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FLEXIBLE PAVEMENT DESIGN

by

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NOTATION AND TERMINOLOGY

p	mean normal stress = $\frac{\sigma_1 + \sigma_2 + \sigma_3}{3}$
q	$= \frac{1}{2}\sqrt{(\sigma_1 - \sigma_2)^2 + (\sigma_2 - \sigma_3)^2 + (\sigma_3 - \sigma_1)^2}$
θ	$\sigma_1 + \sigma_2 + \sigma_3$
σ_1	1st principal stress
σ_2	2nd " "
σ_3	3rd " "
ν	Poisson's ratio
E	Young's Modulus. Several suffices are used, e.g. E_A for Asphalt. Numerical suffices indicate the layer, counting from the pavement surface downward.
h	Layer thickness. Numerical suffices indicate the layer counting from the pavement surface downward.
M_R	Resilient modulus of soils.
h_e	Equivalent thickness of combined asphalt layers.
E_e	Equivalent stiffness of combined asphalt layers.
N	Design life
K_o	Coefficient of horizontal pressure.

Asphalt

A mixture of bitumen and inert mineral matter. In relation to the context of this thesis it is NOT used exclusively to denote Hot Rolled Asphalt as defined in BS 594.

Stiffness

For a bituminous material total strain varies with temperature and time. Thus the parameter "stiffness" is frequently used, and is analogous to Young's modulus, but relates to specific temperature and time conditions.

ABSTRACT

The principal objective of this research, to produce an analytical design method for highway pavements, has been achieved for pavement structures built with either asphalt or unbound granular base layers. The complex multi-layer configuration of pavement structures requires the use of a computer for the derivation of critical stresses and strains, hence the design method, which incorporates analysis of trial structures, has been computerised. The program, which has been named ADEM, from Analytical DEsign Method, calculates the required thickness of an asphalt layer necessary to ensure satisfactory pavement performance for a preselected design life and includes the Shell programs BISTRO and PONOS. Two modes of distress are considered, asphalt fatigue and permanent deformation, and the pavement designed to satisfy the most critical of these. The designer can specify the asphalt mix, whether a three, four or five-layer structure is required, if the unbound layer is to be considered as a linear elastic or non-linear elastic material, the stiffness and, where appropriate, thickness of all layers except the design layer. Thus it can be seen that ADEM offers considerably more versatility than traditional design methods. ADEM was developed from information available in the literature relating to the performance of highway materials and from analytical studies of the behaviour of pavement structures. Where possible existing proven material models have been used, as with asphaltic materials. However, considerable work was necessary to develop the non-linear elastic system for granular materials, as is evident from the three chapters discussing these studies. Some of the investigations led to the conclusion that more information must be obtained before they can be realistically incorporated in a design system. These studies, which include cement treated materials and cumulative

damage considerations have been reported in the appendices because they have not contributed directly to the development of ADEM. Finally, design studies undertaken with ADEM have indicated the importance of compaction in achieving economical asphalt layers, the fact that for certain cases layer thickness can be reduced by increasing the binder content in an asphalt and that whilst linear characterisation of unbound granular layers is perfectly adequate for comparison between different structures, non-linear characterisation should be used if working designs are required.

CHAPTER ONE

INTRODUCTION

1.1 THE JUSTIFICATION FOR THIS RESEARCH

A pavement is probably the most complex structure designed and constructed by engineers. This is because of the unique layered form of the structure and the fact that the layers are of different materials. To add further complexity to the structure none of the constituent materials - which include crack susceptible portland cement bound materials and stress sensitive unbound materials - are linear elastic.

In view of the aforementioned complexity, it is no surprise that traditional pavement design methods are essentially empirical, having evolved from the experience gained from constructing pavements, observing their behaviour and rejecting structural forms and materials which did not perform satisfactorily. This type of evolutionary process can never keep up with changing conditions, though when these changes are slow a reasonable degree of success can probably be maintained. The first signs of the inability of evolution in pavement design to keep pace with changing conditions probably occurred on airfields during World War II. Aeroplanes rapidly became larger and heavier in order to carry greater bomb loads and it became necessary to build stronger and stronger pavements. Subsequently, similar trends with regard to vehicular traffic and in particular the increasing volume of freight transported by road has caused similar problems on highway networks not only in Great Britain, but also worldwide. The economic life of countries depends on their ability to transport goods, the cost of such transport frequently being a major item in the price of the goods. Thus, it is reasonable to expect a high level of investment in providing and maintaining a highway network. However, because of the general economic recession the investment in maintaining the highway network is in real terms decreasing. Hence, there is a need to design pavements which will perform with increased efficiency for conditions beyond previous experience.

This need can only be satisfied through increased understanding of the structural action of a pavement in conjunction with improved knowledge of the performance of pavement materials. The key to improved design lies in the ability to analyse a pavement structure and combine the results of the analysis with the measured performance of paving materials. This type of procedure has been labelled "analytical pavement design". In fact, it is simply the application of the basic principals of engineering design.

The aim of the research reported in this thesis was to produce a working analytical design method for highway pavements to be constructed in Great Britain and to satisfy the need for improved design capability, which, for the reasons set out above, is currently an urgent necessity.

1.2 A BRIEF HISTORY OF ANALYTICAL PAVEMENT DESIGN

Before discussing analytical pavement design in detail, it is appropriate to consider, briefly, the historical development of the method, which emerged relatively recently as a discipline in its own right. Although some early analytical work was published in 1926 by Westergaard (1) relating to concrete pavements, little further development took place until 1943 when Burmister (2) published a paper entitled "The theory of stress and displacements in layered systems and application to the design of airport runways", which indicated developing interest in analysing pavement structures.

Ability to analyse pavements created the need for material descriptors such as representative modulus values and Poisson's ratio.

At this point a major problem was encountered, particularly in relation to flexible pavements since bitumen is, as already noted, a viscoelastic material with temperature dependent properties.

In 1954, Van der Poel (3) laid the foundation for a usable method for determining the stiffness of asphalt with a nomograph for bitumen stiffness. Heukelom and Klomp (4) based a method for calculating the stiffness of a low void content asphalt mix on the Van der Poel nomograph (3), and this work was extended to include mixes with higher void contents by Van Draat and Sommer (5) in 1965. However, knowledge of the stresses and strains within a pavement are no use for design purposes unless further data relating the performance of material to the stress condition is available.

In 1955, Hveem (6) attributed cracking in pavements to repeated flexing of the asphalt, in other words, to fatigue. Thus the need for research into the fatigue behaviour of asphalt was established and significant contributions to knowledge in this area were made by Pell (7), and Monismith et al (8).

During this period interest in an analytical approach to pavements had been growing in the USA. This interest led to the International Conference on the Structural Design of Asphalt Pavements (9) in 1962 which was conceived partially as a forum for discussion of the results of the AASHO road test. This conference represents the birth of the analytical approach to pavement design, gathering papers worldwide, and providing the first international meeting of people concerned specifically with the analytical design of pavements. Many interesting papers were presented at this conference, which laid the foundation for the development of the analytical approach. Of particular value was a paper by Whiffin and Lister (10) which provided much evidence to validate the use of elastic analysis as a tool for designing pavements.

Having started in 1962, International Conferences on the Structural Design of Asphalt Pavements have been held subsequently at five-yearly intervals and chronicle the development of analytical pavement design. The 1967 and 1972 conferences contained many papers on the characterisation of paving materials, necessary as input data for both analysis and design. During this period, pavement analysis was undertaken by the use of tables such as those produced by Jones (11) and Acum and Fox (12), or by the use of charts, as produced by Peattie (13). However, the rapid development of computer technology offered improved computational power and led to the production of programs for analysing pavements, the well known BISTRO (14) program being published in 1968.

Thus the scene was set, material data and analysis tools were available, and as a result, the 1977 International Conference was devoted mainly to papers presenting complete design methods, or subsystems for treating parts of the design problems.

Before passing on, note should be made of the considerable contribution to pavement design technology by the Shell Oil Company who, through their research centres, have provided much of the basic research upon which the analytical design method has been based.

1.3 THE ANALYTICAL DESIGN PROBLEM

According to "A Dictionary of Civil Engineering" (15), structural design consists of two parts, the first of which is structural analysis to "determine what forces are carried by all parts of a structure", and the second part is "the proportioning of the members according to the calculations of the first part". Thus the distinction between analytical pavement design, based on calculation of forces in a pavement structure, and the procedure carried out by the apparently incorrectly titled Road Note No. 29 (16), "A Guide to the Structural Design of Pavements for New

Roads", which does not involve any consideration of forces in the pavement, becomes clear.

Hence, analytical design specifically involves the calculation of stress and strain in an assumed pavement structure to be compared with knowledge of the properties of the materials in that structure to ensure that they do not suffer excessive distress before the pavement has performed satisfactorily for its design life.

Since pavement materials are multi-phase materials, their performance is a function of the proportions of the mix, and hence mix design should become part of the structural design procedure. However, the approach most commonly adopted, and used in this research, has been to specify both design requirements and mix proportions and calculate the required layer thickness. Using this approach has shown that in general British mixes do not offer a balance between fatigue performance and stiffness (the latter to resist deformation). Recent work by Brown (17) has demonstrated that the predictive methods available for assessing the stiffness and fatigue performance of asphalt can be used within the framework of pavement analysis for mix design. Brown's work (17) represents the first attempt to directly relate pavement and mix design and warrants further development. Unfortunately, it was published so recently (late 1978) that it could not be incorporated in this research.

The discussion above concerns only bituminous materials. Since a pavement consists of layers of different materials, it is evident that there can be considerable interaction between the performance parameters for the individual materials. Thus the design of a pavement can be seen to be a complex multi-dimensional problem.

1.4 INTRODUCTION TO THIS RESEARCH

The research reported in this thesis started in September 1975. The aim, as already stated, was to produce a working analytical design method for highway pavements and it is believed that this has been achieved. The approach adopted was to examine the literature concerning the characterisation of pavement materials, their performance, and the general structural requirements of a pavement and extract data to produce a design method. This naturally excluded any experimental research relating to material or pavement performance. The complexity of the problem led inevitably to the design method being produced in the form of a computer program.

It was recognised at the outset that information would be lacking in some areas as to the significance of various parameter connected with the analysis and design of pavements. Thus information had to be generated to provide a basis for decisions relating to the design process. This information has been provided by analytical studies with respect to the parameters under consideration. Hence, this research consists of studies of material and pavement behaviour as indicated by the chapter headings. Many chapters report literature reviews and most discuss the analytical studies which were considered necessary. This is followed by a description of the design program and discussions of results obtained with it. Because of the very wide scope of this work the literature reviews could not always include all available publications. It is, however, hoped that there are no significant omissions.

Most attention has been focussed on areas in which least information is available. Hence, the properties of asphalt receive little attention because they are relatively well defined and regularly used in pavement design literature, whereas unbound granular materials receive considerable attention.

Since the aim of this work was to produce a design method, only studies which contribute directly to this aim are included in the main body of this thesis. In general, the studies which indicated the need for more data before the results could be included in a design system have been reported as appendices. These studies have, however, contributed to the knowledge of the behaviour of pavement systems, and therefore conclusions and recommendations at the end of the main text draw on this information.

In the author's view the aim of this research has been achieved and is embodied in the ADEM3 computer program for pavement design. This computer program is unique in that it calculates the thickness of an asphalt layer necessary to satisfy a prescribed set of design requirements. It should be stressed that ADEM3 is an engineering design method and as such is not infallible. In common with any engineering design method, it contains engineering approximations which are open to question. However, in its development it has considered the wealth of empirically derived data as embodied in the Road Note No. 29 (16) approach as well as most of the experimental data available relating to pavement material behaviour. As such, the ADEM3 design method will withstand considerably more close scrutiny than its wholly empirical counterpart. By developing ADEM3 with reference to observed British pavement performance it contains all the validating evidence which supports the empirically derived design method.

Use of ADEM3 requires little more information than that currently used for designing pavements. In return, a versatile system is available for considering the effect of varying structural form and material characteristics, and for providing working pavement designs. It is suggested that ADEM3 represents a considerable improvement in pavement design technology since it is an implementable analytical design method.

