

CHAPTER 1

INTRODUCTION - HISTORY AND DEVELOPMENT

	<u>PAGE</u>
1.1 INTRODUCTION	2
1.2 DESIGN PROCEDURES	2
1.2.1 California Bearing Ratio	2
1.2.2 State of California	3
1.2.3 AASHO design procedure	4
1.2.4 Theoretical procedure	5
1.3 FAILURE AND DISTRESS MODES	7
1.4 SUMMARY	7
1.5 DISCUSSION	8

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

$$GE = 0,003 2 (TI) (100 - R) \dots\dots\dots (1.1)$$

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 sub-base.	.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

	<u>PAGE</u>
2.1 INTRODUCTION	11
2.2 DESIGN REQUIREMENTS	11
2.2.1 Layer thickness	11
2.2.2 Material equivalency	12
2.2.3 Sound foundation	14
2.2.4 Balanced pavement design	16
2.2.5 Insulation against thermal stresses	19
2.2.6 Non-traffic-associated cracking	19
2.2.7 Design traffic	20
(a) Wheel load	
(b) Load equivalency factors	
(c) Number of load repetitions	
2.2.8 Testing method	22
(a) Direct tensile test	
(b) Indirect tensile test	
(c) Bending test	
2.2.9 Design criteria	26
2.2.10 Fatigue life	30
2.2.11 Properties of field- versus laboratory-prepared materials	33
2.2.12 Variability of materials	34
2.3 CONCLUSIONS	35

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 mm 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.

