RESEARCH REPORT **RR 91/012**

DEPARTMENT OF TRANSPORT

Characterization of the Elastic Properties of Existing Pavements: State of the Art

April 1993

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SINOPSIS:

'n Wye reeks inligting rakende die bepaling en gebruik van die elastisiteitsmoduli van plaveiselmateriale word hierin bespreek. Dit sluit die volgende in, nl. ondersoek na die sensitiwiteit van plaveisellewe as gevolg van veranderings in elastisiteitsmoduli, opdatering van moduli soos verkry vanaf Multi-Diepte Deflektometer toetsresultate, bondige ondersoek na verskeie terugberekeningsmetodes en 'n literatuurstudie wat konsentreer op toepaslike inligting vir Suid-Afrikaanse toestande. Aanbevelings word gemaak rakende die tegnieke en benaderings wat gebruik word om moduli te bepaal in plek van spesifieke waardes. Nadruk word geplaas op die konsep van "ekspert ingryping" in die ontwerp en evaluering van plaveisels as gevolg van die komplekse interaksie van omgewing, belasting, materiaal en konstruksie faktore wat 'n ten volle gerekenariseerde sisteem op hierdie stadium onmoontlik maak. Verder word daar verwys na areas vir verdere werk asook die insluiting van resente publikasies rakende "nuwe" tegnieke vir materiaaltoetsing in die dokument.

SYNOPSIS:

Various aspects of the determination and use of elastic moduli of pavement materials are herein addressed. Included is an investigation into the sensitivity of pavement life to changes in elastic material parameters, an update of moduli derived from Multi-Depth Deflectometer test results, a brief investigation of different backcalculation techniques and a literature survey concentrating on information considered applicable to South African conditions. Recommendations are made regarding techniques or approaches used to determine elastic moduli rather than giving ranges of values for materials. Emphasis is placed on the concept of "expert intervention" in the design and assessment of pavements due to complex interaction of various factors. These include environmental, loading, material and construction considerations. A reliable, fully automated, computerised system taking all these factors into consideration is at present not feasible. Areas where further work is required are noted and recent publications on some of the "new" techniques of material testing are included.

TREFWOORDE:
KEYWORDS: Elastic modulus, mechanistic pavement design, flexible pavements, rigid pavements, back-calculation, material testing, in-situ testing, laboratory testing

VERSLAGKOSTE:
REPORT COST:

DISCLAIMER

The views and opinions expressed in this report are those of the author and do not represent Department of Transport Policy.

This report was reviewed by:

Dr E Horak
Dr P J Strauss

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1. MOTIVATION FOR RESEARCH

The uncertainty in the profession regarding the selection or derivation of appropriate effective elastic moduli for input in the South African mechanistic pavement design procedure necessitates an update of data used therein. At present various methods of testing may be used in the field and/or the laboratory for derivation of elastic parameters for different material types. Couples to this, various types of elastic moduli can be determined or defined from the same test results, (for instance dynamic, effective and static moduli) which may cause confusion. In addition, a great deal of information is available from both literature and Heavy Vehicle Simulator (HVS) test data which has not been compiled and ought to be made available to assist the practitioner in obtaining and using suitable design values.

The document is essentially a "State of the Art" report describing how elastic moduli are obtained and used in the context of South African pavement engineering. It is not a complete answer for mechanistic design problems but does nevertheless highlight various important aspects of South African pavement engineering and gives additional updated information on elastic parameters.

2. RESEARCH PROGRAMME

The research was carried out slightly differently to that originally planned due to the fact that as work proceeded, various (hitherto) secondary aspects became more important. In addition it was found difficult to divide the investigation into categories such as "Review of laboratory procedures to determine elastic properties" without taking into consideration the context within which test results would be used.

A sensitivity analysis was initially carried out to investigate differences in predicted pavement life due to variations in elastic properties of materials in different layers. Structures taken from the catalogue of designs in TRH4[1] were used for the analysis.

Considerable effort also went into finding and obtaining various recent local and international references relating to elastic properties of road building materials. Many of the references, however, gave information that was often very site specific and difficult to correlate to typical pavement conditions in South Africa. There was a lack of information where material test results, appropriate test analysis techniques and resultant pavement performance were related. Information on local pavement engineering techniques was subsequently obtained, where (by definition) South African conditions are taken into consideration.

Current and Heavy Vehicle Simulator[2] (HVS) test results that were carried out subsequent to 1983 (when the document RP/19/83[3] was published) were processed further (where necessary) to give additional information on values of effective elastic moduli.

To supplement the mechanistic analysis of the HVS test data where limited material classification was carried out, test pits were opened at various sites and material characterisation carried out by means of in situ and laboratory tests. In some cases this was found necessary due to limited material classification.

To further investigate and summarise information appropriate to road designers in South Africa, interviews were arranged with engineers in consulting firms noted for their involvement in pavement design. This aspect of the research yielded some very valuable information and also highlighted several areas requiring further attention.

Following interviews with certain consultants additional pavement analyses were carried out *in situ* and on the computer using HVS data to correlate and check the compatibility of techniques to existing transfer functions, failure criteria and general approaches.

Finally, this document was prepared with the aim of synthesising available information into guidelines and recommendations.

3. INTRODUCTION

Whereas most road designers in South Africa use the mechanistic design approach to some extent, there are limited guidelines available that define appropriate moduli that can be confidently used in design. This is due in part to the fact that moduli of most materials vary depending on environmental and loading factors. These can include stress stiffening or softening, strain hardening or softening, daily or seasonal temperature ranges and moisture susceptibility. Work has therefore been carried out on behalf of the Department of Transport to investigate these problems further and to provide guidelines to the profession to assist design engineers.

The project was initially intended to have been more orientated towards a literature survey with the inclusion of recent Heavy Vehicle Simulator (HVS) test data. As work progressed, however, it became apparent that although there is a wealth of published information both locally and internationally concerning elastic moduli, much of this information is site specific and not easily applicable to general cases. This being the case it was decided to concentrate on the immediate South African problems, approaches and techniques. Accordingly, as already indicated, interviews were held with a number of local consulting engineering companies involved in road design and rehabilitation.

The next aspect that became apparent in the investigation was that to meaningfully discuss, use or measure elastic moduli, the entire design and construction process must be borne in mind. For example, to predict expected pavement life a linear elastic computer analysis is typically used to give stresses and strains which are then input into transfer functions to predict the number of standard 40 kN dual wheel loads (E80s) that can be applied to the pavement before reaching some critical state. Moduli used for this input must be compatible with the techniques used for derivation of the transfer functions necessary for converting deflections, stresses and strains to pavement capacity (number of E80s). This is so due to most of the transfer functions being model-dependant.

In addition to the desk studies and interviews mentioned above, field work was carried out to augment information gained from HVS test data, principally for better material characterisation. Also, to investigate the suitability of certain techniques for analysis of existing road pavements, various methods of analysis of field data were used with field data as indicated in Appendix G.

The report is a synthesis of the information from the sources described above. It is a state of the art synopsis of the characterisation and use of elastic material properties in pavement design in South Africa. From the information thus far obtained it is apparent that there are still many areas of uncertainty due to the inherent variability of materials, pavement environments and the effects of wheel loads. Therefore what the report attempts to achieve (*inter alia*) is to make designers more aware of the relative importance of certain aspects of pavement investigation and design processes and to give some guidelines for the derivation and use of elastic parameters of pavement materials.

3.1 TERMINOLOGY

Throughout the main body of the report the term "elastic modulus" refers to the "effective" elastic modulus, i.e. the modulus measured under a dual wheel load of 40 kN (corresponding to an axle load of 80 kN). This is denoted as E_e. In Appendix B (summaries of "relevant" technical papers), terminology may vary. Therefore where "elastic modulus" is mentioned in Appendix B, the meaning should be taken in context of the article wherein it is found.

4. DEFINITION OF THE PROBLEM AND APPROACH

To investigate the significance of variations in elastic parameters, the South African mechanistic method[5] was used to analyse selected pavement structures taken from the TRH4 catalogue. Ranges of elastic moduli and Poisson's Ratios of various materials used in the analysis were taken from literature[3] and used with the multi-layer linear elastic programme ELSYM5. It is appreciated that there are many possible combinations of elastic parameters for any given structure but the values used were considered fairly representative and suitable for giving an indication of the sensitivity of pavement life to variation in elastic parameters.

The governing mechanist parameters/failure criteria were as given in Table 4.1.

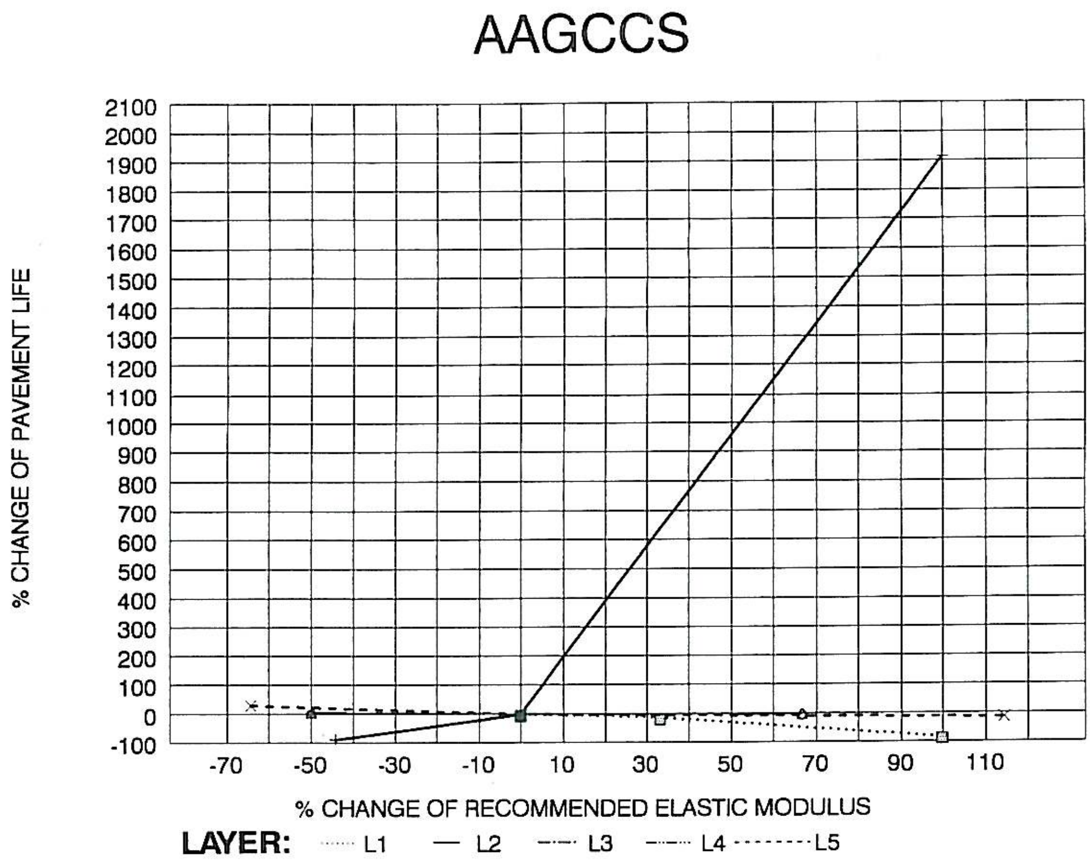
Table 4.1: Mechanistic failure criterion

MATERIAL	FAILURE CRITERIA
Asphalt	Fatigue (tensile strain at bottom of layer).
Cemented	Effective Fatigue (tensile strain at bottom of layer) and crushing at top of layer.
Granular	Factor of Safety (against shear failure - middle of base layers).
Subgrade	Rutting (vertical strain at top of layer)

A problem experienced during the analysis was that of the calculation of negative stresses in granular materials (the magnitude of which violated the material model) due principally to linear elastic assumptions of material behaviour. To deal with this an algorithm was used developed by Raad and Figueroa[6] which helped resolve some of the problems by stress adjustment to maintain the assumed Mohr-Coulomb failure criteria.

A summary of the analysis is given in Figures 4.1 to 4.7 with detailed results in Appendix A. The technique and terminology used is explained using a simple example given at the beginning of the appendix. The influence of changes in elastic moduli on pavement life is shown as "percentage change in elastic moduli" versus "percentage change in pavement life" in Figures 4.1 to 4.7, for the seven selected pavement structures. The figures indicate how important the accurate determination of elastic moduli is for meaningful pavement life prediction. In addition, Figure 4.7 shows how Poisson's ratio can affect pavement life. Furthermore, it is seen that variations of similar magnitude of elastic parameters can have quite different effects on calculated pavement lifes. Considerable time was spent in attempting to derive general rules for successive ratios of moduli and (modulus)*(layer thickness), but due to the number of variables and the limited number of structures used it was not conclusive. Some general guidelines are, however, apparent when the different failure modes are carefully considered. For instance, a brittle, stiff (high elastic modulus) layer should not overlie a relatively flexible (low elastic modulus) material. If this is the case the upper material can fail quickly through the tensile strain/fatigue mode. An example of this is seen in Figure 4.1 where a relatively small increase in stiffness of the granular base directly beneath the asphalt surfacing results in a large increase in pavement life (asphalt being the critical layer in this case). Similar features are seen in Figures 4.2 to 4.7 with different degrees of severity. The concept of pavement balance[7] is here worthy of note. Good pavement balance is where stiffnesses of successive pavement layers are such that there is no excessive build-up of stress between any two layers and strains are compatible. De Beer[7] suggests that a ratio of 2 for successive moduli gives optimum results. This idea should perhaps be further investigated and developed. It should also be noted that the failure mode for light, lightly cemented base pavements is typically crushing. This mode of failure is dependent on the ratio of applied stress (typically tyre pressure) to the unconfined compression strength of the cemented material.

CALCULATED CHANGES IN PAVEMENT LIFE WITH CHANGES IN ELASTIC MODULI.



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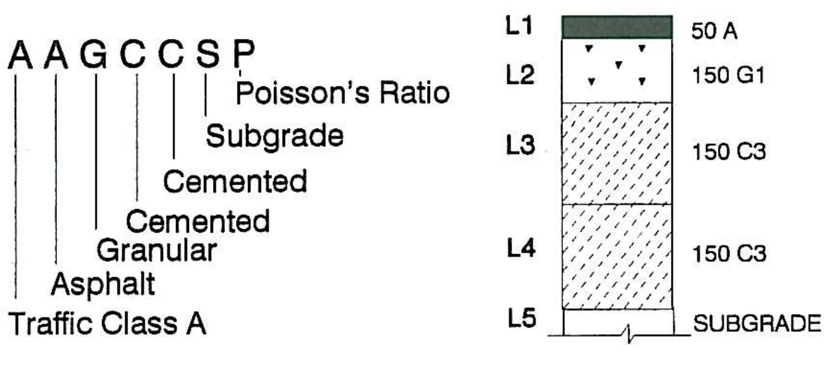
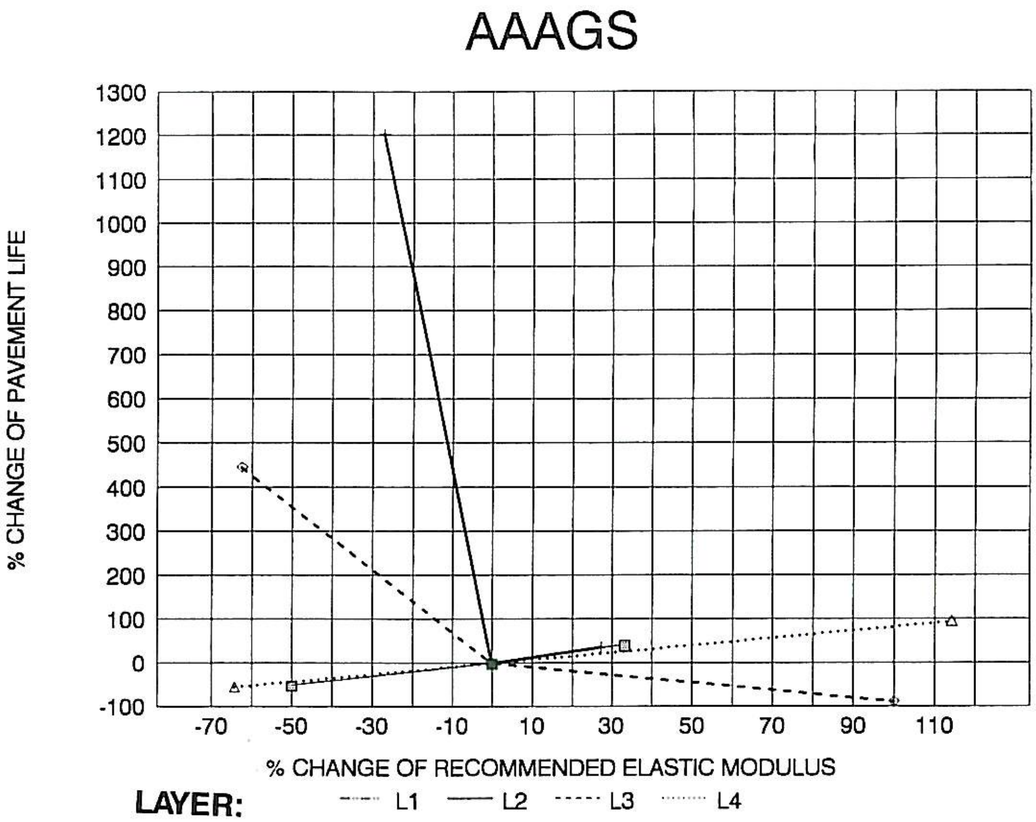


FIGURE 4.1

CALCULATED CHANGES IN PAVEMENT LIFE WITH CHANGES IN ELASTIC MODULI
PAVEMENT AAGCCS



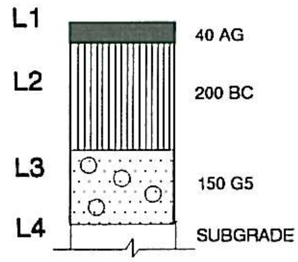
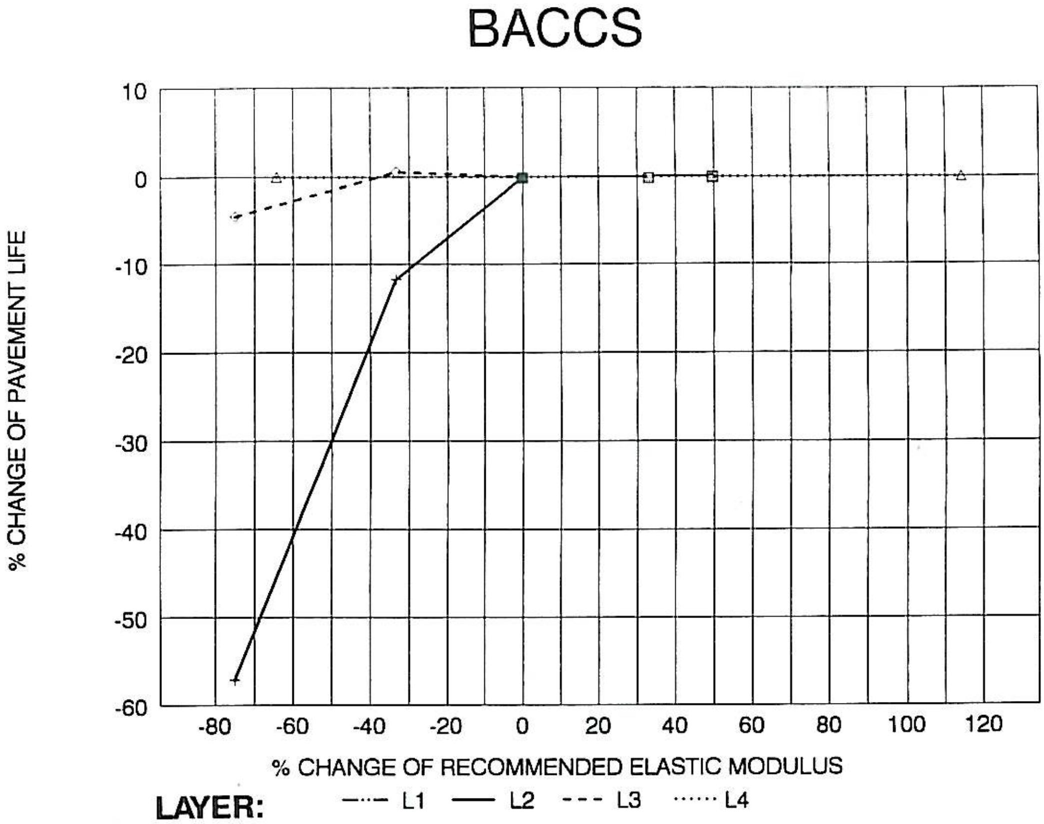
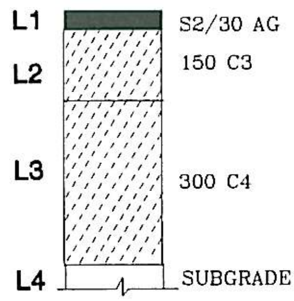


FIGURE 4.2

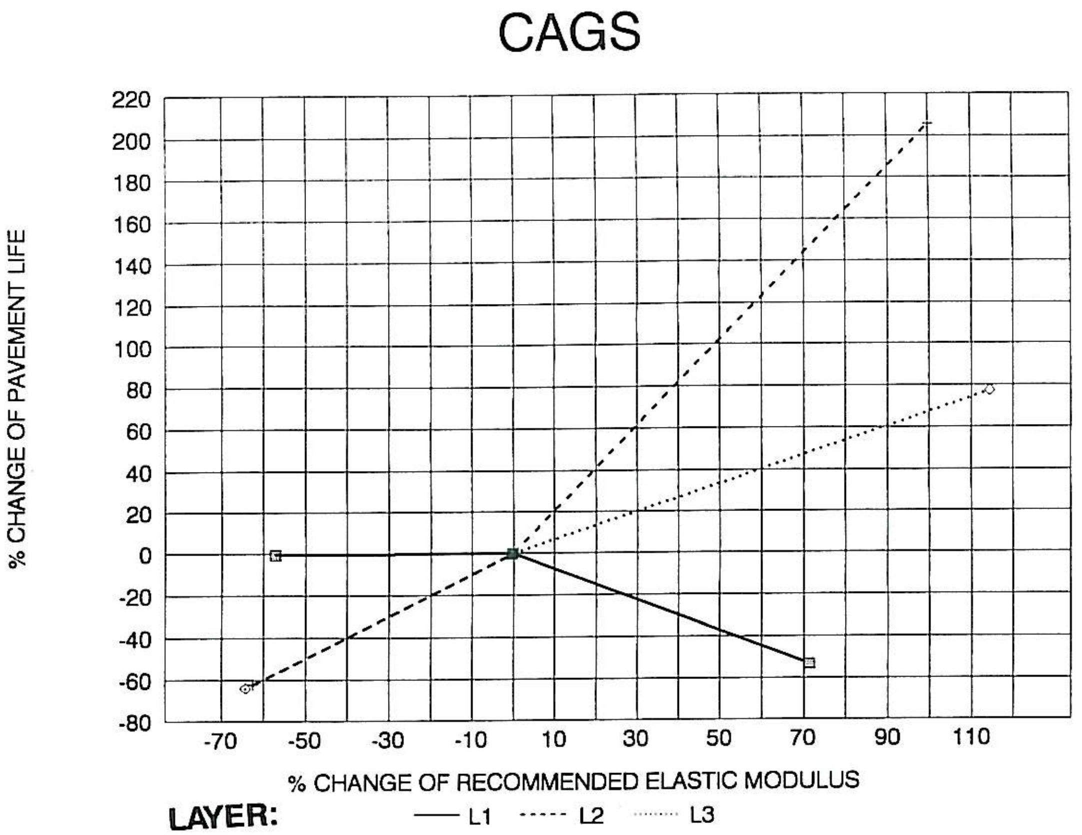
CALCULATED CHANGES IN PAVEMENT LIFE
WITH CHANGES IN ELASTIC MODULI
PAVEMENT AAAGS

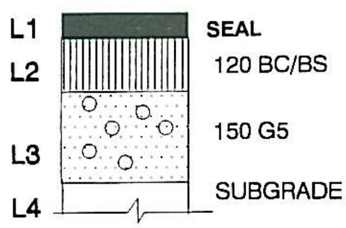




CALCULATED CHANGES IN PAVEMENT LIFE WITH CHANGES IN ELASTIC MODULI.

FIGURE 4.3
CALCULATED CHANGES IN PAVEMENT LIFE WITH CHANGES IN ELASTIC MODULI
PAVEMENT BACCS





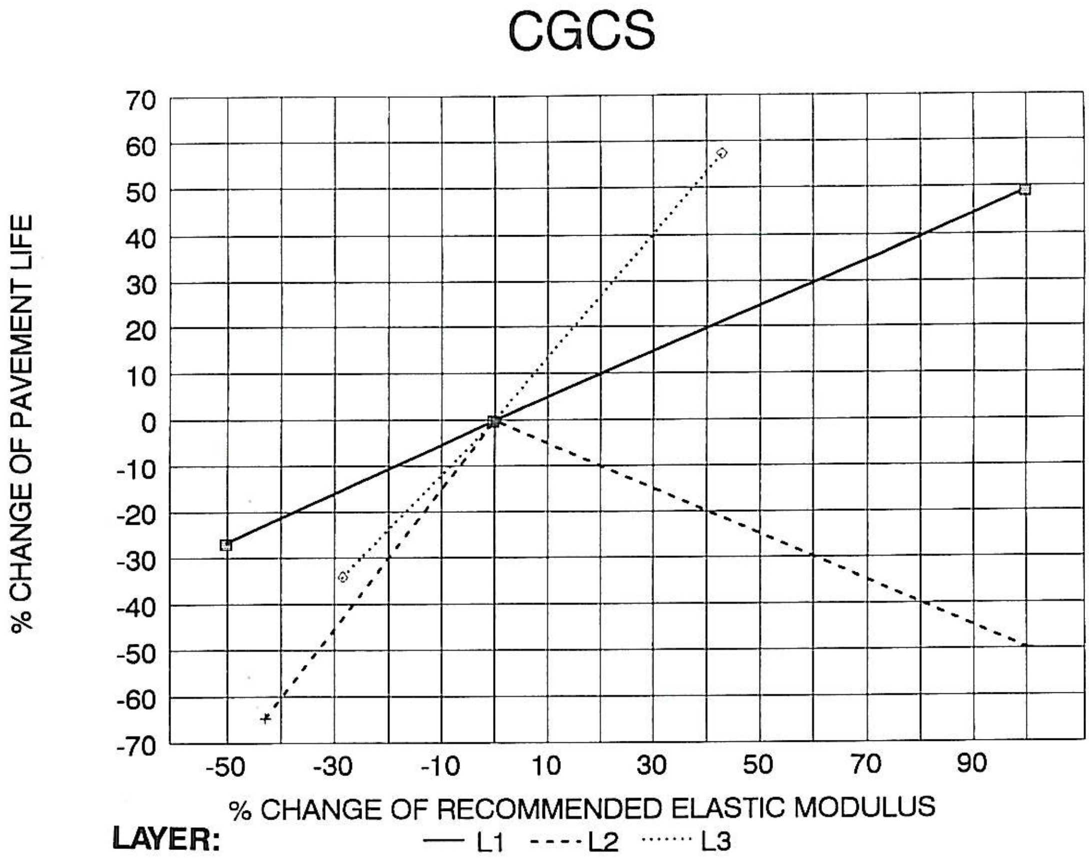
CALCULATED CHANGES IN PAVEMENT LIFE WITH CHANGES IN ELASTIC MODULI.

FIGURE 4.4

CALCULATED CHANGES IN PAVEMENT LIFE WITH CHANGES IN ELASTIC MODULI

PAVEMENT CAGS

CALCULATED CHANGES IN PAVEMENT LIFE WITH CHANGES IN ELASTIC MODULI.



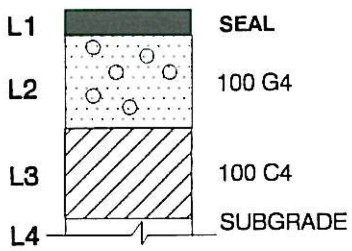
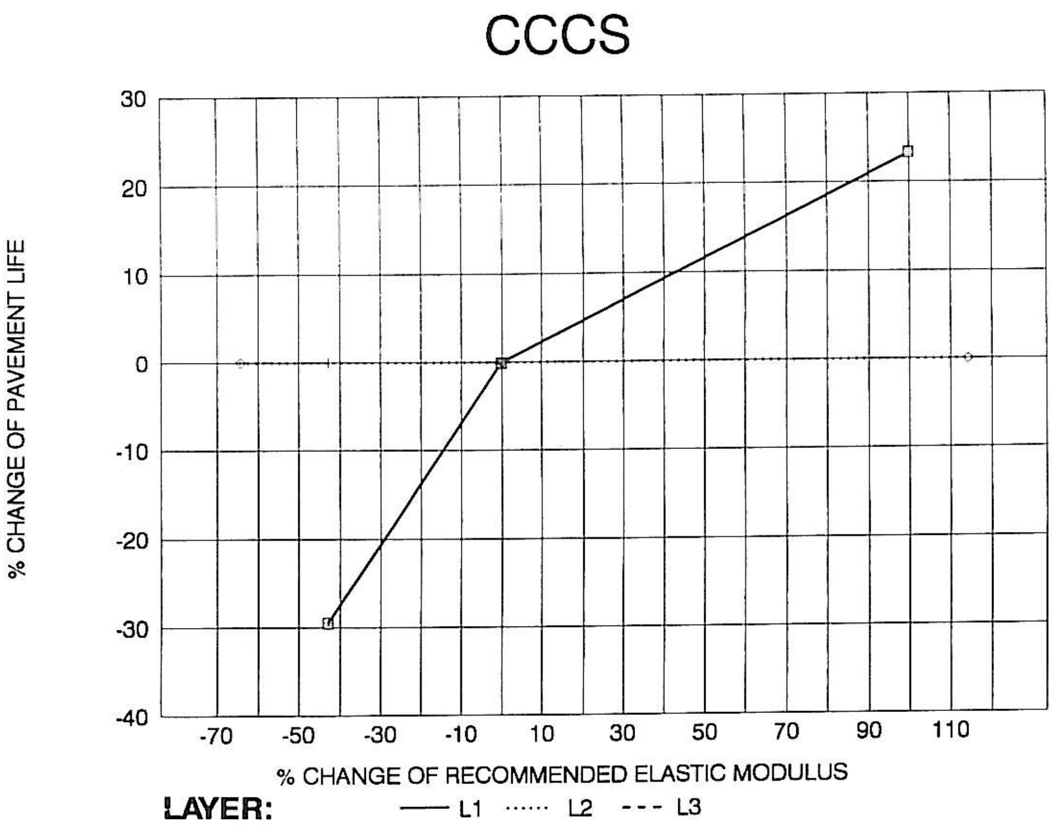


FIGURE 4.5
CALCULATED CHANGES IN PAVEMENT LIFE WITH CHANGES IN ELASTIC MODULI
PAVEMENT CGCS



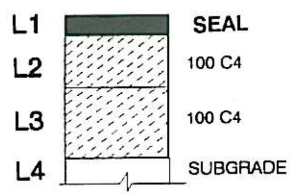


FIGURE 4.6

CALCULATED CHANGES IN PAVEMENT LIFE
WITH CHANGES IN ELASTIC MODULI
PAVEMENT CCCS

CALCULATED CHANGES IN PAVEMENT LIFE WITH
CHANGES IN POISSON'S RATIO.

AAGCCSP

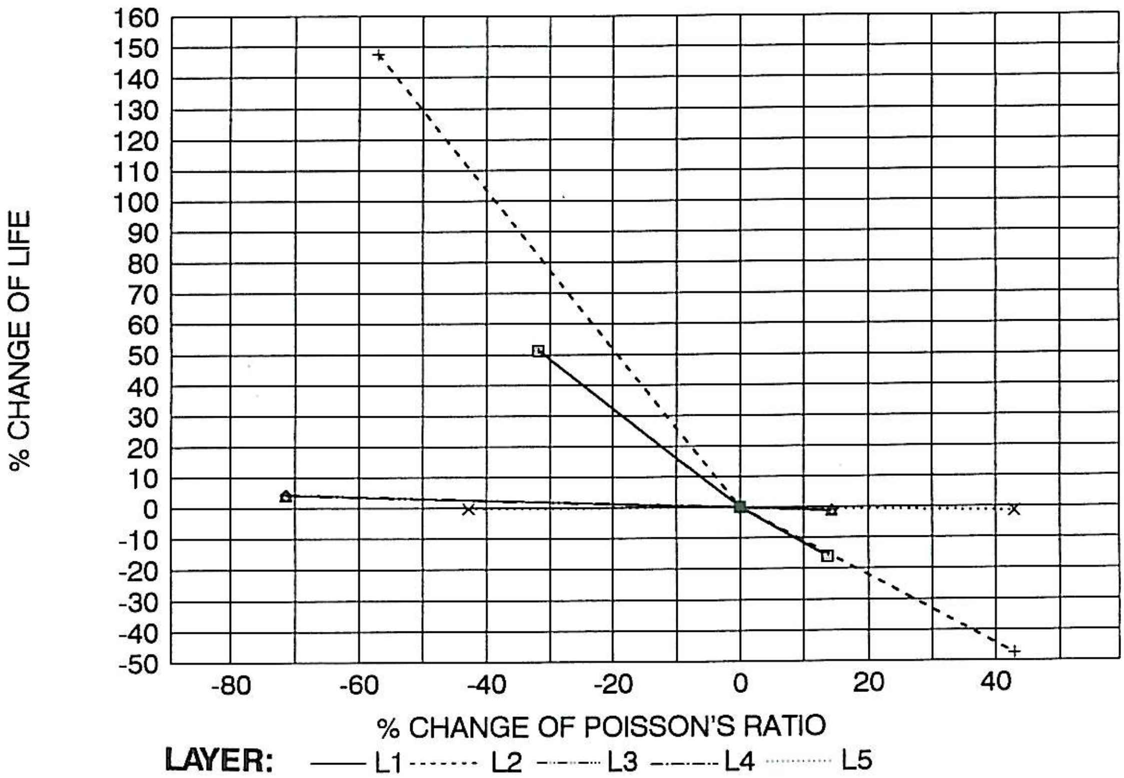


FIGURE 4.7

CALCULATED CHANGES IN PAVEMENT LIFE
WITH CHANGES IN POISSON'S RATIO
PAVEMENT AAGCCSP (See also Fig 1)

AAGCCSP%

5. SUPPLEMENTARY PROCESSING OF HVS DATA

During the course of the investigation load-deflection data from HVS tests over the past 8 years was further analysed and values of E_e derived. These values compliment those found in the tables of recommended elastic moduli given in RP/19/83[3] and Appendix B (item B.5.2). Due to the need for better material characterisation, it was found necessary to go to some of the earlier HVS test positions, take representative samples and in some cases do additional field tests (for example Dynamic Cone Penetrometer- (DCP[7]) and PENCEL Shear Pavement Pressuremeter- (PSPP[9]) tests). Materials samples were tested in the laboratory using some of the procedures recommended in TRH14[10] and THM1[11].

Details of the information obtained from additional analysis of the HVS test results are given in Figures 5.1 to 5.11. Note that as moduli change throughout a pavement life, initial values (at 10 HVS wheel load repetitions - N10) and final HVS reading values (NFinal) are given for guidance. The manner and rate of change of these values varies from pavement to pavement depending on a number of factors (see below) and their interaction.

The following three paragraphs are given as a guide to assist the reader in accessing the information presented in Figures 5.2 to 5.11:

Elastic moduli calculated from multi-depth deflection (MDD) readings using linear elastic theory are given for different Heavy Vehicle Simulator (HVS) test sites in Figures 5.2 to 5.11. Although moduli are generically represented in the graphs, (for example all asphaltic materials are plotted together in Figures 5.2 and 5.3), they are not equal in magnitude. To assist the reader in interpreting the data presented for possible application in practice, the pavement structures from which test results were derived have been given in Figures 5.12 to 5.25.

A key giving the link between the site (and test number) and the figure given on the X-axis in Figures 5.2 to 5.11 is given in Figure 5.1.

To obtain the pavement structure relating to moduli derived from HVS site number 10 in Figure 5.2, for example, Figure 5.1 (the "key") shows that these test results were obtained from the Mariannhill test site, HVS test No 215A3. The pavement structure is given in Figure 5.13. Further details on the history of the change of elastic moduli during the HVS tests are available at the Division of Roads and Transport Technology (DRTT) at the CSIR.

It is important to appreciate that the magnitudes of elastic moduli can change with the number of traffic loads and their intensity (thus inducing stresses and strains within the material), moisture content and temperature. It is therefore difficult to give recommended values for materials that will be within 10 or even 20 per cent of their effective in-situ values. In this regard the effects of possible "mis-estimation" of pavement lives has already been indicated in Figures 4.1 to 4.7.

Generally it appears that for cemented materials the latest data lies on the lower end of the ranges given in RP/19/83[3]. Note, however, that with assessment of existing pavement materials the number of variables (such as material variation, environmental influences, traffic history, means of investigation and interpretation of data), makes it difficult to correlate and update information in RP/19/83[3].

The significant effect of changes of Poisson's Ratio on predicted pavement life is shown in Figure 4.7. The measurement of this material parameter is usually restricted to triaxial testing but with the development of the K-mould (see 7.2 and Appendix D), design values can now be obtained cost effectively.

'X-AXIS No.'	KEY	SITE AND HVS No.
5		RICHMOND - 3 SISTERS 342A2
10		MARIANNHILL 215A3
15		MARIANNHILL 217A3
20		MARIANNHILL 218A3
25		MALMESBURY A2
30		VREDE - MEMEL 267A2
35		VREDE - MEMEL 266A2
40		VREDE - MEMEL 268A2
45		HARRISMITH - VAN REENEN A3
50		WARDEN - VILLIERS 255A3
55		WARDEN - VILLIERS 252A3
60		UMGABABA
65		UMKOMAAS
70		BULTFONTEIN 339A4
75		BULTFONTEIN 306A4
80		BULTFONTEIN 307A4
85		BULTFONTEIN 308A4
90		BULTFONTEIN 309A4
95		ROOIWAL 337A4
100		ROOIWAL 338A4
105		WELKOM 356A2
110		WELKOM 357A2
115		WELKOM 363A2

FIGURE 5.1

KEY SITE AND HVS TEST NO

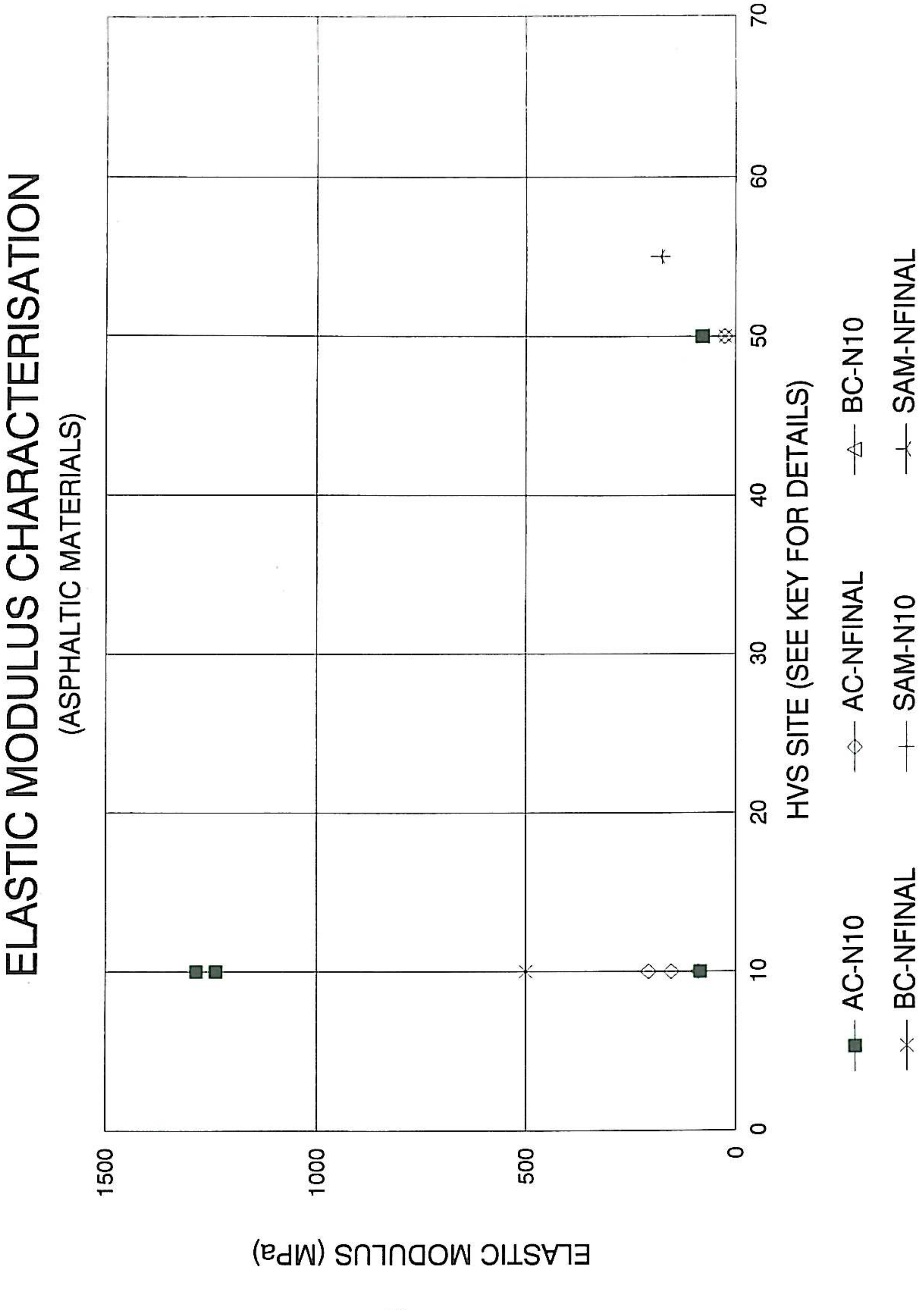
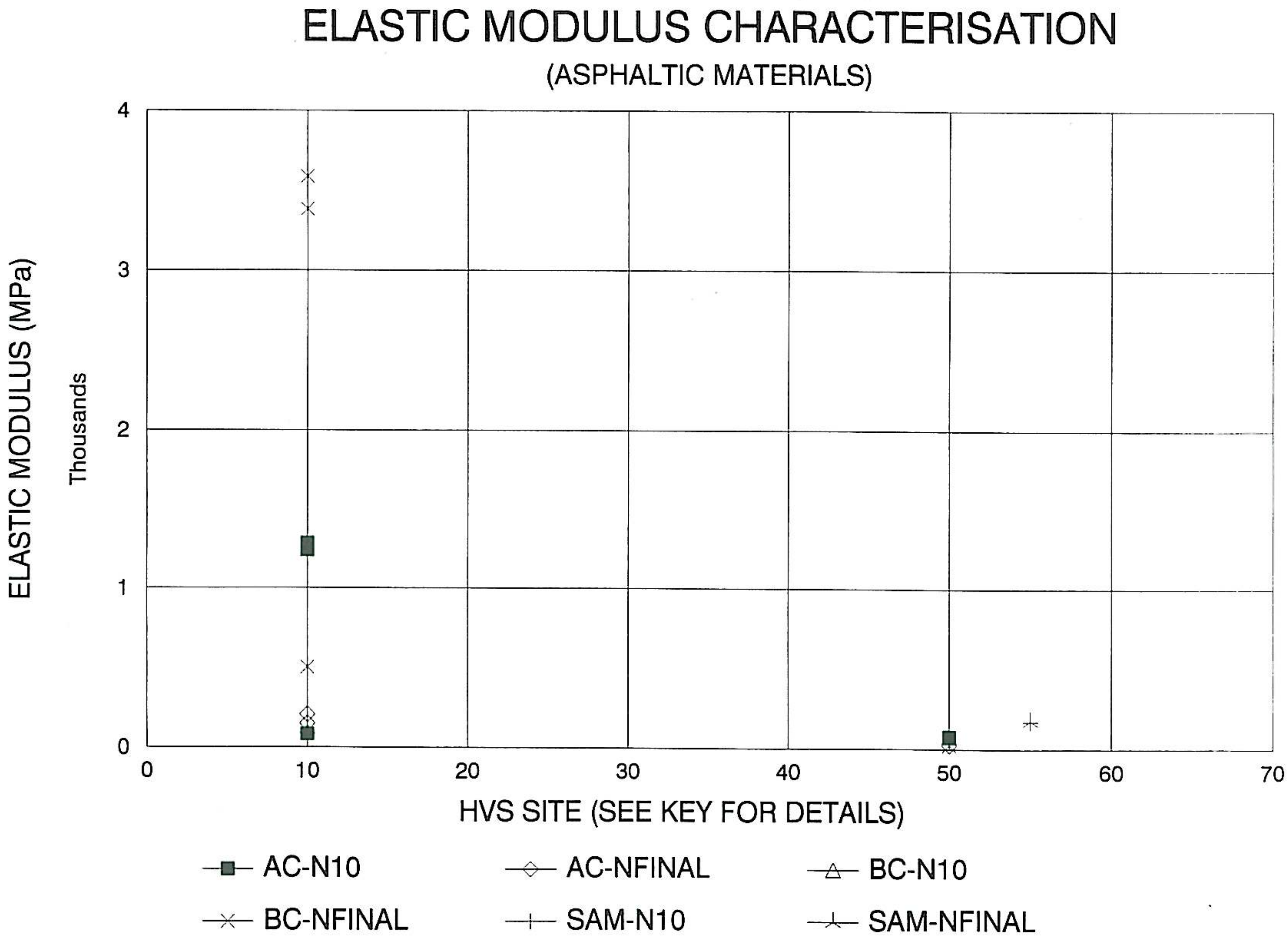


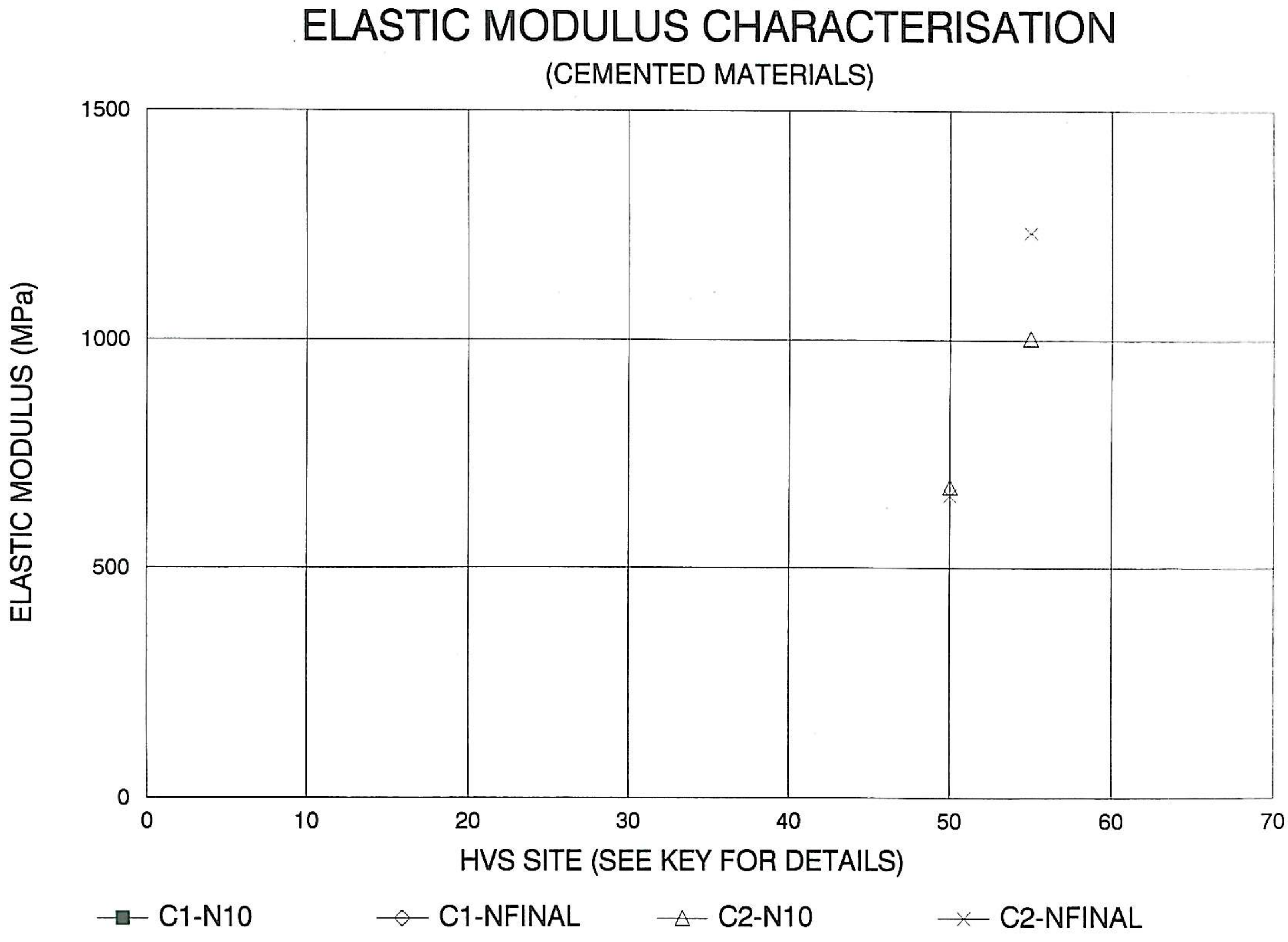
FIGURE 5.2

EASPH.DRW



EASAUTO.DRW

FIGURE 5.3



EC1C2.DRW

FIGURE 5.4

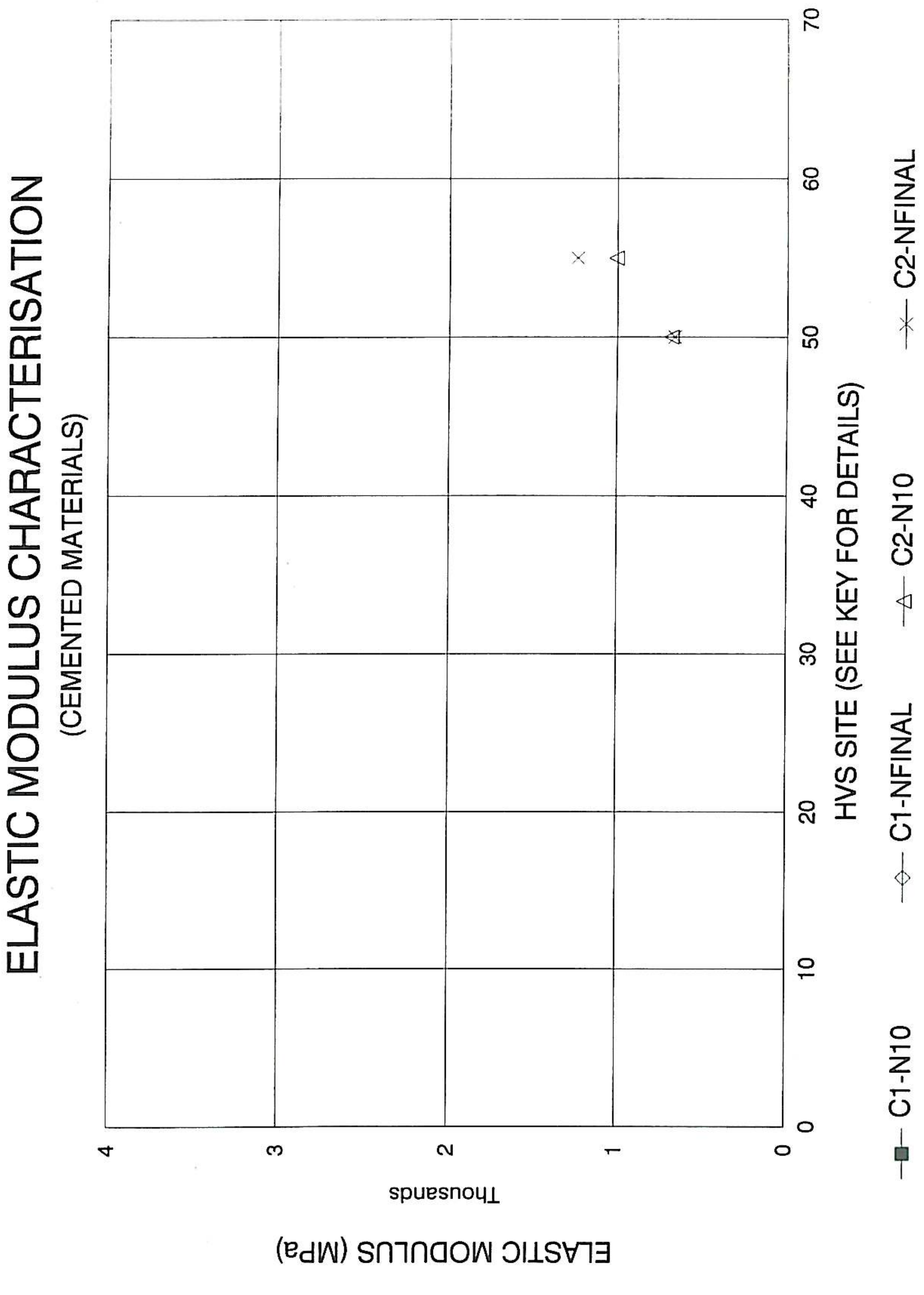
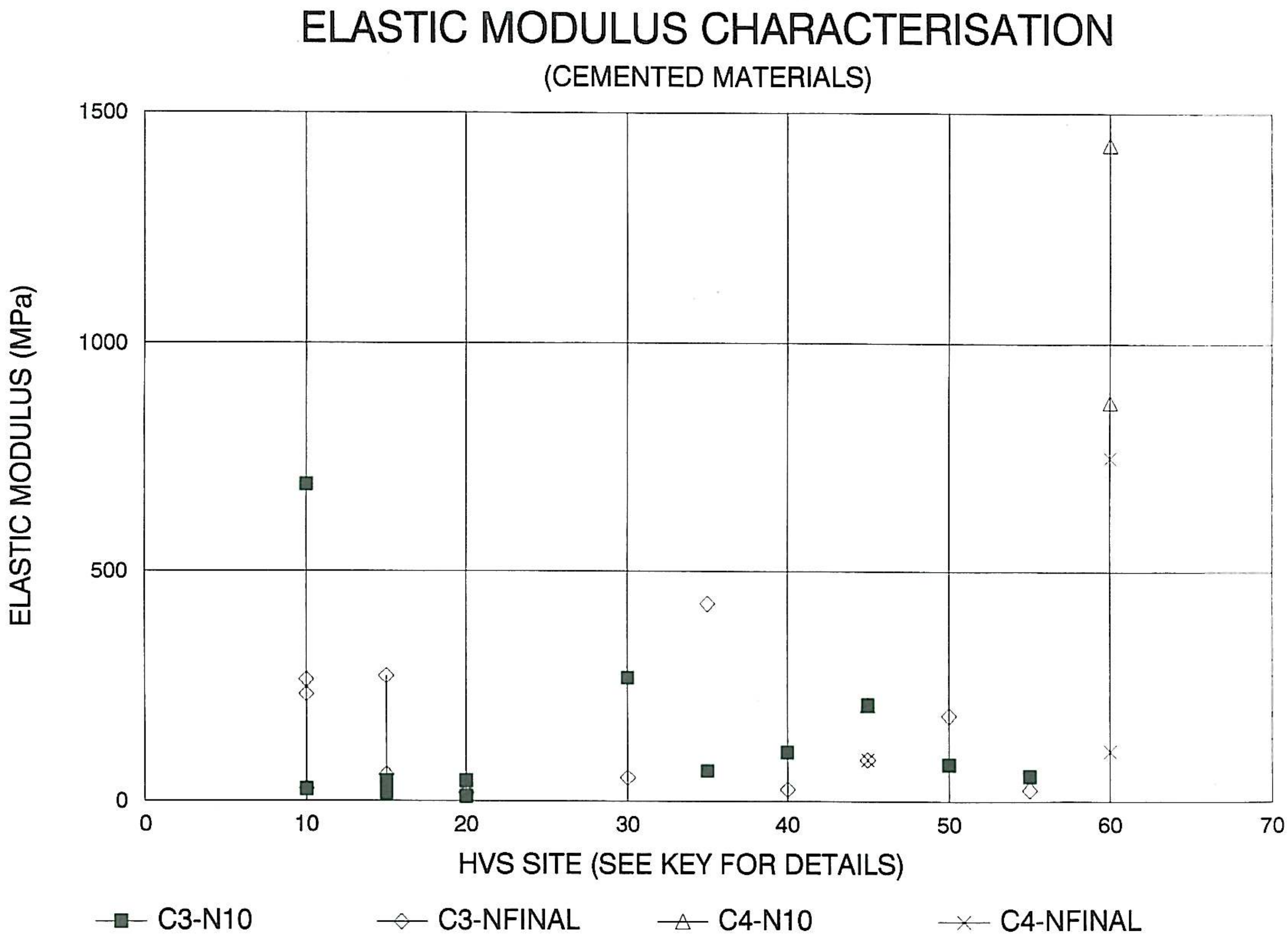


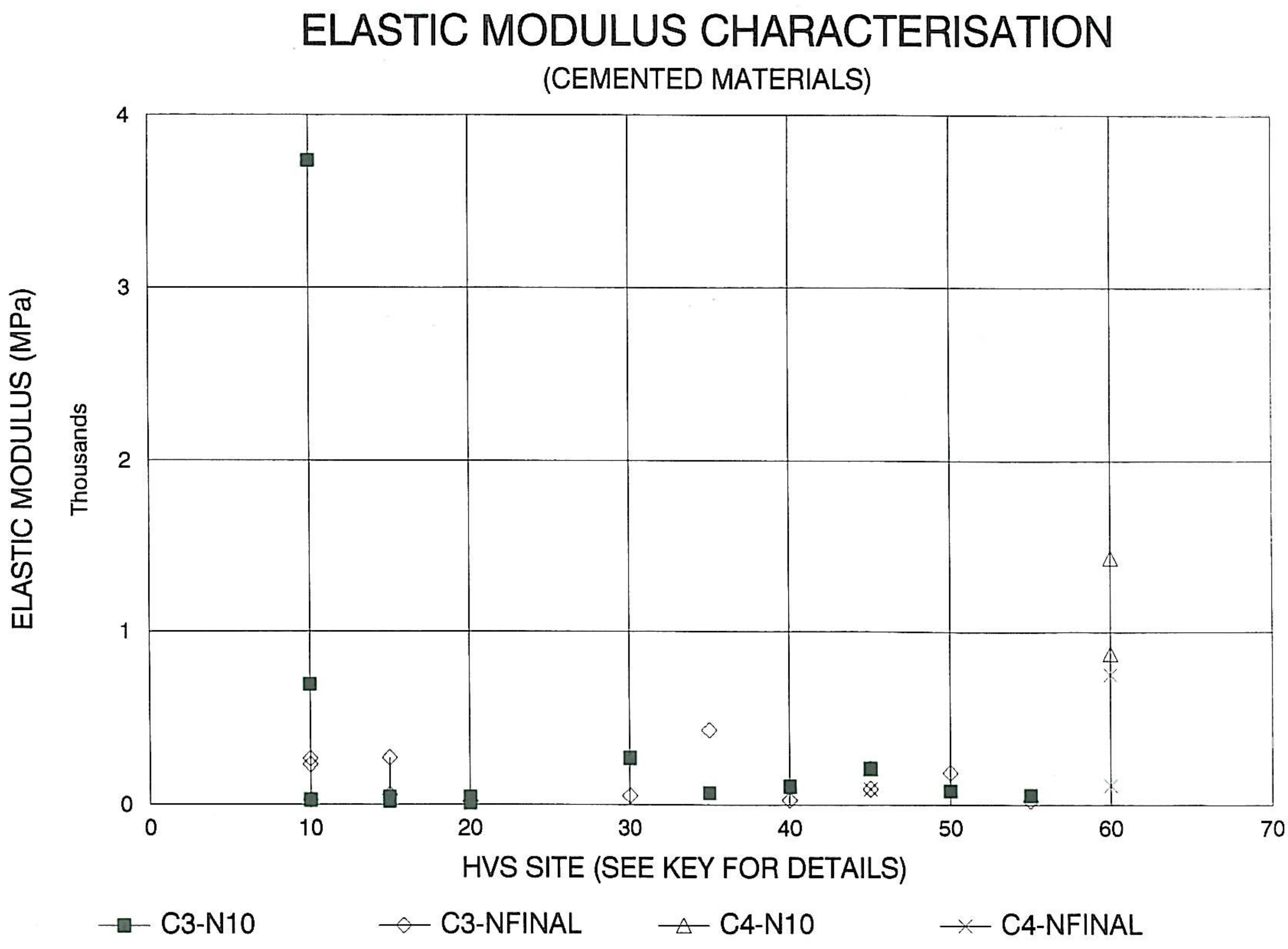
FIGURE 5.5

EC1C2AU.DRW



EC3C4.DRW

FIGURE 5.6



EC3C4AU.DRW

FIGURE 5.7

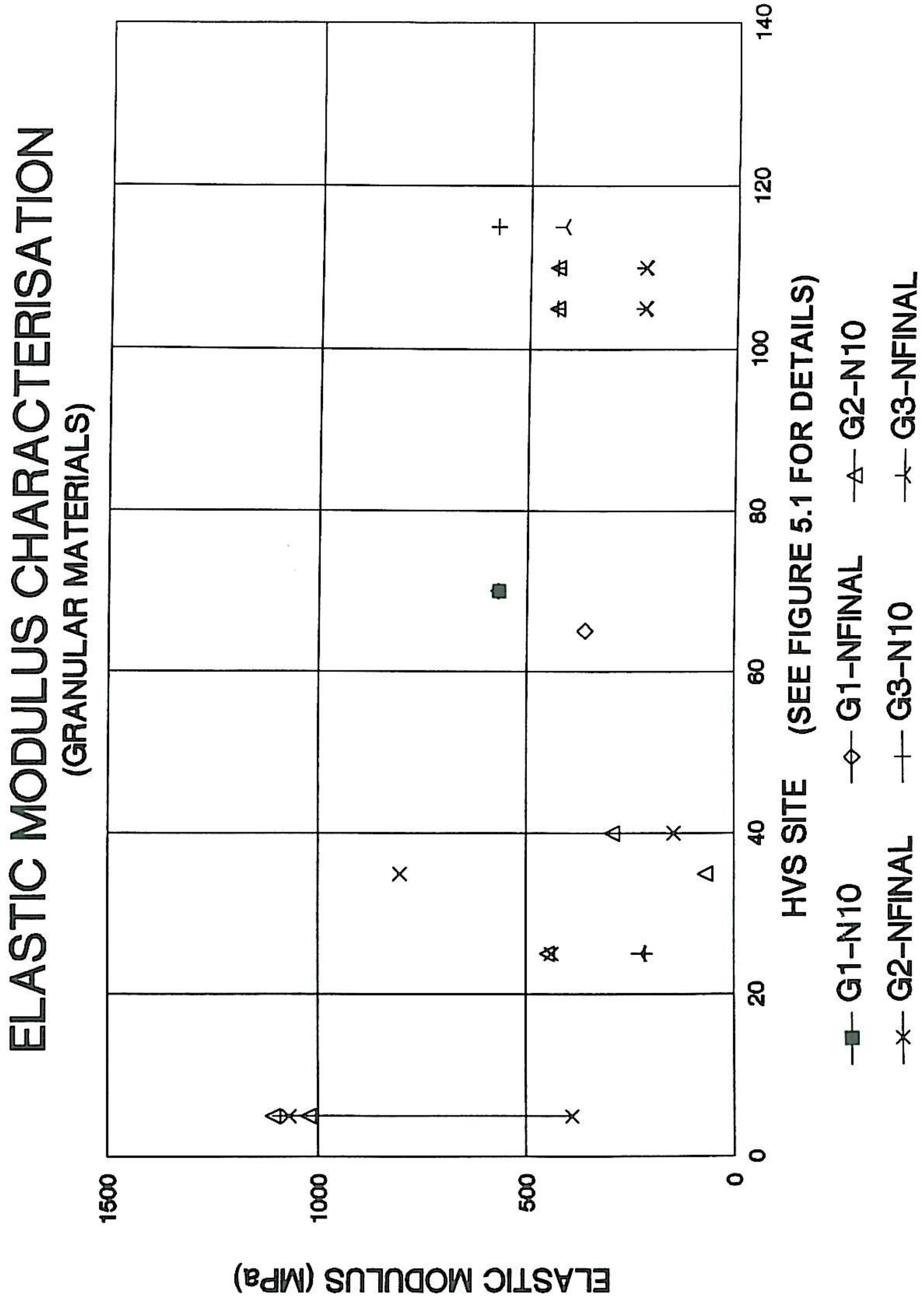

ELASTIC MODULUS CHARACTERISATION
(GRANULAR MATERIALS)
ELASTIC MODULUS (MPa)
0
500
1000
1500
0
20
40
60
80
100
120
140
HVS SITE (SEE FIGURE 5.1 FOR DETAILS)
G1-N10
G1-NFINAL
G2-N10
G2-NFINAL
G3-N10
G3-NFINAL

FIGURE 5.8

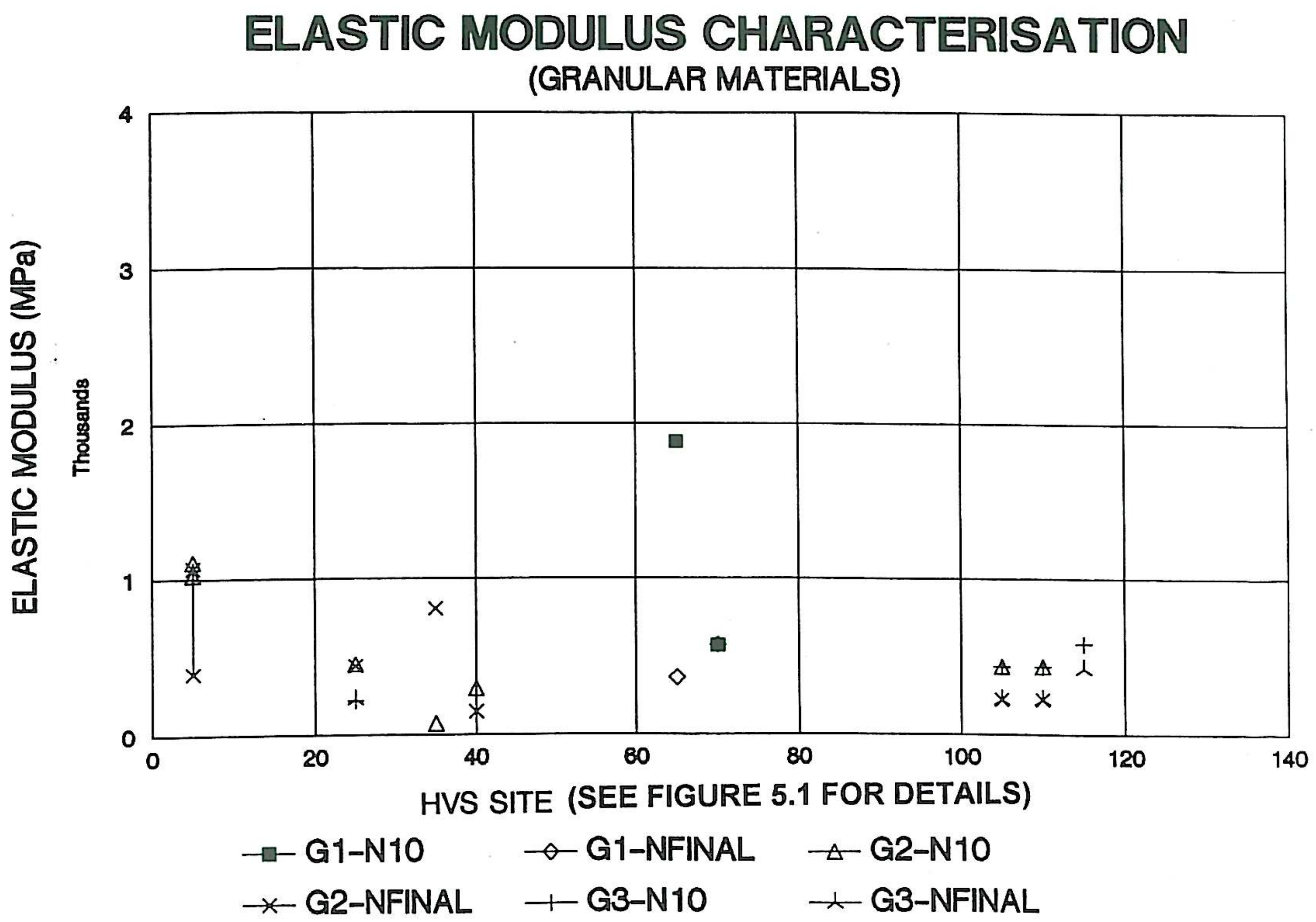

ELASTIC MODULUS CHARACTERISATION
(GRANULAR MATERIALS)
ELASTIC MODULUS (MPa)
Thousands
0
1
2
3
4
0
20
40
60
80
100
120
140
HVS SITE (SEE FIGURE 5.1 FOR DETAILS)
G1-N10
G1-NFINAL
G2-N10
G2-NFINAL
G3-N10
G3-NFINAL

FIGURE 5.9

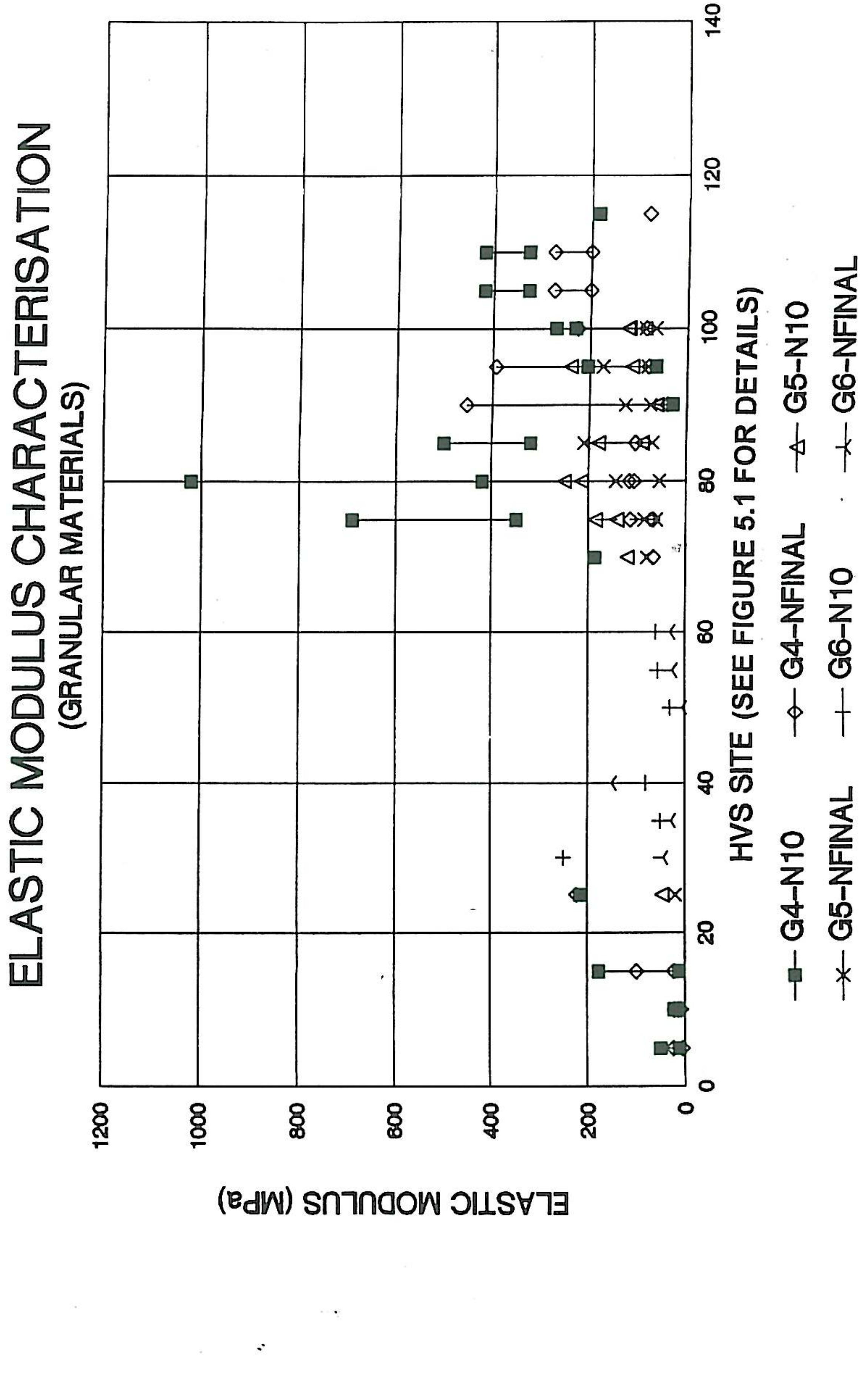

ELASTIC MODULUS CHARACTERISATION
(GRANULAR MATERIALS)
ELASTIC MODULUS (MPa)
0
200
400
600
800
1000
1200
0
20
40
60
80
100
120
140
HVS SITE (SEE FIGURE 5.1 FOR DETAILS)
G4-N10
G4-NFINAL
G5-N10
G5-NFINAL
G6-N10
G6-NFINAL

FIGURE 5.10

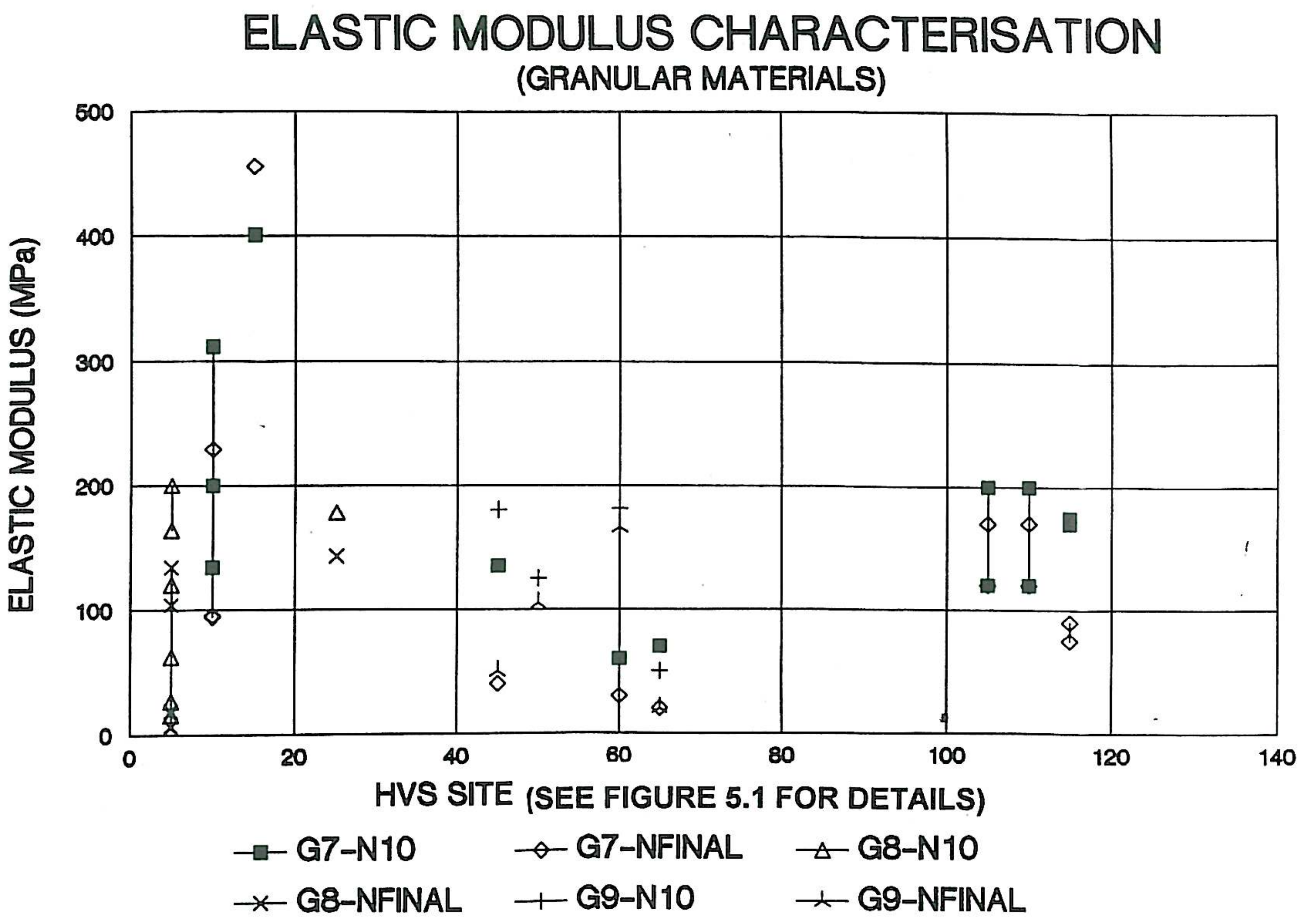

ELASTIC MODULUS CHARACTERISATION
(GRANULAR MATERIALS)
ELASTIC MODULUS (MPa)
0
100
200
300
400
500
0
20
40
60
80
100
120
140
HVS SITE (SEE FIGURE 5.1 FOR DETAILS)
G7-N10
G7-NFINAL
G8-N10
G8-NFINAL
G9-N10
G9-NFINAL

FIGURE 5.11

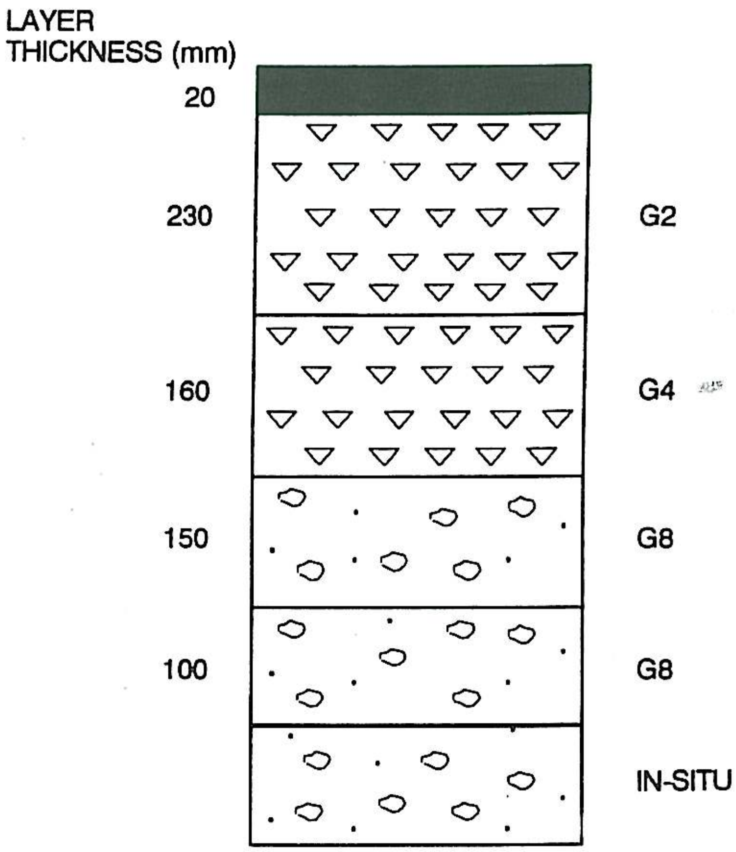


FIGURE 5.12

RICHMOND - 3 SISTERS (TEST SECTION 341)

PAVLA.DRW.

