CHAPTER 8

A DESIGN PROCEDURE FOR PAVEMENTS WITH CEMENT-TREATED LAYERS

PAGE

8.1 INTRODUCTION

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

8.2 THE DESIGN PROCEDURE

8.2.1 Design objective and approach

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

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

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

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

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

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

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

8.2.2 The flow diagram

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

8.2.3 Design traffic and tyre contact pressure

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

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

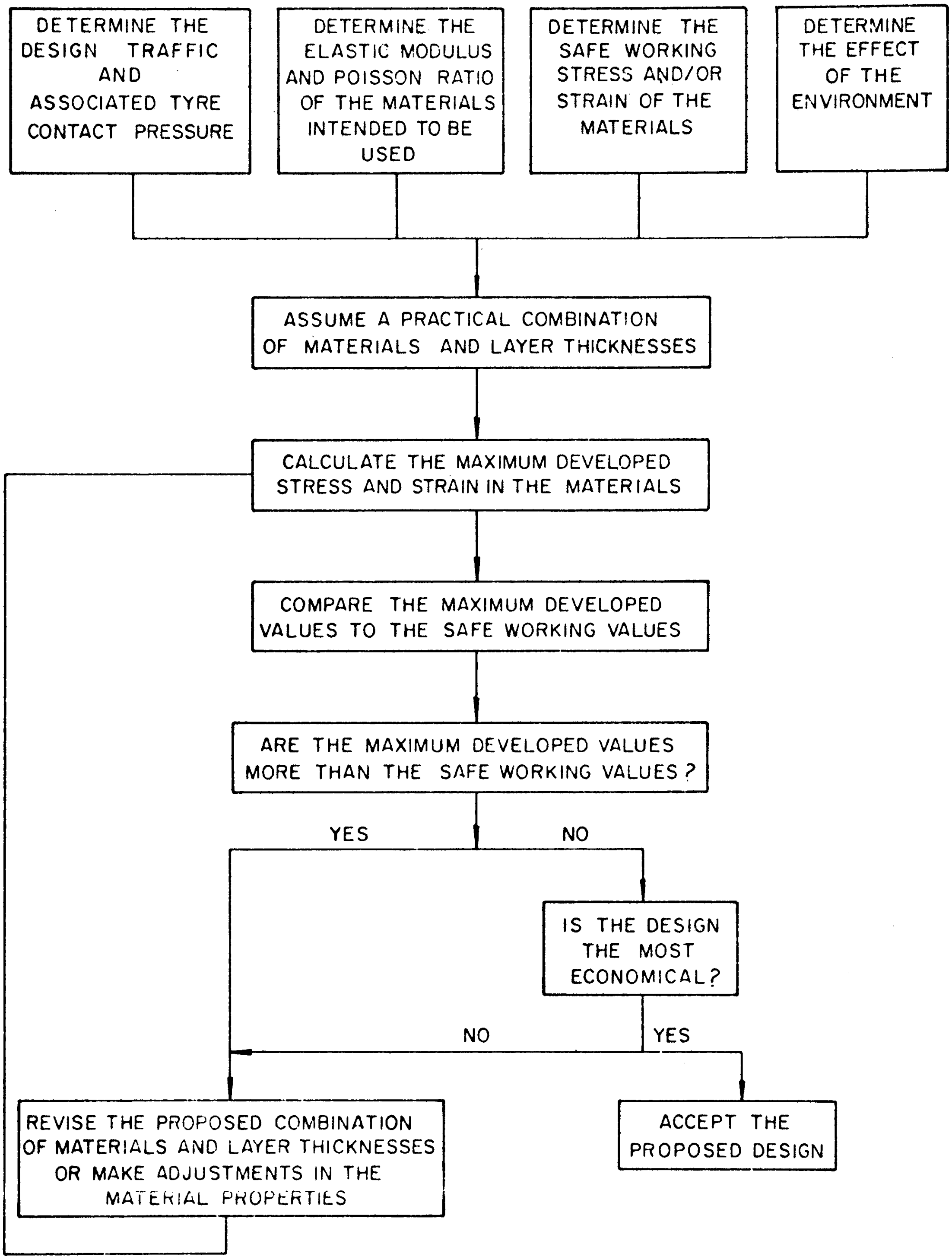

FIGURE 8-1

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

(Based on Brown and Pell, 1972)

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

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

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

TABLE 8.1 : Definition of design traffic

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

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

8.2.4 Elastic modulus and Poisson ratio of materials

8.2.4.1 Bituminous materials

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

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

8.2.4.2 Untreated crusher-run

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

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

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

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

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

8.2.4.3 Cement-treated materials

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

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

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

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

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

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

Cement-treated crusher-run : $E_b = 8\ \sigma_b + 3\ 500$ ..... (8.4)
Cement-treated natural gravel: $E_b = 10\ \sigma_b + 1\ 000$ .... (8.5)