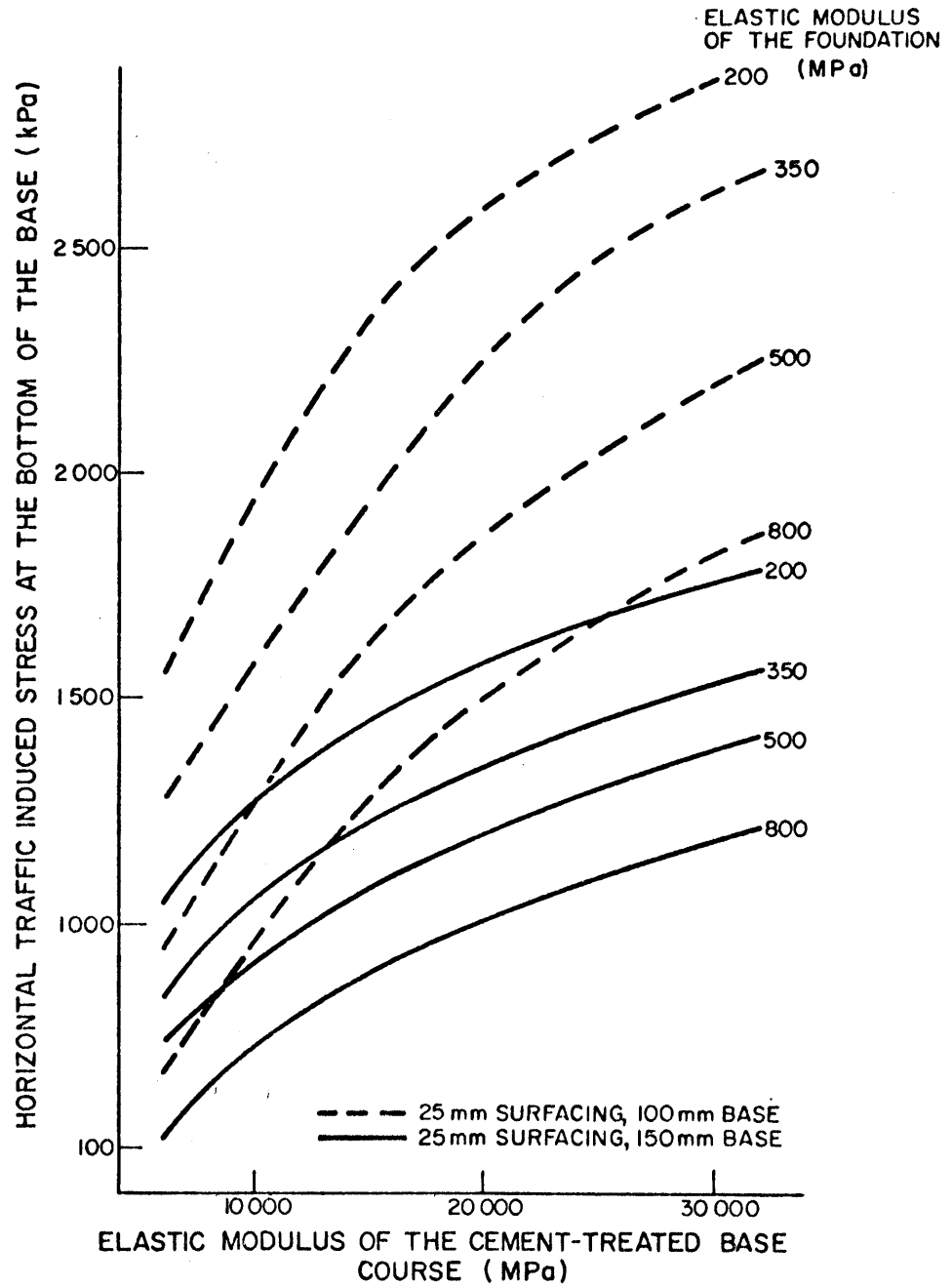


FIGURE 2-1

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

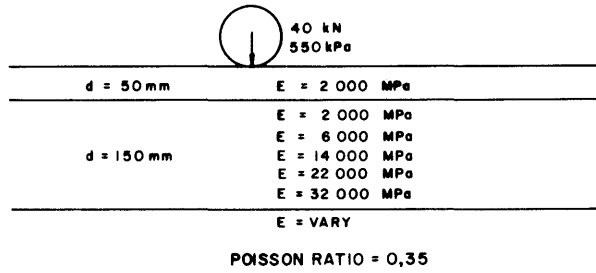


FIGURE 2-2
PAVEMENT DESIGN USED TO DETERMINE
THE INFLUENCE OF THE ELASTIC MODULUS
OF THE FOUNDATION

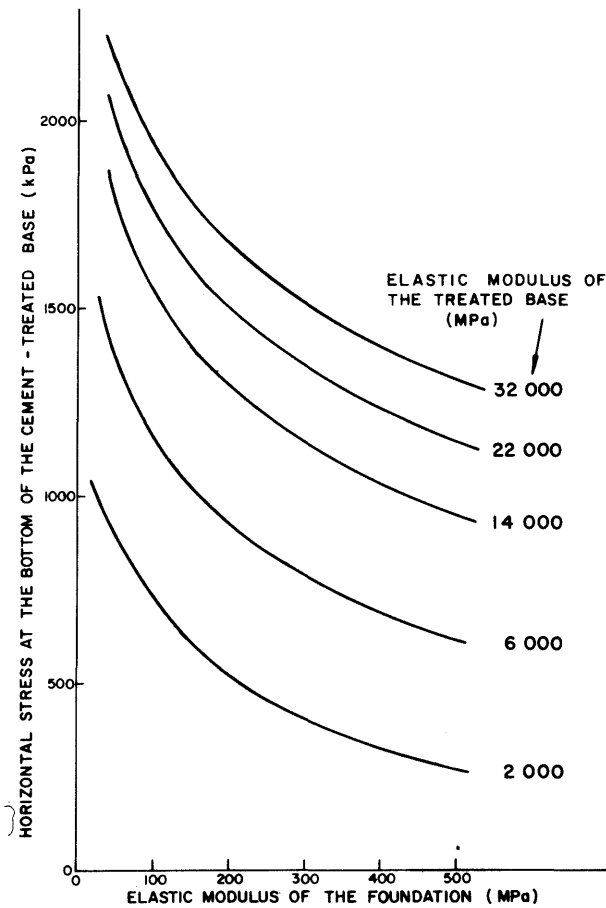


FIGURE 2-3
THE INFLUENCE OF THE ELASTIC MODULUS OF THE
FOUNDATION ON THE TENSILE STRESSES IN THE BASE

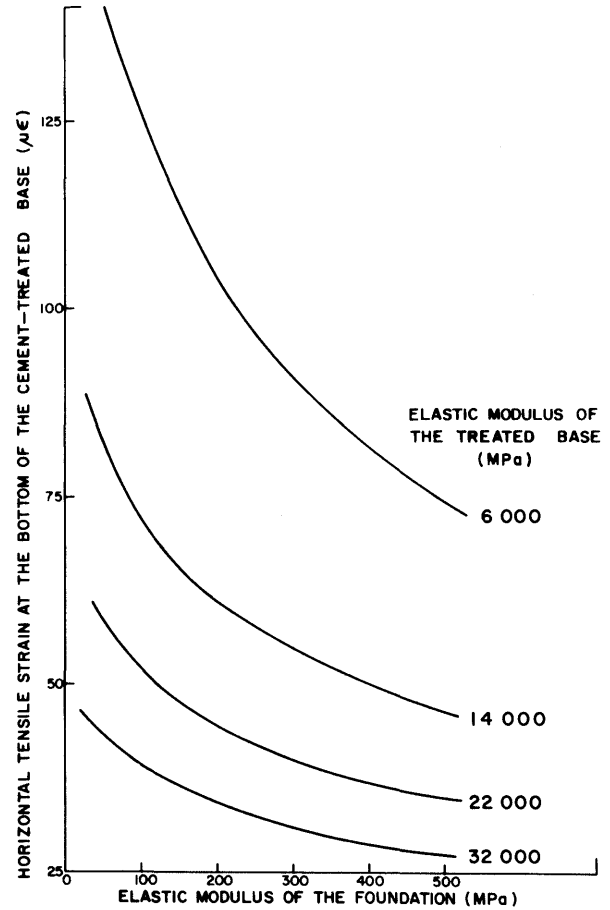


FIGURE 2-4
THE INFLUENCE OF THE ELASTIC MODULUS OF THE