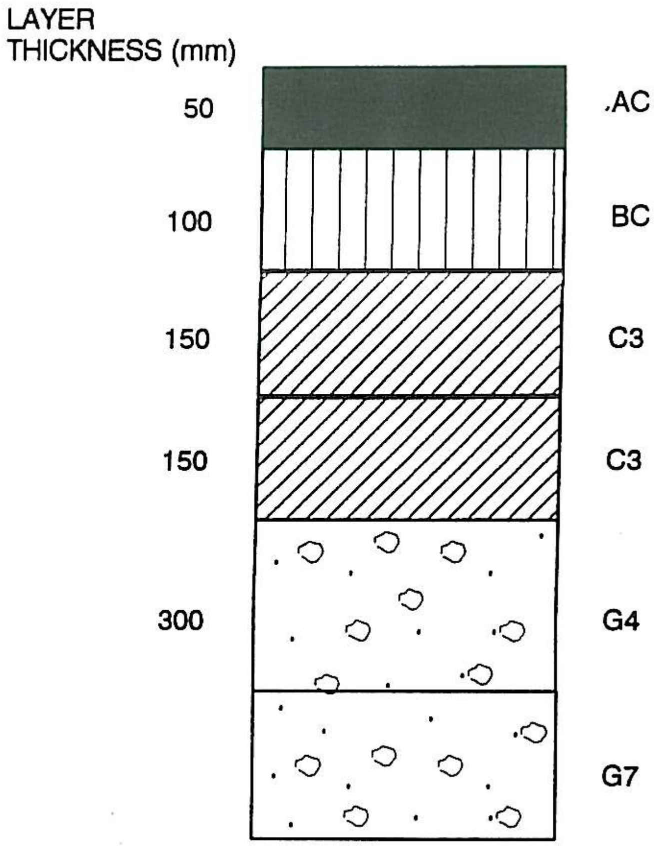


FIGURE 5.13

MARIANNHILL (TEST SECTIONS 215, 217 AND 218)

PAVN.DRW.

LAYER THICKNESS AND MATERIAL CODE ACCORDING TO TRH 14 [4]

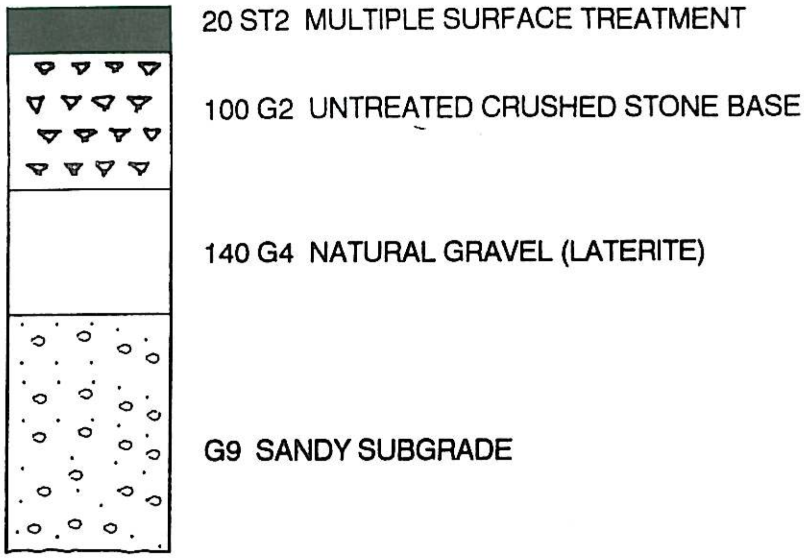


FIGURE 5.14(A)

THE GENERAL PAVEMENT STRUCTURE AT SITE A MALMESBURY

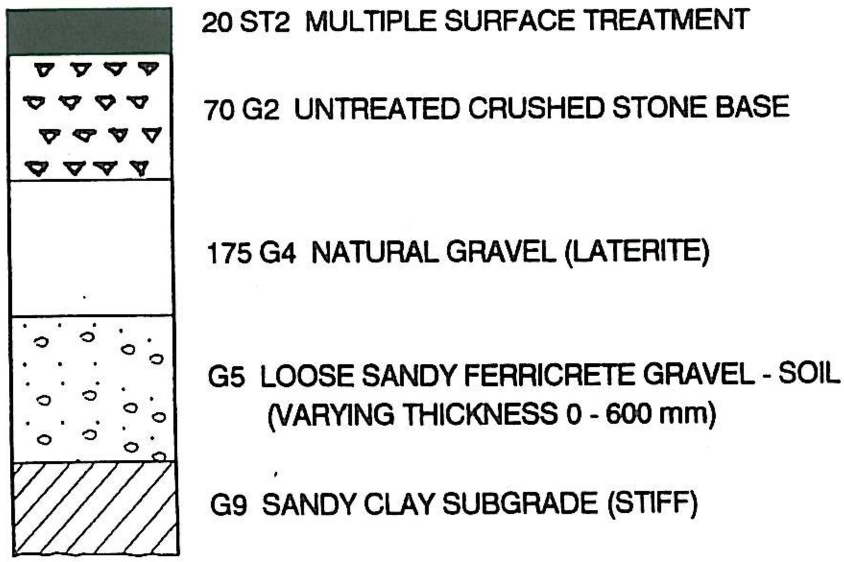
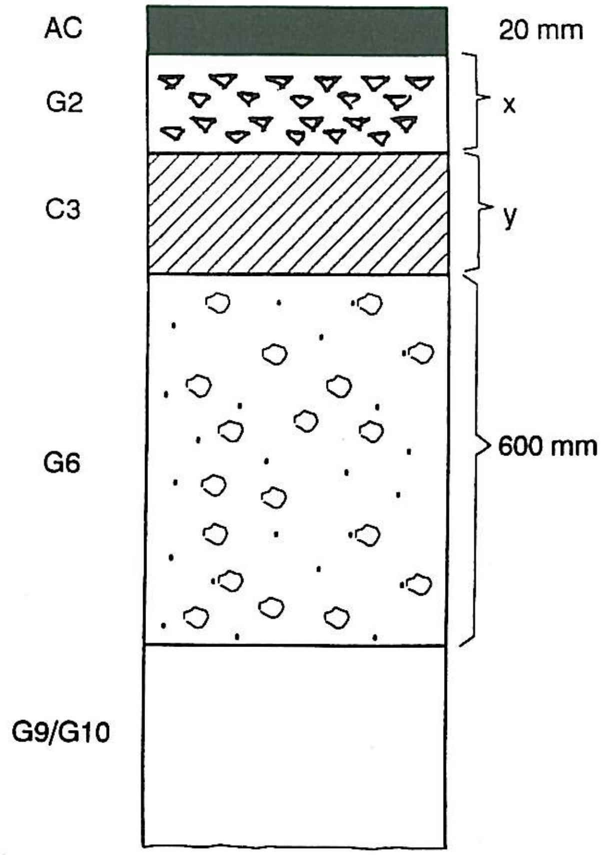


FIGURE 5.14(B)

THE GENERAL PAVEMENT STRUCTURE AT SITE B MALMESBURY

NEW.DRW.



TEST SECTION NO.	x (mm)	y (mm)
266	150	-
267	100	150
268	150	150

FIGURE 5.15

VREDE -MEMEL

PAVB.DRW.

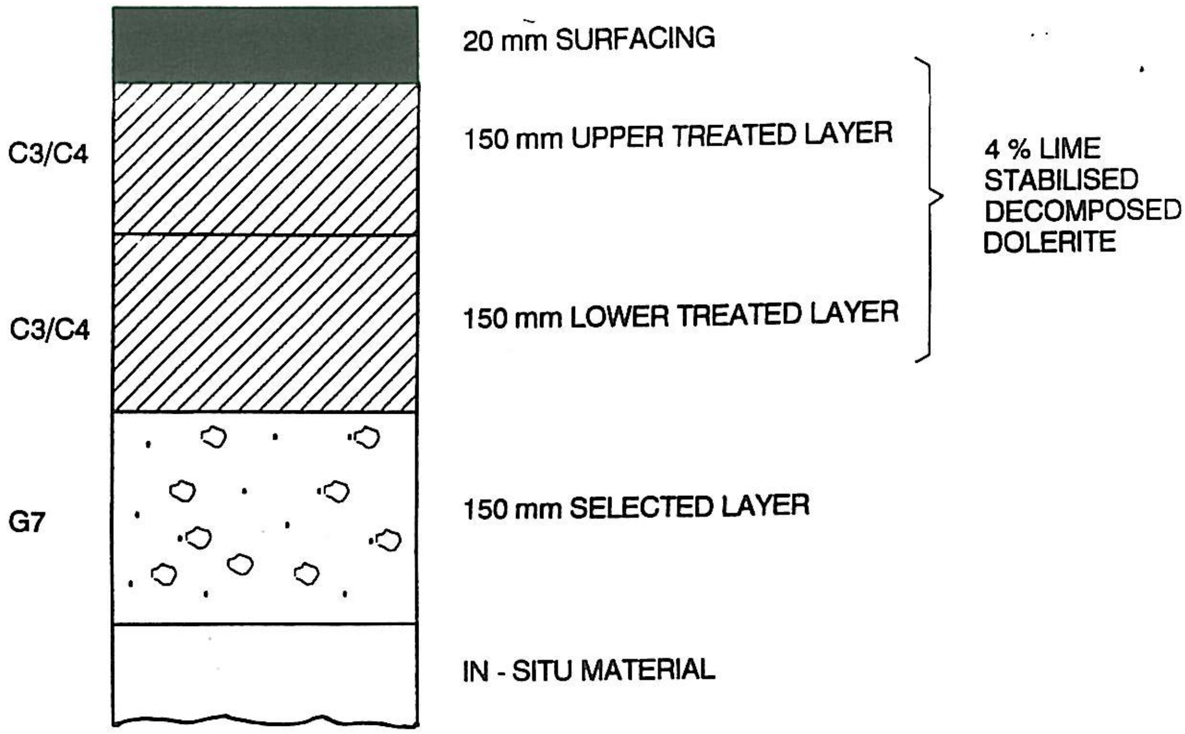


FIGURE 5.16
HARRISMITH / VAN REENEN TEST SECTION 7

PAVCF.DRW.

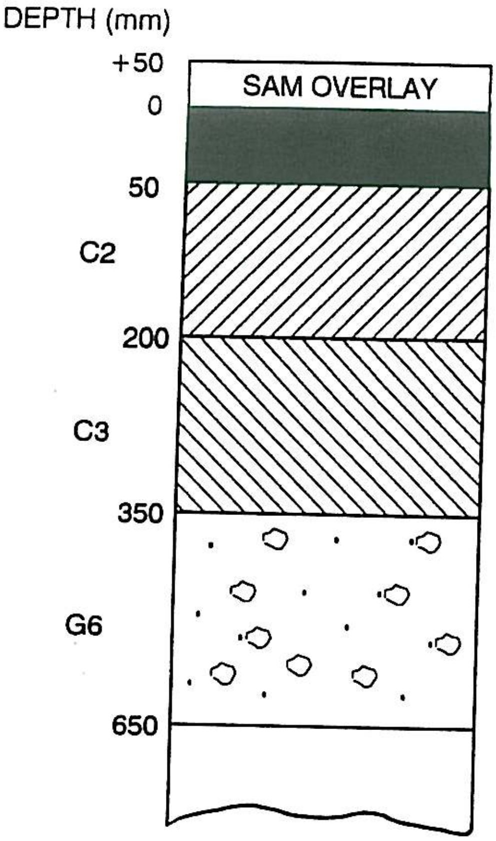


FIGURE 5.17

WARDEN - VILLIERS TEST SECTIONS 252 AND 255

PAVCE.DRW.

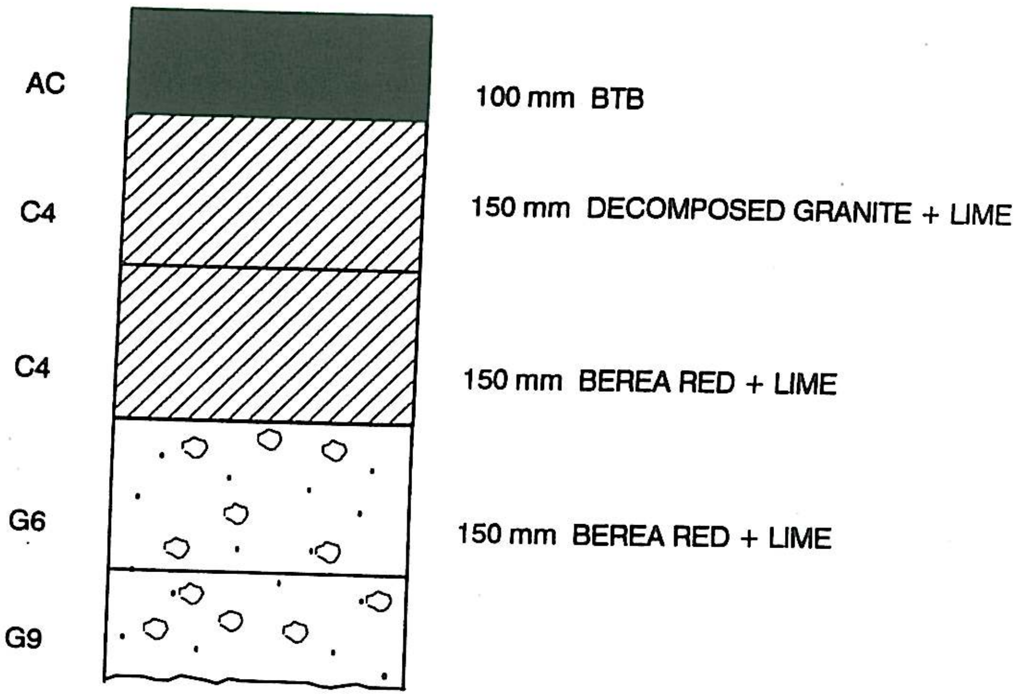
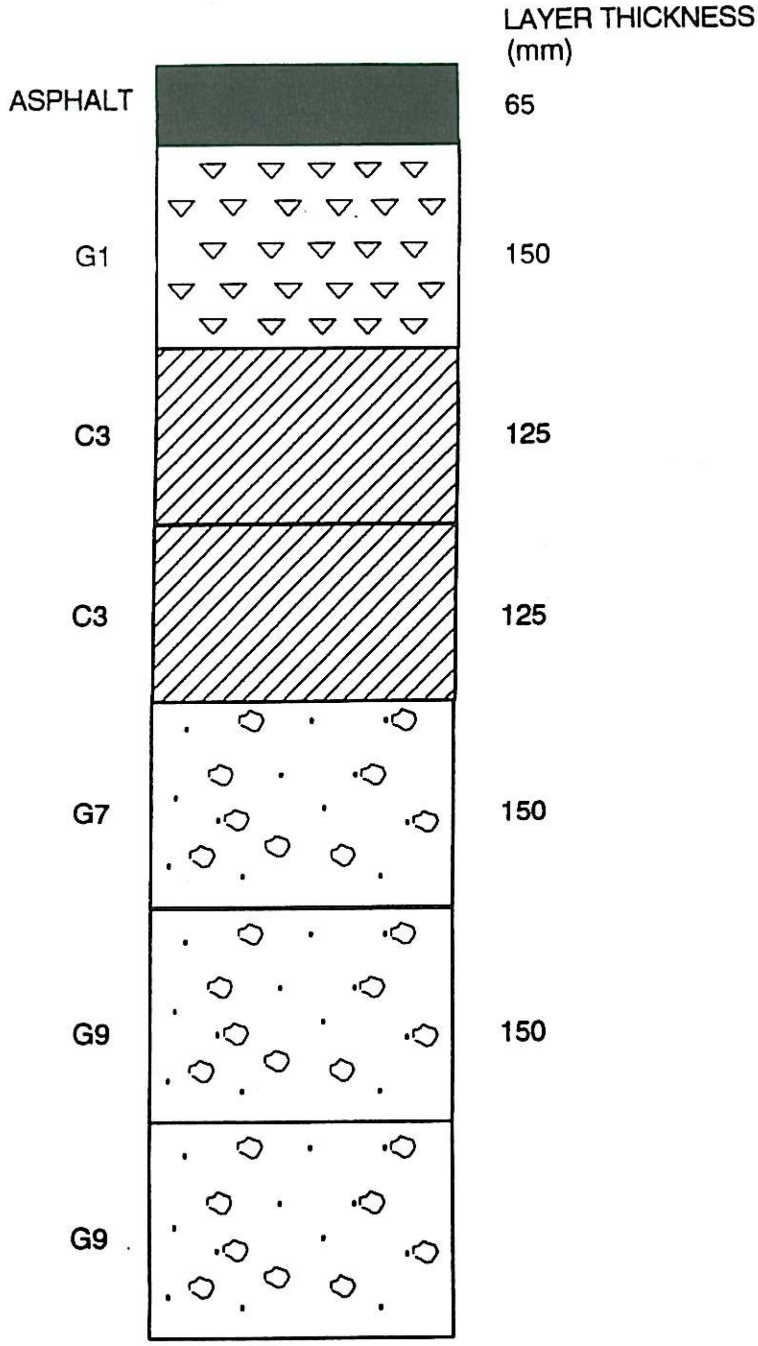


FIGURE 5.18

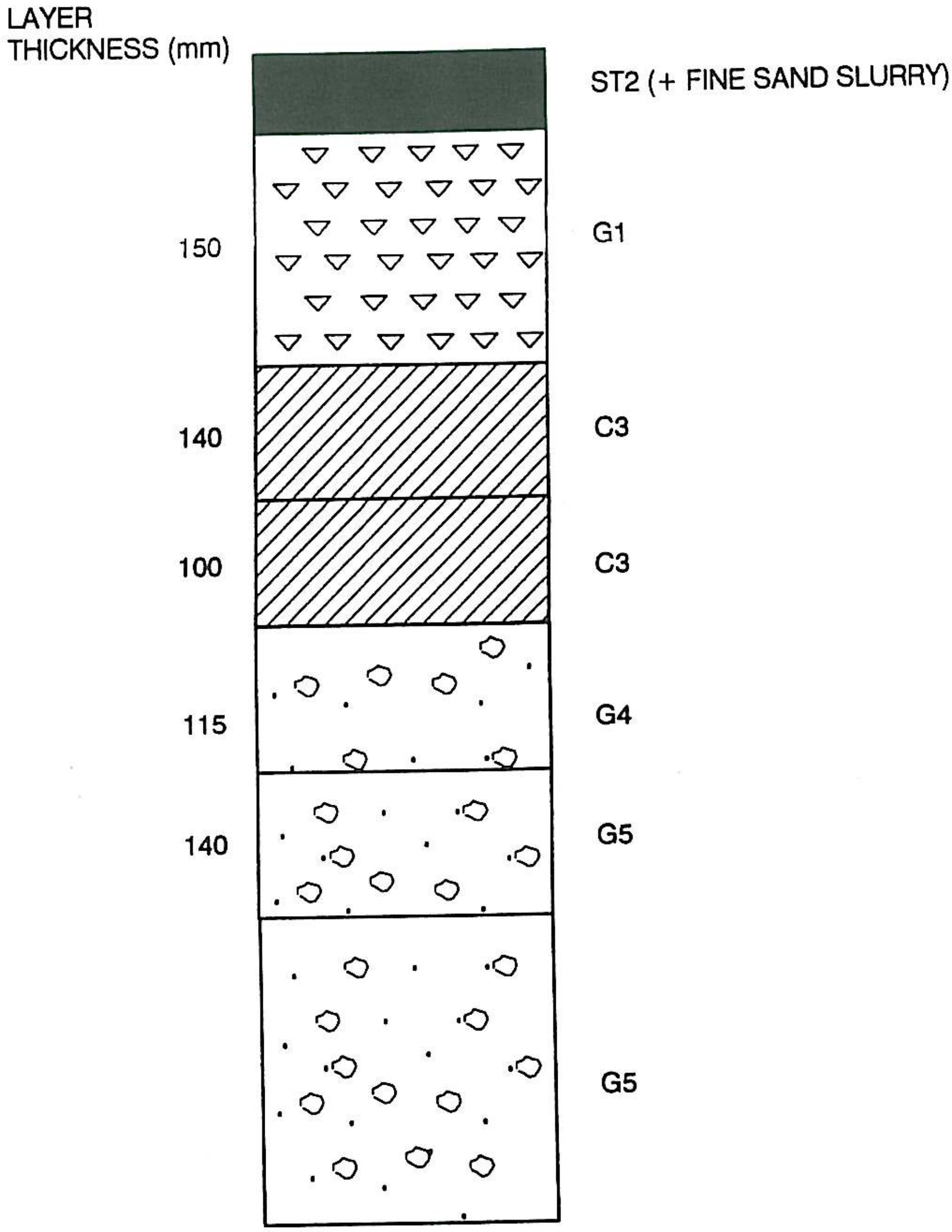
UMGABABA TEST SECTION 225

PAVA.DRW.



PAVCG.DRW.

FIGURE 5.19
UMKOMAAS TEST SECTION



PAVK.DRW.

FIGURE 5.20

BULTFONTEIN TEST SECTION 339

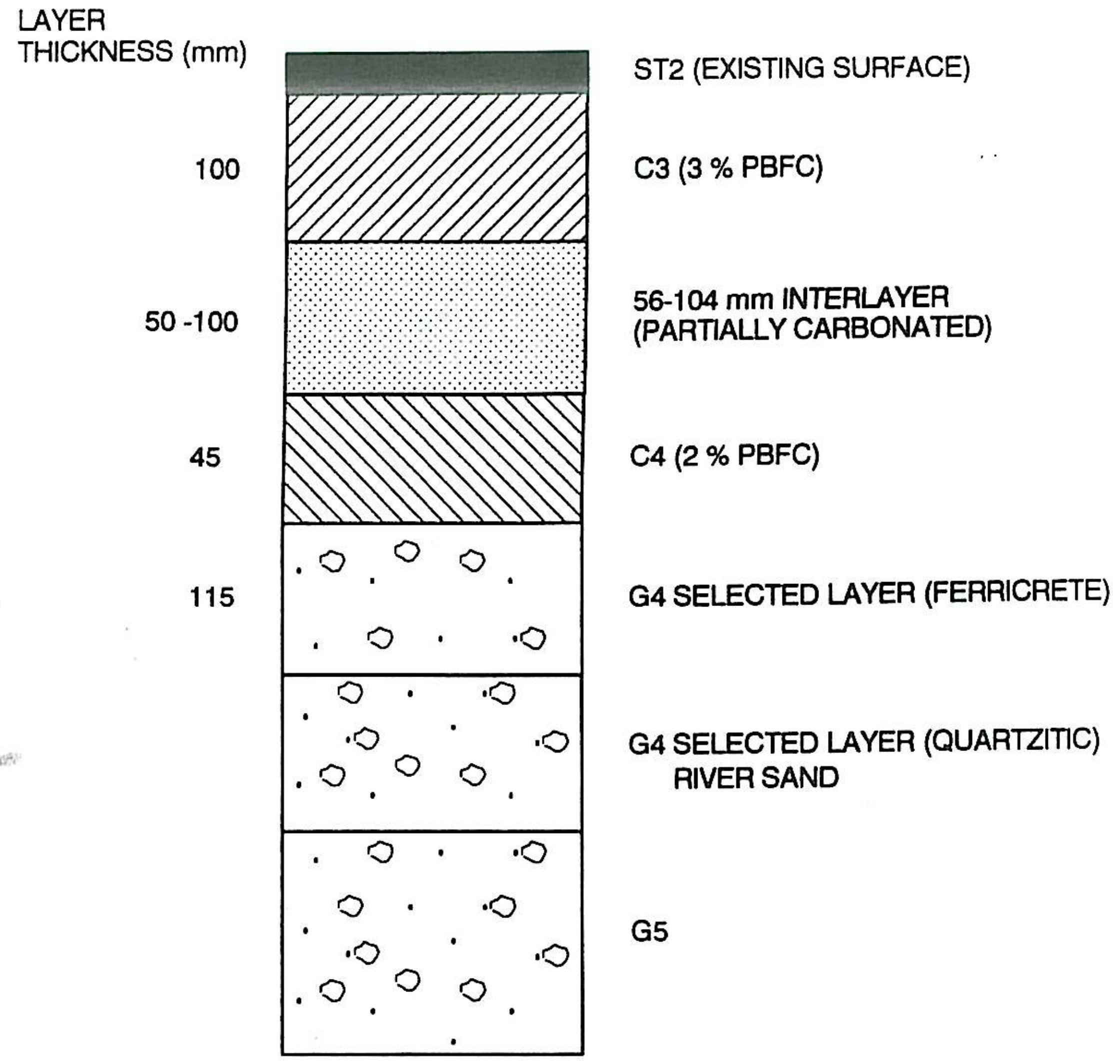
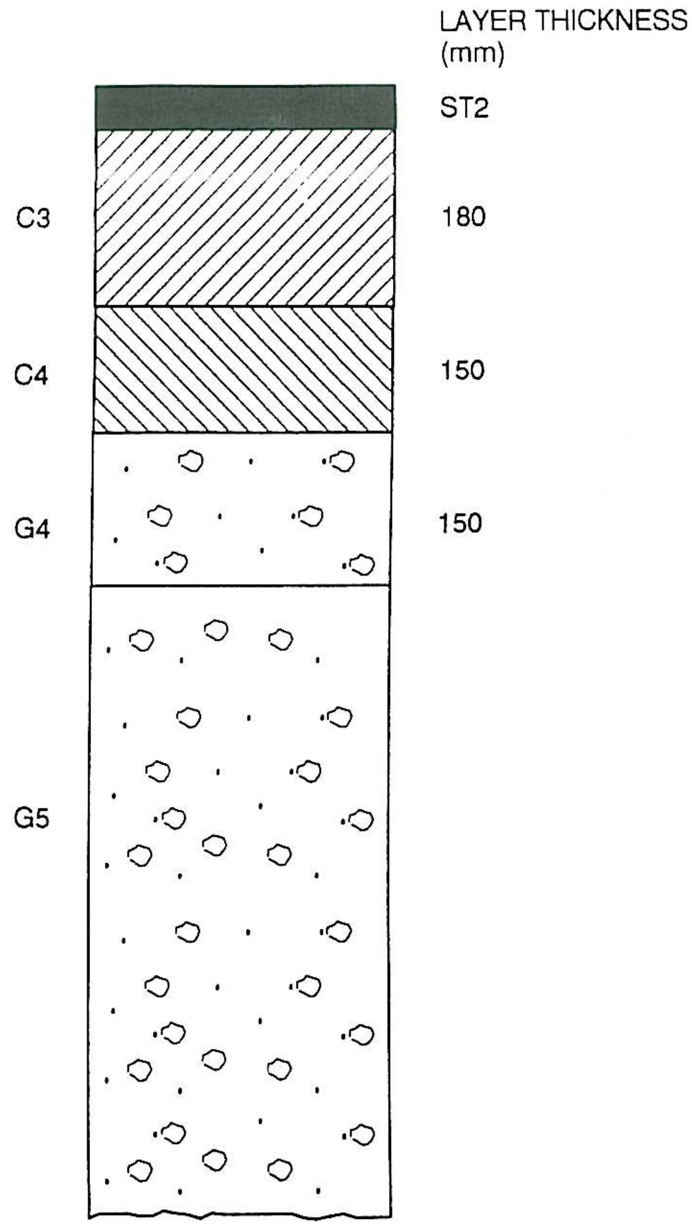


FIGURE 5.21

BULTFONTEIN TEST SECTIONS 306, 307, 308 AND 309

PAVJ.DRW.



PAVEH.DRW.

FIGURE 5.22

HVS SECTION 275A4, : ROOIWAL TEST SECTIONS 337 AND 338

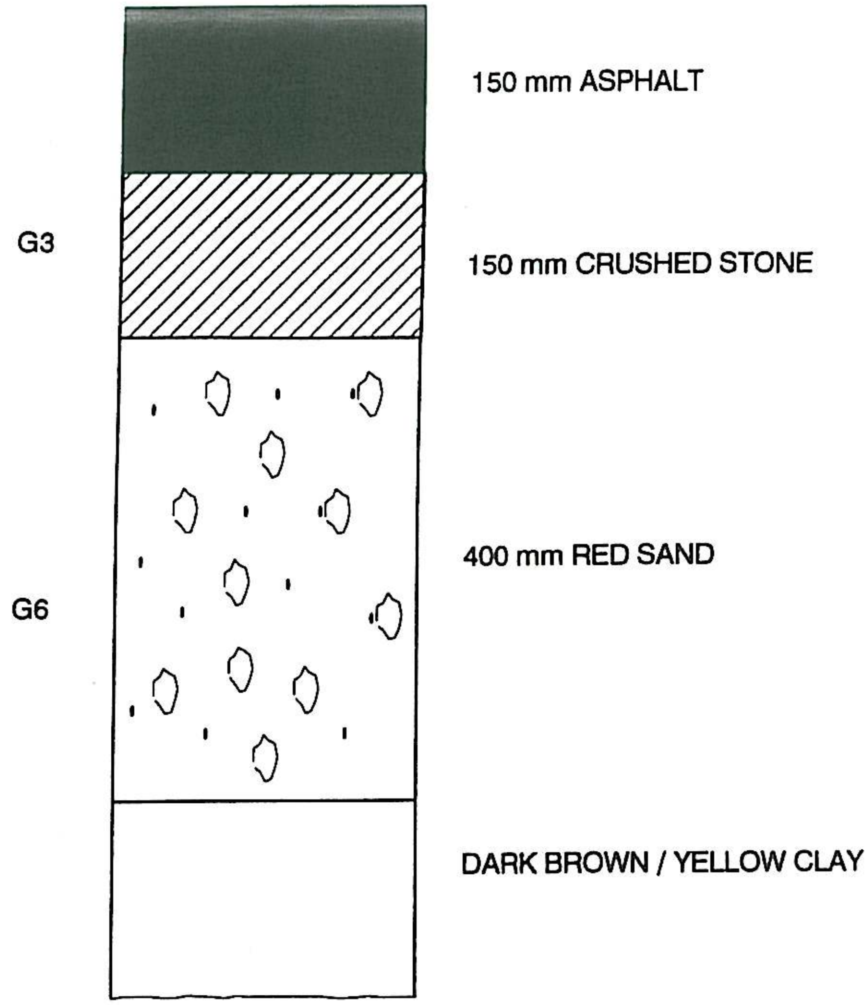


FIGURE 5.23

WELKOM - BLAAUDRIFT TEST SECTION 356A2

PAVO.DRW.

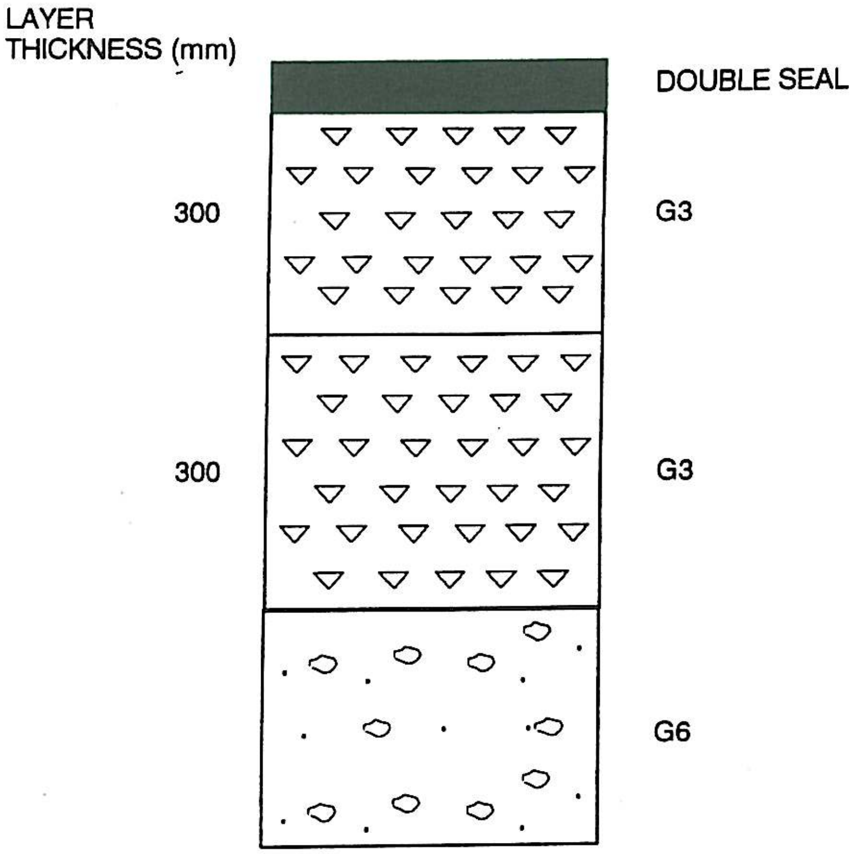


FIGURE 5.24
WELKOM TEST SECTION 363A2

PAVL.DRW.

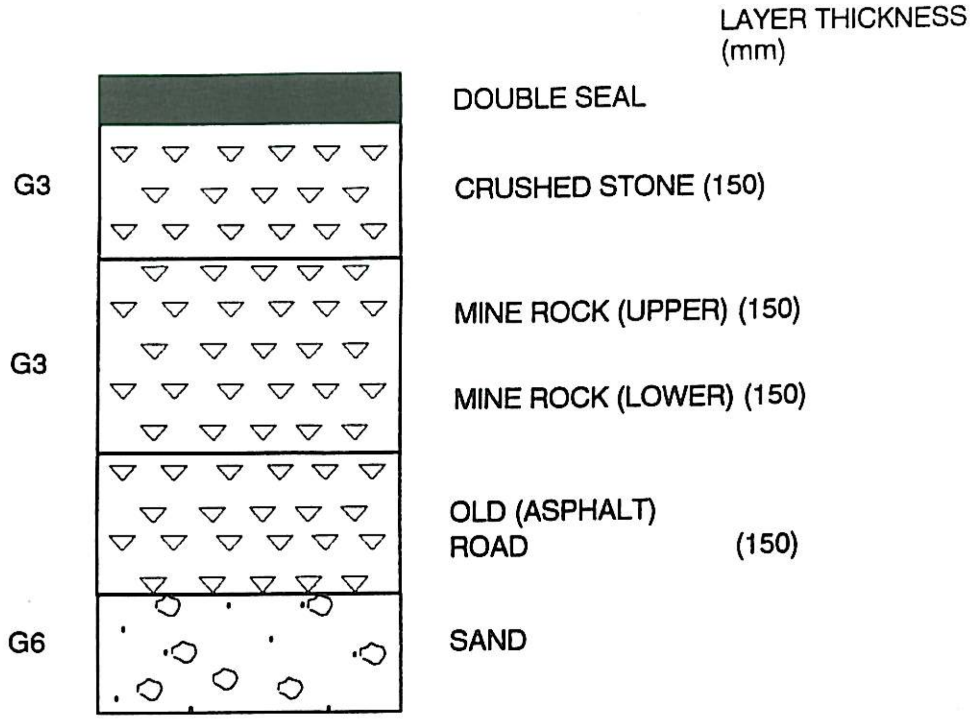


FIGURE 5.25
WELKOM TEST SECTION 357

PAVC.DRW.

6. LITERATURE STUDY FINDINGS

Brief synopses of what was considered the most appropriate information from the publications reviewed are given in Appendix B.

Section 10 (Recommendations) includes the findings of the desk study. Many variables influence the prediction of the moduli of most pavement materials, and, as yet, no general rules can be recommended. The information given here is intended to highlight the influence of some of the variables on moduli, and in addition, give the reader specific information that may be directly relevant to the problem at hand.

7. PRELIMINARY SURVEY OF "NEW" TEST INSTRUMENTS AND TEST METHODS FOR THE DETERMINATION OF ELASTIC MODULI

7.1 THE PENCEL SHEAR PAVEMENT PRESSUREMETER (PSPP)

This device measures the in-situ stress-strain response of materials under load. It is a simple device that can be used in most typical pavement environments but is not suited for measuring asphaltic materials, concrete or very stiff materials in its present form. The basic pressuremeter concept was developed by Louis Menard in the 1950s and '60s and since has been used extensively for the design of shallow and deep foundations, ground anchors, cantilever drilled shaft walls and anchored bulkheads, pavements, ground improvement and compaction control.

The PENCEL pavement pressuremeter was first developed by Briaud and Shields[9] in 1979 to replace tests such as the McLeod plate test[12]. Since then the procedure for and analysis of the PENCEL test have become more sophisticated. It is theoretically possible to predict the change in elastic modulus with load repetitions, time (creep moduli) and to reflect stress or strain dependency (depending on the material type).

The PENCEL pressuremeter probe was bought by the CSIR in 1989 and has since been used on various HVS test sites in South Africa. A special technique of analysis is being developed at the Division of Roads and Transport Technology specifically correlated to HVS test results in order to be compatible with the South African Mechanistic Design Procedure[5]. The pressuremeter has been used to test various categories of granular materials and the analysis technique currently under development is being calibrated for these materials. Appendix C gives a brief synopsis of the apparatus, its use, and typical test results.

For optimum economy, the apparatus is intended for the analysis of pavement subgrades in combination with other methods like the IDM[13] and/or the DCP[7,8]. The PSPP has the potential of providing the designer with reliable subgrade information that is subject to fewer assumptions than are currently used in some bakcalculation techniques (such as semi-infinite depths of subgrades for instance).

The elastic subgrade modulus is required for all pavement designs in some form, and if, as it seems, that this "direct" means of measurement is applicable, it should be considered, possibly in preference to some of the indirect methods of subgrade modulus determination.

7.2 THE K-MOULD

The K-mould is a laboratory apparatus used for measurement of material stress-strain relationships of roadbuilding materials. It shows great promise in the field of obtaining constitutive relations for the development of material models and is described in some detail in Appendix D.

7.3 THE PD-MDD COMBINATION

An alternative way of obtaining deflection measurements throughout the pavement structure is to use Multi-Depth Deflectometers (MDD)[14] in conjunction with a Personal Computer (PC) and data-logging card. The in-depth deflections can be measured at high speed with high resolution and so give a "complete" measure of the deflections basins at each module depth. Once this has been obtained the in-depth deflection basins (or simply peak deflections) can be fitted and elastic moduli derived from the data. Work is at present being conducted at the Division of Roads and Transport Technology (DRTT) to develop reliable methods of analysis using this information. A description of the technique and examples of test results is given in Appendix E.

The combination of the MDD and PC-logging systems is thought to have enormous potential as the "complete" pavement response can be measured for each pavement layer, thus providing the opportunity for more accurate modelling techniques to be developed.

8. INTERVIEWS WITH CONSULTANTS

This phase of the project was extremely fruitful and gave good insight into practical issues regarding means of assessing and designing pavements under various constraints. Discussions were not limited to test methods to determine elastic moduli but included design philosophy in general in order to put elastic moduli into the correct context.

The questionnaire/guideline used as a basis for discussion with the consultants is given in Appendix C.

In every interview it was stated that experience (or "expert intervention") is one of the most important factors in pavement analysis and design. Whereas experience is recognised as being important, however, as much of this intuitive knowledge as possible needs to be quantified. This information would then be more accessible and obtainable for all design engineers.

Novel features of some of the approaches incorporated "unusual" uses of test data with existing established methods. For example iteration between MECDE3, DCP results and tabulated values (e.g. RP/19/83[3]) to obtain representative elastic moduli was used in some instances.

A wide range of techniques for determining elastic moduli and subsequent use was found, ranging from the DCP[7,8], the Impulse Deflectometer (IDM)[4] and backcalculation techniques (both in-house and commercially available methods).

A synthesis of information obtained from interviews held with design engineers of 12 consulting engineering firms now follows.

8.1 GENERAL COMMENTS

It was the stated opinion of a number of the consultants that obtaining and using elastic moduli of pavement materials must be seen in context, and that the complete structural response and overall performance of the pavement are the most significant parameters to be used.

A particularly interesting and practical approach adopted by one or two consultants was that of generating transfer functions for each pavement under investigation (for rehabilitation). This is done by isolating sections of the pavement that are at different stages of their lives and doing in-depth investigation to ascertain why the pavement is in a certain condition after X traffic repetitions under prevailing environmental influences. Once the mechanism(s) of pavement behaviour and "failure" is established, appropriate measures can be carried out to rehabilitate the structure. In addition, the experience and data obtained from investigation of the full scale behaviour of the pavement can be extrapolated to other similar pavement structures. It must be stated, however, that this approach requires expert knowledge of materials and pavement behaviour and should therefore only be attempted when this is available.

The results of long term pavement performance (LTPP) monitoring (Department of Transport Project Report PR 89/134/1) will provide very useful information in this regard and should be continued as long as possible on as many different road structures as possible.

Almost without exception the consultants interviewed all make some use of the published values of elastic moduli in documents such as report RP/19/83[3]. Typically it seems that in most cases (where costs have to be kept to a minimum), materials are classified in terms of the TRH14[10] classification categories and the published values of elastic moduli used for design. Where more funds are available for testing the tabled values are used as a reference against which to compare moduli obtained from bakcalculation or laboratory test results for example.

Strong support for publishing more values of elastic moduli was found although it may be the case that a great deal of published information may tend to replace material testing (laboratory or in situ). This would be a mistake and could lead to serious problems when materials are mistakenly classified and/or inappropriate moduli are used, especially where environmental influences play a large part in determining the magnitude and variation of moduli.

In addition to the difficulties associated with the correct measuring of material properties, pavement response and design considerations, the problem of specifying and constructing pavement layers to provide the desired elastic properties and responses is not addressed at all. The need for better and more direct quantifiable quality control methods on this aspect was seen as very important.

A point that became clear during the interviews was that as many different methods of obtaining moduli as possible should be used and compared. The main limitation on this approach is found to be funding. This then in turn, impacts on the level of confidence in test values (and therefore design). In addition, the use of statistics to help define confidence in various measured or derived parameters is increasingly being used. Various consultants use statistical techniques to process DCP, IDM and other deflection test parameters, although there is some uncertainty as to the level of acceptance of the various measures. In the experience of one consultant different statistical distributions have been found to be applicable for different measurements. For instance Radius of Curvature measurements are found to fit the log-normal distribution best whereas, for instance, measurements of peak deflections are found to be normally distributed.

Further aspects of the application of statistics to road analysis and design will be published in the report of Department of Transport project PR 91/249.

Consideration of the role of economics during the design process was stated by a limited number of those interviewed to be an important factor in the design. Most of the interviewees expressed the opinion that funding, to a greater or lesser extent, determined the accuracy and confidence of a pavement design or assessment. The point was made that even if the best analysis techniques and skills are available, limited test data (due to inadequate funding) cannot be compensated for.

8.2 ASPHALTIC MATERIALS

The selection and/or determination of elastic moduli for asphaltic materials was a topic of much interest and some controversy with the engineers interviewed. In particular, guidance is apparently required with the problem of dealing with the temperature susceptibility of asphalt properties. Cracked asphaltic surfacings also raise problems with the general profession with regard to selection of suitable, representative material parameters. Requests were made at several interviews for additional published values.

In addition to, an inherently connected to this need, 8 out of the 12 companies interviewed specifically referred to the uncertainty regarding fatigue curves/transfer functions for asphalts with and without modified binders.

Correlation of values between laboratory (ITT)[15] values and those derived from in situ deflection measurements with depth shows that the elastic parameters of "new" or relatively uncracked asphalts agree quite well.

Test methods: The ITT[15] is a test extensively used for "routine" stiffness testing. The main criticism of the best mode seems to be that the best method is not necessarily similar to that experienced by the asphalt in situ. Another laboratory method that is gaining more favour due to the seemingly more similar mode of deformation to the field case is the 4-Point beam test[16].

Arbitrary use of documented stiffness values is often the case in practice due to time and cost considerations. When this approach is adopted, however, great care needs to be taken in ensuring that environmental and loading conditions in the field are compatible with those applicable to the values in the tables. It was stated in a number of instances that the maximum modulus used in design for asphalt was seldom, if ever, greater than approximately 1 500 MPa. This figure was arrived at through experience and backcalculation techniques using deflection basins.

From comments made during interviews there seems to be considerable confidence in moduli derived from the SHELL program "BANDS"[17], especially where asphalt overlays are to be designed or existing asphalts are relatively uncracked. The "fundamental" approach of the BANDS approach means that theoretically it is a sound method to apply in many situations (i.e. not being subject to limitations brought about by the usual empirical derivation of relationships). However, it was expressed that even if the BANDS approach is adopted for the prediction of moduli, the uncertainty of fatigue curves (transfer functions) still remains, as does the selection of the most suitable approach for analysis and design. In some instances where bitumen contents are low and materials are relatively dry and brittle, the "Factor of Safety" approach is used. When one should use this approach (in preference to tensile strain criteria) and how applicable it actually is, is however not obvious.

In general where asphalts and bitumen treated materials (BTMs) are present in a road to be rehabilitated, the most usual way of obtaining moduli is by backcalculation of deflection basins or with empirical DCP-CBR relationships.

8.3 CEMENT STABILISED MATERIALS

8.3.1 General comments

In general it seems that most consulting practices are unaware of the "latest" findings regarding the use of elastic moduli for cemented materials. In particular the concept and calculation of effective fatigue life (i.e. the repetitions required to initiate and propagate a crack completely through a cemented layer), and crushing life seems unknown. Ignorance of these aspects of material behaviour can lead to very inaccurate pavement life prediction and therefore little or no confidence in the design approach. At present standard practice seems to be that engineers have their own "in-house" approximations for cracked cemented materials, sometimes using DCP results and always with "experience and intuition".

The effect of cracks in cemented materials was noted as an area that needs investigation. In particular, the manner with which these are dealt with when backcalculating moduli from deflection basins was seen as a problem as was the decision of which modulus range to use from tabled values (i.e. how to decide on how severe cracking is and the size of blocks). Two consulting engineers stated that in their experience elastic moduli for cemented materials in "large blocks" are typically found to be in the order of 600 - 800 MPa (which is considerably lower than most of the recommended values). However, as stated above, to decide on the size and state of blocks of cemented material is not easily and it might be that the values of 600 - 800 MPa actually refer to small blocks.

Doubts were expressed as to the applicability of fatigue curves for moist conditions, and the suitability of parameters for "equivalent" granular materials.

8.3.2 Test methods

Moduli are typically obtained via the DCP-CBR-E relationships where CBR tests are often carried out at in situ moisture contents. The relationship of E = X*CBR (where X may vary between 5 and 20) is often used to estimate moduli. (Selection of values for factor "X" is left to the "experience" of the designer).

Generally, moduli are obtained by backcalculation, i.e. fitting theoretical deflection basins to field measurements (usually with linear elastic multi-layer programs such as CHEV or MECDE3). There is, however, no uniformity in the acceptance criteria of any given accuracy of fitted basins and is left to the discretion of the individual. Different backcalculation techniques are briefly discussed in a paper by Sanders, De Beer and Prozzi submitted for publication in the proceedings of the Annual Transportation Convention (ATC) to be held in Pretoria in August 1992. A transcript of the document is given in Appendix E.

8.4 CRUSHED STONE

8.4.1 General comment

NOTE: The way in which moduli are derived from any test or analytical technique must be compatible with the way in which the moduli are to be used in assessing or designing pavements. If this is not so then relationships between stresses, strains, deflections and pavement life will not be applicable.

At present elastic moduli of granular materials are used in analysis and design of pavements with the Factor of Safety approach[3,5]. Another approach is, however, being developed to deal with the analysis and design of granular pavement materials using a non-linear model to derive elastic moduli from in situ depth deflection measurements. This will be published in due course.

A non-linear approach using the program STATEN[18] for granular materials was found by one of the interviewees to be considerably better suited for prediction of material behaviour than, for example, ELSYM5 (which uses a linear elastic material model).

Typically, empirical CBR-DCP and CBR-elastic modulus relationships are used to obtain values of elastic moduli. Backcalculation techniques are also used (see Appendix E) as are values taken from documentation.

It is interesting to note that during the course of the interviews less uncertainty was expressed in obtaining moduli for granular materials than, for example asphalts or cemented materials.

8.5 SUBGRADE MATERIALS

Various opinions were expressed with respect to subgrades ranging from the questions of natural variation to the belief that South African pavements are generally over-designed but where weak subgrades occur, under-designed. Most of the engineers interviewed, however, had little in-depth comment on subgrades although there was a general realisation of the profound effect that subgrade properties can have on pavement behaviour.

Typically it seems that the test most often used for estimating elastic subgrade moduli is the CBR, either in the laboratory or via the DCP-CBR relationship. CBR values are converted to elastic moduli using empirical relationships such as E = 10 CBR (E in MPa) which may be modified from experience. If IDM results are available then the subgrade modulus automatically calculated by the program is used (usually "E_{AASHTO}"). Should specific tests not be carried out for moduli determination then in general materials are classified according to TRH14[10] and moduli from literature used.

The opinion was generally expressed that there is inadequate guidance for the selection of subgrade moduli, especially where materials are expansive as in certain Orange Free State and Transvaal areas. In addition several engineers expressed that a simple (preferably in situ) subgrade test is required to resolve the question of subgrade parameter definition.

Some specific comments regarding subgrades should be considered: It was stated that if designs in the TRH4 catalogue[1] were used when stiff subgrades were present the pavement would be over-designed. Also, when active subgrades are present "undisturbed" samples should be taken and swell-under-load tests carried out in consolidometers using the surcharge stress expected in situ. If swell or collapse is in excess of 1,5 %, then measures should be taken to improve the subgrade, or material removed if this is not possible. The stabilisation of the moisture condition, as a basic remedy for volumetric changes in active materials, was discussed. Means to prevent moisture changes by consideration of shoulder construction and use of sandy free draining materials was also mentioned as areas for future investigation.

The potential of good subgrade investigation was illustrated with examples of savings made in pavement layer thicknesses on some existing roads.

8.6 CONCRETE

Little was discussed with regard to the determination of the elastic stiffness of concrete. This was so because only a limited number of the consultants interviewed had experience in concrete pavement design, plus the fact that some rigid pavement design methods do not require elastic moduli as input data.

8.7 FLEXIBLE PAVEMENTS

8.7.1 Rehabilitation

In general interviews were mainly concerned with pavement rehabilitation due to present national needs. Most of the comments made in Section 7 are applicable to rehabilitation and only a very brief summary of points relating to the determination of elastic moduli will now be given.

Methods typically used for determination of elastic moduli are as follows:

(a) Literature (for example document RP/19/83[3]) following material characterisation by laboratory testing.

(b) Backcalculation using *in situ* deflection basins (measured by the IDM or the Road Surface Deflectometer (RSD) for instance). Analysis may be carried out by manual interaction with multi-layer pavement analysis programs although there are "automatic" backcalculation programmes available such as WESLEA[19].

(c) Backcalculation of moduli by fitting maximum deflections obtained from Benkleman Beams.

(d) Empirical relationships between DCP-CBR and elastic modulus.

8.7.2 New roads

In general the catalogue method is used with published values of elastic moduli if a design is to be altered due to availability of certain materials, for instance.

Normally a limited amount of testing is carried out in the laboratory. This is mainly to classify material according to TRH14[10].

The DCP may be used to obtain relative measures of material between locations and in some cases in situ CBR values. A range of in-house and published relationships are used.

9. CONCLUDING COMMENTS

9.1 MODULI FROM MULTI-DEPTH DEFLECTOMETERS (MDDs) - HEAVY VEHICLE SIMULATOR TEST DATA

Values of moduli plotted in Figures 5.8 to 5.13 have been derived using Multi-Depth Deflectometer deflections and linear elastic multi-layer theory. These moduli are therefore compatible with transfer functions given in RP/19/83[3]. It is possible that these moduli will not agree with values derived from other test methods and approaches such as the IDM or the triaxial test. As mentioned previously, care must be taken when using moduli obtained from various sources with transfer functions derived under specific conditions.

In general moduli agree with values quoted in earlier literature (e.g. RP/19/83[3]) but as indicated, change with load intensity, moisture and number of repetitions. When carrying out a pavement analysis or design therefore, various combinations of moduli should be used in an attempt to simulate changes in pavement state.

9.2 SENSITIVITY STUDY

Information given in Appendix A (i.e. estimations of the susceptibility of various pavements to changes in elastic moduli and Poisson's ratio) indicates how pavement performance does not depend merely on the magnitude of moduli and Poisson's ratio, but also on pavement geometry and material type. The pavement lives calculated were computed using existing transfer functions and combinations of "failure" modes (e.g. for cemented materials, crushing and effective fatigue were used). Using different assumptions and failure criteria it is possible to obtain different estimations of pavement life for any given structure. It is advisable therefore to treat Appendix A as indicative of possible effects of variations in elastic properties and to take each real pavement situation as a special case, and not generalise.

9.3 ENGINEERING JUDGEMENT

At the moment there seems to be no readily available technique or test method that is suitable for all of the most common pavement problems and situations. Engineering experience is therefore an important component in all aspects of analysis and design.

9.4 BACKCALCULATION OF MODULI

9.4.1 Non-linear dynamic analysis

The advent of powerful and readily available computer facilities has meant that detailed rigorous data analysis is carried out by some consulting engineering firms. This in turn implies that non-linear dynamic analyses can be carried out more easily than before. In addition it has been shown that "static" elastic parameters can be used in dynamic analysis, thus potentially keeping material testing costs at the present level. It seems from the literature that for rigorous analysis of pavement behaviour, dynamic effects are to be incorporated. This in turn implies than new functions linking stresses and strains derived from this new approach to pavement life may need to be formulated.

9.4.2 Multi-layer linear elastic analysis

At present the available multi-layer linear elastic computer programmes that can be used on Personal Computers have one general shortcoming which is the limited number of pavement layers dealt with by the programme (i.e. a general limitation of 5).

9.4.3 Derivation of subgrade moduli

The derivation of subgrade moduli is not at present particularly consistent or reliable due to a number of factors. For instance, when backcalculating subgrade moduli from surface or in-depth deflection measurements, assumptions may be used such as a finite depth of subgrade as opposed to a semi-infinite half-space (which has been "traditionally" used in South Africa to date). This can have a significant effect on derived values (see Figures B2.6a - B.2.6b for specific examples) even before the natural variability of materials is addressed. Other techniques used such as the PENCEL Shear Pavement Pressuremeter, Dynamic Cone Penetrometer or the K-mould also have their drawbacks such as the test mode or direction or the volume of material tested. Possible marked seasonal changes in subgrade properties can also complicate derivation of subgrade properties. Comparison of different test and analysis techniques often give results that do not readily agree. More investigation is required in this direction to avoid large inaccuracies in pavement life prediction. Page A24 in Appendix A, for instance gives an idea of possible discrepancies which can occur, especially on light pavement structures.

9.4.4 The effect of Poisson's ratio on pavement life

Variations in Poisson's ratio can have an important influence on pavement behaviour, depending on the type of structure, and to what degree values vary in certain materials. Section 4, Figure 4.7 and Pages A-48 to A-45 in Appendix A indicate some of the possible effects. It is important to note that if a layer under a cemented or asphaltic layer has a "large" Poisson's ratio (i.e. more “rubber like”), the asphalt or cemented layer will tend to fail in fatigue relatively quickly. Bearing in mind that there seems to be little attention paid to this parameter with respect to testing, pavement analysis and design and in construction, additional work should be carried out to improve the situation. The K-mould discussed in Section 7, and in Appendix D, appears to be a very promising instrument for quick and accurate measurement of this parameter and should be developed further.

10. RECOMMENDATIONS

As a synthesis of the sources and information presented above the following recommendations are made regarding the determination and use of elastic moduli for road pavement assessment and design.

10.1 ANALYSIS AND DESIGN

- Determination of elastic moduli must follow procedures that are complimentary to the design approach that will be used. In particular moduli derived from dynamic tests such as the IDM should not be used with mechanistic design techniques developed for static or creep loading situations unless factored for each site.

- A minimum of assumptions of elastic moduli for road assessment and design should be made. Where possible in situ measurements must be taken, otherwise material should be sampled to permit the accurate laboratory determination of parameters and properties that describe (or can be used to classify materials) elastic moduli of the materials. The frequency of sampling will be dictated by local conditions but should be increased where there is a significant change of pavement condition.

  When tabled values are used such as from report RP/19/83[3] careful assessment of the pavement condition and any likely changes in state should be considered as the range of moduli for any class material is relatively large and can have a significant effect on pavement life.

  The type of pavement structure and in particular the succession of layer material types should help determine the accuracy to which material moduli are to be determined, and therefore what approach to be used. Where a material is prone to failure in tensile strain, for instance, the underlying material should not be relatively soft, otherwise failure can quickly occur.

- Bakcalculation of elastic moduli from deflection measurements is recommended but must be carried out with sufficient knowledge of the background of the technique used in developing the program and any inherent assumptions in the theory. Where possible, programs that can directly incorporate all layers in a structure should be used rather than those which require the combination of layers. In this regard ELMOD[20] should be used with caution.

- For organisations without the benefit of powerful computers, programs using equivalent layer theory (ELT) are suitable for quick assessment of elastic moduli of pavement layers. Once approximate moduli values are obtained, ELSYM5, for instance, can then be used for final iteration to obtain a good fit to measured deflections.

- Where cracked materials are present, i.e. asphalts and cemented layers, backcalculation of deflection basins is especially recommended. For asphalts, this approach is preferred to using the SHELL (BANDS)[17] method because it is suited to new and uncracked layers.

- It seems that typically, from laboratory ITT[15] tests and backcalculation of in situ deflection basins, asphalt moduli lie between 1 000 and 2 000 MPa. This agrees in general with values quoted in RP/19/83[3], although care is needed in choosing appropriate values, taking into account the material state and temperature. New asphalts, as tested in the laboratory appear to have lower moduli than those recommended in RP/19/83[3].

- Information presented in section 5 (updating RP/19/83[3]) regarding elastic moduli of pavement materials to pavement evaluation and design should be applied where possible.

- "Expert intervention is required for "accurate" analysis of pavement behaviour. A pavement engineer must have a good understanding of the general problem addressed, testing and analysis techniques, and not rely on automated "black-box" technology for analysis and design.

- Each pavement situation should be treated as a special case taking into account the effects of material type, environmental factors, traffic loading and any other factor that may influence pavement performance such as construction perhaps.

- Existing pavement analysis programmes such as MECDE3 or ELSYM5 should be upgraded to include transfer functions as a post processing option.

- The effect of the depth to zero deflection (or "apparent rigid layer") on pavement design and analysis should be further investigated and incorporated in the mechanistic design procedure.

- For quick assessment of elastic moduli of pavement layers using the existing transfer functions, deflection basins may be analysed and fitted with the equivalent layer theory (ELT) approach. To subsequently refine the analysis, the ELSYM5 (or similar) programme can be used taking the ELT-derived moduli as "seed" moduli.

10.2 CONSTRUCTION

- Site staff must be trained properly and briefed on critical aspects and parameters for each particular structure. This makes the construction process more reliable and helps to ensure that the design is converted from paper into reality with the highest possible degree of confidence.

- When proving materials for use in construction, as much materials investigation as possible should be carried out. Simple techniques such as the DCP can be effectively used to assess material variability in-situ and representative samples taken for laboratory analysis.

- Quality control techniques need to be developed and applied to ensure that design requirements are met on site. In particular there is a lack of knowledge on how to construct any given material so it will have a certain effective elastic modulus.

- Follow-up (ideally monitoring) of pavement performance and comparison to predicted performance be carried out at every opportunity to test the applicability of present design techniques.

10.3 TESTING

- To obtain a unique and "complete" measure of pavement behaviour, use of the MDD and data logger in conjunction with the IDM (or deflection truck for instance), is recommended. In addition to the surface basin this apparatus gives deflection basins at depth, thus allowing analysis of individual pavement layers. Data obtained from monitoring pavement response to dynamic loads should then be analysed to give dynamic material parameters, describing *inter alia* damping and inertia.

- To obtain the "complete" pavement response suitable for detailed pavement component analysis the use of Multi-Depth Deflectometers (MDDs), a data logging computer and actual traffic loads seems ideal. A "quick" MDD system is being developed at the CSIR that is designed to be quick to install, accurate and suitable for use by the profession in general.

- The approaches given in Table 2 *should* be considered when planning test programmes. It is appreciated that other test methods and approaches may be preferred, depending on the pavement situation, engineers experience and other factors. Literature can and should be used to ensure that no gross errors in calculated moduli are apparent but should not be used exclusively and at the expense of material testing.

Table 10.1: Provisional recommendations for test procedures

MATERIAL	LABORATORY	IN-SITU
Asphalt	ITT 4-Point Beam Test	Backcalculation with deflections from the MDD-PC and/or the IDM.
Granular	CBR Triaxial K-mould	Bakcalculation with deflections from the MDD-PC and/or the IDM, plus PSPP and DCP test results
Cemented	CBR UCS Triaxial K-mould	Backcalculation with deflection from the MDD-PC and/or the IDM, plus DCP test results.
Subgrades	CBR Triaxial K-mould	Backcalculation with deflections from the MDD-PC and/or the IDM seismic test results plus PSPP and DCP test results.

- When testing materials that are temperature dependant such as asphalt, deflection measurements should be carried out at different times of the day to obtain the change of effective moduli with temperature.

- As many appropriate test techniques as possible should be used and with cognisance of environmental conditions, as "complete a history" as possible of pavement performance as possible drawn up.

- From initial apparatus development and test results, provisional use of the K-mould and PENCEL Shear Pavement Pressuremeter is recommended, especially for granular and subgrade materials. Additional development and commissioning of the two instruments should be carried out.

- Relationships should be established between, for example, resilient subgrade modulus and unconfined compression test parameters, grain-size distribution and index properties for the main groups of South African subgrade types encountered in pavement construction. With such a regional or national data base, appropriate values of resilient moduli (that take into account stress dependency) could be obtained merely using "simple" widely-available test procedures.

- The K-mould should be developed further for measurement of Poisson's ratio, bearing in mind the important effect that Poisson's ratio can have on pavement life. The variation of this parameter with stress and moisture can also be quantified with the K-mould, thus potentially providing a designer with realistic values for use in sensitivity analysis.

- Long term pavement performance projects should be continued as long as possible to provide data with which to validate design techniques, assumptions, test techniques and methods of analysis.

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12. REFERENCES

1. Committee for State Road Authorities (CSRA). TRH4: 1985 Structural design of interurban and rural road pavements. Technical Recommendations for Highways (TRH), Department of Transport, Pretoria, South Africa, 1985.

2. Freeme C R. State of the Art on heavy vehicle testing in South Africa. Proceedings of a Symposium presented by the NITRR (now DRTT), CSIR, Pretoria, August, 1984.

3. Freeme C R. Evaluation of pavement behaviour for major rehabilitation of roads. Technical Report RP/19/83, Division of Roads and Transport Technology, CSIR, Pretoria.

4. Ahlborn G. Elastic layered systems with normal loads. Institute of Transportation and Traffic Engineering, University of California at Berkeley, California.

5. Maree J H., and Freeme C R. The mechanistic design method used to evaluate the pavement structures in the catalogue of the Draft TRH14. 1980, Technical Report RP/2/81, Pavement Engineering Technology, NITRR (now DRTT), CSIR, Pretoria, South Africa.

6. Raad L., and Figueroa J L. Load response of transportation support systems. Vol 106, No TE1, Transportation Engineering Journal, ASCE, January 1980.

7. Kleyn E G. Aspekte van plaveiselevaluering en ontwerp soos bepaal met behulp van die dinamiese kegelpenetrometer. M.Ing verhandeling, Universiteit van Pretoria, Pretoria, 1984.

8. De Beer M. Use of the Dynamic Cone Penetrometer (DCP) in the design of road structures. Geotechnics in the African Environment 10th African Regional Conference on Soil Mechanics and Foundation Engineering Incorporating the 3rd International Conference on Tropical and Residual Soils, Maseru, September 1991.

9. Briaud J-L and Shields D H. A special pressuremeter and pressuremeter test for pavement evaluation and design. Geotechnical Testing Journal, GTODJ, Vol 2, No 3, September 1979.

10. Committee for State Road Authorities (CSRA). TRH14: 1985. Guidelines for road construction materials. Technical Recommendations for Highways (TRH), Department of Transport, Pretoria, South Africa, 1985.

11. Committee for State Road Authorities (CSRA). TMH1: Standard Methods of testing road construction materials. 2nd Edition, Department of Transport, Pretoria, South Africa, 1986.

12. McLeod N W. Airport runway evaluation in Canada. Internal Report, Transport Canada, Ottawa, Ontario, Canada, August 1947.

13. Horak E., Maree J H., and Van Wyk A J. Procedures for using Impulse Deflectometer (IDM) Measurements in the structural evaluation of pavements. Paper No 5A3, Session 5A, Proceedings of the Annual Transportation Convention, CSIR, Pretoria, August 1989.

14. De Beer M., Horak E., and Visser A T. The Multi-Depth Deflectometer (MMD) system for determining the effective elastic moduli of pavement layers. Proceedings of the 1st International Symposium on Non-destructive Testing of Pavements and Backcalculation of Moduli, ASTM, Conference, Baltimore, Maryland, USA, June 1988.

15. ASTM D4123-82. Standard test method for Indirect Tension Test for resilient modulus of bituminous mixtures. American Society for Testing and Materials (ASTM), Annual Book of ASTM Standards, Vol 04.03, 1987.

16. Irwin L H., and Galloway B M. Influence of laboratory test methods on fatigue test results for asphaltic concrete, fatigue and dynamic testing of bituminous mixtures. ASTM Special Technical Publication 561, Philadelphia, Pa, USA, June 1974.

17. SHELL International Petroleum Company. BANDS user manual - Bitumen and asphalt nomographs developed by Shell. Version: 1989: Release: ME 1.1, Shell International Petroleum Company Ltd., London, England, January 1990.

18. STATEN. Information regarding the application of this computer program to pavement engineering is available through Dr J P Lourens, Bruinette, Kruger and Stoffberg Inc., Pretoria.

19. Van Cauwelaert F J., Alexander D R., White T D., and Barker W R. Multi-layer elastic program for backcalculating layer moduli in pavement evaluation. Non-destructive testing of pavements and backcalculation of moduli, ASTM STP 1026, A J Bush III and G Y Baladi, Eds., American Society for Testing and Materials, Philadelphia, 1989.

20. Ullidtz P., Battiato G., Larsen B K., and Stubstad R N. Verification of the analytical-empirical method of pavement evaluation based on FWD testing. Proceedings of the 6th International Conference on the Structural Design of Asphalt Pavements, The University of Michigan, Ann Arbor, MI, USA, July 1987.

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An example to aid understanding of information given in Appendix A

Pavement Structure

AN EXAMPLE TO AID UNDERSTANDING OF INFORMATION GIVEN IN APPENDIX A AND FIGURES 4.1 TO 4.7

The terminology and interpretation of previous Figures 4.1 to 4.7 is now explained:

(a) Nomenclature of the pavement structures:

The first letter of the code for each pavement structure represents the design traffic for the pavement[1].

The second and successive letters are taken from the initial letter of the material type for each layer.

For example: A A G C C S

Subgrade
Cemented
Cemented
Granular
Asphalt
Traffic class A

(b) Calculation of 'X' axis value for Figures 4.1 to 4.7

Values on the X-axis of the graphs ('percentage change of recommended elastic modulus') are calculated by selecting values of layer moduli within the ranges given in RP/19/83[3], and dividing these values by the 'recommended' values given in the same document, e.g. for a G1 granular layer over a cemented layer in a 'slab' state a value of 600 MPa could be taken as appropriate within the range 250 - 1000 MPa. The 'recommended' value for this material type and state is 450 MPa.

The X ordinate is therefore 600/450 = 1.33, which equals + 33 % on the X-scale.

(c) Values on the 'Y-axis' (the 'percentage change of pavement life') are calculated by first determining pavement life mechanistically using 'recommended' values of moduli for each layer. Pavement moduli are then systematically varied and pavement lives calculated for each variation. The 'percentage' change in pavement life' is then computed by dividing the life calculated after varying different moduli by the life calculated using 'recommended' values of elastic moduli. For example:

If life calculated by increasing the elastic modulus in layer one by +20 % = 1 x 10^6, and the life determined using recommended values of elastic moduli is 5 x 10^6, then the 'percentage change of pavement life' is

$$\frac{(10^6)}{(5 \times 10^5)} = 2$$

which is represented by +200 % on the graph.

(d) Data given in the graphs indicate the influence of moduli variations of the respective layers.

It is seen that variations of similar magnitude of elastic parameters can have quite different effects on calculated pavement lives. Some general pointers can be drawn from the results if different failure modes are considered. For instance, a relatively brittle stiff layer should not overlie a relatively soft, elastic or plastic material. If this is the case material in the upper layer can fail quickly through the tensile strain/fatigue mode. The concept of pavement balance[7] helps to clarify this point. A well-balanced pavement is one that has ratios of moduli of successive pavement layers such that there is no excessive build-up of stress between any two layers (i.e. a smooth transition). De Beer[8] suggests that a ratio of 2 for successive moduli gives optimum results. This idea should perhaps be further investigated and developed to assist in pavement design.

Not that there can be large differences in pavement life calculated (the Y-axis) with relatively small changes in values of elastic moduli (the X-axis). This is due to the sensitivity of certain of the relationships between stress and strain and pavement life and different ratios of moduli between layers.

Example Pavement Structure: 'AAGCCS' (Figure A1, pages A-4 to A-10)

Note that the structure of each pavement analysed is given on the initial page of calculations.

Figure A1 (Page A-4) shows variations in predictions of pavement life due to changes in elastic moduli of the various pavement layers, and a table indicating the (theoretical) failure mode induced in the pavement with an 80 kN dual wheel axle load. In addition, variation in elastic moduli for the respective layers used in analysis are given.

NOTE: (a) The bituminous seal is not considered as a layer for purposes of structural analysis.

(b) Layer thicknesses are expressed in mm and moduli in MPa.

Page A-5 gives a summary of the calculated pavement lives expressed as a percentage of the pavement life calculated using 'recommended mid-range' values (see Document RP/19/83[3]).

Pages A-6 to A-10 give further details of calculations carried out to obtain values shown in Figure A1, Page A-4.

An explanation of terms used now follows, using page A-6 as an example:

The term L1 (seen at the top of the page) refers to the layer investigated.

'TRUN20', for instance, indicates the computer run number.

Material input parameters and dimensions used for multi-layer linear-elastic computer analysis are given in each of the table boxes on Page A-6. On inspection it is seen that the only moduli that are altered during the analysis described are those of Layer 1, moduli of the other two (structural) layers being kept to typical or 'recommended' values given in Document RP/19/83[3].

The number of repetitions for 'failure' to occur are given in the lower portion of the table boxes, on the left hand side being those due to 'single failure criteria' and on the right hand side, 'combined failure criteria'. It is sometimes necessary to combine failure criteria to sensibly describe a full failure mode. For instance, in the case in question the combined failure criteria used here is that of the cement stabilised layer first cracking due to fatigue and then (when in an 'equivalent' granular state - see reference 3) failing in shear.

Minimum predicted pavement lives are then used in the tables shown on Page A-5 and the graphs in Figure A1 (Page A-4).

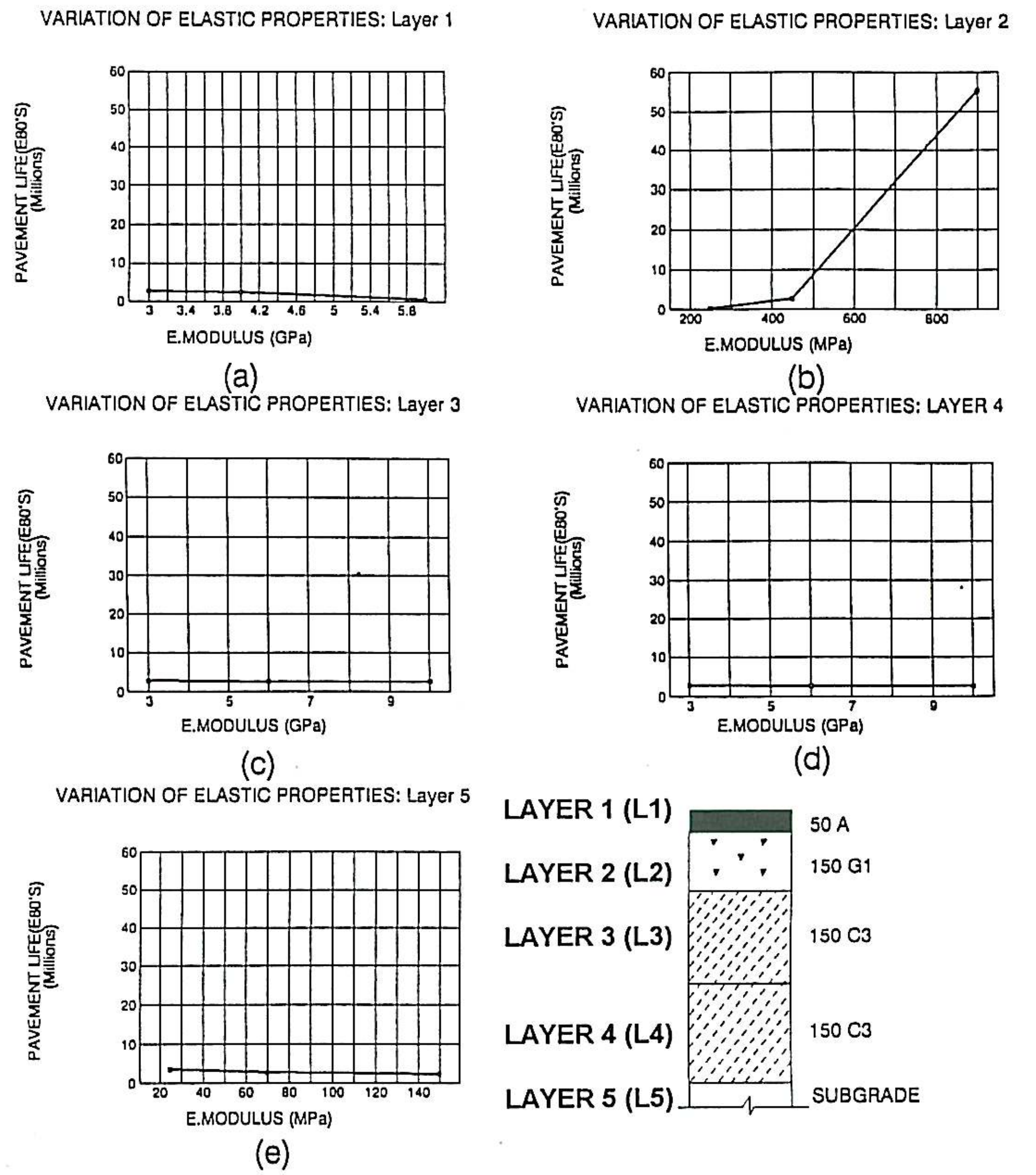
The column "Layer ratio' on Page A-6indicates which layers are used to calculate modular ratios. E x H represents the elastic modulus expressed in MPa, multiplied by the layer thickness (in metres).

It is interesting to note that an increase in modulus for any particular layer does not necessarily imply a greater pavement life. This is illustrated in Graph (b) on Page A-28 for the CGCS structure where the effects of varying the elastic moduli of the cemented material is indicated. The decrease in predicted pavement life when the elastic modulus of the cemented layer increases above 3 500 MPa is seen to be due to a change in failure mode, i.e. from 'Fatigue and FOS (shear failure) to that of crushing. Intuitively this seems to be correct as a stiffer layer will resist deflections to a greater extent, thus inducing stresses within the layer. If compressive strength does not increase commensurately, then crushing will occur at fewer repetitions than if the layer was more flexible.

It is emphasised that the investigation into the sensitivity of pavement life to variations in elastic moduli was carried out simplistically and results may not describe the actual behaviour of any real pavement. In particular it must be remembered that for each computer run only a single parameter was changed which may not be the case in-situ where moisture, temperature and load intensity may incur changes in several parameters simultaneously.

SENSITIVITY OF PAVEMENT LIFE TO CHANGES IN ELASTIC MODULI

AAGCCS



LAYER	ELASTIC MODULUS (MPa)	FAILURE IN:		FIGURE
		MATERIAL	LAYER	
1	1500 4000 6000	Asphalt Asphalt Asphalt	L1 L1 L1	a
2	250 450 900	Asphalt Asphalt Asphalt	L1 L1 L1	b
3	3000 6000 10000	Asphalt Asphalt Asphalt	L1 L1 L1	c
4	3000 6000 10000	Asphalt Asphalt Asphalt	L1 L1 L1	d
5	25 70 150	Asphalt Asphalt Asphalt	L1 L1 L1	e

FIGURE A1
CALCULATED CHANGES IN PAVEMENT LIFE WITH CHANGES IN ELASTIC MODULI
PAVEMENT AAGCCS (DETAILED ANALYSIS)

PAVEMENT
STRUCTURE: AAGCCS

LAYER1

VARIATION IN ELASTIC MODULUS		VARIATION IN PREDICTED LIFE	
MPa	%	E80'S	%
3000	Reference	2.75E+06	Reference
4000	33.33	2.42E+06	-11.99
6000	100.00	5.32E+05	-80.68

LAYER2

VARIATION IN ELASTIC MODULUS		VARIATION IN PREDICTED LIFE	
MPa	%	E80'S	%
250	-44.44	3.71E+05	-86.53
450	Reference	2.75E+06	Reference
900	100.00	5.55E+07	1915.38

LAYER3

VARIATION IN ELASTIC MODULUS		VARIATION IN PREDICTED LIFE	
MPa	%	E80'S	%
3000	-50.00	2.87E+06	4.45
6000	Reference	2.75E+06	Reference
10000	66.67	2.64E+06	-3.95

LAYER4

VARIATION IN ELASTIC MODULUS		VARIATION IN PREDICTED LIFE	
MPa	%	E80'S	%
3000	-50.00	2.92E+06	5.99
6000	Reference	2.75E+06	Reference
10000	66.67	2.67E+06	-3.12

LAYER5

VARIATION IN ELASTIC MODULUS		VARIATION IN PREDICTED LIFE	
MPa	%	E80'S	%
25	-64.29	3.61E+06	31.13
70	Reference	2.75E+06	Reference
150	114.29	2.37E+06	-13.74

MATERIAL	RECOMMENDED E (MPa)
L1 ASPHALT A	1500
L2 GRANULAR G1	450
L3 CEMENTED C3	6000
L4 CEMENTED C3	6000
L5 SUBGRADE	70

Note: To calculate the 'variation in predicted life' the recommended mid-range values of elastic modulus were used.