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

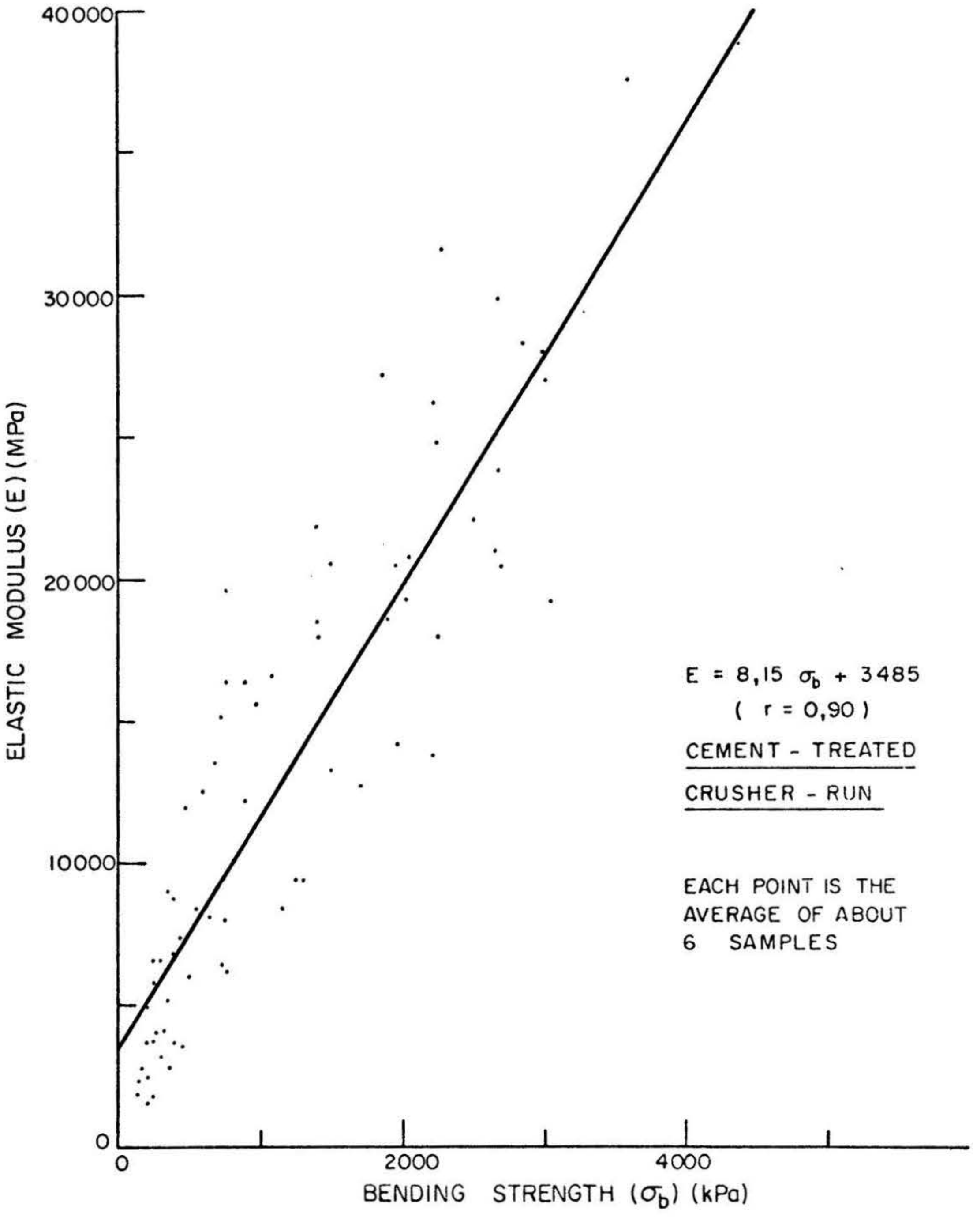

FIGURE 8-2

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

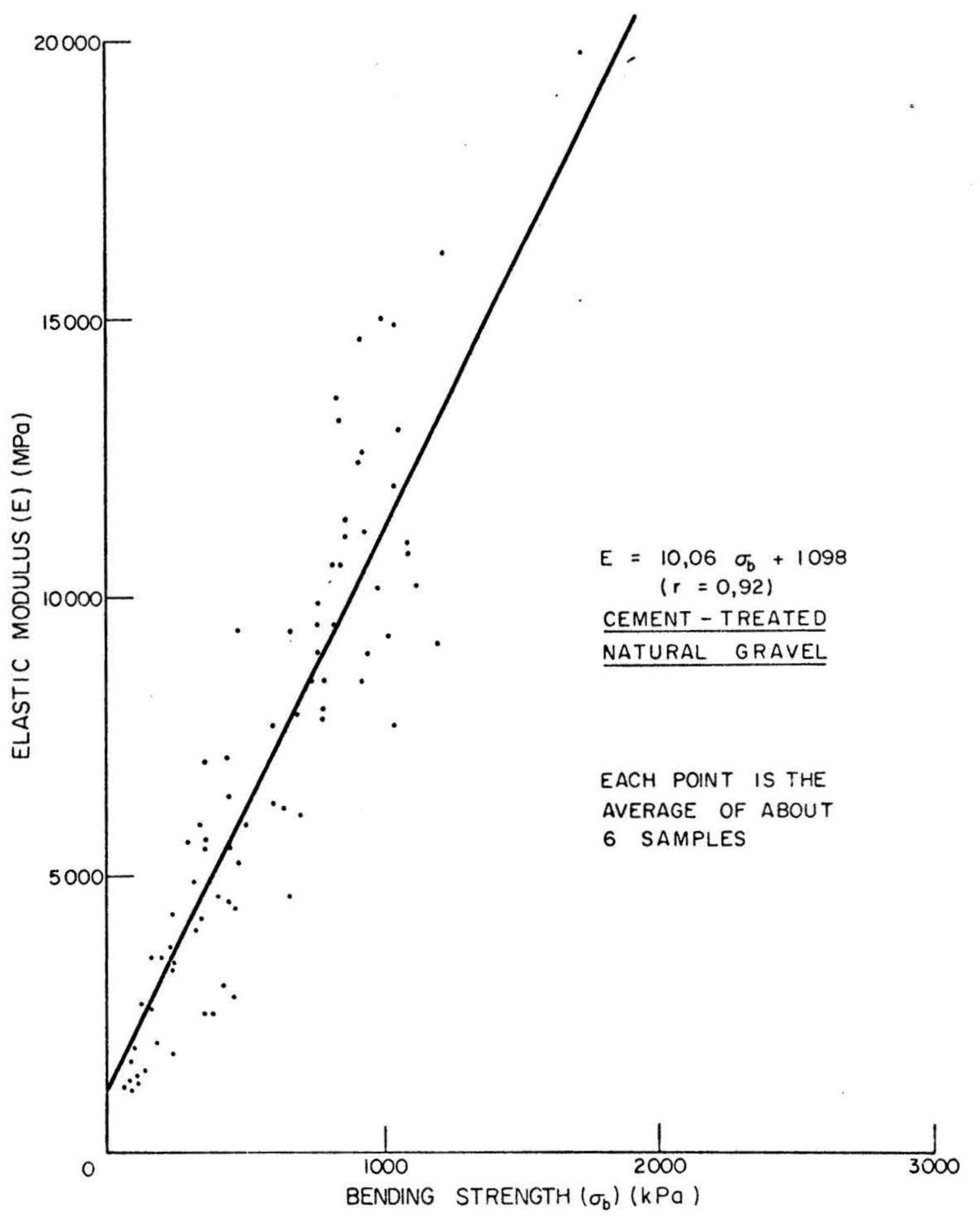

FIGURE 8-3

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

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

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

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

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

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

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

where K_f = material constant

A = unconfined compressive strength in psi

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

C = cement content by weight (per cent)

