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A structural design procedure for cement-treated layers in pavements

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SUMMARY

Cement-treated materials have been used successfully in road pavements since the thirties. In the past the research developments were mainly directed towards, and the emphasis during design was mainly placed on material properties, with very little attention being paid to thickness design. High-speed electronic computers and the appropriate programs became available during the sixties, and since then more effort has been devoted to the requirements for a successful structural layout and the behaviour of a pavement structure. This thesis, which is complementary to these studies, discusses the structural design of pavements having cement-treated layers. Some of the design requirements have been known for some time, three more have been added and finally a design procedure is proposed and verified.

Chapter 1 portrays the development of structural pavement design theory. It indicates how design procedures gradually became more extensive but also more complex. A pavement design procedure which is based on layered elastic theory fits into this development pattern and it has the potential to comply with future requirements of structural design procedures.

The requirements for a structurally well-designed cement-treated layer are summarized in Chapter 2. Some of these were obtained from a literature survey and they include the requirements that a cement-treated layer must be thick; it must be built on a proper foundation while bearing in mind the principles of a balanced design; and it must be designed to withstand the heavy vehicles expected to travel on it. In the thesis attention is paid to some of the other requirements, for example non-traffic-associated and traffic-associated cracking, fatigue behaviour, thermal stresses and the variability in the properties of field- and laboratory-prepared materials. Some other requirements which are mentioned but which will need further investigation, are the material characterization, design criteria and the general variability of construction materials.

Cracking in cement-treated materials is discussed in a somewhat original approach in Chapter 3 and it is pointed out that a clear distinction is necessary between initial, that is non-traffic-associated, and traffic-associated cracking. The occurrence of initial cracking must be accepted as a fact and very little can be done to avoid it or prevent it from occurring! Traffic-associated cracking in cement-treated layers can be

prevented by an appropriate structural analysis and design. This involves doing the analysis for an uncracked pavement and thereafter making some increase in the maximum stress to accommodate the stress increase caused by the initial, non-traffic-associated, crack.

Prismatic solid finite elements are used to calculate the extent of the increase in the tensile stress next to the initial crack (Chapter 4). The various ways of modelling the pavement, and the accuracy of each of these methods, are discussed and the use of the L-model is suggested and justified. The vertical surface deflection and the increase in horizontal tensile and vertical compressive stresses next to the crack in eight different typical structural layouts are calculated and the percentages are reported. The percentage increase in tensile stress seems to be dependent on the width of the crack and the thickness of the cement-treated material, but it does not appear to exceed 40 per cent. It is therefore suggested that the stress calculated in an uncracked pavement should be increased accordingly and this increased value should be used as the design tensile stress.

Thermal stresses in cement-treated layers have always been believed to be very important. A finite difference computer program is used to prove this for uncracked cement-treated layers in Chapter 5. It is also shown that once the layer has cracked, and all properly constructed cement-treated layers do crack, movement can take place at the crack which will prevent the development of excessive thermal stresses. The use of a thermal insulating layer is very beneficial and it seems that the thickness of the layer affects the insulating ability much more than the type of material used. It is therefore recommended that for major roads a 150 mm crusher-run layer should be used as a thermal insulator on top of the cement-treated layer.

Chapter 6 discusses the difference in properties of materials prepared in the field and in the laboratory. It is important that the same quality of material should be prepared in both cases or alternatively, that the designer should know the extent of this difference to enable him to take account of it. Samples recovered from the field indicate little variation in quality during a day's work and the section may be accepted as homogeneous with regard to the evaluated properties. The differences between work performed on different days are extremely significant, even if the sections were constructed with the same materials, by the same construction team and according to the same specifications. Thus sections constructed on different days may not be regarded as being of the same quality and as having the same properties. The variation in properties within a layer is significant and the upper half of the layer seems to have higher values than the lower half. The difference between field- and laboratory-prepared samples is significant, and the field samples generally tend to have lower values than the laboratory-prepared samples; not enough information is available to really indicate how much lower, but 30 per cent is recommended

The Heavy Vehicle Simulator (HVS) was used to correlate the predicted and actual behaviour of pavements with cement-treated layers, and ten HVS tests were performed (Chapter 7). The excellent correlations between the predicted and actual elastic moduli and predicted and actual amount of traffic-associated cracking are described for seven of these tests. The chapter also shows the progress that was made over 5 years in interpreting the results from these tests.

A pavement design procedure which is based on layered elastic theory, and the design requirements developed in this thesis, are outlined in Chapter 8. The definition of failure which is adopted and the design flow diagram with all its subdivisions, are explained. The design procedure incorporates the full spectrum of traffic wheel loading, fundamental material properties and failure criteria. Layered elastic theory is applied to calculate the stresses and strains at the various critical positions and these are compared with allowable values. Some variations in the outline, for example making allowance for mixed traffic and the use of standard designs, are discussed and explained. Finally the proposed procedure is verified by a description of the excellent agreement between the predicted and actual response and behaviour of several pavements. Five worked examples are also included.

The thesis also contains four appendices. The first of these describes a theoretical study of pumping in pavements using the prismatic solids finite element computer program. The second outlines the use of the prismatic solids finite element computer program. In the remaining two the thermal properties required in Chapter 5 are calculated.

OPSOMMING

Sementgestabiliseerde materiale word reeds sedert die dertigerjare met sukses in paaie gebruik. In die verlede het die klem tydens navorsing, ontwikkeling en ontwerp op materiaaleienskappe geval en baie min aandag is gegee aan die ontwikkeling van 'n metode om die vereiste dikte van die lae te bepaal. Sedert die hoëspoed- elektroniese rekenaars en die nodige rekenaarprogramme in die sestigerjare beskikbaar geword het, is meer aandag aan die vereistes van 'n geslaagde strukturele uitleg en die gedrag van die plaveisel gegee. Hierdie proefskrif, wat aanvullend is tot die vorige studies, bespreek die strukturele ontwerp van plaveisels met sementgestabiliseerde lae. Enkele van die ontwerpvereistes is reeds 'n geruime tyd bekend, drie word bygevoeg en bespreek en ten slotte word 'n ontwerpmetode voorgestel en geverifieer.

Die ontwikkeling van plaveiselontwerpteorie word in Hoofstuk 1 bespreek. Daar word aangetoon hoe die ontwerpmetodes geleidelik meer omvattend geword het - maar ook steeds moeiliker om te gebruik. 'n Ontwerpteorie wat gebaseer is op die teorie van gelaagde elastiese sisteme en baie goed in die ontwikkelingspatroon inpas, het die potensiaal om te voldoen aan die toekomstige vereistes van strukturele ontwerpmetodes vir plaveisels.

Die vereistes van 'n struktureel goedontwerpte sementgestabiliseerde laag word in Hoofstuk 2 bespreek. Sekere van hierdie vereistes is met behulp van 'n literatuurstudie bekom en sluit in dat 'n sementgestabiliseerde laag dik moet wees en 'n stewige fondament moet hê. Die beginsels van 'n gebalanseerde ontwerp moet in gedagte gehou word en die laag moet so ontwerp word dat dit die swaar voertuie wat op die pad verwag word, kan dra. In die proefskrif word enkele ander vereistes bespreek naamlik die moontlike voorkoming van aanvanklike barste, ook bekend as krimpingsbarste, en barste wat deur verkeerspannings veroorsaak word, die vermoeidheidslewe, die hantering van termiese spannings en die verskil tussen die eienskappe van materiale wat in die veld en materiale wat in die laboratorium voorberei is. Enkele van die ander vereistes wat genoem word, maar waarvoor verdere studie nog nodig is, is die karakterisering van die materiaal, die ontwerpkriterium en die algemene veranderlikheid van konstruksiemateriale.

Barste in sementgestabiliseerde materiale word op 'n ietwat oorspronklike manier in Hoofstuk 3 bespreek en daar word aangedui dat die verskil tussen aanvanklike barste en barste wat deur verkeerspannings veroorsaak word, baie goed verstaan moet word. Die verskyning van aanvanklike barste moet aanvaar word as 'n feit en baie min kan gedoen word om te voorkom dat die barste op die oppervlak van die pad verskyn! Barste wat deur verkeerspannings veroorsaak word kan vermy word deur 'n toepaslike analise en ontwerp te doen. Dit behels die ontleding van 'n ongebarste plaveisel en die verhoging van die berekende maksimum spanning waardeur dan voorsiening gemaak word vir die toename in spanning wat veroorsaak word deur die aanvanklike bars.

'n Eindige elemente-analise is gebruik om die toename in trekspanning langs die aanvanklike bars te bereken (Hoofstuk 4). Die verskillende maniere waarop die plaveisel gemodelleer kan word met eindige elemente en die akkuraatheid van elkeen van die metodes word bespreek. Daarna word die gebruik van die L-model aanbeveel. Die oppervlakdefleksie en die toename in horisontale trekspanning en vertikale drukspanning langs die bars in agt verskillende maar tipiese strukturele uitlegte word bereken en die persentasietoename in die maksimum horisontale trekspanning vir elke geval word gegee. Die persentasietoename in trekspanning is waarskynlik afhanklik van die wydte van die bars en die dikte van die sementgestabiliseerde lae maar skynbaar oorskry dit nie 40-persent nie. Daarom word daar voorgestel dat die berekende spanning in 'n ongebarste plaveisel dienooreenkomstig vergroot moet word en dat hierdie verhoogde waarde dan gebruik moet word as ontwerptrekspanning.

Daar is nog altyd geglo dat termiese spannings in sementgestabiliseerde lae baie belangrik is. In Hoofstuk 5 word 'n eindige verskille-rekenaarprogram gebruik om dit te bevestig ten opsigte van ongebarste sementgestabiliseerde lae. Daar word ook aangetoon dat sodra die laag gebars het, en alle goedgeboude sementgestabiliseerde lae bars, word die opbou van oormatige hoë termiese spannings voorkom, want daar kan beweging by die bars plaasvind. Dit is baie voordelig om 'n termiese isoleerlaag bo-op 'n sementgestabiliseerde laag te plaas en dit blyk dat die dikte van die laag die isoleervermoë meer beinvloed as die tipe materiaal wat in die laag gebruik word. Daarom word aanbeveel dat 'n klipslaglaag van 150 mm bo-op die sementgestabiliseerde laag van 'n hoofpad gebruik moet word.

In Hoofstuk 6 word die verskil tussen die eienskappe van materiale wat in die veld voorberei is en materiale wat in die laboratorium voorberei is, bespreek. Dit is belangrik dat die kwaliteit van die materiaal in albei gevalle dieselfde moet wees of andersins moet die ontwerper bewus wees van die grootte van die verskil sodat hy daarvoor voorsiening kan maak in die ontwerpstadium. Toetse het aangetoon dat die gedeelte van 'n laag wat op een dag gebou is, as homogeen ten opsigte van die gemete eienskappe aanvaar mag word. Die verskille in sementgestabiliseerde gedeeltes wat op verskillende dae gebou word, is uiters betekenisvol - selfs al word dieselfde materiale, dieselfde konstruksiespan en dieselfde spesifikasie in elke geval gebruik. Daarom mag daar nie aanvaar word dat gedeeltes wat op verskillende dae gebou is dieselfde kwaliteit en eienskappe sal hê nie. Selfs die variasie in eienskappe binne-in die laag is betekenisvol en dit blyk dat die boonste gedeelte van 'n laag hoër waardes het as die onderste gedeelte. Die verskil tussen veld- en laboratoriumvoorbereide monsters is betekenisvol en oor die algemeen neig die veldmonsters om laer waardes te hê as die laboratoriumvoorbereide monsters. Daar is nie duidelikheid oor hoeveel laer nie, maar 30-persent word tans aanbeveel.

Die swaarvoertuignabootser (SVN) is gebruik om die verwagte en werklike gedrag van plaveisels met sementgestabiliseerde lae te korreleer en hiervoor is tien SVN-toetse uitgevoer (Hoofstuk 7). Die uitstekende ooreenstemming, in terme van elastisiteitsmoduli en hoeveelheid verkeersbarste, word vir sewe van die toetse beskryf. Die hoofstuk toon ook die vordering aan wat gedurende die afgelope vyf jaar gemaak is met die interpretasie van die toetsresultate.

'n Plaveiselontwerpmetode wat gebaseer is op die teorie van gelaagde elastiese sisteme en die ontwerpvereistes wat in die proefskrif ontwikkel is, word in Hoofstuk 8 beskryf. Die aanvaarde definisie van swigting en die vloeidiagram vir die ontwerp asook al sy onderafdelings, word verduidelik. Die ontwerpmetode sluit die hele spektrum van wielbelastings, fundamentele materiaaleienskappe en swigtingskriteria in. Die teorie van gelaagde elastiese sisteme word gebruik om die spanning en vervormings op die verskillende kritiese posisies te bereken en daarna word hierdie waardes met die toelaatbare waardes vergelyk. Enkele variasies van die voorgestelde metode, soos byvoorbeeld om voorsiening te maak vir gemengde verkeer en die gebruik van standaardontwerpe, word bespreek en verduidelik. Ten slotte word die voorgestelde metode geverifieer met 'n beskrywing van die uitstekende ooreenstemming tussen die voorspelde en die werklike gedrag van etlike plaveisels. Vyf uitgewerkte voorbeelde word ook ingesluit.

Die proefskrif bevat 4 aanhangsels. Die eerste een beskryf 'n teoretiese studie van pompaksie in plaveisels en hiervoor is 'n spesiale eindige elemente-rekenaarprogram gebruik. Die tweede aanhangsel beskryf die gebruik van die eindige elemente-rekenaarprogram wat in Hoofstuk 4 en aanhangsel A gebruik is. In die oorblywende twee aanhangsels word die termiese eienskappe bereken waarna in Hoofstuk 5 verwys word.

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CONTENTS

CHAPTER 1

INTRODUCTION - HISTORY AND DEVELOPMENT

1.1 INTRODUCTION

Since the arrival of the motor-car, the demands for better roads have continually increased. Economic growth and the associated increase in road traffic, both in numbers of vehicles and the axle loads, have accelerated the demands from the public even further. The development of better roads followed two major lines, namely improved geometric design which ensures flatter slopes and longer horizontal curves, and improved pavement design procedures which ensure smoother, dust-free road surfaces. Both of these developments resulted in more comfortable travel. It is the objective of this chapter to indicate how the theory of pavement design has progressed: to do this some of the existing procedures will be analysed and the development trends will be shown. Currently a more fundamental procedure is being developed and some discussion of this will be given.

1.2 DESIGN PROCEDURES

In pavement design provision should be made to accommodate at least (i) the effect of traffic, which consists of both the number and magnitude of the loads, (ii) the material characteristics, and (iii) the environmental conditions. These parameters may be termed the input requirements and are needed for the design of the pavement.

1.2.1 California Bearing Ratio

The California Bearing Ratio (CBR) pavement design procedure, which was developed in the late thirties, is one of the oldest procedures (Porter, 1938, 1942 and 1949). This procedure defined three traffic categories which were based only on the magnitude of the wheel load, namely 31 kN (7 000 lb) for light traffic, 40 kN (9 000 lb) for medium traffic, and 53 kN (12 000 lb) for the heavy traffic. No specific provision was made for the number of load repetitions. In this procedure the material is characterized by an empirical test which bears very little relationship to the loading conditions experienced by the material in the pavement structure, but the CBR test was adopted after calibration against the actual performance of several existing pavements. The design procedure does not directly include the environmental conditions although practical experience with it has placed limitations on the use of certain materials under certain environmental conditions, for example the use of untreated bases is discouraged in high-rainfall and freeze-thaw areas.

The thickness design is performed by entering a graph corresponding to the appropriate traffic category, for example medium traffic, and reading off the total pavement thickness required for a material of a certain CBR. The thickness of the base and subbase can also be obtained from this curve since it is merely necessary to place a material of a certain thickness over a material having a certain CBR. No provision was, however, made for obtaining the thickness of the bituminous surfacing.

The advantages of this procedure are that (i) it has been used extensively and that a wealth of empirical information has been accumulated on its use, and (ii) that the required laboratory test is relatively quick and easy to perform. The disadvantages are the inability (i) to accommodate modern road-building materials, such as bituminous mixtures, cement- and lime-treated materials (called cementitious materials) and crusher-runs (an excellent quality crushed fresh rock), (ii) to include the environmental conditions, and (iii) to make provision for both the number and magnitude of load applications.

1.2.2 State of California

The State of California design procedure was developed around 1941 and has been used in California, USA since about 1950 (Hveem and Carmany, 1948). It is essentially an empirical pavement design procedure which aims at preventing permanent deformation, often also referred to as plastic deformation or rutting, in the pavement layers.

The number and magnitude of all wheel loads expected during the design period are considered and converted to an equivalent number of 22,2 kN (5 000 lb) wheel loads, which is referred to as the EWL. From this value the destructive equivalent of the traffic is calculated and expressed as a numerical value called the Traffic Index (TI). The subgrade material is characterized by means of the stabilometer and expansion pressure tests, and the structural quality of the material is expressed as the resistance value (R), called the R-value. The R-value of a material is a coefficient representing the shearing resistance to plastic deformation of a material when it is saturated and at a given density. This given density is determined from the expansion pressure tests. Since the thickness design is based on an R-value determined from a saturated sample, the procedure automatically takes account of severe environmental conditions.

A structural layout expected to withstand the traffic loading should be proposed while keeping in mind the minimum thicknesses specified for the various materials. The thicknesses of the materials in the proposed layout, for example bituminous or cement-treated, are then converted to gravel equivalents by using the specified gravel equivalency factors. The gravel equivalent (GE) required over a material with a certain R-value, is calculated from

$$GE = 0{,}003\,2\ (TI)\ (100 - R) \quad \ldots\ldots\ldots\ldots \quad (1.1)$$

where GE = gravel equivalent in feet

TI = traffic index

R = R-value of the material.

Next the gravel equivalents are compared. The gravel equivalent proposed over a material with a certain R-value should correspond to, or exceed, the gravel equivalent required over that material. If this can be achieved throughout the proposed layout it may be accepted.

The advantages of the procedure are that (i) the full traffic spectrum, both number and magnitude, and (ii) the modern road-building materials, are included. The environment is considered by determining the R-value from saturated samples. A disadvantage may be that the procedure is really only used in California and very little experience in the use of the procedure is available outside of this State.

1.2.3 AASHO design procedure

The AASHO Road Test was performed in Illinois, USA, and was completed by the end of 1960. It was designed to develop a pavement design procedure that would overcome the disadvantages of the then existing design procedures. The original AASHO procedure was slightly changed and adopted in South Africa during 1971 (NITRR, 1971). The number and magnitude of all the load applications are included through the calculation of the number of equivalent 80 kN axles (E80). The materials are characterized by their structural coefficients and provision is also made to include the modern road-building materials. The Regional Factor (R), which was defined to take care of the environmental conditions, can be calculated for any particular area and this makes the AASHO procedure universally applicable.

The first step in the design is to calculate the expected total number of equivalent 80 kN axles (E80) during the design life of the pavement. To take account of the environment, an adjustment is made to this number by multiplying it by the Regional Factor (R). The required structural ability, defined in terms of the dimensionless required structural number (RSN), is hereafter calculated from a fairly complex formula which requires the traffic loading in terms of the number of E80's and the level of serviceability of the pavement (p). The level of serviceability (p) defines the requirements of the road user in terms of riding quality and is expressed as the dimensionless present serviceability index (PSI). The solution of this formula can usually be obtained from a table.

The designer should propose a structural layout that is expected to withstand the calculated traffic loading while maintaining the level of serviceability of the pavement above the required minimum. The structural ability of this layout, which is termed the proposed structural number (PSN), is calculated from the product of the structural coefficients and the layer thicknesses.

The design is performed by comparing the required structural number (RSN) with the proposed structural number (PSN). Should the PSN exceed the RSN, the proposed structural layout is acceptable, but if the PSN is less than the RSN, the necessary adjustments should be made in either layer thicknesses or material quality.

The advantages of the procedure are that the full traffic spectrum, the modern road-building materials, the environment, and the requirements of the road user (expressed in terms of the level of serviceability) are included. Although it is by no means a complete treatise of these complex topics, it was a good point of departure. The major disadvantages of the method are (i) the uncertainty about the ability of the Regional factor (R) to define the environment and hence the questionable applicability of this procedure to South African conditions (Otte, 1972), and (ii) the possibility that the designer may land up with an unbalanced layout (section 2.2.4, page 16).

1.2.4 Theoretical procedure

Since the end of the sixties various people have proposed portions, called subsystems, of a theoretical pavement design procedure based on layered elastic theory (Peattie, 1962; Whiffen and Lister, 1962; Freeme, 1972; Brown and Pell, 1972; Monismith, 1973; Mitchell et al, 1974). These subsystems were developed to overcome the shortcomings in the existing design procedures and their development was expedited by the availability of the computer and layered elastic theory programs. They are also called fundamental or mechanistic procedures because they are not based on the results of one or two empirical tests but rather on the three basic steps of any engineering design, namely assumptions, analysis and comparison. This means that some assumptions are made to build a mathematical or theoretical model of the practical problem, after which the model is analysed to determine the stresses, strains and deflections developed, and finally these values are compared with the allowable safe working values of the materials.

If the developed values exceed the safe working values, distress may occur and the design should be altered in an effort either to decrease the developed values or to increase the safe working values.

The theoretical design procedure cannot easily be described in the manner that the previous three were described, because it is much more complex and consists of a number of subsystems each controlling the safety of a particular position or material in the layout. It requires assumptions about the number of load applications and the magnitude of the loads, because these are needed to calculate the allowable and working stresses or strains in the various materials. The materials are characterized in terms of elastic parameters, namely the elastic modulus and Poisson ratio, and their allowable stress or strain values. The environmental conditions are taken care of by adjustments to the material characteristics (for example, elastic modulus) of those materials affected by temperature, (for example, bituminous materials) and by moisture conditions (for example, untreated materials and the subgrade).

One of the initial steps of the design is to obtain the design wheel loads for the various materials to be used in the analysis, for example, the total number of load repetitions for thin bituminous layers (Freeme and Marais, 1973), the number of equivalent 80 kN axles (E80) for untreated materials, and the number and magnitude of the maximum wheel loads for cementitious layers. Another initial step is to prepare a structural layout which will possibly be suitable for the expected traffic. The proposed materials are then characterized in terms of the appropriate elastic parameters and the design criteria for them are based on the expected temperature and number of load applications. The analysis is performed by solving numerous equations which are so complex that a computer is required. The designer knows where the critical positions in the structure are (Peattie, 1962; Whiffen and Lister, 1962) and the design is performed by comparing the calculated stresses or strains at the various positions with the allowable values. If the calculated value at a particular position is less than the allowable value, it may be considered as safe and not likely to show signs of distress. By comparing the stresses and strains the designer can obtain the most critical position(s) and an indication of the likely mode of distress may be deduced. From these comparisons it may also be possible to suggest ways of avoiding overstressing or -straining at certain positions; these would usually include adjustments to the material properties and/or the layer thicknesses.

Some of the advantages of using the theoretical design procedure are that (i) the extent of overdesigning can be minimized because each material can be stressed or strained to its optimum, (ii) materials presently defined as sub-standard may be used economically after they have been adequately characterized in terms of their elastic parameters, and (iii) an indication of the maintenance strategy and requirements can be obtained. The major disadvantage is that very little practical experience on the use of the procedure is currently available.

1.3 FAILURE AND DISTRESS MODES

By definition, a design procedure is aimed at preventing the failure or distress of a structure. The failure of a reinforced concrete beam can easily be defined but this is not so easy for a road pavement. The various road authorities have experienced numerous modes of distress in their pavements and they have each tried to overcome these by designing against them. Several of the existing design procedures were therefore developed to prevent a particular mode of distress, for example the CBR procedure aimed at avoiding rutting in the subgrade (Porter, 1938), the AASHO procedure was developed to ensure an acceptable riding quality on a pavement (measured in terms of PSI), and the Shell procedure to avoid fatigue cracking of the bituminous surfacing and deformation (rutting) of the subgrade (Dormon and Metcalf, 1965). The theoretical procedure, however, is directed at preventing all the known distress modes in pavements by checking the various critical positions in the layout. At this stage the procedure has not been perfected and it is not yet possible to handle all the distress modes, but, in theory, this will be possible once the procedure has been completed. This will mean a major improvement on the older design procedures which usually aimed at preventing only one mode of distress.

1.4 SUMMARY

The theory of pavement design has developed significantly since the early thirties. It started with very simple empirical tests and a single graph or equation, which aimed at preventing one particular form of distress. It has progressed and has recently reached the stage where sophisticated laboratory tests are required to determine one or more relatively basic material characteristics. These tests, along with complex computer programs, will, it is hoped, eliminate all the various modes of distress. The development of pavement design theory can best be seen from the summary in Table 1.1. The table summarizes the procedures as they were originally developed - no provision has been made for subsequent additions arising from empirical usage, for example limitations on the layer thicknesses and specifications of the various materials in terms of, for example liquid limit, plasticity index and grading.

TABLE 1.1 : The development in pavement design theory

	CALIFORNIA BEARING RATIO (CBR)	STATE OF CALIFORNIA	AASHO	THEORETICAL PROCEDURE
TRAFFIC: Number of load repetitions	Not considered.	Any number; is calculated from predicted traffic volume.	Any number; is calculated from predicted traffic volume.	Any number; is calculated from predicted traffic volume but depends on materials, for example . E80 for untreated materials . Maximum wheel load for cementitious materials . Total traffic volume for bituminous surfacing.
Magnitude or unit of loading	Light : 31 kN Medium: 40 kN Heavy : 53 kN	Equivalent 22,2 kN wheel loads.	Equivalent 80 kN axles (E80).	Equivalent 80 kN axles and actual loading.
MATERIALS: Type considered	Untreated.	Treated and untreated.	Treated and untreated.	All road-building materials; also those still to be developed.
Characterized by	CBR	R-value and gravel equivalency factors.	Structural coefficient.	. Elastic modulus and Poisson ratio. . Allowable stress and strain.
ENVIRONMENT:	-	R-value is determined from a saturated sample.	Regional Factor.	. Adjustments in elastic modulus and allowable stresses and strains. . Calculates and includes effect of thermal and moisture stresses.
ANALYSIS:	Single graph.	Equation.	Tables and equation.	Complex equations - must be solved by computer.
OUTCOME OF ANALYSIS:	Total thickness; and thickness of base and subbase.	.Gravel equivalent. .Thickness of various layers in structure.	.Structural number. .Thickness of various layers in structure.	. Number of load repetitions until distress. . Critical position in structure and likely mode of distress. . Probable maintenance requirements.
MODES OF DISTRESS:	Deformation of subgrade.	Permanent deformation in pavement materials.	Loss of riding quality.	All modes of distress.

1.5 DISCUSSION

A theoretical design procedure based on layered elastic theory has been criticized by numerous persons, for example Goetz (1972), although very few have committed their doubts to paper. Some of the critics have expressed their faith in the existing empirical design procedures and further developments of them (Croney, 1972).

Table 1.1 displays the natural growth in complexity and extent of the theory of pavement design and it can be seen that the theoretical procedure has followed the logical sequence. Its development should therefore not be considered as an impossibility or a far-fetched idea. It is admitted that the theory has not yet been perfected and that there are still major tasks to be done, for example the characterization of materials, variation in material quality, definition of failure and distress, and the correlation between the computed response, that is stresses, strains and deflection, and the performance of actual pavements. Even the critics should, however, agree that the computed response correlates very well with that predicted by the theory (Wang, 1968; Dehlen, 1969; Fossberg, 1970; Hicks, 1970; Hofstra and Valkering, 1972; and Thrower et al, 1972).

It therefore appears that the theoretical design procedure based on layered elastic theory has both its opponents and proponents. The author is a proponent and believes that a design procedure which employs the basic structural design philosophy can, and should, be developed to allow pavement designers a degree of confidence comparable to that currently experienced by structural engineers. It is believed that such a procedure would be the only way for pavement designers to cope with the rising demands outlined by Monismith and Finn (1972), namely to (i) make better use of available, often sub-standard, materials, (ii) provide for ever-increasing wheel loads and tyre contact pressures, (iii) evaluate and utilize new materials which might be developed, (iv) define and include the role of construction, and (v) improve the overall reliability of pavement performance prediction.

It is appreciated that such a comprehensive design procedure will consist of numerous complex and extensive subsystems, and that the problems often seem insurmountable, but the apparent success (Monismith and Finn, 1972) of the subsystem predicting fatigue in bituminous surfacings urges one to speed up the development of other subsystems. One of these comprises the design of cementitious layers and it is therefore the overall objective of this thesis to contribute to the earlier attempts by Mitchell and Shen (1967), Nielsen (1968), Hadley et al (1972), and Mitchell et al (1974).

Table 1.1 indicated that the theory of pavement design has progressed significantly since the thirties. This progress, especially in layered elastic theory, will be utilized to develop a subsystem for the design of cement-treated layers.

CHAPTER 2

A REVIEW OF DESIGN REQUIREMENTS

2.1 INTRODUCTION

During the 40 years in which cement-treated materials have been used in pavements, a wealth of empirical information and research developments have been accumulated on aspects contributing to the successful performance of cement-treated materials in pavements. The known information is very valuable during the development of a pavement design procedure and it should be utilized and included in any newly developed thickness design procedure. The objective of this chapter is to summarize published literature and outline the important aspects and design requirements for a structurally well-designed cement-treated layer.

It is accepted, and should go without saying, that to achieve an acceptable pavement materials of an acceptable quality should be used. The treated material should therefore be well-mixed and compacted to provide a strong, hard and durable material in which some tensile strength has developed. The natural material should be free from deleterious materials such as salts and organic materials. Fresh cement and clean water should be used during the construction.

2.2 DESIGN REQUIREMENTS

The important design requirements and parameters are as follows:

2.2.1 Layer thickness

Cement-treated materials should be constructed as a thick layer, because the load-bearing ability increases rapidly when the thickness is increased. This requirement has been looked into by Nussbaum and Larsen (1965). They found that the load-bearing capacity (measured in terms of deflection) of a 100 mm cement-treated layer is about 1,5 times that of a 100 mm granular layer and if the layer thickness is increased to 250 mm, the load capacity increases to 3,3 times that of a granular layer of the same thickness. This means that the structural equivalency of the material is not fixed but it increases with thickness.

This requirement might also be explained along the lines of basic beam-bending theory. The load capacity (measured in terms of the resistance to deflection) of a beam is proportional to the factor EI, where E is the elastic modulus and I is the second moment of area. I is proportional to bd^3 and that means that the load capacity of a beam is proportional to the third power of the depth (d). The same principle applies to a cement-treated base.

Layered elastic theory, as contained in the CHEVRON computer program (Warren and Dieckmann, 1963), enabled a study of the relationship between the developed tensile stress at the bottom of the cement-treated base in a three-layer pavement structure and the elastic modulus of the material. It is also possible to investigate the influence of varying base thickness and subgrade support. Figure 2.1 shows these relationships for two different three-layer pavements, one having a 25 mm surfacing and a 100 mm base, the other a 25 mm surfacing and a 150 mm base. It appears that the developed tensile stress can be nearly halved by increasing the base thickness from 100 to 150 mm. The figure also shows the rapid increase in developed stress in a thin layer (100 mm) when the elastic modulus of the cement-treated layer is increased.

Increasing the base thickness has its limitations, because of construction problems. If in-situ mixing is used, it becomes very difficult to handle the material of a layer thicker than 150 mm on the roadway but this problem may be overcome by central mixing and paver laying. The compaction of a layer thicker than 150 might pose a problem. This difficulty could be overcome by careful use of a powerful vibrating roller, but some work still has to be done on the problem of achieving the required density in a thick cement-treated layer. The advantage of a 200 mm thick cement-treated layer can be seen from Figure 2.1.

2.2.2 Material equivalency

For quite some time material equivalencies have been used by pavement designers to determine the required thickness of a treated layer and also to compare different alternative structural layouts. There are various forms of material equivalencies and they include the AASHO structural coefficients and the gravel equivalencies used by the California Division of Highways and the Asphalt Institute. Mitchell and Freitag (1959) studied some existing airfields with cement-treated bases and observed that cement-treated materials have greater load-spreading capacity on weak subgrades than on stronger (CBR>3) subgrades. Nussbaum and Larsen (1965) and Mitchell and Shen (1967) proved that the structural ability of a cement-treated layer is not constant and that the load-spreading ability depends on (i) the thickness of the cement-treated layer, (ii) the elastic modulus of the cement-treated material, (iii) the elastic modulus of the subgrade, and (iv) the thickness of the asphaltic surfacing. The approach of equating a unit thickness of cement-treated material with a certain thickness of gravel is an unnecessary oversimplification of the problem which introduces an element of inaccuracy and should preferably not be used.

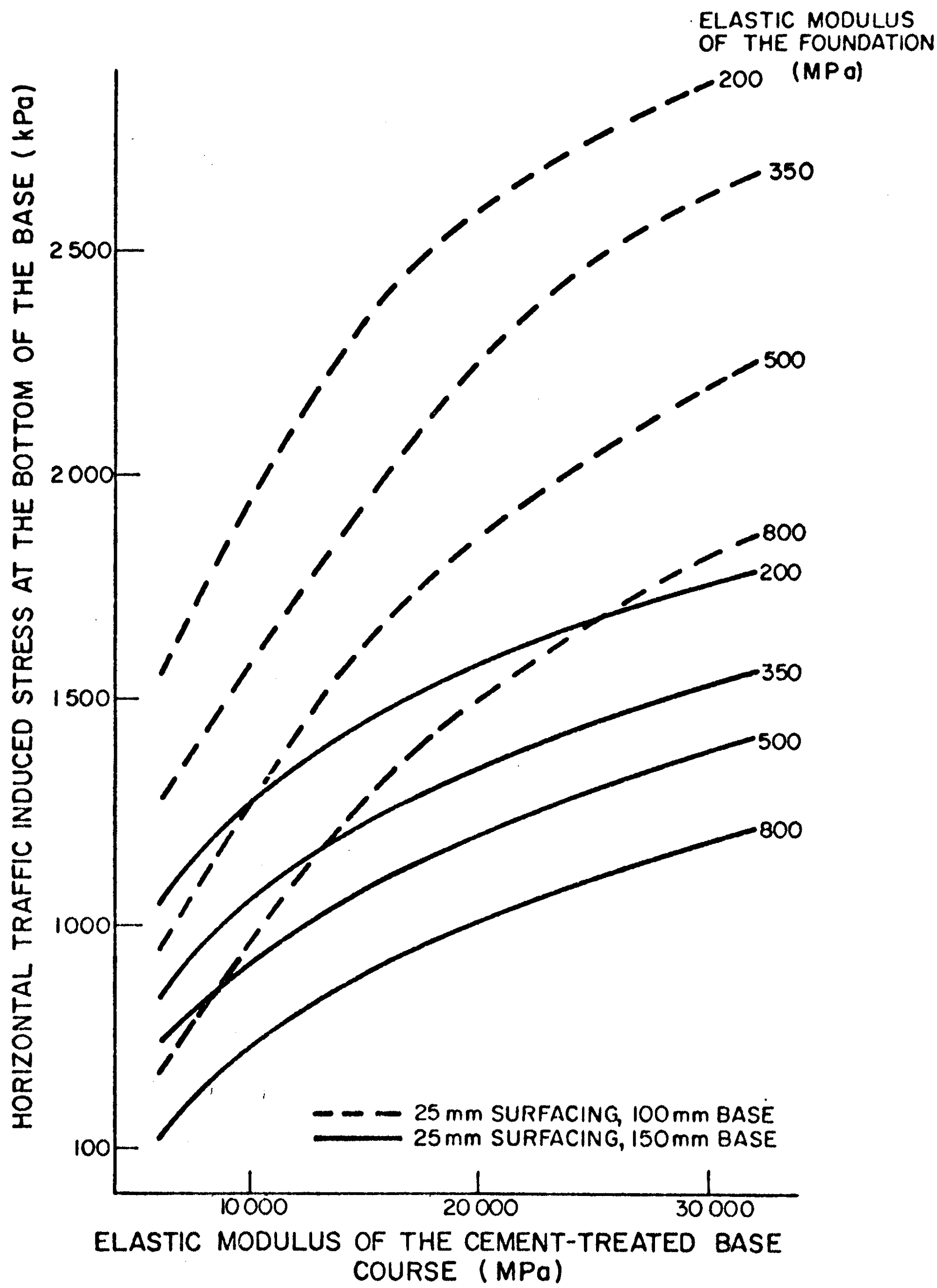


FIGURE 2-1

THE RELATIONSHIP BETWEEN TRAFFIC INDUCED STRESS AND THE MODULUS OF THE CEMENT-TREATED BASE

2.2.3 Sound foundation

The foundation of a cement-treated layer, which in this thesis is defined as all the material and structural layers below a cement-treated layer, usually consists of the subgrade, one or more layers of selected fill material, possibly an untreated subbase and sometimes even another cement- or lime-treated layer. Layered elastic theory, in the form of the CHEVRON program, enabled the author to make a theoretical study of the effect of the foundation's elastic modulus on the tensile stresses and strains at the bottom of a cement-treated layer in a three-layer pavement. Figure 2.2 shows the pavement and the various moduli considered to prepare Figures 2.3 and 2.4. These figures indicate that a sound foundation underneath a cement-treated layer is very necessary to reduce the stresses and strains.

The advantage of increasing the elastic modulus of the foundation and the associated decrease in the tensile stresses in a cement-treated layer, can also be observed in Figure 2.1. If the base has a high elastic modulus (32 000 MPa) the stress can be halved by increasing the foundation's elastic modulus four times, and if the base has a lower elastic modulus (6 000 MPa) the stress can be reduced to about 65 per cent by increasing the foundation's elastic modulus four times. Although it might appear expensive to increase the foundation's elastic modulus four times, this can readily be achieved by using a thick cement-treated subbase. A cement-treated subbase that complies with the PCA soil-cement criteria would also reduce the possibility of pumping in the subbase. The use of a cement-treated subbase is a standard procedure in the design of concrete pavements (PCA, 1973).

Mitchell and Shen (1967) performed some laboratory tests to investigate the relationship between the layer thickness required for a safe design and the elastic modulus of the foundation. They observed that (i) the thickness of the cement-treated layer may be reduced and also (ii) that the quality of the cement-treated material becomes less critical when the elastic modulus of the foundation increases. Their study also indicated that a sound foundation is very important.

The importance of the foundation's elastic modulus requires that this value should be determined much more accurately than is currently done. At present it is assumed that the ruling CBR of South African soils for pavement design purposes is about 10 to 15 and, based on the initial and approximate relationship developed by Heukelom and Klomp (1962), the elastic modulus of the foundation is assumed to be 100 MPa. This is probably less than the actual values, but because nothing better is

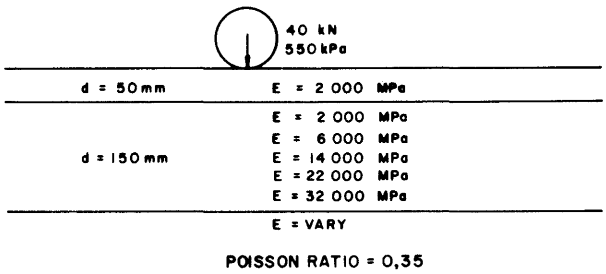


FIGURE 2-2

PAVEMENT DESIGN USED TO DETERMINE THE INFLUENCE OF THE ELASTIC MODULUS OF THE FOUNDATION

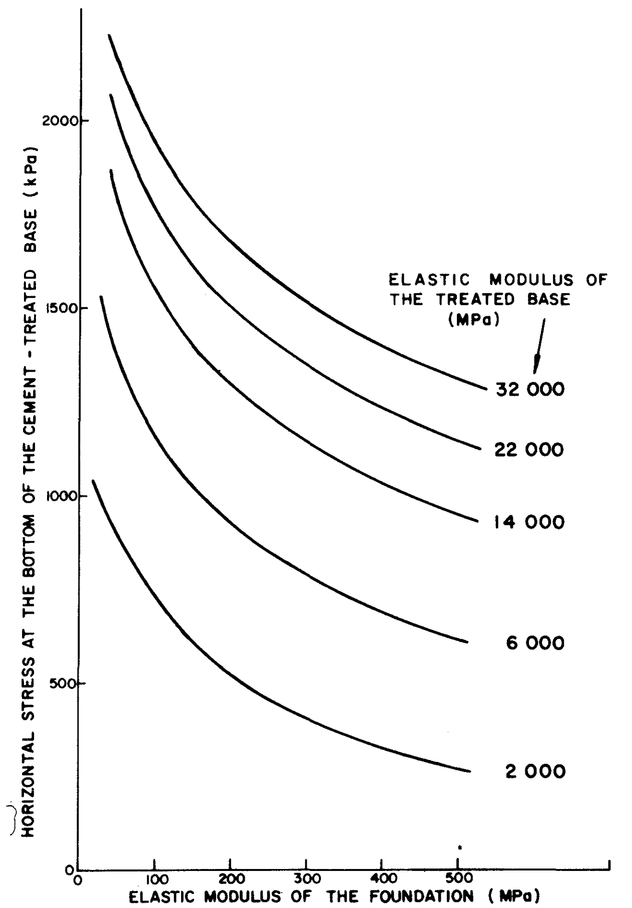


FIGURE 2-3

THE INFLUENCE OF THE ELASTIC MODULUS OF THE FOUNDATION ON THE TENSILE STRESSES IN THE BASE

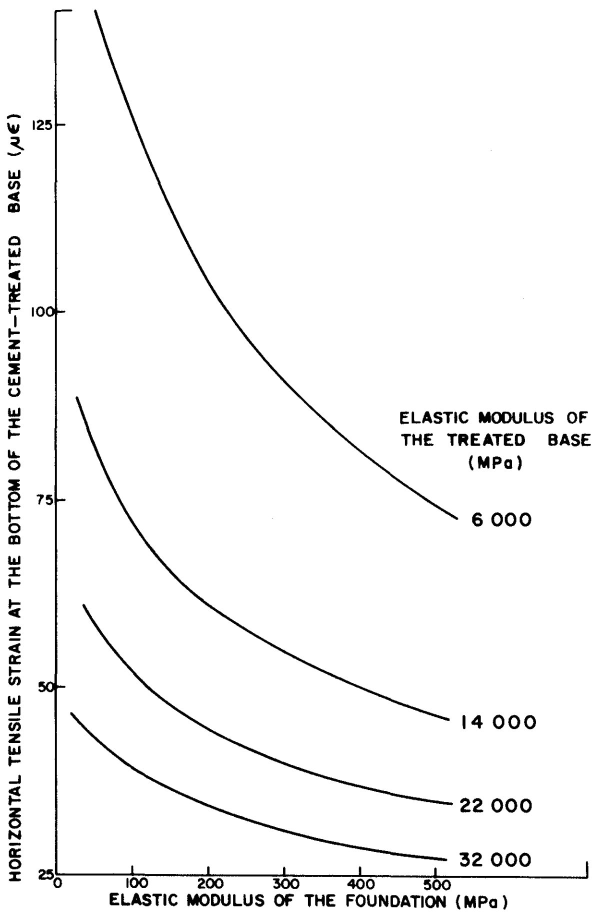


FIGURE 2-4

THE INFLUENCE OF THE ELASTIC MODULUS OF THE FOUNDATION ON THE TENSILE STRAIN IN THE BASE available and to be on the safe side, this conservative value is generally used for the foundation. To overcome this shortcoming the author suggests that the elastic modulus of the completed foundation should be measured in practice after it has been constructed. This can be done by doing either a plate-bearing or a Benkelman beam test (Otte, 1973). If the measured elastic modulus does not agree with that assumed when the design was performed, the measured value should be adopted and the proposed structural layout should be changed if the change can be justified, for example, economically. It is appreciated that the adoption of this suggestion may result in some contractual difficulties, for example, possible delays to the contractor or perhaps changes in the quantities of materials involved on the contract, but the author is certain that these can be handled when necessary. It is believed that this control on the proposed structural layout during the construction stage is justified and to the benefit of the overall reliability and economy of the pavement.

2.2.4 Balanced pavement design

A balanced structural layout can be described as a layout in which all the components of the structure are stressed or strained to just within their allowable limits. Neale (1968) defined a balanced design "...as one in which each layer of the pavement is sufficiently strong to prevent high stresses or strains developing in the underlying layers. If a design is not balanced, excessive stress and/or strain will occur in one or more of the layers in the pavement...". If some of the layers, say the base and subgrade, are strained relatively more than the others, they will probably experience distress sooner and this may result in unacceptable performance of the road and possibly a premature failure. To provide an optimum economical design, all the layers in the structure must be utilized to their full load-bearing capacity.

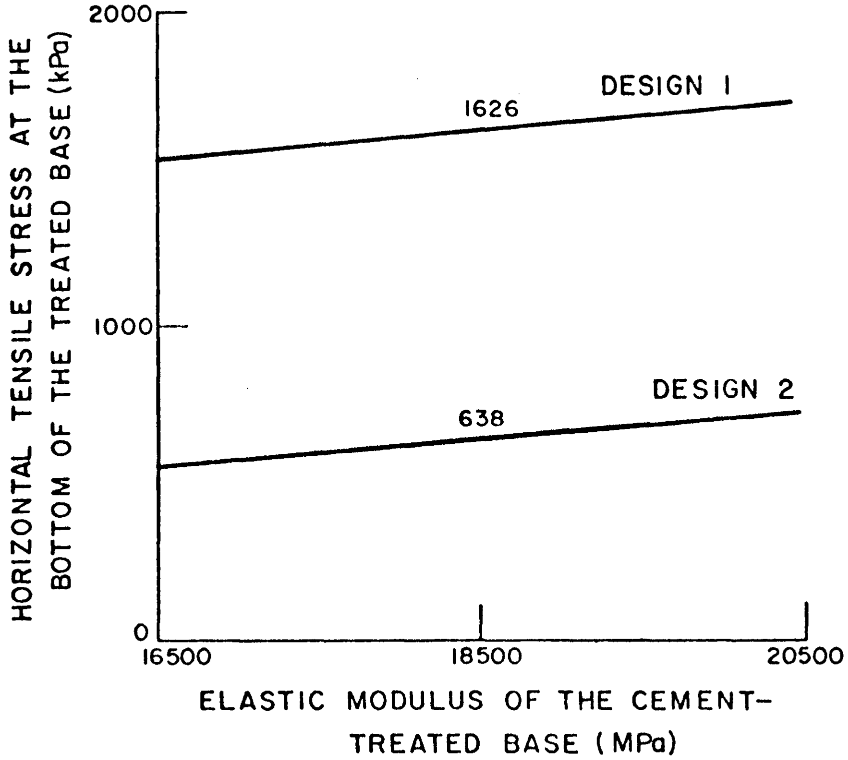
Layered elastic theory provided an opportunity to study the stress-strain behaviour of various structural layouts (Whiffen and Lister, 1962) and this aided the understanding of pavement behaviour. The concept of balanced design provided yet another opportunity for the refinement of the structural design of pavements. Although layered elastic theory and the balanced design concept cannot provide an infallible pavement design procedure which can be applied to all loading circumstances under all environmental conditions, they do provide the design engineer with some rational means of analysing a proposed structure - this will indicate to him that some changes in the layout are more advantageous and effective than others, and also where and why the future distress may occur and possible ways to avoid it or to extend the life of the pavement. Both of these, elastic theory and balanced design, can therefore be used to help avoid an early pavement failure.

There was a stage in South Africa when pavements were designed to have an untreated crusher-run layer between two cement-treated layers (Otte, 1973a). When the concept of balanced design is applied to these layouts it appears that they were unbalanced and that the presence of the untreated crusher-run layer was detrimental to their future behaviour and that it should rather have been omitted. Figure 2.5 shows the layout with and without the untreated layer between the two treated layers, and the tensile stresses at the bottom of the upper cement-treated layer. For an elastic modulus of 18 500 MPa the figure shows that the developed bending stress can be reduced 2,5 times (from 1 626 kPa to 638 kPa) when the untreated layer is omitted. The layout shown as Design 1 should therefore not be used!

Crusher-run bases of about 200 mm were always very popular in some parts of the country, mainly because of their very acceptable performance and the easy supply of unweathered crushed rock from the gold mines. The unacceptable performance of a cement-treated base on a major road carrying very heavy traffic (Otte, 1973a) prompted a return to untreated crusher-run bases, but significantly increased in thickness - 375 mm. These layouts were analysed by layered elastic theory (Otte and Monismith, 1976) and it appears that there is very little advantage, economically or otherwise, in building a thick untreated crusher-run base, that is a layer thicker than about 150 mm. The thickness of the cement-treated subbases should rather be increased.

In both these examples other pavement design procedures, like the AASHO and current Californian procedures, would have indicated opposite trends. In the first example, when the untreated layer was omitted, they would have shown reduced structural numbers and equivalent gravel thicknesses, and therefore would have indicated a drop in life expectancy. In the second example they would have shown an increasing life expectancy with increased crusher-run thickness, and although layered elastic theory and balanced design did not show the opposite, they would have shown that there might be other more economical ways of increasing the life expectancy of the layout.

These examples indicate that some of the basic principles in some of the well-used existing pavement design procedures are invalid. Therefore



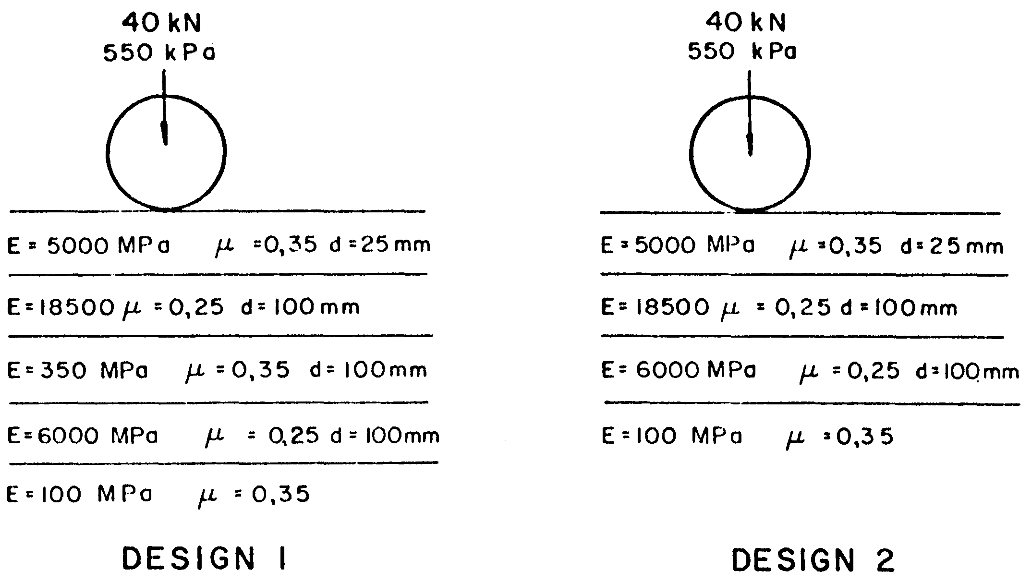


FIGURE 2-5

THE REDUCTION IN TENSILE STRESS AT THE BOTTOM OF A TREATED BASE WHEN THE UNTREATED LAYER IS LEFT OUT the use of these methods is rather questionable when cement-treated materials are used in conjunction with untreated crusher-run. These principles and design procedures therefore had to be re-evaluated and the balanced pavement design concept provided the opportunity. The latter should therefore be included in any pavement design procedure which is to be developed for cement-treated materials.

2.2.5 Insulation against thermal stresses

Lister (1972) showed that 30 per cent of the total stress in a base may be caused by temperature and 70 per cent by the wheel load. Williamson (1974) used elastic theory, an uncracked, infinitely long slab and the thermal properties of field-constructed cement-treated bases to calculate the stresses caused by temperature. He found that the temperature stresses can equal, and indeed exceed, the stresses induced by wheel loadings. This indicates that all the cement-treated bases used in this country could have cracked due to temperature alone, leaving aside the effects of shrinkage and traffic. Bonnot's (1972) theoretical calculations also showed that only a small drop in temperature (about 8 to 11 $^{\circ}$C) is sufficient to cause thermal cracking in an uncracked cement-treated layer.

Lister (1972) has also mentioned that the temperature stresses can be reduced by using a thick cover over the cement-treated base, and that the thickness of the cover is much more important than the type of material used as a cover. This would suggest that a 150 mm untreated crusher-run layer on top of the cement-treated base would be just as efficient a thermal insulator as a more expensive bituminous surfacing. This was confirmed by Williamson (1974).

The effect of temperature stresses in cement-treated layers is studied in some detail in Chapter 5. At this stage it is considered sufficient to state that thermal stresses do play a part in the design of cement-treated layers and that they should be studied with a view to minimizing their adverse effect.

2.2.6 Non-traffic-associated cracking

This type of cracking in cement-treated bases is caused by factors other than traffic and is the result of environmental stresses or a geotechnical problem such as a subsiding embankment. The geotechnical problems are usually dealt with separately and pavement designers should consider the environmental stresses which include drying-shrinkage and temperature stresses. Some people claim that the initial cracks in cement-treated bases are the result of drying-shrinkage (George, 1968) while others claim that they are caused by thermal stresses (Williamson, 1974). No distinction will be made in this thesis (Chapter 3); it is merely accepted that initial cracks are the result of the combined effect of drying-shrinkage and temperature stresses. There have been studies (George, 1968; and Wang, 1973) on possible ways of eliminating or reducing the amount of cracking, but there appears to be no way of avoiding these cracks in properly constructed cement-treated layers. Their presence has to be accepted as a fact and ways must be devised by structural analysts to accommodate them in pavement design.

Finite element analysis has made it possible to study a cracked cement-treated layer and develop a possible method of accommodating the transverse initial crack during the structural design. This is explained and detailed in Chapter 4.

2.2.7 Design traffic

(a) Wheel load

In a previous study (Otte, 1972a; and Otte, 1974) the stress-strain properties of cement-treated materials were investigated and a general stress-strain curve was suggested (Figure 2.6). This curve shows the material to be initially linear-elastic. Above a certain critical loading condition, which is about 35 per cent of the strength and about 25 per cent of the strain at break, microcracking starts as a loss of bond between the aggregate and the matrix of fine material and cement, permanent strains develop and the stress-strain curve becomes non-linear and non-elastic (Shah and Winter, 1966; and Hansen, 1966).

Traffic stresses above this critical value will increase the amount and extent of microcracking which will eventually result in the development of traffic-associated cracking and a reduction in the structural capacity of the material. To prevent traffic-associated cracking, microcracking should therefore be controlled or prevented. This would be achieved if the maximum stress in the material, which is usually induced by the heaviest wheel load, were less than the stress at which microcracking starts. The stresses induced by lighter wheel loads are not detrimental because the material remains linear-elastic and no microcracking will therefore develop in the matrix. This means that the number, and the magnitude of the heavier wheel loads expected on the road are important, and not the lighter loads. A cement-treated base should therefore be designed to carry the heavy axles and the significantly lighter loadings may be disregarded. This procedure will also accommodate overloading if it takes place on the road.

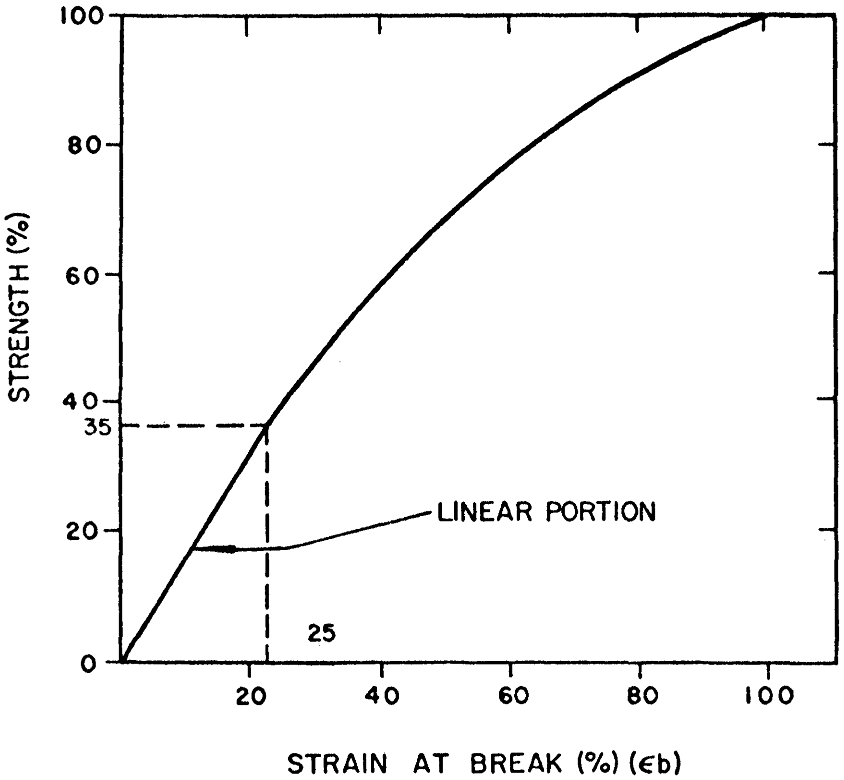


FIGURE 2-6

THE GENERAL STRESS - STRAIN CURVE OF CEMENT-TREATED MATERIALS

(b) Load equivalency factors

It was stated above that the magnitude of the heavier wheel loads is required and not the whole traffic spectrum. This implies that the load equivalency factors of the State of California and AASHO design procedures are not applicable. Using layered elastic theory and a strain criterion, Van Vuuren (1972) calculated load equivalency factors for cement-treated bases and demonstrated that they were dependent on the structural layout of the pavement. Figure 2.7 shows the calculated equivalency factor of an 80 kN wheel load to be in excess of 10 000 in some structural layouts. This means that the existing concept of expressing an axle load spectrum as a certain number of equivalent 80 kN axles for a pavement with cement-treated layers is probably incorrect and not applicable. The current idea of accommodating different wheel load intensities and thereby designing for the expected traffic, is discussed in Chapter 8.

(c) Number of load repetitions

Since the number of expected load repetitions cannot be accommodated by calculating E80, some other way had to be devised to accommodate it. This is detailed further in Chapter 8, but it assumes the applicability of Miner's Law and the procedure developed by the PCA for the design of concrete pavements.

2.2.8 Testing method

The CBR pavement design procedure requires a CBR of 80 for basecourse material. Since this quality of natural material is not available everywhere, cement-treatment has had to be used. In South Africa the CBR test was initially used to evaluate the properties of cement-treated materials (Fossberg and Gregg, 1963) but, since treating the natural materials (soils) required additional expenditure, it was decided that the cement-treated material should be significantly better than the required CBR of 80. To ensure the improvement, a factor of safety of 3 was required and the CBR of the cement-treated base had to be 240. It should however be remembered that the CBR test was developed to test the bearing value of subgrade soils (Porter, 1938) with CBR values generally below 100, and that extrapolating it to test cement-treated bases at CBR values of 240, was taking it a bit too far!

In the USA the Portland Cement Association (PCA) developed the wet-dry and freeze-thaw durability tests for cement-treated materials (Norling, 1973). These tests are rather time-consuming and the unconfined compressive strength test soon overtook them as the most popular testing method in the

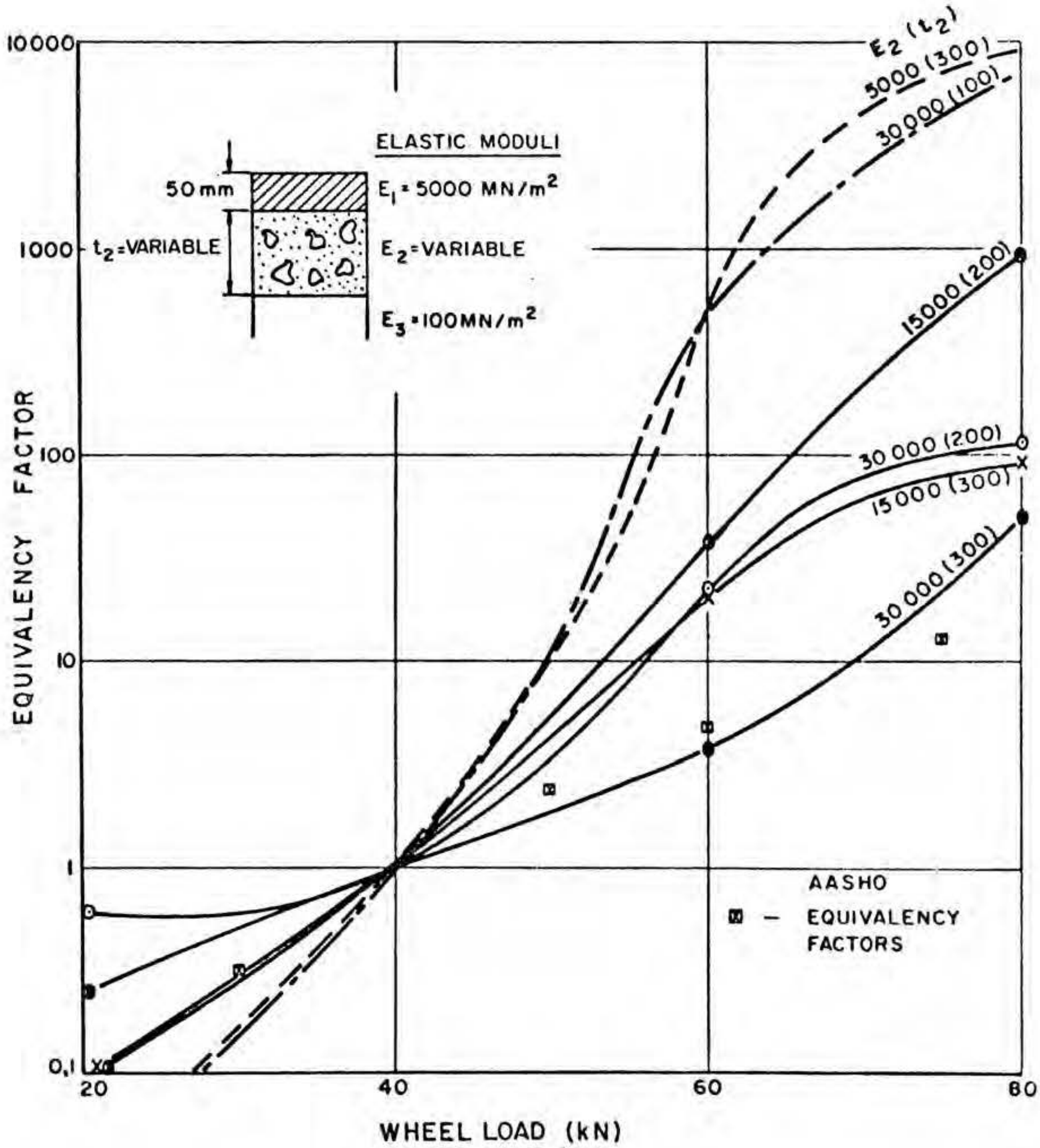


FIGURE 2-7

RELATIONSHIP BETWEEN WHEEL LOAD AND EQUIVALENCY FACTOR FOR VARIOUS CEMENT-TREATED PAVEMENTS

(After Van Vuuren, 1972)

USA (Marais, 1973). South Africa followed suit and the unconfined compressive test has been used since about 1963 to test base and subbase quality cement-treated materials (Fossberg and Gregg, 1963). The CBR test remained and is still sometimes used to test certain cement- and lime-treated soils.

The author believes that even unconfined compressive tests are not really applicable to the structural design of pavements with cement-treated materials. Norling (1973) stated that the compressive strength requirements originally suggested by the PCA were only to observe and prove "...that the cement is functioning normally and that the soil is not interfering with hydration of the cement...". The compressive strength test was suggested as a test for material evaluation and not for structural pavement design purposes. To establish a test for structural design purposes it must be appreciated that the tensile strength of cement-treated materials is much lower than the compressive strength and that this is the weak link - the tensile strength must be evaluated and not the compressive strength.

A measure of the tensile strength can be obtained in at least three ways, namely the direct, indirect and bending tests.

(a) Direct tensile test

The direct tensile test ought to provide the true tensile strength of cement-treated materials (σ_t), but it is a very cumbersome test which will probably never be suitable for routine design and quality control purposes.

(b) Indirect tensile test (BRAZILIAN)

Both Hadley et al (1972) and Mitchell (1976) suggested that the indirect tensile test be used to test cement-treated materials for pavement design work. Hadley et al suggested it because the test is relatively quick and easy to perform, but Mitchell based his suggestion on some finite element work performed by Raad (1976) which indicated that the indirect tensile strength (σ_i) is very much the same as the direct tensile strength (σ_t). The calculations showed the ratio between the indirect tensile strength (σ_i) and the direct tensile strength (σ_t) to vary between 0,8 and 1,0 when the ratio between the elastic modulus in compression (E_c) and tension (E_t) is less than 4, that is $E_c/E_t<4$, and when E_c/E_t is more than 4 the ratio between the strengths varies between 1,0 and only 1,04.

One of the earliest laboratory studies on the relationship between the indirect (σ_i) and direct (σ_t) tensile strengths was performed on concrete (Wright, 1955) and it indicated the ratio as being about 1,5. Johnston and Sidwell (1968) reported that the ratio between the two strengths is dependent on the maximum size and concentration of coarse aggregate in the concrete and that the ratio varies between 1,05 and 1,7. The maximum size aggregate and coarse aggregate concentration may have an effect on the elastic moduli in compression (E_c) and tension (E_t), and if this is correct, the tests by Johnston and Sidwell seem to substantiate Raad's calculations, but the actual ratios differ significantly from the theoretically calculated ratios. Laboratory tests on cement-treated materials reported by Raad (1976) indicated that the ratio varies between about 0,8 and 1,11.

