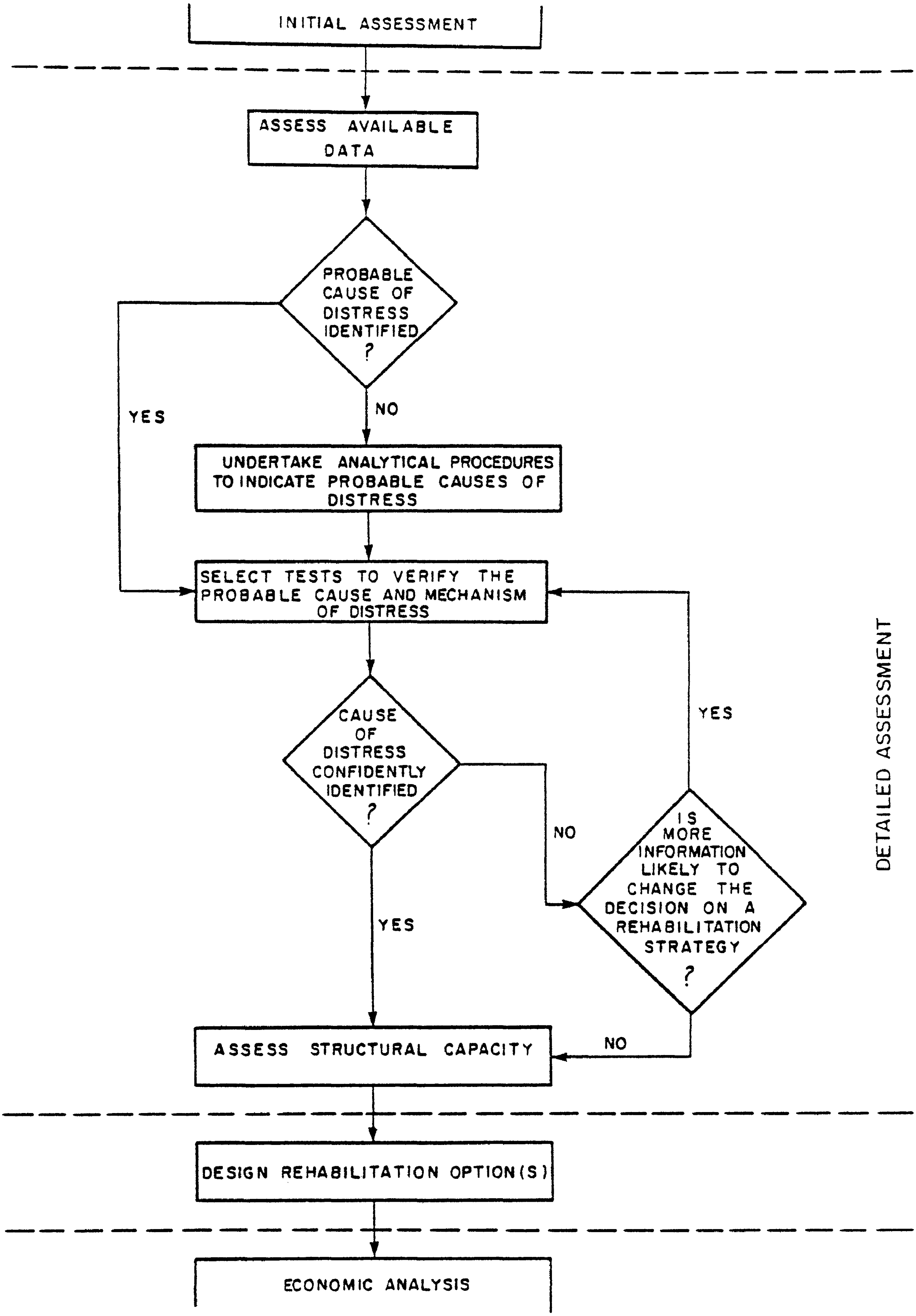

FIGURE 6

FLOW DIAGRAM OF DETAILED ASSESSMENT

Where distress is manifested in a typical manner and this has been observed during the initial assessment, the cause and mechanism of distress may be easy to identify. In this case the results of material tests can be used to make a decision on the rehabilitation strategy required.

However, in general, doubt about the exact cause and mechanism of distress requires the use of an analytical procedure. Information gained during the initial assessment is used to identify the possible causes of distress. An empirical analysis using data on, for example, deflections, DCP's, rutting and traffic, should then enable the most probable cause of distress to be identified. Confidence to proceed to the rehabilitation design stage is gained by verifying this probable cause and mechanism of distress with appropriate tests. A large number of pavement test methods exist and the following are currently used in South Africa :

- Deflection, deflection bowl and radius of curvature;
- In-depth deflections using the Multi Depth Deflectometer (MDD);
- Dynamic Cone Penetrometer (DCP). surveys;
- Test pits and material tests;
- HVS tests (only for very important investigations where cost implications could be very high).

The selection of the test method will depend on the circumstances, local experience, relative cost and the applicability of the test.

If the test selected does not confirm what was expected, a re-assessment of the data is required, but only if more information is likely to change the decision on the rehabilitation strategy to be adopted. More tests may be decided on and this cycle is repeated 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 road can be subdivided into uniform sections if appropriate.

The structural capacity should now be assessed so that appropriate rehabilitation alternatives can be established.

4.1.3 Processing of data

In the detailed assessment, analytical procedures are used to identify the most probable cause(s) and mechanism(s) of distress in each uniform section. For some of these procedures, such as the DCP measurements and structural capacity analyses, the data on certain parameters have to be reprocessed to determine representative values for each uniform section of road.

In such cases, the analyses should take into account the serviceability requirements for the different road categories (see Appendix 2, Table 16). Depending on the category of road, it is regarded as acceptable for a small percentage of the length of a road to perform unsatisfactorily at the end of the rehabilitation design period. This is taken into account by using different levels of data processing for the analytical procedures. The percentile levels recommended for use are given in Table 2 (see TRH4[14], Table 16).

TABLE 2 - Recommended percentile levels for use in data processing

Category of road	Length of road allowed to perform unsatisfactorily at end of design life %	Recommended percentile levels for use in the processing of data
A	5	95
B	10	90
C	20	80

These percentile levels are also recommended for use in designing the different rehabilitation options. Theoretically this results in a design which provides a satisfactory level of serviceability throughout the design life of the rehabilitated road.

In practice it is often found that only the 95th percentile values of parameters for 100m sections are available. In these cases the road is assumed to be homogeneous over the length of a section. The mean value of the available 95th percentile values will now give a good approximation of the actual 95th percentile value of a uniform section of road under investigation. For practical purposes, then, the following values are used :

- A category roads : mean value of the 95th percentile values;
- B category roads : 0,95 x mean value of the 95th percentile values;
- C category roads : 0,90 x mean value of the 95th percentile values.

4.2 Analysis methods

In the analysis it is important to establish whether the pavement has performed in a normal or abnormal way. A pavement normally deteriorates as traffic loads accumulate. When the rate of deterioration is unexpectedly high*, the pavement's behaviour is considered abnormal. Abnormal behaviour can have numerous causes and special care should be taken to determine the reason for the abnormality and to prevent its recurrence.

In cases where distress has occurred in a normal way and where the pavement design does not appear to be adequate for the future traffic, the investigation should concentrate on assessing the strength and quality of the existing pavement layers for use as a foundation for a rehabilitated pavement.

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* Pavements in South Africa are designed for a structural design period varying from 15-30 years, depending on the category of road; refer draft TRH4[14], Table 3.

The situation can be determined by undertaking a load carrying capacity analysis. This will enable a comparison to be made between the observed behaviour and that expected for the design traffic and past accumulated traffic.

The following empirical relations may accordingly be found to be applicable in establishing the mechanism and cause of distress.

4.2.1 The Asphalt Institute curve (see Figure 7)

The general application of this relation, developed by the Asphalt Institute[16, 17, 18], to South African pavements has not as yet been fully established and research is continuing.

This relation gives the minimum expected life for well-designed and well-constructed pavements on the basis of deflection. If a pavement in Zone B fails, this would suggest that there are anomalies such as a base or surfacing of poor quality or an active subgrade.

It should be noted that the term "deflection" is used to describe the elastic deformation (recoverable vertical movement) measured under a standard 40 kN dual wheel load. The deflection value that can be considered to be the representative deflection of a section of pavement is the applicable percentile value (see Table 2) calculated by using all the deflections measured on that section of road.

Furthermore, it is well known that the deflections measured on a pavement vary depending on seasonal changes (e.g. wet or dry season). For analyses or design purposes it is desireable to take these seasonal changes into consideration and to use the maximum deflection that can be measured during the passage of all seasons (during a year) on a specific pavement i.e. the maximum annual deflection (Deflections usually reach a maximum value just after the rainy (wet) season).

With each application of a load on a pavement, a small percentage of the deformation will not recover and will remain as permanent or plastic deformation. The cumulative effect of a number of load applications will cause this permanent deformation to become measurable. For pavement evaluation pruposes this is measured as rut depth as is shown in Figure 53 and used in analyses methods as discussed in section 5.2.3.

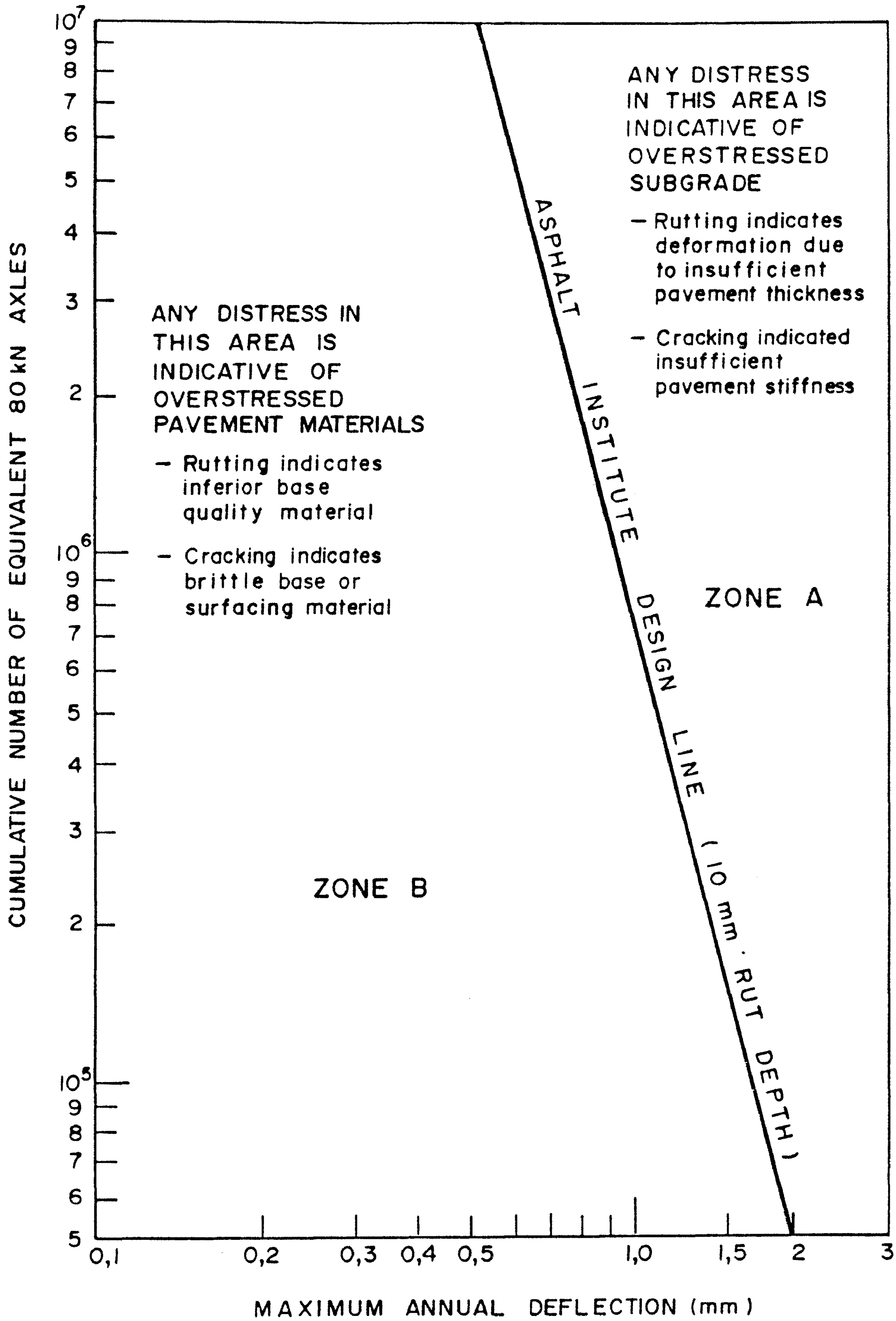

FIGURE 7

DEFLECTION - TRAFFIC RELATION USED FOR ASSESSMENT OF EXPECTED LIFE OF SURFACING AND BASE

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4.2.2 DCP Design Curve (see Figure 8)

With this procedure, largely developed by the Transvaal Roads Department[19, 20, 21], the in-situ CBR of pavement material can be measured with the Dynamic Cone Penetrometer (DCP). The penetration after each blow of the hammer indicates the in-situ CBR at that depth. In this way a useful DCP profile is obtained from which the quality of the various layers can be assessed. Figure 8 gives penetration levels for pavements that have been found to work well in the Transvaal for the given traffic loadings. Generally, points lying to the right of the DCP design curve chosen for the traffic category will represent material of inadequate quality for these conditions.

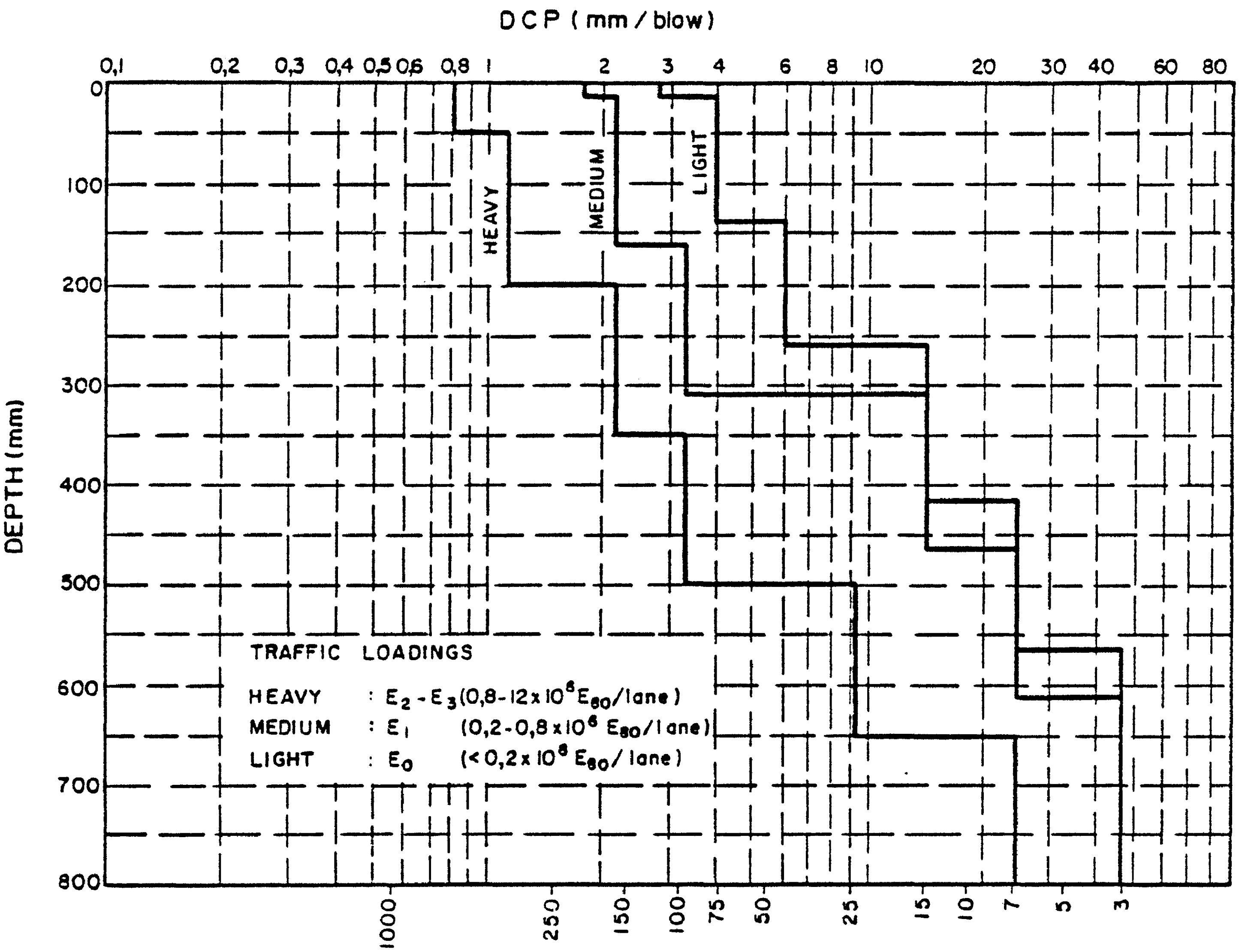

FIGURE 8

DCP - DESIGN CURVES FOR THREE CATEGORIES OF TRAFFIC

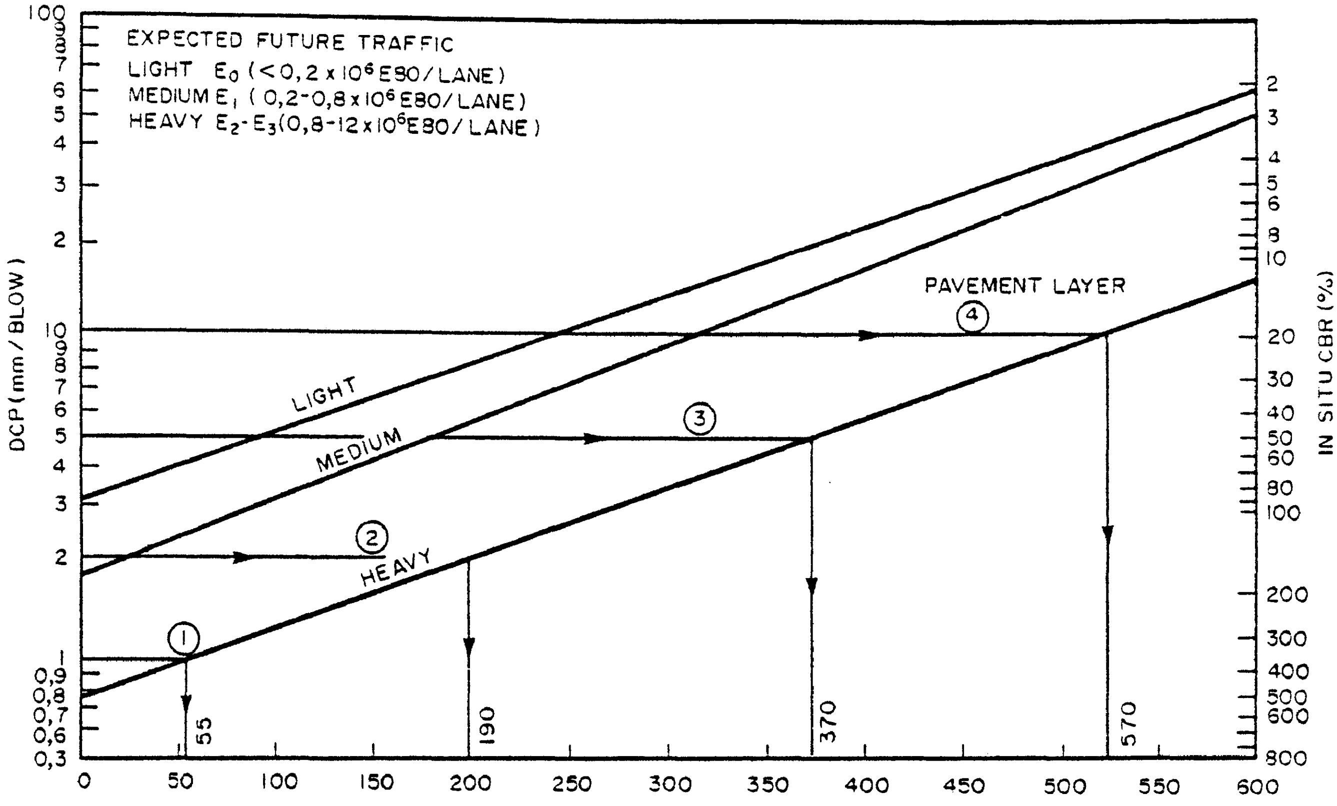

FIGURE 25c

EXAMPLE OF THE USE OF THE COVER REQUIREMENT CHART BASED ON DYNAMIC CONE PENETRATION MEASUREMENTS

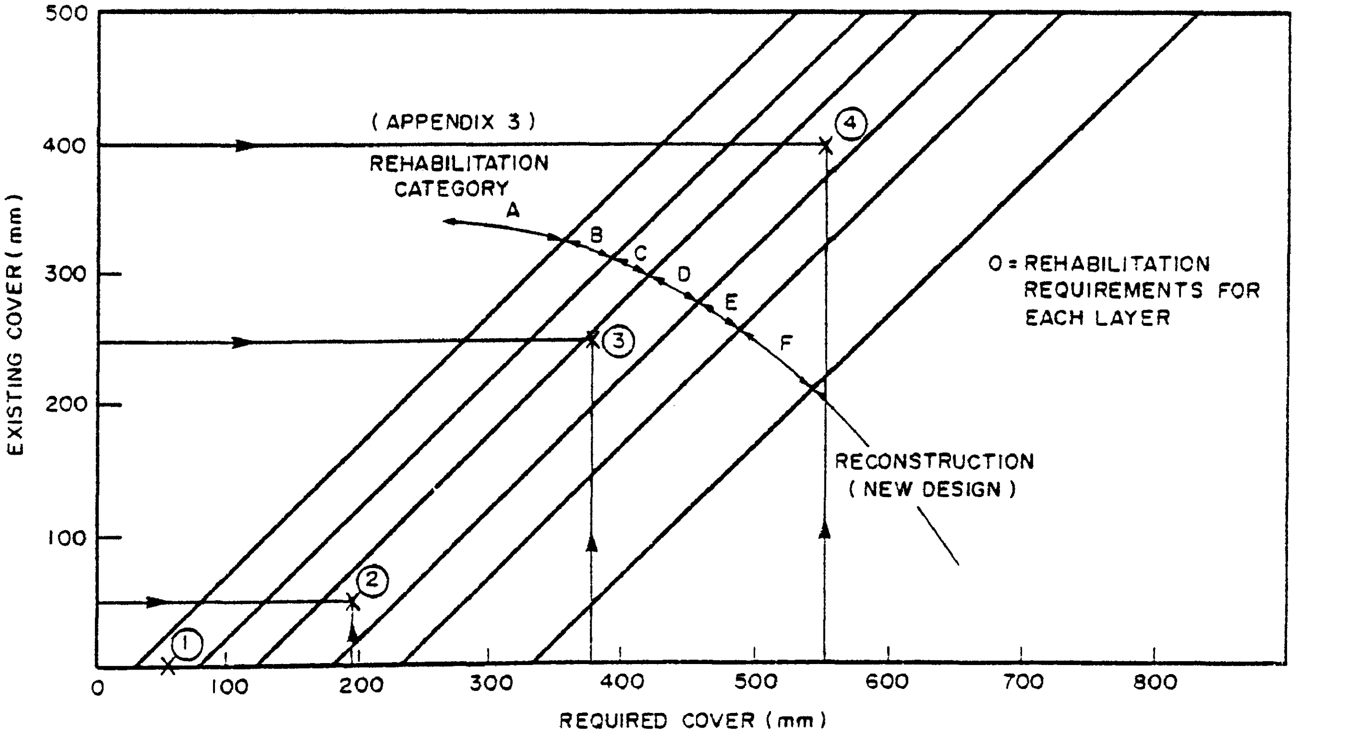

FIGURE 25d

EXAMPLE OF THE USE OF THE REHABILITATION CATEGORY CHART BASED ON COVER REQUIREMENTS

4.2.4 Deflection - riding quality relation (see Figure 10)

In the absence of rut depth measurements, the existance or absence of a relation between riding quality and deflection of a pavement could give give a useful indication of the locality of the problem. The 95th percentile values of deflection and riding quality over 100m lengths are useful for this purpose. Examples of a 'good' and a 'poor' relation are given in Figures 10(a) and 10(b) respectively.

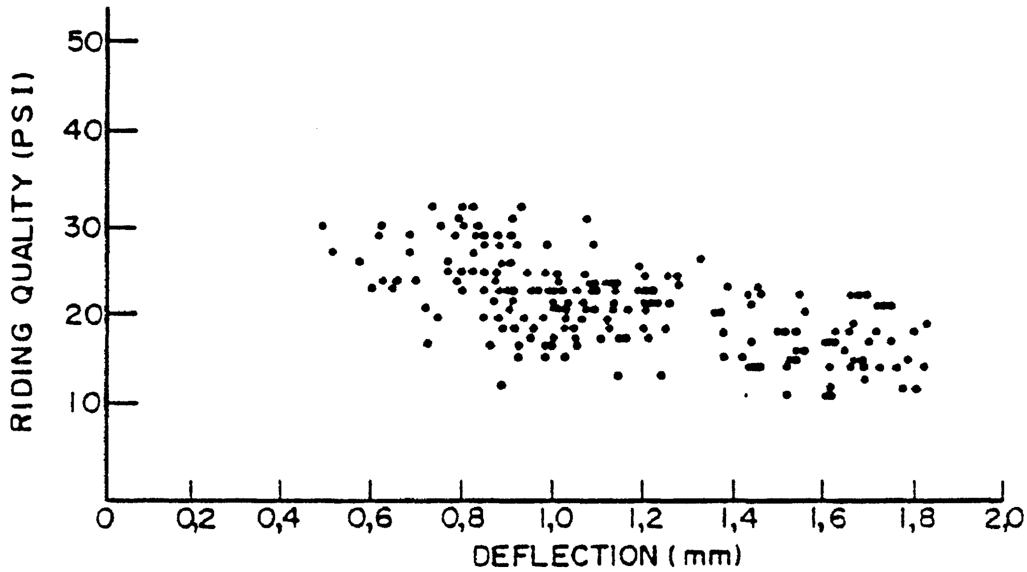

FIGURE 10(a)
EXAMPLE OF A GOOD RELATION BETWEEN DEFLECTION AND RIDING QUALITY
(ROAD P6/1: km 34 - km 52-JUNE 1982[9])

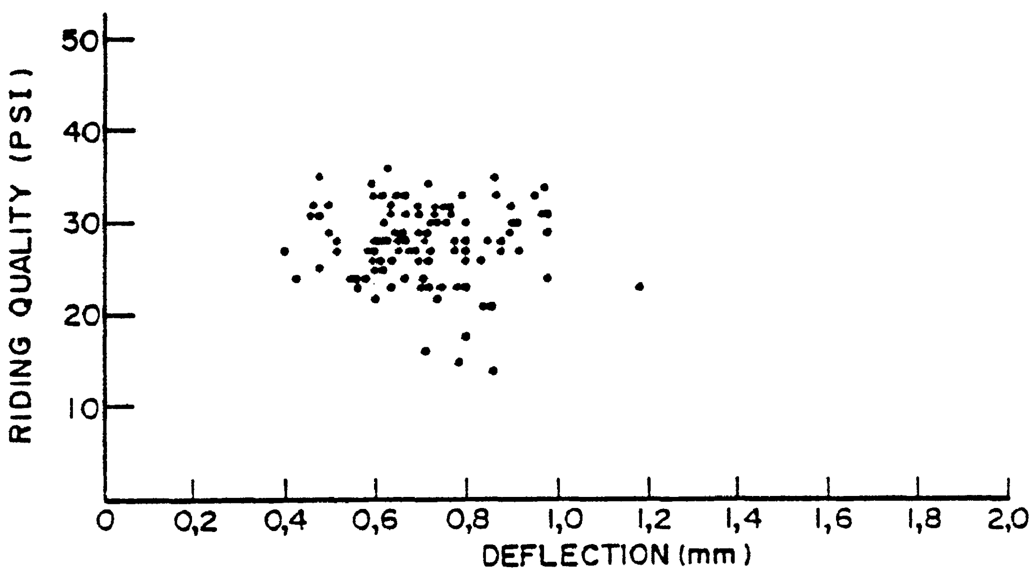

FIGURE 10(b)
EXAMPLE OF A POOR RELATION BETWEEN DEFLECTION AND RIDING QUALITY
(ROAD P6/1: km 20 - km 34 - JUNE 1982[9])

4.3 Identification of the cause and mechanism of distress

The aim of this evaluation procedure is to identify the most probable cause and mechanism of distress. In the guidelines presented here, permanent deformation and cracking are the two modes of distress investigated. (Other modes of distress are primarily associated with surfacing-only problems.) Possible causes of deformation are identified and examined using empirical relations until the most probable cause is established. Where cracking occurs, its characteristics are used to identify the most probable cause of distress.

4.3.1 Deformation in combination with other distress manifestations

4.3.1.1 *Deformation caused by surfacing inadequacies*

Premature rutting or corrugations may be caused by problems in the surfacing only, since under certain conditions asphalt layers may deform. In the analysis, no correlation between rutting and deflection will be found and a DCP survey will show no shear 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. Testing should concentrate on this aspect and should satisfy the creep modulus requirement for the traffic category. The design and selection of hot mix surfacings and bases do not fall within the scope of this document. These aspects are covered in detail in various other documents such as TRH8[23].

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 TRH3[24].

4.3.1.2 *Deformation caused by base or subbase inadequacies*

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

In cases where poor subbase or base quality is the only cause of the rutting, there will be a poor relation between deflection and rut depth.

A further strong indicator of subbase or base shear failure is obtained by demarcating on Figure 11 (Grant[22]) the 10 mm minimum rut depth level and finding the corresponding deflection for the estimated past traffic (obtained from Figure 7). Results that fall above the 10 mm rut depth ordinate and to the left of the deflection value are indicative of poor subbase or base quality.

A DCP survey is often very useful to determine the extent and position of distress in any layer(s) of the pavement.

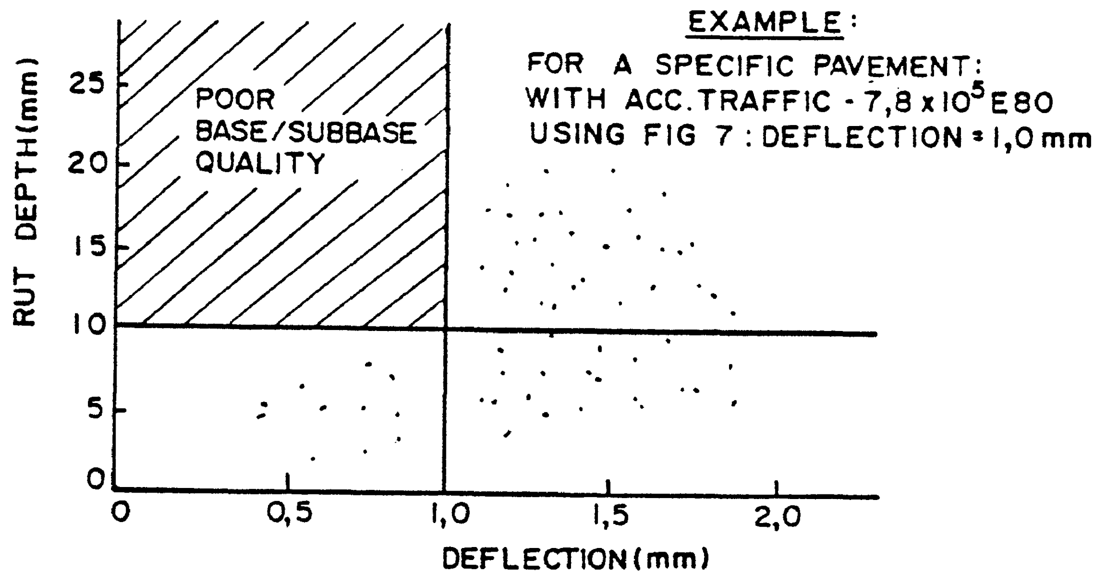

FIGURE 11

EXAMPLE OF DEFLECTION-RUT DEPTH RELATION WITH DEMARCATED POOR BASE OR SUBBASE REGION

4.3.1.3 *Deformation originating in the subgrade due to insufficient load distribution by the pavement layers (traffic associated)*

Overstressing of the subgrade by traffic forces is generally attributed to insufficient load distribution by the pavement layers because of an 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 strenght is exceeded by the stresses imposed on it and with accumulated traffic rutting is eventually formed in the wheeltracks. The rutting in this case is usually fairly wide, covering about half a lane, and due to the depth of the overstressed material and the restriction of the pavement layers, heave 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 clear relation between deflection and rut depth is found, it indicates that the pavement structure has not adequately protected the subgrade. If rut-depth measurements are unavailable, riding quality measurements (PSI) can be used. A relation between deflection and riding quality will also be indicative of insufficient pavement thickness and/or strength. (Refer sub-sections 4.2.3 and 4.2.4).

If, despite a large and general amount of scatter, there is a tendency for the rutting to increase (or the riding quality to decrease) with increasing deflection, then deformation is likely to be occurring in the subgrade (see Figures. 12(a) and 12(b)). Analyses based on DCP surveys and/or the Asphalt Institute curve can be used to confirm the pavement's inadequacy.

4.3.1.4 *Deformation caused by active subgrades 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, there is no rutting in the wheel paths. Riding quality should in this case be independent of traffic and deflection, and plots of deflection versus riding quality will show no discernible relation.

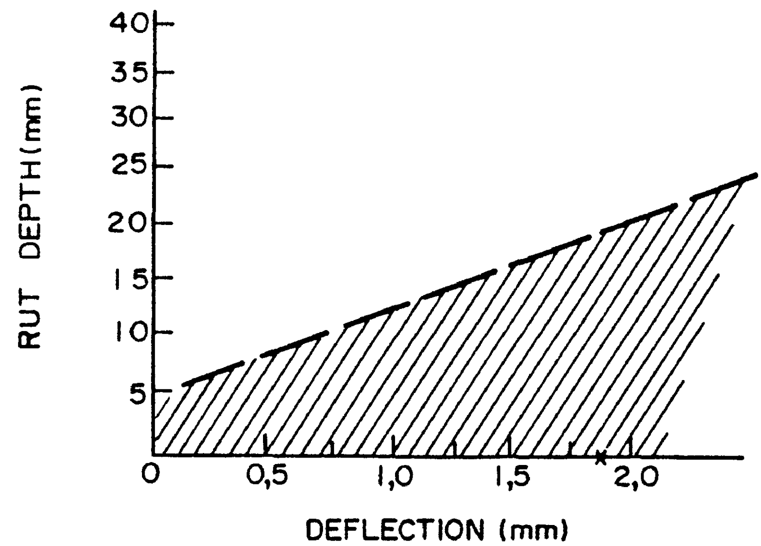

FIGURE 12 (a)

RELATION BETWEEN DEFLECTION AND RUT DEPTH INDICATING AN OVERSTRESSED SUBGRADE

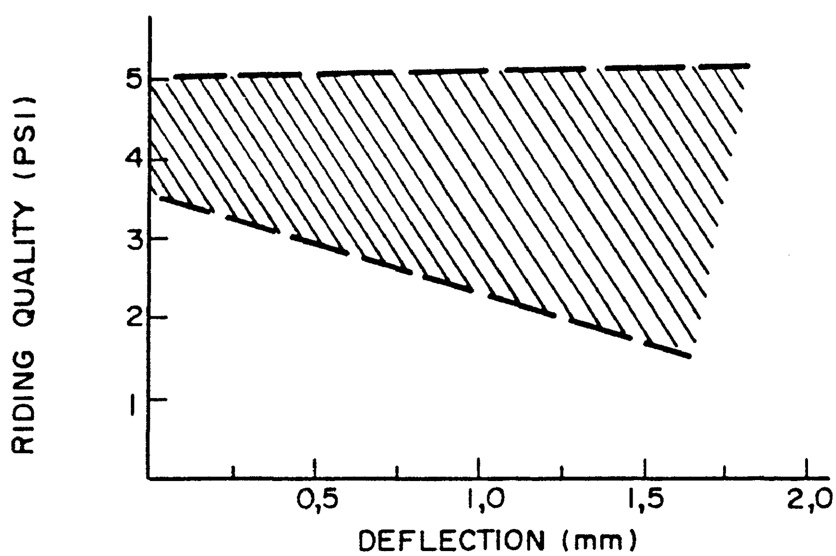

FIGURE 12 (b)

RELATION BETWEEN DEFLECTION AND RIDING QUALITY INDICATING AN OVERSTRESSED SUBGRADE

4.3.1.5 *Deformation caused by post-construction compaction 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. Deflection and rut-depth measurements will show no correlation.

The structural capacity of pavements with shear failure and pavements showing signs of post-construction densification is very different. In the case of shear, the structural capacity will decrease, while deformation will continue 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 the ruts may, if compared with results in the wheel path, give an indication of whether densification has occurred. A stable condition is indicated by a smooth DCP curve.

4.3.2 Cracking as a mode of distress

Cracking occurs when the strain induced in 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 tolerable 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. Grant[22] pointed out that binders in asphalt mixtures weather faster 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 its brittle temperature 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. Ages near the upper limit of ranges given in Table 3, taken from draft TRH4[14], 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 sub-layers. In time, these cracks reflect through the surfacing.

Laboratory tests 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 :

4.3.2.1 Crocodile cracking

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

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

TABLE 3 - Suggested typical ranges of surfacing life periods

Base type	Surfacing type	Typical range of surfacing life (yrs)		
		Road category and traffic		
		A E3-E4	B E2-E3	C E1-E2
Granular	Bitumen and/or slurry seal	-	-	2-8
	PVC-tar single surface treatment	-	-	5-8
	Butimen single surface treatment	-	6-10	8-11
	Butimen double surface treatment	-	6-12	8-13
	Cape seal	-	6-15	8-18
	Open graded asphalt*	4-8	6-10	-
	Thin cont. graded asphalt	8-11	9-14	-
	Thin gap-graded asphalt	8-13	10-15	-
Bituminous	Bitumen sand or slurry seal	-	-	-
	PVC-tar single surface treatment	-	-	5-8
	Bitumen single surface treatment	-	7-11	8-12
	Bitumen double surface treatment	-	7-13	8-14
	Cape seal	-	8-15	8-18
	Open graded asphalt*	4-8	6-10	-
	Thin cont. graded asphalt	8-12	9-15	-
	Thin gap-graded asphalt	8-14	10-16	-
Cemented	Bitumen sand or slurry seal	**	-	-
	PVC-tar single surface treatment	**	-	4-7
	Bitumen single surface treatment	**	4-7	5-8
	Bitumen double surface treatment	**	5-8	5-9
	Cape seal	**	5-10	5-11
	Open graded asphalt*	**	6-8	-
	Thin cont. graded asphalt	**	5-10	-
	Thin gap-graded asphalt	**	6-12	-

\- Surface type not normally used.
* On top of cont. or gap-graded asphalt.
** Base type not used.

Inadequacies of load distribution are difficult to identify from surface inspection alone. In these cases, deflection measurements can be used to give more quantitative information on the origin of 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 avoid including shrinkage cracking or non-traffic-associated cracking when doing this correlation). Often a DCP survey can be used to pinpoint a structurally inadequate layer.

4.3.2.2 *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 and thickness of the pavement layer. Although traffic loading does not initiate this distress, it tends to cause secondary cracking which may result in crocodile cracking.

This distress may not affect the strength of the pavement. It is however, taken to indicate the potential lowering of the structural capacity of a pavement because of the possibility of the ingress of surface water to structural layers, and of pumping.

4.3.2.3 *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 as such should be identified during the initial assessment. Swell 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 be the first indication of structural problems. If this cracking occurs in or close to the wheel paths, it is likely to develop into ladder and thereafter crocodile cracking. Cracks can thus be an indication that the pavement is approaching the end of its service life, and therefore needs further investigation.

4.3.2.4 *Transverse cracking*

Transverse cracking is symptomatic of temperature-associated distress. This cracking is thus not in itself usually 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 is limited to the surfacing layer only.

Transverse cracking as a result of the shrinkage of unbound layers is a rare phenomenon in this country. 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.

4.4 Structural capacity analysis

The object of the structural capacity analysis is to determine the remaining life of the existing pavement. This is particularly relevant where non-strengthening rehabilitation measures appear appropriate or when an 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, strengthening measures should be included in the rehabilitation design.

Various pavement design methods may be used to determine the pavement's structural capacity. For example, the pavement structure may be compared with other familiar designs of known performance or with a catalogue of designs for new pavements such as that presented in TRH4[14]. Alternatively, a mechanistic analysis can be carried out using typical layer moduli for the various materials incorporated in the design.

Two methods are described in this document, based on the parameters, identified in Appendix 2, measuring pavement structure capacity qualities i.e.

- deflection; and
- DCP measurements.

4.4.1 Deflection based method

The use of deflection measurements to assess future pavement behaviour has been well established throughout the world and the availability of the Lacroix deflectograph makes this an attractive procedure in South Africa. However, the use of deflection-life curves (for example the Asphalt Institute curve[17]) can have very little value if used mechanically without assessing the applicability of the method to the pavement under investigation.

Pavement surface deflection is a response of the total pavement structure to an applied load and gives no detailed information of material properties in individual layers. The use of deflections to predict distress when the cause can be attributed to factors such as reflective cracking, weathering, pumping and swelling or expansion of materials is very limited. For example, in pavements with stabilized base materials where reflection cracking could be a main distress mode, the initial measured deflections (should be very low in this case) will give no indication to the future appearance of distress. In these cases traffic-associated distress will only appear in the final breakdown of the pavement and has little to do with the original low deflections measured on the pavement.

However, for pavements which became distressed due to the normal action of traffic, a good correlation can be expected between distress (or life expectancy) and deflections. These pavements usually will consist of simple structures (unstabilized base materials or where the stabilized material has been broken down) where the distress is brought about by the accumulated action of traffic and is free of the influence of external factors such as active subgrades, causing abnormal behaviour. Useful information to the structural capacity of these pavements can be gained through an analysis based on deflection.

In the analysis given here, the Asphalt Institute's design curve[17] is used.

It should be noted that this line represents the lower limit of information gathered from numerous sources as illustrated in Figure 13. For a given total accumulated equivalent traffic loads (Σ E80s) pavements with deflection levels :

- below the shaded area will most probably provide a good service in carrying the traffic;
- above the shaded area are unlikely to provide a good service in carrying the traffic;
- in the shaded area could or could not provide a good service in carrying the traffic.

The Asphalt Institute curve thus gives a conservative relation with a high probability of satisfactory design, where :

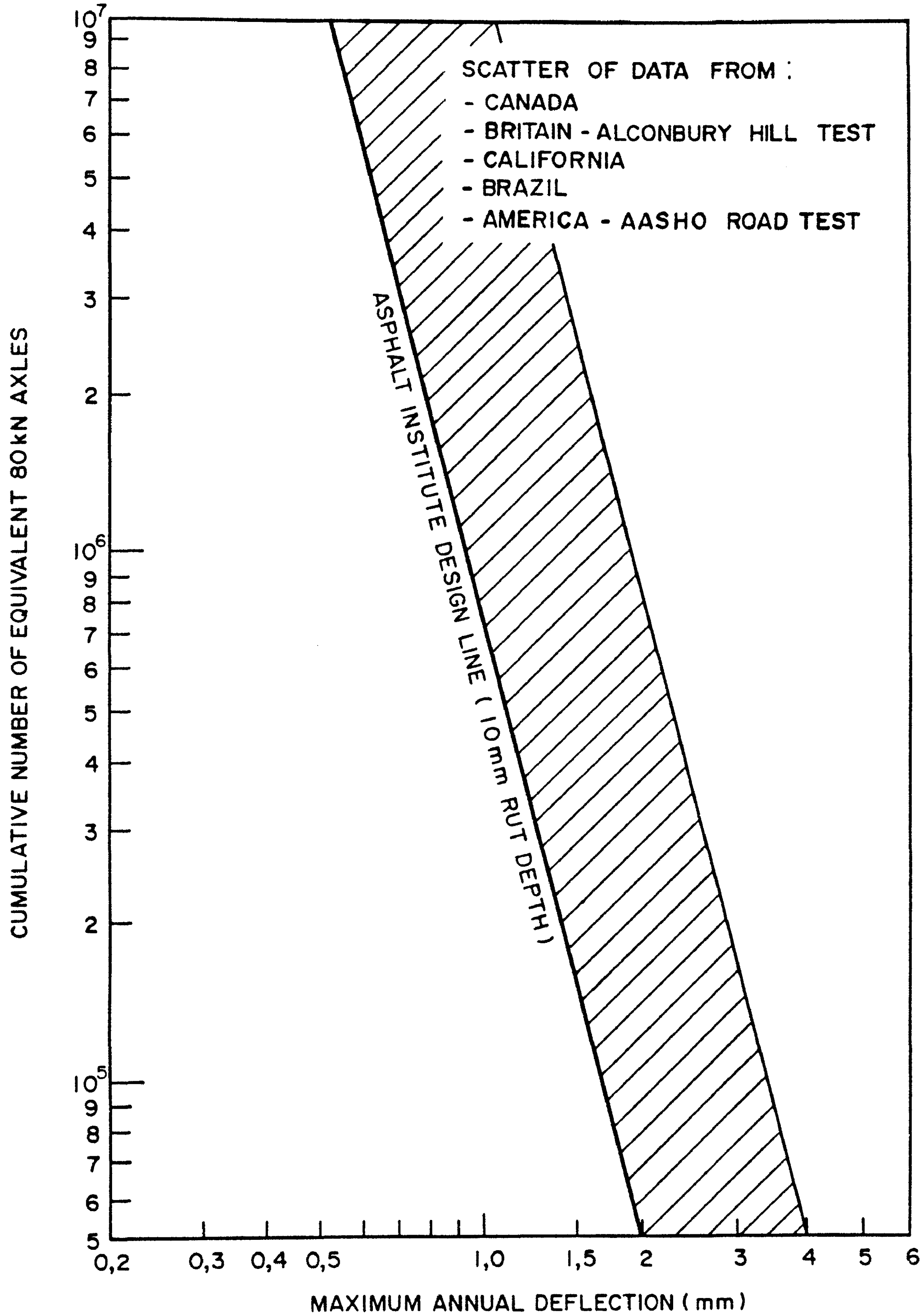

FIGURE 13

SCATTER OF DEFLECTION - TRAFFIC DATA USED IN ESTABLISHING THE ASPHALT INSTITUTE CURVE [17]

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$$N \propto 1/\delta^3$$

N = number of equivalent 80 kN axle loads

δ = maximum annual deflection

From this curve the expected life (N_e) of a length of pavement can be determined by means of deflection measurements. For this analysis the pavement is divided into design sections according to the measured deflection values. A length of pavement is considered as a unit when the coefficient of variation of the deflection on the pavement is less than 0,25*.