FOUNDATION ON THE TENSILE STRAIN IN THE BASE

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,

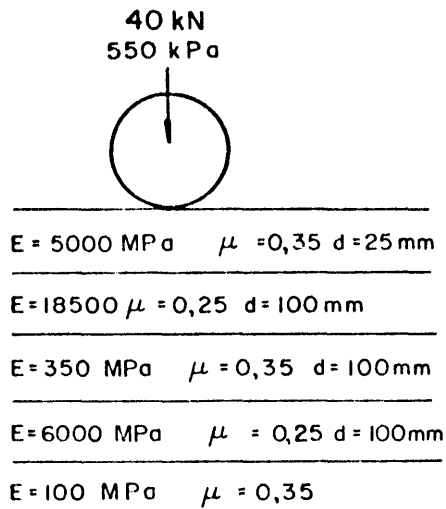
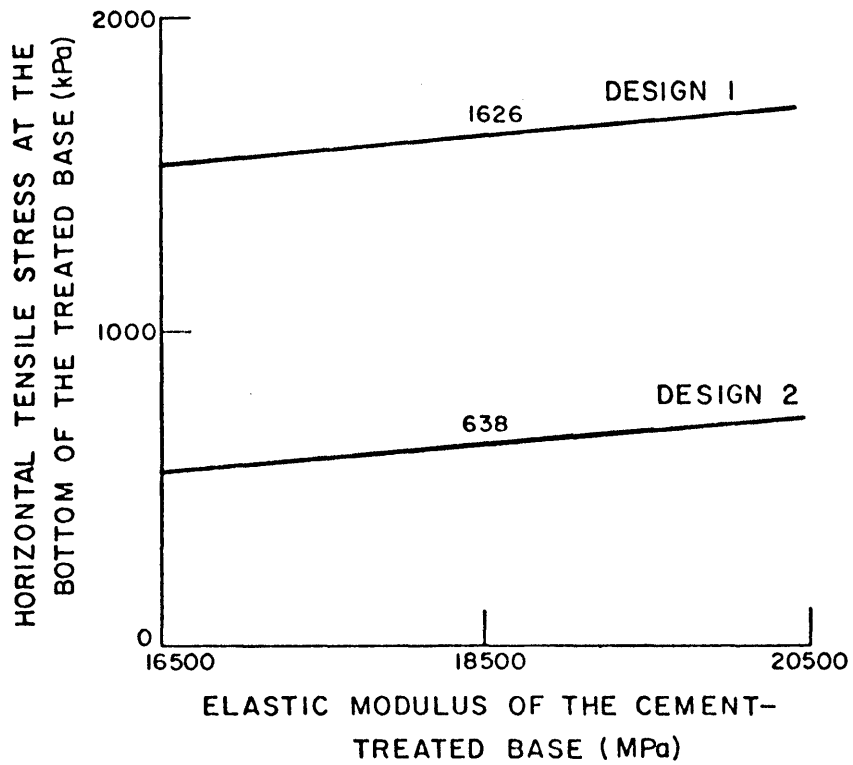
and also where and why the future distress may occur and possible ways to avoid it or to extend the life of the pavement. Both of these, elastic theory and balanced design, can therefore be used to help avoid an early pavement failure.

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!

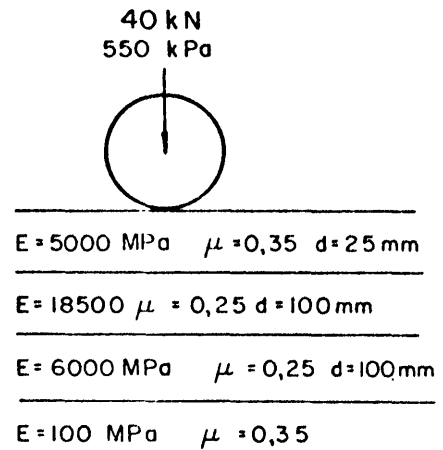
Crusher-run bases of about 200 mm were always very popular in some parts of the country, mainly because of their very acceptable performance and the easy supply of unweathered crushed rock from the gold mines. The unacceptable performance of a cement-treated base on a major road carrying very heavy traffic (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



DESIGN 1



DESIGN 2

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 °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.