CHAPTER TWO

CHOICE OF ANALYTICAL TOOL

2.1 INTRODUCTION

Since this project is concerned with analytical pavement design, the first requirement is an analysis tool. It was decided, at an early stage, to use one of the existing available computer programs as development of yet another analysis program was considered wasteful.

Tables 2.1 and 2.2, reproduced from Barksdale and Hicks (18), list and compare most of the operational computer programs currently available. Two programs missing from the list are ELSYM, a linear elastic program of the classical layer theory type available from C.L. Monismith, University of California, Berkeley, and DEFPAV, a finite element program, capable of treating materials non-linearly.

To satisfy the needs of this research, a well proven analysis program was required, preferably with detailed documentation, since its incorporation into a design package would require major alterations and additions to the program. With these requirements in mind, three basic types of program were considered as discussed below.

2.2 VISCOELASTIC PROGRAMS

Viscoelastic characterisation of pavements does not appear to be widely accepted. This is possibly because of the apparent complexity of the method, or because of dissatisfaction with the viscoelastic models used. Whatever the reason for its lack of general acceptance, this excludes it from consideration for use as a design tool.

Theory	Program	Availability	Source
Linear elastic (classical layered theory)	Shell BISTRO (axisymmetric)	Limited distribution (researchers only)	Shell Oil Company, One Shell Plaza, Houston, Texas 77002
	Chevron (axisymmetric)	No restrictions	R.J. Schmidt, Chevron Research Co., Richmond, California
Linear finite element	General axisymmetric (X1L67)	No restrictions	C.L. Monismith, University of California, Berkeley
	Prismatic space (2-dimensional)	No restrictions	University of California, Berkeley Berkeley
	Extended 2-dimensional (AFPAV)	Under development	R. Pichumani, Kirtland Air Force Base, New Mexico
Nonlinear elastic (classical layered theory)	Chevron 5-layer elastic with iteration	No restrictions	University of California, Berkeley Berkeley
Nonlinear finite element	FePave 1 (iterative)	No restrictions	University of California, Berkeley
	FePave 2 (incremental)	No restrictions	University of California, Berkeley
	Anisotropic incremental iterative	Available March 1973	R.D. Barksdale, Georgia Institute of Technology, Atlanta
Viscoelastic	Linear viscoelastic	No restrictions	F. Moavenzadeh, Massachusetts Institute of Technology, Cambridge

Table 2.1 Operational computer programs and their availability (after Barksdale and Hicks, 18)

Description of features	BISTRO	W1L67	VISAB3	AFPAV	Remarks
Theoretical basis	Burmister's layered system theory	Finite-element analysis of axisymmetric solids	Discrete-element analysis of pavement slabs	3-dimensional analysis of pris-matic solids	AFPAV is called extended 2-dimensional finite-element code and cannot solve general 3-dimensional problems.
Type of pavement analyzed	Rigid and flexi-ble pavements	Rigid and flexi-ble pavements	Rigid pavement only	Rigid and flexible pavements	Except for VISAB3, codes do not distinguish type of pavement
Number of pavement layers	N-layers (N = 10)	N-layers (N = 12)	2	N-layers (n = 15)	VISAB3 can treat only 2 layers, but the value of N can be changed by suitably modifying other codes
Number of applied loads	12 (can be increased)	1 (can be modified to analyze multiple loads)	Unlimited	Unlimited	BISTRO and W1L67 (when modif-ied) use principle of superposition for multiple loads; VISAB3 and AFPAV consider multiple loads simultaneously.
Contact pressure	Uniform over circular area	Uniform over circular area	Concentrated	Uniform over rectangu-lar area	Nonuniform contact pressure can be treated approximately in AFPAV code.

Table 2.2 Comparison of computer programs (after Barksdale and Hicks, 18)

/contd

Description of features	BISTRO	W1L67	VISAB3	AFPAV	Remarks
Joints in rigid pavement	Cannot consider	Cannot consider	Considers approximately	Can consider realistically	Joint analysis using AFPAV code is yet to be developed (1973)
Stress concentrations along joints	No	No	No	Yes	
Edge loads in rigid pavement	Cannot analyze	Cannot analyze	Can analyze	Can analyze	Edge-load analysis using AFPAV code is yet to be developed (1973)
Material properties	Linear elastic	Bilinear elastic	Linear elastic	Linear elastic	AFPAV code can be further developed to consider non-linear material properties.
Nonuniform layer thicknesses and properties	Cannot consider	Cannot consider	Cannot consider	Can consider	Changes referred to are in 1 vertical plane only and those should be constant in the longitudinal direction.
Discontinuities (e.g. culverts) under pavement	Cannot analyze	Cannot analyze	Cannot analyze	Can analyze	Orientation of culvert should be normal to one horizontal axis of gear arrangement.

Table 2.2 contd

2.3 FINITE ELEMENT PROGRAMS

The potential of the finite element method for the solution of difficult analysis problems and the apparent facility with which material characterisation other than linear elastic can be incorporated makes this method very attractive.

However, in order to obtain meaningful answers with a finite element program, it is necessary to keep the proportions of rectangular elements reasonably near to that of a square. In analysing a pavement structure, this can only be achieved by the use of a large number of elements, which means that only very large computers can handle the problems. When non-linear characteristics are introduced into the system and either incremental or iterative techniques are used to solve the structure, the cost of running the program can become very high. By its nature, the finite element analysis produces stresses and strains throughout the structure which are not needed for design since only critical parameters are necessary. Thus, on the basis of their cost, finite element programs were rejected as a design tool.

2.4 ELASTIC LAYER PROGRAMS

Elastic layer programs generally produce solutions for stress and strain at preselected points, within a pavement structure. This makes their use efficient for design purposes, provided that the location of critical stresses and strains can be predefined and offers considerable economy with regard to computing costs. However, they are less versatile with regard to non-linear characterisation of materials and whilst they can be modified to do this they are unable to cope with variations in the horizontal plane.

The most attractive attribute of the elastic layer systems is the fact that some of them are widely used. This has established confidence in the programs and proven their reliability. The BISTRO (14) program is one example of an elastic layer program which has been widely used. It is also an extremely well documented program and has been used extensively at the University of Nottingham. Hence, since confidence had been established in this program it was adopted as the basic analytical tool for this research.

CHAPTER THREE

THE CHARACTERISATION OF BITUMINOUS MATERIALS FOR DESIGN

3.1 INTRODUCTION

Before an analysis of a pavement can be undertaken material parameters are necessary. Since, as indicated in the previous chapter, the BISTRO (14) elastic layer program will be used for pavement analysis, the necessary material parameters are modulus and Poisson's ratio. Having undertaken an analysis of a pavement structure, performance or design criteria are necessary so that design can be undertaken.

This chapter deals with the provision of elastic parameters and failure criteria for bituminous materials. The most obvious and satisfactory way in which this information can be produced is by direct measurement. However, this requires costly equipment, skilled operators and can be very time consuming. Therefore, the work reported in this chapter relates specifically to methods for predicting stiffness and failure criteria.

3.2 DETERMINATION OF STIFFNESS

The most commonly used method for predicting the stiffness of bitumen bound material is the method derived by the Shell researchers (3,4,5) which has recently been modified slightly by Bonnaure et al (19). However, before this method is discussed, it should be noted that there are alternative methods, such as those produced by Shook and Kallas (20) and Francken and Verstraeten (21).

Shook and Kallas (20) propose two regression equations which relate dynamic modulus, measured at a loading frequency of 4 Hz from flexure tests, to mix and temperature parameters. Since they will not be used in this research, these equations have not been reproduced.

The method proposed by Francken and Verstraeten (21) is based on the Van der Poel nomograph for predicting the stiffness of the bitumen (3) and provides equations for relating the stiffness of an asphalt mix to volumetric proportions of the mix and the binder stiffness. Since this method is essentially similar to the Shell procedure and requires similar information it will not be discussed in detail.

Having considered other methods, it was decided to use the Shell method for stiffness prediction. This was because of the general acceptance of the method, and the availability from Shell (23) of a computer program for obtaining bitumen stiffnesses from the Van der Poel nomograph (3).

The existence of the Van der Poel nomograph (3) for the prediction of bitumen stiffness for a range of temperatures and loading times is the key to estimating material stiffnesses. As indicated by Van der Poel (3), the nomograph is only a convenient means of presenting a large number of experimental results, therefore it is essential to consider its accuracy, which is reported as being within a factor of 2. Initially, this appears to be an unreasonably large measure of error. However, when put into the context of the sensitivity of bitumen to temperature, its stiffness will be changed by a factor of 2 for a few degrees variation. Thus, for engineering purposes, the nomograph is sufficiently accurate.

Extending his research on bitumen to bitumen aggregate mixes, Van der Poel (24) showed that the stiffness of a mixture with dense graded aggregate is primarily dependent on the stiffness of the bitumen and the volume concentration of the aggregate. Heukelom and Klomp (4) have extended this work for mixes with less than 3% voids, giving the following expression for mix stiffness:

$$\frac{S_m}{S_b} = \left[1 + \frac{2.5}{n} \times \frac{C_v}{1-C_v}\right]^n \tag{3.1}$$

where S_m = stiffness of the mix (MN/m^2)

S_b = stiffness of the bitumen from the nomograph (MN/m^2)

C_v = the volume concentration of the aggregate, derived from

$$\frac{\text{volume of aggregate}}{\text{volume of aggregate + volume of bitumen}}$$

$$n = 0.83 \log \frac{4\times10^4}{S_b} \tag{3.2}$$

A further extension of this work to include mixes with greater than 3% air voids was reported by Van Draat and Sommer (5). A correction to C_v was derived for void contents above 3% as follows:

$$C_v' = \frac{C_v}{1 + (V_v - 0.03)} \tag{3.3}$$

where V_v = void content.

This corrected aggregate volume concentration (C_v') is substituted into Equation 3.1 as required by the void content.

Further research within the Shell organisation (19) has produced a revised method for calculating mix stiffness. This method still uses the bitumen stiffness nomograph, offering either a nomograph or equations for the calculation. However, this method generally gives similar answers to that obtained from Equations 3.1 and 3.3 above so it has not been adopted.

Having obtained the computerised representation of the Van der Poel (3) nomograph, Brown et al (25) added Equations 3.1 and 3.3 to it, producing a program suitable for calculating mix stiffnesses for a wide range of mix variables.

3.2.1 Implementation of the Shell method for determining mix stiffness

The computer program developed by Brown et al (25) offered several options with regard to the input parameters required to describe the bitumen in the mix. For simplicity a system requiring only that information which would normally be available to a highway engineer was adopted for the design procedure.

Regarding the bitumen, the only parameter generally known to the pavement designer is the penetration grade of binder in the mix. This, however, relates to the binder as supplied to the mixing plant, the nomograph procedure (3) requiring the recovered penetration grade. No published information appears to be available relating initial and recovered penetration, but the recovered penetration is usually assumed to be 65% of the initial penetration (26).

The other parameter required to characterise the bitumen is the recovered softening point. BS 3690 (27) gives tolerances on both penetration and softening point, which allows considerable variation in the stiffness of the bitumen. However, a relationship, generally representative of commercial road building bitumens, has been made available (26) as follows:

$$S.P. = 99.13 - 26.35 \log P \tag{3.4}$$

where S.P. = softening point

P = penetration grade

It should be noted that this relationship is valid for both initial and recovered softening points. In addition, it relates specifically to the softening point derived according to the ASTM method of test (28), which is the appropriate test method for the bitumen stiffness nomograph. This information, necessary to determine bitumen stiffness, is readily available to highway engineers. If the softening point according to the British Standard method (29) is required, it is necessary to subtract 1.5^{o}C from the value obtained via the ASTM method.

The parameters required for the calculation of volumetric properties are aggregate and bitumen specific gravities, binder content and void content. With the exception of the void content this data is readily available. It is relatively easy to measure void content on a core removed from the pavement, but this of course requires that either pavement or at least a trial section has been constructed. If this is impossible, guidance on the void contents likely to be encountered in typical mixes as laid, is available from the work of Leech and Powell (30). Thus the provision of parameters for the calculation of stiffness of a bituminous mix presents no difficulty.

3.3 DETERMINATION OF POISSON'S RATIO

As indicated by Pell and Brown (31), very little information is available on the values of Poisson's ratio for bituminous materials. It is reported (31) that values range from 0.35 to 0.5, the higher values occurring at high temperatures and that for normal conditions a value of 0.4 appears to be appropriate and is recommended for use in design.