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

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

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

8.2.4.4 Foundation and untreated soils

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

$$E = k \cdot CBR \qquad (8.10)$$

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

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

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

8.2.5 Safe working stress and/or strain of materials

8.2.5.1 Bituminous material

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

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

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

8.2.5.2 Cement-treated materials

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

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

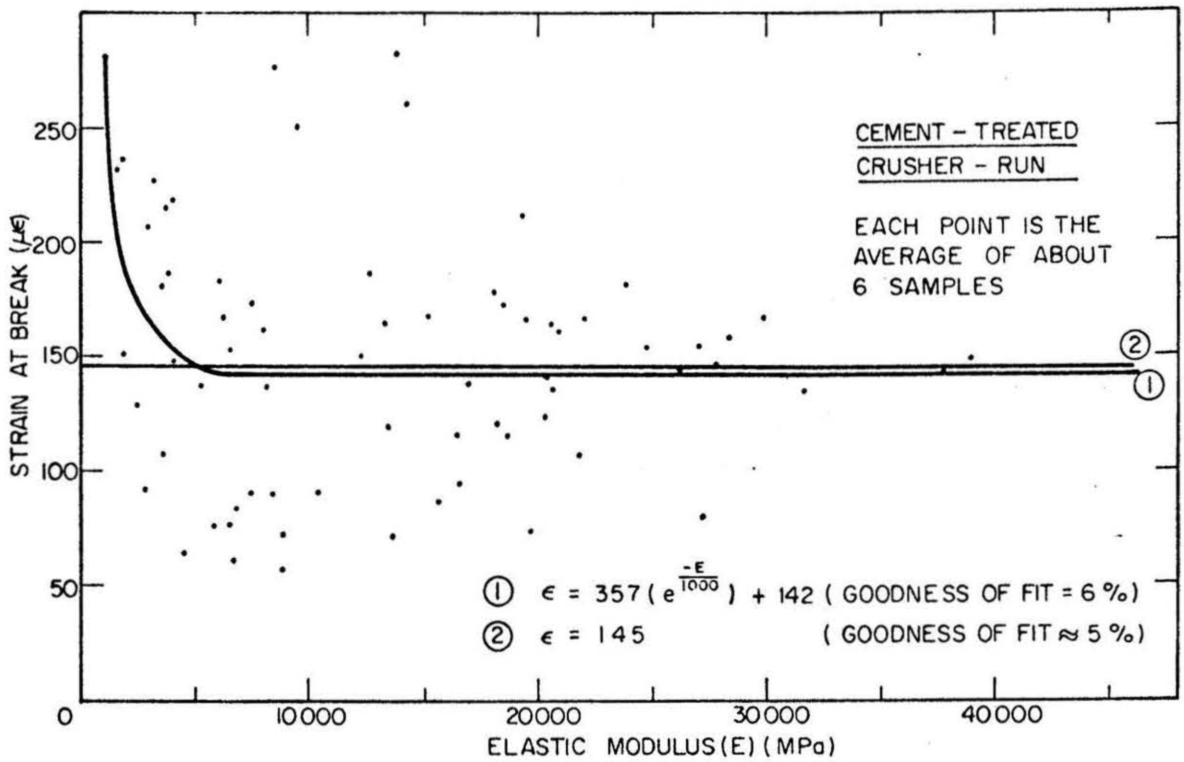

FIGURE 8-4

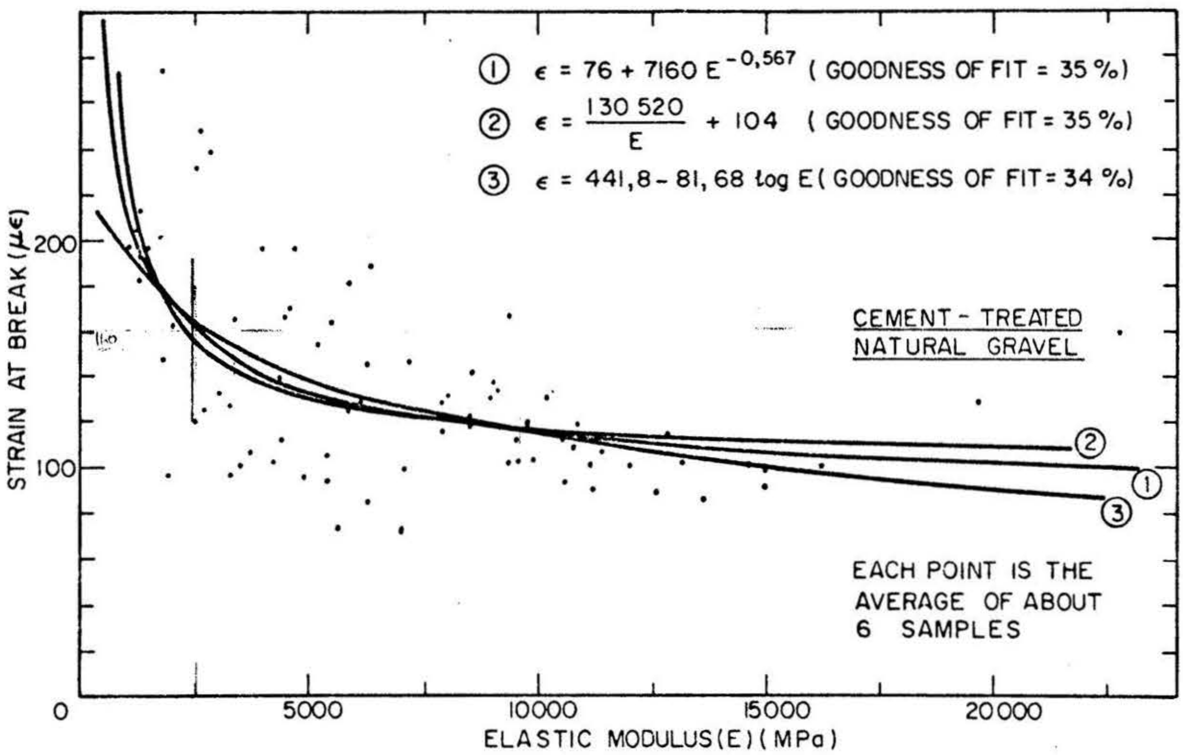

FIGURE 8-5

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

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

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

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

8.2.5.3 Subgrade materials

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

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

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

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

8.2.5.4 Untreated granular materials