For this analysis the total of the past accumulated equivalent traffic and the expected accumulated equivalent traffic for the pavement must be calculated**. The total of the past and expected traffic will give the cumulative design E80 axle loads (N_c)

$$N_e = \Sigma \text{ Past Traffic} + \Sigma \text{ Expected traffic}$$

$$= \sum_{m=o}^{n} \frac{E_o}{(1+i)^m} + \sum_{r=1}^{x} E_o (1+j)^r$$

where:

E_o = Yearly equivalent 80 kN axle loads in the year of investigation

i = Mean E80 growth rate during the past existence of the pavement (percentage/100)

n = age of the pavement (years)

j = Expected E80 growth rate during the rehabilitation design period (percentage/100)

x = Rehabilitation design period (years)

---

* See Appendix 5 for the calculation of the coefficient of variation. Extensive deflection surveys with the Lacroix Deflectograph have shown that uniform sections of road have a coefficient of variation of less than 0,25.

** It is not within the scope of this document to discuss traffic calculations. This aspect is covered in detail in other documents such as draft TRH4[14]

The pavement structure is considered adequate for the design period when the cumulative calculated design load (N_c) is equal to or less than the cumulative predicted design load (N_e) of the pavement, determined from Figure 14 (see example below).

However, when $\frac{N_c}{N_e} > 1$ the option of strengthening should be considered.

If this analysis shows a possible inadequate pavement structure for future traffic needs, overstressing of the pavement due to traffic forces is a possibility. Strengthening of the pavement should be considered to prevent any traffic-induced deformation.

In the case where N_e > Past accumulated E80 loads this analysis can also be used to determine the expected remaining life of the pavement by calculating the number of years before the N_e load is reached.

i.e. when $\frac{N_c}{N_e} = 1$

<u>Example</u>

The following data are available for a length of pavement.

Category of road : A

Deflection (95th percentile)	d_r	$= 1$ mm
Standard deviation of deflections	S	$= 0{,}19$
Coefficient of variations	C_v	$= 0{,}19$
Past E80 loads		$= 6 \times 10^5$ E80
Current annual E80 loads	E_o	$= 36 \times 10^3$ E80
Expected E80 growth rate		= 6 %
Expected E80 loads (20 years)		$= 1{,}4 \times 10^6$ E80

$\therefore N_c = 2{,}0 \times 10^6$ E80

As the coefficient of variation is less than 0,25 the pavement can be considered as a unit for the structural capacity analysis.

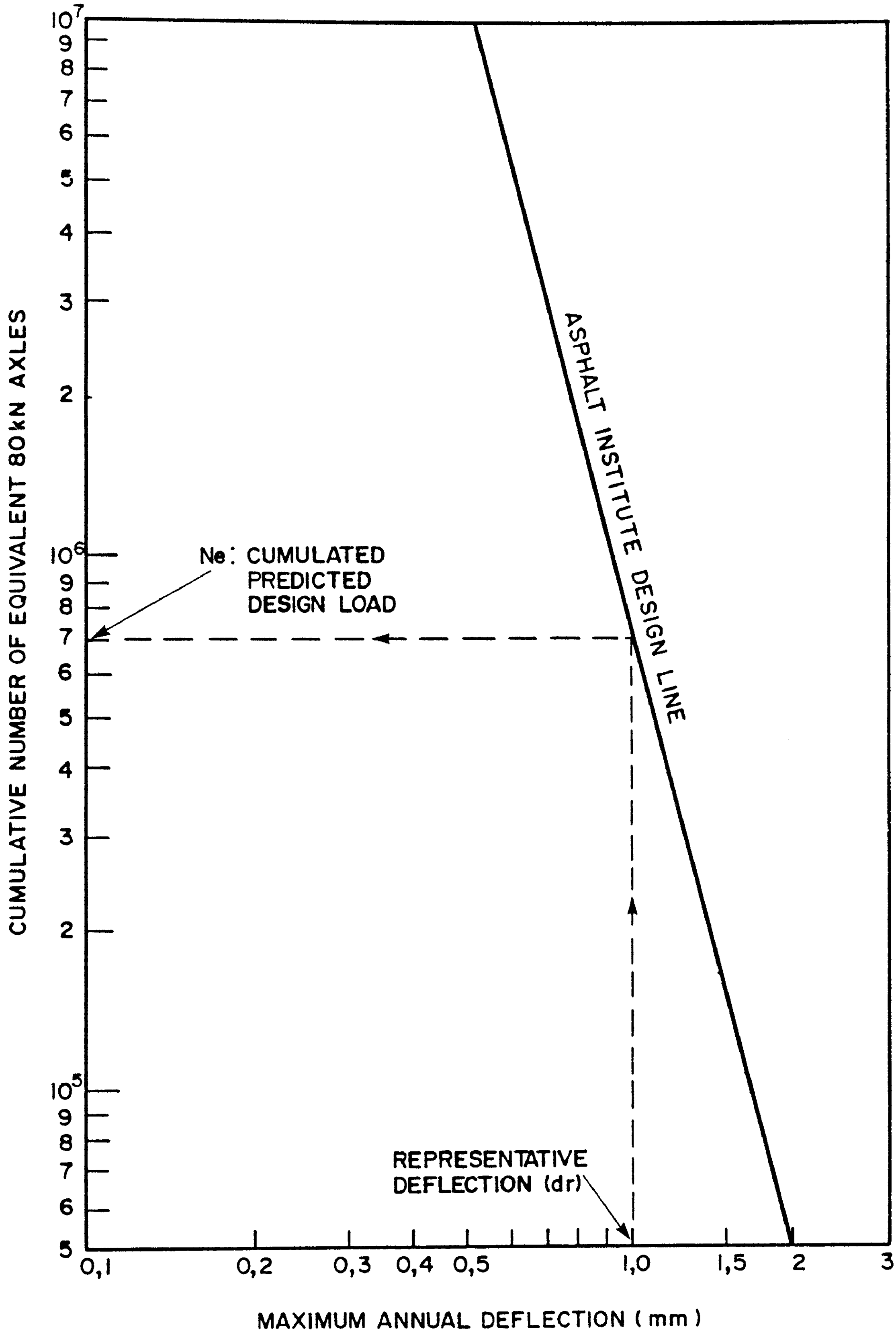

FIGURE 14

RELATION BETWEEN DEFLECTION AND TRAFFIC

940-4-3932a BD

From Figure 14 $N_e = 7{,}0 \times 10^5$ E80

As $\dfrac{N_c}{N_e} = 2{,}9 > 1$

Strengthening of the pavement should be considered for an analysis period of 20 years.

In this example N_e > Past accumulated E80 loads and this method can be used to make an estimate of the expected remaining life of the pavement, i.e. when

$$\frac{N_c}{N_e} = 1$$

In this example
When $N_c = 7 \times 10^5$ E80

Σ Past traffic + Σ Expected traffic = 7×10^5

$$\therefore \quad 6 \times 10^5 + \sum_{r=1}^{x} E_o\,(1+j)^r = 7 \times 10^5$$

$$\therefore \quad \sum_{r=1}^{x} 36\ 000\ (1+0{,}06)^r = 1 \times 10^5$$

$$\therefore \quad \sum_{r=1}^{x} (1{,}06)^r = 2{,}778$$

From this x ≅ 2 years 6 months

Therefore, the structural capacity of the pavement will, according to this analysis, be exceeded in two and a half years time.

4.4.2 Dynamic Cone Penetrometer based method

The Dynamic Cone Penetrometer has been used extensively by the Transvaal Roads Department (TRD), who has primarily been responsible for the development of applications in the use of the measured data. The DCP values have been calibrated against well known strength parameters such as CBR and UCS, and recently pavement evaluation and design procedures based and this data have also been developed[19, 20, 21].

These methods have been developed using data mainly from the Transvaal, but according to Kleyn and Savage[20] enough confidence exists to recommend the use of the DCP as a "practical alternative or supplement to deflection measurements as a method of evaluating existing pavements."

However, the use of the DCP methods are also restricted by certain limitations in applicability. The relations were developed by primarily using data obtained from thin surfaced, unbound gravel roads and is therefore mainly applicable for use on relatively "light" roads and according to Kleyn and Savage[20] with caution, also on pavements with lightly cemented layers (UCS < 3 000 kPa).

According to the TRD method, calculation of the structural capacity of a pavement is based on the DCP structural number (DSN), calculated from DCP data to a depth of 800 mm.

$$\text{Pavement layer DSN} = \frac{h}{DN}$$

$$DSN = \sum_{x=1}^{n} \frac{h_x}{DNx} \quad \text{with} \quad \sum_{x=1}^{n} h_x = 800 \text{ mm}$$

where : h = thickness of a pavement layer

DN = DCP number

= rate of DCP penetration through a pavement layer in mm/blow.

n = number of pavement layers up to a dpeth of 800 mm.

The relations between DSN_{800} and the accumulated equivalent 80kN axles are given in Figure 15[20]. Three relations are given, taking into account the variation in moisture or drainage conditions, described as

- M1 - dry moisture regime or good drainage condition;
- M2 - optimum moisture regime or average drainage condition;
- M3 - wet moisture regime or poor drainage condition.

With the calculated DSN_{800} and the drainage condition of the pavement known, the structural capacity in equivalent 80kN axles (N_c) can be determined by using Figure 15 and the following formula :

$$N_c = C\ (DSN_{800})^{3,5}$$

with

N_c = cumulative calculated design load.
DSN_{800} = DCP structural number to a depth of 800 mm
C = 48×10^{-3} for a M1 condition
= 22×10^{-3} for a M2 condition
= 7×10^{-3} for a M3 condition

No strenghtening of the pavement is needed when the cumulative calculated design load (N_c) is less or equal than the cumulative predicted design load N_e (refer 4.4.1).

However, when $(N_c/N_e) > 1$, the option of strengthening of the pavement should be considered as the structural capacity is inadequate for the design load.

In determining the structural capacity, it is also possible to determine the remaining life (if any) of the pavement by calculating the number of years left before the N_c equivalent traffic is reached.

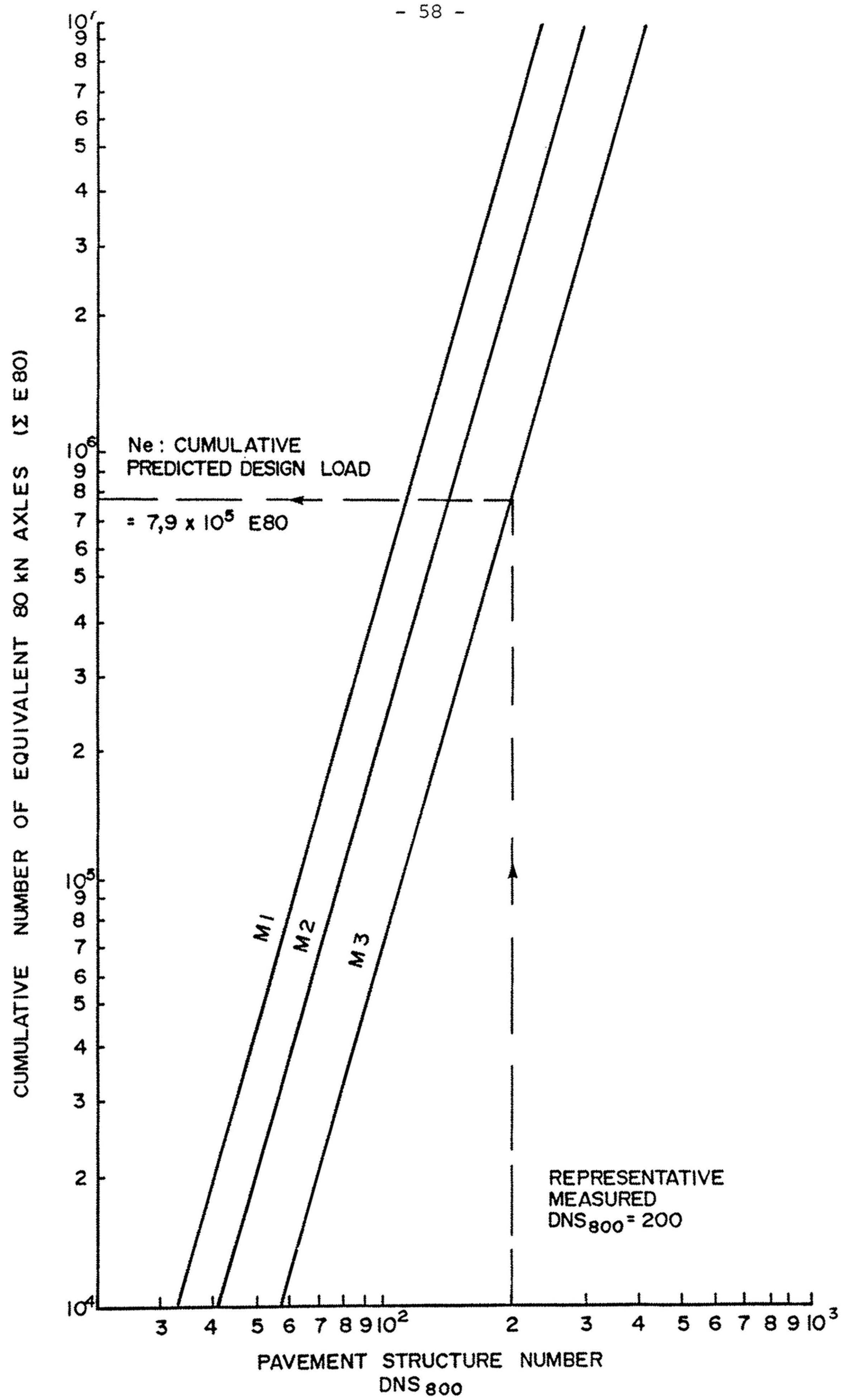

FIGURE 15

RELATION BETWEEN THE PAVEMENT STRUCTURE NUMBER (DSN_{800}) AND THE ACCUMULATED EQUIVALENT E80 AXLES

940-4-4331 HD (DN)

Example

The following data are available for a length of road

Category of road : A

DNS_{800} (95th percentile) = 200

Drainage condition = M_3

Σ Past E80 loads = 5×10^5 E80

Current annual E80 loads = 5×10^4 E80

Future expected E80 growth rate = 8 %

$\therefore N_c = 3 \times 10^6$ E80

From Figure 15

$N_e = 7{,}9 \times 10^5$ E80

$\frac{N_c}{N_e} = 3{,}8 > 1$

Strenghtening of the pavement should be considered for an analysis period of 20 years.

In this example $N_e > \Sigma$ Past traffic and this method can be used to estimate the remaining life of the pavement, i.e. when

$\frac{N_c}{N_e} = 1$

In this case, when :

$N_c = 7{,}9 \times 10^5$

$$\therefore \quad \Sigma \text{ Past traffic} + \Sigma \text{ Future traffic} = 7{,}9 \times 10^5$$

$$\therefore \quad 5 \times 10^5 + \sum_{r=1}^{x} E_o \, (1+j)^r = 7{,}9 \times 10^5$$

$$\therefore \quad \sum_{r=1}^{x} 50\ 000 \, (1+0{,}08)^r = 2{,}9 \times 10^5$$

$$\therefore \quad \sum_{r=1}^{x} (1{,}08)^r = 5{,}8$$

From this $x \cong 4$ years 7 months.

Therefore, the remaining life of this pavement is estimated to be not more than 5 years.

SECTION 5

Rehabilitation design procedure

5.1 Introduction

5.1.1 Objectives and Scope

5.1.2 Approach

5.2 Rectifying deformation

5.2.1 Caused by surfacing inadequacies

5.2.2 Caused by inadequacies in the pavement layers

5.2.3 Originating in the subgrade due to insufficient load distribution by the pavement layers

5.2.4 Caused by active subgrades and collapse settlement

5.2.5 Caused by post-construction compaction of material

5.3 Rectifying cracking

5.3.1 Caused by the shrinkage of treated layers (reflective cracking)

5.3.2 Caused by traffic-induced fatigue of asphalt layers

5.3.3 Caused by shrinkage, thermal forces and/or brittle materials of asphalt layers

5.4 Improving structural capacity

5.4.1 Deflection based method

5.4.2 DCP based method

5. REHABILITATION DESIGN PROCEDURE

5.1 Introduction

5.1.1 Objectives and scope

The objective of the design procedure is to establish applicable rehabilitation alternatives for each uniform length of a pavement. In this process the cause(s) and mechanism(s) of distress of each section, as identified during the detailed assessment, is used to select design methods in which the available information can be used to its full advantage.

The recommended procedure for the design of rehabilitation measures involves :

- the selection of applicable design methods;
- the use of the design methods in establishing rehabilitation needs as related to each cause and mechanism of distress;
- the selection of the most critical design (most conservative) by comparing the rehabilitation needs as determined by each method;
- the design of various rehabilitation options by using as reference the most critical design method.

The recommended procedure is outlined in Figure 16.

5.1.2 Approach

The success of a specific rehabilitation design method depends on its ability to utilize the information, the analyses and the results of the preceding detailed assessment. A number of methods are currently being used in South Africa. These include :

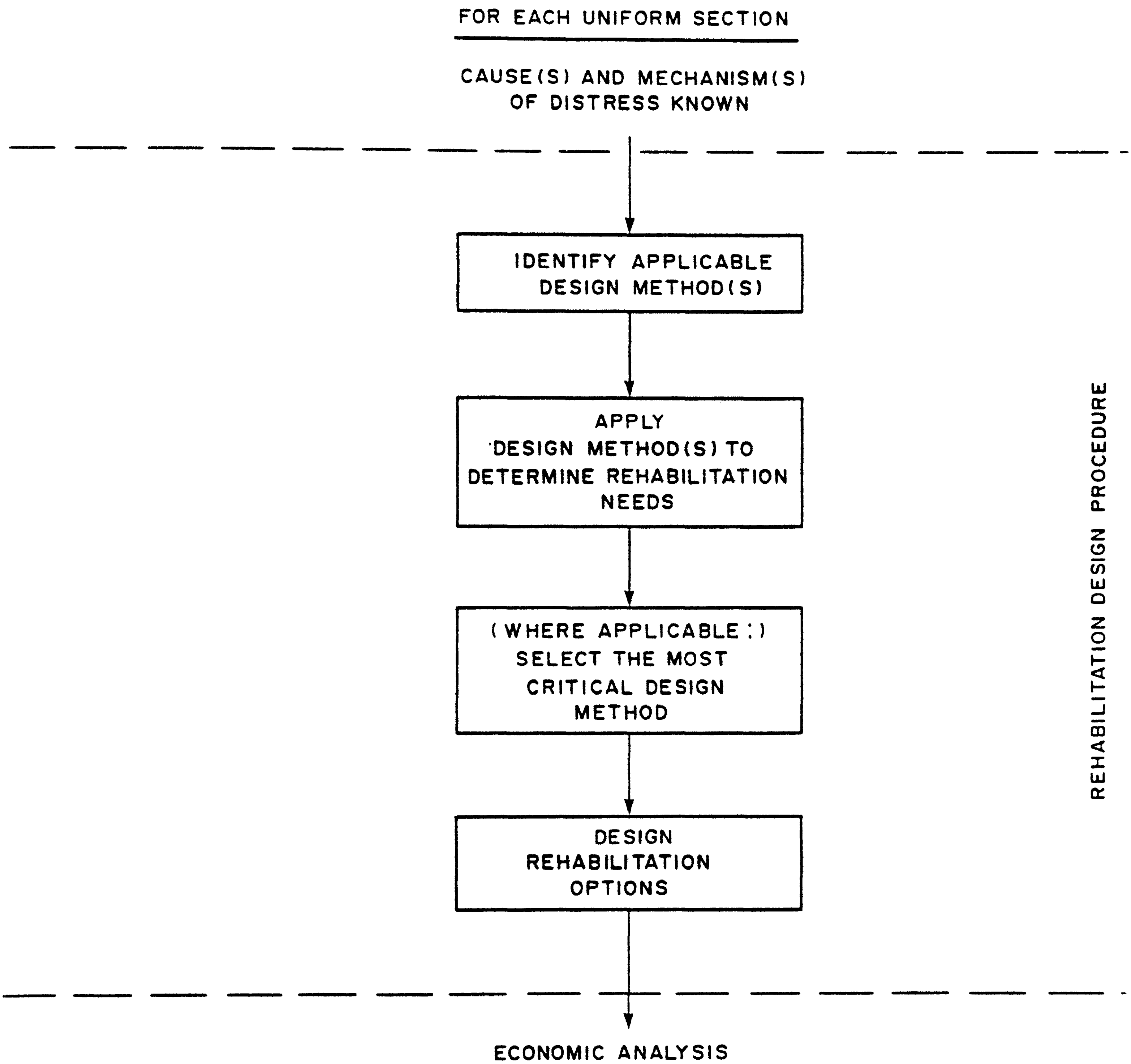

FIGURE 16

FLOW DIAGRAM OF THE REHABILITATION DESIGN PROCEDURE

940-4-4352/1 KR (GJ)

- deflection based methods[16, 17, 18, 22, 29, 30];
- CBR and CBR based methods[26, 27, 28];
- DCP method[19, 20, 21];
- Mechanistic design methods[31, 32];
- Heavy Vehicle Simulator (HVS) method[33, 34].

The method chosen, usually depend on the type of data available and individual experience and preference. However, all these methods have limitations which the user should recognise. Although any of the methods may be used, only those considered most suitable for rectifying specific causes and mechanisms of distress are now described in detail.

5.2 Rectifying deformation

5.2.1 Deformation caused by surfacing inadequacies

Deformation in thick asphalt layers is usually attributed to inadequate mechanical stability. Rehabilitation design should therefore satisfy the stability requirement of the asphaltic layer for the traffic category (see TRH8). This can be achieved either by replacing the asphalt layer or by recycling. The choice of option will depend on the subsequent economic considerations.

5.2.2 Deformation caused by inadequacies in the pavement layers

In this case the shear strength of the pavement materials is not adequate for the traffic stresses imposed on them. The following alternative remedial approaches are applicable :

- reducing the shear stress level in the inadequate layer(s) by overlaying with a sufficient thickness of high quality material;
- improving the quality of the inadequate layer(s).

In the latter case, the material quality can be improved by reworking the material with or without the addition of high quality material or a stabilization agent such as cement or lime. Samples from the inadequate layer(s) should be laboratory tested to determine whether mechanical or chemical treatment can be used to modify the material. These alternatives involves the recycling of the pavement materials, and the modified material should conform to the standards in draft TRH14[35].

The design of an overlay of sufficient thickness is based on the measurement of material properties of the existing pavement layers. The material properties used should give an indication of the shear limits of the material as a design should not allow these limits to be exceeded by the expected future traffic loads. Each inadequate layer should be tested and designed for seperately to ensure an adequate rehabilitation design. Two related design methods, using tests giving a good indication of the shear properties of materials, are recommended for use :

- CBR based method

This method, of which the design curves are given in Figure 17, was developed by the British TRRL[27,28] from the original CBR design curves (see reference 36, Figure 9.1). In this method traffic is explicitly included as a design parameter. (The different traffic categories of the British were transformed into the design parameters for traffic given in draft TRH4[14]) According to Road Note 29[28], the moisture content at which the material samples are tested should, as closely as possible, approximate the natural moisture content in the road. However, this is often difficult to assess and in the absence of this information it is advisable to use the soaked CBR for design purposes.

With available CBR results and the calculated expected future traffic for the rehabilitation design period, Figure 17 is used to obtain the cover required for each pavement layer.

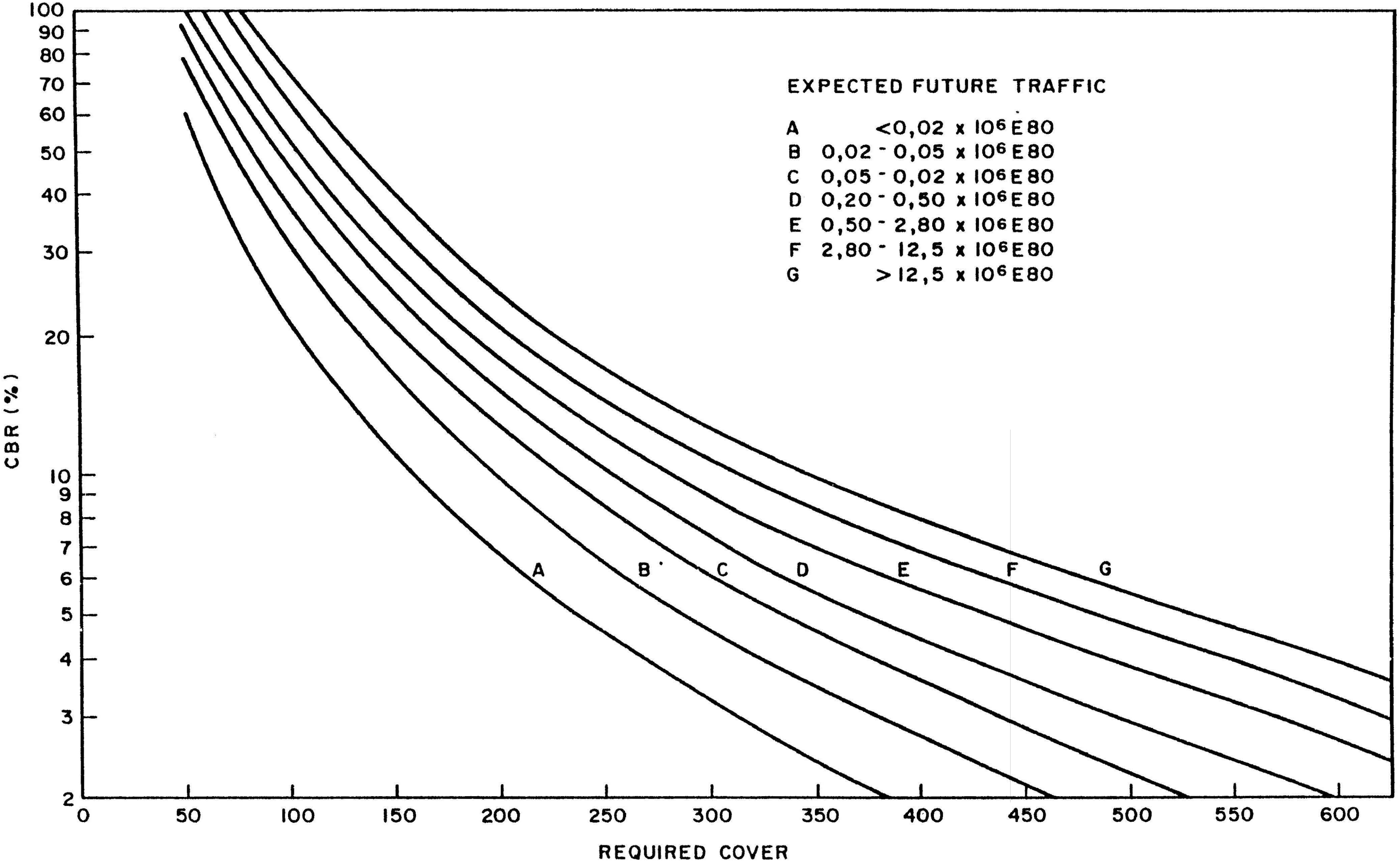

FIGURE 17

COVER REQUIREMENT BASED ON CBR (27,28)

- DCP based method

This method is derived from the TRD[20] curves for the design of new roads in which DCP measurements are used as indicative of the in situ CBR of the material. The convenience of measurement taking makes this a very attractive design method. Provision is made for three traffic categories, i.e. heavy, medium and light, of which the design curves are given in Figure 18.

The measured DCP penetrations and the calculated expected future traffic for the rehabilitation design period are used in Figure 18 to obtain the required cover for each pavement layer. Overlays are not considered a viable option where the existing base course has a DCP penetration of greater than 6 mm/blow. In these cases improvement of the existing material should rather be considered.

Any of the above two methods is suitable for obtaining the required cover for inadequate pavement layers. However, in the case of rehabilitation design, these inadequate layers are already covered by a certain thickness of material which should be taken into account in determining actual rehabilitation needs. This is done in Figure 19, where the required cover and the existing cover is used to determine the applicable rehabilitation category for each inadequate layer. The greater category as determined by any of the pavement layers is considered the most suitable to the rehabilitation needs of the pavement. Applicable designs for the different rehabilitation categories are given in Appendix 3 - Rehabilitation design catalogue.

5.2.3 Deformation originating in the subgrade due to insufficient load distribution by the pavement layers

In these cases it is obvious that the pavement is structurally inadequate and requires strengthening. This can be achieved by reconstruction, partial reconstruction or overlaying.

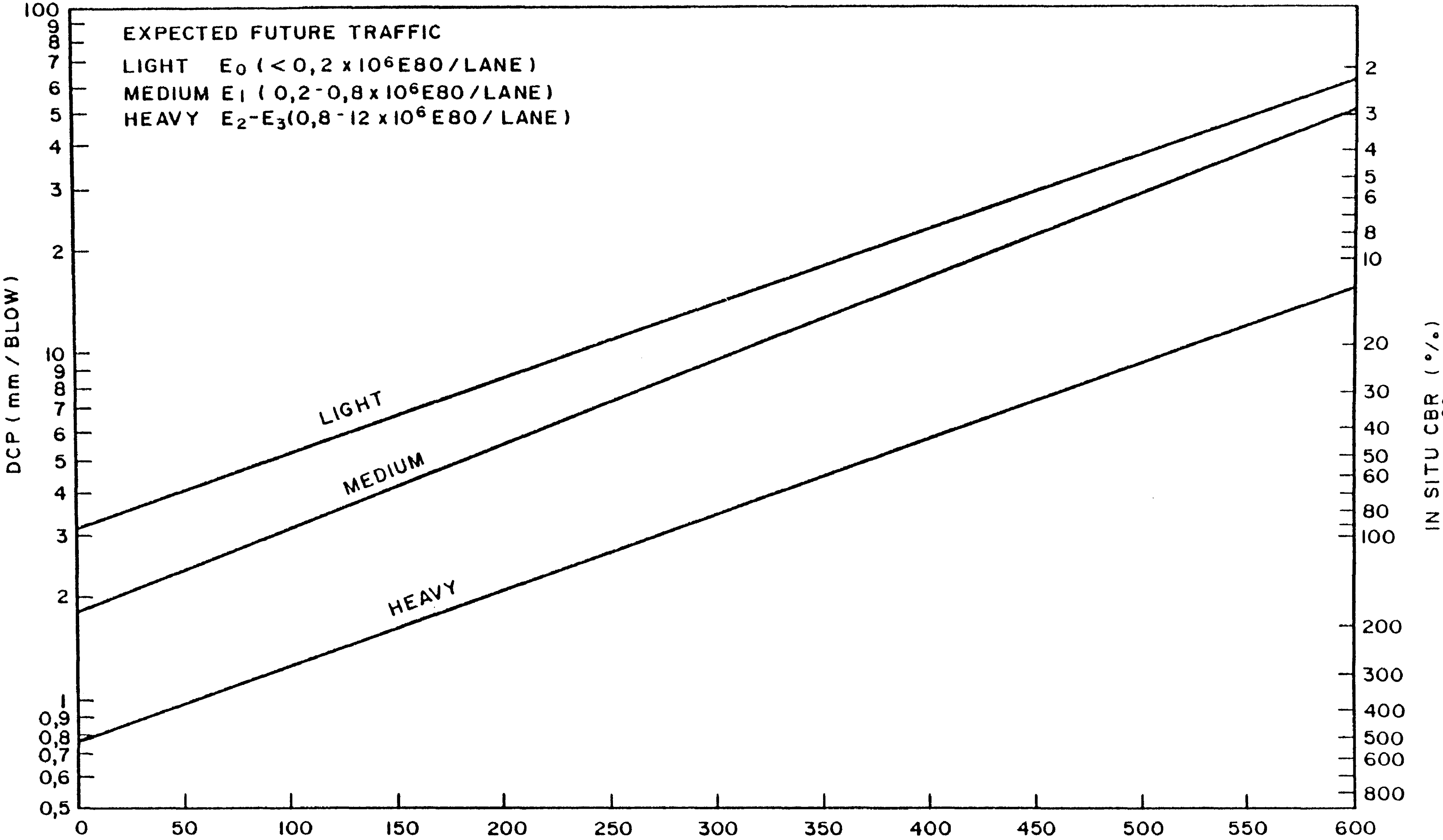

FIGURE 18

COVER REQUIREMENT BASED ON DYNAMIC CONE PENETRATION (20)

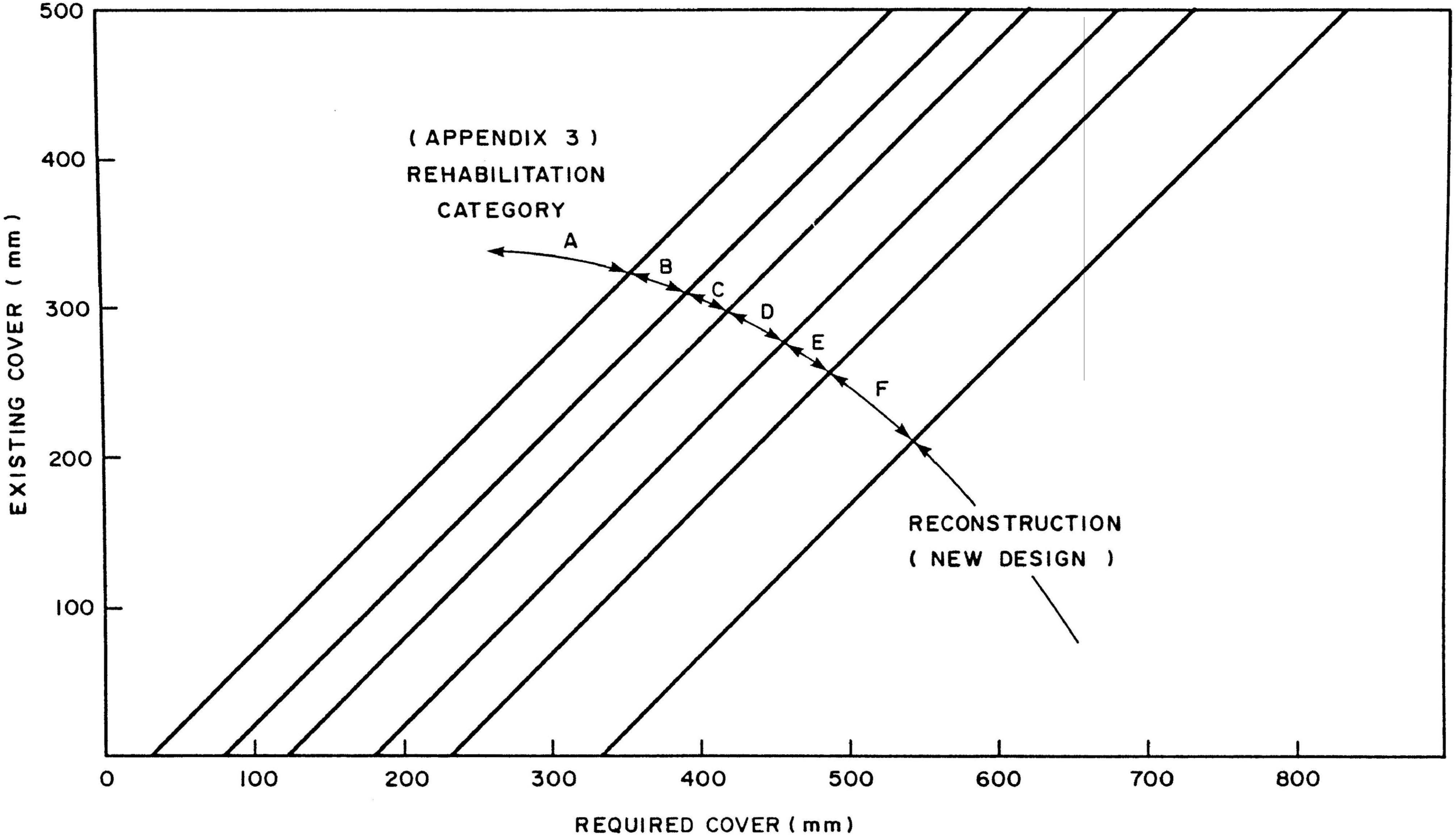

FIGURE 19

REHABILITATION CATEGORY CHART

In the recommended method, based on Grant[22], use is made of the existing relation between deflection and rut depth to determine the design deflection for the length of road. With the design deflection and measured deflection known, the strengthening required for each uniform section can be determined. In the absence of detailed rut depth measurements, use can be made in a similar manner of the relation between deflection and riding quality (see section 4.2.3 and 4.2.4). The former relation is now used to demonstrate the recommended method.

The relation found between deflection and rut depth during the detailed assessment as indicative of deformation originating in the subgrade, is used. The data from this relation is processed using a regression and correlation analysis (see Appendix 5) and an appropriate confidence limit (see Table 2) to obtain a limiting line as shown in Figure 20. This excludes with high confidence the occurrence of rutting in excess of specific depth at various deflections. This limiting line is used to determine the minimum deflection (0,75 mm in the example) that has resulted in a terminal level of rut depth of 10 mm under the past accumulated traffic. This deflection is plotted against the calculated past traffic on Figure 21, to establish the deflection-traffic load relation or design line (parallel to the Asphalt Institute design line) for the particular pavement.

The calculated expected future traffic (ΣE80s) during the rehabilitation design period can now be used to determine the design deflection from the design line of the pavement (Figure 21). An appropriate rehabilitation category can now be determined using Figure 22 which is applicable in South Africa. (Grant, Marais and Uys[37]). Entering Figure 22 with the design deflection and the applicable percentile deflection values (see Table 2) measured over the design subsections of road, the rehabilitation category can be determined. Applicable designs for each rehabilitation category are given in Appendix 3. Suitable design subsections on the pavement are isolated by means of the coeffi-

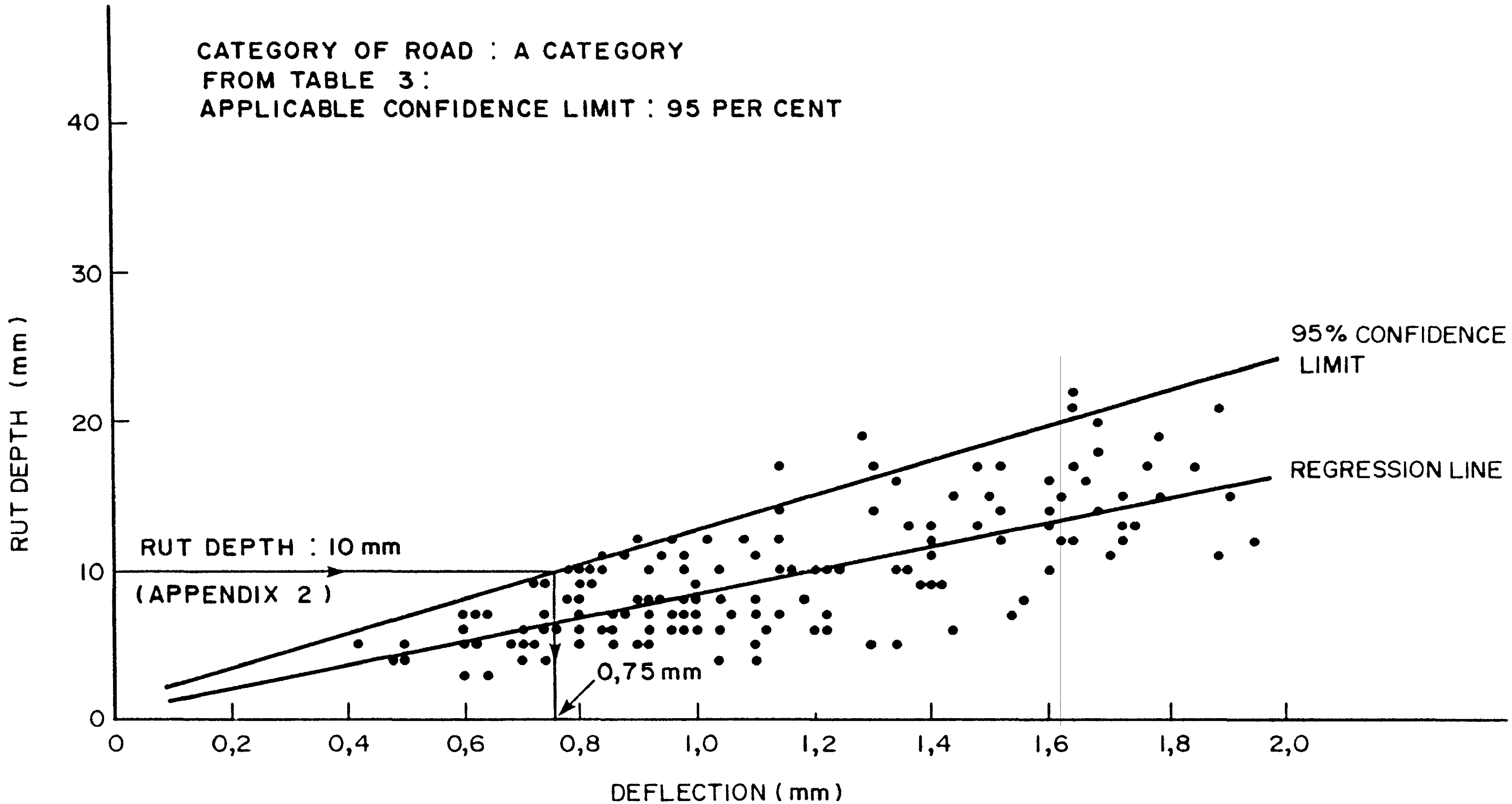

FIGURE 20

EXAMPLE OF THE USE OF THE RUT DEPTH-DEFLECTION RELATION IN ASSESSING THE REHABILITATION NEED OF A LENGTH OF PAVEMENT

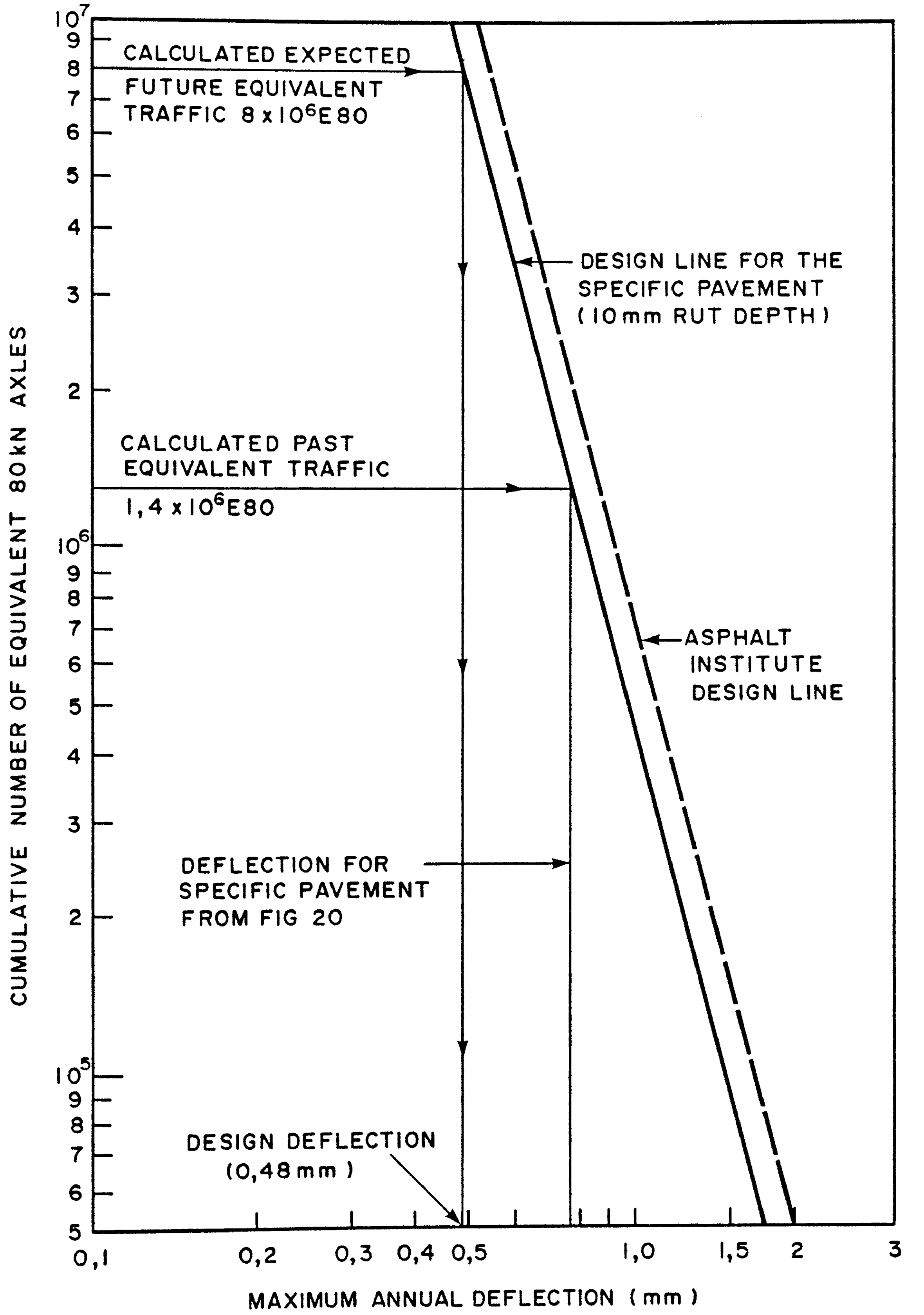

FIGURE 21

EXAMPLE OF THE USE OF THE DEFLECTION-TRAFFIC RELATION TO ESTABLISH CRITERIA FOR DESIGN FOR DEFORMATION DUE TO AN OVERSTRESSED SUBGRADE

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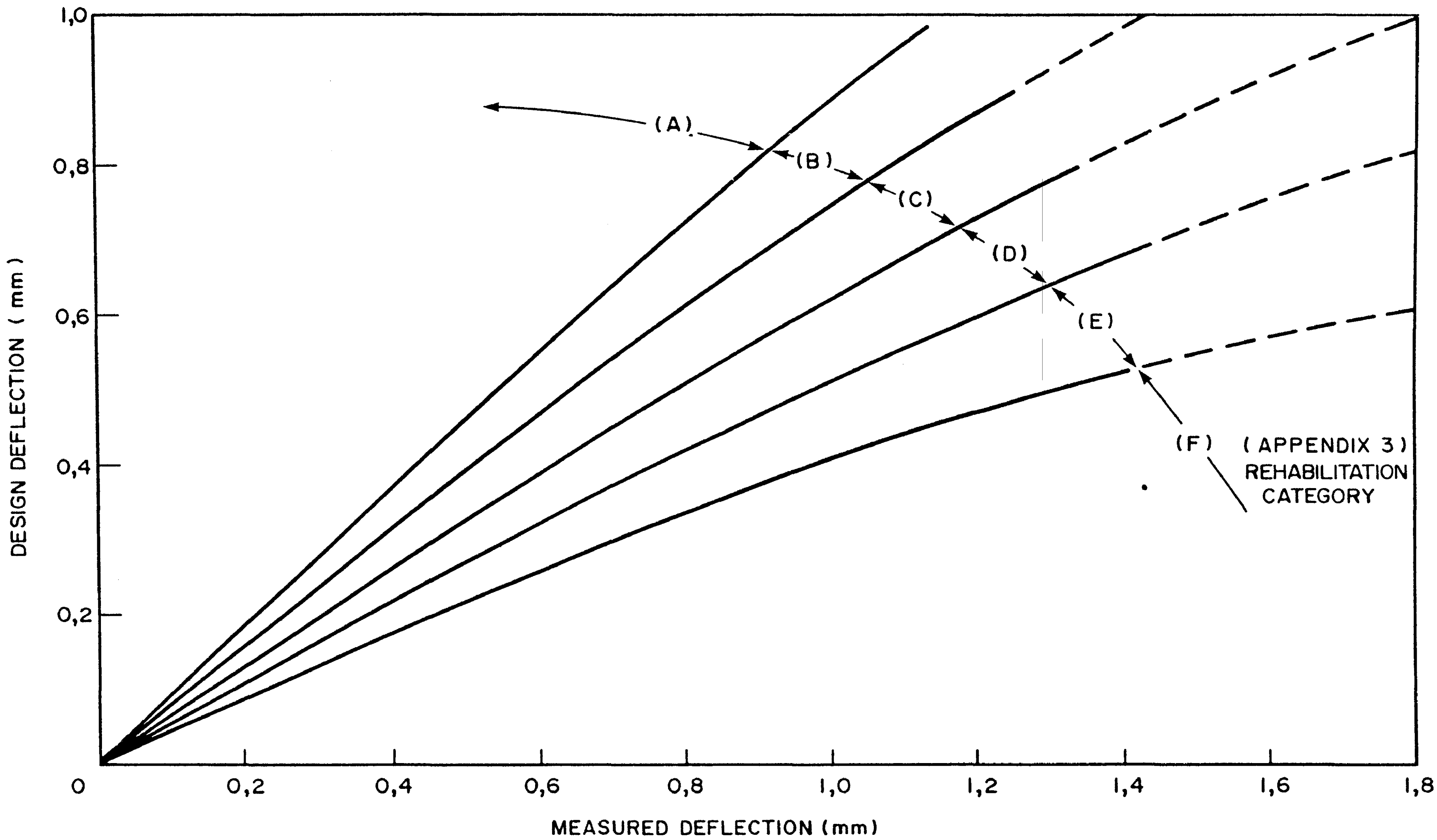

FIGURE 22

REHABILITATION DESIGN CHART (37)

cient of variation (see Appendix 5) of the measured deflections. With the exclusion of localized problem areas, it is generally impractical to vary the design too frequently (at less than about 0,5 km).