3.4 DESIGN CRITERIA FOR BITUMINOUS MATERIALS

This section is concerned solely with the failure of the asphalt layer. The problem of permanent deformation, which may accumulate throughout the layers of the structure will be considered in a separate chapter later in this thesis.

It has become generally accepted that cracking in the asphalt layer of a pavement is a fatigue phenomenon. This type of failure is not commonly observed in British pavements but is a common occurrence in structures elsewhere, particularly in the USA. Considerable research effort has been focussed on the problem of fatigue in bituminous mixes to the extent that more data is available on this aspect of pavement material behaviour than any other. In 1973, a symposium was held during the annual meeting of the Highway Research Board on the "Structural Design of Asphalt Concrete Pavements to Prevent Fatigue Cracking", the proceedings of which have been reported in Special Report 140 of the Highway Research Board (32).

Two important points emerge from this symposium, with which most authorities agree. The first is that the criterion of fatigue crack initiation is one of applied tensile strain (33), with a general relationship defining the fatigue life of the form:

$$N = C\left(\frac{1}{\varepsilon}\right)^m \qquad (3.5)$$

where N = number of applications of load to initiate a fatigue crack

ε = amplitude of applied tensile strain

C and m = factors depending on the composition and properties of the mix.

The second is that linear summation of cycle ratios is a reasonable cumulative damage hypothesis which permits the prediction of service life for a range of conditions from simple laboratory tests. This hypothesis, commonly known as Miner's rule, can be expressed mathematically as:

$$D = \sum_{i=1}^{i} \frac{n_i}{N_i} \qquad (3.6)$$

where D = total cumulative damage

n_i = number of applications at strain level i

N_i = number of applications to cause failure in simple loading at strain level i.

Failure occurs as soon as D equals 1.

Ample evidence to support these two important conclusions is presented in reference 32 and rather than burden this thesis with a list of publications cited by authors of papers in Report 140 (32) readers are referred to the original document should they require confirmation.

Further information, reported by Pell and Cooper (34) led to a convenient method for predicting the fatigue life of a mix from a knowledge of its mix proportions. This method has been reported in the form of a nomograph (35). However, for the purposes of incorporation into the pavement design program, the regression equations reported (35) were used. The two equations used are:

$$\log N = 1.6 - m(3.2 + \log \varepsilon) \tag{3.7}$$

where N = number of cycles to failure at a particular strain level

ε = maximum amplitude of applied strain

m = slope of the fatigue-life relationship which depends on the mix properties

and

$$\log N(\varepsilon = 10^{-4}) = 4.13 \log V_b + 6.95 \log (S.P.) - 11.13 \tag{3.8}$$

where $N(\varepsilon=10^{-4})$ = the number of cycles to failure at a strain of 10^{-4}

V_b = volumetric proportion of bitumen in the mix (%)

S.P. = the initial ring and ball softening point of the bitumen.

Equation 3.8, obtained from a regression analysis on the results of a large number of tests (35), allows the life of any mix to be determined (at a standardised strain of

10^{-4}) from a knowledge of the mix proportions. Substitution of the derived life and standardized strain into Equation 3.7 allows the slope factor, m, to be determined for this mix. This defines the strain-life relationship for this mix and Equation 3.7 can now be used to provide the fatigue life, at a given strain level, or be rearranged to provide the allowable strain at a specified life. The facility to calculate allowable strain provides a design criterion and has been adopted for the development of the design program.

3.4.1 Implementation of the fatigue life prediction system

It should be noted that Equations 3.7 and 3.8 relate specifically to stress controlled laboratory fatigue tests, under continuous loading. Direct application of these fatigue equations to typical British pavements and mixes indicates that they will fail in fatigue whereas observation of British pavements indicates that they do not generally exhibit this mode of failure. Thus it is evident that the laboratory test results cannot be applied directly to pavement performance.

One important difference between laboratory tests and pavement conditions is the difference in loading. Laboratory tests take place under continuous loading, whereas in a pavement there is always a delay between load pulses even on heavily trafficked roads, and periods of the day during which very few loads are applied. Data on the effect of rest periods has been published by several authors (36-40), though the evidence is conflicting. However, the more recent publications (38-40) indicate that fatigue life is increased. The other major difference between Cooper and Pell's (35) laboratory testing, and pavement performance relates to crack propagation. A bituminous layer in a pavement will not be regarded as failed until the cracks have propagated through that layer, whereas the laboratory test (34) leads to abrupt failure of the specimen at the first crack. Brown (41) found that increasing the predicted laboratory fatigue life by a factor of 100 gave reasonable predictions in relation to British pavements. This factor came from combining a five-fold life increase due to rest periods, as indicated by the literature (38-40) with a 20-fold life increase due to crack propagation. There was, however, little justification for the latter factor.

Recent work reported by Van Dijk (42) using a rolling wheel test simulative of pavement action provides useful information with regard to crack propagation time, through reinterpretation of his results. Fig. 3.1 is reproduced from Van Dijk (42) and plots measured strain amplitude as a function of number of wheel passes, or load cycles. The deterioration of a test piece is described with the aid of N_1, N_2 and N_3 points as follows:

(1) Before the fatigue stage, N_1, hairline cracks are initiated.

(2) Between fatigue stages, N_1 and N_2, the hairline cracks widen progressively with accompanying network formation.

(3) At and past the fatigue stage, N_2, real cracks are formed, followed by

(4) A failure of the bottom or top surface, the N_3 stage.

Issue is taken with the definition of crack initiation life, N_1, as the intersection of two extrapolated linear parts. It is suggested that cracking starts as soon as the strain starts to increase at the tangent point marked A in Fig. 3.1, since this indicates that some change has taken place in the material. This point is probably analogous with failure as experienced in the test method used by Cooper and Pell (35). Since the N_2 point from Fig. 3.1 relates to the formation of

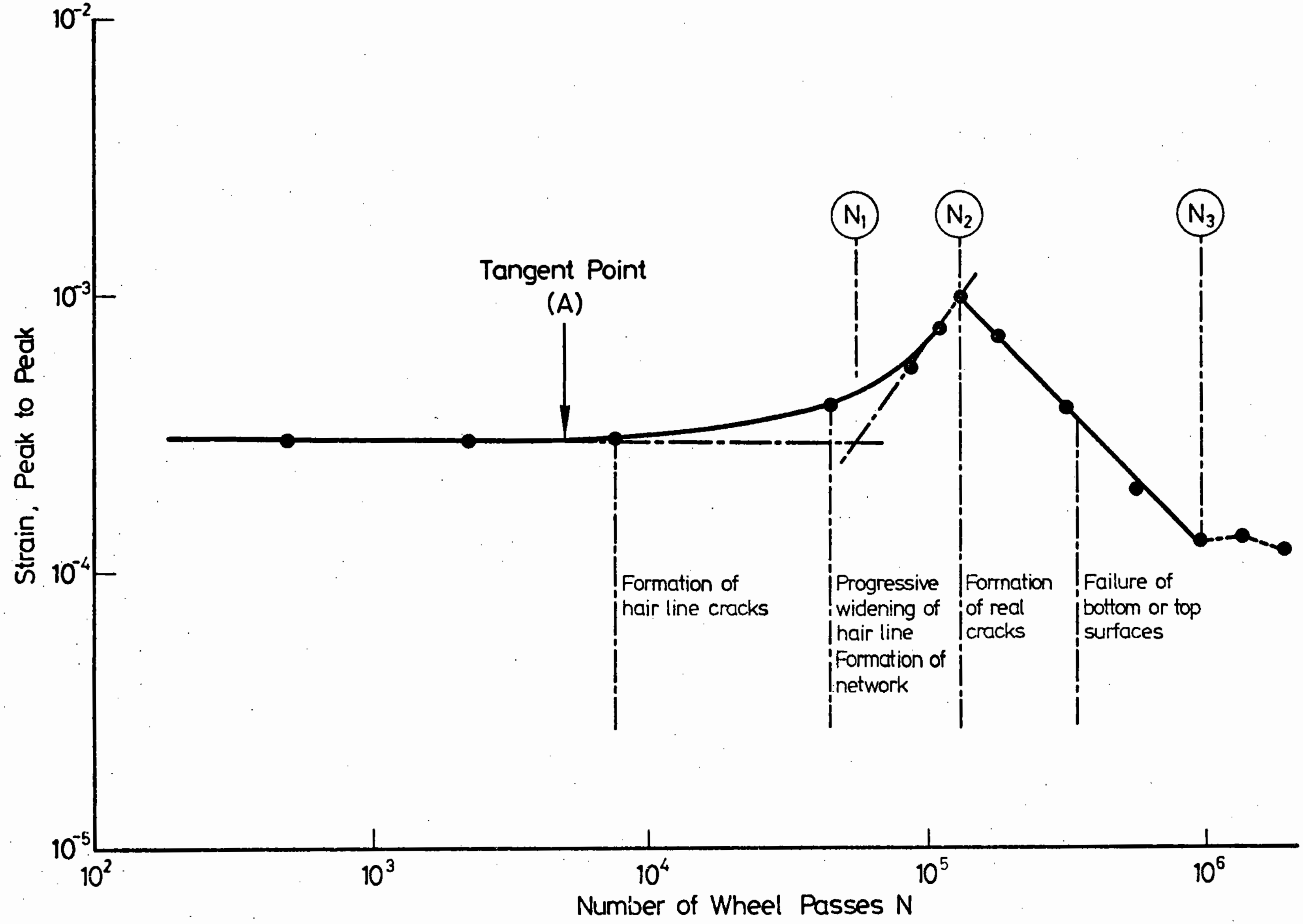

FIG. 3.1 CRACK DEVELOPMENT AS A FUNCTION OF STRAIN READINGS AT EQUIVALENT NUMBER OF WHEEL PASSES (After Van Dijk, 42)

"real cracks" this can be regarded as a limiting service condition. Hence from Fig. 3.1 crack propagation takes place from approximately 5×10^3 to 1.3×10^5 cycles of load. This indicates that a factor of 26 should be applied to crack initiation life to obtain service life. Thus the factor of 20 used by Brown (41) for crack propagation may be conservative, but appears to be of the correct order of magnitude. In consequence, a factor of 100, as recommended by Brown (41) has been adopted in this work to convert laboratory fatigue life to pavement fatigue life.

It should be noted that caution is needed with regard to the application of this factor to design. The conversion gives results judged to be reasonable when all layers in a pavement are assumed to be linear elastic and for an average annual temperature. Also, in its derivation no specific consideration has been made of the distribution of traffic from lane to lane, though this is implied since subjective judgement has been used to verify predictions. Within these confines the factor is perfectly reliable. However, application of this factorised fatigue law in design systems which differ significantly from the one mentioned above, perhaps by using a cumulative damage approach, or specific consideration of traffic distribution between lanes, requires reassessment of the correction factor.

3.5 LOCATION OF THE CRITICAL STRAIN

Results of laboratory material investigation have indicated that tensile strain is the significant parameter in determining the fatigue behaviour of a bituminous material.

Peattie (43), reporting some early work on pavement design, indicated that the conditions at the bottom of the bituminous layer in a flexible pavement were critical, i.e. that the maximum principal tensile strain occurred at the bottom of the asphalt layer, and this has subsequently been generally accepted.

Close examination of Peattie's (43) work indicated that analysis had been accomplished by the use of tabulated solutions under a single wheel load, since versatile computer programs for the analysis of pavements were not available at the time. It was considered to be essential to check that the critical strain was always at the bottom of the asphalt layer, so a series of pavements were analysed under dual wheel load conditions.

Several structures were analysed in detail on a very stiff subgrade (E_s = 300 MN/m^2). The structures consisted of 200 mm layers of granular material and asphalt layers of between 40 and 300 mm thickness, with stiffnesses from 1,000 to 18,000 MN/m^2. The results of the analysis indicated that for certain combinations of stiffness and asphalt layer thickness the maximum principal tensile strain in the bituminous layer was not at the bottom of the asphalt layer, on the axis of symmetry of the wheel load, as has been assumed in the past.