L1

TRUN20		POISSON'S	LAYER	LAYER	E. x H	MODULAR	TRUN20
LAYER	E.MOD	RATIO	THICKNESS	RATIO		RATIOS	
1	2999	0.44	50	1/2	149.95	2.221481	
2	450	0.35	150	2/3	67.5	0.075012	
3	5999	0.35	150	3/4	899.85	1	
4	5999	0.35	150	4/5	899.85	0.012855	
5	70	0.35	1000000		70000		
	SINGLE CRITERIA LIFE				COMBINED FAILURE CRITERIA		
LAYER 1	2.75E+06 ASPH				2.75E+06 ASPHALT		
LAYER 2	5.44E+14 G1				5.44E+14 G1		
LAYER 3	6.57E+07 SUBC3-FATIGUE				7.71E+12 FATIGUE+FOS		
	6.38E+07 SUBC3-CRUSHING				6.38E+07 CRUSHING		
	7.71E+12 SUBC3-FOS				7.71E+12 FATIGUE+FOS		
LAYER 4	4.50E+07 SUBSUBC3-FATIGUE				7.50E+26 FATIGUE+FOS		
	1.16E+08 SUBSUBC3-CRUSHING				1.16E+08 CRUSHING		
	7.50E+26 SUBSUBC3-FOS				7.50E+26 FATIGUE+FOS		
LAYER 5	2.35E+14 SUBGRADE				2.35E+14 SUBGRADE		
	2.75E+06 MINIMUM			MINIMUM	2.75E+06	7.5E+26 MAXIMUM	

TRUN29		POISSON'S	LAYER	LAYER	E. x H	MODULAR	TRUN29
LAYER	E.MOD	RATIO	THICKNESS	RATIO		RATIOS	
1	3999	0.44	50	1/2	199.95	2.962222	
2	450	0.35	150	2/3	67.5	0.075012	
3	5999	0.35	150	3/4	899.85	1	
4	5999	0.35	150	4/5	899.85	0.012855	
5	70	0.35	1000000		70000		
	SINGLE CRITERIA LIFE				COMBINED FAILURE CRITERIA		
LAYER 1	2.42E+06 ASPH				2.42E+06 ASPHALT		
LAYER 2	2.67E+15 G1				2.67E+15 G1		
LAYER 3	6.57E+07 SUBC3-FATIGUE				5.61E+12 FATIGUE+FOS		
	6.54E+07 SUBC3-CRUSHING				6.54E+07 CRUSHING		
	5.61E+12 SUBC3-FOS				5.61E+12 FATIGUE+FOS		
LAYER 4	4.52E+07 SUBSUBC3-FATIGUE				5.94E+26 FATIGUE+FOS		
	1.17E+08 SUBSUBC3-CRUSHING				1.17E+08 CRUSHING		
	5.94E+26 SUBSUBC3-FOS				5.94E+26 FATIGUE+FOS		
LAYER 5	2.98E+14 SUBGRADE				2.98E+14 SUBGRADE		
	2.42E+06 MINIMUM			MINIMUM	2.42E+06	5.9E+26 MAXIMUM	

TRUN106		POISSON'S	LAYER	LAYER	E. x H	MODULAR	TRUN106
LAYER	E.MOD	RATIO	THICKNESS	RATIO		RATIOS	
1	5999	0.44	50	1/2	299.95	4.443703	
2	450	0.35	150	2/3	67.5	0.075012	
3	5999	0.35	150	3/4	899.85	1	
4	5999	0.35	150	4/5	899.85	0.012855	
5	70	0.35	1000000		70000		
	SINGLE CRITERIA LIFE				COMBINED FAILURE CRITERIA		
LAYER 1	5.32E+05 ASPH				5.32E+05 ASPHALT		
LAYER 2	4.15E+16 G1				4.15E+16 G1		
LAYER 3	6.57E+07 SUBC3-FATIGUE				4.25E+12 FATIGUE+FOS		
	6.79E+07 SUBC3-CRUSHING				6.79E+07 CRUSHING		
	4.25E+12 SUBC3-FOS				4.25E+12 FATIGUE+FOS		
LAYER 4	4.55E+07 SUBSUBC3-FATIGUE				4.06E+26 FATIGUE+FOS		
	1.18E+08 SUBSUBC3-CRUSHING				1.18E+08 CRUSHING		
	4.06E+26 SUBSUBC3-FOS				4.06E+26 FATIGUE+FOS		
LAYER 5	4.26E+14 SUBGRADE				4.26E+14 SUBGRADE		
	5.32E+05 MINIMUM			MINIMUM	5.32E+05	4.1E+26 MAXIMUM	

L2

TRUN107		POISSON'S	LAYER	LAYER	E. x H	MODULAR	TRUN107
LAYER	E.MOD	RATIO	THICKNESS	RATIO		RATIOS	
1	2999	0.44	50	1/2	149.95	3.998666	
2	250	0.35	150	2/3	37.5	0.041673	
3	5999	0.35	150	3/4	899.85	1	
4	5999	0.35	150	4/5	899.85	0.012855	
5	70	0.35	1000000		70000		
	SINGLE CRITERIA LIFE				COMBINED FAILURE CRITERIA		
LAYER 1	3.71E+05 ASPH				3.71E+05 ASPHALT		
LAYER 2	6.44E+15 G1				6.44E+15 G1		
LAYER 3	6.54E+07 SUBC3-FATIGUE				3.16E+14 FATIGUE+FOS		
	5.84E+07 SUBC3-CRUSHING				5.84E+07 CRUSHING		
	3.16E+14 SUBC3-FOS				3.16E+14 FATIGUE+FOS		
LAYER 4	4.34E+07 SUBSUBC3-FATIGUE				2.24E+27 FATIGUE+FOS		
	1.15E+08 SUBSUBC3-CRUSHING				1.15E+08 CRUSHING		
	2.24E+27 SUBSUBC3-FOS				2.24E+27 FATIGUE+FOS		
LAYER 5	1.02E+14 SUBGRADE				1.02E+14 SUBGRADE		
	3.71E+05 MINIMUM			MINIMUM	3.71E+05	2.2E+27 MAXIMUM	

TRUN20		POISSON'S	LAYER	LAYER	E. x H	MODULAR	TRUN20
LAYER	E.MOD	RATIO	THICKNESS	RATIO		RATIOS	
1	2999	0.44	50	1/2	149.95	2.221481	
2	450	0.35	150	2/3	67.5	0.075012	
3	5999	0.35	150	3/4	899.85	1	
4	5999	0.35	150	4/5	899.85	0.012855	
5	70	0.35	1000000		70000		
	SINGLE CRITERIA LIFE				COMBINED FAILURE CRITERIA		
LAYER 1	2.75E+06 ASPH				2.75E+06 ASPHALT		
LAYER 2	5.44E+14 G1				5.44E+14 G1		
LAYER 3	6.57E+07 SUBC3-FATIGUE				7.71E+12 FATIGUE+FOS		
	6.38E+07 SUBC3-CRUSHING				6.38E+07 CRUSHING		
	7.71E+12 SUBC3-FOS				7.71E+12 FATIGUE+FOS		
LAYER 4	4.50E+07 SUBSUBC3-FATIGUE				7.50E+26 FATIGUE+FOS		
	1.16E+08 SUBSUBC3-CRUSHING				1.16E+08 CRUSHING		
	7.50E+26 SUBSUBC3-FOS				7.50E+26 FATIGUE+FOS		
LAYER 5	2.35E+14 SUBGRADE				2.35E+14 SUBGRADE		
	2.75E+06 MINIMUM			MINIMUM	2.75E+06	7.5E+26 MAXIMUM	

TRUN21		POISSON'S	LAYER	LAYER	E. x H	MODULAR	TRUN21
LAYER	E.MOD	RATIO	THICKNESS	RATIO		RATIOS	
1	2999	0.44	50	1/2	149.95	1.110740	
2	900	0.35	150	2/3	135	0.150025	
3	5999	0.35	150	3/4	899.85	1	
4	5999	0.35	150	4/5	899.85	0.012855	
5	70	0.35	1000000		70000		
	SINGLE CRITERIA LIFE				COMBINED FAILURE CRITERIA		
LAYER 1	5.55E+07 ASPH				5.55E+07 ASPHALT		
LAYER 2	4.37E+14 G1				4.37E+14 G1		
LAYER 3	6.62E+07 SUBC3-FATIGUE				1.08E+11 FATIGUE+FOS		
	6.61E+07 SUBC3-CRUSHING				6.61E+07 CRUSHING		
	1.08E+11 SUBC3-FOS				1.08E+11 FATIGUE+FOS		
LAYER 4	4.72E+07 SUBSUBC3-FATIGUE				1.28E+26 FATIGUE+FOS		
	1.19E+08 SUBSUBC3-CRUSHING				1.19E+08 CRUSHING		
	1.28E+26 SUBSUBC3-FOS				1.28E+26 FATIGUE+FOS		
LAYER 5	7.16E+14 SUBGRADE				7.16E+14 SUBGRADE		
	4.72E+07 MINIMUM			MINIMUM	5.55E+07	1.3E+26 MAXIMUM	

L3

TRUN37		POISSON'S	LAYER	LAYER	E. x H	MODULAR	TRUN37
LAYER	E.MOD	RATIO	THICKNESS	RATIO		RATIOS	
1	2999	0.44	50	1/2	149.95	2.221481	
2	450	0.35	150	2/3	67.5	0.150050	
3	2999	0.35	150	3/4	449.85	0.499916	
4	5999	0.35	150	4/5	899.85	0.012855	
5	70	0.35	1000000		70000		
	SINGLE CRITERIA LIFE				COMBINED FAILURE CRITERIA		
LAYER 1	2.87E+06	ASPH			2.87E+06	ASPHALT	
LAYER 2	5.44E+14	G1			5.44E+14	G1	
LAYER 3	5.50E+07	SUBC3-FATIGUE			7.91E+14	FATIGUE+FOS	
	6.31E+07	SUBC3-CRUSHING			6.31E+07	CRUSHING	
	7.91E+14	SUBC3-FOS			7.91E+14	FATIGUE+FOS	
LAYER 4	4.33E+07	SUBSUBC3-FATIGUE			2.21E+27	FATIGUE+FOS	
	1.12E+08	SUBSUBC3-CRUSHING			1.12E+08	CRUSHING	
	2.21E+27	SUBSUBC3-FOS			2.21E+27	FATIGUE+FOS	
LAYER 5	7.19E+13	SUBGRADE			7.19E+13	SUBGRADE	
	2.87E+06	MINIMUM		MINIMUM	2.87E+06	2.2E+27	MAXIMUM

TRUN20		POISSON'S	LAYER	LAYER	E. x H	MODULAR	TRUN20
LAYER	E.MOD	RATIO	THICKNESS	RATIO		RATIOS	
1	2999	0.44	50	1/2	149.95	2.221481	
2	450	0.35	150	2/3	67.5	0.075012	
3	5999	0.35	150	3/4	899.85	1	
4	5999	0.35	150	4/5	899.85	0.012855	
5	70	0.35	1000000		70000		
	SINGLE CRITERIA LIFE				COMBINED FAILURE CRITERIA		
LAYER 1	2.75E+06	ASPH			2.75E+06	ASPHALT	
LAYER 2	5.44E+14	G1			5.44E+14	G1	
LAYER 3	6.57E+07	SUBC3-FATIGUE			7.71E+12	FATIGUE+FOS	
	6.38E+07	SUBC3-CRUSHING			6.38E+07	CRUSHING	
	7.71E+12	SUBC3-FOS			7.71E+12	FATIGUE+FOS	
LAYER 4	4.50E+07	SUBSUBC3-FATIGUE			7.50E+26	FATIGUE+FOS	
	1.16E+08	SUBSUBC3-CRUSHING			1.16E+08	CRUSHING	
	7.50E+26	SUBSUBC3-FOS			7.50E+26	FATIGUE+FOS	
LAYER 5	2.35E+14	SUBGRADE			2.35E+14	SUBGRADE	
	2.75E+06	MINIMUM		MINIMUM	2.75E+06	7.5E+26	MAXIMUM

TRUN100		POISSON'S	LAYER	LAYER	E. x H	MODULAR	TRUN100
LAYER	E.MOD	RATIO	THICKNESS	RATIO		RATIOS	
1	2999	0.44	50	1/2	149.95	2.221481	
2	450	0.35	150	2/3	67.5	0.045009	
3	9998	0.35	150	3/4	1499.7	1.666611	
4	5999	0.35	150	4/5	899.85	0.012855	
5	70	0.35	1000000		70000		
	SINGLE CRITERIA LIFE				COMBINED FAILURE CRITERIA		
LAYER 1	2.64E+06	ASPH			2.64E+06	ASPHALT	
LAYER 2	4.15E+14	G1			4.15E+14	G1	
LAYER 3	6.96E+07	SUBC3-FATIGUE			7.71E+10	FATIGUE+FOS	
	4.87E+07	SUBC3-CRUSHING			4.87E+07	CRUSHING	
	7.70E+10	SUBC3-FOS			7.71E+10	FATIGUE+FOS	
LAYER 4	4.64E+07	SUBSUBC3-FATIGUE			2.54E+26	FATIGUE+FOS	
	1.20E+08	SUBSUBC3-CRUSHING			1.20E+08	CRUSHING	
	2.54E+26	SUBSUBC3-FOS			2.54E+26	FATIGUE+FOS	
LAYER 5	5.68E+14	SUBGRADE			5.68E+14	SUBGRADE	
	2.64E+06	MINIMUM		MINIMUM	2.64E+06	2.5E+26	MAXIMUM

L4

TRUN34		POISSON'S	LAYER	LAYER	E. x H	MODULAR	TRUN34
LAYER	E.MOD	RATIO	THICKNESS	RATIO		RATIOS	
1	2999	0.44	50	1/2	149.95	2.221481	
2	450	0.35	150	2/3	67.5	0.075012	
3	5999	0.35	150	3/4	899.85	2.000333	
4	2999	0.35	150	4/5	449.85	0.006426	
5	70	0.35	1000000		70000		

	SINGLE CRITERIA LIFE			COMBINED FAILURE CRITERIA	
LAYER 1	2.92E+06	ASPH		2.92E+06	ASPHALT
LAYER 2	7.96E+14	G1		7.96E+14	G1
LAYER 3	6.22E+07	SUBC3-FATIGUE		1.19E+08	FATIGUE+FOS
	6.19E+07	SUBC3-CRUSHING		6.19E+07	CRUSHING
	5.65E+07	SUBC3-FOS		1.19E+08	FATIGUE+FOS
LAYER 4	3.50E+07	SUBSUBC3-FATIGUE		4.48E+24	FATIGUE+FOS
	1.19E+08	SUBSUBC3-CRUSHING		1.19E+08	CRUSHING
	4.48E+24	SUBSUBC3-FOS		4.48E+24	FATIGUE+FOS
LAYER 5	1.11E+13	SUBGRADE		1.11E+13	SUBGRADE
	2.92E+06	MINIMUM	MINIMUM	2.92E+06	4.5E+24 MAXIMUM

TRUN20		POISSON'S	LAYER	LAYER	E. x H	MODULAR	TRUN20
LAYER	E.MOD	RATIO	THICKNESS	RATIO		RATIOS	
1	2999	0.44	50	1/2	149.95	2.221481	
2	450	0.35	150	2/3	67.5	0.075012	
3	5999	0.35	150	3/4	899.85	1	
4	5999	0.35	150	4/5	899.85	0.012855	
5	70	0.35	1000000		70000		

	SINGLE CRITERIA LIFE			COMBINED FAILURE CRITERIA	
LAYER 1	2.75E+06	ASPH		2.75E+06	ASPHALT
LAYER 2	5.44E+14	G1		5.44E+14	G1
LAYER 3	6.57E+07	SUBC3-FATIGUE		7.71E+12	FATIGUE+FOS
	6.38E+07	SUBC3-CRUSHING		6.38E+07	CRUSHING
	7.71E+12	SUBC3-FOS		7.71E+12	FATIGUE+FOS
LAYER 4	4.50E+07	SUBSUBC3-FATIGUE		7.50E+26	FATIGUE+FOS
	1.16E+08	SUBSUBC3-CRUSHING		1.16E+08	CRUSHING
	7.50E+26	SUBSUBC3-FOS		7.50E+26	FATIGUE+FOS
LAYER 5	2.35E+14	SUBGRADE		2.35E+14	SUBGRADE
	2.75E+06	MINIMUM	MINIMUM	2.75E+06	7.5E+26 MAXIMUM

TRUN101		POISSON'S	LAYER	LAYER	E. x H	MODULAR	TRUN101
LAYER	E.MOD	RATIO	THICKNESS	RATIO		RATIOS	
1	2999	0.44	50	1/2	149.95	2.221481	
2	450	0.35	150	2/3	67.5	0.075012	
3	5999	0.35	150	3/4	899.85	0.600020	
4	9998	0.35	150	4/5	1499.7	0.021424	
5	70	0.35	1000000		70000		

	SINGLE CRITERIA LIFE			COMBINED FAILURE CRITERIA	
LAYER 1	2.67E+06	ASPH		2.67E+06	ASPHALT
LAYER 2	4.35E+14	G1		4.35E+14	G1
LAYER 3	6.57E+07	SUBC3-FATIGUE		6.85E+15	FATIGUE+FOS
	6.45E+07	SUBC3-CRUSHING		6.45E+07	CRUSHING
	6.85E+15	SUBC3-FOS		6.85E+15	FATIGUE+FOS
LAYER 4	5.21E+07	SUBSUBC3-FATIGUE		7.41E+27	FATIGUE+FOS
	1.12E+08	SUBSUBC3-CRUSHING		1.12E+08	CRUSHING
	7.41E+27	SUBSUBC3-FOS		7.41E+27	FATIGUE+FOS
LAYER 5	2.15E+15	SUBGRADE		2.15E+15	SUBGRADE
	2.67E+06	MINIMUM	MINIMUM	2.67E+06	7.4E+27 MAXIMUM

L5

TRUN31 LAYER	E.MOD	POISSON'S RATIO	LAYER THICKNESS	LAYER RATIO	E. x H	MODULAR RATIOS	TRUN31
1	2999	0.44	50	1/2	149.95	2.221481	
2	450	0.35	150	2/3	67.5	0.075012	
3	5999	0.35	150	3/4	899.85	1	
4	5999	0.35	150	4/5	899.85	0.035994	
5	25	0.35	1000000		25000		

	SINGLE CRITERIA LIFE		COMBINED FAILURE CRITERIA	
LAYER 1	3.61E+06 ASPH		3.61E+06 ASPHALT	
LAYER 2	4.05E+15 G1		4.05E+15 G1	
LAYER 3	6.47E+07 SUBC3-FATIGUE		7.35E+12 FATIGUE+FOS	
	5.51E+07 SUBC3-CRUSHING		5.51E+07 CRUSHING	
	7.35E+12 SUBC3-FOS		7.35E+12 FATIGUE+FOS	
LAYER 4	3.90E+07 SUBSUBC3-FATIGUE		1.90E+27 FATIGUE+FOS	
	1.18E+08 SUBSUBC3-CRUSHING		1.18E+08 CRUSHING	
	1.90E+27 SUBSUBC3-FOS		1.90E+27 FATIGUE+FOS	
LAYER 5	1.50E+12 SUBGRADE		1.50E+12 SUBGRADE	
	3.61E+06 MINIMUM	MINIMUM	3.61E+06	1.9E+27 MAXIMUM

TRUN20 LAYER	E.MOD	POISSON'S RATIO	LAYER THICKNESS	LAYER RATIO	E. x H	MODULAR RATIOS	TRUN20
1	2999	0.44	50	1/2	149.95	2.221481	
2	450	0.35	150	2/3	67.5	0.075012	
3	5999	0.35	150	3/4	899.85	1	
4	5999	0.35	150	4/5	899.85	0.012855	
5	70	0.35	1000000		70000		

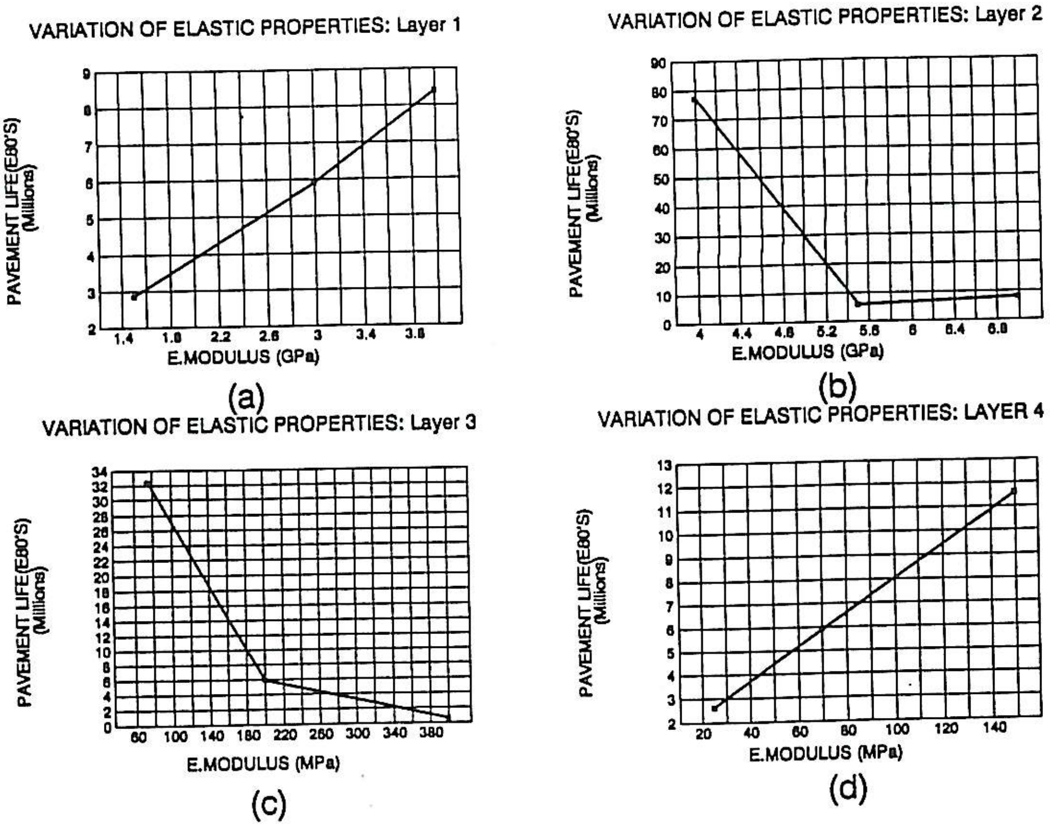
	SINGLE CRITERIA LIFE		COMBINED FAILURE CRITERIA	
LAYER 1	2.75E+06 ASPH		2.75E+06 ASPHALT	
LAYER 2	5.44E+14 G1		5.44E+14 G1	
LAYER 3	6.57E+07 SUBC3-FATIGUE		7.71E+12 FATIGUE+FOS	
	6.38E+07 SUBC3-CRUSHING		6.38E+07 CRUSHING	
	7.71E+12 SUBC3-FOS		7.71E+12 FATIGUE+FOS	
LAYER 4	4.50E+07 SUBSUBC3-FATIGUE		7.50E+26 FATIGUE+FOS	
	1.16E+08 SUBSUBC3-CRUSHING		1.16E+08 CRUSHING	
	7.50E+26 SUBSUBC3-FOS		7.50E+26 FATIGUE+FOS	
LAYER 5	2.35E+14 SUBGRADE		2.35E+14 SUBGRADE	
	2.75E+06 MINIMUM	MINIMUM	2.75E+06	7.5E+26 MAXIMUM

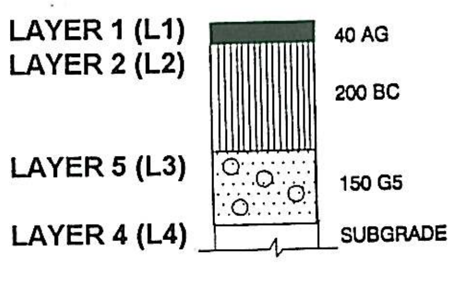
TRUN32 LAYER	E.MOD	POISSON'S RATIO	LAYER THICKNESS	LAYER RATIO	E. x H	MODULAR RATIOS	TRUN32
1	2999	0.44	50	1/2	149.95	2.221481	
2	450	0.35	150	2/3	67.5	0.075012	
3	5999	0.35	150	3/4	899.85	1	
4	5999	0.35	150	4/5	899.85	0.005999	
5	150	0.35	1000000		150000		

	SINGLE CRITERIA LIFE		COMBINED FAILURE CRITERIA	
LAYER 1	2.37E+06 ASPH		2.37E+06 ASPHALT	
LAYER 2	1.74E+14 G1		1.74E+14 G1	
LAYER 3	6.67E+07 SUBC3-FATIGUE		1.41E+13 FATIGUE+FOS	
	6.62E+07 SUBC3-CRUSHING		6.62E+07 CRUSHING	
	1.41E+13 SUBC3-FOS		1.41E+13 FATIGUE+FOS	
LAYER 4	4.96E+07 SUBSUBC3-FATIGUE		8.99E+26 FATIGUE+FOS	
	1.15E+08 SUBSUBC3-CRUSHING		1.15E+08 CRUSHING	
	8.99E+26 SUBSUBC3-FOS		8.99E+26 FATIGUE+FOS	
LAYER 5	1.07E+16 SUBGRADE		1.07E+16 SUBGRADE	
	2.37E+06 MINIMUM	MINIMUM	2.37E+06	9.0E+26 MAXIMUM

SENSITIVITY OF PAVEMENT LIFE TO CHANGES IN ELASTIC MODULI

AAAGS





LAYER	ELASTIC MODULUS (MPa)	FAILURE IN:		FIGURE
		MATERIAL	LAYER	
1	1500 3000 4000	Asphalt Asphalt Asphalt	L2 L2 L2	a
2	4000 5500 7000	Asphalt Asphalt Asphalt	L2 L2 L2	b
3	75 200 400	Asphalt Asphalt Granular	L2 L2 L3	c
4	25 70 150	Asphalt Asphalt Asphalt	L3 L1 L1	d

FIGURE A2
CALCULATED CHANGES IN PAVEMENT LIFE WITH CHANGES IN ELASTIC MODULI
PAVEMENT AAAGS (DETAILED ANALYSIS)

PAVEMENT
STRUCTURE: AAAGS

LAYER1				LAYER2			
VARIATION IN ELASTIC MODULUS		VARIATION IN PREDICTED LIFE		VARIATION IN ELASTIC MODULUS		VARIATION IN PREDICTED LIFE	
MPa	%	E80'S	%	MPa	%	E80'S	%
1500	-50.00	2.87E+06	-51.60	4000	-27.27	7.73E+07	1202.28
3000	Reference	5.93E+06	Reference	5500	Reference	5.93E+06	Reference
4000	33.33	8.42E+06	41.88	7000	27.27	8.05E+06	35.69

LAYER3				LAYER4			
VARIATION IN ELASTIC MODULUS		VARIATION IN PREDICTED LIFE		VARIATION IN ELASTIC MODULUS		VARIATION IN PREDICTED LIFE	
MPa	%	E80'S	%	MPa	%	E80'S	%
75	-62.50	3.24E+07	445.81	25	-64.29	2.62E+06	-55.77
200	Reference	5.93E+06	Reference	70	Reference	5.93E+06	Reference
400	100.00	5.89E+05	-90.08	150	114.29	1.16E+07	95.07

MATERIAL	RECOMMENDED E (MPa)
L1 ASPHALT AG	3000
L2 ASPHALT BC	5500
L3 GRANULAR G5	200
L4 SUBGRADE	70

Note: To calculate the 'variation in predicted life' the recommended mid-range values of elastic modulus were used.

L1 AAAGS

AAAGS26						
LAYER	E.MOD	POISSON'S RATIO	LAYER THICKNESS	LAYER RATIO	E. x H	MODULAR RATIOS
1	1500	0.44	40	1/2	60	0.0545553
2	5499	0.44	200	2/3	1099.8	36.66
3	200	0.35	150	3/4	30	0.0004285
4	70	0.35	1000000	4/5	70000	

	SINGLE CRITERIA LIFE		COMBINED FAILURE CRITERIA	
LAYER 1	2.91E+08 ASPH		2.91E+08 ASPHALT	
LAYER 2	2.87E+06 ASPH-FATIGUE		2.87E+06 ASPHALT	
LAYER 3	5.04E+13 SUBSUB G5-FOS		5.04E+13 GRANULAR	
LAYER 4	6.42E+09 SUBGRADE		6.42E+09 SUBGRADE	
	2.87E+06 MINIMUM	MINIMUM	2.87E+06	5.04E+13 MAXIMUM

AAAGS17						
LAYER	E.MOD	POISSON'S RATIO	LAYER THICKNESS	LAYER RATIO	E. x H	MODULAR RATIOS
1	2999	0.44	40	1/2	119.96	0.1090743
2	5499	0.44	200	2/3	1099.8	36.66
3	200	0.35	150	3/4	30	0.0004285
4	70	0.35	1000000	4/5	70000	

	SINGLE CRITERIA LIFE		COMBINED FAILURE CRITERIA	
LAYER 1	1.68E+08 ASPH		1.68E+08 ASPHALT	
LAYER 2	5.93E+06 ASPH-FATIGUE		5.93E+06 ASPHALT	
LAYER 3	3.46E+14 SUBSUB G5-FOS		3.46E+14 GRANULAR	
LAYER 4	1.48E+10 SUBGRADE		1.48E+10 SUBGRADE	
	5.93E+06 MINIMUM	MINIMUM	5.93E+06	3.46E+14 MAXIMUM

AAAGS25						
LAYER	E.MOD	POISSON'S RATIO	LAYER THICKNESS	LAYER RATIO	E. x H	MODULAR RATIOS
1	3999	0.44	40	1/2	159.96	0.1454446
2	5499	0.44	200	2/3	1099.8	36.66
3	200	0.35	150	3/4	30	0.0004285
4	70	0.35	1000000	4/5	70000	

	SINGLE CRITERIA LIFE		COMBINED FAILURE CRITERIA	
LAYER 1	1.99E+08 ASPH		1.99E+08 ASPHALT	
LAYER 2	8.42E+06 ASPH-FATIGUE		8.42E+06 ASPHALT	
LAYER 3	1.01E+15 SUBSUB G5-FOS		1.01E+15 GRANULAR	
LAYER 4	2.33E+10 SUBGRADE		2.33E+10 SUBGRADE	
	8.42E+06 MINIMUM	MINIMUM	8.42E+06	1.01E+15 MAXIMUM

L2

AAAGS24 LAYER	E.MOD	POISSON'S RATIO	LAYER THICKNESS	LAYER RATIO	E. x H	MODULAR RATIOS
1	2999	0.44	40	1/2	119.96	0.1499874
2	3999	0.44	200	2/3	799.8	26.66
3	200	0.35	150	3/4	30	0.0004285
4	70	0.35	1000000	4/5	70000	

	SINGLE CRITERIA LIFE		COMBINED FAILURE CRITERIA	
LAYER 1	1.07E+08 ASPH		1.07E+08 ASPHALT	
LAYER 2	7.73E+07 ASPH-FATIGUE		7.73E+07 ASPHALT	
LAYER 3	1.00E+12 SUBSUB G5-FOS		1.00E+12 GRANULAR	
LAYER 4	2.56E+09 SUBGRADE		2.56E+09 SUBGRADE	
	7.73E+07 MINIMUM	MINIMUM	7.73E+07	1.00E+12 MAXIMUM

AAAGS17 LAYER	E.MOD	POISSON'S RATIO	LAYER THICKNESS	LAYER RATIO	E. x H	MODULAR RATIOS
1	2999	0.44	40	1/2	119.96	0.1090743
2	5499	0.44	200	2/3	1099.8	36.66
3	200	0.35	150	3/4	30	0.0004285
4	70	0.35	1000000	4/5	70000	

	SINGLE CRITERIA LIFE		COMBINED FAILURE CRITERIA	
LAYER 1	1.68E+08 ASPH		1.68E+08 ASPHALT	
LAYER 2	5.93E+06 ASPH-FATIGUE		5.93E+06 ASPHALT	
LAYER 3	3.46E+14 SUBSUB G5-FOS		3.46E+14 GRANULAR	
LAYER 4	1.48E+10 SUBGRADE		1.48E+10 SUBGRADE	
	5.93E+06 MINIMUM	MINIMUM	5.93E+06	3.46E+14 MAXIMUM

AAAGS27 LAYER	E.MOD	POISSON'S RATIO	LAYER THICKNESS	LAYER RATIO	E. x H	MODULAR RATIOS
1	2999	0.44	40	1/2	119.96	0.0857102
2	6998	0.44	200	2/3	1399.6	46.653333
3	200	0.35	150	3/4	30	0.0004285
4	70	0.35	1000000	4/5	70000	

	SINGLE CRITERIA LIFE		COMBINED FAILURE CRITERIA	
LAYER 1	2.51E+08 ASPH		2.51E+08 ASPHALT	
LAYER 2	8.05E+06 ASPH-FATIGUE		8.05E+06 ASPHALT	
LAYER 3	7.23E+16 SUBSUB G5-FOS		7.23E+16 GRANULAR	
LAYER 4	6.00E+10 SUBGRADE		6.00E+10 SUBGRADE	
	8.05E+06 MINIMUM	MINIMUM	8.05E+06	7.23E+16 MAXIMUM

L3

AAAGS28 LAYER	E.MOD	POISSON'S RATIO	LAYER THICKNESS	LAYER RATIO	E. x H	MODULAR RATIOS
1	2999	0.44	40	1/2	119.96	0.1090743
2	5499	0.44	200	2/3	1099.8	97.76
3	75	0.35	150	3/4	11.25	0.0001607
4	70	0.35	1000000	4/5	70000	

	SINGLE CRITERIA LIFE		COMBINED FAILURE CRITERIA	
LAYER 1	1.28E+08 ASPH		1.28E+08 ASPHALT	
LAYER 2	3.24E+07 ASPH-FATIGUE		3.24E+07 ASPHALT	
LAYER 3	3.24E+33 SUBSUB G5-FOS		3.24E+33 GRANULAR	
LAYER 4	5.52E+10 SUBGRADE		5.52E+10 SUBGRADE	
	3.24E+07 MINIMUM	MINIMUM	3.24E+07	3.24E+33 MAXIMUM

AAAGS17 LAYER	E.MOD	POISSON'S RATIO	LAYER THICKNESS	LAYER RATIO	E. x H	MODULAR RATIOS
1	2999	0.44	40	1/2	119.96	0.1090743
2	5499	0.44	200	2/3	1099.8	36.66
3	200	0.35	150	3/4	30	0.0004285
4	70	0.35	1000000	4/5	70000	

	SINGLE CRITERIA LIFE		COMBINED FAILURE CRITERIA	
LAYER 1	1.68E+08 ASPH		1.68E+08 ASPHALT	
LAYER 2	5.93E+06 ASPH-FATIGUE		5.93E+06 ASPHALT	
LAYER 3	3.46E+14 SUBSUB G5-FOS		3.46E+14 GRANULAR	
LAYER 4	1.48E+10 SUBGRADE		1.48E+10 SUBGRADE	
	5.93E+06 MINIMUM	MINIMUM	5.93E+06	3.46E+14 MAXIMUM

AAAGS29 LAYER	E.MOD	POISSON'S RATIO	LAYER THICKNESS	LAYER RATIO	E. x H	MODULAR RATIOS
1	2999	0.44	40	1/2	119.96	0.1090743
2	5499	0.44	200	2/3	1099.8	18.33
3	400	0.35	150	3/4	60	0.0008571
4	70	0.35	1000000	4/5	70000	

	SINGLE CRITERIA LIFE		COMBINED FAILURE CRITERIA	
LAYER 1	2.16E+08 ASPH		2.16E+08 ASPHALT	
LAYER 2	7.35E+07 ASPH-FATIGUE		7.35E+07 ASPHALT	
LAYER 3	5.89E+05 SUBSUB G5-FOS		5.89E+05 GRANULAR	
LAYER 4	1.75E+10 SUBGRADE		1.75E+10 SUBGRADE	
	5.89E+05 MINIMUM	MINIMUM	5.89E+05	1.75E+10 MAXIMUM

L4

AAAGS22						
LAYER	E.MOD	POISSON'S RATIO	LAYER THICKNESS	LAYER RATIO	E. x H	MODULAR RATIOS
1	2999	0.44	40	1/2	119.96	0.1090743
2	5499	0.44	200	2/3	1099.8	36.66
3	200	0.35	150	3/4	30	0.0012
4	25	0.35	1000000	4/5	25000	

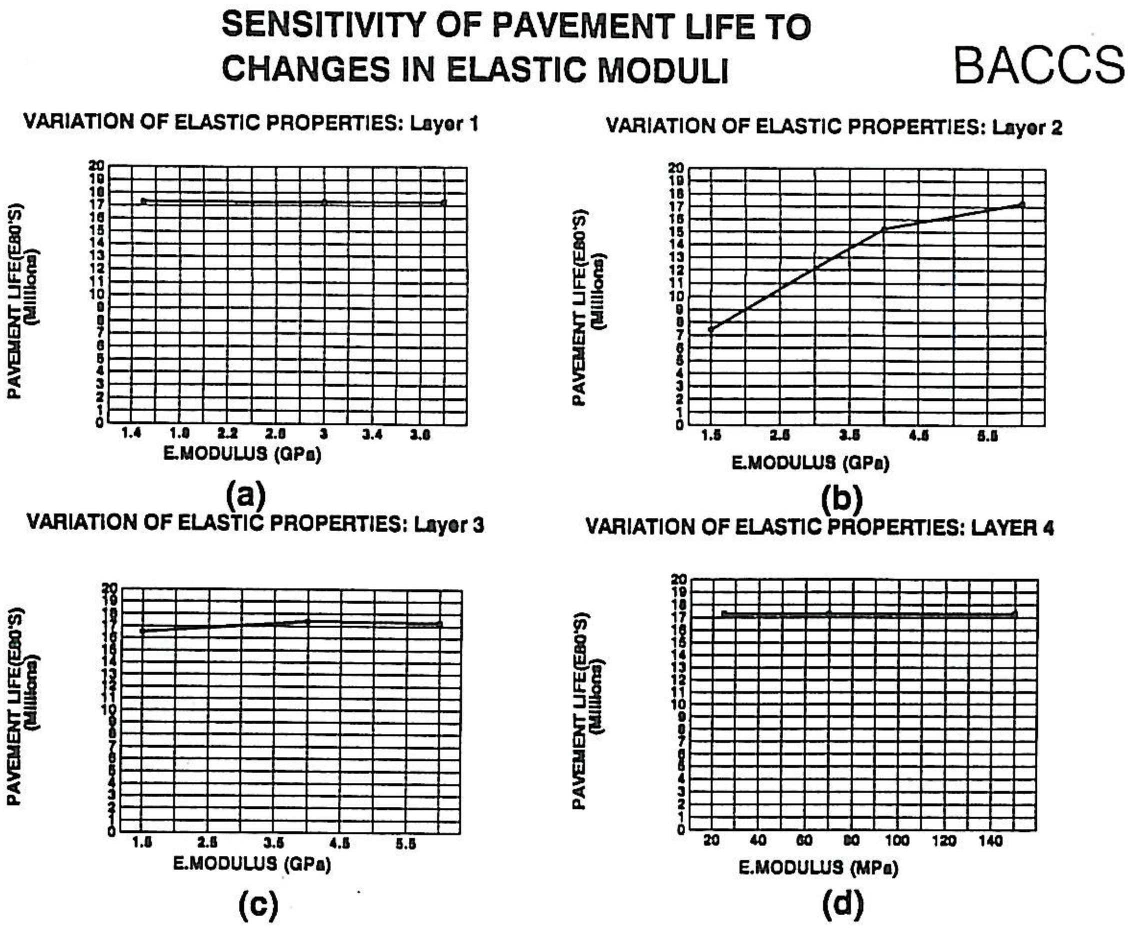
	SINGLE CRITERIA LIFE		COMBINED FAILURE CRITERIA	
LAYER 1	5.73E+07 ASPH		5.73E+07 ASPHALT	
LAYER 2	2.62E+06 ASPH-FATIGUE		2.62E+06 ASPHALT	
LAYER 3	2.62E+09 SUBSUB G5-FOS		2.62E+09 GRANULAR	
LAYER 4	2.65E+07 SUBGRADE		2.65E+07 SUBGRADE	
	2.62E+06 MINIMUM	MINIMUM	2.62E+06	2.62E+09 MAXIMUM

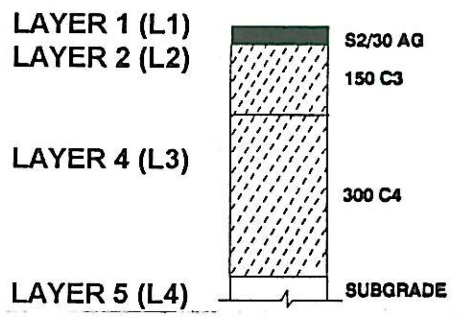
AAAGS17						
LAYER	E.MOD	POISSON'S RATIO	LAYER THICKNESS	LAYER RATIO	E. x H	MODULAR RATIOS
1	2999	0.44	40	1/2	119.96	0.1090743
2	5499	0.44	200	2/3	1099.8	36.66
3	200	0.35	150	3/4	30	0.0004285
4	70	0.35	1000000	4/5	70000	

	SINGLE CRITERIA LIFE		COMBINED FAILURE CRITERIA	
LAYER 1	1.68E+08 ASPH		1.68E+08 ASPHALT	
LAYER 2	5.93E+06 ASPH-FATIGUE		5.93E+06 ASPHALT	
LAYER 3	3.46E+14 SUBSUB G5-FOS		3.46E+14 GRANULAR	
LAYER 4	1.48E+10 SUBGRADE		1.48E+10 SUBGRADE	
	5.93E+06 MINIMUM	MINIMUM	5.93E+06	3.46E+14 MAXIMUM

AAAGS23						
LAYER	E.MOD	POISSON'S RATIO	LAYER THICKNESS	LAYER RATIO	E. x H	MODULAR RATIOS
1	2999	0.44	40	1/2	119.96	0.1090743
2	5499	0.44	200	2/3	1099.8	36.66
3	200	0.35	150	3/4	30	0.0002
4	150	0.35	1000000	4/5	150000	

	SINGLE CRITERIA LIFE		COMBINED FAILURE CRITERIA	
LAYER 1	4.04E+08 ASPH		4.04E+08 ASPHALT	
LAYER 2	1.16E+07 ASPH-FATIGUE		1.16E+07 ASPHALT	
LAYER 3	2.19E+18 SUBSUB G5-FOS		2.19E+18 GRANULAR	
LAYER 4	4.08E+12 SUBGRADE		4.08E+12 SUBGRADE	
	1.16E+07 MINIMUM	MINIMUM	1.16E+07	2.19E+18 MAXIMUM





LAYER	ELASTIC MODULUS (MPa)	FAILURE IN:		FIGURE
		MATERIAL	LAYER	
1	1500 3000 4000	C3 (Crushing) C3 (Crushing) C3 (Crushing)	L2 L2 L2	a
2	1500 4000 6000	C3 (Crushing) C3 (Crushing) C3 (Crushing)	L2 L2 L2	b
3	1500 4000 6000	C3 (Crushing) C3 (Crushing) C3 (Crushing)	L2 L2 L2	c
4	25 70 150	C3 (Crushing) C3 (Crushing) C3 (Crushing)	L2 L2 L2	d

FIGURE A3
CALCULATED CHANGES IN PAVEMENT LIFE WITH CHANGES IN ELASTIC MODULI
PAVEMENT BACCS (DETAILED ANALYSIS)

PAVEMENT
STRUCTURE: BACCS

LAYER1				LAYER2			
VARIATION IN ELASTIC MODULUS		VARIATION IN PREDICTED LIFE		VARIATION IN ELASTIC MODULUS		VARIATION IN PREDICTED LIFE	
MPa	%	E80'S	%	MPa	%	E80'S	%
1500	50.00	1.73E+07	0.04	1500	-75.000	7.43E+06	-57.070
3000	Reference	1.73E+07	Reference	4000	-33.333	1.53E+07	-11.765
4000	33.33	1.73E+07	-0.04	6000	Reference	1.73E+07	Reference

LAYER3				LAYER4			
VARIATION IN ELASTIC MODULUS		VARIATION IN PREDICTED LIFE		VARIATION IN ELASTIC MODULUS		VARIATION IN PREDICTED LIFE	
MPa	%	E80'S	%	MPa	%	E80'S	%
1500	-75	1.65E+07	-4.51272	25	-64.2857	1.73E+07	0
4000	-33.333	1.74E+07	0.490	70	Reference	1.73E+07	Reference
6000	Reference	1.73E+07	Reference	150	114.286	1.73E+07	0

MATERIAL	RECOMMENDED E (MPa)
L1 ASPHALT	3000
L2 CEMENTED C3	6000
L3 CEMENTED C4	6000
L4 SUBGRADE	70

Note: To calculate the 'variation in predicted life' the recommended mid-range values of elastic modulus were used.

PAVEMENT
STRUCTURE: BACCS

LAYER1				LAYER2			
VARIATION IN ELASTIC MODULUS		VARIATION IN PREDICTED LIFE		VARIATION IN ELASTIC MODULUS		VARIATION IN PREDICTED LIFE	
MPa	%	E80'S	%	MPa	%	E80'S	%
1500	50.00	1.73E+07	0.04	1500	-75.000	7.43E+06	-57.070
3000	Reference	1.73E+07	Reference	4000	-33.333	1.53E+07	-11.765
4000	33.33	1.73E+07	-0.04	6000	Reference	1.73E+07	Reference

LAYER3				LAYER4			
VARIATION IN ELASTIC MODULUS		VARIATION IN PREDICTED LIFE		VARIATION IN ELASTIC MODULUS		VARIATION IN PREDICTED LIFE	
MPa	%	E80'S	%	MPa	%	E80'S	%
1500	-75	1.65E+07	-4.51272	25	-64.2857	1.73E+07	0
4000	-33.333	1.74E+07	0.490	70	Reference	1.73E+07	Reference
6000	Reference	1.73E+07	Reference	150	114.286	1.73E+07	0

MATERIAL	RECOMMENDED E (MPa)
L1 ASPHALT	3000
L2 CEMENTED C3	6000
L3 CEMENTED C4	6000
L4 SUBGRADE	70

Note: To calculate the 'variation in predicted life' the recommended mid-range values of elastic modulus were used.

L1

BACCS20 LAYER	E.MOD	POISSON'S RATIO	LAYER THICKNESS	LAYER RATIO	E. x H	MODULAR RATIOS	BACCS20
1	1500	0.44	30	1/2	45	0.0500083	
2	5999	0.35	150	2/3	899.85	0.5000016	
3	5999	0.35	299.999	3/4	1799.6940	0.0257099	
4	70	0.35	1000000	4/5	70000		

	SINGLE CRITERIA LIFE			COMBINED FAILURE CRITERIA		
LAYER 1	2.43E+10	ASPH		2.43E+10	ASPHALT	
LAYER 2	6.57E+07	SUBC3-FATIGUE		1.02E+18	FATIGUE+FOS	
	1.73E+07	SUBC3-CRUSHING		1.73E+07	CRUSHING	
	1.02E+18	SUBC3-FOS		1.02E+18	FATIGUE+FOS	
LAYER 3	5.45E+07	SUBSUBC3-FATIGUE		2.08E+16	FATIGUE+FOS	
	6.89E+07	SUBSUBC3-CRUSHING		6.89E+07	CRUSHING	
	2.08E+16	SUBSUBC3-FOS		2.08E+16	FATIGUE+FOS	
LAYER 4	4.78E+12	SUBGRADE		4.78E+12	SUBGRADE	
	1.73E+07	MINIMUM	MINIMUM	1.73E+07	1.02E+18	MAXIMUM

BACCS17 LAYER	E.MOD	POISSON'S RATIO	LAYER THICKNESS	LAYER RATIO	E. x H	MODULAR RATIOS	BACCS17
1	2999	0.44	30	1/2	89.97	0.0999833	
2	5999	0.35	150	2/3	899.85	0.5000016	
3	5999-	0.35	299.999	3/4	1799.6940	0.0257099	
4	70	0.35	1000000	4/5	70000		

	SINGLE CRITERIA LIFE			COMBINED FAILURE CRITERIA		
LAYER 1	1.16E+10	ASPH		1.16E+10	ASPHALT	
LAYER 2	6.59E+07	SUBC3-FATIGUE		6.26E+16	FATIGUE+FOS	
	1.73E+07	SUBC3-CRUSHING		1.73E+07	CRUSHING	
	6.26E+16	SUBC3-FOS		6.26E+16	FATIGUE+FOS	
LAYER 3	5.61E+07	SUBSUBC3-FATIGUE		1.95E+16	FATIGUE+FOS	
	6.99E+07	SUBSUBC3-CRUSHING		6.99E+07	CRUSHING	
	1.95E+16	SUBSUBC3-FOS		1.95E+16	FATIGUE+FOS	
LAYER 4	4.78E+12	SUBGRADE		4.78E+12	SUBGRADE	
	1.73E+07	MINIMUM	MINIMUM	1.73E+07	6.26E+16	MAXIMUM

BACCS21 LAYER	E.MOD	POISSON'S RATIO	LAYER THICKNESS	LAYER RATIO	E. x H	MODULAR RATIOS	BACCS21
1	3999	0.44	30	1/2	119.97	0.1333222	
2	5999	0.35	150	2/3	899.85	0.5000016	
3	5999	0.35	299.999	3/4	1799.6940	0.0257099	
4	70	0.35	1000000	4/5	70000		

	SINGLE CRITERIA LIFE			COMBINED FAILURE CRITERIA		
LAYER 1	1.27E+10	ASPH		1.27E+10	ASPHALT	
LAYER 2	6.59E+07	SUBC3-FATIGUE		1.25E+16	FATIGUE+FOS	
	1.73E+07	SUBC3-CRUSHING		1.73E+07	CRUSHING	
	1.25E+16	SUBC3-FOS		1.25E+16	FATIGUE+FOS	
LAYER 3	5.52E+07	SUBSUBC3-FATIGUE		1.83E+16	FATIGUE+FOS	
	7.05E+07	SUBSUBC3-CRUSHING		7.05E+07	CRUSHING	
	1.83E+16	SUBSUBC3-FOS		1.83E+16	FATIGUE+FOS	
LAYER 4	4.78E+12	SUBGRADE		4.78E+12	SUBGRADE	
	1.73E+07	MINIMUM	MINIMUM	1.73E+07	1.83E+16	MAXIMUM

L2

BACCS22		POISSON'S	LAYER	LAYER	E. x H	MODULAR	BACCS22
LAYER	E.MOD	RATIO	THICKNESS	RATIO		RATIOS	
1	2999	0.44	30	1/2	89.97	0.3998666	
2	1500	0.35	150	2/3	225	0.1250212	
3	5999	0.35	299.999	3/4	1799.6940	0.0257099	
4	70	0.35	1000000	4/5	70000		

	SINGLE CRITERIA LIFE		COMBINED FAILURE CRITERIA	
LAYER 1	1.25E+09 ASPH		1.25E+09 ASPHALT	
LAYER 2	3.53E+07 SUBC3-FATIGUE		4.61E+10 FATIGUE+FOS	
	7.43E+06 SUBC3-CRUSHING		7.43E+06 CRUSHING	
	4.60E+10 SUBC3-FOS		4.61E+10 FATIGUE+FOS	
LAYER 3	5.08E+07 SUBSUBC3-FATIGUE		8.70E+19 FATIGUE+FOS	
	5.86E+07 SUBSUBC3-CRUSHING		5.86E+07 CRUSHING	
	8.70E+19 SUBSUBC3-FOS		8.70E+19 FATIGUE+FOS	
LAYER 4	4.78E+12 SUBGRADE		4.78E+12 SUBGRADE	
	7.43E+06 MINIMUM	MINIMUM	7.43E+06	8.70E+19 MAXIMUM

BACCS23		POISSON'S	LAYER	LAYER	E. x H	MODULAR	BACCS23
LAYER	E.MOD	RATIO	THICKNESS	RATIO		RATIOS	
1	2999	0.44	30	1/2	89.97	0.1499874	
2	3999	0.35	150	2/3	599.85	0.3333066	
3	5999	0.35	299.999	3/4	1799.6940	0.0257099	
4	70	0.35	1000000	4/5	70000		

	SINGLE CRITERIA LIFE		COMBINED FAILURE CRITERIA	
LAYER 1	5.92E+09 ASPH		5.92E+09 ASPHALT	
LAYER 2	5.86E+07 SUBC3-FATIGUE		3.14E+14 FATIGUE+FOS	
	1.53E+07 SUBC3-CRUSHING		1.53E+07 CRUSHING	
	3.14E+14 SUBC3-FOS		3.14E+14 FATIGUE+FOS	
LAYER 3	5.38E+07 SUBSUBC3-FATIGUE		1.99E+17 FATIGUE+FOS	
	6.57E+07 SUBSUBC3-CRUSHING		6.57E+07 CRUSHING	
	1.99E+17 SUBSUBC3-FOS		1.99E+17 FATIGUE+FOS	
LAYER 4	4.78E+12 SUBGRADE		4.78E+12 SUBGRADE	
	1.53E+07 MINIMUM	MINIMUM	1.53E+07	1.99E+17 MAXIMUM

BACCS17		POISSON'S	LAYER	LAYER	E. x H	MODULAR	BACCS17
LAYER	E.MOD	RATIO	THICKNESS	RATIO		RATIOS	
1	2999	0.44	30	1/2	89.97	0.0999833	
2	5999	0.35	150	2/3	899.85	0.5000016	
3	5999	0.35	299.999	3/4	1799.6940	0.0257099	
4	70	0.35	1000000	4/5	70000		

	SINGLE CRITERIA LIFE		COMBINED FAILURE CRITERIA	
LAYER 1	1.16E+10 ASPH		1.16E+10 ASPHALT	
LAYER 2	6.59E+07 SUBC3-FATIGUE		6.26E+16 FATIGUE+FOS	
	1.73E+07 SUBC3-CRUSHING		1.73E+07 CRUSHING	
	6.26E+16 SUBC3-FOS		6.26E+16 FATIGUE+FOS	
LAYER 3	5.61E+07 SUBSUBC3-FATIGUE		1.95E+16 FATIGUE+FOS	
	6.99E+07 SUBSUBC3-CRUSHING		6.99E+07 CRUSHING	
	1.95E+16 SUBSUBC3-FOS		1.95E+16 FATIGUE+FOS	
LAYER 4	4.78E+12 SUBGRADE		4.78E+12 SUBGRADE	
	1.73E+07 MINIMUM	MINIMUM	1.73E+07	6.26E+16 MAXIMUM

L3

BACCS24							BACCS24
LAYER	E.MOD	POISSON'S RATIO	LAYER THICKNESS	LAYER RATIO	E. x H	MODULAR RATIOS	
1	2999	0.44	30	1/2	89.97	0.0999833	
2	5999	0.35	150	2/3	899.85	1.9996733	
3	1500	0.35	299.999	3/4	449.9985	0.0064285	
4	70	0.35	1000000	4/5	70000		

	SINGLE CRITERIA LIFE			COMBINED FAILURE CRITERIA	
LAYER 1	2.14E+09	ASPH		2.14E+09	ASPHALT
LAYER 2	6.01E+07	SUBC3-FATIGUE		7.96E+11	FATIGUE+FOS
	1.65E+07	SUBC3-CRUSHING		1.65E+07	CRUSHING
	7.96E+11	SUBC3-FOS		7.96E+11	FATIGUE+FOS
LAYER 3	3.39E+07	SUBSUBC3-FATIGUE		7.80E+24	FATIGUE+FOS
	7.64E+07	SUBSUBC3-CRUSHING		7.64E+07	CRUSHING
	7.80E+24	SUBSUBC3-FOS		7.80E+24	FATIGUE+FOS
LAYER 4	4.78E+12	SUBGRADE		4.78E+12	SUBGRADE
	1.65E+07	MINIMUM	MINIMUM	1.65E+07	7.80E+24 MAXIMUM

BACCS25							BACCS25
LAYER	E.MOD	POISSON'S RATIO	LAYER THICKNESS	LAYER RATIO	E. x H	MODULAR RATIOS	
1	2999	0.44	30	1/2	89.97	0.0999833	
2	5999	0.35	150	2/3	899.85	0.7500650	
3	3999	0.35	299.999	3/4	1199.6960	0.0171385	
4	70	0.35	1000000	4/5	70000		

	SINGLE CRITERIA LIFE			COMBINED FAILURE CRITERIA	
LAYER 1	7.20E+09	ASPH		7.20E+09	ASPHALT
LAYER 2	6.73E+07	SUBC3-FATIGUE		1.48E+16	FATIGUE+FOS
	1.74E+07	SUBC3-CRUSHING		1.74E+07	CRUSHING
	1.48E+16	SUBC3-FOS		1.48E+16	FATIGUE+FOS
LAYER 3	4.93E+07	SUBSUBC3-FATIGUE		4.22E+17	FATIGUE+FOS
	7.28E+07	SUBSUBC3-CRUSHING		7.28E+07	CRUSHING
	4.22E+17	SUBSUBC3-FOS		4.22E+17	FATIGUE+FOS
LAYER 4	4.78E+12	SUBGRADE		4.78E+12	SUBGRADE
	1.74E+07	MINIMUM	MINIMUM	1.74E+07	4.22E+17 MAXIMUM

BACCS17							BACCS17
LAYER	E.MOD	POISSON'S RATIO	LAYER THICKNESS	LAYER RATIO	E. x H	MODULAR RATIOS	
1	2999	0.44	30	1/2	89.97	0.0999833	
2	5999	0.35	150	2/3	899.85	0.5000016	
3	5999	0.35	299.999	3/4	1799.6940	0.0257099	
4	70	0.35	1000000	4/5	70000		

	SINGLE CRITERIA LIFE			COMBINED FAILURE CRITERIA	
LAYER 1	1.16E+10	ASPH		1.16E+10	ASPHALT
LAYER 2	6.59E+07	SUBC3-FATIGUE		6.26E+16	FATIGUE+FOS
	1.73E+07	SUBC3-CRUSHING		1.73E+07	CRUSHING
	6.26E+16	SUBC3-FOS		6.26E+16	FATIGUE+FOS
LAYER 3	5.61E+07	SUBSUBC3-FATIGUE		1.95E+16	FATIGUE+FOS
	6.99E+07	SUBSUBC3-CRUSHING		6.99E+07	CRUSHING
	1.95E+16	SUBSUBC3-FOS		1.95E+16	FATIGUE+FOS
LAYER 4	4.78E+12	SUBGRADE		4.78E+12	SUBGRADE
	1.73E+07	MINIMUM	MINIMUM	1.73E+07	6.26E+16 MAXIMUM

L4

BACCS18	POISSON'S	LAYER	LAYER	E. x H	MODULAR	BACCS18
LAYER E.MOD	RATIO	THICKNESS	RATIO		RATIOS	
1 2999	0.44	30	1/2	89.97	0.0999833	
2 5999	0.35	150	2/3	899.85	0.5000016	
3 5999	0.35	299.999	3/4	1799.6940	0.0719877	
4 25	0.35	1000000	4/5	25000		

	SINGLE CRITERIA LIFE		COMBINED FAILURE CRITERIA
LAYER 1	5.03E+09 ASPH		5.03E+09 ASPHALT
LAYER 2	6.52E+07 SUBC3-FATIGUE		3.71E+22 FATIGUE+FOS
	1.73E+07 SUBC3-CRUSHING		1.73E+07 CRUSHING
	3.71E+22 SUBC3-FOS		3.71E+22 FATIGUE+FOS
LAYER 3	4.98E+07 SUBSUBC3-FATIGUE		5.68E+11 FATIGUE+FOS
	6.99E+07 SUBSUBC3-CRUSHING		6.99E+07 CRUSHING
	5.68E+11 SUBSUBC3-FOS		5.68E+11 FATIGUE+FOS
LAYER 4	4.78E+12 SUBGRADE		4.78E+12 SUBGRADE
	1.73E+07 MINIMUM	MINIMUM	1.73E+07 3.71E+22 MAXIMUM

BACCS17	POISSON'S	LAYER	LAYER	E. x H	MODULAR	BACCS17
LAYER E.MOD	RATIO	THICKNESS	RATIO		RATIOS	
1 2999	0.44	30	1/2	89.97	0.0999833	
2 5999	0.35	150	2/3	899.85	0.5000016	
3 5999	0.35	299.999	3/4	1799.6940	0.0257099	
4 70	0.35	1000000	4/5	70000		

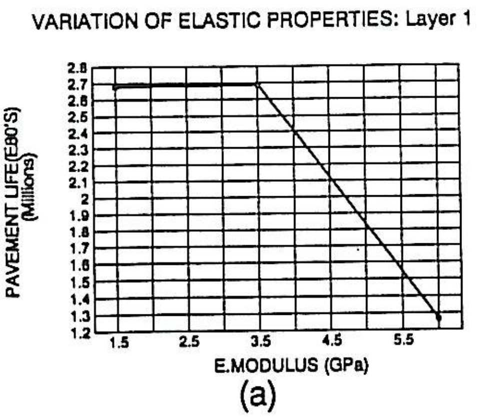
	SINGLE CRITERIA LIFE		COMBINED FAILURE CRITERIA
LAYER 1	1.16E+10 ASPH		1.16E+10 ASPHALT
LAYER 2	6.59E+07 SUBC3-FATIGUE		6.26E+16 FATIGUE+FOS
	1.73E+07 SUBC3-CRUSHING		1.73E+07 CRUSHING
	6.26E+16 SUBC3-FOS		6.26E+16 FATIGUE+FOS
LAYER 3	5.61E+07 SUBSUBC3-FATIGUE		1.95E+16 FATIGUE+FOS
	6.99E+07 SUBSUBC3-CRUSHING		6.99E+07 CRUSHING
	1.95E+16 SUBSUBC3-FOS		1.95E+16 FATIGUE+FOS
LAYER 4	4.78E+12 SUBGRADE		4.78E+12 SUBGRADE
	1.73E+07 MINIMUM	MINIMUM	1.73E+07 6.26E+16 MAXIMUM

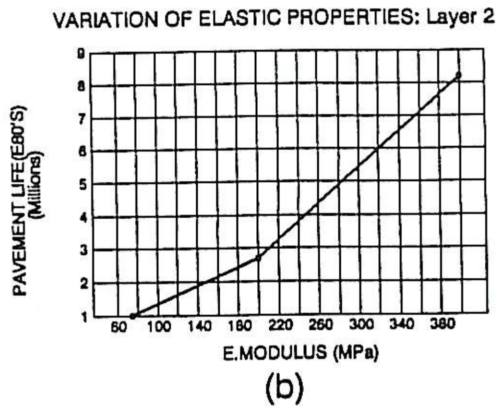
BACCS19	POISSON'S	LAYER	LAYER	E. x H	MODULAR	BACCS19
LAYER E.MOD	RATIO	THICKNESS	RATIO		RATIOS	
1 2999	0.44	30	1/2	89.97	0.0999833	
2 5999	0.35	150	2/3	899.85	0.5000016	
3 5999	0.35	299.999	3/4	1799.6940	0.0119979	
4 150	0.35	1000000	4/5	150000		

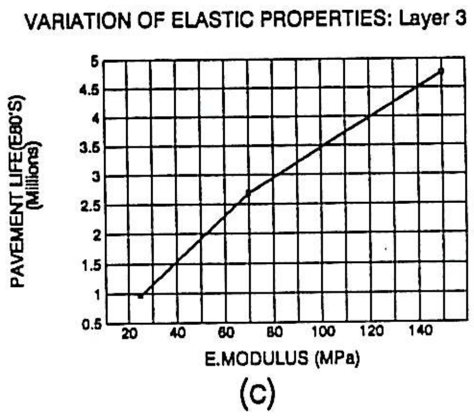
	SINGLE CRITERIA LIFE		COMBINED FAILURE CRITERIA
LAYER 1	2.25E+10 ASPH		2.25E+10 ASPHALT
LAYER 2	6.64E+07 SUBC3-FATIGUE		8.90E+13 FATIGUE+FOS
	1.73E+07 SUBC3-CRUSHING		1.73E+07 CRUSHING
	8.90E+13 SUBC3-FOS		8.90E+13 FATIGUE+FOS
LAYER 3	5.86E+07 SUBSUBC3-FATIGUE		8.51E+19 FATIGUE+FOS
	6.97E+07 SUBSUBC3-CRUSHING		6.97E+07 CRUSHING
	8.51E+19 SUBSUBC3-FOS		8.51E+19 FATIGUE+FOS
LAYER 4	4.78E+12 SUBGRADE		4.78E+12 SUBGRADE
	1.73E+07 MINIMUM	MINIMUM	1.73E+07 8.51E+19 MAXIMUM

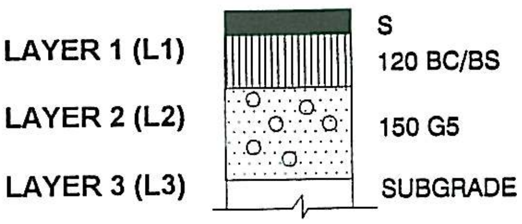
SENSITIVITY OF PAVEMENT LIFE TO CHANGES IN ELASTIC MODULI

CAGS









LAYER	ELASTIC MODULUS (MPa)	FAILURE IN:		FIGURE
		MATERIAL	LAYER	
1	1500 3500 6000	Asphalt Asphalt Asphalt	L1 L1 L1	a
2	75 200 400	Asphalt Asphalt Asphalt	L1 L1 L1	b
3	25 70 150	Subgrade Asphalt Asphalt	L3 L1 L1	c

FIGURE A4
CALCULATED CHANGES IN PAVEMENT LIFE WITH CHANGES IN ELASTIC MODULI
PAVEMENT CAGS (DETAILED ANALYSIS)

PAVEMENT
STRUCTURE: CAGS

LAYER1				LAYER2			
VARIATION IN ELASTIC MODULUS		VARIATION IN PREDICTED LIFE		VARIATION IN ELASTIC MODULUS		VARIATION IN PREDICTED LIFE	
MPa	%	E80'S	%	MPa	%	E80'S	%
1500	-57.143	2.68E+06	-0.304	75	-62.500	1.01E+06	-62.372
3500		2.69E+06		200		2.69E+06	
6000	71.429	1.26E+06	-52.961	400	100.000	8.21E+06	205.511

LAYER3			
VARIATION IN ELASTIC MODULUS		VARIATION IN PREDICTED LIFE	
MPa	%	E80'S	%
25	-64.286	9.68E+05	-63.998
70		2.69E+06	
150	114.286	4.77E+06	77.293

MATERIAL	RECOMMENDED E (MPa)
L1 ASPHALT BC/BS	3500
L2 GRANULAR G5	200
L3 SUBGRADE	70

Note: To calculate the 'variation in predicted life' the recommended mid-range values of elastic modulus were used.

L1

CAGS21	-- E.MOD	POISSON'S RATIO	LAYER THICKNESS	LAYER RATIO	E. x H	MODULAR RATIOS	CAGS21
LAYER							
1	1500	0.44	120	1/2	180	6	
2	200	0.35	150	2/3	30	0.085714	
3	70	0.35	5000	3/4	350		

	SINGLE CRITERIA LIFE		COMBINED FAILURE CRITERIA
LAYER 1	2.68E+06 ASPHALT		2.68E+06 ASPHALT
LAYER 2	5.93E+13 GRANULAR		5.93E+13 FATIGUE+FOS
LAYER 3	2.61E+07 SUBGRADE		2.61E+07 SUBGRADE
	2.68E+06 MINIMUM	MINIMUM	2.68E+06 5.9E+13 MAXIMUM

CAGS9	E.MOD	POISSON'S RATIO	LAYER THICKNESS	LAYER RATIO	E. x H	MODULAR RATIOS	CAGS9
LAYER							
1	3499	0.44	120	1/2	419.88	13.996	
2	200	0.35	150	2/3	30	0.085714	
3	70	0.35	5000	3/4	350		

	SINGLE CRITERIA LIFE		COMBINED FAILURE CRITERIA
LAYER 1	2.69E+06 ASPHALT		2.69E+06 ASPHALT
LAYER 2	8.93E+14 GRANULAR		8.93E+14 FATIGUE+FOS
LAYER 3	1.88E+08 SUBGRADE		1.88E+08 SUBGRADE
	2.69E+06 MINIMUM	MINIMUM	2.69E+06 8.9E+14 MAXIMUM

CAGS22	E.MOD	POISSON'S RATIO	LAYER THICKNESS	LAYER RATIO	E. x H	MODULAR RATIOS	CAGS22
LAYER							
1	5999	0.44	120	1/2	719.88	23.996	
2	200	0.35	150	2/3	30	0.085714	
3	70	0.35	5000	3/4	350		

	SINGLE CRITERIA LIFE		COMBINED FAILURE CRITERIA
LAYER 1	1.26E+06 ASPHALT		1.26E+06 ASPHALT
LAYER 2	1.72E+16 GRANULAR		1.72E+16 FATIGUE+FOS
LAYER 3	9.53E+08 SUBGRADE		9.53E+08 SUBGRADE
	1.26E+06 MINIMUM	MINIMUM	1.26E+06 1.7E+16 MAXIMUM

L2

CAGS20		POISSON'S	LAYER	LAYER	E. x H	MODULAR	CAGS20
LAYER	E.MOD	RATIO	THICKNESS	RATIO		RATIOS	
1	3499	0.44	120	1/2	419.88	37.32266	
2	75	0.35	150	2/3	11.25	0.032142	
3	70	0.35	5000	3/4	350		
	SINGLE CRITERIA LIFE				COMBINED FAILURE CRITERIA		
LAYER 1	1.01E+06 ASPHALT				1.01E+06 ASPHALT		
LAYER 2	3.10E+21 GRANULAR				3.10E+21 FATIGUE+FOS		
LAYER 3	2.26E+08 SUBGRADE				2.26E+08 SUBGRADE		
	1.01E+06 MINIMUM			MINIMUM	1.01E+06	3.1E+21 MAXIMUM	

CAGS9		POISSON'S	LAYER	LAYER	E. x H	MODULAR	CAGS9
LAYER	E.MOD	RATIO	THICKNESS	RATIO		RATIOS	
1	3499	0.44	120	1/2	419.88	13.996	
2	200	0.35	150	2/3	30	0.085714	
3	70	0.35	5000	3/4	350		
	SINGLE CRITERIA LIFE				COMBINED FAILURE CRITERIA		
LAYER 1	2.69E+06 ASPHALT				2.69E+06 ASPHALT		
LAYER 2	8.93E+14 GRANULAR				8.93E+14 FATIGUE+FOS		
LAYER 3	1.88E+08 SUBGRADE				1.88E+08 SUBGRADE		
	2.69E+06 MINIMUM			MINIMUM	2.69E+06	8.9E+14 MAXIMUM	

CAGS23		POISSON'S	LAYER	LAYER	E. x H	MODULAR	CAGS23
LAYER	E.MOD	RATIO	THICKNESS	RATIO		RATIOS	
1	3499	0.44	120	1/2	419.88	6.998	
2	400	0.35	150	2/3	60	0.171428	
3	70	0.35	5000	3/4	350		
	SINGLE CRITERIA LIFE				COMBINED FAILURE CRITERIA		
LAYER 1	8.21E+06 ASPHALT				8.21E+06 ASPHALT		
LAYER 2	6.14E+11 GRANULAR				6.14E+11 FATIGUE+FOS		
LAYER 3	4.62E+08 SUBGRADE				4.62E+08 SUBGRADE		
	8.21E+06 MINIMUM			MINIMUM	8.21E+06	6.1E+11 MAXIMUM	

L3

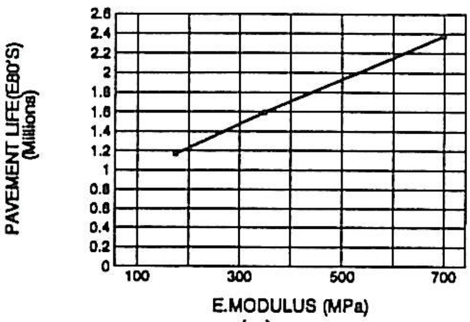
CAGS18		POISSON'S	LAYER	LAYER	E. x H	MODULAR	CAGS18
LAYER	E.MOD	RATIO	THICKNESS	RATIO		RATIOS	
1	3499	0.44	120	1/2	419.88	13.996	
2	200	0.35	150	2/3	30	0.24	
3	25	0.35	5000	3/4	125		
	SINGLE CRITERIA LIFE				COMBINED FAILURE CRITERIA		
LAYER 1	1.19E+06 ASPHALT				1.19E+06 ASPHALT		
LAYER 2	3.90E+11 GRANULAR				3.90E+11 FATIGUE+FOS		
LAYER 3	9.68E+05 SUBGRADE				9.68E+05 SUBGRADE		
	9.68E+05 MINIMUM			MINIMUM	9.68E+05	3.9E+11 MAXIMUM	

CAGS9		POISSON'S	LAYER	LAYER	E. x H	MODULAR	CAGS9
LAYER	E.MOD	RATIO	THICKNESS	RATIO		RATIOS	
1	3499	0.44	120	1/2	419.88	13.996	
2	200	0.35	150	2/3	30	0.085714	
3	70	0.35	5000	3/4	350		
	SINGLE CRITERIA LIFE				COMBINED FAILURE CRITERIA		
LAYER 1	2.69E+06 ASPHALT				2.69E+06 ASPHALT		
LAYER 2	8.93E+14 GRANULAR				8.93E+14 FATIGUE+FOS		
LAYER 3	1.88E+08 SUBGRADE				1.88E+08 SUBGRADE		
	2.69E+06 MINIMUM			MINIMUM	2.69E+06	8.9E+14 MAXIMUM	

CAGS19		POISSON'S	LAYER	LAYER	E. x H	MODULAR	CAGS19
LAYER	E.MOD	RATIO	THICKNESS	RATIO		RATIOS	
1	3499	0.44	120	1/2	419.88	13.996	
2	200	0.35	150	2/3	30	0.04	
3	150	0.35	5000	3/4	750		
	SINGLE CRITERIA LIFE				COMBINED FAILURE CRITERIA		
LAYER 1	4.77E+06 ASPHALT				4.77E+06 ASPHALT		
LAYER 2	3.86E+17 GRANULAR				3.86E+17 FATIGUE+FOS		
LAYER 3	2.33E+10 SUBGRADE				2.33E+10 SUBGRADE		
	4.77E+06 MINIMUM			MINIMUM	4.77E+06	3.9E+17 MAXIMUM	

SENSITIVITY OF PAVEMENT LIFE TO CHANGES IN ELASTIC MODULI

CGCS



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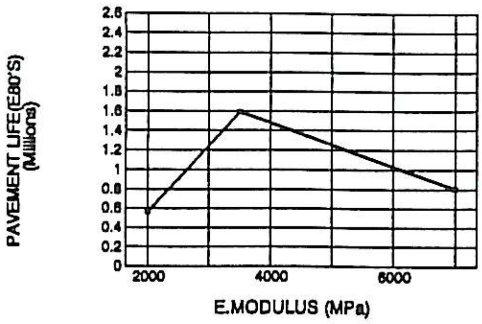

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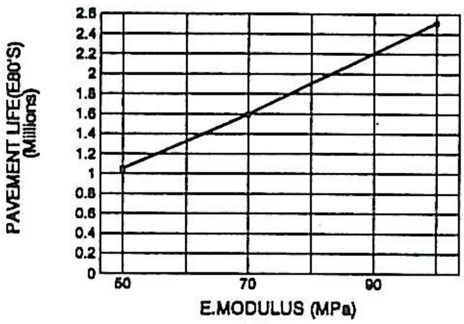

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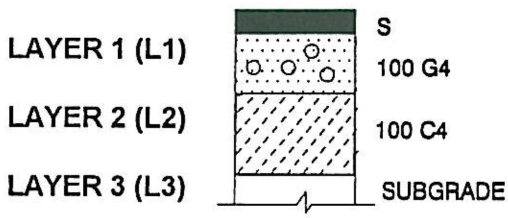


LAYER	ELASTIC MODULUS (MPa)	FAILURE IN:		FIGURE
		MATERIAL	LAYER	
1	175 350 700	C4 (Crushing) C4 (Crushing) C4 (Crushing)	L2 L2 L2	a
2	2000 3500 7000	C4 (Fatigue + FOS) C4 (Crushing) C4 (Crushing)	L2 L2 L2	b
3	50 70 100	C4 (Fatigue + FOS) C4 (Fatigue + FOS) C4 (Fatigue + FOS)	L2 L2 L2	c

FIGURE A5
CALCULATED CHANGES IN PAVEMENT LIFE WITH CHANGES IN ELASTIC MODULI
PAVEMENT CGCS (DETAILED ANALYSIS)

PAVEMENT
STRUCTURE: CGCS

LAYER1				LAYER2			
VARIATION IN ELASTIC MODULUS		VARIATION IN PREDICTED LIFE		VARIATION IN ELASTIC MODULUS		VARIATION IN PREDICTED LIFE	
MPa	%	E80'S	%	MPa	%	E80'S	%
175	-50.000	1.17E+06	-26.660	2000	-42.841	5.64E+05	-64.676
350	Reference	1.60E+06	Reference	3499	Reference	1.60E+06	Reference
700	100.000	2.38E+06	49.277	6998	100.000	8.04E+05	-49.605

LAYER3			
VARIATION IN ELASTIC MODULUS		VARIATION IN PREDICTED LIFE	
MPa	%	E80'S	%
50	-28.571	1.05E+06	-34.068
70	Reference	1.60E+06	Reference
100	42.857	2.51E+06	57.421

MATERIAL	RECOMMENDED E (MPa)
L1 GRANULAR G4	350
L2 CEMENTED C4	3500
L3 SUBGRADE	70

Note: To calculate the 'variation in predicted life' the recommended mid-range values of elastic modulus were used.