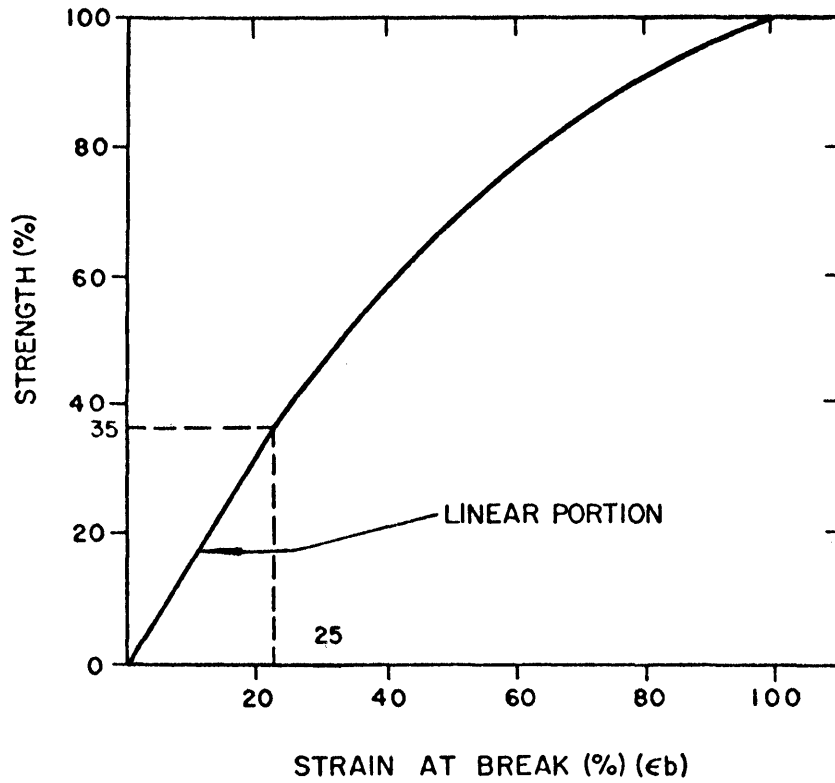


FIGURE 2-6
*THE GENERAL STRESS - STRAIN CURVE OF
CEMENT-TREATED MATERIALS*

(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

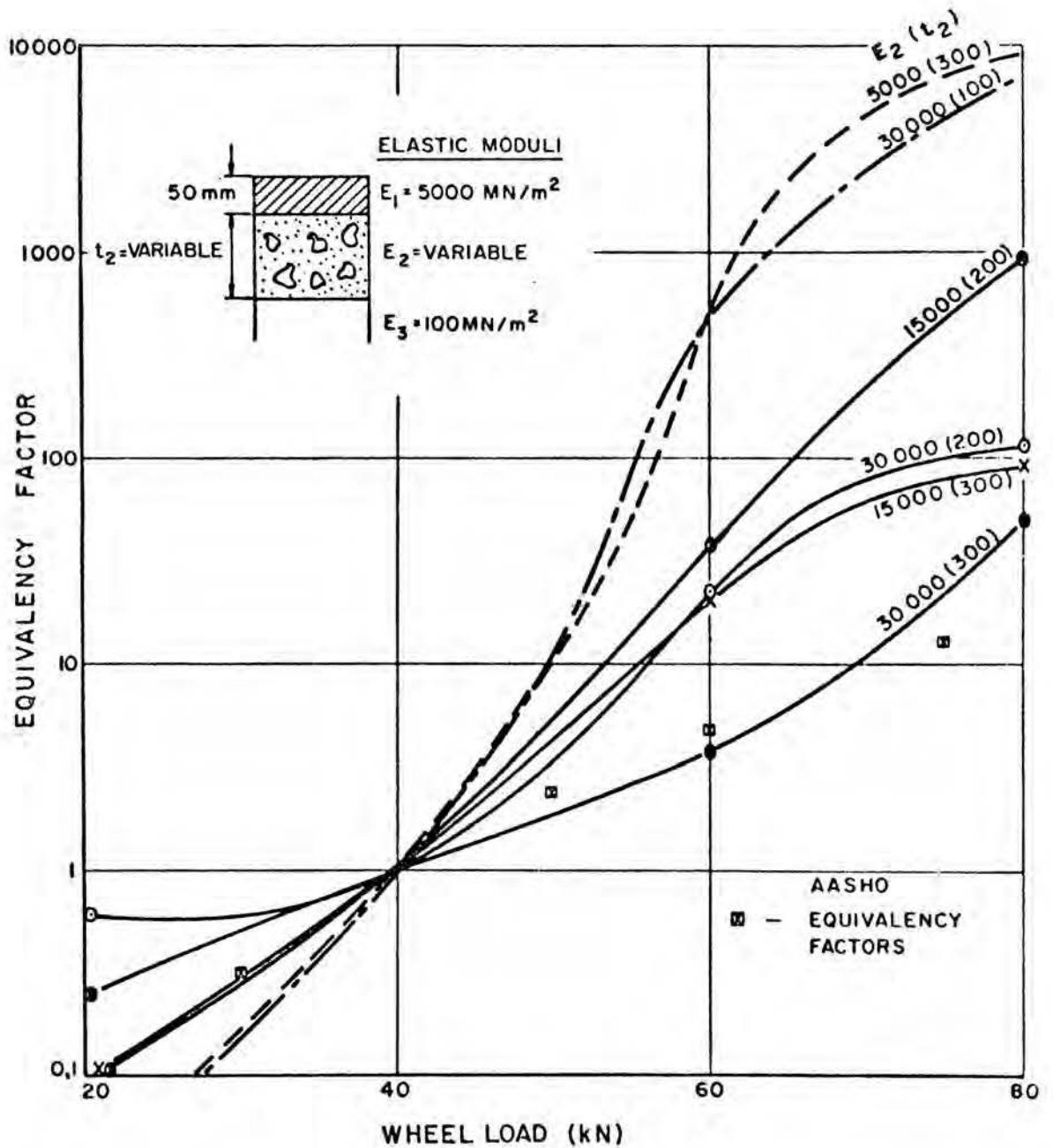


FIGURE 2-7