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

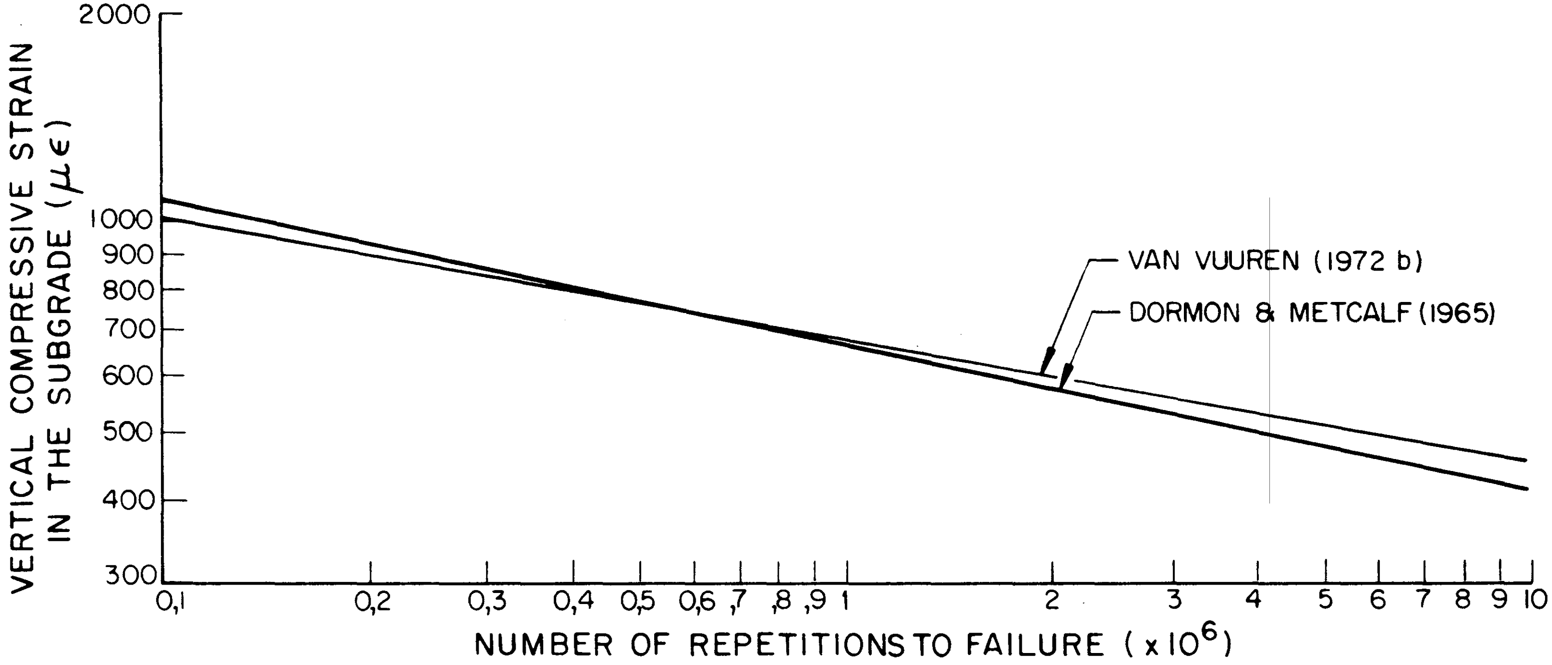

FIGURE 8-6

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

8.2.6 Effect of environment

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

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

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

8.2.7 Selection of a practical combination of materials and layer thicknesses

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

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

8.2.8 Maximum developed stresses and strains in the materials

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

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

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

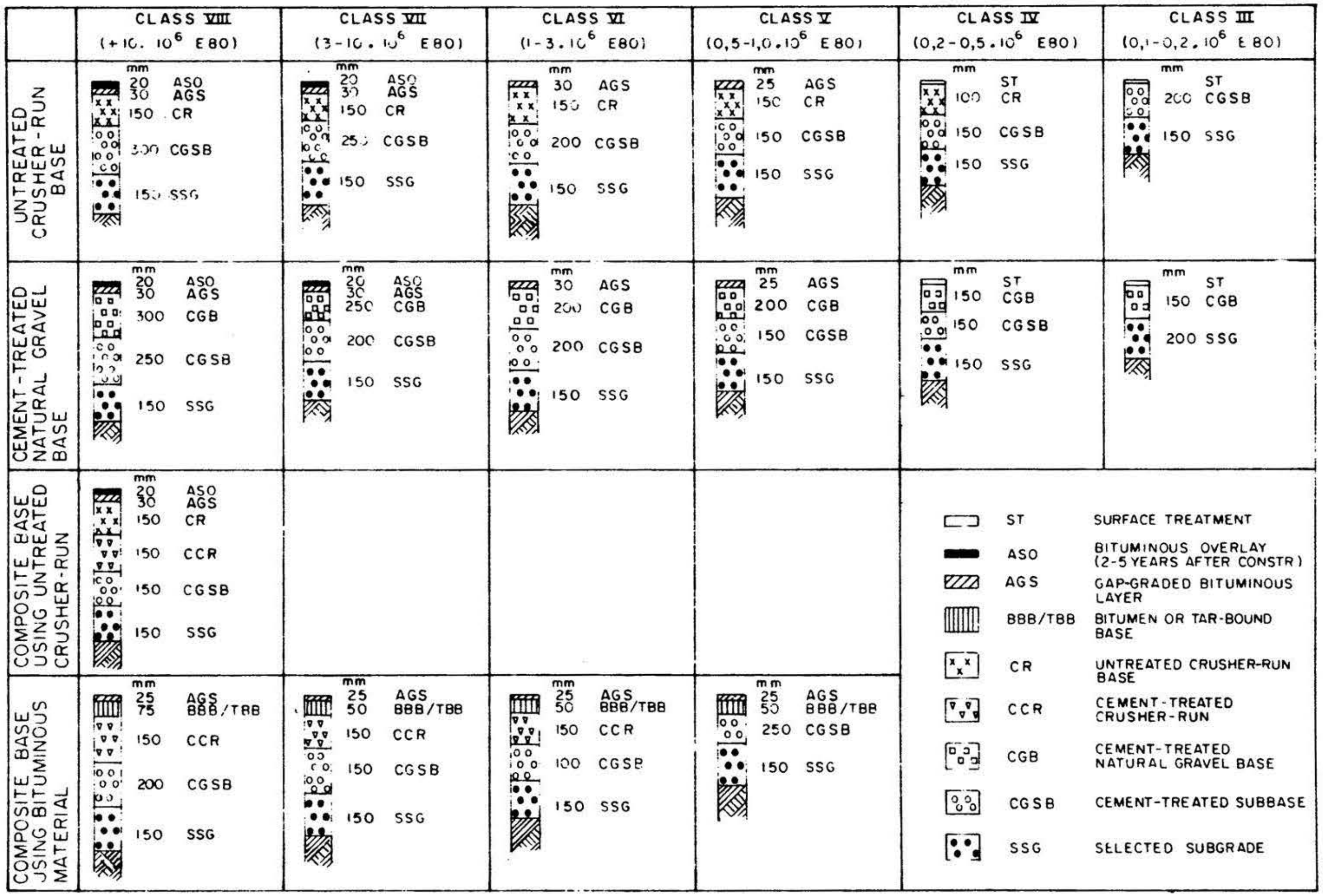

FIGURE 8-7

SUGGESTED LAYOUTS FOR CEMENT-TREATED LAYERS IN PAVEMENTS

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

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

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

(c) the proposed layer thicknesses.

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

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

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

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

8.2.9 Comparison of maximum developed values with safe working values

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

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

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

8.2.10 The post-crack phase

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

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

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

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

8.2.11 Crack-propagation phase

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

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

8.3 VARIATIONS IN DESIGN PROCEDURE

8.3.1 Accommodating mixed traffic

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

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

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

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

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

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

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

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

8.3.2 Use of standard designs

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

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

8.4 VERIFICATION AND ACCURACY OF PROCEDURE

8.4.1 Pavement response

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

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

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

FIGURE 8-8

CONSERVATIVE LAYOUTS FOR CEMENT-TREATED LAYERS IN PAVEMENTS.