L1

TRUN102	–	POISSON'S	LAYER	LAYER	E. x H	MODULAR	TRUN102
LAYER	E.MOD	RATIO	THICKNESS	RATIO		RATIOS	
1	175	0.35	100	1/2	17.5	0.050014	
2	3499	0.35	100	2/3	349.9	0.004998	
3	70	0.35	1000000	3/4	70000		
	SINGLE CRITERIA LIFE				COMBINED FAILURE CRITERIA		
LAYER 1	2.11E+11 G1				2.11E+11 G1		
LAYER 2	1.06E+06 SUBC3-FATIGUE				1.17E+06 FATIGUE+FOS		
	1.71E+06 SUBC3-CRUSHING				1.71E+06 CRUSHING		
	1.15E+05 SUBC3-FOS						
LAYER 3	3.70E+06 SUBGRADE				3.70E+06 SUBGRADE		
	1.15E+05 MINIMUM			MINIMUM	1.17E+06	2.1E+11 MAXIMUM	

TRUN90		POISSON'S	LAYER	LAYER	E. x H	MODULAR	TRUN90
LAYER	E.MOD	RATIO	THICKNESS	RATIO		RATIOS	
1	350	0.35	100	1/2	35	0.100028	
2	3499	0.35	100	2/3	349.9	0.004998	
3	70	0.35	1000000	3/4	70000		
	SINGLE CRITERIA LIFE				COMBINED FAILURE CRITERIA		
LAYER 1	4.23E+13 G1				4.23E+13 G1		
LAYER 2	1.45E+06 SUBC3-FATIGUE				1.60E+06 FATIGUE+FOS		
	4.10E+06 SUBC3-CRUSHING				4.10E+06 CRUSHING		
	1.49E+05 SUBC3-FOS						
LAYER 3	8.38E+06 SUBGRADE				8.38E+06 SUBGRADE		
	1.49E+05 MINIMUM			MINIMUM	1.60E+06	4.2E+13 MAXIMUM	

TRUN103		POISSON'S	LAYER	LAYER	E. x H	MODULAR	TRUN103
LAYER	E.MOD	RATIO	THICKNESS	RATIO		RATIOS	
1	700	0.35	100	1/2	70	0.200057	
2	3499	0.35	100	2/3	349.9	0.004998	
3	70	0.35	1000000	3/4	70000		
	SINGLE CRITERIA LIFE				COMBINED FAILURE CRITERIA		
LAYER 1	1.19E+18 G1				1,19E+18 G1		
LAYER 2	2.15E+06 SUBC3-FATIGUE				2.38E+06 FATIGUE+FOS		
	1.28E+07 SUBC3-CRUSHING				1.28E+07 CRUSHING		
	2.31E+05 SUBC3-FOS						
LAYER 3	2.75E+07 SUBGRADE				2.75E+07 SUBGRADE		
	2.31E+05 MINIMUM			MINIMUM	2.38E+06	1.2E+18 MAXIMUM	

TRUN104		POISSON'S	LAYER	L2 LAYER	E. x H	MODULAR	TRUN104
LAYER	E.MOD	RATIO	THICKNESS	RATIO		RATIOS	
1	350	0.35	100	1/2	35	0.175	
2	2000	0.35	100	2/3	200	0.002857	
3	70	0.35	1000000	3/4	70000		

	SINGLE CRITERIA LIFE		COMBINED FAILURE CRITERIA	
LAYER 1	2.36E+14 G1		2.36E+14 G1	
LAYER 2	3.98E+05 SUBC3-FATIGUE 1.15E+07 SUBC3-CRUSHING 1.66E+05 SUBC3-FOS		5.64E+05 FATIGUE+FOS 1.15E+07 CRUSHING	
LAYER 3	1.53E+06 SUBGRADE		1.53E+06 SUBGRADE	
	1.66E+05 MINIMUM	MINIMUM	5.64E+05	2.4E+14 MAXIMUM

TRUN90		POISSON'S	LAYER	LAYER	E. x H	MODULAR	TRUN90
LAYER	E.MOD	RATIO	THICKNESS	RATIO		RATIOS	
1	350	0.35	100	1/2	35	0.100028	
2	3499	0.35	100	2/3	349.9	0.004998	
3	70	0.35	1000000	3/4	70000		

	SINGLE CRITERIA LIFE		COMBINED FAILURE CRITERIA	
LAYER 1	4.23E+13 G1		4.23E+13 G1	
LAYER 2	1.45E+06 SUBC3-FATIGUE 4.10E+06 SUBC3-CRUSHING 1.49E+05 SUBC3-FOS		1.60E+06 FATIGUE+FOS 4.10E+06 CRUSHING	
LAYER 3	8.38E+06 SUBGRADE		8.38E+06 SUBGRADE	
	1.49E+05 MINIMUM	MINIMUM	1.60E+06	4.2E+13 MAXIMUM

TRUN105		POISSON'S	LAYER	LAYER	E. x H	MODULAR	TRUN105
LAYER	E.MOD	RATIO	THICKNESS	RATIO		RATIOS	
1	350	0.35	100	1/2	35	0.050014	
2	6998	0.35	100	2/3	699.8	0.009997	
3	70	0.35	1000000	3/4	70000		

	SINGLE CRITERIA LIFE		COMBINED FAILURE CRITERIA	
LAYER 1	3.43E+12 G1		3.43E+12 G1	
LAYER 2	5.11E+06 SUBC3-FATIGUE 8.04E+05 SUBC3-CRUSHING 1.31E+05 SUBC3-FOS		5.24E+06 FATIGUE+FOS 8.04E+05 CRUSHING	
LAYER 3	9.05E+07 SUBGRADE		9.05E+07 SUBGRADE	
	1.31E+05 MINIMUM	MINIMUM	8.04E+05	3.4E+12 MAXIMUM

L3

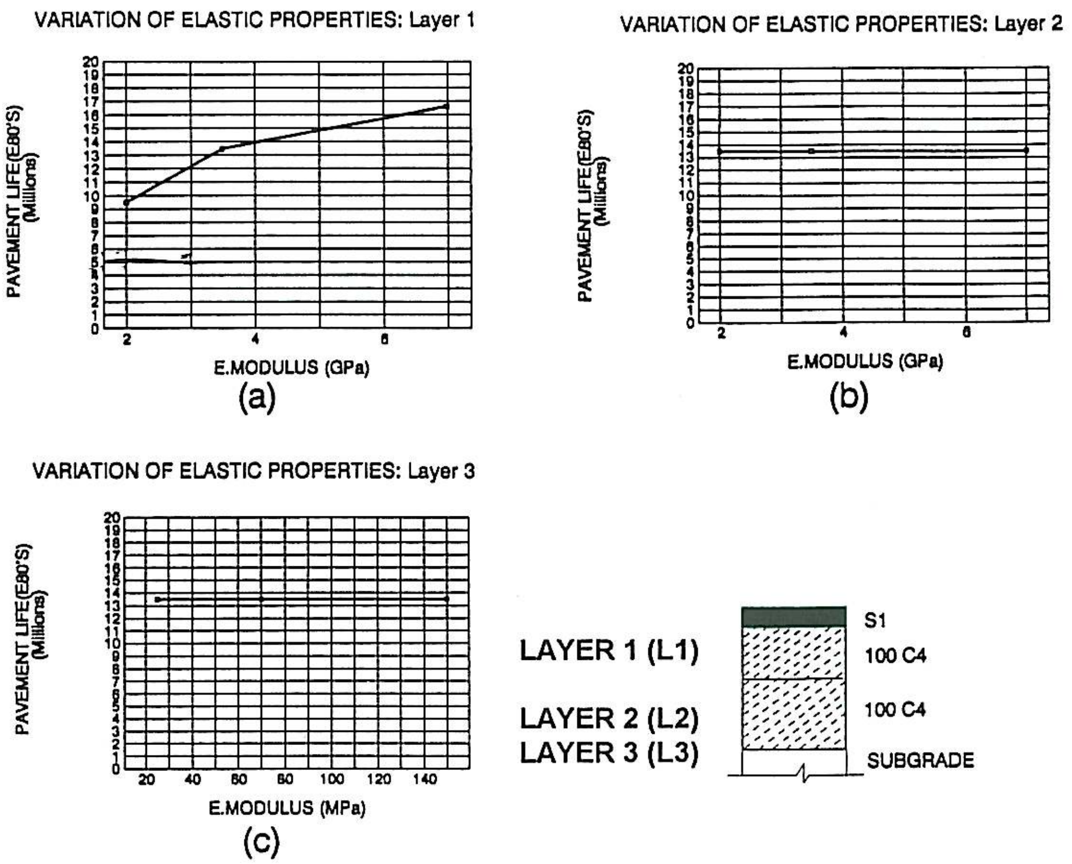
TRUN76		POISSON'S	LAYER	LAYER	E. x H	MODULAR	TRUN76
LAYER	E.MOD	RATIO	THICKNESS	RATIO		RATIOS	
1	350	0.35	100	1/2	35	0.100028	
2	3499	0.35	100	2/3	349.9	0.006998	
3	50	0.35	1000000	3/4	50000		
	SINGLE CRITERIA LIFE				COMBINED FAILURE CRITERIA		
LAYER 1	5.76E+14 G1				5.76E+14 G1		
LAYER 2	8.88E+05 SUBC3-FATIGUE				1.05E+06 FATIGUE+FOS		
	2.67E+06 SUBC3-CRUSHING				2.67E+06 CRUSHING		
	1.64E+05 SUBC3-FOS						
LAYER 3	1.89E+06 SUBGRADE				1.89E+06 SUBGRADE		
	1.64E+05 MINIMUM			MINIMUM	1.05E+06	5.8E+14 MAXIMUM	

TRUN90		POISSON'S	LAYER	LAYER	E. x H	MODULAR	TRUN90
LAYER	E.MOD	RATIO	THICKNESS	RATIO		RATIOS	
1	350	0.35	100	1/2	35	0.100028	
2	3499	0.35	100	2/3	349.9	0.004998	
3	70	0.35	1000000	3/4	70000		
	SINGLE CRITERIA LIFE				COMBINED FAILURE CRITERIA		
LAYER 1	4.23E+13 G1				4.23E+13 G1		
LAYER 2	1.45E+06 SUBC3-FATIGUE				1.60E+06 FATIGUE+FOS		
	4.10E+06 SUBC3-CRUSHING				4.10E+06 CRUSHING		
	1.49E+05 SUBC3-FOS						
LAYER 3	8.38E+06 SUBGRADE				8.38E+06 SUBGRADE		
	1.49E+05 MINIMUM			MINIMUM	1.60E+06	4.2E+13 MAXIMUM	

TRUN77		POISSON'S	LAYER	LAYER	E. x H	MODULAR	TRUN77
LAYER	E.MOD	RATIO	THICKNESS	RATIO		RATIOS	
1	350	0.35	100	1/2	35	0.100028	
2	3499	0.35	100	2/3	349.9	0.003499	
3	100	0.35	1000000	3/4	100000		
	SINGLE CRITERIA LIFE				COMBINED FAILURE CRITERIA		
LAYER 1	4.53E+12 G1				4.53E+12 G1		
LAYER 2	2.38E+06 SUBC3-FATIGUE				2.51E+06 FATIGUE+FOS		
	6.32E+06 SUBC3-CRUSHING				6.32E+06 CRUSHING		
	1.34E+05 SUBC3-FOS						
LAYER 3	4.41E+07 SUBGRADE				4.41E+07 SUBGRADE		
	1.34E+05 MINIMUM			MINIMUM	2.51E+06	4.5E+12 MAXIMUM	

SENSITIVITY OF PAVEMENT LIFE TO CHANGES IN ELASTIC MODULI

CCCS



LAYER	ELASTIC MODULUS (MPa)	FAILURE IN:		FIGURE
		MATERIAL	LAYER	
1	2000 3500 7000	C4 (Crushing) C4 (Crushing) C4 (Crushing)	L1 L1 L1	a
2	2000 3500 7000	C4 (Fatigue + FOS) C4 (Crushing) C4 (Crushing)	L1 L1 L1	b
3	25 70 150	C4 (Fatigue + FOS) C4 (Fatigue + FOS) C4 (Fatigue + FOS)	L1 L1 L1	c

FIGURE A6
CALCULATED CHANGES IN PAVEMENT LIFE WITH CHANGES IN ELASTIC MODULI
PAVEMENT CCCS (DETAILED ANALYSIS)

PAVEMENT
STRUCTURE: CCCS

LAYER1				LAYER2			
VARIATION IN ELASTIC MODULUS		VARIATION IN PREDICTED LIFE		VARIATION IN ELASTIC MODULUS		VARIATION IN PREDICTED LIFE	
MPa	%	E80'S	%	MPa	%	E80'S	%
2000	-42.84	9.52E+06	-29.47	2000	-42.84	1.35E+07	0
3499	Reference	1.35E+07	Reference	3499	Reference	1.35E+07	Reference
6998	100	1.67E+07	23.35	6998	100	1.35E+07	0

LAYER3			
VARIATION IN ELASTIC MODULUS		VARIATION IN PREDICTED LIFE	
MPa	%	E80'S	%
25	-64.29	1.35E+07	0
70	Reference	1.35E+07	Reference
150	114.29	1.35E+07	0

MATERIAL	RECOMMENDED E (MPa)
L1 CEMENTED C4	3500
L2 CEMENTED C4	3500
L3 SUBGRADE	70

Note: To calculate the 'variation in predicted life' the recommended mid-range values of elastic modulus were used.

L1

CCCS16							
LAYER	E.MOD	POISSON'S RATIO	LAYER THICKNESS	LAYER RATIO	E. x H	MODULAR RATIOS	CCCS16
1	2000	0.35	100	1/2	200	0.571591	
2	3499	0.35	100	2/3	349.9	0.004998	
3	70	0.35	1000000	3/4	70000		

	SINGLE CRITERIA LIFE		COMBINED FAILURE CRITERIA	
LAYER 1	6.33E+07 BASE-C4-F.O.S.			
	1.55E+07 BASE-C4-FATIGUE		7.87E+07 BASE-FATIGUE+F.O.S.	
	9.52E+06 BASE-C4-CRUSHING		9.52E+06 BASE-C4-CRUSHING	
LAYER 2	4.00E+06 SUBC3-FATIGUE		1.35E+15 FATIGUE+FOS	
	3.59E+07 SUBC3-CRUSHING		3.59E+07 CRUSHING	
	1.35E+15 SUBC3-FOS			
LAYER 3	3.35E+15 SUBGRADE		3.35E+15 SUBGRADE	
	4.00E+06 MINIMUM	MINIMUM	9.52E+06	3.4E+15 MAXIMUM

CCCS9							
LAYER	E.MOD	POISSON'S RATIO	LAYER THICKNESS	LAYER RATIO	E. x H	MODULAR RATIOS	CCCS9
1	3499	0.35	100	1/2	349.9	1	
2	3499	0.35	100	2/3	349.9	0.004998	
3	70	0.35	1000000	3/4	70000		

	SINGLE CRITERIA LIFE		COMBINED FAILURE CRITERIA	
LAYER 1	5.62E+07 BASE-C4-F.O.S.			
	1.55E+07 BASE-C4-FATIGUE		7.17E+07 BASE-FATIGUE+F.O.S.	
	1.35E+07 BASE-C4-CRUSHING		1.35E+07 BASE-C4-CRUSHING	
LAYER 2	5.35E+06 SUBC3-FATIGUE		9.15E+15 FATIGUE+FOS	
	4.29E+07 SUBC3-CRUSHING		4.29E+07 CRUSHING	
	9.15E+15 SUBC3-FOS			
LAYER 3	3.19E+15 SUBGRADE		3.19E+15 SUBGRADE	
	5.35E+06 MINIMUM	MINIMUM	1.35E+07	9.2E+15 MAXIMUM

CCCS17							
LAYER	E.MOD	POISSON'S RATIO	LAYER THICKNESS	LAYER RATIO	E. x H	MODULAR RATIOS	CCCS17
1	6998	0.35	100	1/2	699.8	2	
2	3499	0.35	100	2/3	349.9	0.004998	
3	70	0.35	1000000	3/4	70000		

	SINGLE CRITERIA LIFE		COMBINED FAILURE CRITERIA	
LAYER 1	1.50E+08 BASE-C4-F.O.S.			
	1.55E+07 BASE-C4-FATIGUE		1.66E+08 BASE-FATIGUE+F.O.S.	
	1.67E+07 BASE-C4-CRUSHING		1.67E+07 BASE-C4-CRUSHING	
LAYER 2	7.53E+06 SUBC3-FATIGUE		5.57E+16 FATIGUE+FOS	
	5.46E+07 SUBC3-CRUSHING		5.46E+07 CRUSHING	
	5.57E+16 SUBC3-FOS			
LAYER 3	4.79E+15 SUBGRADE		4.79E+15 SUBGRADE	
	7.53E+06 MINIMUM	MINIMUM	1.67E+07	5.6E+16 MAXIMUM

L2

CCCS18							CCCS18
LAYER	E.MOD	POISSON'S RATIO	LAYER THICKNESS	LAYER RATIO	E. x H	MODULAR RATIOS	
1	3499	0.35	100	1/2	349.9	1.7495	
2	2000	0.35	100	2/3	200	0.002857	
3	70	0.35	1000000	3/4	70000		

	SINGLE CRITERIA LIFE		COMBINED FAILURE CRITERIA	
LAYER 1	6.67E+08 BASE-C4-F.O.S.			
	1.55E+07 BASE-C4-FATIGUE		6.83E+08 BASE-FATIGUE+F.O.S.	
	1.35E+07 BASE-C4-CRUSHING		1.35E+07 BASE-C4-CRUSHING	
LAYER 2	2.11E+06 SUBC3-FATIGUE		8.33E+15 FATIGUE+FOS	
	4.27E+07 SUBC3-CRUSHING		4.27E+07 CRUSHING	
	8.33E+15 SUBC3-FOS			
LAYER 3	6.22E+13 SUBGRADE		6.22E+13 SUBGRADE	
	2.11E+06 MINIMUM	MINIMUM	1.35E+07	8.3E+15 MAXIMUM

CCCS9							CCCS9
LAYER	E.MOD	POISSON'S RATIO	LAYER THICKNESS	LAYER RATIO	E. x H	MODULAR RATIOS	
1	3499	0.35	100	1/2	349.9	1	
2	3499	0.35	100	2/3	349.9	0.004998	
3	70	0.35	1000000	3/4	70000		

	SINGLE CRITERIA LIFE		COMBINED FAILURE CRITERIA	
LAYER 1	5.62E+07 BASE-C4-F.O.S.			
	1.55E+07 BASE-C4-FATIGUE		7.17E+07 BASE-FATIGUE+F.O.S.	
	1.35E+07 BASE-C4-CRUSHING		1.35E+07 BASE-C4-CRUSHING	
LAYER 2	5.35E+06 SUBC3-FATIGUE		9.15E+15 FATIGUE+FOS	
	4.29E+07 SUBC3-CRUSHING		4.29E+07 CRUSHING	
	9.15E+15 SUBC3-FOS			
LAYER 3	3.19E+15 SUBGRADE		3.19E+15 SUBGRADE	
	5.35E+06 MINIMUM	MINIMUM	1.35E+07	9.2E+15 MAXIMUM

CCCS19							CCCS19
LAYER	E.MOD	POISSON'S RATIO	LAYER THICKNESS	LAYER RATIO	E. x H	MODULAR RATIOS	
1	3499	0.35	100	1/2	349.9	0.5	
2	6998	0.35	100	2/3	699.8	0.009997	
3	70	0.35	1000000	3/4	70000		

	SINGLE CRITERIA LIFE		COMBINED FAILURE CRITERIA	
LAYER 1	1.09E+07 BASE-C4-F.O.S.			
	1.55E+07 BASE-C4-FATIGUE		2.64E+07 BASE-FATIGUE+F.O.S.	
	1.35E+07 BASE-C4-CRUSHING		1.35E+07 BASE-C4-CRUSHING	
LAYER 2	1.28E+07 SUBC3-FATIGUE		5.66E+15 FATIGUE+FOS	
	4.05E+07 SUBC3-CRUSHING		4.05E+07 CRUSHING	
	5.66E+15 SUBC3-FOS			
LAYER 3	7.40E+17 SUBGRADE		7.40E+17 SUBGRADE	
	1.09E+07 MINIMUM	MINIMUM	1.35E+07	7.4E+17 MAXIMUM

L3

CCCS14							CCCS14
LAYER	E.MOD	POISSON'S RATIO	LAYER THICKNESS	LAYER RATIO	E. x H	MODULAR RATIOS	
1	3499	0.35	100	1/2	349.9	1	
2	3499	0.35	100	2/3	349.9	0.013996	
3	25	0.35	1000000	3/4	25000		

	SINGLE CRITERIA LIFE		COMBINED FAILURE CRITERIA
LAYER 1	4.36E+06 BASE-C4-F.O.S.		
	1.55E+07 BASE-C4-FATIGUE		1.98E+07 BASE-FATIGUE+F.O.S.
	1.35E+07 BASE-C4-CRUSHING		1.35E+07 BASE-C4-CRUSHING
LAYER 2	2.59E+06 SUBC3-FATIGUE		7.27E+16 FATIGUE+FOS
	4.48E+07 SUBC3-CRUSHING		4.48E+07 CRUSHING
	7.27E+16 SUBC3-FOS		
LAYER 3	6.71E+14 SUBGRADE		6.71E+14 SUBGRADE
	2.59E+06 MINIMUM	MINIMUM	1.35E+07 7.3E+16 MAXIMUM

CCCS9							CCCS9
LAYER	E.MOD	POISSON'S RATIO	LAYER THICKNESS	LAYER RATIO	E. x H	MODULAR RATIOS	
1	3499	0.35	100	1/2	349.9	1	
2	3499	0.35	100	2/3	349.9	0.004998	
3	70	0.35	1000000	3/4	70000		

	SINGLE CRITERIA LIFE		COMBINED FAILURE CRITERIA
LAYER 1	5.62E+07 BASE-C4-F.O.S.		
	1.55E+07 BASE-C4-FATIGUE		7.17E+07 BASE-FATIGUE+F.O.S.
	1.35E+07 BASE-C4-CRUSHING		1.35E+07 BASE-C4-CRUSHING
LAYER 2	5.35E+06 SUBC3-FATIGUE		9.15E+15 FATIGUE+FOS
	4.29E+07 SUBC3-CRUSHING		4.29E+07 CRUSHING
	9.15E+15 SUBC3-FOS		
LAYER 3	3.19E+15 SUBGRADE		3.19E+15 SUBGRADE
	5.35E+06 MINIMUM	MINIMUM	1.35E+07 9.2E+15 MAXIMUM

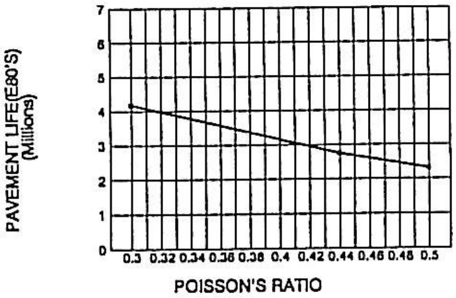
CCCS15							CCCS15
LAYER	E.MOD	POISSON'S RATIO	LAYER THICKNESS	LAYER RATIO	E. x H	MODULAR RATIOS	
1	3499	0.35	100	1/2	349.9	1	
2	3499	0.35	100	2/3	349.9	0.002332	
3	150	0.35	1000000	3/4	150000		

	SINGLE CRITERIA LIFE		COMBINED FAILURE CRITERIA
LAYER 1	6.07E+08 BASE-C4-F.O.S.		
	1.55E+07 BASE-C4-FATIGUE		6.22E+08 BASE-FATIGUE+F.O.S.
	1.35E+07 BASE-C4-CRUSHING		1.35E+07 BASE-C4-CRUSHING
LAYER 2	9.41E+06 SUBC3-FATIGUE		6.21E+14 FATIGUE+FOS
	4.07E+07 SUBC3-CRUSHING		4.07E+07 CRUSHING
	6.21E+14 SUBC3-FOS		
LAYER 3	1.06E+16 SUBGRADE		1.06E+16 SUBGRADE
	9.41E+06 MINIMUM	MINIMUM	1.35E+07 1.1E+16 MAXIMUM

SENSITIVITY OF PAVEMENT LIFE TO CHANGES IN POISSON'S RATIO

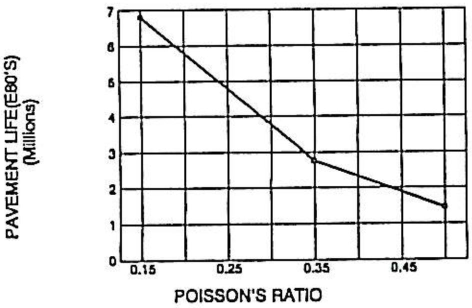
AAGCCSP

VARIATION OF POISSON'S RATIO : Layer 1



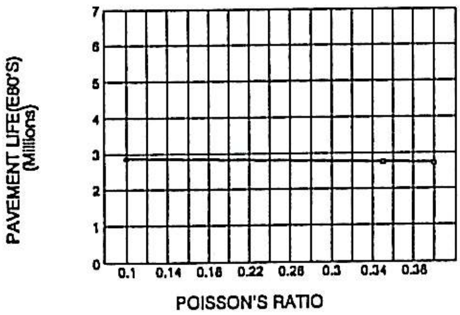
(a)

VARIATION OF POISSON'S RATIO: Layer 2



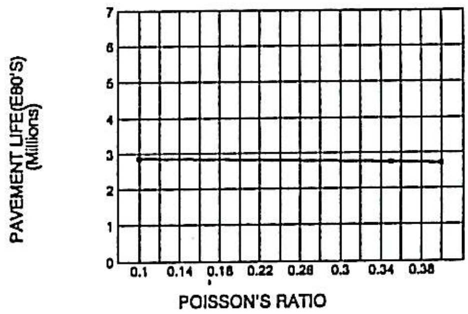
(b)

VARIATION OF POISSON'S RATIO: Layer 3



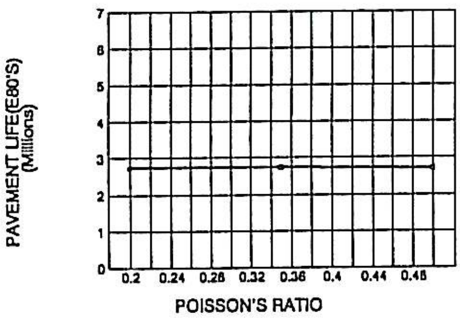
(c)

VARIATION OF POISSON'S RATIO: LAYER 4

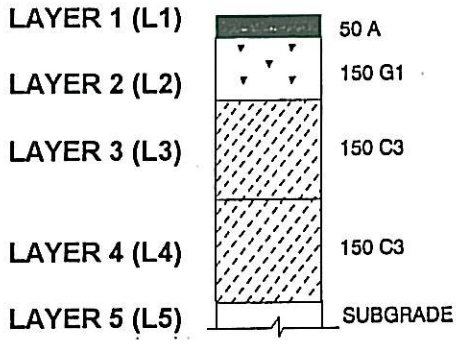


(d)

VARIATION OF POISSON'S RATIO: Layer 5



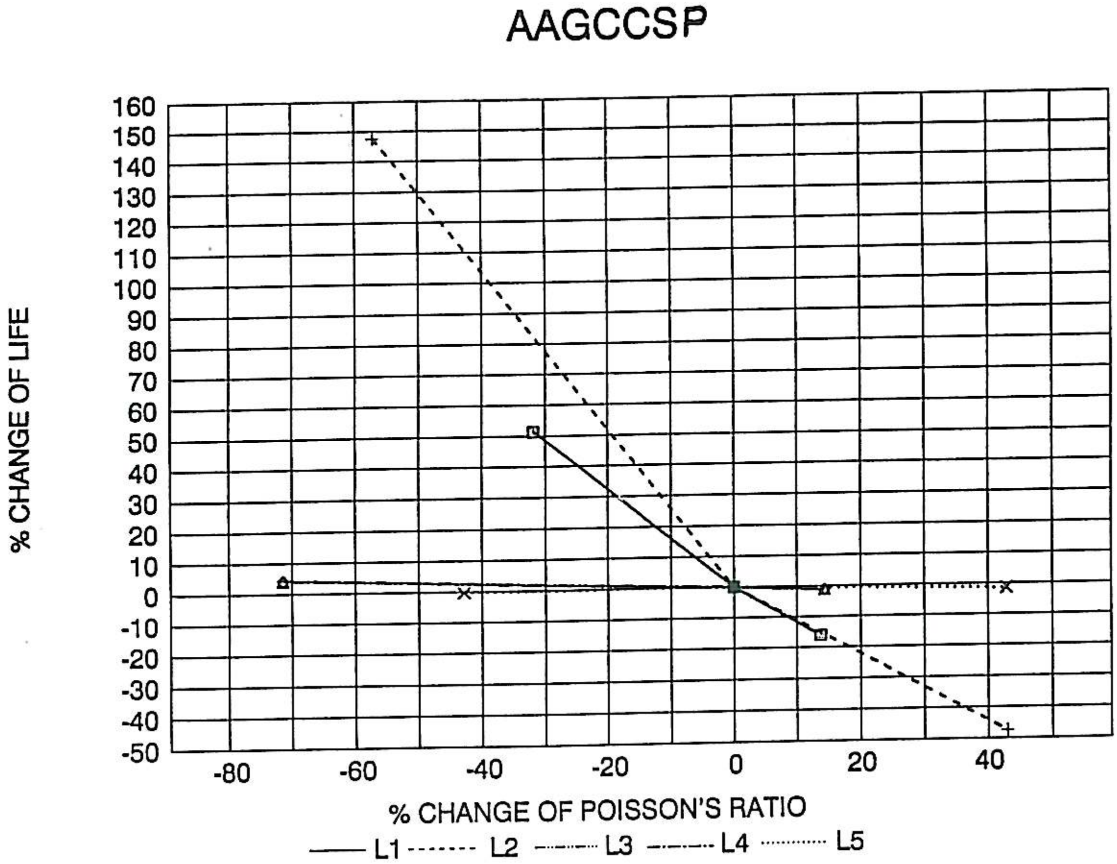
(e)



LAYER	ELASTIC MODULUS (MPa)	FAILURE IN:		FIGURE
		MATERIAL	LAYER	
1	0.30 0.44 0.50	Asphalt Asphalt Asphalt	L1 L1 L1	a
2	0.15 0.35 0.50	Granular Asphalt Asphalt	L2 L1 L1	b
3	0.10 0.35 0.40	Asphalt Asphalt Asphalt	L1 L1 L1	c
4	0.10 0.35 0.40	Asphalt Asphalt Asphalt	L1 L1 L1	d
5	0.20 0.35 0.50	Asphalt Asphalt Asphalt	L1 L1 L1	c

FIGURE A7
CALCULATED CHANGES IN PAVEMENT LIFE WITH CHANGES IN POISSON'S RATIO
PAVEMENT AAGCCSP (SEE ALSO FIGURE A1)

CALCULATED CHANGES IN PAVEMENT LIFE WITH CHANGES IN POISSON'S RATIO.



PAVEMENT
STRUCTURE: AAGCCSP

LAYER1			
VARIATION IN POISSON'S RATION		VARIATION IN PREDICTED LIFE	
MPa	%	E80'S	%
0.3	-31.82	4.17E+06	51.37
0.44	Reference	2.75E+06	Reference
0.5	13.64	2.31E+06	-16.12

LAYER2			
VARIATION IN POISSON'S RATION		VARIATION IN PREDICTED LIFE	
MPa	%	E80'S	%
0.15	-57.14	6.81E+06	147.49
0.35	Reference	2.75E+06	Reference
0.5	42.86	1.45E+06	-47.41

LAYER3			
VARIATION IN POISSON'S RATION		VARIATION IN PREDICTED LIFE	
MPa	%	E80'S	%
0.1	-71.43	2.87E+06	4.15
0.35	Reference	2.75E+06	Reference
0.4	14.29	2.72E+06	-1.15

LAYER4			
VARIATION IN POISSON'S RATION		VARIATION IN PREDICTED LIFE	
MPa	%	E80'S	%
0.1	-71.43	2.87E+06	4.15
0.35	Reference	2.75E+06	Reference
0.4	14.29	2.72E+06	-1.15

LAYER5			
VARIATION IN POISSON'S RATION		VARIATION IN PREDICTED LIFE	
MPa	%	E80'S	%
0.2	-42.86	2.74E+06	-0.29
0.35	Reference	2.75E+06	Reference
0.5	42.86	2.71E+06	-1.43

MATERIAL	RECOMMENDED POISSON'S RATIO
L1 ASPHALT A	0.44
L2 GRANULAR G1	0.35
L3 CEMENTED C3	0.35
L4 CEMENTED C3	0.35
L5 SUBGRADE	0.35

Note: To calculate the 'variation in predicted life' the recommended mid-range values of elastic modulus were used.

L1

TRUN135		POISSON'S	LAYER	LAYER	E. x H	MODULAR	TRUN135
LAYER	E.MOD	RATIO	THICKNESS	RATIO		RATIOS	
1	2999	0.3	50	1/2	149.95	2.221481	
2	450	0.35	150	2/3	67.5	0.075012	
3	5999	0.35	150	3/4	899.85	1	
4	5999	0.35	150	4/5	899.85	0.012855	
5	70	0.35	1000000		70000		

	SINGLE CRITERIA LIFE			COMBINED FAILURE CRITERIA		
LAYER 1	4.17E+06	ASPH		4.17E+06	ASPHALT	
LAYER 2	9.15E+14	G1		9.15E+14	G1	
LAYER 3	6.55E+07	SUBC3-FATIGUE		7.87E+12	FATIGUE+FOS	
	6.19E+07	SUBC3-CRUSHING		6.19E+07	CRUSHING	
	7.87E+12	SUBC3-FOS		7.87E+12	FATIGUE+FOS	
LAYER 4	4.45E+07	SUBSUBC3-FATIGUE		1.26E+27	FATIGUE+FOS	
	1.15E+08	SUBSUBC3-CRUSHING		1.15E+08	CRUSHING	
	1.26E+27	SUBSUBC3-FOS		1.26E+27	FATIGUE+FOS	
LAYER 5	1.81E+14	SUBGRADE		1.81E+14	SUBGRADE	
	4.17E+06	MINIMUM	MINIMUM	4.17E+06	1.3E+27	MAXIMUM

TRUN20		POISSON'S	LAYER	LAYER	E. x H	MODULAR	TRUN20
LAYER	E.MOD	RATIO	THICKNESS	RATIO		RATIOS	
1	2999	0.44	50	1/2	149.95	2.221481	
2	450	0.35	150	2/3	67.5	0.075012	
3	5999	0.35	150	3/4	899.85	1	
4	5999	0.35	150	4/5	899.85	0.012855	
5	70	0.35	1000000		70000		

	SINGLE CRITERIA LIFE			COMBINED FAILURE CRITERIA		
LAYER 1	2.75E+06	ASPH		2.75E+06	ASPHALT	
LAYER 2	5.44E+14	G1		5.44E+14	G1	
LAYER 3	6.57E+07	SUBC3-FATIGUE		7.71E+12	FATIGUE+FOS	
	6.38E+07	SUBC3-CRUSHING		6.38E+07	CRUSHING	
	7.71E+12	SUBC3-FOS		7.71E+12	FATIGUE+FOS	
LAYER 4	4.50E+07	SUBSUBC3-FATIGUE		7.50E+26	FATIGUE+FOS	
	1.16E+08	SUBSUBC3-CRUSHING		1.16E+08	CRUSHING	
	7.50E+26	SUBSUBC3-FOS		7.50E+26	FATIGUE+FOS	
LAYER 5	2.35E+14	SUBGRADE		2.35E+14	SUBGRADE	
	2.75E+06	MINIMUM	MINIMUM	2.75E+06	7.5E+26	MAXIMUM

TRUN136		POISSON'S	LAYER	LAYER	E. x H	MODULAR	TRUN136
LAYER	E.MOD	RATIO	THICKNESS	RATIO		RATIOS	
1	2999	0.5	50	1/2	149.95	2.221481	
2	450	0.35	150	2/3	67.5	0.075012	
3	5999	0.35	150	3/4	899.85	1	
4	5999	0.35	150	4/5	899.85	0.012855	
5	70	0.35	1000000		70000		

	SINGLE CRITERIA LIFE			COMBINED FAILURE CRITERIA		
LAYER 1	2.31E+06	ASPH		2.31E+06	ASPHALT	
LAYER 2	4.40E+14	G1		4.40E+14	G1	
LAYER 3	6.58E+07	SUBC3-FATIGUE		7.30E+12	FATIGUE+FOS	
	6.48E+07	SUBC3-CRUSHING		6.48E+07	CRUSHING	
	7.30E+12	SUBC3-FOS		7.30E+12	FATIGUE+FOS	
LAYER 4	4.53E+07	SUBSUBC3-FATIGUE		5.72E+26	FATIGUE+FOS	
	1.17E+08	SUBSUBC3-CRUSHING		1.17E+08	CRUSHING	
	5.72E+26	SUBSUBC3-FOS		5.72E+26	FATIGUE+FOS	
LAYER 5	2.71E+14	SUBGRADE		2.71E+14	SUBGRADE	
	2.31E+06	MINIMUM	MINIMUM	2.31E+06	5.7E+26	MAXIMUM

L2

TRUN134		POISSON'S	LAYER	LAYER	E. x H	MODULAR	TRUN134
LAYER	E.MOD	RATIO	THICKNESS	RATIO		RATIOS	
1	2999	0.44	50	1/2	149.95	2.221481	
2	450	0.15	150	2/3	67.5	0.075012	
3	5999	0.35	150	3/4	899.85	1	
4	5999	0.35	150	4/5	899.85	0.012855	
5	70	0.35	1000000		70000		
	SINGLE CRITERIA LIFE				COMBINED FAILURE CRITERIA		
LAYER 1	7.24E+06	ASPH			7.24E+06	ASPHALT	
LAYER 2	6.81E+06	G1			6.81E+06	G1	
LAYER 3	6.67E+07	SUBC3-FATIGUE			2.47E+15	FATIGUE+FOS	
	6.48E+07	SUBC3-CRUSHING			6.48E+07	CRUSHING	
	2.47E+15	SUBC3-FOS			2.47E+15	FATIGUE+FOS	
LAYER 4	4.58E+07	SUBSUBC3-FATIGUE			2.45E+26	FATIGUE+FOS	
	1.19E+08	SUBSUBC3-CRUSHING			1.19E+08	CRUSHING	
	2.45E+26	SUBSUBC3-FOS			2.45E+26	FATIGUE+FOS	
LAYER 5	2.77E+14	SUBGRADE			2.77E+14	SUBGRADE	
	6.81E+06	MINIMUM		MINIMUM	6.81E+06	2.4E+26	MAXIMUM

TRUN20		POISSON'S	LAYER	LAYER	E. x H	MODULAR	TRUN20
LAYER	E.MOD	RATIO	THICKNESS	RATIO		RATIOS	
1	2999	0.44	50	1/2	149.95	2.221481	
2	450	0.35	150	2/3	67.5	0.075012	
3	5999	0.35	150	3/4	899.85	1	
4	5999	0.35	150	4/5	899.85	0.012855	
5	70	0.35	1000000		70000		
	SINGLE CRITERIA LIFE				COMBINED FAILURE CRITERIA		
LAYER 1	2.75E+06	ASPH			2.75E+06	ASPHALT	
LAYER 2	5.44E+14	G1			5.44E+14	G1	
LAYER 3	6.57E+07	SUBC3-FATIGUE			7.71E+12	FATIGUE+FOS	
	6.38E+07	SUBC3-CRUSHING			6.38E+07	CRUSHING	
	7.71E+12	SUBC3-FOS			7.71E+12	FATIGUE+FOS	
LAYER 4	4.50E+07	SUBSUBC3-FATIGUE			7.50E+26	FATIGUE+FOS	
	1.16E+08	SUBSUBC3-CRUSHING			1.16E+08	CRUSHING	
	7.50E+26	SUBSUBC3-FOS			7.50E+26	FATIGUE+FOS	
LAYER 5	2.35E+14	SUBGRADE			2.35E+14	SUBGRADE	
	2.75E+06	MINIMUM		MINIMUM	2.75E+06	7.5E+26	MAXIMUM

TRUN132		POISSON'S	LAYER	LAYER	E. x H	MODULAR	TRUN132
LAYER	E.MOD	RATIO	THICKNESS	RATIO		RATIOS	
1	2999	0.44	50	1/2	149.95	2.221481	
2	450	0.5	150	2/3	67.5	0.075012	
3	5999	0.35	150	3/4	899.85	1	
4	5999	0.35	150	4/5	899.85	0.012855	
5	70	0.35	1000000		70000		
	SINGLE CRITERIA LIFE				COMBINED FAILURE CRITERIA		
LAYER 1	1.45E+06	ASPH			1.45E+06	ASPHALT	
LAYER 2	1.66E+29	G1			1.66E+29	G1	
LAYER 3	6.41E+07	SUBC3-FATIGUE			2.11E+09	FATIGUE+FOS	
	5.46E+07	SUBC3-CRUSHING			5.46E+07	CRUSHING	
	2.04E+09	SUBC3-FOS			2.11E+09	FATIGUE+FOS	
LAYER 4	4.40E+07	SUBSUBC3-FATIGUE			3.78E+27	FATIGUE+FOS	
	1.13E+08	SUBSUBC3-CRUSHING			1.13E+08	CRUSHING	
	3.78E+27	SUBSUBC3-FOS			3.78E+27	FATIGUE+FOS	
LAYER 5	1.97E+14	SUBGRADE			1.97E+14	SUBGRADE	
	1.45E+06	MINIMUM		MINIMUM	1.45E+06	1.7E+29	MAXIMUM

L3

TRUN131 LAYER	E.MOD	POISSON'S RATIO	LAYER THICKNESS	LAYER RATIO	E. x H	MODULAR RATIOS	TRUN131
1	2999	0.44	50	1/2	149.95	2.221481	
2	450	0.35	150	2/3	67.5	0.075012	
3	5999	0.1	150	3/4	899.85	1	
4	5999	0.1	150	4/5	899.85	0.012855	
5	70	0.35	1000000		70000		

	SINGLE CRITERIA LIFE		COMBINED FAILURE CRITERIA	
LAYER 1	2.87E+06 ASPH		2.87E+06 ASPHALT	
LAYER 2	1.15E+15 G1		1.15E+15 G1	
LAYER 3	6.81E+07 SUBC3-FATIGUE		1.57E+10 FATIGUE+FOS	
	6.77E+07 SUBC3-CRUSHING		6.77E+07 CRUSHING	
	1.56E+10 SUBC3-FOS		1.57E+10 FATIGUE+FOS	
LAYER 4	4.33E+07 SUBSUBC3-FATIGUE		7.58E+25 FATIGUE+FOS	
	1.17E+08 SUBSUBC3-CRUSHING		1.17E+08 CRUSHING	
	7.58E+25 SUBSUBC3-FOS		7.58E+25 FATIGUE+FOS	
LAYER 5	1.28E+14 SUBGRADE		1.28E+14 SUBGRADE	
	2.87E+06 MINIMUM	MINIMUM	2.87E+06	7.6E+25 MAXIMUM

TRUN20 LAYER	E.MOD	POISSON'S RATIO	LAYER THICKNESS	LAYER RATIO	E. x H	MODULAR RATIOS	TRUN20
1	2999	0.44	50	1/2	149.95	2.221481	
2	450	0.35	150	2/3	67.5	0.075012	
3	5999	0.35	150	3/4	899.85	1	
4	5999	0.35	150	4/5	899.85	0.012855	
5	70	0.35	1000000		70000		

	SINGLE CRITERIA LIFE		COMBINED FAILURE CRITERIA	
LAYER 1	2.75E+06 ASPH		2.75E+06 ASPHALT	
LAYER 2	5.44E+14 G1		5.44E+14 G1	
LAYER 3	6.57E+07 SUBC3-FATIGUE		7.71E+12 FATIGUE+FOS	
	6.38E+07 SUBC3-CRUSHING		6.38E+07 CRUSHING	
	7.71E+12 SUBC3-FOS		7.71E+12 FATIGUE+FOS	
LAYER 4	4.50E+07 SUBSUBC3-FATIGUE		7.50E+26 FATIGUE+FOS	
	1.16E+08 SUBSUBC3-CRUSHING		1.16E+08 CRUSHING	
	7.50E+26 SUBSUBC3-FOS		7.50E+26 FATIGUE+FOS	
LAYER 5	2.35E+14 SUBGRADE		2.35E+14 SUBGRADE	
	2.75E+06 MINIMUM	MINIMUM	2.75E+06	7.5E+26 MAXIMUM

TRUN137 LAYER	E.MOD	POISSON'S RATIO	LAYER THICKNESS	LAYER RATIO	E. x H	MODULAR RATIOS	TRUN137
1	2999	0.44	50	1/2	149.95	2.221481	
2	450	0.35	150	2/3	67.5	0.075012	
3	5999	0.4	150	3/4	899.85	1	
4	5999	0.4	150	4/5	899.85	0.012855	
5	70	0.35	1000000		70000		

	SINGLE CRITERIA LIFE		COMBINED FAILURE CRITERIA	
LAYER 1	2.72E+06 ASPH		2.72E+06 ASPHALT	
LAYER 2	4.43E+14 G1		4.43E+14 G1	
LAYER 3	6.51E+07 SUBC3-FATIGUE		2.39E+13 FATIGUE+FOS	
	6.05E+07 SUBC3-CRUSHING		6.05E+07 CRUSHING	
	2.39E+13 SUBC3-FOS		2.39E+13 FATIGUE+FOS	
LAYER 4	4.56E+07 SUBSUBC3-FATIGUE		1.16E+27 FATIGUE+FOS	
	1.16E+08 SUBSUBC3-CRUSHING		1.16E+08 CRUSHING	
	1.16E+27 SUBSUBC3-FOS		1.16E+27 FATIGUE+FOS	
LAYER 5	2.95E+14 SUBGRADE		2.95E+14 SUBGRADE	
	2.72E+06 MINIMUM	MINIMUM	2.72E+06	1.2E+27 MAXIMUM

L4

TRUN131		POISSON'S	LAYER	LAYER	E. x H	MODULAR	TRUN131
LAYER	E.MOD	RATIO	THICKNESS	RATIO		RATIOS	
1	2999	0.44	50	1/2	149.95	2.221481	
2	450	0.35	150	2/3	67.5	0.075012	
3	5999	0.1	150	3/4	899.85	1	
4	5999	0.1	150	4/5	899.85	0.012855	
5	70	0.35	1000000		70000		
	SINGLE CRITERIA LIFE				COMBINED FAILURE CRITERIA		
LAYER 1	2.87E+06 ASPH				2.87E+06 ASPHALT		
LAYER 2	1.15E+15 G1				1.15E+15 G1		
LAYER 3	6.81E+07 SUBC3-FATIGUE				1.57E+10 FATIGUE+FOS		
	6.77E+07 SUBC3-CRUSHING				6.77E+07 CRUSHING		
	1.56E+10 SUBC3-FOS				1.57E+10 FATIGUE+FOS		
LAYER 4	4.33E+07 SUBSUBC3-FATIGUE				7.58E+25 FATIGUE+FOS		
	1.17E+08 SUBSUBC3-CRUSHING				1.17E+08 CRUSHING		
	7.58E+25 SUBSUBC3-FOS				7.58E+25 FATIGUE+FOS		
LAYER 5	1.28E+14 SUBGRADE				1.28E+14 SUBGRADE		
	. 2.87E+06 MINIMUM			MINIMUM	2.87E+06	7.6E+25 MAXIMUM	

TRUN20		POISSON'S	LAYER	LAYER	E. x H	MODULAR	TRUN20
LAYER	E.MOD	RATIO	THICKNESS	RATIO		RATIOS	
1	2999	0.44	50	1/2	149.95	2.221481	
2	450	0.35	150	2/3	67.5	0.075012	
3	5999	0.35	150	3/4	899.85	1	
4	5999	0.35	150	4/5	899.85	0.012855	
5	70	0.35	1000000		70000		
	SINGLE CRITERIA LIFE				COMBINED FAILURE CRITERIA		
LAYER 1	2.75E+06 ASPH				2.75E+06 ASPHALT		
LAYER 2	5.44E+14 G1				5.44E+14 G1		
LAYER 3	6.57E+07 SUBC3-FATIGUE				7.71E+12 FATIGUE+FOS		
	6.38E+07 SUBC3-CRUSHING				6.38E+07 CRUSHING		
	7.71E+12 SUBC3-FOS				7.71E+12 FATIGUE+FOS		
LAYER 4	4.50E+07 SUBSUBC3-FATIGUE				7.50E+26 FATIGUE+FOS		
	1.16E+08 SUBSUBC3-CRUSHING				1.16E+08 CRUSHING		
	7.50E+26 SUBSUBC3-FOS				7.50E+26 FATIGUE+FOS		
LAYER 5	2.35E+14 SUBGRADE				2.35E+14 SUBGRADE		
	2.75E+06 MINIMUM			MINIMUM	2.75E+06	7.5E+26 MAXIMUM	

TRUN137		POISSON'S	LAYER	LAYER	E. x H	MODULAR	TRUN137
LAYER	E.MOD	RATIO	THICKNESS	RATIO		RATIOS	
1	2999	0.44	50	1/2	149.95	2.221481	
2	450	0.35	150	2/3	67.5	0.075012	
3	5999	0.4	150	3/4	899.85	1	
4	5999	0.4	150	4/5	899.85	0.012855	
5	70	0.35	1000000		70000		
	SINGLE CRITERIA LIFE				COMBINED FAILURE CRITERIA		
LAYER 1	2.72E+06 ASPH				2.72E+06 ASPHALT		
LAYER 2	4.43E+14 G1				4.43E+14 G1		
LAYER 3	6.51E+07 SUBC3-FATIGUE				2.39E+13 FATIGUE+FOS		
	6.05E+07 SUBC3-CRUSHING				6.05E+07 CRUSHING		
	2.39E+13 SUBC3-FOS				2.39E+13 FATIGUE+FOS		
LAYER 4	4.56E+07 SUBSUBC3-FATIGUE				1.16E+27 FATIGUE+FOS		
	1.16E+08 SUBSUBC3-CRUSHING				1.16E+08 CRUSHING		
	1.16E+27 SUBSUBC3-FOS				1.16E+27 FATIGUE+FOS		
LAYER 5	2.95E+14 SUBGRADE				2.95E+14 SUBGRADE		
	2.72E+06 MINIMUM			MINIMUM	2.72E+06	1.2E+27 MAXIMUM	

L5

TRUN138		POISSON'S	LAYER	LAYER	E. x H	MODULAR	TRUN138
LAYER	E.MOD	RATIO	THICKNESS	RATIO		RATIOS	
1	2999	0.44	50	1/2	149.95	2.221481	
2	450	0.35	150	2/3	67.5	0.075012	
3	5999	0.35	150	3/4	899.85	1	
4	5999	0.35	150	4/5	899.85	0.012855	
5	70	0.2	1000000		70000		
	SINGLE CRITERIA LIFE				COMBINED FAILURE CRITERIA		
LAYER 1	2.74E+06	ASPH			2.74E+06	ASPHALT	
LAYER 2	5.35E+14	G1			5.35E+14	G1	
LAYER 3	6.58E+07	SUBC3-FATIGUE			1.15E+13	FATIGUE+FOS	
	6.38E+07	SUBC3-CRUSHING			6.38E+07	CRUSHING	
	1.15E+13	SUBC3-FOS			1.15E+13	FATIGUE+FOS	
LAYER 4	4.55E+07	SUBSUBC3-FATIGUE			6.46E+26	FATIGUE+FOS	
	1.16E+08	SUBSUBC3-CRUSHING			1.16E+08	CRUSHING	
	6.46E+26	SUBSUBC3-FOS			6.46E+26	FATIGUE+FOS	
LAYER 5	2.30E+13	SUBGRADE			2.30E+13	SUBGRADE	
	2.74E+06	MINIMUM		MINIMUM	2.74E+06	6.5E+26	MAXIMUM

TRUN20		POISSON'S	LAYER	LAYER	E. x H	MODULAR	TRUN20
LAYER	E.MOD	RATIO	THICKNESS	RATIO		RATIOS	
1	2999	0.44	50	1/2	149.95	2.221481	
2	450	0.35	150	2/3	67.5	0.075012	
3	5999	0.35	150	3/4	899.85	1	
4	5999	0.35	150	4/5	899.85	0.012855	
5	70	0.35	1000000		70000		
	SINGLE CRITERIA LIFE				COMBINED FAILURE CRITERIA		
LAYER 1	2.75E+06	ASPH			2.75E+06	ASPHALT	
LAYER 2	5.44E+14	G1			5.44E+14	G1	
LAYER 3	6.57E+07	SUBC3-FATIGUE			7.71E+12	FATIGUE+FOS	
	6.38E+07	SUBC3-CRUSHING			6.38E+07	CRUSHING	
	7.71E+12	SUBC3-FOS			7.71E+12	FATIGUE+FOS	
LAYER 4	4.50E+07	SUBSUBC3-FATIGUE			7.50E+26	FATIGUE+FOS	
	1.16E+08	SUBSUBC3-CRUSHING			1.16E+08	CRUSHING	
	7.50E+26	SUBSUBC3-FOS			7.50E+26	FATIGUE+FOS	
LAYER 5	2.35E+14	SUBGRADE			2.35E+14	SUBGRADE	
	2.75E+06	MINIMUM		MINIMUM	2.75E+06	7.5E+26	MAXIMUM

TRUN139		POISSON'S	LAYER	LAYER	E. x H	MODULAR	TRUN139
LAYER	E.MOD	RATIO	THICKNESS	RATIO		RATIOS	
1	2999	0.44	50	1/2	149.95	2.221481	
2	450	0.35	150	2/3	67.5	0.075012	
3	5999	0.35	150	3/4	899.85	1	
4	5999	0.35	150	4/5	899.85	0.012855	
5	70	0.5	1000000		70000		
	SINGLE CRITERIA LIFE				COMBINED FAILURE CRITERIA		
LAYER 1	2.71E+06	ASPH			2.71E+06	ASPHALT	
LAYER 2	4.78E+14	G1			4.78E+14	G1	
LAYER 3	6.55E+07	SUBC3-FATIGUE			3.96E+12	FATIGUE+FOS	
	6.43E+07	SUBC3-CRUSHING			6.43E+07	CRUSHING	
	3.96E+12	SUBC3-FOS			3.96E+12	FATIGUE+FOS	
LAYER 4	4.47E+07	SUBSUBC3-FATIGUE			9.16E+26	FATIGUE+FOS	
	1.17E+08	SUBSUBC3-CRUSHING			1.17E+08	CRUSHING	
	9.16E+26	SUBSUBC3-FOS			9.16E+26	FATIGUE+FOS	
LAYER 5	8.88E+17	SUBGRADE			8.88E+17	SUBGRADE	
	2.71E+06	MINIMUM		MINIMUM	2.71E+06	9.2E+26	MAXIMUM

APPENDIX B

SUMMARISED FINDINGS OF THE LITERATURE STUDY

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APPENDIX B: SUMMARISED FINDINGS OF LITERATURE STUDY

B.1 Asphalt

B.1.1 **Reference**: Maccarrone, S. and Jameson, G.W. Fatigue and Elastic Characterisation of Asphaltic Concrete Mixes. Proc.14 AARB Conference, Canberra, Australia, August 1988.

Abstract:

A laboratory investigation was undertaken to determine the effect of several mix variables on the fatigue life and mix stiffness of three commonly used continuously graded asphalt concrete mixes by the Road Construction Authority. A repeated flexure machine operating in controlled stress stress mode was used to test rectangular beams of about 50 mm square. The factors investigated were bitumen content and viscosity class, air voids,loading time, temperature and tensile stress. The effects of these factors on mix stiffness were shown to agree with work reported by others. The stiffness doubles for every 7% decrease in air voids and every 7% decrease in temperature. For asphaltic concrete temperatures between 10-30˚C, mix stiffness can be reliably estimated from the SHELL 1978 pavement design manual. Stiffness values at various traffic speeds are given for use in mechanistic pavement design. The data suggests that the SHELL fatigue life relationships adopted in the NAASRA Pavement Design Guide tend to underestimate field performance for thin (<50mm) asphaltic concrete surface pavements but are satisfactiry for full depth and deep strength asphaltic concrete pavements. No changes to the NAASRA Guide fatigue relationships are recommended at this stage.

General Points

Test beams were contructed of the continuously graded mix having the constituents detailed in Tables B.1.1 and B.1.2

Table B.1.1 Mineral Composition of Mixes

MIX	Material	% in Mix
A	14mm Rhyolite 10mm " 7mm " 5mm " Fine sand Cement works flue dust	20.0 10.0 30.0 20.5 17.0 2.5
B	14mm Basalt 10mm " 7mm " 5mm " Coarse sand Fine sand Cement works flue dust	31.0 10.0 9.0 28.5 15.0 5.0 1.5
C	14mm Basalt 10mm " 7mm " 5mm " Medium sand Cement works flue dust	26.0 14.0 22.0 14.0 23.0 1.0

Table B.1.2 Mineral Gradation of Mixes

MIX A	Mix B	Mix C	Sieve size
100	100	100	19.0mm
97	97	98	13.2mm
82	79	77	9.5mm
70	62	63	6.7mm
55	55	51	4.75mm
40	42	40	2.36mm
32	31	34	1.18mm
24	23	26	600μm
16	16	14	300μm
8	9	7	150μm
5.3	5.9	5.8	75μm

Relationships between mix stiffness and temperature, loading time, air voids, compaction, bitumen type and viscosity, load applications and tensile stress are given in Figures B.1.1a to B.1.1f.

Conclusions drawn from the study are as follows:

For temperatures between 10° and 30° C, mix stiffness can be reliably estimated from the SHELL(1978) procedure using measured bitumen stiffness.

Mix stiffness decreases with loading time. The effect becomes progressively more significant with increasing temperature. For the class 170 bitumen mixes investigated at Marshall bitumen content and air voids, and at a mix temperature of about 25 C, stiffnesss is estimated to be approximately 2500, 3500 and 4400 MPa at vehicle speeds of 30, 60 and 100 km/h respectively.

Mix stiffness is not stress dependent over the tensile stress range 200-1000 kPa.

Mix stiffness doubles for a 7% decrease in air voids and every 7 C decrease in temperature between 10-30 C.

Mix stiffness was generally at a maximum at the Marshall bitumen design (5%).

For two of the three mix types the stiffness of Class 320 bitumen mixes was about double that of Class 170 mixes; for the other mix type the stiffness was similar for the two bitumen viscosity classes.

With controlled stress testing fatigue life increased with mix stiffness. The fatigue life of Class 320 mixes at the Marshall design bitumen content and air voids had 2-5 times the fatigue life of equivalent Class 170 mixes. For this reason and the higher stiffness of Class 320, 320 mixes were used for base and intermediate courses where the total asphaltic concrete thickness is 150mm or more.

Field performance of asphaltic concrete pavements needs to be monitored and compared with laboratory measurements. This will allow suitable translation factors (between laboratory and field behaviour) to be obtained and pavement design methods refined.

B.1.2 **Reference**: Jameson, G.W., Sharp, K.G. and Vertessy, N.J. Full-Depth Asphalt Pavement Fatigue Under Accelerated Loading. Submitted to the 7th International Conference on Asphalt Pavements to be held at the University of Nottingham, England, 16th-20th August 1992.

Abstract

This paper describes the recent AcceleratedLoading Facility (ALF) trial conducted in Australia on full depth asphalt pavements nominally 120mm thick. The aim of the trial was to investigate the fatigue performance of of dense-graded asphalt. Extensive laboratory and field testing was conducted to complement the ALF trial. Relationships were established between back-calculated asphalt moduli, determined from Falling Weight Deflectometer deflection basins, pavement temperature and the severity and extent of surface cracking. The modulus was found to decrease markedly with increase of loading cycles before surface cracking was apparent. Fatigue relationships, derived for various extents and severities of surface cracking, suggested that, for the trial mix tested under ALF loading, the SHELL fatigue relationship is associated with about 50% of the loaded area having severe fatigue cracking.

General points

The test pavement was constructed approximately 1 year before testing and consisted of a nominal 70mm base course and 50mm wearing course of dense-graded asphalt respectively. The mean particle size distribution and bitumen content of the asphalt mixes are given in Table B.1.2a.

Table B.1.2a Particle size distribution and bitumen content

Wearing course	Base	Sieve size
100	99	19.0mm
96	90	13.2mm
83	78	9.5mm
58	52	4.75mm
34	31	1.18mm
17	18	300μm
8	8	150μm
5.5	5.4	75 μm
5.3	5	Bitumen Content (%)

Mean relative (Marshall laboratory) densities of the wearing course and basecourse were 96.5% and 98.3% respectively. The binder was a class 320 (40/50 penetration bitumen) with a softening point of 58°C.

Test results indicating the relationship between asphalt modulus, bitumen content, trafficking and speed of loading are shown in Table B.1.2b and Figures B.1.2a and B.1.2b.

Table B.1.2b Results of Laboratory Testing of Field Asphalt Cores

Air Voids (%)	Bitumen Content (%)	Effective Volume of Bitumen (%)	Nottingham Asphalt Tester Modulus (MPa)	SHELL Modulus (MPa)
7.2	5.1	10.9	2720	1980
6.6	5.4	11.6	1980	1960
5.7	5.2	11.1	2290	2210
4.3	5.2	11.3	2300	2430
6.3	5.3	11.4	2300	2040

B.1.3 **Reference**: Dohmen, L.J.M. and Molenaar, A.A.A. Full Scale Pavement Testing in the Netherlands, Submitted to the 7th International Conference on Asphalt Pavements to be held at the University of Nottingham, England, 16th-20th August 1992.

Abstract

Response studies have been carried out on several test pavements equipped with strain measurement devices. From the results it appears that the actual strain under a circular uniformly distributed load as produced by a falling weight deflectometer can easily be predicted with linear elastic multi-layer programmes, like for example, BISAR. The agreement between the measured and calculated strain due to a moving dual and single wheel was less. The measured strains were lower than those predicted. It is believed this is mainly due to the non-uniform contact pressure distribution but effects of visco-elasticity might have attributed to this as well.

Furthermore it appeared that there was a very good agreement between the asphalt layer stiffness backcalculated from deflection profiles and the laboratory determined mix stiffness.

General Points

Falling Weight Defectometer (FWD) measurements were carried out in addition to repeated load plate tests on test structures. THe FWD measurements showed (through backcalculation) that the overall stiffness modulus decreased quite rapidly with increasing numbers of load repetitions.

For each of the test pavements layer moduli backcalculated from deflection profiles correspond well with moduli determined from laboratory mixes at similar frequencies and temperatures. A 4-point bending test apparatus was used with cyclic loading.

From analyses it that to reliably backcalculate bituminous layer moduli good fits of peak deflection (Ymax) and suface curvature index are essential.

An apparently contradiction was measured during the testing: that of a rapid decrease of stiffness without an accompanying large increase of strain at the bottom of the layer. It was postulated that this lack of correlation of stiffness and strain measurements was due to a decrease of stiffness in the top of the material (due to the high energy imparted to the material by the high frequency test loading). When the FWD was used (at intervals of 10^5 load repetitions), the increased surface deflection therefore apparently showed the layer to have a lower stiffness than it actually possessed.

As the asphalt thickness reduced (from 240mm to 120mm), so stiffnesses also reduced (due to the relatively larger effect that the surface energy has on the whole layer).

This could be an important observation where other laboratory studies using accelerated loading are carried out.

B.1.4 **Reference**: De Beer, M. Developments in the Failure Criteria of the South African Mechanistic Design Procedure for Asphalt Pavements. Submitted to the 7th International Conference on Asphalt Pavements to be held at the University of Nottingham, England, 16th-20th August 1992.

Abstract

Effective implementation of the widely accepted mechanistic design method for asphalt pavements requires calibrated failure criteria and transfer functions. The paper describes various criteria developed and verified in South Africa in association with the full-scale accelerated testing of pavements, using the Heavy Vehicle Simulator (HVS) technology. Criteria such as the effective fatigue failure, compression (crushing) failure, erodibility of lightly cementitious pavement materials, and subgrade strain are discussed. Aspects of in-situ pavement response measuring techniques are discussed and include dynamic pavement charateristics, asphalt creep response, and temperature and load correction as well as the effects of vehicle speed on pavement response.

The particular relevant point in this paper now referred to is that of correction of IDM-measured deflections to a standard temperature and load (25°C and 40kN respectively). These corrections must be carried out before deflection basins are used for backcalculation of moduli.

The correction curves were drawn up for the pavement structure shown in Figure B.1.4. which is found on the N1 highway between Pretoria and Pietersburg from kM 2.8-3.0.

It is appreciated that the derived relationships are only truly applicable to this particular asphalt mix on this pavement. However the principles are valid for other asphalt mixes and with limited laboratory and/or field testing it should be possible to adjust the values of line gradient 'm' to suit each particular situation.

To correct test measurements to 'standard' readings the following equation has been derived from the test results indicated in Figure B.1.4 :

$$\delta_{25} = \frac{\delta T}{m} \left[\frac{40}{P} \right]$$

Where d_{25} = deflection corrected to reflect conditions at 25°C and 40kN,
dT = measured deflection at T°C,
m = gradient of temperature correction line
and P = the load at which deflections were measured.

B.1.5 **Reference**: Technical notes I/AT/51/90, I/AT/54/91, Asphalt Technology Programme, DRTT, CSIR, November 1991.

General Comments

Research on Heavy Duty Asphalt Pavements (HDAPs) is at present being carried out for the Southern African Bitumen and Tar Association (SABITA), at the Division for Roads and Transport Technology (DRTT). In-situ Heavy Vehicle Simulator (HVS) tests

were carried out on various test sections where different mixes were used.

In-situ depth-deflections were measured under moving wheel loads using Multi-Depth Deflectometers (MDDs) and a PC with a high speed data-logging card. Comparisons have been done for laboratory (ITT) tests using different compaction methods and the field results. A summary of the information is shown in Table B.1.5.

Table B.1.5 Comparison of field and laboratory measurements of stiffness (MPa).

Mix (at $25^{o}C/10Hz$ and ± 20km/h)	Hi-speed MDD	ITT (cores)	ITT-Hugo hammer & Field Mix	ITT-Hugo hammer & Lab.mix
Semi-gap*	1136	1298	1369	1505
Semi-open*	1360	1380	1470	1477
Continuous*	1138	1318	1381	1543

* A 60/70 Pen Sapref bitumen was used with dolorite aggregate.

Note that cores were removed after more than a month of test traffick compaction.

A conclusion of this study was that ITT test-derived stiffnesses can, with reasonable accuracy, predict in-situ moduli backcalculated from deflections (measured by MDDs) using linear elastic theory.

B.1.6 **Reference**: Vlok, M., Optimising Hot-mix Design:The effect of South African Aggregates on the Engineering Properties of an Asphalt Mix, Part of Project Report PR88/019, South African Roads Board, Research and Development Advisory Committee, Department of Transport, Pretoria, March 1991.

General Comment

Research was carried out to investigate the influence of 'fundemental' factors in design of bituminous mixes as the existing empirical methods do not optimize hot-mix design for specific conditions of materials, traffic, environment and pavement support. Factors investigated included shape factor, resilient modulus, Poisson's ratio and indirect tensile strength (ITS). 4 different aggregate types were used in the study: dolerite, norite, quartzite and hornfels. Norite and quartzite from the reef and Pretoria as they are very commonly used in asphalt mixes, and dolerite and hornfels from the Southern and Northern Cape Province because they probably possess the most marked variance in aggregate geometric propoerties from the quarztites and norites.

To control the gradation of the mixes, the aggregate materials of various sizes were combined according to the equation,

$$p = 100(d/D)^M$$

where p = the percentage of material which passes a given sieve with opening d.
D = the maximum size of aggregate of the given asphalt mix.
M = a variable exponent, termed "gradation index".

The maximum aggregate size selected for this study was 19 mm and three gradation indices of 0,40, 0,45 and 0,50 were selected using the above equation.

Tables 1 and 2 below give information regarding aggregate properties to assist the reader in assessing the engineering properties in following tables.

Table B.1.6a: Aggregate rugosity and shape factor of gradings

NORITE							
Grading		0.40		0.45		0.50	
Aggregate size (mm)	Rugosity	% Retained	Wghtd rugosity	% Retained	Wghtd rugosity	% Retained	Wghtd Rugosity
9.5	2.059288	0.123282	0.253873	0.137583	0.283323	0.15165	0.312292
4.75	4.490658	0.21229	0.953322	0.231091	1.03775	0.248476	1.11582
2.36	8.949097	0.16216	1.451188	0.170484	1.525679	0.177041	1.584353
1.18	5.669977	0.12162	0.689582	0.123486	0.700162	0.123845	0.702198
0.6	6.523458	0.090225	0.588577	0.088522	0.57747	0.085787	0.559631
0.3	6.967471	0.070324	0.489977	0.066677	0.464569	0.062445	0.435084
0.15	16.08333	0.053295	0.857165	0.04881	0.785032	0.044155	0.710164
0.075	27.32692	0.04039	1.103741	0.035731	0.976425	0.031223	0.853215
0.053	24.34134	0.126414	3.077085	0.097616	2.376095	0.075378	1.834798
SHAPE FACTOR			9.464511		8.726506		8.107566

Table B.1.6a (Cont): Aggregate rugosity and shape factor of gradings							
DOLERITE							
Grading		0.40		0.45		0.50	
Aggregate size	Rugosity	% Retained	Wghtd rugosity	% Retained	Wghtd rugosity	% Retained	Wghtd rugosity
9.5	2.585232	0.123282	0.318713	0.137583	0.355684	0.15165	0.392052
4.75	4.494553	0.21229	0.954148	0.231091	1.03865	0.248476	1.116788
2.36	6.118828	0.16216	0.992231	0.170484	1.043163	0.177041	1.083281
1.18	9.418488	0.12162	1.145476	0.123486	1.163051	0.123845	1.166432
0.6	13.60458	0.090225	1.22747	0.088522	1.204306	0.085787	1.167102
0.3	15.29131	0.070324	1.075339	0.066677	1.019577	0.062445	0.954867
0.15	14.00304	0.053295	0.746296	0.04881	0.683493	0.044155	0.618309
0.075	20.01607	0.04039	0.808454	0.035731	0.715199	0.031223	0.624952
0.053	24.00906	0.126414	3.035079	0.097616	2.343659	0.075378	1.809751
SHAPE FACTOR			10.30321		9.566781		8.933533
QUARTZITE							
Grading		0.40		0.45		0.50	
Aggregate size	Rugosity	% Retained	Wghtd rugosity	% Retained	Wghtd rugosity	% Retained	Wghtd rugosity
13.2							
9.5	3.547302	0.123282	0.437319	0.137583	0.488048	0.15165	0.53795
4.75	11.92414	0.21229	2.531376	0.231091	2.75556	0.248476	2.962861
2.36	13.18092	0.16216	2.137421	0.170484	2.247138	0.177041	2.333558
1.18	8.820521	0.12162	1.072751	0.123486	1.08921	0.123845	1.092377
0.6	1.753057	0.090225	0.158169	0.088522	0.155184	0.085787	0.15039
0.3	7.493186	0.070324	0.526947	0.066677	0.499622	0.062445	0.467912
0.15	13.35255	0.053295	0.711628	0.04881	0.651742	0.044155	0.589586
0.075	20.43729	0.04039	0.825467	0.035731	0.73025	0.031223	0.638104
0.053	29.48707	0.126414	3.727576	0.097616	2.878398	0.075378	2.222672
SHAPE FACTOR			12.12866		11.49515		10.99541

Table B.1.6a (Cont.) : Aggregate rugosity and shape factor of gradings.

HORNFELS

Grading		0.40		0.45		0.50	
Aggregate size(mm)	Rugosity	% Retained	Wghtd rugosity	% Retained	Wghtd rugosity	% Retained	Wghtd rugosity
9.5	0.75294	0.123282	0.092825	0.137583	0.103592	0.15165	0.114185
4.75	17.84187	0.21229	3.787650	0.231091	4.123092	0.248476	4.433273
2.36	17.35197	0.16216	2.813799	0.170484	2.958236	0.177041	3.072002
1.18	16.05233	0.12162	1.952283	0.123486	1.982237	0.123845	1.988000
0.6	15.78329	0.090225	1.424044	0.088522	1.397170	0.085787	1.354009
0.3	16.61428	0.070324	1.168374	0.066677	1.107788	0.062445	1.037479
0.15	16.11154	0.053295	0.8588668	0.04881	0.786409	0.044155	0.711410
0.075	22.04889	0.04039	0.890560	0.035731	0.787834	0.031223	0.688422
0.053	24.36678	0.126414	3.080300	0.097616	2.378578	0.075378	1.836715
SHAPE FACTOR			16.06850		15.62494		15.23549

Table B.1.6b : Computed Densities.

Aggregate type	Grading index	Avg loose volume ml	Avg Shaken Volume ml	Loose Bulk Density (LBD) kg/m^3	Shaken Bulk Density (SBD) kg/m^3	Aggregate mass g	Apparent Density kg/m^3	LBD (%AD) %	SBD (%AD) %
DOLERITE	0.5	2125	1850	4000	1882.353	2162.162	2960	63.59	73.04
	0.45	2062	1775	3901	1891.853	2197.746	2960	63.91	74.24
	0.4	2150	1850	3900	1813.953	2108.108	2960	61.28	71.21
QUARTZ	0.5	2325	2125	4000.4	1720.602	1882.541	2710	63.49	69.46
	0.45	2212	1925	3999.2	1807.548	2077.506	2710	66.69	76.66
	0.4	2587	2225	3999.8	1545.757	1797.663	2710	57.03	66.33
HORNFELS	0.5	2325	2100	4000.7	1720.731	1905.095	2770	62.12	68.77
	0.45	2237	1925	4009.1	1791.777	2082.649	2770	64.68	75.18
	0.4	2312	2050	4000.1	1729.773	1951.268	2770	62.44	70.44

NORITE	0.5	2175	1862	3999.6	1838.897	2147.436	2973	61.85	72.23
	0.45	2250	1850	4004.7	1779.867	2164.703	2973	59.86	72.81
	0.4	2175	1837	4000.1	1839.126	2176.925	2973	61.86	73.22

Table B.1.6b (cont.) : Computed Densities.

Binder origin and percentage content were kept constant to reduce the number of variables, i.e. a Natref 80/100 penetration at 5% binder content was used.

Indirect tension (ITT) tests were used to determine resilient moduli from which the results in Table B.1.6c were obtained:

Table B.1.6c : Summary of Indirect Tensile Test Results

Combinations	Aggregate/Grading	Resilient modulus (MPa)	Poisson ratio	Indirect Tensile Strength (kPa)	Strain at max stress	Shape factor
Combining materials	NORITE	2757	0.42	820	0.82	8.8
	DOLERITE	2697	0.5	904	0.80	9.6
	QUARTZITE	2529	0.24	1008	0.63	11.5
	HORNFELS	2557	0.46	974	0.48	15.6
Combining grading values	0.50	2601	0.32	868	0.76	10.8
	0.45	2627	0.50	992	0.63	11.4
	0.40	2677	0.28	920	0.72	12.0

Conclusions

Resilient modulus is not affected by the shape factor to any major degree. This seems to be due to the relatively small strains developed during the resilient modulus test being less than that required to mobilise the effects of aggregate particle interaction. The binder is therefore the controlling material property indicating that the resilient modulus test is mainly an indication of binder stiffness.

B.1.7 **Reference**: Vlok, M., Optimising Hot-mix Design: The effect of Briquette Dimensions on Indirect Tensile Testing, Part of Project Report PR88/019, South African Roads Board, Research and Development Advisory Committee, Department of Transport, Pretoria, March 1991.

General Comment

The effect of Briquette size on the measurement of engineering properties of asphalt was investigated with specific reference to tensile strength and elastic parameters. Briquettes were cut in two different ways resulting in either the centre portion of the sample being tested or half of the original briquette. A continuously graded mix using quartzite with a NATREF 80/100 penetration bitumen was used in the experimental programme. Table B.1.7a gives details of the grading used for the test specimens.

Table B.1.7a : Aggregate grading

SIEVE SIZE mm	CUMULATIVE PERCENTAGE PASSING
19.00	100
13.20	92
9.50	81
4.75	60
2.36	46
1.180	34
0.600	22
0.300	16
0.150	12
0.075	7

The above properties were estimated by assuming the validity of Hondros' elastic equations[B.1.6] and the condition of planar stress.

The final forms of the equations utilized were as follows:

Modulus of Elasticity E

$$E = \frac{P}{X_T}\left[\int_R^R \frac{\sigma rx}{P} - \nu \int_R^R \frac{\sigma\Theta x}{P}\right]$$

where

$\frac{P}{X_T}$ = the least squares line of best fit between load P and total deformation X_T for loads in the initial linear (or nearly linear) portion of the load-deformation curve.

ν = Poisson's ratio, as determined from the equation expressed below.

Poisson's Ratio ν

$$\nu = \frac{\left[\int_{-R}^{R} \frac{\sigma rx}{P} \quad \overline{R} \int_{-R}^{R} \sigma rx\right]}{\left[\overline{R} \int_{-R}^{R} \sigma\Theta x + \int_{-R}^{R} \sigma\Theta x + \int_{-R}^{R} \sigma\Theta y\right]}$$

(D-10)

where

$$\int_{-R}^{R} \sigma ry \qquad \int_{-R}^{R} \sigma rx =$$

integration of radial stresses in the y and x-directions- see Hondro's relationships[B.1.6].

$$\int_{-R}^{R} \sigma\Theta x \qquad \int_{-R}^{R} \sigma\Theta y =$$

integration of radial stresses in the x and y-directions, and

$$\overline{R} = \frac{Y_T}{X_T} =$$

the least squares line of best fit between vertical deformation Y_T and the corresponding horizontal deformation X_T for loads in the linear or nearly linear portion of the load deformation curves (generally up to about 50 percent of maximum load).

RESULTS

The results of the engineering properties as tested with the Asphalt Testing System are reflected in table B.1.7b below :

Table B.1.7b : Engineering properties of briquettes

SAMPLE	DIAMETER mm	THICKNESS mm	LOAD N	E MODULUS MPa	POISSON	TENSILE STR kPa
NOMINAL 150 mm DIAMETER						
HUGO COMPACTED - FULL SIZE						
M1	149	104	4104.5	1709.6	1.1	
M4	149	112	4078.2	2124.5	1.4	
M9	149	106	4106.9	1957.2	1.1	
M10	150	114	4080.0	2339.8	1.6	
M11	USED TO ASCERTAIN LOAD					
M12	149	102	4183.8	2641.6	1.5	
HALVED SAMPLE						
M1L	150	52	4019.6	2542.6	0.9	
M1R	149	46	3979.9	2356.8	0.8	
M12L	150	48	4006.2	2283.4	0.8	
M12R	149	54	4023.3	2087.7	0.8	
M1RL	150	22	966.3	2391.4	0.4	442.9
M1LL	149	25	1542.5	990.1	0.9	458.4
M12LL	149	23	1525.4	1965.0	0.2	
CENTRE RETAINED SAMPLE						
M4.1	150	49	4009.2	2701.3	0.9	
M9.1	149	50	3984.2	2417.4	0.9	
M10.1	149	47	4021.4	2735.7	0.7	
M4.2	149	31	1559.0	2429.9	0.7	514.4
M4.2	150	34	1548.0	2059.7	0.6	596.8
NOMINAL 100 mm DIAMETER BRIQUETTE						
HUGO COMPACTED & CORED - FULL SIZE						
M2	99	91	1120.7	1169.4	0.2	
M3	92	98	1266.6	1433.6	0.3	
M5	99	90	1225.7	1343.9	0.4	
M6	98	91	1088.3	958.8	0.3	
M7	99	91	1250.7	1504.9	0.3	
M8	99	92	1116.4	1316.8	0.2	

HALVED SAMPLE						
M3L	99	42	1449.7	1698.6	0.3	837
M3R	99	43	1435.7	1750.1	0.3	
M6L	99	44	1444.8	2397.3	0.3	762
M6R	99	44	1466.2	2440.0	0.2	
M3RL	99	22	998.0	1781.4	0.3	
M6RL	99	19	994.3	2783.0	0.2	981.5
M6RR	99	18	995.6	4070.5	0.3	1143.2
CENTRE RETAINED SAMPLE						
M2.1	99	53	1435.7	1489.2	0.3	
M5.1	99	53	1450.9	2127.0	0.3	
M8.1	99	53	1438.1	1890.9	0.4	
M2.2	99	31	1635.9	2629.3	0.3	912.7
M5.2	99	32	1646.3	2915.8	0.4	924.4
M8.2	99	34	1652.4	1801.7	0.2	851

Table B.1.7b (Cont.) : Engineering properties of briquettes

CONCLUSIONS AND RECOMMENDATIONS

Sample thickness does influence the quantified engineering properties and certain trends in relation to the thickness can be observed. The briquette dimensions of a sample to be tested should therefore not be assumed but accurately measured to improve the present formulae in combination with a theoretical investigation and a data base.

Where accurate results of engineering properties are to be ascertained it would be advisable to compact larger samples and core these as the effect of mould sidewall friction could then be eliminated.

It is recommended with the knowledge of the defined trends that a larger study be initiated to quantitatively define the roll of the briquette thickness on the engineering properties of the asphalt. Furthermore the effect of the frictional forces of briquette compaction should be investigated further as these have an effect on the results of engineering properties obtained.

B.2 **SUBGRADE**

B.2.1 **Reference**: Jameson, G.W., Sharp, K.G. and Vertessy, N.J. Full-Depth Asphalt Pavement Fatigue Under Accelerated Loading, Submitted to the 7th International Conference on Asphalt Pavements to be held at the University of Nottingham, England, 16th-20th August 1992.
See B.1.1 above for an abstract of the paper and general points regarding the backround to and findings of the tests.
The stress dependant nature of the subgrade was taken into account to allow test results of the Accelerated Loading Facility and a Falling Weight Deflectometer to be compared. Resilient moduli of the materials was measured in the laboratory over a range of deviator stresses. Results have been expressed in the general form

$$E = E_{max}(1-q/300)^p$$

Where E = subgrade resilient modulus (MPa)
E_{max} = maximum subgrade resilient modulus (MPa)
q = deviator stress (kPa)
and p = constant

A typical plot of results for the imported clay fill is given in Figure B.2.1. Results showed that "p" values ranging from 0 at the top of the subgrade and 2.2 for lower subgrade layers were appropriate.

B.2.2 **Reference**: Brown, S.F. and O'Reilly, M.P., The relationship between California Bearing Ratio and Elastic Stiffness for Compacted Clays, Ground Engineering, October 1990, London.
The paper shows that there is no unique relationship between resilient Young's Modulus and CBR due to the influence of deviator stress. In addition the resilient Young's modulus depends on the soil suction and soil type. Where soils are compacted at moisture contents greater than the plastic limit the relationship between resilient Youngs's modulus and CBR for a given pulse magnitude of deviator stress is $E_r = 17.6*CBR^{0.64}$ (MPa) which is similar as that proposed by TRRL, i.e. $E_r = 10*CBR$(MPa). This equation does not however take into acount soil type.
The study was carried out in the laboratory using triaxial samples with pneumatically-applied deviator stresses at 1 Hz.
Relationships between suction and deviator stress wre investigated for three soils: Keuper marl, Gault clay and London clay. The relationship developed between resilient modulus E_r, material parameters A and B that have ranges as shown in Table

B.2.2., the deviator stress pulse q_r and suction S is as follows:

$$E_r = \frac{q_r}{A} \left(\frac{S}{q_r} \right)^r$$

Typical CBR-E mod. relationships for compacted clay:

E = 10 CBR Applicable for deviator stresses of 60 MPa but not necessarily for others.

$E = 17.6\ CBR^{0.64}$

Note that soil type and stress level must be taken into account as the E_r vs CBR relationship is not constant due to the influence of deviator stresses.

B.2.3 **Reference**: Drumm, E.C., Boating-Poku, Y. and Johnson Pierce, T. Estimation of Subgrade Resilient Modulus from Standard Tests. Journal of Geotechnical Engineering, Vol.116, No.5, May, 1990.

Abstract: Mechanistic pavement design procedures based on elastic layer theory require the specification of elastic moduli for each material in the pavement section. Repeated load tests yielding a resilient modulus are frequently used to characterise the soil subgrade. Due to difficulties associated with cyclic testing, approximate methods are often used for design estimates of resilient modulus. These approximations are often based only on shear strength measures and do not account for the dependence on the magnitude of cyclic deviator stress. A procedure is described to relate the soil-index properties and the moduli obtained from the unconfined compression tests, to resilient modulus. Two statistical models are described and demonstated for 11 soils from throughout the state of Tennessee. One model provides an estimation of the breakpoint resilient modulus, or the modulus at a deviator stress of 6psi (41 kPa). The second model provides a general nonlinear relationship for the modulus of fine-grained soils as a function of deviator stress. Both models are demonstrated for a range of soils and are shown to provide a good characterisation of the response for the soils investigated. Similar relationships can be developed for other subgrade soils, and may prove useful to agencied that use deterministic design procedures but lack the capability for high-production repeated-load testing.

General Comments

The difference in load response of granular and cohesive materials is noted with particular mention made of bi-linear models that can be used to characterise the elastic response of cohesive materials. The influence of moisture content, dry density and index properties on fine- grained soils is discussed.

The AASHTO pavement design guide (1986)[B.2.5] is quoted as saying that agencies involved in pavement design should establish correlations between standard soil tests such as the California Bearing Ratio (CBR) and index properties to obtain design values of resilient moduli.
The AASHTO pavement design guide (see section B.2.5) suggests that the resilient modulus of fine-grained soil can be estimated as

$$E_r(\text{psi}) = 1500 \text{ CBR}$$

Other similar relationships are also mentioned for other unbound materials. It is noted that the CBR is a measure of strength and does not necessarily have to correlate to resilient modulus. Also, it should be noted that these simple relations do not take into account stress-dependency, and so may be limited in applicability.

It is proposed that the initial modulus may be a better measure of E_r than quantities related to shear strength. E_r is obtained from the initial portion of the stress-strain curbe (and therefore corresponds to small levels of stress and strain). The paper shows for the low plasticity silts and clays tested (from Tenessee) that a hyperbolic assumption of material stress-strain behavior fits test data well.
A bilinear representation of E_r vs $\sigma_{deviator}$ was fitted to test data to obtain the minimum and breakpoint modulus for each material. This data was then statistically compared to unconfined compression test parameters, index properties and grain-size distribution. The resulting stastical model was then used to predict E_r for the soils in the test program, giving satisfactory predictions of breakpoint modulus.
As E_r is dependant on the magnitude of deviator stress, a hyperbolic relationship between these properties was developed for the test data. Subsequent comparisons between measured and predicted stiffnesses showed good correlation.

If found applicable in South Africa, this technique would give laboratories lacking sophisticated testing facilities a means of predicting resilient moduli suffuciently accurately for pavement design.
To make the approach available to the general profession, extensive testing should be done initially on the main soil types experienced in South Africa. This would establish

the basic equations which could then be modified according to local environmental conditions.