RELATIONSHIP BETWEEN WHEEL LOAD AND
EQUIVALENCY FACTOR FOR VARIOUS CEMENT-
TREATED PAVEMENTS
(After Van Vuuren, 1972)

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

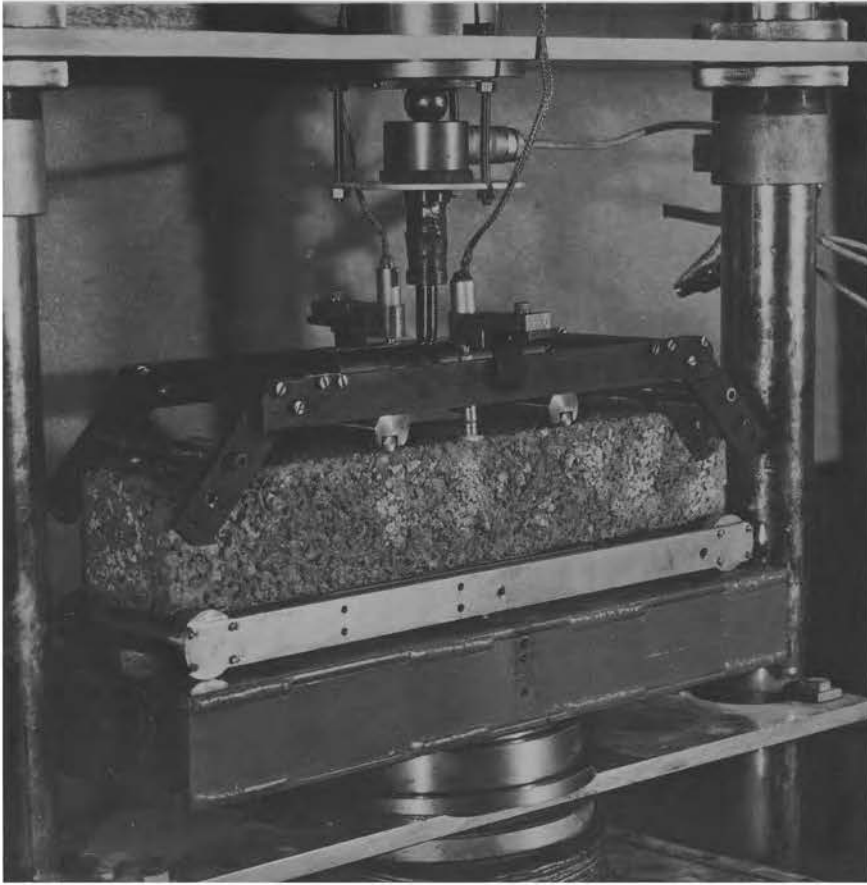


FIGURE 2.8
The bending test apparatus.