During 1972 the British Department of the Environment changed their standard specification and adopted the indirect tensile test for quality control purposes on concrete in roads. This change generated some opposition and it seems to have reopened the discussion on the applicability of the indirect tensile test (Hannant et al, 1973; Walker, 1974; Orr, 1975; and Williams, 1976), the repeatability of the test (Sherriff, 1975) and the influence of the testing machine on the results (Foote, 1975). Due to these discussions the author is hesitant to use the indirect tensile test.

(c) Bending test

The bending test, also called the flexural test, is the third possible test for obtaining the tensile strength, which is also often referred to as the modulus of rupture. The bending test assumes the applicability of the beam bending theory and that the material has the same elastic modulus in compression and tension, $E_c=E_t$. It is appreciated that the moduli in tension and compression may not be the same, $E_c \neq E_t$, but when the layered elastic theory, for example CHEVRON, is applied it is assumed that the two moduli are the same because only one modulus is required to characterize cement-treated materials, irrespective of whether the material is in compression or tension. This assumption was also made by both Hadley et al (1972) and Mitchell (1976) when they applied layered elastic theory. The author therefore reasons that if this assumption on the moduli is made when the theoretical analysis is performed, it is valid to make the same assumption when the materials are tested. He therefore prefers the bending test.

Another reason why this test is preferred, and this was also mentioned by Morgan and Scala (1968), Hannant (1972) and Walker (1974), is that the layer bends under a wheel load and the test therefore simulates the actual field condition so much better than the other two test methods. The bending test is also relatively quick and easy to perform, and since it is preferred and used for quality control on concrete road construction, it ought to be applicable to cement-treated materials.

The apparatus for performing the bending test is shown in Figure 2.8. The size of the beam sample depends on the maximum size aggregate; if a 25 mm aggregate has been used the beam should be at least 75 x 75 x 450 mm, preferably 100 x 100 x 600 mm. If the maximum size aggregate has been 16 or 13,2 mm, a 50 x 50 x 300 mm sample should be used. The load is applied gradually and measured throughout the duration of the test with a load cell. The resultant deflection of the beam sample is measured at the middle of the span by two linear variable differential transformers (LVDT's). Compression at the reaction points has been eliminated from the deflection measurements by measuring the deflection of the beam relative to a fixed datum, namely the top of the beam prior to load application. The output from the three electronic instruments is continuously recorded throughout the test and this is used to calculate the bending stress, elastic modulus and strain at break.

The bending stress is defined as the bending moment (M) divided by the section modulus (Z) of the beam. By using the load and the resulting deflection, and by applying basic elastic beam-bending theory, the corresponding elastic modulus can be calculated. The strain is calculated as the stress divided by the corresponding elastic modulus. A stress-strain curve is then drawn and the slope of the initial straight-line portion is taken as the static linear elastic modulus in bending, abbreviated to elastic modulus. The bending strength (σ_b) is the maximum bending stress recorded and the strain at break (ε_b) is the strain corresponding to the bending strength.

2.2.9 Design criteria

When pavement design used to be merely a matter of materials evaluation, the design criterion was either a minimum CBR or a minimum unconfined compressive strength. With the introduction of a more fundamental approach to the structural design of pavements, via elastic theory, it became necessary to obtain design criteria for the various materials. Whiffen and Lister (1962) compared the calculated tensile stress to the tensile (flexural) strength and hence used stress as the design criteria, although they also reported on the tensile strain "...corresponding to the onset of

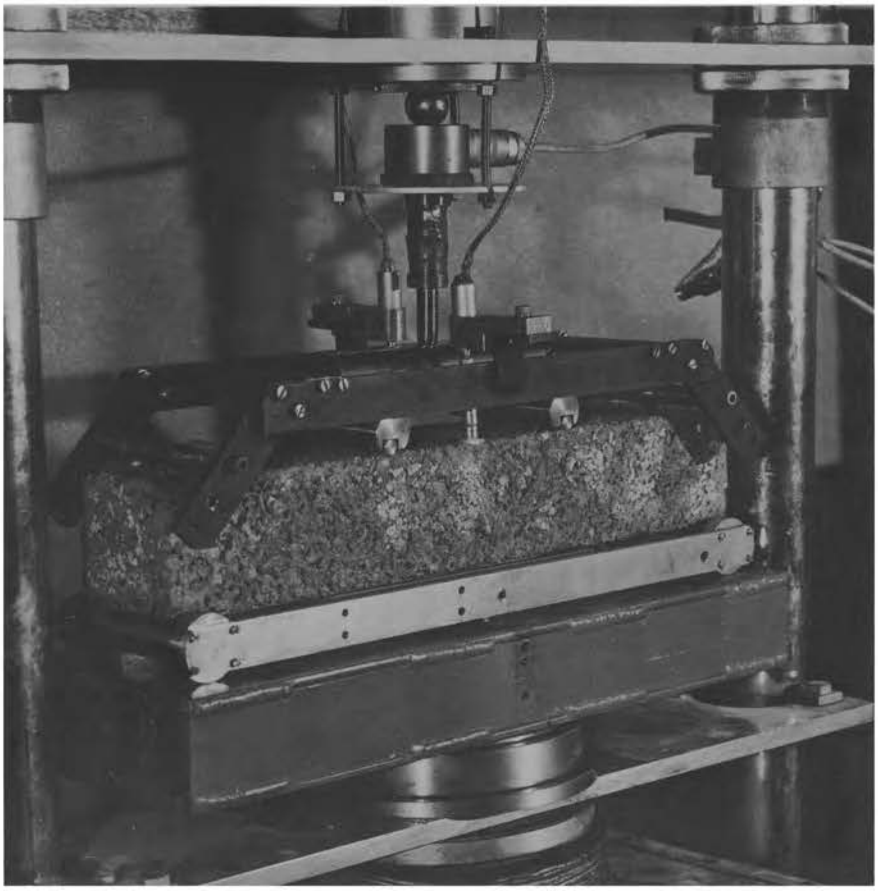


FIGURE 2.8
The bending test apparatus.

hair cracking...", that is, at failure of the beam. Saunders (1964) prepared a theoretical discussion on how elastic theory can be applied to cement-treated materials and he considered horizontal tensile stress at the bottom of the cement-treated layer as a "...most important factor..." and used it as the design criterion. Mitchell and Shen (1967) suggested allowable stress and strain values for cement-treated materials (soil-cement) but used stress as the design criterion because they maintained that due to the linearity between stress and strain and due to the effect of the Poisson ratio on strain, the limiting stress should be reached before the limiting strain. Thompson et al (1972) used stress to evaluate traffic-associated cracking in lean concrete bases. Fossberg et al (1972) used both stress and strain values to compare calculated and measured responses of instrumented cement-treated pavements, but they did not propose a design criterion. Lister (1972) argued that "...failure in cement-bound materials is related more closely to a strain criterion than to one of simple stress.." but he used stress throughout his paper. Hadley et al (1972) used both stress and strain as design criteria, in the design curves they suggested. Wang (1968), Bonnot (1972) and the author (Otte, 1974) observed that the strain at failure varies much less than the strength. This provides an opportunity to specify a design criterion which remains fairly constant and that is why the author (Otte, 1974) suggested that strain should be used as the design criterion and not stress. Strain is also used as the design criterion for asphaltic materials and the subgrade soils. Practising engineers generally find it much easier to visualize stress than strain, which could mean that stress may have to be used as a practical design criterion.

Stress can be used when a particular cement-treated material on a particular project is evaluated, such as the analyses by Whiffen and Lister (1962) and Thompson et al (1972). Strain may be used for the initial design and evaluation of a proposed structural layout when cement-treated materials are considered in general.

A certain part of the confusion as to whether stress or strain should be used as the criterion, originates from the lack of a true understanding of the stress-strain properties and fracture of cement-treated material under biaxial loading conditions. The discussion on stress and strain as design criteria in the previous paragraphs, and the presently used allowable design values were all obtained under essentially uniaxial loading conditions whereas the materials in a pavement are loaded at least biaxially.

One of the first studies on the biaxial performance of cement-treated materials was by Pretorius (1970). He reviewed some of the known failure theories and after a limited laboratory study concluded that "...no single simple failure criterion for the strength of soil-cement under general biaxial stress..." existed at that stage. He nevertheless decided to utilize the distortion energy theory of Von Mises and then developed equation (2.1) to define the failure of cement-treated materials in the tension-compression and tension-tension stress states:

$$\frac{\tau_o}{\sigma_c} = 0{,}0857 + 1{,}157 \left(\frac{\sigma_o}{\sigma_c}\right) - 0{,}512 \sqrt{\frac{I_3}{\sigma_c^{\,3}}} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (2.1)$$

where $\tau_o = \frac{1}{3}\sqrt{(\sigma_1-\sigma_2)^2 + (\sigma_2-\sigma_3)^2 + (\sigma_3-\sigma_1)^2}$; the octahedral shear stress

$\sigma_o = \frac{1}{3}(\sigma_1+\sigma_2+\sigma_3)$; octahedral normal stress

σ_c = unconfined compressive strength

$I_3 = \sigma_1\sigma_2\sigma_3$; the third stress invariant

σ_1, σ_2 and σ_3 = the principal stresses.

Abboud (1973) studied the failure of cement-treated materials under relatively high confining stresses. He stated that "...the existence of the tensile zones (hair cracks) in the hardened material together with the knowledge that failure may be related to stress concentrations around these cracks suggests that the Mohr-Coulomb failure criteria may not be the best tool to be used for failure prediction in these materials...". He performed triaxial tests and found the modified Griffith failure criterion to predict the failure of cement-treated soils in tension and compression "...in a very satisfactory manner...". Raad (1976) followed this through and observed that the Griffith failure criterion applies very well to the tensile and very low stress ranges, while the modified Griffith criterion is more applicable to higher stress levels, that is when the confining stress (σ_3) (for example in a triaxial test) is greater than about 0,1 times the unconfined strength (σ_c) (Mitchell, 1976). From this work the failure of cement-treated materials under biaxial loading conditions can be represented by

$$\sigma_1 = \sigma_c + 5\sigma_3 \quad \text{(2.2)}$$

where σ_1 = major principal stress

σ_3 = minor principal stress (compressive)

σ_c = unconfined compressive strength.

From the above it appears that a failure criterion for cement-treated materials under biaxial loading has been developed. Only once this work has been completed and fully evaluated would it be possible to develop and propose a design criterion and test method for the design of cement-treated materials. In the meantime it is proposed that uniaxial tensile strain be used as the design criterion.

2.2.10 Fatigue life

A literature study (Otte, 1972a) on the static failure of concrete indicated that failure usually starts when the applied stress (σ) has reached about 35 per cent of the strength (σ_b), that is at a stress level (σ/σ_b) of 0,35 (Figure 2.6). The failure starts with microcracking and a loss of bond at the interface between the aggregate and the matrix of fine material and cement and these microcracks propagate under load until the sample collapses. From this it can be concluded that the material would be able to withstand an unlimited number of load repetitions while the stress ratio (σ/σ_b) remains below 0,35 because the applied stress is too low to start the development of microcracking.

Research on the fatigue behaviour of cement-treated materials and concrete performed by 9 researchers was reviewed by Otte (1972a). The conclusions and observations of the various researchers varied but for practical pavement design purposes it may be accepted that concrete and cement-treated materials will withstand about one million load repetitions before failing in fatigue if the applied stress (σ) is about 50 per cent of the strength (σ_b), that is σ/σ_b=0,5. This relationship was also assumed by the PCA when they developed their pavement design procedure (PCA, 1973).

This observation from the fatigue work on cement-treated materials and concrete may be explained from the study on static failure outlined in the first paragraph of this section. If the material is loaded to 50 per cent of the strength (σ_b) microcracking will occur, but at an acceptable rate. The number of load repetitions (N) required to propagate these cracks through the material at this stress ratio (σ/σ_b=0,5) is usually fairly high, around one million, and this represents an acceptable fatigue life. The number of load repetitions to failure (N_f) may vary from sample to sample and this may be the reason why the results from the various researchers on fatigue differed.

It can therefore be concluded that a cement-treated material can withstand one million load repetitions if the stress ratio (σ/σ_b) is 0,5 and an unlimited number of repetitions if the stress ratio is 0,35. The corresponding strain ratios ($\varepsilon/\varepsilon_b$) (Otte, 1972a and 1974) were measured as about 0,33 and 0,25 respectively.

In this thesis strain is used as the design criterion. If a linear relationship between the strain ratio ($\varepsilon/\varepsilon_b$) and the logarithm of the number of load repetitions to failure (N_f) (Pretorius, 1970) is assumed, it is possible to prepare a figure (Figure 2.9a) relating a strain ratio to a number of load repetitions before fatigue failure. This relationship can also be expressed by equation (2.3), namely

$$\varepsilon/\varepsilon_b = 1 - 0{,}11 \log N_f \quad \ldots\ldots \quad (2.3)$$

If a linear relationship between the logarithm of the strain ratio ($\varepsilon/\varepsilon_b$) and the logarithm of the number of load repetitions to failure (N_f) is assumed Figure 2.9b and equation (2.4) may be proposed

$$\varepsilon/\varepsilon_b = N_f^{-0{,}079} \quad \ldots\ldots \quad (2.4)$$

Equations (2.3) and (2.4) may also be written as

$$N_f = 10^{9{,}1(1-\varepsilon/\varepsilon_b)} \quad \ldots\ldots \quad (2.5)$$

and $$N_f = (\varepsilon/\varepsilon_b)^{-12{,}66} \quad \ldots\ldots \quad (2.6)$$

Since the fatigue curve of cement-treated materials is relatively flat, if compared for instance with that of bituminous materials, future research and possible changes in the numerical values proposed in equations (2.3) to (2.6) are not considered very urgent and/or of practical importance. It is therefore suggested that these figures and equations be accepted as general fatigue curves and that they may be used for practical pavement design purposes. In this thesis equations (2.3) and (2.5) will be used.

This was the accepted way of considering fatigue in cement-treated materials, until Raad (1976) very recently suggested a change. He applied the Griffith and modified Griffith theories to the failure of cement-treated materials (section 2.2.9), and from this he proposed a new approach to the fatigue life of cement-treated materials which incorporates the

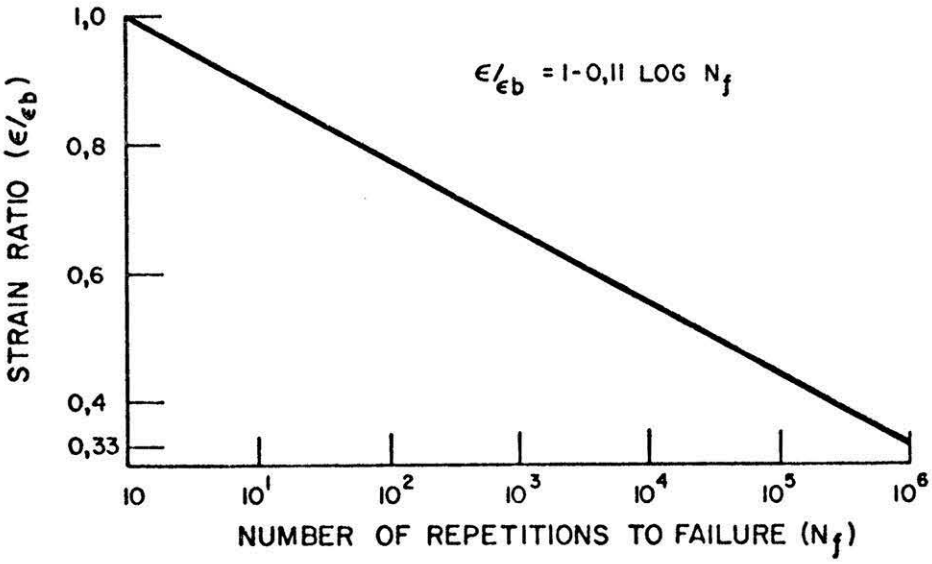


FIGURE 2-9a

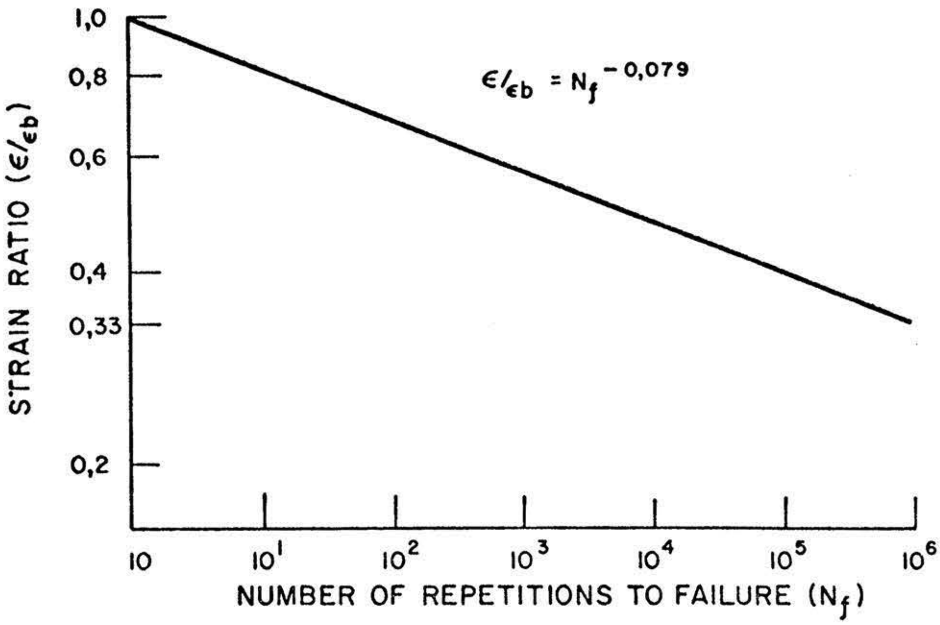


FIGURE 2-9b

GENERAL FATIGUE CURVES FOR CEMENT-TREATED MATERIALS biaxial loading condition. He proposed a relationship between the principal major and minor stresses (σ_1 and σ_3) and the tensile strength of the material (T) and then defined a stress level F/T

where $$F = \frac{(\sigma_1-\sigma_3)^2}{8(\sigma_1+\sigma_3)} \quad \text{when} \quad \sigma_1+3\sigma_3 \geqslant 0 \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (2.7)$$

Note: + for compressive stresses
- for tensile stresses

$$F = -\sigma_3 \quad \text{when} \quad \sigma_1+3\sigma_3 < 0 \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (2.8)$$

He proposed a relationship between the stress level F/T and the number of load repetitions to failure (N_f) which is independent of the shape, width (time of loading or duration) and frequency of the applied stress pulses. He has applied his proposed relationship to some of the published fatigue work on cement-treated materials and it seems to fit the laboratory results obtained by several researchers much better than the simple relationship of stress or strain level versus the number of load repetitions. The application of his laboratory study to pavement design needs further evaluation, especially after the indications by Walker et al (1977) that it does not really apply to the behaviour of actual pavements.

2.2.11 Properties of field- versus laboratory-prepared materials

It is very important that the construction team should produce the quality of material that the designer assumed when he performed the design. If the difference in material properties is significant, the structure may either fail or be totally overdesigned.

To ensure reasonable agreement, the stabilizing agent should be properly and uniformly distributed throughout the material since the amount of stabilizer is one of the major factors contributing to increased strength and quality. This problem has been studied ever since cement-treated materials came into use (Robinson, 1952; Fossberg and Gregg, 1963), and it has been largely solved by improvements in the construction technique and in the mixing machines.

There are also other important construction details such as the mixing, compaction and curing technique and these usually differ from construction site to construction site. Their total and combined effect may be sufficient to result in significant differences in the quality of field- and laboratory-prepared materials, and it is considered important to know how significant these differences are. For a proper design, the design engineer should be aware of the extent of these differences and he should allow for them. At present no allowance is made during the design stages for construction differences and Chapter 6 has been compiled to evaluate their influence.

2.2.12 Variability of materials

When engineers consider a road as having failed it often happens that it has really only failed over certain short sections and not over its entire length. This means that there are long sections still completely acceptable. Certain sections were therefore originally not as strong and durable as the remaining part of the road and this introduces the problem of variability along a road.

The reason for some of the variability resulting in premature distress in certain sections may be poor construction techniques, for example poor backfilling at structures, faulty plant, insufficient or non-existent subsurface drainage, and construction difficulties at the cross-over from cut to fill sections (Grant, 1974). It is considered possible that these sources of variability and failure on roads may be eliminated or reduced by more accurate construction control (backfilling at structures) and better design (subsurface drainage). The structural pavement design engineer should therefore not be too concerned about these sources of variation over the length of the project. He should be concerned about the variability of the project which is dependent on the material variability, for example variation of borrow-pit materials, and he should try to accommodate this in his design procedure.

Kühn et al (1974) collected some information on the variability of South African bituminous, untreated and treated materials and they showed that the coefficients of variation varied between about 0,6 per cent for the percentage compaction to about 80 per cent for the flow of bituminous materials. Otte (1974) gave some results to show the variability in the quality of cement-treated crusher-run between different contracts, although the contractors were working to the same specification. Grant (1974) presented a plot of the variation in surface deflection along a road and he showed the significant differences in the average deflection if the sample is taken only slightly differently. The author ran tests with the La Croix deflectograph over a completed subgrade and the variation in deflection along the road agreed remarkably well with that shown by Grant (1974).

These abovementioned examples of variability in the natural and imported materials confirm the statement by Hudson et al (1974) that materials variation will very often overshadow other factors in the structural design. They suggested a design procedure in which the variability of the materials and subgrade is accommodated and where the designer may "...quantify the goodness of the design by specifying a level of reliability...".

At this stage during the development of a structural design procedure the author considers it as still very difficult to include the variability of road-building materials fully. No effort to do it has therefore been made in this thesis. The procedures proposed by Hudson et al (1974) may be used as the basis for future developments and the author is confident that they will be sufficiently refined in due course. In the meantime the presence of the variabilities must be appreciated and it should be realised that the design method proposed in Chapter 8 cannot accommodate it. The method is based on average material properties, environmental conditions and traffic, and although it is fairly accurate in predicting the load response and behaviour, it has its limitations when used to predict the serviceability of a pavement.

2.3 CONCLUSIONS

(1) From published literature it is possible to outline some of the important aspects and requirements that should be remembered when the structural design of a cement-treated layer is performed.

- (a) The layer should be constructed as thick as possible and necessary, both practically and economically.
- (b) The load-bearing ability of the material may not be quantified by a fixed constant such as a structural coefficient or a gravel equivalency factor since it varies and is dependent on various aspects.
- (c) A sound foundation or support for the cement-treated layer is very necessary and it contributes significantly to the safety thereof.
- (d) All structural layouts must adhere to the concept of a balanced design since it will improve the safety and economics of the layout.
- (e) Temperature stresses in the cement-treated layer may be important and their possible influence should be minimized.
- (f) The presence of non-traffic-associated, also referred to as initial, cracking should be accepted and allowance should be

made during the structural design to minimize the possible adverse effect thereof.

(g) The layer should be designed to withstand the heavy axles expected on the road and the lighter loadings may be disregarded. This is because the AASHO load equivalency factors are not applicable to cement-treated layers.

(h) For structural design purposes the cement-treated material should be tested in a bending test and not in an unconfined compression, direct tensile, indirect tensile, or a CBR test.

(i) The failure criterion for the material has not been finally established. Some authors have suggested stress, and some strain, as a design criterion. In this thesis uniaxial tensile strain will be used, although bending strength ought to be acceptable in some cases for a particular material and project. Significant progress has been made towards the development of a biaxial failure criterion for cement-treated materials, but the general applicability of this criterion still has to be verified.

(j) The fatigue life of cement-treated materials should be evaluated and included in a pavement design procedure. It is suggested that the simple relationship between uniaxial strain level and the number of load repetitions be used until engineers become more conversant with the biaxial loading conditions.

(k) The difference between materials constructed in the field and those prepared under ideal conditions in a laboratory may be significant and some allowance must be made to accommodate this.

(l) The variability of the construction materials, both the natural materials in the subgrade and those imported for the selected and treated upper layers, is important when the overall performance of the road is to be predicted.

(2) It can be appreciated that none of the existing pavement design procedures (for example, CBR, State of California or AASHO) can make provision for all these requirements. They may therefore be considered inadequate and not completely suitable for the structural design of pavements with cement-treated layers.

CHAPTER 3

CRACKING IN CEMENT-TREATED MATERIALS

3.1 INTRODUCTION

Cracking in cement-treated materials has been, and will continue to be, a major cause of concern. To some people a crack in a material is a sure sign of weakness, but this need not necessarily always be true. The cracking in the cement-treated crusher-run on Special Road S12 was followed by severe distress (Otte, 1973a) and this led to lengthy discussions amongst South African engineers on cracking in cement-treated materials. Especially important were the discussions on possible ways to avoid cracks and, if they could not be avoided, what to do once they had occurred.

Overseas studies on the performance of pavements with cement-treated materials (Brewer and Williams, 1968; and Wright, 1969) were very favourable and reported that the materials behaved satisfactorily. They also claimed that cracking should not be "...regarded as a serious defect...". The increasing use of layered elastic theory in pavement design, wave propagation measurements by Jones (1963) and the interpretation of these measurements by Pell and Brown (1972) have sparked a further interest in the cracking of cement-treated materials.

Jones (1963) observed that the cement-treated materials in a particular pavement cracked badly and the average elastic modulus was reduced from about 27 600 MPa to 410 MPa over about 4 years. Pell and Brown (1972), after interpreting these results, questioned the relevance of laboratory tests on uncracked samples and suggested that the appropriate elastic modulus for pavement design purposes should be only 500 MPa. This relatively low elastic modulus, which corresponds to the value commonly used for untreated crusher-run, introduced economics and questioned the structural benefits of using cement-treated materials. Pell and Brown (1972) also mentioned that an investigation would be needed to account properly for the effect of cracks or perhaps, if possible, to control or eliminate them.

This chapter is a general discussion on aspects of cracking in cement-treated materials and it puts forward a philosophy on how cracking should be accommodated in pavement design. An attempt is made to analyse the phenomenon of cracking and the rather pessimistic suggestion of Pell and Brown.

3.2 TYPES OF CRACKING

It is always very important to have a clear understanding of why a crack has developed in a pavement. From this understanding it is possible to analyse the likely causes and consequences and hence to understand the behaviour of the pavement. When cement-treated materials are used, this is even more important because it is so easy to confuse the various types of cracking.

There are two major types, namely initial and traffic-associated cracking, and one should distinguish clearly between them. If this is not possible confusion will arise over material and structural design aspects.

A possible example of such confusion is the following statement by George (1974): "Cracking within the base is, therefore, the first cause of structural deterioration in a cement-bound pavement." It really depends on the definition of structural deterioration, but there are numerous pavements (Otte, 1973a) that have exhibited initial cracking which have performed superbly with no subsequent traffic-associated cracking. It is considered inaccurate to claim that these pavements have undergone structural deterioration.

3.2.1 Initial cracking

Initial cracking is caused by factors other than traffic and is usually the result of either environmental stresses or a geotechnical problem such as a subsiding embankment or heaving clays. In this chapter only environmental stresses are considered, which include drying-shrinkage and temperature stresses. Some people claim that the initial cracks are the result of drying-shrinkage (George, 1968) and Pretorius (1970) has even developed a finite element computer program that can predict the crack spacing from laboratory-determined shrinkage and creep curves, while others claim that it is caused by thermal stresses (Williamson, 1974). In this thesis no discussion will be made on why the cracks occur, or at what spacing and width, it is merely accepted as a fact that initial cracks will occur - probably as a result of the combined effect of drying-shrinkage and tempe rature stresses. There have been studies on possible ways of eliminating or reducing the amount of cracking in the material (George, 1968; Wang, 1973; and Norling, 1973a) but the author believes that a properly constructed cement-treated material will crack and this fact has to be accepted. Figure 3.1 shows the extent and width of initial cracking in cement-treated crusher-run.

3.2.1.1 The effect of construction on initial cracks

Some engineers consider it possible to avoid initial cracking merely by juggling with the construction procedure. On one particular contract the initial cracking in a cement-treated crusher-run layer was avoided,

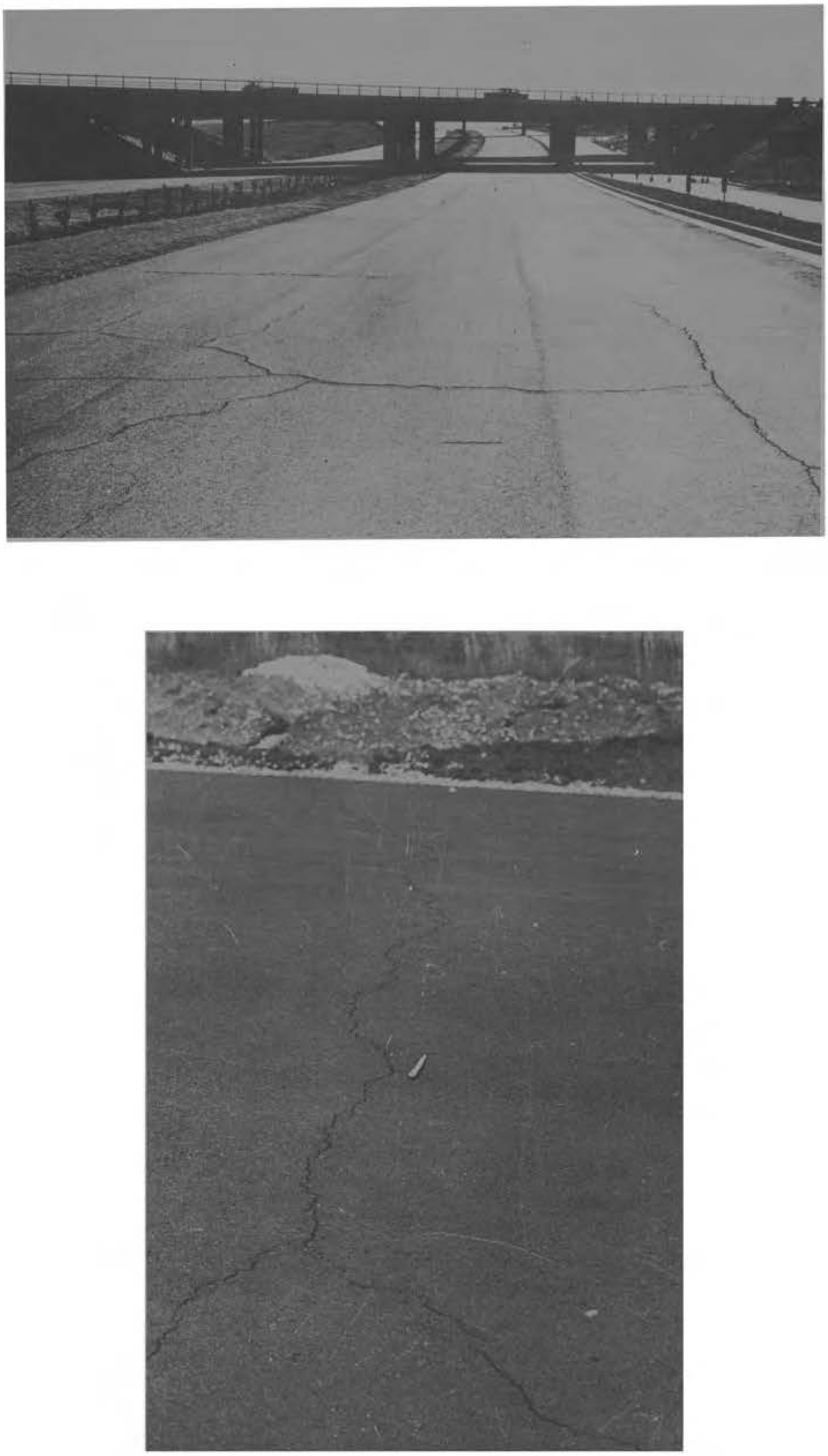


Figure 3.1

The extent and width of initial cracking in cement-treated crusher-run.

probably because the layer was given additional watering, compaction and brooming (called slushing) immediately after its construction. In this particular case all the required cement was added to the material initially but, in an over-zealous effort to reduce or avoid the cracking, slushing was allowed and most of the cement was washed out and broomed away.

On another contract the effect of construction and its variabilities on the extent of cracking in cement-treated materials was vividly displayed when two adjoining and virtually untrafficked sections showed a significant difference in the amount of cracking. The one section, that is the upper third of Figure 3.2, had numerous initial cracks and these formed rectangular blocks of about 2 m by 2 m on the surface. In the other section, that is the lower two-thirds of Figure 3.2, just about no cracks appeared on the surface of the pavement. Block samples (Figure 3.3a) were recovered from these two sections and sawn into beams in the laboratory (Figure 3.3b). The bending strengths, strain at break and elastic moduli of the beams were determined according to the procedure described in section 2.2.8(c) (page 25) and the results are summarised in Table 3.1.

TABLE 3.1 : Physical properties of intact material recovered from cracked and uncracked cement-treated crusher-run

	BENDING STRENGTH (kPa)	STRAIN AT BREAK (με)*	STATIC ELASTIC MODULUS IN BENDING (MPa)
Cracked section+	1 330(28)	251(13)	9 655(21)
Uncracked section	405(22)	216(16)	3 715(35)
Averages are reported and coefficients of variation (in per cent) are shown in brackets)			
+These were not cracked samples. They were intact samples cut from an area between cracks on a section of road that showed cracking. *με is microstrain, that is 10^{-6} mm per mm.			

The Student's t-test, at a 5 per cent level of significance, showed the differences in the bending strength, strain at break and elastic modulus to be significant and that the cracked section had higher values than the uncracked section and this implies a difference in material quality. The difference can also be observed in Figure 3.3 (a) and (b) which shows that the material from the uncracked section (left) was relatively soft and when sawn the edges cumbled and ravelled, while the material from the cracked section (right) was relatively hard and strong, resulting in sharp and well-defined edges.

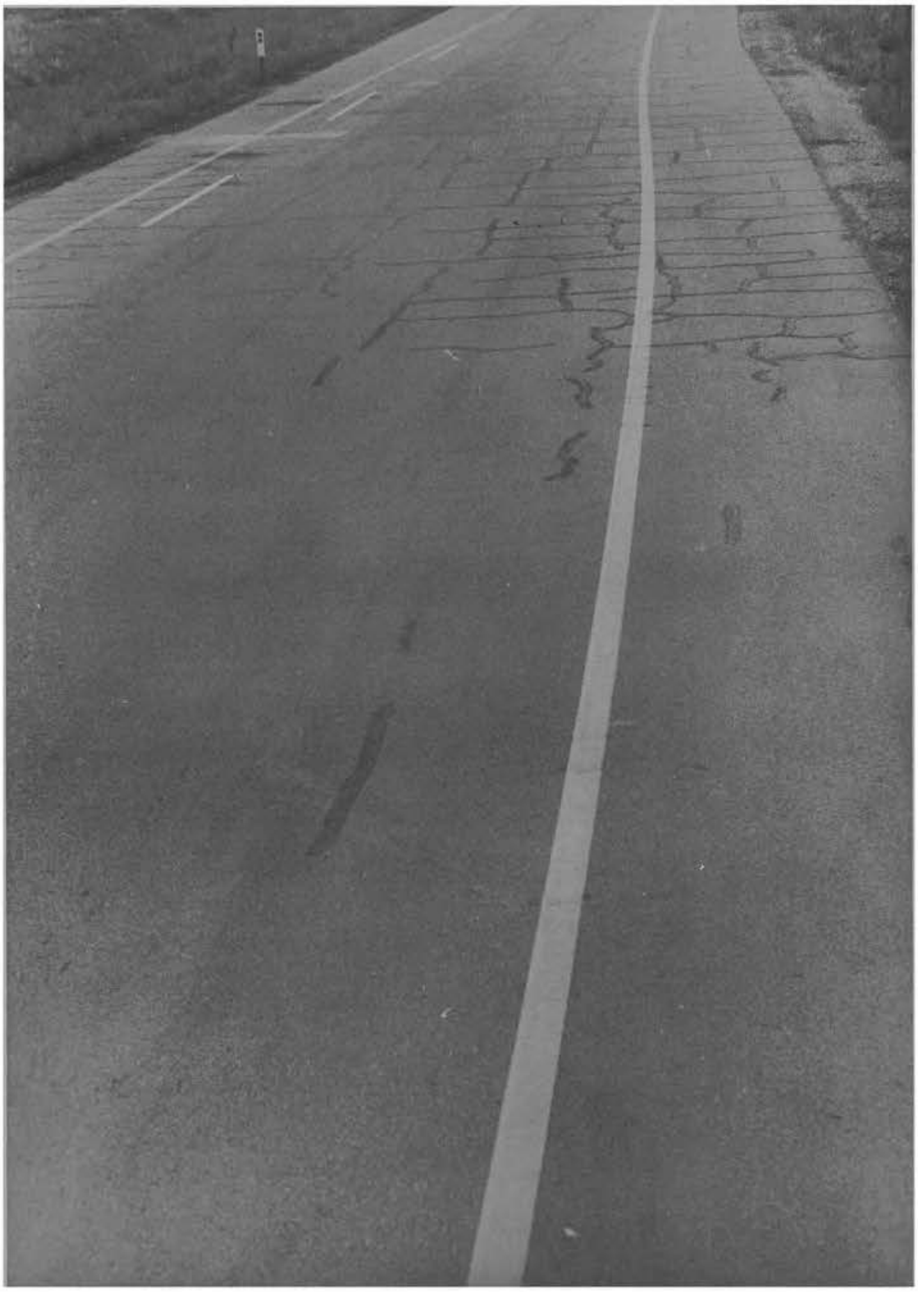


Figure 3.2

Two adjoining sections — the one cracked and the other uncracked.
The cracks were sealed with bitumen emulsion.

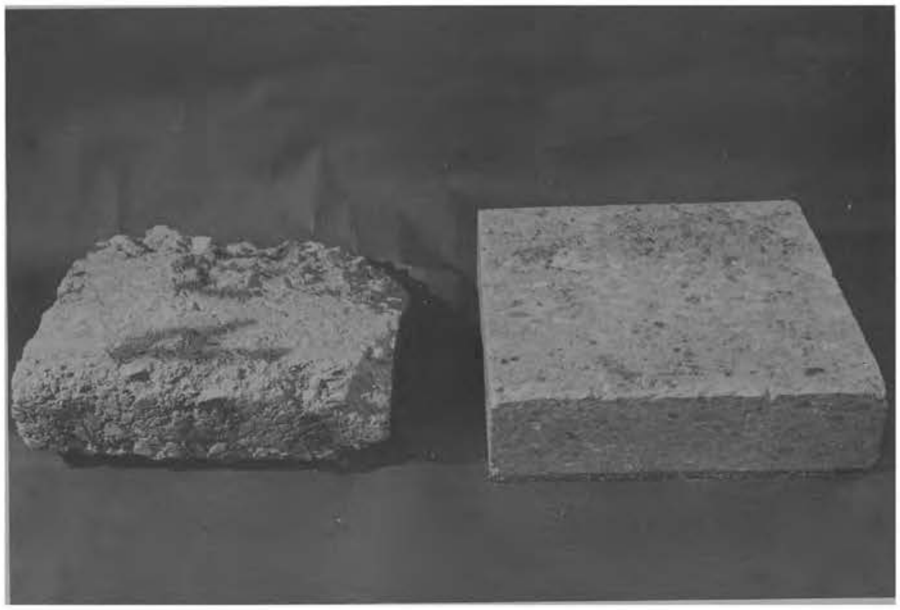


Figure 3.3 (a)

Blocks recovered from the uncracked (left) and cracked (right) sections.

Figure 3.3 (b)

Beams sawn from the uncracked (left) and cracked (right) sections

The two sections shown in Figure 3.2 were nominally the same since they were part of a contract performed on standard specifications with the usual control by the client, but somewhere something changed, probably during construction, and two vastly different end products were achieved right next to each other. The differences observed in cracking and material quality tie in with the general belief that a strong treated layer will have a few wide cracks at a wide spacing, whereas a weaker material will have numerous narrow cracks at a closer spacing.

The importance of a proper construction procedure cannot be overemphasized. Poor quality treatment can be visualised if the contractor keeps on mixing and agitating the materials for too long; or if the trucks bringing in the following layer of material are allowed onto the newly constructed layer too soon; or if slushing is allowed; or if an ineffective curing technique, or no curing at all, is employed. These examples, which from time to time do occur on construction sites, emphasize the need for a proper construction procedure. In the first two instances the initial cementitious bonds will be broken and the ultimate properties of the material will be unknown, but in the other instances, the cementitious bonds may not even be formed. Yamanouchi (1973) did however claim that letting traffic onto the newly constructed sections "...is better..." as far as reducing cracking is concerned, but the author believes that under these conditions the long-term performance and behaviour of the cement-treated layer and the pavement would be rather suspect and unpredictable.

3.2.1.2 Reflection of initial cracks

It is possible to have cracks in the material but to hide it from the public eye. This can be done by using the upside-down design (Otte and Monismith, 1976) or a thick bituminous surfacing layer (Norling, 1973a). In these cases the cracks are present in the lower cement-treated layers but their reflection to the surface is prevented or retarded, and since they cannot be seen, they cause very little concern and are largely forgotten. In South Africa an untreated crusher-run layer placed over the cement-treated layer has been found to dampen reflection cracking effectively. Some British engineers have reported (PIARC, 1971) that it takes about 1 to 2 years for cracks to reflect through a 50 mm bituminous surfacing and that it appears more quickly through high stone-content wearing courses than through those with low stone contents.

3.2.1.3 Thermal stresses and initial cracking

Engineers used to believe that temperature-induced stresses were very significant in cement-treated layers (Lister, 1972). In Chapter 5 this is confirmed for newly-constructed uncracked pavement layers and it is shown that thermal stresses may be the cause of initial cracking. Once the initial cracks have formed, the thermal stresses become negligible relative to the traffic-associated stresses, because movement can take place at the cracks and this will prevent a stress build-up. In the past it was only possible to calculate warping and end-restraint stresses but the computer program (Williamson, 1972 and 1972a) used to prepare Chapter 5 was capable of accommodating the crack width. It made provision for the thermal expansion first to close the crack opening and the remaining expansion to cause an increase in stress. The initial cracks should therefore really be regarded as advantageous because they reduce the thermal stresses in pavements. Thermal stresses are significant insofar as their contribution to the occurrence of initial cracking is concerned, but after cracking (and cement-treated layers usually are cracked), they become unimportant and may be disregarded for pavement design purposes.

3.2.1.4 Initial cracks and ingress of water

One of the major disadvantages of the reflected initial cracks is that water can penetrate them and may become trapped in the pavement. This may lead to softening of the materials in the pavement, overstressing of the other layers and distress in the pavement. If the water could be kept out or be drained away through either a free-draining subgrade (for example sand) or specially provided drainage layers, the pavement would remain relatively dry. If this is possible the crack will probably have a minor influence with only a small effect on the pavement's behaviour.

This can be argued from both theory (Appendix A) and experience (Otte, 1973a) but a recent test with the Heavy Vehicle Simulator (HVS) (Van Vuuren, 1972a and 1973) provided an opportunity to prove it (Chapter 7). To prepare for the test, which was performed on a particular section of National Route N4/1, between Pretoria and Bronkhorstspruit, a saw-cut was made right through the cement-treated base. This produced a very wide crack with no particle interlock and hence no load transfer across it. After 93 000 repetitions of the 55 kN wheel load, the deflection next to the crack increased from 158 μm to about 290 μm. During the following 82 000 repetitions 142 mm of rain fell and the deflection increased to about 960 μm. The deflection was thus approximately doubled during the period of no rainfall, but it was increased about 3,5 times after the rain. This marked increase can be appreciated even more if it is compared with the response of the other test points on the test section (Figure 7.14, page 145). From this test it may be argued that the wide crack had relatively little influence on the deflection (which may be taken as a measure of behaviour) while the section was kept dry, but as soon as water penetrated the pavement the crack had an adverse effect.

A theoretical study (Appendix A) showed that pumping, which can happen after ingress of water, is not always the cause of the traffic-associated cracking - it merely accelerates the rate of cracking. This means that it is possible to handle ingress of water and softening of the lower layers theoretically and analytically. The possible adverse effect of the cracks should therefore not be accommodated by the significant reduction in elastic modulus proposed by Pell and Brown (1972). It should rather be handled by (i) providing some form of blanketing layer which will prevent the cracks from reflecting to the surface and hence prevent water from penetrating, and/or (ii) by a sophisticated theoretical analysis and provision for maintaining structural integrity of the pavement materials after water penetrates and softens the areas adjoining the crack.

3.2.1.5 Pavement response and initial cracking

The effect of an initial crack on the surface deflection and stress distribution in a pavement, called the response, was studied by Fossberg, Mitchell and Monismith (1972a). They compared the response of two instrumented pavement sections, the one cracked and the other uncracked. Their result indicated that "...Vertical deflections were greater by about 20 per cent, and the subgrade stresses directly under the load were greater by at least 50 per cent in the cracked section than in the uncracked pavement. Cracking had a large influence on horizontal strains near the crack in the base, but had only a small influence on strains in the asphalt concrete surfacing.' They also made a saw cut through the 125 mm bituminous surfacing and 212 mm stiff soil-cement base, and performed plate bearing tests at 600 and 200 mm from the cut. Loading 600 mm from the cut increased the vertical stress in the subgrade near the cut by about 40 per cent but it had a negligible effect on the vertical deflections. Loading 200 mm from the cut increased the vertical stress in the subgrade by about 100 per cent and the vertical deflections by at least 60 per cent.

The 60 per cent increase in vertical deflection agrees remarkably well with measurements taken on National Route N4/1 (Table 7.11, page 144) when preparing for the HVS test mentioned earlier. Before the cut was sawn the average Benkelman beam deflection next to the proposed cut was about 100 μm, and after the cut was sawn it increased to 158 μm. Notwithstanding the amount of scatter obtained during the measurements, the 58 per cent increase was calculated to be significant at the 5 per cent level of significance.

The increase in deflection caused by a saw-cut (that is a wide crack) is significant, but the increase caused by a crack would be very difficult to detect because of load transfer across it. This ties in with the conclusion of Fossberg et al (1972a) that "...within the reasonable range of experimental error, deflection measurements cannot be used to detect cracking..." in a cement-treated layer in a pavement. Thus it does appear that cracking, as distinct from a wide saw-cut, has only a limited effect on one of the parameters of pavement response, the deflection, but it has a significant effect on stress distribution.

3.2.1.6 Cracking and material characterization

A method sometimes suggested for incorporating the effect of initial cracking is to reduce the elastic modulus of the cement-treated material - even down to the equivalent of an untreated crusher-run (Pell and Brown, 1972). This produces a material characterization problem because of uncertainty about what to choose as design criteria. It is no longer possible to use the strain values which were obtained for intact materials (Otte, 1974) since the calculated strain in the cement-treated layer increases significantly with a reduction in the elastic modulus (Figure 2.4, page 15). This will indicate all the cement-treated layers as being overstrained and likely to fail! Neither is it logical to do the structural analysis with an equivalent crusher-run, and to expect it to comply with the design criteria of intact materials.

This also leads to economic considerations. If it is possible to do the design and safeguard the other materials in the structure with an equivalent crusher-run, why then is it necessary to provide a cement-treated layer, especially since it is generally more expensive than an untreated layer? The cement-treated layer is often only required as a working platform during construction, and/or because of some deficiencies in the quality of the available natural materials. Nevertheless, whatever the reason for its provision, it is considered sound economics that if a certain material is provided, every effort should be made to maintain the strength and stiffness of the material throughout the life of the pavement. This can only be beneficial.

3.2.1.7 Design to accommodate initial cracks

Although there have been attempts to reduce or eliminate initial cracking (George, 1968; and Wang, 1973) and although it is important to study shrinkage and creep and thermal stresses in cement-treated materials (Dunlop, 1973; Pretorius, 1970), there appears to be no way of avoiding initial cracks. Their presence has to be accepted as a fact! The major consideration is what happens to the serviceability of the pavement after the development of the cracks - this means that once the cracks are there, what can be done to accommodate them and safeguard the pavement? Further research on ways of eliminating the cracks, by people interested in the material aspects, should not necessarily be stopped, but ways must be devised by structural analysts to accommodate cracks in pavement design. The author believes that the clear distinctions that have been drawn recently between material characterization (that is material aspects) and structural analytical aspects, provide the opportunity for the accommodation of the initial cracks. Pell and Brown (1972) believed that the provision for the cracks should be made in the material characterization phase, but the author believes that this should be done during the structural analysis.

Although the author agrees that the testing and material characterization should represent the field conditions as closely as possible (that is the soil mechanics approach), the inclusion of an initial crack may not be represented by an arbitrary reduction in the elastic modulus of the material - it must be accommodated during the structural analysis by the proper stress distribution calculations (that is the structural engineering approach).

The initial crack must be accommodated by a more accurate modelling of the cracked pavement structure and hence a more fundamental structural analysis rather than by an arbitrary reduction in elastic modulus. The comments by Marais (1974) and Pell (1974) that the effects of shrinkage and thermal stresses were neglected during a study of the material properties of cement- and lime-treated materials (Otte, 1974) were therefore irrelevent. It is appreciated that these effects should be considered but not when the fundamental stress-strain properties of the material are studied; it must be included where it belongs and that is during the structural analysis. Chapter 4 describes how this should be done.

3.2.2 Traffic-associated cracking

To include and consider traffic-associated cracking (Figure 3.4) in cement-treated materials is much more difficult than to include the non-

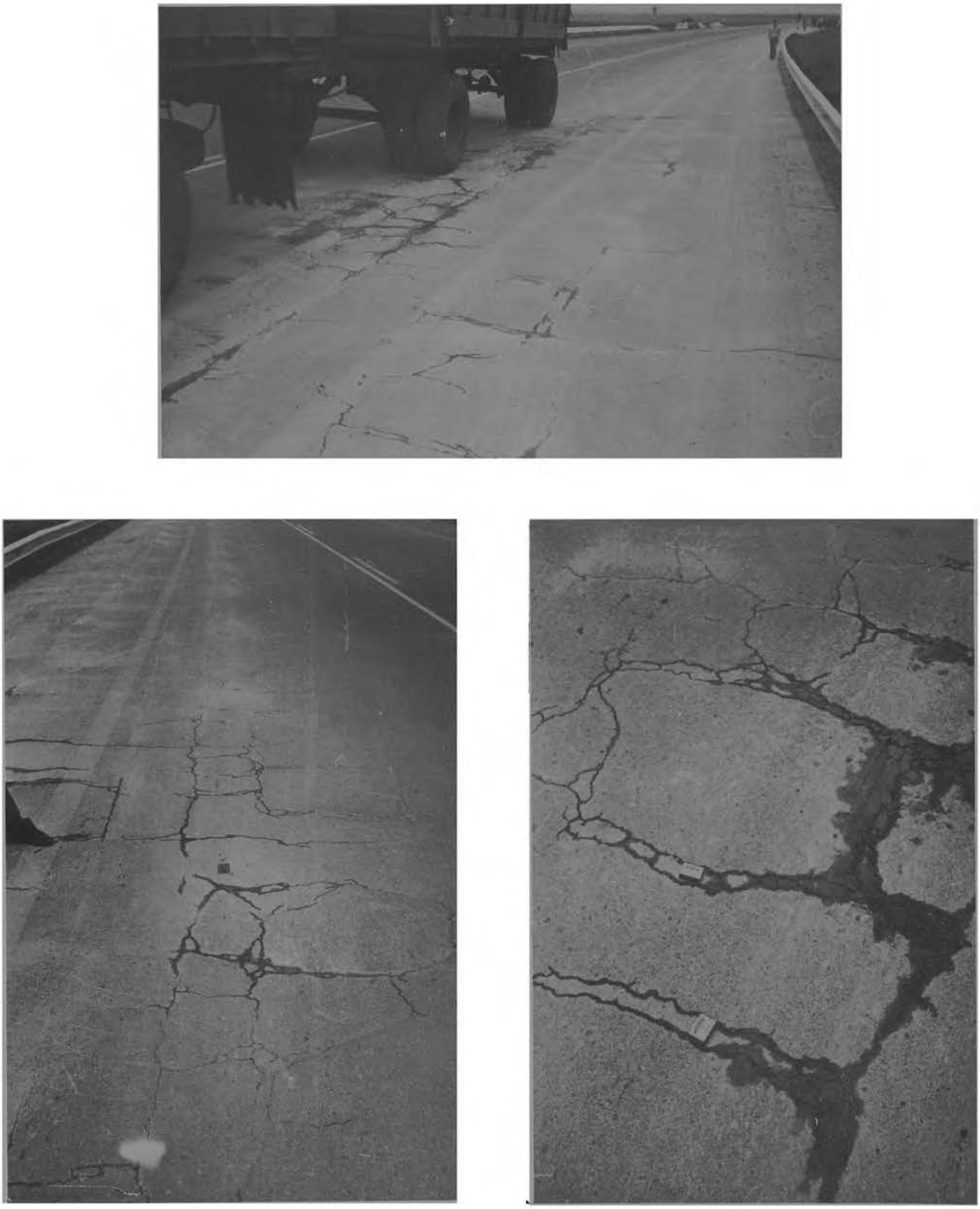


Figure 3.4

Traffic-associated cracking in cement-treated crusher-run. The dark spots are water.

traffic-associated initial cracking. The crack propagation phase in bituminous materials has been studied by various people (Freeme and Marais, 1973), but there has been very little similar study for cement-treated materials. This is probably because cement-treated materials are brittle and the crack propagation phase ought to be very short.

It is therefore suggested that the relatively short crack propagation phase be neglected for structural design purposes. The material should be considered as either (i) intact, but with the initial cracks, or (ii) cracked because of traffic loading. While the cement-treated material is intact it should be handled as explained in section 3.2.1. Once severe traffic-associated cracking has taken place the material may be considered as the equivalent of an untreated material (Otte, 1972b).

When performing the pavement design it is considered incorrect to assume that all cement-treated layers have been completely cracked by traffic-associated stresses. The instance from which Pell and Brown (1972) drew their conclusion was the cracking and unacceptable performance of a thin cement-treated base (75 mm) on the Alconbury Hill experiment. This layer was severely overstressed by traffic (Thompson et al, 1972) and traffic-associated cracking was inevitable. Similar cases of overloading, with the resulting severe traffic-associated cracking, were reported by the author (Otte, 1972b and 1973a). It is incorrect, and it would be very expensive, to consider all cement-treated layers as completely cracked just because traffic-associated cracking developed in a few particularly overstressed sections!

There are numerous cement-treated layers which have exhibited initial cracking (and several have been tested under the Heavy Vehicle Simulator, Chapter 7) but which have shown no signs of traffic-associated cracking. The reason why they did not have traffic-associated cracking is that the materials were not overstressed by the traffic loads!

3.3 DISCUSSION

Cracking in cement-treated materials has always aroused different feelings in people. To the layman, and some engineers, this is a sure sign of weakness or failure, but this is not always true. There are very sound reasons for cracking, both initial and traffic-associated, and provided these are understood and taken into account, cracking should cause very little concern.

The development of initial cracks should be accepted as an unavoidable fact. Cracks should be accommodated during the structural design stage and not during material characterization. By performing a proper structural analysis it is considered possible to avoid traffic-associated cracking and cracking can hence be stopped after the initial cracks have developed. The layer may thus be considered as consisting of large blocks of intact cement-treated material. It is therefore considered inaccurate, unnecessary and uneconomical to equate the structural ability of the material with that of untreated crusher-run.

3.4 CONCLUSIONS

(a) It is very important that a clear distinction should be drawn between initial cracking and traffic-associated cracking. Frequently this distinction is overlooked and this creates serious confusion, especially when cracking in cement-treated materials is considered.

(b) Initial cracking, also called non-traffic-associated cracking, in a properly constructed cement-treated layer has to be accepted as a fact and ways should be found to accommodate it during the structural design of pavements.

(c) Provision for initial cracks in cement-treated layers should be made by doing a proper structural analysis and not by reducing the elastic modulus of the material.

(d) The elastic modulus of the cement-treated material should be obtained frcm tests on intact samples.

(e) It is considered possible to prevent traffic-associated cracking by applying an appropriate design procedure.

CHAPTER 4

THE ANALYSIS OF A CRACKED ROAD PAVEMENT

4.1 INTRODUCTION

The treatment of road-building materials with cement or lime has always been very popular because of the increased strength of the treated material. Both of these materials do however have the tendency to exhibit initial cracking soon after construction (Otte, 1976). For structural design purposes they should therefore be considered in the same way and a general term for both, treated material, will be used in this chapter.

The extent and width of the initial cracks in pavements with cement-treated bases were shown in Figure 3.1 (page 40). The author measured cracks of up to 10 mm wide in cement-treated crusher-run, but the width generally depends on the quality of the treated material and the position of the layer in the structural layout.

The effect and influence of initial cracks on the performance of pavements could, until recently, only be studied and evaluated by visual observation and speculation (Brewer and Williams, 1968; Otte, 1973a; George, 1974). It is difficult to quantify the effect of a crack since cracked layers are no longer continuous and the available computer programs, for example CHEVRON and BISTRO, are therefore really no longer suitable to do the analysis. This fact increased the complexity of the design of treated layers immensely, but a recent development in the finite element analysis technique may be one of the keys to the solution of the problem.

The objectives of this chapter, which includes a fair amount of detail, are (i) to indicate the application of finite element analysis to the study of a pavement containing a cracked treated layer, (ii) to make a tentative recommendation on the procedure that should be adopted to determine possible stress increases, (iii) to study previous applications of the finite element program to this problem, and (iv) to determine the extent of the stress increase in some of the typical layouts used in South Africa. To be able to follow and comprehend this chapter easily some understanding of finite element analysis is necessary, and it is assumed that the reader has this.

4.2 PRISMATIC SOLID FINITE ELEMENTS AND CRACKED TREATED LAYERS

4.2.1 The program

The finite element program used in this study was developed by Wilson and Pretorius (1970) and it employs constant strain prismatic solids. The latter were defined as three-dimensional solids which have constant two-dimensional geometric shapes and infinite third dimensions. The loading into the third dimension is achieved by a Fourier series and this makes the program essentially three-dimensional.

Figure 4.1 explains some of the terminology, for example period length and loaded length, required to run the program. It should be noted that since the prismatic solids are continuous in the Z-direction, the crack can only occur in the YZ-plane. Some additional explanations of the program and information on the preparation of the input data are given in Appendix B.

4.2.2 Layout considered

It was decided to use only one structural layout to study the various ways of analysing a pavement containing a cracked treated layer. The chosen design is shown in Figure 4.2 but since the thin surfacing was considered to play only a minor role in the stress distribution, it was finally decided to analyse the structure shown in Figure 4.3.

The elastic moduli used for the various materials are given in Table 4.1 and a constant Poisson ratio of 0,35 was used for all the materials.

TABLE 4.1 : Elastic moduli of the materials considered in this analysis

MATERIAL	ELASTIC MODULUS (MPa)
Crusher-run	500
Treated layer	4 000
Subbase	500
Selected fill	200
Subgrade	100

4.2.3 Loading condition

The maximum legal wheel load in South Africa is 40 kN and a representative tyre contact pressure is 500 kPa. The prismatic solids program requires the load to be applied to a rectangular area, and an area of 270 x 296,3 mm was chosen. The CHEVRON program calculated the radius of the circular loaded area to be 157,81 mm.

4.2.4 Finite element representation

After careful planning and incorporation of the suggestions by Dehlen (1969) for the finite element representation of a pavement, it was decided to use the mesh shown in Figure 4.4. The total length of the section under consideration was taken as 9 000 mm with a height of 7 025 mm. The element sizes were chosen as shown, six elements underneath the load, six

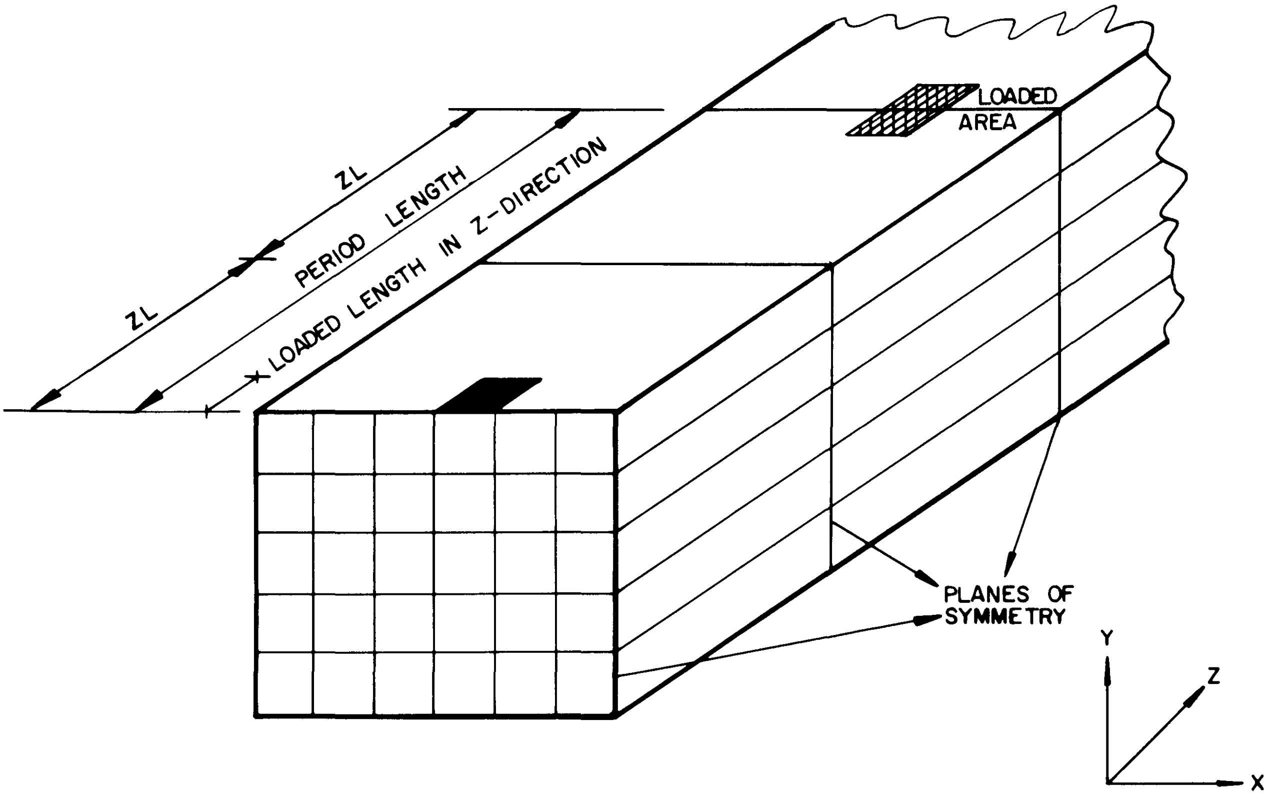


FIGURE 4-1

DEFINITION OF TERMINOLOGY REQUIRED TO RUN THE PRISMATIC SOLID FINITE ELEMENT PROGRAM.

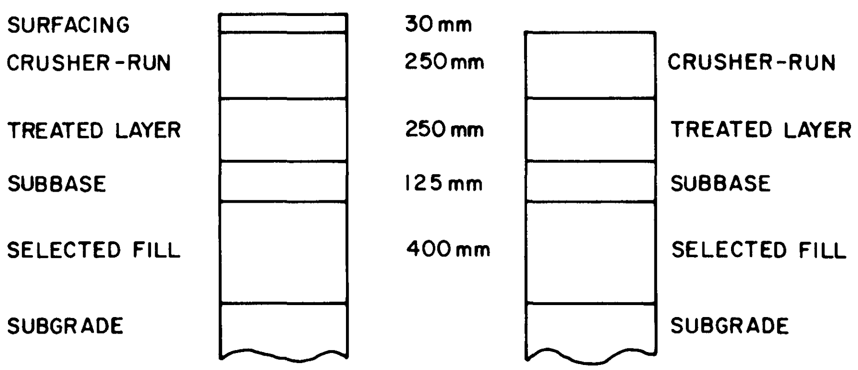


FIGURE 4-2 FIGURE 4-3

STRUCTURAL LAYOUTS CONSIDERED IN THE ANALYSIS OF A PAVEMENT CONTAINING A CRACKED TREATED LAYER

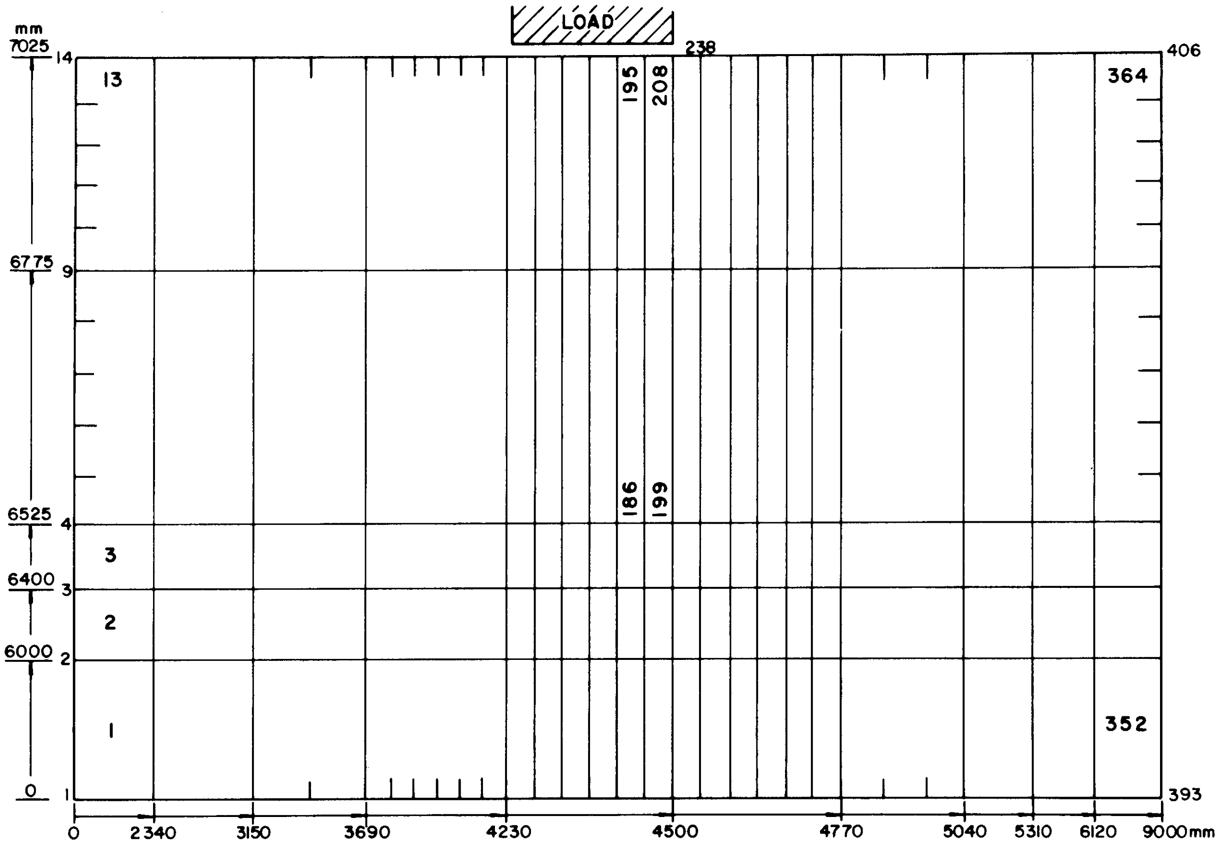


FIGURE 4-4

FINITE ELEMENT MESH - UNCRACKED CONDITION elements in twice the loaded width (540 mm) on the left hand side of the loaded area and six in the loaded width (270 mm) on the right hand side of the load. The two upper layers were each divided into five 50 mm layers.