B.2.4 **Reference**: Manual M10: Concrete Pavement Design (Draft), Department of Transport Chief Directorate: National Roads 1990, Pretoria.
To obtain subgrade stiffnesses the triaxial shear test is recommended using appropriate deviator stresses. If these test results are not available, then the DCP-CBR-Elastic modulus could be used (with care).

Curves of CBR and UCS versus elastic moduli given in this document were derived using linear elastic layered theory with finite element analysis. A relationship between subgrade modulus and CBR is given in Figure B.2.4a.

Some 'typical' relationships that can also be used are

for cohesive soils ($6<PI<20$) : $M_r=158-0.4566(PI)-0.0779(\%OMC)-0.1424(S200)$

for granular soils ($0<PI<10$ and $CBR>15$): $M_r=2.6885-0.02555(\%OMC)$

and for stabilised materials see Figure B.2.4b.

Then, to obtain a combined subgrade stiffness (MPa):

$$\textit{Combined Subgrade Stiffness} = \left[\frac{M_r.\delta_u+E_s.\delta s}{\delta u + \delta s}\right]$$

Where M_r is the resilient modulus of the unstabilised subgrade,
δ_u is the thickness of the unstabilised subgrade
E_r is the resilient modulus of the stabilised subgrade
and δ_s is the thickness of the stabilised subgrade.

B.2.5 **Reference**: AASHTO Guide for Design of Pavement Structures 1986, American Association of State Highway and Transportation Officials, Washington, D.C., USA. Section 1.5: ROADBED SOIL. The resilient modulus of soil E_r can be estimated using AASHTO Test Method T274 or the following expressions:

E_r(psi) = 1500 CBR

E_r(psi) = A+B*(R-value),

Where A and B are constants with ranges of values

A=772 to 1155,
B=369 to 555

and R-value= The 'Resistance R-value' calculated in accordance with AASHTO test method T190-78 (1982).

It is suggested that to obtain E_r, tests should be carried out on representative samples in stress and moisture conditions simulating those of the primary moisture seasons. Alternatively, seasonal resilient moduli may be determined from correlations of soil properties. i.e. clay content, moisture and PI for example.

The resilent modulus is a measure of the elastic properties of soils recognising certain non-linear characteristics. It can be directly used in the design of rigid pavements but is converted to a 'modulus of subgrade reaction' for use in rigid or composite pavement design.

Section 5.2.3 (Rehabilitation Methods With Overlays-General Overlay Methodology-Materials and Environmental study).

The use of deflection basins generated by steady state, dynamic or impulse loads is suggested. Deflections measured with the outer geophones of the basins can be used to derive subgrade moduli. It is important to obtain deflections from geophones where deflections are not influenced by the upper pavement layers. Bearing this in mind it is also important to ensure that geophones are placed close to the minimum distance from the falling weight to satisfy this criterion, otherwise predictive errors will occur.

When the outer geophone is located at a distance $1<a_e<6$, (where a_e is the effective radius of the stress bulb at the pavement-subgrade interface and r is the radial offset distance),

the subgrade modulus can be directly estimated from the relationship:

$$E_{sg} = \frac{(PS_f)}{(d_r r)}$$

Where E_{sg} = in-situ modulus of elasticy of the subgrade layer,
P = the dynamic load (in poinds) of the NDT device used to obtiain deflections,

d_r = the measured NDT deflection at a radial distance of r from the NDT plate load centre
r = the radial distance from the plate load centre to point of d_r measurement,
and
S_f = the subgrade modulus prediction factor.

For a given nondestructive test, the load P, outer deflection d and the radial distance r are known values. The S_f factor is a function of Poisson's Ratio (ν), and has the values given in Table B.2.5.

Table B.2.5 Poisson's ratio versus S_f

Poisson's Ratio	S_f
0.50	0.2686

For r/a_e ratios less than 1 approximate values of S_f are given in Figure B.2.5a.

<u>Estimating the Effective radius of the subgrade stress zone (a_e)</u>: If the r/a_e ratio is greater than 1, for any radial deflection measurement beyond the subgrade-pavement interface (a_e), the modulus is proportional to load, deflection and radial distance. To use this technique specific a_e values for a given NTD device must be estimated. This value is determined by the following expression:

$$a_e = \frac{a_c}{F_b}$$

Where a_c = the radius of the NDT plate, and
F_b = the deflection factor which is a function of the subgrade Poisson Ratio, ν,
and the pavement's effective thickness plate radius ratio (H_e/a_c)

Figure B.2.5b gives F_b plotted against the H_e/a_c ratio as a function of Poisson's ratio. An expression to calculate the effective transformed pavement thickness is given in the design guide.

A method to locate the outer geophone for subgrade modulus determination is given in the manual and is now summarised.

Because the a_e value is dependent upon both layer thickness and modulus, every pavement structure has it's own minimum (ideal) a_e value. As the pavement becomes thicker and/or stiffer, so the a_e value and deflection basin increases.
Where it is impossible to locate the geophone at a great enough distance from the load to obtain deflections resulting only from the subgrade, an approximate solution for the subgrade modulus prediction can be made using the (previously quoted) equation:

$$E_{sg}=\frac{(PS_f)}{(d_r r)}$$

Where the S_f value is now not constant but is function of the r/a_e value. To compute the a_e value as precisely as possible, the following steps should be followed:

(1) Assume layer moduli values (E_i),
(2) Compute the H_e value for the pavement,
(3) Compute the H_e/a_c ratio.
(4) Determine the F_b value from the appropriate figure in the manual,
(5) Compute a_e a_c/F_b
(6) Compute the r/a_e ratio and if much greater than 1 use constant values of S_f given in Figure 5.2.5b. If the $0.5<r/a_e<1$, use the approximate S_f values given in Figure 5.2.5b.
(7) Calculate the predicted subgrade modulus.

Note that if inaccurate assumptions for layer moduli are used to compute the a_e value and the resultant r/a_e ratio is <1, an unknown error may be introduced in to the calculations.

Assessing Nonlinear Subgrade Behaviour

Two methods are given:

(a) Take two deflection measurements from beyond the a_e value (r/a_e must be greater than 1 for both deflection readings). If the subgrade is linear, then two adjacent deflection measurements (at the same load) will give the following:

For linear subgrades: $dr_{n-1}/dr_n = r_n/r_{n-1}$,

and for non-linear subgrades: $dr_{n-1}/dr_n > r_n/r_{n-1}$

(b) This approach uses deflection measurements from a single geophone beyond the a_e radius, but at different loads. If material is linear then $P_1/P_2 = d_{1r}/d_{2r}$.
If material is non-linear the ratio of loads will not be equal to the deflection ratios. The manual gives suggestions of how to adjust for material nonlinearity through estimation of deviator stress at the subgrade-pavement interface and deriving eqations describing field non-linearity from plotting Log E versus Log deviatior stress.

B.2.6 **Reference**: Rohde, G.T., The Mechanistic Analysis of Pavement Deflections on Subgrades Varying with Depth, PhD Thesis, Office of Graduate studies of Texas A&M University, December 1990.

Abstract

Nondestructive deflection testing (NDT) has become an integral part of the structural evaluation of pavements. Interpretation of the measured defelction data is extremely complex and the analysed pavement is often modelled as a multilayered elastic system. In this model the subgrade is usually defined as uniformly stiff and infinitely thick, or a rigid layer is placed at an arbitrary depth. The actual subgrade on which the tested pavement structure is founded varies considerably from this model. It is not infinitely thick, and whether the subgrade is sedimentary or residual in nature, its stiffness normally changes with depth. This change in stiffness can be due to shallow bedrock, material differences, the stress history, or an apparent increase in stiffness due to the stress dependant behaviour of most soils.

In this study a method is developed to determone the apparent depth to a rigid layer from surface deflections is developed. This method is based on Boussinesq's equation and is related to a three layer linear elastic sytem through an extensive

regression analysis. The procedure is validated by comparing the predicted rigid layer depths from surface deflections on five pavement sections to that obtained through penetration testing and seismic refraction analysis.

The methodology is further extended to pavement systems where the subgrade stiffness increases with depth. A nonlinear elastic backcalculation technique based on a finite element approach, is used to illustrate the change in apparent stiffness with depth on a sandy and clay subgrade. It is shown that a three layer linear elastic sytem with an apparent rigid layer can be used to model the increasing stiffness with depth. This developed procedure is compared to existing backcalculation models. Monthly collected falling weight deflectometer (FWD) deflection data on ten in-service pavement sections are analysed and the results are compared in terms of the available laboratory data. The inclusion of an apparent rigid layer into the pavement model led to considerable improvements in the backcalculated layer moduli. The procedure is also evaluated on two pavement sections instrumented with multidepth deflectometers. The backcalcualted moduli are used to predict defletions within the pavement. THese predictions are compared to actual measured deflections to determine how accurate the pavement was modelled.

General Comment

Amongst the wealth of concise information contained within the document, the apparent 'depth to a rigid layer' concept stands out as a particularly important consideration for elastic subgrade moduli determination. An analysis technique is developed that uses graphs of deflection versus 1/(geophone offset distance) to help define the apparent depth to rigid base. Equations are developed for pavements with different asphalt surfacing thicknesses and checked on site by drilling and probing.

The influence of the apparent depth to rigid layer is illustrated in Figures B.2.6a-6.2.6d. Points on these graphs have been computed using MDD in-depth deflection data for the respective pavements in combination with linear elastic multilayer theory. With the appreciation that this material model is not entirely appropriate, the graphs nevertheless show a significant change in elastic moduli with different rigid layer depth assumptions.

The implication of an incorrect assumption of semi-infinite subgrade depth is (*inter alia*) a probable unconservative design where the subgrade is assumed to be stiffer than it actually is. Furthermore, the likely variation in pavement life can be inferred from Appendix B.

B.3 CONCRETE

From recent findings (see DOT report PR88/215 April 1991 it seems that (due to the design procedure) the elastic modulus of concrete is usually not as critical a parameter as for some of the other materials. Rather the subbase and subgrade stiffness (or support) is typically more important than $E_{concrete}$. Having said this however, the most accurate method of rigid pavement analysis and design is apparently the Finite Element technique which requires (*inter alia*) material stiffness as input data. In this case therefore the accurate and representative determination of $E_{concrete}$ is important.

There are many factors affecting the magnitude of elastic concrete stiffness. These factors include the aggregate type, age of the concrete, moisture content, load rate and the percentage of the various constituents in the mix. There are therefore a great variety of possible concrete stiffnesses for 'similar' mixes. In addition to the considerable number of factors influencing concrete stiffness directly, the widely-used correlations between strength and stiffness are also subject to variations in the above material properties.

To be confident in a design approach that uses stiffness as a design input therefore, some testing is required. One of the standard test procedures such as from BS1881 or ASTM C469-65, for example is suggested.

The following summaries of papers are thought appropriate for consideration when deriving concrete stiffness.

B.3.1 **Reference**: Foxworthy, P.T. and Darter, M.I., Illi-Slab and FWD Deflection Basins for Characterisation of Rigid Pavements. Non-Destructive Testing of Pavements and Backcalculation of Moduli, ASTM STP 1026, A.J. Bush III and G.Y. Baladi, Eds., American Society for Testing and Materials, Philadelphia, 1989.

Abstract

Deflection-based nondestructive evaluation methods for highway and airfield pavements rely wholly on mechanistic models of pavement behaviour under load to characterise certain fundamental properties of individual pavement features (pavement section of similar thickness, construction history, and traffic). Once these key pavement parameters are quantified, areas of immediate concern can be identified for maintenance or rehabilitation, and an evaluation of future performance of the entire feature can be made. The procedure sounds simple enough, until one tries to accomplish the task for an entire airfield which might contain well in excess of 200 distinct features. This paper details how the deflection basin created at the centre of a rigid pavement slab under loads produced by the falling weight deflectometer (FWD) can be used in conjunction with the ILLI-SLAB finite element model to backcalculate the two key parameters needed to characterise a classical Westergaard rigid pavement, a dynamic Young's Modulus (E) of the concrete surface, and a composite dynamic modulus of subgrade reaction (k) for the supporting layers of the system. The deflection basin is described in terms of two independent variables, the maximum deflection under the centre of the FWD loading plate (D0) and the cross sectional 'area' of the basin. The independent nature of these two variables is critical to the uniqueness of the backcalculated parameters. Using the ILLI-SLAB model, ranges of dynamic E and k that bound the actual field values are input to the computer, along with the actual FWD load, to produce a graphical solution. An iterative computer solution is then outlined that makes the task of backcalculating dynamic E and k for several hundred features more manageable. A correlation is presented that relates dynamic k values to traditional static k values determined from plate- bearing tests. Finally, comparisons between measured deflections in the field and predicted deflections using backcalculated parameters on the computer are made at centre slab to verify the accuracyand repeatability of th technique for a wide variety of temperatures and thicknesses. It is only after the validity of the technique is established that confidence can be placed in the calculated stresses due to actual loads and, therefore, the evaluation itself.

Findings and general comments

At low FWD load levels (31-35 kN) a poorer consistancy of backcalculated elastic moduli is found than when higher loads are used. Backcalculated elastic moduli are also often found to be unrealistically high at low stress levels.

The best consistency of results was found at temperatures between 7 and 32^{o}C.

ILLI-SLAB was successfully used to model and predict FWD deflections at slab centres for various types of aircraft loadings.

The FWD was found to be very suitable for the rapid collection of large volumes of rigid pavement evaluation data. All seven deflection geophones were found to be required to adequately define deflection basins on this type of pavement.

B.3.2 **Reference**: Alexander, D.R. Kohn, S.D. and Grogan, W.P., Nondestructive Testing Techniques and Evaluation Procedures for Airfield Pavements, Non-Destructive Testing of Pavements and Backcalculation of Moduli, ASTM STP 1026, A.J. Bush III and G.Y. Baladi, Eds., American Society for Testing and Materials, Philadelphia, 1989. 1992.

Abstract

Research efforts at the Waterways Experimental Station have resulted in the development of a methodology based upon a multilayered elastic model and limiting stress/strain criteria which use deflection basin mesurements obtained by applying a load to the pavement surface with a nondestructive testing (NDT) device.

Growing acceptance of the NDT methodology has led to the formulation of a comprehensive nondetructive testing and evaluation procedure for airfield pavements which has been used by and is under consideration for adoption by the Army and is also being used by the AirFirce and Navy (there is presently no official DA published manual or standard for NDT, but one is currently being drafted). A testing scheme is presented for airfield pavements that includes recommendations for the number of tests, location of tests, magnitude of NDT loadings, and geophone spacings. In addition to NDT data, other information such as layer thicknesses, surface and five-day mean air temperatures, and portland cement concrete flexural strengths are required for an accurate evaluation. The computer program BASIN has been developed to graphically and statistically analyse NDT deflection data and to select a representative deflection basin to be used for the evaluation of a pavement section. A description of the modulus backcalculation process using the computer program BISDEF is provided. The final evaluation is based upon a projection of the total number of passes of each type of aircraft type that a pavement will be expected to support over its design life. A method is described for determining the critical aircraft and design pass level for a given projected aircraft mixture. Allowable aircraft loads, strengthening overlay requirements, etc. are then obtained using the moduli from BISDEF and the program AIRPAVE, which compares stresses and strains within a layered system to appropriate limiting criteria. Limiting stress/strain criteria included in AIRPAVE have been calibrated with performance data to ensure consistancy with current Corps of Engineer design criteria.

The use and significance of the pavement classification number (PCN) determined from the allowable load rating for expressing the load-carrying capacity of a pavement by a single unique number without specifying a particular aircraft is discussed. Sample data have been used to demonstrate evaluation techniques and the presentation of

results.

The NDT procedure presented here provides a rapid and versitile method for determining the structural capacity of a pavement system. Rigid, flexible, and composite pavements consisting of stabilized or unstabilized layers can be evaluated by using data from a variety of commercially available NDT equipment. While developed to meet specific military requirements, the procedures are equally applicable to the evaluation of civil airports and could be modified to include the evaluation of roads and streets.

<u>General Comments</u>

Due to the temperature dependency of asphalt, it is suggested that laboratory temperature-stiffness relationships are set up and used with measured and/or predicted field temperatures.

An evaluation procedure is presented that is valid for rigid, flexible and composite pavements. It is based on a layered linear elastic model that characterises multilayered pavement systems.

As part of the pavement evaluation procedure elastic moduli are backcalculated from deflection basins. To do this the pavement is modelled as a layered system incorporating a rigid base at approximately 6 metres depth and using assumed values of Poisson's ratio. BISDEF is then used to produce a set of moduli that closely models the measured deflections. The set of moduli are computed using a technique that varies individual values of moduli and, after each variation , computes a new set of deflections. A set of equations are developed that define the slope and intercept of the equation

$$\log(\text{Deflection}) = A_{ji} + S_{ji}(\log E_i)$$

Where A = intercept,

S = slope,

j=1 to the number of deflections, and

i=1 to the number of layers with unknown modulus values.

The initial value for the subgrade modulus is estimated as

$$E = 59314.82(D72)^{-0.98737}$$

Where E = subgrade modulus, psi, and

D72 = deflection, mils measured at a distance of 72 in. from a loading of 25000lb (111 206 N).

If the surfacing layer is thin or if BISDEF gives unrealistic values, a modulus is usually assigned to the asphalt or concrete material. Asphalt moduli are chosen on the basis of laboratory test results which incorporate the variation of stiffness with temperature for a given loading frequency.

It is claimed that normally BISDEF requires 3 iterations to produce a set of moduli that gives a deflection basin where each deflection is within 3% of measured values.

Use of the technique on military pavements has shown it to be a viable method of testing and evaluating airfield pavements.

B.3.3 **Reference**: Alexander, M.G, Prediction of Elastic Modulus for Design of Concrete Structures, The Civil Engineer in South Africa, June 1985

Abstract

The prediction of the elastic modulus of concrete for design is complicated by the variety of factors affecting this parameter. THe influences of aggregate stiffness and concrete strength are incorporated in a new design approach which is being included in the new BS1881 (originally CP 110). The application of this new appraoch to South African conditions is discussed with specific reference to local aggregates and cements. Aggregates have been grouped according to their elastic properties so that designers can make better estimates of concrete modulus than might have previously been possible. The ideal approach to elastic modulus prediction by means of the concept of two-phase models is also introduced.

General Comments

Note that stiffer aggregates result in higher moduli of concretes in which they are used. Also, for a given aggregate the elastic modulus increases with the strength of the concrete.

Simplified design rules may state a single value of concrete modulus, i.e. 30 GPa at 28 days regardless of other factors. The table below shows how this value could apply to grades of concrete between 20 and 60 MPa. An alternative method is based on the relationship between elastic modulus and compressive strength, including the effect of aggregate stiffness. In general a mean value for the 28-day elastic modulus can be estimated from the expression:

$$E_{28}(\text{GPa}) = K_o + 0.2f_{28}.$$

Where K_o can be taken from Table B.3.3a

Table B.3.3a Values of concrete elastic modulus. (E_{28} values in GPa).

Aggregate Group					
f_{28} (MPa)	Group 1 Mean K_o=15GPa (K_o range=10-19 GPa)		Group 2 Mean K_o=15GPa (K_o range=20-30 GPa)		Modulus values with K_o=20 GPa
	Mean	Range	Mean	Range	
20			29	24-33	24
25	19	14-23	30	25-35	25
30	20	15-24	31	26-36	26
40	21	16-25	32	27-37	27
50	22	17-26	33	28-38	28
60	23	18-27	35	30-40	30
		(*)	37	32-42(**)	32

* No modulus values are quoted for high strength concrete using low modulus aggregates, since these aggregates are also likely to be of low strength. Such low strength aggregates will affect the performance of high strength values.

** A maximum practical value of elastic modulus likely to be achieved in practice is considered to be 42 GPa. Quoted results for elastic modulus are seldom above this value.

The suggested aggregate groups therefore comprise the following:

Group 1: (Mean K_o=15 GPa, K_o range=10-19 GPa).
Aggregates with low to medium values of elastic modulus such as karoo sandstones

and poorer quality granites.

Group 2: (Mean K_o=25 GPa, K_o range=20-30 GPa).
Aggregates with medium to high values of elastic modulus such as good quality granites, quartzites and quartzitic sandstones, basic igneous rocks and dolomites.

To estimate elastic concrete moduli at ages other than at 28 days, the following equation may be used:

$$E_{time} = E_{28}(0.4 + 0.6 f_t/f_{28})$$

Vaues of f_t/f_{28} for different ages of concrete can be taken from Table B.3.5b

Table B.3.3b Stress ratios for concrete at different ages.

Age t (days)	3	7	28	90	365
Western Province OPC	0.61	0.82	1.00	-	-
Eastern Province OPC	0.58	0.78	1.00	-	-
Other SA OPC	0.50	0.70	1.00	(1.10)	(1.15)
PC15	0.46	0.66	1.00	-	-
PBFC	0.41	0.56	1.00	(1.20)	(1.25)
RHC	0.55	0.74	1.00	-	-

Where OPC = Ordinary Portland cement,
PC15 = OPC containing up to 15% blast furnace slag,
PBFC = Portland blast furnace cement
and RHC = Rapid Hardening Cement.

The above values represent averages for a c:w ratio of 1.67 at the various ages.

A direct comparison of compressive strength vs elastic concrete modulus where the elastic properties of the aggregate is not taken into account is also given:

$$E(GPa) = 4.9 * f_c^{0.5} \text{ (MPa)}$$

Where moduli were determined in accordance with BS 1881:Part 5:1970.

An alternative prediction model is given by Hobbs, (Hobbs, D.W., The Dependence of the Bulk Modulus, Young's Modulus, Creep, Shrinkage and Thermal Expansion of Concrete upon Aggregate, Materiaux et Constructions, Vol.4, No. 20, 1971, pp 107-114):

$$E_c = E_m \frac{[(1-V_a)E_m + (1+V_a)E_a]}{[(1+V_a)+(1-V_a)E_a]}$$

Where E_c = modulus of the composite
E_m = modulus of the matrix
E_a = modulus of the embedded aggregate
V_a = volume fraction of the aggregate.

B.3.4 **Reference:** BS1881:Part 5:1970, Methods of Testing Hardened Concrete for other than Strength, British Standards Institution, London.

Two tests are mentioned for the determination of elastic modulus:

(a) The static test (to obtain E_{cs})

This test is carried out on prepared (or cored) cylindrical concrete test specimens with diameters of 150mm and being 300mm high. Specimens are prepared as detailed in Part 3 of BS1881.

Values of strain under prescribed load cyles are derived from extensometer readings. To obtain values of elastic modulus from results, corresponding points of stress and strain are plotted for each extensometer. Provided the slopes of the two lines do not differ by more than 15%, the average slope is taken and expressed (to the nearest 500 MPa) as the modulus of elasticity. If the discrepancy is greater than 15%, then the test is repeated.

(b) The dynamic test (to obtain E_{cd})

This is a non-destructive test and the specimens tested can therefore be used for other test procedures.

BS1881:Parts 3 and 4 describe preperation of test beams and the sawing of specimens from hardened concrete respectively.

A dynamic variable frequency excitor is fixed to the one end of the beam and a vibration pick-up fixed to the other end. Frequencies are modified until resonance is obtained in the fundamental mode of longitudinal vibration.

Values of the dynamic modulus are calculated using the relation

$$E_{cd} = 4n^2l^2w*10^{-12}$$

Where l = specimen length (mm)
n = natural frequency of the fundamental mode of longitudinal vibration of the specimen in hertz,
and w = density in kilogrammes per cubic metre.

Various relationships between static and dynamic moduli have been suggested by different sources. The one proposed by SABS 0100 is probably suitable for most uses and is expressed as follows:

$$E_c = 1.25\ E_{cq} - 19\text{(GPa)}$$

Where E_c and E_{cq} are the static and dynamic moduli respectively.

B.3.5 **Reference:** SABS 0100:Part 1-1980, Code of Practice for The Structural Use of Concrete, Part-1: Design, The Council of the South African Bureau of Standards, Pretoria.

Directly-relevant information regarding elastic moduli is to be found in Appendix D: Elastic Deformation of Concrete.

Much of the information above regarding $E_{concrete}$ being dependant on the crushing strength of the concrete and the elastic properties of the aggregate. If concrete made with natural aggregates which have a density of 2300 kg/m^3 or more, static and dynamic moduli may be taken from Table B.3.5.

Table B.3.5. Moduli of Elasticity

Compressive Strength, f_{ou} (MPa)	Static Modulus ($E_{c'}$		Dynamic Modulus (E_{cq})	
	Mean Value	Typical Range	Mean Value	Typical Range
20	25	21-29	35	31-39
25	26	22-30	36	32-40
30	28	23-33	38	33-43
40	31	26-36	40	35-45
50	34	28-40	42	36-48
60	36	30-42	44	38-50

Other relationships that have been found between E and concrete strength include the empirical relationship developed by the American Concrete Institute (ACI) Committee 318, (Building Code Requirements for Reinforced Concrete, Detroit, American Concrete Institute, 1977, ACI Publication 318-77).

$$E_{ct}=0.043(\rho^3(f'_c)t$$

Where

E_{ct} is the secant modulus of elasticity at age t (days),

ρ is the density of the concrete in kg/m^3

and f'_c is the cylinder strength (MPa) of the concrete at age t, determined from tests on 152*305mm cylinders.

B.3.6 **Reference**: AASHTO Guide for Design of Pavement Structures 1986, American Association of State Highway and Transportation Officials, Washington, D.C., USA.

Section 2.3.3. To estimate the elastic modulus of concrete the American Concrete Institute (ACI) suggest (for normal weight portland cement concrete):

$$E_c = 57000(f'_c)^{0.5}$$

Where E_c=PCC modulus (in psi)
and f'_c = PCC compressive strength (in psi) as determined using AASHTO T22, T140, or ASTM C39.

B.3.7 **Reference:** Parrott, L.J., Simplified Methods of Predicting the Deformation of Structural Concrete, Development Report 3, Cement and Concrete Association, Wexham Springs, U.K., 1979.

The relationship between elastic modulus and 28 day cube strength is quoted as

$$E_{28} = C_o + 0.2\, f_{28}$$

where C_o=f(Eaggregate) and varies between approximately 5 and 30.

As increases in elastic moduli with age for a given concrete depend mainly upon continued hydration of the cement and the associated reduction in porosity of the cement paste, they are related to increases in cube strength. A relationship between these parameters is given as

$$E_{time} = E_{28}(0.4 + 0.6(f_t/f_{28}))$$

Where f_t is the cube strength at age t days.

Then combining the above relationships:

$$E_t = (C_o+0.2f_{28})(0.4+0.6(f_t/f_{28})$$

(where f_t/f_{28} varies with cement type and c/w ratio)

If f_t/f_{28} is not known the values in Table B.3.7 can be used and a sensitivity analysis carried out.

Table B.3.7 f_t/f_{28} vs Age of concrete

Age,t (days)	3	7	14	28	90	180	365	730
f_t/f_{28}	0.5	0.7	0.85	1.0	1.17	1.22	1.28	1.33

B.3.8 **Reference:** Illi-back: a Closed-Form Backcalculation Procedure For Rigid Pavements. Department of Civil Engineering, University of Illinois, USA.

This computer program uses a rigorous, theoretically sound and efficient baclcalculation procedure applicable to two-layer rigid pavement systems. A unique feature of the program is that in addition to giving backcalculated parameters it also evaluates the degree to which the system behaves as predicted by theory.

The backcalculation procedure considers a two-layer system consisting of a rigid pavement slab resting on an elastic solid (ES) or a dense liquid (DL) foundation. Solutions are obtained from the analysis of geophone deflections at 0, 305, 610, and 915mm using theory proposed by Westergaard[B.3.10a], Hogg[B.3.10b] and Ioannides[B.3.10c], plus the concept for determining the Area of the deflection basin, first proposed by Hoffman and Thompson[B.3.10d].

B.4 CEMENT STABILISED MATERIALS

Cemented materials behave elastically up to approximately 35% of the ultimate material strength at which time microcracking begins to occur. Thereafter, material acts non-linearly and non-elastic. The following ways of estimating elastic moduli of cemented materials refer to moduli obtained over the initial portion of the stress-strain curve corresponding approximately to the initial 25% strain.

B.4.1 **Reference**: Jordaan, G.J., Analysis and Development of Some Pavement Rehabilitation Design Methods, PhD Thesis, University of Pretoria, Pretoria, 1987.

Ranges of elastic moduli are quoted that were derived from Heavy Vehicle Simulator test results on a pavement with a strongly cemented base. Suggested ranges of moduli for different classes of cemented materials are given in Table 6.4.1. THe pavement consisted of a 35mm wearing course, a 230mm cement-stabilised (C2) basecourse and a 200mm granular (G5) subbase on in-situ sand.

Table B.4.1 Recommended Values of Elastic Modulus

Material Class	Recommended Modulus Range (MPa)
C1	800-1000
C2	500-800
C3	500-800
C4	400-600

B.4.2 **Reference:** Cementitious Stabilisers in Road Construction, Technical Recommendations for Highways, Draft TRH13, Department of Transport, Pretoria, 1986.

The recommendations shown in Tables B.4.2a and B.4.2b are obtained from Appendix A

Table B.4.2a Estimation of Elastic Modulus from Flexural Strength.

Material Type	Equation
Cement-treated crushed stone	$E=8*\sigma_b + 3500$
Cement-treated natural gravel	$E=10*\sigma_b + 1000$
Lime-treated natural gravel	$E=17*\sigma_b - 900$

Table B.4.2b Estimation of Elastic Modulus from Unconfined Compressive Strength.

Material Type	Equation
Cement-treated crushed stone	$E=4.16*(\sigma_c)^{0.88} + 3484$
Cement-treated natural gravel	$E=5.13*(\sigma_c)^{0.88} + 1098$
Lime-treated natural gravel	$E=8.56*(\sigma_c)^{0.88} - 927$

Where E = Elastic modulus (MPa)

σ_b = Flexural strength (kPa)

and σ_c = Unconfined compressive strength (kPa)

Correlations between UCS, CBR and DCP penetration can be used to give relationships between unconfined compression strength and elastic modulus. Figure B.4 shows two relationships between UCS and elastic modulus. It should be noted that the 'UCS(DCP)-Elastic modulus' curve was generated from in-situ data and thus should be more applicable to working pavements than the laboratory-derived relationship.

B.5 TECHNIQUES APPLICABLE TO VARIOUS MATERIAL TYPES

B.5.1 **Reference**: Greenstein, J. and Berger, L., Using NDT Aided by an Expert System to Evaluate Airport and Highway Systems, Non-Destructive Testing of Pavements and Backcalculation of Moduli, ASTM STP 1026, A.J. Bush III and G.Y. Baladi, Eds., American Society for Testing and Materials, Philadelphia, 1989. 1992.

Abstract

The paper presents the methodology currently used by the authors for the evaluation of flexible and rigid airport and highway pavements. Using this methodology, one can determine economically and accurately the elastic modulus of (a) the subgrade; (b) the subbase and base materials, and (c) the asphalt layer of a flexible pavement. To improve the accuracy and to reduce the sensitivity of the calculated moduli of old pavements with non-uniform characteristics, the subgrade modulus is calculated, and then the combined modulus of the pavement structure is completed. In the third step the moduli of the asphalt and granular materials are determined. The alternative way of calculating the three moduli exactly and simultaneously is sensitive to minor changes in pavement performance of either deflection basin or layer thickness. The Hogg model of a thin plate on an elastic foundation, the Burmister two-layer model, the Odemark-Ullidtz method of 'equivalent thicknesses', and the theory of the strength of materials are all used to calculate the elastic moduli of the subgrade and pavement layers. Comparison of the elastic moduli determined by the simplified methods and such 'exact' computer programs as CRANLEY showed very good agreement. The Hertz-Westergaard models are used in the evaluation of rigid pavements. An expert system in airport in highway evaluation products is used to minimise the testing errors and to better calibrate the nondestructive testing (NDT) results with standard test pits.

The NDT methodology presented in this paper is independent of equipment type and has been successfully used with the Pavement Profiler, the falling weight deflectometer, the 16-kip vibrator, and the Benkelman Beam. This NDT methodology has been used worldwide as: (a) a design tool of pavement strengthening and rehabilitation, and (b) a quality tool to analyse and predict the performance of adequate and marginal materials that do not meet standard specifications.

General Comments

The authors claim to have success in using various models for different layers in pavement structures. For subgrades the Hogg model has been extensively used and compared with values obtained from in-situ testing. The technique can be applied without the need of powerful computers and has the advantage that prior knowledge of the layers above the subgrade is not required for establishing it's stiffness. Note is made of calculated subgrade stiffnesses that showed good correlation with 'exact' solutions obtained from the CRANLEY program (that uses a three layer elastic model) and the Odemark-Ullidtz simplified multilayer linear elastic model. In all cases a rigid base under a finite subgrade thickness was assumed, approximating (more realistically in most cases) subgrade response with a stiffness that increases with depth.

For rigid pavements the Hertz-Westergaard method of computing vertical distortion and stresses on an elastic plate on a dense fluid subgrade is used to determine the modulus of subgrade reaction. Concrete stiffnesses are obtained using the Hertz-Westergaard theory, which from examples cited appears to be sufficiently accurate for design purposes.

An important observation is also made by the authors that engineers using a backcalculation procedure must exercise great care and carefully apppraise answers at each stage to ensure gross errors are not made. Use of data bases and an expert system approach to help define material properties and the design process.

B.5.2 **Reference**: Freeme, C.R. Evaluation of Pavement Behaviour for Major Rehabilitation of Roads, Technical Report RP/19/83, Division of Roads and Transport Technology, CSIR, Pretoria, 1983.

The following tables are taken directly from RP/19/83.

Table B.5.2a: Moduli of granular materials for rehabilitation

CODE	MATERIAL DESCRIPTION	ABBREVIATED SPECIFICATION	OVER CEMENTED LAYER SLAB STATE	OVER GRANULAR LAYER OR EQUIVALENT	WET STATE (GOOD SUPPORT)	WET STATE (POOR SUPPORT)
G1	High quality crushed stone	86 - 88 % ARD impermeable	450 (250 - 1 000)	300 (175 - 600)	250	240
G2	Crushed stone	100 to 102 % Mod AASHTO	400 (200 - 800)	250 (150 - 450)	230	220
G3	Crushed stone	98 - 100 % Mod AASHTO	350 (200 - 800)	250 (125 - 400)	220	200
G4	Gravel base quality	CBR $\nless$ 80 PI $\ngtr$ 6	300 (175 - 600)	225 (100 - 375)	200	180
G5	Gravel	CBR $\nless$ 45 PI $\nless$ 10 - 15	250 (150 - 450)	200 (75 - 350)	180	160
G6	Gravel low quality subbase	CBR $\nless$ 25	225 (100 - 400)	200 (50 - 300)	150	140

Poisson's ratio 0,35

Table B.5.2b: Moduli of cemented materials for rehabilitation

ORIGINAL CODE	UCS (MPa) PRE-CRACKED STATE	PARENT MATERIAL		PRE-CRACKED STATE GPa (range)	POST-CRACKED STATE (MPa)				
					LARGE BLOCKS	SMALL BLOCKS			
						DRY STATE		WET STATE	
							EQUIV. CODE		EQUIV. CODE
C1	6 - 12	Crushed stone	G2	14 (7 - 30)	3 00	600	EG1	500	EG1
		Crushed stone	G3	12 (6 - 30)	2 800	600	EG1	400	EG2
C2	3 - 6	Crushed stone	G2	10 (4 - 14)	2 500	500	EG1	300	EG2
			G3	8 (14 - 14)	2 400	450	EG2	250	EG3
		Gravel	G4	6 (3 - 12)	2 200	450	EG2	250	EG3
C3	1,5 - 3	Gravel	G4	5 (3 - 10)	2 000	400	EG3	180	EG4
			G5	4,5 (3 - 9)	2 000	350	EG4	160	EG5
			G6	4 (2 - 8)	2 000	300	EG4	140	EG6
			G7	3,5 (2 - 7)	1 500	250	EG5	120	EG7
			G8	3 (2 - 6)	1 200	200	EG5	90	EG8
C4	0,75 - 1,5		G4	4 (2 - 7)	2 000	350	EG3	180	EG4
			G5	3,5 (2 - 6)	2 000	300	EG4	160	EG5
			G6	3 (2 - 6)	2 000	250	EG4	140	EG6
			G7	2,5 (1 - 5)	1 000	200	EG5	120	EG7
			G8	2 (1 - 4)	1 000	170	EG5	90	EG8
			G9	1,5 (0,5 - 3)	500	150	EG6	70	EG9
			G10	1 (0,5 - 2)	500	125	EG6	45	EG10

Poisson's ration 0,35

Table B.5.2c: Approximate stiffness values for bituminous materials

MATERIAL GRADING	DEPTH FROM SURFACE	STIFFNESS (MPa) FOR TEMPERATURE AND STATE							
		GOOD STATE OR NEW MATERIAL		STIFF DRY MIXTURE		VERY CRACKED STATE		LENSES OF UNSTABILIZED OR STRIPPED MATERIAL	
		20 °C	40 °C	20 °C	40 °C	20 °C	40 °C	20 °C	40 °C
Gap-graded	0 - 50	4 000	1 500	5 000	1 800	1 000	500	1 500	1 000
	50 - 150	6 000	3 500	7 000	4 000	1 000	500	2 500	1 500
	150 - 250	7 000	5 500	8 000	6 000	1 000	500	3 000	2 000
						(equivalent granular state -EG3)			
Continuously graded	0 - 50	6 000	2 200	7 000	4 000	750	500	2 000	1 500
	50 - 150	8 000	5 500	9 000	6 000	1 000	750	3 000	2 000
	150 - 250	9 000	7 500	10 000	8 000	1 000	750	3 000	2 000
						(equivalent granular state -EG2)			

Table B.5.2d: Effective moduli for concrete

SLAB STATE (GPa) & (range)	GRANULAR STATE (MPa) & (range)
30 (10 - 50)	1 000 (500 - 1 500) Equivalent granular state EG1)

Table B.5.2e: Moduli of subgrade materials

			RESILIENT MODULUS		
CODE	SOAKED	MATERIAL	WET STATE	DRY STATE	EQUIV. CODE
G7	$\nless$ 15	Gravel-soil	120	240	EG5
G8	$\nless$ 10	Gravel-soil	90	180	EG6
G9	$\nless$ 7	Gravel-soil	70	140	EG7
G10	$\nless$ 3	Gravel-soil	45	90	EG8

Table B.5.2f: Approximate effective stiffness values for bituminous materials

MATERIAL GRADING	LAYER THICKNESS	STIFFNESS (MPa) FOR TEMPERATURE AND STATE							
		GOOD STATE OF NEW MATERIAL		STIFF DRY MIXTURE		VERY CRACKED STATE		LENSES OF UNSTABILIZED OR STRIPPED MATERIAL	
		20 °C	40 °C	20 °C	40 °C	20 °C	40 °C	20 °C	40 °C
Gap-graded	0 - 50	4 000	1 500	5 000	1 800	1 000	500	1 500	1 000
	50 - 150	6 000	3 500	7 000	4 000	1 000	500	2 500	1 500
	150 - 250	7 000	5 500	8 000	6 000	1 000	500	3 000	2 000
						(equivalent granular state)			
Continuously graded	0 - 50	6 000	2 200	7 000	4 000	750	500	2 000	1500
	50 - 150	8 000	5 500	9 000	6 000	1 000	750	3 000	2000
	150 - 250	9 000	7 500	10 000	8 000	1 000	750	3 000	2000
						(equivalent granular state)			

Table B.5.2g: Approximate stiffness values at representative vehicle speeds and surface temperatures

OPERATING SPEED (km/h)	DEPTH FROM SURFACE (mm)	STIFFNESS (MPa) FOR STATED TEMPERATURE			
		Gap-graded asphalt		Continuously graded asphalt	
		Temp 20 °C	Temp 40 °C	Temp 20 °C	Temp 40 °C
80 - 100	0 - 50	4 000	1 500	6 000	2 200
	50 - 150	6 000	3 500	8 000	5 000
	150 - 250	7 000	5 500	9 000	7 500
40 - 60	0 - 50	4 000	1 500	5 000	2 000
	50 - 150	4 500	3 000	6 000	4 000
	150 - 250	5 000	4 000	6 500	5 500
Poisson's ratio = 0,44					

B.5.3 **Reference**: South African Roads Board, Nonlinear dynamic Analysis and design of Road Pavements, Interim report IR 90/030/1, South African Roads Board, Department of Transport, Pretoria, 1991.

Although not specifically addressing elastic material parameters the analyses carried out in the course of the project depended heavily on elastic values of moduli and Poisson's ratio, which were deduced from computer analysis of various pavement structures.

This reference highlights differences between static linear elastic pavement analysis and dynamic non-linear analysis, on an experimental concrete pavement loaded statically and dynamically. Computer analysis of asphalt pavements was also carried out and showed that 'static' moduli and Poisson's ratios can accurately predict pavement response if mass and damping characteristics are taken into account. In relation to the present project one of the most important findings of the earlier work is that the way elastic moduli are derived is not perhaps as significant as earlier thought. Rather, it seems, that different material behaviour appears to be more influenced by the mass inertial of the bodies and damping.

Differences between and implications of static and dynamic loads are indicated as are the effects of pavement geometry. Consequences of different pavement geometry are shown to be important as different aspects of pavement behaviour become dominant with changes in geometry. The spread of load also changes with loading speed and geometry and can significantly influence stresses in the various layers.

A significant finding of the project was that laboratory experiments using stationary impulse loads induced quite different reactions in pavement layers to those induced with moving loads.

The report implies that some back-calculation techniques currently in use are not necessarily accurate and should be reviewed, and also that pavement behaviour as typically understood by pavement engineers may actually be due to other reasons, as yet not fully understood. Dynamic effects should therefore be incorporated into the analysis of pavement structures and transfer functions (describing pavement life as functions of stress, strain and deflection, for instance), should be reevaluated.

B.5.4 **Reference**: Roesset, M. and Stokoe, K.H., Dynamic Behaviour of Pavement Structures, Reference material of 'a short course' held at the Department of Civil Engineering at the University of Stellenbosch, May 1990.

The use of seismic techniques for field determination of material properties is discussed at length with some specific references and case histories relating to pavements, and the Falling Weight Deflectometer (or in South African terminology, the Impulse Deflectometer-IDM).

It seems that great possibilities exist for the use of seismic techniques in material parameter evaluation and when properly applied have a number of commendable points. For instance: (a) seismic mehtods can be used for large or small zones of material, (b) techniques have a sound theoretical basis (elastic theory),
(c) the analytical method used closely represents the type of loading used, and
(d) measured wave velocities are independant of equipment used.

In addition to the above, in-situ seismic tests test material in an essentially undisturbed state thus automatically incorporating fabric effects, stress states and other factors such as anisotropy.

Seismic disturbances are induced by a source which propagates compression and shearing waves. Measurement of seismic waves typically enails monitoring wave particle motions (velocities or accelerations) using seismic receivers. The time required for waves to travel given distances is measured and then initial tangent moduli can be calculated (if an isotropic homogeneous medium is assumed) for:

(a) The Shear Modulus

$$G = \rho V_s^2$$

and

(b) the Constrained modulus

$$M = \rho V_p^2$$

Where V_s is the shear wave velocity,
V_p is the compression wave velocity,
ρ is the mass density of the soil ($\rho = \gamma/g$, where γ is total weight
and g is gravitational acceleration.

Little information regarding succcessful documented use of seismic techniques in South African pavement engineering exists, possibly through general ignorance of the techniques and the relative dificulty in obtaining the necesssary equipment for practical field use.

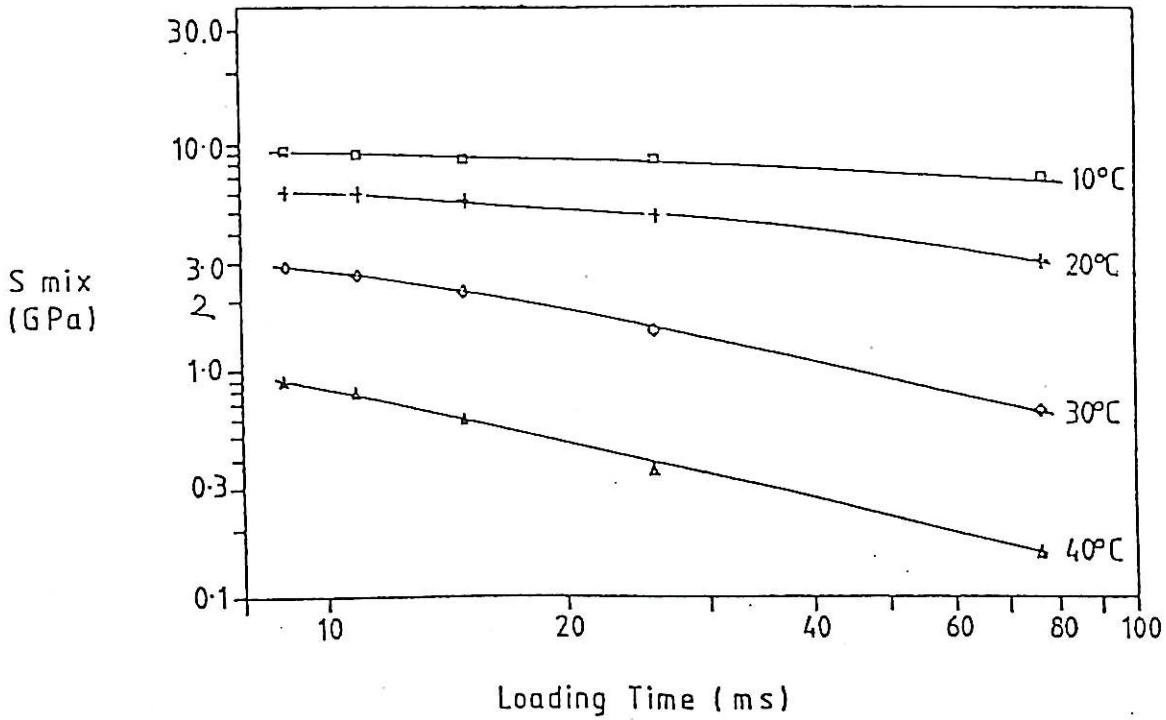


FIGURE B.1.1a EFFECT OF LOADING TIME ON MIX STIFFNESS (MIX B)
(After Maccarrone and Jameson, 1988[B.1.1])

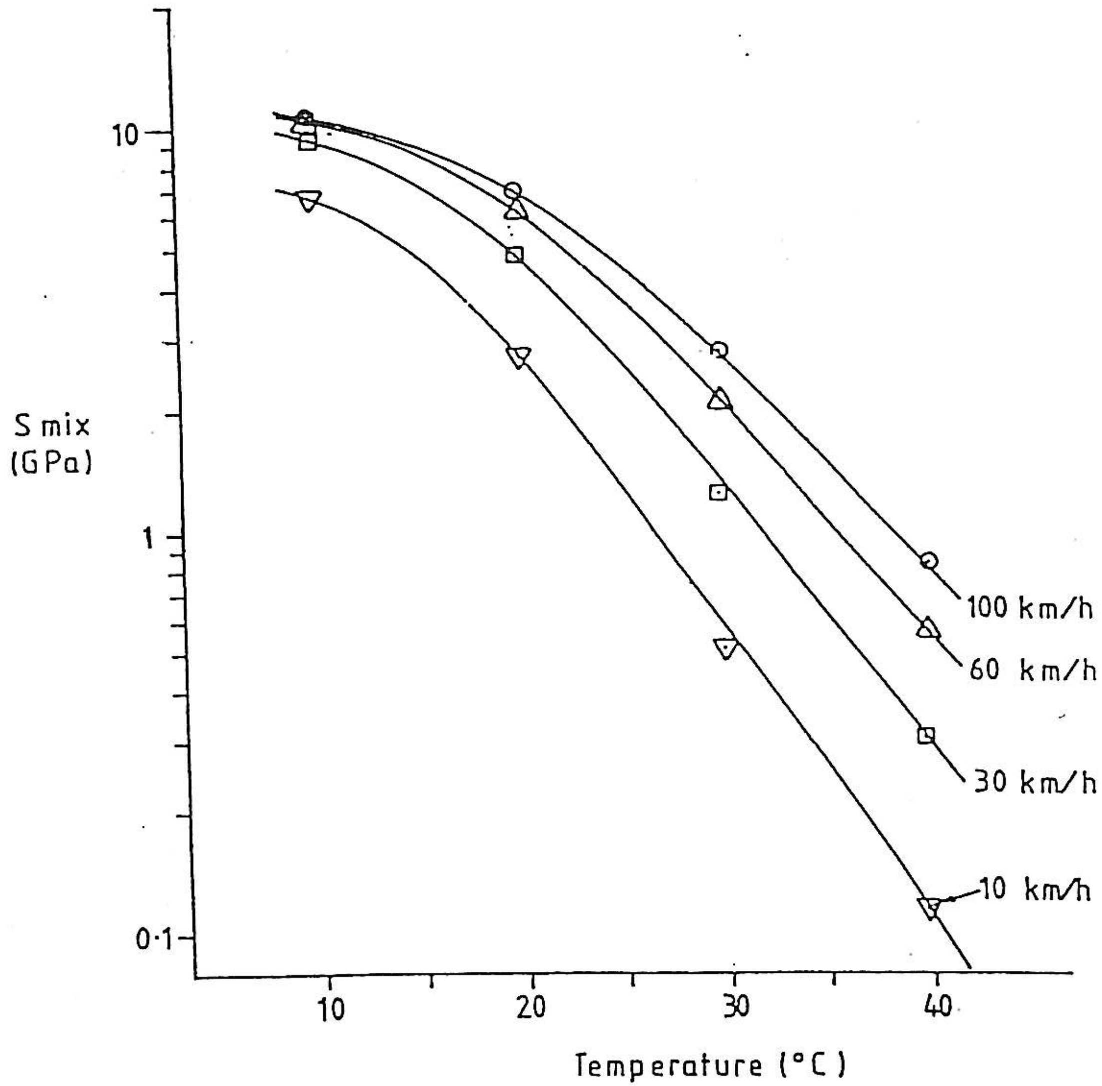


FIGURE B.1.1b AVERAGE STIFFNESS MODULI OF THE THREE MIXES AS A FUNCTION OF TEMPERATURE AND VEHICLE SPEED
(After Maccarrone and Jameson, 1988[B.1.1])

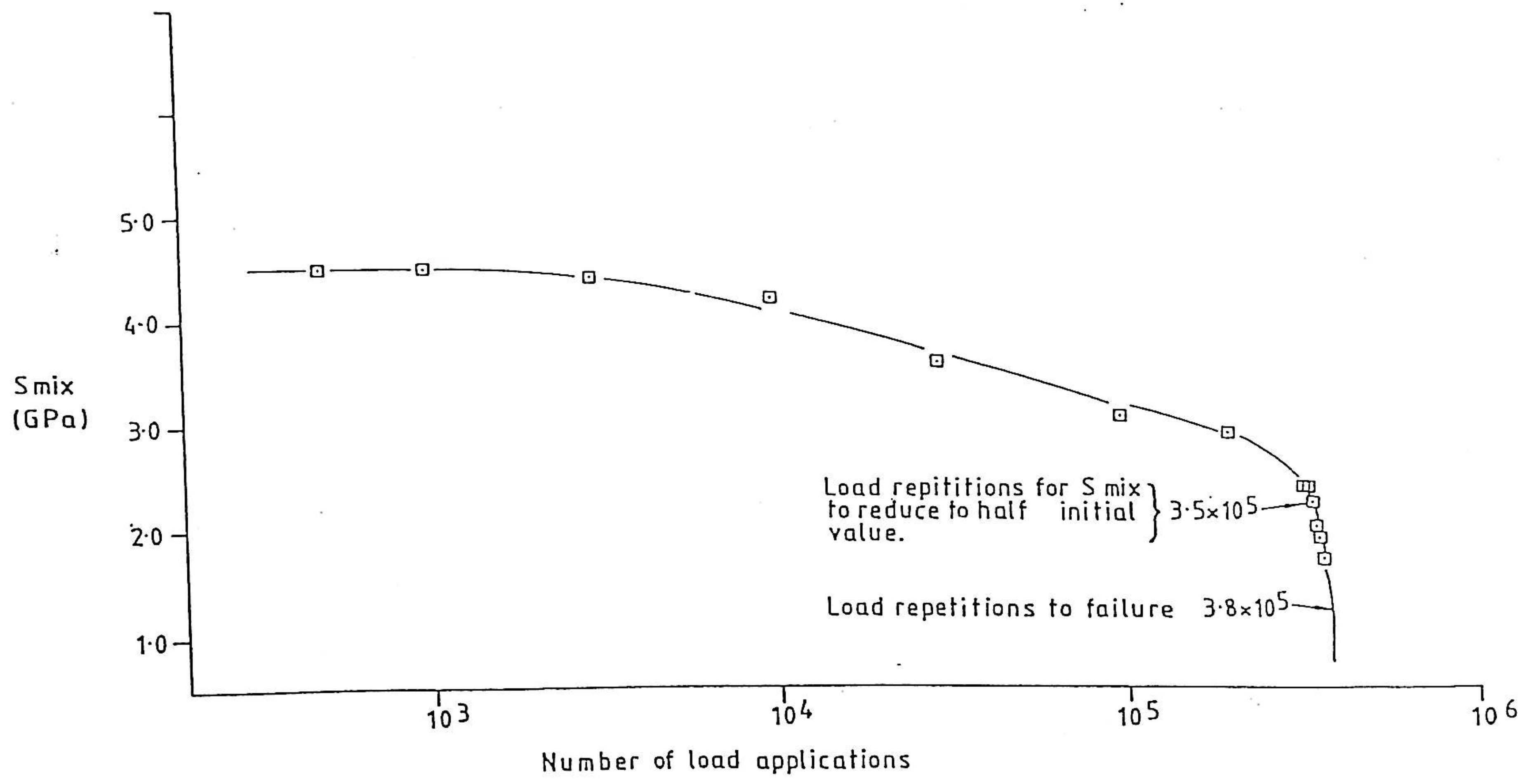


FIGURE B.1.1c TYPICAL RELATIONSHIP BETWEEN BEAM STIFFNESS (SMIX) AND NUMBER LOAD APPLICATIONS
(After Maccarrone and Jameson, 1988[B.1.1])

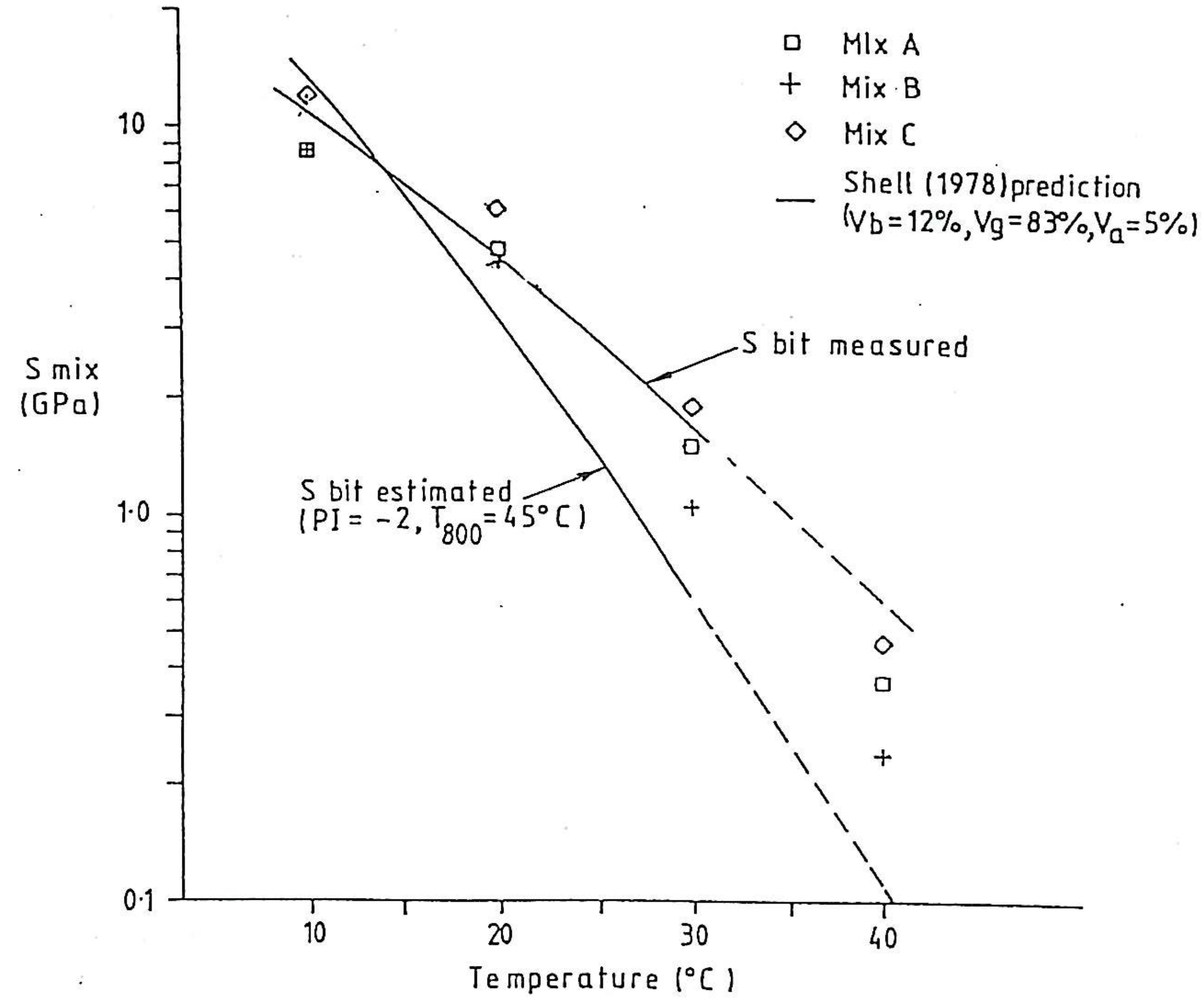


FIGURE B.1.1d EFFECT OF TEMPERATURE ON MIX STIFFNESS (SMIX)
(LOADING TIME 16 ms) (After Maccarrone and Jameson, 1988[B.1.1])

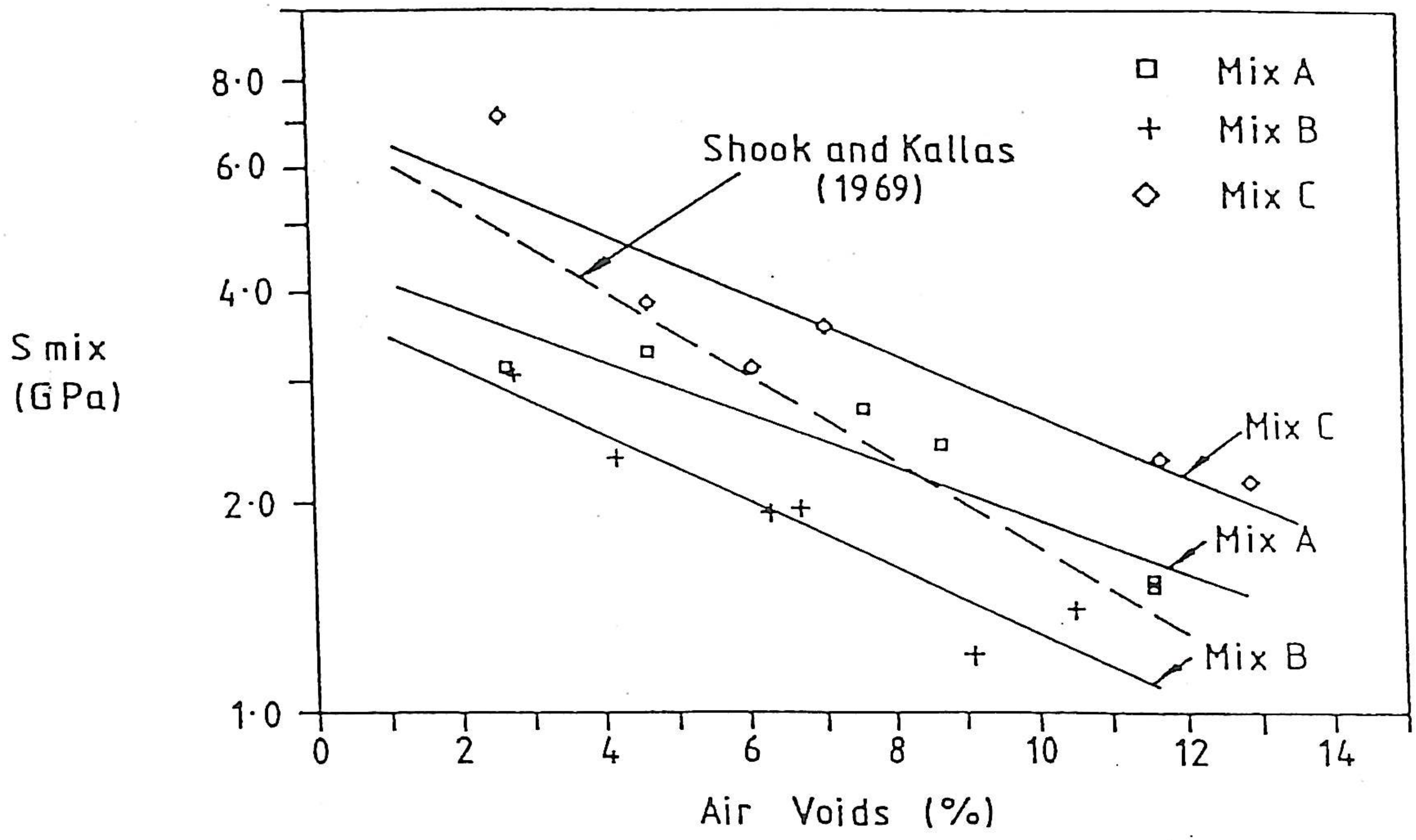


FIGURE B.1.1e EFFECT OF AIR VOIDS ON MIX STIFFNESS (SMIX))
(VARYING COMPACTIVE EFFORT)
(After Maccarrone and Jameson, 1988[B.1.1])

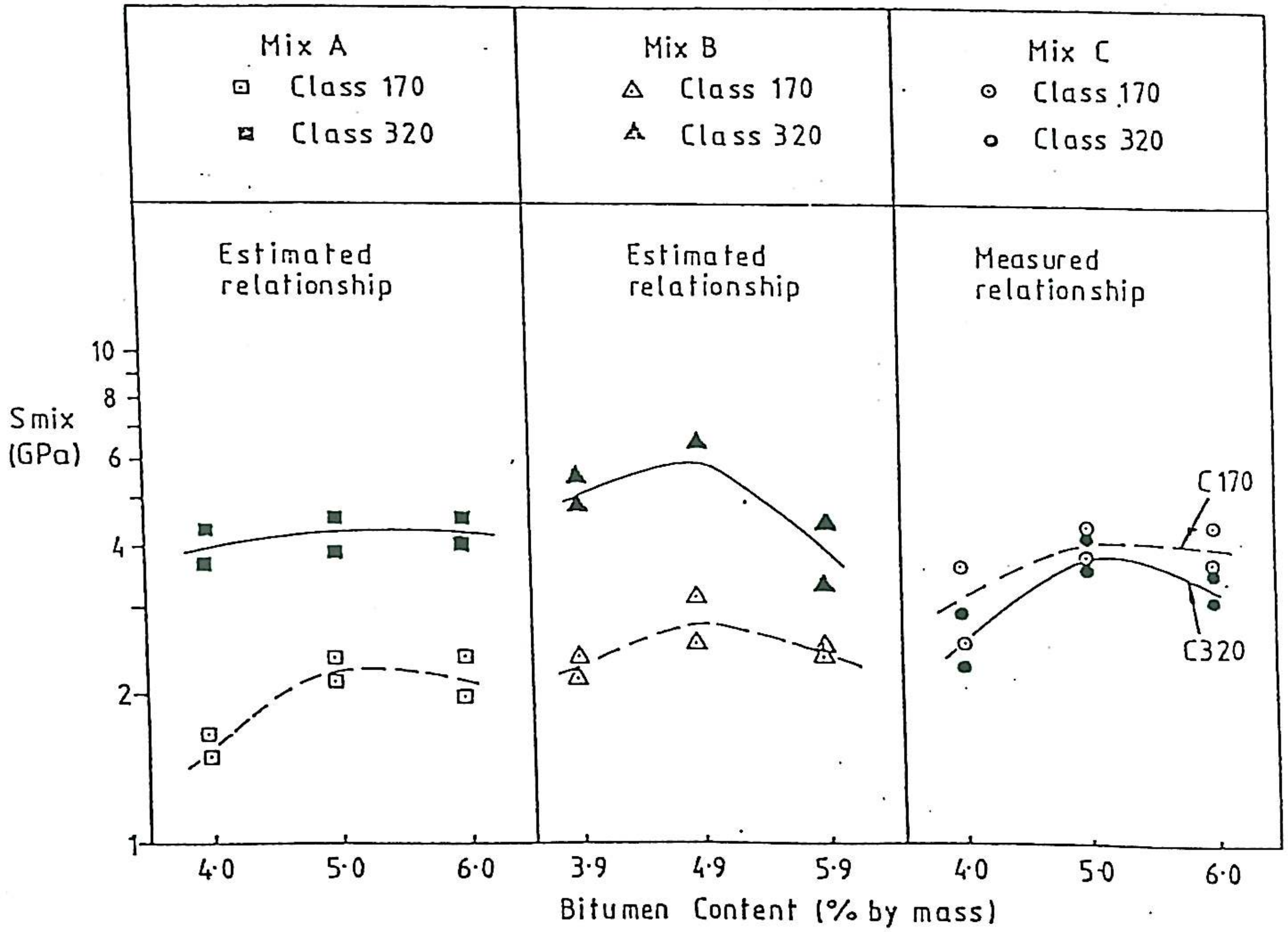


FIGURE B.1.1f EFFECT OF BITUMEN CONTENT AND VISCOSITY CLASS MIX STIFFNESS (SMIX) AT 20 °C (CONSTANT COMPACTIVE EFFORT)
(After Maccarrone and Jameson, 1988[B.1.1])

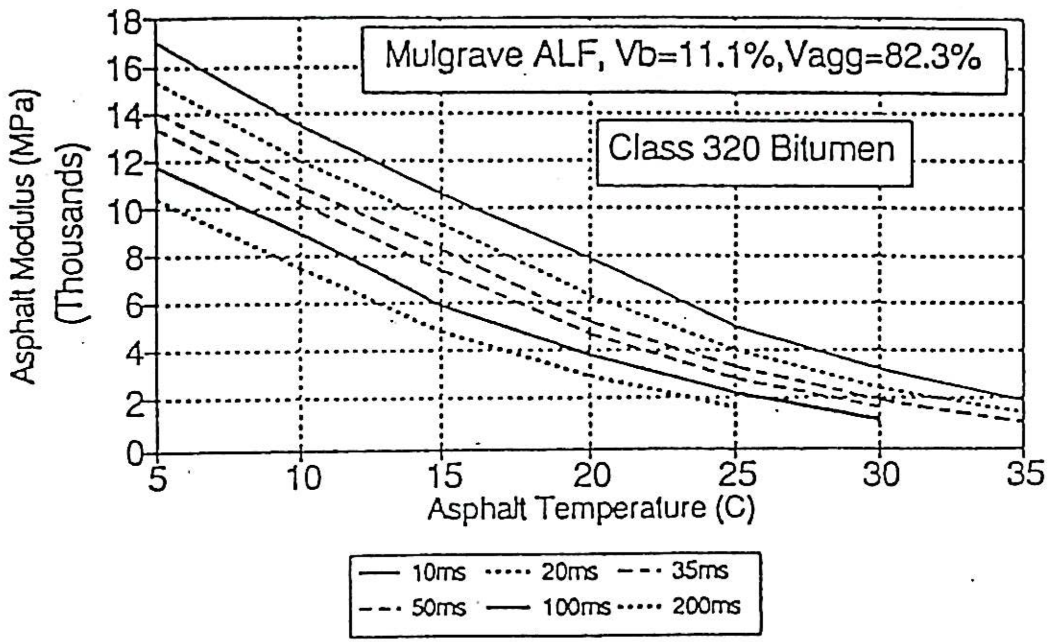


FIGURE B.1.2a SHELL ASPHALT MODULUS PREDICTIONS FOR THE TRIAL MIX
(After Jameson, Sharp and Vertessy, 1992[B.1.2])

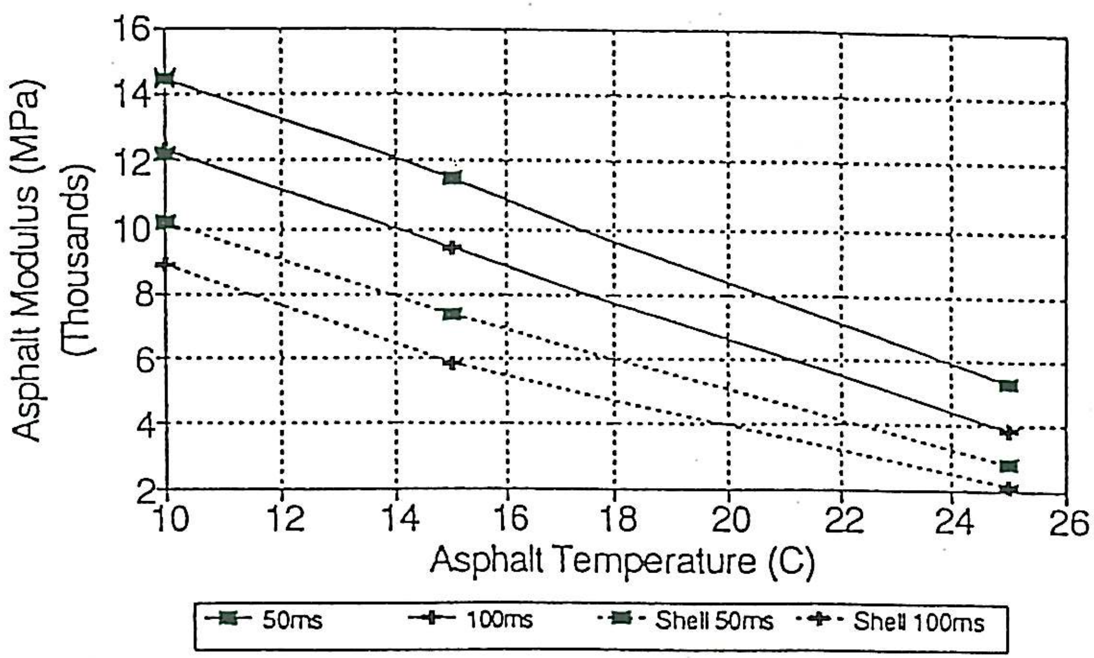


FIGURE B.1.2b LABORATORY MEASURED ASPHALT MODULUS ON LABORATORY COMPACTED CORES
(After Jameson, Sharp and Vertessy, 1992[B.1.2])

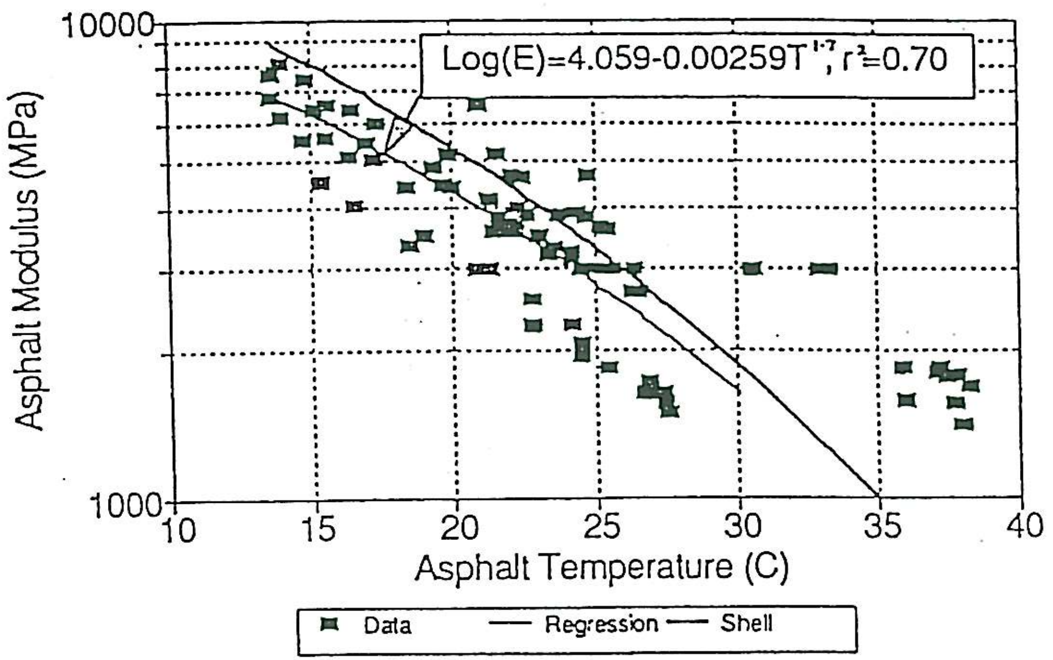


FIGURE B.1.2c BACKCALCULATED FWD MODULUS OF UNTRAFFICKED ASPHALT
(After Jameson, Sharp and Vertessy, 1992[B.1.2])

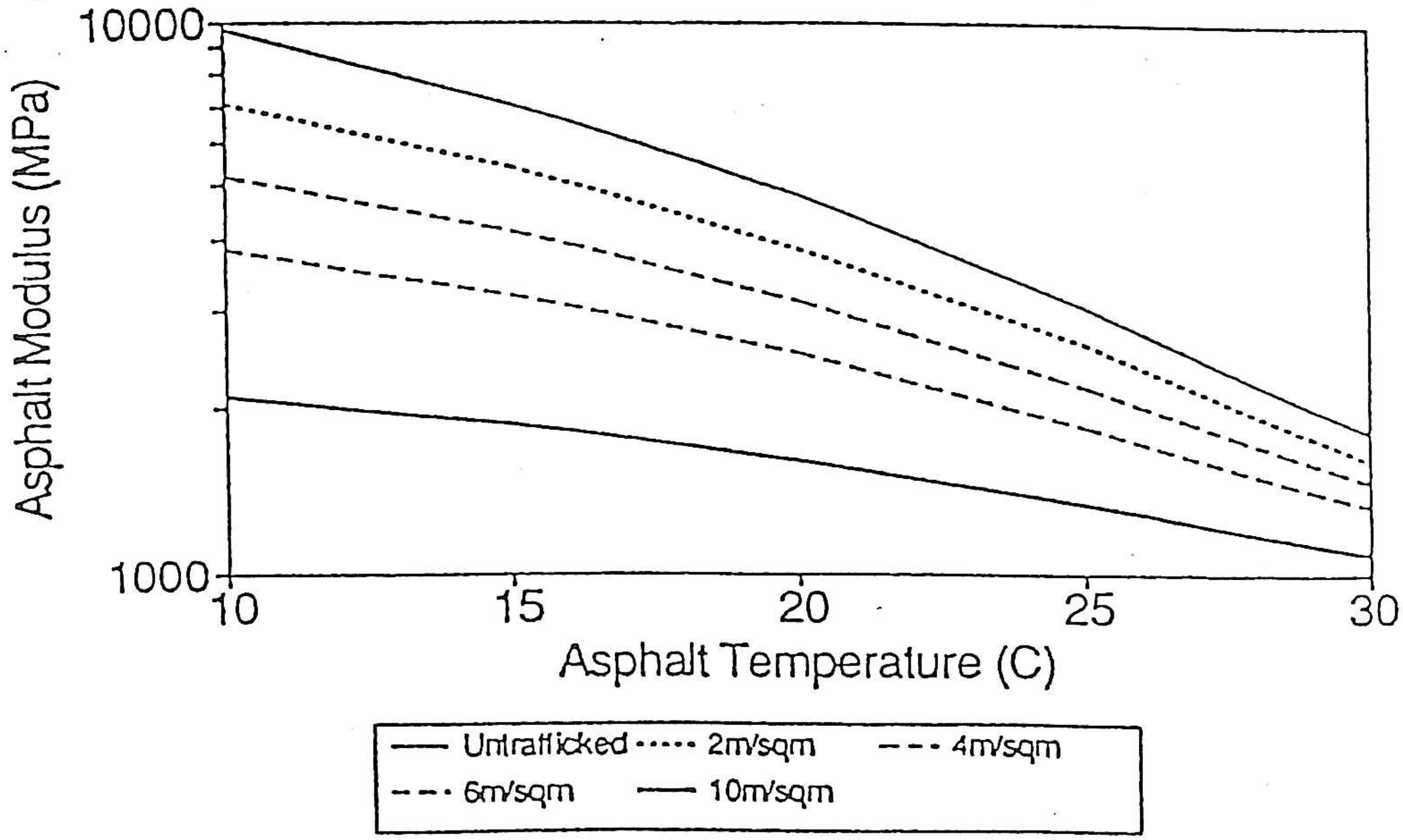


FIGURE B.1.2d BACKCALCULATED FWD MODULUS VARIATION WITH TEMPERATURE AND CRACKING SEVERITY
(After Jameson, Sharp and Vertessy, 1992 [B.1.2])

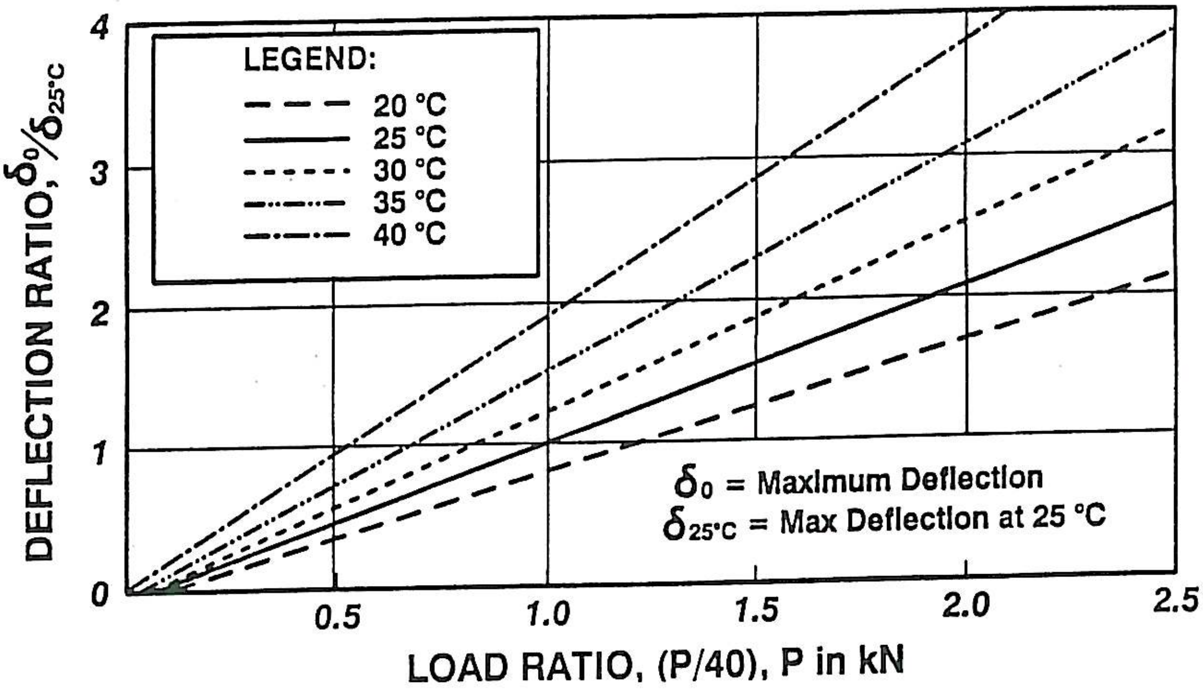


FIGURE B.1.4 TEMPERATURE AND LOAD CORRECTION FOR FWD MEASURED SURFACE DEFLECTION FOR AN ASPHALT BASED PAVEMENT (NORMALIZED FOR 40 kN AND 25 °C)
(After De Beer[B.1.4])