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:

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

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

$\sigma_c =$ unconfined compressive strength

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

σ_1, σ_2 and $\sigma_3 =$ 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

$$\sigma_1 = \sigma_c + 5\sigma_3 \dots\dots\dots (2.2)$$

- 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 $\sigma/\sigma_b=0,5$. 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 ($\sigma/\sigma_b=0,5$) 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 (ϵ/ϵ_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 (ϵ/ϵ_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

$$\epsilon/\epsilon_b = 1 - 0,11 \log N_f \dots\dots\dots (2.3)$$

If a linear relationship between the logarithm of the strain ratio (ϵ/ϵ_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

$$\epsilon/\epsilon_b = N_f^{-0,079} \dots\dots\dots (2.4)$$

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

$$N_f = 10^{9,1(1-\epsilon/\epsilon_b)} \dots\dots\dots (2.5)$$

and
$$N_f = (\epsilon/\epsilon_b)^{-12,66} \dots\dots\dots (2.6)$$

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

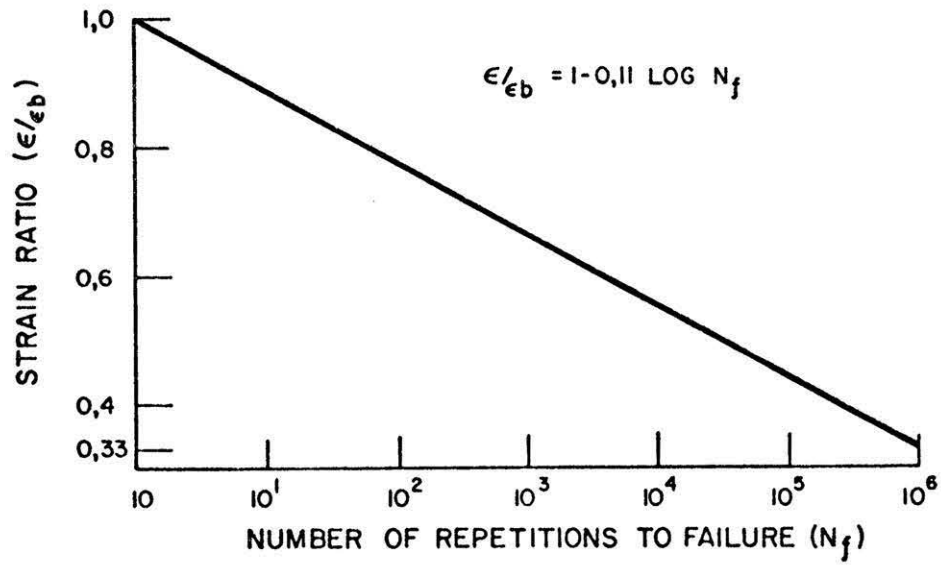


FIGURE 2-9a

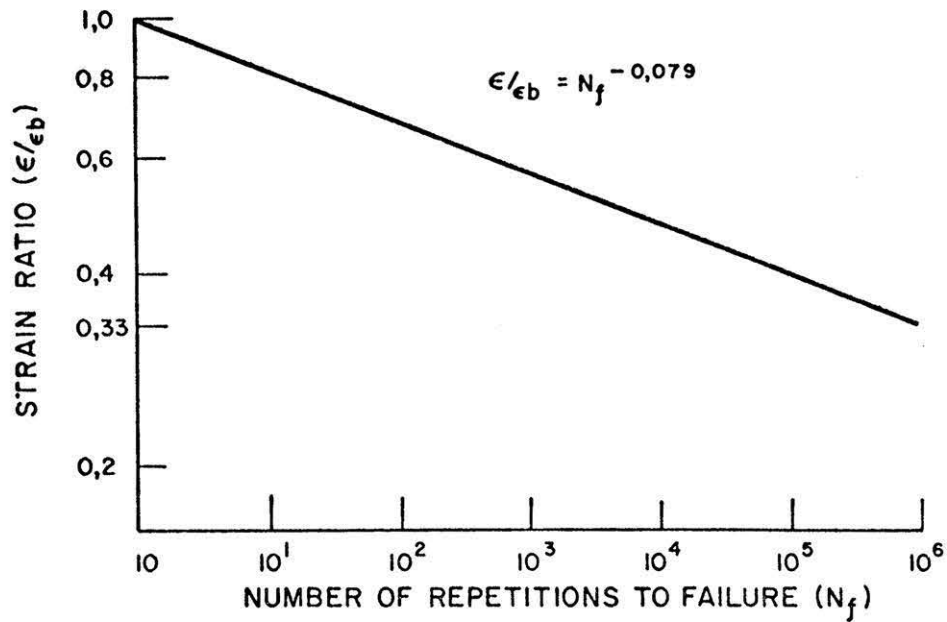


FIGURE 2-9b

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

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

Note: + for compressive stresses