It was, therefore, considered desirable to extend this study to define limits within which the assumption that the limiting strain was at the bottom of the layer could be applied. To this end, further structures were analysed on subgrades of modulus 150 and 75 MN/m^2 and the lower limit for the modulus of the asphalt reduced to 500 MN/m^2. The 200 mm thick granular layer was maintained for all structures with a modulus of 2.5 times that of the subgrade. The results of this study are presented in Fig. 3.2 which should be used to check that the design calculations are in fact valid.

Claessen et al (44) have also checked the location of the maximum principal tensile strain, and have derived a factor C for checking the validity of the assumption:

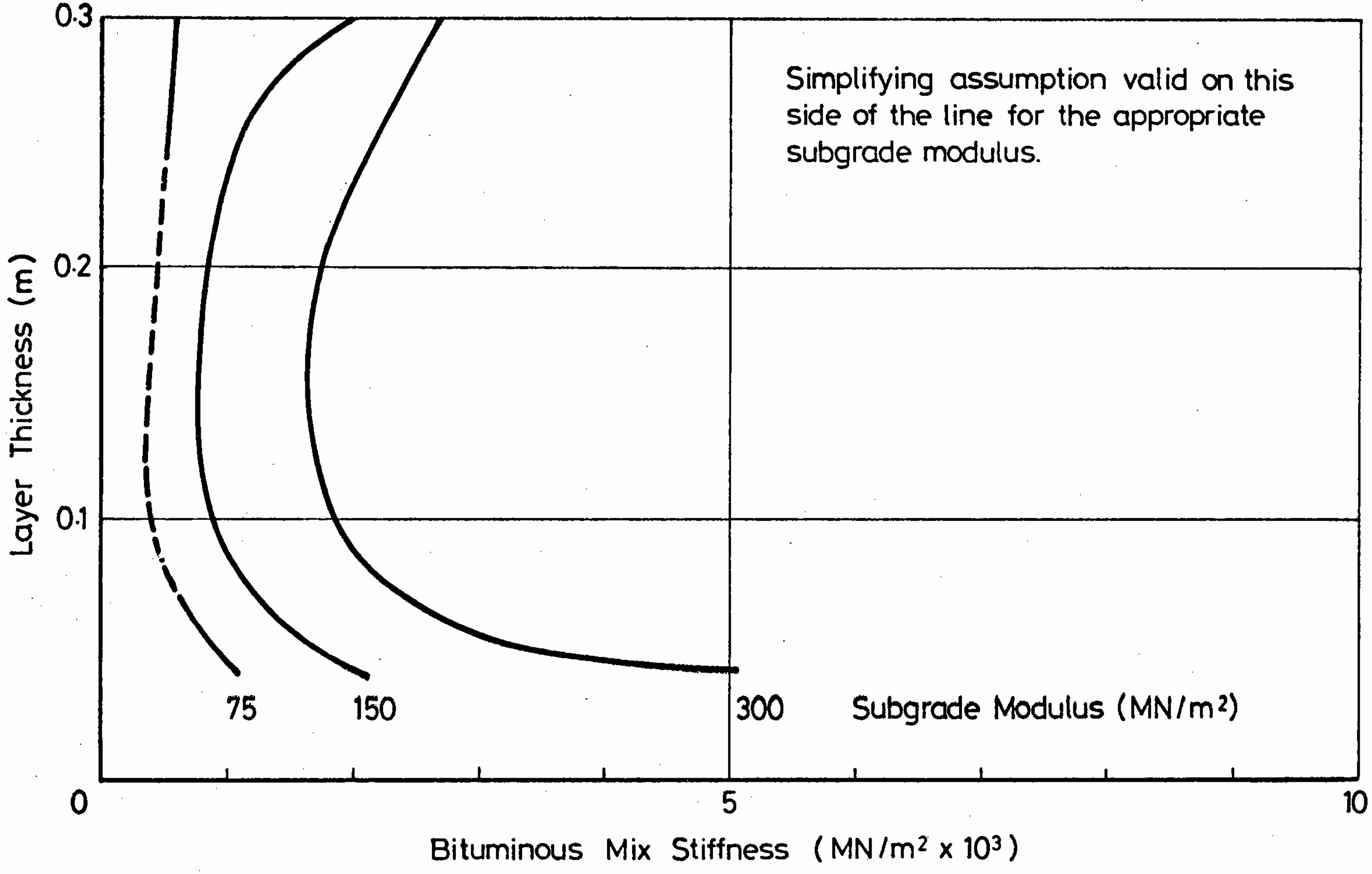

FIG. 3.2 ENVELOPES OF VALIDITY FOR SIMPLIFYING ASSUMPTION

$$C = h_1 \frac{E_1}{E_2} \quad \text{mm} \tag{3.9}$$

where h_1 = thickness of the asphalt surfacing

E_1 = stiffness of the asphalt surfacing

E_2 = stiffness of the granular layer.

If C is greater than 133 mm, then the limiting tensile strain is not at the bottom of the layer. A comparison between these two limiting conditions indicated that the Claessen et al (44) approach always lay within the zone established from Fig. 3.2 as invalid. It is, therefore, recommended that Fig. 3.2 be used in preference because in being more conservative, it is the safer.

CHAPTER FOUR

THE CHARACTERISATION OF UNBOUND GRANULAR MATERIALS FOR DESIGN

4.1 INTRODUCTION

The literature relating to the behaviour of unbound granular material which has been reviewed in this Chapter, indicates that there are basically two approaches to characterising these materials.

The first approach treats the layer in a linear elastic manner and relates the stiffness of a granular layer to the stiffness of the underlying layer.

The second approach is based on the results of triaxial testing, which has indicated that the modulus of unbound granular material is stress dependent.

It appears to be generally accepted that the stress dependent approach is more truly representative of the material, and it has been used for pavement analysis (45). However, no information is available from the literature which allows an assessment to be made of the significance of non-linear characterisation of granular materials with respect to pavement design. Therefore, examples of the two approaches have been selected for comparative studies.

It was found that the simple elastic approach was directly and easily implementable through existing analysis programs. The CHEVRON elastic layer program, mentioned in Table 2.1, is available in a form which incorporates an iterative process for the non-linear elastic analysis of unbound layers. However, this program is limited in the number of layers that it can consider and having decided to use BISTRO (14) as a basis for the design program the use of a second analysis program, CHEVRON, for non-linear iteration was not sensible. Therefore, an iterative procedure was incorporated into BISTRO (14), together with a number of refinements to the material characterisation not incorporated in CHEVRON. The development of this procedure required that self-weight stresses be incorporated in the analysis, which, in turn, necessitated investigations related to the determination of coefficients of horizontal pressure for self-weight stresses. The work relating to the development of a non-linear analysis tool is also reported in this chapter.

4.2 LITERATURE REVIEW

4.2.1 Simple elastic characterisation

Heukelom and Klomp (46) reported that the modulus of a granular layer was between 1.5 and 3.5 times that of the underlying soil. This information was derived from surface wave propagation measurements on normally constructed pavements, and has formed the basis for virtually all linear elastic characterisations of unbound granular layers. A widely accepted modular ratio between the granular layer and the underlying subgrade is 2.5, and this ratio has been adopted in all linear elastic studies in this research.

However, Claessen et al (47) prefer to use the development by Dormon and Metcalf (48) of the original work (46) in which:

$$E_G = k_2 E_s \tag{4.1}$$

where E_G = modulus of granular layer

E_s = modulus of subgrade

$k_2 = 0.2 \quad h_G^{0.45}$

and $2 < k_2 \leq 4$

h_G = thickness of granular layer (mm)

Barker et al (49) have extended Equation 4.1 through a series of analytical studies to provide curves for predicting the modulus of a layer from the modulus of the layer beneath. This work is intended for very thick granular layers in structures with unbound bases and sub-bases, so the curves have not been reproduced here. However, this work is interesting since it recognises the fact that the stiffness of an unbound granular material must have an upper limit, and sets this at 690 MN/m^2 for base materials and 275 MN/m^2 for sub-base materials.

The only design criterion suggested for granular layers is the one derived by Brown and Pell (50) which requires the horizontal tensile stress to be limited to no more than 0.5 times the vertical wheel load stress plus the horizontal overburden pressure.

Little information is available with regard to Poisson's ratio, but Pell and Brown (51) recommend that a value of 0.3 is used for elastic analyses.

4.2.2 Non-linear characterisation

A detailed review of the literature by Hicks (45) identified the following factors as influencing the resilient response of granular materials:

(1) Stress level (mean normal stress or confining stress).

(2) Density

(3) Grading, aggregate type and particle shape.

(4) Moisture content.

(5) Frequency of loading and number of load applications.

These headings will be used as the basis for the review of the literature on unbound granular materials.

Stress level: There appears to be agreement between researchers in this field, in that all studies report an increase in stiffness with increasing stress level. This is also true for measurements carried out in full scale pavements.

Boyce (52), when reviewing the literature on resilient properties of granular material, produced a table comparing the relationships derived by various sources. This table, with some additions, has been reproduced as Table 4.1, and shows that most studies indicate that resilient modulus (E) is affected by stress level as follows:

$$E = K_1'(\sigma_3)^{K_2'} \tag{4.2}$$

$$E = K_1(\theta)^{K_2} \tag{4.3}$$

where K_1', K_2', K_1 and K_2 are material constants.

Hicks (45), in an extensive study, found higher correlation coefficients when resilient modulus was expressed as a function of normal stress (θ) than in terms of confining stress (σ_3).

Smith and Nair (60) when reviewing the literature, found that all data which they examined could be bounded by the two extremes, K_1 = 2,000, K_2 = 0.6 and K_1 = 5,000, K_2 = 0.6 for E_r expressed in psi.

Table 4.1 notes that the relationship indicated by Boyce et al (62) is for certain restricted conditions. These conditions require that the repeated stress is small compared with the mean normal stress. The complete resilient stress-strain relationship treats the material anisotropically and therefore cannot be applied to a BISTRO (14) type analysis. Since the modulus stress relationship is of a complex nature the publication of Boyce et al (62) should be referred to for details of this model.

It should be noted that as indicated by Hicks (45), the constant K_2 in Equations 4.2 and 4.3 is dimensionless and that K_1 has units of stress$^{(1-K_2)}$. Unless a reference specifically stated otherwise, this was assumed to be the case for all such relationships. It is therefore evident that care must be exercised in converting from one set of units to another.

Source	Material	Relationship
Biarez (53)	Uniform sand	$E_r = K'(p)^{K_2'}$ $(0.5 < K_2' < 0.6)$
Dunlap (54)	Partially saturated, well graded aggregate	$E_r = K_1 + K_2(\sigma_3)$
Williams (55)	Uniform sand	$E_r = K_1 + K_2(\sigma_3)^{1/3}$
Hicks (45) and others at Berkeley	Aggregate base	$E_r = K_1(\sigma_3)^{K_2}$ or $K_1'(\theta)^{K_2'}$
Moore et al (56)	Crushed limestone base	$E_r = K_1 + K_2(\sigma_3)^2$
Allen and Thompson (57)	Gravel and crushed stone	$E_r = K_1(\sigma_3)^{K_2}$ or $K_1'(p)^{K_2'}$
Hardin and Black (58)	Dry sand	$E_r = K_1'(p)^{K_2'}$
Robinson (59)	Uniform dry sand	$E_r = K_1'(p)^{K_2'}$ $(K_2' = 0.48 - 0.6)$
Smith and Nair (60)	Graded crushed gravel	$E_r = K_1'(\theta)^{K_2'}$ $(\theta = 3 \times p)$
Brown and Pell (61)	Graded crushed stone	$E_r = K_1'(\theta)^{K_2'}$
Boyce et al (62)	Graded crushed limestone	$E_r = K_1'(p_m)^{K_2'}$ p_m = mean value (under restricted conditions)

N.B. The above 'K values' do not all have the same meaning.

Table 4.1 Modulus-stress relationships for granular materials

Density: There does not appear to be any consistent relationship between resilient modulus and density, though the trend observed by Hicks (45), Kennedy (63) and Coffman et al (64) in tests on various types of graded aggregate is for modulus to increase with increasing density.

Grading, aggregate type and particle shape: The most obvious effect of these variables relates to the compaction of the material, thereby affecting the density which can be achieved for a given compactive effort. If materials are compared at the same relative densities, there appears to be little effect on the resilient modulus.

Hicks (45), comparing crushed and partially crushed aggregate, found that the coefficients K_1 and K_2 were similar at comparable relative densities.