5.2.4 Deformation caused by active subgrades and collapse settlement

Since these movements* are not induced by repeated traffic stresses, measures aimed primarily at strengthening the pavement should be excluded. The primary design consideration should be to determine the most economical way of correcting the longitudinal surface profile. Nevertheless, reconstruction or an overlay will be required in order to correct the surface profile of the road. Attention should also be given to improve the existing drainage (both surface and subsoil) of the road.

5.2.5 Deformation caused by post-construction compaction of material

In this case the structural capacity of the pavement improves as the layers densify under traffic. Partial reconstruction, recycling or an asphalt overlay could be used to restore the surface profile. Measures in excess of a minimum overlay will only be required in cases where an exceptional increase in the future traffic is expected. (see section 5.4 - Improving Structural capacity).

5.3 Rectifying cracking

5.3.1 Cracking caused by the shrinkage of treated layers (reflective cracking)

Shrinkage cracking is brought about by non-traffic associated factors i.e. environmental factors, including expansion, contraction or shrinkage of, mainly, treated materials. These

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* Procedures for estimating the heave potential of subgrades are available [38,39]

cracks are usually formed in the base and/or subbase and, with time, propogate through to the surface to be known as reflective cracking.

This problem is dealt with by :

(i) covering with bituminous and/or granular layers, or

(ii) curing the source of the problem, i.e. removing or recycling the existing cracked surfacing.

(i) Covering of cracks with a bituminous and/or granular layer

The covering of reflective cracking serves as a relatively temporary solution as the cracks would, in time, again reflect through to the surface. Recently bitumen-rubber, geo-fabrics and other special products have become available in South Africa. Many of these products are claimed to prevent or slow down crack propagation. However, at this stage these materials have as yet not been proved to be economically viable solutions in South Africa.

Crack propagation through asphalt surfacings for pavements with less than 100 mm of cracked material may, according to Grant[22], vary from as little as 10 mm per year to more than 25 mm per year. This rate of propagation depends mainly on the asphalt material quality and the amount of movement of the cracked layers and pumping present in the existing pavement.

Where an asphalt overlay is proposed as the remedial measure, a mixture with a specified bitumen film thickness of not less than 10 μm is recommended. Design methods applicable in this case depend on the distress severity and structural requirements of the pavement. Where secondary traffic associated cracking with rutting in excess of a warning level (10 mm) is present the original treated layer could be assumed to have been broken down to the state of a granular layer. In these cases the applicable design method as described in section 5.2, rectifying deformation, should be used. The characteristics of bound layers are, when required, determined by doing unconfined compressive strength (UCS) tests on samples taken from these layers. In these cases, the relation between UCS and CBR, given in Figure 23 as established by the TRD, is used to determine the equivalent CBR of these layers.

Where rutting is not a problem, but the pavement is considered structurally inadequate to carry the expected future traffic, the CBR based method given in section 5.2.1 together with Figure 23 for the bound layers, are used to determine the appropriate rehabilitation category.

The design methods described, will ensure that the pavement is structurally adequate, but will not entirely solve the problem of reflective cracking. Pavements with bound layers in excess of 150 mm, will require very thick overlays - rehabilitation category E or F in Appendix 3 - to allow for a crack-free life of a reasonable length. These overlays may well prove not to be economically viable.

Where no strengthening or improvement of the surface profile is required, holding actions such as crack sealing or a reseal to prevent the ingress of water into the pavement are often considered viable options. However, these methods are only applicable where movement between cracked blocks and pumping have as yet not started.

(ii) Curing the source of the problem

Because of the thickness of overlays required in the case of reflective cracking, alternative options such as ripping and recompaction, recycling or reconstruction can be economically more attractive and should therefore also be considered. These options, in contrast to an overlay, will present a permanent solution to the problem.

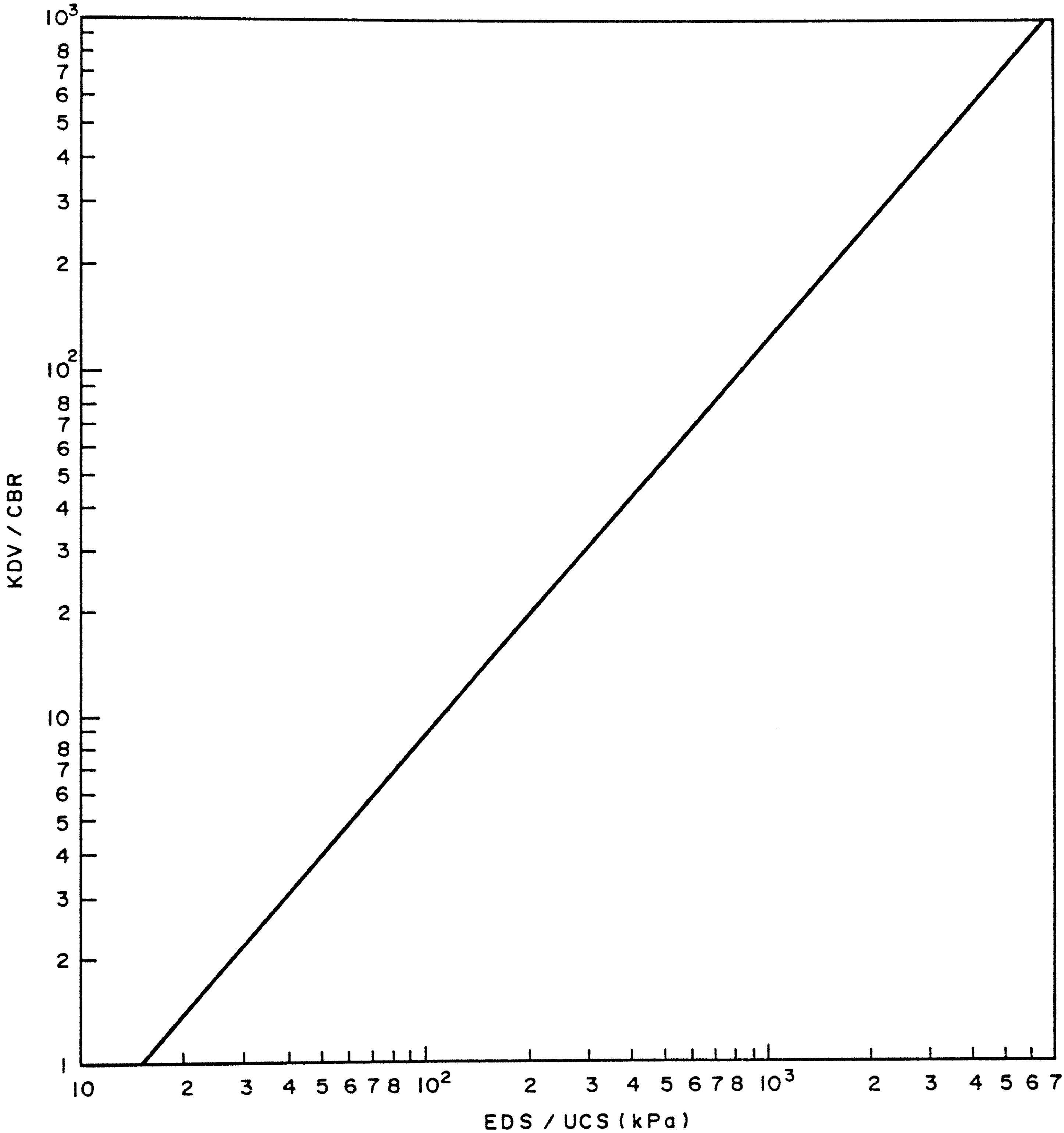

FIGURE 23

GENERAL RELATION BETWEEN UCS AND CBR (TRD[20])

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Where secondary, traffic associated cracking is as yet not present and the deflections measured are not in a severe condition (see Appendix 2), the removal of the source of the reflective cracking through the ripping and recompaction of the bound layer will also reduce the load bearing capacity of the pavement. According to Grant[22] a broken up and recompacted layer will be equal to half the thickness of the original bound layer. Grant[22] recommended the use of Figure 24 to assess the effect of ripping and recompaction on the structural capacity by calculation of the expected increase in deflection measurements. The thickness of the bound layer to be ripped and recompacted and the representative measured deflection of the pavement are needed as input data.

In Figure 24, the representative measured deflection is projected from the vertical axis to intersect with the line indicating the thickness of the bound layer. This gives an indication, on the horizontal axis, of the deflection that would occur if the bound layer was to be removed. The projection of this deflection to a line, representing half the thickness of the original bound layer will now, on the vertical axis to the right, give an indication of the deflection that can be expected by ripping and recompaction of the bound layer. This deflection is used in section 5.4 to determine the additional cover required to carry the calculated expected future traffic.

Where the bound layer has already been broken down by traffic, but ripping and recompaction is still considered necessary to relieve any stress concentrations, no decrease in the bearing capacity is expected. The measured surface deflections with the methods recommended in 5.3.1(i) are then used to determine any additional cover required to make the pavement structurally sound for the rehabilitation design period.

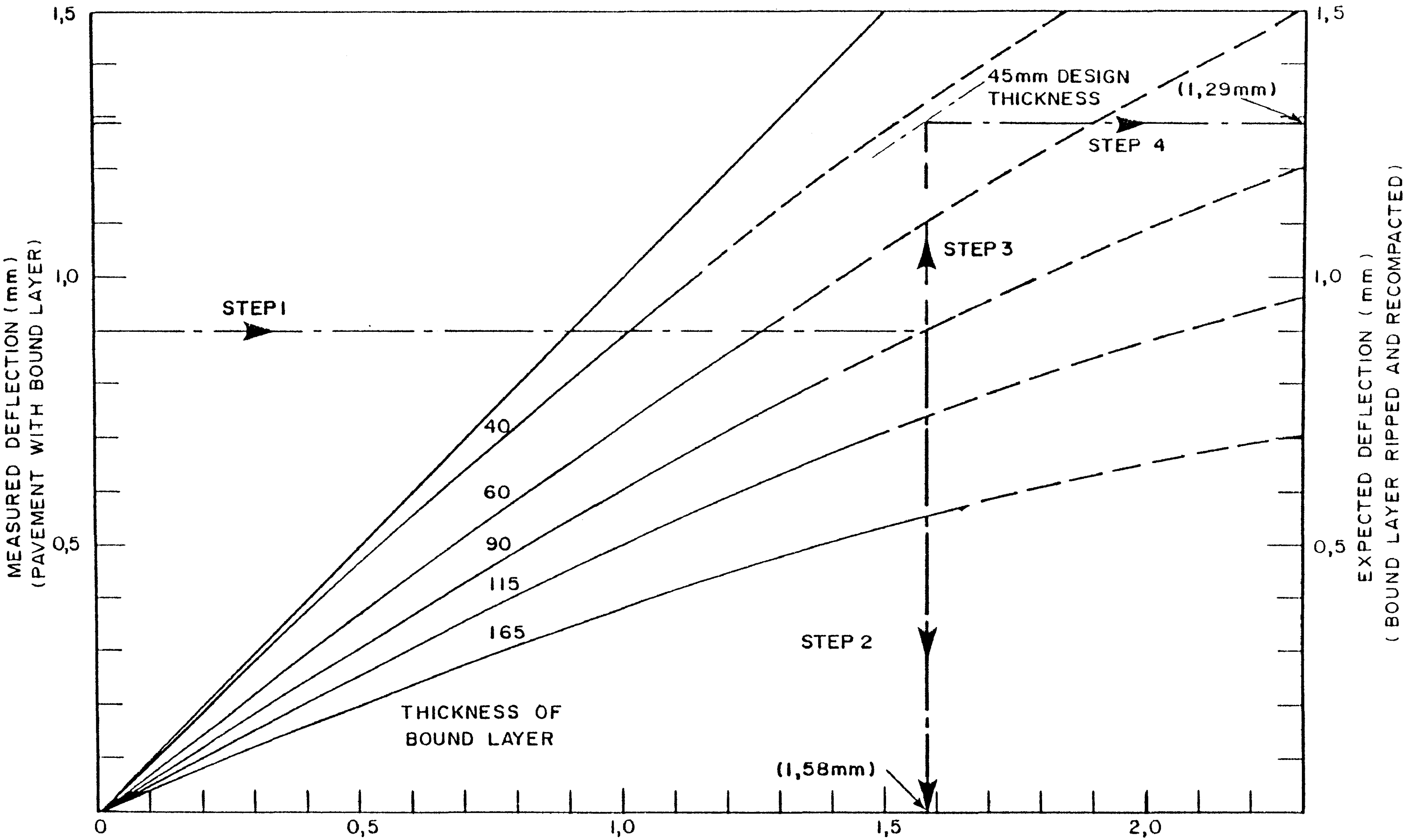


FIGURE 24

DEFLECTION REDUCTION DESIGN CHART [37]

The use of Figure 24 is demonstrated in the following example.

Example

Assume a cracked layer thickness of 90 mm, a representative deflection of 0,8 mm and design traffic of 7 x 10^5 E80.

In Figure 24, Steps 1 and 2 give the deflection that would occur if the cracked layer were to be removed as 1,58 mm (Deflection A). Steps 3 and 4 give the deflection of the treated layer as 1,29 mm following ripping and recompaction (Deflection B).

An applicable overlay thickness can be obtained by using section 5.4

5.3.2 Caused by traffic-induced fatigue of asphalt layers

This cracking is usually the result of a combination of factors, including a relative flexible pavement (high deflections) and poor fatigue resistance properties of the asphalt layer. Remedies should therefore centre around the stiffening of the pavement (reduce the flexibility - the deflection) or the improvement of the fatigue properties of the layer.

Stiffening of the pavement should only be considered when the structural capacity of the pavement is inadequate for the calculated expected future traffic. In these cases section 5.4 should be consulted to design an overlay of sufficient thickness. Reflection of cracks through to the surface may present a problem and section 5.3.1 should also be consulted.

Where the structural capacity is adequate for the future traffic, replacement, recycling or covering of the existing asphalt layer to get a rehabilitated product with adequate fatigue resistance properties can be considered. The resistance of asphalt layers to fatigue can vary considerably. Grant[22] pointed out that fatigue resistance increases with an increase in binder content and a decrease in void content. However, factors such as aging of the material (aggravated by an open mix) or a loss in ductility (causing brittleness) as a result of an excess of heat during the preparation and placing of a layer, cause a rapid decrease in the fatigue resistance of an asphalt layer. The design of the asphalt should conform to the design criteria given in TRH8. An overlay with a volume binder concentration of greater than 10 per cent and low voids to prevent rapid aging of the binder should be safe against fatigue cracking.

5.3.3 Caused by shrinkage, thermal forces and/or brittle materials of asphalt layers

Non-traffic associated factors such as asphalt material properties and environmental conditions are responsible for this cracking and strengthening or stiffening of the pavement is not required. A seal or thin overlay is usually used to seal the surface to prevent the ingress of water into the pavement. However, propagation of the existing cracks through to the surface of the new overlay may present a problem. Ripping and recompaction or recycling could be viable alternative options in this case. The design methods recommended in section 5.3.1 (ii) on the curing of reflective cracking should be used to determine the effect of removing or ripping of the layer. A new overlay should conform to the design criteria given in TRH8[23].

5.4 Improving structural capacity

Pavements with an inadequate structural capacity as identified in section 4.4 and which are still in a sound condition concerning deformation, are dealt with in this section. As no serious deformation is as yet present, existing distress formations cannot be used to design applicable overlays or other rehabilitation alternatives (see section 5.2.3). Generally expected trends in pavement behaviour must be used in these cases.

Two methods are recommended for use in section 4.4 for structural capacity analysis. The same principles are now applicable in the design of applicable rehabilitation options for pavements with inadequate structural capacity to carry the calculated expected future traffic.

5.4.1 Deflection based method

Section 4.4.1 gives the background to the Asphalt Institute's curve, discussing its applicability for use in predicting the life of a pavement. Using the same principles, this curve can now be used with the calculated expected future traffic to obtain a design deflection for the improvement of the structural capacity of the pavement. This design deflection and the deflection representative of the pavement (see Table 2) are now used in Figure 22 to find an applicable rehabilitation category. Rehabilitation options for the different rehabilitation categories are given in Appendix 3.

<u>Example</u> :

The example used in Section 4.4.1 to illustrate the calculation of the structural capacity of the pavement is now used to determine the rehabilitation needed to ensure that the structural capacity is sufficient for a required 20 years of trafficking.

From section 4.4.1

Category of road A

Deflection (95th percentile)	d_r = 1 mm
Standard deviation of deflection	S_d = 0,19
Past E80 loads	= 6×10^5 E80
Present annual E80 loads	E_o = 36×10^3 E80
Expected E80 growth rate	= 6 %
Expected future E80 (20 years)	= $1,4 \times 10^6$ E80

$$\therefore \quad \frac{N_c}{N_e} = 2,9 > 1$$

Therefore, it is evident that structural strengthening of the pavement will be required to improve its structural capacity and thereby to increase its expected life.

Since the pavement will be rehabilitated and thereby improved, only the expected future traffic for the rehabilitation design period is used to obtain a design deflection from the Asphalt Institute curve as is shown in Figure 25(a). The design deflection obtained (0,86 mm) and the measured deflection (1,0 mm) are then used in Figure 22 as shown in Figure 25(b) to obtain the rehabilitation category required to sufficiently improve the structural capacity of the pavement

From Figure 25(b)

Rehabilitation Category B in Appendix 3 is the remedial action required in this case.

5.4.2 DCP based method

DCP measurements can also be used to obtain adequate rehabilitation designs. In this case section 5.2.2 (DCP based method) is applicable. The calculated expected future traffic and the DCP measurements to a depth of 800 mm are used to find an appropriate rehabilitation category. Appendix 3 gives various rehabilitation options including the thickness of asphalt required, for each category.

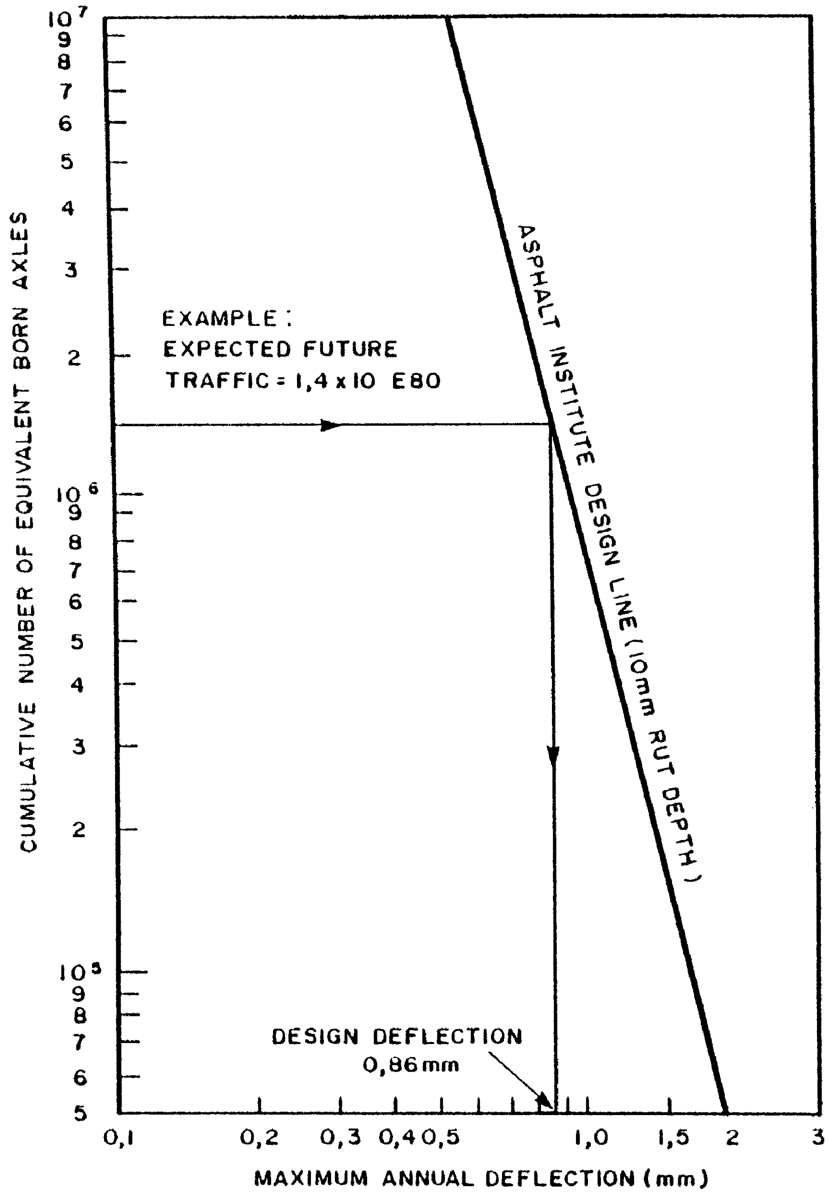


FIGURE 25a

EXAMPLE OF THE USE OF THE ASPHALT INSTITUTE CURVE

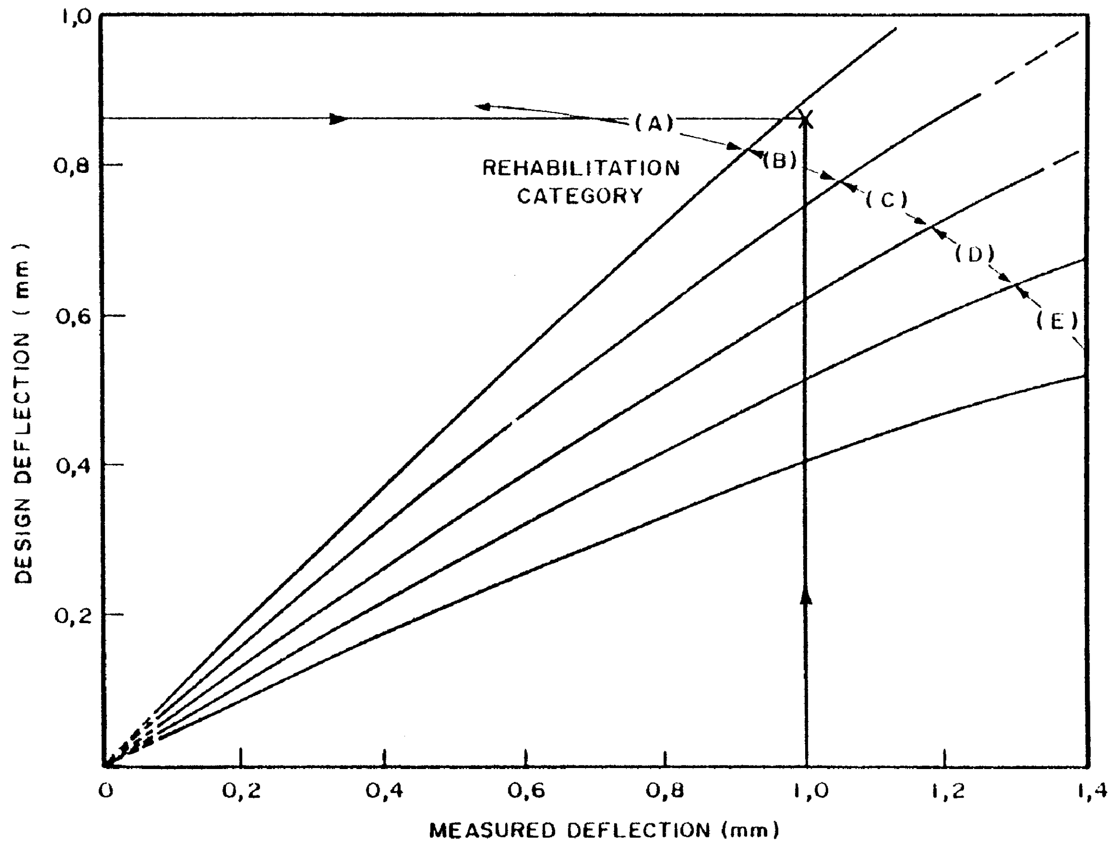


FIGURE 25b

EXAMPLE OF THE USE OF THE REHABILITATION DESIGN CHART (DEFLECTION BASED)

Example :

The following data are available for a section of road :

Category of road A

Drainage condition - wet moisture regime = M3

Σ Past traffic = 5×10^5 E80

Present annual traffic E_o. = 50×10^3 E80

Future expected E80 growth rate = 8 %

Σ Future expected traffic (20 years) = $2,5 \times 10^6$ E80

From an analysis of the DCP measurements taken on the section of road the following were found

Pavement layer (x)	Thickness (h_x)	95th Percentile values of the DCP measurements (mm/blow) (DN_x)
1	50	1,0
2	200	2,14
3	150	5,0
4	200	10,0
5	200	30,0

From the available data :

$$N_c = \Sigma \text{ Past traffic} + \text{Future traffic}$$
$$= 5 \times 10^5 + 2,5 \times 10^6 \text{ E80}$$
$$= 3,0 \times 10^6 \text{ E80}$$

$$DSN_{800} = \sum_{x=1}^{n} \frac{h_x}{DN_x}$$

$$= \frac{50}{1,0} + \frac{200}{2,14} + \frac{150}{5,0} + \frac{200}{10,0} + \frac{200}{30,0}$$

$$= 200$$

This is the same section of pavement used in the example in section 4.4.2 for a structural capacity analysis. In that example it was illustrated that pavement strengthening is necessary for a design period of 20 years.

Using the data from the DCP measurements (95th percentile) and Figures 18 and 19 in section 5.2.2 the required rehabilitation category can be determined as shown in Figures 25(c) and 25(d).

From Figure 25(c) and 25(d)

Pavement layer	Required cover	Existing cover	Rehab Category
1	55	0	B
2	190	50	D
3	370	250	D
4	525	400	D

Therefore, rehabilitation Category D in Appendix 3 will give the required remedial action.

(Note : Although pavement layer 5 is not shown in the analysis on Figures 25(c) and 25(d) because of a restriction on the size of the figures, it would require remedial action of rehabilitation Category C).

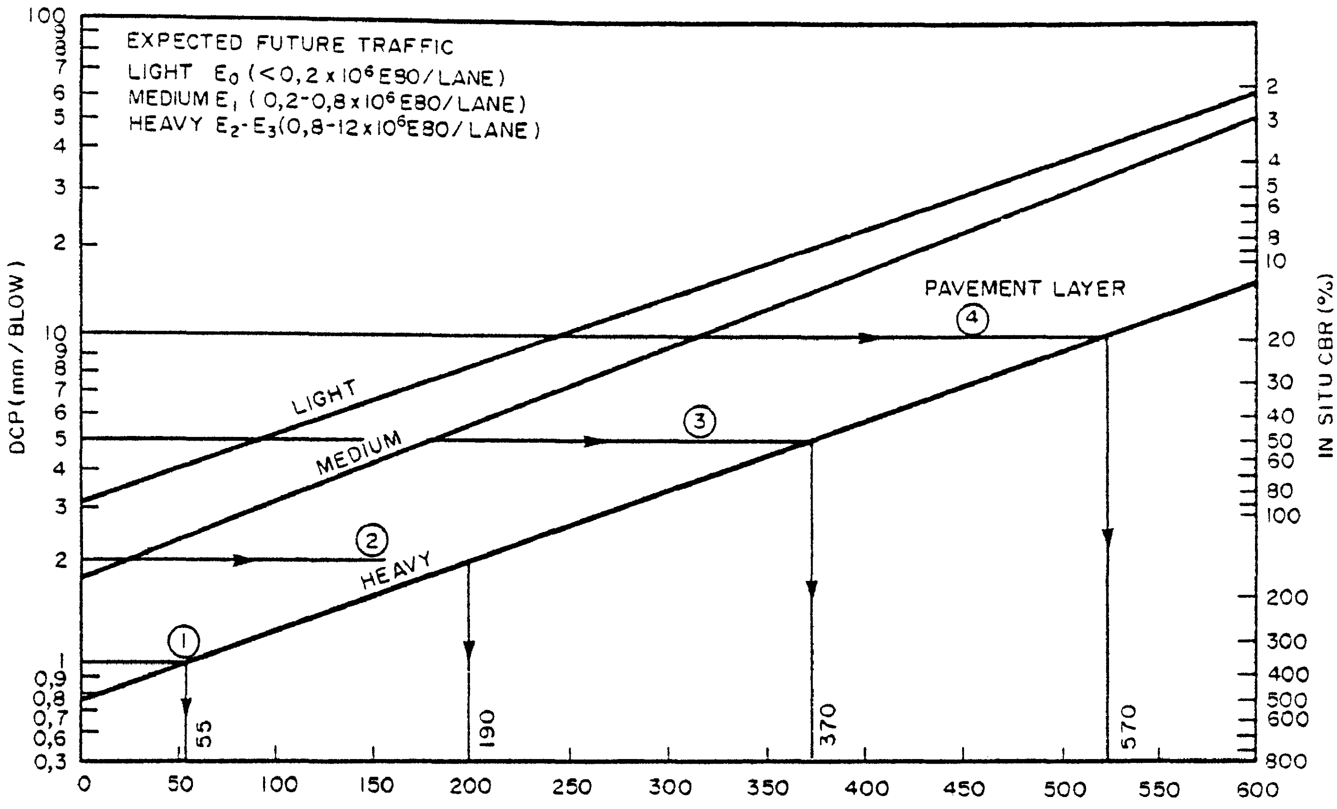


FIGURE 25c

EXAMPLE OF THE USE OF THE COVER REQUIREMENT CHART BASED ON DYNAMIC CONE PENETRATION MEASUREMENTS

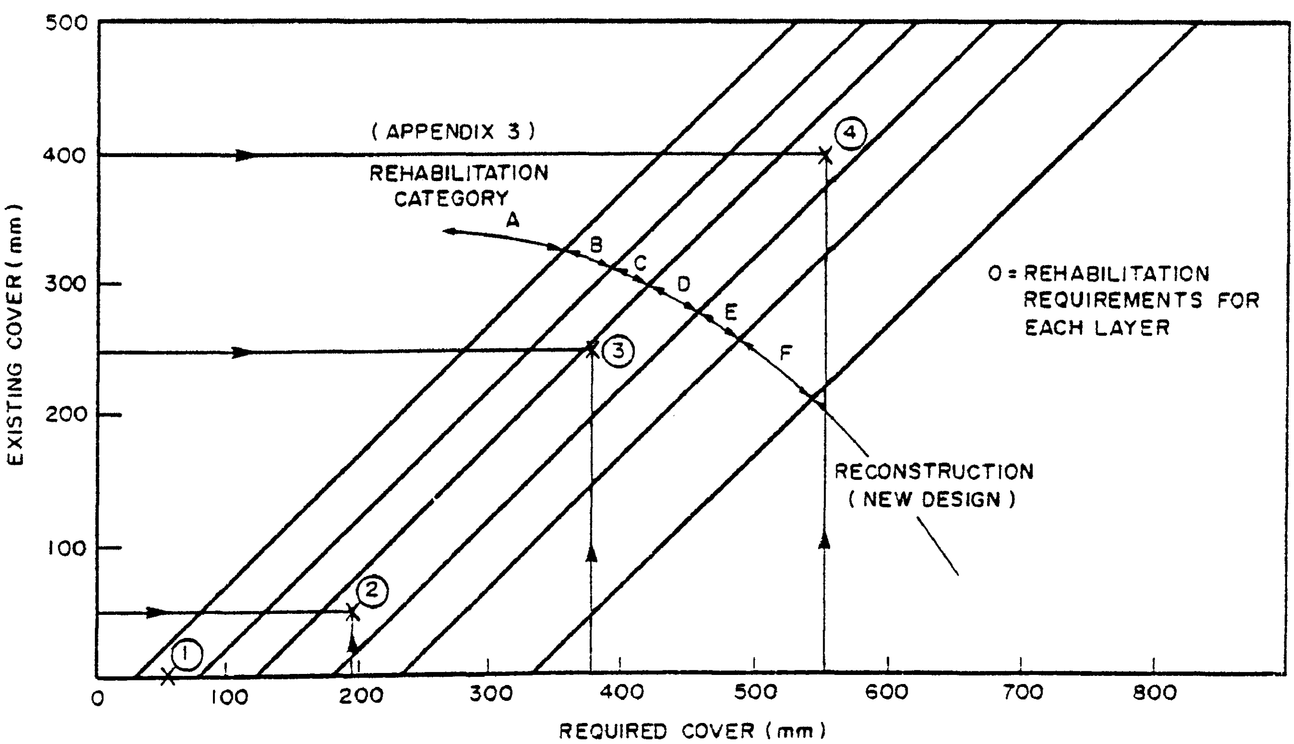


FIGURE 25d

EXAMPLE OF THE USE OF THE REHABILITATION CATEGORY CHART BASED ON COVER REQUIREMENTS

SECTION 6

Economic Analysis

6.1 Introduction

6.2 Agency Costs

6.2.1 Rehabilitation costs
6.2.2 Maintenance costs
6.2.3 Future capital costs
6.2.4 Salvage value

6.3 Road user costs

6.3.1 Delay costs due to rehabilitation activities
6.3.2 Vehicle running costs
6.3.3 Accident costs
6.3.4 Time costs

6.4 Principles of the economic analysis

6.4.1 Present worth of costs
6.4.2 Specific aspects

6.4.2.1 Analysis period
6.4.2.2 Inflation
6.4.2.3 Discount rate

6.5 Incorporating uncertainty

6.5.1 Example

6. ECONOMIC ANALYSIS

6.1 Introduction

The economic analysis is used as part of the rehabilitation procedure in the final stages of project evaluation. All applicable rehabilitation alternatives, as designed in the preceding sections, are now compared on an economic basis.

The economic analysis should only be regarded as an aid in the decision-making process. Other factors such as political or strategic considerations are also taken into account and are often decisive in the process of decision making. Nevertheless, the economic analysis provides an important basis for sound and objective decision-making.

In practice, road authorities are often faced with a number of projects competing for limited funds. In these cases, an economic analysis with the emphasis on a cost-benefit study often provides the motivation needed to secure funds for specific projects. For the purpose as an aid to decision making at a rehabilitation project level, the complexity of the economic analysis is much reduced by many common factors such as traffic conditions and project lengths.

The objective of the economic analysis as part of the rehabilitation design procedure is to determine the most economical of the appropriate remedial options as designed. This is done by taking into account agency costs as well as road user costs[40, 41, 42]. These include :

- Agency costs :
  initial rehabilitation costs (including costs for traffic accommodation);
  maintenance costs during the analysis period (including adminstration costs);
  future capital costs (e.g. stage remedial action);
  salvage value at the end of the analysis period.

- Road user costs:
  - delay costs due to rehabilitation activities;
  - vehicle running costs;
  - accident costs;
  - time costs.

The different rehabilitation options are compared by calculating the total of the present worth of the costs listed above (PWOC), for each option. This calculation must also provide for incorporating the probability of different outcomes and different resultant actions during the analysis period, for each option. This is facilitated by the use of decision trees and Bayesian analysis as suggested by Curtayne and Servas[3].

The recommended procedure is outlined in Figure 26.

6.2 Agency Costs

6.2.1 Rehabilitation costs

This cost item is the initial cost involved in the rehabilitation of the section of road, using the option under investigation. Current unit costs should be used in the calculations which should include all relevant capital cost items associated with the rehabilitation construction, such as the costs involved in the accommodation of traffic.

6.2.2 Maintenance costs

Although the rehabilitation of the road provides in the structural requirements of the pavement for the duration of the design period, routine maintenance will still be needed. Costs associated with maintenance should be included in the economic analysis and will include costs involved in the regular inspection of the pavement, administration costs and any reparation due to natural aging and weathering of the pavement. These include the repairing of skid resistance and the sealing of cracks which can occur after a period of time.

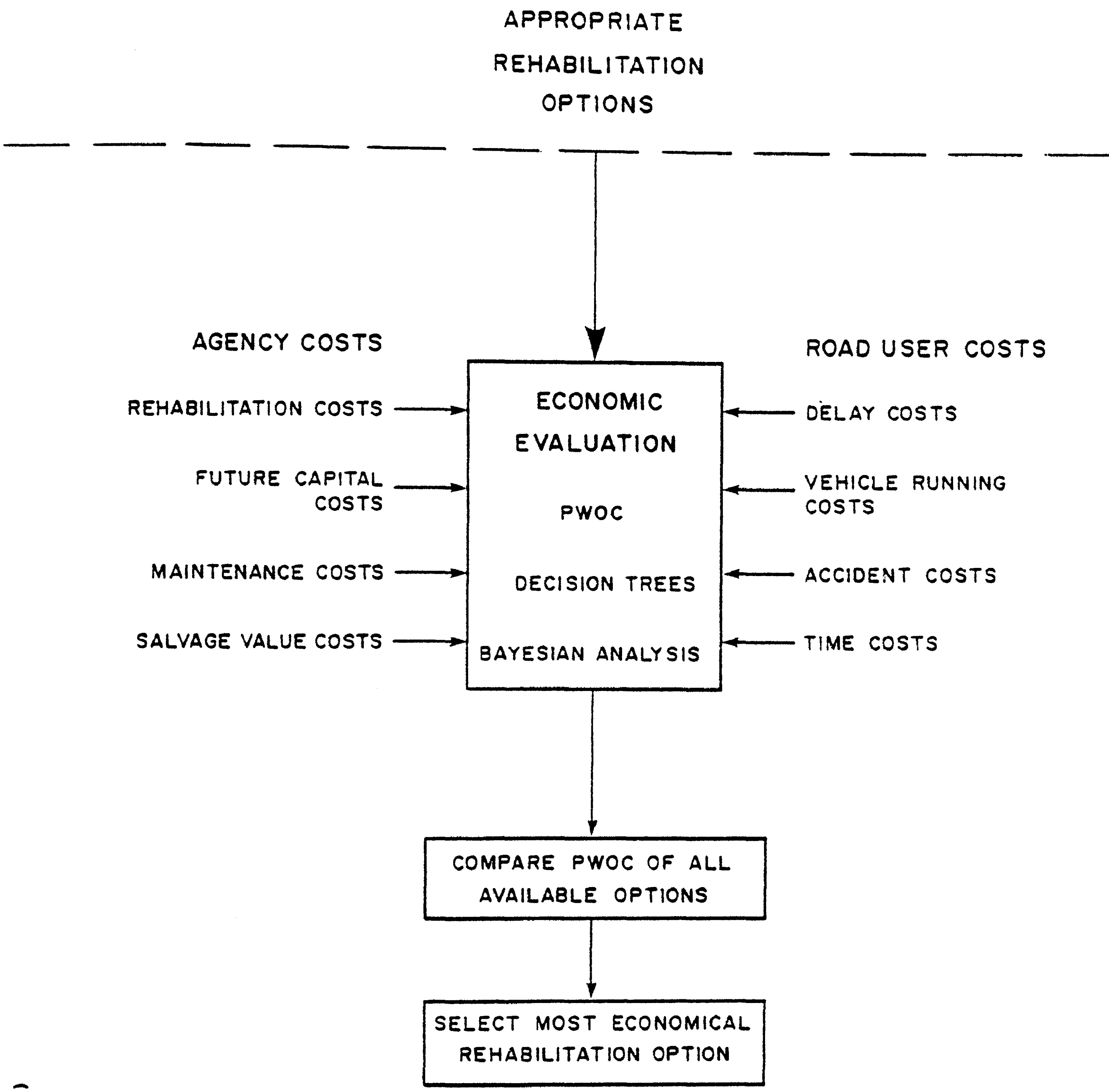


FIGURE 26

FLOW DIAGRAM OF THE ECONOMIC ANALYSIS AS PART OF PAVEMENT REHABILITATION DESIGN

Many of these costs are difficult to assess, but rehabilitation options are often known to react differently with time due to the susceptibility of certain materials to aging, cracking or weathering. Therefore, the process of the economic analysis should provide for the likelihood of these additional costs occurring. In the calculations these costs are included as current unit costs.

6.2.3 Future capital costs

This cost item includes all major capital costs foreseen during the rehabilitation design period as the result of the adoption of a specific rehabilitation option or strategy. These costs occur when, due to monetory or political reasons, a holding action with future iterim rehabilitation actions are considered or in the consideration of stage rehabilitation construction. These costs are incorporated by using current unit costs.

This cost item could also include costs due to major rehabilitation involvement associated with the appearance of distress or non-maintenance of the road resulting in possible structural damage. In these cases the postponement of effective maintenance may result in the requirement of structural strengthening. The possibility of the need for interim rehabilitation cannot be disregarded and taking into account the category of road, the policy of the road authority and the rehabilitated pavement structure, provision should be made for including these possible further actions.

6.2.4 Salvage value

The recent development in the technology for the recycling of materials, especially bituminous materials, in South Africa emphasized the need to include the salvage value of a pavement in an economic analysis. This value of the pavement at the end of the analysis period is dependant on various factors, including :

- the anticipated use of the material or pavement at the end of the analysis period. These could involve :

  (a) the recycling of the material;
  (b) the removing and selling of the material;
  (c) the rehabilitation of the pavement and hence re-use of the material as a foundation for a new road;
  (d) the abandoning of the road with or without restoring the original vegitation;
  (e) combinations of the above;

- the volume, the type, the age and the expected life of the material;
- the market value of the pavement or material at the end of the analysis period.

It is clear that the salvage value is often difficult to assess due to various unknown factors. However, due to the length of the analysis period and the use of a discount rate in the calculations, small differences in salvage value estimates usually have very 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 could 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 as recommended by Epps, Finn and Monismith[40], where :

$$\text{Salvage value} = (1 - \frac{A}{B}) \text{ Cost of new overlay}$$

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

However, the above method can be misleading if the road is expected to be rehabilitated again at the end of the analysis period. The mear existance of the road, the fact that the road has already been established and is carrying traffic is worth a lot and this could be a main contribution to the salvage value of a road. In these cases the difference between the costs in constructing a new road and the cost in rehabilitation and/or upgrading of the existing road would best represent the salvage value of the pavement.

Since the object of the economic analysis as part of the rehabilitation procedure is to determine the most economical of the available rehabilitation options, the salvage value could be 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. All costs are determined by using current unit costs.

6.3 Road user costs

Road user costs are usually difficult to determine and hence are often neglected or disregarded in the economical evaluation of transportation facilities. This can lead to gross inaccuracies in calculating total costs, as these costs, taken over the design period, are usually far in excess of the agency costs.

However, in the economic analysis as part of the rehabilitation design procedure, many of the road user cost items can be disregarded. In this case, the economic analysis is used to compare the available rehabilitation options, with each option providing an acceptable riding surface during the design period along the same route. It follows, that many of the road user cost items will be equal for all the available options and therefore, would have no influence on the comparatable total economical costs of the alternatives. In this case, the inclusion of these cost items in the analysis will serve very little purpose.

6.3.1 Delay costs due to rehabilitation activities

This cost item can be significant, especially with the rehabilitation of a heavily trafficked road such as an interurban freeway. Again, only the differences between the costs for the various options are of importance.

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

Delay costs, due to the longer time spend on the road, the slower speeds and the more congested traffic conditions, will involve time costs, vehicle running costs and accident costs. The compatable effect of all these contributory items should be included in a thorough study.

6.3.2 Vehicle running costs

This cost item taken over the design period, can usually be disregarded in the economic analysis because it is usually the same for the different available options. This is due to the meeting of design requirement by the various options, e.g. the provision of an acceptable service over the design period along the same route.

Vehicle running costs comprise of the following components[41, 42] :

- fuel costs;
- oil costs;
- tyre costs;
- depreciation costs; and
- parts and maintenance costs.

Visser[43] and Schutte[44] give procedures for the calculation of these costs together with data (1983) applicable in South

Africa. In the calculation of these costs as part of delay costs due to rehabilitation activities, some of the components, such as depreciation costs are, offcourse, not applicable. Each component needs careful consideration and should not be blindly incorporated into any analysis.

6.3.3 Accident costs

This cost item is determined by the accident frequency and accident unit costs. Schutte[44] and Visser[43] give information applicable in South Africa for use in calculating accident costs.

Taken over the rehabilitation design period, these costs will usually be equal for the various rehabilitation options and can be disregarded. However, during the rehabilitation construction period, some options may cause traffic congestion, resulting in an increase in the number of accidents and accident cost.

6.3.4 Time costs

This cost item is the most difficult to assess, but is only of consequence as part of delay costs due to rehabilitation activities.

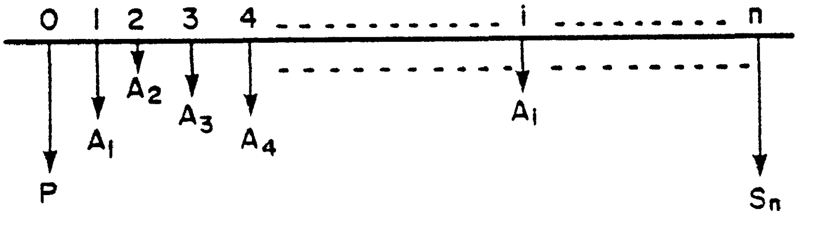
Time is only of value when the time saved on one activity is used productively for another or similar activity. Practically it is impossible to assess whether every individual is using any time saving productively. Therefore, this cost item is to a great extent determined by making various assumptions. Schutte[44] gives figures for the average value of time in South Africa.