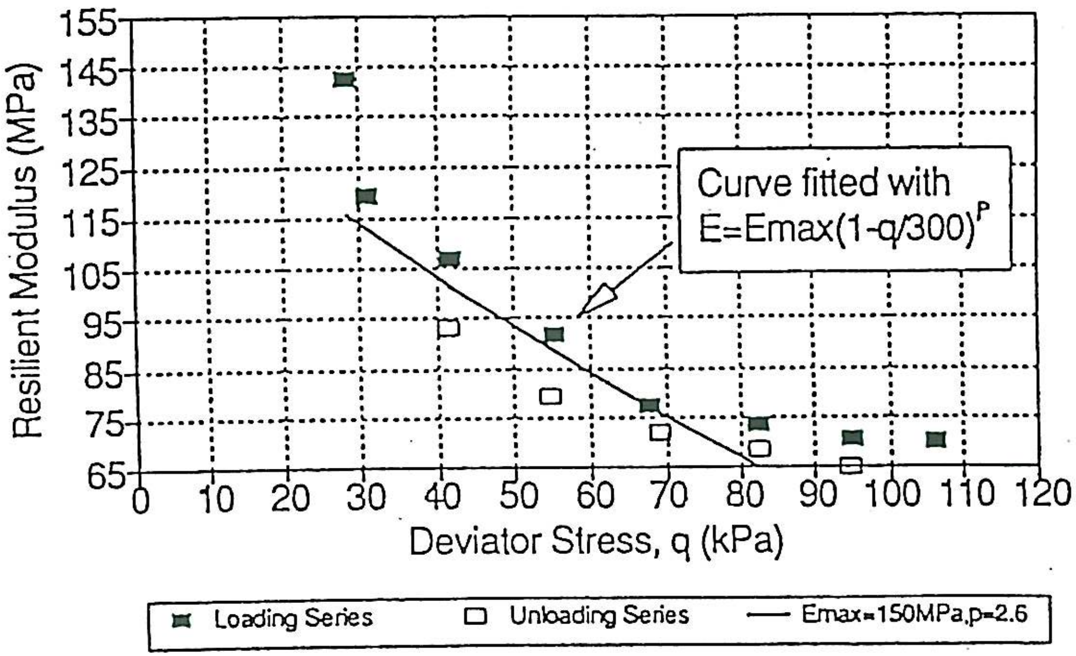


FIGURE B.2.1 TYPICAL IMPORTED SUBGRADE RESILENT MODULUS VARIATION WITH DEVIATOR STRESS

(After Jameson, Sharp and Vertessy[B1.2])

For Granular soils: (with CBR > 15 and PI 0 - 10)

Log Mr = 2,6885 - 0,02555(% W)

This relationship is plotted in figure B.2.4a

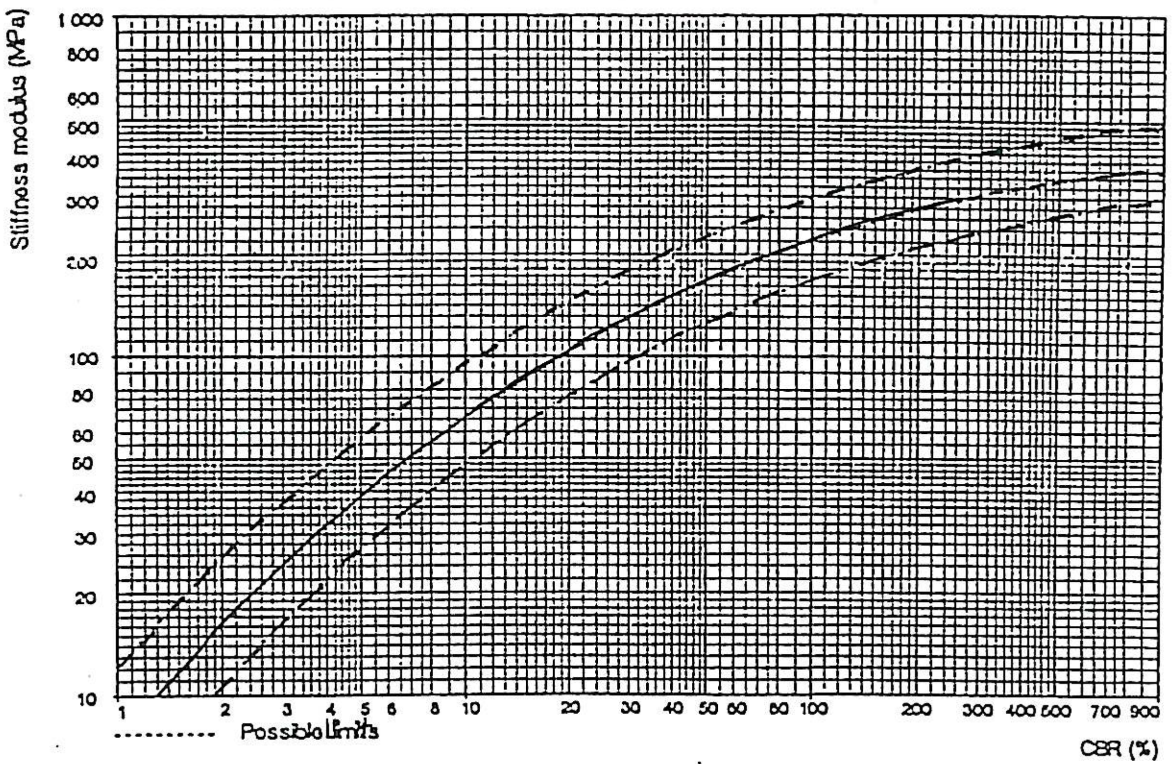


FIGURE B.2.4a DETERMINATION OF SUBGRADE MODULUS OF STABILISED MATERIALS (After Concrete Manual M10[B.2.4])

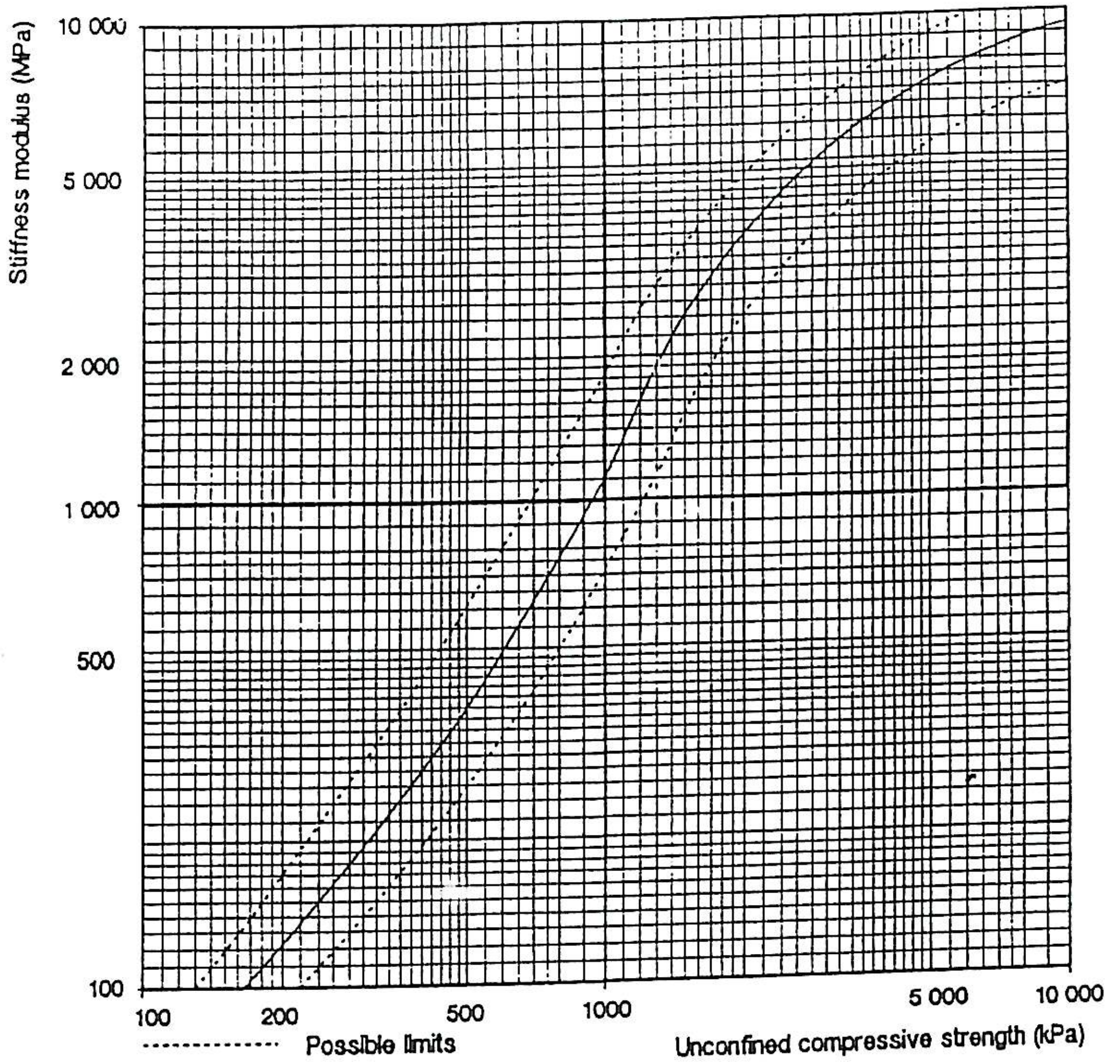


FIGURE B.2.4b DETERMINATION OF STIFFNESS MODULUS OF STABILISED MATERIALS (After Concrete Manual M10[B.2.4])

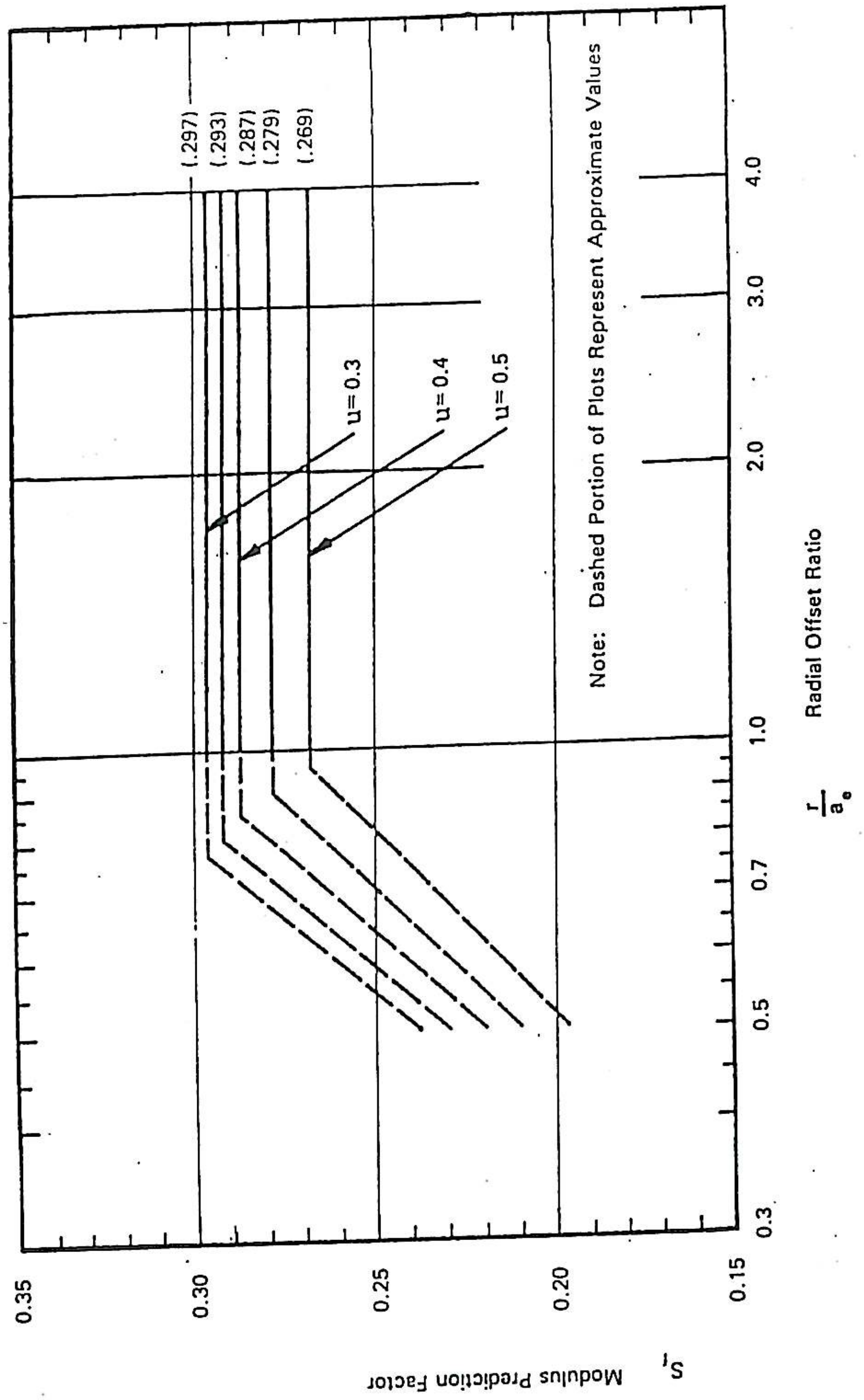


FIGURE B.2.5a SUBGRADE MODULUS PREDICTION FACTOR VERSUS RADIAL OFFSET RATIO (After AASHTO[5.2.5])

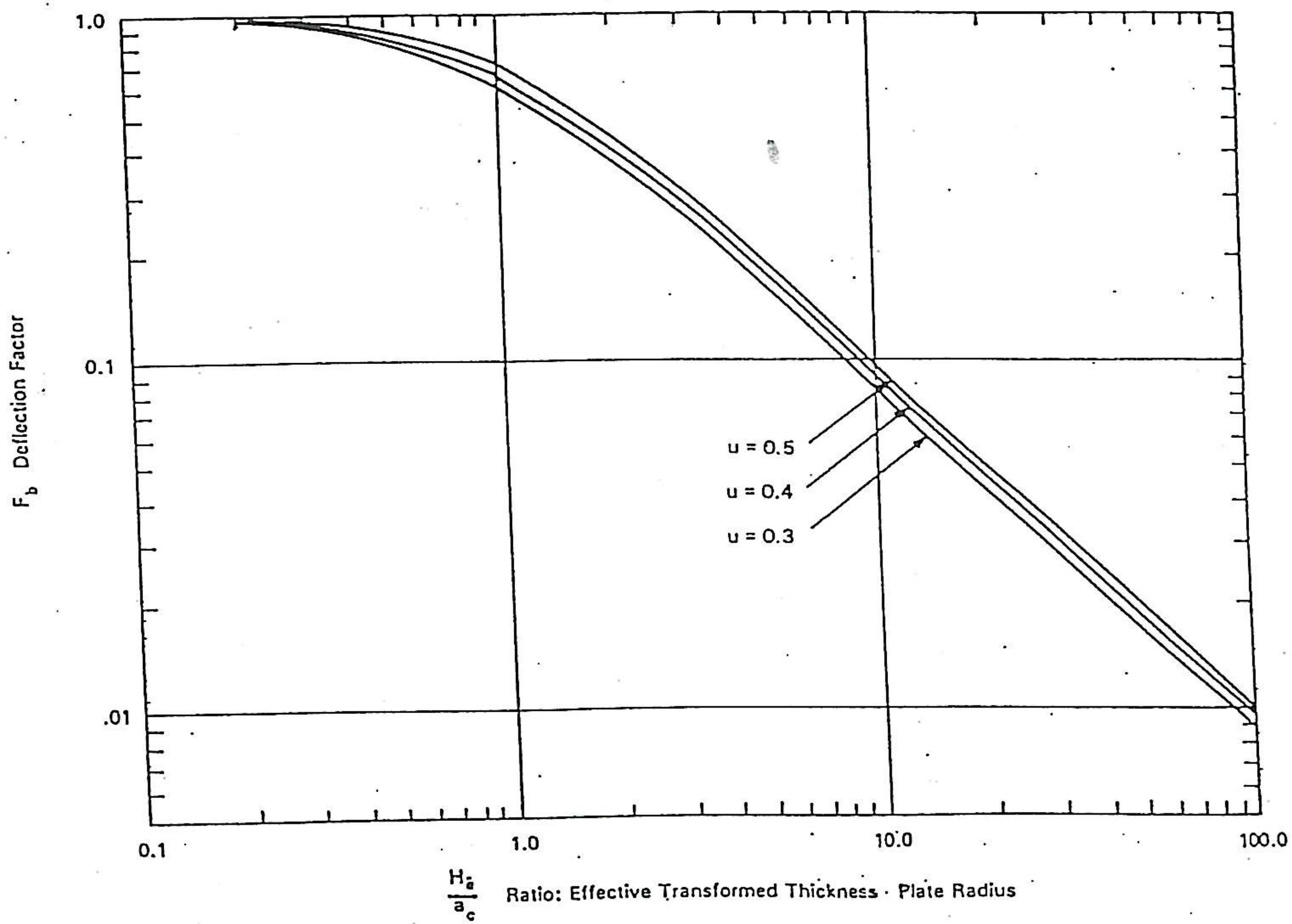


FIGURE B.2.5b DEFLECTION FACTOR VERSUS EFFECTIVE DEPTH RATIO (After AASHTO [5.2.5])

SENSITIVITY OF BACK-CALCULATED ELASTIC SUBGRADE MODULUS
TO ASSUMED DEPTH OF RIGID LAYER.
HVS TEST 309A4 BULTFONTEIN [40kN]

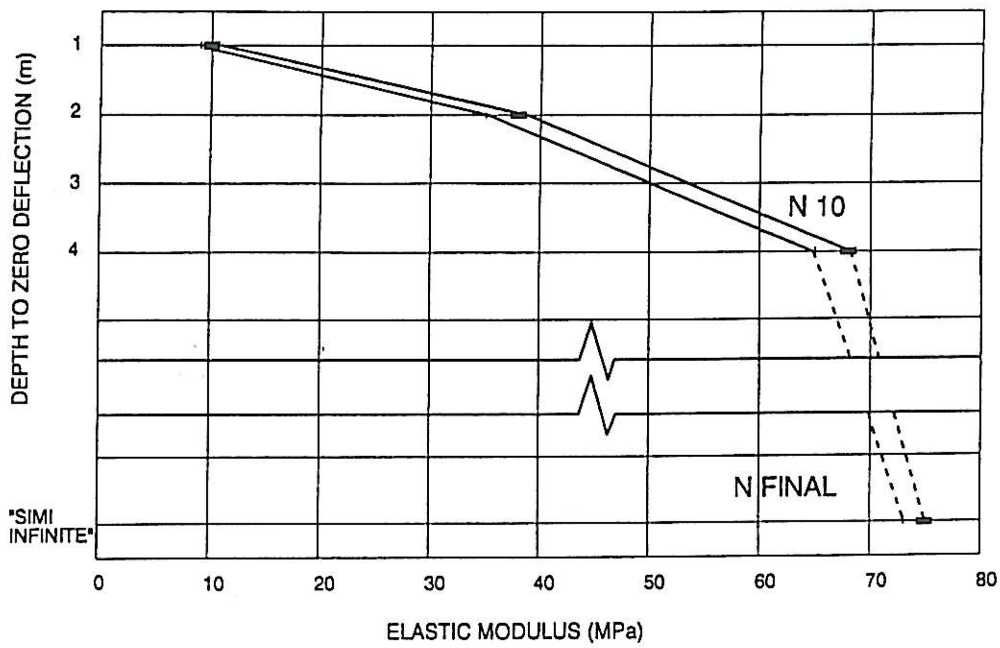
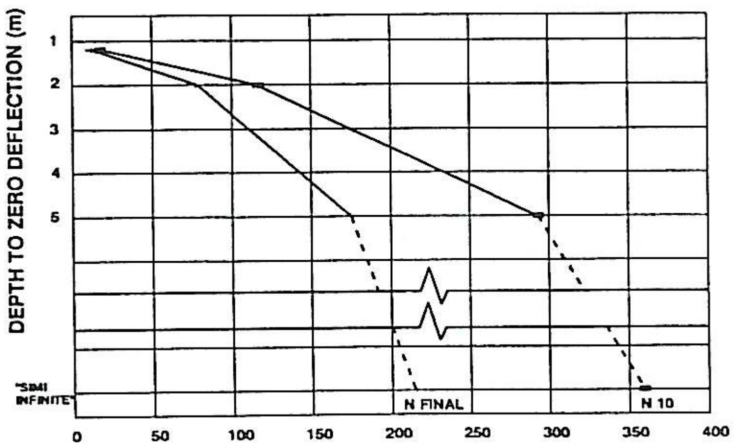


FIGURE B.2.6a SENSITIVITY OF BACK-CALCULATED ELASTIC SUBGRADE MODULUS TO ASSUMED DEPTH OF RIGID LAYER

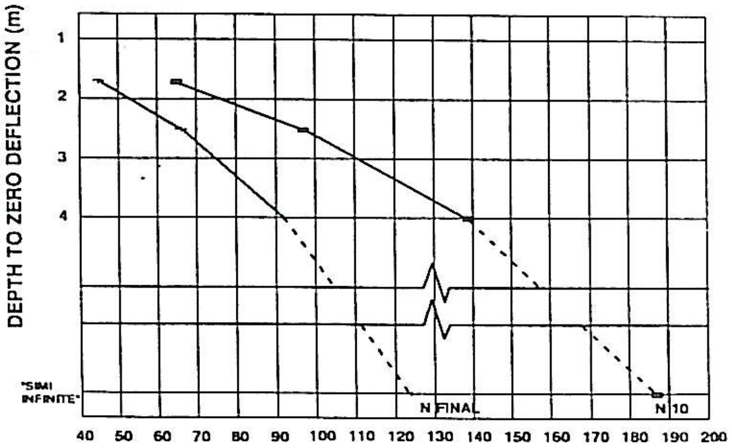
309A4

SENSITIVITY OF BACK-CALCULATED ELASTIC SUBGRADE MODULUS TO ASSUMED DEPTH OF RIGID LAYER.

HVS TEST 357A2 AND 363A2 WELKOM (40kN)



P 357A2



P 363A2

FIGURE B.2.6b SENSITIVITY OF BACK-CALCULATED ELASTIC SUBGRADE MODULUS TO ASSUMED DEPTH OF RIGID LAYER

SENSITIVITY OF BACK-CALCULATED ELASTIC SUBGRADE MODULUS TO ASSUMED DEPTH OF RIGID LAYER.

HVS TEST 341A4 3-SISTERS [40kN]

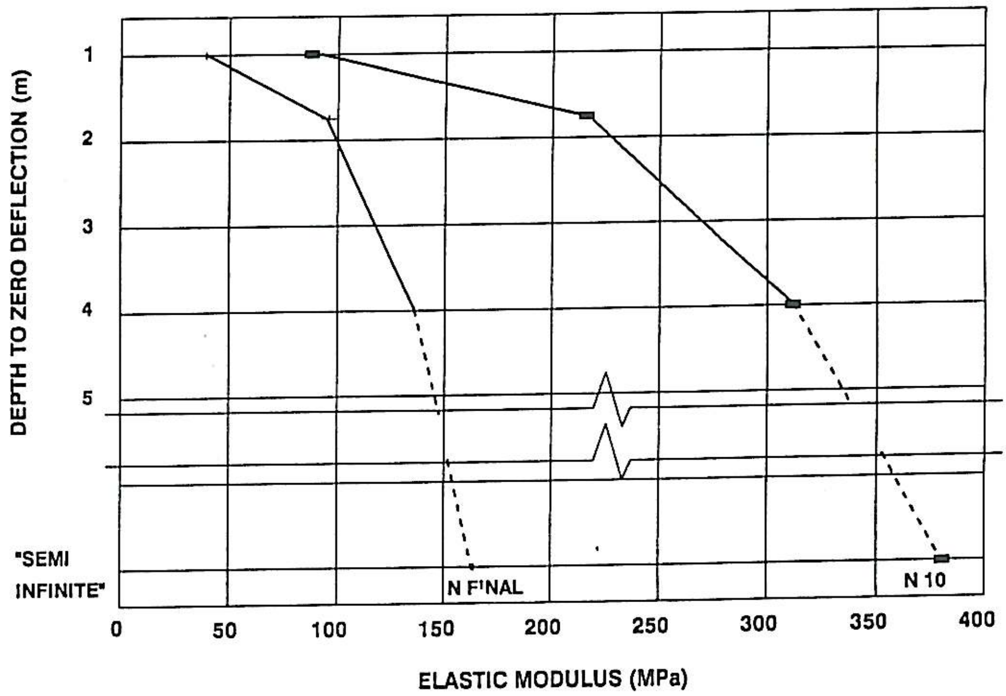
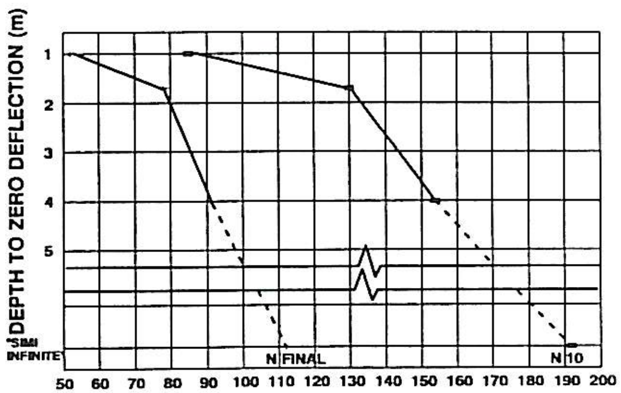


FIGURE B.2.6c SENSITIVITY OF BACK-CALCULATED ELASTIC SUBGRADE MODULUS TO ASSUMED DEPTH OF RIGID LAYER

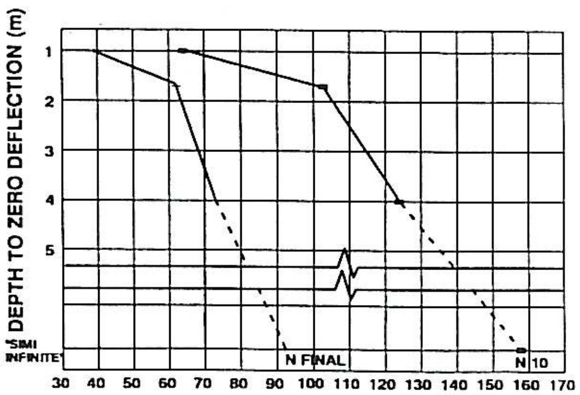
342A4

SENSITIVITY OF BACK-CALCULATED ELASTIC SUBGRADE MODULUS TO ASSUMED DEPTH OF RIGID LAYER.

HVS TEST P337 AND P338 ROOIWAL (40kN)



P337



P338

FIGURE B.2.6d SENSITIVITY OF BACK-CALCULATED ELASTIC SUBGRADE MODULUS TO ASSUMED DEPTH OF RIGID LAYER

ESTIMATING ELASTIC MODULI FROM UCS

(CEMENTED MATERIALS)

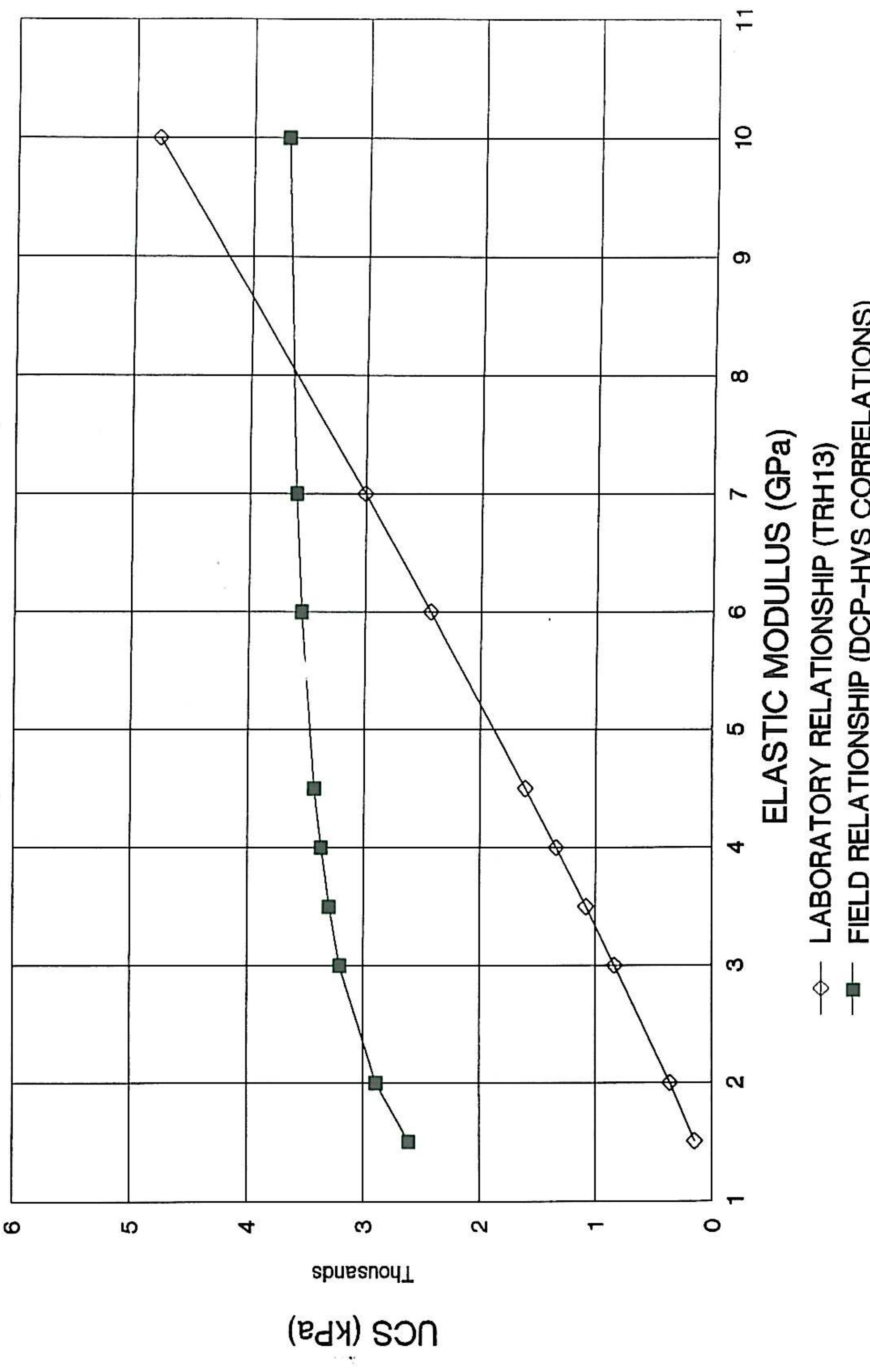


FIGURE B . 4

UCS-E

REFERENCES

B.1.1 Maccarrone S and Jameson G W. Fatigue and elastic characterisation of asphaltic concrete mixes. Proceedings of the 14th AARB Conference, Canberra, Australia, August 1988.

B.1.2 Jameson G W., Sharp K G., and Vertessy N J. Full-depth asphalt pavement fatigue under accelerated loading. Submitted to the 7th International Conference on Asphalt Pavements to be held at the University of Nottingham, England, 16th - 20th August 1992.

B.1.3 Dohmen L J M., and Molenaar A A A. Full scale pavement testing in the Netherlands. Submitted to the 7th International Conference on Asphalt Pavements to be held at the University of Nottingham, England, 16th - 20th August 1992.

B.1.4 De Beer M. Developments in the failure criteria of the South African mechanistic design procedure for asphalt pavements. Submitted to the 7th International Conference on Asphalt Pavements to be held at the University of Nottingham, England, 16th - 20th August 1992.

B.1.5 Technical Notes I/AT/51/90, I/AT/54/91. Asphalt Technology Programme, DRTT, CSIR, November 1991.

B.1.6 Vlok M. Optimising hot-mix design: The effect of South African aggregates on the engineering properties of an asphalt mix. Part of Project Report PR 88/019, South African Road Board, Research and Development Advisory Committee, Department of Transport, Pretoria, March 1991.

B.1.7 Vlok M. Optimising hot-mix design: The effect of briquette dimensions on Indirect Tensile Testing. Part of Project Report PR 88/019, South African Roads Board, Research and Development Advisory Committee, Department of Transport, Pretoria, March 1991.

B.2.1 Jameson G W., Sharp K G., and Vertessy N J. Full-depth asphalt pavement fatigue under accelerated loading. Submitted to the 7th International Conference on Asphalt Pavements to be held at the University of Nottingham, England, 16th - 20th August 1992.

B.2.2 Brown S F., and O'Reilly M P. The relationship between California Bearing Ratio and elastic stiffness for compacted clays. Ground Engineering, October 1990, London.

B.2.3 Drumm, E C., Boating-Poku Y., and Johnson Pierce T. Estimation of subgrade resilient modulus from standards tests. Journal of Geotechnical Engineering, Vol, 116, No 5, May 1990.

B.2.4 Manual M10: Concrete Pavement Design (Draft). Department of Chief Directorate: National Roads 1990, Pretoria.

B.2.5 AASHTO Guide for design of pavement structures 1986. American Association of State Highway and Transportation Officials, Washington, D.C., USA.

B.2.6 Rohde G T. The mechanistic analysis of pavement deflections on subgrades varying with depth. Ph.D Thesis, Office of Graduate studies of Department of Civil Engineering, Texas A & M University, December 1990.

B.3.1 Foxworthy P T., and Darter M I. Illi-slab and FWD deflection basins for characterisation of rigid pavements. Non-destructive testing of pavements and backcalculation of moduli, ASTM STP 1026, A J Bush III and G Y Baladi, Ed., American Society for Testing and Materials, Philadelphia, 1989.

B.3.2 Alexander D R., Kohn S D., and Grogan W P. Non-destructive testing techniques and evaluation procedures for airfield pavements. Non-destructive testing of pavements and backcalculation of moduli, ASTM STP 1026, A J Bush III and G Y Baladi. Eds., American Society for Testing and Materials, Philadelphia, 1989.

B.3.5 Alexander M G. Prediction of elastic modulus for design of concrete structures. The Civil Engineer in South Africa, June 1985.

B.3.6 BS1881: Part 5: 1970. Methods of testing hardened concrete for other than strength. British Standards Institution, London.

B.3.7 SABS 0100: Part 1 - 1980. Code of practice for the structural use of concrete, Part 1: Design. The Council of the South African Bureau of Standards, Pretoria.

B.3.8 Parrott L J. Simplified methods of predicting the deformation of structural concrete. Development Report 3, Cement and Concrete Association, Wexham, Springs, U.K. 1979.

B.3.9 Illi-back: A closed-form backcalculation procedure for rigid pavements. Department of Civil Engineering, University of Illinois, USA.

B.3.9a Westergaard, H M. Stresses in concrete pavements computed by theoretical analysis. Public Roads, Vol 7, No 2., April 1926.

B.3.9b Hogg A H A. Equilibrium of a thin plate, symmetrically loaded, resting on an elastic foundation of infinite depth. Philosophical Magazine, Series 7, Vol 25, March 1988.

B.3.9c Ioannides A M., Barenberg E J., and Lary J A. Interpretation of Falling Weight Deflectometer results using the principles of dimensional analysis. Proceedings of the 4th International Conference on Concrete Pavement Design and Rehabilitation, Purdue University, W Lafayette, Indiana, USA, 1989.

B.3.9.d Hoffman M S., and Thompson M R. Mechanistic interpretation of nondestructive pavement testing deflections. Civil Engineering Studies, Transportation Engineering Series No 32, Illinois.

B.4.1 Jordaan G J. Analysis and development of some pavement rehabilitation design methods. Ph.D Thesis, University of Pretoria, Pretoria, 1987.

B.4.2 Cementitious stabilisers in road construction, Technical Recommendations for Highways, Draft TRH13. Department of Transport, Pretoria, 1986.

B.5.1 Greenstein J., and Berger L., Using NDT aided by an expert system to evaluate airport and highway systems. Non-destructive testing of pavements and backcalculation of moduli, ASTM STP 1026, A J Bush III and G Y Baladi, Eds., American Society for Testing and Materials, Philadelphia, 1989.

B.5.2 Freeme C R. Evaluation of pavement behaviour for major rehabilitation of roads. Technical Report RP/19/83, Division of Roads and Transport Technology, CSIR, Pretoria, 1983.

B.5.3 South African Roads Board. Non-linear dynamic analysis and design of road pavements. Interim Report IR 90/030/1, South African Roads Board, Department of Transport, Pretoria, 1991.

B.5.4 Roesset M., and Stokoe K H. Dynamic behaviour of pavement structures. Reference material of "a short course" held at the Department of Civil Engineering at the University of Stellenbosch, May 1990.

APPENDIX C

QUESTIONNAIRE/GUIDELINES FOR DISCUSSIONS WITH CONSULTING ENGINEERS

QUESTIONNAIRE/GUIDELINES FOR DISCUSSION WITH CONSULTING ENGINEERS

Topics discussed with consulting engineers were presented as follows:

C.1 HOW DOES YOUR COMPANY DESIGN ROAD PAVEMENTS?

* SA MECHANISTIC DESIGN?
* OTHER MECHANISTIC APPROACH?
* CATALOGUE METHOD?
* OTHER TECHNIQUE?

(A) Flexible Pavements
(B) Rigid Pavements

C.2 WHAT APPROACH DO YOU FOLLOW TO OBTAIN ELASTIC PARAMETERS (E and Poisson's Ratio)?

* LABORATORY TESTING - WHICH TESTS?
* HOW MANY SAMPLES?
* IN-SITU TESTING: RSD + BASIN FITTING BY HAND?
* RSD + BASIN FITTING BY PROPRIETARY PROGRAM?
* RSD + BASIN FITTING BY IN-HOUSE PROGRAM?
* RSD + MAX. DEFLECTION FITTING?
* IDM + PROPRIETARY ANALYSIS PROGRAM?
* IDM + IN-HOUSE ANALYSIS PROGRAM?
* IS THE DEPTH TO RIGID LAYER TAKEN INTO CONSIDERATION?
* DCP?
* PLATE-JACKING TESTS?
* DOCUMENTED GUIDELINES?
* COMBINATION OF METHODS - WHICH ONES FOR:

(C) New Roads
(D) Roads to be rehabilitated?

C.3 'CONFIDENCE FACTOR'

HOW CONFIDENT ARE YOU IN YOUR:

* (a) APPROACH?
* (B) USE OF TRANSFER FUNCTIONS?

(from deflections to elastic moduli and from stresses/strains/deflections to pavement life prediction)

FOLLOW-UP OF DESIGN & CONSTRUCTION?

C.4 PITFALLS AND PROBLEMS

IN YOU OPINION WHERE ARE THE BIGGEST PROBLEMS EXPERIENCED IN PAVEMENT ANALYSIS/DESIGN?

* NEW INSTRUMENTS/TEST METHODS REQUIRED?
* FOR WHICH MATERIALS IN PARTICULAR?
* NEW/MODIFIED METHODS OF ANALYSIS?
* NEW/MODIFIED TRANSFER FUNCTIONS?
* OTHER?

APPENDIX D

THE USE OF THE DRTT K-MOULD IN DETERMINING THE ELASTIC MODULI OF UNTREATED ROADBUILDING MATERIALS

BY C J SEMMELINK

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THE USE OF THE DRTT K-MOULD IN DETERMINING THE ELASTIC MODULI OF UNTREATED ROADBUILDING MATERIALS

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SUMMARY

The paper briefly describes the 152,4 mm K-mould, and the principle on which it works. The method in which the 12 materials were tested is summarized. This is followed by a discussion on the theoretical approach of data evaluation. The actual results obtained are then discussed briefly and this is followed by conclusions and recommendations.

OPSOMMING

Die referaat bespreek kortliks die 152,4 mm K-druksel, en die beginsels waarop dit werk. Die metode waarvolgens die 12 materiale getoets is, word opgesom. Dit word gevolg deur 'n bespreking van die teoretiese benadering tot data-evaluering. Die werklike resultate wat verkry is, word dan kortliks bespreek en dit word gevolg deur gevolgtrekkings en aanbevelings.

INTRODUCTION

The K-mould may be described as any of several mechanical devices that automatically increase the lateral restraint on a soil specimen as it is being vertically loaded. The result is a confined compression test but with a constant or controlled horizontal modulus rather than a constant or controlled horizontal stress. An advantage over the triaxial stress path method is that the horizontal stress need not be calculated in advance on the basis of elastic theory and an assumed K_o, but seeks its own value. Another advantage is that axially rigid but laterally flexing walls distribute strain uniformly through the specimen rather than to allow bulging in the middle, as typically occurs in the triaxial test[1].

A unique feature of a K-test system is that by following along either a K_o consolidation stress path or K_f shear failure stress path or something in between, a travelling Mohr circle is obtained that traces the entire envelope from one test on a single specimen[1].

The original concept of the K-mould was developed at the Iowa State University under the guidance of professor R L Handy. The current American version consists of a 100 mm (4 inch) internal diameter single-split mould (with a greater wall thickness at the back, in the zone of maximum bending moment) which takes normal Proctor samples. Since local practice uses 152,4 mm diameter samples, it was felt that the right approach would be to develop a K-mould of similar diameter so that the sample used to determine moisture-density curves can also be utilized to determine the parameters E, υ, c, ϕ, K_1 and K_2. Professor Handy was kind enough to send some design drawings for the basic manufacture of a 152,4 mm diameter K-mould. This is a larger version of the prototype 100 mm diameter K-mould.

The second prototype K-mould (see Figure 1) developed at the DRTT consists of an internal thick-walled cylinder (with an internal diameter of 152,4 mm) made up to eight equal case-hardened circular segments. Each segment is mounted on two horizontal shafts which fit into two radially mounted linear ball bearings to allow each segment to move freely in a radial direction. The linear bearings are mounted on an outer thick-walled cylinder which also support sixteen mountings housing the disc springs which apply radial forces to the internal segmented cylinder.

This paper discusses the results obtained with the second prototype K-mould, known as the DRTT K-mould on twelve different materials.

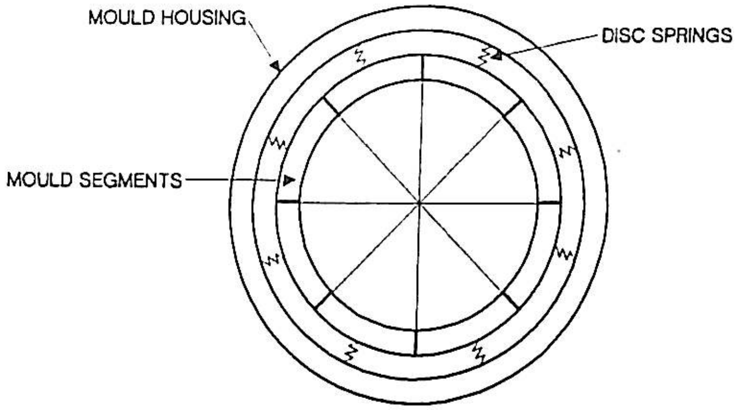

Figure 1 - Schematic presentation of the DRTT K-mould

For the sake of convenience, the materials were divided into three main groups, as follows:

- fine materials
- natural gravels
- crushed stone materials

Four materials were selected from each group for the initial analysis (see Table 1).

Table 1 - List of materials used in this investigation and code

MATERIAL CATEGORY	MATERIAL	CODE	MATERIAL	CODE
Fine Material	Montmorrilonte clay	KBAB	Red sandy clay	SPK
	Silty sand	SILK	Slightly plastic sand	KDW
Natural Gravel	Dolerite soil	DENS7	Red chert soil	LENC
	Decomposed dolerite	NPAB	Quartzite gravel	TP2
Crushed Stone	Crushed quartzite	FERRO	Crushed granite	ROSS
	Crushed dolerite	NPAA	Crushed tillite	NPAE

METHOD OF TESTING

An intensive study on the compactability of materials[2] clearly showed that the bearing capacity of materials is strongly influenced by the density level and the moisture content of the materials. It was therefore decided to evaluate each of the materials in this study at different levels of density and moisture contents. The density levels chosen for the fine materials and natural gravels were 90 %, 95 % and 100 % mod.AASHTO, expressed as a percentage of the "solid" density. The density levels for the crushed stone were approximately equivalent to 95 %, 100% and 105 % mod.AASHTO (see Table 2).

TABLE 2: EXPRESSION OF DIFFERENT LEVELS OF mod.AASHTO DENSITY IN TERMS OF "SOLID" DENSITY

MATERIAL SAMPLE	DENSITY LEVELS			
	90 % * mod.AASHTO (% AD)	95 % * mod.AASHTO (% AD)	100 % * mod.AASHTO (% AD)	MDD (vibratory) (% AD)
KBAB	53,62	56,60	59,58	59,11
SPK	62,19	65,40	69,10	71,01
SILK	63,38	66,90	70,42	73,72
KDW	68,78	72,60	76,42	78,12
DENS7	71,83	75,82	79,81	84,28
LENC	67,40	71,15	74,89	79,59
NPAB	68,45	72,25	76,05	77,44
TP2	69,48	73,34	77,20	82,14
FERRO	76,82	81,12	85,34	86,79
ROSS	78,38	82,77	87,07	86,64
NPAA	77,87	82,23	86,51	88,04
NPAE	80,99	85,52	89,97	88,33

* Respective density levels for crushed stone are approximately 95 %, 100 % and 105 % mod.AASHTO

Because the compaction study also showed that there is a critical moisture content (CMC) for each material at which the highest CBR values (unsoaked) are achieved for any level of density, it was decided to evaluate the materials at moisture contents of 0,75 CMC, CMC, 1,25 CMC, 1,50 CMC and 1,75 CMC. Some of the materials, such as the black clay samples, were also evaluated at moisture contents equal to or greater than 2,00 CMC to achieve a totally saturated sample (see Table 3).

Table 3 - Sample number for different combinations of density and moisture content

MOISTURE CONTENT % CMC *	DENSITY LEVEL		
	90 % mod.AASHTO	95 % mod.AASHTO	100 % mod.AASHTO
75	1	6	11
100	2	7	12
125	3	8	13
150	4	9	14
175	5	10	15
≥ 200	16	17	18

* CMC = Critical Moisture Content

The amount of dry material was calculated for a compacted sample height of 100 mm. To control the compacted sample height, use was made of an infra red sensor, which automatically switches off the vibratory compaction table as soon as the set height is reached (100 mm for fine materials and natural gravels, and 95 mm for crushed stone material). For most of the materials, it is virtually impossible to compact to the required density level when the moisture content of the material is approximately equal to the CMC of the material. To limit degradation of these materials during compaction, the compaction period was limited to four minutes maximum, if the required height had not been reached by then.

For the research reported the samples were tested in the K-mould immediately after compaction. In other cases samples were left to air-dry to lower moisture contents to specific values or to cure cement and lime-stabilized materials before testing.

THE K-MOULD TEST PROCEDURE

Immediately before testing the average sample height is determined by taking four measurements at approximately 90° intervals with a vernier height gauge. The sample is then transferred from the compaction mould to the K-mould by means of a special extruder press. This is normally only used in the case of dry, granular materials, where the sample tends to fall apart if removed without lateral support. In the case of more cohesive samples which don't collapse when the lateral support is removed, the sample is transferred manually. The side wall as well as the top and bottom end plates of the K-mould are lubricated with a silicon lubricant spray prior to transferring each sample. This is done to limit the effects of end and side friction.

The disc-spring mountings (see Figure 2) are usually unlocked so that the compacted sample can easily be slided into the K-mould, after which all the disc-spring mountings are locked again prior to transfer. For these series of tests the position of the disc-springs were such that the springs had no pretension loading on them when the mountings were locked.

The K-mould plus sample is then transferred to the Baldwin press where the K-mould is placed on the footing which contains the bottom load cell. The disc-spring mountings are unlocked briefly to ensure that the bottom load plate of the K-mould with the sample on top is making contact with the load transfer shaft of the bottom load cell (see Figure 2). Once contact has been established the disc-spring mountings are locked again.

The adjustable mounting arm for the vertical deformation meters is then brought in position and locked in this position (see Figure 2). The shaft of each of the vertical deformation meters is then moved up and down while checking on a multimeter whether it is registering properly. The steel trace of the horizontal deformation meter is also pulled slightly while checking with a multimeter whether it is registering properly (see Figure 2). All four deformation meters and the two load cells are then connected to an analogue casette recorder. The K-mould plus footing is then raised or the top load plate lowered by means of the press until the load transfer shaft of the upper load cell is just above the top load plate of the K-mould. The recorder is then switched on, after which the sample is rapidly loaded to a maximum load of approximately 200 kN or until the disc-springs have completely been compressed (which ever occurs the first), after which the recorder is stopped.

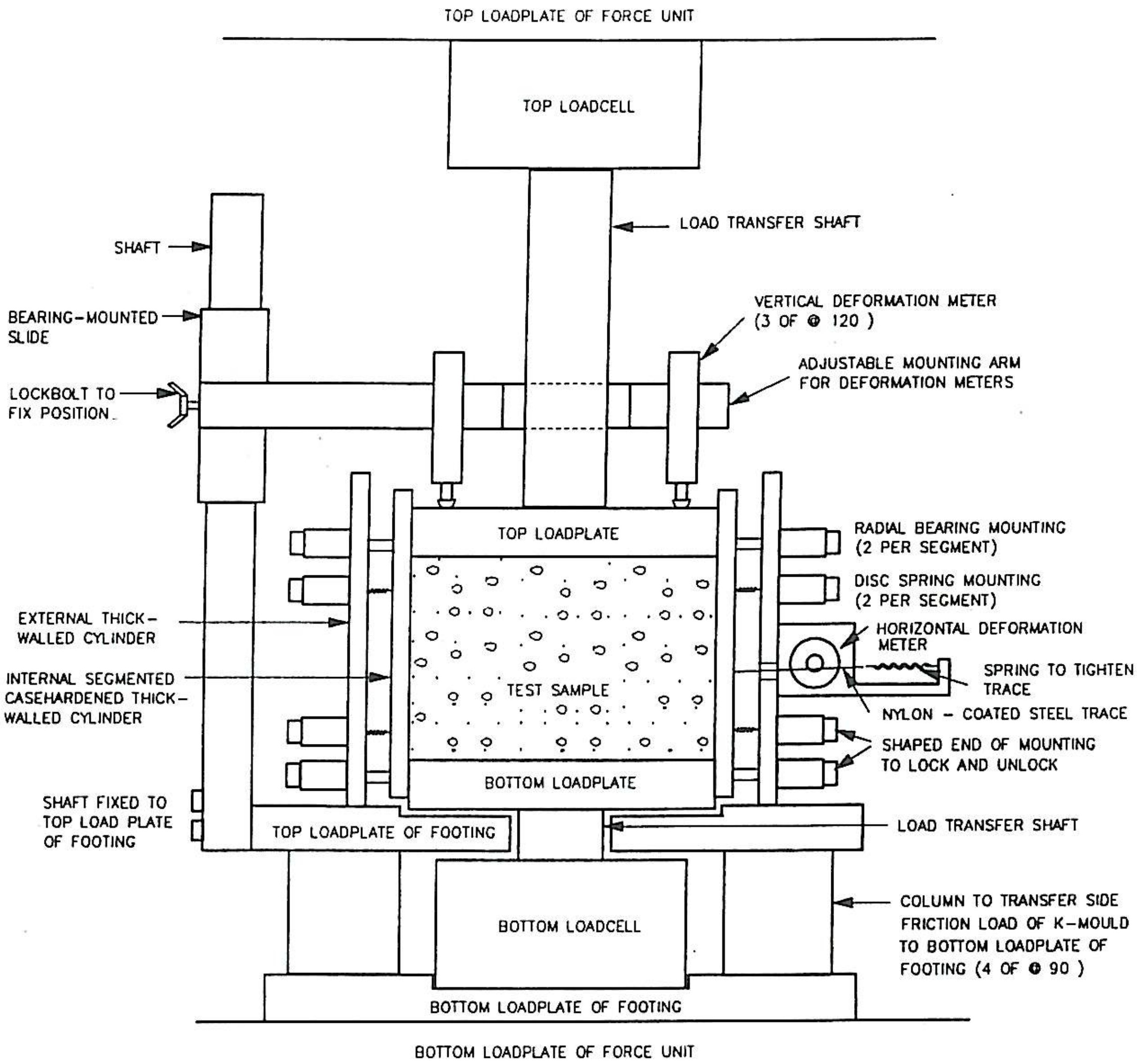

Figure 2 - **Schematic side view of K-mould set-up**

Although for this series of tests a Baldwin press was used for loading, a pulsating load press (i.e Instron) can also be used. Recently some work was done on some asphalt samples applying a pulsating load with a loading time of 0,1 seconds and a recovery time of 0,9 seconds. The applied load varied between about 3 kN and 10 kN. The results look very favourable. A special adaptor piece was also used which made it possible to test 100 mm diameter samples, which is the normal core size for asphaltic materials.

No serious problems or limitations have been experienced with the use of the K-mould up till now. In the case of materials with a low stability some of the material tended to squeeze through the slots between the segments when the lateral support of the mould became greater than the stiffness of the material itself (usually when the disc springs had completely been compressed).

The analogue data is then transferred to the computer by means of an analogue digital interface. Once this has been done, the data is processed to correct the mV signals to loads (kN) and deformations (mm). These values are then processed to determine the elastic properties of the material.

THEORETICAL APPROACH TO DATA EVALUATION

In the case of the triaxial compression test the modulus of resilience is calculated by using the effective vertical stress and dividing this by the vertical strain. In the case of the triaxial compression test the effective stress σ_1 of the applied load is equal to σ_d (i.e the deviator stress = applied load/area) plus σ_3 because σ_3 also works in on the top load plate of the sample. In the case of the K-mould σ_3 does not work in on the top load plate and therefore the effective vertical stress is equal to σ_1 (= applied load/area). Because of some side friction, even though the mould has been lubricated with a silicon lubricant, the bottom load is slightly smaller than the top load. This is also the case in pavement layers. Therefore, σ_1 is taken as the average of $\sigma_{1\,top}$ and $\sigma_{1bottom}$. More will be said about this later.

In general it was found that the plot of σ_1 (Sigma-1) against e_1 (Epsilon-1) was well described by a third degree function of e_1 (see Figure 3). The r^2-values for this relation for almost every sample evaluated was above 0,99 (see Example in Appendix A).

The smooth line in Figure 3 is the best fitting curve of the respective third degree function. The elastic modulus (E-1) for the K-mould and modulus of resilience (M_R) for the triaxial compression test as they are known, in road engineering is actually the slope of this curve ($=d\sigma_1/de_1$).

From Figure 3, it follows that

$$\sigma_1 = A.e_1 + B.e_1^2 + C.e_1^3 + D \qquad \text{..................1}$$

where

σ_1 = stress level (kPa) (either σ_1 for K-mould or $\sigma_1 = \sigma_d + \sigma_3$ for triaxial compression test)

e_1 = vertical strain

A..D = regression coefficients

and $E = d\sigma_1/de_1 = A + 2B.e_1 + 3C.e_1^2$ 2

The values of A, B and C for Equation 2 are the same as for Equation 1 and are determined through regression analyses of the data.

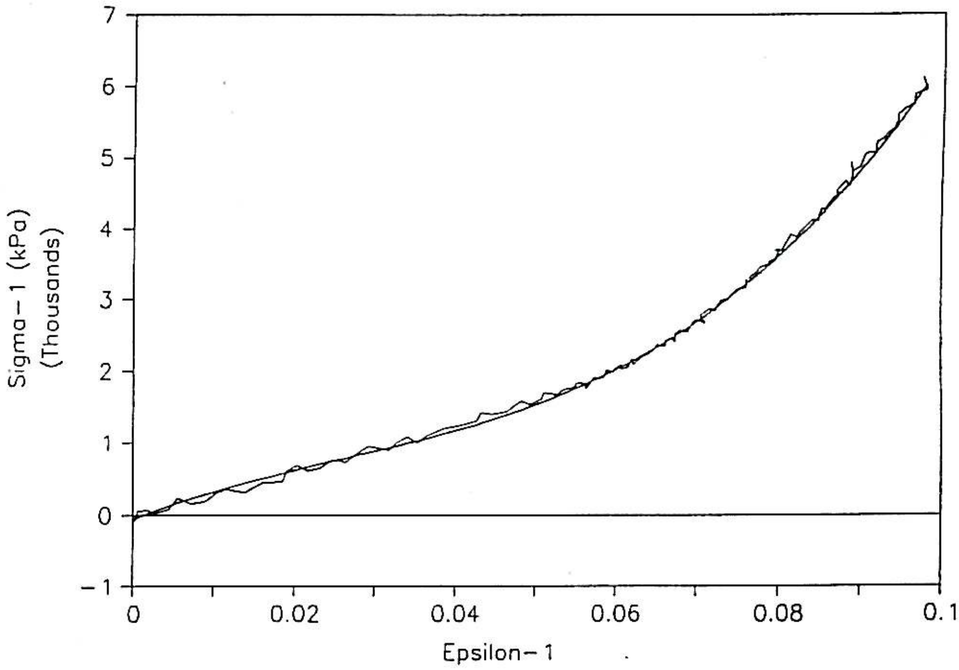

Figure 3 - **Example of the relation between Sigma-1 and Epsilon-1**

Because the sample mass, moisture content, height and diameter are known at the start of the loading cycle, it is possible to determine the percentage of "solid" density of the material on a continuous basis as the sample height and diameter changes with loading. As was mentioned in the compaction study[2] the "solid" density of the materials depends on the nature of the material. For most materials the apparent density is used (= ARD x 1000 kg/m^3), but for porous materials, for which there is a substantial difference in the values of the bulk relative density (BRD) and the apparent relative density (ARD), the dry bulk density is used (= BRD x 1000 kg/m^3).

By plotting E-1 against dry density (% AD or % DBD) it is possible to see what effect density has on the elastic moduli of the materials (see Figure 4).

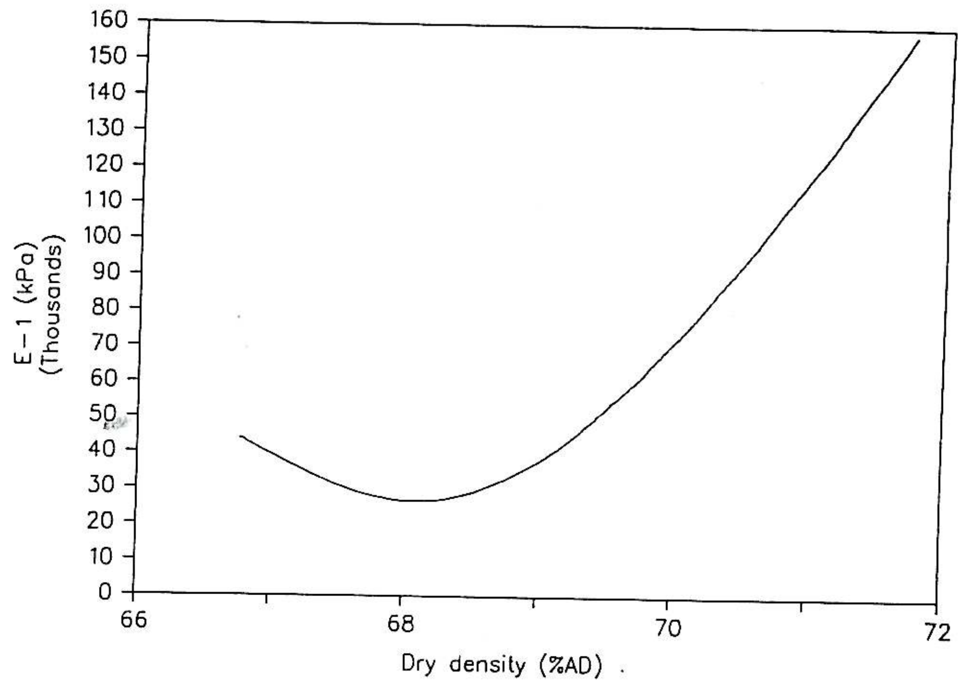

Figure 4 - Example of the relationship between E-1 and dry density (% AD)

It is evident from the smooth curve in Figure 4 that a very good relationship exists between the E-value and the dry density (% AD or % DBD) at a particular moisture content. The curve is generally parabolic in nature. Therefore, by doing a regression analysis between these parameters, it may be possible to estimate E-values from the dry density and moisture content (see Figure 5).

Apart from being able to determine the elastic modulus of the material the measurements of the K-mould can also be used to determine the elastic parameters c, ϕ and υ as well as the values of K_1 and K_2.

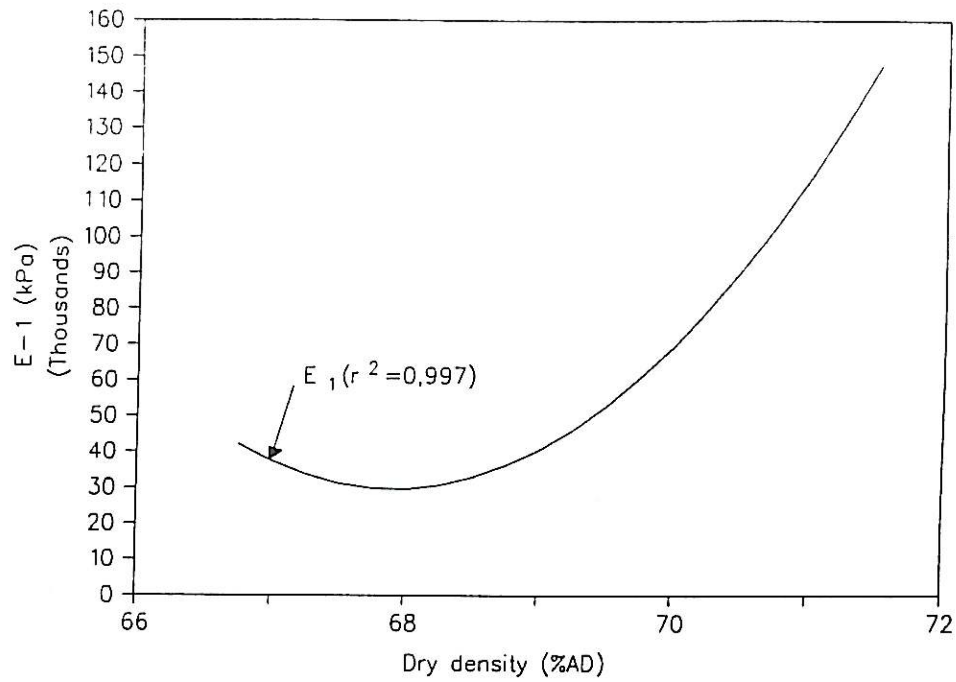

Figure 5 - **Example of the theoretical relationship between E-1 (E = f (DD, DD²)) and dry density (% AD) for Figure 4**

The values of c and ϕ are determined from the $q(=(\sigma_1 - \sigma_3)/2)$ against $p(=(\sigma_1 + \sigma_3)/2)$ plot. By means of regression analysis the following equation is determined:

$$q = m.p + a \quad (3)$$

From equation (3) both ϕ and c are calculated by solving:

$$\sin \phi = m \quad (4)$$

and

$$c = a. \sec \phi \quad (5)$$

where ϕ = friction angle of soil

c = adhesion component of friction force of soil (cohesion)

Poisson's ratio, υ, is solved by the following equation:

$$\upsilon = (e_1.\sigma_3 - e_3.\sigma_1)/(e_1.(\sigma_1 + \sigma_3) - 2\, e_3.\sigma_3) \quad (6)$$

where σ_1 = vertical stress

σ_3 = lateral stress (radial stress)

e_1 = vertical strain

e_3 = lateral strain (radial strain)

υ = Poisson's ratio

To determine K_1 and K_2 a plot is made of log E (MPa) against log θ (kPa) where $\theta = (\sigma_1 + 2.\sigma_3)$. By doing regression analysis of the straight-lined portions of this graph the following equation is solved

$$\log E = K_2.\log \theta + K_1 \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (7)$$

See Appendix B for an example of these results.

DISCUSSION OF THE RESULTS

Originally the vertical stress was to be calculated from the average vertical load as determined by the top and bottom load cells of the K-mould. However, at the start of the testing programme the bottom load cell failed and for the greater part of the research project the vertical load had to be calculated from the top load only. When the crushed stone materials were tested, the bottom load cell had been replaced and the average vertical stress could be calculated. Because the E-values using the average stress for the crushed stone materials seemed low, they were recalculated using the top load only. This increased their E-values by approximately a third. These values are also in agreement with the triaxial test.

However, it came to our notice very recently that Maree[3] had found that the E-moduli as determined by the repeated loading triaxial compression test gave higher values than the E-moduli of the material in the road itself as determined by means of back-calculation of the E-moduli from the deflection measurements. The difference in value is also approximately a third. This means that the E-moduli as calculated from the average vertical stresses of the K-mould are actually closer to the real E-moduli in the road than those measured by the repeated loading triaxial compression test.

As mentioned earlier, density levels and moisture content were again shown to have a very definite effect on the elastic modulus of the materials (see Figure 6 to 8 as examples). By comparing the E-values at the same dry densities (% AD or % DBD) it is clear what tremendous effect a small change in moisture content can have on the strength of materials.

In general the E-values as measured with the DRTT K-mould seem to tie in well with presently used values for materials[3] (see Table 4).

Table 4 - Comparison of E-values from K-mould with standard E-values*

MATERIAL	ESTIMATED SOAKED CBR	STANDARD E-VALUES * (mPa)		K-MOULD E-VALUES (mPa)		
		WET	DRY	WET	DRY	SATU-RATED
KBAB	3	45	90	30	70	10
SPK	7	70	140	70	95	25
SILK	10	90	180	40	140	20
KDW	12	90	180	100	180	20
		Mini-mum	Maxi-mum	100 % mod	Mini-mum	Maxi-mum
DENS7	36	50	300	180 - 260 +	30	200
LENC	7	30	150	40	3	100
NPAB	16	50	250	50 - 70	0	180
TP2	20	50	250	50 - 80	20	150
				86 % AD	Mini-mum**	Maxi-mum
FERRO	200 +	175	1 000	250 - 300	100	500
ROSS	200 +	175	1 000	120 - 270	45	330
NPAA	200 +	175	1 000	160 - 230	30	285
NPAE	200 +	175	1 000	175 - 200	30	400

* See reference 4 - Tables 2, A1 and A5

** Densities around 74 % AD to 78 % AD

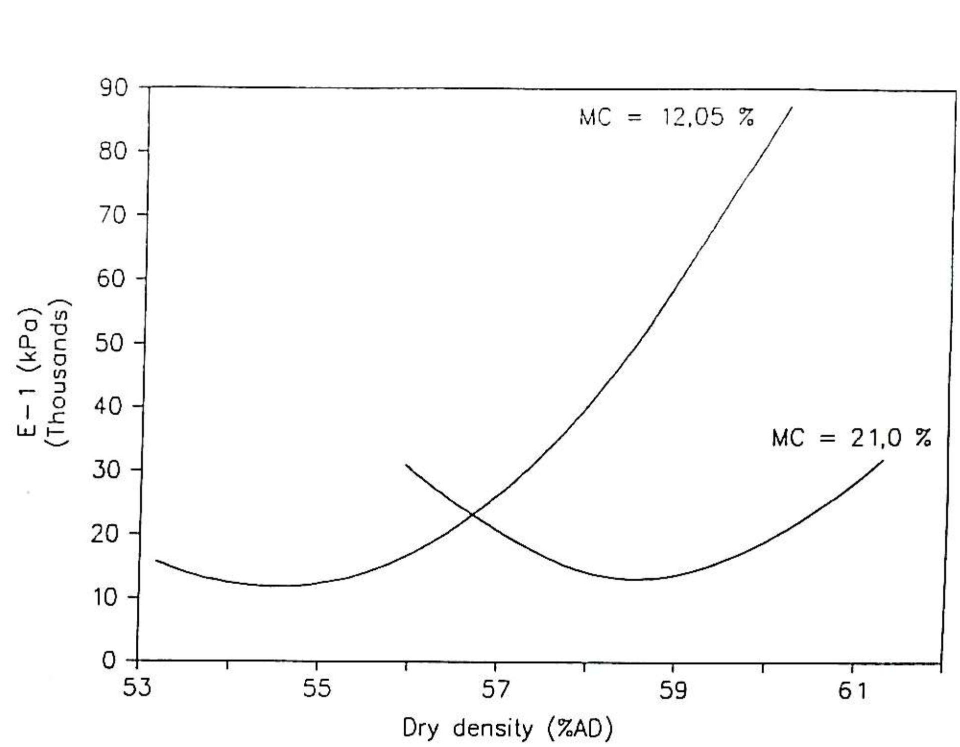

Figure 6 - Measured values for E_1 and against dry density (% AD) for black clay for different moisture contents

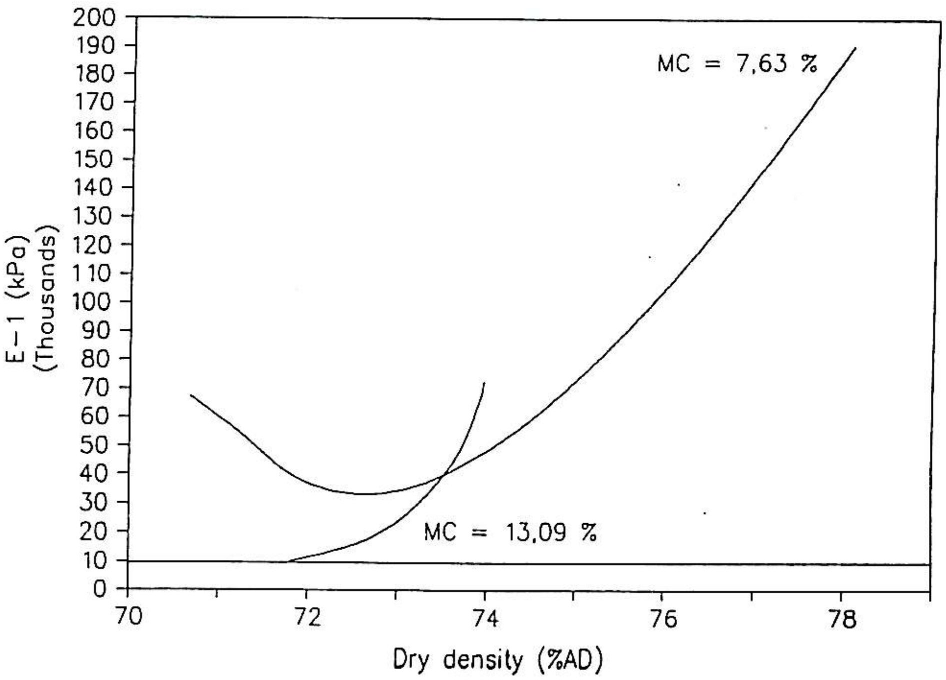

Figure 7 - Measured values for E_1 against dry density (% AD) for decomposed dolerite for different moisture contents

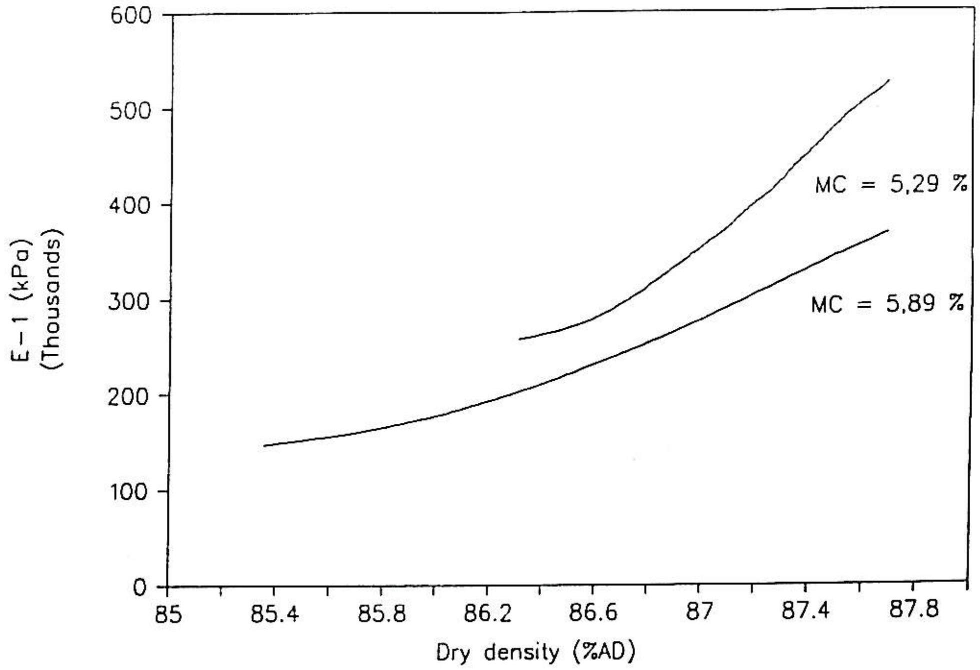

Figure 8 - Measured values for E_1 against dry density (% AD) for quartzite crushed stone for different moisture contents

Most of the curves of E-moduli against dry density seem to have a fairly flat or negative slope at low density levels. This is most probably the area in which plastic deformation or re-orientation of the material's particles is taking place with the subsequent increase in density (see Table 2). Under the repeated load triaxial system this would be attributed to "plastic deformation" of the material. At a certain stage, when the density of the material approaches the maximum dry density obtained on the vibrating table, the relation between the E-moduli and dry density virtually becomes a straight line.

CONCLUSIONS AND RECOMMENDATIONS

It is possible to determine the elastic moduli of untreated roadbuilding materials with a great degree of accuracy with the K-mould in a single loading cycle. The high r^2-values also obtained between the E-moduli and the dry density show that the density of the material has a tremendous influence on the elastic performance of the material. The different E-moduli at the same densities for different moisture contents also show the detrimental effect of high moisture contents.

All roadbuilding materials should therefore be compacted to the highest density practically possible. A maximum allowable moisture content during construction should be specified and proper precautions for surface and subsurface drainage (where required) should be taken on all roadbuilding projects to ensure optimal performance from the road.

The variation in the E-moduli of the materials also emphasises the fact that it is necessary to determine the E-moduli of the actual material to be used if the aim is to design and construct more optimally.

Seeing that both the elastic modulus and the bearing capacity are functions of dry density (% AD or % DBD) and moisture content, it would seem possible to determine the elastic modulus indirectly from the bearing capacity. It would then be relatively easy to verify during construction whether the design criteria (with respect to E-values) are satisfied, by doing in-situ CBR determinations on the finished layerwork. If the design criteria are not satisfied the design can then timeously be adjusted to prevent future failure or alternatively, this procedure could be included in acceptance control systems.

ACKNOWLEDGEMENT

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REFERENCES

1. Handy R L and Fox D E, K-tests for subgrade and base evaluation. ATC Proceedings, 3 - 7 August 1987, Session 4A, Page VI, ISBN 0798842148.

2. Semmelink C J An index for describing the compactability of untreated roadbuilding materials Research Report DPVT-141, Division of Roads and Transport Technology, CSIR, 1990.

3. Maree J H Aspekte van die ontwerp en gedrag van plaveisels met korrel-materiaalkroonlae D.Sc Eng Thesis, University of Pretoria, May 1982.

4. Freeme C R Evaluation of pavement behaviour for major rehabilitation of roads Research Report RP/19/83, National Institute for Transport and Road Research, CSIR, 1983.

C J SEMMELINK

APPENDIX A

FERRT14 (MC=5.29%)(corrected)
Sigma-1 vs Epsilon-1

Regression Output:

Constant	-297.639
Std Err of Y Est	106.9418
R Squared	0.999400
No. of Observations	107
Degrees of Freedom	103

X Coefficient(s)	247853.2	1354823.	32089289
Std Err of Coef.	8870.646	464528.8	6930697.