In another study (Otte, 1975) an axisymmetrical finite element program was used to analyse the mesh representing the uncracked pavement (Figure 4.4) and it was compared with a layered elastic theory analysis, namely CHEVRON. The differences in the stress values in the vicinity of the load were calculated to be about 7 per cent. This indicates that Figure 4.4 is an accurate and acceptable finite element representation of the pavement.

To run the prismatic solid finite element program it was necessary to make decisions on the following input requirements described in Appendix B.

(a) The nodal points along the two outer vertical boundaries were constrained to move only in the vertical direction (code = 1) whereas those along the bottom boundary were fixed (code = 3) and could not move at all. All the other nodal points were free to move in any direction.

(b) The load was applied at seven nodal points, two of 3 333,38 and five of 6 666,75 N. This resulted in a total load of very nearly 40 kN.

(c) The period length, which is the spacing in the Z-direction between the centres of two loads (Figure 4.1) was chosen after a CHEVRON analysis. The analysis showed that the surface deflection at 4 000 mm offset from the centre of the loaded area was only about 8 per cent of the maximum deflection and the horizontal tensile stress was less than 0,1 per cent of the maximum value. It was then decided to use a period length of 4 000 mm.

(d) The loaded length in the Z-direction was 148,15 mm and this indicated about 14 harmonics to represent the load. Only 11 harmonics were used with the 11th contributing less than 2 per cent to the maximum nodal point deflection (281 μm at nodal point 196).

4.2.5 Analysis of a pavement containing a cracked treated layer

To model a pavement containing a cracked treated layer the finite element mesh shown in Figure 4.5 was used. This mesh will hereafter be referred to as Case A. It was necessary to spend some time in determining the optimum procedure for numbering the nodal points in order not to exceed the specification on the maximum difference between nodal points defining an element (see Appendix B). The procedure that was accepted can be obtained from the nodal point numbers given in Figure 4.5.

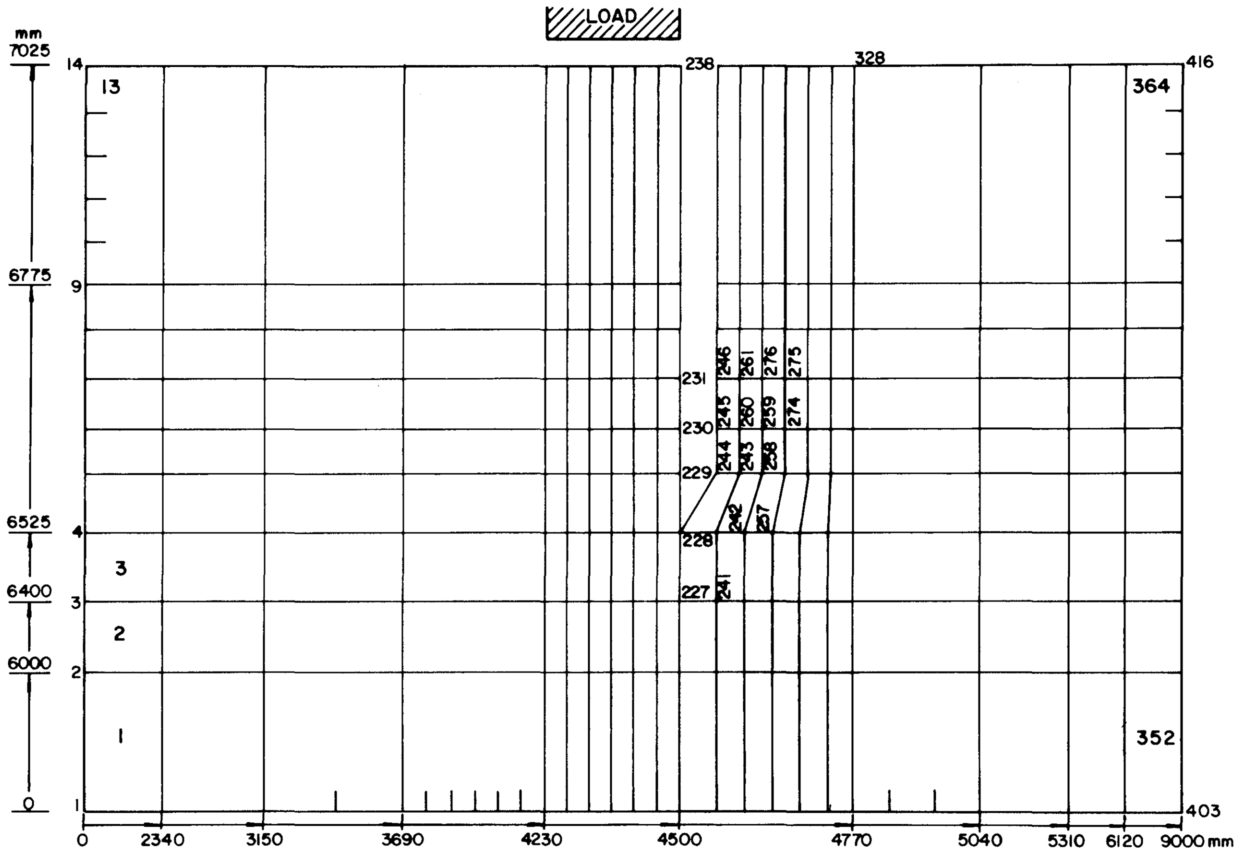


FIGURE 4-5

FINITE ELEMENT MESH - CASE A

The crack width was assumed to be 6 mm which represents a wide crack with no load transfer across it. The crack spacing was assumed to be 4 500 mm, which agrees with practical observations (Otte, 1973a).

The lower four nodal points on each of the two outer vertical boundaries were constrained and could move only in the vertical direction. The other (that is, the upper) nodal points on the outer vertical boundaries were allowed to move freely since they formed the edges of other cracks. The nodal points along the sides of the crack were allowed to move in any direction but those along the bottom boundary of the mesh were fixed and could not move at all (see Appendix B).

In a previous study (Otte, 1975) the stresses parallel to the crack (σ_{zz})* and those perpendicular to the crack (σ_{xx}) in both the cracked (Case A) and uncracked conditions were compared at various offsets from the centre of the load. It was observed that (i) the maximum tensile stress in the treated layer occurred at the bottom of the layer; (ii) the σ_{xx} stress was not significantly affected by the introduction of the crack, it decreased by 20 per cent; (iii) the direction of the maximum tensile stress changed from being perpendicular to the crack (in the X-direction) to parallel to the crack (in the Z-direction); (iv) the σ_{zz} stress increased significantly so that it exceeded the maximum tensile σ_{xx} by about 80 per cent, and (v) a more accurate comparison of the maximum stresses may be possible if the maximum values could be obtained from accurately prepared stress contour maps since the position of the maximum stress will probably not coincide with the centre point of one of the elements.

From observations (i), (iii) and (iv) in the previous paragraph it was concluded that the maximum horizontal tensile stress in a cracked cement-treated layer will occur at the bottom of the cement-treated layer and it will act parallel to the crack (σ_{zz}). This agrees with the conclusions by Pretorius (1970).

4.2.5.1 <u>An alternative method (L-model)</u>

A large vertical tensile stress at the bottom and next to the crack, but on the opposite side of the load, was observed during the analysis of Case A. This large vertical stress may result in a loss of bond between the treated layer and the subbase, and the formation of a horizontal crack between these two layers. A pavement containing a cracked treated layer should therefore really be modelled by including the narrow horizontal

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* σ_{zz} is a normal stress parallel to the z-axis which acts on a plane perpendicular to the z-axis.

crack between the two layers. If this is done the material above the horizontal crack and to the right of the vertical crack shown in Figure 4.5 may be eliminated from the model since there is no structural continuity between the layers. This suggested the finite element model shown in Figure 4.6. It will hereafter be called the L-model. This type of model was also used by Pretorius (1970) although a much finer, and hence possibly a more accurate mesh, was used in this study.

The maximum tensile stress in Case A was about 224 kPa and in the L-model it was calculated as 199 kPa. The small difference in tensile stress and the high vertical tensile stress next to the crack in Case A (Figure 4.5) prompted the suggestion that the L-model should be used. Additional advantages of the L-model are that it is easier to change the depth of the crack (see later) and that the computer time is about 35 per cent less than that for the Case A analysis. The CPU time to analyse the L-model on an IBM S 370/158 computer running under OS R21,8 was about 14 minutes.

4.2.5.2 A third alternative method

It is also possible to analyse a pavement containing a cracked treated layer by applying the principle of axisymmetrical and asymmetrical loading. Figure 4.7 is a schematic drawing explaining the principle, namely that a one-unit load next to the vertical crack may be represented by an axisymmetrical load of 0,5 units and an asymmetrical load of 0,5 units. Two analyses are performed, one for the axisymmetrical load and another for the asymmetrical load, each having a load of 0,5 units. When the calculated stresses are added together the outcome is the stress when a one-unit load is placed next to the crack. By doing it this way the principle of superposition is assumed, which implies that the two 0,5 unit loads on the left hand side of the crack cancel out and the two on the right hand side produce a one-unit load next to the crack.

Figure 4.8 shows the boundary conditions that were used, namely the nodal points on the vertical cracked faces were allowed to move freely (code = 0); those on the bottom boundary were fully constrained (code = 3); the nodal points on the lower outer vertical edges for the axisymmetrical load were allowed to move only vertically and for the asymmetrical load they were allowed to move only horizontally (see Appendix B). The mesh layout and the element sizes were the same as used in Figure 4.4.

A detailed comparison between the stresses calculated for this loading condition and those of Case A (Figure 4.5) has been done elsewhere

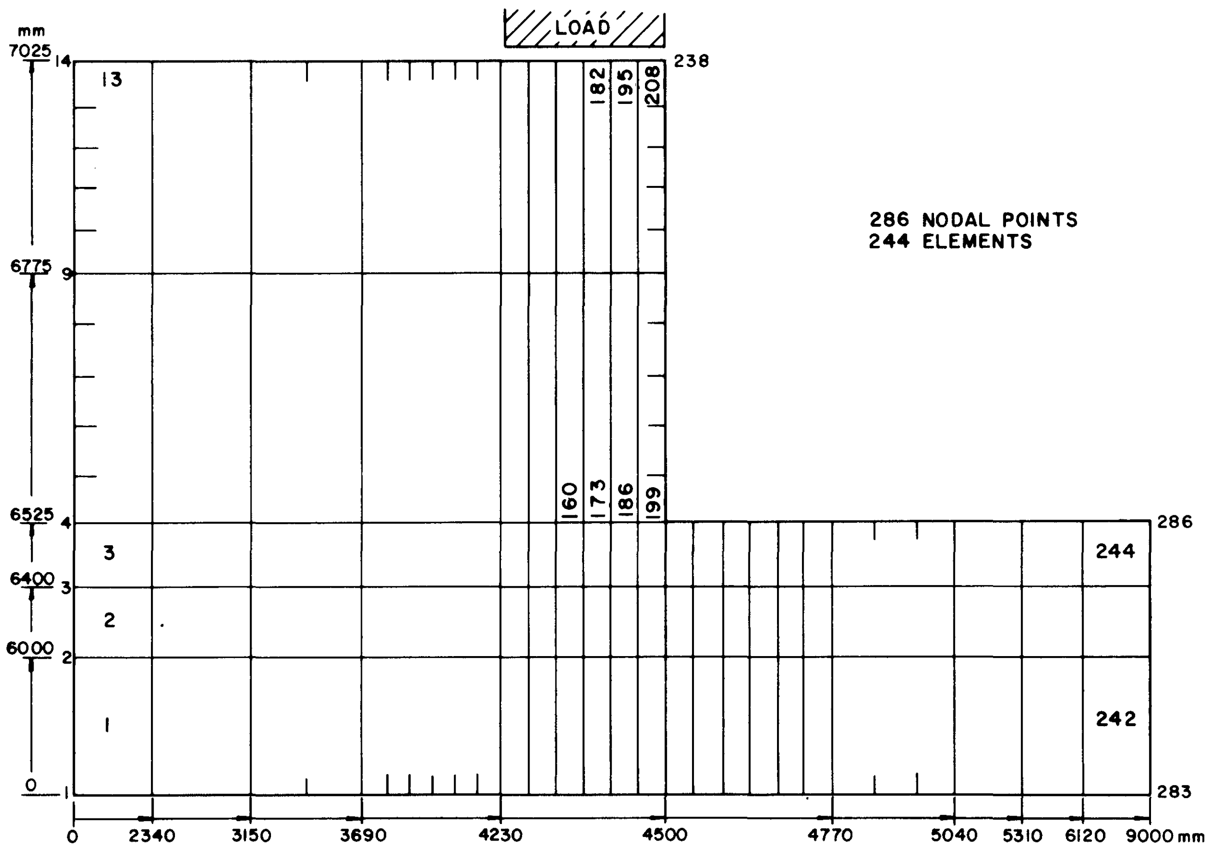
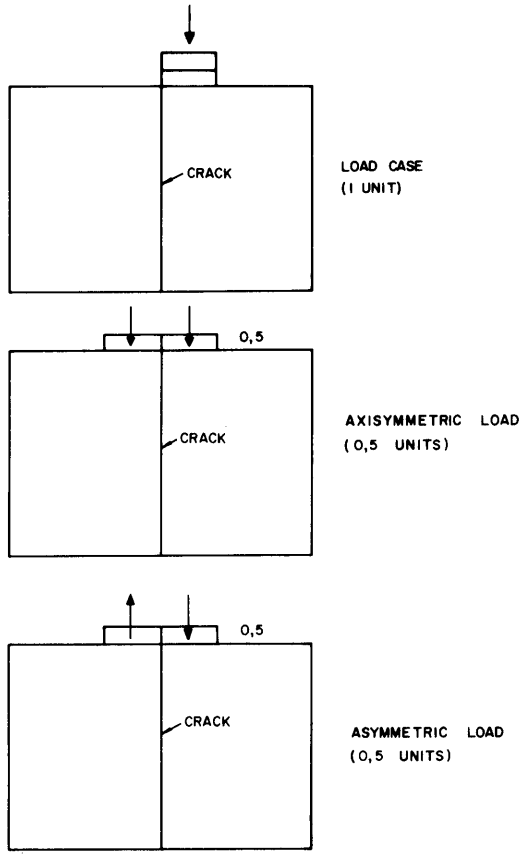


FIGURE 4-6

FINITE ELEMENT MESH OF L-MODEL



A LOAD OF 1 UNIT NEXT TO THE VERTICAL CRACK IS EQUAL TO AN AXISYMMETRICAL LOAD OF 0,5 UNITS PLUS AN ASYMMETRICAL LOAD OF 0,5 UNITS

FIGURE 4-7

LOADING PRINCIPLE FOR ANALYSIS OF A PAVEMENT CONTAINING A CRACKED TREATED LAYER

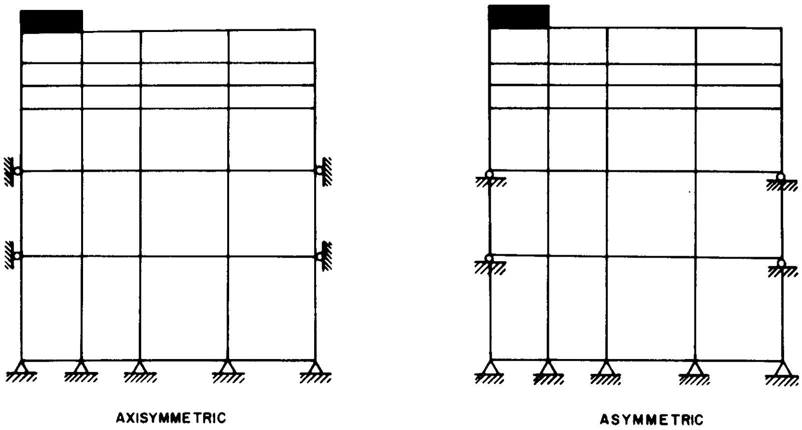


FIGURE 4-8

BOUNDARY CONDITIONS FOR AXISYMMETRIC AND ASYMMETRIC LOADS

(Otte, 1975). It was observed that the differences in stress near the crack were usually below 10 per cent but in some of the outlaying elements with low stress values the difference increased to around 50 per cent with a maximum difference of 170 per cent. The difference between the maximum stresses calculated for the two conditions was only 8 per cent, and this was considered very acceptable. Assuming the model in Figure 4.5 (Case A) to be 'correct', it may be concluded that loading next to the edge of a vertical crack may be represented by the sum of an axisymmetrical and asymmetrical analysis, each with half of the load intensity.

It therefore seems that both the L-model and the axisymmetrical plus asymmetrical loading may be used to analyse a pavement containing a cracked treated layer. Otte (1975) compared the stresses calculated by these two methods at the bottom of the treated layer. The agreement in the σ_{xx} values was generally very poor while the differences in the σ_{yy} and σ_{zz} values were generally about 10 per cent. Since the maximum horizontal tensile stress always seems to be a σ_{zz} value, either of the two analysis methods may be accepted. There are also other factors to be considered, such as computer time and the manual labour involved in summing the outcome of the axisymmetrical and asymmetrical loadings. It is therefore suggested that the L-model should be used in preference to this loading method to represent a pavement containing a cracked treated layer, and it will be used in the remainder of this chapter.

4.2.6 Increase in tensile stress

The increase in maximum horizontal tensile stress caused by the crack can be defined and calculated as the difference between the maximum stress obtained from the appropriate finite element model and the maximum calculated by a CHEVRON analysis. To determine the maximum stress from the finite element models, stress contour maps were prepared (Otte, 1975). This was found to be very time-consuming and the development of an easier alternative was very desirable.

Further analyses indicated that the most convenient way to determine the increase in horizontal tensile stress caused by the crack would be to do a finite element analysis of the L-model and to locate the maximum horizontal tensile stress. It is usually a σ_{zz} value in the centre of a finite element at the bottom of the treated layer. The next step is to record σ_{zz} in the centre of the finite element directly above the finite element containing the maximum σ_{zz}, and it will generally have a lower value. The stress gradient between these two points should then be projected downwards to calculate the maximum stress (σ_{mc}) at the bottom of the treated layer. It is suggested that this value (σ_{mc}) should be taken as the maximum σ_{zz} in the treated layer. A CHEVRON analysis should hereafter be performed to calculate the maximum tensile stress for the uncracked case (σ_{mu}). The increase in maximum horizontal tensile stress as a result of the crack is then expressed as the difference between the two stress values, that is $\sigma_{mc}-\sigma_{mu}$.

To illustrate the previous suggestion, the maximum tensile stress for a pavement containing a cracked treated layer (hereafter referred to as the cracked condition) was calculated. The maximum σ_{zz} occurred at the centre of element 173 (Figure 4.6) and was 199 kPa. The σ_{zz} at the centre of element 174 was 105 kPa. The two centres are 50 mm apart and this resulted in a stress gradient of 1,88 kPa per mm. The bottom of the layer is 25 mm below the centre of element 173 resulting in a stress of 199 + (1,88 x 25) = 246 kPa (σ_{mc}). The stress contour map (Otte, 1975) indicated a maximum value of about 250 kPa, and this agrees very reasonably with the suggested method.

A CHEVRON analysis of the uncracked pavement indicated the maximum horizontal tensile stress (σ_{mu}) to be about 213 kPa and it occurred underneath the centre of the wheel. The stress increase, as a result of the crack, is therefore about 33 kPa, from 213 to 246 kPa, and that is an increase of about 15 per cent.

It can happen that a steeper stress gradient may occur between some of the adjoining elements, and coupled with the particular stress values, this may result in a larger stress increase. Although these exceptions do not occur very frequently, it is essential that the analyser should also investigate the stress gradients in the surrounding elements during his search for the maximum stress.

If the suggestion of calculating the maximum horizontal tensile stress (σ_{mc}) is adopted, it is not necessary to prepare the stress contour map although the principle of its preparation is used. It is also implicitly assumed that the maximum stress will occur directly beneath the centre of the finite element having the maximum σ_{zz} value, and, based on the previous analyses, it is believed that this is a valid assumption.

4.2.7 Surface deflection

Figure 4.9 shows the surface deflection calculated by three finite element models. The uncracked pavement (Figure 4.4) produced a fairly symmetrical deflection pattern with the maximum value occurring underneath the centre

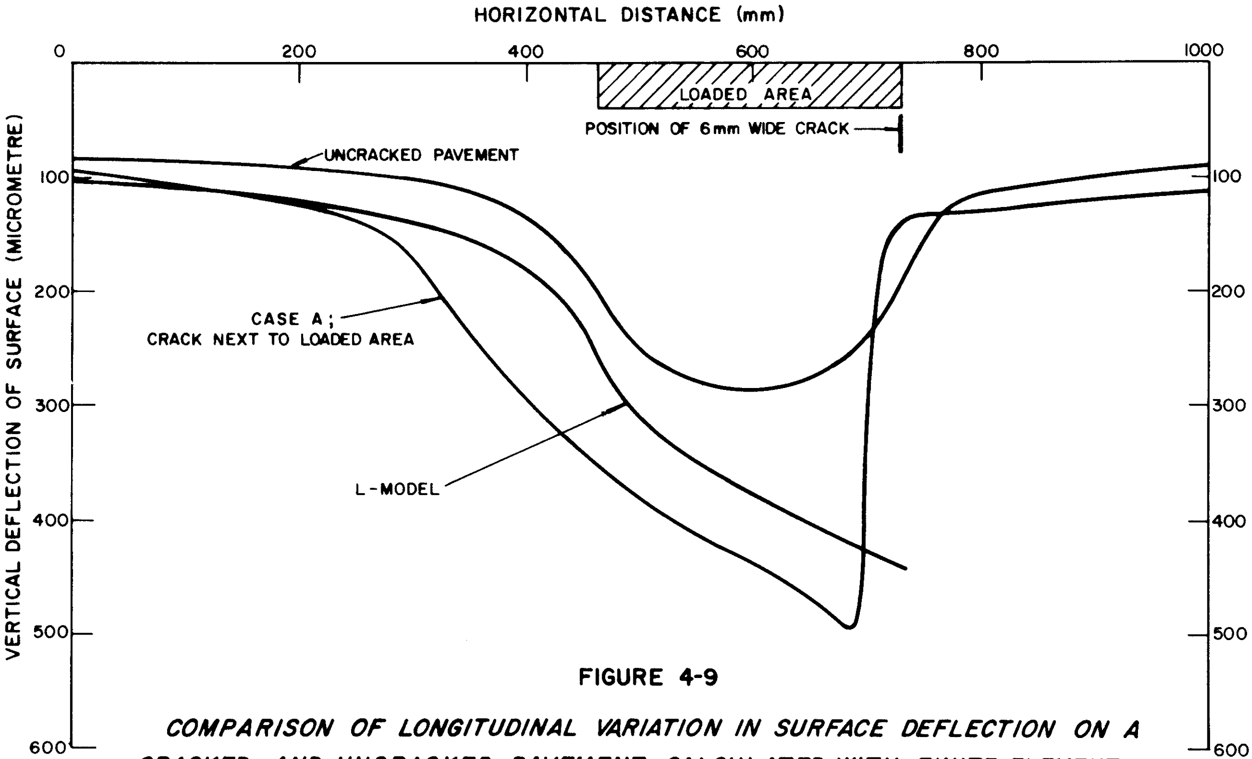


FIGURE 4-9

COMPARISON OF LONGITUDINAL VARIATION IN SURFACE DEFLECTION ON A CRACKED AND UNCRACKED PAVEMENT, CALCULATED WITH FINITE ELEMENT ANALYSIS of the loaded area. The patterns for Case A and the L-model are very similar with a slight difference in the absolute value.

The variation in deflection across the crack is quite interesting. Although a wide crack was assumed in Case A, and no load transfer was supposed to take place across it, the unloaded right hand side deflected about 140 micrometres. This means that some load transfer did take place; probably through the subbase and subgrade in the form of the high vertical tensile stresses which occurred to the right of the crack and at the bottom of the treated layer (see 4.2.5.1, page 59). The amount of load transfer which does take place in Case A is another reason why the L-model (Figure 4.6) should preferably be used.

4.2.8 Vertical stress in the subgrade

If a wide crack is formed in a treated layer the loss of horizontal load transfer across the crack may result in a significant increase in vertical compressive stress in the lower-lying materials next to the crack (Fossberg et al, 1972a). This may cause deformation of the subgrade next to the crack which may lead to an unacceptable riding quality on the road. In the design of a pavement containing a cracked treated layer it may therefore be important to take due notice of the increase in vertical compressive stress and the possibility that deformation may develop. The prismatic solids finite element program was also used to investigate this phenomenon (Otte, 1975) and it was observed that the possibility of subgrade deformation after cracking, and next to the crack in some structural layouts, may not be disregarded (see later).

4.3 STRESS DISTRIBUTION IN CRACKED TREATED LAYERS

4.3.1 Previous use of program

The prismatic solids finite element program was previously used by Pretorius (1970) to study the fatigue behaviour of a cement-treated layer. Recently Luther et al (1974) studied reflection cracking through bituminous overlays with the aid of this program.

Pretorius (1970) analysed the pavement shown in Figure 4.10 using the finite element mesh shown in Figure 4.11. The wheel load was assumed to be two 20 kN loads, applied to two square areas of 203 x 203 mm (8 inches by 8 inches) and the centres were 304,8 mm (12 inches) apart. To obtain the maximum stress between the two loads of the 40 kN dual wheel, he calculated the stress parallel to the crack at Z=152,4 mm (Appendix B) under a 20 kN load, and doubled it. This value was compared with the results of a

ASPHALTIC CONCRETE	E = 4000 MPa υ = 0,35	d = 75 mm
CEMENT-TREATED BASE	E = 19300 υ = 0,16	d = 200 mm
FOUNDATION	E = 69 AND 11 υ = 0,45	

FIGURE 4-10

PAVEMENT ANALYSED BY PRETORIUS (1970)

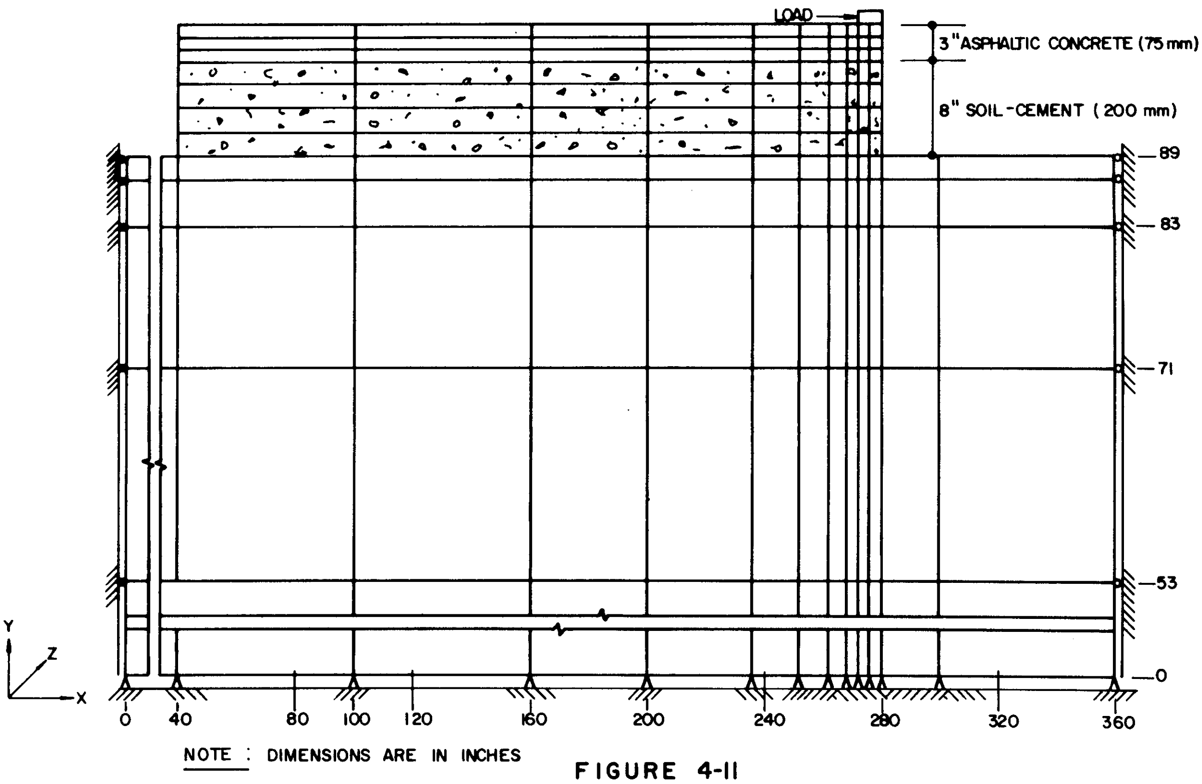


FIGURE 4-11

FINITE ELEMENT MESH FOR LOAD AT AN EDGE (After Pretorius, 1970)

CHEVRON analysis and it was possible to calculate the stress increase. The increase in horizontal tensile stress caused by the crack was calculated as about 46 per cent on the 69 MPa subgrade and as about 60 per cent on the 11 MPa subgrade (Mitchell et al, 1974).

The mesh shown in Figure 4.11 was analysed by the author and the maximum tensile stress between the wheels (on a 69 MPa subgrade) was calculated as 1 022 kPa. The CHEVRON analysis indicated a stress of 770 kPa and hence a 33 per cent stress increase. The corresponding figures reported by Pretorius (1970) were about 758 and 517 kPa which resulted in the increase of about 46 per cent mentioned above. The stress increase calculated by the author was less than that calculated by Pretorius. The discrepancy in the calculated stress values may be due to some small differences, for example in the loading conditions.

The finite element mesh shown in Figure 4.12 was used to analyse the structure shown in Figure 4.10 in an effort to determine the effect of the mesh size. A 69 MPa subgrade was used. The loading system was changed to that described in section 4.2.4; hence one 40 kN load on a 270 x 296,3 mm area was used. It was considered acceptable to make this change in the loading condition since CHEVRON indicates that the maximum tensile stress would increase by about 9 per cent when the loading is changed from two 20 kN loads to one 40 kN load. The maximum horizontal tensile stress in the finite element analysis occurred at the bottom of element 189 and was 1 306 kPa. The CHEVRON analysis with a corresponding loading system indicated a maximum tensile stress of 840 kPa and the stress increase because of the crack was 55 per cent.

From the foregoing it seems that the refinement in the mesh (Figure 4.11 to 4.12) resulted in a 28 per cent increase in the calculated maximum tensile stress - from 1 022 to 1 306 kPa. Using the finer, and probably more accurate, mesh is therefore significant.

4.3.2 Layouts considered

It was the intention to study several typical structural layouts to obtain the percentage increase in stress. There is, however, such a multitude of possible combinations with respect to layer thickness and elastic properties that only two basic structural sections were chosen, namely one with and one without a crusher-run base. The bituminous surfacing was omitted in all the analyses. From this multitude of possible layouts eight were selected and analysed. In some of them the elastic moduli of the treated layers were varied while the properties of the other layers were

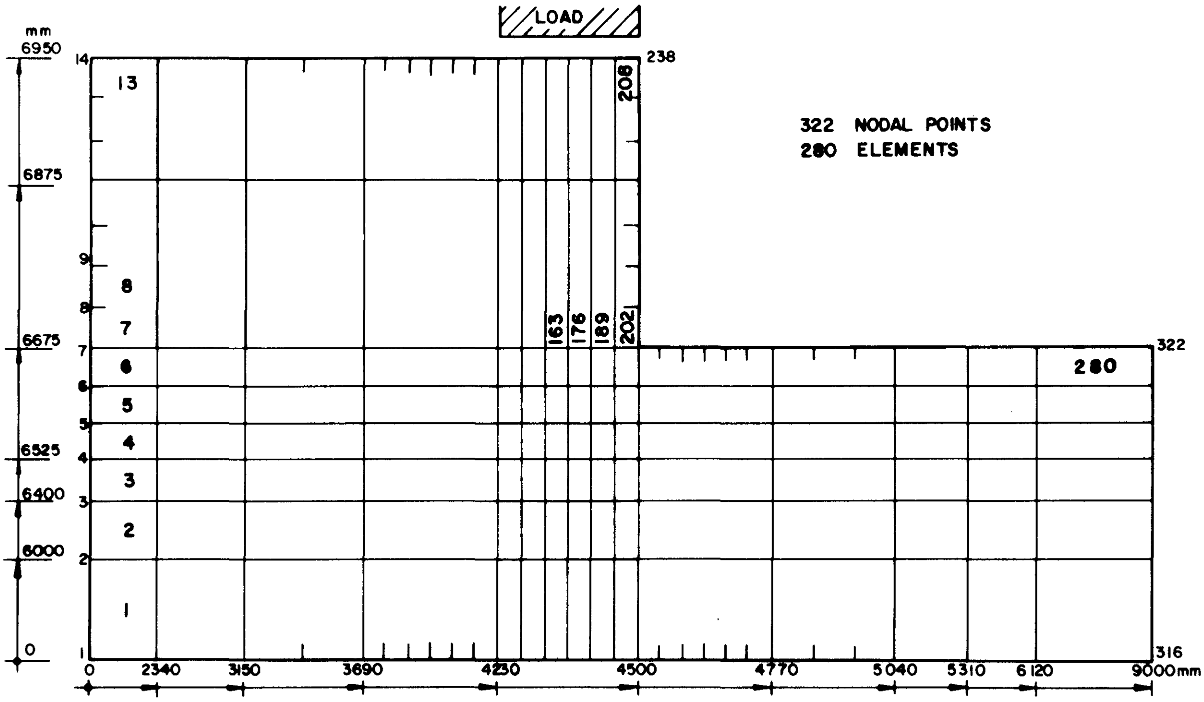


FIGURE 4-12

IMPROVED FINITE ELEMENT MESH

kept constant. A constant Poisson ratio of 0,35 was assumed for all the materials.

Figure 4.13 illustrates the selected thicknesses, elastic moduli and material types in the various layouts. The depth of the crack considered in each analysis is shown by the jagged vertical line. The thicknesses were based on South African experience and the moduli were obtained from published literature. Layouts A to C are typical of the designs used in the Transvaal where an untreated crusher-run layer is used on top of, usually, a cement-treated subbase (Otte and Monismith, 1976). In layout D a cement-treated crusher-run base was included between the treated and untreated layer. Layout E contains two treated layers, either cement or lime, while the upper treated layer in layout F will usually be a cement-treated crusher-run layer. Layout G was used on two roads during the sixties (Special Road S12 and a part of Route N4) but their performances were not entirely satisfactory (Otte, 1973a). Layout H is essentially the same as G but with thicker layers. Both have an untreated layer between two treated layers.

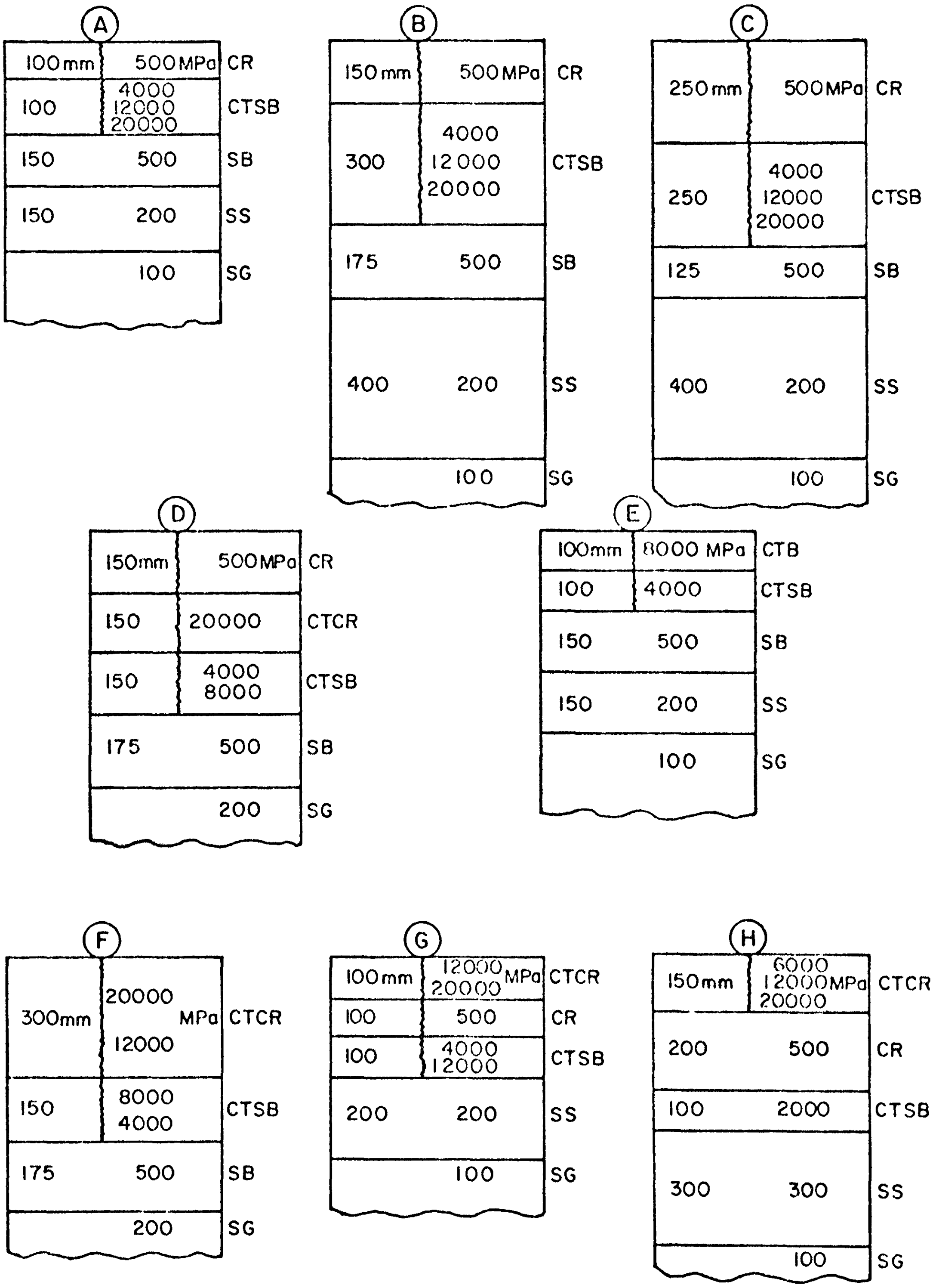
4.3.3 Results

The finite element model shown in Figures 4.6 and 4.12 (L-model) was adopted and modified to accommodate the changes in the depth of the crack. It was analysed 21 times to cover the combinations shown in Figure 4.13. The method described in section 4.2.6 was used to calculate the maximum horizontal tensile and vertical compressive stresses in the cracked pavement.

4.3.3.1 Tensile stress in treated layers

In all cases the maximum tensile stress occurred at the bottom of the treated layer and it acted parallel to the crack. This confirms the observation by Pretorius (1970). The maximum stress usually occurred between the centre of the loaded area and the crack, whilst it always occurred directly beneath the centre of the loaded area in CHEVRON. Tables 4.2 and 4.3 contain the maximum stresses as recorded for the eight layouts; they show the increase in maximum horizontal tensile stress at the bottom of the treated layer and the surface deflection according to CHEVRON.

It seems that the maximum increase in tensile stress (39,3 per cent) as a result of the crack occurred in the treated base of layout D (Table 4.3) while a stress decrease was noticed in layouts A and H. Another interesting observation is the relatively high stress increases in layouts with thick (250 to 300 mm) upper layers in relation to the small increases in layouts



CR = CRUSHER-RUN BASE
CTCR = CEMENT-TREATED CRUSHER-RUN BASE
SB = SUBBASE
SS = SELECTED SUBGRADE
SG = SUBGRADE
CTSB = CEMENT-TREATED SUBBASE
CTB = CEMENT-TREATED BASE
(THE VERTICAL LINE IN EACH LAYOUT SHOWS THE DEPTH OF THE CRACK)

FIGURE 4-13
SCHEMATIC DRAWING OF ANALYSED LAYOUTS

TABLE 4.2 : Maximum horizontal tensile stresses in cracked and uncracked pavements

LAYOUT	ELASTIC MODULUS OF TREATED LAYER (MPa)	MAXIMUM HORIZONTAL TENSILE STRESS IN TREATED LAYER (kPa)		PERCENTAGE INCREASE IN TENSILE STRESS DUE TO CRACK	SURFACE DEFLECTION FROM CHEVRON (μm)
		Finite element analysis (cracked pavement)	CHEVRON program (uncracked)		
A	4 000	595	658	-9,6	394
	12 000	1 418	1 438	-1,3	356
	20 000	1 964	1 903	3,2	335
B	4 000	254	217	17,1	273
	12 000	476	364	30,8	240
	20 000	587	430	36,5	227
C	4 000	254	213	19,2	322
	12 000	509	382	33,2	298
	20 000	644	466	38,2	288

TABLE 4.3 : Maximum horizontal tensile stresses in cracked and uncracked pavements

LAY-OUT	ELASTIC MODULUS (MPa)		MAXIMUM HORIZONTAL TENSILE STRESS IN TREATED LAYER (kPa)				PERCENTAGE INCREASE IN STRESS DUE TO CRACKS		SURFACE DEFLECTION BY CHEVRON (μm)
			Base		Subbase				
	Base	Sub-base	Finite element analysis (cracked)	CHEVRON (un-cracked)	Finite element analysis (cracked)	CHEVRON (un-cracked)	Base	Sub-base	
D	20 000	4 000	596	447	199	161	33,3	23,6	203
	20 000	8 000	333	239	337	256	39,3	31,6	194
E	8 000	4 000	In compression		514	513	-	0,2	248
F	20 000	8 000	289	225	202	153	28,4	32,0	69
	12 000	4 000	290	232	142	113	25,0	25,7	82
G	12 000	4 000	1 570	1 352	489	411	16,1	19,0	258
	12 000	12 000	1 430	1 214	933	805	17,8	15,9	237
	20 000	4 000	2 147	1 829	412	351	17,4	17,4	238
	20 000	12 000	1 967	1 657	820	697	18,7	17,6	219
H	6 000	2 000	675	719	-	-	-6,1	-	221
	12 000	2 000	1 008	1 042	-	-	-3,3	-	194
	20 000	2 000	1 271	1 294	-	-	-1,8	-	176

A and E which had thin (100 mm) upper layers. This observation did not quite fit Layout G, but since this design and layout H should preferably not be used in pavement design (see 2.2.4, page 16) very little attention will be paid to them in the remaining part of the chapter. They were mainly analysed as part of an ongoing study to evaluate their previously unacceptable performance (Otte, 1973a). In both of them the increase in stress due to the cracks was less than 20 per cent, which is not considered to be excessive.

The possible existence of a relationship between the increase in tensile stress due to cracks and the thickness of the upper layers led to the preparation of Figure 4.14. Area A represents layouts A to D, that is those with an untreated base, while area B represents layouts E and F. No sound statistical relationship could be computed between the variables, but the figure may be used to obtain an indication of the expected increase in maximum horizontal tensile stress due to cracks once the surface deflection has been calculated.

4.3.3.2 <u>Vertical compressive stress in lower layers</u>

Table 4.4 shows the vertical stresses and strains in the two untreated layers underlying the cracked treated layer. In some of the layouts, for example A, B, C and E, they are the subbase and selected subgrade and, in others, for example D and F, they are the subbase and subgrade. These values were calculated by finite element analysis (for the cracked pavement) and CHEVRON (for the uncracked pavement). The strain for the finite element analysis was calculated by increasing the strain obtained from the CHEVRON analysis by the same ratio as the increase calculated for the corresponding vertical stresses.

The table shows that the increase in vertical stress due to cracking is dependent on the layout and that it is significant - the increase varies between about 2 and 15 times. For subgrade-quality materials Dormon and Metcalf (1965) have suggested a vertical strain below 650 $\mu\varepsilon$ to withstand about one million load repetitions. Assuming this order of strain to hold also for both subbase and selected subgrade-quality materials, it seems that the strains in the first layer below the cement-treated layer of layouts A, E and G, when cracked, are rather excessive and deformation will probably occur before a million load repetitions. The other layouts may withstand one million load repetitions without severe rutting and deformation next to the crack, but the strains are approaching the allowable limit, and should water penetrate into the lower layers, severe deformation will

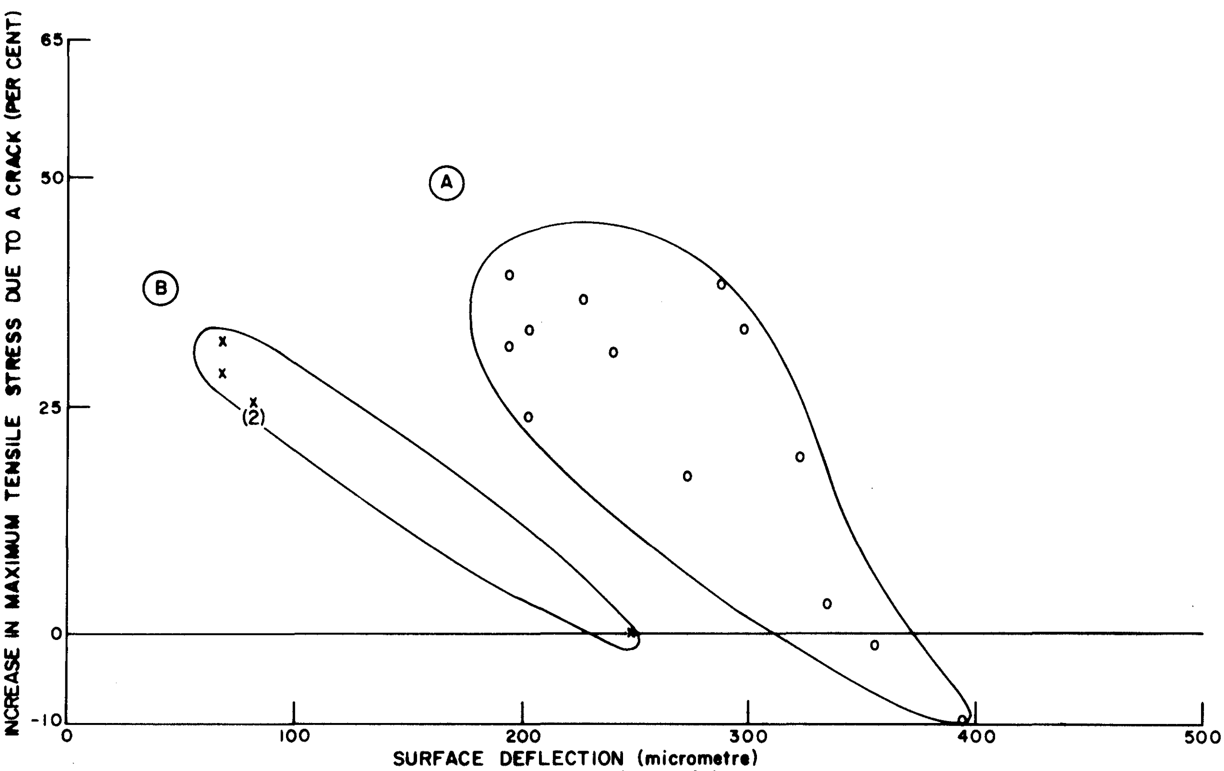


FIGURE 4-14

RELATIONSHIP BETWEEN SURFACE DEFLECTION AND PERCENTAGE INCREASE IN TENSILE STRESS

TABLE 4.4 : Increase in vertical stress and strain in cracked and uncracked pavements

LAYOUT	FIRST LAYER UNDERLYING CEMENT-TREATED LAYER					SECOND LAYER UNDERLYING CEMENT-TREATED LAYER				
	Vertical stress (kPa)		Vertical strain (με)			Vertical stress (kPa)		Vertical strain (με)		
	Finite elements	CHEVRON	Finite elements	CHEVRON	Ratio	Finite elements	CHEVRON	Finite elements	CHEVRON	Ratio
A	500	161	1 028	331	3,1	110	60	634	346	1,8
	467	111	951	226	4,2	99	47	565	268	2,1
	439	88	888	178	5,0	91	40	516	227	2,3
B	238	34	588	84	7,0	70	16	403	92	4,4
	190	19	450	45	10,0	55	10	303	55	5,5
	162	13	399	32	12,5	46	8	242	42	5,8
C	265	31	675	79	8,5	96	19	520	103	5,0
	231	19	559	46	12,2	80	12	440	66	6,7
	207	14	503	34	14,8	70	10	357	51	7,0
D	189	25	461	61	7,6	58	14	298	72	4,1
	179	19	452	48	9,4	53	12	261	59	4,2
E	344	81	845	199	4,2	39	19	417	203	2,1
F	113	11	277	27	10,3	35	7	165	33	5,0
	134	16	343	41	8,4	42	9	219	47	4,7
G	208	40	1 045	201	5,2	50	21	493	207	2,4
	194	32	897	148	6,1	45	18	433	173	2,5
	191	34	961	171	5,6	46	18	473	185	2,6
	177	28	815	129	6,3	42	16	410	156	2,6

probably take place. The strains in layout H are well below 300 με which is probably due to the intact treated subbase. The low strains in this layout (H) result in a negligible chance of rutting and because of this they were not reproduced in Table 4.4.

4.4 DISCUSSION

4.4.1 Horizontal tensile strain

In the development of a design procedure for a pavement containing a treated layer the presence of cracks has always caused some concern because it has been difficult to evaluate their effect. Tables 4.2 and 4.3 and Figure 4.14 quantify the increase in horizontal tensile stress due to the crack. The increased stress acts parallel to the crack and occurs at the bottom of the treated layer. The increase seems to be dependent on the material properties and structural layout, but it will probably not exceed about 40 per cent. During the design process it is therefore possible to make provision for the increased stress and to design for it. This can be done by calculating the maximum horizontal tensile stress (σ_t) for the uncracked structural layout, using say CHEVRON. This stress (σ_t) is then increased, for example, 1,4 times and the new value (σ_n) is taken as the design horizontal tensile stress for the material.

4.4.2 Vertical compressive strain

Wide cracks and the corresponding loss of load-transfer result in significant increases in the vertical compressive strains in the lower layers, and the increased strains approach the presently accepted design criterion for relatively dry materials, that is materials at their natural moisture contents. If water penetrates the cracks and becomes trapped in the lower layers it is therefore highly probable that deformation will take place. To avoid this, the penetration of water should be prevented by providing adequate drainage and/or by sealing the cracks effectively. It may also be possible to allow for the strain increase during the design stage and the data in Table 4.4 may be utilized.

The author is not aware of any deformation problems in pavements containing treated layers that can be directly ascribed to the increased vertical stress or strain next to the crack (Wright, 1969; Lewis and Broad, 1969; and Otte, 1973a). This may be because of various reasons such as some amount of load-transfer across the crack which means that the increased vertical strain is not as severe as indicated in Table 4.4; sufficient drainage to prevent wetting of the lower layers; very little attention being paid to this mode of distress even in a severely distressed pavement because it is not really considered as contributory or because it is extremely difficult to isolate and define the various modes of distress.

4.4.3 Crack length, depth and width

In this chapter the length of the crack was considered to be the width of the pavement, that is a transverse crack across the pavement; the depth of the crack was varied, depending on the layout, but it was always taken through the cement-treated layers; and a wide crack was considered (6 mm) across which no load-transfer was assumed to take place. In practice things are different.

The length of the cracks vary and there are shorter cracks which do not extend across the pavement. Shorter cracks were, however, not considered in this chapter because the prismatic solid finite element program used to perform the analysis can only handle infinitely long cracks extending all the way along the YZ-plane, see Figure 4.1. The increase in stress calculated next to a short crack would probably be less than the increase calculated next to a crack extending all the way across the pavement. This implies that recommendations for stress or strain increases based on the analyses performed in this chapter, are conservative.

Cement-treated materials are brittle, and once a crack has formed it grows very rapidly. This implies that a crack which does not extend through the depth of the layer is not in a stable condition and that it will grow relatively quickly to become a full-depth crack and be in a stable condition. That is why only full-depth cracks were considered in this chapter.

In the analyses only very wide cracks were considered and no load-transfer was assumed to take place across them. In practice there are materials in which the cracks are relatively narrow, from hairline to 2 mm, and some degree of load-transfer will very probably occur across them. In materials known to exhibit narrow cracks, for example some cement-treated natural gravels and lime-treated soils, it is suggested that about 50 per cent load-transfer be assumed and that the stress increases be taken as 50 per cent of those calculated for wide cracks.

4.4.4 Increases recommended for design

If a pavement has to be designed and it is not considered feasible to perform the finite element analysis described in this chapter, the available information can be utilized. Since the increases seem to be dependent on the structural layout and the material properties - which also provide an indication of the crack width - Paterson (1976) interpreted the information in Tables 4.2, 4.3 and 4.4 and proposed Table 4.5. This table indicates the required increases in the results of an uncracked analysis (that is by CHEVRON) and its use is recommended to accommodate the initial crack during structural pavement design work.

Section A.3.3 in Appendix A suggests that these increases are equally applicable to calculated stresses and strains.

TABLE 4.5 : Suggested increases in calculated maximum stresses and strains to accommodate initial cracking

TYPE OF CRACKING	TOTAL THICKNESS TREATED MATERIAL (mm)	MAXIMUM HORIZONTAL TENSILE STRESS*	MAXIMUM VERTICAL STRESS	
			First underlying layer	Second underlying layer
No cracking expected (for example less than 2 per cent lime or cement)		1,0	1,0	1,0
Moderate cracking; crack widths less than 2 mm (for example natural materials with lime or 2-3 per cent cement)	$\leqslant$200	1,10	2,5	1,5
	>200	1,20	7,0	3,5
Extensive cracking; crack widths more than 2 mm (for example crushed stone with 4-6 per cent cement)	$\leqslant$200	1,25	5,0	2,5
	>200	1,40	14,0	7,0

*Parallel and adjacent to initial crack.

4.5 CONCLUSIONS AND RECOMMENDATIONS

(a) When the prismatic solids finite element program was used to analyse a carefully and properly constructed mesh for an uncracked pavement (Figure 4.4), the differences between the calculated stresses and those obtained from a CHEVRON analysis were generally less than 10 per cent. It is therefore recommended that the prismatic solids finite element program may be used to analyse a pavement containing a cracked treated layer. The finite element mesh should be modelled along the principles that were used to prepare Figure 4.4, and examples of these are Figures 4.5, 4.6 and 4.12.

(b) There are various ways to model a pavement containing a cracked treated layer but the L model is suggested because of the possibility of a loss of vertical bond, and hence continuity, between the treated layer and subbase. The axisymmetric plus asymmetric loading conditions may be used but are not recommended because of the amount of manual labour involved.

(c) After cracking the maximum tensile stress acted parallel to the crack (σ_{zz}) and it occurred near the centre of the loaded area at the bottom of the treated layer.

(d) To calculate the stress increase as a result of the crack it is suggested that the L-model should be used to locate the finite element with the maximum σ_{zz} value. From the stress in the finite element above the one having the maximum σ_{zz}, the stress gradient and the maximum value for the structure can be determined (σ_{mc}). Hereafter the CHEVRON analysis is performed to obtain the maximum value for an uncracked pavement (σ_{mu}). The stress increase as a result of the crack is determined as the difference between the two maximum values, $\sigma_{mc}-\sigma_{mu}$.

(e) A very wide crack with no load transfer across it causes a definite increase in the maximum horizontal tensile stress at the bottom of the treated layer. The increase seems to be dependent on the materials and structural layout. If very wide cracks are expected in the cement-treated material the tensile stress calculated for the uncracked condition should be increased 1,25 times (if the expected total thickness of the treated layers is less than or equal to 200 mm), and by 1,4 times if the expected total thickness is more than 200 mm. If moderate cracking is expected, the corresponding increases should be only 1,1 and 1,2 times respectively. It is suggested that these increases be accepted and that the treated layer be designed accordingly.

(f) The increase in the vertical stress and strain in the subgrade as a result of the initial crack is significant and the possibility of subgrade deformation may not be excluded. It is suggested that more attention should be paid to it, for example by providing adequate drainage. The increases reported in Table 4.5 should be used for pavement design work.

(g) This computer analysis contributed to a better understanding of the stress distribution and possible behaviour of a pavement containing a cracked treated layer. Further expansion and work along these lines may eventually result in a complete understanding thereof.

CHAPTER 5

THERMAL STRESSES AND INSULATION

PAGE

5.1 INTRODUCTION

When the first pavement design theories were developed, those during the thirties and forties, traffic-associated distress was the major consideration. An example is the California Bearing Ratio (CBR) procedure that was developed to overcome traffic-associated deformation of the subgrade and which was based on only wheel load intensity. Later, when a regional pavement design procedure (for example the method from the AASHO Road Test) was to be extended to a wider geographical area, it was realised that environmental factors do play an important part. Non-traffic-associated (that is environmental) distress is therefore presently considered equally as important as traffic-associated distress. The growing interest in the effect of the environment, currently sub-divided into moisture and thermal effects, can be seen in the proceedings of the three International Conferences on the Structural Design of Asphalt Pavements - the first and second contained no reports on these subjects, while the third (1972) had four.

This chapter deals with only one aspect of environment, namely the effect of temperature on a pavement containing a treated layer. In this chapter the term 'treated' is used to mean either cement- or lime-treatment (Otte, 1976). There are a number of ways of calculating the temperature distribution but the finite difference technique developed by Williamson (1972a) was used. He developed a mathematical model and prepared a computer program as the first phase of a research project on the effects of environmental factors. This chapter presents some of the results of the second phase which is the civil engineering application of the model. The objectives are to study qualitatively (i) the diurnal variation in stress in the treated layers, (ii) the insulating ability of various thicknesses and types of material, and (iii) the effect of a crack on the thermal stresses.

5.2 BACKGROUND

The importance of thermal gradients on concrete, that is rigid, slabs has been known since the twenties (Westergaard, 1927). One of the first studies (Lister, 1972) on the importance of thermal stresses in pavements containing treated layers mentioned that "...the stresses developed on cooling, together with those due to shrinkage on setting and hardening of the cement after construction, are responsible for the characteristic regular pattern of transverse cracking observed in cement-bound bases...";

and that the thermal stresses are "...comparable to the magnitude of traffic induced stresses...". Williamson (1972a) described a practical case which tended to confirm that the initial cracking in a cement-treated layer is due to temperature gradients in the material.

It is therefore very important to try to minimize the temperature gradient. Possible ways are by the use of insulating materials, heating or cooling elements, or the control of energy influx into the pavement (Williamson, 1972b). Williamson claimed that the use of heating or cooling elements in pavements would be impractical and very expensive, and he described some ways of reducing the energy influx, for example by having a light-coloured surface. The insulation method seems practical but it depends on the physical properties of the material, for example specific heat, density and thermal conductivity. These properties are in turn affected by others, such as the degree of compaction, moisture content and geological origin. This makes the number of variables virtually unlimited. He discussed the use of bituminous layers to insulate the treated layer against severe temperature gradients. The author believes that the untreated crusher-run base, which is very often used on top of the treated layer (Otte and Monismith, 1976) is also suitable as an insulation layer. Since crusher-run materials are cheaper than bituminous materials, it is worthwhile to investigate their insulating properties with the possibility of using them in preference to bituminous materials.

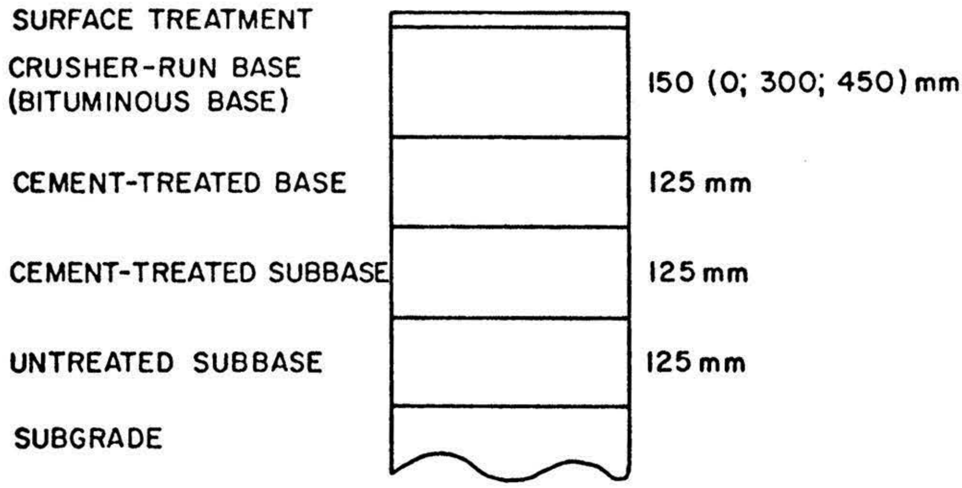
5.3 ANALYSIS

5.3.1 Structural layout

The basic structural layout considered in the analyses is shown in Figure 5.1; it consists of a surface treatment, a 150 mm untreated crusher-run base, two 125 mm cement-treated natural gravel layers, and a 125 mm untreated subbase. During the analyses the thickness of the crusher-run base was increased to 300 and 450 mm and it was also removed completely. The analyses were also repeated using a bituminous base instead of the crusher-run. The Poisson ratios of the materials were fixed at 0,35 and the elastic moduli are given in Table 5.1.

5.3.2 Computer program

A slightly modified version of the Macro Environmental Simulation Model (MESM) of Williamson (1972) was used. The major modification was the way in which the initial temperature distribution was specified and calculated (Otte, 1976a).



THE VARIATIONS WHICH WERE CONSIDERED DURING THE ANALYSIS ARE SHOWN IN BRACKETS

FIGURE 5-1

THE BASIC STRUCTURAL LAYOUT FOR THE THERMAL ANALYSIS

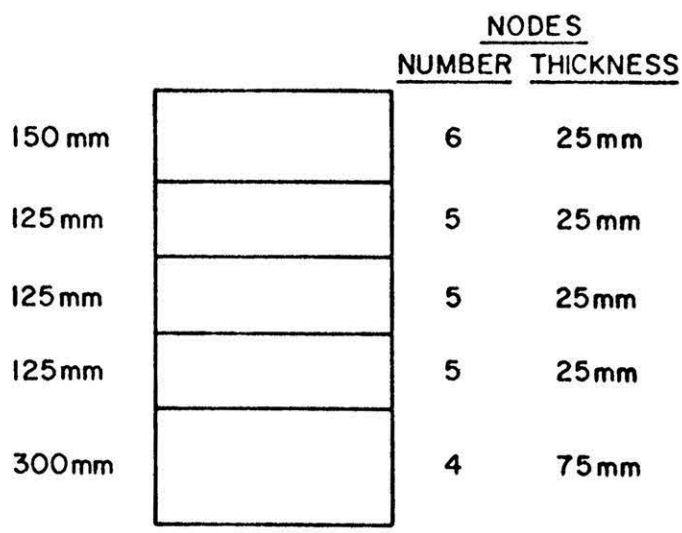


FIGURE 5-2

SUBLAYERING INTO NODES

TABLE 5.1 : Elastic moduli of materials used in the thermal analysis

MATERIAL	ELASTIC MODULUS (MPa)
Crusher-run	500
Cement-treated natural gravel base	8 000
Cement-treated natural gravel subbase	4 000
Untreated subbase	300
Subgrade	100
Bituminous material	3 000

The modified program required the pavement to be divided into sublayers (nodes) and 25 were chosen: 6 for the upper layer, 5 for each of the following three layers, and 4 for the subgrade (Figure 5.2). The thickness of the upper layers and the number of nodes were such that the sublayers were 25 mm thick. The thermal effect of the subgrade, that is the depth which affects the calculated temperatures, was considered to be 300 mm and it was divided into four 75 mm sublayers. When the thickness of the upper layer was increased, the number of sublayers remained 6, resulting in the sublayer thickness increasing to 50 and 75 mm. Some analytical work during which the thick upper layers were subdivided into more than six 25 mm sublayers, indicated a significant difference from that obtained when six relatively thick sublayers were used, that is when the number rather than the thickness of the layers was increased. The variation in results may be a computation problem and it was decided to accept six thicker sublayers rather than numerous 25 mm sublayers. This may have had an effect on the actual thermal stress values which are reported later.