6.4 Principles of the economic analysis

6.4.1 Present worth of costs (PWOC)

The different cost items, as discussed in the preceding sections, need to be meaningfully combined into a single figure for each rehabilitation alternative. The present worth of cost method (PWOC) is suggested for use for this purpose. With this method all costs occurring some time in the future are discounted to the present worth of costs by using an acceptable discount rate. The rehabilitation option which produces the least expected PWOC should be selected as the best applicable rehabilitation option.

The costs during the rehabilitation design period for a rehabilitation option are graphically shown in Figure 27.



P = INITIAL REHABILITATION COSTS (INCLUDING DELAY COSTS)

A_i = ANY FUTURE COSTS DURING THE ANALYSIS PERIOD OCCURING i YEARS FROM THE TIME OF THE INITIAL REHABILITATION (ie. MAINTENANCE COSTS, FUTURE CAPITAL COSTS, ETC.)

S_n = SALVAGE VALUE COSTS AT THE END OF THE ANALYSIS PERIOD

n = ANALYSIS PERIOD

FIGURE 27

COST-FLOW DIAGRAM OF A REHABILITATION OPTION

The present worth of costs of the option shown in Figure 27 is obtained by using the following formula

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

with PWOC = Present worth of costs

P = Initial rehabilitation costs

A_i = Relevant costs occurring during the analysis period after i years

S_n = Salvage value costs at the end of the analysis period n years later

n = Analysis period

r = discount rate

6.4.2 Specific aspects

6.4.2.1 *Analysis period*

The analysis period is the time span over which the economic analysis is used to compare different alternative options 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 economic analysis is done by using the PWOC method, the analysis period must be the same for all the alternative options within a specific rehabilitation study.

6.4.2.2 *Inflation*

When all cost items escalate in cost 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, some materials escalate in costs at much higher or lower rates that the average inflation rates (e.g. petrol from 1974-1983). In these cases these differences can be taken into account by applying the approach followed in the following example.

Assume that asphalt is expected to escalate in cost at an annual rate of 15 per cent for the next 10 years, while 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:

(Assume a current discount rate of 8 %)

Current cost (1983) of a 50 mm of ashpalt overlay = R50 000/km

Discount rate = 8 per cent

The expected 1993 value of a 50 mm asphalt overlay in 1993 money = R50 000 $(1+0,15)^{10}$

= R202 278

The 1993 value of a 50 mm asphalt overlay in 1983 money

= R202 278 $(1+0,10)^{-10}$

= R77 987

The PWOC (1983) of a 50 mm asphalt overlay placed in 1993

= R77 987 $(1+0,08)^{-10}$

= R36 123

6.4.2.3 *Discount rate*

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

- the effective rate of borrowing money; and
- the rate of return that money can earn if invested.

At present, it is recommended that a rate of 8 per cent to 10 per cent is used for the purpose of the economic analysis[42].

6.5 Incorporating uncertainty

With the discussion of the various cost items, mention has repeatedly been made of the need to provide for the possible occurance of an event in the future, in the economic analysis. This is done by incorporating the principles of the Bayesian analysis and decision trees as suggested by Curtayne and Servas[3] into the PWOC method. These principles are contained in Appendix 4.

The uncertainty that exists in the future behaviour of any rehabilitation alternative warrants the inclusion of these aids to the process of decision-making into the economic analysis. Although a designed option may structurally provide the need of a pavement for the next, 20 years, for example, the expected life of the asphalt surfacing (see Table 3) may only be about 10 years. Therefore, it is probable that some action will be needed to seal possible cracks or to restore a possible loss in skid resistance. Varying quality in materials, construction techniques or personal experience of conditions may further warrant the inclusion of other possible actions after a period of time.

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 into a formal analysis procedure.

The practical application of this system is illustrated by the following example.

6.5.1 Example

In this hypothetical example one rehabilitation option is analysed using decision trees and the Bayesian theory in calculating the PWOC of the option. Various outcomes and appropriate actions are considered over periods of 10 years and 20 years.

The following data are used :
Rehabilitation option = 50 mm of asphalt overlay
Analysis period = 20 years
Discount rate = 8 per cent

Current unit costs :

Single seal	R 7 000/km
Double seal	R10 000/km
Asphalt : 25 mm (Thin overlay)	R25 000/km
50 mm	R50 000/km
SAM (Rubber-bitumen overlay)	R12 000/km
SAMI (SAM + Asphalt overlay) : 25 mm	R37 000/km

In this example it is considered that the state of the pavement after 10 years would either be :

- Condition A : very bad, i.e. cracked and spalling (subjectively considered - 20 % probability) or
- Condition B : bad, i.e. cracked (subjectively considered - 40 % probability)
- Condition C : good, i.e. sound (subjectively considered - 40 % probability)

If, after 10 years the pavement will receive some treatment, the following options will be available :

- For condition A where the pavement is assumbed to be in a very bad state i.e. cracked and spalling, an ordinary seal would prove inadequate and a thin overlay (25 mm) or even a thin overlay with a Bitumen Rubber interlayer (SAM) will be considered appropriate remedial actions;
- For condition B where the pavement is assumed to be only cracked, a seal, or a Bitumen Rubber (SAM) seal, or even a thin overlay will be considered appropriate remedial actions;
- For condition C where the pavement is assumed to be in a sound condition the options of an ordinary seal or to do nothing will be considered.

The appropriate remedial action to be taken in each case is determined by taking the initial costs into account and by considering the state of the pavement after another 10 years, i.e. 20 years from now. For every possibility, the cost to bring the state of the pavement to the same level as the other possibilities is calculated and taken as the salvage value costs.

With these costs know, it is now possible to determine the PWOC of each appropriate remedial action that will be considered after 10 years, depending on the state of the pavement. The available option with the least PWOC for each different pavement condition is taken as the most appropriate for that case (The calculations of the PWOC for each option in this example is given later).

The decision tree with the various outcomes and probability of occurrance and PWOC of the different appropriate actions for this example, are shown in Figure 28. This data with current unit costs and salvage values are summarized in Table 4.

- Condition A

The possible outcome after 10 years of a cracked and spalling pavement, is now discussed in detail :

In this case two remedial actions are considered appropriate i.e. a thin overlay (option A1) or a SAMI layer (option A2). In each case the condition of the pavement after a further 10 years is analysed. The final condition of the road surface after 20 years for the purpose of assessing salvage values must now be considered.

Several final conditions may result, for example :

- cracked and deformed
- only cracked;
- only deformed;
- sound.

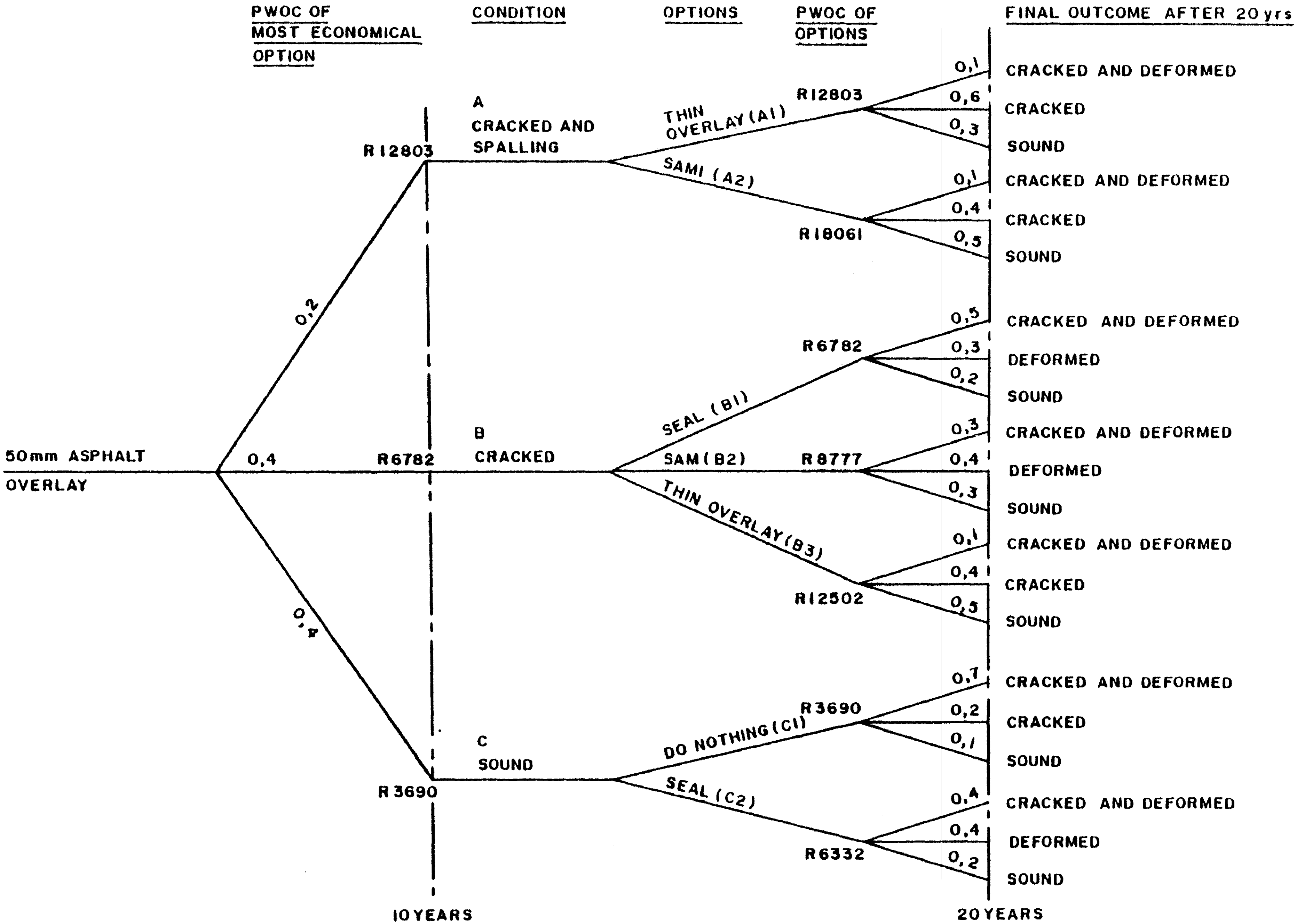


FIGURE 28

DECISION TREE GIVING POSSIBLE OUTCOMES OF A REHABILITATION OPTION

- 103 -

TABLE 4 - Probability of occurrance, various outcomes and relative costs per kilometer of a rehabilitation option

Initial act	Condition After 10 years	Probability of occurance	Second act (options)		Condition after 20 years	Probability of occurance	Salvage value costs (R/km)
50 mm Asphalt (R50 000)	Cracked with Spalling (A)	0,2	(A1)	Thin overlay (25 mm) (R25 000)	Cracked & Deformed	0,1	R15 000
					Cracked	0,6	R 7 000
					Sound	0,3	-
			(A2)	SAMI (R37 000)	Cracked & Deformed	0,1	R15 000
					Cracked	0,4	R 7 000
					Sound	0,5	-
	Cracked (B)	0,4	(B1)	Seal (R7 000)	Cracked & Deformed	0,5	R20 000
					Deformed	0,3	R15 000
					Sound	0,2	R10 000
			(B2)	SAM (R12 000)	Cracked & Deformed	0,3	R20 000
					Deformed	0,4	R15 000
					Sound	0,3	R10 000
			(B3)	Thin overlay (R25 000)	Cracked & Deformed	0,1	R15 000
					Cracked	0,4	R 7 000
					Sound	0,5	-
	Sound (C)	0,4	(C1)	Do Nothing (R 0)	Cracked & Deformed	0,7	R20 000
					Deformed	0,2	R12 000
					Sound	0,1	R 8 000
			(C2)	Seal (R 7 000)	Cracked & Deformed	0,4	R20 000
					Deformed	0,4	R12 000
					Sound	0,2	R 8 000

For condition A1 the final conditions of :

- cracked and deformed;
- only cracked; and
- sound

have been considered as possible and their probability of occurance for each option is now assessed :

(Note that for condition C the final condition of "only cracked" is not considered but "only deformed" is considered in its stead.)

- Final outcome : cracked and deformed

Since rehabilitation design is firstly based on minimizing deformation, the initial rehabilitation design of 50 mm asphalt should keep deformation within acceptable limits for most of the design period of 20 years. As both of the options (A1 + A2) under consideration (Thin overlay a SAM1) add a further structural layer to the pavement, the possibility of the pavement being cracked and deformed after 20 years, will be small (for both options). In both cases subjectively considered here to be 10 %.

- Final outcome : cracked only :

In this condition A case, the pavement is badly cracked and spalling after 10 years and with the addition of a thin overlay (option A1) or SAMI layer (option A2), it is reasonable to expect with a high probability, that the pavement will also be cracked after a further 10 year period. However, a SAMI layer is designed to have greater resistance to cracking, and the probability of this option (A2) being cracked should be smaller than that for an ordinary overlay (A1).

Therefore;

- probability of the option with thin overlay (option A1) being cracked after 10 years is subjectively considered to be 60 %;
- probability of the option with the SAMI (option A2) being cracked after 10 years is subjectively considered to be 40 %.

- Final outcome : sound

The analysis of this possible outcome leaves the following probabilities in each case after a further 10 years or total 20 years :

- Thin overlay - 30 %
- SAMI - 50 %

In the assessment of the PWOC the salvage value is based on the final outcome. The salvage value of the best condition ("Sound") is considered as prime and the costs of bringing the end values of the other final outcomes to this condition, must now be determined.

In this case, to estimate the salvage value costs for Condition A, the sound conditions of both the SAMI option and Thin overlay option are considered to be equal. The costs to upgrade the distressed conditions (namely cracked only or cracked and deformed) to the same condition as the sound condition, is now determined. The following is assumed :

- to upgrade a cracked and deformed condition to a sound condition will cost R15 000/km;
- to upgrade a cracked condition to a sound condition will cost R7 000/km.

The PWOC of the options when the pavement is in a cracked condition with spalling after 10 years, are determined as follows :

Option A1 : Thin overlay

PWOC = Initial costs discounted for 10 years + (Σ Probability of occurance x Salvage value-costs discounted for 20 years)

$$= R25\ 000\ (1{,}08)^{-10} + \left|0{,}1\ (R15\ 000)(1{,}08)^{-20} + 0{,}6\ (R7\ 00)(1{,}08)^{-20} + 0{,}3(0)\ (1{,}08)^{-20}\right|$$

$$= R11\ 580 + \left|R322 + R901 + R0\right|$$

$$= R11\ 580 + R1\ 223$$

$$= R12\ 803$$

Option A2 : SAMI layer

PWOC = Initial costs discounted for 10 years + (Σ Probability of occurance x Salvage value costs discounted for 20 years)

$$= R37\ 000\ (1{,}08)^{-10} + \left|0{,}1\ (R15\ 000)(1{,}08)^{-20} + 0{,}4\ (R7\ 000)(1{,}08)^{-20} + 0{,}5\ (0)(1{,}08)^{-20}\right|$$

$$= R17\ 138 + \left|R322 + R601 + R0\right|$$

$$= R17\ 138 + R923$$

$$= R18\ 061$$

Therefore, the thin overlay is the most economical alternative if the pavement is cracked with spalling (Condition A) after a period of 10 years. As shown in Figure 28, the PWOC of the thin overlay, i.e. R12 803, is now used for the calculation of the PWOC of the initial design of 50 mm of asphalt overlay.

The same procedure is followed for the other conditions (Condition B and Condition C) which are possible after a 10 year period. The final outcomes with their probability of occurance and salvage value costs are shown in Table 4 and are used for the economical analysis.

- Condition B

Pavement in a only cracked condition.

Option B1 : Seal

PWOC = Initial costs discounted for 10 years + (Σ Probability of occurance x Salvage value costs discounted for 20 years)

$= R7\ 000\ (1{,}08)^{-10} + |0{,}5\ (R20\ 000)(1{,}08)^{-20}$
$+ 0{,}3\ (R15\ 000)\ (1{,}08)^{-20} + 0{,}2\ (R10\ 000)(1{,}08)^{-20}|$
$= R3\ 242 + |R2\ 146 + R965 + R429|$
$= R3\ 242 + R3\ 540$
$= R6\ 782$

Option B2 : SAM

PWOC = Initial costs discounted for 10 years + (Σ Probability of occurance x Salvage value costs discounted for 20 years)

$= R12\ 000\ (1{,}08)^{-10} + |0{,}3\ (R20\ 000)(1{,}08)^{-20}$
$+ 0{,}4\ (R15\ 000)(1{,}08)^{-20} + 0{,}3\ (R10\ 000)(1{,}08)^{-20}|$
$= R5\ 558 + |R1\ 287 + R1\ 287 + R644|$
$= R5\ 558 + R3\ 218$
$= R8\ 776$

Option B3 : Thin overlay

PWOC = Initial costs discounted for 10 years + (Σ Probability of occurance x Salvage value cost, discounted for 20 years)

$= R25\ 000\ (1{,}08)^{-10} + |0{,}1\ (R15\ 000)(1{,}08)^{-20}$
$+ 0{,}4\ (R7\ 000)(1{,}08)^{-20} + 0{,}5\ (0)(1{,}08)^{-20}|$
$= R11\ 580 + |R322 + R601 + R0|$
$= R11\ 580 + R922$
$= R12\ 502$

Therefore, the option of a seal (B1) is the most economical alternative if the pavement is in a cracked constitution only (Condition B) after 10 years. As shown in Figure 28, the PWOC of the seal, i.e. R6 782, is now used for the calculation of the PWOC of the initial design of 50 mm of asphalt overlay.

- Condition C

Pavement in a sound condition.

Option C1 : Do Nothing

PWOC = Initial costs discounted for 10 years + (Σ Probability of occurance x Salvage value costs discounted for 20 years)

$$= R0\ (1{,}08)^{-10} + \left|0{,}7\ (R20\ 000)(1{,}08)^{-20} + 0{,}2\ (R12\ 000)(1{,}08)^{-20} + 0{,}1\ (R8\ 000)(1{,}08)^{-20}\right|$$

$$= 0 + \left|R3\ 004 + R515 + R171\right|$$

$$= 0 + R3\ 690$$

$$= R3\ 690$$

Option C2 : Seal

PWOC = Initial costs discounted for 10 years + (Σ Probability of occurance x Salvage value costs discounted for 20 years)

$$= R7\ 000\ (1{,}08)^{-10} + \left|0{,}4\ (R20\ 000)(1{,}08)^{-20} + 0{,}4\ (R12\ 000)(1{,}08)^{-20} + 0{,}2\ (R8\ 000)(1{,}08)^{-20}\right|$$

$$= R3\ 242 + \left|R1\ 716 + R1\ 030 + R343\right|$$

$$= R3\ 242 + R2\ 089$$

$$= R6\ 331$$

The option of doing nothing (C1) is the most economical alternative if the pavement is found to be in a sound condition after 10 years. As shown in Figure 28, the PWOC of the do nothing option, i.e. R3 690, is used in calculating the PWOC if the 50 mm asphalt overlay.

Therefore, the PWOC of a 50 mm of asphalt overlay in this case is :

PWOC = Initial cost + (Σ Probability of occurance of action x Minimum PWOC of action)

= Initial cost + |(Probability of pavement cracked and spalling x PWOC of Thin overlay) + (Probability of pavement cracked x PWOC of seal) + (Probability of pavement in sound condition x PWOC of do nothing)|

= R50 000 + |0,2 (R12 803) + 0,4 (R6 782) + 0,4 (R3 690)|

= R50 000 + R6 749

= R56 749

≅ R56 700

In a full rehabilitation study each appropriate rehabilitation option should be analysed in a similar manner as shown in the example above before the PWOC of the options are compared in order to select the best applicable rehabilitation design.

SECTION 7

CASE STUDY : IMPLEMENTATION OF THE REHABILITATION DESIGN PROCEDURE

7.1 REPORT 1

Pavement rehabilitation study :
Initial assessment of route P6/1.

7.2 REPORT 2

Pavement rehabilitation study :
Detailed assessment, Rehabilitation design and Economic Analysis of Section 2 of route P6/1.

7.1 REPORT 1

Pavement rehabilitation study : Initial assessment P6/1

7.1.1 INTRODUCTION

7.1.2 OBJECTS AND SCOPE

7.1.3 GATHERING OF INFORMATION

7.1.3.1 Preliminary investigation

(a) Pavement structure
(b) Traffic data

7.1.3.2 Pavement surveillance

(a) Riding quality survey
(b) Deflection survey
(c) Rut depth survey

7.1.3.3 Detailed visual inspection

7.1.4 EVALUATION

7.1.4.1 Section 1A : km 20,0 to km 28,0
7.1.4.2 Section 1B : km 28,0 to km 29,0
7.1.4.3 Section 1C : km 29,0 to km 31,4
7.1.4.4 Section 2 : km 31,4 to km 34,7
7.1.4.5 Section 3 : km 34,7 to km 35,7
7.1.4.6 Section 4 : km 35,7 to km 37,6
7.1.4.7 Section 5 : km 37,6 to km 45,2
7.1.4.8 Section 6 : km 45,2 to km 46,2
7.1.4.9 Section 7A : km 46,2 to km 50,0
7.1.4.10 Section 7B : km 50,0 to km 51,2
7.1.4.11 Section 8 : km 51,2 to km 52,2
7.1.4.12 Section 9 : km 52,2 to km 53,2

7.1.5 CONCLUSIONS AND RECOMMENDATIONS

7.1.6 CONDITION SHEETS

7.1.1 INTRODUCTION

The identification of Bronkhorstspruit as a development area, as part of the government's decentralization policy, necessitated an investigation into the adequacy of the roads giving access to this area. The Transvaal Roads Department undertook this investigation and identified route P6/1, among others, as being in need of rehabilitation. The location of this route is given in Figure 29.

The expected increase in traffic from the Witwatersrand put high priority on a rehabilitation study of the P6/1 route, which is the main road link with this densely populated region. To ensure the full utilization of the existing road facility within a climate of financial restraint, a systematic investigation was required to identify the most suitable rehabilitation measures. The procedure contained in sections 1 to 6 adheres to these requirements, and with the co-operation of the Transvaal Roads Department (TRD) through the NITRR, this road from km 20 to 53 was used to test the practical applicability of the procedures.

7.1.2 OBJECTS AND SCOPE

The main objective of the initial assessment, as described in the section 3, is to identify sections of the road which:

- require no action within the next five years;
- have a surfacing-only problem;
- have localized problems;
- require further testing (e.g. sections with possible structural inadequacies or sections that may require attention within the next five-year period.)

The initial assessment is thus a limited investigation which is used to ensure that subsequent detailed assessments are undertaken only on appropriate sections of the road. To achieve this a systematic procedure of information gathering and evaluation is required.

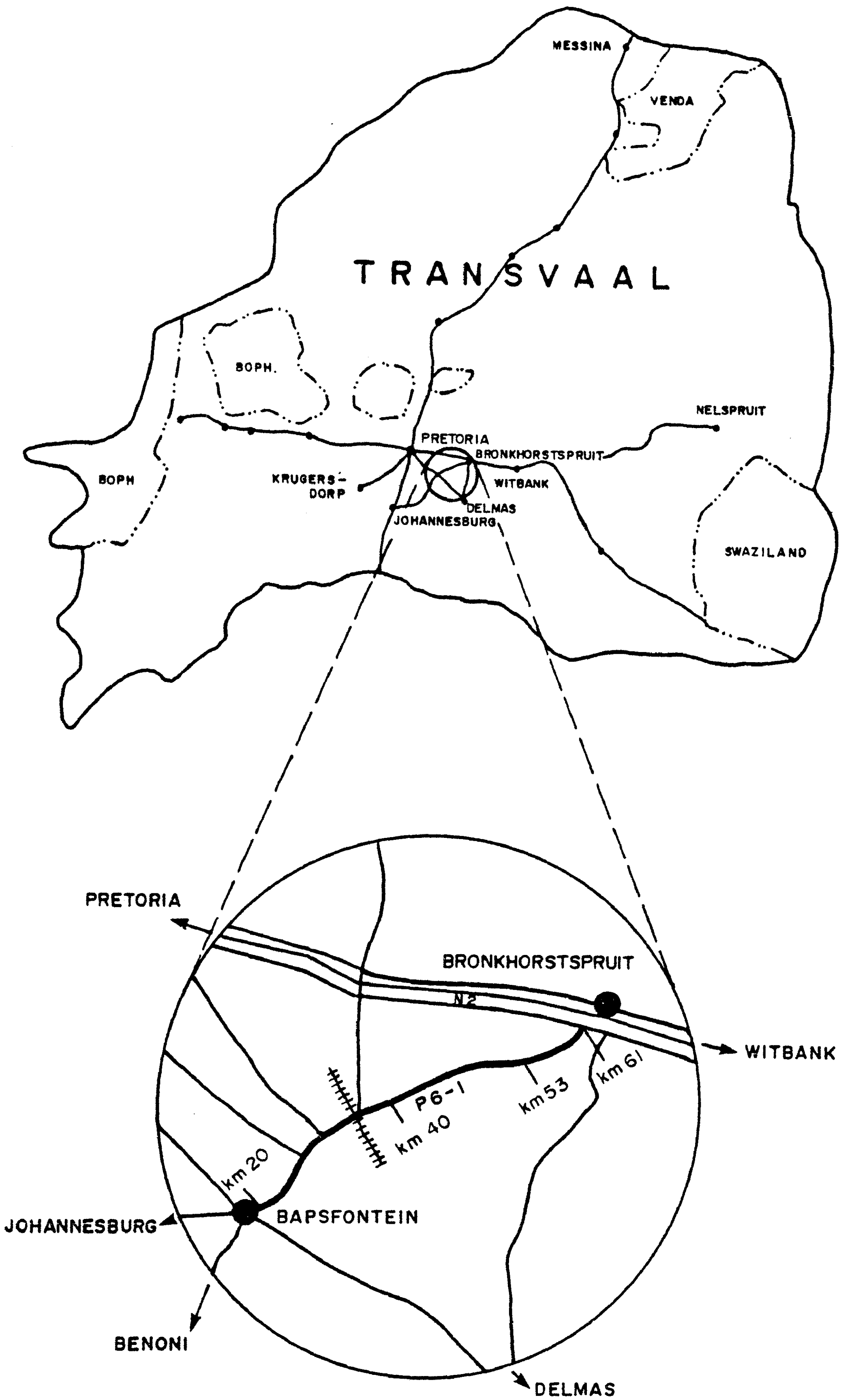


FIGURE 29

LOCATION PLAN OF ROUTE P6/1

7.1.3 GATHERING OF INFORMATION

7.1.3.1 Preliminary Investigation

Research into existing records yielded a limited amount of information as no routine testing or surveying had previously been undertaken on this road. However, the basic pavement construction and traffic data were available.

(a) Pavement structure

Road P6/1 was constructed in 1952/53 with a width of 6,1m (20ft). The pavement consists of a light structure containing:

- several seals which have been placed on top of the pavement during the last 20 years, resulting in a surfacing thickness of 45-50 mm;
- 150 mm natural gravel base (G5);
- 150 mm natural gravel or soil gravel subbase (G6-G8).

This structure was built on the in-situ material, a clayey silt, which varied considerably in quality, as can be concluded from the deflection measurements shown in Figure 31.

Part of the road, from km 35,3 to km 37,6, was rebuilt in 1972 with a width of 7,3 m (24 ft), and consists of a double seal and two 150 mm layers of stabilized natural gravel on the in-situ subgrade material. In 1982 a road-over-rail bridge was constructed at km 35,2, resulting in the reconstruction of a section of the road from km 34,7-km 35,7.

(b) Traffic data

Data on the total number of vehicles per 24 hr are available, covering the period from 1965. An analysis of this data is given in Table 5. Figures for 1954-1964 were extrapolated for the 1965-1968 period.

TABLE 5 - Traffic data for route P6/1

Year	Total E80s
1973 - 1982	$1,4 \times 10^6$
1969 - 1972	
1965 - 1968	$0,9 \times 10^6$
1954 - 1964	
	$2,3 \times 10^6$

P6/1 carries most of its traffic on the southbound lane (Bronkhorstspruit-Bapsfontein) and a 60-40 percent division of the total number of E80s for the south and northbound lanes has been estimated. Total past accumulated equivalent traffic per lane thus amounted to:

southbound lane - $1,4 \times 10^6$ E80
northbound lane - $0,9 \times 10^6$ E80

This traffic is applicable to the whole road except the rebuilt section from km 35,3 to km 37,6. For this section the following figures are used:

south bound lane - $8,4 \times 10^5$ E80
north bound lane - $5,6 \times 10^5$ E80

From the available data, the total expected equivalent traffic over a rehabilitation design period of 20 years is calculated to be $4,1 \times 10^6$ E80. P6/1 is accordingly classified as a Category B* road (major rural road) and future rehabilitation should provide for E_3 (3,0-12,0 - $\times 10^6$E80) traffic.

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* See Appendix 2, Table 16

7.1.3.2 Pavement surveillance

To make rational decisions quantitative information on the performance of the pavement is needed. In liaison with the TRD it was decided to undertake riding quality, deflection and rut depth surveys on this route. The results of the processed data from these surveys are contained in the condition sheets in sub-section 7.1.6, Figures 33 + 34.

(a) Riding quality survey

Riding quality surveys were undertaken on both the southbound and the northbound lanes of the P6/1 route. The results were in line with what was expected, owing to the fact that the southbound lane carries most of the traffic. Figure 30 shows that the riding quality of the southbound lane is generally much lower than that of the northbound lane. This difference is represented by the black areas in this Figure. The warning and sound criteria for a category B road (2,5 and 2,0 PSI respectively) are also shown on this figure.

(b) Deflection survey

The deflection survey was undertaken along the southbound lane of the P6/1 route. This survey showed up definite trends along the route explaining the occurrence of the distress in various sections. By means of this survey uniform sections could easily be identified. Figure 31 shows the graphical output of the deflection survey with the demarcated uniform sections. The boundaries of these sections were adjusted in the final analysis of the initial assessment, as shown on the condition sheets in sub-section 7.1.6, Figures 33 + 34.

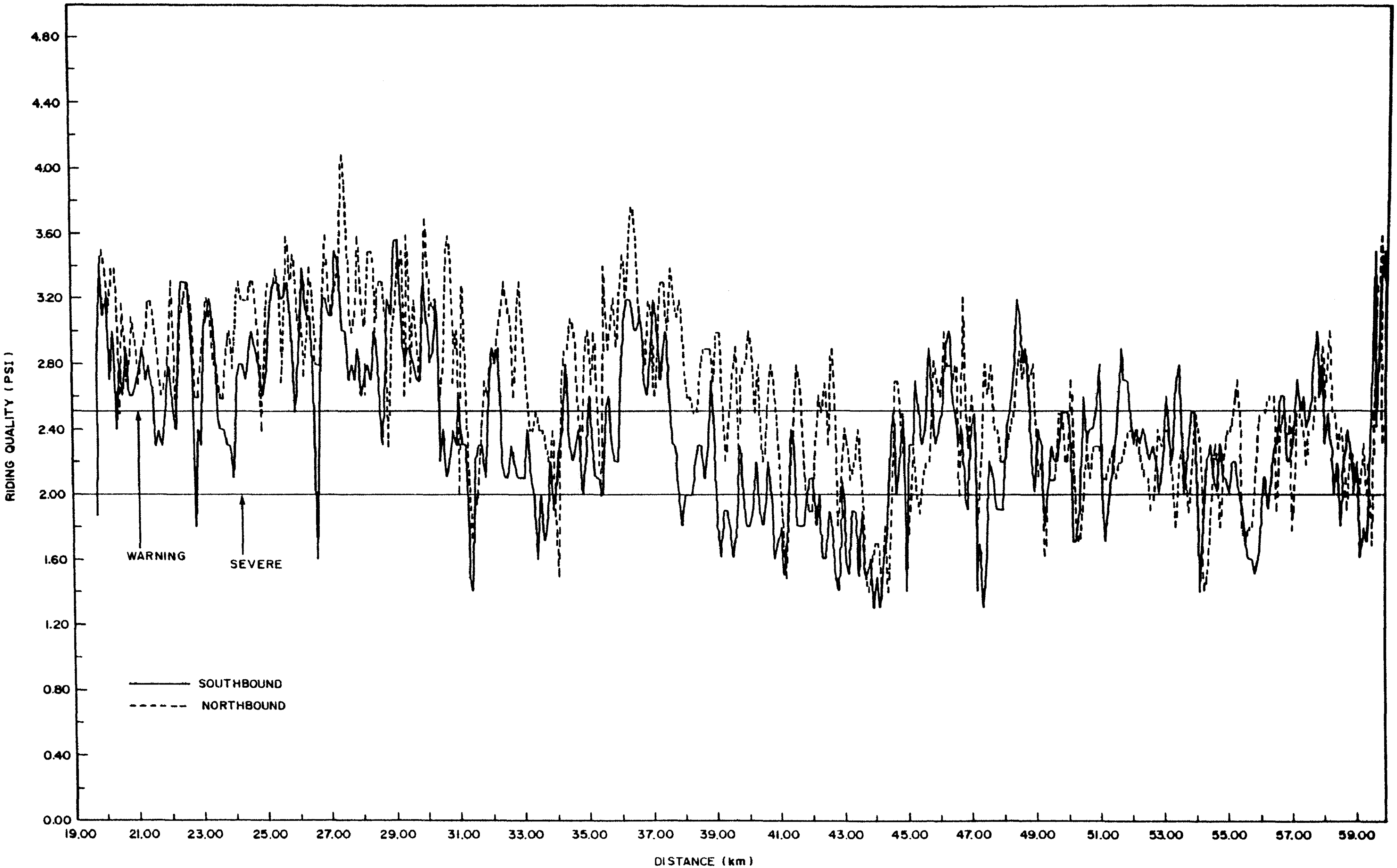


FIGURE 30

GRAPHICAL OUTPUT OF THE RIDING QUALITY OF P 6/1

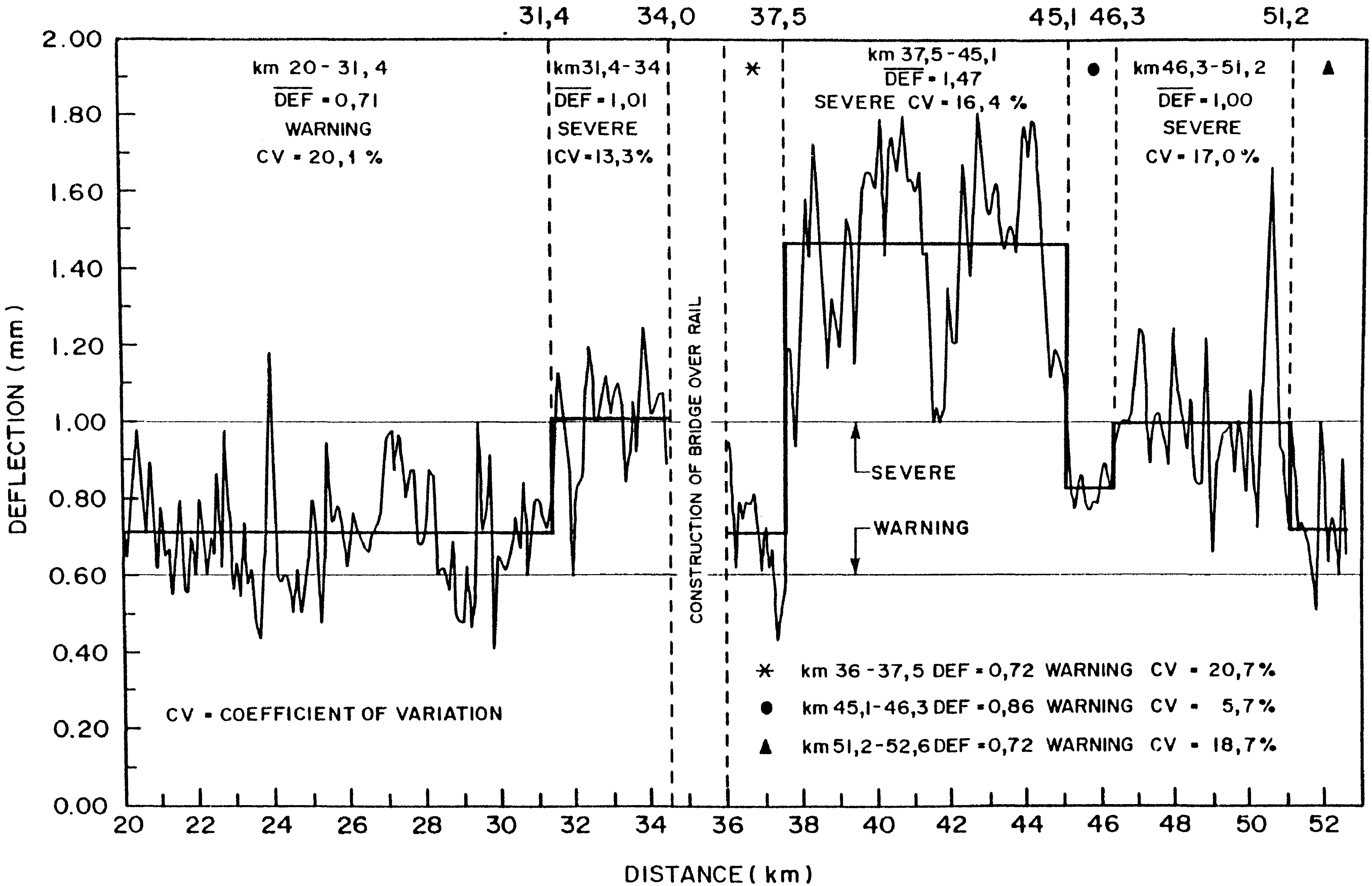


FIGURE 31

P6/1: DEFLECTION SURVEY WITH DEMARCATED UNIFORM SECTIONS

(c) Rut depth survey

The rut depth was measured by a locally developed rut depth meter attached to the Lacroix deflectograph. This instrument measures the rut depth in the outside wheel track. However, for this pavement, the recorded rut depth does not reflect the worst condition as much deeper ruts were observed near the centre of the road. This can be explained by the very narrow lane width (over most of the road) which results in the centre of the road being used in particular by heavy traffic from both directions. As accurate measurements are not available, this survey is not shown.

7.1.3.3 Detailed visual inspection

The detailed visual inspection identified long sections of road with severe forms of distress, especially cracking. Most of the cracking according to the criteria in Appendix 2 was classified as severe. However, some of the sections (e.g. km 20-28 and km 29-31,4) exhibited cracking typical of a surfacing-only problem, whereas others exhibited severe crocodile cracking. Care should thus be taken in the interpretation of the processed data as contained in the condition sheets in sub-section 7.1.6.

This assessment, because of the very narrow lane widths, required the separate recording of "edge breaking". The visual tracks on the gravel shoulders and the edge breaking indicated that trucks often leave the surfaced carriageway or stay very close to its edge to avoid oncoming traffic or during passing manoeuvres. Dangerous traffic conditions thus occur which may prove to be an important factor in the final decision about the best rehabilitation option.

This is a very important condition which is not measured by an instrument or routine inspection concentrating on pavement distress in general. From a safety point of view the expected increase in traffic volume could warrant widening of the pavement. Although this is an important fact which should influence a final decision on the particular road, this report now only concentrates on the structural requirements of the pavement.

The general impression gained from the visual inspection was that the pavement was urgently in need of rehabilitation.

7.1.4 EVALUATION

Table 6 contains a summary of the processed data of the various surveys undertaken on the P6/1 route. This data is shown graphically on the Condition Sheets in sub-section 7.1.6. A statistical analysis of the pavement mechanical surveillance data as regards riding quality, deflection and rut depth is also given in Table 7.

In broad outline, distress manifestation on the P6/1 can be divided into either:

- surfacing problems with typical temperature-associated transverse cracking as the main manifestation of distress (km 20-31,4), or
- structural problems involving subgrade deformation and/or shear distress of the pavement layers (km 31,4-53,2).

Various distinct sections can be identified in each of the above. In order to evaluate and identify appropriate uniform sections, the condition sheets in sub-section 7.1.6 and the data contained in Tables 6 and 7 should be referred to constantly.

TABLE 6 - Summary of the Processed data of the condition of the P6/1 route as contained in the condition sheets in Section 7.1.6

Uniform section			Visually assessed condition in % of 100 m lengths of a section								Measured condition in % of 100 m lengths of a section					
No	Distance (km)	Length (km)	Cracking		Disintegration		Edge breaking		Deformation		Riding quality		Deflection		Rutting	
			W*	S**	W	S	W	S	W	S	W	S	W	S	W	S
1A	20,0-28,0	8,0	16	65	5	15	14	58	1	31	21	3	81	1	23	1
1B	28,0-29,0	1,0	40	40	-	10	-	60	10	30	20	-	70	-	30	-
1C	29,0-31,4	2,4	17	46	4	-	17	54	-	4	33	8	79	4	4	-
2	31,4-34,7	3,3	21	39	9	15	3	73	6	21	52	33	30	64	42	-
3	34,7-35,7	1,0	BRIDGE UNDER CONSTRUCTION ----------------------------								BRIDGE UNDER CONSTRUCTION ----------					
4	35,7-37,6	1,9	-	-	-	-	-	16	-	-	16	-	84	-	5	-
5	37,6-45,2	7,6	17	63	8	24	4	74	5	29	20	79	3	97	74	4
6	45,2-46,2	1,0	10	20	-	-	20	60	-	-	70	20	100	-	30	-
7A	46,2-50,0	3,8	18	37	3	3	8	76	-	11	55	32	47	53	26	-
7B	50,0-51,2	1,2	-	-	-	-	-	92	33	-	42	58	50	50	17	-
8	51,2-52,2	1,0	10	-	-	-	10	70	-	-	90	10	90	10	-	-
9	52,2-53,2	1,0	10	80	-	-	-	100	10	40	70	30	50	-	-	-

* W = Warning Condition
** S = Severe Condition

TABLE 7 - Statistically processed data obtained from the Instrument Surveys

Uniform section			Riding quality (PSI)				Deflection (mm)				Rut depth (mm)			
No	Chainage (km)	Length (mm)	Mean (PSI)	Standard Deviation	W** %(1)	S** %(2)	Mean* (mm)	Standard* deviation	W %(3)	S %(4)	Mean* (mm)	Standard* Deviation	W %(5)	S %(6)
1A	20,0-28,0	8,0	2,8	0,36	17,8	1,3	0,72	0,144	77,0	2,6	7,2	3,2	18,9	-
1B	28,0-29,0	1,0	2,8	0,26	13,0	0,1	0,652	0,136	64,2	0,5	7,5	4,5	28,5	0,3
1C	29,0-31,4	2,4	2,7	0,49	27,0	7,8	0,693	0,141	73,1	1,5	5,1	1,8	0,4	-
2	31,4-34,7	3,3	2,2	0,34	53,7	27,1	1,009	0,136	47,1	47,2	8,5	3,2	31,2	0,1
3	34,7-35,7	1,0	-	-	-	-	-	-	-	-	-	-	-	-
4	35,7-37,6	1,9	2,8	0,33	15,3	0,6	0,708	0,148	95,3	2,4	6,1	1,7	1,4	-
5	37,6-45,2	7,6	1,9	0,31	36,0	61,0	1,453	0,239	2,9	97,1	12,4	3,6	70,5	1,7
6	45,2-46,2	1,0	2,6	0,22	40,7	0,2	0,822	0,047	100,0	-	8,3	1,4	11,5	-
7A	46,2-50,0	3,8	2,3	0,43	40,4	25,8	0,984	0,127	55,0	44,8	8,4	2,5	26,1	-
7B	50,0-51,2	1,2	2,2	0,38	48,1	30,2	1,053	0,269	37,4	57,9	7,3	1,8	12,5	-
8	51,2-52,2	1,0	2,4	0,30	50,6	7,4	0,739	0,153	77,5	4,4	4,7	1,1	-	-
9	52,2-53,2	1,0	2,3	0,16	86,8	5,4	0,724	0,120	83,8	1,1	6,0	1,9	1,6	-

* Mean and standard deviation of the 95 percentile values over a section Normal Distribution assumed
** W = Warning Condition; S = Severe Condition

(1) $P(2<RQ\leq 2{,}5)$ - Probability of riding quality being at warning level
(2) $P(RQ\leq 2)$ - Probability of riding quality being at severe level
(3) $P(0{,}6\leq D\leq 1{,}0)$ - Probability of deflection being at warning level except for section 4 where $P(0{,}4\leq D\leq 1{,}0)$
(4) $P(D\geq 1{,}0)$ - Probability of deflection being at severe level
(5) $P(10\leq RD\leq 20)$ - Probability of rut depth being at warning level
(6) $P(RD\geq 20)$ - Probability of rut depth being at severe level

The following uniform sections were identified along the route:*

Section 1A from km 20,0 to km 28,0
Section 1B from km 28,0 to km 29,0
Section 1C from km 29,0 to km 31,4
Section 2 from km 31,4 to km 34,7
Section 3 from km 34,7 to km 35,7
Section 4 from km 35,7 to km 37,6
Section 5 from km 37,6 to km 45,2
Section 6 from km 45,2 to km 46,2
Section 7A from km 46,2 to km 50,0
Section 7B from km 50,0 to km 51,2
Section 8 from km 51,2 to km 52,2
Section 9 from km 52,2 to km 53,2

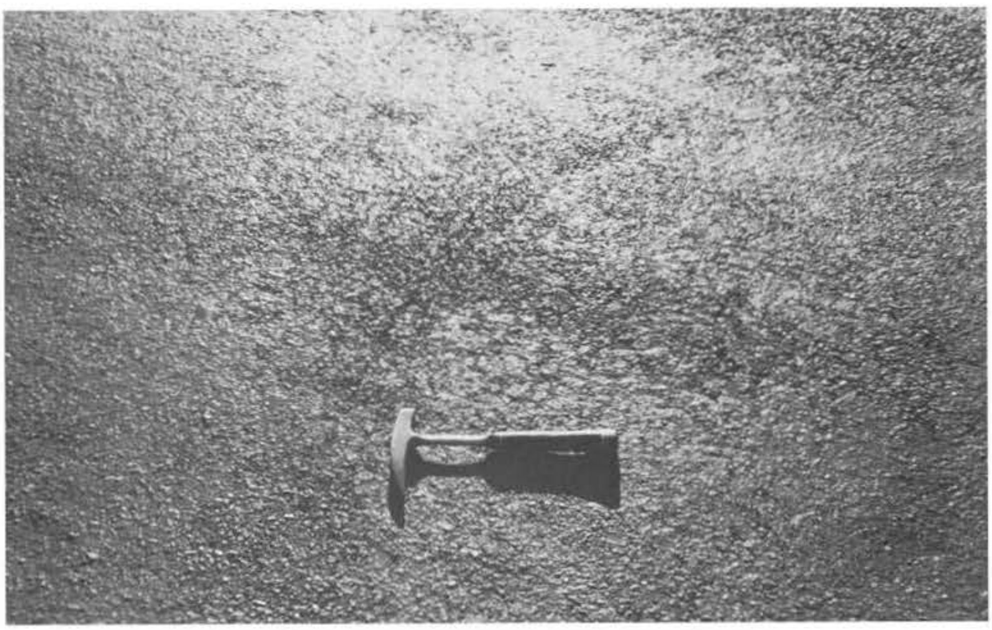
Where a subdivision of sections was made, the sections were very similar with regard to the measured condition, but differed substantially visually. Since edge breaking is considered a problem over the whole length, except for Section 4 (reconstructed section), it is not mentioned in the detailed evaluations of the sections which follow.