Moisture content: When material test results are analysed on the basis of total stress, the coefficient K_1 has been found to decrease with increasing degree of saturation (65-67), K_2 remaining approximately constant. Smith and Nair (60) report that if the analysis is based on effective stress, the modulus is independent of degree of saturation, thus indicating that changes in modulus are probably caused by changes in pore water pressure.

Haynes and Yoder (68) reported similar results from tests on both gravel and crushed stone base course material, the resilient modulus at a degree of saturation of 97% being about one half of the value at 70%. However, the stresses utilised by Haynes and Yoder (68) (σ_3 = 15 psi, $\Delta\sigma_d$ = 70 psi) are higher than those encountered in typical highway pavements.

Seed et al (69) conducted plate load tests on a prototype pavement and found that the resilient deformation in the base course increased by approximately 50% when the granular base was saturated. Hicks (45), in undertaking similar tests, noted no increase in surface deflection, but this could be attributed to the lower ambient temperatures when testing the saturated pavement, causing increased stiffness in the asphalt surfacing.

It is unfortunate that the effect of moisture on resilient modulus has not been thoroughly researched since there appears to be a significant effect, and an approach to pavement design which considers seasonal variations necessitates consideration of moisture changes.

Frequency of loading and number of load applications: Lashine et al (70), testing crushed stone, concluded that frequency of loading has little or no effect on resilient properties within the range 0.01 to 10 Hz. Williams (55) and Robinson (59), both testing sand, reached similar conclusions.

Morgan (71), Lashine et al (70), and Kennedy (63) have recorded a change in resilient properties after a large number of load applications (up to 10^6). More et al (56) reported that after 2.5×10^6 load applications the resilient modulus of crushed limestone was still increasing, but suggested that this may have been due to increasing suction forces caused by moisture loss.

Poisson's ratio: Little information appears to be available concerning Poisson's ratio for granular material. Morgan (71) has reported values ranging from 0.2 to 0.4. Hicks' detailed study (45) indicated that the resilient Poisson's ratio varied with the stress ratio (σ_1/σ_3). He also found that Poisson's ratio is affected by grading and density but not in any discernable pattern, and that it decreased with increasing degree of saturation. Boyce (52) has reported a value of 0.29 for certain restricted stress conditions.

Plate load tests were undertaken by Hicks (45) and when compared with non-linear analysis indicated that if an average value of Poisson's ratio of 0.3 was used, satisfactory predictions could be obtained. Hence, a constant Poisson's ratio of 0.3 has been adopted.

A failure criterion for unbound granular material: Barker (73), discussing the need for a failure criterion for granular materials, used the ratio of the principal stresses (σ_1/σ_3) with limiting values of either 6 or 8.

Boyce (52) reported tests to failure for several stress paths, and found that the principal stress ratio at failure was a function of the stress path, varying from 6.3 to 9.5 for the one material investigated. However, presentation of these results as a ratio of stress invariants (q/p) reduced this scatter, values ranging from 2.16 to 2.2.

From tests undertaken in South Africa on Derdepor crushed stone, Maree (85) concluded that the maximum safe deviator stress was 1,000 kN/m^2 for a confining stress of 100 kN/m^2. When expressed as a ratio of invariants this produces a failure criterion of 2.3. Since Boyce (52) and Maree (85) worked quite independently, the agreement in these results is gratifying, and supports the use of an upper limit of q/p = 2.2. It can be shown by the construction of a Mohr's circle that the limiting relationship of q/p = 2.2 corresponds to an angle of shearing resistance of 54°.

4.2.3 Analyses using stress dependent models for granular materials

Hicks (45) undertook analytical studies of the San Diego Road Test for comparison with measured parameters in the pavement. It was concluded from this work that the CHEVRON5L program (classical elastic layer) when modified to undertake non-linear analysis provided reasonable predictions of longitudinal and lateral surface strain profiles. It was also concluded (45) that observed differences in pavement response could be completely accounted for by varying K_1.

Further comparisons were made (45) between the CHEVRON5L program, the FePav finite element program and the results of plate load tests. Both programs provided satisfactory deflection predictions, but FePav was more accurate with regard to vertical subgrade stress. Measured tensile strains at the bottom of the asphalt layer fell between the values predicted by the two programs, and neither program gave reliable prediction of the vertical stress 75 mm down into the unbound base layer. However, considerable difficulty is likely to be encountered in obtaining reliable measured stresses in granular materials, and so comparisons may not be reliable.

Smith (72), having assumed that the granular material could be accurately characterised by a function similar to Equation 4.3, undertook an analytical study to provide equivalent single value moduli for granular layers. Two equivalent moduli were derived, one to produce the same vertical strain on the subgrade and the other to produce the same horizontal tensile strain at the bottom of the asphalt.

Regression analyses were undertaken on the results of the analyses and it was found that satisfactory relationships were obtained if separate equations were derived for the different granular layer thicknesses. For a granular layer thickness of 102 mm:

$$\log E_{2et} = 0.912 - 0.481 \log h_1 - 0.131 \log E_1 + 0.4 \log E_3 + 0.751 \log K_1 \tag{4.4}$$

$$\log E_{2ev} = 0.884 - 0.436 \log h_1 - 0.116 \log E_1 + 0.373 \log E_3 + 0.755 \log K_1 \tag{4.5}$$

For a layer thickness of 254 mm:

$$\log E_{2et} = 1.111 - 0.525 \log h_1 - 0.157 \log E_1 + 0.265 \log E_3 + 0.9 \log K_1 \quad (4.6)$$

$$\log E_{2ev} = 0.9 - 0.417 \log h_1 - 0.128 \log E_1 + 0.298 \log E_3 + 0.844 \log K_1 \quad (4.7)$$

For a layer thickness of 406 mm:

$$\log E_{2et} = 1.194 - 0.527 \log h_1 - 0.176 \log E_1 + 0.172 \log E_3 + 1.013 \log K_1 \quad (4.8)$$

$$\log E_{2ev} = 0.877 - 0.426 \log h_1 - 0.121 \log E_1 + 0.216 \log E_3 + 0.933 \log K_1 \quad (4.9)$$

where E_{2et} is the granular modulus to produce the same asphalt strain,

E_{2ev} is the granular modulus to produce the same vertical subgrade strain,

h_1 is the thickness of the asphalt layer,

E_1 is the stiffness of the asphalt layer,

E_3 is the stiffness of the subgrade.

Smith (72) reports that his predictions are in the same general range as those produced by the Shell method (Equation 4.1) and correlate well with the work of Barker et al (49). However, it should be noted that Barker et al (49) provide an upper limit to derived stiffness values, whereas Smith (72) has not applied any limits.

Barker (73) analysed a typical airfield pavement containing thick unbound layers. A finite element computer program, using an incremental procedure to cater for non-linearity, was used for the work, the purpose of which was to compare non-linear analyses with and without failure criteria. Barker's results (73) show that when using the K-θ model (Equation 4.3) without a failure criterion, significant tensile stresses appeared in the unbound material which was considered to be impossible. A failure criterion based on the ratio of the major to minor principal stresses and applied to the analysis effectively removed these tensile stresses and a stress distribution judged to be realistic and reasonable was obtained, thus underlining the need for a failure criterion.

It should be noted that the conclusion regarding failure criteria, offered by Barker (73) casts some doubt on the validity of the simplification offered by Smith (72).

4.3 THE DEVELOPMENT OF A SYSTEM FOR NON-LINEAR ANALYSIS OF GRANULAR MATERIALS

4.3.1 Introduction

Having decided that non-linear analysis of unbound granular material warranted investigation, the next stage was to consider in detail the approach to be used.

The first requirement was an analytical tool, the natural choice being the BISTRO program (14) which had to be modified to include a modulus derivation routine.

In order to account for variation in stress with depth in the granular layer, it was divided into four sub-layers of equal thickness, and the stresses derived at the centre of each sub-layer used for the modulus iteration.

Initial attempts to produce an iterative scheme were unsuccessful. This was due to the fact that the BISTRO analysis system (14) assumes that the pavement layers are weightless. The inclusion of self-weight stresses, which are frequently of similar magnitude to the wheel load stresses resulted in a reliable convergent iteration procedure.

4.3.2 The determination of self-weight stresses

Vertical stresses are simply calculated from the product of density and depth. However, horizontal stresses are not so straightforward. The usual method is to use a "coefficient of earth pressure at rest (K_o)" which is the ratio of horizontal to vertical stress under zero lateral strain conditions. Values for K_o may be obtained from the literature or by use of appropriate relationships.

For solution of an elastic system:

$$K_o = \frac{\nu}{1-\nu} \tag{4.10}$$

and this relationship is used in the analyses reported by Monismith et al (74). Brooker and Ireland (75) relate K_o to the results of material testing, recommending:

$$K_o = 0.95 - \sin\phi \tag{4.11}$$

for normally consolidated clay, and

$$K_o = 1 - \sin\phi \tag{4.12}$$

for cohesionless soil.

Values for various clays and sands have been recommended by Crofts et al (76).

A scan of some literature relating to the development of horizontal stresses during the construction of embankments indicated that the horizontal pressure could frequently exceed the vertical, and sometimes by a considerable margin (77-83). It was also indicated (80-82) that the stress history of a given layer was important in relation to deciding a value of K_o, and the use of an overconsolidation ratio was suggested (82). (The overconsolidation ratio is the ratio of the maximum stress at any time in the stress history of the material to the stress currently existing.)

Development of K_o values: The first stage in the development of appropriate K_o values was to estimate the stress history resulting from compaction of the material. Information obtained from the TRRL (84) concerning contact pressures during rolling enabled an estimate to be made of compaction stresses in granular layers of various thicknesses. From these estimates, and the calculation of vertical self-weight stresses a plot of overconsolidation ratio against depth was developed, an example being shown in Fig. 4.1. Use of these plots in conjunction with Fig. 4.2, derived from Lamb and Whitman (83), allowed the production of graphs of K_o as a function of depth for the various thicknesses of material considered. Fig. 4.3 is an example of this type of plot. Hence, a K_o value can be obtained for any point in a layer of prescribed thickness.

For the purposes of this analysis the granular layer was divided into four sub-layers. Hence, the K_o-depth plots, produced for several layer thicknesses, were similarly sub-divided and an average K_o for each sub-layer taken from the graph. Thus, Fig. 4.4 was produced, from which the average K_o value can be determined for each of the four sub-layers of an unbound layer between 100 and 700 mm in total thickness.

4.3.3 The non-linear model

As indicated in the literature review, the modulus of granular material is a function of the state of stress in that material. The most comprehensive study of the properties of granular materials of varying composition has been reported by Hicks (45). Thus, his relationship, in which:

$$E = K_1 \theta^{K_2} \tag{4.13}$$

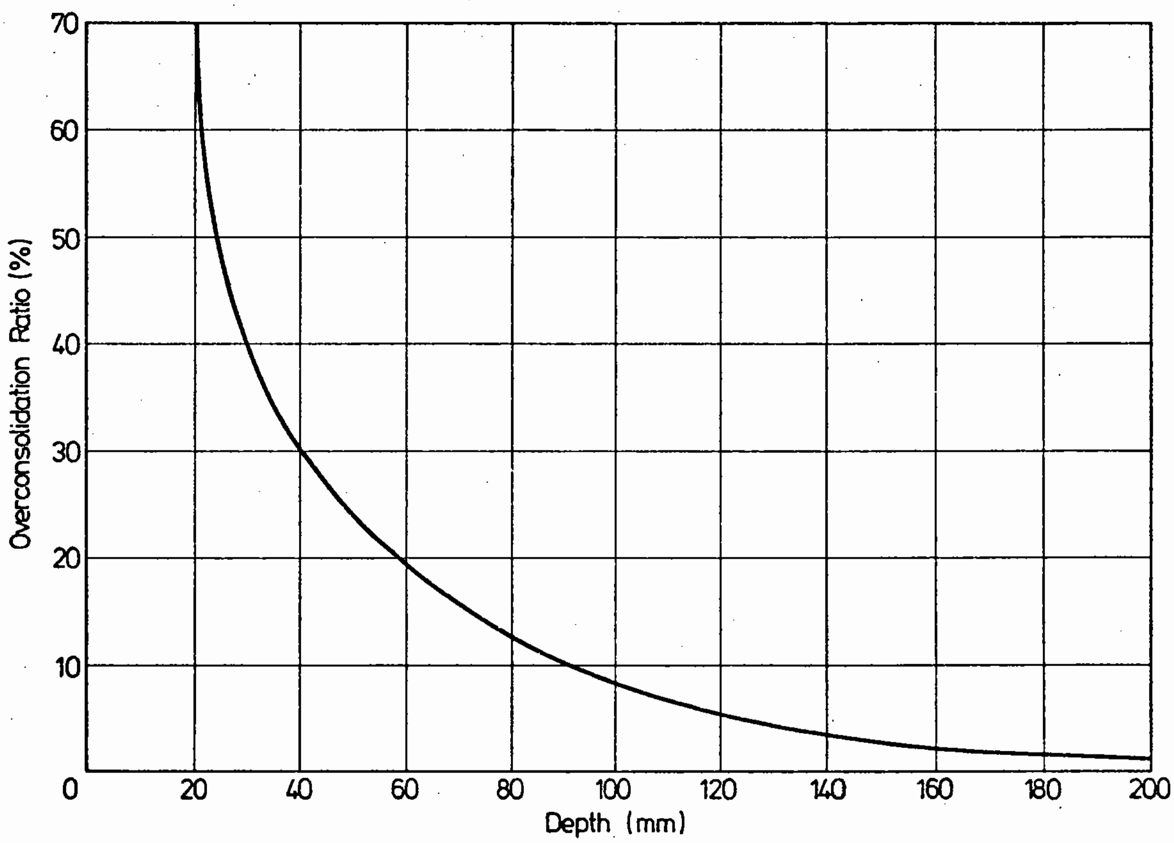

FIG. 4.1 OVERCONSOLIDATION RATIO AS A FUNCTION OF DEPTH

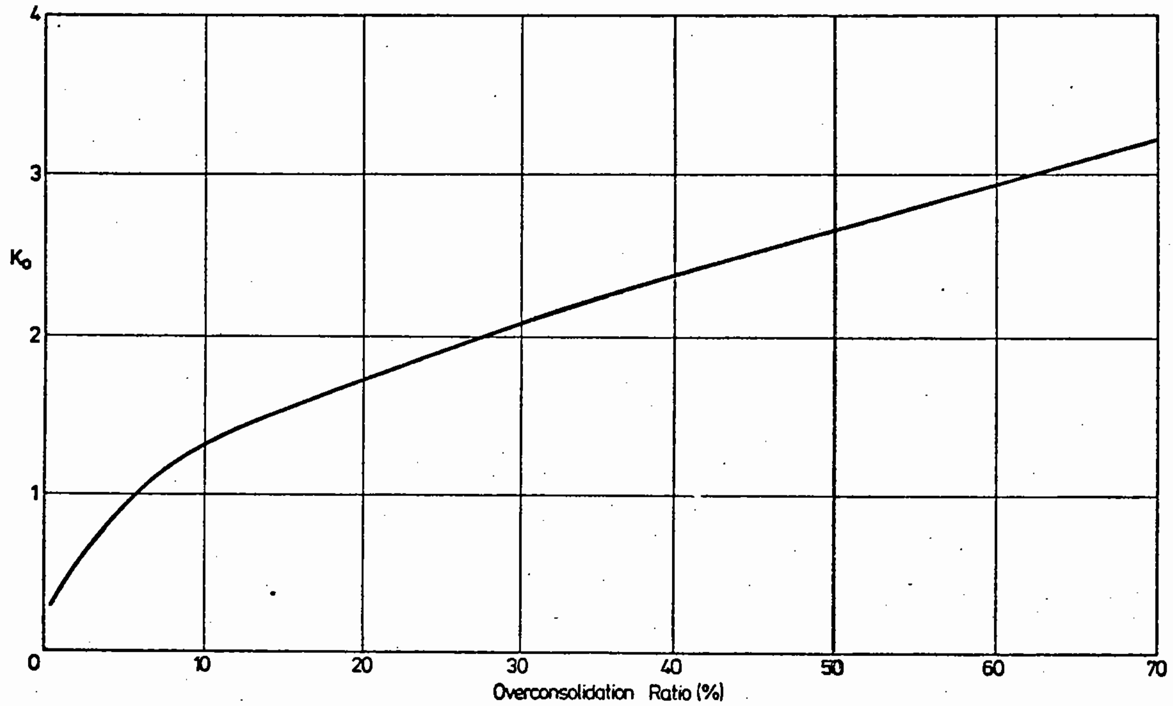

FIG. 4.2 K_0 AS A FUNCTION OF OVERCONSOLIDATION RATIO (After Lamb and Whitman, 83)

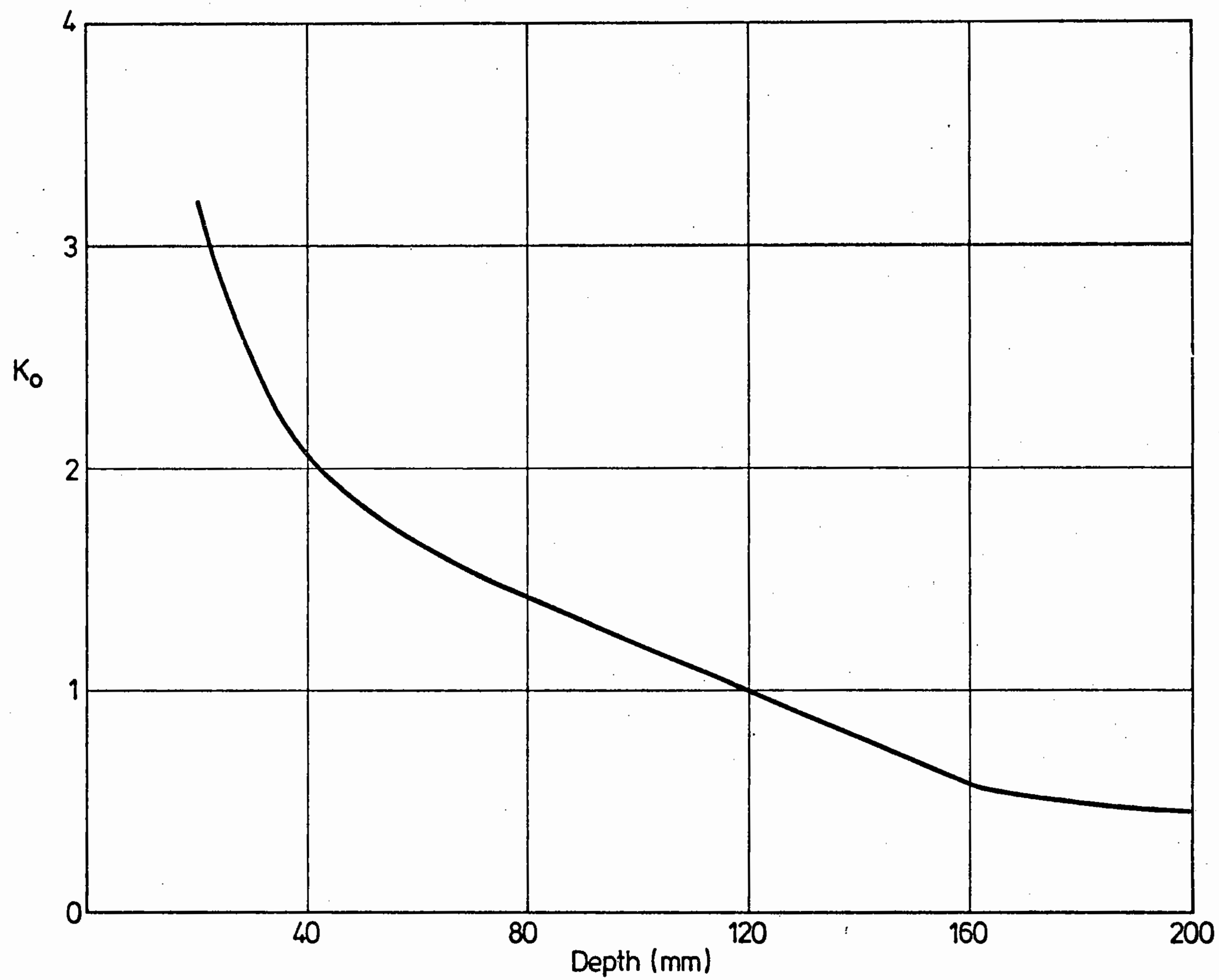

FIG. 4.3 K_o AS A FUNCTION OF DEPTH FOR A 200 mm LAYER

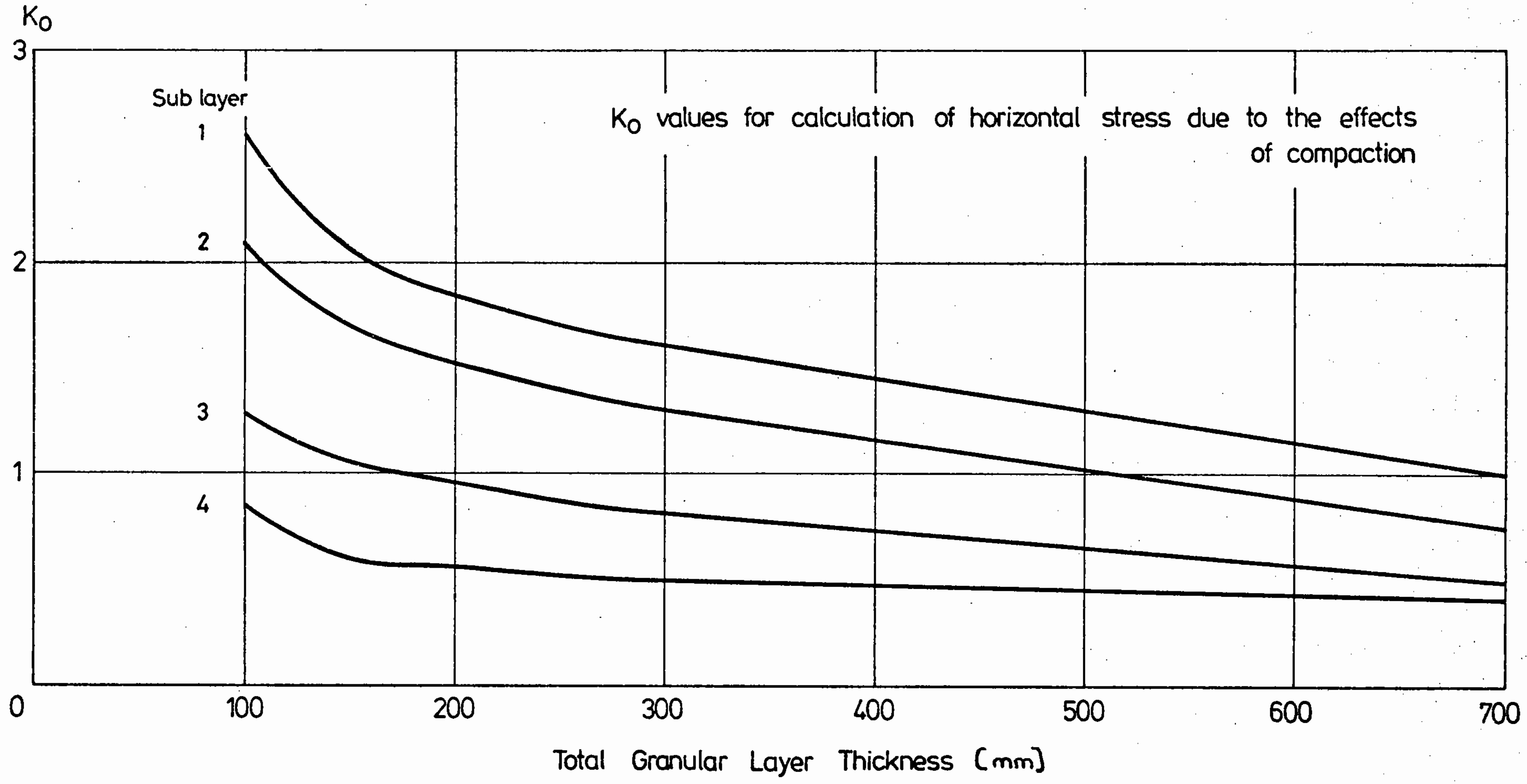

FIG. 4.4 K_o VALUES FOR A RANGE OF GRANULAR LAYER THICKNESSES CONSIDERED AS FOUR SUB-LAYERS

where E = modulus

θ = 3 × mean normal stress (p)

K_1, K_2 = material constants,

has been adopted for this research.