The analyses were performed with the data set stored in the computer program for the average mean climatical data of Pretoria, South Africa. The temperature calculation was started with a uniform temperature distribution of 20 °C, but during the analysis it was adjusted very rapidly and hence complied with the temperature gradient for Pretoria.

5.3.3 Material properties

Two types of crusher-run were considered; according to their thermal conductivities, the one was a granite and the other a quartzite (Bullard, 1939). They were each considered at two densities - at 88 and 82 per cent of the solid density of the particles and were called dense and loose (see

Appendix C). In the study the quartzite was considered as the basic material and the granite was compared with it. The thermal conductivity and specific heat of the materials in the other layers were obtained from the catalogue of average mean values contained in the computer program, because the number of variables had to be kept to a minimum. Table 5.2 contains a summary of the thermal properties used during the analyses.

TABLE 5.2 : Thermal properties used during the analyses

PROPERTY	BITUMINOUS MATERIAL	CRUSHER-RUN				TREATED BASE AND SUBBASE	UNTREATED SUBBASE	SUBGRADE
		Granite		Quartzite				
		Loose	Dense	Loose	Dense			
Conductivity (cal/cm.s.°C)	0,007	0,0032	0,0042	0,0067	0,0093	0,005	0,003	0,002
Specific heat (cal/g°C)	0,20	0,20	0,20	0,20	0,20	0,20	0,20	0,18
Density (g/cc)	2,2	2,14	2,31	2,2	2,36	2,1	1,9	1,8

The absorptance and emittance were fixed at 0,95 which implies a dark surface such as a prime or a thin surface treatment on the crusher-run. Williamson (1972a) evaluated the importance of these coefficients and since the absorption coefficient has a significant influence on the surface temperature it was decided to keep these values constant throughout the analyses.

The coefficient of thermal expansion of the two cement-treated layers was fixed at $4{,}5.10^{-6}$ per ^{o}C. Based on previous laboratory studies (Otte, 1974) and assuming the tensile strength to be half the flexural strength, the tensile strengths of the upper and lower cement-treated natural gravels were fixed at 350 and 120 kPa, respectively. The coefficient of subgrade restraint was fixed at 0,90.

5.4 DIURNAL AND ANNUAL VARIATIONS

The results from the computer analyses can be presented in many ways and it is possible to prepare several graphs showing the diurnal and annual variations in stress through the treated materials. Since this is a design-orientated study, only the stresses at the bottom of the treated layer are considered because it is expected that when the traffic-associated stresses are included, this position will experience the maximum total stress.

5.4.1 Diurnal variation

Figure 5.3 shows that the magnitude of the thermal stress in the upper treated layer varies considerably throughout an average day in December. Unlike the traffic-associated stresses, the maximum thermal stress seldom occurs at the bottom of the treated layer. The variation in stress with depth is small in comparison with the diurnal variation.

Figure 5.4 shows a typical variation in stress at the bottom of each of the two treated layers. These locations experienced tensile stresses in the morning, around midday they were stress-free, and in the afternoon they went into compression. The time of the highest tensile stress, which is the critical time with regard to cracking, is during the morning with a peak around 7 to 8 o'clock. This same pattern also applied to each of the other months of the year. The critical time for cracking differs significantly from what Lister (1972) calculated, since he found it to be the early afternoon.

Figures 5.5 and 5.6 show typical variations in the number of equivalent 80 kN axles during typical days on six South African roads (note the difference in vertical scale). Since the heavy axles contribute significantly to the number of equivalent axles, these distributions may be taken as an indication of the hourly distribution of heavy axles. This information was obtained with the Axle Weight Analyser (AWA) (Basson et al, 1972) and consisted of hourly counts during day-time but only an average from dusk till dawn. It shows that the load distribution varies from road to road with very little of a standard distribution although the low night-time averages indicate that the bulk of the heavier traffic travels during the day when both the tensile and compressive thermal stress peaks occur. Since the tensile stress peak contributes to failure and the compressive peak cannot 'heal' the damage done during the tensile peak, they do not really cancel out and the effect of the tensile peaks is cumulative. It would therefore be beneficial if the traffic loading, and hence the traffic-associated tensile stresses, could be minimized during the periods of peak thermal tensile stresses and maximized during periods of peak thermal compressive stresses, for example between 13h00 and 16h00. Since the traffic seems to be slightly lighter during night-time, it would also be beneficial if the structural layout and material properties could be changed in such a way that the peak tensile stress would occur during the night and not during the day.

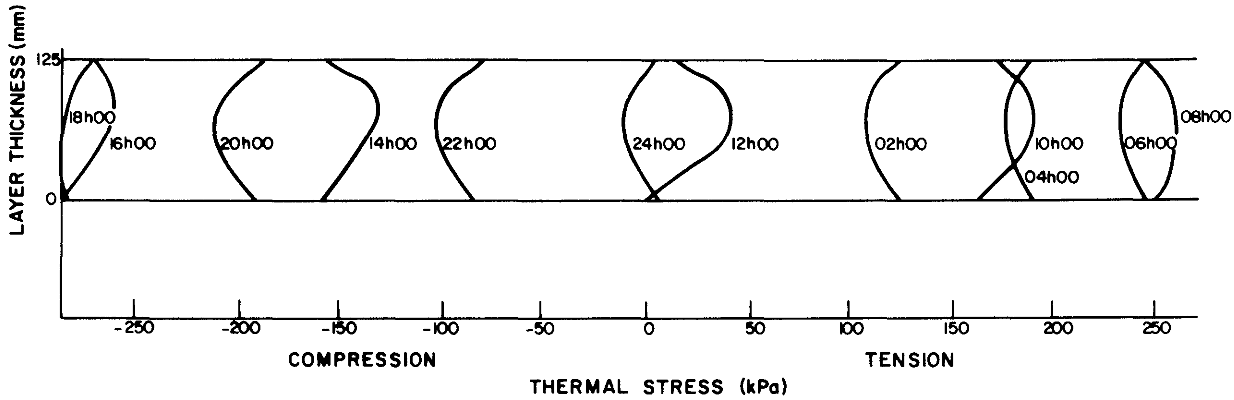


FIGURE 5-3

DIURNAL STRESS DISTRIBUTION WITH DEPTH IN THE UPPER UNCRACKED TREATED LAYER

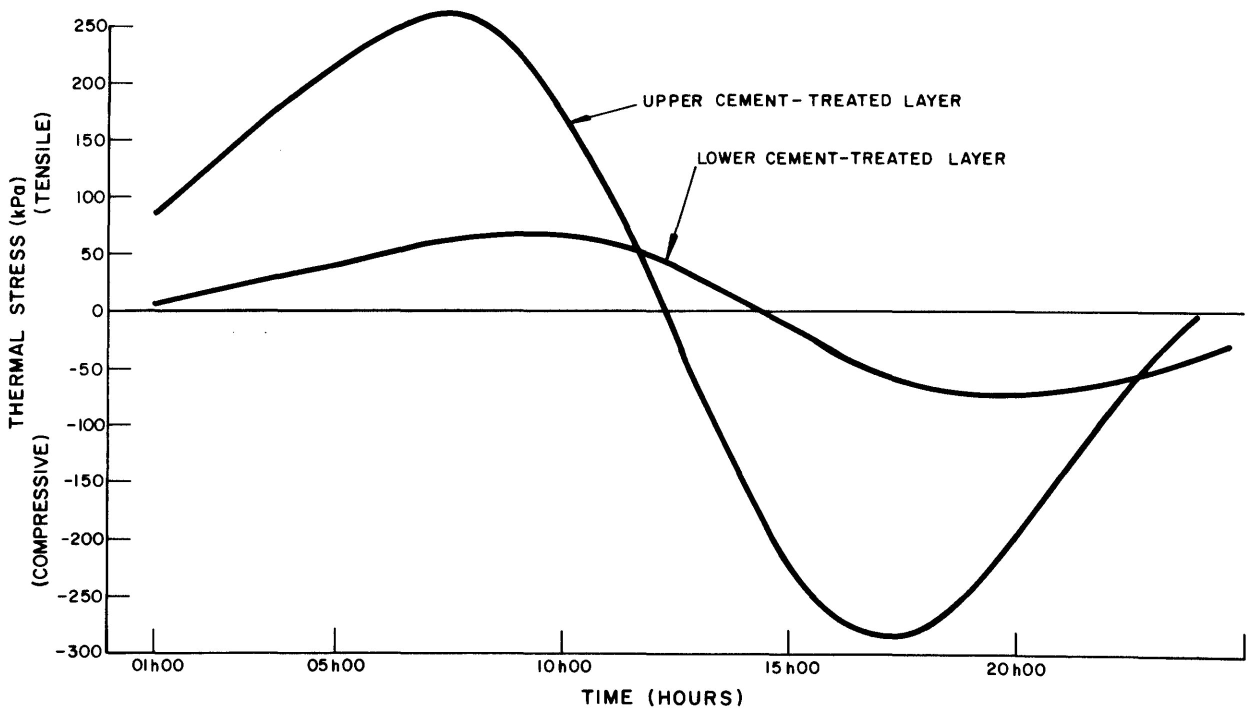


FIGURE 5-4

TYPICAL DIURNAL VARIATION IN THERMAL STRESS DURING DECEMBER IN TWO UNCRACKED CEMENT-TREATED LAYERS UNDER 150mm CRUSHER-RUN

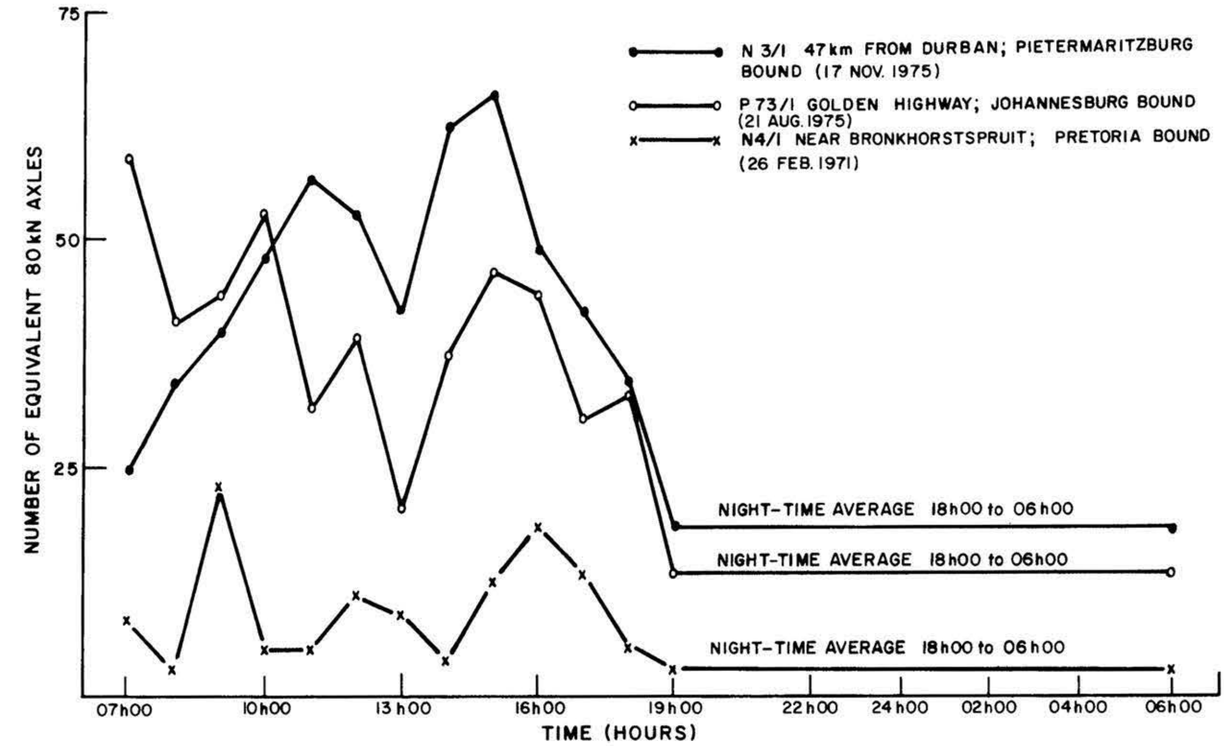


FIGURE 5-5

DIURNAL VARIATION IN EQUIVALENT TRAFFIC ON THREE ROADS

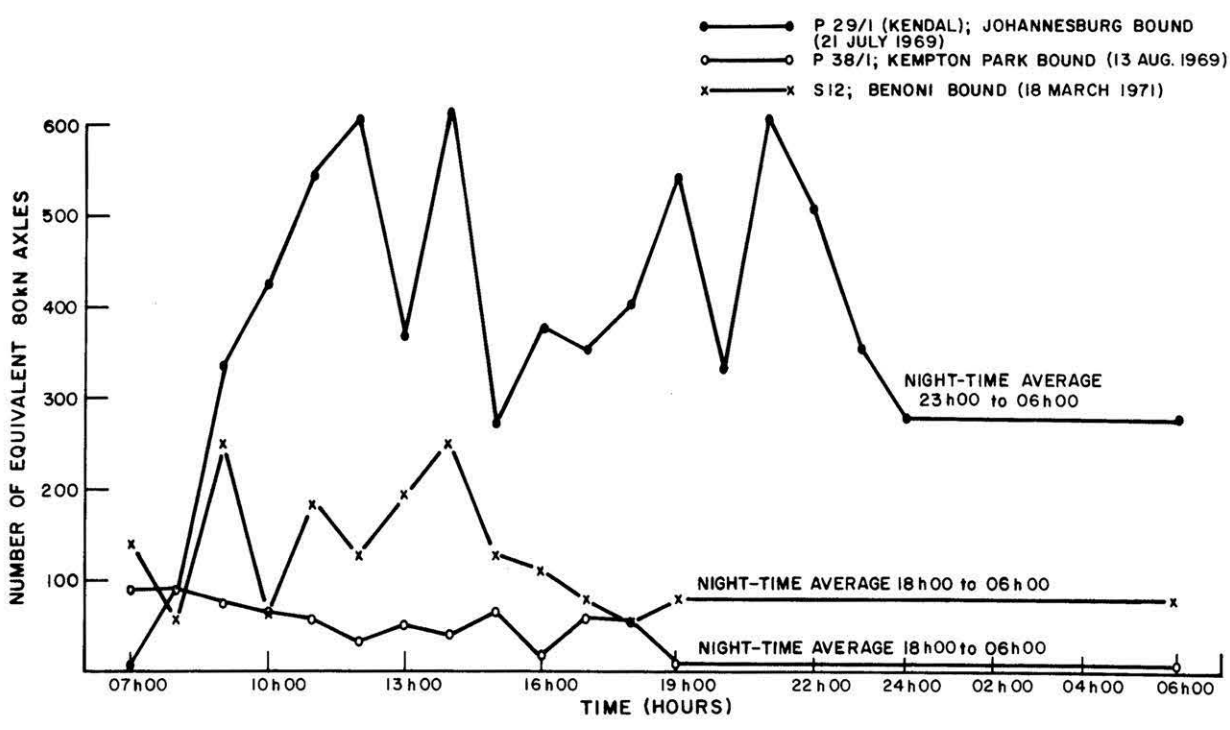


FIGURE 5-6

DIURNAL VARIATION IN EQUIVALENT TRAFFIC ON THREE ROADS

5.4.2 Annual variations

Figure 5.7 shows the theoretical annual variation in the daily maximum tensile and compressive stress at the bottom of the two treated layers of the basic structural layout when it is located in Pretoria. Average material properties were used for the crusher-run in this layout (Williamson, 1972) and the thermal conductivity and density were taken as 0,006 cal/cm.s.^{o}C and 2,2 g/cc respectively. The figure shows that both the maximum tensile and compressive stresses occur during December and the minima during June. This same tendency was observed for all the designs analysed in this study and, unless otherwise stated, the tensile stress values reported in this chapter are the maxima which occurred during December.

5.4.3 Combined traffic-associated and thermal stresses

The traffic-associated tensile stresses under a 40 kN wheel load (520 kPa tyre contact pressure) were calculated, using the CHEVRON computer program, as -509 and 209 kPa for the top and bottom of the upper treated layer, and as 67 and 331 kPa for the top and bottom of the lower treated layer. These stresses were added to the thermal stresses and presented as Figure 5.8. It shows that although the thermal stresses are highest in the upper layer, the total stresses (traffic-associated plus thermal) are highest in the lower layer. The bottom of the lower layer is thus the critical position in practice.

5.4.4 Discussion

It appears that the maximum thermal stresses in the two cement-treated layers developed during the mornings of December. It should be stressed that these are only average values and the occurrence of the true maxima are dependent on numerous environmental and climatic factors.

The thermal stresses in the lower cement-treated layer were very much less than those in the upper layer. Once the traffic-associated stress had been added, the stress position was reversed and the maximum total tensile stress occurred at the bottom of the lower layer.

5.5 THERMAL INSULATION

The use of insulating materials to attenuate temperature gradients is practical and it is considered advisable that they should be used to insulate treated layers in pavements. The insulating ability of the various materials should however be studied before practical recommendations can be made on how they should be used.

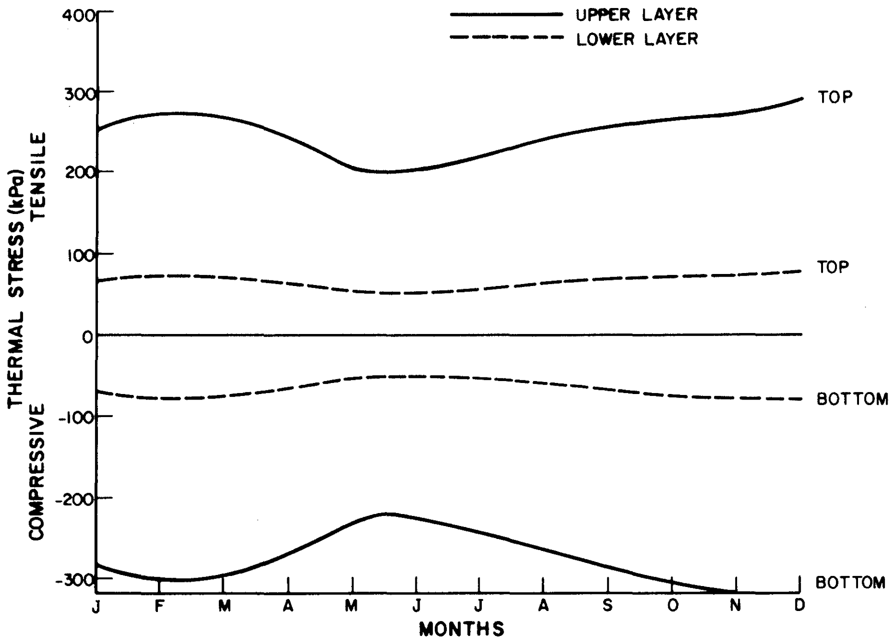


FIGURE 5-7

ANNUAL VARIATION IN DAILY MAXIMUM THERMAL STRESS AT TOP AND BOTTOM OF UNCRACKED LAYERS.

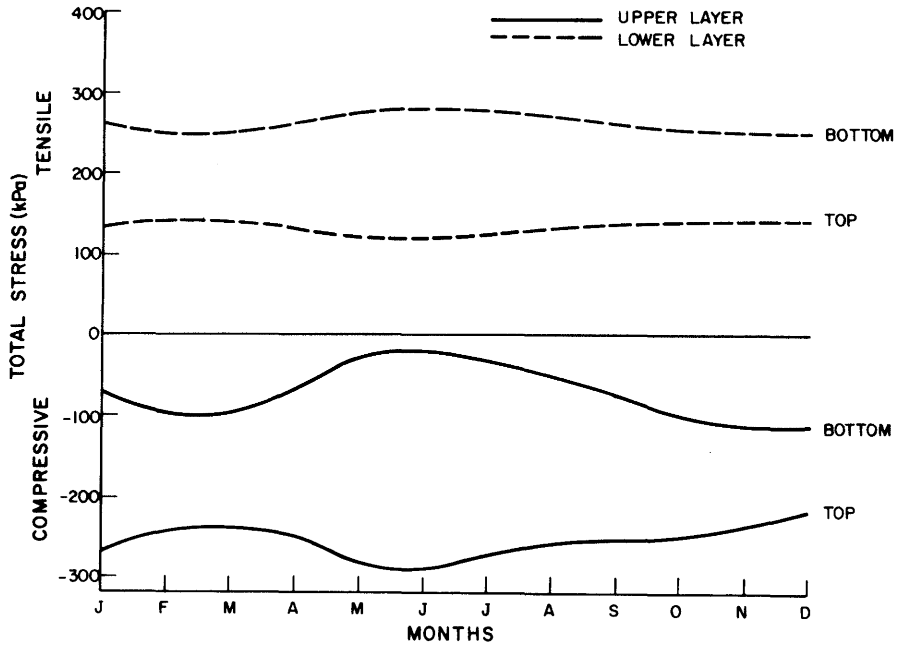


FIGURE 5-8

ANNUAL VARIATION IN DAILY MAXIMUM TOTAL STRESS AT TOP AND BOTTOM OF UNCRACKED LAYERS.

Heat transfer in road pavements may be described as an unsteady state problem since the heat input, for example the solar radiation, does not remain constant throughout the day but it varies from time to time. This problem is therefore also covered by the unsteady-state one-dimensional heat conduction formula (Hsu, 1963)

$$\frac{\delta T}{\delta t} = D\frac{\delta^2 T}{\delta x^2} \quad \text{.......................................} \quad (5.1)$$

where T = temperature
t = time
x = distance
D = diffusivity

The amount of heat transfer is governed by the thermal diffusivity which is defined as

$$D = \frac{k}{s\rho} \quad \text{...} \quad (5.2)$$

where k = thermal conductivity
s = specific heat
ρ = density

The diffusivity is a combination of the material's ability to conduct the heat through it (conductivity) and to store or retain it (density times specific heat).

In a pavement with an insulating layer the amount of heat transfer through this layer and into the treated layers is the matter for concern. The thermal diffusivity of the insulating layer should be low since this will result in the least heat transfer and hence the maximum insulation, and this in turn will cause smaller thermal gradients in the lower layers. A reduction in the diffusivity can be achieved by (i) reducing the conductivity so that the material cannot conduct the heat through it, or (ii) by increasing the density and specific heat which will enable the material to retain more heat in itself. One should therefore theoretically be able to reduce the diffusivity, and hence increase the insulating ability, of say a soil or crusher-run, by increasing its density. The opposite, however, usually happens after an increase in density. The diffusivity actually increases because the increase in conductivity, as a result of the removal of air from between the grains, exceeds the increase in density (see Appendix C).

5.5.1 Material type and quality

Figure 5.9 shows the effect of changes in the diffusivity of the insulating material on the maximum thermal tensile stresses in the basic layout (Figure 5.1). The changes in diffusivity were brought about by considering two types of crusher-run (quartzitic and granitic) at two densities, and one bituminous material. The figure shows that the thermal tensile stresses at the bottom of the two layers decreases only slightly with an increase in diffusivity. This figure may be used to evaluate the effect of further changes in the density and rock-type of the crusher-run insulating layer. It seems, however, that the insulation is relatively independent of these parameters and that a change in rock-type and density will produce only second order changes in the thermal stress values.

5.5.2 Thickness of insulator

Figure 5.10 shows the reduction in thermal stress with an increase in the thickness of the insulator. When this figure is compared with Figure 5.9 it becomes clear that the thickness of the insulator is much more important than the type and quality of the material because there is very little difference between using a bituminous material and a crusher-run as the insulator. In all instances the insulation succeeded in preventing thermal cracking of the treated layers because the thermal stresses were below the tensile strengths specified in section 5.3.3. An analysis in which the insulation was removed showed that thermal cracking would occur very quickly in both of the two treated layers - even before sunrise. The program predicted the crack spacing at about 11 metres; this result differs from the author's practical observations of generally 3 to 6 m. The relatively large spacings may be a result of the relatively high tensile strengths that were specified, 350 and 120 kPa.

An interesting observation is the shape of the curves for the two materials in Figure 5.10 - convex for quartzite and concave for granite. One should however not rely very quantitatively on these curves and the stress values shown in the figure because, as was explained in section 5.3.2, the thickness and number of sublayers used for the insulator has an effect and this does influence the values and the shape of the curves.

5.5.3 Cracked treated layers

It was mentioned above that both of the two treated layers will crack if no insulator is used. Figure 5.10 and all the previous figures were prepared for intact treated layers and are therefore of very little practical

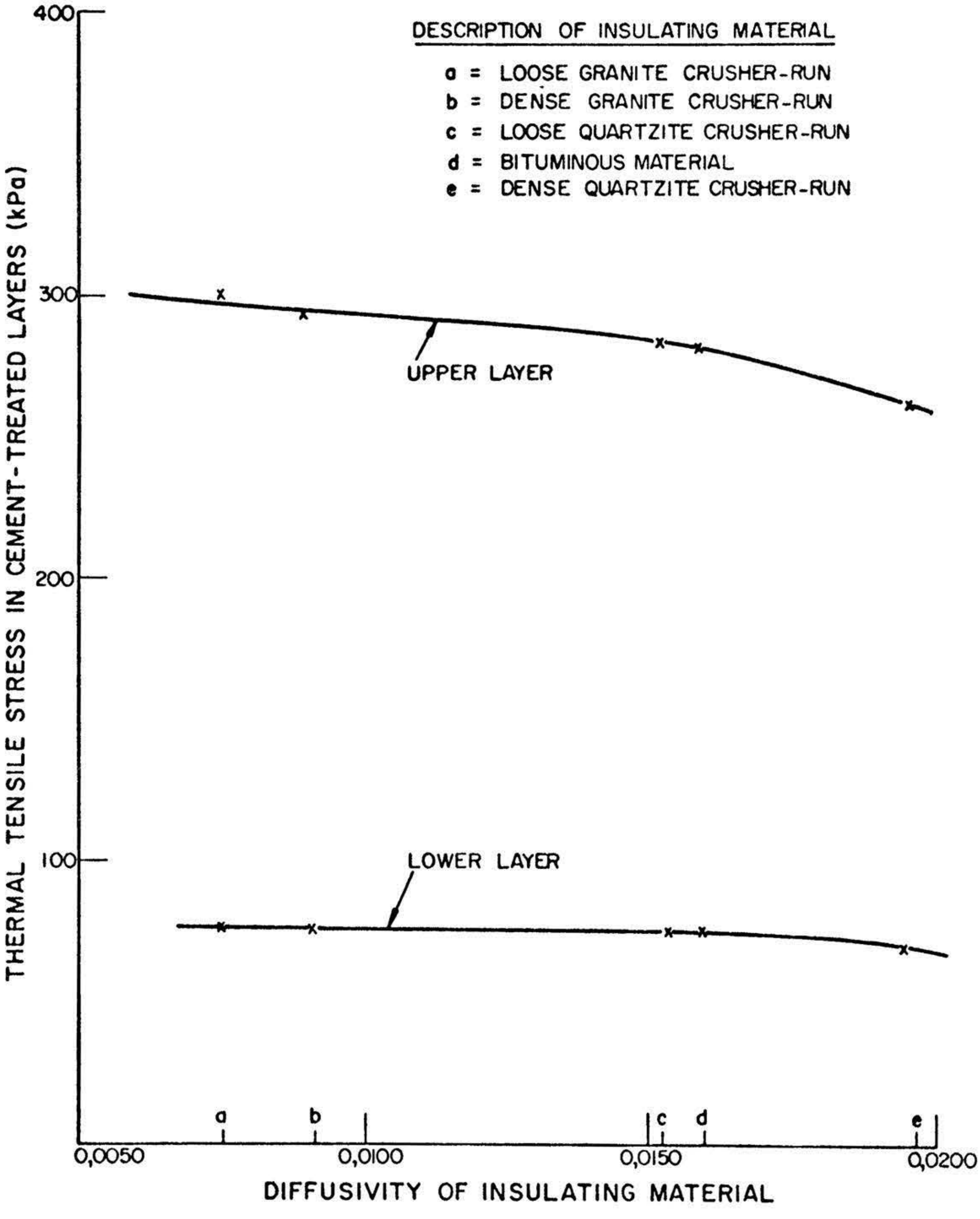


FIGURE 5-9

EFFECT OF DIFFUSIVITY ON MAXIMUM THERMAL TENSILE STRESS

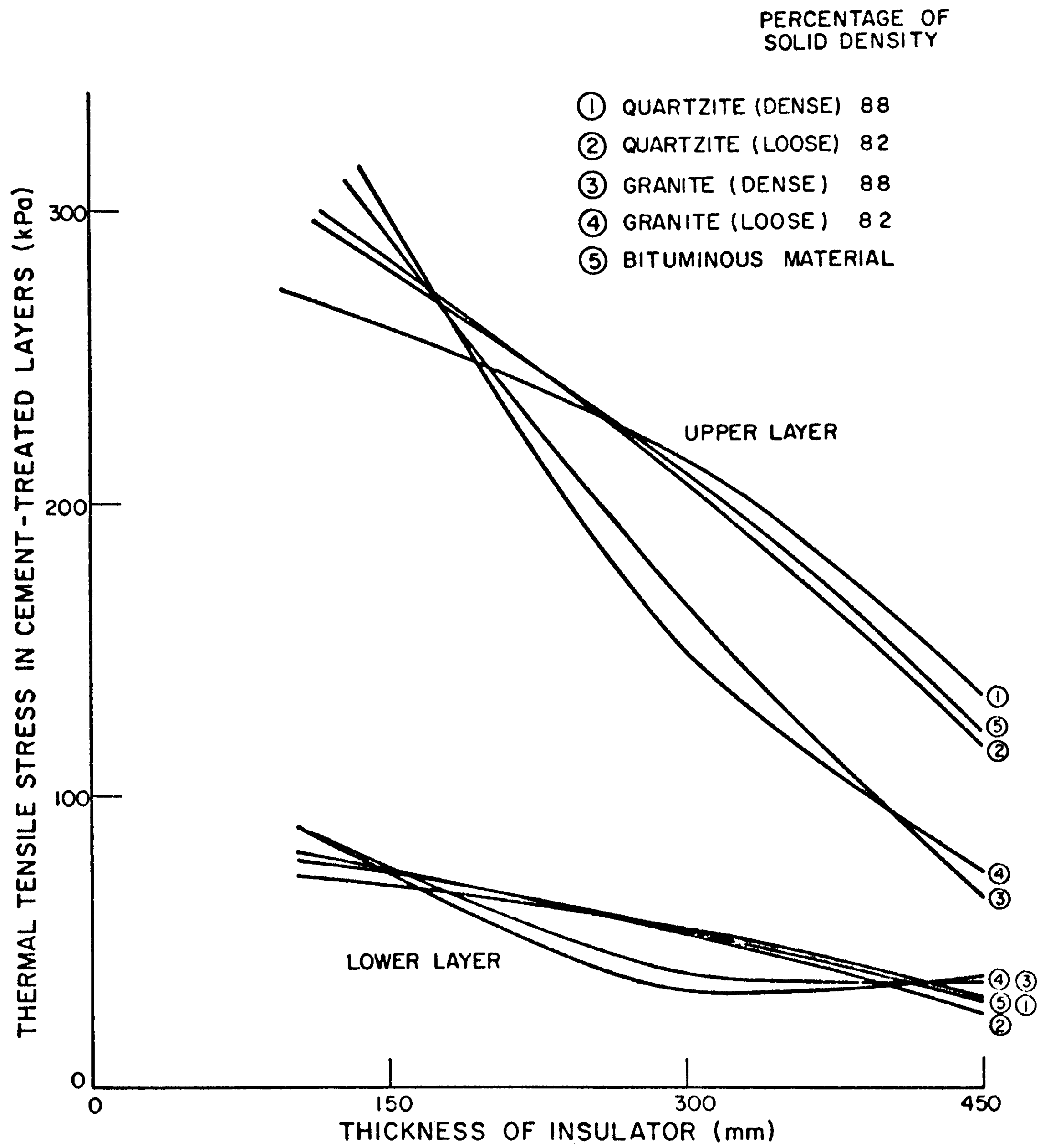


FIGURE 5-10

EFFECT OF INSULATING LAYER ON THERMAL TENSILE STRESS significance since there often is quite a delay between the construction of the treated layer and the completion of the crusher-run insulator. During this period the treated material hardens and, depending on the hardening rate, the temperature cycles and the material quality, it will crack. If temperature is not the cause of the cracking, then shrinkage will be, but cracking is virtually inevitable. To obtain a fairly practical result on insulation, the analyses had to be repeated and cracked treated layers were assumed. Based on the results of the analyses with no insulating layer, the crack spacing and width were taken as 11 metres and 0,3 mm respectively.

The analyses of cracked treated layers showed a marked reduction in maximum thermal tensile stresses. This is quite probably due to the presence of the crack which allows some movement to take place and hence relieves the build-up in stress. The analysis with no insulating layer indicated the maximum thermal tensile stresses at the bottom of the two cracked treated layers to be very high; 150 mm dense quartzitic crusher-run was capable of reducing these values to less than about 140 and 2 kPa respectively, while 450 mm crusher-run reduced both to below 4 kPa.

The crack spacing in cement-treated natural gravel bases in South Africa varies from road to road and it is very difficult to define an average spacing. It was, however, felt that the 11 m used in the previous analyses was too long because the tensile strengths used to calculate it corresponded to those attained only after the material had been in place for some time. The material is more likely to crack at an earlier age while the tensile strength is still low, and this will result in closer crack spacings. Based on the author's observations 6 m was chosen as the average. Assuming the crack width as 0,3 mm, a 150 mm crusher-run insulator succeeded in keeping the maximum tensile stresses below 11 and 5 kPa respectively. When the crack width was significantly reduced, to only 0,1 mm, the maximum tensile stresses increased to about 175 and 45 kPa respectively. The calculated thermal movements at the crack openings were very small, only about 0,1 to 0,3 mm, and this may explain why 150 mm crusher-run is usually adequate to prevent completely the cracks in cement-treated natural materials from appearing on the road surface.

Figure 5.10 indicates that an insulator of about 450 mm is necessary to reduce the thermal stresses in uncracked treated layers to less than about 140 kPa. From the previous discussion it appears that this can also be achieved by only 150 mm crusher-run, provided the layer is slightly cracked and movement may take place. Since it is virtually impossible to avoid cracking in a properly constructed cement-treated layer it will always have this form of stress relief. It is therefore suggested that 150 mm crusher-run is adequate and should be provided as a thermal insulator on cement-treated materials because (i) this is a practical thickness from a construction point of view, (ii) it will prevent the cracks in the treated layers from propagating to the surface, and (iii) it is usually cheaper than a comparable thickness of bituminous material.

5.6 THERMAL STRESSES IN OTHER PAVEMENTS

The previous analyses were done for only one design layout. To study the applicability of the observations to other layouts it was decided to analyse the structural layout that was used on Special Road S12 between Cloverdene and Argent, and on National Route N4/1 between Pretoria and Bronkhorstspruit (Otte, 1973a).

The pavements consist of 25 mm asphaltic concrete surfacing, 100 mm cement-treated crusher-run, 100 mm untreated crusher-run, 100 mm cement-treated natural gravel subbase and some selected subgrade. The material properties and other details required to run the computer program are summarized in Appendix D.

Assuming an uncracked cement-treated crusher-run base (elastic modulus = 18 500 MPa) it was calculated that the layer would crack at 19,9 m spacing and the crack openings would be about 1 mm. The associated maximum thermal tensile stress after cracking was about 500 kPa. The large crack spacing was caused by the high tensile strength and, because the cracks would probably have formed while the strength was still low, it was decided to repeat the analysis. During the earlier crack measurements on the pavements (Otte, 1973a), the crack spacing and width were recorded as about 4 m and 1 mm respectively. The analysis was repeated for a pavement cracked accordingly and the maximum thermal tensile stress was less than 10 kPa.

The same general observations therefore apply to both types of structural layout and hence possibly to all layouts with cement-treated layers. The thermal stresses are very high in an uncracked layer, but in a cracked layer, and all properly constructed cement-treated layers do crack initially, movement can take place at the crack openings and this reduces the thermal stresses significantly.

5.7 DISCUSSION

The theoretical analysis indicated that thermal stresses were very detrimental in uncracked treated layers and that their magnitudes were comparable to those of the traffic-associated stresses. They should therefore be reduced and a practical way is to provide an insulating layer, for example crusher-run or bituminous materials, on top of the treated layer. Theory also indicated the thickness of the insulator to be much more important than the type of material used and its thermal properties.

A properly constructed cement-treated layer will crack at a relatively early age and the crack spacing will be much less than that calculated from the material properties after hardening. After initial cracking the chances of further thermal cracking become very slim because the slabs are short (4 to 6 m), movement can take place which eliminates a build-up of thermal stress, and the tensile strength continues to increase. The amount of thermal stressing in these shorter slabs is very small, and in comparison with traffic-associated stresses it may be neglected. The relative unimportance of thermal stresses in cracked treated layers may be one of the reasons why no need has ever arisen for considering them in the existing pavement design procedures.

5.8 CONCLUSIONS

(a) The computer program of the Macro Environmental Simulation Model (MESM) was found to be very suitable for the analysis of temperature distributions and the associated thermal stresses in pavements. It is thought to be the only program capable of computing thermal stresses in a cracked pavement layer.

(b) The position of the maximum thermal tensile stress during the day varies throughout the depth of the pavement. It may however be assumed that in uncracked treated layers the maximum value required for pavement design purposes occurs at the bottom of the treated layers during the mornings in December. In cracked treated layers there are more variations and, depending on the layout, the maximum thermal stress may occur at virtually any hour.

(c) The thickness of the insulator is much more important than the type of material used. The selection of the insulating material is therefore not governed by its thermal properties but by others, for example its load-bearing ability.

(d) The usual construction sequence and the material properties are such that cement-treated layers will crack, either due to temperature or shrinkage, and they should be considered as cracked. The small movements of the fine cracks are sufficient to eliminate a build-up in thermal stresses and the resulting stresses are significantly smaller than those calculated for uncracked layers. The importance and possible influence of thermal stresses on pavement performance is therefore significantly reduced by the fine cracks which are present in the material.

(e) The amount of thermal movement at the cracks is relatively small (0,1 to 0,3 mm) and 150 mm crusher-run is considered adequate to dampen the cracks completely and prevent them from appearing on the pavement's surface.

(f) The use of a 150 mm crusher-run layer on top of the treated layer will also help to insulate the treated layer against thermal stresses.

(g) The outcome of this study was quite unexpected in the light of previous thinking on thermal stresses in treated layers. Previously thermal stresses were considered to be very important but this study has shown it to be true only for uncracked treated layers. Once the cracks have developed the thermal stresses become virtually negligible in comparison with the traffic-associated stresses. Thermal stresses may therefore be considered as unimportant in cracked treated layers.

CHAPTER 6

VARIATIONS IN QUALITY ARISING DURING CONSTRUCTION

6.1 INTRODUCTION

It is important that the properties of the materials produced during construction should correspond to those assumed by the designer when he does the design. This correspondence is generally assumed because it is accepted that the instituted construction controls, for example field density tests and compliance with the construction methods outlined in the specification, are adequate to ensure it. Variations in the specifications of consulting engineers and differences between the construction techniques, that is mixing, compacting and curing, used by the various constructing organizations, both public and private, are considered significant and it is believed that their effect should be evaluated. The effect of all these aspects on the properties of cement-treated materials has been studied extensively - but individually. The author believes that the combined effects and interaction should be studied under the general heading of construction technique.

The objectives of this chapter are therefore (i) to study the variation in the properties of cement-treated materials constructed in the field, and (ii) to compare the properties of field-constructed materials with those of materials prepared in a laboratory under ideal conditions. The outcome can possibly be utilized in the development of a rational pavement design method since it will indicate to the designer what allowance he should make to accommodate the construction process.

6.2 LITERATURE REVIEW

One of the first studies on the difference between field strengths and the design values was performed by Robinson (1952). In his case the difference was largely caused by an insufficient distribution of the cement through a silty clay. He showed that if the mixing efficiency could be increased and a more even distribution of the cement could be obtained, less cement would be required to comply with the specified strength. Since this represented about a 30 per cent reduction in total additive required, it could mean a significant financial saving on large contracts. This serves to show the importance of efficient mixing in reducing the difference in strength between field- and laboratory-prepared samples.

Mitchell and Freitag (1959) reported that British engineers "...found that normal construction methods result in a field strength equal to about 60% of the laboratory strength for a given cement treatment...". The cement content should therefore be determined as the amount necessary to obtain a laboratory compressive strength equal to the required strength divided by 0,6. This implies that if a strength of 1 700 kPa is required, the laboratory strength should be 2 800 kPa. The recommendation by Ingles and Metcalf (1972) seems to have been based on this work.

Wang (1968) performed compressive and bending tests on both field- and laboratory-prepared cement-treated materials. He compared the strengths and elastic moduli and Table 6.1 summarizes his results. He could not recover beam samples for testing from the materials treated with 6 per cent cement until two months after their construction and no beam samples could be recovered from the 3 per cent cement section because the materials were too weak.

TABLE 6.1 : Comparison in properties of field- and laboratory-prepared specimens (from Wang, 1968)

MATERIAL PROPERTIES			FIELD-PREPARED	LABORATORY-PREPARED	RATIO OF FIELD- TO LABORATORY-PREPARED
Strengths	Unconfined compressive strengths (kPa)	3% cement	140- 340	410- 760	0,33-0,45
		6% cement	415-1 030	760-1 720	0,55-0,60
	Bending strengths (kPa)	3% cement	*	100- 280	*
		6% cement	*- 450	380- 660	*-0,69
Elastic Modulus	Compression (MPa)	3% cement	70- 660	280-1 030	0,25-0,64
		6% cement	140-1 170	1 000-2 200	0,13-0,53
	Bending (MPa)	3% cement	*	410-1 240	*
		6% cement	*-1 720	900-3 030	*-0,57

*Too weak to be sampled and tested.

Wang (1968) obtained the same densities in the field as in the laboratory; nevertheless both the strength and elastic modulus of the field samples were only about 50 to 60 per cent of the corresponding laboratory samples. He gave the possible reasons for the difference as -

(a) better mixing in the laboratory than in the field;

(b) curing conditions in the field not as effective as in the laboratory;

(c) disturbance of field samples during cutting and extraction.

He stated that "...Among the possible causes, the effect of low efficiency mixing seems to be a major factor. In addition, the differences in curing condition might be quite significant...". This implies that he was not very sure of the cause of the differences.

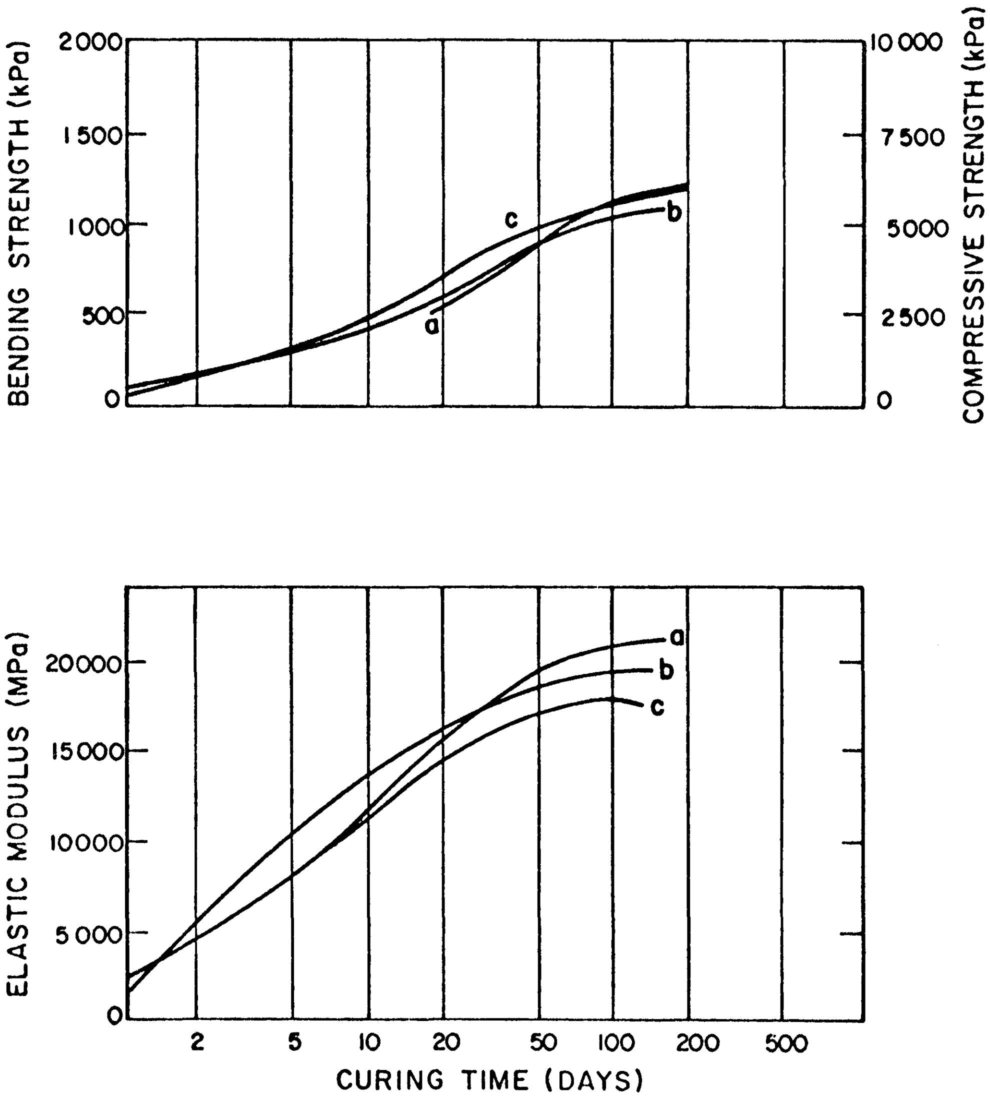
Fossberg (1970) recorded the differences between three construction conditions namely (a) mixing in a ready-mix concrete truck (called truck mixing) and field compaction, (b) truck mixing and laboratory compaction, and (c) laboratory mixing and compaction. About the same densities were obtained in all three conditions and the recorded strength and elastic modulus values are shown against curing time in Figure 6.1. The differences were very small and he obtained about the same strength and elastic modulus for all three construction conditions. He did however observe structural anisotropy in the field-compacted materials, that is the elastic modulus in the direction of compaction was about 1,5 times lower than in the other two directions, an effect which was not found in the laboratory-prepared samples.

Structural anisotropy was also observed by Otte (1972a). Measurements on cement-treated crusher-run samples from 6 different freeway contracts indicated less anisotropy than recorded by Fossberg (1970). The dynamic elastic moduli in the direction of the compaction (called Modulus A) and perpendicular to this plane (called Modulus B) were recorded with an ultrasonic tester. The measurements were taken on wet and dry samples and the results are reported in Table 6.2.

TABLE 6.2 : Structural anisotropy in cement-treated crusher-run recovered from freeway contracts

CONTRACT	NO. OF SAMPLES TESTED	WET SAMPLES			DRY SAMPLES		
		Modulus A	Modulus B	Anisotropy	Modulus A	Modulus B	Anisotropy
1	6	34 600 (2,3)	37 000 (4,6)	1,069	25 100 (5,3)	28 700 (3,1)	1,143
2	12	30 100 (4,0)	35 900 (4,3)	1,192	23 200 (14,5)	27 800 (11,2)	1,198
3	12	25 200 (6,5)	29 200 (7,4)	1,158	17 300 (13,5)	19 500 (12,0)	1,127
4	12	29 700 (6,3)	31 900 (6,6)	1,074	20 700 (7,7)	23 900 (11,9)	1,154
5	9	27 444 (7,2)	30 178 (5,5)	1,099	23 666 (7,2)	27 266 (5,8)	1,152
6	10	29 000 (5,0)	32 200 (3,6)	1,110	25 000 (4,2)	30 400 (3,3)	1,216

The numbers in brackets are the coefficients of variation in per cent.



a = TRUCK MIXING AND FIELD COMPACTION
b = TRUCK MIXING AND LABORATORY COMPACTION
c = LABORATORY MIXING AND COMPACTION

FIGURE 6-1

EFFECT OF MIXING AND COMPACTION ON ELASTIC PROPERTIES OF A SOIL-CEMENT (after Fossberg 1970)

Anisotropy is real and it can affect the relationship between field- and laboratory-prepared samples since the outcome of the comparison depends on the direction in which the properties of the field samples were determined.

The results of a previous study (Otte, 1974) indicated a strong possibility that the construction technique has a significant influence on the properties of cement-treated materials. Samples were taken from fourteen pavements which were built by different contractors and supervised by various authorities and consulting engineers. Although all the contracts were constructed to the same nominal specification and thus regarded by engineers to be of the same quality, the differences in the material properties were significant. The bending strength varied between about 400 and 4 400 kPa, that is a factor of 10; the strain at break varied between about 113 and 251 $\mu\varepsilon$, a factor of 2,2; and the elastic modulus varied by a factor of 10,5 between about 3 700 and 38 900 MPa. These results indicate that cement-treated materials should not be regarded as having the same structural capacity just because they were built to the same specification.

On two adjoining cement-treated crusher-run sections, described in section 3.2.1.1 (page 39), the material quality differed significantly. Although the same material, specification and construction team applied to both, the one had a high elastic modulus and was cracked while the other one had a low elastic modulus and was uncracked. This significant difference in materials quality could really only have been caused by the construction variations.

The literature survey seems to indicate that the important parameters to ensure reasonable agreement between field- and laboratory-prepared samples are (i) cement content and the uniformity of its distribution, (ii) density, (iii) delay between mixing and compaction, and (iv) efficient curing. If good agreement between the field and laboratory conditions can be maintained for these parameters, the material properties ought to agree also.

6.3 DESCRIPTION OF CONTRACTS

A brief description and location of the 10 contracts evaluated in the study are given in Table 6.3. It contains the road number, the layer thickness, the maximum dry density at Mod. AASHO compaction and the optimum moisture content of the untreated soil, the average field density of the material, the approximate construction date, and the use of the material in the structural layout.

TABLE 6.3 : Description of the contracts sampled for a study of field versus laboratory properties of cement-treated materials

CONTRACT NO.	ROAD NUMBER AND LOCATION	LAYER THICKNESS (mm)	MAXIMUM DRY DENSITY (kg/m^3)	OPTIMUM MOISTURE CONTENT (%)	AVERAGE FIELD DENSITY (%)	CONSTRUCTION DATE	LAYER
1	Residential streets in Lynnwood Glen	150 150 150	1 838 1 770 1 833	17,3 20,5 16,5	95 95 96	November and December 1975	Subbase
2	S-12, Argent to Kendal	150 150 150	2 140 2 140 2 160	7,1 7,1 7,0	96 97 97	February and March 1976	Lower subbase Upper subbase Lower subbase
3	P158/2, Diepsloot to Mnandi	150	2 020	10,1	97	7 April 1976	Subbase
4	P56/2, Vergenoeg to Vaal River	150	1 935	12,4	95	20 April 1976	Subbase
5	Apron at Jan Smuts Airport	120	1 965	14,4	97	9 December 1975	Base
6	P88/1, Vereeniging	150	2 160	7,5	98	17 June 1976	Subbase
7	P106/1, Pretoria North	150	1 960	9	96	29 April 1976	Base
8	N1/23, Pienaars River to Warmbaths	150	2 030	13,6	95	26 May 1976	Lower selected subgrade
9	P5/1, Kinross	150	2 200	9	96	18 May 1976	Subbase
10	P73/1, Evaton	150	1 865	15,0	96	7 May 1976	Subbase

Contract 2 consisted of a number of experimental sections. In this study each experimental section was considered individually and they will be referred to by their S-numbers (NITRR, 1973) as for example 2(S6) or 2(S10). The third material mentioned under contract 2 is a lateritic soil that was used on sections 2(S7) and 2(S8). The first two materials are grindstone that was used on the other sections of the contract but the percentage compaction and the stabilizing agent (see later) were varied for the upper and lower subbase.

The reasons for stabilizing the various materials varied. This explains why the type of stabilizer and specified strength criteria varied. Table 6.4 summarizes the percentage and type of stabilizer used and the criteria aimed for.

TABLE 6.4 : Percentage and type of stabilizer and criteria

CONTRACT NO.	PERCENTAGE OF STABILIZER	TYPE OF STABILIZER	SPECIFIED STRENGTH CRITERIA
1	3 to 3,5	Lime	Reduction of plasticity index; did not aim for increased strength
2	3,75 4,0 5	50-50 mixture of Slagment and Portland cement	UCS*= 1 200 kPa after 7 days UCS*= 1 725 kPa " " " UCS*= 1 500 kPa " " "
3	3,5	Portland blast furnace cement	UCS*= 1 500 kPa after 7 days
4	4	Lime	CBR > 70
5	4	Lime	CBR > 70
6	4	Portland blast furnace cement	UCS*= 1 500 kPa after 7 days
7	4	50-50 mixture of Slagment and Portland cement	UCS*= 1 500 kPa after 7 days
8	3	50-50 mixture of Slagment and lime	Reduction of plasticity index; did not aim for increased strength
9	4	50-50 mixture of Slagment and lime	CBR > 160; laboratory values were around 180 to 200
10	4	50-50 mixture of Slagment and lime	CBR > 160; laboratory values on 7 samples were 174, 167, 201, 163, 154, 191 and 146
*UCS = Unconfined compressive strength			

On all the contracts the mix-in-place technique was used with disc harrows and motorgraders to do the mixing. The materials were usually compacted with grid rollers. After compaction the layers were usually kept moist for about 7 days, but if it was possible the tar prime coat was applied sooner. On contract 2 it was a condition of the contract that the curing membrane should be applied immediately after the final compaction or very early the next morning.

6.4 OUTLINE OF STUDY

After arrangements with the client, through the consulting engineer, the site was visited and block samples (about 600 x 600 mm) were sawn from the treated pavement layer. This was done somewhere between 7 and 27 days after construction of the layer but usually between 20 and 27 days after construction. On one or two contracts it was necessary to remove the blocks relatively early and they were slightly moistened, sealed in plastic and stored in a humid room at about 20 ^{o}C. About 27 days after construction the blocks were sawn into 6 or 7 beam samples (75 x 75 x 450 mm), allowed to soak in water for about 24 hours, and tested in flexure according to the procedure outlined in section 2.2.8(c). In this chapter these samples will be referred to as field samples.

Usually more than one block was removed from a particular section. This was necessary to cope with possible collapsing of the blocks during subsequent handling. From some sections it was possible to recover blocks, but it was not possible to saw them into beams because (i) the block had a fine crack which only showed up when it was sawn, or (ii) the matrix was too weak to hold the larger (+75 mm) stones and when sawn they pulled out and the beams crumbled, or (iii) the material was too soft under wet cutting with a diamond blade. If any of these failures occurred it was not possible to obtain field samples.

During a visit to a site a 40 kg sample of the untreated soil and a sample of the stabilizing agent actually used by the contractor were obtained. The relevant soil constants, such as maximum dry density and optimum moisture content, for the material used on the contract were also obtained from the site office.

These soil constants were used throughout the study and no checks were made on the properties of the particular soil sample. It is appreciated that some variations can occur, but it was considered a just representation of construction practice. From each soil sample, that is for each contract, 8 beam samples (75 x 75 x 450 mm) were made. The samples were compacted for about 3 to 4 minutes on a table vibrating at 50 Hz, but the soil had to be placed in about 3 or 4 equal layers with tamping in order to work the predetermined mass into the mould. The samples were cured in a 100 per cent relative humidity room and tested at the same age as the corresponding field samples, which was usually 28 days. Hereafter these will be referred to as the laboratory-prepared samples.

Throughout the study it was endeavoured to compact the beams to the average density and percentage compaction measured by the resident engineer when he approved the construction of the layer. The differences between the materials on which soil constants were determined and those from which the samples were taken to prepare the laboratory specimens, and the difference between the specified optimum moisture content and that required by the vibrating compaction technique used in this study, did however result in lower densities being achieved in the laboratory (Table 6.5).

TABLE 6.5 : Difference in field and laboratory densities

CONTRACT	MAXIMUM DRY DENSITY OF MATERIAL (kg/m^3) (see Table 6.3)	FIELD SAMPLES		LABORATORY SAMPLES	
		Density (kg/m^3)	Percentage relative compaction	Density (kg/m^3)	Percentage relative compaction
1	1 770 to 1 838	*	-	-	-
2	2 140	2 064(2,2)	96	1 941(1,4)	91
	2 140	2 120(2,2)	99	1 954(1,8)	91
	2 160	2 083(5,9)	96	1 931(1,8)	89
3	2 020	1 968(0,1)	97	1 917(1,1)	95
4	1 935	*	-	1 777(1,8)	92
5	1 965	*	-	1 732(2,0)	88
6	2 160	*	-	1 960(1,1)	91
7	1 960	2 009(4,5)	102,5	1 803(2,2)	92
8	2 030	*	-	1 876(1,0)	92
9	2 200	2 114(3,3)	96	1 968(0,8)	89
10	1 865	*	-	1 712(1,2)	92

Numbers in brackets are the coefficient of variation in per cent.
*No field samples could be obtained; see 6.4

The averages and coefficients of variation of the 6 or 7 beams sawn from the field samples and those prepared in the laboratory, were calculated. For each contract the corresponding bending strengths, strains at break and elastic moduli in bending were compared.

6.5 RESULTS

The information obtained from the different contracts can be presented in various ways and it is possible to make various interesting observations.

6.5.1 Variation during a day's work

On five sections it was possible to saw beam samples from at least two different blocks. Since these blocks were constructed on the same day as part of the same section, the variations in their properties will indicate the variation during a day's work.

The contractor worked some of the adjoining experimental sections on contract 2 on the same day and although they are considered as separate contracts in this study, their variations may also be considered as variation during a day's work. They may therefore be included under this heading.

Table 6.6 contains the results and it appears that there was very little variation during a day's work. Statistically significant differences, at the 1 per cent level of significance, were only calculated for the bending strength and strain at break on contract 3, the elastic modulus on sections S5 and S6 of contract 2, and the bending strength between 2 of the 3 samples on contract 7. This implies that a section constructed on a particular day may be taken as homogeneous.

6.5.2 Variation in work performed on different days

Samples were recovered from the lower 150 mm of the cement-treated subbase on five sections of contract 2. These were all constructed with the same material and by the same construction team, but on different days. Table 6.7 shows the variations in material properties.

Engineers would normally regard these materials (Table 6.7) as having the same properties and structural capacity, because they were all constructed of the same materials, by the same contractor and according to the same specifications. A closer study of Table 6.7 will reveal that this assumption is incorrect since the bending strength varied by a factor of about 3,3, the strain at break varied by about 1,9 times, and the elastic modulus varied 2,3 times. Materials exhibiting these orders of variation should not be regarded as of the same quality!

TABLE 6.6 : Variation during a day's work

CONTRACT	NUMBER OF BEAM SAMPLES	BENDING STRENGTH (kPa)	STRAIN AT BREAK (με)	ELASTIC MODULUS (MPa)
2(S7)	8	785 (11,4)	129 (11,2)	7 788 (7,7)
	7	917 (21,4)	141 (13,2)	8 450 (8,7)
2(S8)	5	782 (18,4)	116 (15,4)	8 500 (4,3)
	7	785 (26,4)	130 (12,4)	7 992 (16,1)
3	6	502 (13,2)*	124 (16,4)*	5 900 (4,5)
	5	345 (7,1)	92 (14,5)	5 460 (2,1)
7	7	464 (17,3)*	237 (17,6)	2 825 (9,6)
	7	362 (17,1)	231 (27,4)	2 550 (17,1)
	6	385 (31,3)	249 (22,1)	2 608 (12,5)
9	5	129 (35,0)	172 (27,3)	1 810 (41,0)
	5	192 (37,3)	196 (20,1)	2 570 (28,5)
	6	173 (36,5)	225 (29,1)	2 190 (25,0)
2(S3)	6	843 (10,1)	92 (7,2)	10 683 (9,9)
2(S4)	7	928 (20,0)	89 (17,3)	11 264 (11,5)
2(S5)	5	632 (34,1)	128 (22,1)	6 199 (27,2)*
2(S6)	7	762 (19,2)	120 (11,4)	8 564 (12,3)
2(S16)	8	826 (20,9)	111 (17,3)	9 525 (15,1)
2(S17)	8	854 (16,3)	100 (16,0)	11 100 (18,7)

*Statistically significant variation at the 1 per cent level of significance.
The numbers in brackets are the coefficients of variation in per cent

TABLE 6.7 : Variation in quality of work performed on different days

CONTRACT	NUMBER OF BEAM SAMPLES	BENDING STRENGTH (kPa)	STRAIN AT BREAK (με)	ELASTIC MODULUS (MPa)
2(S3)	6	367 (35,7)	70 (7,5)	7 033 (24,5)
2(S6)	9	938 (12,5)	130 (15,0)	9 005 (7,6)
2(S10)	10	1 035 (24,1)	99 (13,0)	11 996 (13,3)
2(S17)	4	762 (11,1)	102 (26,1)	9 937 (20,0)
2(S19)	5	1 216 (24,0)	100 (17,1)	16 240 (2,5)

Numbers in brackets are the coefficients of variation in per cent

6.5.3 Variation within a layer

On two sections of contract 2 (sections S20 and S21) it was possible to saw and divide the blocks recovered from the lower 150 mm of the cement-treated subbase in such a way that beams could be sawn from both the upper and lower 75 mm of the layer. These will be referred to as the upper and lower halves respectively. The results are reported in Table 6.8.

TABLE 6.8 : Variation within a layer

CONTRACT	POSITION	NO. OF SAMPLES	BENDING STRENGTH (kPa)	STRAIN AT BREAK ($\mu\varepsilon$)	ELASTIC MODULUS (MPa)
2(S20)	upper	6	320 (16,0)	94 (18,3)	4 925 (24)
	lower	6	245 (17,0)	96 (16,1)	3 308 (15,3)
2(S21)	upper	5	361 (13,0)	104 (23,0)	5 600 (38,5)
	lower	5	162 (17,1)	120 (14,0)	2 590 (44,2)
The numbers in brackets are the coefficient of variation in per cent					

The student's t-test, at a 1 per cent level of significance, showed the difference in bending strength between the upper and lower halves to be significant for both the contracts. The differences in the strain at break were not significant. The elastic moduli for contract 2(S20) were significantly different at a 1 per cent level but on contract 2(S21) the difference was only significant at a 5 per cent level of significance.

From this study it may be concluded that on both contracts, although they were constructed on the same day, the upper half had both a higher bending strength and a higher elastic modulus than the lower half of the cement-treated layer.

6.5.4 Variation between field- and laboratory-prepared materials

Practical problems when sawing beams from the recovered blocks or during the laboratory preparation of the samples, resulted in only 4 contracts yielding information that can be utilized to compare field- and laboratory-prepared materials. This information is summarized in Table 6.9.

The table indicates that the properties of the field samples are generally lower than those of the laboratory samples. This does not, however, apply to contract 5, but this is probably due to the low density achieved in the laboratory. If generally higher densities could have been obtained in the laboratory, the differences between the field and laboratory samples may have been greater.

TABLE 6.9 : Variation in field- and laboratory-prepared materials

CONTRACT	FIELD-PREPARED			LABORATORY-PREPARED		
	Bending strength (kPa)	Strain at break (με)	Elastic modulus (MPa)	Bending strength (kPa)	Strain at break (με)	Elastic modulus (MPa)
2	Results were reported in Tables 6.6 and 6.7			817(13,0) 862(15,1) 753(22,0)	111(11,2) 106(7,1) 116(23,2)	10 625(10,0) 11 475(3,6) 9 417(3,4)
3	502(13,2) 345(7,1)	124(16,4) 92(14,5)	5 900(4,5) 5 460(2,1)	443(21,1)	97(11,0)	7 080(25,0)
5	221(49,5)	122(22,2)	2 750(19,1)	73(48,0)	86(28,0)	1 535(13,4)
6	Too many large stones in block; could not be sawn.			481(5,0)	101(13,2)	9 380(23,1)
7	464(17,3) 362(17,1) 385(31,3)	237(17,6) 231(27,4) 249(22,1)	2 825(9,6) 2 550(17,1) 2 608(12,5)	462(23,3)	138(25,0)	4 422(21,0)
8	Block too weak to resist the action of the blade when beams were cut			619(18,3)	189(16,0)	4 586(8,0)
9	129(35,0) 192(37,3) 173(36,5)	172(27,3) 196(20,1) 225(29,1)	1 810(41,0) 2 570(28,5) 2 190(25,0)	777(9,1)	161(7,6)	7 149(6,4)

The numbers in brackets are the coefficient of variation in per cent

Comparing the results from contracts 7 and 8 makes interesting reading. The specified and required strength for contract 7 was 1 500 kPa while contract 8 was treated only to reduce the plasticity. This difference was borne out by the field samples because the quality of the material from contract 7 was better than that from contract 8; for example it could at least withstand the action of the saw. Statistically speaking the quality of the laboratory-prepared samples from the two contracts was the same. This implies that the material on contract 8 could have been prepared to obtain a better-quality treated material in the field. The economics of such an improvement depend on the particular site, but it is believed that it would have been economical to utilize the full potential of the treated material.