7.1.4.1 Section 1A: km 20,0 - km 28,0

This section mainly exhibits severe forms of typical temperature- associated transverse cracking across both lanes, except in the wheel paths where traffic has sealed these cracks. A surfacing problem therefore exists.

* Some sections were redivided into subsections even although the instrument measurements were similar over the whole length, because differences were detected during the detailed visual inspection.

However, in the south-bound lane longitudinal cracking is evident in some places. This may be indicative of structural problems - a possibility that cannot be disregarded, since the mean deflection value for this section is at warning level. Further investigation in this regard is considered necessary. As this lane is carrying most of the traffic, it can be assumed that with the accumulation of further traffic similar crack patterns will develop in the north-bound lane.



PHOTOGRAPH 1: Km 20,8 - Typical temperature-associated transverse cracking evident on Section 1A with an average spacing of about 3 m

PHOTOGRAPH 2: Km 21 - Typical transverse crack shown with trafficking closing the crack in the wheelpaths (Section 1A)

A deflection-life analysis with a 50 per cent probability of the pavement lasting the predicted period, gives the following results[29] :

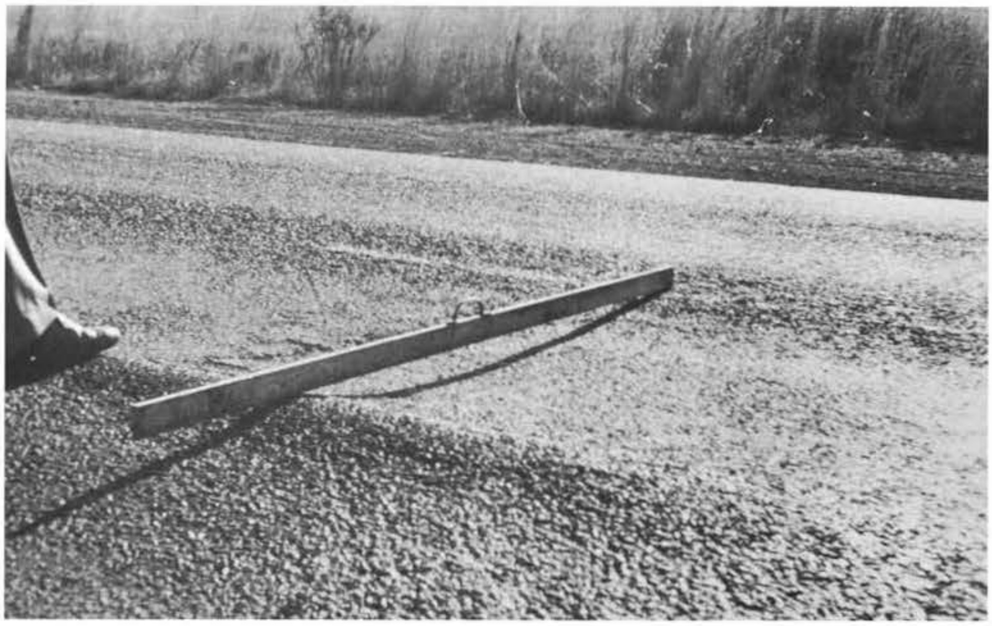
- 10% Annual E80 growth rate - remaining life 5 years
- 12% " " " " - " " 4 years
- 15% " " " " - " " 3 years

This analysis gives an indication of the condition of the pavement with regard to the subgrade conditions. It is therefore very likely that failure of the pavement in this section due to an overstressed subgrade will occur within the next 5 years.

The shear failure in the adjacent section (km 28-29) indicates that the pavement layer strength in some areas is inadequate for the traffic load. The development within the next 5 years of shear failure in this section, due to pavement layer inadequacies , should therefore also be considered.

Because of the above, this section is selected for further testing. It is suggested that a DCP survey be undertaken on this section to determine the properties of the pavement layers.

7.1.4.2 Section 1B: Km 28,0 - km 29,0



PHOTOGRAPH 3: Km 28,7 - Serious distress in the form of pavement layer failure is very evident in this section

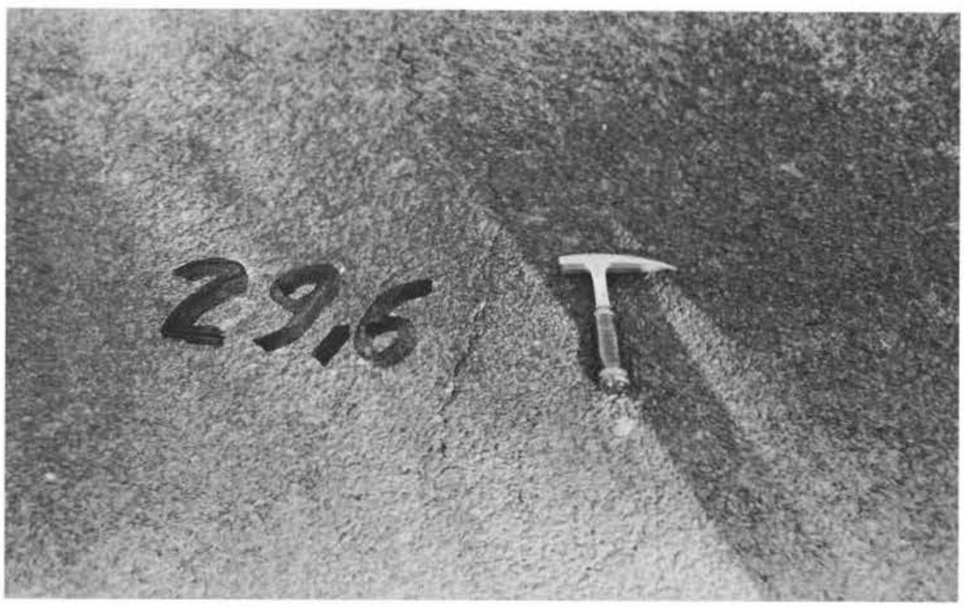
Riding quality, deflection and rut depth measurements, with transverse cracking typical of a surfacing problem, are very similar to that of Section 1A.

However, this section differs from Section 1A in that serious shear distress of the pavement layers and crocodile cracking are evident on the southbound lane. This is particularly severe from km 28,5 to km 29,0, where the gradient causes heavy vehicles to move very slowly. This section has been subjected to continuous maintenance as indicated from the number of existing patches. The evidence of severe structural inadequacies confirms the need for further testing on adjacent sections of the road constructed at the same time and using the same materisl.

Although pavement layer shear failure is considered to be the primary cause of distress here, further testing is needed to verify this. Deflections in this section are at warning level, and with the reduced load-spreading abilities of the distressed pavement layers, subgrade deformation is very likely to occur.

This section is selected for further testing and both the pavement layer and subgrade properties should be investigated. A DCP survey is considered an essential part of any further investigation.

7.1.4.3 Section 1C: Km 29,0 - km 31,4



PHOTOGRAPH 4: Km 29,6 A typical transverse crack in the middle of a lane on Section 1C

This section is very similar to Section 1A. An surfacing problem exists, with the temperature-associated transverse cracking clearly visible between the wheel paths and near the edge of the road. Nevertheless, secondary longitudinal cracking is evident in some places, possibly indicating structural inadequacies.

A deflection-life analysis with a 50 per cent probability of the pavement lasting the predicted period, gives the following results[29] :

10% Annual E80 growth rate - remaining life - 5 years
12% Annual E80 growth rate - remaining life - 4 years
15% Annual E80 growth rate - remaining life - 3 years

However, it is likely that this section will fail before the above mentioned periods because of shear distress of the pavement layers. Because section 1B with similar deflection measurements is already exhibiting shear failure, further testing of this section should be done. A DCP survey is considered necessary to determine pavement layer properties.

This section is therefore selected as requiring further testing because of the likelihood of serious distress appearing within 5 years.

7.1.4.4 Section 2: Km 31,4 - km 34,7

PHOTOGRAPH 5: Km 34,2 - Secondary crocodile cracking develo-ping around a transverse crack

PHOTOGRAPH 6: Km 33,60 - Wide rutting with crocodile cracking indicating subgrade deformation

PHOTOGRAPH 7: Km 32,5 - "heaving" of the surface and base material between the wheelpaths indicating pavement layer shear distress

The low riding quality and the high deflection values obtained are indicative of the general condition of the section. Transverse cracking associated with a surfacing problem and longitudinal and crocodile cracking associated with both base and/or subbase shear distress and subgrade deformation were evident.

This section is seriously distressed and requires urgent rehabilitation, and is therefore selected for further testing. Because of the manifestation of combinations of distress modes, any further investigation should examine both the pavement layers and the subgrade conditions.

As widening of the riding surface of this road should receive serious consideration, partial or total reconstruction of this section is possibly warranted.

7.1.4.5 <u>Section 3: Km 34,7 - 35,7</u>

This section has been excluded from the analysis as at the time of the investigations it was being reconstructed to fit in with a road over rail bridge.

7.1.4.6 <u>Section 4: Km 35,7 - km 37,6</u>

This section was rebuilt in 1972 to a width of 7,4 m and contains two 150 mm stabilized layers with a double seal. No visible distress is evident, but as the measured deflections are high for a pavement with stabilized layers, it must be assumed that the layers are either very lightly stabilized or the layers are in a post-cracked phase due to traffic or environmental effects.

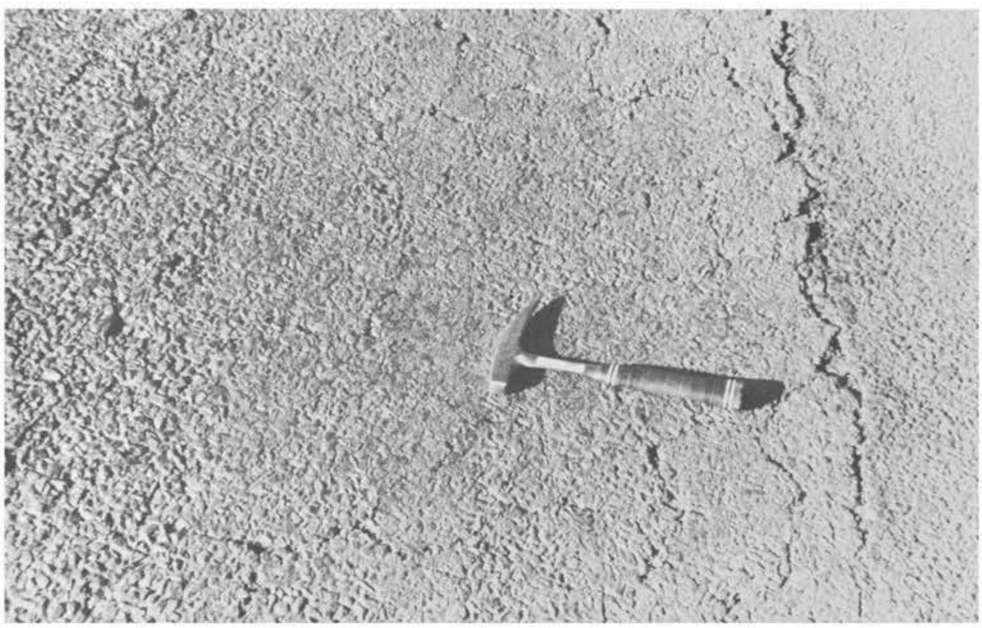
Taking the above into account, a deflection-life analysis with a 50 per cent probability of the pavement lasting the predicted period, gives the following results[29] :

10% Annual E80 growth rate: remaining life - 6 years
12% " " " " " " - 5 years
15% " " " " " " - 4 years.

However, as no visual distress is evident at this stage, it is highly unlikely that this section will fail within the next 5 years, and this section is therefore considered to require no action.

7.1.4.7 Section 5: Km 37,6 - km 45,2

PHOTOGRAPH 8: Km 39,6 - Extensive patching with severe distress evident



PHOTOGRAPH 9: Km 39,6 - Close-up of the surfacing of Photo 8 - severe crocodile cracking

PHOTOGRAPH 10: Km 40,0 - Extensive patching, rutting, cracking and breaking up of the surface

PHOTOGRAPH 11: Km 44,2 - Severity of the distress in this section is clearly shown by the rutting, patching and cracking patterns

This section is considered to be the worst on the P6/1 route, with a mean riding quality of 1,9 PSI and a mean deflection of 1,45 mm. The conditions observed showed that subgrade deformation was the main cause of distress along the whole length of this section.

Because of the dominant effect of the subgrade deformation, distress associated with shear of the pavement layers is not as pronounced, although evident in some places. Further investigations should include a DCP survey to establish the properties of the pavement layers.

However, because of the high deflection values it is believed that a deflection-based rehabilitation design should prove the most suitable in this case.

This whole section is considered to require rehabilitation urgently and is therefore selected for further investigation.

PHOTOGRAPH 12: Localized problem area Km 43,6 - km 44,3 - Patches and undulations clearly visible

PHOT .PH 13: Km 43,6 - Close-up of the surface of the road in Photograph 12, showing severe crocodile cracking

Within Section 5, an area with a localized problem was identified, from km 42,2 to km 44,4. The surface of the road is very low in comparison with the farming land on either side, and from km 43,5 to km 44,4 the road goes through a marshy area. It is obvious from the extensive patching that this section of road constantly requires routine maintenance. Raising the road to at least 1m above the abutting land together with the improvement of drainage facilities should be considered.

7.1.4.8 Section 6: km 45,2 - km 46,2

This section differs considerably from the previous section. Typical temperature-associated transverse cracking, with small areas of longitudinal cracking, is evident. In appearance this section can be compared to Sections 1A and 1C, although it is slightly worse in both riding quality and deflection measurements.

A deflection-life analysis with a 50 per cent probability of the pavement lasting the predicted period, gives the following results[29] :

10%	Annual	E80	growth	rate	- expected	remaining	life	- 4 years
12%	"	"	"	"	"	"	"	- 3 years
15%	"	"	"	"	"	"	"	- 3 years

Although no shear distress of the pavement layers is evident as yet, the possibility of this occurring in the near future cannot be disregarded, because such distress has been found in adjacent sections. A DCP survey as part of a successive detailed assessment is considered to be appropriate.

Because of the likelihood of severe distress within the next 5-year period, this section is selected for further testing.

7.1.4.9 Section 7A: Km 46,2 - km 50,0

PHOTOGRAPH 14: Km 46,8 - Serious subgrade-associated deformation is evident in this section

Transverse and secondary longitudinal cracking, with patches of crocodile cracking, is evident on this section. Although no shear distress in the pavement layer could be identified, deformation was evident along the section, as would be expected from the high deflections measured.

This section is considered to have failed owing to an overstressed subgrade and is therefore selected for further investigation. The possibility of shear distress in the pavement layers should not be disregarded in future investigations, which should include a DCP survey.

Within this section there is a localized problem from km 47,0 to km 47,2. The road is situated next to a large donga which may have had an influence on the condition of the pavement. Further investigation is needed.

7.1.4.10 Section 7B: km 50,0 - km 51,2

The measured condition of this section as given in Table 7 is very similar to that of the previous section. Cracking, is limited however, to three small patches of crocodile cracking. Deformation due to an overstressed subgrade is very evident. The section may only appear to be in a better condition than Section 7A and, although it is not determinable at present, better fatigue properties of the surfacing may be the reason. This can only be established by a successive detailed assessment. The subgrade deformation, the low riding quality and high deflections measured on this section, warrant the selection of this section for further testing.

7.1.4.11 Section 8: km 51,2 - km 52,2

Very little cracking was observed in this section although in places rutting could clearly be seen near the centre line. It is suspected that this section may, like Section 7B, have relatively good surfacing properties.

A deflection-life analysis with a 50 percent probability of the pavement lasting the predicted period, gives the following results[29] :

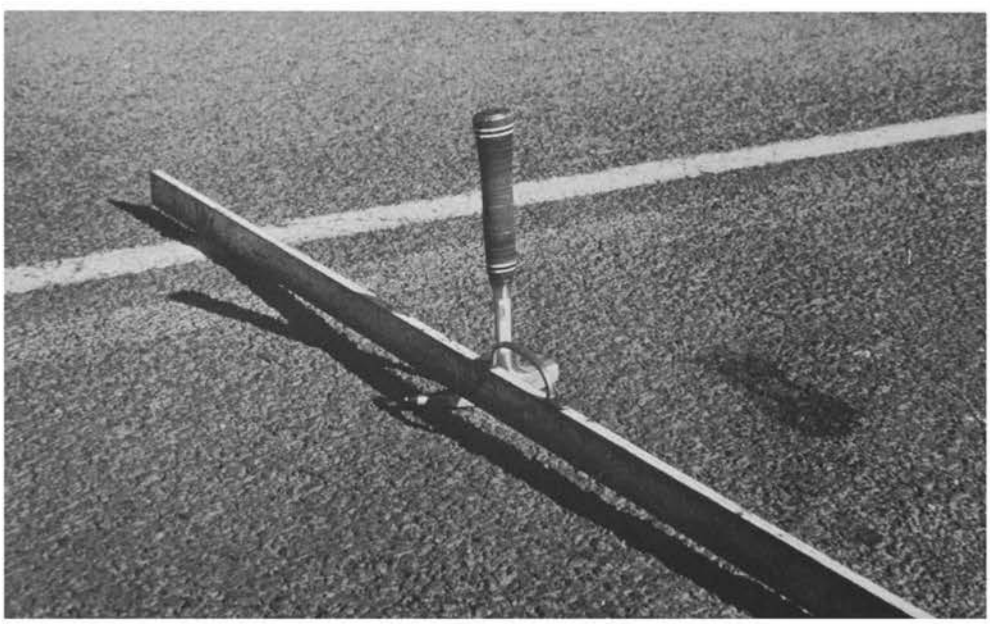
- 10% Annual E80 growth rate - remaining life 5 years
- 12% Annual E80 growth rate - remaining life 4 years
- 15% Annual E80 growth rate - remaining life 3 years

It is therefore possible that severe distress may occur within 5 years.

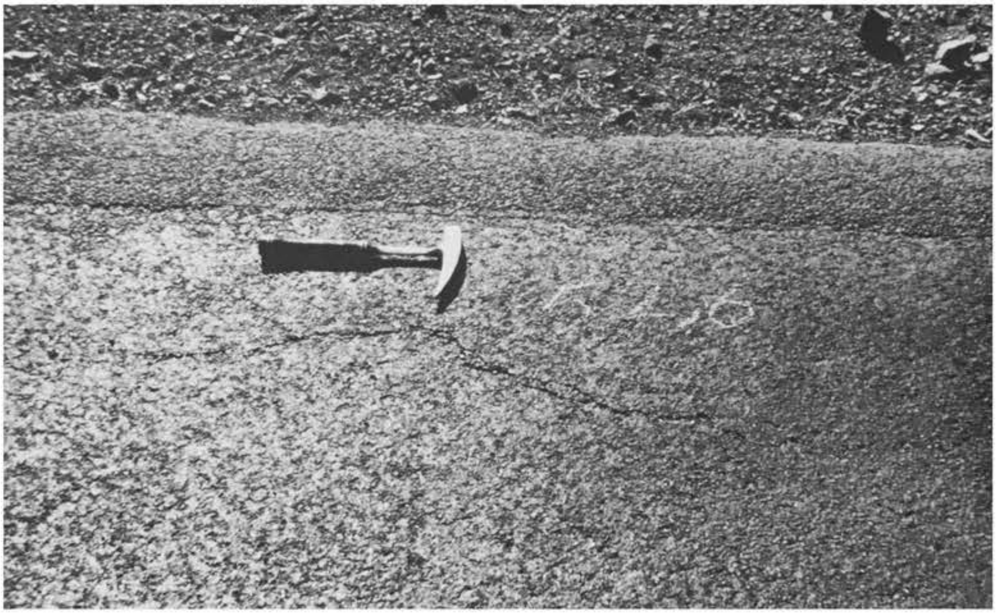
This analysis based on deflection measurement gives an indication of primarily the subgrade conditions and, to determine pavement layer properties, a DCP survey is suggested as part of a successive detailed assessment.

This section is selected for further investigation because of the possibility of serious distress developing within 5 years.

7.1.4.12 <u>Section 9: km 52,2 - km 53,2</u>



PHOTOGRAPH 15: Km 52,6 Rutting as a result of subgrade deformation is very evident in this section



PHOTOGRAPH 16: Km 52,6 - Longitudinal cracking

This section differs substantially from the previous section. Severe forms of transverse and secondary longitudinal and even crocodile cracking are evident. A prominent feature is the very obvious rutting patterns characteristic of subgrade deformation, observed near the centre line (wide deformation bowl with no evidence of heave on sides).

Although no shear distress was observed, the possible occurrence of this cannot be totally disregarded and further investigations should include a DCP survey to establish the properties of the pavement layers. The presence of undulations from km 52,2 to km 52,7 may also be an indication of subgrade problems (active subgrade) and needs investigation.

This section is selected for further testing because of existing structural inadequacies.

7.1.5 CONCLUSIONS AND RECOMMENDATIONS

The investigations undertaken on the P6/1 route confirm that rehabilitation is required. The road carries heavy traffic of which 25 to 30 percent is classified as heavy. From the detailed visual inspection it is clear that the carriageway is too narrow leading to the deterioration and breaking away of the road edge, and to dangerous traffic conditions. This aspect needs serious consideration in any detailed analysis. However, it is not within the scope of this report to reach any decision on the safety aspects of the road; the object being to report on the condition of the pavement.

A general impression of a pavement urgently in need of rehabilitation was obtained. However, the pavement can be divided into uniform sections differing substantially in condition, urgency of rehabilitation and rehabilitation needs. Although most of the sections are identified for further testing, it is likely that further investigations will prove that a "holding action" can be sufficient for some of these sections.

According to the recorded conditions the sections can be classified with regard to the urgency or priority of rehabilitation. The priority rating from 1 to 5, described in section 3, Table 1, is used in Table 8.

TABLE 8 - Priority listing of uniform sections according to the recorded condition of the P6/1 route

Priority	5	4	3	2	1
Uniform section	5;2	1B; 9;7A	6;7B;1A;1C;8	4	3

Rehabilitation should not be postponed for sections with a priority rating of 4 or 5. These sections are in a serious state of distress causing dangerous driving conditons. Sections with a priority rating of 3 have also been selected for further testing, because of the possibility of serious distress developing within the next 5 years. A detailed assessment of these sections may result in the postponement of the rehabilitation of these sections or may show that a "holding action" such as a seal will be sufficient. Sections with a priority rating of 2 require no action, but need re-investigation in 5 years, while sections with a priority rating of 1 will not require structural strengthening during the rehabilitation design period.

A total of 16,7 km of the P6/1, with a priority rating of 4 or 5, thus requires further testing while 13,6 km was selected for further testing with a priority rating of 3. The distribution of these sections along the P6/1 can be seen in Figure 32.

No accurate traffic analysis of the P6/1 route exists and it is recommended that a TWAC survey be undertaken as part of a detailed assessment. Because of serious shear distress of the pavement layers in some sections, it is also advised that a DCP survey be part of further investigations to determine the properties of these layers.

Finally, it is recommended that rehabilitation should include the widening of the carriageway as a priority.

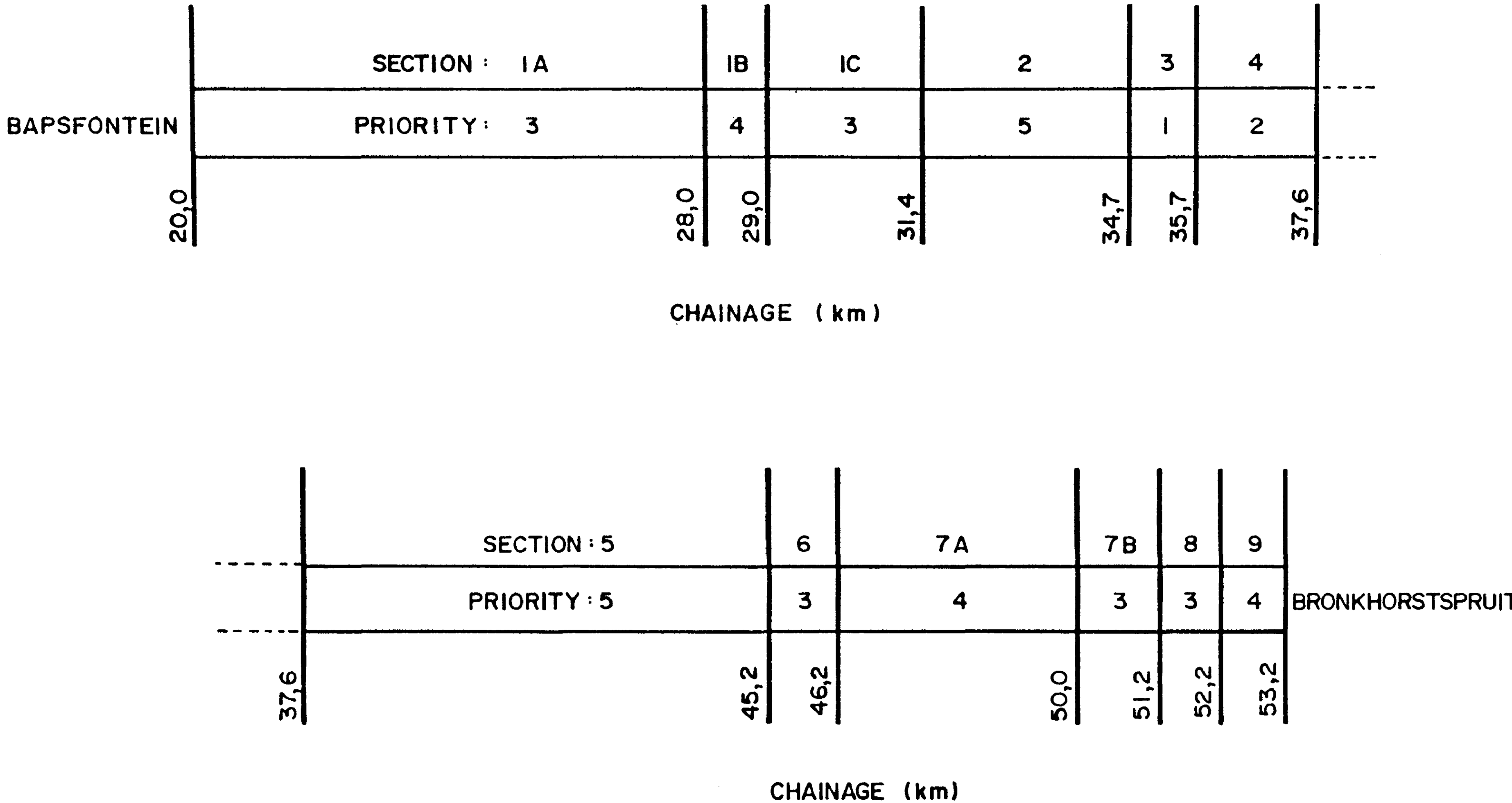


FIGURE 32

URGENCY RATING OF REQUIRED ACTION ON VARIOUS SECTIONS OF THE P6/1 ROUTE

7.1.6 CONDITION SHEETS

7.1.6.1 Figure 33

7.1.6.2 Figure 34

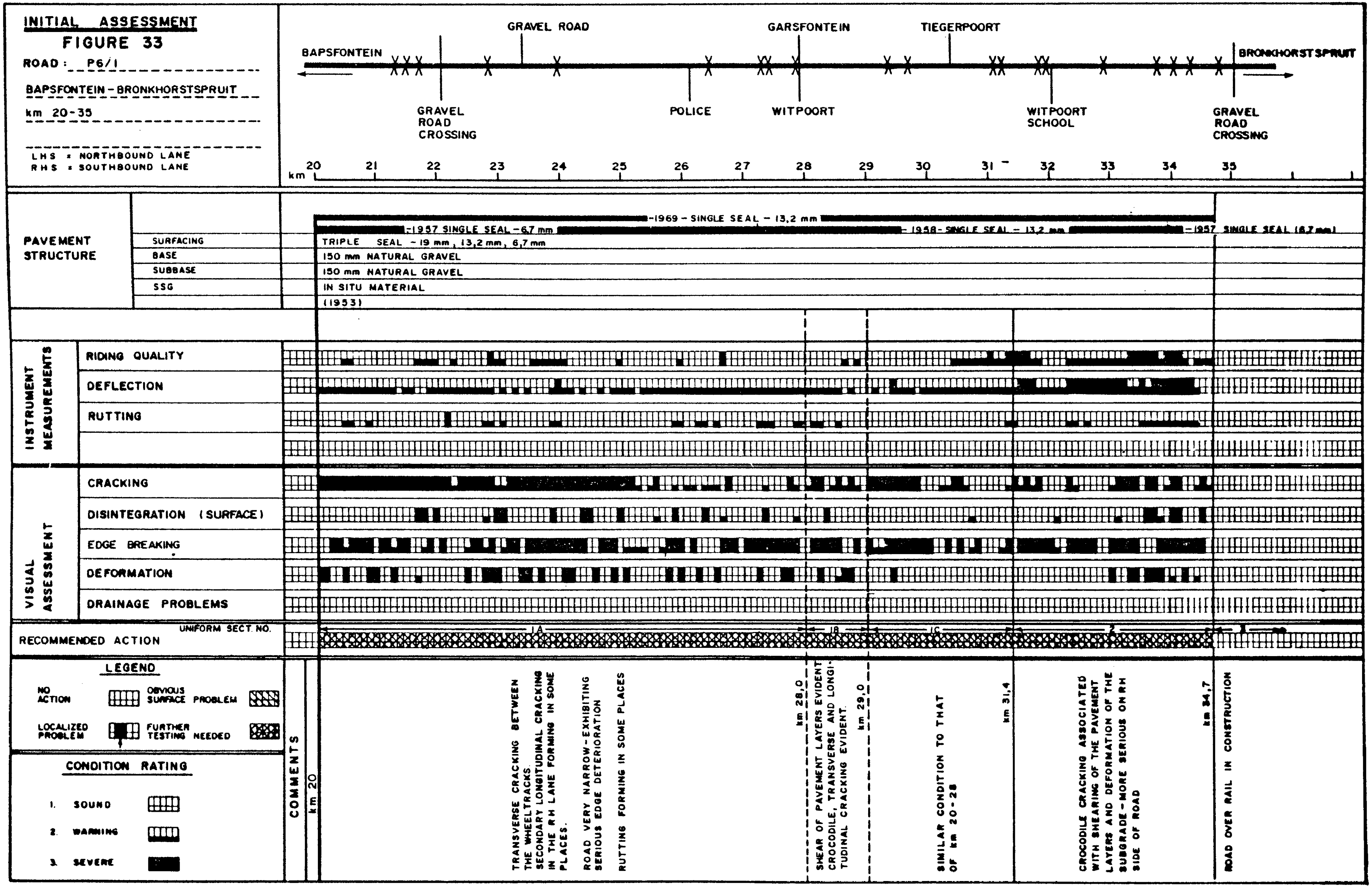


FIGURE 33

CONDITION SHEET : P6/1 - km 20-35

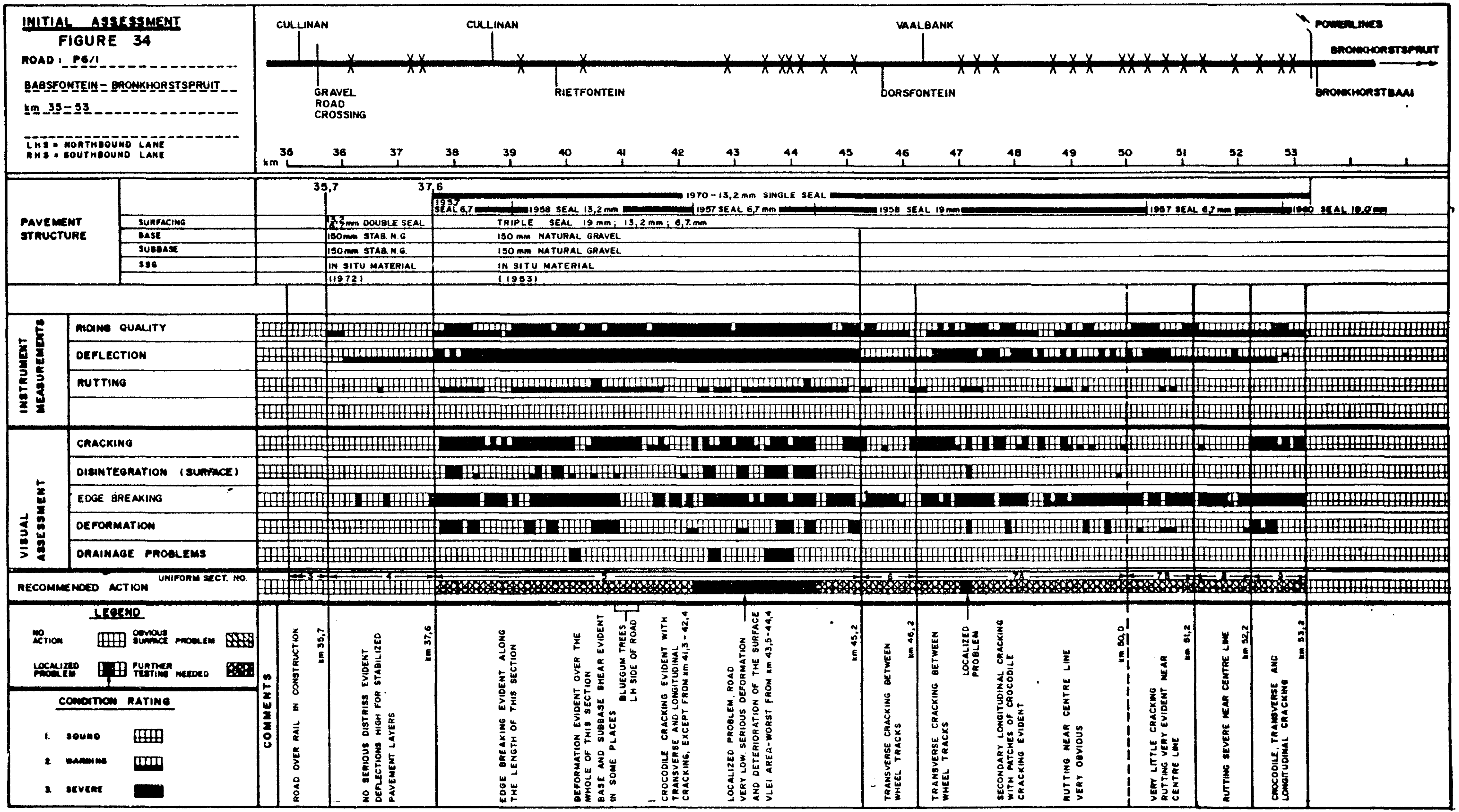


FIGURE 34

CONDITION SHEET : P6/1 - km 35 - 53

7.2 REPORT 2

Pavement rehabilitation study : Detailed assessment, Rehabilitation design and Economic analysis of Section 2 of route P6/1

7.2.1 INTRODUCTION

The initial assessment of route P6/1:

- divided the route into uniform sections;
- identified possible causes and mechanisms of distress;
- where appropriate, recommended tests needed to verify the possible causes and mechanisms of distress; and
- permitted a subjective urgency rating for remedial action on distressed sections.

A uniform section, Section 2 of route P6/1, is used to illustrate the further implementation of the rehabilitation design approach needed to determine the most applicable rehabilitation alternative with confidence. This section, km 31,4 - km 34,7, has an urgency rating of 5, i.e. it requires urgent rehabilitation. The initial assessment showed this section to be subjected to subgrade related deformation as well as pavement layer shear distress.

It should be noted that this report deals only with the rehabilitation needs of the existing paved area. Aspects relating to the need for the widening and/or re-alignment of the road have been specifically excluded.

7.2.2 DETAILED ASSESSMENT

7.2.2.1 <u>Objectives and Scope</u>

The objectives of the detailed assessment are:

- to determine with confidence the causes and mechanisms of distress, and
- to assess the structural capacity of the pavement.

These objectives are achieved through a process of analytical procedures and, if required, additional testing. The initial assessment recommended that DCP measurements were needed to verify preliminary findings. These DCP measurements, together with the data collected in the initial assessment, are used in the analysis. (Normally it is not considered necessary to use all the analytical procedures contained section 4. However, in this example, all these procedures are used in order to illustrate their application.)

7.2.2.2 Analysis of data

(a) Use of the Asphalt Institute curve

The known past accumulated traffic loading of 1,4 x 10^6 E80 (section 7.1) is used, as shown in Figure 35, to obtain the corresponding deflection of 0,83mm. Uniform sections on route P6/1 with representative deflection values of less than 0,83 mm fall in Zone B, while those sections with higher deflections fall in Zone A (see Figure 7). Section 2 falls in Zone A since the mean value of the 95th percentile values of deflection is 1,01 mm (see Table 35). Therefore, the distress on the section can mainly be attributed to an overstressed subgrade, with:

- rutting indicating deformation due to insufficient pavement thickness; and
- cracking indicating insufficient pavement stiffness.

The use of the Asphalt Institute curve confirms the preliminary findings of the initial assessment.

(b) Use of the DCP design curves

In this procedure the measured DCP values are compared with the appropriate design curve. In this case, with a calculated design traffic load of 4,1 x 10^6 E80 (section 7.1) and a cumulative past traffic load of 1,4 x 10^6 E80, the curve for heavy traffic loadings (08-12 x 10^6 E80) in Figure 8, is applicable. In Figure 36 the 90th percentile (see Table 2) values of the DCP measurement are plotted against the design curve.

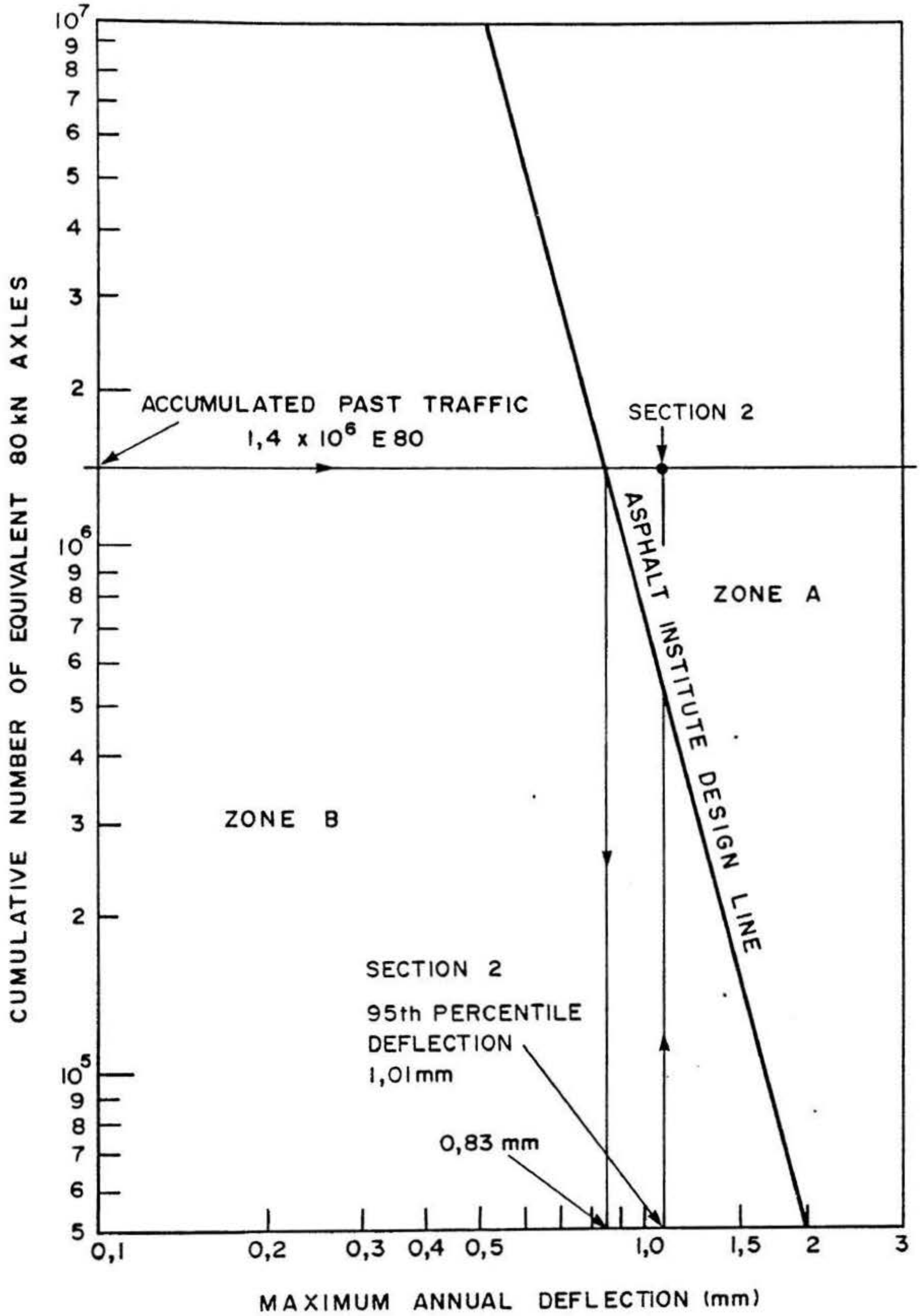


FIGURE 35

RELATION BETWEEN DEFLECTION AND TRAFFIC

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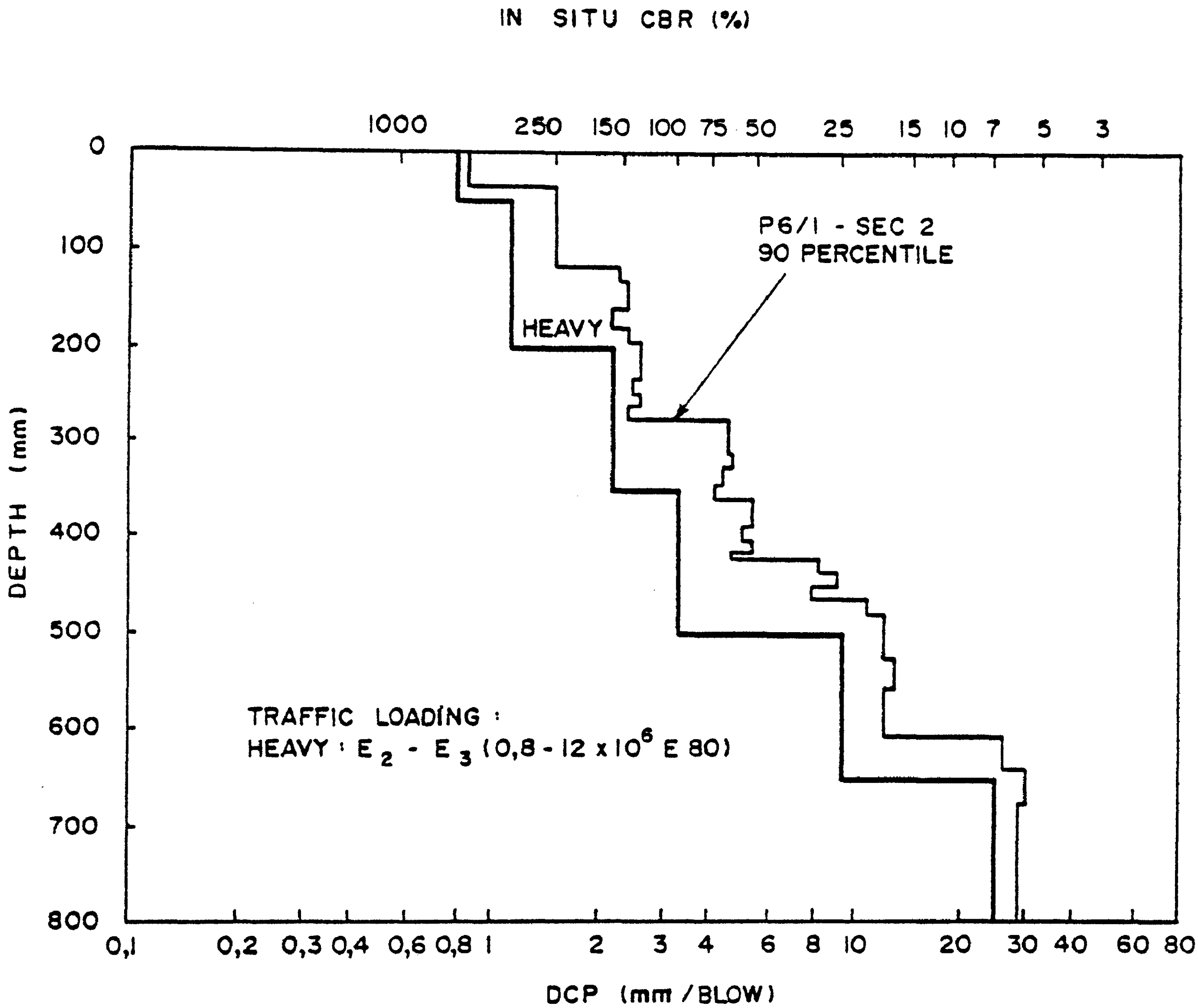

FIGURE 36

DCP DESIGN CURVE FOR HEAVY TRAFFIC LOADINGS

940-4-4350/13a H.D. (KR)

Figure 36 shows that the pavement material properties are totally inadequate. This analysis also confirms the observations in the initial assessment.

(c) Deflection - rut depth relations

This procedure requires the 95th percentile values of deflection and rut depth over 100 m lengths to be plotted. The data of all the sections with similar problems along the route are combined into one graph. For route P6/1 the results of this analysis are given in Figures 37 and 38.

Section 2 is represented in Figure 37, where a general increase in rut depth with a general increase in deflection is shown. This analysis again confirms that deformation originating in the subgrade due to insufficient load distribution by the pavement layers is the main cause and mechanism of distress.

In the absence of rut depth measurements, riding quality can also be used for similar investigations. This is illustrated by the corresponding riding quality versus deflection graphs in Figures 39 and 40. Here again for section 2, represented in Figure 39, there is a general decrease in PSI with deflection, confirming a deformation of the subgrade.

7.2.2.3 <u>Structural capacity analysis</u>

The preceding analysis clearly illustrated that the existing pavement was not designed to carry the past cumulative traffic loads. Distress was thus due to the normal cumulative effect of the traffic. Therefore, use of the Asphalt Institute curve should give a good indication of the life expectancy of the pavement.