E-1 vs DD(%AD)(86.3-87.7)

Regression Output:

Constant	5.7E+08
Std Err of Y Est	3269.069
R Squared	0.998174
No. of Observations	98
Degrees of Freedom	95

X Coefficient(s)	-1.3E+07	78129.83
Std Err of Coef.	453843.4	2606.224

APPENDIX B

FERRT14 (MC=5.29%)(corrected)

phi and c (intercept computed)(p<=4000kPa)

Regression Output:

Constant	15.40399		
Std Err of Y Est	7.643164		
R Squared	0.999945		
No. of Observations	67		
Degrees of Freedom	65		
		phi	c
X Coefficient(s)	0.836784	56.80209	28.13347
Std Err of Coef.	0.000769		

phi and c (intercept computed)(p>4000kPa)

Regression Output:

Constant	-181.524		
Std Err of Y Est	6.903145		
R Squared	0.999853		
No. of Observations	41		
Degrees of Freedom	39		
		phi	c
X Coefficient(s)	0.885693	62.33695	-390.988
Std Err of Coef.	0.001715		

APPENDIX B (CONTINUED)

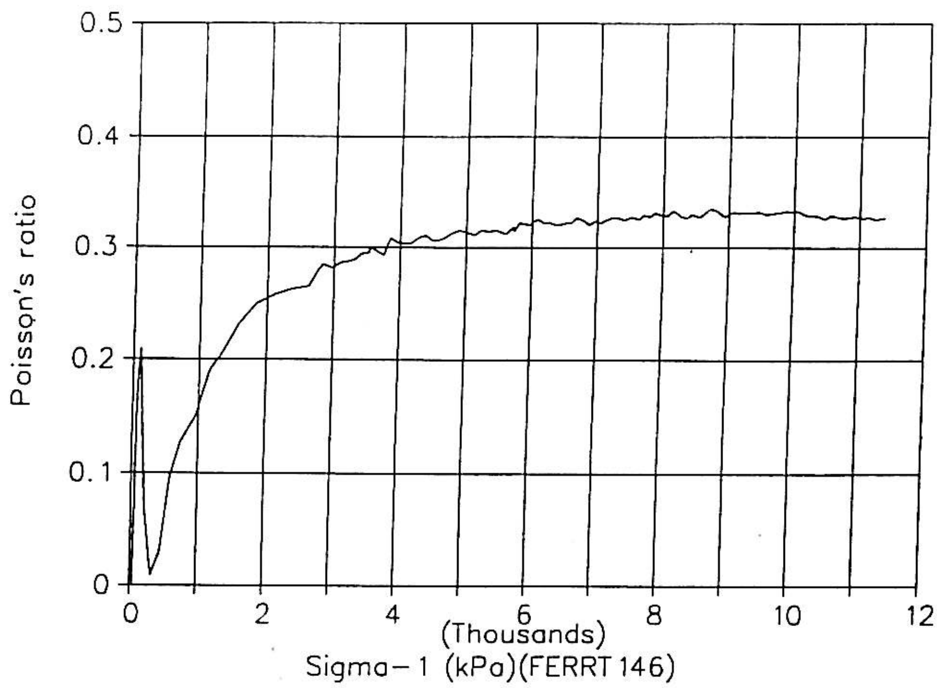

Figure B1 - Example of Poisson's ratio against Sigma-1

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FERRT14 (MC=5.29%)(corrected)

log(E-1)(MPa) vs log(Theta)(kPa)(log(Theta)<3.6)
            Regression Output:
Constant                        2.305290
Std Err of Y Est                0.011142
R Squared                       0.851266
No. of Observations                   16
Degrees of Freedom                    14

X Coefficient(s)   0.043744
Std Err of Coef.   0.004886

log(E-1)(MPa) vs log(Theta)(kPa)(log(Theta)>3.6)
            Regression Output:
Constant                        0.972569
Std Err of Y Est                0.009481
R Squared                       0.982855
No. of Observations                   90
Degrees of Freedom                    88