 - for tensile stresses

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

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.

Kllhn 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

	<u>PAGE</u>
3.1 INTRODUCTION	38
3.2 TYPES OF CRACKING	38
3.2.1 Initial cracking	39
3.2.1.1 The effect of construction on initial cracks	
3.2.1.2 Reflection of initial cracks	
3.2.1.3 Thermal stresses and initial cracking	
3.2.1.4 Initial cracks and ingress of water	
3.2.1.5 Pavement response and initial cracking	
3.2.1.6 Cracking and materials characterization	
3.2.1.7 Design to accommodate initial cracks	
3.2.2 Traffic-associated cracking	48
3.3 DISCUSSION	50
3.4 CONCLUSIONS	51

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 temperature 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 ($\mu\epsilon$)*	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. [*] $\mu\epsilon$ 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.

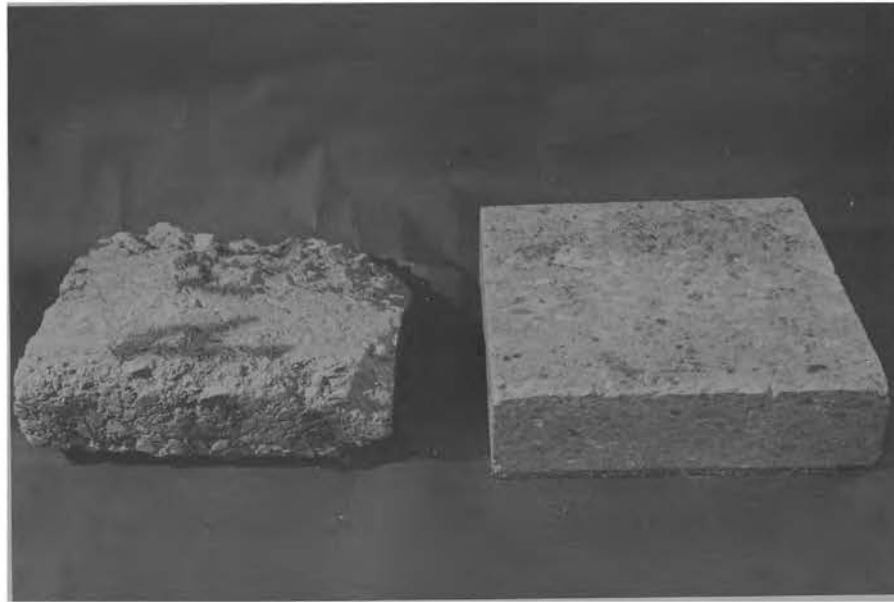


Figure 3.3 (a)

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 over-emphasized. 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 Bronkhorstspuit, 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 overstressed 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 irrelevant. 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 from tests on intact samples.
- (e) It is considered possible to prevent traffic-associated cracking by applying an appropriate design procedure.