This relationship was derived from stress conditions well away from failure. However, since analysis can indicate failure conditions at certain locations in a granular layer, a failure criterion should be included in the analysis to avoid the derivation of unrealistic moduli.

The criterion incorporated in the model uses a limiting q/p ratio of 2.2. If this limit is exceeded, an arbitrarily low value of modulus (3 MN/m^2) is inserted into the calculation procedure.

Since it is not considered to be physically possible for the stiffness of the granular material to reduce abruptly when failure occurs, a further modification to the model was necessary. This was achieved by setting a lower stress ratio limit at q/p = 1 and between this value and q/p = 2.2 interpolating between the modulus appropriate to the mean normal stress (Equation 4.13) and the failure modulus, according to the distance across the zone bounded by q/p = 1 and q/p = 2.2.

It was found necessary to damp the oscillation of the iteration procedure in order to obtain convergence. This was achieved by summing the modulus used by calculating the stresses and the modulus derived from these stresses and dividing the result by a damping factor.* Fig. 4.5 is a q/p plot indicating the relationships used in the various zones and Fig. 4.6 is a flow chart of the modulus determination procedure.

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* Values for this factor range from 2 to 5.

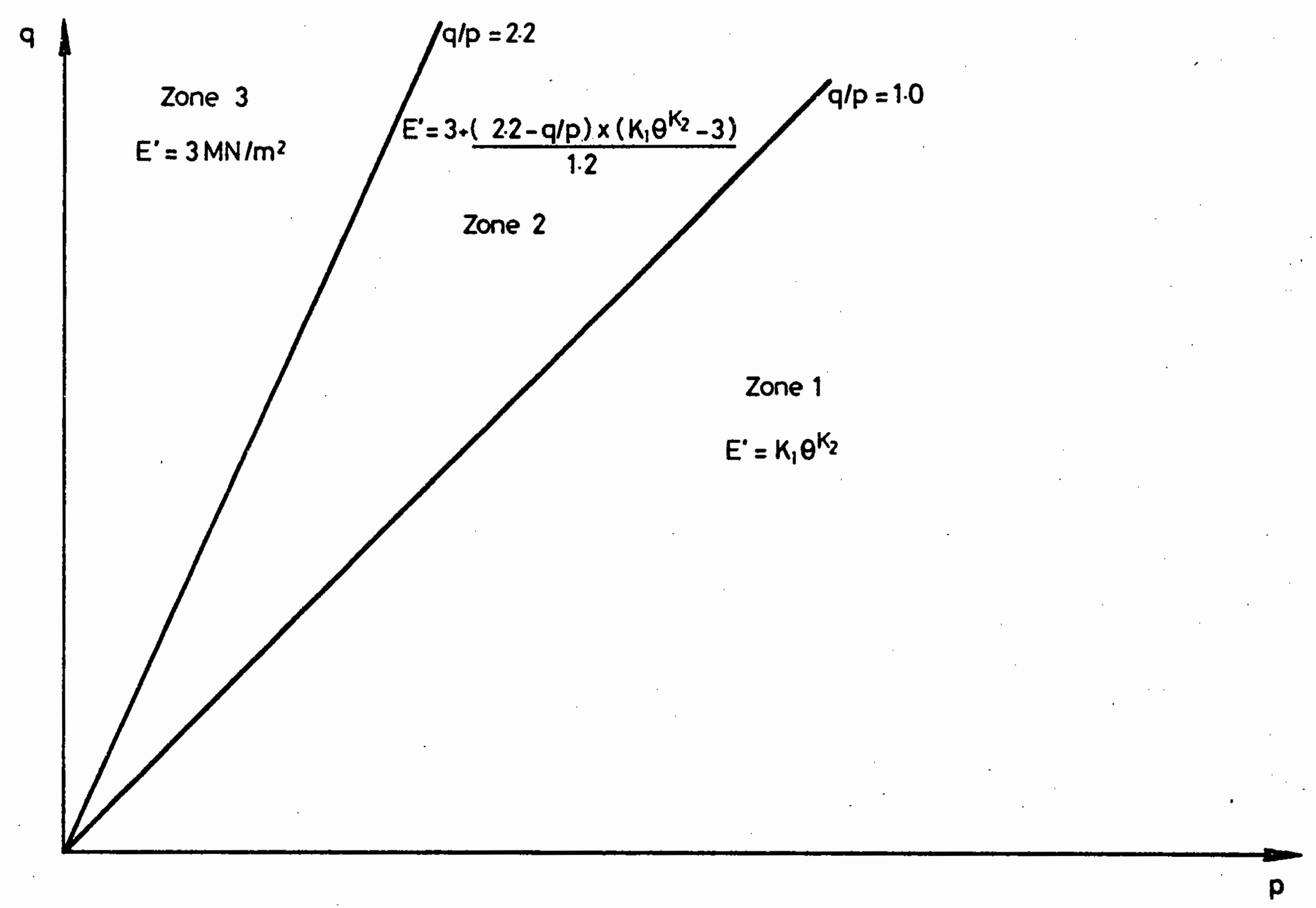


FIG. 4.5 FORMULAE FOR THE DERIVATION OF GRANULAR MATERIAL MODULUS FOR THREE ZONES IN p-q STRESS SPACE

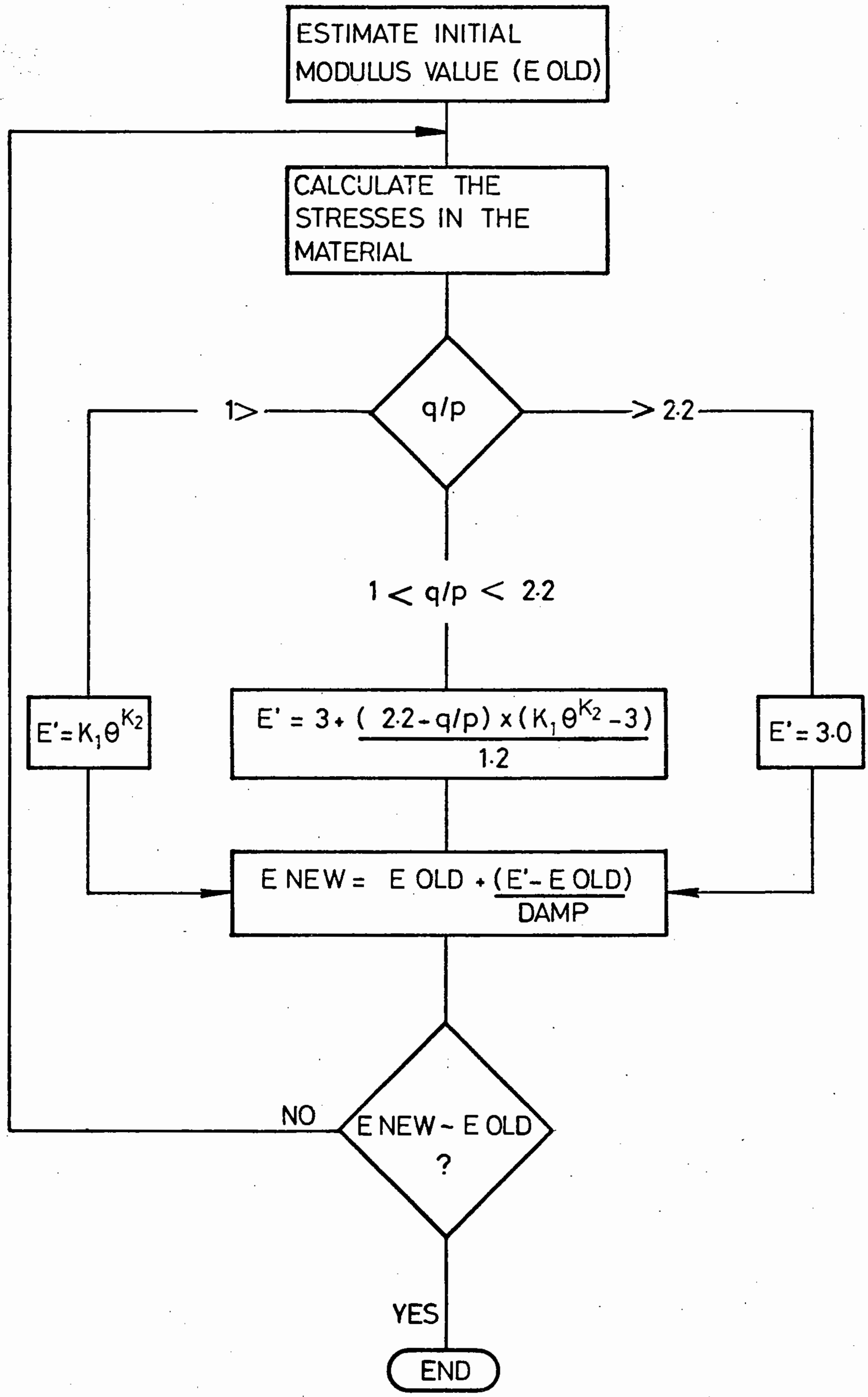


FIG. 4.6 FLOW CHART OF MODULUS DETERMINATION PROCEDURE FOR UNBOUND GRANULAR MATERIALS

4.4 THE EFFECT OF INTRODUCING A FAILURE CRITERION

A comparison was made between stress state, derived moduli, and primary response parameters for a range of structures analysed using non-linear theory, with and without the failure criterion. Nine pavement systems were investigated, as detailed in Fig. 4.7. The structures studied all had a subgrade of CBR 5% and a surfacing stiffness of 7,000 MN/m^2.

A detailed analysis of the stress state in the granular layer, and the implications thereof is given in later sections of this chapter. This section considers only the necessity for application of the failure criterion. The nine structures have been sub-divided into three sets of three, each set having three granular layer thicknesses, 200, 450 and 700 mm. The sub-division is on the basis of surfacing thickness which is 50 mm for structure 1, 200 mm for structure 2 and 400 mm for structure 3. In addition, the quality of the granular material as characterised by the value of K_1 in Equation 4.13 was varied.

4.4.1 The effect of the failure criterion on stress state

Figs 4.8, 4.9 and 4.10 compare the stress distributions, in p-q stress space, for structures 1, 2 and 3 respectively, each figure containing plots for the three thicknesses of granular material.

Structure 1 (Fig. 4.8) with only 50 mm surfacing has the most highly stressed granular layer. In this structure the deviator stress is high and carries the granular material into zone 3 as defined in Fig. 4.5. It is, therefore, evident that application of the failure criterion significantly modifies the stresses in these systems as is shown in Fig. 4.8.