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

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

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

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

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

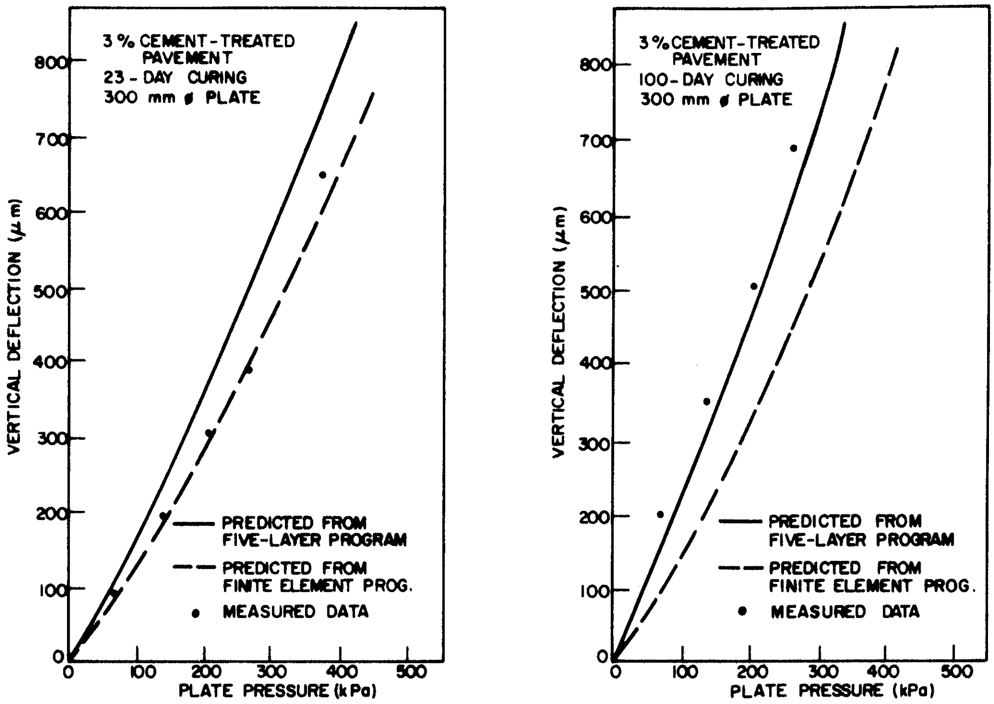

FIGURE 8-9

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

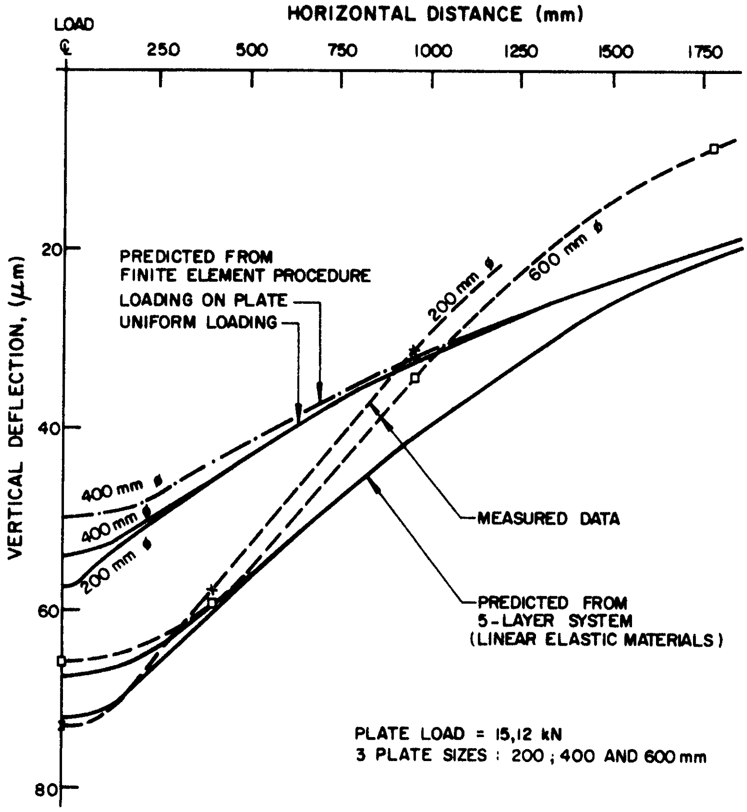

FIGURE 8-10

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

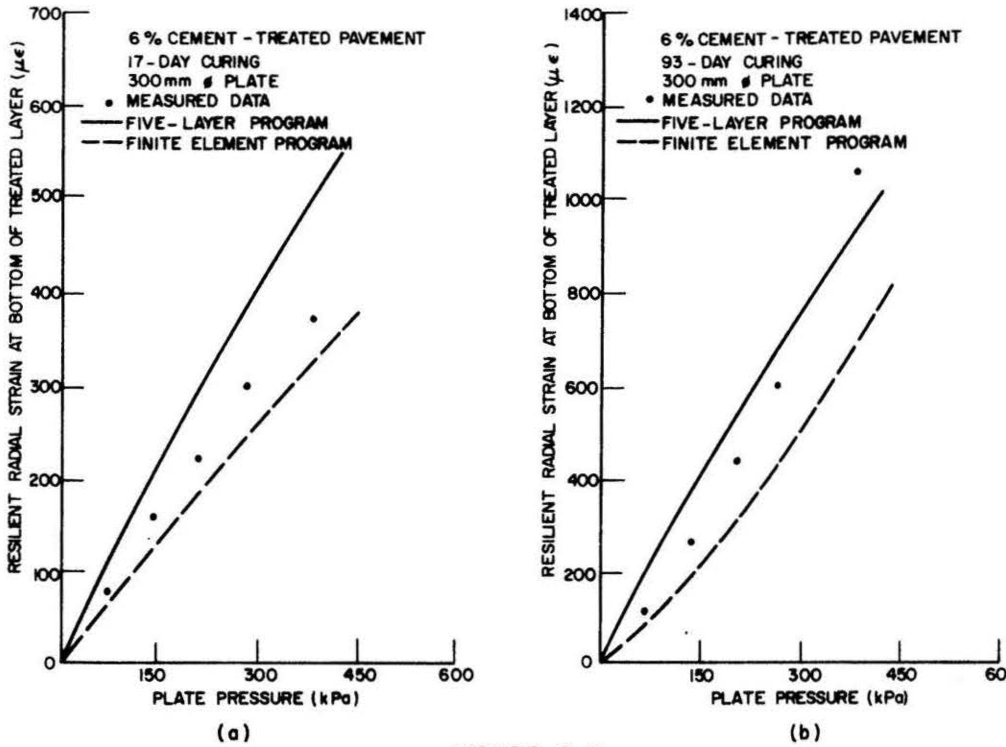

FIGURE 8-11

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

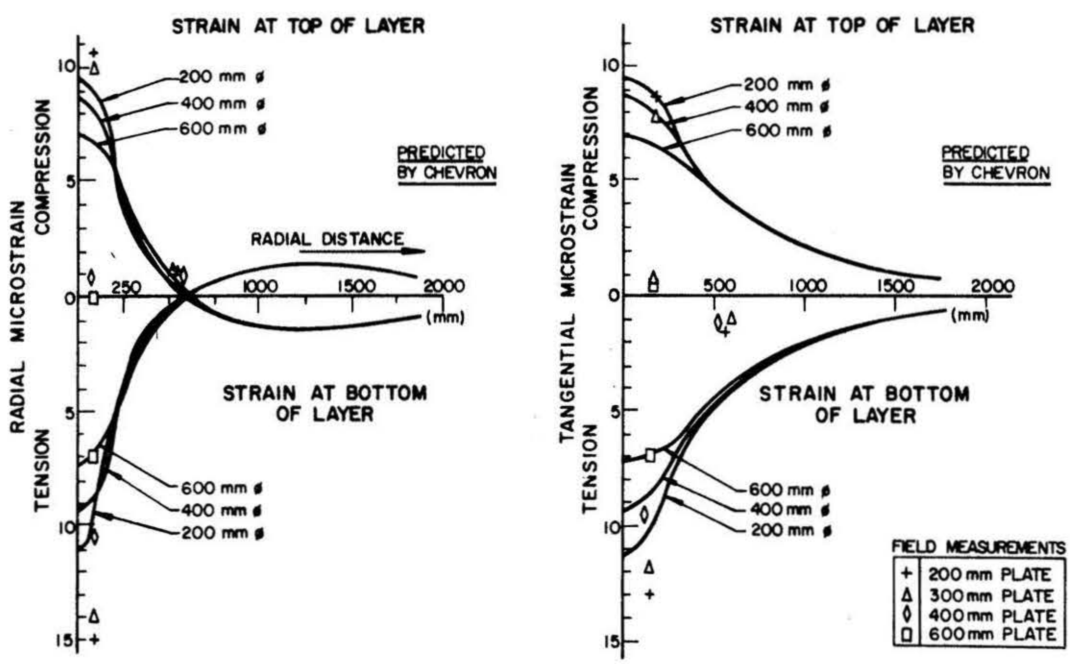

FIGURE 8-12

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

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

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

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

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

8.4.2 Pavement behaviour

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

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

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

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

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

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

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

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

8.5 EXAMPLES

8.5.1 Comparison with State of California procedure

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

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

The suggested procedure (Figure 8.1) is as follows:

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

Elastic moduli and Poisson ratio (section 8.2.4):

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

Safe working strain values:

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

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

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

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

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

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

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

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

8.5.2 Comparison with PCA procedure for industrial pavements

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

The design data are:

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

Straddle carrier: (load A)

wheel load	: 127 kN
tyre contact pressure	: 983 kPa
number of repetitions (n_a)	: 300 000

Forklift: (load B)

equivalent single wheel load	: 160 kN
tyre contact pressure	: 760 kPa
number of repetitions (n_b)	: 25 000.

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

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

Asphaltic concrete:	5 000 MPa
Soil-cement	: 9 000 MPa
Subgrade (CBR=3)	: 30 MPa

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

8.5.3 Upside-down design

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

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

Design traffic:

Bituminous layer : 15 million load applications (T)

Granular materials : 10 million E80

Cement-treated layers: 5 000 repetitions (n) of an 80 kN wheel load (about one 80 kN wheel per day for 10 years).

Tyre contact pressure: 520 kPa.

Elastic moduli and Poisson ratio:

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

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

Allowable design values:

Bituminous materials : ε_t = 300 με (Table 8.2)

Cement-treated crusher-run: 86 με (strain ratio corresponding to n = 5 000 is 0,59; ε_b assumed to be 145 με)

Cement-treated subbase : 103 με (ε_b taken as 175 με; Figure 8.5)

Subgrade : ε_c = 460 με (Table 8.3)

Granular material : zero tensile strain at bottom of layer.

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

Suggested layout: (Figure 8.7)

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

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

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

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

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

Recommendation:

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

8.5.4 Analysis of standard design

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

TABLE 8.6 : Expected traffic distribution on a proposed road

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

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

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

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

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

Safe working strains and strain at break (ε_b):

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

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

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

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

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

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

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

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

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

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

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

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

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

8.5.5 Airport pavement

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

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

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

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

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

Safe working strains:

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

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

Suggested practical layout:

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

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

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

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

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

The layout now changes to -

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

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

The layout now changes to -

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

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

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

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

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

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

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

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

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

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

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

8.6 RECOMMENDATION

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

CHAPTER 9

CONCLUSIONS AND RECOMMENDATIONS

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

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

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

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

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

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

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

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

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

RECOMMENDATIONS FOR FURTHER WORK

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

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

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

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

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

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