6.5.5 Variation in compressive strength

After performing the bending test, 150 mm long samples were sawn from the ends of the beams and tested in compression, although this was not done for all the field samples. The compressive strengths and the ratio between these are given in Table 6.10. The results indicate a significant difference between the field and laboratory samples, but gives no clear indication of which method produced the highest compressive strength.

TABLE 6.10 : Variation in compressive strength of field- and laboratory-prepared materials

CONTRACT	FIELD SAMPLES (kPa)	LABORATORY SAMPLES (kPa)	RATIO
2	4 290(33,2) 6 367(22,0) 2 128(63,0)	2 443(6,7) 2 340(10,2) 4 050(13,1)	1,75 2,72 0,52
3	1 210(21,4)	1 830(9,1)	0,66
4	-	432(16,0)	-
5	-	488(28,2)	-
6	-	1 353(14,2)	-
7	2 303(28,3)	2 195(27,2)	1,05
8	282(28,4)	2 286(12,1)	0,12
9	917(42,1)	2 203(9,2)	0,42
10	559(32,1)	293(102,5)	1,91
Numbers in brackets are coefficient of variation in per cent			

6.6 DISCUSSION

The samples recovered from the various contracts indicated very little variation across a section constructed on a particular day (Table 6.6), but significant variations occurred across sections constructed on different days, that is at different occasions (Table 6.7). This implies that a specific section, constructed in one operation, may be considered homogeneous. However, different sections of a contract which were constructed on different days, may not be regarded as the same and the completed contract may therefore not be regarded as homogeneous. Neither may a layer be accepted as homogeneous in the vertical direction, since the upper part seems to have better material properties than the lower part.

The study of the difference between field- and laboratory-prepared samples generally indicated better material properties in the laboratory-prepared samples. This was to be expected since they were prepared under ideal conditions. The lower values for the field samples indicate that because of the construction technique, the full potential of the materials is not being utilized. Some research and development on construction techniques and procedures may therefore prove worthwhile.

The difference in material properties between the field- and laboratory-prepared materials, is an even more important consideration when the future application and implementation of a structural pavement design procedure based on layered elastic theory is considered. In the past an unconfined compressive strength has been specified as a materials requirement and the construction controls were a method specification together with a check on the specified compressive strength and density. If the contractor complied with these specifications the materials fitted into the original design definitions and the structural pavement design aspect virtually took care of itself, since it was an empirically developed procedure based on successful previous applications of this structural layout with this type of material. When applying the principle of balanced pavement design and layered elastic theory, it is important to know exactly the properties of the materials in the structural layout, and therefore to know to what extent the contractor can produce the quality of the laboratory-prepared samples. To utilize the materials and structural layout fully, the designer must know the amount of reduction he should allow for the difference between the laboratory and field values - this is the factor of safety in material properties. Not enough information is available in Table 6.9 to indicate the amount of reduction that should be allowed and more study on many more contracts will be required to obtain these numbers. In the meantime it seems that the field bending strength is somewhere between 20 and 150 per cent of the laboratory bending strength. The corresponding numbers for the strain at break and elastic modulus are between 63 and 180 and somewhere between 25 and 150 per cent respectively.

6.7 CONCLUSIONS

(a) Variation in material properties on a project as a result of the construction process is real and very significant. Pavement designers should take cognizance of this and somehow include it during the design stages.

(b) From the limited number of samples taken on a contract and the limited number of contracts suitable for this study it appears that:

(i) The variation in material properties in a section constructed during a particular day is not significant and for pavement design purposes the section may be regarded as homogeneous.

(ii) The variation in material properties in sections constructed on different occasions or days is significant and these sections may not be considered the same, even when the same materials, contractor and specification apply to all the sections.

(iii) The material properties are not constant throughout the depth of the layer and the upper half seems to have higher values than the lower half.

(iv) The properties of materials constructed on a road by a contractor are significantly poorer than the values obtained on similar materials prepared in a laboratory. From this study it is not possible to indicate the degree of this difference.

(c) To try to predict the future behaviour of a pavement from cement-treated material properties obtained in a laboratory test, appears to be misleading. The extent of the difference between the design properties and the properties of the material produced by the contractor is unknown and it seems to vary from contractor to contractor. Neither is the difference constant during the construction period since it varies from day to day. Until these differences have been studied and quantified accurately, for example by tightening up the specification on the standard deviation of materials quality, it seems a very difficult task to predict accurately the long-term behaviour of a pavement containing cement-treated materials.

6.8 RECOMMENDATION

Until more specific recommendations become available for practical pavement design work, it is recommended that the values of the properties of field-prepared cement-treated materials be taken as 70 per cent of those of laboratory-prepared materials. A 30 per cent reduction in the laboratory values is thus recommended.

CHAPTER 7

HEAVY VEHICLE SIMULATOR TESTS ON CEMENT-TREATED BASES

PAGE

7.1 INTRODUCTION

The first Heavy Vehicle Simulator (HVS)(Van Vuuren, 1972a and 1973) (Figure 7.1) was commissioned during early 1971. Since then it has been used to simulate the action of heavy vehicles on various types of pavement, which included untreated crusher-run bases, upside-down designs and cement-treated bases. To date no HVS tests have been performed on cement-treated natural gravel bases or on any lime-treated materials. A total of 24 HVS tests was completed in the 5 years between March 1971 and March 1976.

This chapter describes the results of the HVS tests which were performed on pavements with thin bituminous surfacings and cement-treated crusher-run bases. It also attempts to show the progress that was made in designing and interpreting the results from the various HVS tests.

7.2 TEST PROCEDURE

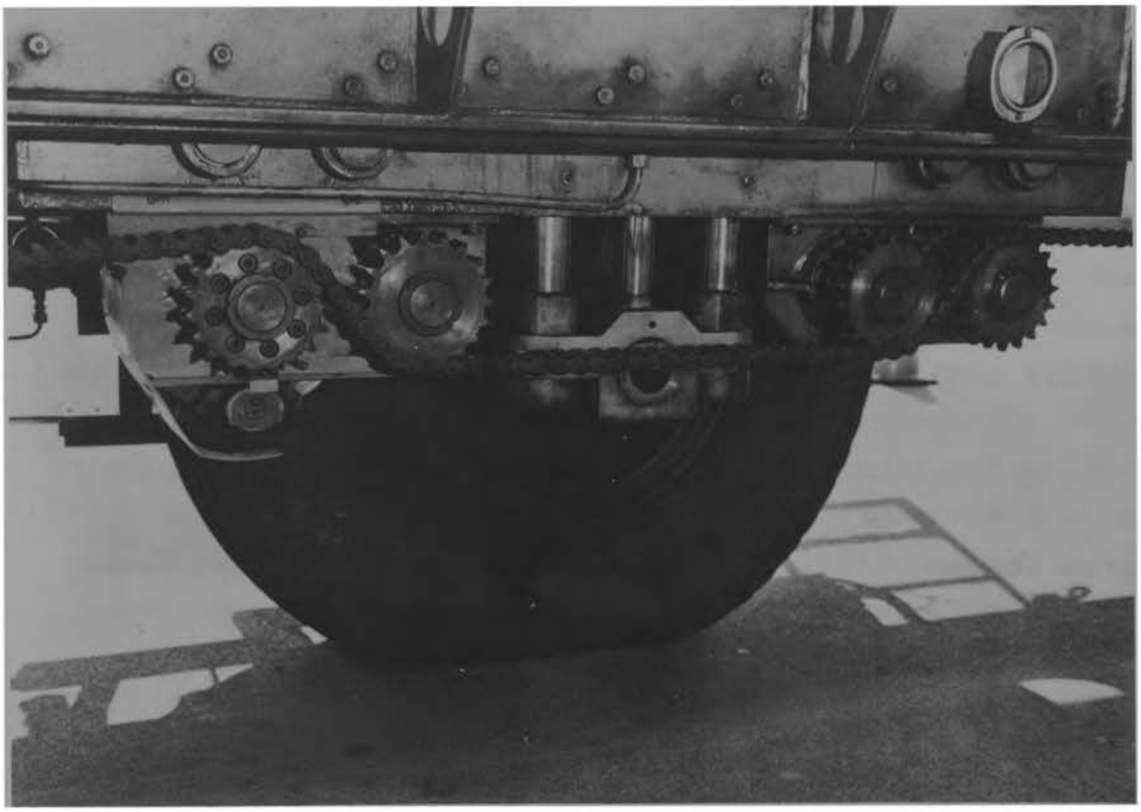
The same HVS was used on all the test sections described in this chapter, but the loading and tyre contact pressure of the single test wheel were varied. This information is summarized in Table 7.1. The same tyre was not used on all the sections, but the same size, namely 1100x20x14 ply, was used throughout.

TABLE 7.1 : Wheel load and tyre contact pressure for the various test sections

TEST SECTION	WHEEL LOAD (kN)	TYRE CONTACT PRESSURE (kPa)
Base 1 on S12 (January 1972)	65	750
Base 2 on S12 (November 1972)	40	620
Johannesburg Eastern Bypass	75	790
Eerste Fabrieke (January 1975)	55	710
Kloof-Stanhope (March 1972)	50	680
Base 1 on S12 (November 1973)	65	750
Base 2 on S12 (January 1974)	65	750
Eerste Fabrieke (August 1975)	55	710

The HVS covered an area about 1 m wide and 6 m long, which was divided into five evenly spaced major test points, 1 m apart and called A, B, C, D and E. The test section was 5 m long and the five test points were in the centre of the 1 m wide strip. At regular intervals the HVS trafficking was interrupted to perform Benkelman beam and Dehlen curvature meter

Overall view



Chain drive on test wheel

FIGURE 7.1
The heavy vehicle simulator.

(Dehlen, 1962) tests under a standard 40 kN dual wheel load. At the same time the transverse vertical deformation under a 2 m straightedge was measured on each of the 5 transverse lines. The maximum deformation on each of these transverse lines was obtained and the average maximum deformation for the site was calculated from the five maxima.

In HVS work it is considered that one repetition has been applied when the loaded test wheel has travelled longitudinally over the 6 m long test section. The test wheel also creeps transversely over the section in increments which can be varied between 200 and 100 mm, and it therefore requires about 10 to 20 repetitions to do one full coverage of the test section. Since the section is only 1 m wide, a considerable amount of stress overlap may occur in certain pavements, but the extent thereof is probably a function of the structural layout and depth. The relationship between the number of load repetitions by the HVS and the number of equivalent stress repetitions at a particular point is therefore unknown and it requires further study in future. At present this unknown relationship should be kept in mind when interpreting and evaluating HVS test results.

7.3 RESULTS

7.3.1 Base 1 of Cloverdene-Argent experiment on Route S12 (11 to 27 January 1972)

This test was actually performed on a section of the Cloverdene-Atlas contract which has the same structural design as Base 1, namely 25 mm asphaltic concrete surfacing, 100 mm cement-treated crusher-run base, 100 mm untreated crusher-run and 100 mm cement-treated natural gravel subbase. The reason for choosing a site on an adjoining contract was that it was not possible to obtain a crack-free section on Base 1 and at that stage (1972) it was considered very important to have a crack-free section. The averages of the measurements taken during the test are reproduced as Table 7.2.

In this test the various layers beneath the cement-treated layers were considered to be equivalent to one homogeneous layer and the elastic properties were obtained from Benkelman beam tests and layered elastic theory analyses done with the CHEVRON computer program. It was considered that the equivalent elastic modulus of the foundation remained the same throughout the test and that changes in the surface deflection were due only to changes in the elastic modulus of the cement-treated layer.

From the initial Benkelman beam deflection (327 μm), the elastic moduli of the 25 mm surfacing (5 000 MPa) and 100 mm cement-treated base (5 000 MPa) and a constant Poisson ratio of 0,35, the elastic modulus of

TABLE 7.2 : Measurements recorded during HVS test - Base 1 on S12

REPETITIONS	DEFLECTION (μm)	RADIUS OF CURVATURE (m)	MAXIMUM DEFORMATION (mm)	RAINFALL (mm)
0	327(9)	187(25)	0	0
100	336(7)	201(20)	0,3	0
200	343(8)	193(9)	-	0
300	324(10)	179(14)	0,7	0
1 500	395(14)	126(11)	1,5	0
2 800	406(3)	125(11)	1,8	0
4 000	405(3)	123(12)	2,3	15
7 000	440(2)	122(11)	2,0	9,5
10 000	442(4)	117(11)	1,9	0
20 000	478(5)	106(9)	2,2	0
35 000	454(6)	108(22)	2,4	93
50 000	504(3)	100(13)	2,8	14
65 000	502(4)	88(6)	3,1	1,5
80 000	550(6)	79(8)	3,2	5,8
100 000	506(5)	91(11)	3,2	0
115 000	604(5)	79(8)	3,4	3,5
The numbers in brackets are the coefficient of variation in per cent				

the foundation was calculated as 180 MPa (Otte, 1972b). It was also calculated that the tensile strain in the cement-treated base under a 65 kN wheel load would be about 300 με which is very high but is caused by the relatively low elastic modulus of the cement-treated material (Figure 2.4, page 15). Since the developed strain (300 με) would exceed the strain at break of the cement-treated material, expected to be only about 160 με, traffic-associated cracking should occur very quickly. At that stage (1972) it was decided to apply more repetitions than theoretically required because it was necessary to fracture the cement-treated crusher-run and revert it back to an equivalent untreated crusher-run. After 80 000 repetitions a block sample was removed (Otte, 1972b) and the cement-treated crusher-run was completely broken into little pieces of about 100 x 100 mm. This test, which was the first one with the HVS, indicated that pavement distress can be predicted and that the HVS can be used to simulate traffic on pavements.

7.3.2 Base 2 of Cloverdene-Argent experiment on Route S12 (20 to 27 November 1972)

The previous HVS test indicated that the machine can be used to study pavement behaviour. The purpose of this test was to expand on the previous one and to try and apply layered elastic theory to a section that would not crack and would stand up to traffic loadings.

The procedure was about the same as that previously mentioned and the averages of the measurements taken during the test are reproduced as Table 7.3. Note the increase in the coefficients of variation as compared with the previous test. A block sample of the 150 mm thick cement-treated crusher-run was removed from a point adjoining but outside the test area and the elastic properties were determined (elastic modulus = 20 500 MPa, strain at break (ε_b)=162 $\mu\varepsilon$). It was assumed that this represented the material at the HVS test site. The surface deflection at test point A was about 231 μm. After assuming the elastic modulus of the 25 mm surfacing to be 3 000 MPa and all the Poisson ratios to be 0,35, layered elastic theory analyses were performed to obtain the equivalent elastic modulus of the lower layers which would produce the same surface deflection. The modulus turned out to be 120 MPa. The corresponding strain (ε) in the cement-treated crusher-run under a 40 kN wheel load was only about 57 $\mu\varepsilon$. The strain ratio ($\varepsilon/\varepsilon_b$) was about 0,35 and according to theory (equation (2.5), page 31) this material would be able to withstand about 800 000 repetitions before fatigue cracking was likely to occur.

TABLE 7.3 : Measurements recorded during HVS test - Base 1 on S12

REPETITIONS	DEFLECTION (μm)	RADIUS OF CURVATURE (m)	RAINFALL (mm)
0	216(43)	1 062(28)	0
32	220(30)	605(33)	0
94	234(22)	656(38)	0
200	262(29)	615(20)	0
400	244(27)	625(17)	0
800	249(25)	645(28)	0
1 600	240(30)	597(32)	0
3 000	217(35)	554(31)	0
6 000	199(53)	718(19)	0
9 000	210(34)	712(16)	0
12 000	199(48)	587(29)	0
The numbers in brackets are the coefficient of variation in per cent			

The load was applied and after 12 000 repetitions a block sample was removed from the trafficked area at test point A and its elastic properties were determined (elastic modulus = 18 000 MPa, strain at break (ε_b) = 178 $\mu\varepsilon$). When the elastic properties before and after the test were compared, it was clear that the test section, as represented by the one sample, was unaffected by the 12 000 repetitions of the 40 kN wheel load.

Unfortunately major repairs had to be done to the HVS and the test was stopped. It was the intention to recover 2 more blocks during the test from which it would have been possible to produce a plot of the change in elastic modulus versus number of repetitions. It was hoped that such a plot would indicate the point of failure and that this could have been compared with the theoretical failure point predicted by the design procedure proposed in Chapter 8.

7.3.3 Johannesburg Eastern Bypass (21 November to 20 December 1974)

The test was performed on the section of the road with a 150 mm cement-treated crusher-run base and 150 mm cement-treated natural gravel subbase. The surfacing consisted of 30 mm asphaltic concrete and a 25 mm BS 594-type bituminous overlay. Severe initial cracking did reflect through the 30 mm asphaltic concrete, but it was largely damped by the overlay at the time when the HVS test was performed.

The purpose of the test was still to try and correlate theory and practice. By doing an analysis along the lines described in section 7.3.2 the elastic modulus of the lower layers (foundation) was calculated as 175 MPa. Assuming the elastic modulus of the 55 mm bituminous surfacing as 3 000 MPa and measuring that of the cement-treated crusher-run as 28 000 MPa, the strain in the cement-treated layer (ε) under a 75 kN wheel load was calculated to be about 65 $\mu\varepsilon$. The strain at break (ε_b) of the material was about 160 $\mu\varepsilon$ and according to equation (2.5) (page 31) the section should withstand about 255 000 repetitions, that is about 4 million equivalent 80 kN axles, before fatigue cracking could be expected.

It was the intention to follow the procedure which was described in section 7.3.2 for Base 2, namely to recover blocks after a certain number of predetermined repetitions, and to follow the change in elastic modulus of the cement-treated base. To recover the sample and adequately refill the hole, was considered to be too time-consuming. It was then decided to apply the predetermined number of load repetitions, advance the machine 3 metres and recover the samples from the section that would not be tested again. This meant that the middle 3 m long section was part of both trafficking spells and it was possible to obtain three 3 m long sections, which had had different traffic loadings, in a distance of only about 9 to 10 metres. The 10 test points (A to J) were 1 m apart.

The deflection and radius of curvature measurements recorded at each test point during the test are reproduced in Table 7.4. Note that points A, C and E were trafficked until about 187 000 repetitions. Then the machine was advanced 3 m and the trafficking was continued on points E, G and I for another 93 000 repetitions. This resulted in point E, which was part of both tests, having to sustain about 280 000 repetitions.

TABLE 7.4 : Measurements recorded during HVS test - Johannesburg Eastern Bypass

REPETITIONS	DEFLECTION (μm) AT POINTS:			RADIUS OF CURVATURE (m) AT POINTS:			MAX. DEFORMATION (mm) AT POINTS: A, B, C, D, E, F	RAINFALL (mm)
	A	C	E	A	C	E		
0	148	145	158	1 199	1 199	1 199	0	50
400	198	178	176	-	-	-	-	6,5
1 000	153	153	160	1 023	1 023	1 258	0,14	3,0
4 000	159	135	148	1 076	1 195	1 084	0,43	0
10 000	143	150	159	1 097	1 337	1 076	0,86	0
30 000	175	161	148	1 613	1 306	930	1,43	22
100 000	203	193	216	674	686	651	2,43	53
140 000	199	210	246	703	735	570	3,43	0
174 000	238	232	267	619	587	596	4,0	0
186 877	229	225	-	563	700	609	4,71	0
190 877	-	-	249	-	-	584	D E F only	0
196 877	-	-	227	-	-	533	5,78	0
235 565	-	-	227	-	-	584	5,28	0
270 020	-	-	208	-	-	728	5,95	0
279 650	-	-	237	-	-	669	4,68	0

REPETITIONS	DEFLECTION (μm) AT POINTS:		RADIUS OF CURVATURE (m) AT POINTS		MAX. DEFORMATION (mm) AT POINTS: G, H, I, J	RAINFALL (mm)
	G	I	G	I		
0	158	156	1 106	1 219	0	0
400	150	152	899	1 084	0	0
1 000	182	175	1 113	1 136	0,5	5
4 000	205	197	957	964	0,25	7
10 000	194	188	784	766	1,0	11
48 688	216	180	726	723	1,8	0
83 143	173	200	766	980	2,5	0
92 773	205	194	751	997	2,0	0

It was assumed that traffic-associated fatigue cracking had occurred in the cement-treated base if the elastic properties showed a marked change after trafficking. Table 7.5 shows the changes that took place after the quoted number of repetitions. Samples 1 and 2 were recovered from areas outside of the testing area and may be assumed to be representative of the intact material while samples 3 and 4 were removed from the trafficked area at points I and E respectively. The table shows no statistically significant change in elastic modulus and bending strength up to 93 000 repetitions, but a marked change after 280 000 repetitions of the

75 kN wheel load. The strain at break shows no significant change.

TABLE 7.5 : Material properties before and after trafficking with the HVS - Johannesburg Eastern Bypass

SAMPLE NO.	REPETITIONS	ELASTIC MODULUS (MPa)	STRAIN AT BREAK ($\mu\varepsilon$)	BENDING STRENGTH (kPa)
1	- *	25 717(17)	157(18)	2 865(26)
2	- *	29 900(26)	166(19)	2 695(10)
3	93 000	27 100(16)	153(18)	3 040(29)
4	280 000	13 300(28)	163(19)	1 510(26)

The numbers in brackets are the coefficients of variation in per cent.

*These samples were not trafficked since they were taken outside of the test area.

Assuming the AASHO load equivalency factors to hold for this design, and since no visible cracking developed during the test, it may be concluded that this section of the Johannesburg Eastern Bypass can be counted on to withstand about 4 to 5 million equivalent 80 kN axles before any visible traffic-associated distress will develop.

7.3.4 Eerste Fabrieke on Route N4/1 (Pretoria to Bronkhorstspruit) (January to May 1975)

Three HVS tests were performed within a distance of about 60 m on this portion of Route N4/1 because a larger statistical sample on one pavement was desired. The structural layout was the same as Base 1 on Route S12, namely 25 mm asphaltic concrete surfacing, 100 mm cement-treated crusher-run base, 100 mm untreated crusher-run, 100 mm cement-treated natural gravel subbase and the necessary selected and fill layers. The applied wheel load was 55 kN and the three tests were stopped after 243 000, 237 000 and 290 320 repetitions respectively. Tests 1 and 2 were performed during the rainy season and pumping from the lower layers through the cracks in the surface started after about 188 000 repetitions on site 1 and after about 54 000 repetitions on site 2. On both of these sections pumping lasted for the entire duration of the test and it appeared that the amount of pumping increased after each shower of rain. Test 3 was performed during a relatively dry period and no pumping occurred.

Tables 7.6, 7.7 and 7.8 contain the Benkelman beam, Dehlen curvature meter, average maximum deformation and rainfall data accumulated during the three tests. Figures 7.2 and 7.3 contain the average deflection and radius of curvature measurements for the five test points on each of the three test sections, as well as the average for the three sites. Figure 7.4 contains the average maximum deformation measured on each of the three sites.

TABLE 7.6 : Measurements recorded during HVS test - Eerste Fabrieke (Site 1)

REPETITIONS	DEFLECTION (µm)						COEFFICIENT OF VARIATION (%)	AVE.MAX DEFORMATION (mm)	RAINFALL (mm)
	A	B	C	D	E	Average			
0	156	198	175	225	223	195	15	0	-
40	167	223	177	219	204	198	13	0,2	-
100	135	217	152	183	215	180	20	1,0	-
400	202	273	210	260	246	238	13	1,4	5
1 000	175	260	200	267	254	231	18	1,6	-
4 000	204	342	267	358	388	312	24	1,2	6
8 000	225	371	288	400	515	360	31	2,6	-
25 000	250	454	356	502	744	461	40	2,8	-
61 000	321	683	542	558	825	585	32	4,4	84
98 000	350	554	477	763	1 150	659	47	4,8	-
133 500	288	560	479	492	796	523	35	5,8	-
188 000	296	675	583	371	600	505	32	7,6	125
243 000	363	813	483	402	740	560	36	11,2	52
	RADIUS OF CURVATURE (m)								
0	1 524	1 136	1 043	867	803	1 075	26		
40	1 097	1 371	831	1 136	890	1 065	20		
100	1 043	1 238	824	1 267	952	1 065	18		
400	867	1 063	633	1 567	784	983	37		
1 000	890	1 136	609	1 177	890	940	24		
4 000	910	1 055	484	1 567	540	911	48		
8 000	803	784	406	1 647	406	809	63		
25 000	766	890	255	549	279	548	52		
61 000	890	140	119	107	207	292	115		
98 000	568	246	157	83	120	235	83		
133 500	507	227	92	104	95	205	87		
188 000	515	383	109	130	161	260	69		
243 000	588	203	92	162	40	217	100		

- means no rainfall during that period.

7.3.4.1 Behaviour of the test sections during HVS testing

Pumping: From the recorded observations it is not possible to determine whether the pumping had any influence on the test sections, that is whether it accelerated the increase in deflection and decrease in radius of

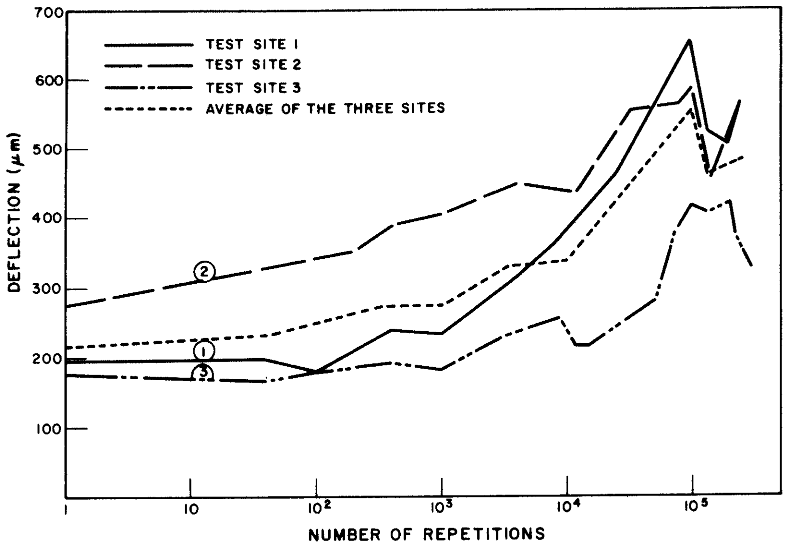


FIGURE 7-2

THE INCREASE IN AVERAGE DEFLECTION ON THREE TEST SITES

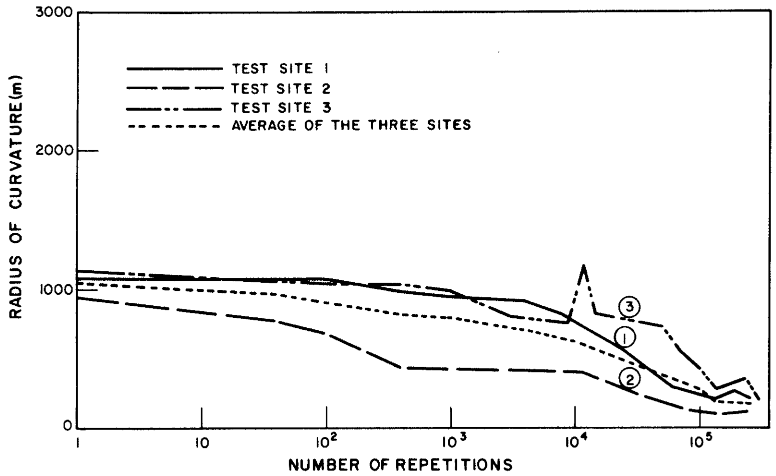


FIGURE 7-3

THE DECREASE IN AVERAGE RADIUS OF CURVATURE ON THREE TEST SITES

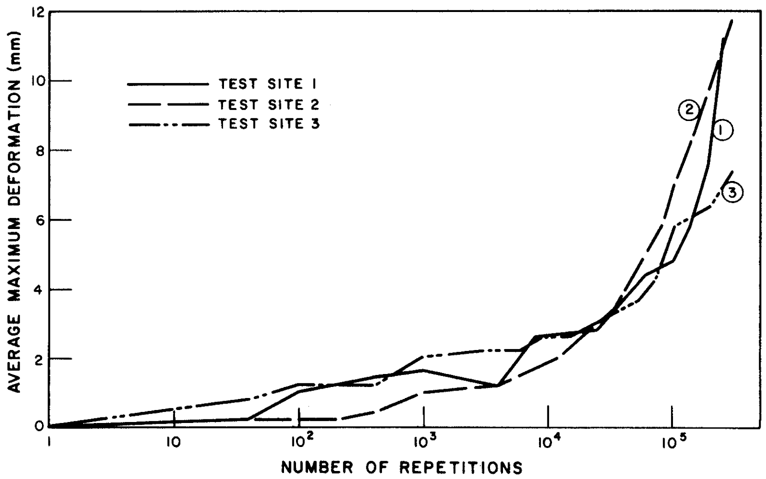


FIGURE 7-4

THE INCREASE IN AVERAGE MAXIMUM DEFORMATION ON THREE TEST SITES

TABLE 7.7 : Measurements recorded during HVS test - Eerste Fabrieke (Site 2)

REPETITIONS	DEFLECTION (μm)						COEFFICIENT OF VARIATION (%)	AVE.MAX. DEFORMATION (mm)	RAINFALL (mm)
	A	B	C	D	E	Average			
0	265	285	323	258	233	273	12	0	-
40	342	317	388	321	267	327	13	0,2	10
200	363	371	396	329	296	351	11	0,2	-
400	433	379	438	383	304	387	14	0,4	-
1 000	475	375	450	363	350	403	14	1,0	-
4 000	492	425	567	425	333	448	19	1,2	15
12 000	471	388	567	408	342	435	20	2,0	-
34 000	513	392	642	542	679	554	20	3,4	8
80 000	517	446	629	675	554	564	16	5,8	16
100 000	513	504	704	675	541	587	16	7,0	12
140 000	513	438	413	438	471	455	8	8,4	32
237 000	579	425	625	721	479	566	21	11,8	30
	RADIUS OF CURVATURE (m)								
0	1 029	1 062	587	1 029	1 029	947	21		
40	844	672	609	867	914	781	17		
200	540	731	499	969	701	688	27		
400	402	522	268	470	506	434	24		
1 000	323	506	332	568	451	436	25		
4 000	296	522	191	558	464	406	39		
12 000	189	313	165	383	343	279	35		
34 000	185	184	132	392	323	243	45		
80 000	134	117	68	181	174	135	34		
100 000	112	120	65	105	175	115	34		
140 000	113	100	73	52	161	100	42		
237 000	160	109	62	63	214	122	54		

- means no rainfall during that period.

curvature. The reason for this rather inconclusive statement is because there were no statistically significant changes, at the 5 per cent level of significance, in the recorded deflection and radius of curvature between about 100 000 and 250 000 repetitions. On site 2 there was not even a significant change in deflection between 4 000 and 237 000 repetitions.

From Figures 7.2 and 7.3 it appears that the changes in deflection and radius of curvature on site 3 were smaller than those on sites 1 and 2. It must, however, be remembered that on site 3 the initial deflection was lower and the initial radius of curvature was higher than on the other two. This, and the scatter in the measurements (see Tables 7.6 to 7.8), make it rather difficult to draw a conclusion about the effect of pumping on the behaviour of the sections.

TABLE 7.8 : Measurements recorded during HVS test - Eerste Fabrieke (Site 3)

REPETITIONS	DEFLECTION (µm) A	B	C	D	E	Average	COEFFICIENT OF VARIATION (%)	AVE.MAX. DEFORMATION (mm)	RAINFALL (mm)
0	142	229	188	163	146	174	21	0	32
40	175	217	163	138	142	167	19	0,6	-
100	154	250	171	154	167	179	22	1,2	-
400	179	267	179	150	179	191	23	1,2	-
1 000	146	263	188	142	171	182	27	2,0	-
3 000	200	363	229	175	163	226	36	2,2	-
6 000	213	388	217	200	204	244	33	2,2	-
9 000	213	433	229	188	200	253	40	2,6	-
12 000	163	358	213	150	196	216	39	2,6	-
15 000	171	400	209	146	150	215	49	2,6	-
50 000	227	458	265	221	225	279	36	3,6	-
74 000	304	454	354	392	383	377	15	4,2	32
98 000	329	420	338	535	460	416	21	5,8	-
131 526	342	438	380	442	425	405	11	5,8	-
194 558	304	417	429	446	502	420	17	6,4	-
220 000	292	438	358	396	388	374	14	*	-
290 320	240	457	328	322	272	324	26	7,4	-
	RADIUS OF CURVATURE (m)								
0	1 567	160	499	1 828	1 647	1 140	66		
40	1 499	153	433	1 828	1 385	1 060	68		
100	1 647	160	332	1 647	1 432	1 044	71		
400	1 267	120	299	1 938	1 524	1 030	77		
1 000	1 288	134	237	2 194	1 113	993	85		
3 000	1 029	127	208	1 828	803	799	87		
6 000	979	111	207	1 828	664	758	92		
9 000	914	98	165	1 599	979	751	83		
12 000	1 097	114	135	4 124	338	1 162	147		
15 000	1 010	91	197	1 938	844	816	91		
50 000	1 113	84	96	1 647	700	728	92		
74 000	447	113	91	1 499	464	523	110		
98 000	700	108	74	803	470	431	77		
131 526	274	103	146	470	406	280	57		
194 558	538	80	102	457	568	349	69		
220 000	167	104	165	844	558	368	88		
290 320	207	140	140	280	513	216	37		

- means no rainfall during that period.

* means no measurement taken.

It appears that although pumping occurred and although a large volume of fine material was pumped out, Benkelman beam deflections and radius of curvature measurements were not sensitive enough to pick up the changes, if any, caused by pumping during the HVS tests.

From Figure 7.4 it also appears as if the pumping had no effect on the recorded average maximum deformation because it is not possible to observe any meaningful change in the deformation pattern after the onset of pumping. The deformations increased continuously throughout the three tests and small final deformations (about 10 mm) were recorded.

Cracking: There were only a few visible cracks on the surface after the completion of trafficking on the three sites. This may be because of the concentrated traffic and some oil and diesel fuel spillage resulted in them being closed. The 25 mm surfacing was removed and the visible cracks in the base were marked with white chalk lines. There were several cracks in the base and these can be seen in Figure 7.5. On site 1 point A was crack-free, the area around point B was cracked into 100 mm square blocks and another major crack occurred between points D and E. Severe pumping occurred during the test, and the circle on the right-hand side of the picture, at point E, shows an area that pumped extensively. All the test points on site 2 had a fair amount of cracking and loose stones were found on the surface of the cement-treated crusher-run base. This could have been caused by the trafficking or they could be rough areas which came about during construction when the stones on the surface of the base were not properly compacted and cemented. Site 3 had a rough area with loose stones around point B, some cracking at points C and D but very little visible cracking at points A and E.

7.3.4.2 The structural analyses and the behaviour of the test sections

Some of the Benkelman beam and radius of curvature measurements recorded on the three sites can be used to assist in the correlation of a pavement's predicted and actual behaviour, that is to correlate theory and practice. The actual pavement consisted of at least five different layers and materials, but to handle it analytically with the limited amount of available information, namely only surface deflection and radius of curvature, it had to be simplified into a three-layered structure. The chosen three-layered structure consisted of a 25 mm bituminous surfacing (elastic modulus 3 000 MPa), an equivalent cement-treated base (100 mm) and the foundation.

Site 1

Site 2

Site 3

FIGURE 7.5

Condition of test sites after asphalt surfacings were removed. Cracks in base were marked with white chalk lines.

In Figure 7.6 the fine lines show the calculated theoretical relationship between the radius of curvature and surface deflection for the equivalent three-layered structure, with different moduli for the two lower layers. The load was taken as two 20 kN loads (contact pressure 520 kPa) 343 mm apart. The surface deflection was calculated at the mid-point of the line joining the centres of the two loaded areas - this corresponds to the Benkelman beam test condition. The radius of curvature was calculated from the difference between the deflection at this point and that at a point 127 mm away on the longitudinal axis - this corresponds to the test condition for the Dehlen curvature meter (Dehlen, 1962).

The deflections and radii of curvature recorded at test points A and B of site 1 are also shown on Figure 7.6. The measured information for point A indicates that the elastic modulus of the cement-treated crusher-run base (E_2) was probably reduced during the first 100 repetitions, thereafter it remained fairly constant and at the end of the test, that is after 243 000 repetitions, it was around 20 000 MPa. This agrees perfectly with the observed behaviour of point A. No cracking was visible at the end of the test (Figure 7.5) and a block sample (600 x 600 mm) of the cement-treated base (E_2) removed at point A at the end of the test had an elastic modulus of 19 260 MPa! (Average of 5 samples; standard deviation of 5 590 MPa.)

Point B, which was only one metre away from point A, behaved significantly differently. When the measurements taken at point B are compared with the theoretical lines on Figure 7.6, it appears that the elastic modulus of both the equivalent cement-treated base (E_2) and the foundation (E_3) reduced during the first 25 000 repetitions. The elastic modulus of the foundation (E_3) was reduced from about 150 MPa to about 50 MPa. After 25 000 repetitions the elastic modulus of the foundation (E_3) remained fairly constant, but the elastic modulus of the equivalent cement-treated base (E_2) was significantly reduced to somewhere around 2 000 to 6 000 MPa. The significant reduction in elastic modulus of the equivalent cement-treated base (E_2) was confirmed by the practical condition of the cement-treated crusher-run base at the end of the test - it was cracked into 100 x 100 mm pieces and no block sample could be sawn from the pavement at this point.

The behaviour of test points C, D and E are shown in Figure 7.7. On all three the elastic moduli of both the equivalent cement-treated crusher-run layer (E_2) and the foundation (E_3) were reduced during the test. No attempt was made to saw blocks from any of these points after completion of the test.

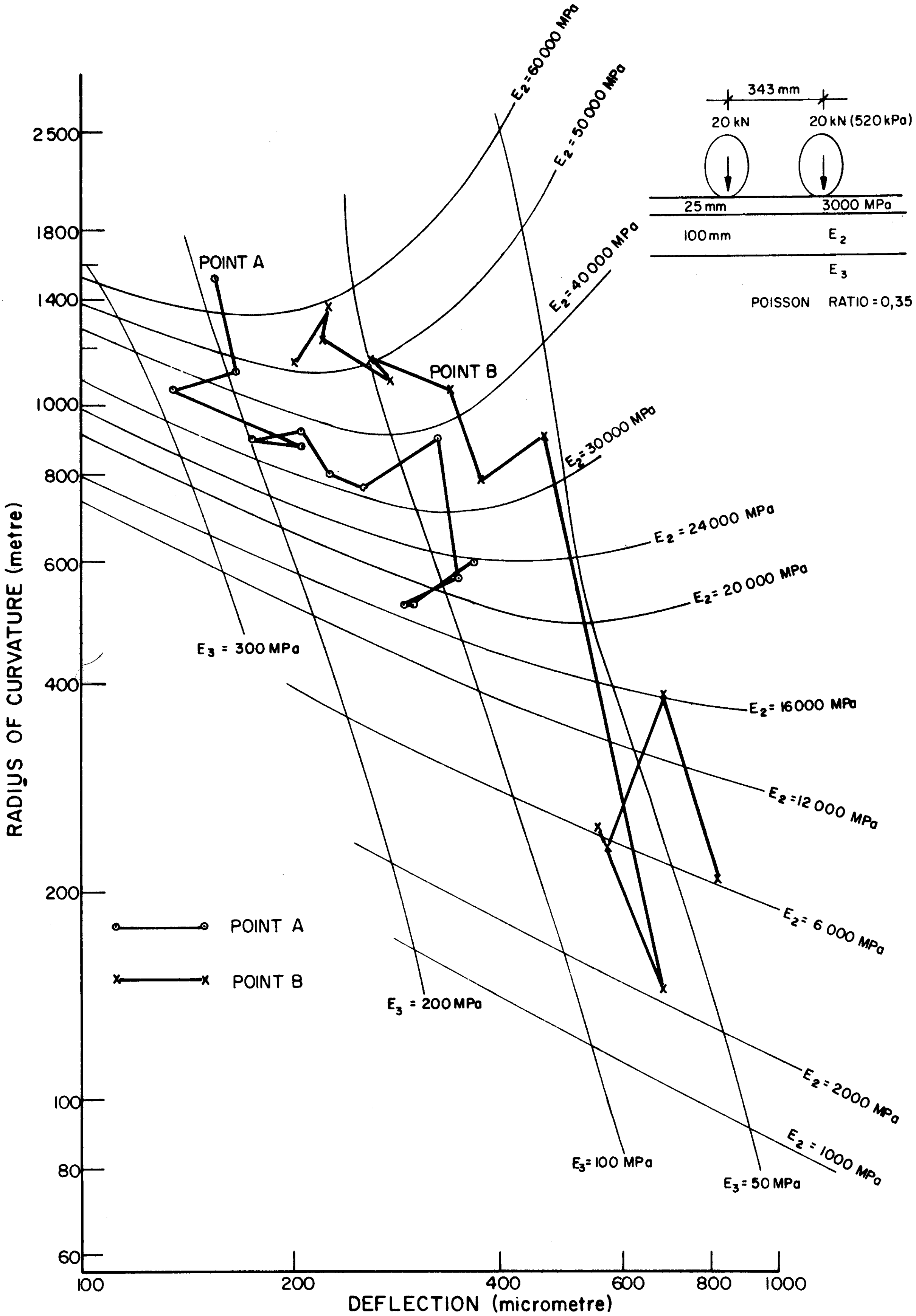


FIGURE 7-6

RELATIONSHIP BETWEEN RADIUS OF CURVATURE AND DEFLECTION FOR A THREE-LAYERED PAVEMENT (SITE 1)

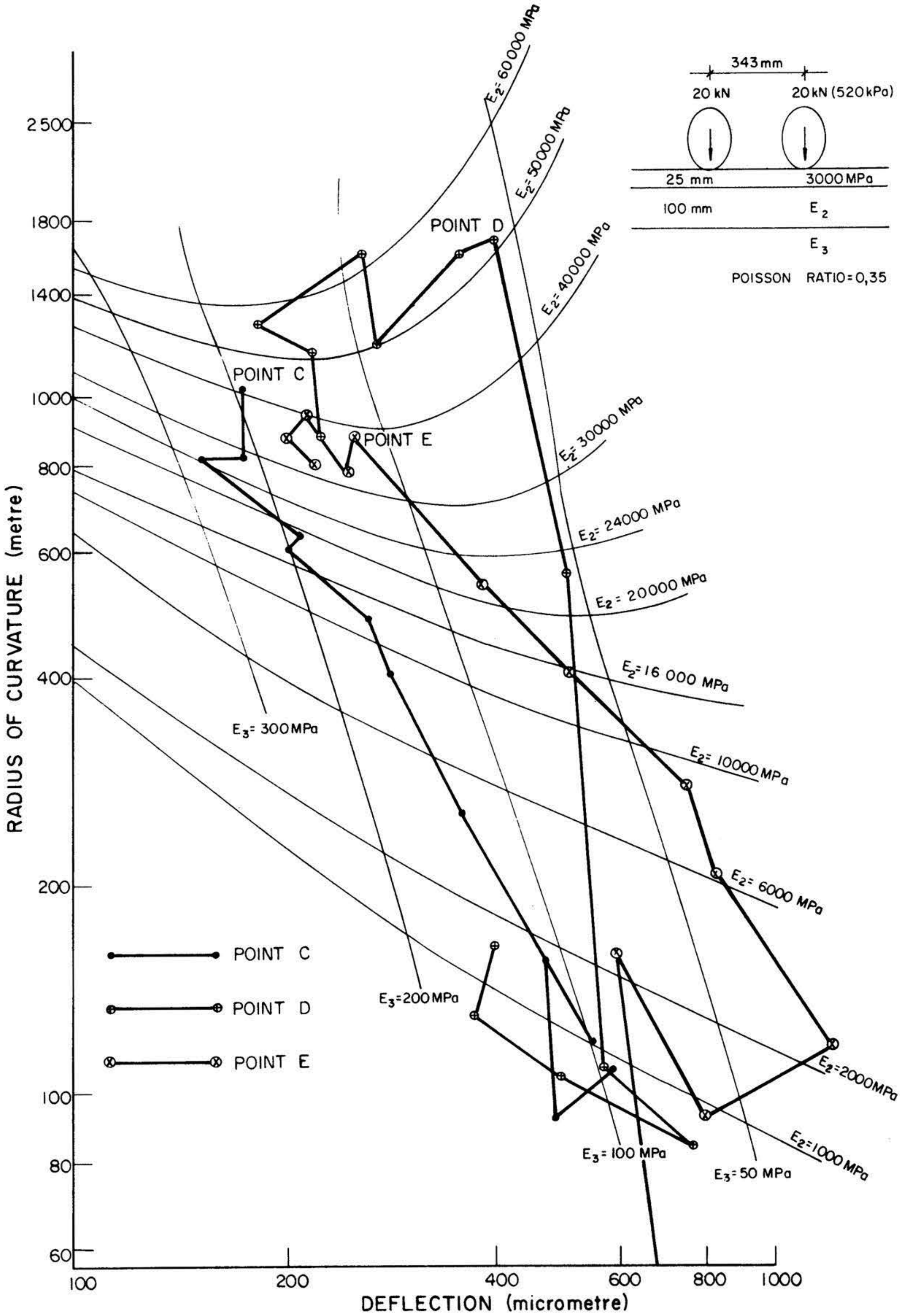


FIGURE 7-7

RELATIONSHIP BETWEEN RADIUS OF CURVATURE AND DEFLECTION FOR A THREE-LAYERED PAVEMENT. (SITE 1)

Test site 2 had a fair amount of cracking and these cracks appeared to be very well distributed over the site (Figure 7.5). Figures 7.8 and 7.9 seem to confirm the assumption of relative homogeneity for this site since the plots of recorded deflection and radius of curvature at the 5 test points are very much the same. The elastic modulus of the cement-treated layer (E_2) was reduced to below 1 000 MPa and this was confirmed when three 600 x 600 mm block samples were sawn from the pavement after completion of the test. The samples were removed at test points C, D and E and schematic drawings of the cracking patterns at the bottom of the cement-treated crusher-run base can be seen in Figure 7.10. These drawings confirm that the cement-treated layer was in fact cracked into little pieces, especially at test points D and E, and that its elastic modulus would be of the order of about 1 000 MPa.

Figures 7.11 and 7.12 show the data recorded on site 3. The relatively high elastic moduli at points D and E during the initial stages of the test and the relatively low moduli at point B cannot be easily explained, but it does not support the assumption of homogeneity over the relatively short (5 m) test section.

Block samples were also sawn at test points B, C and E after completion of the test. The schematic cracking patterns at the bottom of the cement-treated crusher-run layer appear in Figure 7.13. The extent of cracking at point B seems to tie in with the relatively low elastic modulus (less than 1 000 MPa) calculated for point B (Figure 7.11).

7.3.4.3 Conclusions and observations

From the trafficking and testing on these three sites it was concluded that -

(i) it is difficult to make reliable predictions about pavement behaviour, for example, pumping and surface cracking, from Benkelman beam and Dehlen curvature meter tests because they only measure surface phenomena;

(ii) the amount of variation in material quality and pavement behaviour are significant, both along the 5 m long test section and between the three test sites, and

(iii) excellent agreement between the calculated and recorded elastic moduli of the cement-treated base was observed at all the points (8) which were sampled after completion of the test.

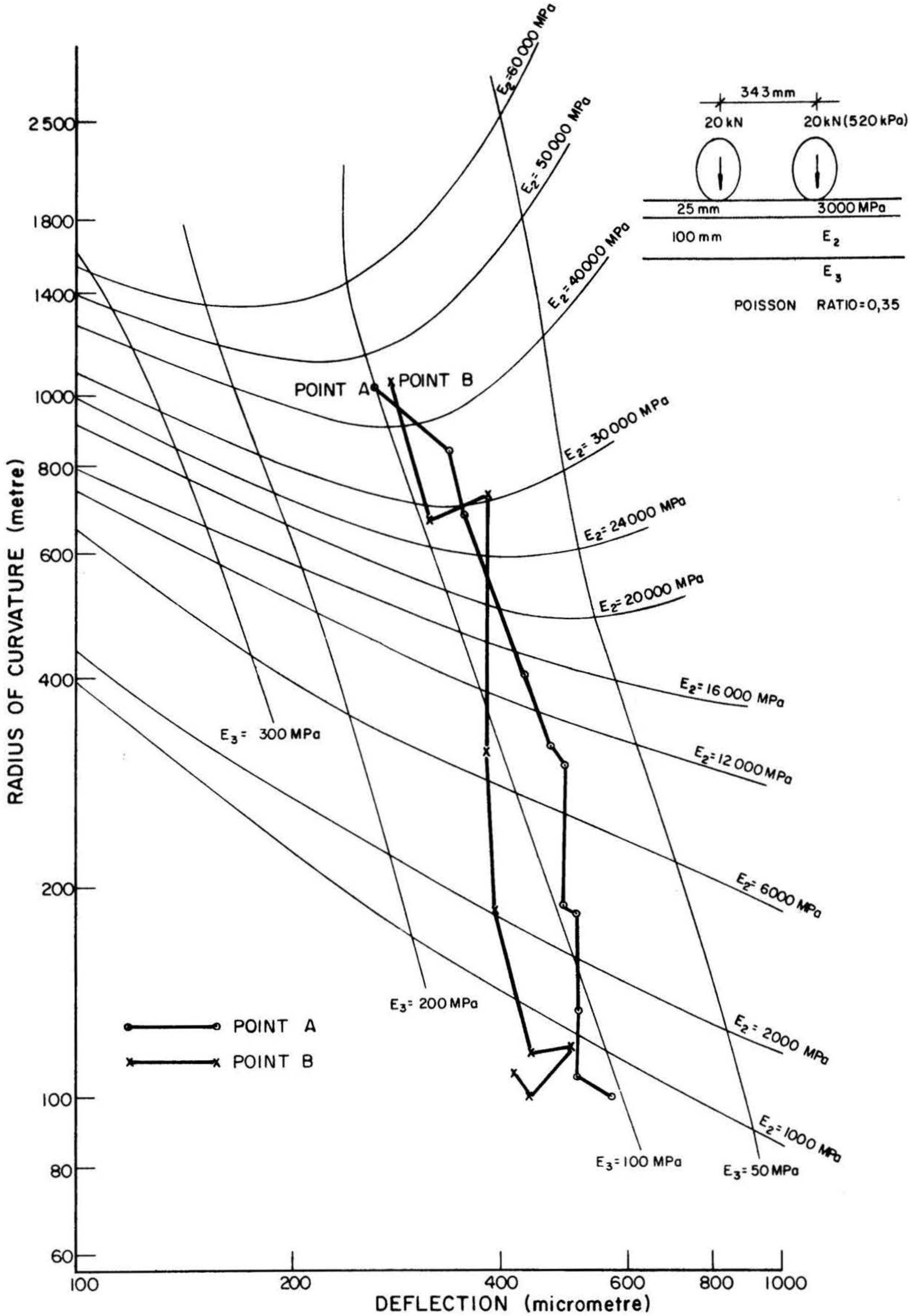


FIGURE 7-8

RELATIONSHIP BETWEEN RADIUS OF CURVATURE AND DEFLECTION FOR A THREE-LAYERED PAVEMENT. (SITE 2)

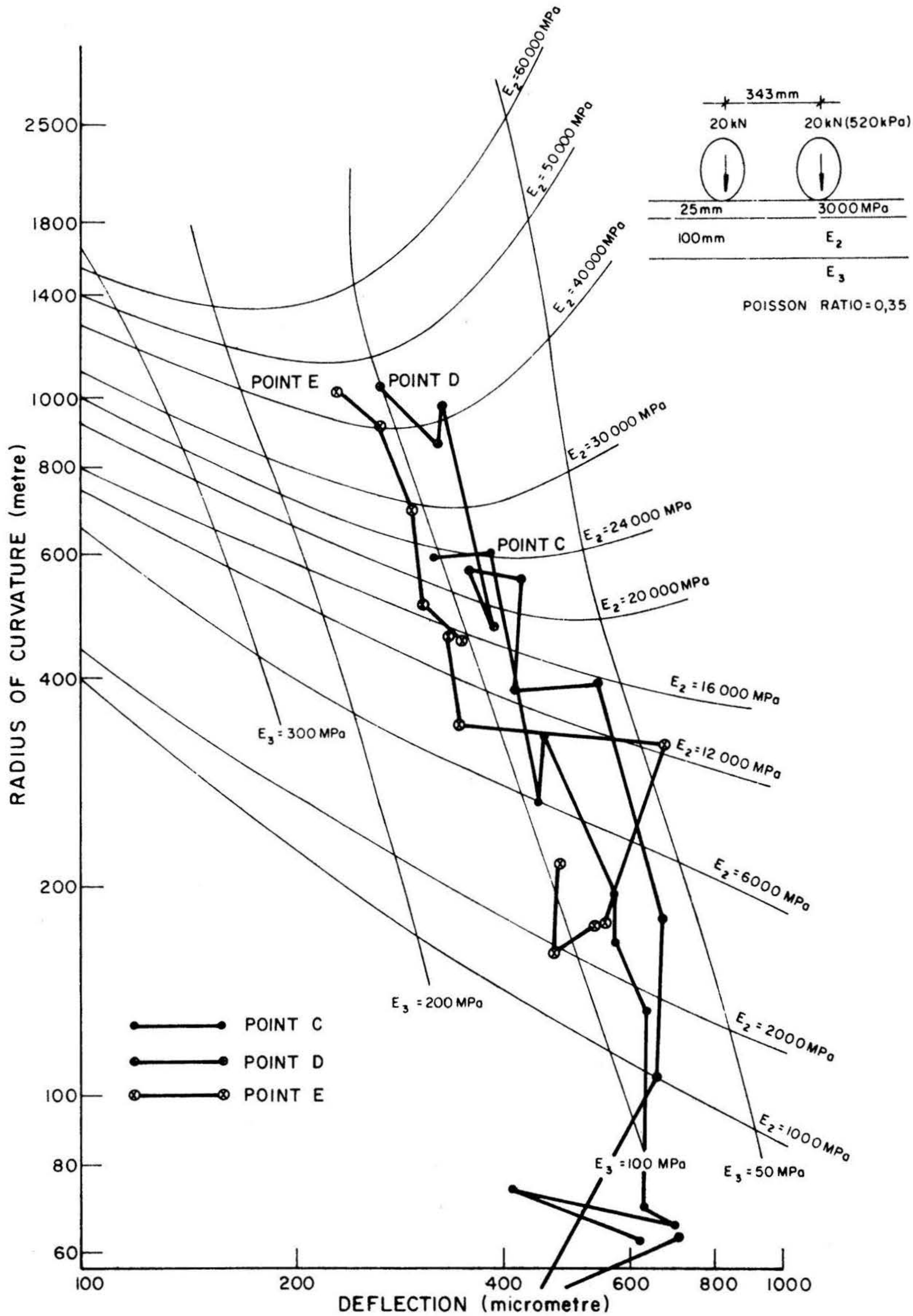


FIGURE 7-9

RELATIONSHIP BETWEEN RADIUS OF CURVATURE AND DEFLECTION FOR A THREE-LAYERED PAVEMENT. (SITE 2)

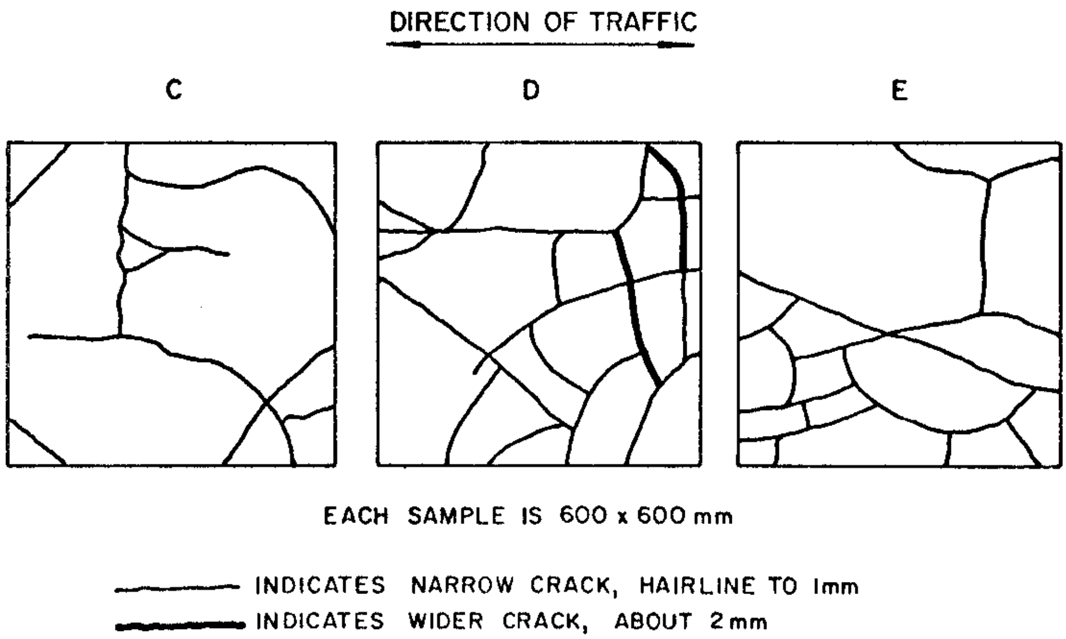


FIGURE 7-10

SCHEMATIC DRAWING OF VISIBLE CRACKS AT THE BOTTOM OF THE CEMENT-TREATED CRUSHER-RUN AFTER COMPLETION OF THE HVS-TEST (SITE 2)

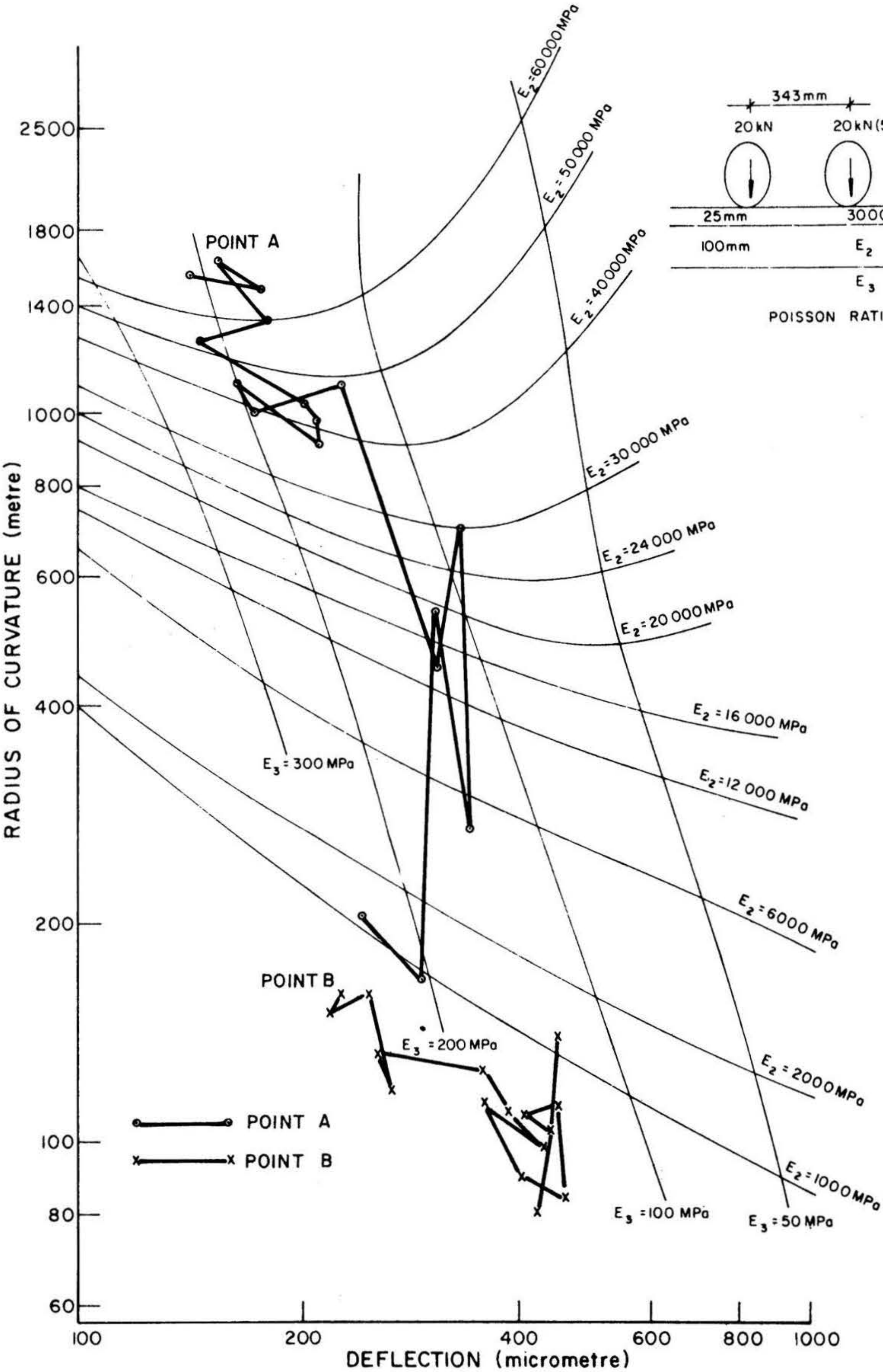


FIGURE 7-11

RELATIONSHIP BETWEEN RADIUS OF CURVATURE AND DEFLECTION FOR A THREE-LAYERED PAVEMENT. (SITE 3)

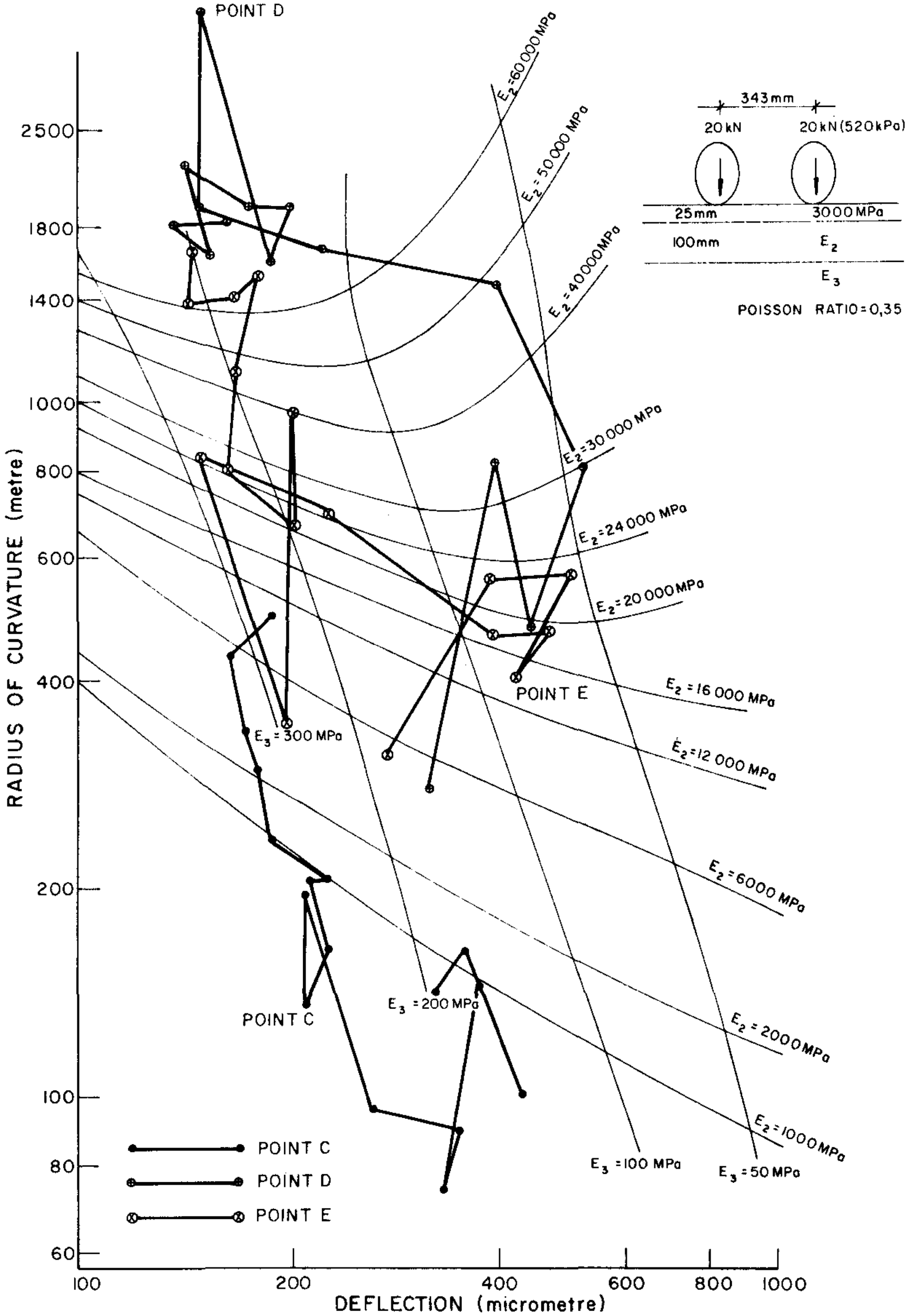


FIGURE 7-12

RELATIONSHIP BETWEEN RADIUS OF CURVATURE AND DEFLECTION FOR A THREE-LAYERED PAVEMENT. (SITE 3)

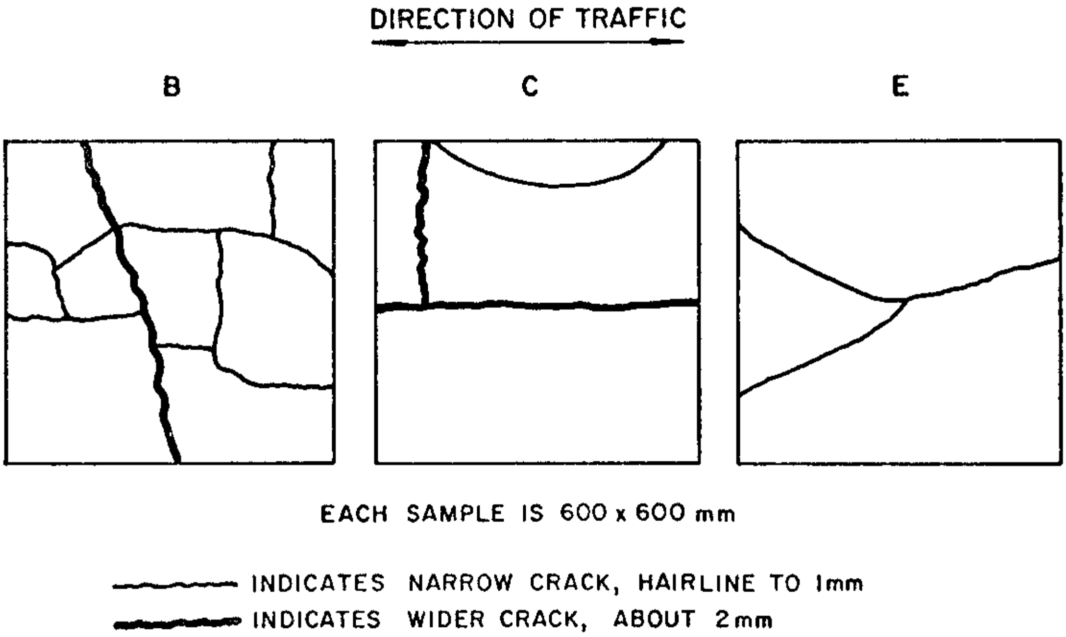


FIGURE 7-13

SCHEMATIC DRAWING OF VISIBLE CRACKS AT THE BOTTOM OF THE CEMENT-TREATED CRUSHER-RUN AFTER COMPLETION OF THE HVS-TEST (SITE 3)

7.3.5 Miscellaneous tests on cement-treated bases

(a) Kloof-Stanhope on Route N3 (16 March 1972)

The structural layout of this section would consist of 50 mm bituminous surfacing, 150 mm bituminous base, 100 mm cement-treated crusher-run subbase and the necessary selected layers and subgrade. It was calculated and predicted that the lower layers would not stand up to traffic loading without the 200 mm bituminous material on top of them. To verify this an HVS test was performed on top of the completed cement-treated crusher-run subbase.