From the initial assessment:

- Past accumulated equivalent traffic = 1,4 x 10^6 E80
- Expected equivalent traffic (20 years) = 4,1 x 10^6 E80

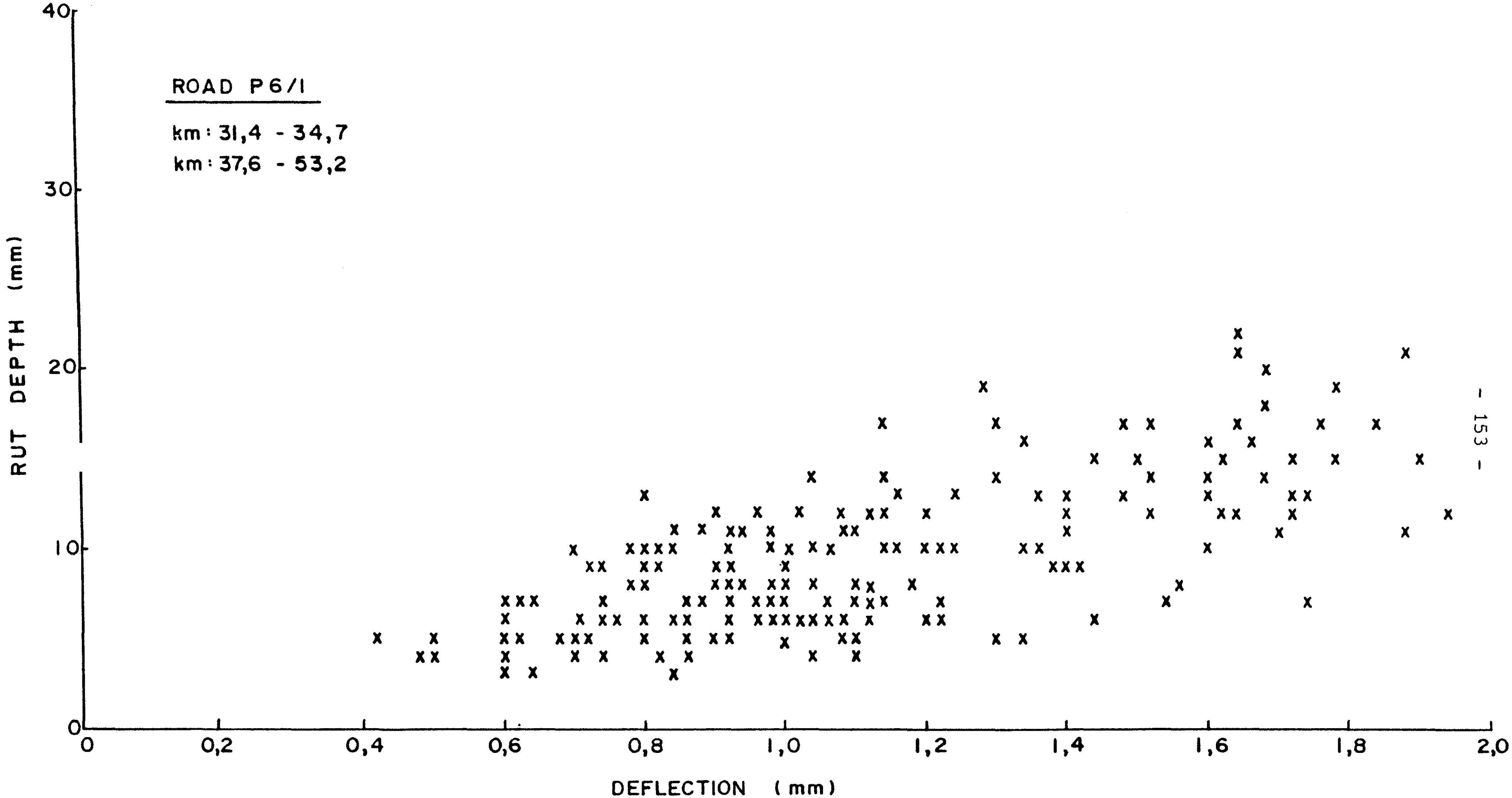

FIGURE 37

DATA SHOWING A GENERAL INCREASE IN RUT DEPTH WITH A GENERAL INCREASE IN DEFLECTION MEASUREMENTS

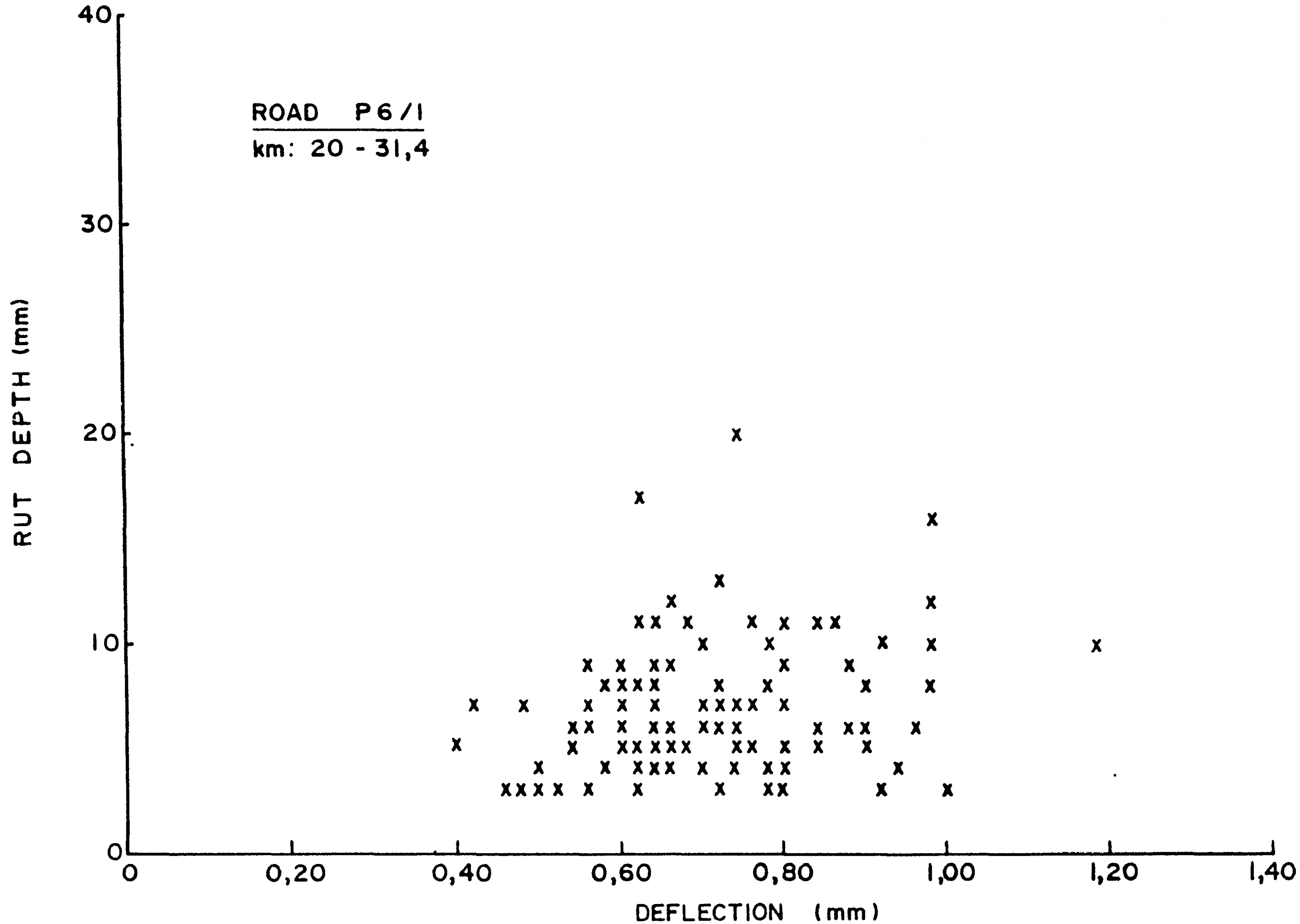

FIGURE 38

DATA SHOWING A POOR CORRELATION BETWEEN RUT DEPTH AND DEFLECTION MEASUREMENTS

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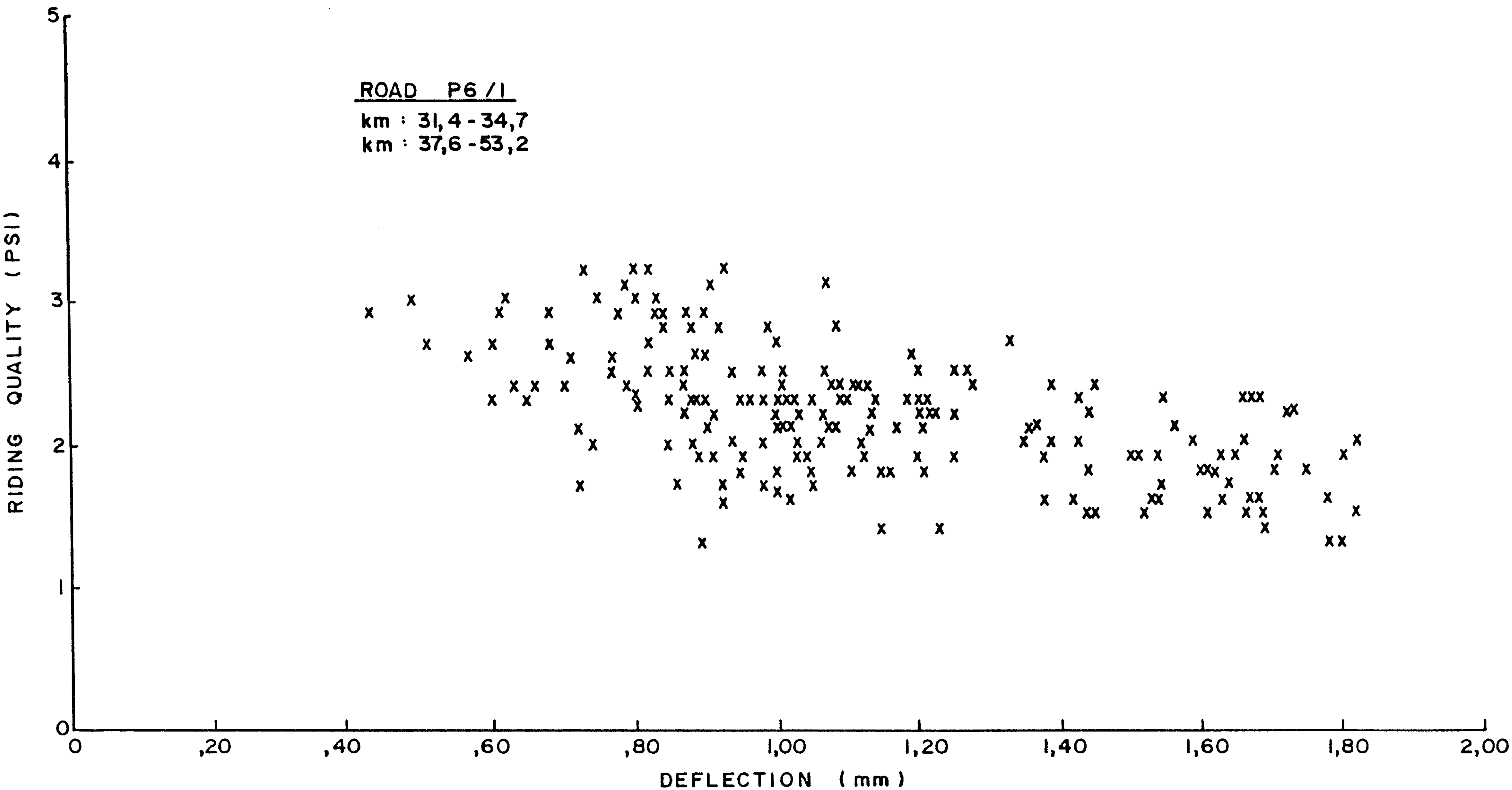

FIGURE 39

DATA SHOWING A GENERAL DECREASE IN RIDING QUALITY WITH A GENERAL INCREASE IN DEFLECTION

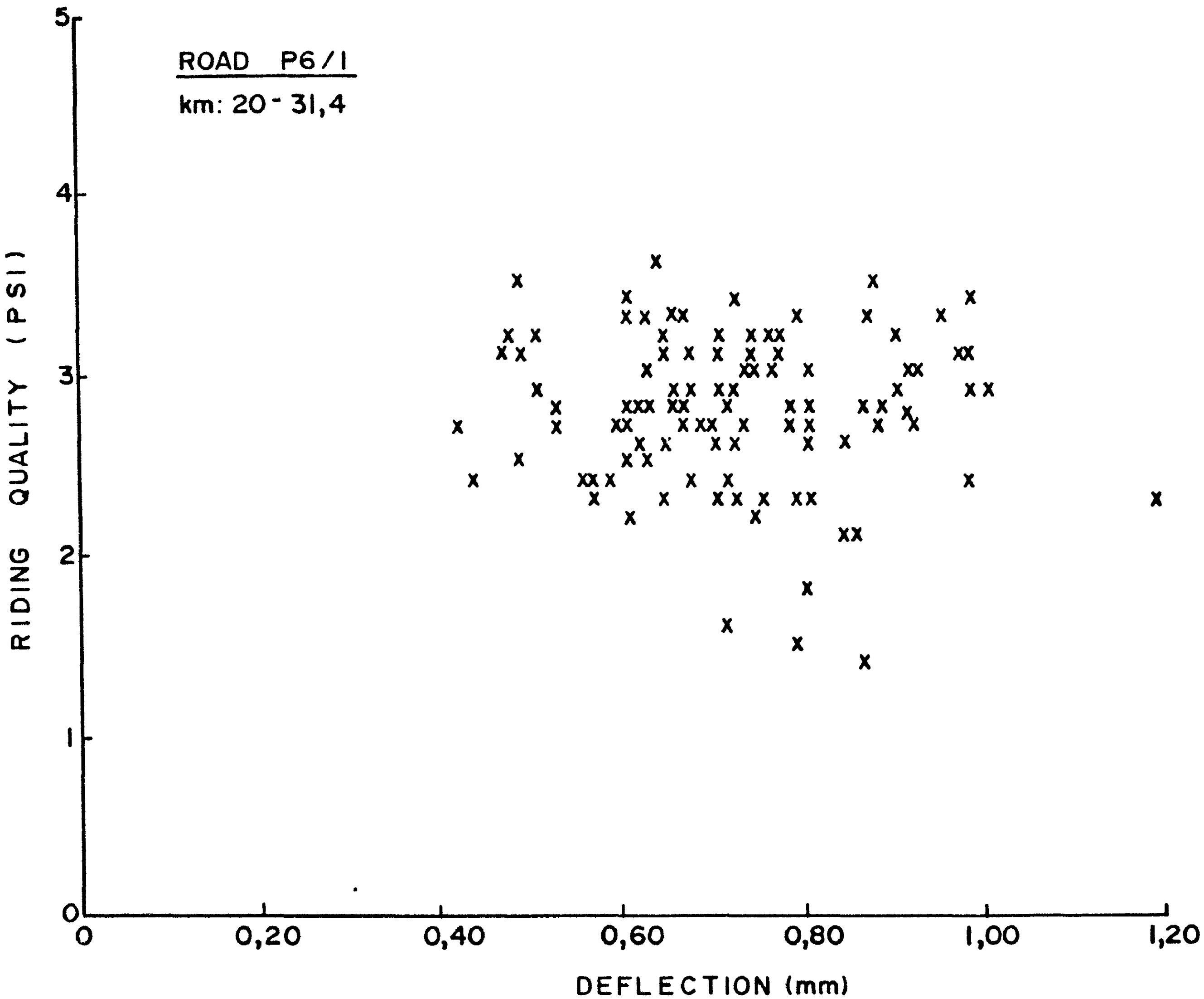

FIGURE 40

DATA SHOWING A POOR CORRELATION BETWEEN RIDING QUALITY AND DEFLECTION MEASUREMENTS

If adequate, a pavement must be able to carry the cumulative equivalent design traffic (N_c) without any strengthening measures.

In this case

$$N_c = \text{Past traffic + future traffic}$$
$$= 5{,}5 \times 10^6 \text{ E80}$$

The predicted design load (N_e) is obtained by using the representative measured deflection of 1,01 mm in Figure 41.

From Figure 41 :

$$N_e = 6{,}7 \times 10^5 \text{ E80}$$

$$\frac{Nc}{Ne} = 8{,}2 > 1$$

Therefore, the pavement needs strengthening. In fact, this can be concluded by using past traffic alone

$$\frac{\text{Past Traffic}}{N_e} = 2{,}1 > 1$$

The capacity of the existing structure has already been exceded 2,1 times. From the distress observed on this section during the initial assessment, this finding is not unexpected.

7.2.2.4 Conclusions

In the analysis the preliminary findings of the initial assessment were verified. The cause(s) and mechanism(s) of the distress on section 2 have thus confidently been determined to include both:

- deformation and resultant cracking originating in the subgrade due to insufficient load distribution by the pavement layers; and

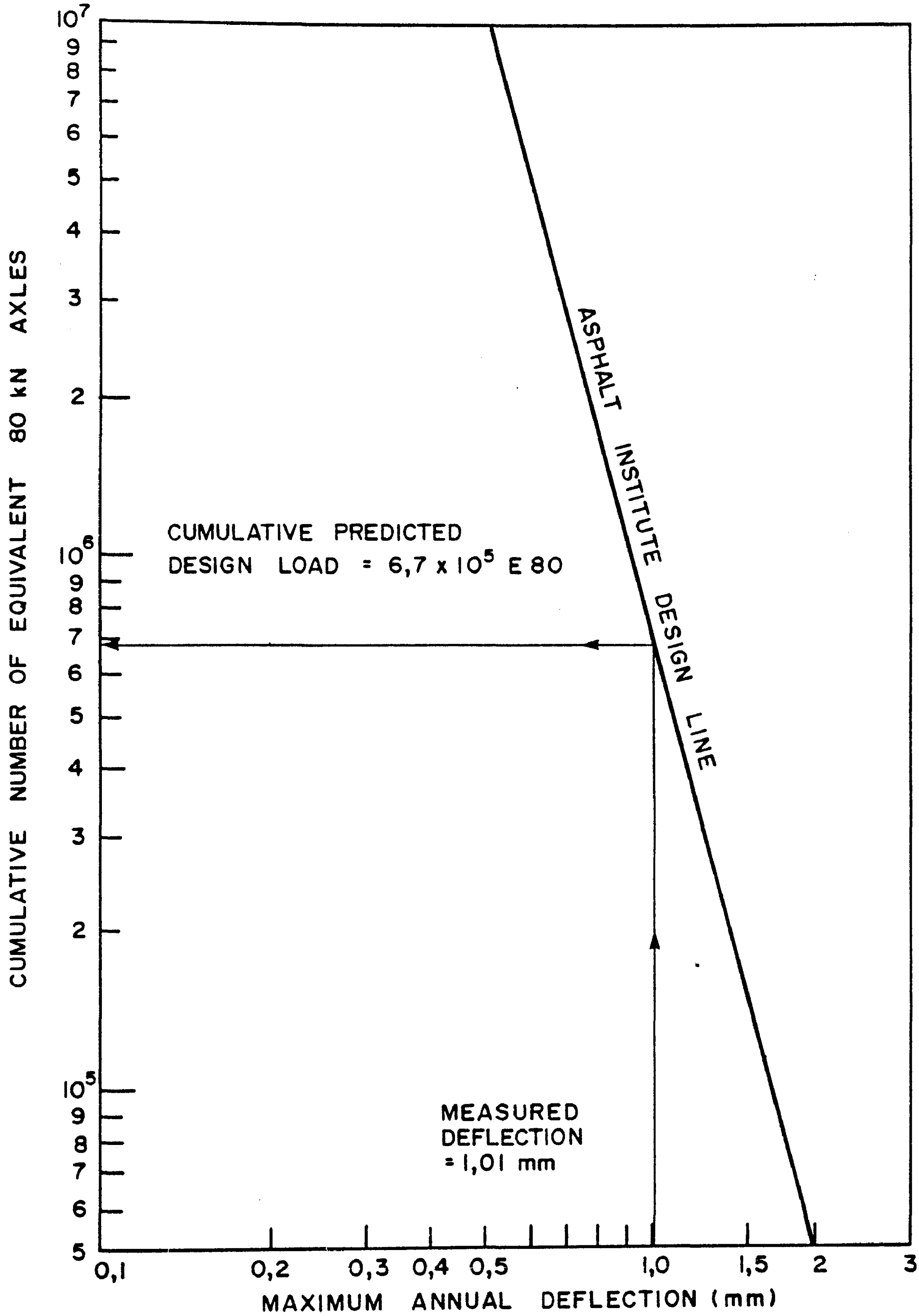

FIGURE 41

RELATION BETWEEN DEFLECTION AND TRAFFIC

- heave and cracking due to shear distress of the pavement layers as a result of inadequate base and subbase material properties.

7.2.3 REHABILITATION DESIGN

7.2.3.1 Objectives and scope

The objective of the rehabilitation design is to establish applicable rehabilitation alternatives.

The detailed assessment has determined that the existing structure has insufficient load-bearing capacity and that the pavement layer material is of inadequate quality. Both these factors are considered in the rehabilitation design. The one found to be most critical will determine the required remedial action.

The structural capacity analysis (section 7.2.2.3) findings indicate that holding actions such as seals, cannot be considered viable alternatives. Strengthening of the pavement is definitely required. With this in view two of the design methods recommended in section 4 are applicable in this case, i.e. those providing for:

- deformation caused by base and subbase inadequacies; and
- deformation originating in the subgrade due to insufficient load distribution by the pavement layers.

7.2.3.2 Remedy for deformation caused by inadequacies in the pavement layers

The procedure followed makes use of DCP measurements as shown in Table 9.

TABLE 9 - Representative measured DCP values of section 2, route P6/1

Pavement layer	Depth (mm)	90th percentile values of DCP measurements (mm/blow)
1	0- 30	0,9
2	30-120	1,41
3	120-270	2,20
4	270-420	4,40

The required cover for each pavement layer is obtained from Figures 42 to 44, and the results are summarized in Table 10.

TABLE 10 - Cover requirements for section 2 of route P6/1

Pavement layer	Required cover (mm)	Existing cover (mm)	Extra cover needed (mm)
1	-	-	-
2	125	30	95
3	210	120	90
4	350	270	80

The 95 mm extra cover needed by pavement layer 2 (base) determines the design requirements of the pavement.

Apart from using a 95 mm granular overlay, the required strengthening can be achieved by using comparable bituminous overlays, by replacing or improving the existing pavement layer material, and by other methods.

7.2.3.3 Remedy for deformation originating in the subgrade

The detailed assessment (Figure 37) showed that there was a direct relation between the rut depth and deflection measurements. This relation is used in the analysis. In Figure 45 the 90 per cent confidence limit (B category road - see Table 2) is used and the deflection corresponding to the allowable rut depth of 10 mm (Appendix 2) is determined as 0,815 mm. This is the minimum deflection which resulted in a

PAVEMENT LAYER : 2

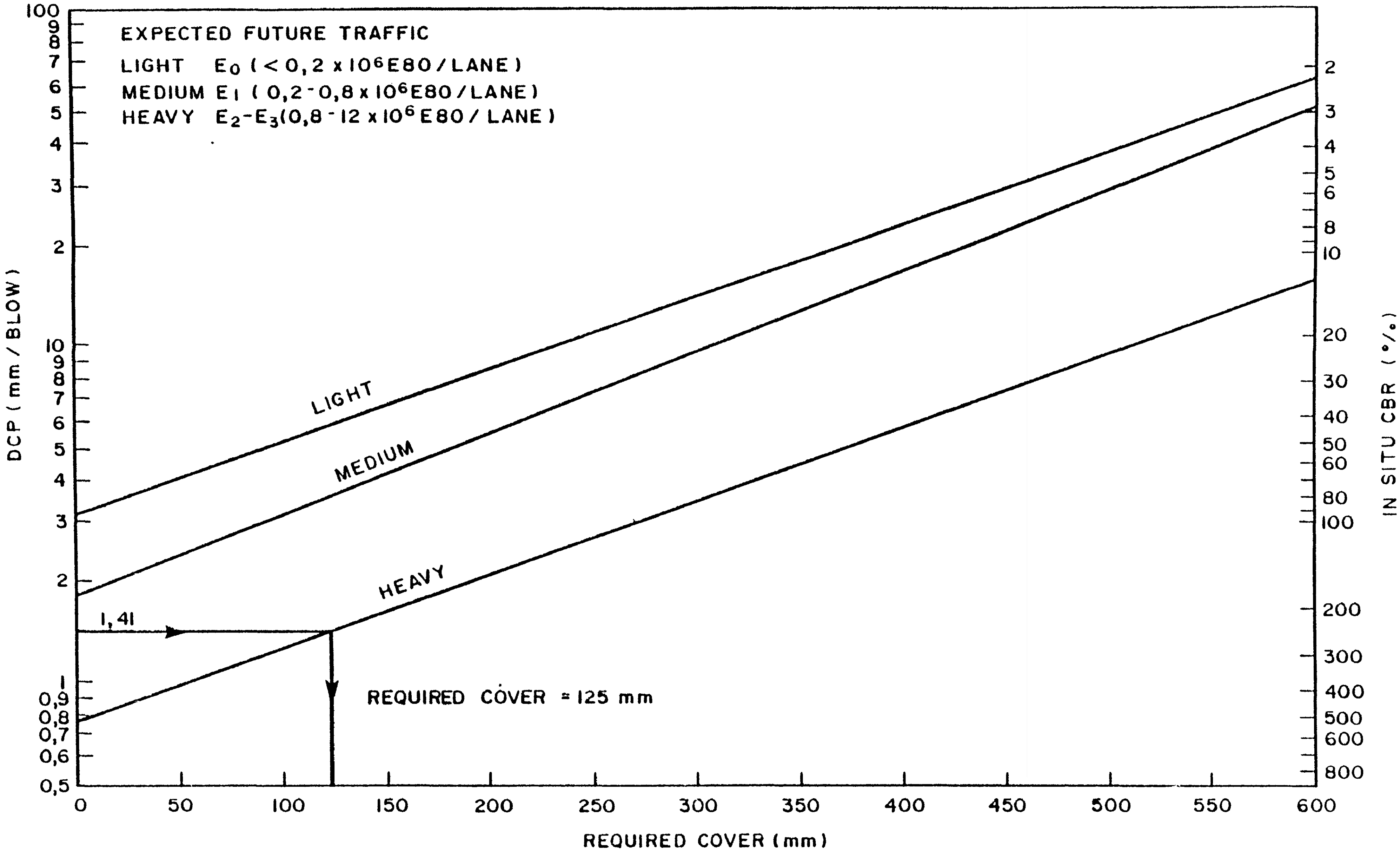

FIGURE 42

COVER REQUIREMENT BASED ON DYNAMIC CONE PENETRATION [20]

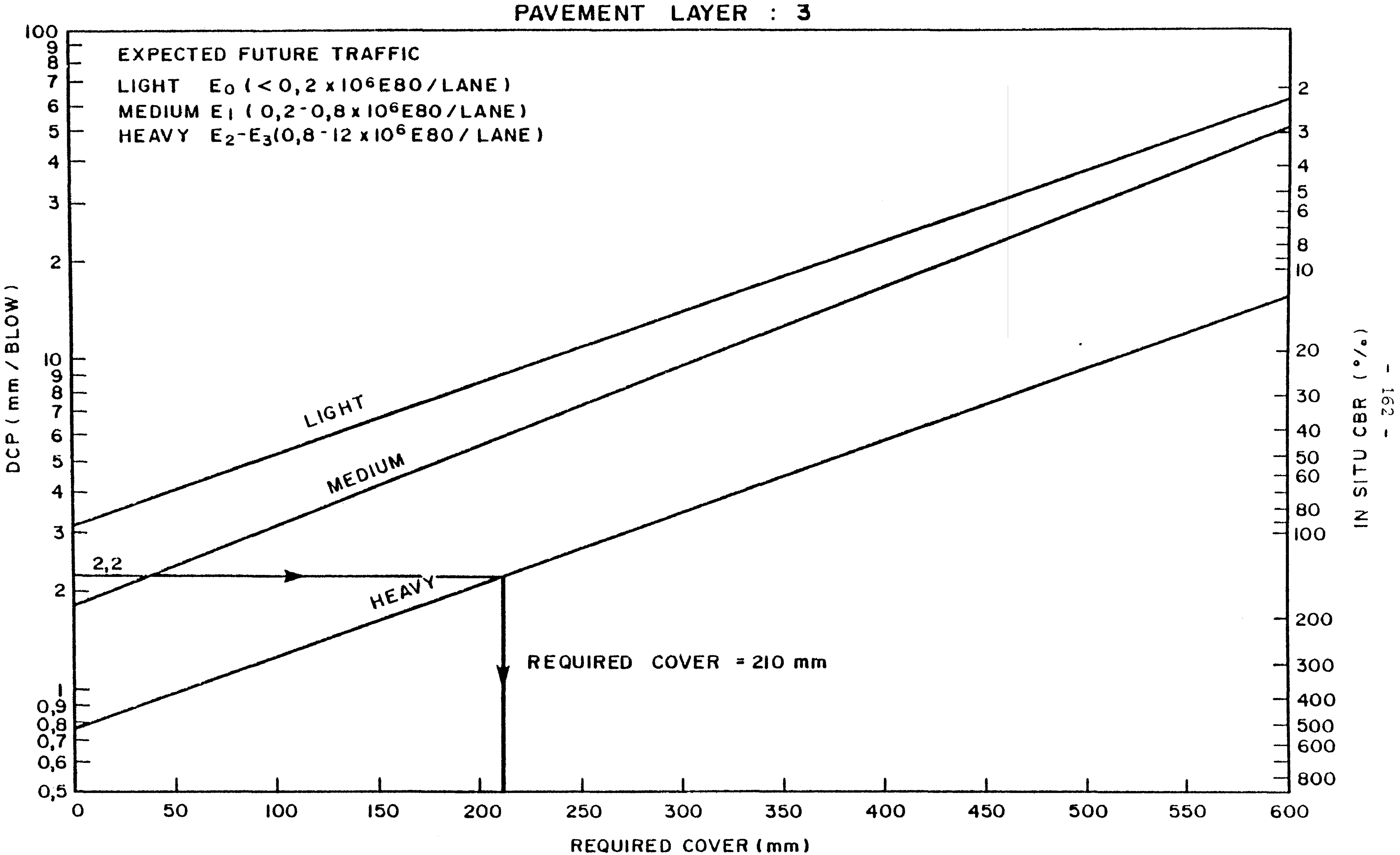

FIGURE 43

COVER REQUIREMENT BASED ON DYNAMIC CONE PENETRATION [20]

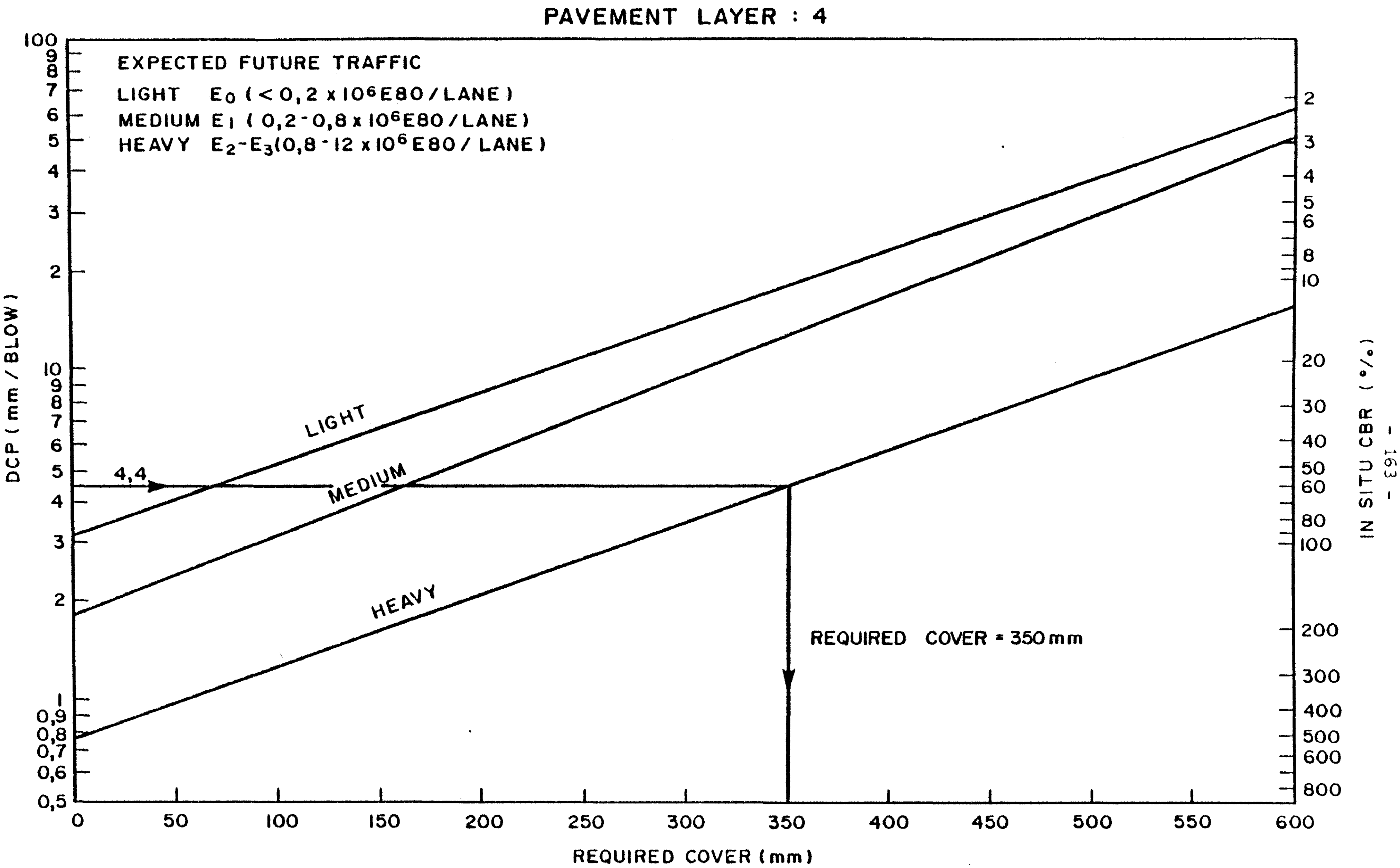

FIGURE 44

COVER REQUIREMENT BASED ON DYNAMIC CONE PENETRATION [20]

rut depth of 10 mm after a past cumulative traffic loading of 1,4 x 10^6 E80.

This deflection is plotted against the calculated past cumulative traffic as shown in Figure 46, to establish a design line for this section of route P6/1. This design line is used to determine the design deflection, 0,63 mm, which corresponds to the calculated future cumulative traffic loading.

The design deflection and the representative measured deflection are used in Figure 47 to find the applicable rehabilitation category, i.e. category C. According to Appendix 3, a nominal asphalt overlay thickness of 60 mm is required.

7.2.3.4 Applicable rehabilitation options

The preceding sub-sections showed that deformation originating in the subgrade due to insufficient load distribution by the pavement layers constitutes the most critical condition. Therefore the appropriate remedial action is found to be 60 mm of asphalt overlay. Other equivalent options such as the following can also be considered: (An equivalent factor of 2 for asphalt versus granular material is used)

- removal of existing seals, in-situ stabilization of 100 mm of the natural gravel base, and a crusher run overlay of 120 mm with a double seal;
- as above but replacing the crusher run overlay with an appropriate thickness of granular material;
- removal of existing seals, in-situ stabilization of 100 mm of the natural gravel base and a crusher run overlay of 100 mm with a 30 mm asphalt overlay;
- as above but replacing the crusher run overlay with an appropriate thickness of granular material;
- reconstruction.

It should be noted that:

(a) Hot mix recycling is not applicable

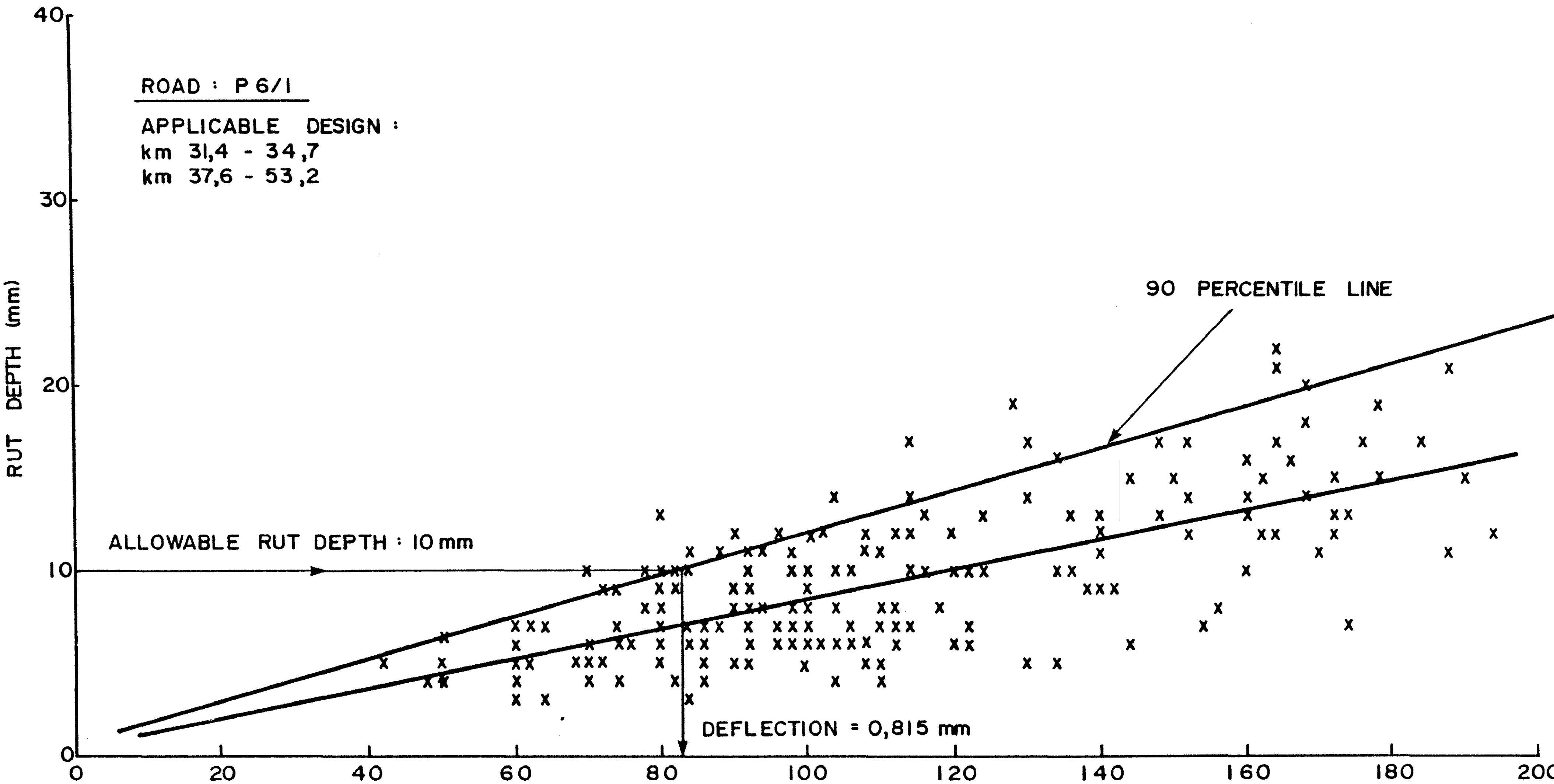

FIGURE 45

THE USE OF THE RUT DEPTH DEFLECTION RELATION OF ROUTE P6/1 FOR REHABILITATION DESIGN

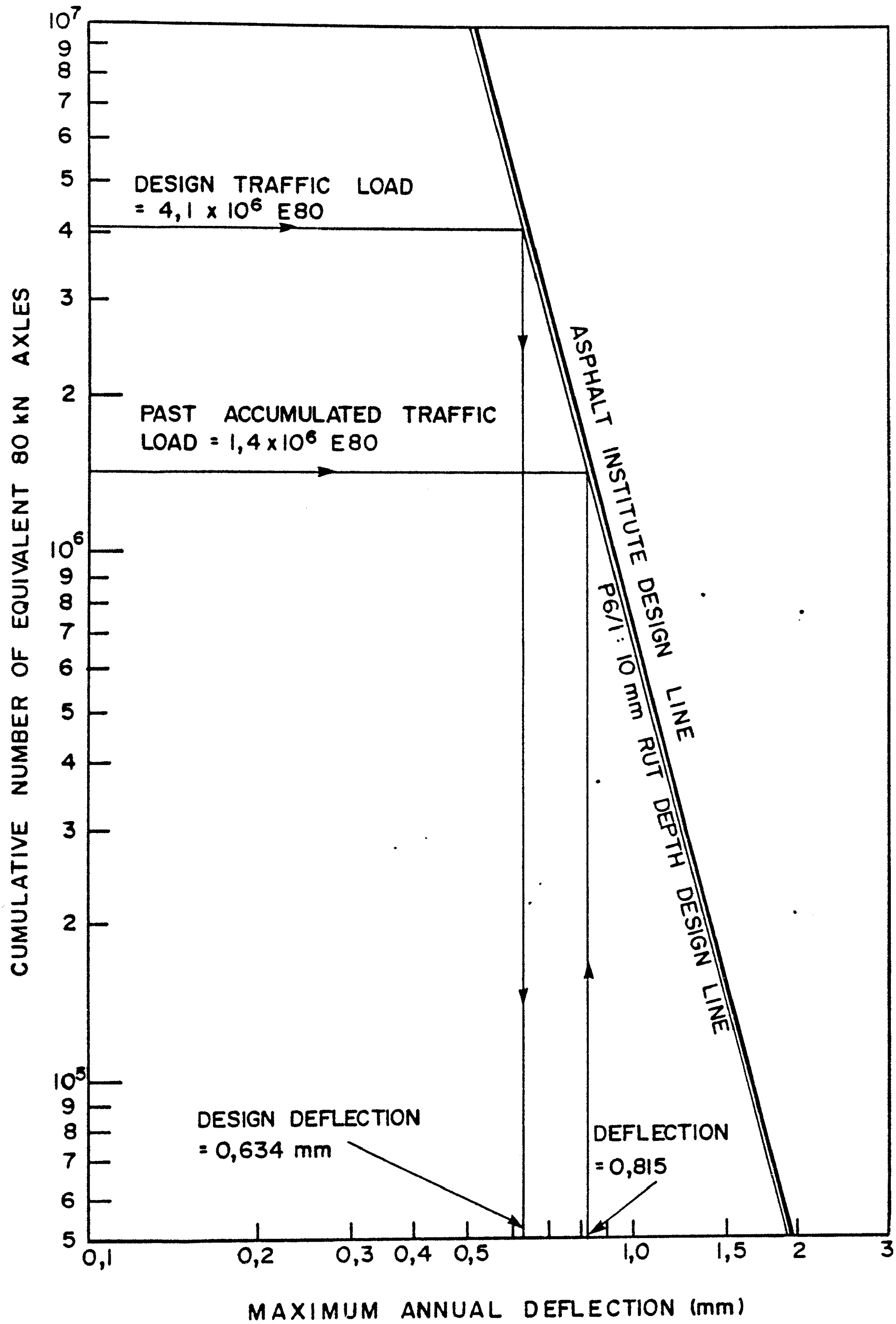

FIGURE 46

RELATION BETWEEN DEFLECTION AND TRAFFIC

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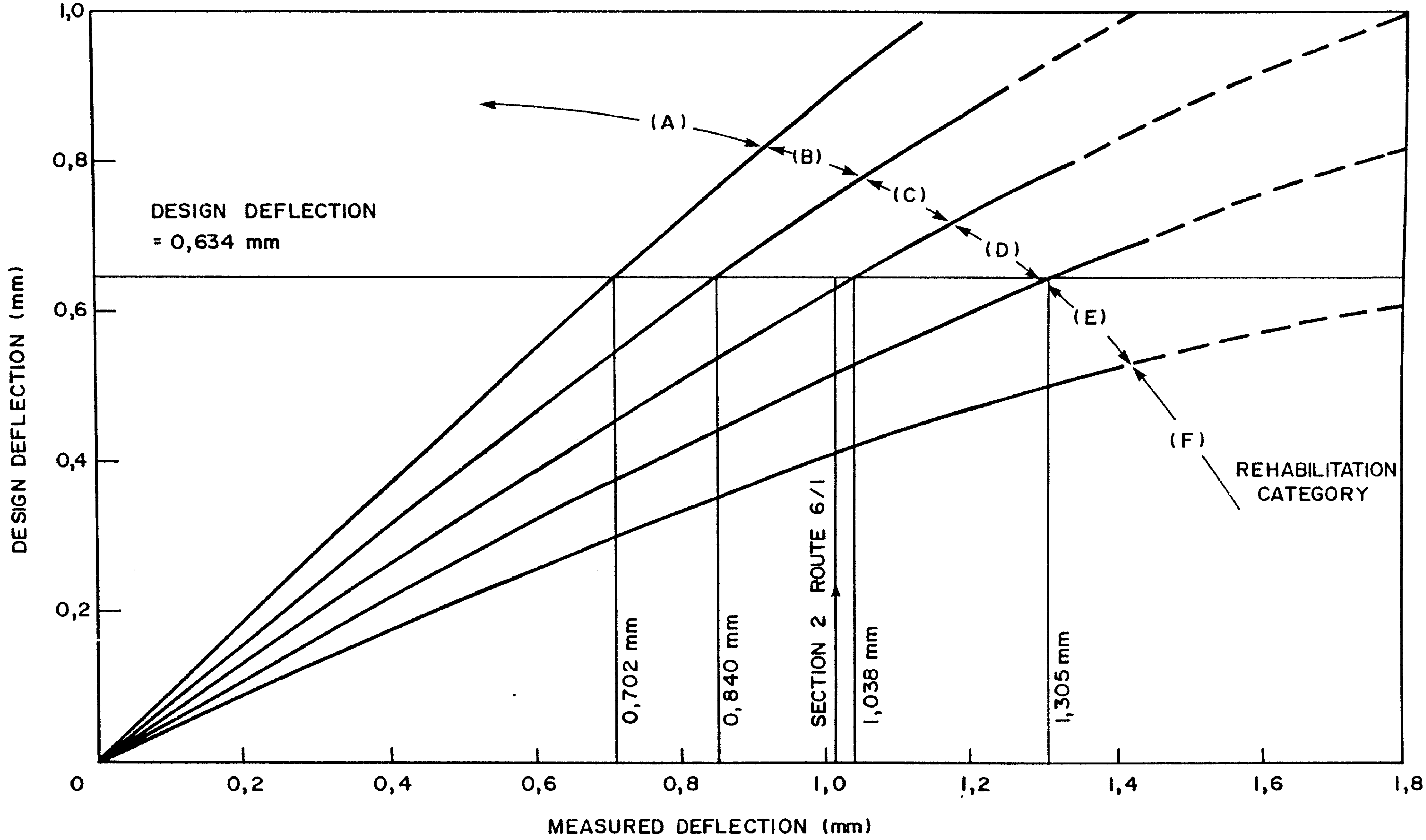

FIGURE 47
REHABILITATION DESIGN CHART

(b) The existing seals may be utilized in the in-situ stabilization.