X Coefficient(s)   0.411918
Std Err of Coef.   0.005799
```

APPENDIX E

PAVEMENT RESPONSE MEASURING SYSTEM

BY M DE BEER

This paper was submitted for publication at the 2nd International Symposium on the State of the Art of Pavement Response Monitoring Systems for Road and Airfields, Cold Regions Research and Engineering Laboratory, Corps of Engineers, Hanover, New Hampshire, September 1991

PAVEMENT RESPONSE MEASURING SYSTEM

M de Beer,[1] Member, SAICE

ABSTRACT: Multi-depth Deflectometer (MDD) technology is becoming more widely used to quantify pavement response under static and dynamic loading. This paper describes recent research in South Africa with the MDD system under an abnormal heavy vehicle (14 ton per axle). Three typical in situ heavy duty pavements were evaluated and some of the more important findings are reported here. The paper demonstrates that pavement response on the surface, as well as in depth, is adequately measured, facilitating the quantification of pavement response under these heavy vehicles. Furthermore, effective elastic moduli have been back-calculated from the depth deflections from which tensile strains have been calculated within the asphalt and cemented layers. Finally, a summary is given on the effect of increased pavement temperature and vehicle speed on pavement response (deflections and strains).

INTRODUCTION

The Multi-depth Deflectometer (MDD) technology originated in South Africa during the 1970s in association with the accelerated full-scale pavement test facilities called the Heavy Vehicle Simulator (HVS) (De Beer et al. 1988, Freeme et al. 1987). For the past 12 to 15 years more than 1.4 billion equivalent standard 40 kN dual wheel load repetitions (80 kN per axle) have been applied to a wide range of pavement structures in South Africa. Most of these pavement sections have been evaluated with MDD instruments within the structures, from which effective elastic moduli have been back-calculated for modelling purposes. In association with the deflection measurements, plastic or permanent deformation has also been measured (De Beer, 1986, 1990). MDD technology was also transferred to the Texas Transportation Institute (TTI) in the USA during 1987 and was implemented with great success (Scullion et al., 1989). Details of the MDD system are given elsewhere (De Beer et al., 1988).

The aim of this paper is to present some of the latest results on pavement response under abnormal heavy loads up to 14 tons per axle. The deflection measurements have been done with the MDDs on three typical heavy duty pavements near Pretoria in the Transvaal.

MEASURING SYSTEM

The MDD pavement response measuring system used in this evaluation is illustrated in Figure 1. The figure indicates that the MDD modules were installed at the layer interfaces, wit the central core anchored at a depth of approximately 2,0 to 3,0 metres.

On all the pavement sections, MDDs were also installed flush with the surface in order

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to measure total surface deflections. AC-LVDTs were used and therefore a conditioner is needed for signal control purposes. Data capturing was done with a "PC30" - analogue to digital conversion circuit board using an Olivetti M19. The data was triggered using a piezo electric film strip (10 mm x 40 mm) mounted on the road surface approximately 2 to 6 metres in line with the MDD location, depending on vehicle speed. For low speeds, data capturing was done at a rate of 125 readings per channel per second (4 channels), and for high speeds (>40 km/h^{-1}) the sampling rate varied between 250 to 375 readings per channel per second. The piezo electric film strip enabled the calculation of vehicle speed and lateral load position (no triggering if the load misses the strip) and position of load relative to the MDD deflection profile.

The data were stored directly on LOTUS spreadsheets, and with the aid of a second micro-computer (Olivetti M19) the data can be validated for errors or unacceptable noise, etc., before final saving.

The use of a second computer is highly recommended as it saves useful time, especially when measurements are made at remote locations, or at varying pavement temperature, speed, etc.

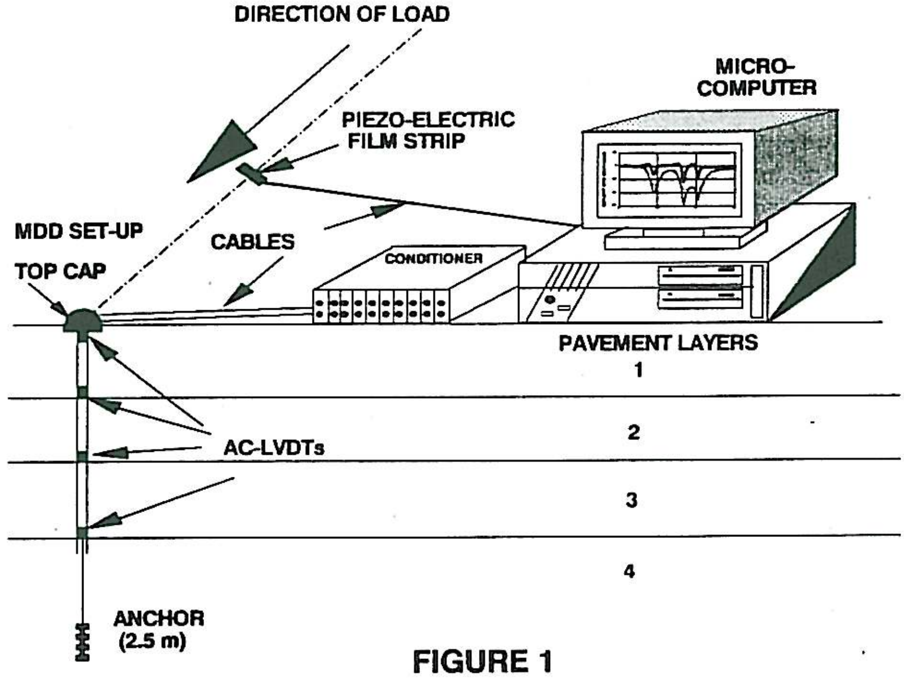


FIGURE 1

PAVEMENT RESPONSE MEASURING SYSTEM

Pavement Structures and Instrumentation

The three heavy duty pavement structures evaluated in this study are illustrated in Figures 2, 3 and 4. Figure 2 illustrates the asphalt base section (Section 1). The pavement consists of two 35 mm semi-gap-graded asphalt surfacings (overlays) with a 120 mm continuously-graded asphalt base layer. The asphalt base is supported by two lightly cementitious subbases of 170 mm and 150 mm each. These subbases were supported by a 115 mm selected layer on top of the subgrade. In Figure 3, the structural detail of Section 2 (175 mm crushed stone base), also on a cementitious subbase (180 mm), is illustrated.

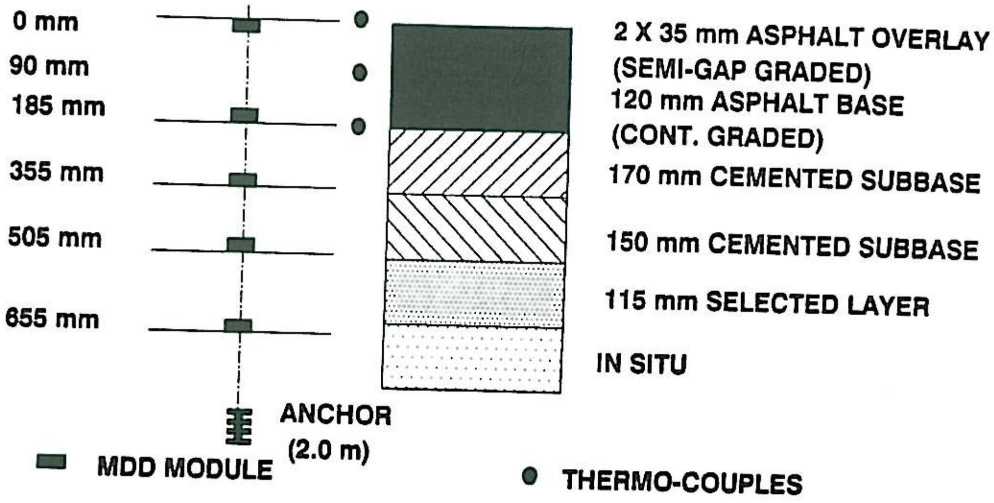


FIGURE 2

PAVEMENT AND MDD LAYOUT ON THE ASPHALT BASE PAVEMENT (SECTION 1)

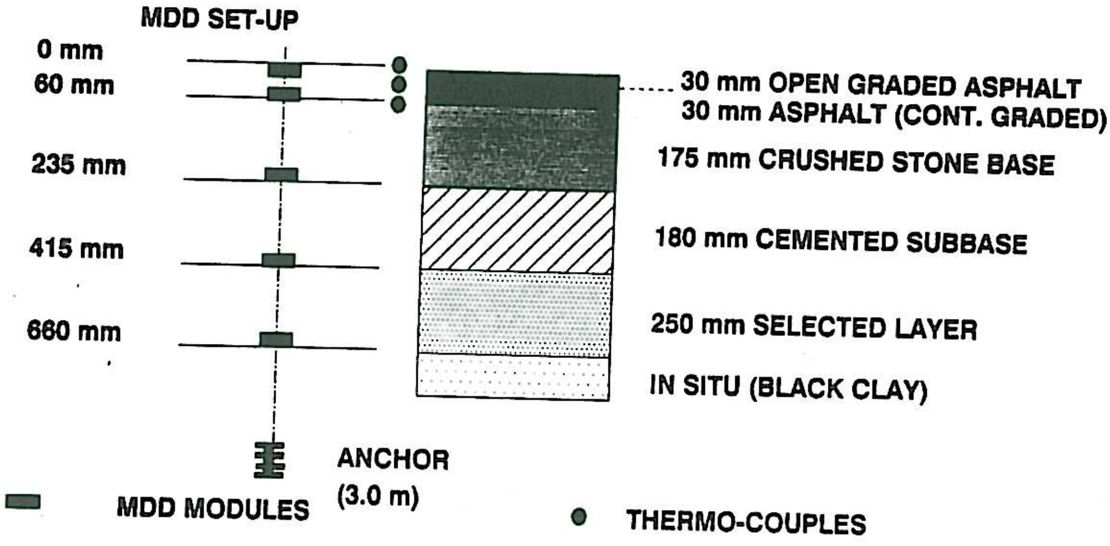


FIGURE 3

PAVEMENT AND MDD LAYOUT ON THE GRANULAR BASE PAVEMENT (SECTION 2)

Figure 4 illustrates the structural detail of Section 3, the strongly cemented (but cracked) base pavement supported by a crushed stone subbase (upside down design). The anchor depths of the MDDs varied between 2,0 m and 3,0 m. To quantify possible deflection of the anchor under the 14 ton axle loading, the results of two sets of MDDs anchored at 2,0 m and 3,0 meters were compared and showed no significant differences. This test was done on the granular pavement with the two MDDs approximately 3 metres apart. Based on this finding, no further attempts were made on this project to better quantify possible anchor deflection. (Anchor deflection will result in an overestimation of the effective elastic moduli of the subgrade, and hence possible underestimation of stresses and strains within the structural pavement layers, without any major influences on the back-calculated moduli of the upper pavement layers.)

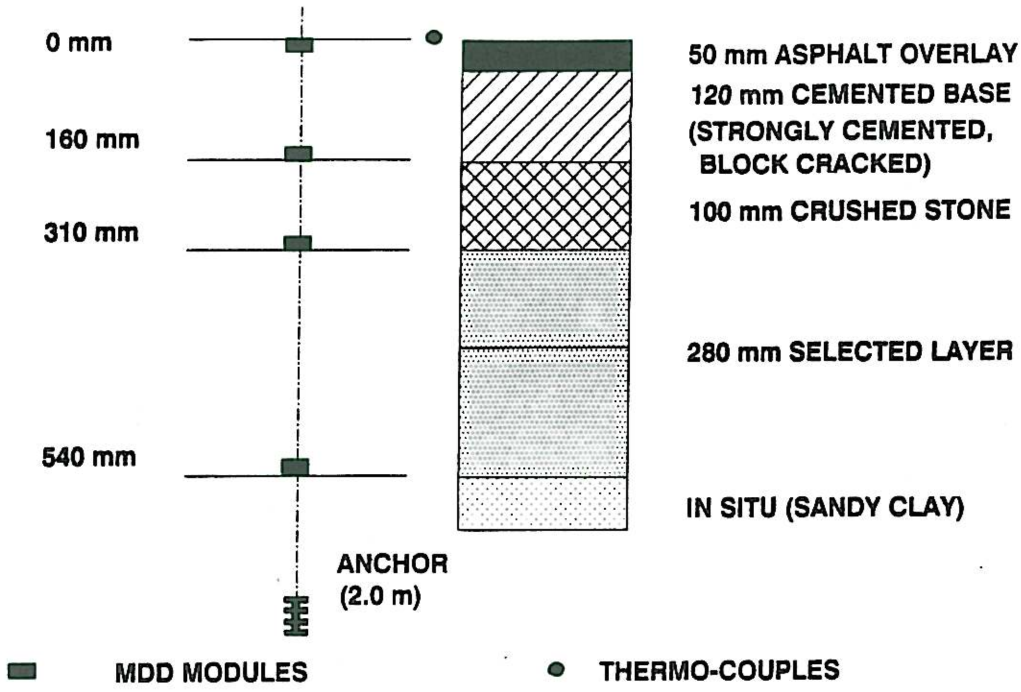


FIGURE 4

PAVEMENT AND MDD LAYOUT ON THE CEMENTED BASE PAVEMENT (SECTION 2)

PAVEMENT RESPONSE

Asphalt Base Pavement (Section 1)

In Figure 5 typical MDD deflection response under the 42 ton tridem axle test vehicle (14 ton per axle) is illustrated. The figure illustrates well-defined deflection influence lines at various depths within the pavement. This measurement was done at a vehicle speed of 78,5 km/h, with asphalt temperature (top) at 59 degrees celsius. Highly non-symmetrical ("banana-shaped") surface deflection basins were noticed on this pavement. The "banana-shape" was a direct result of the visco-elastic properties of the asphalt base layer, as is discussed in more detail later.

Granular Base Pavement (Section 2)

Figure 6 illustrates typical MDD deflection response (under the same loading as the previous Section 1) on the granular base pavement. In this case, deflection basins are almost symmetrical, and also well-defined. Also, in this case, the temperature of the 60 mm asphalt surfacing was approximately 23 degrees celsius, and the test vehicle speed 8,9 km/h.

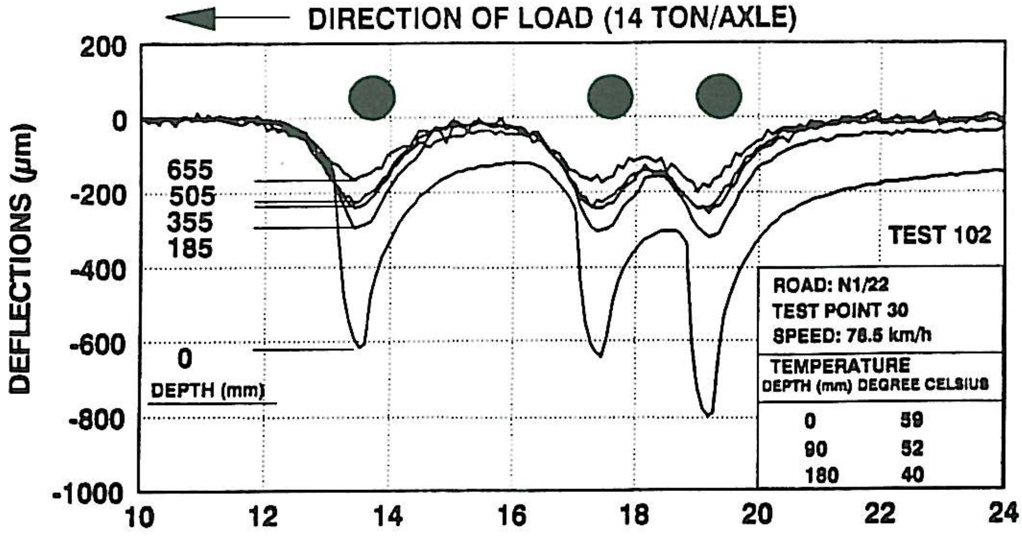


FIGURE 5

TYPICAL MDD DEFLECTION RESPONSE ON THE ASPHALT BASE PAVEMENT (SECTION 1)

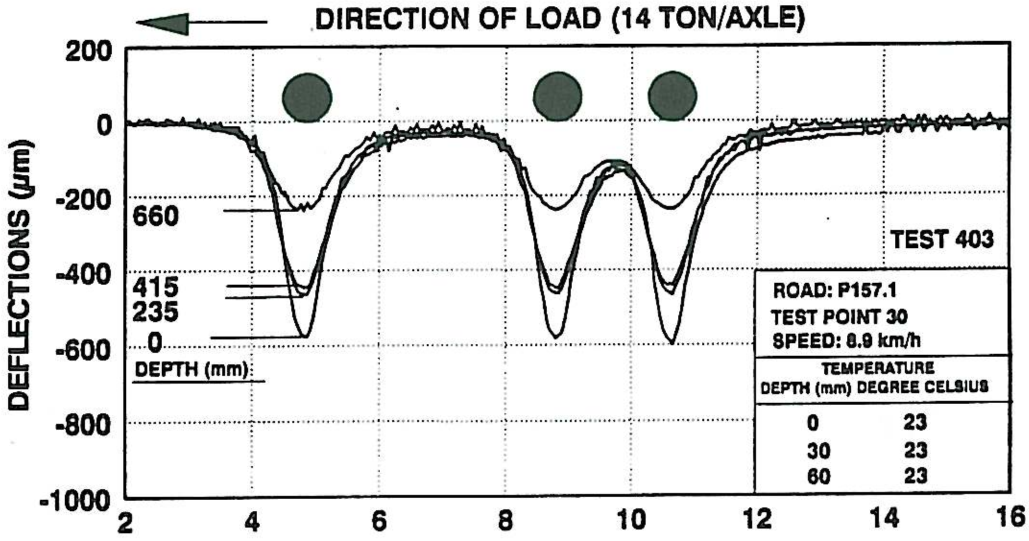


FIGURE 6

TYPICAL MDD DEFLECTION RESPONSE ON THE GRANULAR BASE PAVEMENT (SECTION 2)

Cemented Base Pavement (Section 3)

Figure 7 illustrates a typical MDD response on the cemented base pavement under the 14 ton per axle vehicle. The deflection basins are also much more symmetrical than those of the asphalt base section (asphalt surfacing in this case was 50 mm). The vehicle test speed was 3,2 km/h, with asphalt temperature of 31,2 degrees celsius. Another difference in the shape of the deflection basins in this case is that they are relatively wider than those on the granular and asphalt base pavements. This is due to the wider load-spreading capability of the strongly cemented layers (6-12 MPa unconfined compressive strength). The relatively high deflection (or vertical strain) within the asphalt and cemented base layer suggests weaknesses within or between these layers (as was captured with the two top MDDs). Test pit detail of this pavement indicated weaknesses and moisture accumulation between the thin asphalt surfacing and the cemented base, as well as longitudinal and transverse cracking in the cemented layer.

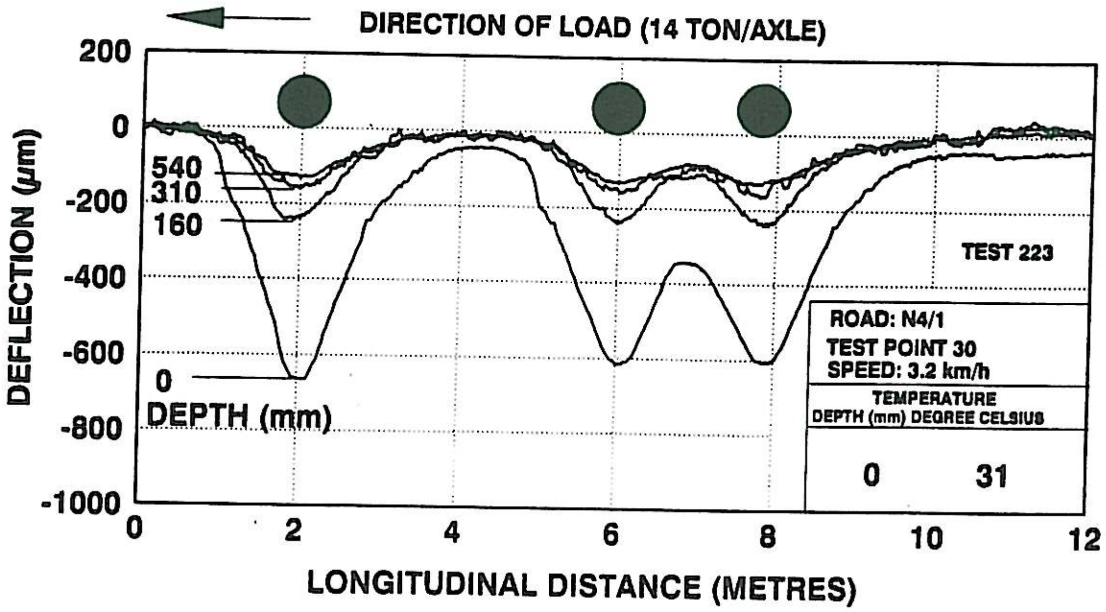


FIGURE 7

TYPICAL MDD DEFLECTION RESPONSE ON THE CEMENTED BASE PAVEMENT (SECTION 3)

It is worthwhile to mention that incorrect identification of layer thickness leads to incorrect placing of certain MDD modules, which may result (normally) in a gross error of the individual layer moduli. However, such moduli are viewed as "combined effective moduli" and may be used as such in the modelling of the pavement structure, but one must be aware of possible errors in the calculation of stresses and strains in these cases. Figure 8 illustrates the MDD response of the granular pavement under a typical 6-axle vehicle transporting logs.

In this case, as with the asphalt pavement (Section 1), the increase in deflection towards the rear axles is well illustrated. This increase in deflection, however, is a function of pavement rebound (elasticity), vehicle speed and distance between the axles.

VERTICAL STRAIN

Owing to the relatively well-defined deflection influence lines at various depths (at layer interfaces), vertical compressive (and tensile) strains can be easily computed. For the vertical strain, the relative deflection between two MDDs is divided by the distance between these MDDs. The shape, however, of the relative deflection profile within a layer, is the same as the shape of the strain profile. The relative deflection profiles within the asphalt, granular and cemented base pavements are illustrated in Figures 9(a), (b) and (c). Figure 9(a) clearly indicates the typical visco-elastic behaviour of the asphalt layer. A more detailed study of the deflection creep response of the asphalt layer is given in Figure 10. The figure indicates the familiar asphalt creep response (normally measured in well-controlled laboratory conditions). The creep response shows the following characteristics:

- Dilation ("swelling") in front of moving wheel (vertical strain) of approximately 125 $\mu\epsilon$).

- Initial elastic response
- Delayed elastic response (creep characteristics)
- Immediate elastic recovery
- Delayed elastic recovery
- Permanent deformation

Although most of the characteristics ofthe creep response of asphaltic materials are well known (Ruth, 1989), the dilation (vertical tensile strain) in the front and on the sides of a moving wheel appear somewhat strange. However, this has occurred with most of the tests with the heavy vehicle (14 tons per axle) on the pavements where the asphalt thickness has been in excess of 60 mm, and with relatively rigid support.

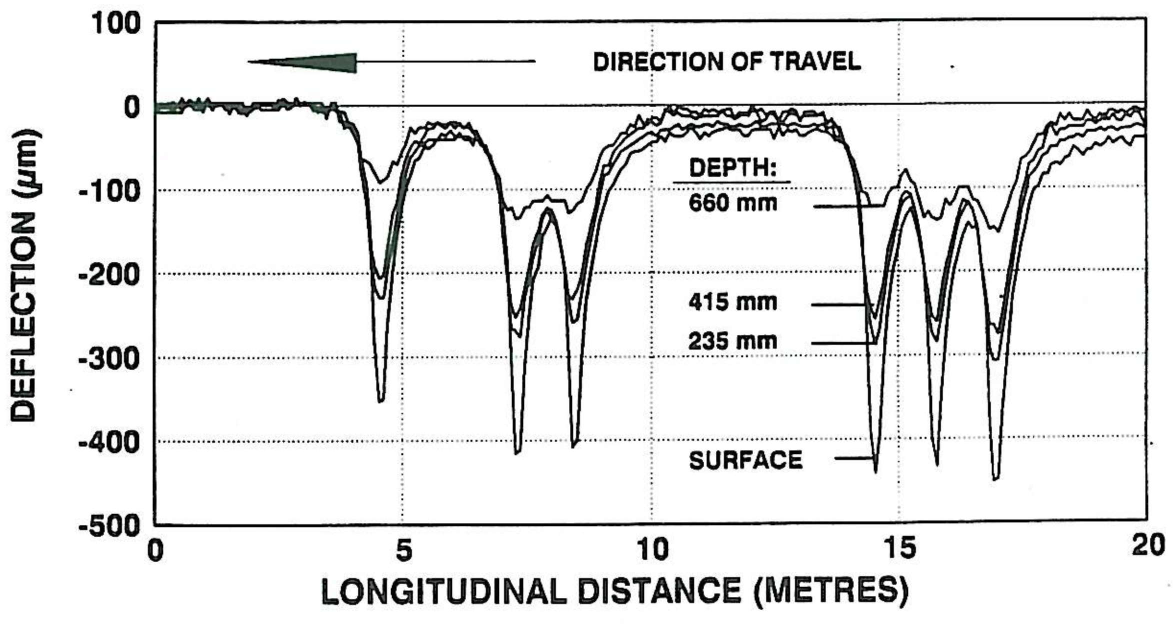


FIGURE 8

TYPICAL MDD DEFLECTION RESPONSE ON THE GRANULAR BASE PAVEMENT RESULTING FROM A TYPICAL 6 AXLE TRUCK

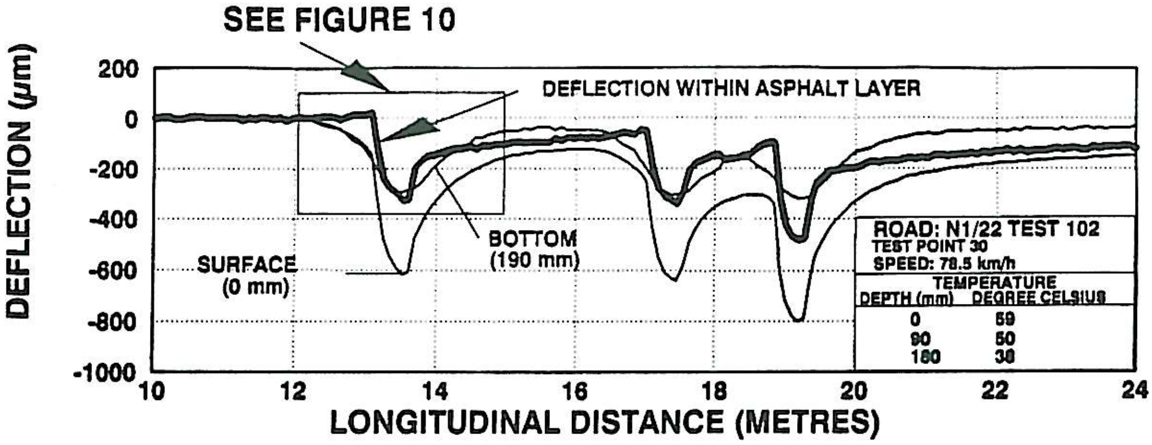


FIGURE 9a

VERTICAL DEFLECTION (STRAIN) IN THE ASPHALT BASE LAYER (SECTION 1)

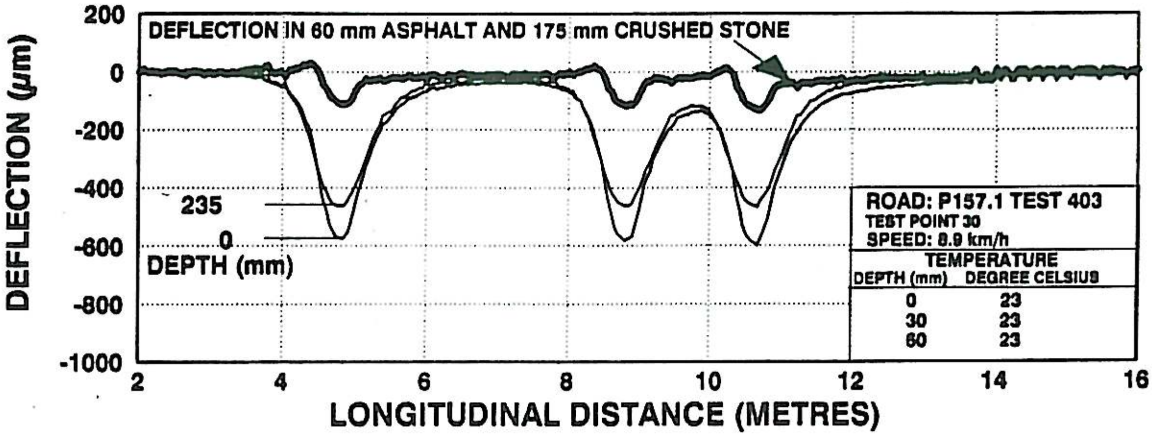


FIGURE 9b

VERTICAL DEFLECTION (STRAIN) IN THE GRANULAR BASE LAYER (SECTION 2)

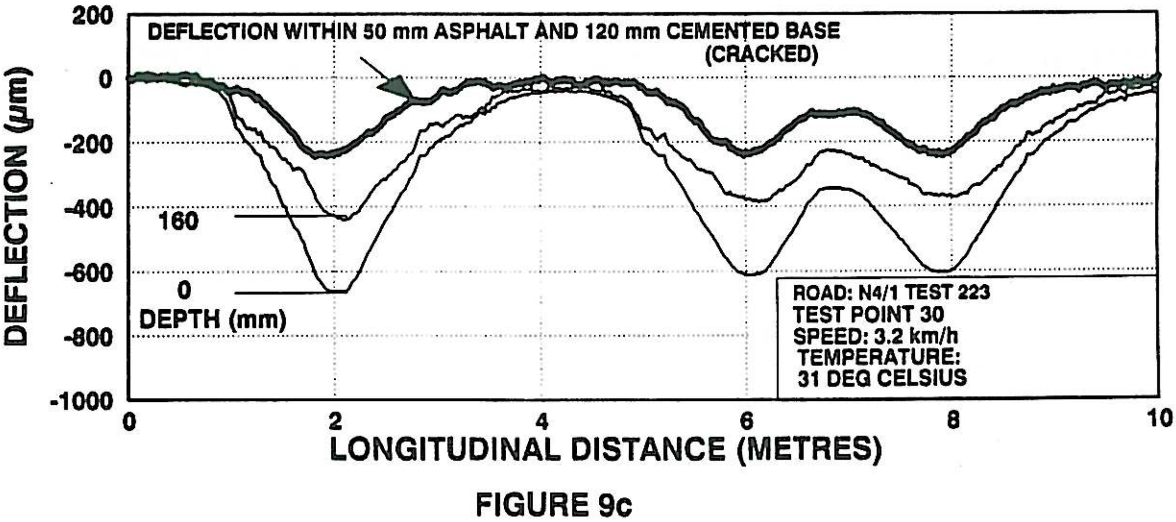


FIGURE 9c

VERTICAL DEFLECTION (STRAIN) IN THE CEMENTED BASE LAYER (SECTION 3)

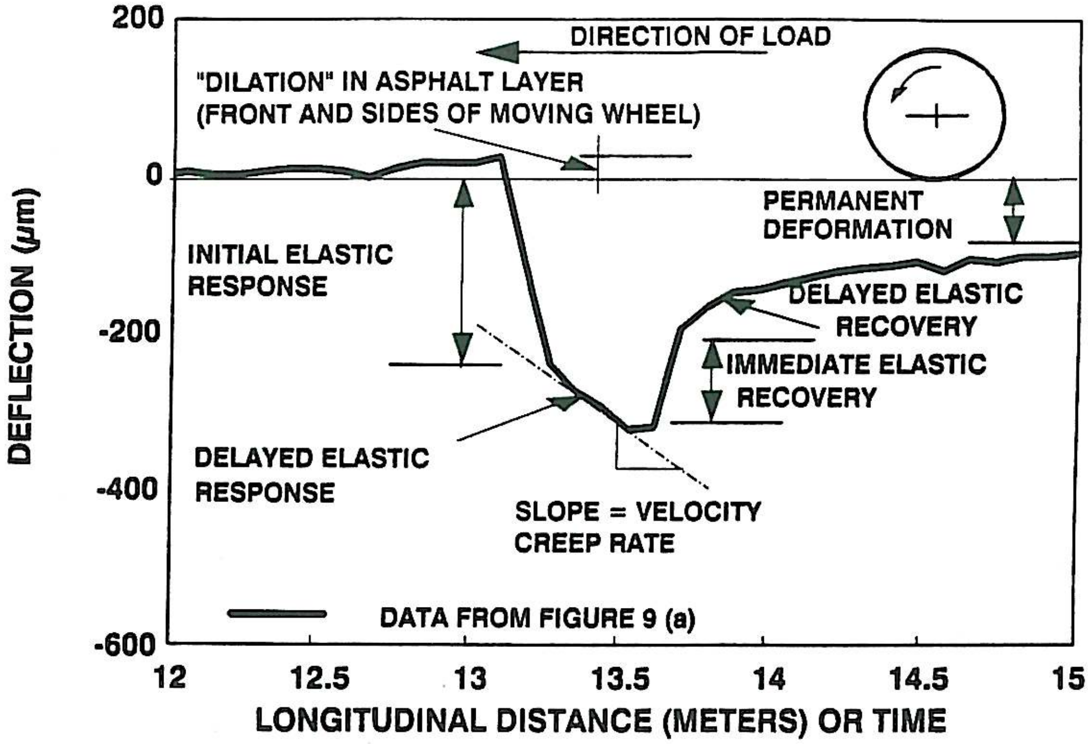


FIGURE 10
MEASURED CREEP RESPONSE OF THE ASPHALT LAYER
(DEFINITIONS AFTER RUTH, 1989)

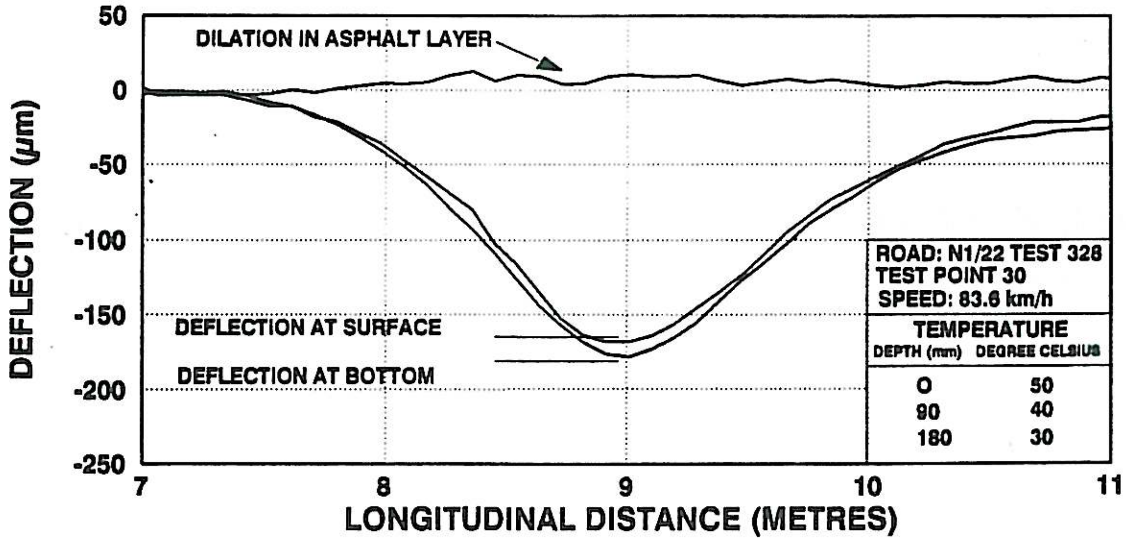


FIGURE 11
MEASURED DILATION WITHIN THE ASPHALT LAYER (14 TON/AXLE LOAD)

It is also interesting to note that dilation occurred in front of each axle (see also Figures 9(a) and (b)). This behaviour did not occur on the cemented base pavement with the 50 mm asphalt. See Figure 9(c). This "dilation" of the asphalt was also noticed when the load bypassed the MDD test position. See Figure 11. It is believed that with a higher sampling rate (>2000 readings per channel per second) the shape of the creep response curve (Figure 10) will be better defined, which will facilitate the calculation of the visco-elastic properties of the asphalt, such as creep, strain rate, moduli, etc. A "static creep test" was conducted on the asphalt base pavement with the 42 ton test vehicle (7 ton per wheel) on the MDD position for approximately 13 minutes. Figure 12 clearly indicates the creep response of the asphalt layer compared with that of the lower cemented and in situ granular layers.

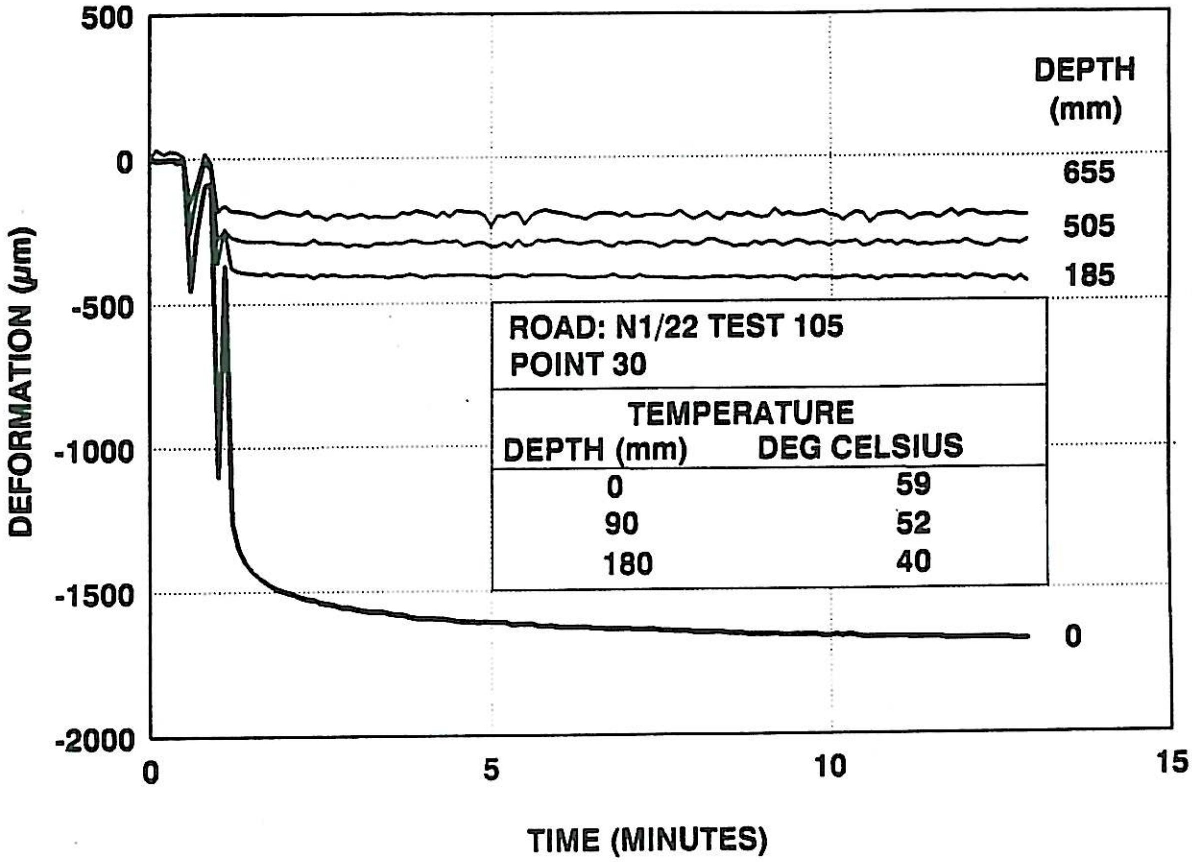


FIGURE 12

RESULT OF A STATIC CREEP TEST UNDER THE 14 TON/AXLE TEST VEHICLE ON THE ASPHALT BASE PAVEMENT (SECTION 1)

DEFLECTION BASIN PARAMETERS

For the purpose of the study with the 42 ton tridem axle test vehicle, it was necessary to define certain prominent deflection basin parameters, based on a typical measured deflection influence profile. These basin parameters are indicated in Figure 13.

The parameters are:

- Peak deflections: T1, T2 and T3
- Valleys: V1, V2 and V3
- Rebounds: R1 and R2
- Distance between axles: L1 and L2
- Total Area: As
- Phase differences: α_1, α_2 and α_3 (which is the distance between the position of the centre of the load and the peak deflections T1, T2 and T3).

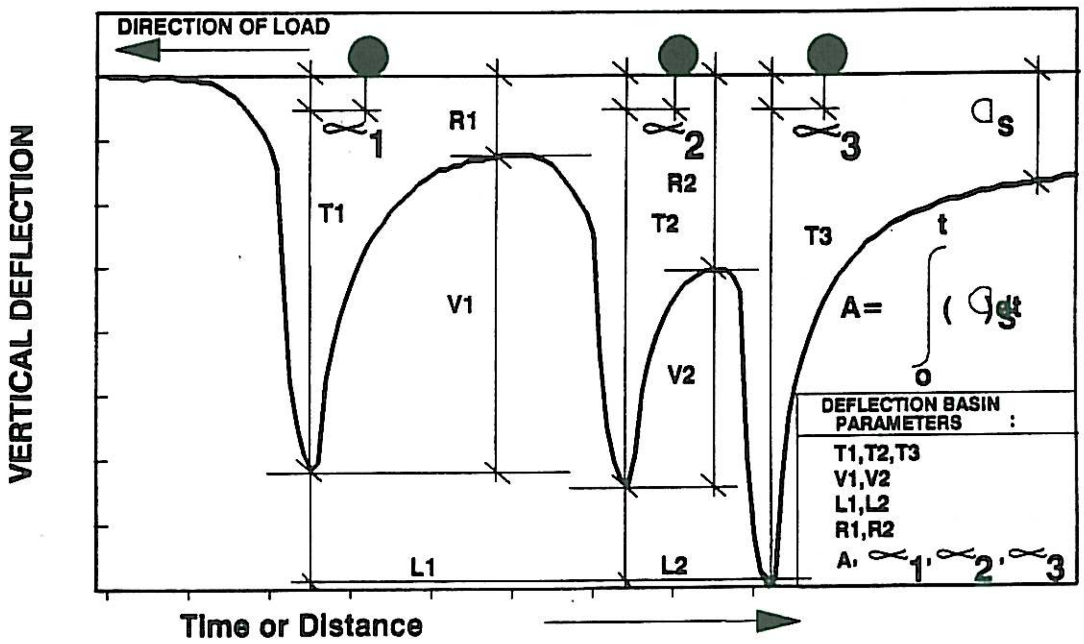


FIGURE 13

DEFINITION OF THE DEFLECTION BASIN PARAMETERS FOR THE FOR THE TEST VEHICLE

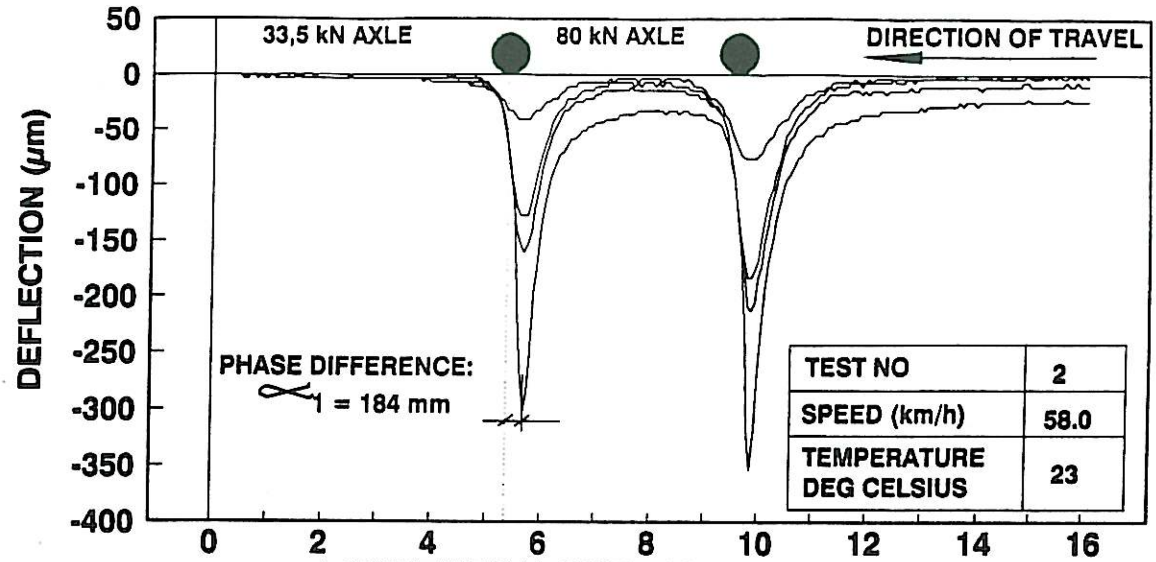


FIGURE 14

PHASE DIFFERENCE BETWEEN THE LOAD POSITION AND PEAK DEFLECTION ON THE GRANULAR BASE PAVEMENT FOR A TEST VEHICLE MOVING AT 58 km/h

For back-calculation of moduli purposes, the average (or chosen percentile) values of the peak deflections T1, T2 and T3 are used. (Future studies will concentrate on the rebounds, valleys, and function of variables such as pavement type, temperature, speed, distance between axles, etc.). Another interesting finding has been made concerning the phase difference. Super-positioning of the load position onto the deflection influence profile (transforming the profile to distance from time domain) has resulted in a measurable lateral distance between the centre the load and the peak deflections. A typical measurement using a normal vehicle with a 80 kN rear axle at a speed of 58 km/h is illustrated in Figure 14.

In this case, the phase difference α_1 = 184 mm. Detailed analysis of the data indicated that the phase difference varies with vehicle speed. A typical result at a range of speeds for the granular pavement (with the front axle 33,5 kN)) is illustrated in Figure 15.
The figure indicates "negative" phase differences (peak deflection following the load), and that it is strongly related to speed of loading. A phase difference of up to 200 mm at a speed of 80 km/h was measured on the granular pavement. This phase difference is directly related to damping (and inertia) of the pavement system, and is a function of the damping characteristics of all the layers in the pavement structure. Rigorous theoretical analysis of a similar pavement by Lourens (1991) where a damping ratio, D=5 % was assumed for all the layers, also indicated similar response. This, however, is an area for further research, and the implications for design and evaluation resulting from this phenomenon, as well as from the dilation characteristics of the asphaltic material in Figure 10, is at this stage unclear, and

needs to be addressed with further research.

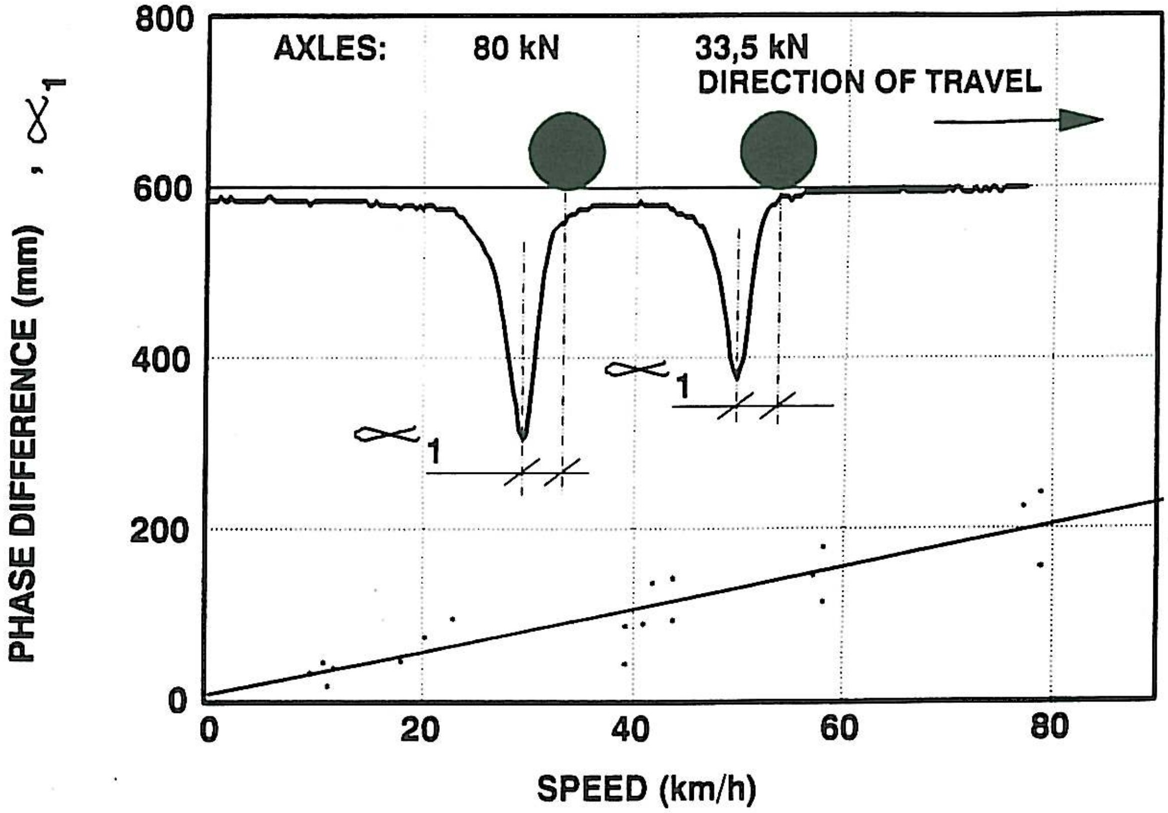


FIGURE 15

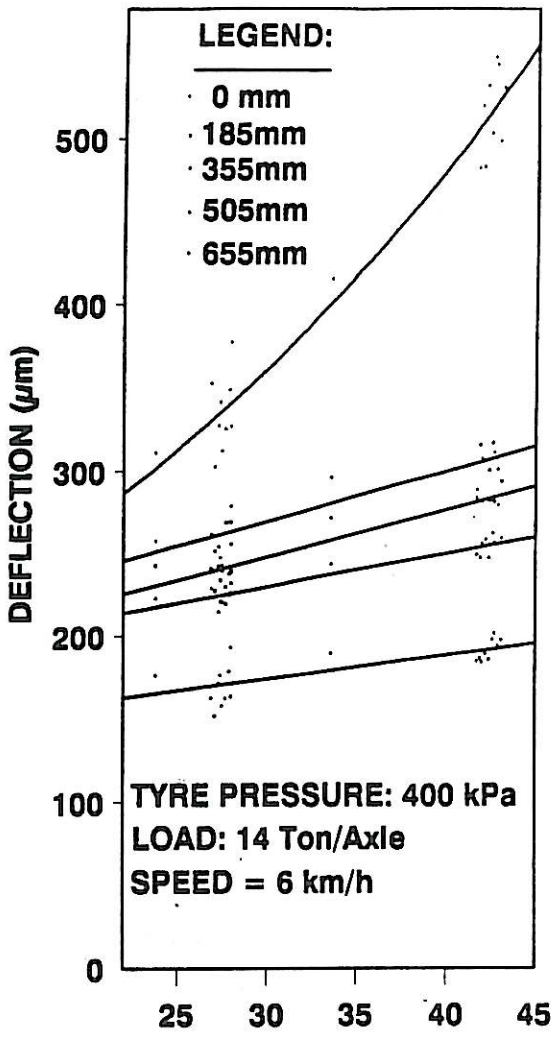
PHASE DIFFERENCE BETWEEN AXLE LOAD CENTRE AND POSITION OF PEAK DEFLECTION ON THE GRANULAR BASE PAVEMENT (SECTION 2)

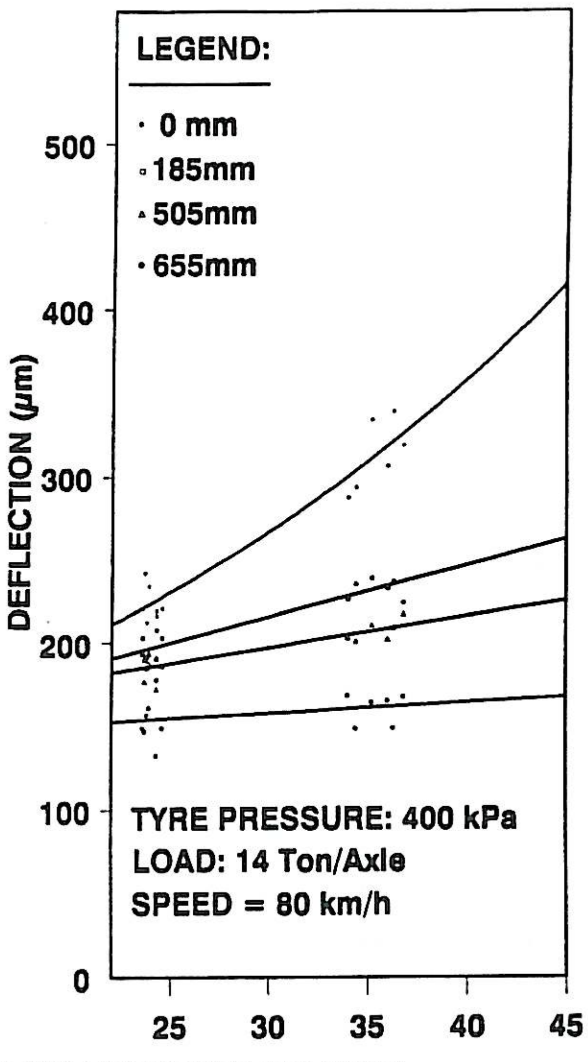
Effects of Temperature and Speed

Temperature Effects

Figures 16 and 17 illustrate the effects of increased asphalt temperature on the MDD deflection of the 190 mm thick asphalt base pavement section. These figures clearly indicate the increase in deflection within the asphalt layer (as the effective elastic modulus of the asphalt decreases as a result of increased temperature).

As a result of this decrease (1856 MPa to 285 MPa, ie 85 per cent) in asphalt elastic modulus, the effective load spreading capability decreased, hence the increase in deflections





PAVEMENT TEMPERATURE (DEGREES CELSIUS)

FIGURE 16: 6 km/h

FIGURE 17: 80 km/h

DEFLECTION WITHIN ASPHALT BASE PAVEMENT AT VARIOUS DEPTHS AS A FUNCTION OF TEMPERATURE AND SPEED

throughout the depth of the pavement structure. Note the increased deflections within all the lower supporting layers, including the subgrade. Figure 18 illustrates the temperature effects on the computed tensile and vertical strains (from effective elastic moduli back-calculated from the MDD deflections) within the pavement. The figure indicates that the tensile strain decreases at the bottom of the asphalt and is associated with an increase in tensile strain at the bottom of the cemented base layer. Also the vertical compressive strain on the subgrade shows increase as a result of an increase in the temperature of the asphalt base layer. Although the vertical compressive strain on the subgrade is relatively small, the increase in tensile strain at the bottom of the cemented subbase is significant. At an asphalt temperature of 20 degrees celsius (68 Fahrenheit), the effective fatigue life of the cementitious subbase layer (De Beer, 1989, 1990) is more than 50 x 10^6 40 kN dual wheel load repetitions, and with the increase in temperature to 45 degrees Celsius (113 Fahrenheit), the effective fatigue life is reduced to 6 x 10^6 load repetitions. This is a clear indication of the importance of environmental conditions such as temperature in the design and evaluation of this type of asphalt base pavements.

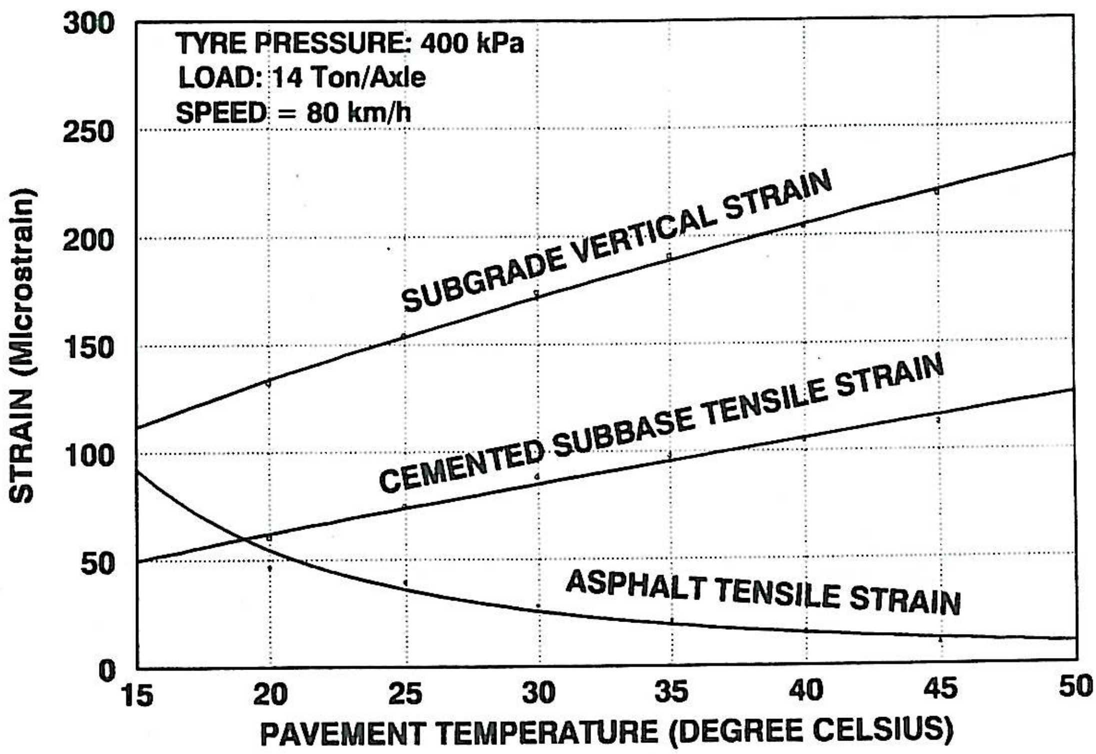


FIGURE 18

STRAIN WITHIN THE ASPHALT BASE PAVEMENT STRUCTURE AS A FUNCTION OF TEMPERATURE

Effects of Speed

As with the deflection measured at different temperatures on the same pavement and loading conditions, the MDD system is ideally suited to measure the effects of vehicle (load) speed on vertical deflection. In Figure 19 the deflection response of the asphalt base pavement at a range of vehicle speeds (5 to 80 km/h) is illustrated. The deflections were normalised (corrected) for an asphalt temperature of 30 degrees Celsius (86 Fahrenheit). These deflections reduced with an increase in speed. The decrease in deflection predominantly occurred within the asphalt base layer (190 mm thick). As with temperature, the effects of the deflections in the lower supporting cemented layers and the subgrade are also affected. In this case, the deflections in all the layers decreased with an increase in speed of the loading. The back-calculated effective elastic moduli of the asphalt layer increased (stiffened) with increase in vehicle speed. This increase (691 MPa to 1158 MPa, ie 68 per cent) in asphalt modulus effectively increases the load-spreading capability of the asphalt layer and hence reduced deflections in the lower layers. Recent rigorous analysis of moving wheel loads at different speeds indicated relatively large reductions in vertical stresses in the pavement layers and hence reduced vertical deflections (Lourens, 1991). This aspect needs further research and more attention should be given to the dynamic characteristics, such as factors of damping and inertia of pavement layers. The data in Figure 19 indicates that the reduction in deflections is according to a power law (Deflection = αv^b α,b=constants, v=vehicle speed). This was also true for the deflections within the cemented base layers. According to these results, increases occurred in the effective elastic moduli of all the layers. In Figure 20, the tensile strains at the bottom of both the asphalt and cementitious subbase, and the vertical compressive strain on the subgrade are illustrated as a function of vehicle speed. In general, a decrease in all the strains in the pavement occurred with vehicle speed. In this case, for example, the effective fatigue life of the cemented subbase increased from 12 x 10^6 E80s to 21 x 10^6 E80s, with an increase of vehicle speed from 5 km/h to 80 km/h. From these results, the "better" supporting conditions from the lower layers effectively reduce the tensile strain at the bottom of the asphalt layer, regardless of the stiffening in asphalt elastic moduli.

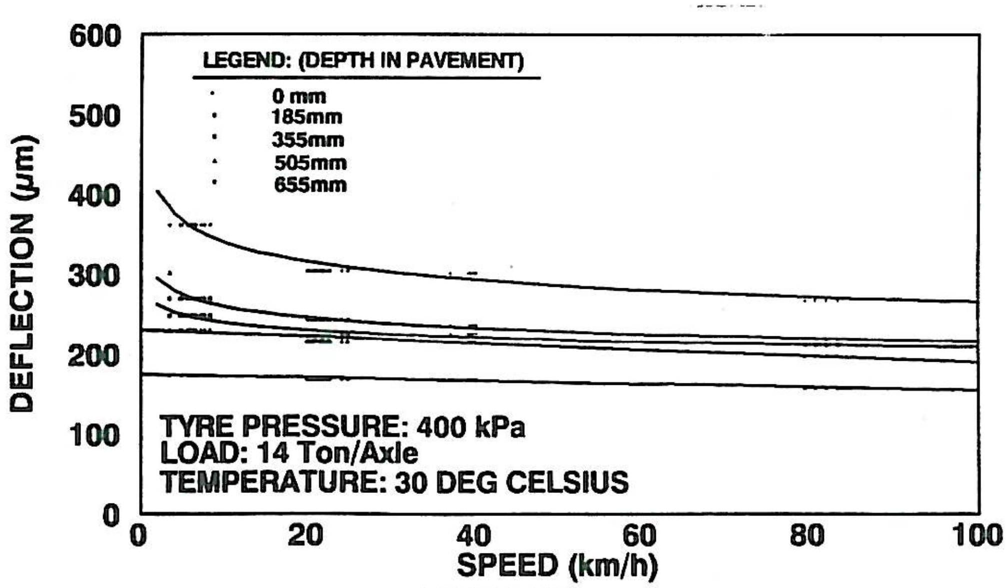


FIGURE 19

EFFECT OF VEHICLE SPEED ON DEFLECTION WITHIN THE ASPHALT PAVEMENT AFTER TEMPERATURE CORRECTION

M DE BEER

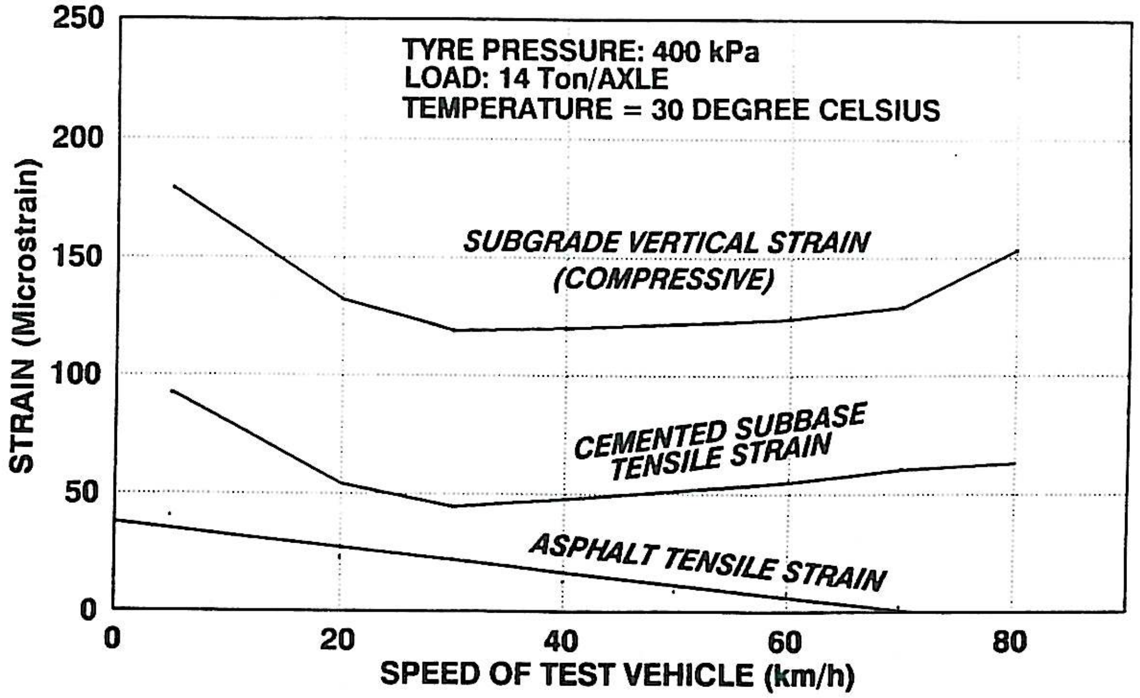


FIGURE 20

STRAIN WITHIN THE ASPHALT BASE PAVEMENT AS A FUNCTION OF SPEED

Conclusions and Recommendations

This paper describes a pavement response measuring system based on the multi-depth deflectometer (MDD) technology. Deflections at different depths on three types of heavy-duty pavement designs in South Africa were measured with this system, using a 42 ton tridem axle test vehicle (14 ton per axle).

The resulting deflection influence lines were well-defined, and enabled the back-calculation of layer moduli from which stresses and strains, and hence aspects of pavement structural "life", can be computed.

These deflection influence lines also enable direct comparison between different pavement types, and therefore enhance the understanding of effects such as speed and temperature on pavement response. In general, the effects of elastic and visco-elastic pavement materials are clearly identifiable, as well as the influence of the one on the other. The pure visco-elastic response of the asphalt layer is not totally reflected in the "banana-shaped" surface deflection influence lines because of the more elastic subbase and subgrade support conditions. (Compare Figure 5 and Figure 10). This study showed the relatively easy quantification of the effects of pavement temperature and vehicle (loading) speed on pavement response. From these measurements it is clear that a pavement's structural life is not a constant, but varies according to temperature and speed effects. More research, however, is needed to incorporate these effects into the design stage of pavements.

It is believed that this monitoring system has, in general, the potential for better correlations between field and laboratory behaviour of pavements and materials.

It is recommended that the following research areas receive further attention and investigation:

- the effect of the phase difference between the position of the moving load and the maximum deflection,
- dilation in asphalt layers, including the effects of load, speed and temperature
- the effects of vehicle speed and temperature on all the pavement layers, and
- dynamic characteristics (damping and inertia) of pavements.

ACKNOWLEDGEMENT

The Director of the Division of Roads and Transport Technology (DRTT) of the CSIR is thanked for permission to publish this paper.

APPENDIX. REFERENCES

De Beer, M (1986). Behaviour of cementitious subbase layers in bitumen base road structures. M Eng thesis, University of Pretoria, South Africa, 1986.

De Beer, M, Horak, E, Visser, A T (1988). The Multi-depth Deflectometer (MDD) system for determining the effective elastic moduli of pavement layers. Proceedings of the First International Symposium on Nondestructive Testing of Pavements and Back-calculation of Moduli, ASTM Publication STP 1026, ASTM Conference, Baltimore, Maryland, USA, 26 June to 1 July 1988, pp70-89.

De Beer, M (1989). Resilient response of pavements with lightly cementitious layers. Research Report DPVT 38, DRTT, CSIR, Pretoria, April, 1989.

De Beer, M (1990). Aspects of the design and behaviour of road structures incorporating lightly cementitous layers. PhD dissertation, University of Pretoria, South Africa, 1990.

Lourens, J P (1991). Nonlinear dynamic analysis and design of road pavements. Interim Report IR 90/030/1, South African Roads Board, RDAC, January 1991.

Ruth, B (1989). Course notes on asphalt technology, University of Stellenbosch, June 1989.

Scullion, T and Bush III, A J (1989). Use of the Multi-depth Deflectometer for deflection measurements. Paper intended for the "State of the art of pavement response monitoring systems for roads and airfields", U S Army Cold Regions Research Laboratory, Hanover, New Hampshire, March 1989.

APPENDIX F

MECHANISTIC PAVEMENT DESIGN USING A PENCEL PAVEMENT PRESSUREMETER (PSPP)

BY P J SANDERS

This paper was published in the Proceedings of the 10th Regional Conference for Africa on Soil Mechanics & Foundation Engineering and the 3rd International Conference on Tropical and Residual Soils, held in Maseru, Lesotho, 23 - 27 September 1991

MECHANISTIC PAVEMENT DESIGN WITH THE PENCEL SHEAR PAVEMENT PRESSUREMETER (PSPP)

SYNOPSIS

A new means of obtaining elastic moduli for pavement design is presented, the PENCEL Shear Pavement Pressuremeter (PSPP). The instrument is being developed from two existing test apparatuses (the Pencel pavement pressuremeter and the Handy Borehole Shear Tester) and at present yields values of elastic moduli within 10% of those derived from Heavy Vehicle Simulator (HVS) results at standard E80 axle loads. As the South African Mechanistic design procedure has largely been developed around HVS testing, moduli obtained from the pressuremeter are therefore taken as being suitable for use in mechanistic pavement design.

A method of obtaining *in situ* measures of shear strength (using the PSPP) in direct shear is being improved after initial results have indicated certain deficiencies in the prototype test apparatus.

The financial implications of using incorrect values of subgrade modulus are briefly mentioned and a comparison made between PSPP and HVS test results from a number of sites. These results are used to illustrate an iterative technique for predicting elastic moduli under traffic loading, using a multi-layer linear elastic computer program and equations derived from PSPP results. Test results from three other sites are also quoted.

(1) INTRODUCTION

The mechanistic pavement design method developed at the Division of Roads and Transport Technology (formerly NITRR) of the CSIR in Pretoria (Freeme, Maree and Viljoen, 1982)[1] has been used to good effect for the design of many pavements in the Republic. The method requires stresses and strains to be evaluated for each layer in order to predict pavement life (by applying limiting criteria to each particular case). To obtain these values, typically a multi-layer linear elastic program is used to analyse the proposed structure which requires relatively simple input data, such as proposed pavement layer thicknesses, wheel loads and elastic material properties (Poisson's ratio and elastic modulus). The effective elastic modulus E_e of the subgrade (the modulus relating to a dual wheel load of 40 kN - see de Beer *et al.*, 1988)[2] has a great influence on the prediction of stresses and strains within layers and, whereas typical values are available for use for most of the materials used in the upper pavement layers, little guidance has been available on suitable values for subgrades and selected materials. The inherent variability of *in situ* materials and the variation of their properties with, *inter alia*, moisture, stress and load repetitions makes it difficult to provide a designer with 'typical' values that will be adequate for use (see the following section for an indication of the effect of varying E_e on pavement life).

Due to the need for appropriate values of E_e for design of new pavements and the analysis of existing structures (for rehabilitation purposes), a relatively simple test method is being developed using an apparatus that is new to Southern Africa - the PENCEL Shear Pavement Pressuremeter (PSPP). The paper gives a brief introduction to the test method and a sample of results obtained from sites in Welkom, Bultfontein, Silverton and Standerton.

While it is appreciated that there are existing laboratory and field tests capable of measuring the elastic modulus of materials, e.g. the triaxial test and the plate-loading test, it is quite expensive to carry out the tests, and designers often do not have the funds or time available. The PSPP apparatus on the other hand offers the following features:

(a) It is relatively inexpensive, although giving relationships to predict the variation of elastic modulus with loads of different magnitudes, strain conditions and repeated loading.

(b) It requires only two people to carry out a test.

(c) It is quick (approximately one test per hour).

(d) It can be used *in situ* (for existing pavement assessment or quality control on new construction) or in the laboratory (for investigation of the relationship between density, strength and elasticity).

(e) Data analysis can be carried out on a pocket computer (although a PC Spread Sheet is an advantage).

(f) Tests are carried out on *in situ* material, hence testing representative material and avoiding the need for expensive and sometimes difficult sampling and laboratory test procedures to be carried out.

(g) As tests are relatively cheap they can be repeated at different times of the year should there be uncertainty over the behaviour of material. For instance, to finalise design, it may be necessary to repeat tests in the wet season if initial tests were carried out in dry conditions.

(2) THE IMPORTANCE OF SUBGRADE ELASTIC MODULUS TO PAVEMENT PERFORMANCE

To illustrate the effect that a variation in elastic modulus can have on the prediction of pavement life, a simple model of a road structure is used, shown in Figure 1, comprising (a) a thin surfacing, (b) two granular layers of approximately G2 quality (TRH14, 1985)[3] and (c) the subgrade. This structure is used as it represents the structure of a road shoulder in Welkom that was tested by the Heavy Vehicle Simulator (HVS) - Sanders (1990)[4] from which correlations of elastic moduli from the PSPP and the HVS have been carried out.

Typical gradings of materials tested in the road at Welkom are shown in Figure 2.

The HVS has been used to test roads for several years in South Africa and has contributed to a large degree to the formulation of the mechanistic design procedure used on the subcontinent. It is therefore reasoned that if elastic moduli derived from the PSPP agree with values obtained from back-calculation of HVS results, PSPP values are appropriate for pavement design or analysis.

The financial implications of using inappropriate values of elastic modulus for design or analysis can be great. For instance, if a road was built with the structure described above, the cost for 10 km would be approximately R2 million. Figure 3 indicates how pavement life may be reduced by 50% or more by using incorrect values of elastic modulus. This, in turn, would incur large rehabilitation or maintenance costs before they were planned in the pavement management system, hence reducing funds available for new works. A further implication therefore is that poor road design through the use of inappropriate elastic moduli can significantly affect municipal, regional and possibly national budgets.

(3) THE PENCEL SHEAR PAVEMENT PRESSUREMETER

A schematic diagram of the PENCEL Shear Pavement Pressuremeter is shown in Figure 4. The apparatus is being developed from the PENCEL Pavement Pressuremeter (Briaud and Shields, 1979)[5] and the test principle developed by Professor Handy for the Borehole Shear Tester (Handy and Fox, 1967)[6].

The measurement of material properties with the pressuremeter is discussed at length by Mair and Wood (1987)[7] and Baguelin *et al.* (1977)[8] so only relevant test principles are very briefly summarised here:

(a) The pressuremeter probe is placed at the desired depth by predrilling a hole or by self-boring,

(b) The probe is expanded into the cavity walls by applying an internal pressure by means of gas or hydraulic fluid,

(c) Measurements of applied pressure and the resulting volume changes of the probe are noted whilst following suitable loading and unloading cycles,

(d) After careful calibration of the test apparatus (to isolate the effects of system compliance and membrane resistance), elastic and plastic parameters can be derived from test data by the application of (*inter alia*) expanding cavity theory and Hooke's laws (Timoshenko and Goodier, 1934[9]).

Briaud and Cosentino (1990)[10] have recommended a test method whereby it is possible to derive equations predicting values of elastic modulus when subjected to variations in stress and strain and creep and cyclic loading.

The equations take the following form:

For stress: $E = K\,(\theta/Pa)^n$ . . . . (A)

Where θ is a stress function (normally taken as the average mean principal stress, but for these test results θ was taken as vertical stress - see Section 6 for comment).

Pa is atmospheric pressure (taken as 101.3 kPa)
and n and K are coefficients obtained from linear regression of a plot of log E vs log (θ/Pa)

For strain: $1/E = a + b\varepsilon$ - after Kondner (1963)[12] . . . . (B)

Where a and b are coefficients obtained from regression of a plot of 1/E vs (cavity strain) and ε is cavity strain.

Suggestions by Riggins (1981)[13] have been adopted to deal with the effects of time dependancy on elastic moduli (creep):

$$E_{t1}/E_{t2} = (t_1/t_2)^{-nr}$$

Where E_{t1}, E_{t2} are moduli calculated over time intervals t_1 and t_2 and nr is the rate coefficient.

The effects of repetitive loading on elastic modulus can also be quantified using a model proposed by Idress *et al.* (1978)[14]:

$$E_n/E_i = N^{-nc}$$

Where E_n and E_1 are secant moduli to the top of the N^{th} cycle and the 1^{st} cycle respectively, and nc is the cyclic exponent.

See Figure 5 for an illustration of a typical test curve indicating sections of the test used to investigate the influences on E_e mentioned above.

To date only results using the stress and strain models have been correlated with HVS test results.

To derive elastic moduli between any two points on a PSPP test curve, the equation suggested by Briaud and Cosentino (1990)[10] has been used:

$$E=(1+\mu)(\sigma_2-\sigma_1)\left[\frac{(1+\delta r_2/R_0)^2+(1+\delta r_1/R_0)^2}{1+\delta r_2/R_0)^2-(1+\delta_1/R_0)^2}\right]$$

where μ = Poisson's ratio,

δr_1 and δr_2 are increases in probe radius corresponding to the two points between which the modulus is to be determined,

Ro is the radius of the deflated probe,

and σ_1 and σ_2 are the pressures applied to the borehole walls for the points considered (see Figure 4).

(4) A PROPOSED DESIGN METHOD

As earlier stated, the objective of developing the PSPP is to provide a means of obtaining suitable values of elastic moduli for the mechanistic design of road and airfield pavements. However, on inspection of the technique of predicting E_e from PSPP test results, it is seen that the elastic moduli obtained from either equation (A) or (B) will depend on the values of stress or strain used in the formulae.

To determine appropriate values to be used in the formulae, the following technique is proposed:

(a) Prediction equations are derived from test results .

(b) A value of elastic modulus is chosen as a "seed" value - E_{es} (a typical unload cyclic modulus seems appropriate).

(c) The seed value is used with a multi-layer elastic pavement analysis program (e.g. CHEVRON or ELSYM - requiring only the computing power of a personal computer to be used).

(d) Stresses or strains are taken from the computer output and used with the equations obtained in (a) to predict $E_{e\ predicted}$.

(e) The value of $E_{e\ predicted}$ is compared with E_{es} and if not found to agree within, say 10%, a new E_{es} is chosen and the procedure repeated until acceptable agreement is found.

A schematic representation of the procedure is given in Figure 7.

(5) AN EXAMPLE OF TEST RESULTS

At the HVS test site 363A2 on the R30 in Welkom, two PSPP tests were carried out in a newly constructed road shoulder with the structure shown in Figure 1. Figures 5 and 6 give the field test curves obtained.

The following equations were derived from corrected field curves:

At 0.30m, $\log E_e = 2.9146 + 1.35(\theta/Pa)$

and at 0.75m, $\log E_e = 3.01876 + 1.35(\theta/Pa)$

Following the procedure suggested in (4), results of the iterations to obtain appropriate elastic moduli for use in mechanistic pavement design are shown in Figures 8 and 9. Superimposed on Figures 8 and 9 are the values of elastic moduli obtained from back analysis of HVS test results.

The following ratios of $E_{epredicted}/E_{eHVS}$ were obtained:

at 0.30m:- $E_{ep}/E_{eHVS} = 1.10$
and at 0.75m:- $E_{ep}/E_{eHVS} = 1.02$

NOTE: The use of $\sigma_v = \theta$ in equation (A) must be justified theoretically because, when deriving equation (A), θ is taken as

$\theta = 1/3\ (0.8\ \sigma_{radial} + \sigma_{overburden})$ - see Briaud and Cosentino (1990)[10], i.e. an average value of principal stresses.

No *in situ* direct shear tests were carried out at Welkom due to the apparatus being under development when the PSPP tests took place. However, a comparison of shear parameters obtained from the PSPP aparatus and laboratory direct shear tests has been done for a sandy material found at an HVS test site at Bultfontein, near Pretoria.

From initial tests using the apparatus, it was found (Sanders, 1990)[15] that the friction angle derived from the PSPP was approximately twice the magnitude of the friction angle obtained from the laboratory shear box. When the PSPP was examined, the discrepancy was found to be due to an incorrect value of shear being used, i.e. approximately double the area used in calculation was actually acting on the borehole walls. The underestimation of this area therefore increased calculated values of shear stress. This shortcoming of the test apparatus is being addressed and commissioning of the improved apparatus will begin shortly.

To illustrate the use of the PSPP on other pavement structures, test results are given in Figures 10, 11 and 12.

6) COMMENTS ON TEST RESULTS

6.1 Field Test Curves

The field test curves shown in Figures 5 and 6 demonstrate typical features of this type of pressuremeter test, namely, a region of little or no contact between the probe and borehole walls (from zero to 40 cc in Figure 5), followed by an increase in resistance to expansion. A relatively straight line relation between volume change and applied pressure in this zone indicates a relatively linear elastic material response.

Unload-reload cycles, the five-minute creep test and the final unloading sequence are all indicated in Figure 5 to clarify terminology.

6.2 Correlation of $E_{e\ predicted}$ to E_{HVS}

Figures 8 and 9 show how the iteration technique gives values of $E_{e\ predicted}$ that agree to within 10% of E_e derived from analysis of Heavy Vehicle Simulator results.

The iteration process is straightforward to use and, with careful examination of test curves to select suitable "seed" values of E_e for use in the computer analysis, only requires a few runs to find an acceptable answer (i.e. approximately one hour is sufficient once raw field data has been corrected).

6.3 General Comments

The close agreement between measured and predicted values of E_e indicate that for the material tested and within the stress ranges investigated, linear elastic theory seems appropriate.

To further improve the accuracy of results, future development work must concentrate on improving the insertion technique of the instrument to obtain a close initial fit between the deflated probe and borehole walls. This in turn will give more accurate readings at low stresses and hence coefficients for prediction equations that relate more closely to stress regimes experienced under wheel loads (normally between 10 and 500 kPa).

Theoretical justification is required for the successful use of $\theta = \sigma_{vertical}$ when predicitng E, as opposed to taking θ as the average principal stress when deriving equations.

Figures 10 to 12 indicate that the PSPP is an effective tool for use in measuring elastic moduli over a range of materials. The apparently close agreement between PSPP, IDM (Falling Weight Deflectometer) and DCP-derived moduli holds great promise for more effective use of the materials for investigation and quality control.

<u>NOTE:</u> To obtain accurate results from the PSPP test, careful calibration of the test apparatus is essential. In addition to this, loading sequences must be adjusted to suit materials tested during each test.

(7) <u>FURTHER USE OF THE PSPP</u>

In addition to using the PSPP to investigate existing road pavements for rehabilitation design applications, the probe can be used to investigate the properties of materials proposed for use in construction. To do this, materials should be compacted at different densities (corresponding to a range of expected 'as built' densities) in moulds large enough to minimise confining effects. PSPP tests can then be carried out and a dynamic cone penetrometer (DCP) used in the same material. This provides a correlation between DCP penetration rates, material density and effective elastic moduli. Using the correlations obtained, the DCP can then be used more effectively as a quality control tool on site.

A smaller probe is at present being constructed at the CSIR which will allow higher pressures to be used and provide a means of measuring *in situ* parameters of pavement materials constructed in 150 mm thick layers. In addition to this, the new probe will be small enough to be used horizontally in the walls of large diameter boreholes, i.e. measuring values of elastic moduli in the vertical direction. These values should be more appropriate than horizontal moduli for many geotechnical engineering purposes, especially where properties of layered materials alter rapidly with depth.

(8) <u>CONCLUDING COMMENTS</u>

A new approach giving a means of obtaining representative values of elastic moduli has been suggested using the PENCEL Shear Pencel Pressuremeter. The apparatus consists of a simple monocel pressuremeter probe (developed by Professors J-L Briaud and D H Shields in North America) and a shear- teeth attachment that enables direct *in situ* measurement of shear strength parameters.

The test technique is relatively straightforward and follows recommendations by Briaud and Cosentino (1990)[10], yielding values of elastic moduli (E) and equations that predict the effects of stress, strain, creep and cyclic loading on E.

Test results carried out in granular materials in a road at Welkom have been presented and compared with Heavy Vehicle Simulator test results. Close agreement between test results is observed (i.e. within 10%).

The implication of the close agreement between test results is that a road designer can quickly obtain valid measures of elastic moduli with little expense for use in mechanistic pavement design. A further implication of the work is that with more accurate measures of subgrade modulus being used in design and analysis, prediction of pavement life is made more reliable, hence allowing better financial planning and pavement management.

9. REFERENCES

1. Freeme, C R, Maree, J H and Viljoen, A W (1982). Mechanistic design of asphalt pavements and verification of designs using the Heavy Vehicle Simulator. Proc. 5th Conf. on the Structural Design of Asphalt Pavements, Vol. I, pp 156-173, Delft 1982.

2. De Beer, M, Horak, E and Visser A T (1988). The multi-depth deflectometer (MDD) system for determining the effective elastic moduli of pavement layers. ASTM Special Technical Publication (STP) 1026.

3. TRH14 (1985). Technical Guidelines for Road Construction Materials. Division of Roads and Transport Technology, CSIR, Pretoria.

4. Sanders, P J (1989). Technical Note: 1/FP/16/90, PET, DRTT, CSIR, Pretoria.

5. Briaud, J-L, and Shields, D H (1979). A Special Pressuremeter and Pressuremeter Test for Pavement Evaluation and Design. Geotechnical Testing Journal, GTJODJ, Vol.2, No.3, September 1979.

6. Handy, R L and Fox, N S. (1967). A Soil Borehole Direct Shear Test Device. Highway Research News, No.27, Spring 1967, USA.

7. Mair, R J and Wood, D M (1987). PRESSUREMETER TESTING methods and interpretation. CIRIA Ground Engineering Report: In-situ Testing. Butterworths, London, 1987.

8. Baguelin, F, Jezequel, J F and Shields, D H (1978). The pressuremeter and foundation engineering. Book 4 in Trans. Tech. Publications Series on Rock and Soil Mechanics, 1st Edition 1977.

9. Timoshenko, S P and Goodier, J N (1934). Theory of elasticity. McGraw Hill, 1934.

10. Briaud, J -L and Cosentino, P J (1990). Pavement Design with the Pavement Pressuremeter. Proc. 3rd International Conference on Pressuremeters, University of Oxford, England, April 1990.

11. Duncan, J M and Chang, C -Y (1970). Non-linear analysis of stress and strain in soils. Journal of the Soil Mechanics and Foundation Engineering Division, ASCE, Vol. 96, SM5, 1970.

12. Kondner, R L (1963). Hyperbolic stress-strain response: Cohesive soils. Journal of the Soil Mechanics and Foundation Engineering Division, ASCE, Vol. 89, SM1, 1963.

13. Riggins, M (1981). Visco-elastic characteristics of marine sediments in large scale simple shear. Ph.D dissertation, Civil Engineering, Texas A & M University, 1981.

14. Idriss, I M, Dobry, R and Sings, R.D. (1978). Non-linear behaviour of soft clays during cyclic loading. Journal of the Geotechnical Engineering Division, ASCE, Col. 104, GT12, 1978.

15. Sanders, P J (1990). A Preliminary Investigation into the Determination of the Elastic Moduli of Road Materials at Bultfontein using the Pencil Shear Pavement Pressuremeter. Report No. DPVT/C 168.1, PET, DRTT, CSIR, Pretoria.

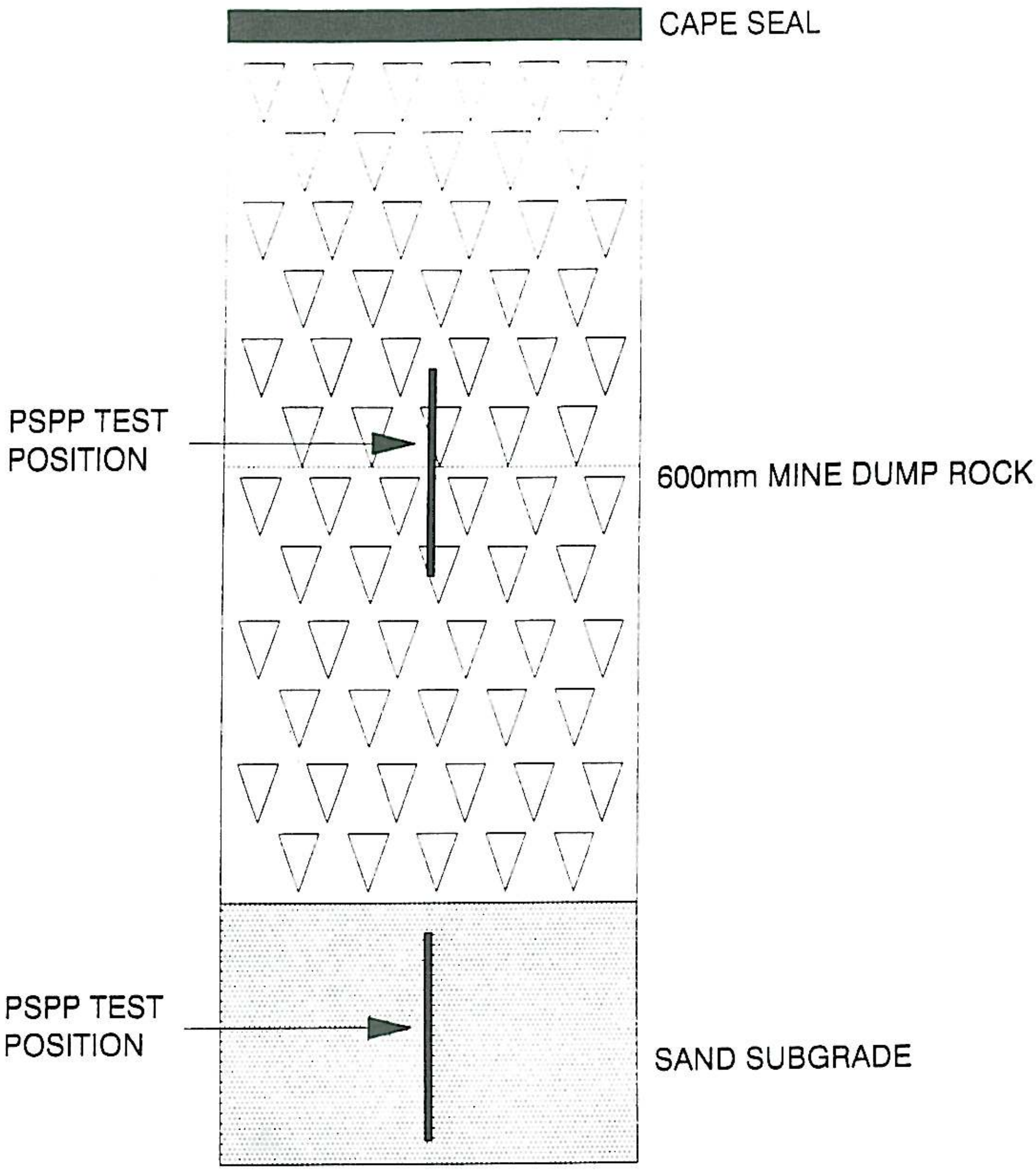


FIGURE 1

PAVEMENT STRUCTURE TESTED BY HVS AT WELKOM R30 kM 12.55

363STRU.DRW

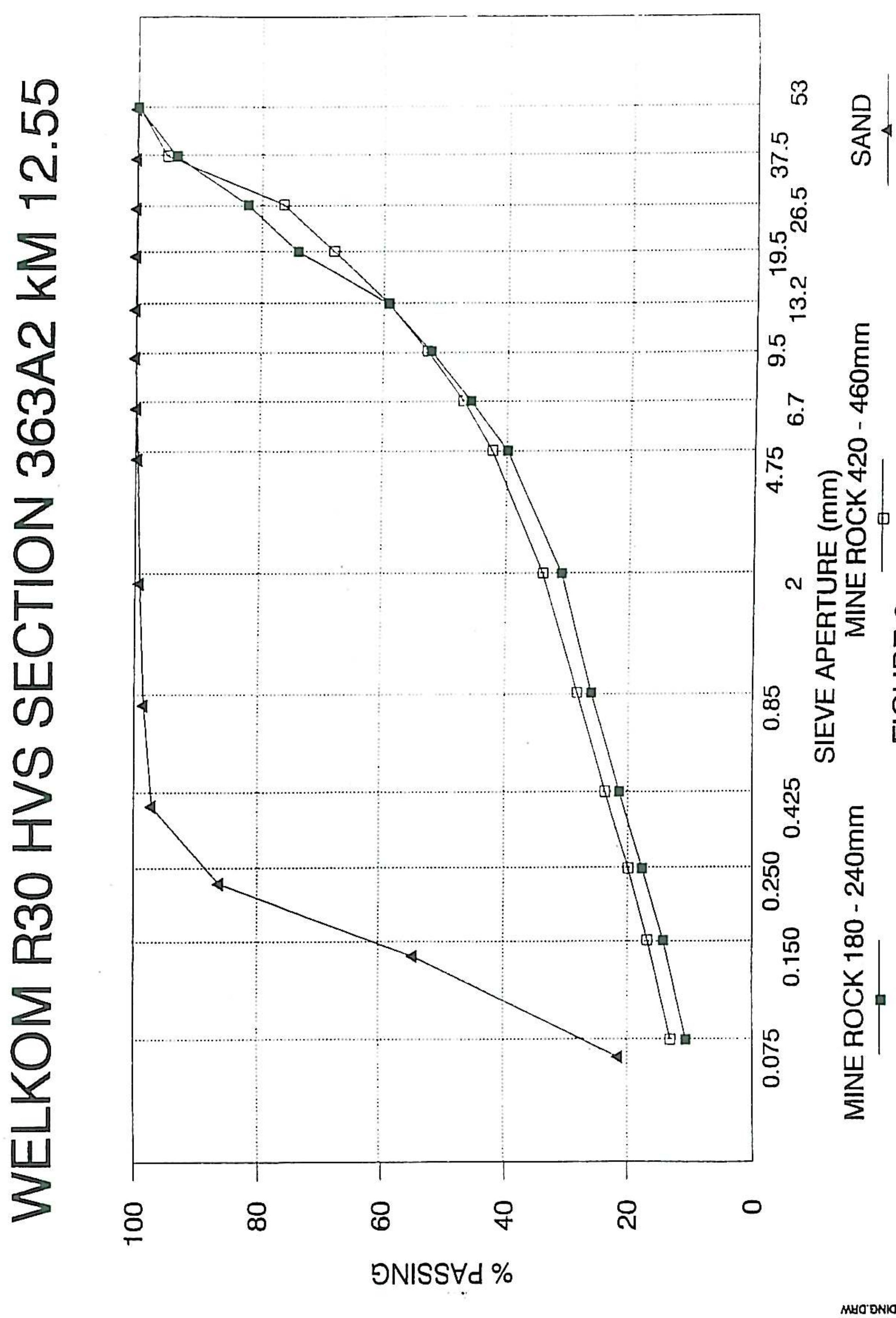


FIGURE 2
GRADING ANALYSIS

GRADING.DRW

WELKOM R30 HVS SECTION 363A2 kM 12.55

MINE ROCK LAYER THICKNESSES = 225mm

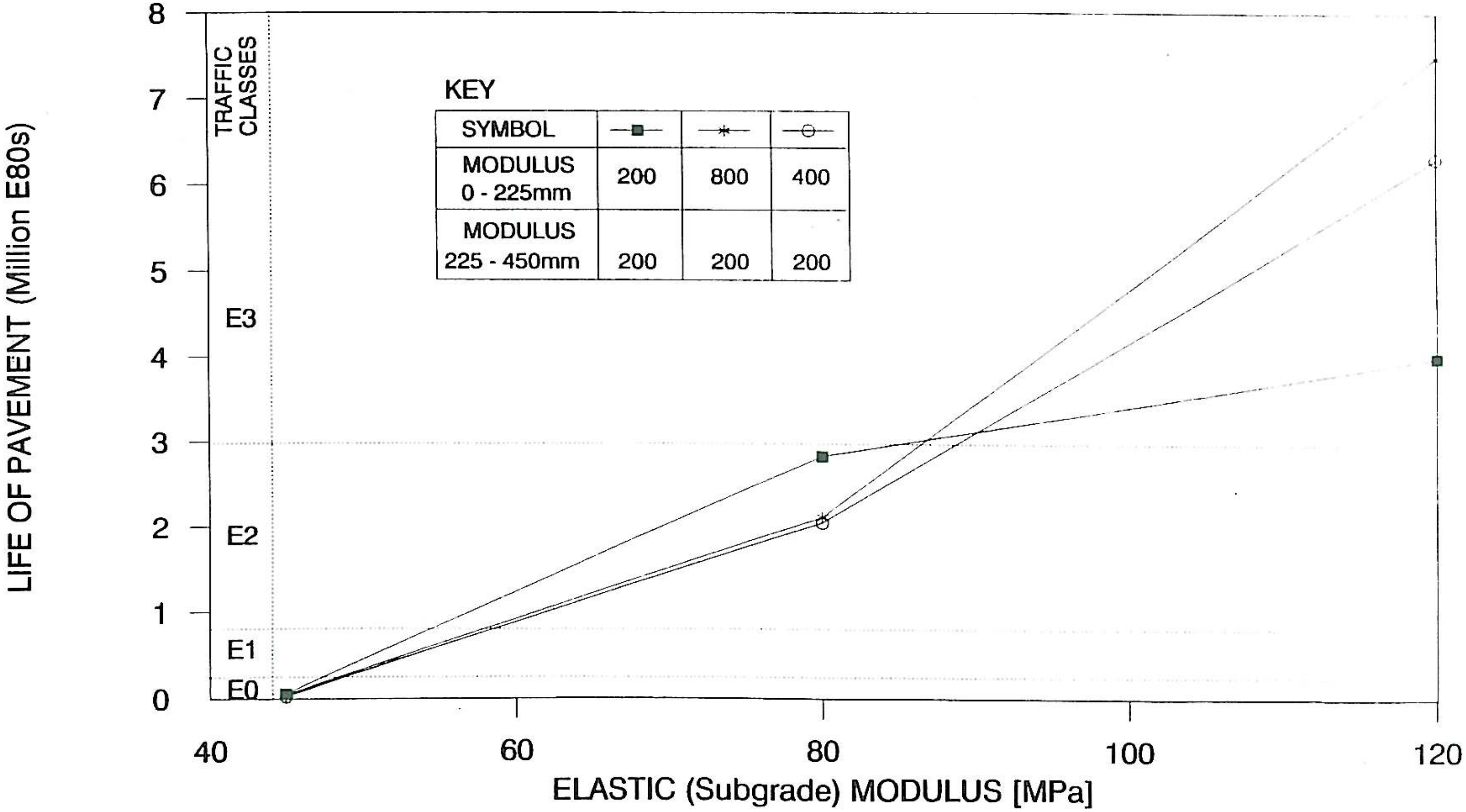
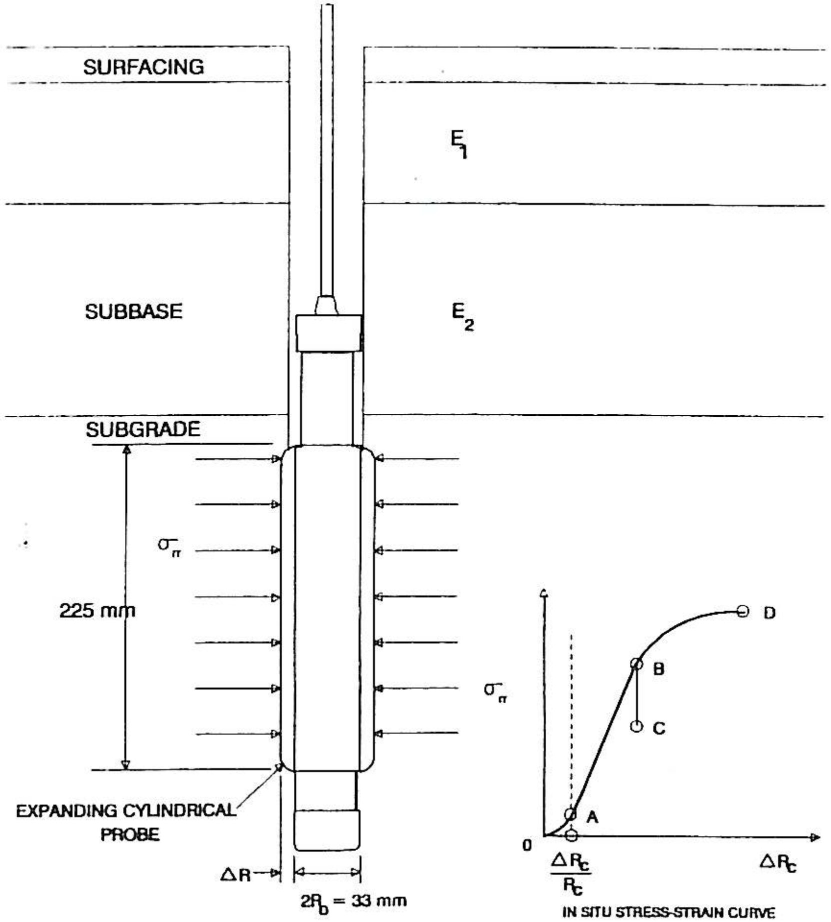


FIGURE 3
EFFECT OF VARYING $E_{subgrade}$ ON PAVEMENT LIFE

FOS-WET.DRW



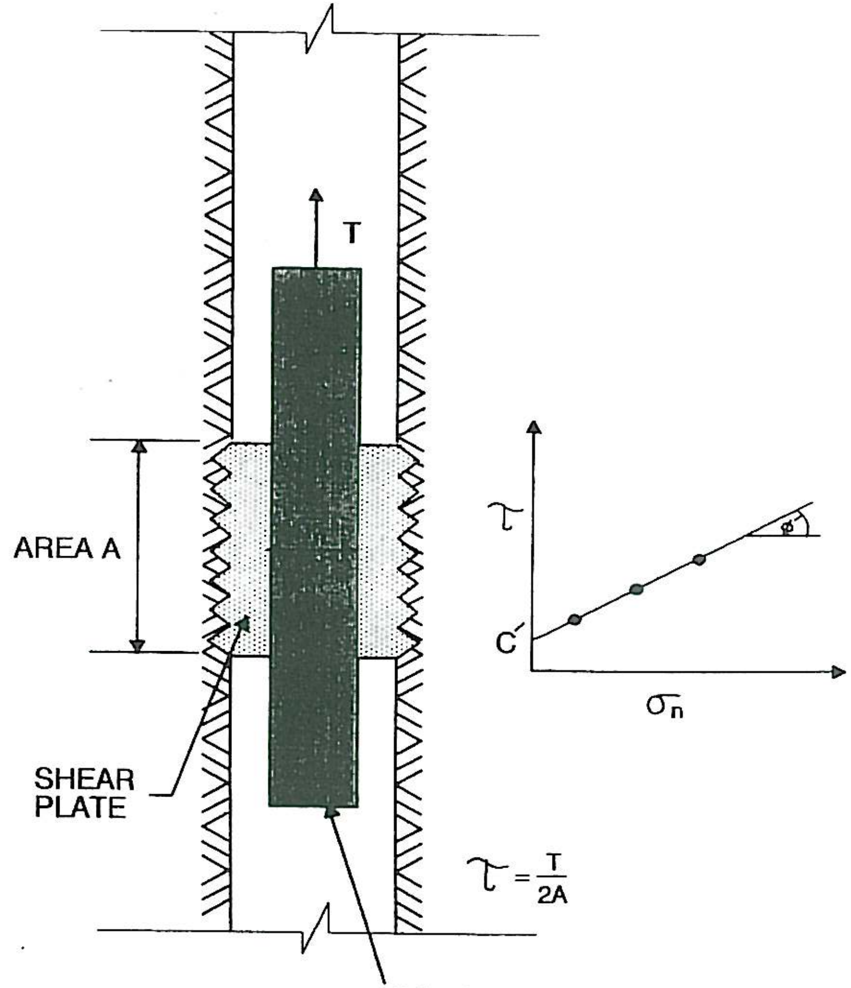


FIGURE 4

TYPICAL PAVEMENT PRESSUREMETER TEST
AND PAVEMENT CROSS SECTION
(AFTER BRIAUD AND COSENTINO, 1990)

PENCEL - SHEAR APPARATUS

PENOH1.DRW

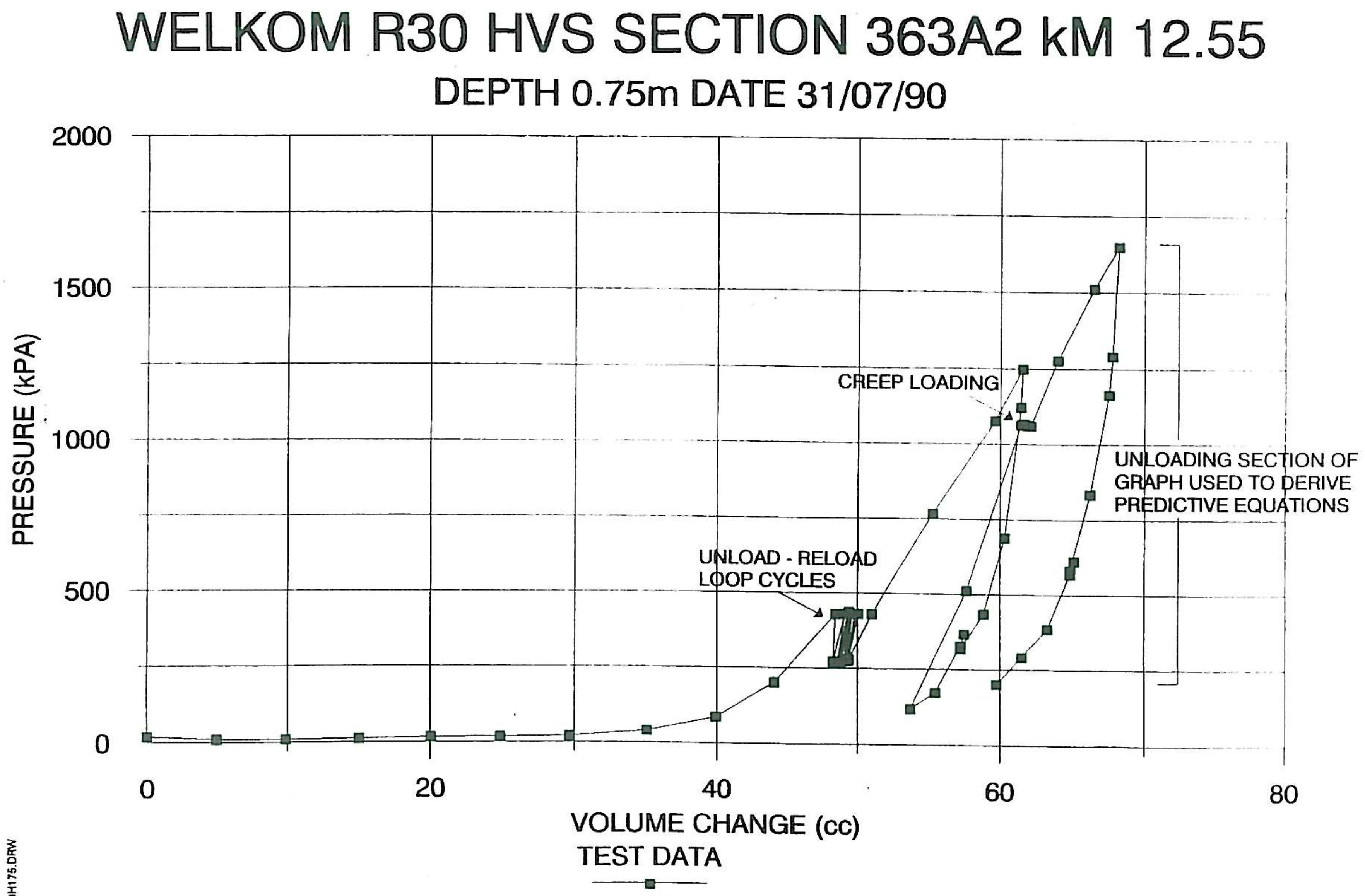


FIGURE 5
CORRECTED PRESSUREMETER CURVE

WR30H175.DRW

WELKOM R30 HVS SECTION 363A2 kM 12.55

DEPTH 0.30m DATE 31/07/90

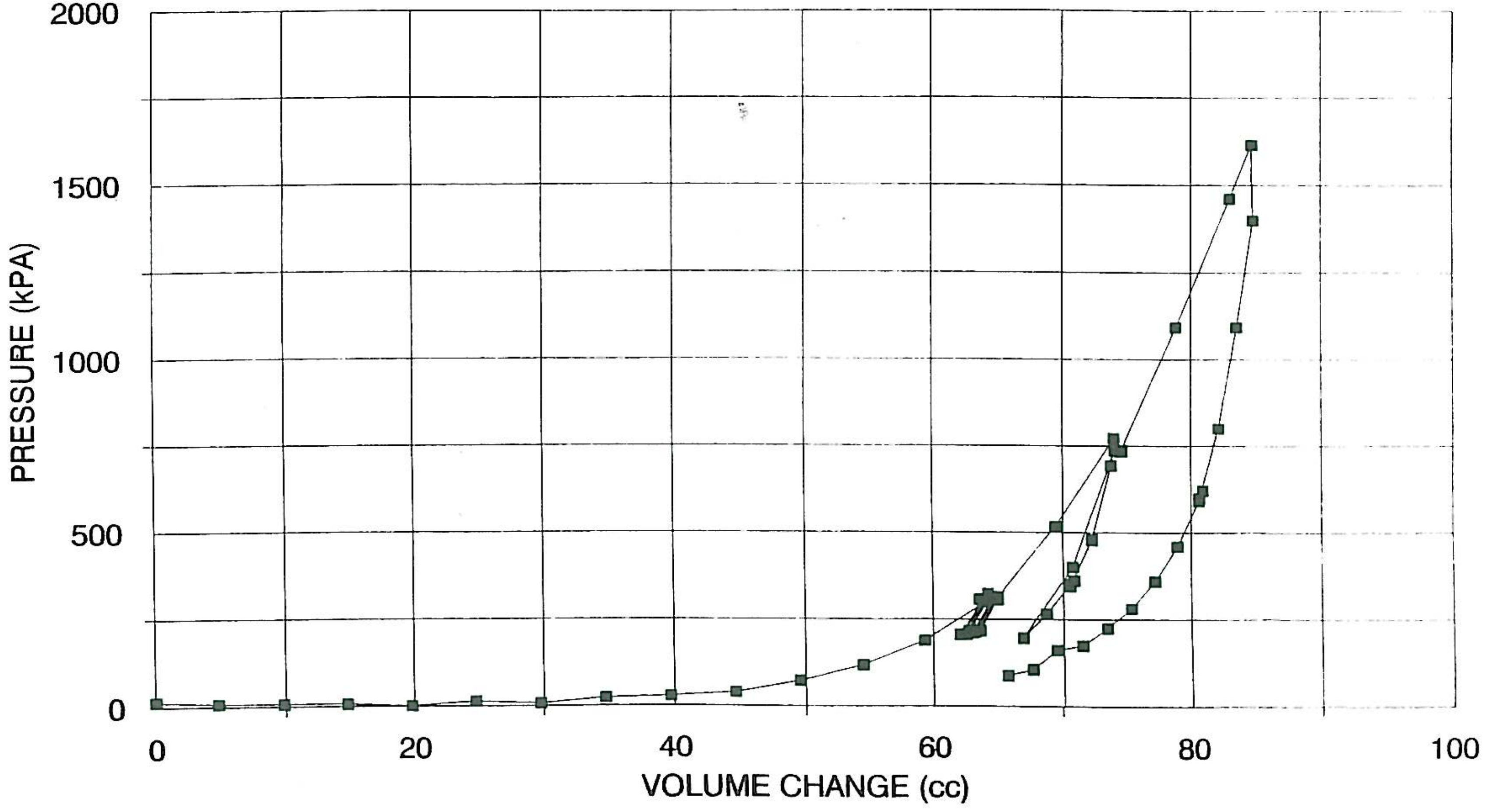
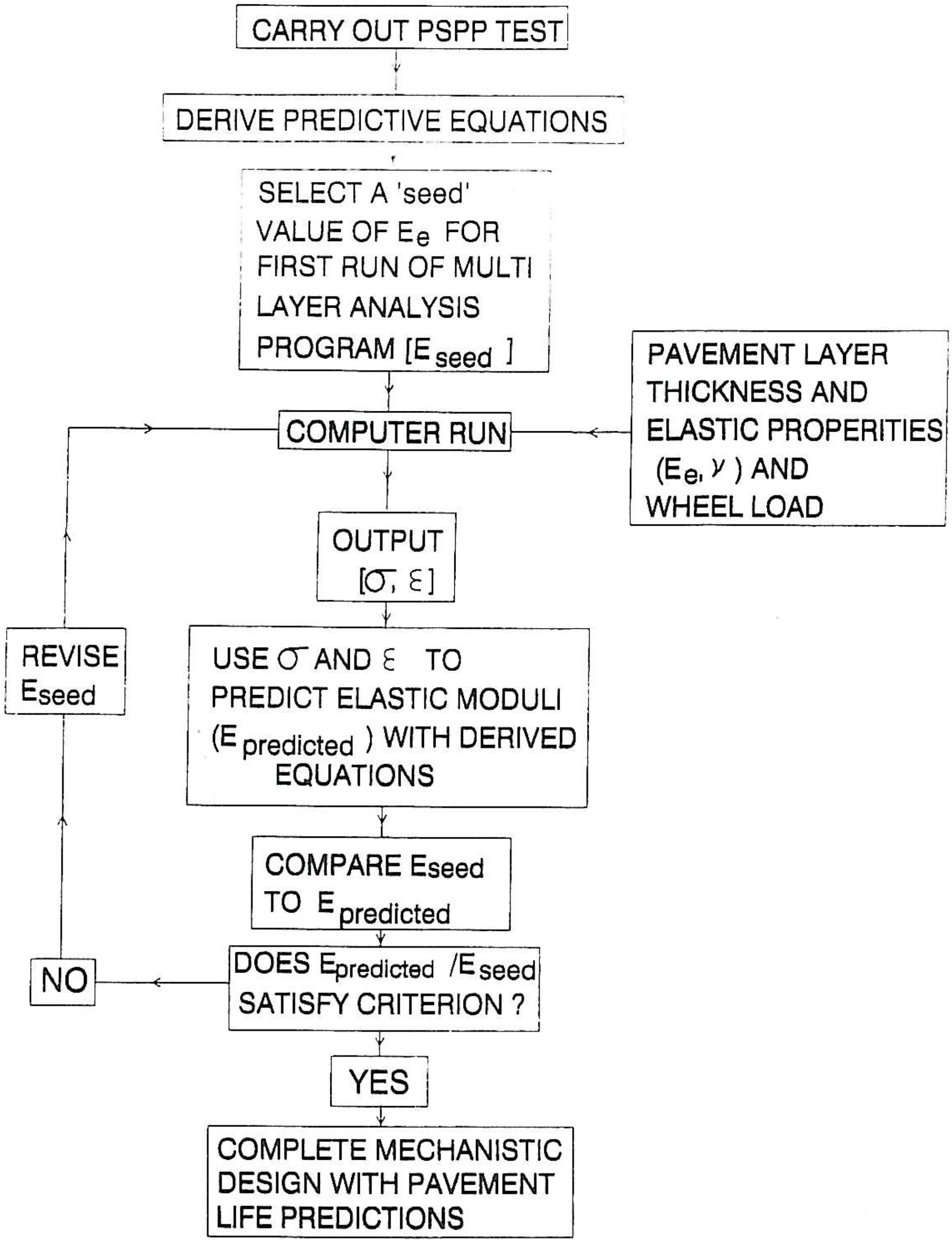


FIGURE 6

CORRECTED PRESSUREMETER CURVE

WR30H230 DRW



NOTE : IF PAVEMENT LIFE PREDICTION IS UNSATISFACTORY OR THE STRUCTURE APPEARS TOO EXPENSIVE, FURTHER ADJUSTMENTS TO LAYERS AND MATERIALS CAN CARRIED OUT AND THE ANALYSIS REPEATED.

FLOW-CH DRW

FIGURE 7

ITERATIVE DESIGN PROCEDURE

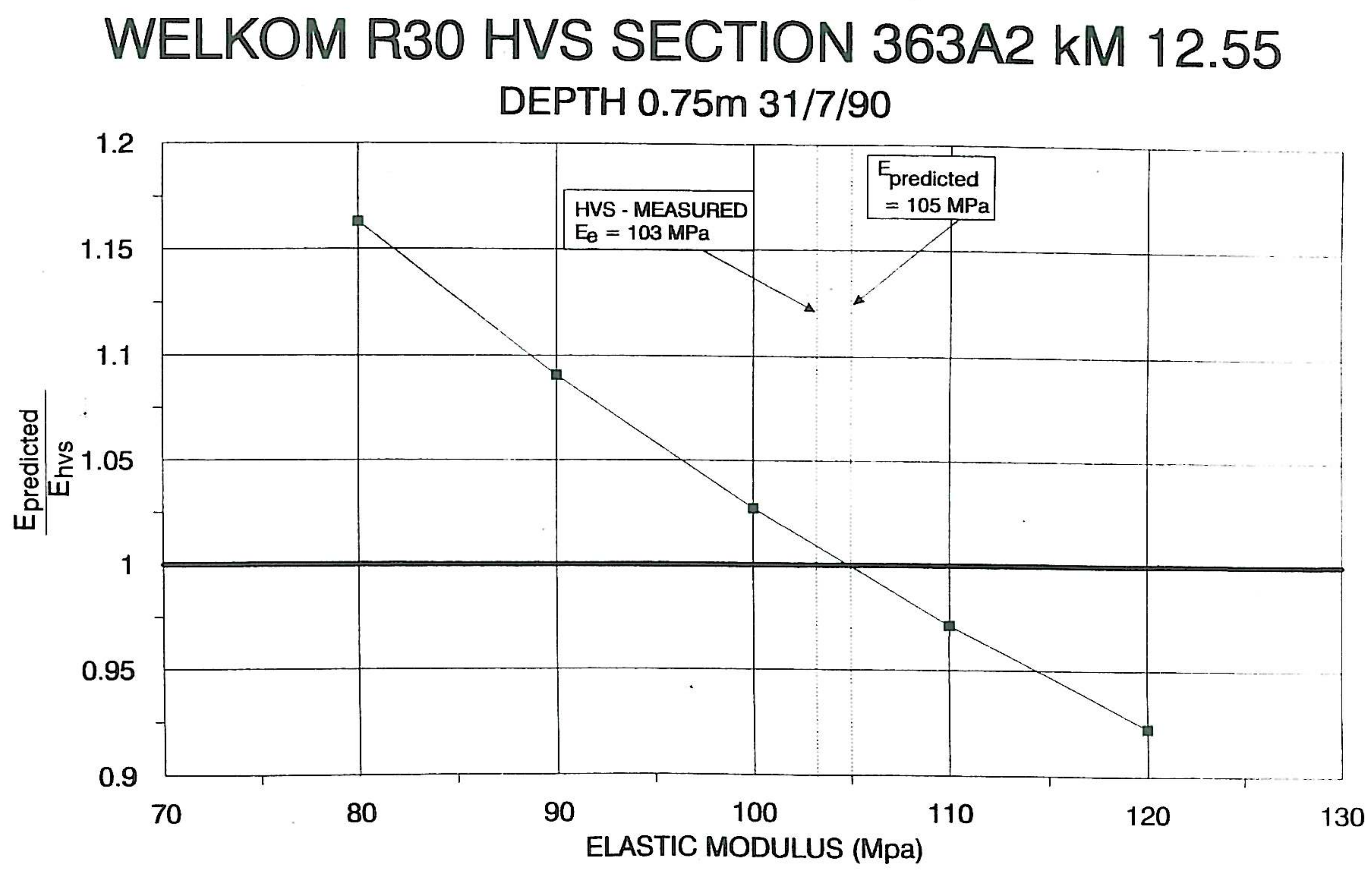


FIGURE 8
DETERMINATION OF E_e BY ITERATION.

E80-SAND.DRW

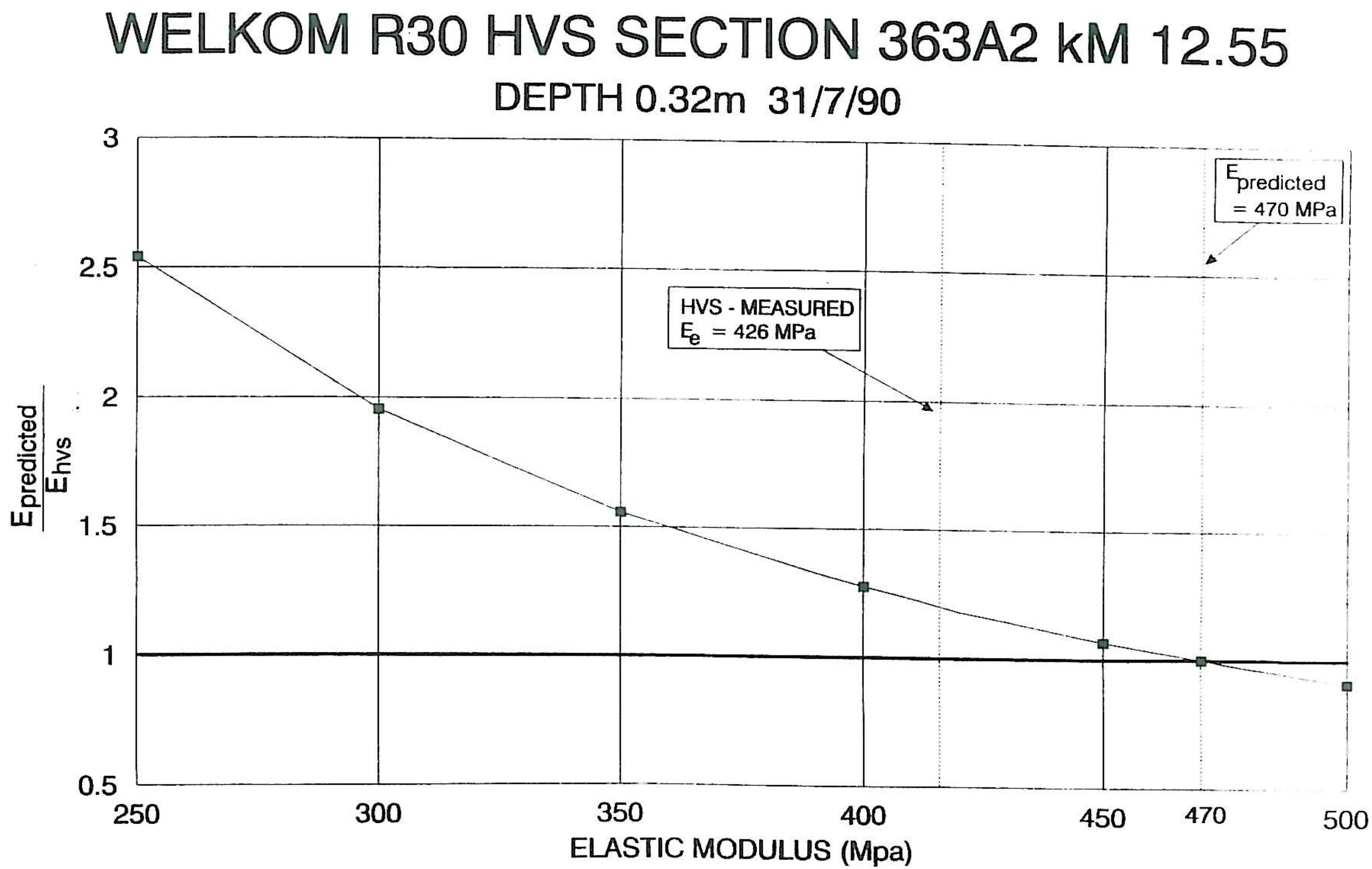


FIGURE 9
DETERMINATION OF E_e BY ITERATION.

E80-MINE.DRW

E(PENCEL)vsE(HVS)-BULTFONTEIN

CORRELATION OF E(PSPP) AND E(HVS) WITH TRAFFIC LOADING

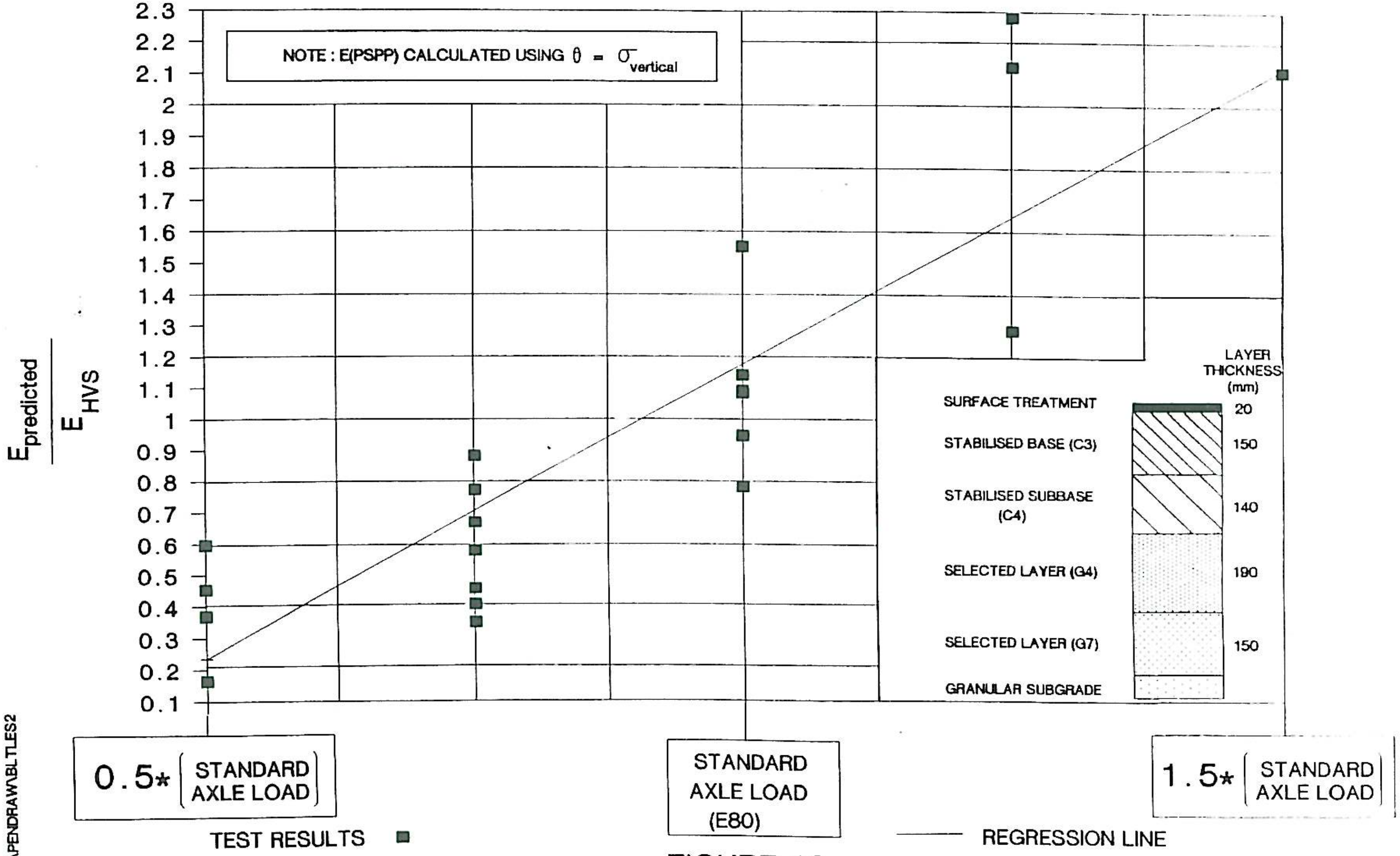


FIGURE 10

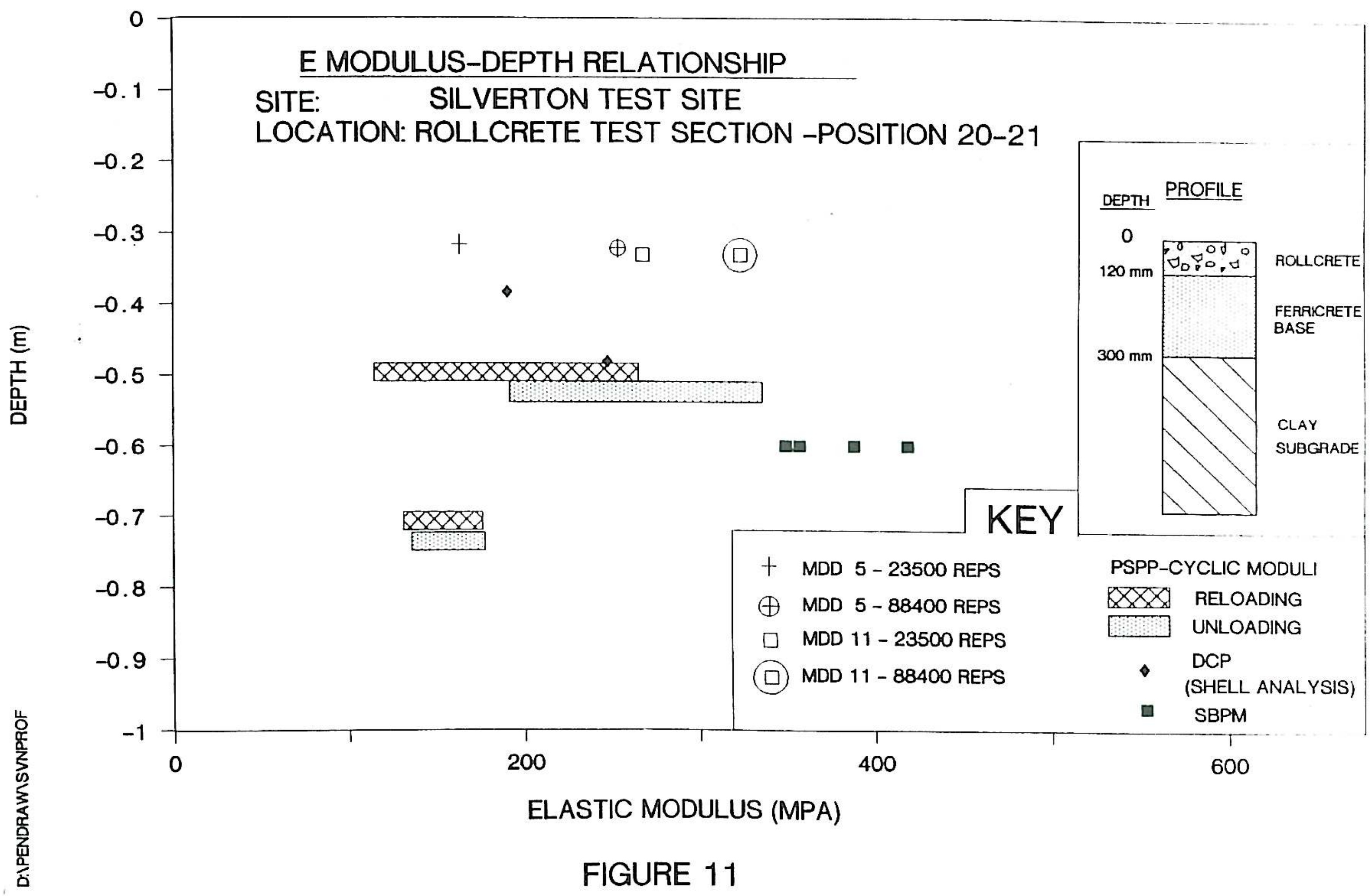

E MODULUS-DEPTH RELATIONSHIP
SITE: SILVERTON TEST SITE
LOCATION: ROLLCRETE TEST SECTION -POSITION 20-21
DEPTH (m)
0
-0.1
-0.2
-0.3
-0.4
-0.5
-0.6
-0.7
-0.8
-0.9
-1
0
200
400
600
ELASTIC MODULUS (MPA)
DEPTH
PROFILE
0
120 mm
300 mm
ROLLCRETE
FERRICRETE BASE
CLAY SUBGRADE
KEY
MDD 5 - 23500 REPS
MDD 5 - 88400 REPS
MDD 11 - 23500 REPS
MDD 11 - 88400 REPS
PSPP-CYCLIC MODULI
RELOADING
UNLOADING
DCP
(SHELL ANALYSIS)
SBPM
D:\PENDRAW\SVNPROF

FIGURE 11

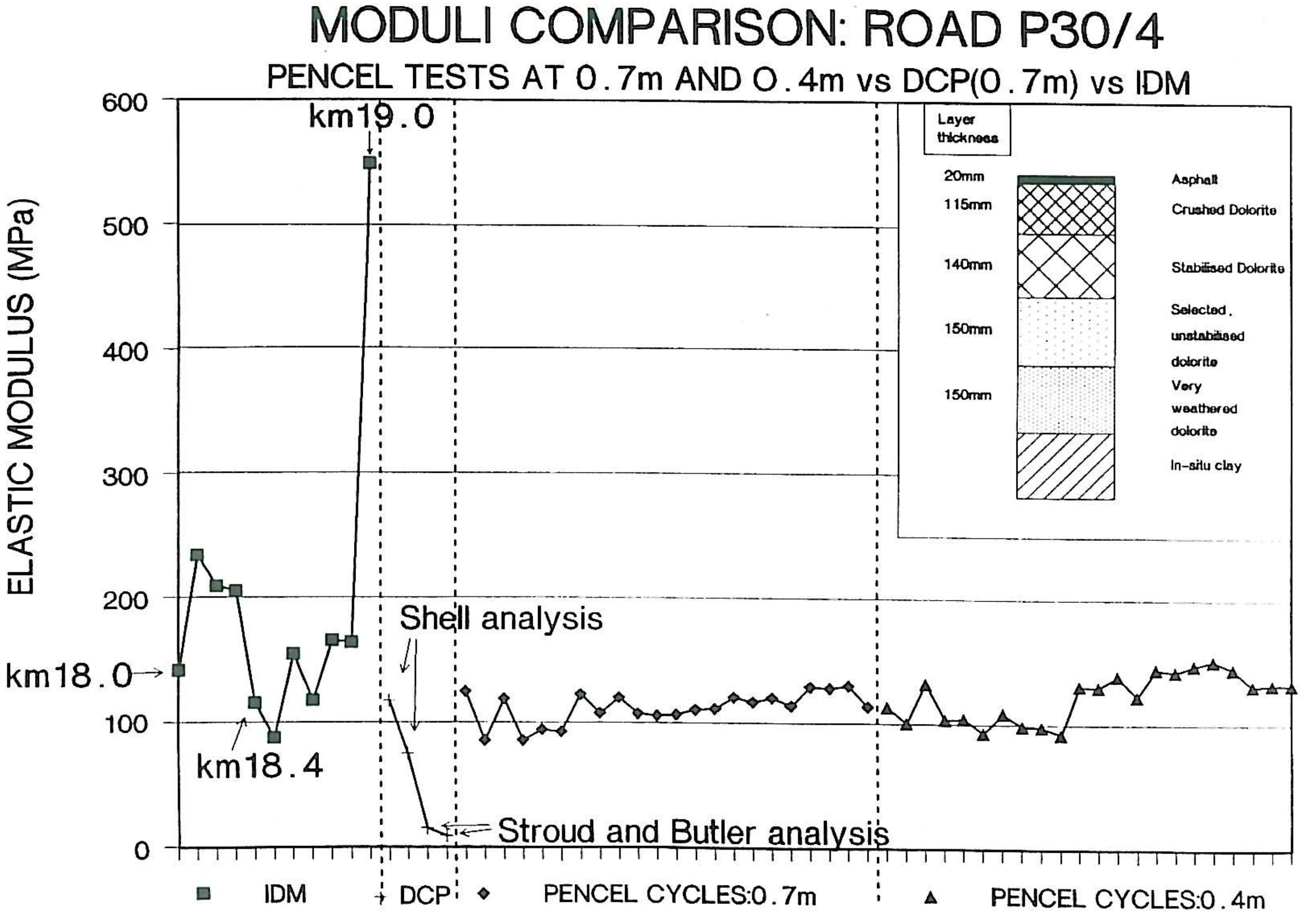


FIGURE 12

d:\pendraw\std-les

APPENDIX G

BACKCALCULATION OF EFFECTIVE ELASTIC MODULI OF PAVEMENT MATERIALS

BY P J SANDERS, M DE BEER AND J PROZZI

This paper has been accepted for publication in the Proceedings of the Annual Transportation Convention to be held at the University of Pretoria, June 1992

BACK-CALCULATION OF EFFECTIVE ELASTIC MODULI OF PAVEMENT MATERIALS

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SYNOPSIS

A perennial problem for road designers using the SA mechanistic design approach is that of the selection of appropriate elastic moduli for pavement materials. A selection of techniques to resolve this problem are briefly discussed and the results of their use on a particular structure compared. Finally, tentative recommendations are made regarding moduli determination for pavement design.

OPSOMMING

'n Deurlopende probleem vir plaveiselontwerpers wat die SA Meganiestiese ontwerpmetode gebruik, is die keuse van geskikte elastisiteitsmoduli vir die plaveiselmateriale. Hierdie referaat bespreek 'n keuse van sekere tegnieke om die probleem op te los, en die resultate word vergelyk op 'n spesifieke struktuur. Voorlopige aanbevelings rakende moduli-bepaling word ook gemaak.

INTRODUCTION

With today's tight financial constraints, provincial and national road authorities concentrate more on the rehabilitation of existing in-service pavements than previously. The Road Needs Study, which was reported on during ATC'9[1], concluded that definite needs exist for the surfacing of roads in the developing areas and for the maintenance and rehabilitation of the existing road network. There are various approaches that can be used for pavement rehabilitation, ranging from a seal overlay holding action to major rehabilitation (or reconstruction). These approaches include the SHELL overlay method[2], the Dynamic Cone Penetrometer (DCP) method[3], the South African mechanistic design approach[4] and the TRH4 catalogue design method[5].

If the South African mechanistic design process is used for rehabilitating a pavement, various factors may influence the rehabilitation design decisions. These are the expected future traffic, available materials, the type of pavement to be rehabilitated, remaining (residual) structural capacity and, obviously, the overall economic viability of any particular option.

One of the most important steps in the rehabilitation design process is to establish the residual life of an existing pavement. Once this has been done, appropriate measures can be put into effect to achieve the desired pavement life and serviceability. To infer the life of a pavement from test results is, however, not always straightforward and requires special attention.

This paper deals with some aspects of "back-calculation" of elastic moduli from in-situ test measurements. Normally these values are compared to published values or independent measurements from instruments like the Dynamic Cone Penetrometer (DCP)[6], the K-mould[7] or triaxial test apparatus[8], for example. Some of the methods to obtain moduli of the various pavement layers are discussed and compared to DCP and published values. The potential influence of using inappropriate moduli in pavement life predictions is also mentioned in the paper. Lastly, tentative recommendations for the establishment of design moduli are made for South African conditions.

WHY BACK-CALCULATE ELASTIC MODULI?

The answer to this question can be divided in two parts: firstly, why do we need elastic moduli, and secondly, why use the back-calculation approach to derive this parameter?

Elastic moduli are typically required for mechanistic design techniques such as the SA mechanistic design procedure to provide an input into setting up a mathematical material model that can be used for calculation of stresses, strains and deflections (i.e. response parameters) in layered systems. These, in turn, are used to predict pavement life, through the use of suitable transfer functions[9]. It follows therefore that calculated stresses and strains depend on the type of material model used (eg non-linear or linear elastic), as well as input data like load configuration, for example. Furthermore, it is very important that stresses and strains used as input to the transfer functions are compatible with those used to derive these functions. In this respect linear elastic theory compatible with the current SA Mechanistic Design method should be used.

There are various reasons for using the back-calculation approach. However, in short, it could be argued that back-calculating, or extracting material parameters from the deflection response under load of an existing pavement structure, is one of the most fundamental and accurate approaches possible, depending on the applicability of techniques used. This is so because the response parameter (usually deflection) reflects the total pavement response which automatically takes into consideration temperature, moisture and other environmental factors at the time of measurement.

If, on the other hand, materials are sampled from pavement structures and tested in the laboratory, there is often a problem in reconstituting specimens to in-situ (undisturbed) conditions and then testing them in a manner simulating the field environment. This may not be very significant for some materials (like asphalt perhaps) but for untreated materials, quite different material behaviour may result with relatively small changes in moisture and density. It is therefore necessary to measure the in-situ conditions from which the specimens were taken accurately. This in itself can be a difficult process. Besides these difficulties there is also the problem of applying the measured data to a model that can reliably and realistically simulate pavement behaviour.

However, if material and construction data are available and the traffic and environmental history known, representative parameters can be deduced from measurements of the present condition of the road. Intuitively it follows that there can be no better way of assessing a pavement structure than to measure full-scale behaviour under real loading conditions. Typically in South Africa, Falling Weight Deflectometer (FWD), or Impulse Deflectometer (IDM)[10], La Croix Deflectograph[11] or the Multi-Depth (MDD) deflections are used for back-calculation[12] purposes.

METHODS OF BACK-CALCULATION

There are various methods of back-calculating elastic moduli from deflection data, some carry out 'automatic' deflection fits, whereas others rely on manual iteration. Some of the techniques are now briefly described. It should be noted that these are all static simulations of wheel loads and do not take into account dynamic considerations, such as damping and inertia, for instance.

MANUAL ITERATION TECHNIQUES

Chevron[13]

The Chevron series of program codes are of the oldest and are relatively well known and well used. The code is based on a linear elastic material model with the multi-layer theory. The program was initially developed for mainframe applications, although versions are available that run on Personal Computers (PC's). An altered version of CHEV is available at the Division of Roads and Transport Technology at the CSIR to back-calculate elastic moduli from in-depth maximum deflection measurements. At present the program uses maximum deflection measurements only to calculate moduli, but will be modified in time to fit full deflection basins at depth.

Mich-Pave[14]

This program uses a novel technique to calculate stresses, strains and deflections in pavement layers. A combination of Finite Elements and multi-layer linear elastic theory is used in a package that runs on a PC.

The program code was developed to improve computational efficiency, especially where non-linear material models are used. It consists of a Finite Element (FE) mesh to a depth of approximately 1,27 m over a flexible boundary. This, in turn, overlies linear elastic layers.

The FE model allows the use of linear or non-linear material models and the mesh is placed close to the wheel load (where stresses are relatively high) as this is where non-linearity is more significant. Subgrade non-linearity is not taken into consideration, but generally stresses below the flexible boundary fall within the linear zone or are only slightly non-linear.

The Mich-Pave code can be used to back-calculate linear and non-linear material properties manually, but can be very time consuming, especially when non-linear material parameters[15,16,30] are being determined.

Automatic Iteration

There are a number of computer program codes available that automatically fit full deflection basins. Three are now briefly described. One uses partially non-linear elastic equivalent layer theory (i.e. the incorporation of non-linear subgrades) and the others, linear elastic multi-layer theory.

Elmod[17]

This code has been developed for use with IDM deflection data and is based on the "equivalent layer theory[18]" (ELT). ELMOD has a limitation of 4 layers and some constraints such as the combination of all asphalt layers and layers less than 100 m thick which are to be combined with those adjacent.

The main principle of ELT is that it converts multi-layer structures into a succession of single layer structures on which Boussinesq's theory[19] can be applied to calculate deflections. The main advantage of ELT is that calculations are quick and can be carried out on a PC, or even on a Lotus spreadsheet.

WESDEF[20]

WESDEF is a multi-layered linear elastic program that can take up to 20 loads and 5 layers with varying friction coefficients on layer interfaces. The program was developed to give the PC user an efficient and accurate means of back-calculating pavement layer moduli from measured surface deflections. Convergence has been optimised for speed and accuracy by parametric analysis and correlation to numerical integration steps.

Padal[21]

PADAL uses multi-layer linear elastic theory (BISTRO[22]) for pavement layers in conjunction with a non-linear subgrade assumption to calculate deflections. These are then matched with measured deflection basin. Tam and Brown[23] found that PADAL could be used 'with confidence' in practical situations but obtained poor correlation with ELMOD for subbases in three layered structures.

COMPARISON OF TECHNIQUES USED TO OBTAIN ELASTIC MODULI

An example of the comparison of the results of different approaches to determine moduli is given in Table 1. IDM data obtained from a cement stabilised pavement near Pretoria (Road 2212), has been analysed with WESDEF, Equivalent Layer Theory (ELT) and CHEV15F. Recommended ranges of moduli from the document "Evaluation of pavement behaviour for major rehabilitation of roads" [4] and values obtained from the DCP[24] are also given.

Table 1: Comparison of moduli derived by different techniques. (Values in MPa)

DCP-derived values	CHEV15F - derived moduli	First Approximation from Equivalent layer theory	Ranges of values from RP/19/83[4]	WESDEF-derived values	Material Type[20]
1838	1200	985	2000-3000	2500	Asphalt*
845	700	2018	1500-2000	630	C3
223	450	258	1000-2000	378	C4
186	52	48	225	46	G5
131	130	173	200	317	G5
111	300	348	200	317	G6

* The asphalt surfacing was medium graded with a binder content of 5,5 %, a voids content of 5,8 % and binder penetration of 34 at 25° C.

Estimates of pavement life can be predicted from stresses and strains calculated from, for example, CHEV15, using the presently available transfer functions[4] (functions converting stresses and strains to pavement life). Using the sets of moduli in Table 1, pavement lives have been predicted and are given in Table 2.

Table 2: Predictions of pavement life

Source of moduli used for calculations:	DCP-derived values	CHEVRON-derived moduli	Approx. from Equivalent layer theory (ELT)	Ranges of values from RP/19/83	WESDEF-derived values
Pavement life prediction* (Million E80s)	6,4	3,5	2,1	13,0	1,4

* Note that pavement life predictions are obtained by taking the smallest allowable number of repetitions for any of the pavement components. In the case of the pavement structure analysed the following failure criterion were used:

Asphalt - maximum tensile strain[4],
Cement stabilised layers - effective fatigue, crushing[25] and Factor of Safety[4]
Granular layers - Factor of Safety
Subgrade - vertical strain[4]

DISCUSSION

A substantial difference in predictions exists between the methods used above. This is probably to be expected because of lack of compatibility between the way (and basic theory) in which moduli are derived and resultant stresses and strains used with transfer functions. These were largely set up from moduli derived from peak deflections and not deflection basins based on the multi-layer theory.

The predicted pavement lives range between 1,4 and 13 million E80s, with those derived from the DCP and the literature being somewhat higher than values obtained from CHEV15, Equivalent Layer Theory and WESDEF. There are possibly several reasons for these discrepancies, including the limitations in data used to set up transfer functions and the DCP-elastic modulus relationship as well as the different numerical techniques and accuracies used in the back-calculation process. Non-linearity and dynamic effects are also factors that are not taken into account during analysis which, depending on the stress range, pavement type and environmental conditions, can play a significant role in accurate pavement assessment.

The largest predicted pavement life is obtained through use of assumed values from the literature (RP/19/83[4]). It should be noted that a variety of answers can be obtained by altering the moduli within a realistic range. An answer that is closer to the others could be obtained with more iterations. The prediction of pavement life obtained from moduli from the DCP is also considerably higher than the others (except of course for the literature-derived values). This is to be expected as the DCP measures some form of dynamic bearing capacity and is only empirically correlated to elastic moduli based on linear elastic multi-layer theory using limited data[24].

When using back-calculated values it should be remembered that there is no unique set of moduli that can be used to simulate a deflection basin. Some 'expert' intervention is normally needed in the process of deriving moduli from test data[26]. With respect to the various possibilities of different sets of moduli, Figure 1 conceptually illustrates why this may be so, ie there are 'local' minima and a 'global' minimum of the error function (difference between measured and calculated deflection) for different moduli solutions to a specific deflection basin. Therefore 'expert' input is needed at least to define appropriate seed values and ranges of moduli for the different layers.

Another significant point to be borne in mind is that data from a single IDM test was used in this study, and this may not have been truly representative of the pavement. This could help explain the discrepancy between predictions obtained from IDM data versus the DCP (which was a typical test result over the test site) and the literature - derived values (which are supposedly 'typical' for the respective materials). It follows that to derive representative parameters from test data, they must always be well screened and accurate to ensure it's relevance to the problem under investigation, specifically moduli back-calculation.

An additional factor that may influence the derivation of elastic moduli from surface deflection basins is that of the assumption of semi-infinite subgrade depths. Rohde[27] has dealt with this aspect at some length and has indicated how significant differences of elastic moduli can be obtained by incorrect assumptions regarding the depth to zero deflection (ie the depth in effect to a rigid base). The effect of overestimating the depth to zero deflection is to overestimate the value of elastic moduli of the subgrade. Consequently stresses and strains are inaccurately calculated in pavement layers and therefore give poor predictions of pavement life.

An example of the differences in elastic moduli that can be obtained by varying the depth to zero deflection is given here for a test site on Road R30 near Welkom. Analysis of test data obtained from Heavy Vehicle Simulator (HVS)[28] tests indicates that the depth to zero deflection appears to lie in the order of 1,5 to 3 metres. From Figure 2 this implies that moduli could be overestimated by a factor of two or three which can result in inaccurate predictions of pavement life.

The extent by which pavement life predictions can vary depends on the pavement structure and loading, therefore no general rules are immediately apparent. If the subgrade is the limiting component determining pavement life, however, the variation of life expectancy will be more closely related. For the pavement tested on the R30 near Welkom, the variation in subgrade modulus did not make a great difference in the prediction of pavement life owing to the fact that the limiting criterion was found to be that of asphalt, and regardless of the variation in subgrade modulus, tensile strain in the asphalt did not alter significantly.

On a practical note, the overall object of testing and data analysis is to establish an understanding of the likely behaviour a pavement when loaded. It follows then that to deal with requirements regarding the design of rehabilitation measures and the subsequent pavement behaviour, links between theoretical and real behaviour are required. Practically speaking this means accurate material design and construction through appropriate and good quality control. Therefore, if a layer modulus of

'X' MPa is required, the means must exist for the material to be laid with confidence over the entire length of pavement. In many cases the uncertainties associated with subsequent construction of pavement layers outweigh all others, so that aspects of non-linearity and the suitability of transfer functions, for instance, become secondary considerations.

SUMMARY and RECOMMENDATIONS

Much has been made of the possible inaccuracies of different methods of data analysis and the consequent effects on the accuracy of pavement life prediction. The following points are derived from consideration of the above.

(i) In general, a designer should ensure that as much accurate and representative test data is available for analysis. As little 'blind estimation' as possible should be used thus reducing uncertainty in the pavement designs.

(ii) No single method should be used for back-calculation to the exclusion of all others. Deflection basin measurements, DCP tests and a selection of analysis techniques, as well as experience, should be used to obtain some degree of confidence in a condition of a pavement or design recommendations.

(iii) The analysis of deflection basins is complex and due cognisance must be paid to using suitable 'seed' moduli to start iteration in programs such as WESDEF. If unsuitable 'seed' moduli are used, inaccurate final answers may be obtained due to a lack of convergence between predicted and measured basins.

Seed moduli may be obtained by various methods, for example, the empirical DCP relationship between penetration per blow and elastic moduli, values from Report RP/19/83[4] and, for lower pavement layers, the PENCEL shear Pavement Pressuremeter(PSPP)[29].

(iv) A 'Black-Box' technique of obtaining suitable elastic moduli for pavement assessment and design is not available. Considerable risks exist in the use of this approach where a designer has limited experience and cannot appraise results with sufficient confidence. A rational approach including assessment of the type and quality of test data as well as all assumptions and their likely effect on the final design, should therefore be used.

(v) Equivalent layer theory (ELT) appears to be a useful tool to obtain moduli from deflection basin analysis, especially for lower pavement layers. It is recommended, however, that to be confident that derived moduli are appropriate for use in pavement design or analysis, the ELT-derived values should be used in conjunction with multi-layer linear elastic methods (such as CHEV15 or Elsym for example) to predict deflections which can then be compared with measured values.

(vi) The effect of ignoring a finite depth to a rigid base (or zero deflection) typically gives artificially high values of subgrade moduli. This, in turn, can lead to inaccurate estimates of pavement life. However, as transfer functions were derived without taking this phenomenon into consideration, this may be compensated for when applying transfer functions. Problems do arise, however, when comparing moduli derived from different test methods and/or analyses.

(vii) Following from above, more work needs to be conducted on transfer functions and their application. It seems probable that HVS-MDD test data could be used to derive depths to zero deflection after which test results could be reanalysed and existing transfer functions adjusted.

(viii) Before analysing IDM (or other) test results the accuracy and appropriateness of the data must be carefully assessed. In particular the effect of environmental changes (such as temperature with asphalt layers) should be taken into consideration. This could be effected by testing at different temperatures for asphaltic materials, for instance, and at different moisture contents for subgrades and granular materials.

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REFERENCES

1 Meyer R G. An analysis of government spending on roads over the past fifteen years. ATC91: Keynote Address, Annual Transportation Convention, CSIR, Pretoria, South Africa, 1991.

2 Claasssen A I M and Ditmarch R. Pavement evaluation and overlay design. The SHELL method. Proc. 4th International Conference on the Structural Design of Asphalt Pavements, Vol. 1, University of Michigan, Ann Arbour, Michigan, 1977.

3 Jordaan G J. Guidelines towards the use of a rehabilitation design method based on Dynamic Cone Penetrometer (DCP) measurements as developed in South Africa. Research Report DPVT 43, Division of Roads and Transport Technology, CSIR, Pretoria, 1989.

4 Freeme C R. Evaluation of pavement behaviour for major rehabilitation of roads. Technical Report RP/19/83, Pavement Engineering Technology, Division of Roads and Transport Technology (formally NITRR), CSIR, Pretoria, September 1983.

5 Committee for State Road Authorities (CSRA). TRH4:1985 Structural Design of Interurban and Rural Road Pavements. Technical Recommendations for Highways (TRH), Department of Transport, Pretoria, South Africa, 1985.

6 Kleyn E G. The use of the Dynamic Cone Penetrometer (DCP). Transvaal Roads Department, Report L2/74, Pretoria, 1975.

7 Semmelink C J. The K-Mould: Its theory and application for determining design parameters of road building materials. Report DPVT-137, Division of Roads and Transport Technology, CSIR, Pretoria, April 1990.

8 Barker W R and Brabston W N. Development of a structural design procedure for flexible airport pavements. Report FAA-RD-74-199, Federal Aviation Administration, Washington, D.C., 1975.

9 De Beer M. Developments in the failure criteria of the South African mechanistic design procedure for asphalt pavements. Prepared for the 7th International Conference on Asphalt Pavements: Design, Construction and Performance, to be held at the University of Nottingham, England, August 1992.

10 Horak E, Maree J H and Van Wyk A J. Procedures for using Impulse Deflectometer (IDM) measurements in the structural evaluation of pavements. Proc. Annual Transportation Convention (ATC), Session 5, Pretoria, 1989.

11 Technical Information Bulletin. Descriptif Technique-Deflectographe Lacroix-LPC, MAP (Measurements Apparatuses and Products) s.a.r.l. 29 route d'Illfurth, Spechbach-le-Bas, 68720, Illfurth France.

12 De Beer M, Horak E and Visser A T. The Multi-Depth Deflectometer (MDD) system for determining the effective elastic moduli of pavement layers. 1st International Symposium on non-destructive testing of pavements and back-calculation of moduli, Baltimore, USA, June 1988.

13 Michelow J. Analysis of stresses and displacements in an N-layered elastic system under a load uniformly distributed on a circular area. CHEVRON Computer Program, California Research Organisation, 1963.

14 Van Cauwelaert F J, Alexander D R, White T D and Barker W R. Multi-layer elastic program for back-calculating layer moduli in pavement evaluation, Special Technical Publication 1026, ASTM, Philadelphia, 1989.

15 Duncan J M and Chang C -Y. Non-linear analysis of stress and strain in soils. Journal of the Soil Mechanics and Foundation Engineering Division, ASCE, Vol. 96, 1970.

16 Kondner R L. Hyperbolic stress-strain response: Cohesive soils. Journal of the Soil Mechanics and Foundation Engineering Division, ASCE, Vol.89 SM1, 1963.

17 Ullitdz P. Overlay and stage design. Proceedings, 4th International Conference on the Structural Design of Asphalt Pavements, Vol. 1, University of Michigan, Ann Arbour, Michigan, 1977.

18 Odemark N. Investigations as to the elastic properties of soils design of pavements according to the theory of elasticity. Staten Vaeginstitut, Stockholm, Sweden, 1949.

19 Boussines J V. Applications des potentials a l'etude de l'equilibre et du mouvement des solides elastiques. Gautier-Villans, Paris, 1885.

20 Van Cauwelaert F J, Alexander D R, White T D and Barker W R. Multi-layer elastic program for back-calculating layer moduli in pavement evaluation. Special Technical Publication 1026, ASTM, Philadelphia, 1989.

21 Brown S F, Tam W S and Brunton J M. Structural evaluation and overlay design: Analysis and implication. Proceedings, 6th International Conference on the Structural Design of Asphalt Pavements, University of Michigan, Ann Arbour, Michigan, 1987.

22 Peutz M G F, Van Kempen H P M and Jones A. Layered systems under normal surface loads. Highway Research Record 228, 1968, National Research Council, Washington, D.C., 1968.

23 Tam W S and Brown S F. Back-analyzed elastic stiffness: Comparison between different evaluation procedures. Non-destructive Testing of

Pavements and Back-calculation of Moduli, STP 1026, ASTM, Philadelphia, 1989.

24 De Beer M. Use of the Dynamic Cone Penetrometer (DCP) in the design of road structures. Geotechnics in the African Environment 10th African Regional Conference on Soil Mechanics and Foundation Engineering Incorporating the 3rd International conference on Tropical and Residual Soils, Maseru, September 1991.

25 De Beer M. Aspects of the design and behaviour of road structures incorporating lightly cementitious layers. Ph.D. Thesis, Department of Civil Engineering, University of Pretoria, South Africa, 1990.

26 Lytton R L. Back-calculation of pavement layer properties. Non-destructive Testing of Pavements and Back-calculation of Moduli, STP 1026, ASTM, Philadelphia, 1989.

27 Rohde G T. The mechanistic analysis of pavement deflections on subgrades varying in stiffness with depth. PhD Thesis, Department of Civil Engineering, Texas A&M University, December 1990.

28 Freeme C R, Maree J H and Viljoen A W. Mechanistic design of asphalt pavements and verification of designs using the heavy vehicle simulator. Proc. 5th Conf. on the Structural Design of Asphalt Pavements, Vol. 1, Delft, 1982.

29 Briaud J-L and Shields D H. A special Pressuremeter and pressuremeter test for pavement evaluation and design. Geotechnical Testing Journal, GTODJ, Vol.2, No.3, Sept. 1979.

30 De Beer M, Kleyn E G and Wolff H. Behaviour of various rehabilitation options of a cracked-and-seated semi-rigid pavement during accelerated testing. Prepared for the 7th International Conference On Asphalt Pavements to be held at Nottingham University, August 17-21, 1992.

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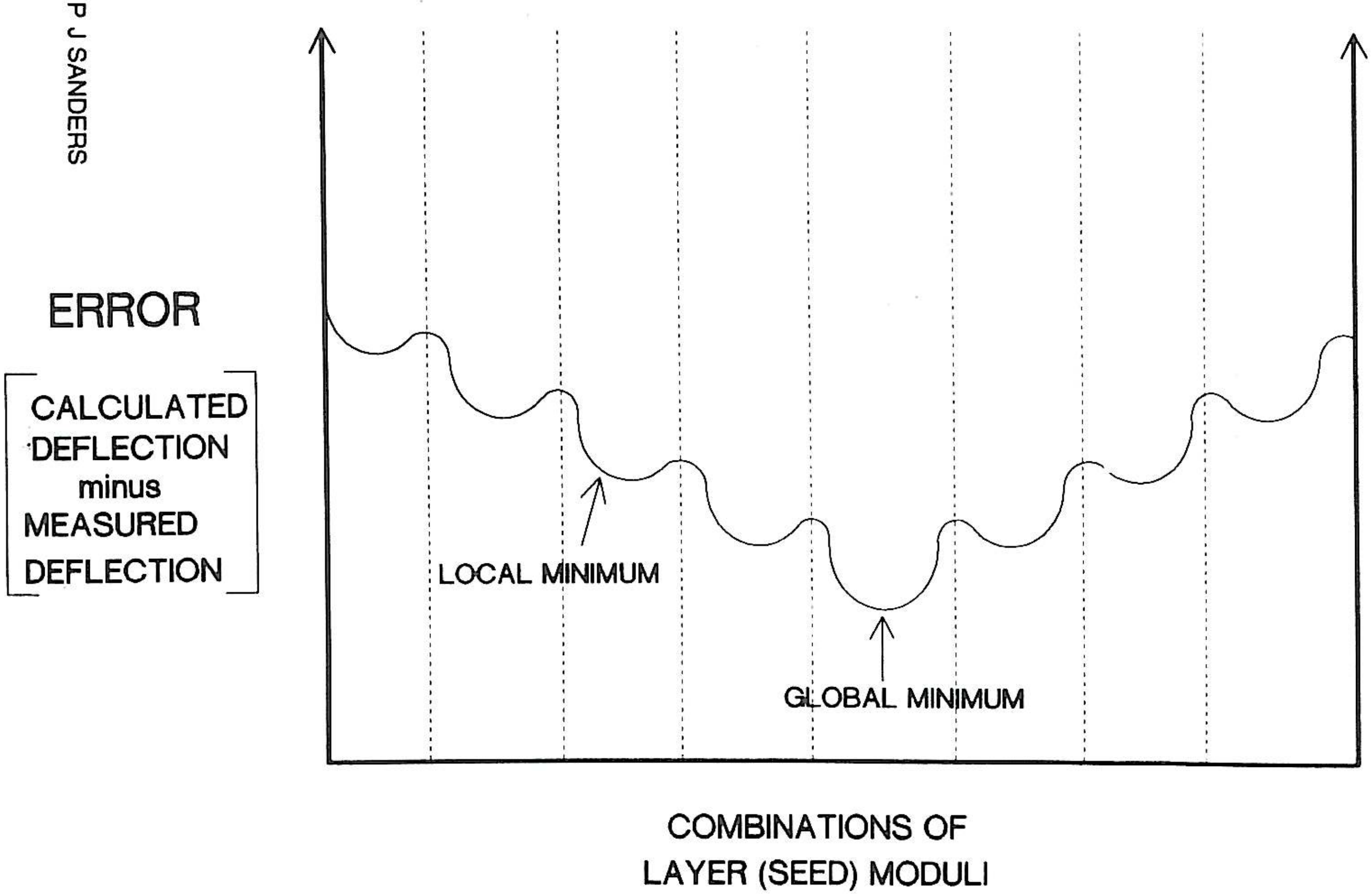
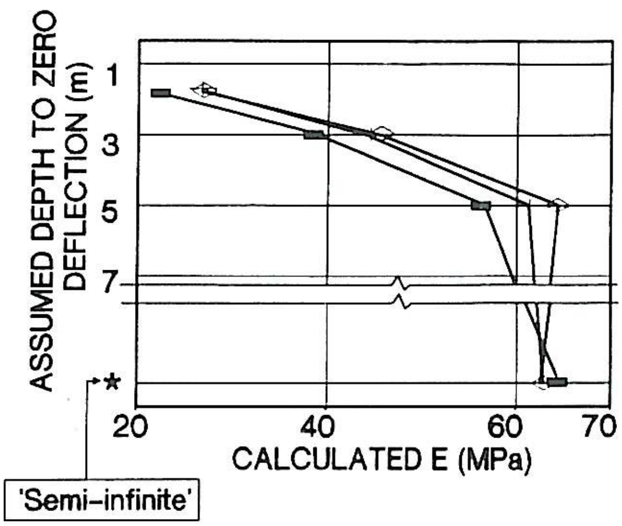


FIGURE 1

INFLUENCE OF SEED MODULI ON ACCURACY OF DEFLECTION BASIN FITTING



FINAL DEFLECTION MEASUREMENTS

Symbol	Wheel Load (kN)
▬	40
◇	60
+	100

FIGURE 2

BACKCALCULATION OF ELASTIC SUBGRADE MODULUS