It was also thought possible to use an ultra-short range seismic device to determine at what stage traffic-associated cracking in the cement-treated layer began. This was not successful since visible cracking developed after 220 repetitions of the 50 kN wheel load and yet the ultra-short range seismic device did not indicate the development of any cracks. It was then decided to stop the test.

This test was nevertheless very important because the behaviour of the pavement correlated very well with the theoretical prediction. The average surface deflection of the test area was about 448 μm. After performing a layered elastic theory analysis on the two-layer system, the stiffness of the lower layer which produced a surface deflection of about 448 μm, was calculated to be about 82 MPa. To perform this analysis the cement-treated crusher-run subbase was sampled and the elastic modulus was measured as 16 400 MPa. Under a 50 kN load, at 680 kPa tyre contact pressure, the strain at the bottom of the cement-treated layer was calculated as about 170 με. When measuring the elastic modulus, the strain at break (ε_b) of this material was measured as about 114 με and from theory (section 2.2.10) it was predicted that microcracking would develop during the first load application. The section did however withstand 220 repetitions before severe visible cracking developed in the cement-treated crusher-run layer and the test was ended. The surface deflection had then increased to about 563 μm.

It must be stressed that the test was performed on the cement-treated subbase, before the thick bituminous layers were put on. The early failure of the subbase should therefore not necessarily be taken as an indication of the expected performance of this road.

This design, as it was during the test, may be considered as the weakest structure tested to date with the HVS.

(b) Base 1 on Route S12 (26 November 1973 to 22 February 1974)

This test was part of a project for performing an HVS test on all the sections of the Cloverdene-Argent experiment on route S12 (Van Vuuren, 1972a). The wheel load was 65 kN and 285 000 repetitions were applied. Severe traffic-associated cracking and pumping were observed and the test confirmed that this type of design will pump heavily after rain.

The measurements taken during the test are reproduced as Table 7.9. Note the relatively large vertical deformation (18 mm) which occurred. This may have been caused by the severe pumping.

TABLE 7.9 : Measurements recorded during HVS test - Base 1 on S12 (second test)

REPETITIONS	DEFLECTION (μm)	RADIUS OF CURVATURE (m)	MAXIMUM DEFORMATION (mm)	RAINFALL (mm)
0	261(31)	445(32)	0	0
500	339(30)	357(38)	0,6	0
1 500	400(37)	307(32)	0,4	0
3 000	382(31)	348(32)	0,6	0
6 700	412(29)	284(44)	0,6	0
23 321	436(14)	278(36)	1,0	10
39 505	476(12)	240(77)	1,8	0
62 505	510(8)	229(49)	3,4	10
85 787	501(8)	212(52)	4,8	11
141 991	504(15)	148(60)	6,4	0
177 872	444(12)	128(37)	9,0	12
208 941	600(19)	162(46)	12,2	0
241 953	585(11)	116(37)	13,6	0
285 453	581(13)	72(59)	18,2	10
The numbers in brackets are the coefficients of variation in per cent.				

(c) Base 2 on Route S12 (17 January to 4 July 1974)

This test was also performed as part of the test series on the experimental sections at Cloverdene on Route S12. The reason why it was abandoned after only 14 000 repetitions of a 65 kN wheel load is not clear and very little can be learned from this test. The measurements taken during the test are reproduced in Table 7.10. The exceptionally large coefficients of variation for the radius of curvature (up to 80 per cent) were caused by a crack at point B which resulted in very low radii of curvature at this point.

TABLE 7.10 : Measurements recorded during HVS test - Base 2 on S12 (second test)

REPETITIONS	DEFLECTION (μm)	RADIUS OF CURVATURE (m)	MAXIMUM DEFORMATION (mm)	RAINFALL (mm)
0	164(17)	643(24)	0	10
1 000	219(36)	287(56)	0,2	0
3 000	266(12)	261(50)	0,4	0
7 511	293(17)	192(63)	0,6	14
11 000	248(19)	204(58)	0,8	9
14 312	351(15)	123(79)	0,6	0
The numbers in brackets are the coefficients of variation in per cent.				

7.3.6 Eerste Fabrieke on Route N4/1 (Pretoria to Bronkhorstspruit) (1 August to 27 November 1975)

The purpose of this test was to evaluate the effect of a crack on the results of an HVS test and also to verify the theory on the increase in stress proposed in Chapter 4. Since the amount of particle interlock across a crack cannot be determined easily it was decided to eliminate this complication completely. A 5 mm wide 125 mm deep transverse saw joint was therefore cut right through the 25 mm surfacing and 100 mm cement-treated crusher-run base, and it was extended about 1 m on either side of the HVS test area. The number of surface deflection measurements was increased and they were recorded much more frequently than in the previous tests. The measurements recorded during the test are reproduced as Table 7.11 and it should be noted that the saw-cut was at point C. The radius of curvature measurements are rather erratic and should preferably be disregarded.

The surface deflection at point C (100 μm) was used to calculate the elastic modulus of the subgrade according to the method explained in section 7.3.2 and it turned out to be about 400 MPa. The elastic moduli of the 25 mm surfacing, 100 mm cement-treated crusher-run base, 100 mm untreated crusher-run and 100 mm cement-treated subbase required to do the calculation for the subgrade's elastic modulus were taken as 3 000, 21 000, 500 and 3 000 MPa. Using these moduli and a constant Poisson ratio of about 0,35, the tensile strain in the cement-treated crusher-run base under a 55 kN wheel load was calculated as 57 $\mu\varepsilon$. According to Table 4.3 (page 72) the wide saw-cut would have increased the calculated 57 $\mu\varepsilon$ by about 17 per cent to about 67 $\mu\varepsilon$ (ε). The strain at break (ε_b) of the material at this site was measured as about 173 $\mu\varepsilon$. The strain ratio ($\varepsilon/\varepsilon_b$) next to

TABLE 7.11 : Measurements recorded during HVS test - Eerste Fabrieke

REPETITIONS	DEFLECTION (μm)						COEFFICIENT OF VARIATION (%)	RAINFALL (mm)
	A	B	C	D	E	Average		
0*	123	100	100	84	133	108	18,3	0
0**	131	134	158	154	136	143	8,7	0
40	131	144	198	163	138	155	17,4	0
100	120	145	214	169	149	159	22,0	0
200	118	135	176	171	169	154	16,8	0
400	110	119	185	150	149	143	20,8	0
700	143	123	164	141	148	144	10,2	0
1 000	139	133	161	136	158	145	9,0	0
2 000	120	127	183	148	160	148	17,3	0
4 000	124	115	184	155	150	146	18,8	0
7 000	130	140	144	123	139	135	6,3	0
10 000	143	134	113	124	141	131	9,5	0
20 000	146	160	221	200	207	187	17,2	0
30 000	202	170	248	210	225	211	13,7	0
70 000	158	180	191	185	221	187	12,2	0
93 212	125	165	290	185	240	201	-	30
142 000	213	236	820	438	390	419	-	80
175 000	240	258	960	465	423	469	-	32
184 665	248	296	671	396	365	395	-	48
	RADIUS OF CURVATURE (m)							
0*	1 499	1 247	2 032	1 663	1 206	1 529	22,1	0
0**	1 663	1 306	0	2 195	1 131	1 574	30,0	0
10 000	2 032	1 247	2 888	-	-	2 056	40,0	0
30 000	3 291	2 993	0	766	587	1 909	75,0	0
93 212	3 658	2 351	-	672	716	1 849	78,0	30
184 665	1 062	1 176	506	147	155	609	80,3	190

*Before the saw-cut.
**After the saw-cut.

the crack is about 0,39 and according to equation (2.5) (page 31) the material is likely to carry about 350 000 load repetitions before fatigue cracking is likely to occur.

The surface deflection versus number of repetitions are reproduced in Figure 7.14. During the first 93 000 repetitions of the 55 kN wheel load the deflection at the saw-cut (point C) increased at the same rate as at the other four points. During the following 82 000 repetitions (from 6 October 1975 to about 25 November 1975) 142 mm rain fell and the deflection at point C increased rapidly to about 960 μm. From this figure it

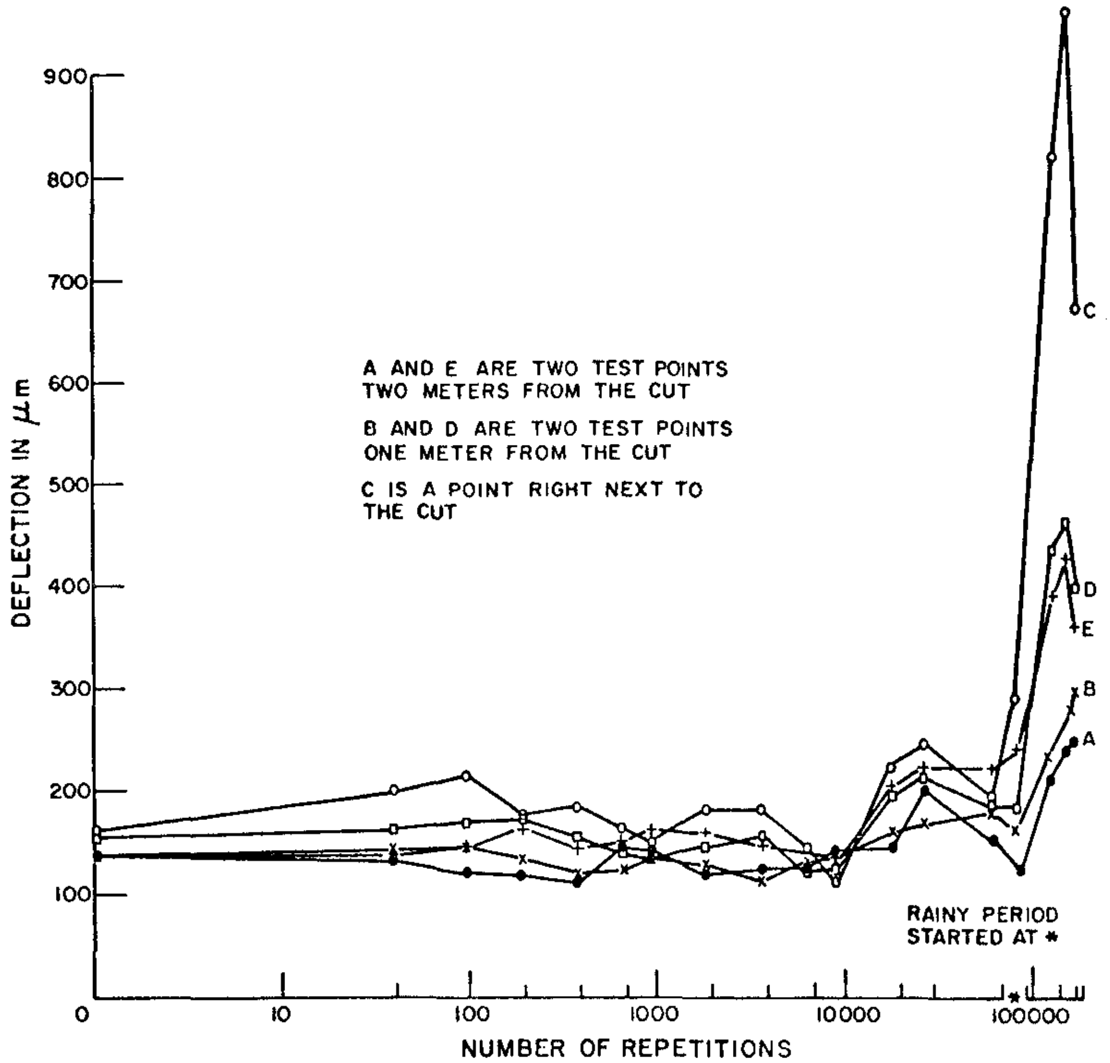


FIGURE 7-14

EFFECT OF RAIN ON SURFACE DEFLECTION NEXT TO A SAW CUT appears that a wide saw-joint, and for that matter any crack, would have very little effect on the surface deflection of this road, provided the water could be kept out of the pavement. As soon as rain fell water could easily penetrate through the 5 mm cut, it apparently softened the lower layers (mainly the crusher-run), the elastic modulus was reduced and the deflection increased rapidly.

The reduction in elastic modulus of the lower layers very probably resulted in an increased strain in the cement-treated crusher-run layer next to the crack. This would increase the calculated strain ratio (0,39) which implies that cracking would have started much sooner than the predicted 350 000 repetitions.

After 184 000 repetitions when the trafficking was stopped, no cracks were visible on the surface. Two blocks (600 x 600 mm), one on each side of the saw-cut, were sawn and the cracks at the bottom of the cement-treated crusher-run can be seen in Figure 7.15. The cracks developed as was predicted by the prismatic solid finite element analysis (Chapter 4), namely perpendicular to the saw-cut, and no cracks developed parallel to the cut.

Block samples were also removed at test points A and E and the schematic crack patterns are shown in Figure 7.16 along with those at point C. Both these test points had higher initial deflections (123 and 133 µm) than point C (100 µm) and hence probably a lower elastic modulus for the foundation, resulting in higher strains in the cement-treated crusher-run.

If the strain in the cement-treated crusher-run was 74 µε (that is only 10 per cent more than 67 µε) the strain ratio would be about 0,43 and this would require about 154 000 repetitions to the onset of cracking. This relatively small increase in strain may explain why cracks were visible after 184 000 repetitions.

7.4 DISCUSSION

Kloof-Stanhope was the weakest structure tested - it cracked fairly early and conformed to the theoretical prediction. The first test on Base 2 of the experimental section on Route S12 was unfortunately never completed. It was calculated and shown that the pavement of the Johannesburg Eastern Bypass was very strong. After it had withstood about 4 million equivalent 80 kN axles a sample with a lower elastic modulus was recovered from the road. The sample still had a relatively high modulus (13 300 MPa) and no cracks could be seen on the surface. It may therefore be assumed that the

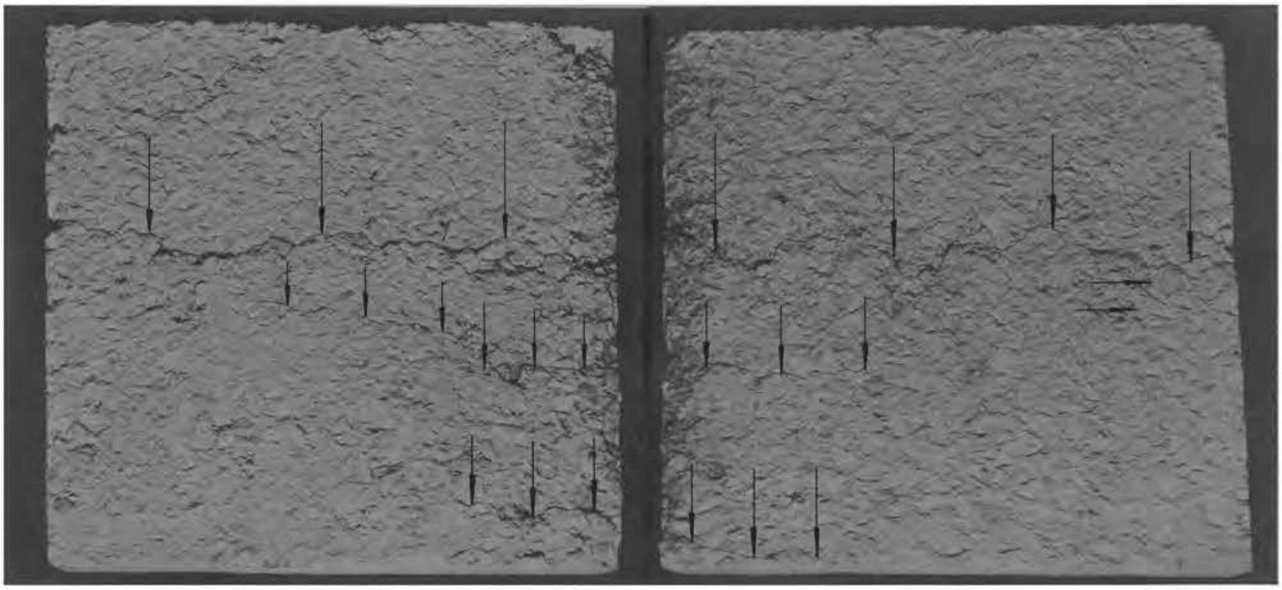


FIGURE 7.15

Visible cracks at the bottom of the cement-treated crusher-run. They are perpendicular to the saw cut.

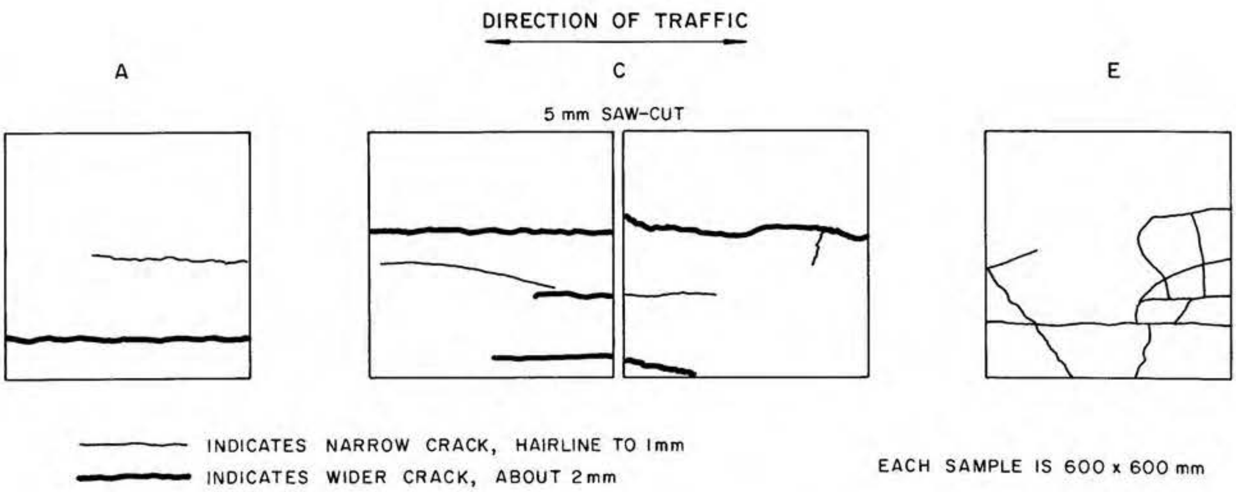


FIGURE 7-16

SCHEMATIC DRAWING OF VISIBLE CRACKS AT THE BOTTOM OF THE CEMENT-TREATED CRUSHER-RUN AFTER COMPLETION OF THE HVS-TEST road performed as predicted - it safely and successfully carried the overload. The three tests at Eerste Fabrieke on Route N4/1 were fairly extensive but the variability in material quality over the 5 m test length appeared to be a very significant factor. In an attempt to correlate theory and practice for the tests at Eerste Fabrieke, more measurements, for example plate bearing tests, were taken but this seemed to impede the analyses. More effort should therefore be devoted to ways and means of measuring the elastic moduli of materials in situ.

It was during the detailed analysis performed on the results of one of the tests at Eerste Fabrieke (section 7.3.4) that it was realised that the destructive testing procedure of removing block samples should not be used. This is because there are large variations at each site and if a change in say elastic modulus is recorded, one cannot be sure whether it was caused by the trafficking, or whether it was because of the variation in materials quality over the HVS test site. It was realised that a non-destructive device is needed to monitor the changes continuously at a particular point during an HVS test. Such a device is the Multidepth Deflectometer (MDD) which is currently being developed and evaluated by the National Institute for Transport and Road Research (NITRR) of the CSIR.

If the test on Base 2 of road S12 (section 7.3.2) could have been completed successfully, a plot of elastic modulus versus number of repetitions would have been possible. It was hoped that such a plot would enable one to determine the number of repetitions to failure for the cement-treated material. Repeated loading work on cement-treated materials did, however, show that the elastic modulus of cement-treated materials remains virtually constant and that it suddenly drops off at the failure point which indicates a brittle failure for the material (Figure 7.17). To prepare a graph of the change in elastic modulus which would accurately describe the sudden drop-off at the failure point, would therefore require a large number of test points and the plot expected from the test on Base 2, prepared from only 4 points, would therefore not have been sufficient. It is when this type of study has to be done that the Multidepth Deflectometer (MDD) may be very advantageous because a virtually continuous monitoring of the elastic modulus would be possible. Since the MDD can simultaneously measure the elastic modulus of a number of layers in the structure, it may be possible to observe the changes in their material properties continuously and from this determine which layer distresses (fractures) first and what happens to the stress distribution after the distress.

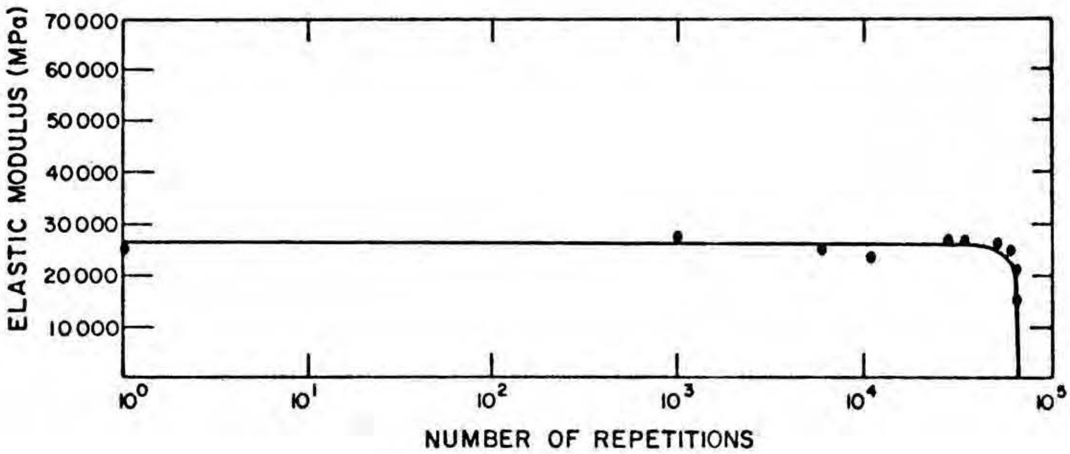


FIGURE 7-17

REDUCTION IN ELASTIC MODULUS OF CEMENT-TREATED MATERIAL DURING REPEATED LOADING

7.5 FUTURE WORK

The purpose of all the HVS tests was to correlate theory and practice. Since the first test on S12 it was realised that it would not be easy because of the destructive nature of the existing test procedures and the variability on a site. It was hoped to use available non-destructive seismic devices to record the onset of cracking but this method failed to produce positive results. It was also hoped that a plot of elastic modulus versus number of repetitions would show the failure point, but too few points were taken to produce a reliable figure. It is hoped that the proposed future use of a Multidepth Deflectometer will overcome these problems.

The study to evaluate the effect of a crack on the behaviour of a pavement should be continued. The one test on a saw-cut (section 7.3.6) showed that water should be kept out of the pavement and it is recommended that this test should be repeated during the dry season to determine whether the significant increase in deflection next to the crack (point C) was really due to the ingress of water. The testing should also be repeated on actual cracks which do have a certain amount of particle interlock and there should also be some control sections. At least three of each test should be done to obtain statistically reliable results, and this means that at least nine sections are needed. The Pretoria area has about 4 dry months per year and at about 2 tests per dry season it would take about 4 seasons, that is 4 years, to do this study. It should also be complemented with studies during rainy seasons to evaluate the effect of water in the cracks properly.

7.6 CONCLUSIONS

(a) It is very easy to monitor changes in Benkelman beam deflections and Dehlen curvature meter results during an HVS test, but they are not sufficient to produce all the information required for pavement design purposes. These measurements once again confirmed that the deflection increases and the radius of curvature decreases with an increase in the number of repetitions.

(b) A drop in elastic modulus is a very valid indicator of cracking in cement-treated materials. Initially it was thought possible to determine the failure point, and hence correlate theory and practice, from a plot of the material's elastic modulus versus the number of load repetitions. Subsequent laboratory work showed that cement-treated materials really exhibit brittle fracture and that a large number of points would be required to prepare such a plot. The destructive testing procedure used during the testing on Bases 1 and 2 on road S12, and which would have produced 3 or 4 points on a graph, is therefore not sufficient. A much more expensive, non-destructive and continuous monitoring device is needed, for example the Multidepth Deflectometer.

(c) Elastic theory indicates that Benkelman beam deflections and Dehlen curvature meter results may be utilized when the pavement structure can be simplified into an equivalent three-layered structure. It is then possible to study the changes in the elastic properties of the materials in the equivalent structure. This was confirmed by HVS test results because the agreement between predicted and measured elastic moduli on eight test points was remarkably good!

(d) Layered elastic theory and the elastic properties of the various materials in the structural layouts were used to predict the behaviour of some of the sections tested under the HVS. The predictions that 2 sections, namely Base 1 on road S12 and the Kloof-Stanhope section on Route N3, would undergo traffic-associated cracking during the first few repetitions of the test wheel, were substantiated by recovered samples and visual observations at the end of the tests. It was also predicted that two other sections, namely the Johannesburg Eastern Bypass and the section with the saw-cut at Eerste Fabrieke on Route N4, would withstand a substantial amount of traffic, namely about 255 000 and 350 000 repetitions respectively of the 75 kN and 55 kN wheel loads respectively before traffic-associated fatigue cracking could be expected to occur. At the end of the two tests, after 280 000 and 184 000 repetitions respectively, there were no signs of visible cracking on the surface, but in the first case microcracking had commenced as evidenced by a significant drop in the material's elastic modulus, and in the second case visible cracks had developed in the cement-treated layer.

CHAPTER 8

A DESIGN PROCEDURE FOR PAVEMENTS WITH CEMENT-TREATED LAYERS

PAGE

8.1 INTRODUCTION

The requirements for a well-designed structural layout containing a cement-treated layer were discussed and outlined in Chapter 2. Some of these requirements are very important and existing pavement design procedures are hardly adequate with respect to these requirements. The existing procedures also suffer from numerous shortcomings, which include giving insufficient credit to layer thickness, their insensitivity to the layer sequence which may lead to an unbalanced layout, and the incorrect handling of traffic loadings. The objective of this chapter is to outline a design procedure which is considered more suitable to the structural design of pavements with cement-treated layers than the existing procedures.

8.2 THE DESIGN PROCEDURE

8.2.1 Design objective and approach

The objective of structural pavement design is to provide a pavement with an acceptable riding quality throughout the pavement's design life of usually between 10 and 20 years. The author believes that in pavements with cement-treated layers this design objective can largely be met if traffic-associated cracking of the treated materials can be controlled. This is because a loss of riding quality usually follows after this type of cracking in the cement-treated material (Otte, 1973a), but it should be appreciated that the onset of traffic-associated cracking does not result in an immediate and significant drop in the pavement's riding quality and that the pavement may continue to carry traffic successfully for a considerable period of time after the onset of traffic-associated cracking.

Traffic-associated cracking during the pavement's design life can be controlled in at least three ways.

The one approach is to prevent even the first traffic-associated microcrack (see 2.2.7, page 20) in the matrix of the cement-treated material and to equate its development to termination of the pavement's design life. This implies an infinitely long design life for the cement-treated material since it will remain intact throughout its life. This is considered to be an unnecessarily conservative approach because in practical pavement design it is generally accepted that a material should not be designed to carry traffic for an infinitely long period and it is also accepted that the pavement can still carry traffic after the onset of traffic-associated microcracking.

A second alternative design approach is to accept the fatigue life concept which implies that cement-treated materials can carry a limited number of load repetitions before failing due to fatigue. In this approach the treated material is considered to undergo traffic-associated cracking once the fatigue capacity of the material has been consumed by the load applications (equations 2.3 to 2.6 in section 2.2.10). During the structural design process it should therefore be endeavoured to ensure that the treated material's fatigue capacity will not be exceeded and that traffic-associated cracking will not occur before the pavement has carried the design traffic.

The third alternative design approach, and probably the more practical and even less conservative one, is to accept traffic-associated cracking in the treated layer before the pavement has carried the full design traffic. The period of remaining life after traffic-associated cracking has occurred in the cement-treated layer, will be called the post-crack phase in this thesis. This is a very important phase in the overall design life of pavements with cement-treated layers and, depending on the material properties and traffic conditions, it can vary between 20 to 80 per cent of the total design life - it should therefore not be ignored! To properly analyse and include the contribution of this post-crack phase in the design life is very complex. It is currently only possible to make tentative suggestions on (i) how the post-crack phase can be included during the design, and (ii) the research work still to be done to include it with any degree of confidence.

A very important aspect when accepting the traffic-associated cracking before the pavement has carried the design traffic (that is to consider and include the post-crack phase) is to ensure that the pavement will not deform excessively with a consequent loss in riding quality during this phase. It is possible that deformation and a loss of riding quality may occur because of the reduction in the elastic modulus of the cement-treated layer and the consequent increase in vertical compressive strain within the lower layers. It is therefore important that all the layers in the pavement should be able to successfully carry the remainder of the design traffic after the onset of traffic-associated cracking in the treated layer(s).

The selection of the appropriate design approach for a specific project remains with the designer or the client (usually a road authority), and depends largely on the conditions of the particular project such as prestige, environment, traffic volume, etc. It would however appear realistic to choose either the second or third alternative approach. Engineers and clients are often inclined to think that the acceptance of the second approach is too conservative and that it would result in a very much more expensive pavement. This is however not always true. After the additional surety and larger factor of safety is properly considered, it may be preferable to design against the development of traffic-associated cracks before the pavement has carried the design traffic rather than to design for a post-crack phase with the (at present) unpredictable life within this phase. This argument would be particularly valid where heavily trafficked high prestige pavements are concerned.

8.2.2 The flow diagram

Various people (Freeme, 1972; Brown and Pell, 1972; Monismith, 1973; and Mitchell et al, 1974) have proposed theoretical pavement design procedures based on layered elastic theory, and have illustrated the procedure, or parts thereof, by means of a flow diagram. Figure 8.1 shows the flow diagram of a design procedure for pavements with cement-treated layers which is based on the suggestion by Brown and Pell (1972). On the following pages this figure will be discussed.

8.2.3 Design traffic and tyre contact pressure

To control the amount and rate of microcrack development in the matrix of fine material and cement, a cement-treated layer should be designed to withstand the number of heavy wheel loads expected on the pavement. The number and magnitude of significantly lighter wheel loadings are not important (section 2.2.7).

The maximum legal wheel load permitted in South Africa is 40 kN, but this is often exceeded as was previously shown (Otte, 1973a). To predict the heaviest wheel load expected is rather difficult. Traffic weighing surveys on roads in the vicinity of the proposed new road will give an indication of the maximum wheel loads of present traffic, but predictions of future maximum wheel loads will require predictions of future developments in commodities carried, truck design, vehicle mass legislation, and law enforcement. If reliable estimates cannot be made on this basis, then use must be made of the presently available knowledge on the overloading taking place on the roads in the country. It is suggested that a design wheel load of 60 kN (50 per cent overloaded) should be used for 'normal' roads and 80 kN for roads expected to carry heavy traffic. The frequency of the design wheel load on the pavement should be determined by considering factors such as the effectiveness of law enforcement with respect to vehicle loading, and the

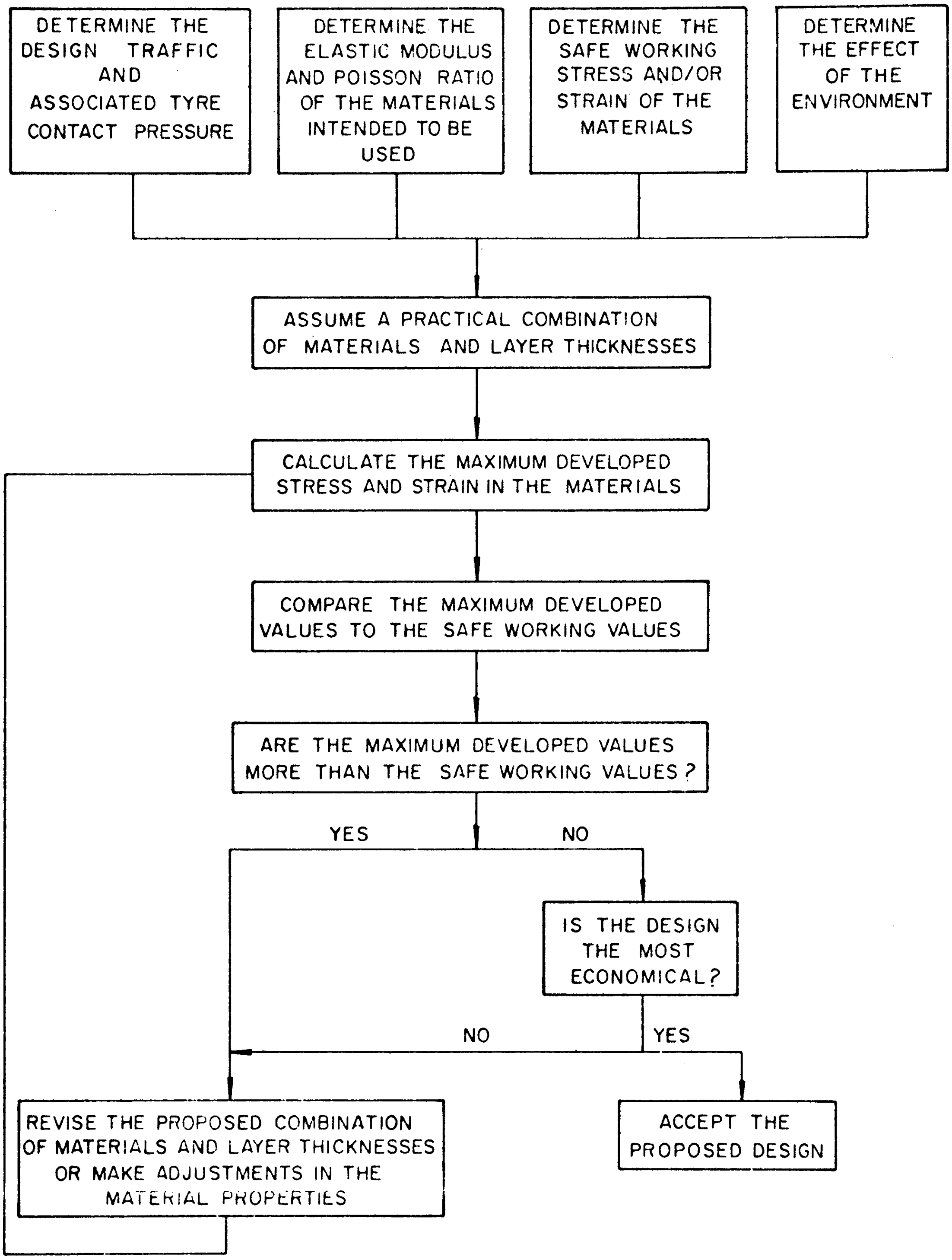


FIGURE 8-1

THE FLOW DIAGRAM OF A DESIGN PROCEDURE FOR PAVEMENTS WITH CEMENT-TREATED BASES

(Based on Brown and Pell, 1972)

number of laden trucks. It is nevertheless recommended that the full traffic spectrum, that is the estimated number of repetitions (n) in each axle load group, should be measured and used in the design. This method is explained later, in section 8.3.1.

To design the thin bituminous surfacing (less than 50 mm) the suggestions by Freeme and Marais (1973) are accepted and the total number of axle repetitions (T), irrespective of load, is used as the design traffic. The traffic loading on the untreated materials and subgrade is expressed in terms of the number of equivalent 80 kN axles (E80) and to obtain this number it is assumed that the Californian equivalency factors do hold (Van Vuuren, 1972).

The design traffic for the proposed design procedure is dependent on the material type and it is therefore expressed as at least three different numbers (Table 8.1).

TABLE 8.1 : Definition of design traffic

MATERIAL TYPE	DESIGN TRAFFIC
Bituminous	The total number of axle repetitions (T) (for example 5 million).
Cement-treated	The number of heavy wheel loads (for example 5 000; 80 kN) (preferably the full traffic spectrum)
Untreated and subgrade	Number of equivalent 80 kN axles (E80) (for example 1 million)

Van Vuuren (1974) reported on a survey which determined the tyre pressures of vehicles. Using this and a relationship between tyre inflation pressure and tyre contact pressure, he proposed an average tyre contact pressure of 520 kPa. This pressure will be used as the standard in this procedure.

8.2.4 Elastic modulus and Poisson ratio of materials

8.2.4.1 Bituminous materials

It is known that these materials are visco-elastic and that they have a temperature-dependent elastic modulus. When designing pavements with cement-treated materials it is suggested that the visco-elastic nature of the bituminous material be accommodated by determining its elastic modulus at only one loading rate. This rate should correspond to highway-type loading rates and it is suggested that 80 to 100 km/h be used. The temperature dependence may be accommodated by taking moduli at two temperatures, namely an average warm value (30 °C) and a cold value (0 °C). It is suggested that the lower elastic modulus, which corresponds to the higher temperature, be used to design the cement-treated and untreated materials in the layout because the load-bearing ability of the bituminous material is then at a minimum. The higher elastic modulus should be used when designing the thickness of a bituminous layer on top of an untreated layer because this corresponds to a condition of maximum strain at the bottom of the bituminous layer.

These suggestions and simplifications are considered justified because the elastic modulus of the bituminous surfacing has only a small effect on the stresses developed in the various layers of this kind of pavement. This is because the bituminous layer is usually relatively thin (less than 50 mm) and it has a relatively low elastic modulus when compared with some of the cement-treated materials. For practical pavement design and from research on South African bituminous materials (NITRR, 1977), it is suggested that 2 000 and 6 000 MPa may be used for gap-graded (BS 594-type) bituminous mixes at 30 $^{\circ}$C and 0 $^{\circ}$C respectively. The corresponding values for asphaltic concrete mixes are 5 000 and 12 000 MPa. The suggested Poisson ratio is 0,44.

8.2.4.2 Untreated crusher-run

This material has a stress-dependent modulus which means that the equivalent elastic modulus is dependent on the support of adjacent layers and the applied load.

Biarez (1962) defined the elastic modulus, or resilient modulus (E_R), of granular materials as

$$E_R = K_1 \theta^{K_2} \qquad \text{.......................................} \quad (8.1)$$

where K_1 and K_2 are material constants, and θ is the sum of the principal stresses. Typical values of K_1 and K_2 for a good quality crusher-run are about 5 and 0,75 respectively (Maree, 1977).

When used in conjunction with relatively stiff cement-treated materials, untreated crusher-run performs only a minor load-carrying function because it has a lower elastic modulus than the cement-treated material. This allows one to choose a relatively fixed value and it is suggested that 500 MPa be used for general design purposes. The elastic modulus may however be as low as 350 MPa under light wheel loads or possibly higher than 600 MPa when it is carrying heavy traffic and is used on top of a stiff cement-treated layer. A Poisson ratio of 0,35 is typical (Maree, 1977).

8.2.4.3 Cement-treated materials

The elastic modulus in bending (E_b) of cement-treated materials varies greatly since it depends on the material, amount of cement, age, density, curing etc. The range is from about 3 000 MPa for weakly cemented natural gravel, to 38 000 MPa for high-quality cement-treated crusher-run. It is important to know what the elastic modulus of the treated layer will be and it is suggested that the value should be determined according to the laboratory procedure described in section 2.2.8(c) (Otte, 1972a and 1974). Even if it is not possible or feasible to do the complete test and hence measure the elastic modulus (E_b), it is possible to obtain an indication thereof if either the bending strength (σ_b) or the unconfined compressive strength (σ_u) can be obtained.

During the course of this research project at least 70 block samples were sawn from cement-treated crusher-run layers. These blocks were sawn into beam samples (usually 6) and tested in bending. A linear regression analysis produced equation (8.2) as the relationship between the average elastic modulus (E_b) and average bending strength (σ_b) of these block samples. The regression coefficient is 0,90. (Note: Elastic modulus in MPa and bending strength in kPa.)

$$E_b = 8{,}15\ \sigma_b + 3\ 485 \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (8.2)$$

Similar results on 79 field- and laboratory-prepared cement-treated natural gravel materials produced equation (8.3). The regression coefficient is 0,92.

$$E_b = 10{,}06\ \sigma_b + 1\ 098 \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (8.3)$$

Figures 8.2 and 8.3 summarize this information and it is suggested that the following rounded-off values (equations (8.4) and (8.5)) be used for practical pavement design purposes.

Cement-treated crusher-run : $E_b = 8\ \sigma_b + 3\ 500$ ..... (8.4)

Cement-treated natural gravel: $E_b = 10\ \sigma_b + 1\ 000$ .... (8.5)

The relationship between bending strength (σ_b) and unconfined compressive strength (σ_u) is not unique. A summary of the work by seven researchers (Otte, 1972a) indicated that the ratio σ_b/σ_u varies between 0,12 and 0,43. A reasonable average seems to be 0,2. In a review paper Mitchell (1976) suggested the following relationship between bending strength (σ_b) and unconfined compressive strength (σ_u).

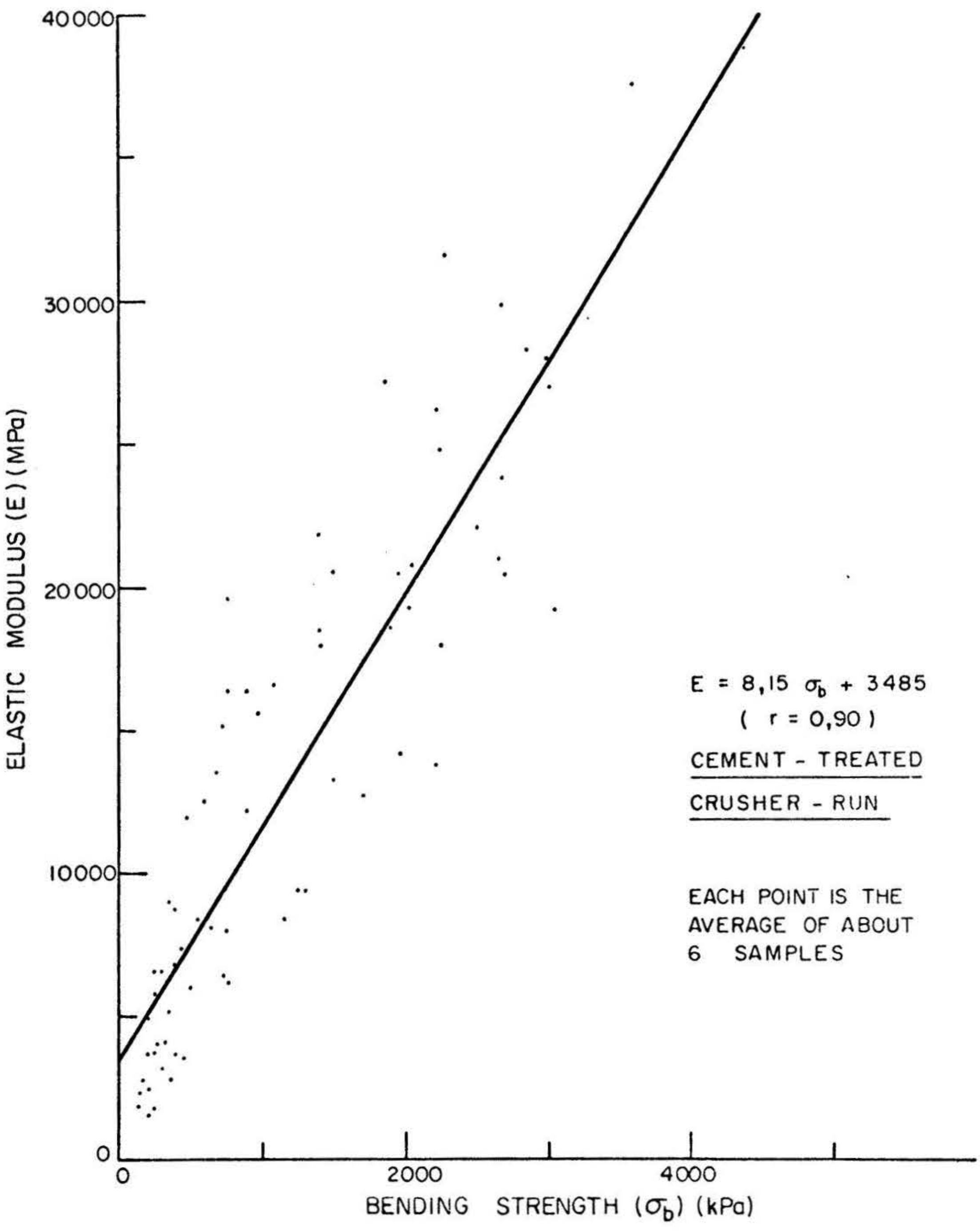


FIGURE 8-2

RELATIONSHIP BETWEEN BENDING STRENGTH AND ELASTIC MODULUS OF CEMENT-TREATED CRUSHER-RUN.

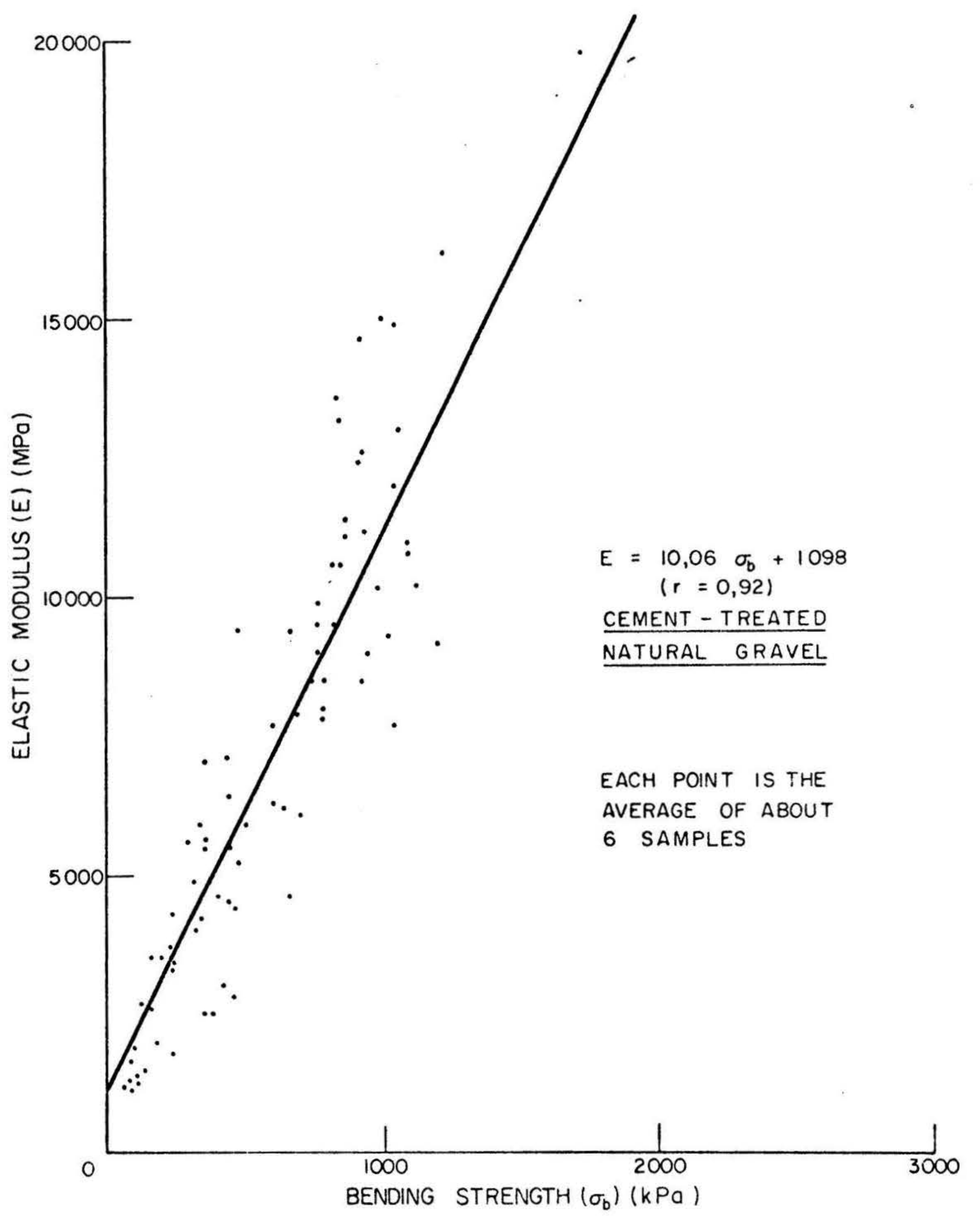


FIGURE 8-3

RELATIONSHIP BETWEEN BENDING STRENGTH AND ELASTIC MODULUS OF CEMENT-TREATED NATURAL GRAVEL.

$$\sigma_b = 0{,}51\ (\sigma_u)^{0{,}88} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (8.6)$$

In the relationships between E_b and σ_b (equations (8.2) and (8.3)), σ_b may therefore be replaced by 0,2 σ_u or by the relationship suggested by Mitchell. If the latter is accepted the following two equations may be used to relate E_b and σ_u.

Cement-treated crusher-run : $E_b = 4{,}16\ (\sigma_u)^{0{,}88} + 3\ 485$..(8.7)

Cement-treated natural gravel: $E_b = 5{,}13\ (\sigma_u)^{0{,}88} + 1\ 098$..(8.8)

Another approximate method used to obtain the elastic modulus in bending (E_b) was suggested by Mitchell (1976). He reviewed the published information on cement-treated soils and suggested equation (8.9).

$$E_b = K_f \cdot 10^{A.m} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (8.9)$$

where K_f = material constant

A = unconfined compressive strength in psi

m = $0{,}04\ (10)^{-0{,}186C}$

C = cement content by weight (per cent)

If the various material constants (K_f) can be obtained this suggestion may prove useful for practical pavement design.

If none of the three approximate relationships for obtaining the elastic modulus given above can be utilized, and if no other information is available, it is suggested that 20 000 MPa may be used for cement-treated crusher-run, and 9 000 and 3 500 MPa for cement-treated natural gravel of base and subbase quality respectively. The Poisson ratio is not critical in the analysis (Fossberg, 1970; and Otte, 1974) and may be taken as 0,35.

When the elastic modulus of a cement-treated material is selected for design purposes the following two points should be taken into account: (i) Construction variations, since Chapter 6 indicates that the eleastic modulus is very variable and that it can vary by up to 200 per cent within a particular project (page 110). The study also shows that the elastic modulus does not appear to be constant throughout the depth of the layer, and that the upper part has a higher elastic modulus than the lower part (page 111). (ii) The selection of a low value for the elastic modulus in order to make allowance for cracking or to make provision for poor construction can result in an excessive value for the computed strain (Figure 2.4 on page 15 shows high strains for low moduli). This may falsely indicate an overstrained condition in the cement-treated materials. These two factors, that is the construction variations and the effect of a reduced elastic modulus, should be fully appreciated when the elastic modulus of a cement-treated material is selected.

8.2.4.4 Foundation and untreated soils

In order to do the design the engineer must obtain information on the elastic modulus of the foundation. The latter was defined (see section 2.2.3, page 14) as the combination of the layers below the cement-treated layer and it represents the overall effectiveness of all the lower layers (usually the selected subgrade and subgrade) in supporting the cement-treated layer. In section 2.2.3 it was suggested that although the elastic modulus of the foundation may be assumed for the initial design, it should be verified during the construction stage by performing either a plate bearing or a Benkelman beam test on the completed foundation. If this fairly involved but accurate method is deemed unnecessary, the elastic modulus of the untreated foundation materials may be estimated from the approximate relationship suggested by Heukelom and Klomp (1962). They utilized dynamic testing and proposed the relationship between the elastic modulus and the CBR of untreated subgrade and subbase quality materials as

$$E = k \cdot CBR \quad \text{.......................................} \quad (8.10)$$

The elastic modulus is expressed in MPa while k varies between 5 and 20, with an average around 10. This implies that the approximate elastic modulus of soils is 10 times the CBR value.

A more recent and apparently more accurate suggestion to obtain the elastic modulus of soils was made by Kirwan and Snaith (1976). They have proposed that the elastic (or resilient) modulus of partially saturated cohesive subgrade soils depends on two parameters, the relative compaction (R_c) and the relative moisture content (R_w). R_c was defined as the ratio between the in-situ dry density and the modified AASHO maximum dry density, and R_w as the ratio between the in-situ moisture content and the modified AASHO optimum moisture content. The elastic modulus of the soil at a 20 kPa deviator stress is then obtained from a chart based on R_c and R_w. They have verified this procedure for only one soil, but it does seem to hold promise for the future.

The Poisson ratio is usually not critical to the analysis of the upper layers, and 0,35 is suggested for general use.

8.2.5 Safe working stress and/or strain of materials

8.2.5.1 Bituminous material

The allowable tensile strain in these materials has been measured by various people and summarized by Freeme and Marais (1973). The safe working strain of bituminous mixes, reproduced in Table 8.2 (NITRR, 1977) is not a constant value for a certain number of load repetitions but it is affected by various physical factors, such as the stiffness and void content. The tensile strain at the bottom of the bituminous surfacing of a pavement containing a cement-treated base is usually rather low and not near to the critical level. It is suggested that the values reported in Table 8.2 be adopted for the design of thin surfacings.

TABLE 8.2 : Maximum permissible tensile microstrain in thin (less than 50 mm) bituminous surfacings (from NITRR, 1977)

NUMBER OF LOAD REPETITIONS (T)	GAP-GRADED BITUMINOUS MATERIAL			ASPHALTIC CONCRETE		
	Stiffness (MPa)			Voids in mix (%)		
	2 400	5 000	8 000	2	5	9
10^3	1 000	840	810	1 060	970	880
10^4	720	610	570	700	620	580
10^5	510	430	400	440	410	360
10^6	350	300	280	290	270	240
10^7	250	210	200	190	175	150
10^8	180	150	140	120	115	100

8.2.5.2 Cement-treated materials

The allowable tensile strain (ε) in cement-treated materials depends on the strain at break (ε_b) of the material and the number of load repetitions (N_f) (equation (2.3), page 31). The strain at break (ε_b) of the cement-treated material intended for use in the pavement should be determined by performing bending tests on beam samples (section 2.2.8(c)). If the bending test cannot be performed, or in the case of only a preliminary design, it is suggested that an approximate method be used. Figures 8.4 and 8.5 summarize information on strain at break (ε_b) accumulated during the course of this research project. It is suggested that these values be used for the approximation.

The accumulated data was fitted to a number of non-linear functions using least squares and the function with the smallest residual sum of squares was selected for representation of the data. (The goodness of fit

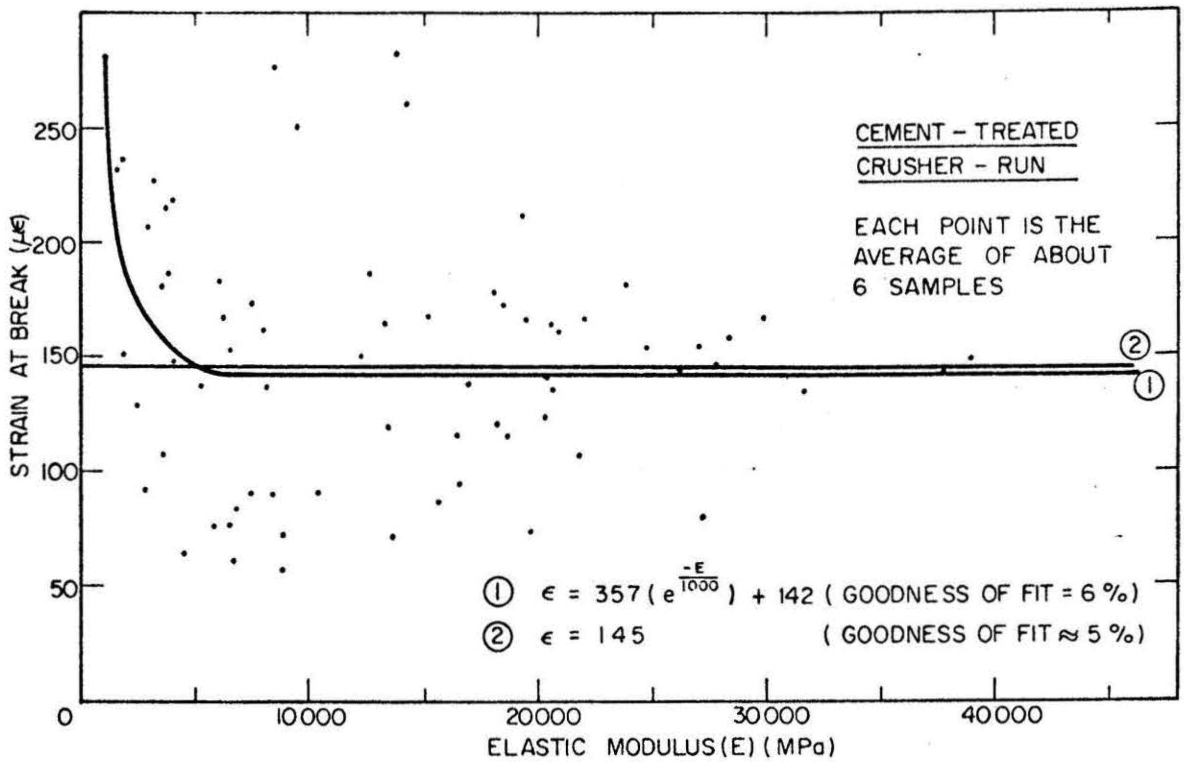


FIGURE 8-4

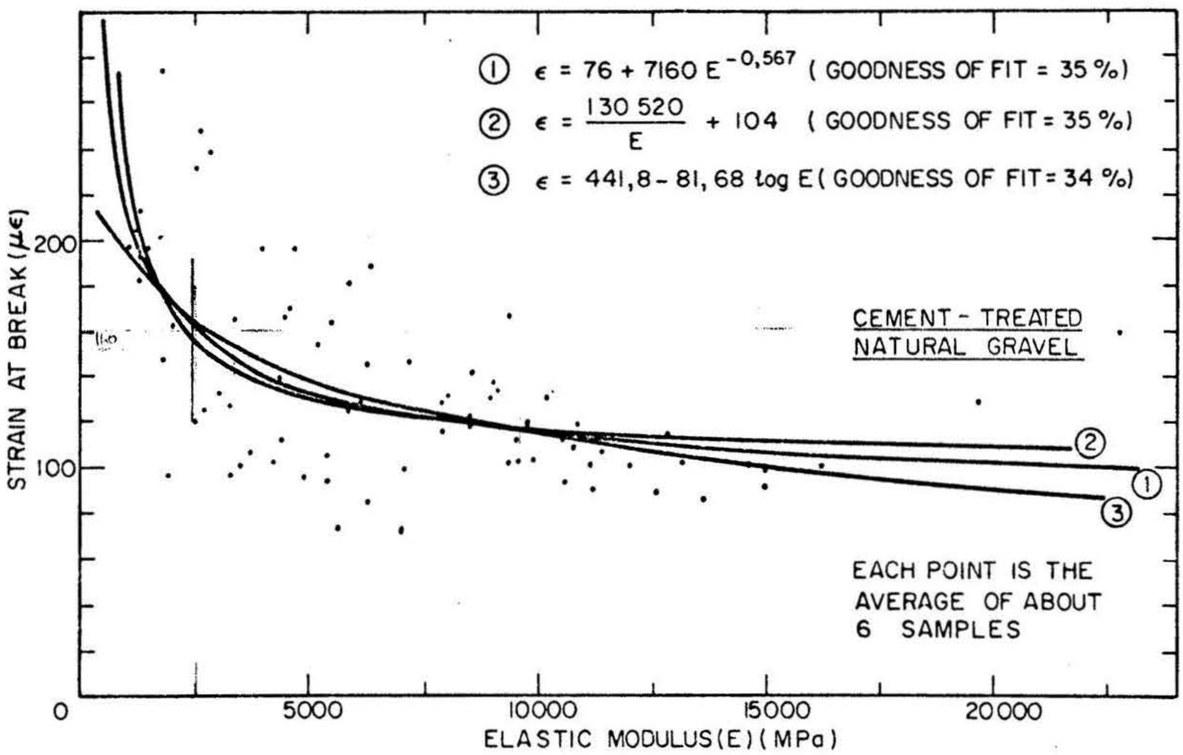


FIGURE 8-5

RELATIONSHIP OF ELASTIC MODULUS AND STRAIN AT BREAK FOR CEMENT-TREATED MATERIALS.

is a statistical concept denoting the accuracy of the fitted curve, and if the curve fits the data perfectly the goodness of fit is 100 per cent.) The amount of scatter in the data of the cement-treated crusher-run (Figure 8.4) is significant and the curve that fits this data the best has a goodness of fit of only 6 per cent. It is therefore suggested that the average value (145 $\mu\varepsilon$) should be used. The strain at break of the cement-treated natural gravel (Figure 8.5) increases with a decrease in elastic modulus and at least three lines with a goodness of fit of about 35 per cent are possible. It should therefore be appreciated that estimating the strain at break for cement-treated natural gravel from the elastic modulus is not very accurate.

It may be pertinent to mention here that average values of varying parameters have generally been accepted in this thesis. The introduction of statistically determined alternative values with confidence limits other than 50 per cent have purposely been avoided because of the relatively small number of results at this stage. Working on average values does not restrict research workers in future to use other confidence limits since the data is available and any confidence limit can be calculated and used once the problem of a total probabilistic approach to pavement design has been solved. In the meantime the author chose to work on average values since (i) this ties in with practical experience of pavement design because if a higher confidence limit is used it will result in an unnecessarily conservative approach and hence very thick cement-treated layers, and (ii) it is the approach that has been adopted by all the other workers in the field such as Shell, the Asphalt Institute, Portland Cement Association and various overseas universities (Dormon and Metcalf, 1965; PCA, 1973; Hadley et al, 1972; Mitchell et al, 1974; Pell and Brown, 1972; and Freeme and Marais, 1973) while the statistical aspects of material variation and the probabilistic approach to pavement design have been investigated by various other researchers (Hudson et al, 1974; McCullough, 1976; Darter, 1976; and Moavenzadeh, 1976).