7.2.4. ECONOMIC ANALYSIS

7.2.4.1 Objectives and scope

The rehabilitation design section has identified a number of possible remedial options. The following are considered the most appropriate for adoption in this project and are consequently examined in the economic analysis:

- 60 mm asphalt overlay
- 120 mm crusher-run overlay with a double seal
- 100 mm crusher-run overlay with 30 mm of asphalt overlay.

The last two options also include the removal of the existing surfacing and the stabilization of 100 mm of the in-situ base-course material.

The objective of the economic analysis is to rank these available options in accordance with economic criteria by examining their respective costs and consequences. This is accomplished by taking into account:

- capital costs, which consists of initial rehabilitation costs, maintenance costs and salvage value; and
- road user costs such as accident costs, time delay costs and vehicle operating costs.

For the purpose of this analysis, road user costs are assumed to be equal for the options under consideration. Therefore, these costs are not included in the analysis.

In the analysis various possible outcomes are considered after periods of 10 and 20 years by using the concept of decision trees.

The following factors are used in the analysis:

- analysis period : 20 years
- discount rate : 8 per cent

7.2.4.2 Current unit costs

The following unit costs are used in the analyis:

- Single surface seal		R 6 710/km
- Double surface seal		R11 160/km
- Asphalt overlay :	25 mm	R25 160/km
	30 mm	R30 200/km
	60 mm	R60 390/km
- SAM (bitumen/rubber seal)		R13 420/km
- SAMI (SAM + Asphalt overlay):	30mm	R42 620/km
	60mm	R73 810/km
- Ripping + Removal of seals		R 1 380/km
- Ripping + Recompaction (100 mm)+ In-situ stabilization (100 mm) (emulsion)		R 5 000/km
- Granular base layer:	100 mm	R 5 000/km
	120 mm	R 6 000/km
- Crusher run base	100 mm	R17 570/km
	120 mm	R21 080/km

7.2.4.3 Salvage values

The salvage values of the possible outcomes within a rehabilitation option are determined as the costs involved in bringing these possible outcomes to the same end condition.

However, the salvage value of each option at the end of the analysis period may differ owing to the use of different materials in the rehabilitation process. For example, the salvage value of 60 mm of asphalt will be higher than the salvage value of 120 mm of crusher run material. In this study it is assumed that the salvage values of:

- Option 1 - 60 mm of asphalt : R12 000/km
- Option 2 - 120 mm of crusher run : R 3 000/km
- Option 3 - 100 mm of crusher run with 30 mm of asphalt : R 8 500/km

Therefore, in this case the salvage value cost of all the possible outcomes of option 2, when compared with option 1, should be increased by R9 000 (R12 000 - R3 000). In the same way, the salvage value costs of all the possible outcomes of option 3, when compared with option should be increased by R3 500 (R12 000 - R3 500).

7.2.4.4 Analysis of the options

(a) Option 1

Remedial initial action : 60 mm asphalt overlay.

For this remedial option two pavement conditions are considered possible after 10 years, i.e.

- Condition 1A - a cracked condition with a probability of occurance of 50 % (subjectively determined);
- Condition 1B - a sound condition with a probability of occurance of 50 % (subjectively determined).

If the pavement is cracked after 10 years (condition 1A) the following alternative remedial actions will be considered :

- a seal (option 1A1);
- a SAM seal (option 1A2);
- a thin overlay (option 1A3);
- a SAMI overlay (option 1A4).

If the pavement is in a sound condition after 10 years, (condition 1B) the following actions could be appropriate:

- do nothing (option 1B1);
- a seal (option 1B2);

- a thin overlay (option 1B3).

In each case and for each appropriate remedial option, the various different outcomes with probabilities of occurance are determined as shown in Figure 48. These values are subjectively derived by comparing the different structural influences such as the effect of a further structural layer (thin overlay), compared to a seal layer. The possible pavement conditions, the probabilities of occurance and the influences of the various remedial actions with salvage value costs are summarized in Table 11.

Using the data in Table 11, the PWOC of the different remedial actions for each possible pavement condition after 10 years are now determined

Condition 1A:

Pavement - Cracked

Option 1A1 :

Remedial option : Seal

PWOC (seal)

= Initial costs discounted for 10 years + (Σ Probability of occurance x Salvage value costs discounted for 20 years)

$= \text{R6 710 } (1{,}08)^{-10} + |0{,}8\ (\text{R23 150})(1{,}08)^{-20}$

$+ 0{,}2\ (\text{R12 600})(1{,}08)^{-20}|$

$= \text{R3 108} + |\text{R3 973} + \text{R541}|$

$= \text{R3 108} + \text{R4 514}$

$= \text{R7 622/km}$

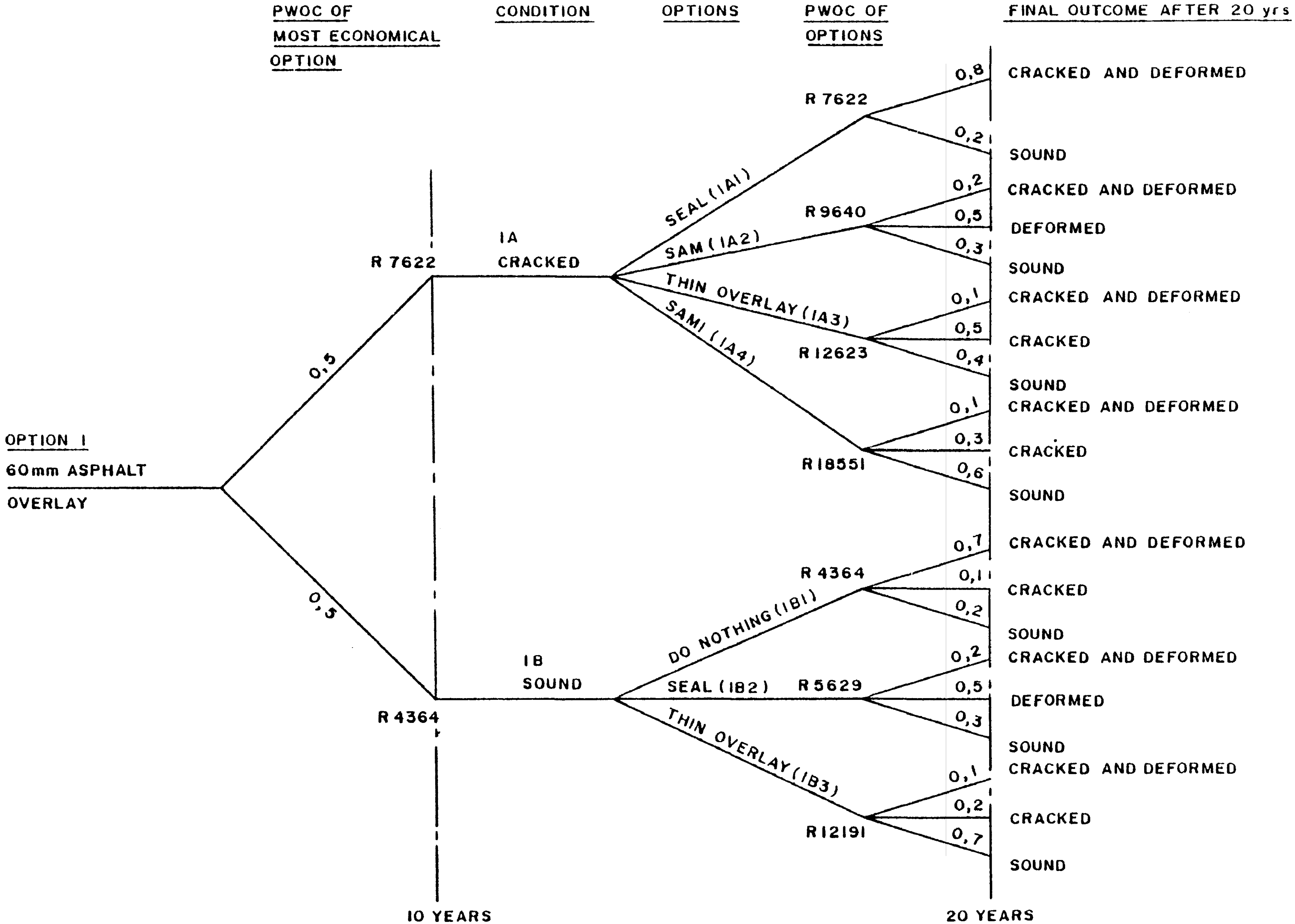

FIGURE 48

DECISION TREE OF OPTION ONE

TABLE 11 - Possible outcomes as option 1 with all relevant costs/km

Initial act	Condition after 10 years	Probability of occurance	Second act (option)		Condition after 20 years	Probability of occurance	Salvage value costs
60 mm Asphalt Overlay (R60 390) (1)	Cracked (1A)	0,5	(1A1)	Seal (R 6 710)	Cracked & Deformed	0,8	R23 150
					Sound	0,2	R12 600
			(1A2)	SAM (R13 420)	Cracked & Deformed	0,2	R23 150
					Deformed	0,5	R15 100
					Sound	0,3	R12 600
			(1A3)	Thin overlay (25 mm) (R25 160)	Cracked & Deformed	0,1	R11 600
					Cracked	0,5	R 6 710
					Sound	0,4	-
			(1A4)	SAMI (25 mm) (R38 580)	Cracked & Deformed	0,1	R11 600
					Cracked	0,3	R 6 710
					Sound	0,6	-
	Sound (1B)	0,5	(1B1)	Do-Nothing (Ro)	Cracked & Deformed	0,8	R23 150
					Deformed	0,1	R10 100
					Sound	0,1	R 8 100
			(1B2)	Seal (R 6 710)	Cracked & Deformed	0,2	R15 100
					Deformed	0,5	R12 600
					Sound	0,3	R 8 100
			(1B3)	Thin overlay (25 mm) (R25 160)	Cracked & Deformed	0,1	R11 600
					Cracked	0,2	R 6 710
					Sound	0,7	-

Option 1A2

Remedial option : SAM

PWOC (SAM)

= Initial costs discounted for 10 years + (Σ Probability of occurance x Salvage value costs discounted for 20 years)

$= R13\ 420\ (1{,}08)^{-10} + |0{,}2\ (R23\ 150)(1{,}08)^{-20}$
$+ 0{,}5\ (R15\ 100)(1{,}08)^{-20} + 0{,}3\ (R12\ 600)(1{,}08)^{-20}|$

$= R6\ 216 + |R993 + R1\ 620 + R811|$

$= R6\ 216 + R3\ 424$

= R9 640/km

Option 1A3 :

Remedial option : Thin overlay

PWOC (Thin overlay)

= Initial costs discounted for 10 years + (Σ Probability of occurance x Salvage value costs discounted for 20 years)

$= R25\ 160\ (1{,}08)^{-10} + |0{,}1\ (R11\ 600)(1{,}08)^{-20}$
$+ 0{,}5\ (R6\ 710)(1{,}08)^{-20} + 0{,}4\ (0)(1{,}08)^{-20}|$

$= R11\ 654 + |R249 + R720 + R0|$

$= R11\ 654 + R969$

= R12 623/km

Option 1A4 :

Remedial option : SAMI

PWOC (SAMI)

= Initial costs discounted for 10 years + (Σ Probability of occurance x Salvage value costs discounted for 20 years)

$= R38\ 580\ (1{,}08)^{-10} + |0{,}1\ (R11\ 600)(1{,}08)^{-20}$
$+ 0{,}3\ (R6\ 710)(1{,}08)^{-20} + 0{,}6\ (0)(,108)^{-20}|$

$= R17\ 870 + |R249 + R432 + R0|$

$= R17\ 870 + R681$

= R18 551/km

It is seen that a seal (option 1A1) is the most economical alternative if the pavement is in a cracked condition after a period of 10 years. As shown in Figure 48, the PWOC of the seal, i.e. R7 622/km, is now used in calculating the PWOC of Option 1.

Condition 1B :

Pavement condition : Sound :

Option 1B1 :

Remedial option : Do nothing

PWOC (Do nothing)

= Initial costs discounted for 10 years + (Σ Probability of occurance x Salvage value costs discounted for 20 years)

$= R0\ (1{,}08)^{-10} + \left|0{,}8\ (R23\ 150)(1{,}08)^{-20} + 0{,}1\ (R10\ 000)(1{,}08)^{-20} + 0{,}1\ (R8\ 100)(1{,}08)^{-20}\right|$

$= 0 + \left|R3\ 973 + R217 + R174\right|$

= 0 + R4 364

= R4 364/km

Option 1B2 :

Remedial option : Seal

PWOC (Seal)

= Initial costs discounted for 10 years + (Σ Probability of occurance x Salvage value costs discounted for 20 years)

$= R6\ 710(1{,}08)^{-10} + \left|0{,}2\ (R15\ 100)(1{,}08)^{-20} + 0{,}5\ (R12\ 600)(1{,}08)^{-20} + 0{,}3\ (R8\ 100)(1{,}08)^{-20}\right|$

$= R3\ 108 + \left|R648 + R1\ 352 + R521\right|$

= R3 108 + R2 521

= R5 629/km

Option 1B3 :

Remedial option : Thin overlay

PWOC (Thin overlay)

= Initial costs discounted for 10 years + (Σ Probability of occurance x Salvage value costs discounted for 20 years)

$= R25\ 160(1{,}08)^{-10} + |0{,}1\ (R11\ 600)(1{,}08)^{-20} + 0{,}2\ (R6\ 710)(1{,}08)^{-20} + 0{,}7\ (0)(1{,}08)^{-20}|$

$= R11\ 654 + |R249 + R288 + R0|$

= R11 654 + R537

= R12 191/km

From the above three calculations, it is clear that the do nothing alternative (option 1B1) is the most economical remedial option if the pavement is in a sound condition after a period of 10 years. As shown in Figure 48, the PWOC of the do nothing option, i.e. R4 364/km, is now used in calculating the PWOC of Option 1.

Therefore,

PWOC of Option 1

= Initial costs + (Σ Probability of occurance x PWOC of most economical remedial action)

$= R60\ 390 + |0{,}5\ (R7\ 622) + 0{,}5\ (R4\ 364)|$

= R60 390 + R5 993

= R66 383

≅ R66 400/km

(b) Option 2

Remedial initial action: removal of existing seals, in situ stabilization of 100 mm of the natural gravel; 120 mm crusher run with a double seal.

This option only provides for a double seal as surfacing material. For a B category road this is not normally considered adequate for a long period of time and deterioration can therefore be expected within 10 years. Taking this into account, three conditions are considered possible after 10 years of service, i.e. :

- Condition 2A - badly deteriorated surface with a probability of occurance of 40 % (subjectively determined);
- Condition 2B - a cracked surface with a probability of occurance of 40 % (subjectively determined);
- Condition 2C - a sound pavement with a probability of occurance of 20 % (subjectively determined).

The following alternative remedial actions will in each case be appropriate :

If the surfacing of the pavement is badly deteriorated after 10 years :

- a thin overlay (option 2A1)
- a SAMI layer (option 2A2)

If the surfacing of the pavement is cracked after 10 years :

- a seal (option 2B1); or
- a SAM layer (option 2B2); or
- a thin overlay (option 2B3); or
- a SAMI layer (option 2B4).

If the pavement is in a sound condition after a 10 year period :

- a do nothing approach (option 2C1); or
- a seal (option 2C2); or
- a thin overlay (option 2C3)

In each case the most economical alternative is determined in order to find the best approach. This is done, as in Option 1, by taking into account the possible end conditions of the pavement after a further 10 years, by determining the salvage values cost and probability of occurances as shown in Figure 49 and Table 12. Using this data the PWOC of each alternative for each pavement condition is now determined.

Condition 2A :

Pavement Condition : Surfacing badly deteriorated.

Option 2A1 :

Remedial option : Thin overlay

PWOC (Thin overlay)

= Initial costs discounted for 10 years + (Σ Probability of occurance x Salvage value costs discounted for 20 years)

$= R25\ 160(1{,}08)^{-10} + |0{,}1\ (R20\ 600)(1{,}08)^{-20}$

$+ 0{,}6\ (R15\ 710)(1{,}08)^{-20} + 0{,}3\ (R9\ 000)(1{,}08)^{-20}|$

$= R11\ 654 + |R442 + R2\ 022 + R579|$

$= R11\ 654 + R3\ 043$

$= R14\ 697/km$

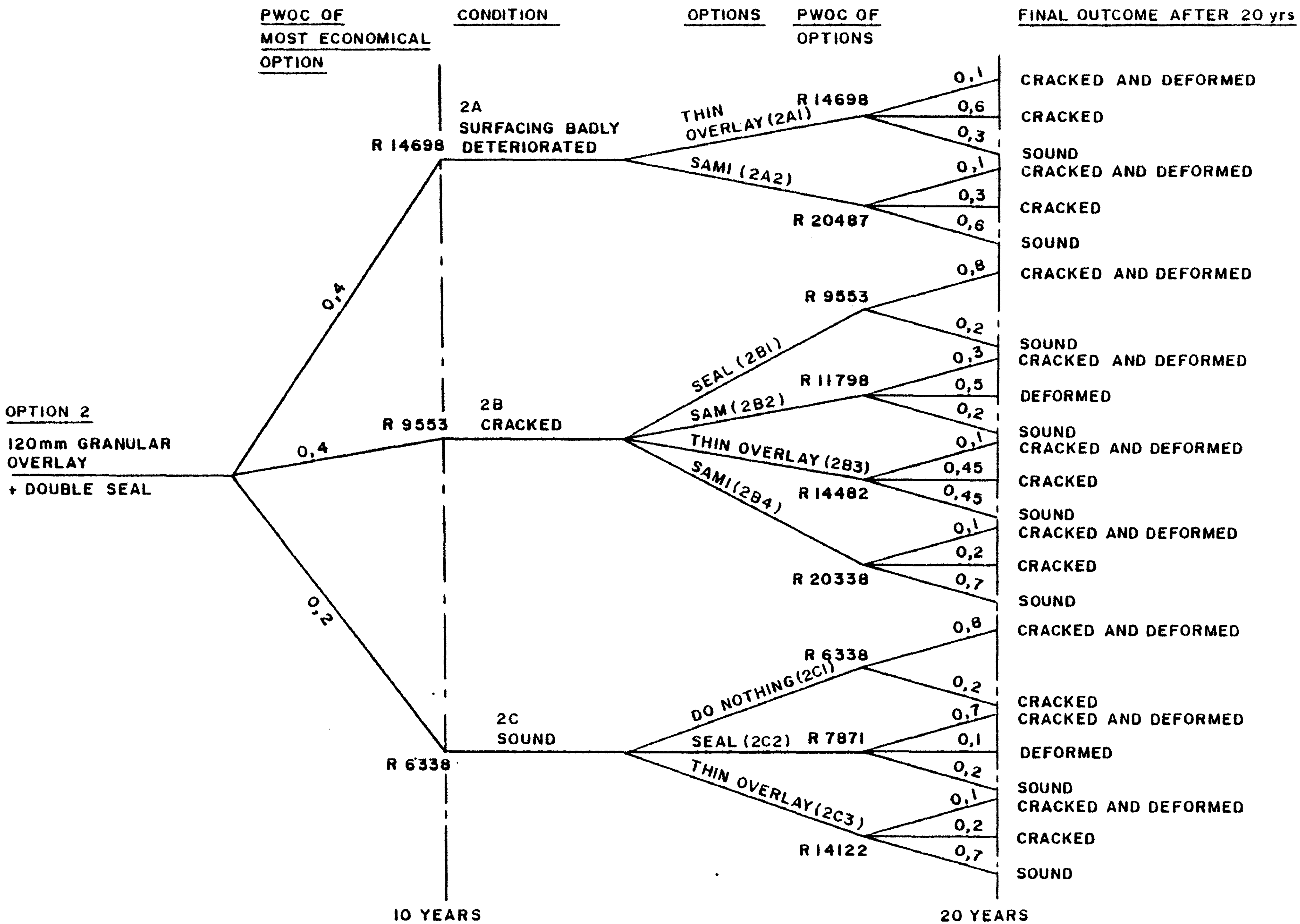

FIGURE 49

DECISION TREE OF OPTIONS TWO

TABLE 12 - Possible outcomes as option 2 with all relevant costs/km

Initial act (option)	Condition after 10 yrs	Probability of occurance	Second act (option)		Condition after 20 years	Probability of occurance	Salvage value costs
120 mm	Surfacing	0,4	(2A1)	Thin overlay	Cracked & Deformed	0,1	R20 600
Crusher	badly de-			(25 mm)	Cracked	0,6	R15 710
Run +	teriorated			(R25 160)	Sound	0,3	R 9 000
Double	(2A)						
seal			(2A2)	SAMI	Cracked & Deformed	0,1	R20 600
(R38 620)				(25 mm)	Cracked	0,3	R15 710
(2)				(R38 580)	Sound	0,6	R 9 000
	Cracked	0,4	(2B1)	Seal	Cracked & Deformed	0,8	R32 150
	(2B)			(R 6 710)	Sound	0,2	R21 600
			(2B2)	SAM	Cracked & Deformed	0,3	R32 150
				(25 mm)	Deformed	0,5	R24 100
				(R13 420)	Sound	0,2	R21 600
			(2B3)	Thin overlay	Cracked & Deformed	0,1	R20 600
				(25 mm)	Cracked	0,45	R15 710
				(R25 160)	Sound	0,45	R 9 000
			(2B4)	SAMI	Cracked & Deformed	0,1	R20 600
				(25 mm)	Cracked	0,2	R15 710
				(R38 580)	Sound	0,7	R 9 000
	Sound	0,2	(2C1)	Do nothing	Cracked & Deformed	0,8	R32 150
	(2C)			(R o)	Cracked	0,2	R19 100
			(2C2)	Seal	Cracked & Deformed	0,7	R24 100
				(R6 710)	Deformed	0,1	R19 100
					Sound	0,2	R17 100
			(2C3)	Thin overlay	Cracked & Deformed	0,1	R20 600
				(25 mm)	Cracked	0,2	R15 710
				(R25 160)	Sound	0,7	R 9 000

Option 2A2 :

Remedial option : SAMI overlay

PWOC (SAMI)

= Initial costs discounted for 10 years + (Σ Probability of occurance x Salvage value costs discounted for 20 years)

$= R38\ 580(1,08)^{-10} + |0,1\ (R20\ 600)(1,08)^{-20}$
$+ 0,3\ (R15\ 710)(1,08)^{-20} + 0,6\ (R9\ 000)(1,08)^{-20}|$

$= R17\ 870 + |R442 + R1\ 011 + R1\ 159|$

$= R17\ 870 + R2\ 612$

$= R20\ 482/km$

It is seen that the most economical option after 10 years if the surfacing of the pavement has badly deteriorated, is a thin overlay (option 2A1). As shown in Figure 49 the PWOC of the thin overlay, i.e. R14 697/km, is now used in calculating the PWOC of Option 2.

Condition 2B :

Pavement condition : Cracked

Option 2B1 :

Remedial option : Surface seal

PWOC (Seal)

= Initial costs discounted for 10 years + (Σ Probability of occurance x Salvage value costs discounted for 20 years)

$= R6\ 710(1,08)^{-10} + |0,8\ (R32\ 150)(1,08)^{-20}$
$+ 0,2\ (R21\ 600)(1,08)^{-20}|$

$= R3\ 108 + |R5\ 518 + R927|$

$= R3\ 108 + R6\ 445$

$= R9\ 553/km$

Option 2B2 :

Remedial option : SAM seal

PWOC (SAM)

= Initial costs discounted for 10 years + (Σ Probability of occurance x Salvage value costs discounted for 20 years)

$= R13\ 420(1,08)^{-10} + |0,3\ (R32\ 150)(1,08)^{-20} + 0,5\ (R24\ 100)(1,08)^{-20} + 0,2\ (R21\ 600)(1,08)^{-20}|$

$= R6\ 216 + |R2\ 069 + R2\ 585 + R927|$

$= R6\ 216 + R5\ 581$

$= R11\ 797/km$

Option 2B3 :

Remedial option : Thin overlay

PWOC (Thin overlay)

= Initial costs discounted for 10 years + (Σ Probability of occurance x Salvage value costs discounted for 20 years)

$= R25\ 160(1,08)^{-10} + |0,1\ (R20\ 600)(1,08)^{-20} + 0,45\ (R15\ 710)(1,08)^{-20} + 0,45\ (R9\ 000)(1,08)^{-20}|$

$= R11\ 654 + |R442 + R1\ 517 + R869|$

$= R11\ 654 + R2\ 828$

$= R14\ 482/km$

Option 2B4 :

Remedial option : SAMI

PWOC (SAMI)

= Initial costs discounted for 10 years + (Σ Probability of occurance x Salvage value costs discounted for 20 years)

$= R38\ 580(1,08)^{-10} + |0,1\ (R20\ 600)(1,08)^{-20} + 0,2\ (R15\ 710)(1,08)^{-20} + 0,7\ (R9\ 000)(1,08)^{-20}|$

$= R17\ 870 + |R442 + R674 + R1\ 352|$

$= R17\ 870 + R2\ 468$

$= R20\ 338/km$

Therefore, if the pavement is in a cracked condition after 10 years, the most economical remedial action would be to seal with an ordinary seal (option 2B1). As shown in Figure 49 the PWOC of the seal, i.e. R9 553/km, is now used for calculating the PWOC of Option 2.

Condition 2C :

Pavement condition : Sound

Option 2C1 :

Remedial option : Do nothing

PWOC (Do nothing)

= Initial costs discounted for 10 years + (Σ Probability of occurance x Salvage value costs discounted for 20 years)

$= R0\ (1{,}08)^{-10} + |0{,}8\ (R32\ 150)(1{,}08)^{-20} + 0{,}2\ (R19\ 100)(1{,}08)^{-20}|$

$= R0 + |R5\ 518 + R820|$

= R6 338/km

Option 2C2 :

Remedial option : Seal

PWOC (Seal)

= Initial costs discounted for 10 years + (Σ Probability of occurance x Salvage value costs discounted for 20 years)

$= R6\ 710\ (1{,}08)^{-10} + |0{,}7\ (R24\ 100)(1{,}08)^{-20} + 0{,}1\ (R19\ 100)(1{,}08)^{-20} + 0{,}2\ (R17\ 100)(1{,}08)^{-20}|$

$= R3\ 108 + |R3\ 619 + R410 + R734|$

= R3 108 + R4 763

= R7 871/km

Option 2C3 :

Remedial option : Thin overlay

PWOC (Thin overlay)

= Initial costs discounted for 10 years + (Σ Probability of occurance x Salvage value costs discounted for 20 years)

$= R25\ 160\ (1{,}08)^{-10} + \left|0{,}1\ (R20\ 600)(1{,}08)^{-20} + 0{,}2\ (R15\ 710)(1{,}08)^{-20} + 0{,}7\ (R9\ 000)(1{,}08)^{-20}\right|$

$= R11\ 654 + \left|R442 + R674 + R1\ 352\right|$

$= R11\ 654 + R2\ 468$

$= R14\ 122/km$

Therefore, if the pavement is in a sound condition after 10 years, it is most economical to do nothing (option 2C1) at that stage. As shown in Figure 49, the PWOC of the do nothing option, i.e. R6 338/km, is now used for calculating the PWOC of Option 2.

The different possible pavement conditions have now been analysed and the PWOC of Option 2 can be determined.

PWOC of Option 2

= Initial costs + (Σ Probability of occurance x PWOC of most economical remedial action)

$= R38\ 620 + \left|0{,}4\ (R14\ 698) + 0{,}4\ (R9\ 553) + 0{,}2\ (R6\ 338)\right|$

$= R38\ 620 + \left|R5\ 879 + R3\ 821 + R1\ 268\right|$

$= R38\ 620 + R10\ 968$

$= R49\ 588$

$\cong R49\ 600/km.$

(c) Option 3

Remedial initial action: removal of existing seals;

in-situ stabiliation of 100 mm of the natural gravel; 100 mm crusher run with 30 mm of asphalt overlay.

This option provides for the removal of any existing in-situ stress concentrations and also eliminates the problem of reflection cracking. Therefore, the risk of cracking should be less than that of Option 1. Two pavement conditions are considered possible after a period of 10 years, i.e.

- Condition 3A - a cracked condition with a probability of occurance of 40 per cent (subjectively determined);
- Condition 3B - a sound condition with a probability of occurance of 60 per cent (subjectively determined).

The following appropriate alternative remedial actions could in each case be considered.

- Condition 3A :

If the pavement is cracked after 10 years :
- a Seal (option 3A1); or
- a SAM (option 3A2); or
- a Thin overlay (option 3A3); or
- a SAMI (option 3A4)

- Condition 3B :

If the pavement is in a sound condition after 10 years :
- a do nothing approach (option 3B1); or
- a seal (option 3B2); or
- a thin overlay (option 3B3).

In each case the most economical approach is selected as the best approach. This is done by considering for each action the possible conditions of the pavement after a further 10 years by determining the salvage value costs and probability of occurances as shown in Figure 50 and Table 13. Using this data the PWOC of each alternative for each pavement conditions is now determined.

- Condition 3A :

Pavement condition Cracked

- Option 3A1 :

Remedial option : Seal

PWOC (Seal)

= Initial costs discounted for 10 years + (Σ Probability of occurance x Salvage value costs discounted for 20 years)

$= R6\ 710\ (1{,}08)^{-10} + |0{,}6\ (R26\ 650)(1{,}08)^{-20} + 0{,}3\ (R18\ 600)(1{,}08)^{-20} + 0{,}1\ (R16\ 100)(1{,}08)^{-20}|$

$= R3\ 108 + |R3\ 431 + R1\ 197 + R345|$

$= R3\ 108 + R4\ 973$

$= R8\ 081/km$

- Option 3A2 :

Remedial option : SAM

PWOC (SAM)

= Initial costs discounted for 10 years + (Σ Probability of occurance x Salvage value costs discounted for 20 years)

$= R13\ 420\ (1{,}08)^{-10} + |0{,}2\ (R26\ 650)(1{,}08)^{-20} + 0{,}6\ (R18\ 600)(1{,}08)^{-20} + 0{,}2\ (R16\ 100)(1{,}08)^{-20}|$

$= R6\ 216 + |R1\ 144 + R2\ 394 + R691|$

$= R6\ 216 + R4\ 229$

$= R10\ 445/km$

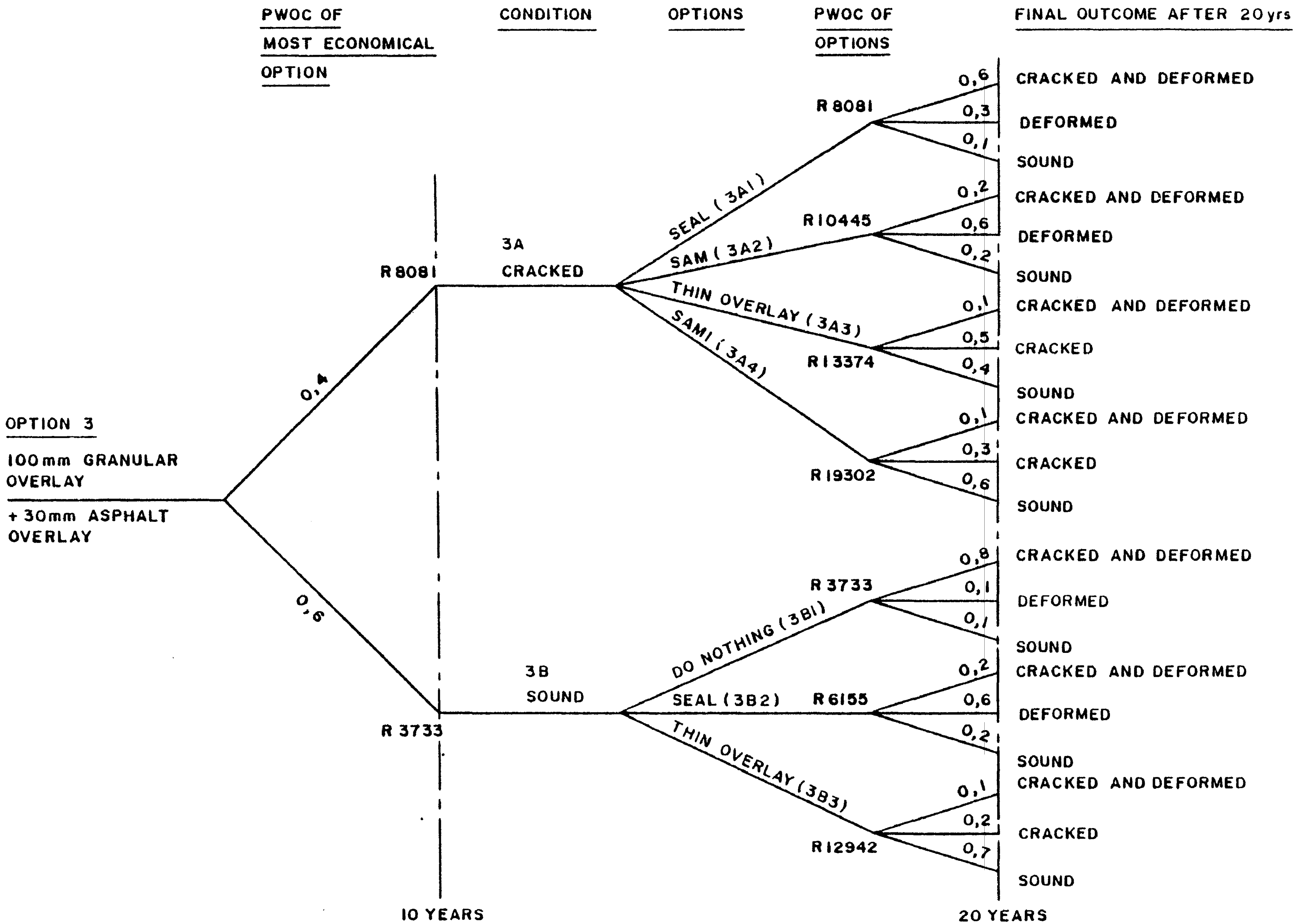

FIGURE 50

DECISION TREE OF OPTION THREE

- 187 -

TABLE 13 - Possible outcomes as option 3 with all relevant costs/km

Initial act (option)	Condition after 10 years	Probability of occurance	Second act (option)		Condition after 20 years	Probability of occurance	Salvage value costs
100 mm Crusher run + 30 mm Asphalt overlay (option 3)	Cracked (3A)	0,4	(3A1)	Seal (R 6 710)	Cracked & Deformed	0,6	R26 650
					Deformed	0,3	R18 600
					Sound	0,1	R16 100
			(3A2)	SAM (R13 420)	Cracked & Deformed	0,2	R26 650
					Deformed	0,6	R18 600
						Sound	0,2 R16 100
			(3A3)	Thin overlay (25 mm) (R25 160)	Cracked & Deformed	0,1	R15 100
					Cracked	0,5	R10 210
					Sound	0,4	R 3 500
			(3A4)	SAMI (25 mm) (R38 580)	Cracked & Deformed	0,1	R15 100
					Cracked	0,3	R10 210
					Sound	0,6	R 3 500
	Sound (3B)	0,6	(3B1)	Do-Nothing (Ro)	Cracked & Deformed	0,8	R18 600
					Cracked	0,1	R13 600
					Sound	0,1	R11 600
			(3B2)	Seal (R 6 710)	Cracked & Deformed	0,2	R18 600
					Deformed	0,6	R13 600
					Sound	0,2	R11 600
			(3B3)	Thin overlay (25 mm) (R25 160)	Cracked & Deformed	0,1	R15 100
					Cracked	0,2	R10 210
					Sound	0,7	R 3 500

- Option 3A3 :

Remedial option : Thin overlay

PWOC (Thin overlay)

= Initial costs discounted for 10 years + (Σ Probability of occurance x Salvage value costs discounted for 20 years)

$= R25\ 160\ (1{,}08)^{-10} + |0{,}1\ (R15\ 000)(1{,}08)^{-20}$
$+ 0{,}5\ (R10\ 210)(1{,}08)^{-20} + 0{,}4\ (R3\ 500)(1{,}08)^{-20}|$

$= R11\ 654 + |R324 + R1\ 095 + R300|$

$= R11\ 654 + R1\ 717$

$= R13\ 373/km$

- Option 3A4 :

Remedial option : SAMI

PWOC (SAMI)

= Initial costs discounted for 10 years + (Σ Probability of occurance x Salvage value costs discounted for 20 years)

$= R38\ 580\ (1{,}08)^{-10} + |0{,}1\ (R15\ 100)(1{,}08)^{-20}$
$+ 0{,}3\ (R10\ 210)(1{,}08)^{-20} + 0{,}6\ (R3\ 500)(1{,}08)^{-20}|$

$= R17\ 870 + |R324 + R657 + R451|$

$= R17\ 870 + R1\ 432$

$= R19\ 302/km$

Therefore, the most economical remedial alternative if the pavement is found to be cracked after 10 years, is a seal (option 3A1). As shown in Figure 50, the PWOC of the seal, i.e. R8 081/km, is now used for calculating the PWOC of Option 3.

- Condition 3B

Pavement condition : Sound

- Option 3B1

Remedial option : Do nothing.

PWOC (Do nothing)

= Initial costs discounted for 10 years + (Σ Probability of occurance x Salvage value costs discounted for 20 years)

$= R0\ (1{,}08)^{-10} + |0{,}8\ (R18\ 600)(1{,}08)^{-20}$
$+ 0{,}1\ (R13\ 600)(1{,}08)^{-20} + 0{,}1\ (R11\ 600)(1{,}08)^{-20}|$

$= R0 + |R3\ 192 + R292 + R249|$

$= R3\ 733/km$

- Option 3B2 :

Remedial option : Seal

PWOC (Seal)

= Initial costs discounted for 10 years + (Σ Probability of occurance x Salvage value costs discounted for 20 years)

$= R6\ 710\ (1{,}08)^{-10} + |0{,}2\ (R18\ 600)(1{,}08)^{-20}$
$+ 0{,}6\ (R13\ 600)(1{,}08)^{-20} + 0{,}2\ (R11\ 600)(1{,}08)^{-20}|$

$= R3\ 108 + |R798 + R1\ 751 + R498|$

$= R3\ 108 + R3\ 047$

$= R6\ 155/km$

- Option 3B3 :

Remedial option : Thin overlay

PWOC (Thin overlay)

= Initial costs discounted for 10 years + (Σ Probability of occurance x Salvage value costs discounted for 20 years)

$= R25\ 160\ (1{,}08)^{-10} + |0{,}1\ (R15\ 100)(1{,}08)^{-20}$
$+ 0{,}2\ (R10\ 210)(1{,}08)^{-20} + 0{,}7\ (R3\ 500)(1{,}08)^{-20}|$

$= R11\ 654 + |R324 + R438 + R526|$

$= R11\ 654 + R1\ 288$

$= R12\ 942/km$

It is seen that the most economical option if the pavement is found to be in a sound condition after a period of 10 years, will be to do nothing (option 3B1). As shown in Figure 50, the PWOC of the do nothing option, i.e. R3 733/km, is now used for calculating the PWOC of Option 3.

The different possible pavement conditions have now been analysed and the PWOC of Option 3 can be determined.

PWOC of Option 3

= Initial costs + (Σ Probability of occurance x PWOC of most economical item)

= R54 150 + |0,4 (R8 081) + 0,2 (R3 733)|

= R54 150 + R5 472

= R59 622/km

≅ R59 600/km

7.2.4.5 Conclusions and Recommendations

The calculated present worth of costs of the different options are:

- Option 1: 60 mm asphalt overlay - R66 400/km
- option 2: 120 mm crusher run + double seal - R49 600/km
- option 3: 100 mm crusher run + 30 mm asphalt - R59 600/km

Therefore, it is recommended that option 2 be used in the rehabilitation of section 2 of the P6/1. This consists of:

- ripping and removing of the existing seals;
- in-situ stabilization of 100 mm of the existing base-course as the new sub-base;
- construction of a 120 mm crusher-run base course; and
- a double seal.

SECTION 8

THE EPILOGUE

8.1 PRACTICAL IMPLEMENTATION OF THE RECOMMENDED REHABILITATION PROCEDURES

8.1.1 Initial assessment of a rural road

8.1.1.1 Review

8.1.2 Initial assessment of a freeway route

8.1.2.1 Gathering of information
8.1.2.2 Evaluation
8.1.2.3 Review

8.2 Consequences of implementation
8.3 Conclusions and recommendations

8. THE EPILOGUE

8.1 PRACTICAL IMPLEMENTATION OF THE RECOMMENDED REHABILITATION PROCEDURE

In the development of the recommended procedure specific attention was given to current South African practice. These are based on the use of equipment and techniques familiar to engineers in this country. Consequently, these procedures in the document are readily applicable here.

As demonstrated in section 7, it is recommended that an investigation into pavement rehabilitation should comprise two reports :

- the initial assessment of the pavement conditions with definite proposals for rehabilitation, including the need for future investigations;
- the detailed assessment, identifying rehabilitation possibilities, with recommendations for optimum rehabilitation measures based on an economic analysis of options.

The reports can be combined, but this is not recommended as the initial assessment gives the road authority advance warning of the situation concerning a particular road, and thereby allowing an early determination of likely needs and relative urgency. Moreover, by dividing the rehabilitation investigation procedure into two parts, the investigating agency can fully assess all available data before proceeding with further testing.

To date, the recommended procedure has only been implemented in its entirety on the P6/1 rehabilitation project (Section 7). However, part of the recommended approach, the initial assessment, has been successfully implemented on various projects.

The usefulness of the initial assessment was demonstrated in the following two investigations undertaken through the NITRR :

- in co-operation with the Transvaal Roads Department an initial assessment was done on a rural road, Route P6/1, from Bapsfontein to Bronkhorstspruit (section 7);
- in co-operation with the Department of Transport (DOT) an initial assessment was done on a freeway, Route N2, sections 10 and 11, near Port Elizabeth[10].

In accordance with the approach recommended, the above investigations were undertaken in order :

to provide :

- a complete record of the behaviour and condition of the pavement;
- information required for further investigation of the sections;

to identify :

- sections requiring no action within the next 5-year period;
- sections exhibiting an obvious surfacing-only problem;
- sections with localized problems;
- sections with probable structural inadequacies identified for further testing (including sections that may require attention within the next 5-year period);

and to suggest or recommend :

- the possible cause(s) and mechanism(s) of distress;
- further testing to verify the possible cause(s) and mechanism(s) of distress;
- appropriate rehabilitation design procedures where applicable.

8.1.1 Initial assessment of rural road P6/1

The report on the initial assessment of route P6/1 is given in section 7.1 as a case study.

8.1.1.1 Review

Apart from the deflection, rut depth and riding quality measurements, two engineers in training spent three days to complete the detailed visual inspection. It took an engineer less than a month to evaluate the results and write the report. This limited effort produced a complete record of the condition of the road including all the processed data and the results of the pavement evaluation.

The evaluation showed that there were several distinct uniform sections. Although the first impression was of a seriously distressed pavement, according to the initial assessment only 50 per cent of the length of the road required immediate attention (including localized problem areas), 41 per cent of the road had a surfacing-only problem where a holding action such as a seal would be adequate, and 9 per cent of the length of the road required no action. It must be noted that the uniform sections identified represent viable lengths of pavement with regard to rehabilitation logistics (minimum lengths of a section - 1 km).

A minimum of further testing was suggested for some sections which might have had structural inadequacies (Axle Weight Analyser and Dynamic Cone Penetrometer survey). For other sections positive identification was made and applicable rehabilitation measures were recommended. From this analysis it was clear that the adoption of a single rehabilitation measure (blanket approach) would have proved inappropriate for structural or economic reasons.

8.1.2 Initial assessment of freeway route : N2/10 & 11

The recommended approach was also demonstrated in a rehabilitation investigation on sections 10 and 11 of National Freeway, Route 2. This road (effective length 150 km) was identified by the PMS system of the DOT as possibly being in need of rehabilitation, and was considered especially suitable for investigation as it presented a wide variety of conditions and problems.

8.1.2.1 Gathering of information

The route consists of 35 km of single carriageway and 58 km of dual carriageway. Various structural combinations, including sections with a cement-treated base (CTB), sections with a bituminous treated base (BTB) and sections with a natural gravel base, occur along the route which was constructed between 1950 and 1974. Part of the freeway traverses the Port Elizabeth urban area, so there is a considerable variation in the volume of traffic between interchanges. The total past accumulated equivalent E80 loads varied between $3{,}5 \times 10^5$ and $1{,}9 \times 10^6$.

The preliminary investigation was facilitated by the availability of the required construction and pavement structure data captured in the Pavement Management System (PMS) data base. The PMS also enabled an assessment of past pavement behaviour to be made as data from a number of condition surveys were available. However, no recent information was available and to update the PMSs, data rut depth, deflection and riding quality surveys were undertaken. This information was supplemented by a detailed visual inspection which gave valuable clues as to the causes and mechanisms of distress in various sections[10].

8.1.2.2 Evaluation

The variations in traffic, pavement structure and age of the different sections of the pavement made the evaluation of this road theoretically much more complicated than that of the P6/1 road. However, the recommended evaluation of individual parameters takes these differences into account, and a graphic presentation of data (as in Figure 54) enabled uniform sections to be identified.

In all, twenty different uniform sections were identified[10]. Eleven were judged to need no further action during the next five-year period, and the other nine sections (total 36 km) were selected for further action. These included sections with localized problem areas requiring urgent major rehabilitation as well as sections where a holding action such as a seal should prove sufficient.

8.1.2.3 Review

The availability of data such as as-built data and deflection, rut depth and riding quality measurements through the existing PMS, meant that little extra field work was necessary to complete the initial assessment of the freeway sections. The systematic procedure adopted enabled an initial assessment of the existing conditions and likely future needs to be made with confidence.

By implementing the recommended procedures, further investigation and possible rehabilitation were reduced by 76 per cent resulting in substantial savings. A further reduction in costs was brought about by the positive identification of the causes of distress in some cases and by suggesting relevant tests for others[10].

8.2 CONSEQUENCES OF IMPLEMENTATION

As a result of the above-mentioned studies the DOT has recognized the potential of the recommended approach and is actively in the process of training personnel to undertake similar investigations. In fact, some investigations are already in progress, i.e. on the N1 in the Transvaal and the N3 in the Orange Free state.

The recent involvement of the Heavy Vehicle Simulator (HVS) in pavement rehabilitation projects required of its personnel to evaluate the roads under investigation to determine the condition of its test sections in relation to the rest of the road. As a result the approach of the initial assessment was demonstrated to them on the N3 section 9 (Warden - Villiers). The initial assessment has now been adopted as the standard procedure for pavement evaluation on HVS pavement rehabilitation projects.

The implementation of the latest methodology, although still on a very limited scale, has had important consequences. The systematic approach to data processing and evaluation has shown that some existing types of equipment and procedures used in South Africa, are inadequate. One such example is the equipment used for continuous mechanical measurement of rut depth. It consists of a number of stationary wheels, representing a straight edge, and one measuring wheel in line with the left rear wheel of the towing vehicle. In this way reasonable accuracy in measuring rut depth is achieved if the left rear wheel of the towing vehicle travels in the middle of the rut. The P6/1 investigation shown that this seldom, if ever, happens in cases where the rutting occurs very close to the edge of a narrow road. On the P6/1, despite the presence of large rut tracks, the measurements showed rutting to be negligible.