The number of load repetitions can be determined from traffic surveys (Basson et al, 1972) and if reliable estimates can be made on the traffic in each axle load group, the variation to be suggested in section 8.3.1 should be followed. If the breakdown into the various axle groups cannot be obtained, but the total number of repetitions (N_f) by the heavy wheel load (Table 8.1) can be estimated, equation (2.3) should be used to calculate the corresponding strain ratio ($\varepsilon/\varepsilon_b$). Thereafter the allowable tensile strain (ε) is calculated from the strain ratio ($\varepsilon/\varepsilon_b$) and the strain at break (ε_b).

8.2.5.3 Subgrade materials

Limited information is available on the safe working values in the subgrade materials. Dormon and Metcalf (1965) have published criteria (Table 8.3) which they calculated from the results of the AASHO Road Test built in Illinois, USA. Van Vuuren (1972b) did an evaluation of their allowable vertical strains in the subgrade for South African conditions, and since they agree (Figure 8.6), it is suggested that these values be used in South Africa at present. Although they have been suggested some time ago (1965) these values may still be used while the deformation subsystems are being developed further (Monismith et al, 1977; Brown and Bell, 1977; and Kirwan et al, 1977).

It usually happens that the computed vertical compressive strains in the subgrade are much less than the values reported in Table 8.3 and Figure 8.6, and this indicates that ruts deeper than 20 mm are not likely to occur. This agrees with practical observations on South African pavements.

TABLE 8.3 : Permissible maximum vertical compressive strain in subgrade soils (from Dormon and Metcalf, 1965)

NUMBER OF EQUIVALENT REPETITIONS (E80)	PERMISSIBLE VERTICAL COMPRESSIVE STRAIN ($\mu\varepsilon$)
1.10^5	1 000
3.10^5	830
1.10^6	650
3.10^6	570
1.10^7	460
over 10^7	400

8.2.5.4 Untreated granular materials

These materials have a very limited, if any, tensile strength, and this value can hardly be measured and relied on in pavement design. If an untreated crusher-run base is used on top of a cement-treated layer, the tensile stresses in the crusher-run base are usually very low (Otte and Monismith, 1976), resulting in a very limited possibility of the base being overstressed. If high tensile stresses do develop it is suggested that the design criterion proposed by Pell and Brown (1972) be applied which states that the tensile stress should not exceed 0,5 times the vertical compressive stress. This design criterion is currently under scrutiny by the National Institute for Transport and Road Research (NITRR) of the CSIR and it is expected that an improved and more versatile criterion will be developed during 1977 and 1978.

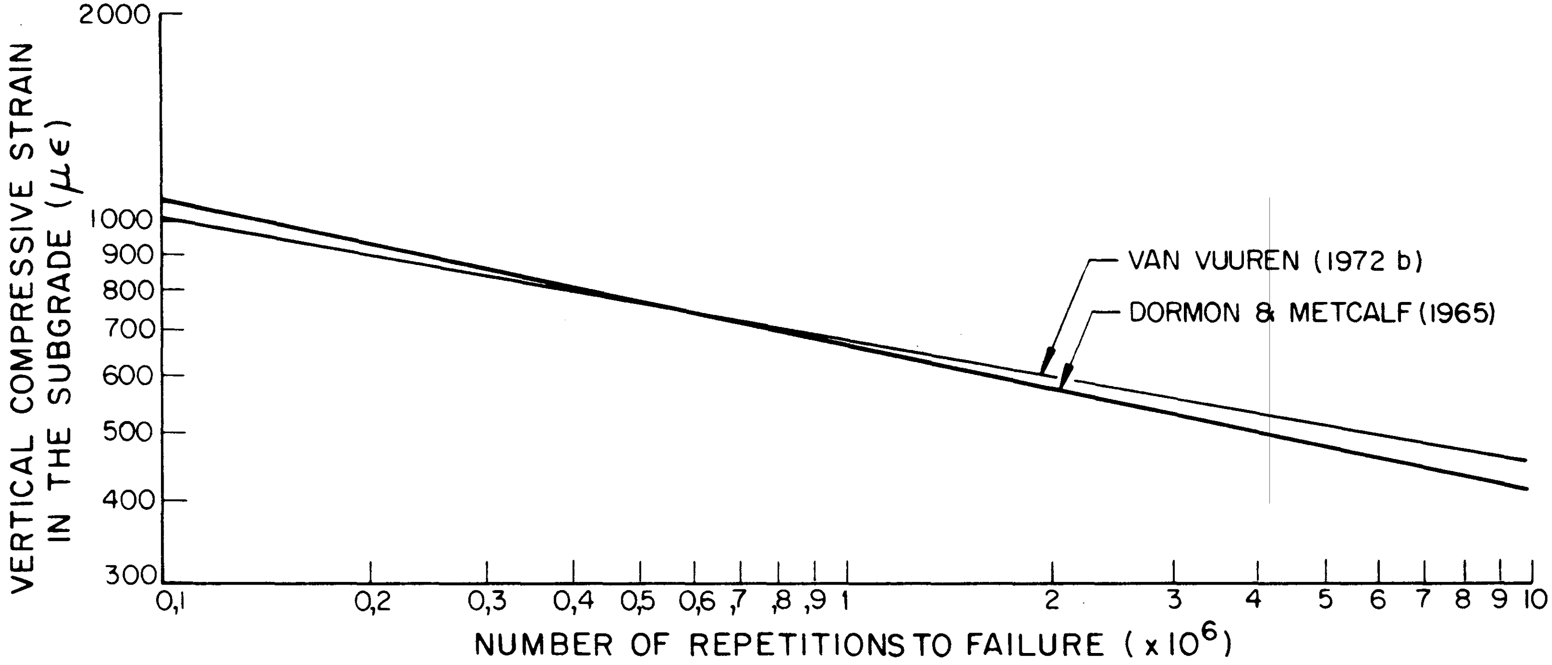


FIGURE 8-6

SAFE WORKING STRAIN IN THE SUBGRADE FOR 10 mm PERMANENT DEFORMATION.

8.2.6 Effect of environment

In Chapter 5 it was shown that thermal stresses are important in a continuous and uncracked cement-treated layer. Once cracking has occurred movement can take place at the cracks and the thermal stresses will be significantly reduced. It was shown that the use of an untreated crusher-run layer as a thermal insulator may, however, have some advantages in reducing the thermal stresses in cement-treated materials.

The effect of changes in moisture content and soil suction, that is negative pore-water pressure, has a considerable influence on the performance of untreated materials in pavements. In arid and semi-arid areas soil suction is generally high and this can increase the load-bearing ability of fine-grained subgrade soils significantly. Richards (1973) investigated the effect of changes in soil suction on the load-bearing ability of subgrade soils and proposed a non-linear finite element model to study the effect of transient moisture conditions in pavements (Richards, 1974). Richards and Pappin (1977) then applied these principles to the behaviour of a particular experimental road in New South Wales, Australia which was constructed during 1973, but failed very soon after completion. They indicated how abnormally heavy rainfall affected the predicted moisture conditions, and hence the elastic properties of the subgrade soils, and that the pavement failed because it was underdesigned for the moisture conditions which actually existed. It is considered that this investigation really indicates how soil suction in subgrades can be utilized and applied in pavement design and it is recommended that it be used to obtain the elastic properties of untreated materials.

The influence and interaction of all the various environmental factors on the performance of pavements are however still being studied and no firm conclusions can be drawn. At this stage only limited suggestions can be made which include an untreated crusher-run base to provide some thermal insulation to the cement-treated layer and proper drainage to prevent softening of the lower layers. Frost heave and thawing are fortunately not applicable to pavements in Southern Africa and need therefore not be considered.

8.2.7 Selection of a practical combination of materials and layer thicknesses

A practical combination for a certain area is to a large extent governed by available funds and existing practice and experience. It has been shown in Chapter 2 that it is desirable to (i) build a thick cement-treated layer, (ii) support it on a relatively stiff foundation, and (iii) to cover it with another layer to dampen reflective cracking and provide some thermal insulation. It is suggested that these guidelines be followed.

Suggested layouts for pavements with cement-treated layers appear in Figure 8.7. These layouts are given for several traffic categories which were defined in terms of E80, although it is the heavy and overloaded traffic that does the damage to cement-treated materials. The definition of these categories, which was based on practical experience and the nomenclature of TRH 4 (NITRR, 1977), considered the extent of overloading and heavy wheel loads which normally occur in the traffic streams of the given intensities, but the increasing load equivalency factors described in section 2.2.7 (page 22) and observed by Paterson (1977) were not incorporated directly when the E80 were calculated. From practical experience gained by the use of these designs it is expected that the proposed layouts will carry the traffic for about 10 to 15 years without requiring major maintenance. These suggestions are also very heavily dependent on construction practice which includes the thicknesses to which pavement layers are usually constructed by the various road-building authorities, for example 100, 125 or 150 mm. A difference in construction practice may therefore dictate slight changes in the proposed layer thicknesses.

8.2.8 Maximum developed stresses and strains in the materials

A multi-layered elastic system is assumed and this implies that -

- the materials are linear-elastic, homogeneous and isotropic;
- inter-layer continuity exists between the various layers;
- the loading conditions are axisymmetric;
- the load is applied as a uniformly distributed pressure over a circular area;
- in the horizontal direction all the layers are infinite in extent, and
- in the vertical direction the bottom layer is semi-infinite but the upper layers have uniform and finite thicknesses.

A review of the historical development in layered elastic theory was prepared by Nielsen (1970). He very aptly summarized the mathematical and numerical procedures used by those who have worked on the development of the theory since 1943. Layered elastic theory consists of several complex equations and to solve these several computer programs had to be prepared. Some of these are CHEVRON (Warren and Dieckmann, 1963), BISTRO (Peutz, Van Kempen and Jones, 1968), the recent update of BISTRO called BIZAR, ELSYM5 (Ahlborn, 1973), PSLAY-1, CHEV4 (NITRR, 1977a) and a number of finite element programs. Any one of these can be used, but the author prefers CHEVRON because it is fast and easy to use and interpret.

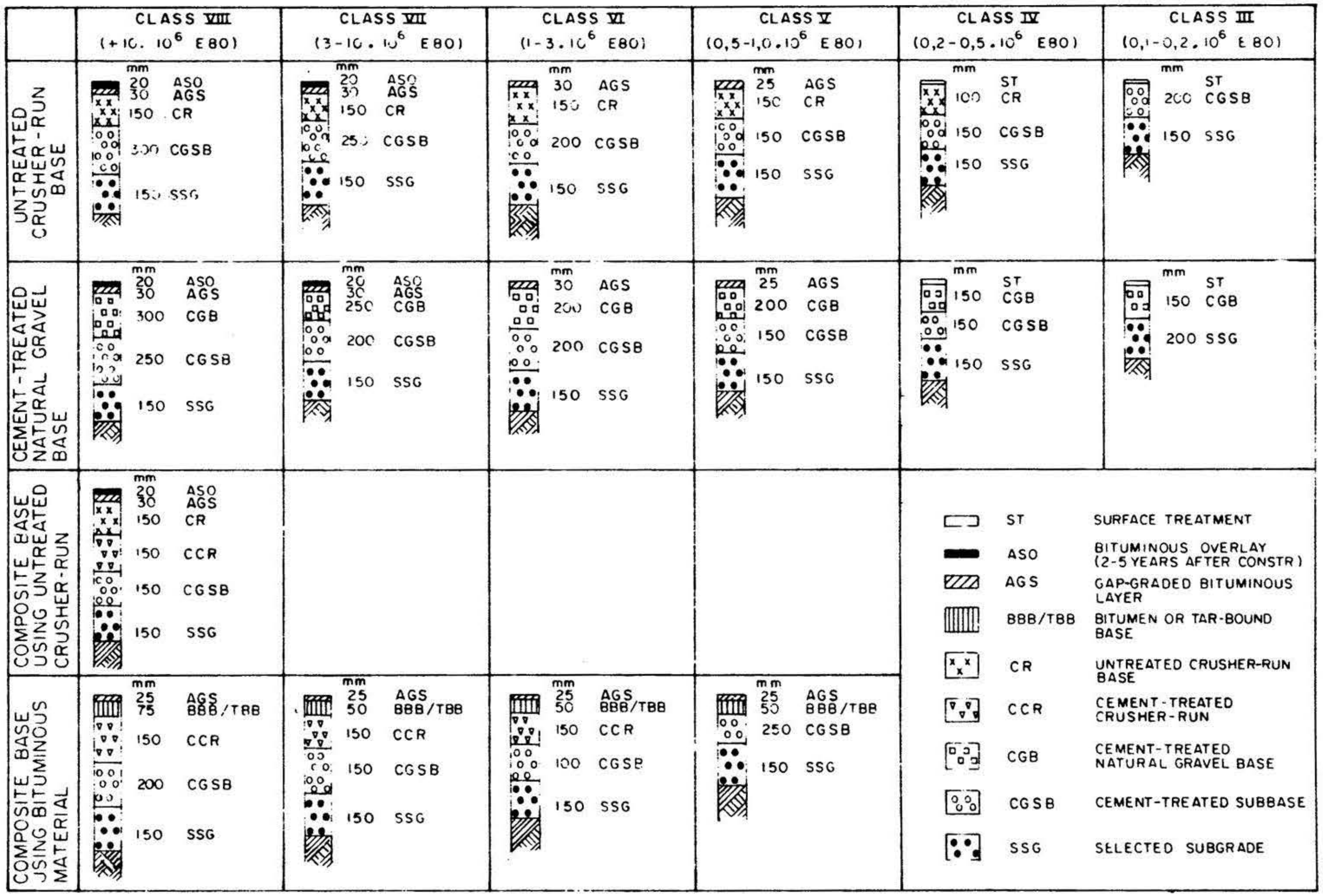


FIGURE 8-7

SUGGESTED LAYOUTS FOR CEMENT-TREATED LAYERS IN PAVEMENTS

To calculate the developed stresses and strains in the pavement materials, it is necessary to have -

(a) the design wheel load and tyre contact pressure;

(b) the elastic modulus and Poisson ratio of the materials intended to be used, and

(c) the proposed layer thicknesses.

The computer program calculates the vertical, radial and tangential stresses and strains, and the deflections. These values can be obtained at any predetermined horizontal distance from the centre of the wheel load and at any depth in the pavement, but are usually only requested at the interface between the various layers.

The heavy wheel loads are usually carried on dual wheels. Pretorius (1970) used the superposition principle and calculated that the maximum stress in a cement-treated base occurs in the centre between the two wheels of a dual wheel. In the proposed procedure it is therefore assumed that the heavy wheel loads are acting on single wheels and that the maximum stress and strain in the cement-treated base will develop under the centre of this single wheel. Thus a single load program, such as CHEVRON or CHEV4, is acceptable for this design procedure.

Since it seems virtually impossible to prevent the development of initial cracking in cement-treated materials, the design engineer must make provision for this phenomenon while doing the structural analysis. In Chapter 4 (Table 4.5, page 78) it was shown that the crack resulted in an increase in horizontal tensile stress and the maximum stress occurs at the bottom of the cracked layer and acts parallel to the initial crack. The extent of the stress increase seems to be dependent on the width of the cracks and the thickness of the cement-treated layers. If wide cracks are expected, that is cracks wider than 2 mm, and the total thickness of cement-treated material is expected to be 200 mm or less, the suggested increase is 1,25 times, but if the thickness is more than 200 mm the suggested increase is 1,40 times. If moderate cracks are expected, that is from hairline to 2 mm, the suggested increases are only 1,1 and 1,20 times respectively. If no cracking is expected to occur, the calculated stresses and strains need not be increased, but this is not recommended since a properly treated and well-constructed cement-treated layer should show some cracking (Chapter 3). If the base is of a material that will blanket reflection of cracks to the surface (for example crusher-run) it is suggested that the category of moderate cracking applies. The increase in the vertical compressive strain in the layer below the cracked treated layer is considerable - up to 14 times. The magnitude of the increase in vertical strain shown here should not be regarded as very accurate, but it may be used as an interim guide to warn the designer against possible deformation in the subgrade. The values calculated in the CHEVRON analysis are increased according to Table 4.5 (page 78) and used as design developed values. By adopting and applying these increases in stress and strain the use of the expensive prismatic solid finite element analysis can be avoided.

To design the cement-treated and untreated materials the computer analysis is performed using the lower of the two suggested elastic moduli for the bituminous material, namely 2 000 or 5 000 MPa depending on whether it is a gap-graded mix or an asphaltic concrete. The analysis should however be repeated while using the higher of the two suggested moduli (6 000 or 12 000 MPa) to ensure that the bituminous material will not be overstrained during the winter.

8.2.9 Comparison of maximum developed values with safe working values

The requirements of a structurally well-designed pavement, called a balanced design, were outlined in Chapter 2. The design requires the developed stresses and strains in every material, and at every point, to be less than the safe working values and if this is so the material may be considered safe against overstressing or overstraining.

In pavements with cement-treated layers it is usually found that the bituminous surfacing and the subgrade are safe against overstraining because the developed strains in these layers are less than the safe working values. The developed strain in the cement-treated layers is usually the critical value. This implies that the whole pavement structure will be safe if the developed strains in the cement-treated layers are less than the allowable strains.

If the developed strain nowhere exceeds the allowable safe working values, and the designer is sure that the proposed layout will result in the most economical design that he can produce under the prevailing conditions, he may accept the design. If the developed values are more than the safe working values, adjustments are required in the proposed layer combinations and/or thicknesses and/or material properties. After these have been made the calculations are repeated. The layout must continously be revised until it is satisfactory.

8.2.10 The post-crack phase

In pavements with cement-treated layers the calculations often show that the cement-treated material will experience traffic-associated cracking before the road has carried the design traffic. As stated in section 8.2.1 this can be perfectly acceptable provided that the pavement will not experience an unacceptable loss in riding quality as a result of the cracking before it has carried the design traffic. If it is accepted that the treated material will crack, the analysis for the post-crack phase should also be done but a reduced elastic modulus should be used for the now cracked cement-treated layer. To accurately model the cracked material produces difficulties since the effective elastic modulus in the vertical direction is virtually unaffected by the cracks, but in the horizontal direction it is significantly reduced because of the loss of cementation across the crack. The cracked cement-treated layer is therefore anisotropic and cannot be handled by CHEVRON. To overcome this complication it is suggested that finite element analysis should be used since it can handle anisotropic materials. If the appropriate finite element computer programs are not available it is suggested that the cracked cement-treated layer should be assumed as an equivalent, untreated, isotropic material with an elastic modulus of between 1 500 and 500 MPa (depending on the original material, the site and the structural layout) and the analysis should be repeated by using CHEVRON. This analysis will show increased strains in the other layers of the pavement (especially the lower layers) and they should be designe to withstand the increased values.

Pavements with one treated layer need to be analyzed only twice; once before the treated layer has cracked and once after it has experienced traffic-associated cracking. The tensile strain (ε) calculated at the bottom of the cement-treated layer (before it has experienced traffic-associated cracking) is used to calculate the traffic (N_1) (from equation 2.5, page 31) that will produce traffic-associated cracking in the cement-treated material. This value (N_1) is subtracted from the total design traffic (N_d) to obtain the remainder of the traffic (N_d-N_1) that must still be carried during the post-crack phase.

The second analysis is performed on the layout after assuming that the treated layer has cracked and with the elastic modulus of the treated layer reduced to that of an equivalent untreated material. The outcome of this analysis is used to verify that the pavement can carry the remainder of the traffic (N_d-N_1) without its riding quality being reduced to an unacceptably low limit.

In pavements with more than one treated layer the traffic (N_1) required to produce traffic-associated cracking in the lower treated layer is calculated from equation (2.5) (page 31). This layer's elastic modulus is then reduced to that of an equivalent untreated material and the traffic (N_2) to produce traffic-associated cracking in the treated layer above it is calculated (also from equation (2.5)). This process is repeated, if necessary, until N_1+N_2..... equals or exceeds the design traffic (N_d). (See example 5, page 195.)

8.2.11 Crack-propagation phase

The crack-propagation phase is the period of time that it takes the traffic-associated crack to propagate from the bottom of the layer upwards through the treated material. (This phase should not be confused with the growth and interconnection of microcracks in the matrix of the cement-treated material, page 30.) At present it is very difficult to calculate the length of the crack-propagation phase in cement-treated materials with any degree of accuracy, but since it is a brittle material, the crack-propagation time may be taken as infinitely short, and equated to zero. This is why in this thesis (section 3.2.2) the author considers cement-treated materials to be either (i) intact, but with the initial crack, or (ii) cracked because of traffic and hence as an equivalent untreated material. Neglecting the vertical crack-propagation time is on the conservative side, but it is considered to be only a very small proportion of the total design life.

To calculate the time for the horizontal crack-propagation and the development of the ladder-type cracking pattern (Pretorius, 1970 and Figure 3.4, page 49) is very difficult and complex. It can currently not be included in the structural design procedure as considerable further research work on failure criteria is necessary before it can be included with any degree of accuracy.

8.3 VARIATIONS IN DESIGN PROCEDURE

8.3.1 Accommodating mixed traffic

The procedure as suggested in section 8.2 makes provision for a certain number of repetitions by the heaviest wheel load expected on the pavement. It is very often possible to perform a traffic survey which will provide reliable information on the estimated number of repetitions (n) in each axle load group, for example with the AWA (Axle Weight Analyser) (Basson et al, 1972). If such a breakdown can be achieved a more accurate design may be possible.

The accommodation of mixed traffic can be handled in several ways:

One method is to perform a computer analysis for each axle load group with a corresponding mean tyre contact pressure. This method involves rather elaborate and expensive computer analyses.

An alternative method is to perform only one analysis for the 40 kN wheel load and the strains for the remaining groups are then computed by multiplying with a load factor for each group. These load factors unfortunately depend not only on the tyre pressure but also the tyre contact area. If a constant tyre pressure is assumed for all the axle load groups (varying contact area) the load factors are not only dependent on the load but also the structural layout. If a constant contact area is assumed (varying contact pressure) the load factors are independent of the structural layout and simply vary directly with the axle load. In practice as the axle load groups change so do the tyre pressures (contact pressure) as well as the contact area and the exercise becomes quite complex.

Notwithstanding the rather elaborate and expensive computer analysis of the first method the author still favours this approach and each axle load group should be analysed with a selected contact pressure and contact area. Even here an average condition will have to be accepted for each group since the relationship between load, contact area and tyre pressure varies from tyre to tyre and road to road (Van Vuuren, 1974).

Thus, although complex, it is possible to calculate an acceptable strain (ε) in the pavement layout for each axle load group. Depending on the strain at break (ε_b) of the material, the strain ratio ($\varepsilon/\varepsilon_b$) can be calculated for each axle loading and from equation (2.5) the corresponding allowable number of repetitions to failure (N_f). To determine whether fatigue failure is likely to occur as a result of cumulative damage, Miner's hypothesis (Miner, 1945) should be applied. This hypothesis (equation (8.11)) states that if the sum of the ratios between the estimated (or actual) (n) and the allowable (N_f) number of load repetitions in each axle load group is less than unity (1), the material is not likely to fail in fatigue.

$$\Sigma\left(\frac{n}{N_f}\right) < 1 \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (8.11)$$

If the sum of the ratios exceeds 1 the layout should be changed to reduce the calculated strain and hence increase the allowable number of repetitions (N_f), or alternatively the number of estimated (or actual) repetititons (n) must be reduced. The applicability of Miner's hypothesis to concrete was also assumed when the PCA developed their concrete pavement design procedure (PCA, 1973).

8.3.2 Use of standard designs

To apply the proposed design procedure in practice the design engineers must all have information and experience concerning the relevant material properties and computer programs. This can lead to unnecessary duplication of facilities. To overcome this it might be desirable for the road authorities to adopt a few standard designs, to do the necessary calculations on these and to publish the information in the form of standard tables or graphs. The suggestions given in Figure 8.7 were recently expanded and included as part of the catalogue of acceptable designs in the South African pavement design manual (NITRR, 1977).

Virtually all the layouts illustrated in Figure 8.7 accept that traffic-associated cracking will occur before the pavement has carried the full design traffic. In Figure 8.8 the layouts have been made slightly thicker and the construction costs are thus between 5 and 15 per cent more expensive than those in Figure 8.7. It was calculated however that traffic-associated cracking is far less likely to occur in these layouts before they have carried the full design traffic (Otte, 1977). The occurrence of traffic-associated cracking is very dependent on the assumed traffic spectra, construction variations, and material properties and it is therefore possible to predict traffic-associated cracking even in some of the layouts suggested in Figure 8.8. The decision on whether to accept a layout from Figure 8.7 or Figure 8.8 should be made by either the designer or the client and depends on the conditions of the particular project, e.g. prestige of project, environmental conditions, traffic volume, etc. When the greater surety and increased factor of safety is compared with the relatively small increase in the construction cost it may be worthwhile to accept the more conservative layouts suggested in Figure 8.8.

8.4 VERIFICATION AND ACCURACY OF PROCEDURE

8.4.1 Pavement response

It is a basic assumption that the proposed design procedure is able to predict a pavement's response, which has been defined as the stresses, strains and deflections developed in the pavement by the action of a load on the surface. To verify this assumption Wang (1968) and Fossberg (1970) built instrumented test sections and performed plate bearing tests on them.

	CLASS VIII ($+10 \ 10^6$ E80)	CLASS VII ($3-10 \cdot 10^6$ E80)	CLASS VI ($1-3 \cdot 10^6$ E80)	CLASS V ($0,5-1,0 \cdot 10^6$ E80)	CLASS IV ($0,2-0,5 \cdot 10^6$ E80)	CLASS III ($0,1-0,2 \cdot 10^6$ E80)
UNTREATED CRUSHER-RUN BASE	mm 20 ASO 30 AGS 150 CR 425 CGSB 150 SSG	mm 20 ASO 30 AGS 150 CR 400 CGSB 150 SSG	mm 30 AGS 150 CR 350 CGSB 150 SSG	mm 25 AGS 150 CR 300 CGSB 150 SSG	mm ST 100 CR 250 CGSB 150 SSG	mm ST 150 CR 200 CGSB 150 SSG
CEMENT-TREATED NATURAL GRAVEL BASE	mm 20 ASO 30 AGS 300 CGB 250 CGSB 150 SSG	mm 20 ASO 30 AGS 250 CGB 250 CGSB 150 SSG	mm 30 AGS 250 CGB 200 CGSB 150 SSG	mm 25 AGS 200 CGB 200 CGSB 150 SSG	mm ST 200 CGB 150 CGSB 150 SSG	mm ST 150 CGB 200 CSSB 150 SSG
COMPOSITE BASE USING UNTREATED CRUSHER-RUN	mm 20 ASO 30 AGS 150 CR 150 CCR 250 CGSB 150 SSG	mm 20 ASO 30 AGS 150 CR 150 CCR 200 CGSB 150 SSG	mm 30 AGS 150 CR 150 CCR 150 CGSB 150 SSG			
COMPOSITE BASE USING BITUMINOUS MATERIAL	mm 25 AGS 75 BBB/TBB 150 CCR 250 CGSB 150 SSG	mm 25 AGS 50 BBB/TBB 150 CCR 225 CGSB 150 SSG	mm 25 AGS 50 BBB/TBB 150 CCR 150 CGSB 150 SSG	mm 25 AGS 50 BBB/TBB 150 CCR 125 CGSB 150 SSG		

ST	SURFACE TREATMENT
ASO	BITUMINOUS OVERLAY (2-5 YEARS AFTER CONSTR)
AGS	GAP-GRADED BITUMINOUS LAYER
BBB/TBB	BITUMEN OR TAR-BOUND BASE
CR	UNTREATED CRUSHER-RUN BASE
CCR	CEMENT-TREATED CRUSHER - RUN
CGB	CEMENT-TREATED NATURAL GRAVEL BASE
CGSB	CEMENT-TREATED SUBBASE
SSG	SELECTED SUBGRADE

FIGURE 8-8

CONSERVATIVE LAYOUTS FOR CEMENT-TREATED LAYERS IN PAVEMENTS.

They recorded the response of these sections and compared the measured deflections and strains with those calculated by both CHEVRON and a finite element analysis. Examples to show the excellent comparisons that they obtained are reproduced as Figures 8.9 to 8.12.

The author (Otte, 1973b) performed field studies on four pavements (A, B, C and D) with cement-treated bases to study the applicability of the proposed procedure to the prediction of surface deflection for a pavement. No strainmeters or pressure cells had been installed in the pavements. On each pavement the Benkelman beam deflection was obtained from at least 9 measurements which were equally spaced within a rectangle of about 300 x 300 mm. A diamond saw was then used to cut a block sample (600 x 600 mm) of the surfacing and cement-treated base at the exact position at which the Benkelman beam tests were performed. This sample was taken to the laboratory and sawn into at least 6 beam samples to determine the elastic modulus in bending according to the procedure outlined in section 2.2.8(c) (page 25). The average layer thicknesses of the surfacing and base were also recorded. After the 600 x 600 mm block sample had been removed a plate bearing test (Otte, 1973) was performed on top of the layer directly beneath the cement-treated base to calculate the elastic modulus of the foundation (section 2.2.3, page 14). The elastic modulus of the surfacing at 20 ^{o}C was taken as 3 000 MPa for gap-graded mixes and as 6 000 for asphaltic concrete (Freeme and Marais, 1973). A constant Poisson ratio of 0,35 was used throughout the analyses. The various average elastic moduli and layer thicknesses for the four pavements are given in Table 8.4.

TABLE 8.4 : Elastic moduli and layer thicknesses of materials in four pavements

PAVEMENT	BITUMINOUS MATERIAL		CEMENT-TREATED BASE		FOUNDATION (MPa)
	Elastic modulus (MPa)	Thickness (mm)	Elastic modulus (MPa)	Thickness (mm)	
A	3 000	25	20 500	150	141
B	6 000	19	9 655	133	132
C	6 000	19	3 715	133	111
D	6 000	50	37 670	120	92

The values reported in Table 8.4 were used to calculate the surface deflection under a 40 kN wheel load (tyre contact pressure 430 kPa) and in Table 8.5 these are compared with the measured Benkelman beam deflections.

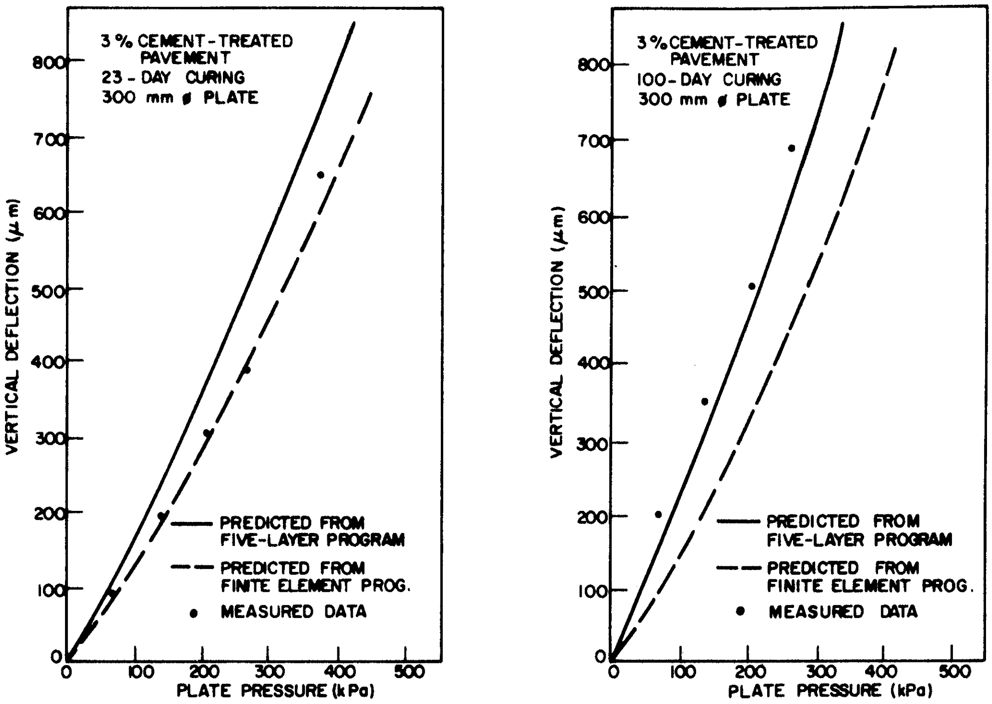


FIGURE 8-9

COMPARISON OF MEASURED AND PREDICTED VERTICAL DEFLECTION (AFTER WANG, 1968)

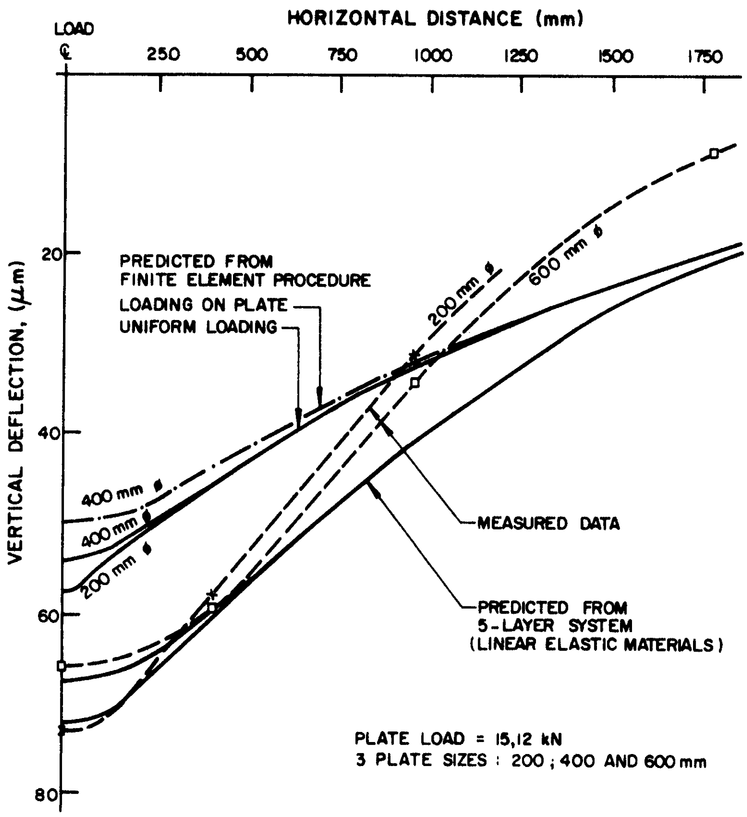


FIGURE 8-10

COMPARISON OF MEASURED AND PREDICTED VERTICAL DEFLECTION ON TOP OF A SOIL-CEMENT BASE (after Fossberg, 1970)

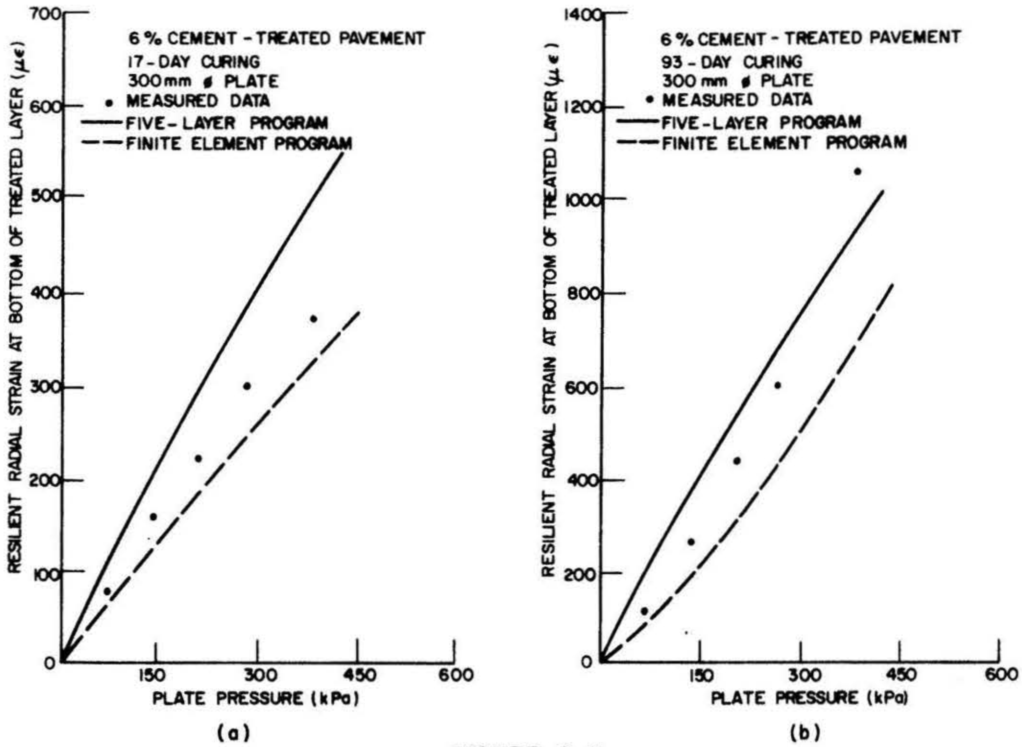


FIGURE 8-11

COMPARISON OF MEASURED AND PREDICTED STRAIN.
(AFTER WANG, 1968)

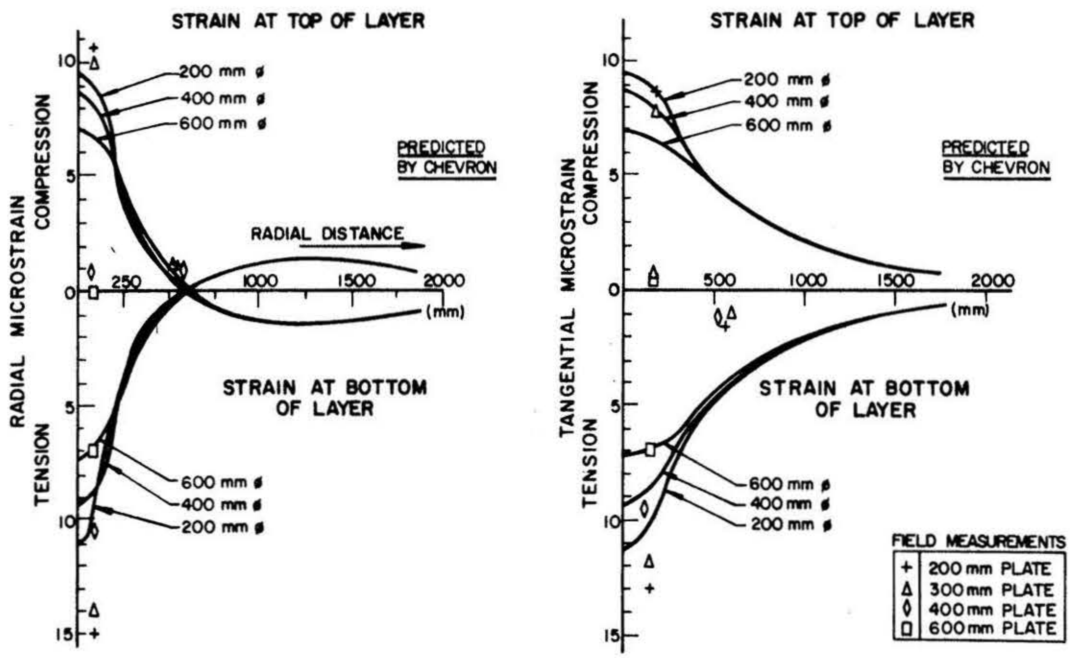


FIGURE 8-12

COMPARISON OF MEASURED AND PREDICTED STRAIN IN SOIL - CEMENT BASE.
(VERTICAL LOAD = 15,12 kN)
(AFTER FOSSBERG, 1970)

TABLE 8.5 : Comparison of calculated and measured deflections for four pavements

PAVEMENT	CALCULATED DEFLECTION (μm)	MEASURED DEFLECTION (μm) (Benkelman beam deflection)	DIFFERENCE (%)
A	189	243	-22
B	248	216	+15
C	342	260	+32
D	239	269	-11

The differences varied between +32 and -11 per cent. The 32 per cent difference is a little too high, but the other three are relatively low and perfectly acceptable for pavement design work.

From the field studies by Wang (1968), Fossberg (1970) and the author (Otte, 1073b), it may be concluded that the proposed design procedure is able to reasonably predict a pavement's response. Both the deflection and horizontal strains can be predicted to within about ±30 per cent and this is considered a reasonable accuracy.

8.4.2 Pavement behaviour

It is important that the proposed procedure should be able to predict pavement response, but engineers also require it to be able to predict pavement behaviour, which is defined as the change in the pavement's condition with time thoughout its design life. Throughout the pavement's design life the condition of the pavement should remain acceptable to both the engineer and the road user. For pavements with cement-treated bases this is usually achieved if traffic-associated cracking can be prevented because a severe drop in the serviceability of the pavement usually follows very closely on the onset of traffic-associated cracking (Otte, 1973a). Traffic-associated cracking and behaviour are therefore considered to be closely related.

To study whether the proposed procedure can predict pavement behaviour would require the accurate monitoring of several full-scale experiments throughout their design lives (Thompson et al, 1972; and Van Vuuren, 1972a). This is very time-consuming and often very unlikely to produce realistic results in practice. The author was fortunate in having a Heavy Vehicle Simulator (HVS) (Van Vuuren, 1972a and 1973) at his disposal to verify the prediction of pavement behaviour and several pavements with cement-treated bases were tested. The details of these tests were given in Chapter 7. In the following paragraphs the data of seven of these tests are analysed and it is indicated that the proposed procedure may be used to predict pavement behaviour.

The first application of the proposed design procedure to pavements with cement-treated bases is described in section 7.3.1. It was predicted that the base would be completely overstrained and it was bound to fail within one or two repetitions of the 65 kN wheel load because the calculated strain (ε) was about 300 $\mu\varepsilon$ and the strain at break (ε_b) of the material was expected to be only about 160 $\mu\varepsilon$. It was decided to subject the pavement to overloading and after 80 000 repetitions a block sample was removed and the cement-treated base was in fact severely cracked. Small blocks (100 x 100 mm) (Otte, 1972b) had formed which may be considered as the structural equivalent of untreated crusher-run.

On a second pavement (section 7.3.5(a)) the tensile strain (ε) in the cement-treated crusher-run layer under a 50 kN load was calculated to be about 170 $\mu\varepsilon$. The strain at break (ε_b) of the material was measured as only about 114 $\mu\varepsilon$. Since the strain induced by the wheel load exceeds the strain at break, it was expected that cracking would develop during the first load application and that visible and severe traffic-associated cracking would occur very quickly. After 220 repetitions of the test wheel over the 1 m wide test section, visible traffic-associated cracking developed in the cement-treated crusher-run layer and the test was ended.

It was prediced that a third pavement (section 7.3.3) would be able to withstand a virtually unlimited number of 40 kN wheel loads. Under a 75 kN wheel load (about double the legal wheel load) the tensile strain was calculated as 65 $\mu\varepsilon$. The strain at break (ε_b) of the material was measured as 160 $\mu\varepsilon$ (Table 7.5, page 124). The strain ratio was 0,406 and according to the proposed procedure (equation (2.5), page 31), the pavement should have been able to withstand about 255 000 repetitions of the 75 kN wheel load before the onset of traffic-associated cracking, that is fatigue failure. To ensure that the HVS test could be completed within a reasonable time (three months), it was decided to subject the pavement to the 75 kN wheel load. After 280 000 repetitions the test was stopped and there were no signs of visible cracking but Table 7.5 indicates a statistically significant reduction in elastic modulus for the cement-treated crusher-run. This is probably because some traffic-associated cracking had occurred within the material.

The fourth example in which the proposed procedure was used to predict behaviour is detailed in section 7.3.6. In this test a saw-cut was made across the test section to simulate a very wide crack with no load transfer across it. After taking account of the proposed stress increase next to a crack (Chapter 4) the strain ratio was calculated as 0,39. According to equation (2.5) (page 31) the section should have been able to withstand about 350 000 repetitions of the 55 kN wheel load before traffic-associated cracking could be expected. The test was stopped after 184 000 repetitions and no cracking was visible at that stage. A significant amount of rain (about 142 mm) had fallen during the last 82 000 repetitions of the test and the deflections next to the crack had increased rapidly (Figure 7.14, page 145). This increase was probably caused by the rain-water softening the foundation, resulting in a reduction of its elastic modulus with a consequent increase in the strain developed next to the crack. The condition calculated next to the crack (tensile strain = 67 $\mu\varepsilon$) would therefore probably no longer have applied and traffic-associated cracking would have occurred much sooner than the predicted 350 000 repetitions. Four block samples sawn from the pavement at test points A, C and E after 184 000 repetitions have confirmed that traffic-associated cracking had started in the cement-treated crusher-run base (Figures 7.15 and 7.16, page 147). The cracks in the two blocks next to the saw cut (at point C) occurred as was predicted, namely perpendicular to the cut.

The fifth example applies to a pavement (Eerste Fabrieke, section 7.3.4) where three HVS tests were performed. The relationship between the recorded deflection and radius of curvature (for example, Figure 7.6) was utilized to predict the elastic moduli of the materials throughout the test. After completion of the HVS test block samples (600 x 600 mm) were sawn at eight of the fifteen points to measure the elastic modulus of the cement-treated crusher-run base. At seven of these points it was predicted that the cement-treated crusher-run base would be cracked, since the elastic modulus was expected to be only about 1 000 MPa, and this was confirmed by the amount of cracking observed in the material. At the eight point the elastic modulus was predicted to be between about 18 000 and 22 000 MPa and it was measured as 19 260 MPa.

When the results of the seven HVS tests described in the previous five paragraphs are summarized, they make very interesting reading. An early failure was predicted on two of the sites and severe traffic-associated cracking did occur soon after the trafficking was started. It was predicted that two of the other sections would withstand a fair amount of traffic, about 255 000 and 350 000 repetitions respectively, and when the HVS tests were stopped, after 280 000 and 184 000 repetitions respectively, no visible cracking had appeared on the surface. In the first case cracking had however commenced as evidenced by a significant drop in the material's elastic modulus, and in the other case visible cracks had developed at the bottom of the cement-treated layer. In the fifth case the elastic modulus and cracking were predicted with remarkable accuracy. The excellent agreement between theory and practice in all the examples quoted is considered as sufficient proof of the ability of the proposed procedure to predict traffic-associated cracking and hence pavement behaviour.

8.5 EXAMPLES

8.5.1 Comparison with State of California procedure

The data from an example in Part 7 of the State of California planning manual is utilized to explain the application of the procedure suggested in Figure 8.1. A four-lane divided highway has to be designed on a subgrade with an R-value of 10 and the expected traffic index (TI) is 8,0. A traffic index (TI) of 8,0 corresponds to about 4,4 million equivalent 22,2 kN (5 000 lb) wheel loads. The California procedure requires a gravel equivalent of 707 mm and the suggested layout is -

75 mm asphaltic concrete,
180 mm Class A cement-treated base
240 mm Class 2 subbase (R-value of subbase = 50).

The suggested procedure (Figure 8.1) is as follows:

Design traffic: A divided highway with TI=8 is not expected to carry overloaded vehicles and the maximum legal wheel load (40 kN) is suggested as the design wheel load. The corresponding tyre contact pressure is accepted as 520 kPa. Assume 0,5 million 40 kN wheel loads which is equivalent to about 4,4 million 22,2 kN wheel loads.

Elastic moduli and Poisson ratio (section 8.2.4):

- Asphaltic concrete: 5 000 MPa.
- Class A cement-treated base corresponds very closely to South African cement-treated crusher-run: elastic modulus = 20 000 MPa.
- Subbase: an R-value of 50 corresponds to a CBR of about 15 (PCA, 1973) and this corresponds to an elastic modulus of about 150 MPa.
- Subgrade: an R-value of 10 corresponds to a CBR of about 2 (PCA, 1973) and an elastic modulus of about 20 MPa.
- Poisson ratio for all materials equals 0,35.

Safe working strain values:

- Asphaltic concrete: ε_t = 200 $\mu\varepsilon$ (Table 8.2).
- Cement-treated crusher-run: ε_t = 54 $\mu\varepsilon$ (equation (2.5) and Figure 8.4) (n = $0{,}5.10^6$).
- Subbase and subgrade materials: ε_c = 800 $\mu\varepsilon$ (Table 8.3).

Effect of the environment: Provide adequate drainage and adjust elastic moduli if necessary.

Practical layout: As suggested by the State of California procedure.

Maximum strains: A CHEVRON analysis indicates (i) a horizontal compressive strain of 25 $\mu\varepsilon$ at the bottom of the asphaltic concrete, (ii) a horizontal tensile strain of 42 $\mu\varepsilon$ at the bottom of the cement-treated crusher-run, and (iii) vertical compressive strains of 95 $\mu\varepsilon$ and 227 $\mu\varepsilon$ at the top of the two layers underlying the cement-treated layer.

If the increases suggested in Table 4.5 (page 78) are applied, the design tensile strain in the cement-treated crusher-run becomes 42 x 1,25 = 52,5 $\mu\varepsilon$ and in the two underlying layers 95 x 5 = 475 $\mu\varepsilon$ and 227 x 2,5 = 568 $\mu\varepsilon$.

Developed and safe working values: The calculated and increased strain in the cement-treated layer is less than the safe working strain. The vertical compressive strain in the two untreated lower layers is less than the allowable 800 $\mu\varepsilon$ and no rutting or subgrade deformation is likely to develop.

Since the asphaltic concrete is placed on a rigid cement-treated layer it is in compression under the wheel load and no traffic-associated distress is expected in this material.

Acceptance: The layout suggested by the State of California procedure can therefore be accepted. Initial cracking may develop and reflect through the surfacing but since the strain ratio if 52,5/145 traffic-associated cracking in the cement-treated base is only expected after the pavement has carried about 0,6 million 80 kN axles, (the design traffic), that is one hundred 80 kN axles per day for 15 years.

8.5.2 Comparison with PCA procedure for industrial pavements

The PCA proposed a design procedure for pavements carrying heavy industrial vehicles (PCA, 1975). The worked example in the PCA manual in which a pavement is designed to carry heavy straddle carriers and forklifts is compared with the procedure proposed in Figure 8.1.

The design data are:

Subgrade modulus k = 27,1 MPa/m (= 100 pci).

Straddle carrier: (load A)

wheel load : 127 kN

tyre contact pressure : 983 kPa

number of repetitions (n_a) : 300 000

Forklift: (load B)

equivalent single wheel load : 160 kN

tyre contact pressure : 760 kPa

number of repetitions (n_b) : 25 000.

According to the PCA design procedure (PCA, 1975), 50 mm asphaltic concrete and 375 mm soil-cement will be sufficient to carry the load.

The author's proposed design procedure requires elastic moduli and the following values were chosen (section 8.2.4):

Asphaltic concrete: 5 000 MPa

Soil-cement : 9 000 MPa

Subgrade (CBR=3) : 30 MPa

The CHEVRON computer program calculated the tensile strains at the bottom of the soil-cement layer for loads A and B as 85 and 101 $\mu\varepsilon$ respectively. Figure 8.5 shows that the strain at break (ε_b) of a cement-treated material with an elastic modulus of 9 000 MPa is about 117 $\mu\varepsilon$ and this layer is therefore expected to carry only about 18 repetitions of load B (equation (2.5)) before traffic-associated cracking will occur. The pavement suggested by the PCA procedure is therefore likely not to be able to carry the expected traffic and the suspected reason is the very low elastic modulus (30 MPa) of the lean clay subgrade.

The design data were also used to calculate the thickness of concrete required should it be decided to construct a concrete pavement (PCA, 1973). Depending on the quality of the concrete and whether a cement-treated subbase is to be included, the required thickness of concrete varies between about 325 and 375 mm. The required thickness of concrete and soil-cement is therefore very much the same. This does not seem to be a realistic solution! The suggested layout of 50 mm asphaltic concrete and 375 mm soil-cement appears to be insufficient for the expected traffic loading - especially on such a weak subgrade. It is therefore recommended that the layout suggested by the PCA procedure should not be constructed.

The author's proposed procedure was applied to obtain an alternative design. The following layout is suggested:

50 mm asphaltic concrete
150 mm cement-treated crusher-run
300 mm cement-treated gravel base (soil-cement)
150 mm weakly cemented gravel subbase
300 mm natural gravel selected subgrade
in-situ subgrade.

The wheel loads and tyre pressures are as defined above. The expected elastic moduli (E) and Poisson ratios (μ) are -

Asphaltic concrete	: E = 5 000 MPa;	μ = 0,35
Cement-treated crusher-run	: E = 20 000 MPa;	μ = 0,35
Cement-treated gravel base	: E = 9 000 MPa;	μ = 0,35
Cement-treated gravel subbase	: E = 1 500 MPa;	μ = 0,35
Selected subgrade	: E = 300 MPa;	μ = 0,35
In-situ subgrade	: E = 30 MPa;	μ = 0,35

The allowable horizontal tensile strain in the asphaltic concrete is 400 $\mu\varepsilon$ (Table 8.2). According to equation (2.3) (page 31) the allowable strain ratios ($\varepsilon/\varepsilon_b$) for the cement-treated materials to withstand 300 000 and 25 000 load repetitions of loads A and B are 0,40 and 0,52 respectively. According to Figures 8.4 and 8.5 the tensile strain at break (ε_b) of the cement-treated crusher-run (E = 20 000 MPa), cement-treated gravel base (E = 9 000 MPa) and the weakly cemented subbase (E = 1 500 MPa) are 145, 117 and 190 $\mu\varepsilon$ respectively. Multiplying these values by the strain ratios of 0,40 and 0,52 for loads A and B the allowable tensile strain for the three layers are thus: 58, 47 and 76 $\mu\varepsilon$ for load A and 75, 61 and 99 $\mu\varepsilon$ for load B. The allowable vertical compressive strain in the selected subgrade and in-situ subgrade (Table 8.3) is 830 $\mu\varepsilon$.

The layout was analysed with the CHEVRON computer program for both loads A and B. The calculated horizontal strain in the asphaltic concrete and the vertical strains in the selected subgrade and in-situ subgrade are respectively -47, -93 and -193 $\mu\varepsilon$ for load A and -48, -108 and -222 $\mu\varepsilon$ for load B. When these values are compared with the allowable strain as calculated above, namely 400, 830 and 830 $\mu\varepsilon$ respectively, it appears that they are well below the allowable. These materials should therefore not show any signs of distress and the cement-treated layers would seem to be the critical ones, if any. The tensile strains calculated at the bottom of the three cemented-treated layers are respectively 15, 46 and 57 $\mu\varepsilon$ under load A and 9, 53 and 67 $\mu\varepsilon$ under load B. If it is assumed that wide initial cracks will develop only in the cement-treated crusher-run (Figure 3.1, page 40) and not in the lower layers, the calculated strain values in this layer, namely 15 and 9 με for loads A and B must be increased by 1,4 (Table 4.5) to become 21 and 13 με respectively. A comparison of the calculated and allowable tensile strains in the three cement-treated layers can be made as follows:

	LOAD A		LOAD B	
	Calculated	Allowable	Calculated	Allowable
Cement-treated crusher-run base:	21 με	57 με	13 με	75 με
Cement-treated gravel base :	46 με	47 με	53 με	61 με
Cement-treated gravel subbase :	57 με	76 με	67 με	99 με

The allowable tensile strains for each of the wheel loads are always greater than the calculated values. The number of repetitions to failure (N_f) (equation (2.5)) for the three cement-treated materials in the suggested layout, based on the actual strain ratios calculated in the materials, are 60.10^6; 333 000 and $2,3.10^6$ under load A. The corresponding number of repetitions (N_f) under load B are 200.10^6; 95 000 and $0,7.10^6$. This design layout would thus be satisfactory if only one of the wheel loads were to be applied.

Accommodating both wheel loads (that is, mixed traffic, section 8.3.1) changes the situation. When applying Miner's hypothesis (equation (8.11)) to each cement-treated layer to determine the effect of cumulative fatigue damage, the following is found:

$$\text{Cement-treated crusher-run} \quad : \Sigma\left(\frac{n}{N_f}\right) = \left(\frac{300\ 000}{60.10^6} + \frac{25\ 000}{200.10^6}\right) < 1$$

$$\text{Cement-treated gravel base} \quad : \Sigma\left(\frac{n}{N_f}\right) = \left(\frac{300\ 000}{333\ 000} + \frac{25\ 000}{95\ 000}\right) > 1$$

$$\text{Cement-treated gravel subbase}: \Sigma\left(\frac{n}{N_f}\right) = \left(\frac{300\ 000}{2,3.10^6} + \frac{25\ 000}{0,7.10^6}\right) < 1$$

The sum of the ratios in the cement-treated gravel base is more than unity and the layer is therefore likely to suffer some distress as a result of cumulative fatigue damage. This may be rectified by either (i) increasing the elastic modulus which will reduce the calculated strain, (ii) increasing the strain at break (ε_b) to at least 120 με, or (iii) rearranging the proposed layout.

Based on the third alternative approach suggested in section 8.2.1, it is recommended that the proposed layout and the traffic-associated cracking be accepted. This is because the cracking will only occur after the pavement has carried about 260 000 and 20 000 repetitions of loads A and B respectively. During the post-crack phase the pavement is expected to successfully carry the small amount of remaining traffic (only 40 000 and 5 000 repetitions respectively) without unacceptable deformation of the subgrade and loss in riding quality.

8.5.3 Upside-down design

In Chapter 5 it was suggested that 150 mm crusher-run should be used as a basecourse over cement-treated materials to dampen reflective cracking and to act as a thermal insulator. Cement-treated materials are relatively cheap in South Africa while bituminous materials are fairly expensive. This situation lends itself ideally to the use of the so-called upside-down pavement, that is a pavement layout consisting of a thin bituminous layer, a granular basecourse and a cement- or lime-treated subbase. This layout, which has been used with a fair amount of success in parts of Australia, the USA and Japan (Lilley, 1972; Norling, 1973a; and Yamanouchi, 1973) has also been constructed with considerable success in South Africa and has currently become practically a standard layout for highway pavements.

The following example shows the application of the proposed procedure to the design of an upside-down pavement which has a cement-treated crusher-run layer and is expected to carry overloaded traffic.

Design traffic:

Bituminous layer	: 15 million load applications (T)
Granular materials	: 10 million E80
Cement-treated layers:	5 000 repetitions (n) of an 80 kN wheel load (about one 80 kN wheel per day for 10 years).

Tyre contact pressure: 520 kPa.

Elastic moduli and Poisson ratio:

Bituminous material (BS 594-type)	: 1 500 MPa
Crusher-run base	: 500 MPa
Cement-treated crusher-run	: 15 000 MPa
Cement-treated subbase	: 2 000 MPa
Selected subgrade and subgrade (foundation):	100 MPa

The Poisson ratio of all materials is assumed to be 0,35.

Allowable design values:

Bituminous materials	: ε_t = 300 με (Table 8.2)
Cement-treated crusher-run:	86 με (strain ratio corresponding to n = 5 000 is 0,59; ε_b assumed to be 145 με)
Cement-treated subbase	: 103 με (ε_b taken as 175 με; Figure 8.5)
Subgrade	: ε_c = 460 με (Table 8.3)
Granular material	: zero tensile strain at bottom of layer.

Environment factors: Appropriate adjustments were made to the elastic moduli and allowable strains. The upside-down design should also preferably not be used in wet climates unless proper provision is made for drainage.

Suggested layout: (Figure 8.7)

35 mm bituminous material (BS 594-type)
150 mm crusher-run base
150 mm cement-treated crusher-run
150 mm cement-treated subbase
selected materials and subgrade.

Analysis: The suggested layout was analysed with CHEVRON. The horizontal tensile strains, under the 80 kN wheel load and at the bottom of the four upper layers, were calculated and shown to be -7, -40, +47 and + 95 $\mu\varepsilon$ (- compression, + tension). If moderate cracking is accepted in both of the cement-treated layers, the tensile strains are increased 1,2 times (Table 4.5) to 56 and 114 $\mu\varepsilon$ respectively.

Comparison: The calculated horizontal tensile strains in the bituminous material and cement-treated crusher-run (-7 and 56 $\mu\varepsilon$) are less than the allowable values (300 and 86 $\mu\varepsilon$). The calculated vertical compressive strain in the subgrade (less than 300 $\mu\varepsilon$) is also less than the allowable value (460 $\mu\varepsilon$). The calculated horizontal tensile strain in the cement-treated subbase (114 $\mu\varepsilon$) exceeds the allowable value (103 $\mu\varepsilon$) and since the strain ratio is about 0,65 (i.e. 114/175) the layer can be expected to carry only about 1 500 repetitions (N_f) ($N_f = 10^{9,1(1-114/175)}$) before traffic-associated cracks will form within the layer itself. At this stage the road will still have an acceptable riding quality and no traffic-associated cracks would have appeared on the surface.

It is now necessary to repeat the CHEVRON analysis for the post-crack phase but with a reduced elastic modulus for the cement-treated subbase; the layer is assumed to have deteriorated considerably and that its elastic modulus after cracking is only 500 MPa. The calculated horizontal tensile strains in the three upper layers are -25, -51 and 71 $\mu\varepsilon$. The vertical compressive strains on the two lower layers after cracking are 160 and 348 $\mu\varepsilon$.

After increasing the calculated tensile strain in the cement-treated crusher-run base (71 $\mu\varepsilon$) (when the cement-treated subbase is cracked) by 1,2 to 85 $\mu\varepsilon$, the strain ratio ($\varepsilon/\varepsilon_b$) is calculated as 0,59. According to equation (2.3) (page 31), this strain ratio can allow about 5 000 repetitions (the design traffic) before traffic-associated cracks will develop. Traffic-associated cracking is therefore not expected in the cement-treated crusher-run layer - not even after the cement-treated subbase has cracked. If the cracked cement-treated subbase is accepted as the equivalent of a granular material and the calculated vertical compressive strain (ε_c) (160 $\mu\varepsilon$) is increased 2,5 times (Table 4.5) to 400 $\mu\varepsilon$, it appears that rutting of about 10 mm can be expected after the pavement has carried the design traffic, namely 10 million E80 (see Table 8.3).

Recommendation:

Accept the suggested structural layout. The cracking in the cement-treated subbase may be accepted since it is not likely to affect the riding quality before the pavement has carried the design traffic.

8.5.4 Analysis of standard design

A traffic study was performed for a particular project and the traffic distribution is given in Table 8.6. Assuming the AASHO equivalency factors, it represents an equivalent traffic of 0,52 million E80.

TABLE 8.6 : Expected traffic distribution on a proposed road

WHEEL LOAD (kN)	EXPECTED NUMBER OF REPETITIONS (n)
10	5.10^7
20	3.10^6
30	6.10^5
40	3.10^4
50	2.10^2
60	1.10^2

In Figure 8.7 it is suggested that a surface treatment, 150 mm cement-treated natural gravel base, 150 mm cement-treated natural gravel subbase, and 150 mm selected subgrade would carry between 0,2 and 0,5 million E80 on a subgrade with CBR=5. To verify the proposed layout, the suggested design procedure may be used.

Design traffic: A dual wheel load of 40 kN (tyre contact pressure = 520 kPa) with wheel spacing of 350 mm is used. The same contact area is assumed, that is varying contact pressures, and the load factor is therefore simply the ratio between the wheel loads.

Elastic moduli (E) and Poisson ratio (μ):

Surfacing (high-temperature region)	: E = 1 500 MPa; μ = 0,44
Cement-treated gravel base	: E = 7 500 MPa; μ = 0,35
Cement-treated gravel subbase	: E = 3 000 MPa; μ = 0,35
Selected subgrade (CBR=10)	: E = 100 MPa; μ = 0,35
Subgrade (CBR=5)	: E = 50 MPa; μ = 0,35.

Safe working strains and strain at break (ε_b):

Surfacing	: ε_t = 270 $\mu\varepsilon$ (Table 8.2)
Cement-treated base	: ε_b = 120 $\mu\varepsilon$ (Figure 8.5)
Cement-treated subbase	: ε_b = 150 $\mu\varepsilon$ (Figure 8.5)
Selected subgrade and subbase	: ε_c = 700 $\mu\varepsilon$ (Table 8.3).

Effect of environment: The elastic modulus of the surfacing is taken as a minimum and it is assumed that adequate drainage is provided.

Practical layout: The layout mentioned above is used, but since the traffic is slightly more than 0,5 million E80, it was decided to use a 30 mm surfacing instead of the surface treatment.

Maximum strains: The horizontal strains at the bottom of the three upper layers were calculated to be -46, 31 and 74 $\mu\varepsilon$ (- compressive, + tension). The vertical compressive strains on the top of the two lower layers are less than 200 $\mu\varepsilon$.

The total thickness of cement-treated material exceeds 200 mm and since it is assumed that wide cracks may develop, the tensile strain in the base between the wheels (31 $\mu\varepsilon$) is increased 1,4 times (Table 4.5) to 43 $\mu\varepsilon$.

Developed and safe working strains: The surfacing is in compression and hence not likely to show signs of distress. The vertical compressive strains in the lower untreated layers are less than the allowable and subgrade deformation is therefore unlikely. The application of Miner's hypothesis (equation (8.11)) to the two cement-treated layers is done in Table 8.7.