Now, as rut depth is one of the main parameters used for the identification of the causes and mechanisms of distress reasonable accuracy of measurement is a prerequisite for successful implementation. The need to improve or replace the existing equipment was therefore demonstrated. The acquisition of new equipment from overseas was considered, but as this would have been costly, it was decided to develop locally a rut depth measuring device, the AURUSCAN (Automatic Rut depth Scanner)*. This instrument consists of a reference frame with three wheels and one measuring wheel which "scans" the pavement. In this way the profile of the road surface is determined every six meters with the measuring wheel following the track of a sine wave between the two reference wheels.

Implementation of the recommended procedure requires information from pavement surveys to be processed in a specific way to enable an effective subsequent analysis and evaluation to be made. Experience to date has highlighted the need to automate the processing of data. The installation of a computerized system with the

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* This instrument is being developed by the NITRR

storage of data on magnetic tape will not only eliminate manual processing but will also add scope to the use of information. For example, in the case of the Lacroix deflectograph this approach will permit calculations of the radius of curvature (RC) for every deflection measurement. The radius of curvature of a pavement is indicative of the quality of the upper structural layers of a pavement and is therefore a valuable source of information. It is likely that the general availability of such data will have a definite influence on the further development or refinement of rehabilitation procedures in South Africa.

8.3 CONCLUSIONS AND RECOMMENDATIONS

In this document a systematic procedure of pavement investigation, evaluation, analysis and resultant rehabilitation design is described. No formal method of analysis or evaluation is excluded, provided the method is relevant to the case and the information obtained through the application of the method can benefit the analysis in terms of its probable influence on the decision on the best rehabilitation strategy to follow. Much emphasis is also placed on the taking of decisions in terms of the economical consequences in finding the best solution to a problem. It is believed that this procedure provides a workable method that can be used to good effect in pavement rehabilitation design.

However, this thesis should be seen as a first effort in producing a comprehensive rehabilitation procedure for South Africa. Implementation and practical use will lead to the identification of shortcomings and the need for improvements in the given procedure. Therefore, refinements and changes can be expected as new knowledge becomes available through further research and input.

The limited use of the procedure has already identified shortcomings in existing survey equipment. Further equipment improvements can be expected to follow. This rehabilitation procedure also lends itself to automation through the computerization of data analysis methods and the writing of applicable computer software has been identified as a future priority.

Although this thesis contains what is believed to be the current most practical applicable methods of pavement rehabilitation analysis and design in South Africa, these methods are in general based an empirical relations developed through years of experience. However, it has been recognized that a preferred method of pavement analysis would be based on the rapid evaluation of pavements using the established elastic and inelastic theory[45]. Although some of the recommended methods contained in the thesis are based on the mechanistic approach to pavement evaluation, the formal use of this procedure, incorporating elastic theory, has to a great extent been excluded.

Various organizations over the world have in recent years developed procedures which formally incorporate the mechanistic approach into pavement evaluation and design[46]. However, these procedures have had limited success in the accurate prediction of pavement behaviour[46] and it is my opinion that more research is required before these procedures can successfully be incorporated into a formal rehabilitation procedure. Therefore, it is recommended that future research be concentrated on the development of methods for the formal incorporation of the mechanistic design procedure, using elastic theory, into pavement rehabilitation design. In doing so, a practical procedure should be developed for the accurate determination of material properties to fully exploit the possibilities of such a system. For example : at present some of these properties can only be determined accurately by measuring in-depth deflections. This is an expensive and complicated procedure, considered to be impractical for general pavement evaluations. Therefore, it is clear that much scope exists for future research which can lead to improvements or supplementary pavement evaluation, analysis and design techniques.

A P P E N D I X 1

THE DETAILED VISUAL INSPECTION

APPENDIX 1 - THE DETAILED VISUAL INSPECTION

1. INTRODUCTION

A detailed visual inspection survey forms an important part of the initial assessment in rehabilitation design. This appendix gives the procedure to be followed in such an inspection.

2. BASIC REQUIREMENTS FOR THE VISUAL INSPECTION

To permit observation of the required detail, it is recommended that the visual survey be carried out at walking speed.

The essential aspects of the inspection should be done by one person who has a comprehensive knowledge of pavement distress and its causes.The remainder of the routine activities, such as measurement taking, may require the assistance of additional staff.

The different modes and types of distress are discussed in detail in TRH6[25], (pages 24 - 37). This document should be studied before the visual inspection is done.

The actual visual inspection should also be carefully planned. A form for recording all the relevant information must be prepared. 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.

In addition, when records of deflection measurements are available, these will be valuable to the observers in indicating pavement areas requiring special attention.

3. RECORDING OF DISTRESS IN PAVEMENTS

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

(a) location of distress
(b) mode and type of distress
(c) degree and extent of distress
(d) position and spacing of distress
(e) pertinent construction details and deficiencies
(f) topographical, geological and vegetational clues to the cause of distress.

3.1 Location of distress

The inspection must provide an accurate record of the position of each particular mode and type of distress that is evident.

3.2 Mode and type of distress

With respect to visible evidence of distress TRH6[25] identifies four main modes. These are:

(i) Deformation or unevenness
(ii) Surface cracking
(iii) Surface disintegration
(iv) Smoothing of the surface texture.

These modes of distress are manifested in several typical ways. These types are listed in Table 14, together with the codes used to identify the types on an inspection form (see Figure 51).

It must be noted that different types of distress within the same mode are generally brought about by different causes. It is therefore important that individual types are 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 14 - 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

3.3 Degree and extent of distress

The degree of distress is an indication of the seriousness of the problem. Explanations of the degrees into which each type of distress is classified, are given in Table 15.

TABLE 15 - Classification of degrees of distress

Class	Description
Degree 1	Distress difficult to discern unaided
Degree 2	Easily discernible distress but of little immediate consequence
Degree 3	Notable with respect to possible consequences
Degree 4	Important with respect to possible consequences
Degree 5	Extreme with respect to possible consequences

More specific criteria for individual distress types are given in TRH6[25] (pages 24 - 37).

The extent of distress can be given as a proportion of either the length or area of the pavement affected.

3.4 Position and spacing of distress

The position of distress is given in relation to the width of a traffic lane. Examples of this are: on the shoulders in the wheelpaths 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.

3.5 Pertinent construction details and deficiencies

Visible construction details in the areas of distress, such as the occurrence of distress in 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.

3.6 Topographical, geological and vegetational clues to the cause of distress

Topographical and geological observations may provide useful indications of insufficient drainage. In addition, drainage problems tend to have a marked effect on the immediate environment. Examples of such indications are given below:

(a) 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.

(b) 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. This will cause water 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.

(c) 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.

4. THE INSPECTION FORM

A typical form for the visual inspection is given in Figure 51. For practical reasons it is suggested that codes be used to identify the different types of distress. On this form the position and extent of distress must be shown graphically, and space is provided for this purpose. Figure 52 gives an example of a completed form.

The example in Figure 52 shows that the following information was gathered during the inspection of the first 150 m:

Longitudinal cracking near the shoulders; degree 3,
km 16,025 - 16,050
Crocodile cracking over the whole width of the lane; degree 4,
km 16,075 - 16,100
Rutting in both wheelpaths; degree 3,
km 16,075 - 16,125
Edge breaking; degree 5,
km 16,010 - 16,050
Pothole near the edge of the road; degree 4,
km 16,080
Patch near the edge of the road; 5m long by 1m wide
km 16,080
Blocked drain; km 16,100
The length km 16,0 - 16,11 is in fill with a maximum height of 1 m.
Lushness of growth of grass and reeds -
km 16,075 - 16,125

940-4-3914a K.T.

FIGURE 51
DETAILED VISUAL INSPECTION FORM

DATE : _ _ _ _ _ _ _ _ _ _ _ ROAD _ _ _ _ _ _ _ _ _ _ _ DIRECTION _ _ _ _ _ _ _ _ _ _ PAGE No _ _ _ _ _ _ _ _ _ _ _

NAME : _ _ _ _ _ _ _ _ _ _ _ LANE : _ _ _ _ _ _ _ _ _ _ _ CHAINAGE _ _ _ _ _ _ _ _ _ _

NB : READ INSTRUCTIONS BEFORE USE

CHAINAGE (km)			
CRACKING	POSITION	ROAD WIDTH	
	TYPE		
	DEGREE		
	SPACING (m)		
DEFORMATION	POSITION	ROAD WIDTH	
	TYPE		
	DEGREE		
	SPACING (m)		
BREAKING-UP OF SURFACE	POSITION	ROAD WIDTH	
	TYPE		
	DEGREE		
SMOOTHING OF SURFACE	POSITION	ROAD WIDTH	
	TYPE		
	DEGREE		
CONSTRUCTION DETAILS AND DEFICIENCIES	POSITION	ROAD	
	CUT / FILL		
TOPOGRAPHY VEGETATION GEOLOGY	POSITION	ROAD	
	TYPE		

FIGURE 52

AN EXAMPLE OF A USED DETAILED VISUAL INSPECTION FORM

DATE: 29/4/82 ROAD: P12/1 DIRECTION: S N PAGE No: 1

NAME: J SMITH LANE: SLOW CHAINAGE: 16,0 - 16,5

NB: READ INSTRUCTIONS BEFORE USE

CHAINAGE (km)		16,0 – 16,1	16,1 – 16,2	16,2 – 16,3	16,3 – 16,4	16,4 – 16,5
CRACKING	POSITION (ROAD WIDTH)					
	TYPE	L C		T		
	DEGREE	3 4		3		
	SPACING (m)			2		
DEFORMATION	POSITION (ROAD WIDTH)					
	TYPE		RU	CO		
	DEGREE		3/3	2		
	SPACING (m)			2		
BREAKING-UP OF SURFACE	POSITION (ROAD WIDTH)	X				
	TYPE	EB PH	PA		PA	EB
	DEGREE	5 4				5
			5x1		10x1	
SMOOTHING OF SURFACE	POSITION (ROAD WIDTH)					
	TYPE				BL	
	DEGREE				3	
CONSTRUCTION DETAILS AND DEFICIENCIES	POSITION (ROAD)					
		BLOCKED DRAIN				
	CUT/FILL	F1	C1		F4	
TOPOGRAPHY VEGETATION GEOLOGY	POSITION (ROAD)	XXX				
	TYPE		GRASS REEDS			

- 208 -

APPENDIX 2

PAVEMENT EVALUATION: CRITERIA AND PROCEDURES FOR USE IN SOUTH AFRICA

APPENDIX 2 - PAVEMENT EVALUATION : CRITERIA AND PROCEDURES FOR USE IN SOUTH AFRICA

1. INTRODUCTION

A prerequisite for optimum pavement rehabilitation design is an effective system for evaluating the condition of a pavement. To achieve this, one must be able to express the condition of the pavement in measurable, and hence comparable, terms.

The assessment of both the serviceability and the structural capacity qualities of a pavement should be covered in an evaluation study. The relative influence and importance of each of these two approaches must be considered in the laying down of minimum performance levels for any one of the measurable parameters. Criteria established in this way, will ensure the identification of structurally inadequate lengths of pavement as well as lengths with an unacceptable riding surface.

The relative importance of each factor depends on the category of road and the parameter under investigation. For example, the more important a road, the more important will be its level of serviceability. This is taken into account by setting higher limiting criteria for serviceability parameters such as riding quality for higher categories of road. At the same time these considerations are not relevant to structural capacity, which has other parameters, such as pavement deflection.

Pavement condition is assessed by both visual and mechanical means. Visual assessments involve recording of visible signs of distress such as cracking, deformation, disintegration, smoothing of the surface, and observing possible causes of distress such as suspected drainage problems.

Various mechanical surveying methods are used on a routine basis in South Africa. Of these, riding quality, rut depth and skid resistance measurements are used for assessing both the serviceability and structural capacity of pavements. Conjointly, deflection and Dynamic Cone Penetrometer (DCP) measurements are used for assessing pavement structural capacity only.

With few exceptions, limited research has been undertaken in South Africa in relating these parameters to the life expectancy of a pavement. However, much experience had 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 relations. The criteria recommended here are based on a study of these relations and other established overseas, and on limits accepted in general practice.

2. BASIS FOR ESTABLISHING EVALUATION CRITERIA

In the past, although individual road authorities used criteria in pavement evaluations, no uniform system existed. The most noteworthy attempt in this direction has been the system developed for the Department of Transport by Rickwood and Curtayne[47], in which some criteria were established, albeit only for National Roads. This work has been used as a basis for the development of the comprehensive set of criteria contained in this thesis.

In establishing these evaluation criteria, the category of road and the pavement structure were taken into account. Furthermore, these criteria are based on an acceptable condition rating.

In this report, three different categories of pavement condition are used:

sound condition - the measured pavement condition is considered good;

warning condition - the measured pavement condition is approaching the minimum acceptable level for a particular parameter;

severe condition - the measured pavement condition is below the minimum acceptable level for a particular parameter.

2.1 Category of road

The road categories applicable in South Africa were established with the publication of draft TRH4[14], and are shown in Table 16.

TABLE 16 - Road categories in South Africa[14]

	Road category		
	A	B	C
Traffic	$E_3 - E_4$ $3\text{-}50\times10^6$ E80/lane	$E_2 - E_3$ $0{,}2\text{-}12\times10^6$ E80/lane	$E_1 - E_2$ $<3\times10^6$ E80/lane
Description	e.g. interurban freeways, major interurban roads	e.g. interurban collectors, major rural roads, major industrial roads	Lightly trafficked rural roads, strategic roads
Importance	Very important	Important	Less important
Serviceability level	Very high	High	Moderate

2.2 Pavement structure

TRH4[14] and Kennedy and Lister[29] identified the following pavement structures, exhibiting different structural characteristics :

Pavements with - cement-treated materials
- bituminous-treated materials
- aggregates exhibiting cementation
- natural gravel or untreated materials.

3. THE USE OF VISUAL ASSESSMENTS IN THE EVALUATION OF PAVEMENTS

For the successful application of a set of meaningful criteria in pavement evaluation, detailed measurements are needed. The visual inspection therefore requires detailed recording of visible distress such as shown in Figure 51.

On this form, distress is recorded according to a scale, allowing the different criteria to be applied accurately. This form is designed for individual pavement rehabilitation assessments and is not recommended for network-level investigations as required in pavement management systems (PMSs).

Criteria for the analysis of recorded data relating to particular distress manifestations are given below:

3.1 Cracking

For the evaluation procedure, cracking from degree C_3 to C_5 as identified in TRH6[25] is considered for various types of distress. This type of deformation is measured as a percentage of a unit length (usually 100 m).

(a) Crocodile cracking, map cracking, block cracking (with a spacing of less than 0,3 m) and combinations thereof

Cracking $<$ X	Sound condition
X $\leq$ Cracking $<$ Y	Warning condition
Y $\leq$ Cracking	Severe condition

Road category	X%	Y%
A	3	12
B	5	15
C	10	20

(b) Longitudinal cracking

Cracking $<$ X	Sound condition
X $\leq$ Cracking $<$ Y	Warning condition
Y $\leq$ Cracking	Severe condition

Road category	X%	Y%
A	40	80
B	40	80
C	50	80

(c) Crack patterns other than that described in a and b and/or combinations of crack patterns

Measured as a percentage of a unit length (usually 100 m)

$$\frac{E_c}{S_c} + \frac{E_d}{0,6} < X \qquad \text{Sound condition}$$

$$X \leq \frac{E_c}{S_c} + \frac{E_d}{0,6} < Y \qquad \text{Warning condition}$$

$$Y \leq \frac{E_c}{S_c} + \frac{E_d}{0,6} \qquad \text{Severe condition}$$

E_d = extent of unsealed cracking as described in (a)
E_c = extent of unsealed cracking other than E_d
S_c = spacing of cracking in E_c

Road category	X%	Y%
A	5	20
B	10	40
C	20	50

3.2 Deformation

Deformation from degree C3 to C5, identified according to TRH6, is considered. This type of distress is measured as a percentage of a unit length (usually 100m).

	Deformation $< X$	Sound condition
$X \leq$	Deformation $< Y$	Warning condition
$Y \leq$	Deformation	Severe condition

Road category	X%	Y%
A	3	12
B	5	15
C	10	20

3.3 Disintegration

(a) Patching

Given that

L_p = % length covered by patching

A_L = Area of patches

$L = L_p + \sqrt{A_L}$

then

$L < X$	Sound condition
$X \leq L < Y$	Warning condition
$Y \leq L$	Severe condition

Road category	X%	Y%
A	5	20
B	10	40
C	20	50

(b) Loss of stone (ravelling)

Degrees C3 to C5, in accordance with TRH6, are considered. This distress is measured as a percentage of a unit length (usually 100 m)

Ravelling < X	Sound condition
X ≤ Ravelling < Y	Warning condition
Y ≤ Ravelling	Severe condition

Road category	X%	Y%
A	20	50
B	20	50
C	40	70

3.4 Smoothing of the surface

Degrees C_3 to C_5, in accordance with TRH6, are considered. This distress is measured as a percentage of a unit length (usually 100 m).

Smoothing < X	Sound condition
X ≤ Smoothing < Y	Warning condition
Y ≤ Smoothing	Severe condition

Road category	X%	Y%
A	20	50
B	30	50
C	40	60

3.5 Drainage problems

Obvious problems such as blocked side drains, lack of drainage facilities and standing water next to the road, should be recorded.

Should these problems occur in addition to any severe or warning conditions of pavement distress, drainage problems are considered severe. If there is no obvious pavement distress, a warning condition should be recorded.

4. MECHANICAL SURVEYS USED FOR PAVEMENT EVALUATION

4.1 Assessment of serviceability and/or structural capacity qualities

4.1.1 Riding quality measurements

In South Africa riding quality is measured with various pieces of equipment such as:

- the PCA roadmeter, which measures riding quality in PSI with values ranging from 1(poor) to 5(good);
- the Linear Displacement Integrator (LDI) a measuring device which measures the displacement of the rear axle in mm per unit length;
- the Photologger, which measures the riding quality in mm per unit length with an accelerometer mounted on the rear axle of the vehicle which contains it.

Correlation studies between the different instruments, as determined by Visser[48], are used in relating PCA measurements to the CPI and Photologger measurements. The 95th percentile values of the measured riding quality over a certain distance (usually 100 m) is considered.

For the PCA roadmeter:

	$RQ > X$	Sound condition
$X \geq$	$RQ > Y$	Warning condition
$Y \geq$	RQ	Severe condition

For the LDI and Photologger

	$RQ < X$	Sound condition
$X \leq$	$RQ < Y$	Warning condition
$Y \leq$	RQ	

	PCA (PSI)		LDI (m/km)		PHOTOLOGGER (mm/km)	
ROAD CATEGORY	X	Y	X	Y	X	Y
A	3,0	2,5	1,5	2,0	130	190
B	2,5	2,0	2,0	2,6	190	260
C	2,0	1,5	2,6	3,3	260	340

4.1.2 Rut depth measurements

Rut depth is measured in millimetres (see Figure 53) and percentile values over a unit length (usually 100 m) are considered.

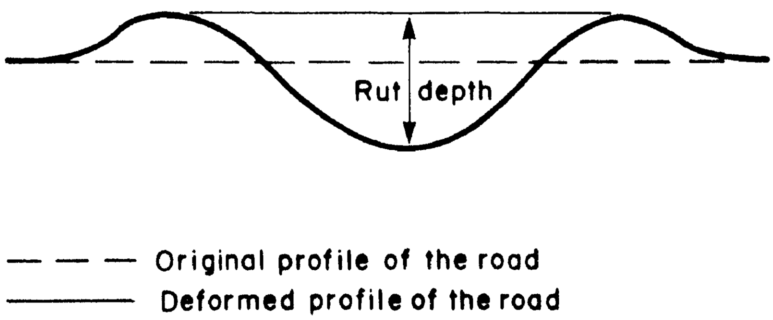


FIGURE 53

MEASUREMENT OF RUT DEPTH

	Rut depth < X	Sound condition
X ≤	Rut depth < Y	Warning condition
Y ≤	Rut depth	Severe condition

Road category	X	Y
A	10	20
B	10	20
C	10	20

4.1.3 Skid resistance measurements

Although several pieces of equipment are available for measuring skid resistance[49], the apparatus most widely used in South Africa is the SCRIM which measures the sideways force coefficient at speeds of 80 km/h (SFC80). The 95th percentile values of the measured skid resistance over a certain distance (usually 100 m) are considered. These criteria are based on a study by Gorden[49].

$$
\begin{array}{llll}
 & SFC_{80} & > X & \text{Sound condition} \\
X \geq & SFC_{80} & > Y & \text{Warning condition} \\
Y \geq & SFC_{80} & & \text{Severe condition}
\end{array}
$$

Road category	X	Y
A	0,50	0,45
B	0,45	0,40
C	0,40	0,35

4.2 Assessment of structural capacity qualities

4.2.1 Deflection measurements

Deflection is measured in millimetres and the 95th percentile values over a certain distance (usually 100 m) are considered. The type of basecourse material can have a marked influence on this parameter (see Kennedy and Lister[29]) and is therefore also included as an additional factor.

$$
\begin{array}{llll}
 & \text{Deflection} & < X & \text{Sound condition} \\
X \leq & \text{Deflection} & < Y & \text{Warning condition} \\
Y \leq & \text{Deflection} & & \text{Severe condition}
\end{array}
$$

	Basecourse material							
	GB-NC		GB-C		BTB		CTB	
Road category	X	Y	X	Y	X	Y	X	Y
A	0,4	0,9	0,3	0,7	0,3	0,6	0,3	0,5
B	0,6	1,0	0,4	1,0	0,4	0,7	0,4	0,6
C	0,9	1,4	0,7	1,2	0,6	0,9	0,5	0,7

GB-NC - Non-cemented granular base

GB-C - Granular base with aggregates exhibiting cementation

BTB - Bituminous-treated base (tar and bitumen)

CTB - Cement-treated base

4.2.2 Dynamic Cone Penetrometer (DCP) measurements

DCP measurements are measured in mm/blow. In order to evaluate the whole pavement it is necessary to determine the DCP structural number of the pavement up to a depth of 800 mm (DSN_{800}) by applying the following formula developed by Kleyn and Savage[20] :

$$\text{Pavement } DSN_{800} = \sum_{n=1}^{x} \frac{h_n}{DN_n} \quad \text{with} \quad \sum_{n=1}^{x} h_n = 800 \text{ mm}$$

where h_n = the thickness of the n-th pavement layer

DN_n = the DCP number of the n-th pavement layer, i.e. the penetration rate of the DCP in mm/blow

x = the number of pavement layers

Moisture can have a marked effect on DCP measurements and must be taken into account. Three conditions are provided for, i.e.

(i) a dry moisture regime or good drainage condition (M1)

(ii) an optimum moisture regime or average drainage condition (M2)

(iii) a wet moisture regime or poor drainage condition (M3).

The 95th percentile values of the calculated DSN_{800} of the DCP measurements taken within a certain length of pavement are taken into account. These criteria are based on evaluation methods developed by the TRD[19, 20, 21].

	$DSN_{800} > X$	Sound condition
$X \geq$	$DSN_{800} > Y$	Warning condition
$Y \geq$	DSN_{800}	Severe condition

	Drainage condition					
Road Category	M1		M2		M3	
	X	Y	X	Y	X	Y
A	380	170	470	210	650	290
B	250	80	310	95	435	135
C	170	65	210	80	290	110

5. EVALUATION PROCEDURE[9, 10]

In the evaluation procedure, decisions should not be based on the measured values of a single parameter. In general, it is necessary to combine all the results and other relevant information. This is achieved by using a form such as that given in Figure 54. This type of visual presentation has a number of advantages, i.e.

- It makes all the relevant information such as the type of structure and a plan of the road, available at a glance;
- it is easy to interpret (darker areas indicate a more severe condition);
- it is compact for use on site; and
- it provides space for recording any other relevant information.

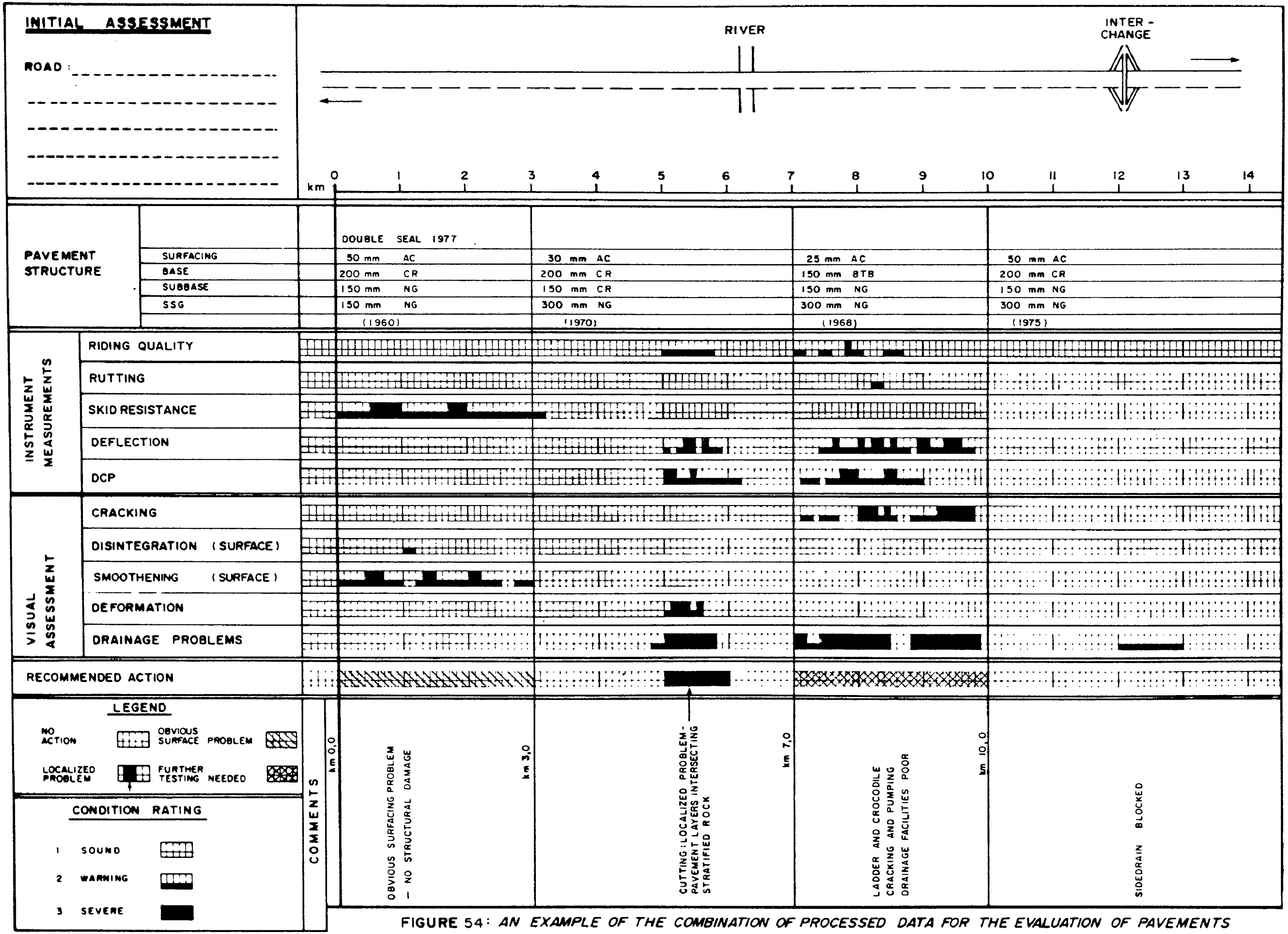


FIGURE 54: *AN EXAMPLE OF THE COMBINATION OF PROCESSED DATA FOR THE EVALUATION OF PAVEMENTS*

A P P E N D I X 3

REHABILITATION DESIGN CATALOGUE

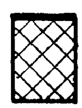
APPENDIX 3 - REHABILITATION DESIGN CATALOGUE

This appendix gives comparable structural rehabilitation alternatives for each rehabilitation category. (Based on Grant[22]) the quality of the material in the different layers should conform to the standards given in TRH14.

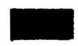
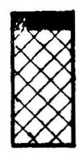
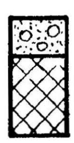
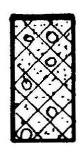
Asphalt overlay as well as different partial reconstruction alternatives are given for each rehabilitation category. The partial reconstruction options are given as guidelines and need not be adhered to strictly. For example, before embarking on the ripping and recompaction of existing pavement layers, the applicability thereof should be considered, taking into account e.g. the existing pavement structure and the quality of the material in the pavement layers.

Permutations with layers of different materials can provide various alternatives to the options given in this appendix and materials such as bitumen-rubber, geo-fabrics and the different techniques and products for crack sealing should also be considered where applicable, to find the most economical alternative.

REHABILITATION CATEGORY A

ASPHALT OVERLAY	SURFACE TREATMENT OR LEVELING COURSE
PARTIAL RECONSTRUCTION (THIN SURFACING + GRANULAR BASE)	 100 mm SURFACE TREATMENT RIP + RECOMPACT

REHABILITATION CATEGORY B

ASPHALT OVERLAY	 40 mm NOMINAL THICKNESS (25 mm MINIMUM THICKNESS)
PARTIAL RECONSTRUCTION (THIN SURFACING + GRANULAR BASE)	 30 mm ASPHALT OVERLAY 150 mm RIP + RECOMPACT
	 50 100  150 mm SURFACE TREATMENT (a) RIP 100 mm (b) IMPORT 50 mm BASE (c) MIX + RECOMPACT

940-4-4382/2 B.D.

REHABILITATION CATEGORY C

ASPHALT OVERLAY	60 mm NOMINAL THICKNESS (45 mm MINIMUM THICKNESS)
PARTIAL RECONSTRUCTION (THIN SURFACING + GRANULAR BASE)	50 100 30 mm ASPHALT OVERLAY 150 mm (a) RIP 100 mm (b) IMPORT 50 mm BASE (c) MIX + RECOMPACT
	AFTER 6 YEARS : 30 mm ASPHALT OVERLAY SURFACE TREATMENT 100 mm RIP + STABILIZE IN PLACE
	AFTER 6 YEARS – 30 mm ASPHALT OVERLAY SURFACE TREATMENT 100 mm BASE MATERIAL (IMPORT) 100 mm RIP + RECOMPACT

940-4-4382/3 B.D.

REHABILITATION CATEGORY D

ASPHALT OVERLAY	90 mm NOMINAL THICKNESS (75 mm MINIMUM THICKNESS)
PARTIAL RECONSTRUCTION (THIN SURFACING + GRANULAR BASE)	30 mm ASPHALT OVERLAY 100 mm BASE MATERIAL (IMPORT) 100 mm RIP + RECOMPACT
	AFTER 6 YEARS: 30 mm ASPHALT OVERLAY SURFACE TREATMENT 150 mm RIP + STABILIZE IN PLACE
	AFTER 6 YEARS : 30 mm ASPHALT OVERLAY SURFACE TREATMENT 150 mm BASE MATERIAL (IMPORT) 100 mm RIP + RECOMPACT

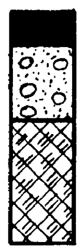
940-4-4382 / 4 B.D.

REHABILITATION CATEGORY E

ASPHALT OVERLAY	115 mm NOMINAL THICKNESS (100 mm MINIMUM THICKNESS)
PARTIAL RECONSTRUCTION (THIN SURFACING + GRANULAR BASE)	40 mm ASPHALT OVERLAY 150 mm BASE MATERIAL (IMPORT) 100 mm RIP + RECOMPACT
	40 mm ASPHALT OVERLAY 150 mm RIP + STABILIZE IN PLACE

940-4-4382/5 B.D.

REHABILITATION CATEGORY F

ASPHALT OVERLAY	 165 mm NOMINAL THICKNESS (150 mm MINIMUM THICKNESS)
PARTIAL RECONSTRUCTION (THIN SURFACING + GRANULAR BASE)	 40 mm ASPHALT OVERLAY 100 mm BASE MATERIAL (IMPORT) 150 mm RIP + STABILIZE IN PLACE

940-4-4382/6 B.D.

A P P E N D I X 4

CONCEPTS IN DECISION THEORY

APPENDIX 4 - CONCEPTS IN DECISION THEORY

The object of this appendix is to give a basic introduction to the concepts of the Bayesian analysis and Decision trees recommended by Curtayne and Servas[3] for use in the economic appraisal of alternative rehabilitation options. The concept of the Bayesian approach is described in detail in references 51 and 52.

With pavement rehabilitation design, considerable doubt as to the condition of the pavement after a certain period exists. This is due to various uncertainties such as the variation in the quality of material, construction techniques and uncertain traffic conditions. To provide for the incorporation of 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 judgement based on experience of conditions and materials to be incorporated in the system with observed data to obtain a balance estimation of the expected cost of an alternative. In the case of any given act (rehabilitation option) the engineer often has some knowledge of the possible outcome or outcomes (conditions) as expected after a certain period of time. He may also expect some outcome to be more likely to occur than others, and therefore could subjectively assign probabilities of occurence to each possible outcome. For example, in Figure 55, illustrated by a branch of a decision tree, a given act α, can have three possible outcomes with values Θ_1, Θ_2 and Θ_3, which it is judged can occur with respective probability of occurence of P_1, P_2 and P_3.

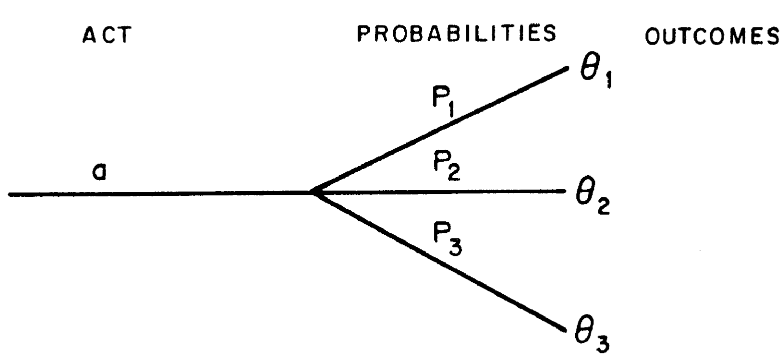


FIGURE 55

EXAMPLE OF A GIVEN ACT WITH THREE POSSIBLE OUTCOMES

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

$$E(\alpha) = P_1\Theta_1 + P_2\Theta_2 + P_3\Theta_3$$

$$= \sum_i P_i\Theta_i$$

with $\sum_i P_i = 1$

In practice one act is generally followed by another act after a period of time (e.g. overlay) followed by a seal when the overlay is cracked after a period of time). The principle illustrated in Figure 55 can now be extended on to include a further act with again possible outcomes as illustrated with the decision tree in Figure 56. The information in Figure 56 is contained in Table 17.

TABLE 17 - The resultant outcomes and acts for two time periods for any one initial act as illustrated in Figure 56

Initial Act	Probability	Outcome	Second Act	Probability	Outcome
α_1	P_{11}	Θ_{11}	b_{111}	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_{1223}	Θ_{1221} Θ_{1222} Θ_{1223}
			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}

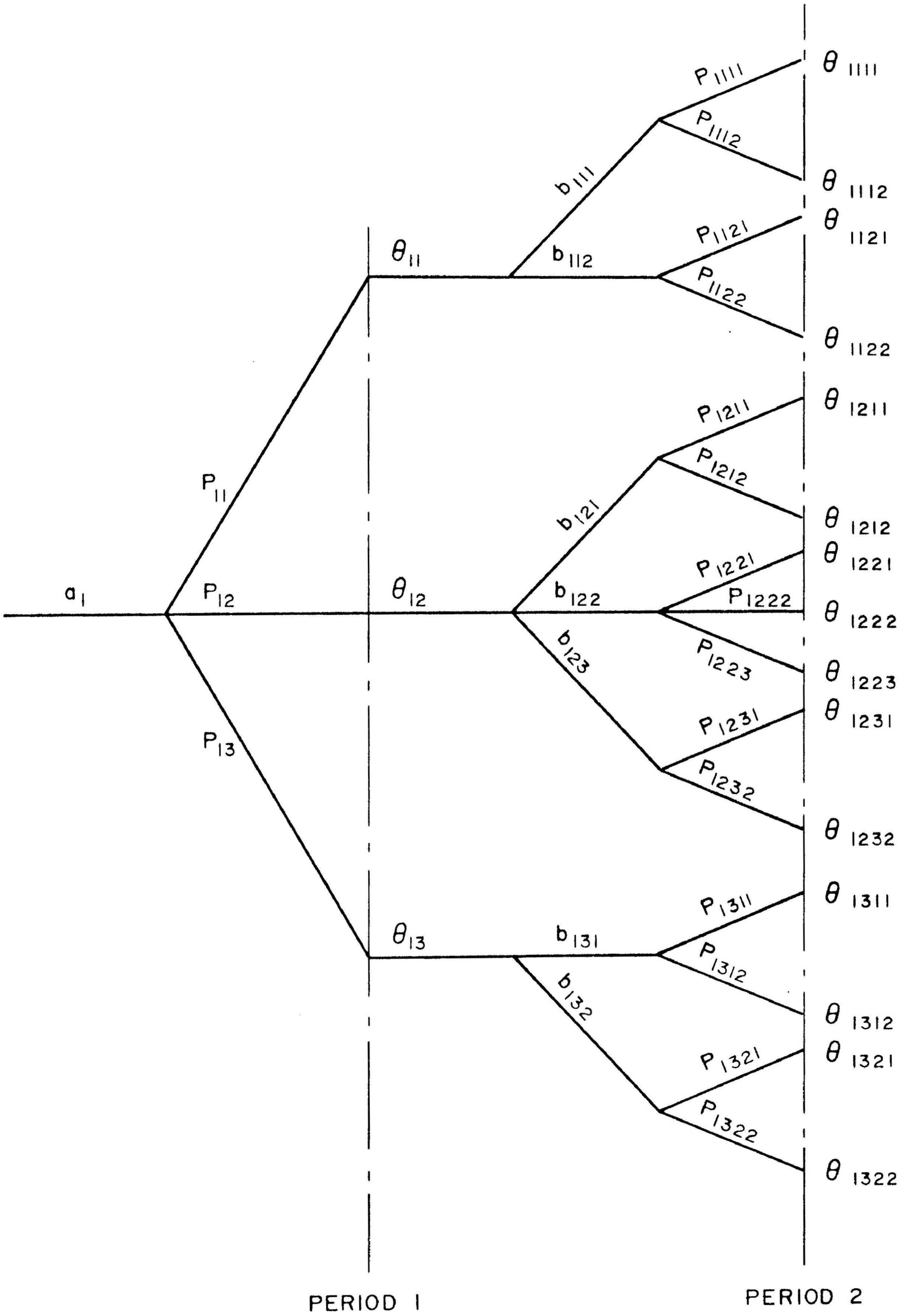


FIGURE 56

DECISION TREE WITH OUTCOMES AND POSSIBLE SECOND ACTS FOR TWO TIME PERIODS

The expected value of the example contained in Table 18 is given by :

$$
\begin{aligned}
E(a_i) = {} & P_{11} \max\{(P_{1111}\ \Theta_{1111} + P_{1112}\ \Theta_{1112}); (P_{1121}\ \Theta_{1121} + P_{1122}\ \Theta_{1122})\} \\
& + P_{12} \max\{(P_{1211}\ \Theta_{1211} + P_{1212}\ \Theta_{1212}); (P_{1221}\ \Theta_{1221} + P_{1222}\ \Theta_{1222} \\
& + P_{1223}\ \Theta_{1223}); (P_{1231}\ \Theta_{1231} + P_{1232}\ \Theta_{1232})\} \\
& + P_{13} \max\{(P_{1311}\ \Theta_{1311} + P_{1312}\ \Theta_{1312}); (P_{1321}\ \Theta_{1321} \\
& + P_{1322}\ \Theta_{1322})\} \\
= {} & \sum_i P_{1i} \max \{\sum_j P_{1i1j}\ \Theta_{1i1j};\ \sum_j P_{1i2j}\ \Theta_{1i2j};\ \sum_j P_{1i3j};\ \Theta_{1i3j}\}
\end{aligned}
$$

with $\sum_i P_{1i} = 1$

$$\sum_i P_{1i1j} = 1; \quad \sum_j P_{1i2j} = 1; \quad \sum_j P_{1i3j} = 1$$

The example illustrated in Figure 56 and Table 18 would represent the analysis of only one rehabilitation alternative. All available options should, in a similar manner, be analysed. The expected value of each applicable option, analysed over the same period of time can then be compared to determine the option with the least expected value.

A P P E N D I X 5

STATISTICAL EQUATIONS

APPENDIX 5 - STATISTICAL EQUATIONS

COEFFICIENT OF VARIATION

The coefficient of variation (C) of a single variable (e.g. deflection) is derived by

$$Cv = \frac{S}{\bar{x}}$$

where, for n observations

$$S \text{ (Standard deviation)} = \sqrt{\frac{\sum_{i=1}^{n} (x_i - \bar{x})^2}{n - 1}}$$

$$\bar{x} \text{ (Mean value)} = \frac{\sum_{i=1}^{n} x_i}{n}$$

CONFIDENCE LIMIT

A confidence limit of a variable Y, given the value x of another variable X, is determined by regression analysis. In general, the scatter of Y about the regression line is dependent on the variable X. For the applications in this document the standard deviation of this scatter is assumed to be proportional to X (e.g. the scatter of rut depth increases linearly with an increase in deflection).

For n sets of observations x_i (e.g. deflections) and y_i (e.g. rut depth) the confidence limit is obtained by:

$$C(Y\ x) = E(Y\ x) + Sy\ x.A$$

where $E(Y\ x) = a + bx$

$Sy\ x = s.x$

$A = p.$ Percentile value of the t-distribution obtained from Table 18 (e.g. for a 95 per cent confidence limit $p = 0{,}95$)

$$a = \frac{\sum_{i=1}^{n} w_i y_i - b \sum_{i=1}^{n} w_i x_i}{\sum_{i=1}^{n} w_i}$$

$$b = \frac{\sum_{i=1}^{n} w_i \left(\sum_{i=1}^{n} w_i y_i x_i\right) - \left(\sum_{i=1}^{n} w_i y_i\right)\left(\sum_{i=1}^{n} w_i x_i\right)}{\sum_{i=1}^{n} w_i \left(\sum_{i=1}^{n} w_i x_i^2\right) - \left(\sum_{i=1}^{n} w_i x_i\right)^2}$$

$$s^2 = \frac{\sum_{i=1}^{n} w_i (y_i - a - bx_i)^2}{n - 2}$$

$$w_i = \frac{1}{x_i^2}$$

FIGURE 57 illustrates the symbols used.

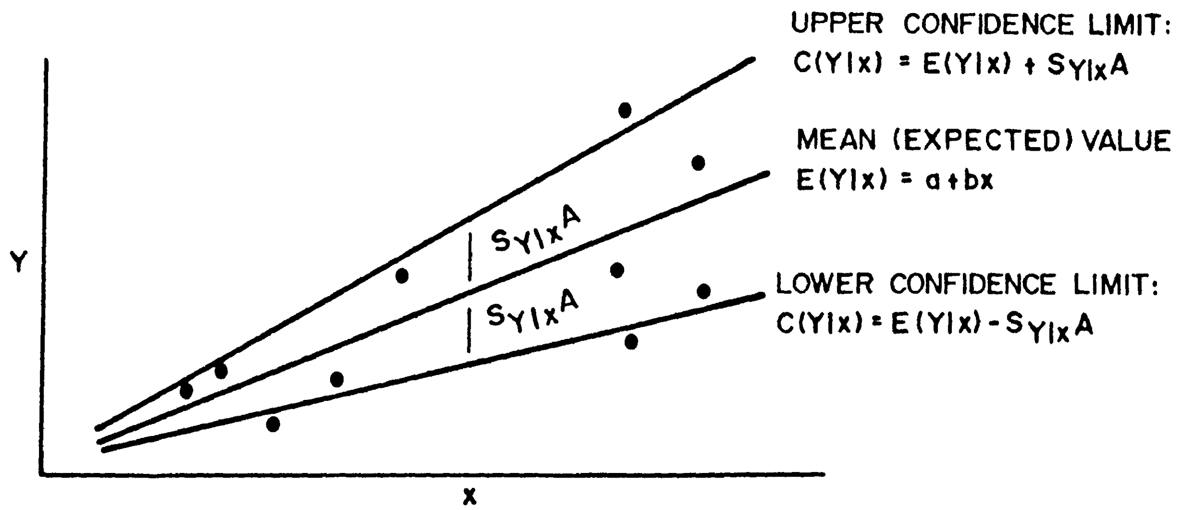


FIGURE 57

ILLUSTRATION OF STATISTICAL TERMS

TABLE 18 p-Percentile values of the t-distribution (After Brownlee, 1960)

f \ p	0,750	0,900	0,950	0,975	0,990	0,995	0,999
1	1,000	3,078	6,314	12,706	31,821	63,657	318
2	0,816	1,886	2,920	4,303	6,965	9,925	22,3
3	0,765	1,638	2,353	3,182	4,541	5,841	10,2
4	0,741	1,533	2,132	2,776	3,747	4,604	7,173
5	0,727	1,476	2,015	2,571	3,365	4,032	5,893
6	0,718	1,440	1,943	2,447	3,143	3,707	5,208
7	0,711	1,415	1,895	2,365	2,998	3,499	4,785
8	0,706	1,397	1,860	2,306	2,896	3,355	4,501
9	0,703	1,383	1,833	2,262	2,821	3,250	4,297
10	0,700	1,372	1,812	2,228	2,764	3,169	4,144
11	0,697	1,363	1,796	2,201	2,718	3,106	4,025
12	0,695	1,356	1,782	2,179	2,681	3,055	3,930
13	0,694	1,350	1,771	2,160	2,650	3,012	3,852
14	0,692	1,345	1,761	2,145	2,624	2,977	3,787
15	0,691	1,341	1,753	2,131	2,602	2,947	3,733
16	0,690	1,337	1,746	2,120	2,583	2,921	3,686
17	0,689	1,333	1,740	2,110	2,567	2,898	3,646
18	0,688	1,330	1,734	2,101	2,552	2,878	3,610
19	0,688	1,328	1,729	2,093	2,539	2,861	3,579
20	0,687	1,325	1,725	2,086	2,528	2,845	3,552
21	0,686	1,323	1,721	2,080	2,518	2,831	3,527
22	0,686	1,321	1,717	2,074	2,508	2,819	3,505
23	0,685	1,319	1,714	2,069	2,500	2,807	3,485
24	0,685	1,318	1,711	2,064	2,492	2,797	3,467
25	0,684	1,316	1,708	2,060	2,485	2,787	3,450
26	0,684	1,315	1,706	2,056	2,479	2,779	3,435
27	0,684	1,314	1,703	2,052	2,473	2,771	3,421
28	0,683	1,313	1,701	2,048	2,467	2,763	3,408
29	0,683	1,311	1,699	2,045	2,462	2,756	3,396
30	0,683	1,310	1,697	2,042	2,457	2,750	3,385
40	0,681	1,303	1,684	2,021	2,423	2,704	3,307
60	0,679	1,296	1,671	2,000	2,390	2,660	3,232
120	0,677	1,289	1,658	1,980	2,358	2,617	3,160
∞	0,674	1,282	1,645	1,960	2,326	2,576	3,090

f = degree of freedom (n - 2)

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