From Table 8.7 it appears that the cement-treated subbase will experience traffic-associated cracking because of cumulative fatigue damage before it has carried the full design traffic. The analysis was therefore repeated for the post-crack phase but with a reduced elastic modulus (1 000 MPa) for the cement-treated subbase. The strains in the bituminous material and cement-treated base were respectively calculated as 63 and 65 $\mu\varepsilon$. The strain in the cement-treated base (65 $\mu\varepsilon$) is increased 1,4 times to 91 $\mu\varepsilon$ since wide cracks

TABLE 8.7 : Application of Miner's hypothesis to accommodate mixed traffic

WHEEL LOAD (kN)	EXPECTED NUMBER OF REPETITIONS (n) (Table 8.6)	CALCULATED STRAIN (ε)	($\varepsilon/\varepsilon_b$)	NUMBER OF REPETITIONS TO FAILURE (N_f) (eq. 2.5)	$\Sigma(\frac{n}{N_f})$
Cement-treated base (ε_b=120 $\mu\varepsilon$)					
10	5.10^7	10,75	0,09	∞	
20	3.10^6	21,5	0,18	∞	
30	6.10^5	32,25	0,27	$43,9.10^5$	0,14
40	3.10^4	43	0,36	$66,7.10^4$	0,04
50	2.10^2	53,75	0,45	$1\ 011.10^2$	0,002
60	1.10^2	64,5	0,54	153.10^2	0,006
					Σ=0,188
Cement-treated subbase (ε_b=150 $\mu\varepsilon$)					
10	5.10^7	18,5	0,12	∞	
20	3.10^6	37	0,25	∞	
30	6.10^5	55,5	0,37	$5,4.10^5$	1,11
40	3.10^4	74	0,49	$4,4.10^4$	0,68
50	2.10^2	92,5	0,62	29.10^2	0,07
60	1.10^2	111	0,74	2.10^2	0,50
					Σ=2,36

are expected. The vertical compressive strain on the top of the cracked treated subbase and selected subgrade is 230 and 275 $\mu\varepsilon$. Since the strain ratio in the cement-treated base is 0,76 (i.e. 91/120) it would be able to carry only about 150 legal wheel loads before it would also suffer traffic-associated cracking and a drop in elastic modulus.

The analysis for the post-crack phase was continued but with two cracked cement-treated layers (elastic moduli 2 000 and 1 000 MPa) and the vertical compressive strain on the top of the subgrade was calculated as 360 $\mu\varepsilon$ which is low (Table 8.3) and therefore not critical with respect to the development of rutting.

If a higher elastic modulus is initially assumed for the cement-treated base, say 9 000 MPa, traffic-associated cracks will take a little longer to develop, but both the cement-treated layers will also develop traffic-associated cracks before the layout has carried the full design traffic ($0,5.10^6$ E80). Rutting in the subgrade is still not expected to occur.

Acceptance: The suggested structural layout may be accepted. Initial cracks may develop very soon after construction and they will reflect through the 30 mm surfacing. Traffic-associated cracking is expected to develop but very little deformation (rutting about 5 mm) is expected to occur before the pavement has carried the design traffic. If the designer is not prepared to accept traffic-associated cracking it is suggested that a 200 mm cement-treated base should be used instead of the 150 mm cement-treated base (Figure 8.8).

8.5.5 Airport pavement

An airport pavement has to be designed for 4 million departures of a Boeing 747.

Design traffic: Accepting the Load Repetition Factors suggested by the PCA (1973), the 4 million departures is converted to an equivalent 1,12 million repetitions for structural pavement design purposes.

The load on the aircraft's strut is 900 kN and this represents 225 kN per wheel. The tyre contact pressure is assumed as 1 275 kPa. The four wheels per strut are spaced in a rectangle of 1 120 by 1 470 mm.

Elastic moduli (E) and Poisson ratio (μ):

Surfacing (high temperature region)	: E = 1 500 MPa; μ = 0,43
Bitumen-treated base	: E = 1 500 MPa; μ = 0,43
Cement-treated gravel base	: E = 8 000 MPa; μ = 0,35
Cement-treated subbase	: E = 3 000 MPa; μ = 0,35
Subgrade (CBR=30)	: E = 300 MPa; μ = 0,35

Safe working strains:

Surfacing and bitumen-treated base	: ε_t = 70 $\mu\varepsilon$ (NITRR, 1977)
Cement-treated base	: ε_b = 120 $\mu\varepsilon$ (Figure 8.5)
Cement-treated subbase	: ε_b = 152 $\mu\varepsilon$ (Figure 8.5)
Selected subgrade and subgrade	: ε_c = 650 $\mu\varepsilon$ (Table 8.3)

Effect of environment: The airport is in a flat area with a high water-table and it was therefore decided not to use untreated crusher-run. The possibility of high horizontal shear stresses when the aircraft has to brake suddenly during take-off confirmed this decision. The elastic modulus of the bituminous materials was taken as a minimum namely 1 500 MPa.

Suggested practical layout:

100 mm bituminous material
450 mm cement-treated base
450 mm cement-treated subbase
subgrade (CBR=30).

Maximum strains: The horizontal strains at the bottom of the three upper layers were calculated with ELSYM5 as -40, +33 and 60 $\mu\varepsilon$ respectively. The vertical strain on the top of the subgrade is only -69 $\mu\varepsilon$ (- compression, + tension).

The particular cement-treated base material is expected to experience only moderate cracking and the 100 mm bituminous material is expected to dampen the reflection of these cracks very effectively. The calculated tensile strains in the cement-treated materials were therefore not increased because it was considered to be too conservative.

Developed and safe working strains: The surfacing is in compression and not likely to show distress. The vertical compressive strain in the subgrade is less than the allowable strain and subgrade deformation is not expected.

The bottom 150 mm of the cement-treated subbase is expected to carry about 322 000 repetitions (N_1) before it is likely to undergo traffic-associated cracking ($N_1 = 10^{9,1(1-60/152)}$); from equation (2.5) (page 31). If the vertical crack propagation time through the 150 mm layer is assumed to be zero and it is assumed that the elastic modulus of the lower 150 mm reduces to 1 000 MPa after cracking, the structural analysis can be continued.

The layout now changes to -

100 mm bituminous material	: E = 1 500 MPa
450 mm cement-treated base	: E = 8 000 MPa
300 mm cement-treated subbase	: E = 3 000 MPa
150 mm cracked cement-treated subbase	: E = 1 000 MPa
Subgrade	: E = 300 MPa.

The tensile strain at the bottom of the 300 mm cement-treated subbase was calculated as 56 $\mu\varepsilon$. According to equation (2.5) it was calculated that traffic-associated cracking can occur after 550 000 repetitions (N_2) ($N_2 = 10^{9,1(1-56/152)}$). It was again assumed that the vertical crack propagation time through the 150 mm layer was zero and that the elastic modulus would decrease to about 1 000 MPa.

The layout now changes to -

100 mm bituminous material	: E = 1 500 MPa
450 mm cement-treated base	: E = 8 000 MPa
150 mm cement-treated subbase	: E = 3 000 MPa
300 mm cracked cement-treated subbase	: E = 1 000 MPa
Subgrade	: E = 300 MPa.

The tensile strain at the bottom of the intact 150 mm cement-treated subbase was calculated as 60,5 $\mu\varepsilon$. This strain level corresponds to about 300 000 repetitions (N_3) before the onset of traffic-associated cracking.

When continued along the line explained above, and when the full 450 mm cement-treated subbase was assumed as cracked (and with an equivalent elastic modulus of 1 000 MPa), the tensile strain at the bottom of the cement-treated base (E = 8 000 MPa) was calculated as 61 $\mu\varepsilon$. Since the strain at break (ε_b) of the cement-treated base is 120 $\mu\varepsilon$, the number of repetitions to traffic-associated cracking (N_4) at the bottom of the cement-treated base is only 30 000.

The total number of load repetitions before the onset of traffic-associated cracking at the bottom of the cement-treated base is therefore $N_1+N_2+N_3+N_4$ = 1,2 million. This number is slightly in excess of the design traffic, viz. 1,12 million.

At this stage, that is when the design traffic has been carried, the compressive strain at the bottom of the surfacing is 44 $\mu\varepsilon$ and the vertical compressive strain on the top of the subgrade has increased to 116 $\mu\varepsilon$. No distress in these layers is therefore expected.

The lower strain at break (ε_b) of the cement-treated base (120 $\mu\varepsilon$) and the remaining comparatively thin intact cement-treated material (only 300 mm), results in a relatively rapid crack propagation through the cement-treated base after the cement-treated subbase and the lower 150 mm of the cement-treated base have cracked. It is therefore recommended that this relatively short propagation time through the remaining thickness of the cement-treated base should not be calculated and considered as contributing to the pavement's life.

Alternative: If greater surety is desired and the design has to be more conservative, the cement-treated subbase should be 600 mm thick. With all the cement-treated layers intact, the tensile strain at the bottom of the 600 mm subbase was calculated as 54 $\mu\varepsilon$. This represents about 740 000 repetitions to the onset of traffic-associated cracking (N_5).

The values of N_1 to N_4 calculated previously will be slightly more for this layout, but assuming them to be the same, 1,12 million repetitions is expected to be reached after only about 320 mm of the cement-treated subbase has experienced traffic-associated cracking ($N_5+N_1+0,1N_2$). This results in about a further 725 000 repetitions

$(N_4+N_3+0,9N_2)$ before the layout has reached the same amount of cracking as in the original suggested layout.

The alternative layout (that is with a 600 mm cement-treated subbase) is therefore expected to carry about 1,94 million repetitions before it has reached the same amount of traffic-associated cracking as that which the original layout would reach after about 1,2 million repetitions, i.e. 1,62 times more. The increase in initial cost if the additional 150 mm cement-treated subbase is constructed is only about 10 per cent. To spend 10 per cent more money for an increase in life of 62 per cent is considered worthwhile!

Acceptance: If funds are limited to cover absolute requirements only, accept the layout as suggested initially. No distress or deformation is expected to occur before the pavement has carried the design traffic. Depending on the availability of funds the alternative layout should be seriously considered because of the added advantage of a 62 per cent increase in life for a mere 10 per cent increase in construction cost.

8.6 RECOMMENDATION

The proposed structural pavement design procedure for cement-treated layers based on layered elastic theory seems to be accurate in predicting both pavement response and behaviour. It is therefore recommended that it be accepted and used for practical pavement design purposes. Considerably more research is however necessary before it would be able to accurately predict the pavement's future performance in terms of a serviceability index, that is riding quality.

CHAPTER 9

CONCLUSIONS AND RECOMMENDATIONS

The conclusions and recommendations relevant to a particular chapter have been listed at the end of that chapter. This chapter contains only the overall recommendations and conclusions from the study.

1. Structural pavement design theory has been and is advancing rapidly. A design method based on a sound theoretical basis is therefore not entirely impossible. At present layered elastic theory is being used as the theoretical basis and the wider application thereof to pavement design is developing very rapidly.

2. The published literature indicates that none of the existing pavement design procedures is really suitable for the structural design of pavements with cement-treated layers. This is because there are certain very important design aspects which should be, but are not, accurately accounted for in the existing procedures. A cement-treated layer should be thick; it should be placed on a sound foundation and be strong enough to withstand the stresses and strains induced by the heavy wheel loads expected on the pavement. The balanced design concept aims at preventing a layer from being overstressed and this is a must in any procedure. Other very relevant aspects, for example thermal insulation, testing methods, design criteria, fatigue life and material variability, should also be considered and included.

3. Initial cracking in cement-treated materials, caused either by drying-shrinkage or thermal stresses, has to be accepted as a fact. There seems to be very little that can be done to prevent it from occurring! Allowance for these initial cracks should be made during the structural design stage by means of a proper structural analysis and the calculation of the expected increase in stress and strain next to the crack. The allowance for the initial crack should not be made during the material's characterization phase by, for example, reducing the elastic modulus or by an adjustment to the allowable stresses and strains. Traffic-associated cracking, caused by traffic stresses, is very detrimental since a loss in riding quality usually follows this type of cracking - it should therefore be controlled.

4. A prismatic solids finite element computer program provided the opportunity to calculate the expected increase and hence design against the development of traffic-associated cracking. The analyses indicate that the maximum tensile stress in a pavement with a very wide crack is about 1,4 times the maximum stress calculated for an uncracked pavement and that it occurs at the bottom of the treated layer and acts parallel to the crack. The amount of the increase depends on the crack width and the thickness of the cement-treated layers. It is recommended that the structural analysis should be performed on an uncracked pavement by the use of, for example, the CHEVRON computer program and that the computed stresses and strains in the cement-treated layers should be increased to make provision for the crack. When the increased values are used as design values, the onset of traffic-associated cracking ought to be prevented.

5. Thermal stresses are important in uncracked, cement-treated materials and they most probably contribute to the onset of initial cracking. Once the layer has cracked, movement can take place which eliminates the build-up of thermal stresses. Thermal stresses are therefore not important in a cracked layer, and since all properly constructed cement-treated layers will crack, thermal stresses may be neglected for structural pavement design purposes. It is nevertheless recomended that a 150 mm crusher-run insulator should be placed on top of the cement-treated layer in major roads.

6. The variation in quality between cement-treated materials produced in the field by the construction process, and those prepared in a laboratory and used by the design engineer, is significant. The differences vary from site to site and it is currently very difficult to recommend on how to include them during the design stage. It is, however, recommended that more effort should go into ensuring better conformity between field- and laboratory-prepared materials since the reported variations must have serious economic implications.

7. The Heavy Vehicle Simulator (HVS) tests were performed in an attempt to correlate theory and practice. The agreement between the predicted and actual behaviour, which was defined as the amount of traffic-associated cracking, is excellent.

8. A structural pavement design procedure for cement-treated layers is proposed in this thesis. The aim can be to either prevent traffic-associated cracking in the cement-treated material before the pavement has carried the design traffic, or the cracking can be accepted and the post-crack phase be included and considered. A decision on which approach should be used rests with either the designer or client. The procedure is based on layered elastic theory and it seems to be eminently suitable for the design of pavements with cement-treated materials because the agreement between the predicted and actual response and behaviour is really excellent. Its use for practical pavement design purposes is therefore recommended.

RECOMMENDATIONS FOR FURTHER WORK

1. Triaxial testing, fracture mechanics and finite element studies are necessary to finalise -
   (a) the establishment of a design criterion for cement-treated materials, and
   (b) vertical and horizontal crack propagation in the material.

2. Finite element studies and field testing are necessary to verify the material characterization during the post-crack phase. Methods should be devised to accommodate the anisotropy and predict the elastic modulus of the equivalent untreated material.

3. Studies may be continued on the prevention of initial cracking, but this is not considered to be very necessary for structural pavement design purposes. This research energy can be spent more fruitfully to verify the applicability of the theoretically calculated increases in stress and strain adjacent to the crack.

4. The statistical variations in material quality and the difference in properties between field- and laboratory-prepared samples require more accurate definition and inclusion in the proposed design procedure.

5. Much more research effort is required on the definition of environmental loading conditions and the prediction of future traffic loading.

6. The proposed procedure should be expanded and verified, to include also the prediction of long-term performance, that is the prediction of riding quality under both traffic and environmental loading.

APPENDIX A

PUMPING IN A CRACKED PAVEMENT

A.1 INTRODUCTION

Cracks in cementitious layers can allow water to penetrate into the lower layers. If the water gets trapped and cannot drain away through either specially provided drainage layers or a free-draining subgrade, for example sand, it may accumulate and soften the lower layers. This may reduce their elastic moduli resulting in possible overstressing of the upper (cementitious) layers. Under certain conditions it is possible that the water may be pumped out by traffic taking with it some of the fine material from the lower layers. If this is allowed to continue for a long enough period a void may form near the crack.

Pumping is a very serious matter since total distress and a loss of riding quality usually follows very soon after it has started (Otte, 1973a). There has been much speculation on this and many people have been led to believe that pumping actually causes the distress. The prismatic solid finite element program (explained in Chapter 4) provided an opportunity to study this phenomenon analytically.

A.2 ANALYSIS

Only layout H (Figure 4.13, page 71) was considered in this part of the study. After a finite element analysis had been performed to determine the effect of the crack, the mesh was changed and the analysis was repeated but a void was included (Figure A.1). The void (135 mm wide and 5 mm deep) was assumed to represent pumping, or the effect thereof, and it was taken to run across the pavement.

During the study the elastic modulus of the treated base was increased from 12 000 MPa to 50 000 MPa and that of the untreated crusher-run was reduced from 500 MPa to 80 MPa. It is appreciated that moduli of 50 000 and 80 MPa are extremely exceptional, but the major objectives in choosing them were to obtain modular ratios of 100 and 250.

To investigate the effect of progressive softening of the lower layers in the vicinity of the crack the modulus of the second pavement layer, that is the one containing the void, was reduced as shown in Figure A.2. The outcome of this analysis was compared with that when the void was modelled into a layer with a uniform elastic modulus (as in Figure A.1).

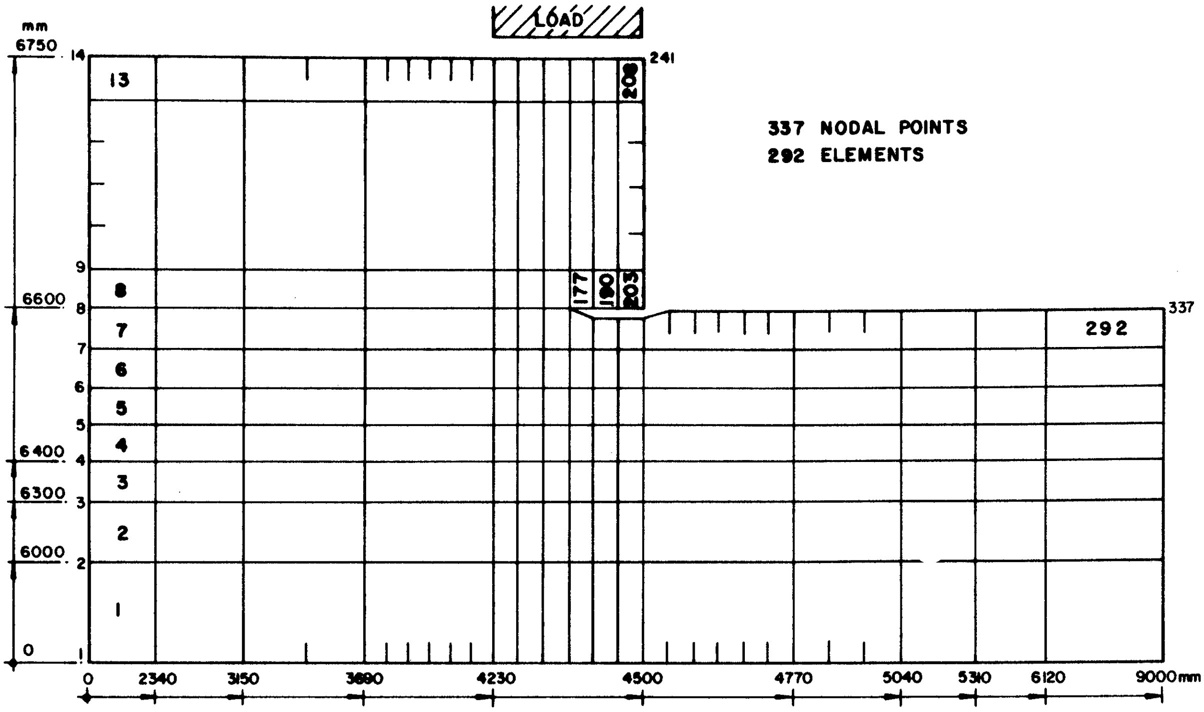


FIGURE A-1

FINITE ELEMENT MODEL AFTER PUMPING

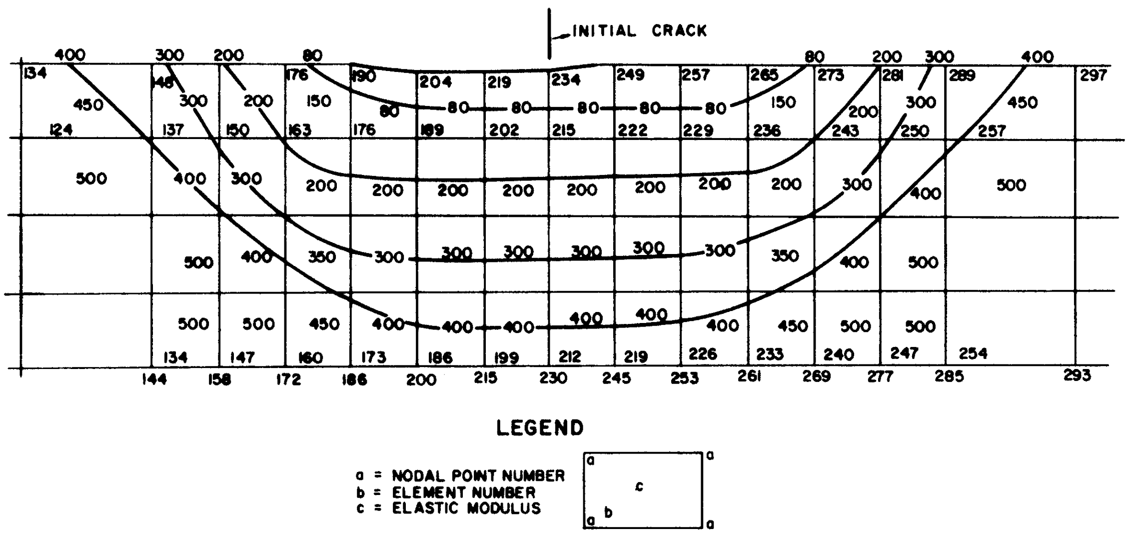


FIGURE A-2

THE REDUCTION IN ELASTIC MODULUS OF THE UNTREATED LAYER IN THE VICINITY OF THE CRACK

A.3 RESULTS

A.3.1 Uniform elastic modulus for support

A.3.1.1 Maximum tensile stress

The maximum tensile stresses in two cracked cases, one without (Figure 4.6) and one with (Figure A.1) the void, were calculated as described in section 4.2. They occurred at the bottom of the treated layer and acted parallel to the crack (σ_{zz}). The increase in stress caused by the void was calculated as the ratio between the two maxima and is reported in Table A.1.

TABLE A.1 : Increase in maximum tensile stress at the bottom of a cracked layer as a result of a void

ELASTIC MODULUS OF TREATED LAYER (MPa)	MODULAR RATIO	MAXIMUM TENSILE STRESS (kPa) IN THE LAYLR		RATIO
		Without void	With void	
6 000	12	675	1 205	1,79
12 000	24	1 007	1 488	1,48
20 000	40	1 271	1 691	1,33
50 000	100	1 784	2 121	1,19
20 000	250	1 987	2 254	1,13

The table indicates that the maximum tensile stress increases (from 675 to 1 987) and the ratio between the stresses decreases (from 1,79 to 1,13) with an increase in modular ratio. This information is also reproduced as Figure A.3. It indicates a significant increase in stress below a modular ratio of about 100 and the minimum increase caused by the void seems to be about 1,10 times at very large modular ratios.

A.3.1.2 Comparison of σ_{zz} and σ_{xx} stresses

In the analyses with the void the maximum stress perpendicular to the crack (σ_{xx}) occurred in the centre of one of the elements at the top of the treated layer and some distance away from the edge of the initial crack. Table A.2 compares these values with the maximum stresses parallel to the crack (σ_{zz}) at the bottom of the layer. In all five cases the stresses perpendicular to the crack (σ_{xx}) were much lower than those parallel to the crack (σ_{zz}).

A.3.2 Progressive softening of support

The analysis with the progressively softening support (Figure A.2) was done for a cement-treated layer with an elastic modulus of 12 000 MPa.

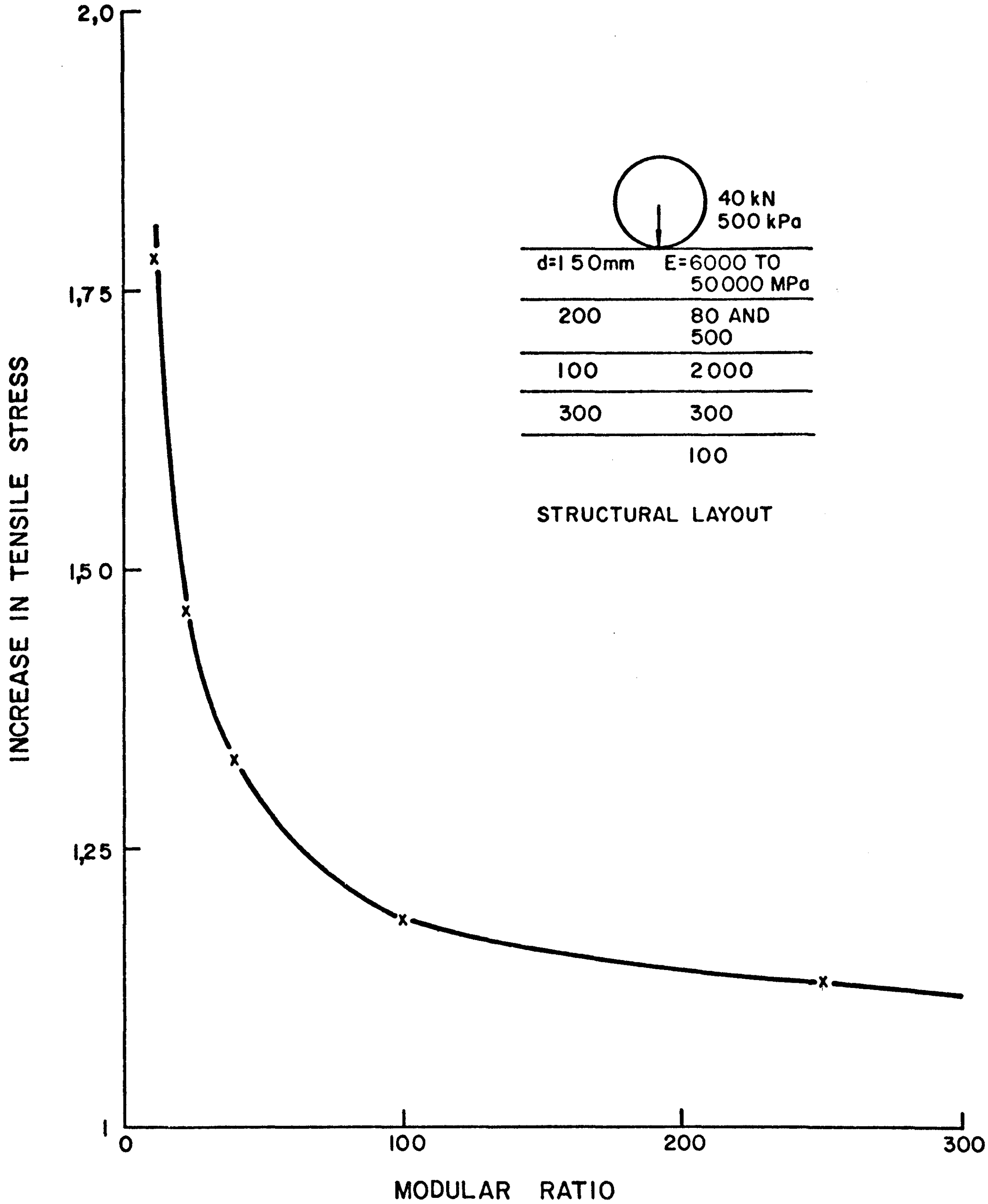


FIGURE A-3

RELATIONSHIP BETWEEN INCREASED TENSILE STRESS AND MODULAR RATIO DUE TO THE PRESENCE OF A VOID

TABLE A.2 : Comparison of maximum stresses perpendicular and parallel to the crack after the void has developed

ELASTIC MODULUS (MPa)	MODULAR RATIO	MAXIMUM TENSILE STRESS (kPa)		RATIO
		Perpendicular to crack (σ_{xx}) (Top of layer)	Parallel to crack (σ_{zz}) (Bottom of layer)	
6 000	12	398	1 205	3,03
12 000	24	416	1 488	3,58
20 000	40	430	1 691	3,93
50 000	100	456	2 121	4,65
20 000	250	628	2 254	3,59

The maximum tensile stress occurred at the same position as in the previous analyses and Table A.3 contains the results to evaluate the effect of the progressive softening before and after the void developed.

Table A.3 indicates that for both cases, with and without the void, the stresses for the progressively softening support were higher than for the uniform support, 1 399 and 1 691 versus 1 008 and 1 488 respectively. This could have been expected from the work by Pretorius (1970). It also appears that the stress increase caused by the pumping and the formation of the void was reduced on the progressive softening support, from 1,48 to 1,21 times.

TABLE A.3 : Effect of progressive softening on maximum tensile stress (kPa)

	CRACKED WITH NO VOID	VOID HAS DEVELOPED	RATIO
Uniform support	1 008	1 488	1,48
Progressive softening	1 399	1 691	1,21

A.3.3 Maximum strain

The failure criterion for cement-treated materials is often expressed in terms of strain (section 2.2.9) and the maximum tensile strain for each analysis was therefore calculated. In all twelve cases the maximum tensile strain occurred in element 203 and it acted parallel to the crack, that is ε_{zz}. The principle of linear extrapolation that was applied for the stress calculation (section 4.2.6) was not used. The values reported in Table A.4 occurred at the centre of the element, 12,5 mm above the bottom of the layer and 12,5 mm from the edge of the crack. The strains (ε_{zz}) were calculated from equation (A.1) and the stresses (σ_{zz}, σ_{yy} and σ_{xx}) were obtained from the finite element analysis (μ = 0,35).

$$\varepsilon_{zz} = \frac{1}{E}\left[\sigma_{zz} - \mu\ (\sigma_{yy}+\sigma_{xx})\right] \quad \text{......................... (A.1)}$$

<u>TABLE A.4</u> : <u>Maximum tensile strain in a cracked cement-treated layer</u>

ELASTIC MODULUS OF TREATED LAYER (MPa)	MODULAR RATIO	MAXIMUM TENSILE STRAIN ($\mu\varepsilon$)		RATIO
		Without void	With void	
6 000	12	94	164	1,74
12 000	24	67	100	1,49
20 000	40	51	68	1,33
50 000	100	28	34	1,21
20 000	250	80	91	1,14
12 000	+	95	115	1,21

The ratios calculated in Tables A.1 and A.4 are very much the same. This means that the increase caused by the void formed after pumping is the same no matter whether stress or strain is used as the design criterion.

A.4 <u>DISCUSSION</u>

If it is assumed that pumping was very severe and that a void had formed near the initial crack, then Table A.1 indicates an even further increase in tensile stress over that already shown in Tables 4.2 and 4.3. The maximum tensile stress after the void has formed occurred at the same position as before the void had formed and it acted in the same direction. This means that the factors responsible for the typical traffic-associated cracking pattern had not changed and that the pattern itself would therefore probably not be changed by the presence or absence of pumping in the pavement. This seems to indicate that pumping did not <u>cause</u> the traffic-associated cracking, neither did it change the mode of cracking or the cracking pattern, but since it caused an additional increase in stress it <u>accelerated</u> the distress, that is it caused the traffic-associated cracking to occur sooner.

It is however also possible to argue that the pumping did cause the cracking. In some cases the stress or strain may have been relatively low before pumping and the design would probably have tolerated a considerable amount of traffic, which, depending on the traffic pattern, may mean a life of about 5 to 10 years. After pumping the stress and strain may have increased and cracking would occur much sooner, say after 6 to 12 months. Under these conditions, that is when the pavement's life is reduced from a few years to only a few months, it would be reasonable to argue that the pumping had actually caused the cracking. This is because pumping had increased the stress or strain to beyond the point where cracking would not have occurred before the pavement had largely served its functional life. The strains reported in Table A.4 do however indicate that there may not be such a large reduction in expected life (see equation 2.5, page 31). Although the strain ratios are fairly high for modular ratios 12 and 24, the difference in the practical life, that is the time in months or years before the material can be expected to fail in fatigue, is relatively small for all six cases. The above reasoning of pumping causing failure because of a significant reduction in life is therefore not really applicable to these examples.

Table A.2 seems to indicate that traffic-associated cracking does not occur as a result of the σ_{xx} stresses since they are significantly smaller than the σ_{zz} stresses. The pavement with the void next to the crack partially acted as a cantilever resulting in increased tensile stresses near the top of the layer but the stresses at the bottom and parallel to the crack (σ_{zz}) exceeded these by far and the traffic-associated cracking would therefore occur perpendicular to the initial transverse crack. The original theory of the failure mode (Figure A.4) stated that a cantilever would be formed after pumping and this would cause the cracking to start at the top of the treated layer. Table A.2 seems to indicate that this is incorrect. The failure mode, both the previous and the current concept, is explained schematically in Figure A.5.

It was seen (Table A.3) that progressive softening of the lower layers increased the stresses to beyond those developed on a uniform support and it will therefore cause cracking to occur much sooner. Progressive softening is, however, a fairly accurate representation of what can happen when water accumulates in the lower layers. This, and the discussion on the effect of the void on the maximum tensile stress (Table A.1) indicate that the prevention of pumping and the formation of a void is very beneficial in terms of increasing the life of the pavement. Special attention should therefore be paid to avoid the ingress of water and the pumping out of fines.

The design analyzed indicated that the severity of the pumping may be reduced by increasing the modular ratio (Figure A.3) and it therefore

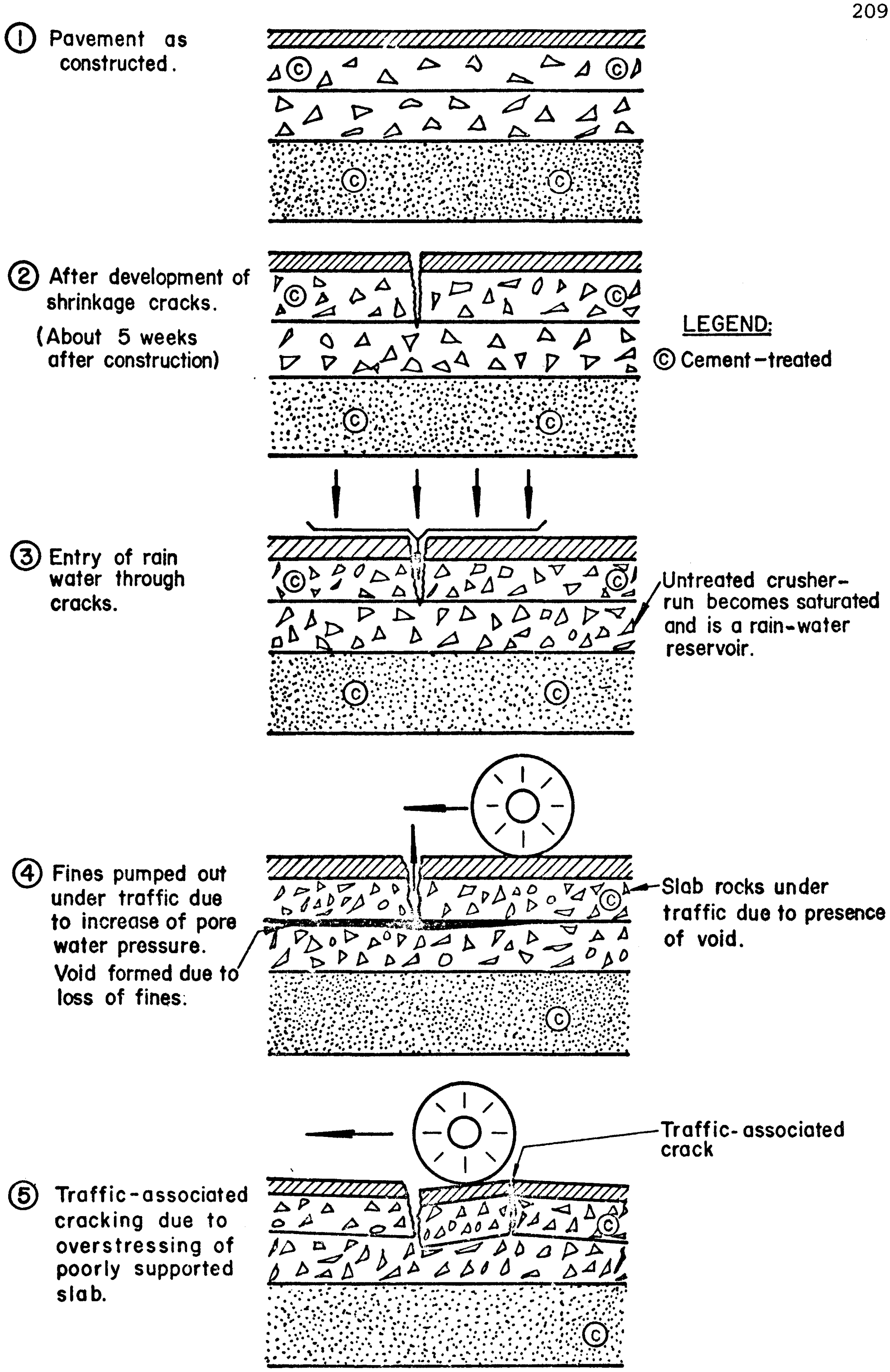
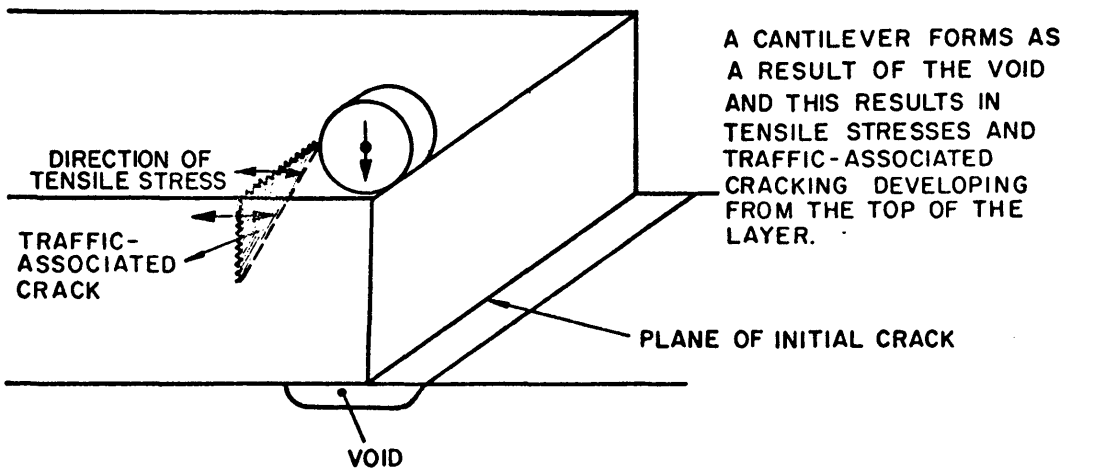
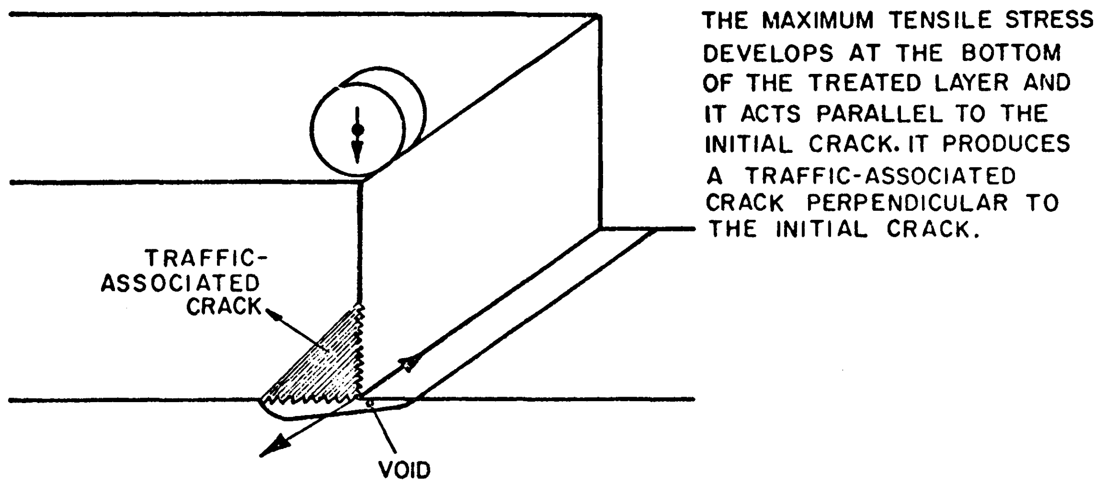


FIGURE A-4

SCHEMATIC DIAGRAM OF MODE OF FAILURE OF A CEMENT-TREATED BASE



PREVIOUS CONCEPT



CURRENT CONCEPT

FIGURE A-5

THREE-DIMENSIONAL SCHEMATICS OF FAILURE MODE ASSOCIATED WITH PUMPING AND TRAFFIC-ASSOCIATED CRACKING appears to be advantageious to strive to the maximum modular ratio. It should also be appreciated that the tensile stress increases significantly with an increase in modular ratio (Table A.1 and Figures 2.1 and 2.3, pages 13 and 15) and that this could cause overstressing of the material and override the advantages of the previous argument. The two apparently conflicting requirements on modular ratio should be fully appreciated and every designer should evaluate his layout in terms of these. It is nevertheless recommended that the modular ratios should be kept to a minimum at all times (section 2.2.4).

A.5 CONCLUSION

(a) Load-associated cracking after excessive pumping is caused by high tensile stresses that develop at the bottom of the treated layer and which act parallel to the initial crack. It is not the result of stresses that were developed because the treated layer cantilevered over a void that was formed after pumping out of the fine material.

(b) Pumping does not cause traffic-associated cracking in treated layers; it merely accelerates the development of these cracks and makes them occur sooner. In cases where severe pumping was followed by traffic-associated cracking these cracks would have occurred anyway but the pumping made them occur much sooner. This is the reason why pumping is often thought to have caused distress in the form of extensive traffic-associated cracking in the treated layer.

APPENDIX B

THE PRISMATIC SOLIDS FINITE ELEMENT PROGRAM

A finite element computer program capable of handling prismatic solids was developed around 1969 and 1970 by E.L. Wilson and P.C. Pretorius (1970). The input requirements are essentially the same as for the other finite element programs that were developed by Wilson but there are some aspects of the input data that may constitute problems and these will be explained in this Appendix. It also contains a description of the input data.

It is assumed that the reader has some understanding of finite element analysis.

B.1 INPUT

B.1.1 Number of harmonics

Subroutine STIFF has a statement

$$\text{TM} = \text{ZL*SIN}[\text{XHAR*PI*T(N)/ZL}] / [\text{XHAR*PI*T(N)}] \quad \ldots\ldots\ldots\ldots\ (\text{B.1})$$

where T(N) is the loaded length in the Z-direction (see later)

PI = 3,141 2 when $\sin^{-1}$ is calculated in radians

= 180 when $\sin^{-1}$ is calculated in degrees

ZL = one half of the period length (see later)

XHAR = one less than the number of harmonics

TM = a parameter that should ideally be equal to zero. If T(N) or XHAR is zero then the parameter is not defined as above; if T(N) becomes zero then TM equals one and when XHAR becomes zero then TM becomes 0,5.

This equation is needed to calculate the number of harmonics (that is the number of terms in the Fourier series) that are required to represent the load accurately. To illustrate the use of the equation, assume T(N) as 150 mm and half of the period length as 2 400 mm. If TM should be zero, then (XHAR*PI*T(N)/ZL) should be zero since Sin 0° = Sin 180° = 0. Therefore

$$\frac{\text{XHAR*T(N)}}{\text{ZL}} \text{ should be 1 and XHAR} = \frac{\text{ZL}}{\text{T(N)}} = \frac{2\ 400}{150} = 16$$

Seventeen harmonics will be required since the number is given by XHAR + 1.

B.1.2 ZL-length

The ZL-length is half of the period length and the period length is the distance between two consecutive loads as indicated in Figure 4.1 (page 55).

For the analysis of pavement structures the distance between the loaded areas should be sufficient to ensure that there will be no interaction between two adjacent loaded areas and this means a long period length. The longer the period length the more the number of harmonics required to represent the load accurately but since this will be costly on computer time, the period length should not be made unnecessarily long. Using a stiff cement-treated layer, it is suggested that the period length should be of the order of 3 to 4 metres.

B.1.3 Material properties

Only one elastic modulus and one Poisson ratio can be specified for each layer of material. This implies homogeneous, isotropic, linear elastic materials.

B.1.4 Codes at nodal points

Care must be taken in the allocation of the boundary condition code numbers. A code of 0 indicates that the nodal point is free to move both vertically and horizontally, as is the case for all nodal points within the boundaries, and along the free surface. A code of 1 signifies freedom of movement in the Y-direction only (as for vertical boundaries and any vertical lines of symmetry) and one of 2 signifies freedom of movement in the X-direction only, while a code of 3 prohibits movement in both the X- and Y-directions. The presence of any cracks allows movement in a direction perpendicular to the crack.

B.1.5 Loaded length in Z-direction

The loaded length in the Z-direction is half of the total loaded length in the Z-direction. This is because of symmetry around the axis that goes through the centre of the loaded area (see Figure 4.1).

If the loaded area is rectangular it should be decided which side falls along the Z-axis and the loaded length in the Z-direction will again be half of the total load in the Z-direction.

B.1.6 Load application

The load on the structure is always applied as concentrated nodal point loads. A force working in the positive direction is given a + sign and one in the negative direction is designated negative, for example a force working downwards is negative.

If a total load of x kN is applied to a structure and it is allowable to use symmetry (around the y-axis), the load on the section of the structure under consideration in the finite element analysis is only $\frac{x}{2}$ kN. An example is shown in Figure B.1. If the load is specified as a pressure (load per area) the pressure remains the same on both sides of the axis, only the total load (kN) is halved.

If the load is applied as a uniformly distributed load, for example tyre or footing pressure, it must be converted and represented as concentrated line loads acting on the nearby nodal points. The length of these line loads are the loaded length in the Z-direction (section B.1.5). The concentrated forces on the nodal points are calculated from equation (B.2) which is

Nodal point force = (Applied pressure or stress) x (2 times loaded length in Z-direction) x (the effective width over which the stress acts) .... (B.2)

In this equation the term (2 x loaded length in the Z-direction) may be replaced by the <u>total</u> length in the Z-direction.

Equation (B.2) can best be explained by an example. Assume an applied stress of 520 kPa over an area of 280 x 280 mm and that in the finite element mesh this load will be carried on 5 nodal points (Figure B.2). Because of symmetry around the vertical axis (y) the distance between nodal points 1 and 5 is 140 mm and when equal spacing is assumed, the loads are 35 mm apart. The loaded length in the Z-direction is also 140 mm, it is the half of 280 mm. The concentrated (line) loads at the nodal points are therefore calculated as follows:

Nodal point 1 and 5 : (0,520)*(2*140)*(17,5) = 2 548 N
Nodal point 2 to 4 : (0,520)*(2x140)*(35) = 5 096 N

The total load at the 5 nodal points is 20,384 kN. (Note: it is about 50 per cent of the total applied load.)

B.1.6 Longitudinal distance

These distances are measured from the centre of the load along the Z-axis and any number of these distances can be fed in (all on separate cards). Stresses and displacements are printed at each plane so defined.

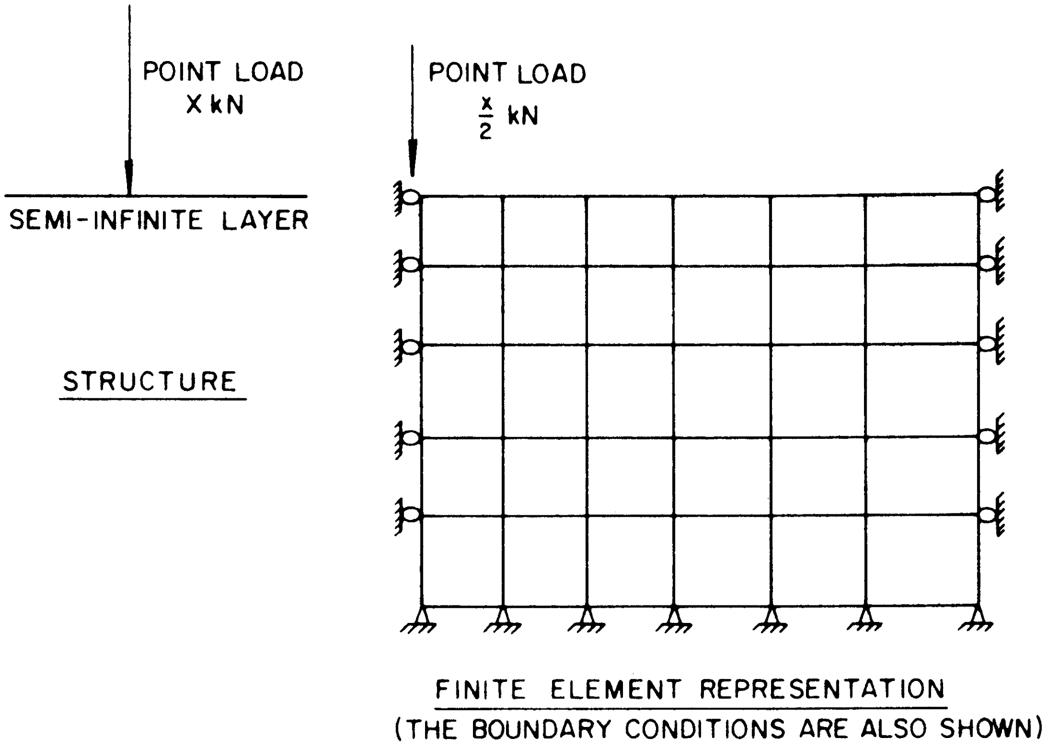


FIGURE B-1

THE LOADING OF AN AXISYMMETRIC STRUCTURE

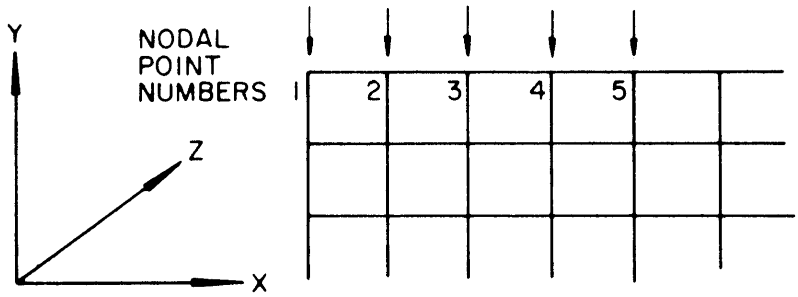


FIGURE B-2

CALCULATION OF CONCENTRATED NODAL POINT LOADS

B.2 OUTPUT

The output contains the nodal point displacements in the X-, Y- and Z-directions for each harmonic. The contributions of each harmonic to the displacement at the various individual nodal points are totalled and the total displacement of each nodal point and the stresses in each element at the specified longitudinal distances, are printed. The stresses given are σ_{xx}, σ_{yy}, σ_{zz}, τ_{xy}, τ_{xz} and τ_{yz}. The normal stresses (σ_{xx}, σ_{yy} and σ_{zz}) are parallel to the X-, Y- and Z-axes respectively, and they act on planes perpendicular to these axes, called the x-, y- and z-planes. τ_{xy}, τ_{xz} and τ_{yz} are the shearing stresses. The first subscript associates the shearing stress with a plane which is perpendicular to the axis designated by the subscript, while the second subscript designates the direction of the shearing stress, for example τ_{xy} is a shear stress on the x-plane in the y-direction.

The sign convention in the program is the normal one, namely tensile stresses are positive and compressive stresses are negative.

B.3 DIMENSIONS FOR INPUT AND OUTPUT

The original program by Wilson and Pretorius was metricated by using scale factors in the input and output format specifications. The nodal point forces should be specified in N and this is then changed to mN by the scale factor. The elastic moduli should be specified in MPa and this is converted to kPa while all distances should be specified in millimetres. The output dimensions are micrometres for the nodal point displacements and kPa for the stresses. The dimensions to be used in the metricated version of the program are summarized in Table B.1.

TABLE B.1 : Input and output dimensions

INPUT	OUTPUT
Nodal point force - N	Stress - kPa
Elastic modulus - MPa	Displacements - micrometres
Distance - mm	

B.4 COMPUTER PROGRAM INPUT

The first step is to select a finite element representation of the two-dimensional cross-section of the body. Elements and nodal points are then numbered in two numerical sequences, each starting with one. The following group of punched cards numerically defines the two-dimensional structure to be analyzed:

(i) Identification card - (18A4)

Columns 1 - 72 This card contains the heading or other relevant information which is to be printed with the results

(ii) Control card - (4I5, F10.2)

Columns	1 - 5	Number of nodal points (450 max)
	6 - 10	Number of elements (400 max)
	11 - 15	Number of different materials (12 max)
	16 - 20	Number of harmonics
	21 - 30	ZL - length

The ZL-length is half of the period length. The period length is the length to the next load.

(iii) Material property information (I5, 2F10.0)

One card for each material:

Columns	1 - 5	Material identification - any number from 1 to 12
	6 - 15	Elastic modulus (MPa)
	16 - 25	Poisson ratio

(iv) Nodal point data - (I5, F5.0, 5F10.0)

One card for each nodal point with the following information:

Columns	1 - 5	Nodal point number
	6 - 10	Code number
	11 - 20	X-ordinate (mm) (usually horizontal)
	21 - 30	Y-ordinate (mm) (usually vertical)
	31 - 40	UX
	41 - 50	UY
	51 - 60	Loaded length in Z-direction (i.e. half of the total length of the load). (If the nodal point is not loaded the value is zero.)

If the number in column 10 (code number) is

0, UX is specified X-load
UY is specified Y-load

NB. Load in Newton (N)

1, UX is specified X-displacement
UY is specified Y-load

2, UX is specified X-load
UY is specified Y-displacement

3, UX is specified X-displacement
UY is specified Y-displacement

All loads are total forces acting at the nodal point. Loaded length is half the total loaded length in the Z-direction. Nodal point cards must be in numerical sequence and the first and last nodal points must have cards. If cards are omitted, the omitted nodal points are generated at equal intervals along a straight line between the defined nodal points. The boundary code (column 10), UX, UY, and loaded length for the generated nodal points are all set equal to <u>zero</u>.

(v) <u>Element properties: (6I5)</u>

One card for each element.

Columns	1 - 5	Element number
	6 - 10	Nodal point I
	11 - 15	Nodal point J
	16 - 20	Nodal point K
	21 - 25	Nodal point L
	26 - 30	Material identification.

<u>NOTE</u>: (a) The nodal points I, J, K and L defining the element must be ordered <u>counter</u>-clockwise. The maximum difference between nodal point numbers defining any one element is 17.

(b) Element cards must be in numerical sequence. If cards are omitted, the information for the omitted elements is generated by incrementing by one the preceding, I, J, K and L. The material identification number for the generated elements is set equal to the value on the first card.

(c) No provision was made for triangular elements.

(vi) <u>Longitudinal distance: (F10.0)</u>

Column 1 - 10 ZZ - length.

This distance defines the cross-sections in the longitudinal (Z) direction where stresses (kPa) and strains are to be printed. Nodal point displacements and element stresses for the defined finite element mesh are obtained.

APPENDIX C

CALCULATION OF THERMAL CONDUCTIVITIES

Williamson (1972c) discussed various theories to calculate the thermal conductivity of composite materials, such as crusher-runs and asphaltic concretes, and concluded that "...the geometric mean equation is capable of undertaking such calculations to an accuracy of about 90 per cent...". The equation is

$$k_{mix} = (k_{agg})^m \cdot (k_{bind})^n \cdot (k_{void})^p \cdot (k_{water})^q \text{ ...... (C.1)}$$

where k_{mix} = thermal conductivity of the composite material

k_{agg} = thermal conductivity of the aggregate

k_{bind} = thermal conductivity of the binder: cement, lime or bitumen. Taken as 0,002 7 cal/cm.s.^{o}C for cement

k_{void} = thermal conductivity of air. Taken as 0,000 063 cal/cm.s.^{o}C

k_{water} = thermal conductivity of water. Taken as 0,001 5 cal/cm.s.^{o}C

m,n,p & q = percentage of unity for the various phases in the material, e.g. 0,9 and 0,04.

C.1 CRUSHER-RUN

Two rock types were considered for the crusher-run, quartzite and granite. Bullard (1939) gave the thermal conductivities of the rocks as about 0,016 and 0,006 5 cal/cm.s.^{o}C and the solid densities (specific weights) as 2,69 and 2,62 g/cc respectively.

The specifications for crusher-run basecourses call for compaction to 88 per cent of solid density. The remaining 12 per cent of the volume is taken up by air voids and water and these were assumed to be 8 and 4 per cent respectively. The moisture content may change during the day and the year, but 4 per cent moisture is a reasonable average value. The percentage binder equals zero, since an untreated crusher-run was considered.

Quartzite: $k_{mix} = (0{,}016)^{0{,}88} \cdot (0{,}000\ 063)^{0{,}08} \cdot (0{,}001\ 5)^{0{,}04}$

$= 0{,}009\ 3$ cal/cm.s.^{o}C

Granite : $k_{mix} = (0{,}006\ 5)^{0{,}88} \cdot (0{,}000\ 063)^{0{,}08} \cdot (0{,}001\ 5)^{0{,}04}$

$= 0{,}004\ 2$ cal/cm.s.^{o}C

Often a lower density is achieved in practice and, assuming the same moisture content, the void ratio would increase. To investigate the effect of a change in density 82 per cent of solid density was assumed:

Quartzite: $k_{mix} = (0{,}016)^{0{,}82} \cdot (0{,}000\ 063)^{0{,}14} \cdot (0{,}001\ 5)^{0{,}04}$
$= 0{,}006\ 7$ cal/cm.s.°C

Granite : $k_{mix} = (0{,}006\ 5)^{0{,}82} \cdot (0{,}000\ 063)^{0{,}14} \cdot (0{,}001\ 5)^{0{,}04}$
$= 0{,}003\ 2$ cal/cm.s.°C

In the previous calculations only two densities were considered. To study the full spectrum of densities and to evaluate the effect of small changes in water content and void ratio, Figure C.1 was prepared. In preparing the figure it was assumed that the sum of the percentages of solid density, void ratio and water content should equal 100 per cent. The figure shows the density and rock type to be important but the water content and the void ratio has a relatively minor effect. The reason for this is that both air and water have relatively low thermal conductivities.

C.2 BITUMINOUS MATERIALS

Williamson (1972) reported the thermal conductivity of an asphaltic concrete with quartzitic aggregate as 0,007 1. The bitumen content was about 5 per cent and the void ratio was about 11 per cent. Saal et al (1940) reported the thermal conductivity of bitumen as about 0,000 4. When a dry mixture is assumed, that is water content equals zero, the percentage of solid density of the aggregate (m) can be calculated from equation (C.1) as 0,84 that is 84 per cent of solid density. The applicability of the equation was again confirmed by these calculations since the volume percentages totalled a hundred.

The void ratio of the abovementioned mixture was high since the specifications usually call for it to be between 2 and 6 per cent. A more practical mix would be one with only 5 per cent voids and hence a higher percentage of solid density for the aggregate. The thermal conductivity of such a mix was calculated as

$$k_{mix} = (0{,}016)^{0{,}9} \cdot (0{,}000\ 4)^{0{,}05} \cdot (0{,}000\ 063)^{0{,}05}$$
$$= 0{,}01 \text{ cal/cm.s.}^{\circ}\text{C}$$

When applying the equation to the same mixture, but with a granitic aggregate, the conductivity was calculated as 0,004 5 cal/cm.s.°C.

To reduce the number of variables the average values for bituminous materials quoted by Williamson (1972) will be used in the analyses. These are 0,007 cal/cm.s.°C and 2,2 g/cc.

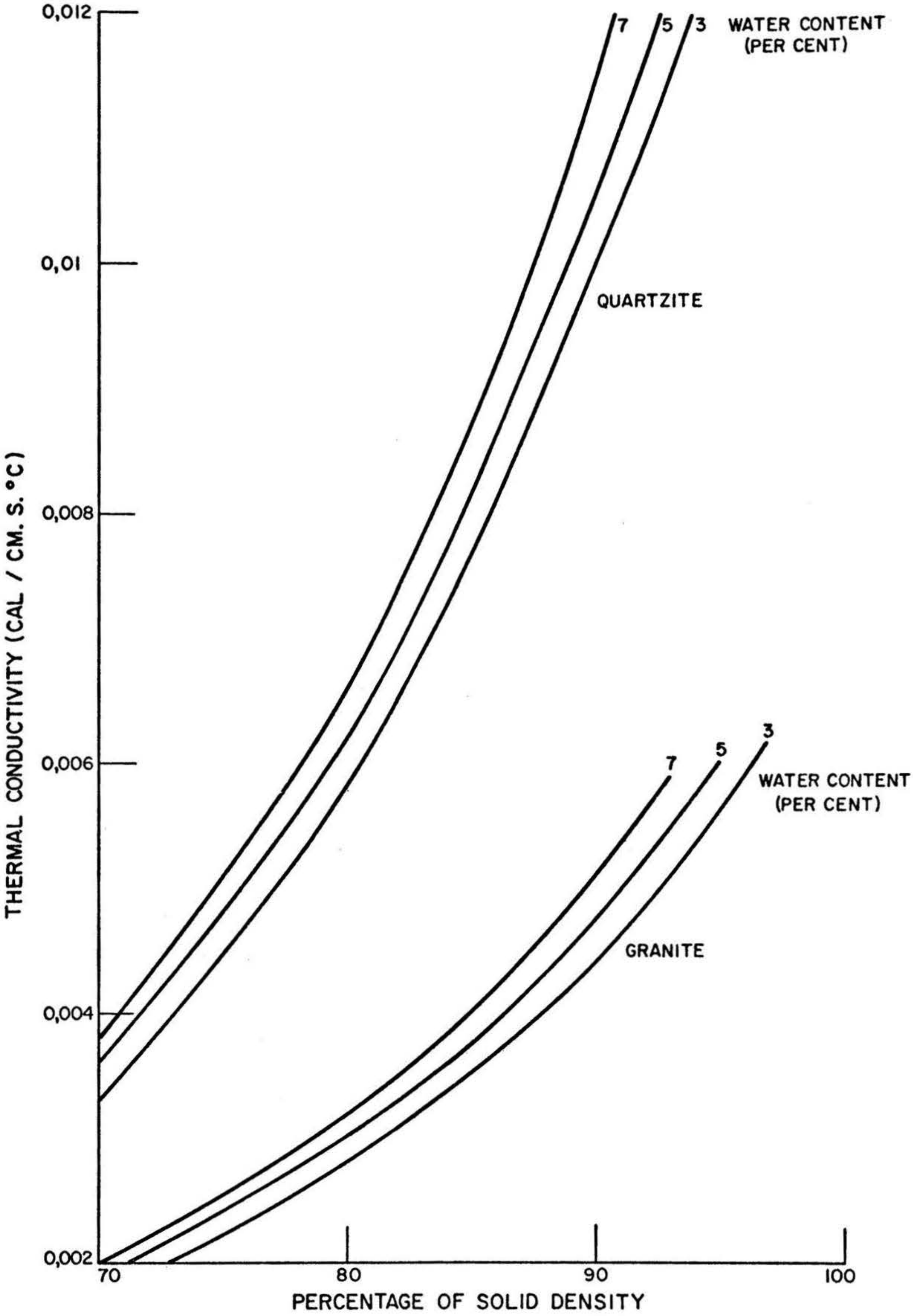


FIGURE C-1

EFFECT OF DENSITY, WATER CONTENT AND ROCK TYPE ON THE THERMAL CONDUCTIVITY OF CRUSHER-RUN

C.3 CONCLUSIONS

(a) The thermal conductivity of quartzitic and granitic crusher-run basecourses should be taken as 0,009 3 and 0,004 2 cal/cm.s.^{o}C when compacted to 2,36 and 2,31 g/cc respectively.

(b) At lower densities (2,2 and 2,14 g/cc) the conductivities should be reduced to 0,006 7 and 0,003 2 cal/cm.s.^{o}C.

(c) The effect of changes in void ratio and water content seems to be fairly small and for the sake of minimizing the number of variables, only 4 per cent moisture was considered.

(d) The thermal conductivity of an asphaltic concrete made with quartzitic and granitic aggregates was calculated as 0,010 and 0,004 5 cal/cm.s.^{o}C respectively. Both the void ratio and bitumen content were fixed at 5 per cent. In Chapter 5 an average value was used, namely 0,007 cal/cm.s.^{o}C.

APPENDIX D

DATA REQUIRED TO ANALYSE THE PAVEMENTS OF ROADS S12 AND N4/1

To model the pavement for the computer program it was subdivided into 21 nodes; one 25 mm node for the sufacing, four 25 mm nodes for each of the next four layers, and an additional four 75 mm nodes for the selected subgrade. The other information was described in Chapter 5, but the material properties were changed according to Table D.1.

TABLE D.1 : Material properties

PROPERTY	BITUMINOUS MATERIAL	CEMENT-TREATED CRUSHER-RUN (QUARTZITE)	CRUSHER-RUN (QUARTZITE)	CEMENT-TREATED NATURAL GRAVEL	SUBGRADE
Conductivity	0,010	0,010 8	0,009 3	0,005	0,002 5
Specific heat	0,2	0,2	0,2	0,2	0,19
Density (g/cc)	2,3	2,36	2,36	2,0	1,85
Elastic modulus (MPa)	3 000	18 500	500	4 000	150

The tensile strength and coefficient of linear expansion of the cement-treated crusher-run were 690 kPa and $8.10^{-6}/^{o}C$ respectively (Otte, 1974, and Williamson, 1972).

The thermal conductivity of the cement-treated quartzitic crusher-run was calculated according to equation (C.1) given in Appendix C

$$k_{mix} = (0,016)^{0,88} . (0,002\ 7)^{0,04} . (0,000\ 063)^{0,04} . (0.001\ 5)^{0,04}$$

$$= 0,010\ 8\ \text{cal/cm.s.}^{o}C$$

In this calculation compaction to 88 per cent of solid density was assumed and 4 per cent for each of the other three components.

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