It should be noted that the variation in stress through the layer, as indicated by the length of the p-q plot at any particular granular

Structure 1

$E = 7{,}000\ MN/m^2$

$\nu = 0.4$

$h = 50\ mm$

$K_1 = 600$		200 mm
$K_2 = 0.6$	$h =$	450 mm
$\nu = 0.3$		700 mm

CBR = 5

Structure 2

$E = 7{,}000\ MN/m^2$

$\nu = 0.4$

$h = 200$

$K_1 = 600$		200 mm
$K_2 = 0.6$	$h =$	450 mm
$\nu = 0.3$		700 mm

CBR = 5

Structure 3

$E = 7{,}000\ MN/m^2$

$\nu = 0.4$

$h = 400$

$K_1 = 600$		200 mm
$K_2 = 0.6$	$h =$	450 mm
$\nu = 0.3$		700 mm

CBR = 5

FIG. 4.7 PAVEMENT SYSTEMS ANALYSED TO INVESTIGATE THE EFFECT OF GRANULAR MATERIAL FAILURE CRITERIA

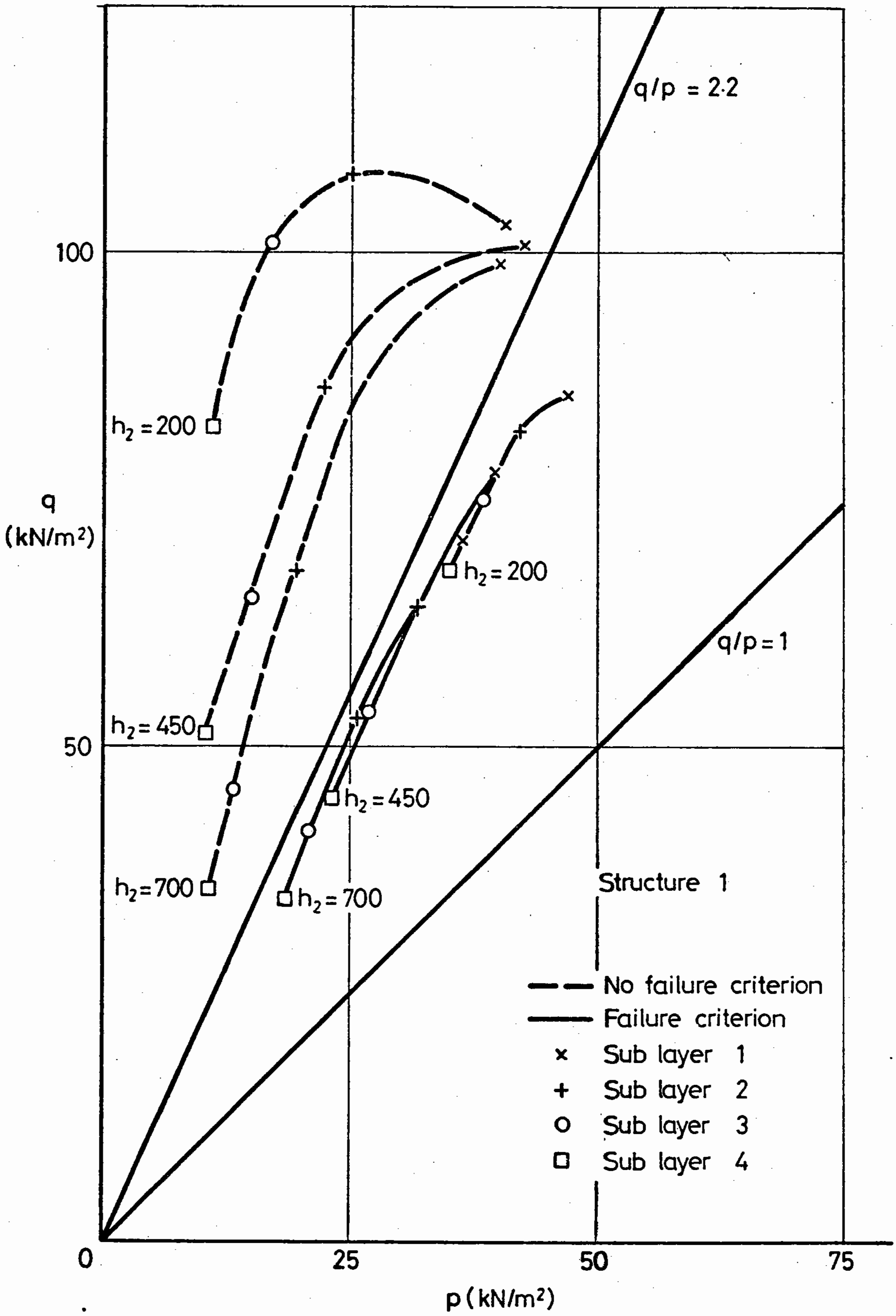


FIG. 4.8 GRANULAR LAYER STRESS DISTRIBUTION FOR STRUCTURE 1

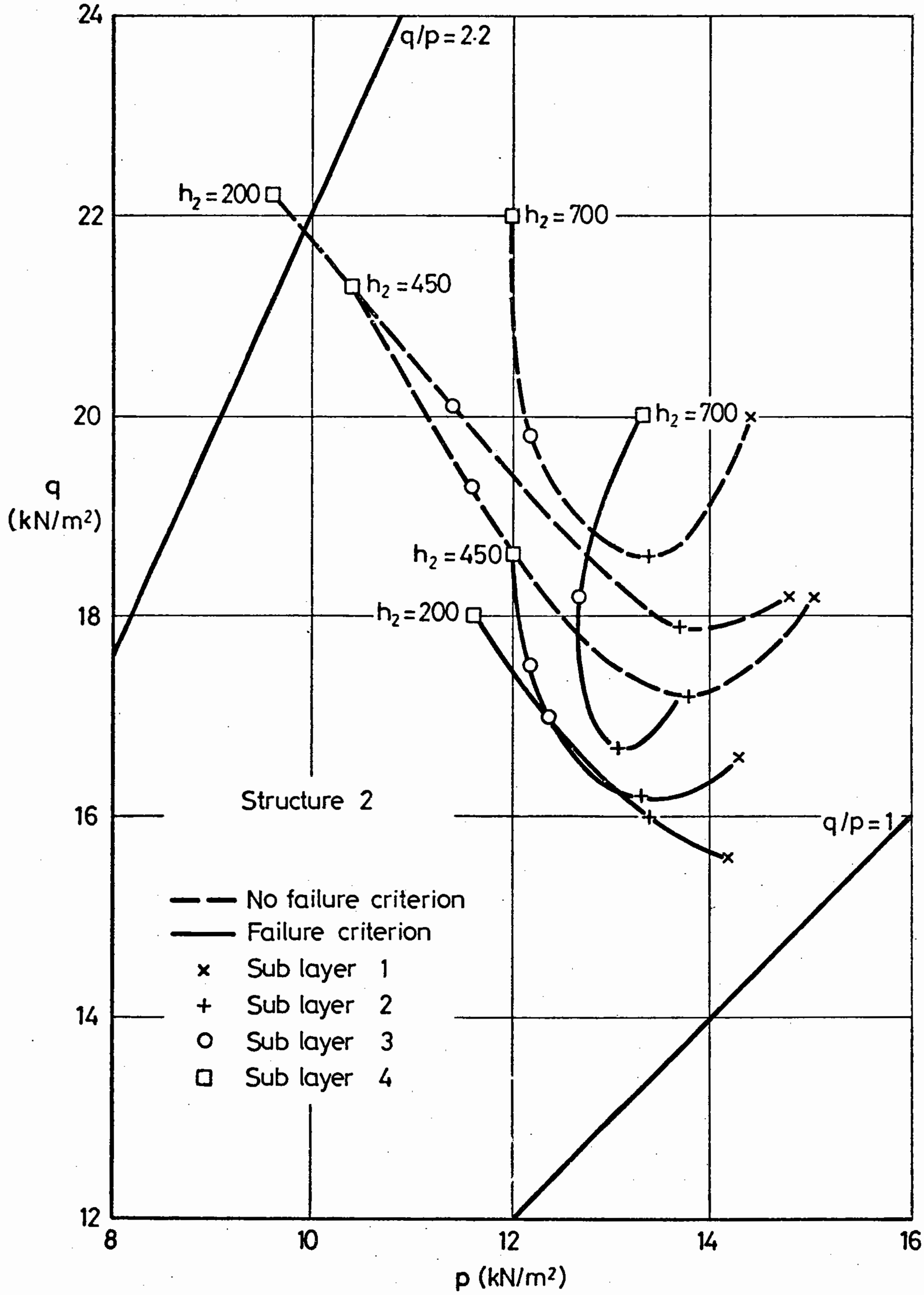


FIG. 4.9 GRANULAR LAYER STRESS DISTRIBUTION FOR STRUCTURE 2

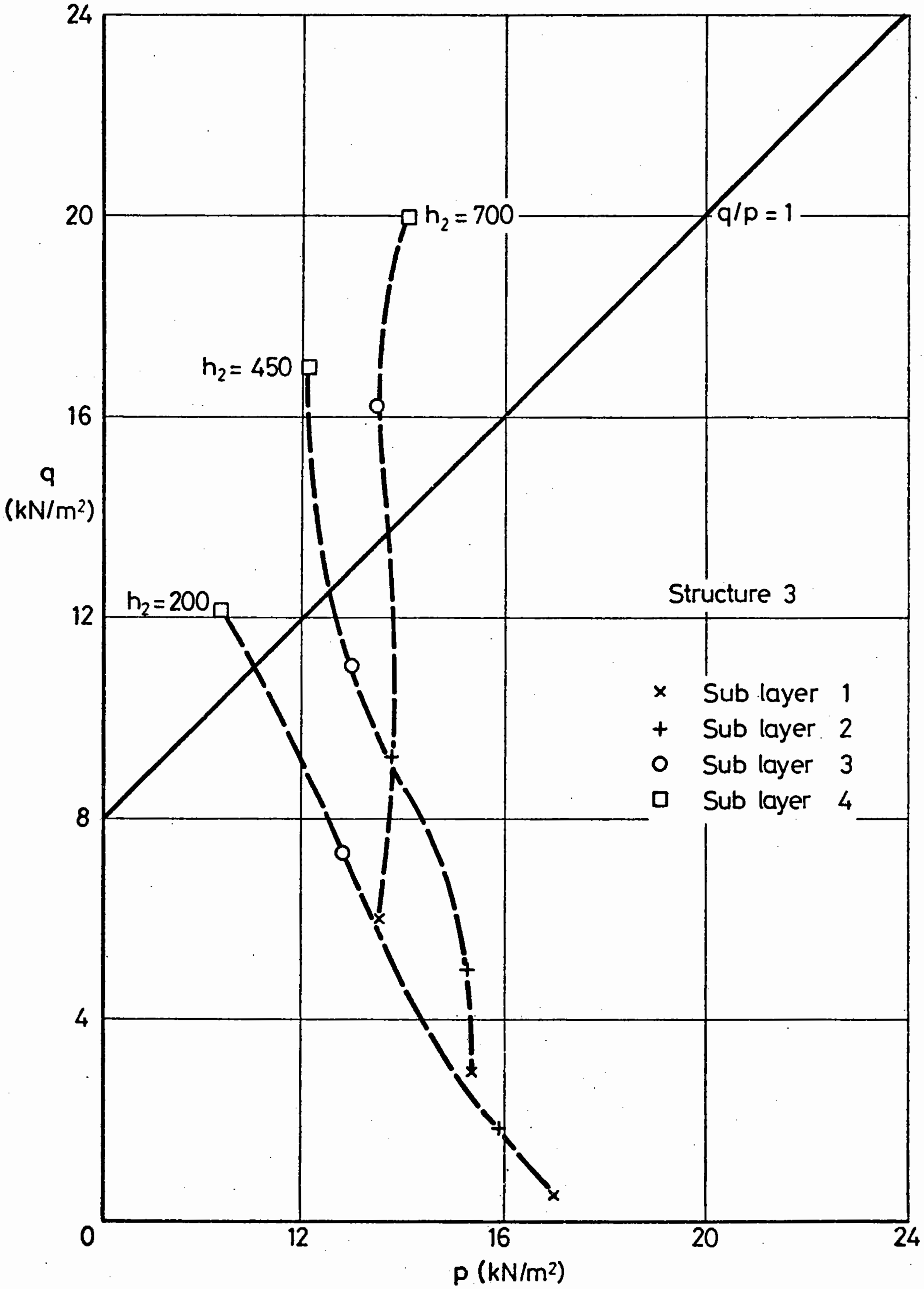


FIG. 4.10 GRANULAR LAYER STRESS DISTRIBUTION FOR STRUCTURE 3

Note: Stress distribution with and without failure criteria are identical.

layer thickness, is much reduced by the application of the failure criterion. Even after application of the failure criterion, the granular layer is very close to failure throughout its depth.

With a 200 mm asphalt surfacing (Fig. 4.9) the stresses within the granular material in structure 2 are lower than those in structure 1, and, with the exception of the fourth sub-layer of the 200 mm base system, the stresses are below failure level. Thus there is once more a significant modification to the stress field within the granular layer.

Structure 3 has 400 mm of surfacing and the stresses in this system (Fig. 4.10) are the lowest of the three. It is mostly in zone 1 (Fig. 4.5) of the model and hence not subject to modification by the failure criterion, as confirmed in Fig. 4.10. Some sub-layers do fall in zone 2, but since the stresses are small the modification to the stress distribution is not significant.

4.4.2 The effect of the failure criterion on derived moduli

The most significant modifications to the derived moduli occur in the highly stressed system of structure 1, as would be expected. The reduction is considerable, the modified values being between 33% and 50% of the unmodified ones, as shown in Fig. 4.11.

The modification is much less pronounced for structure 2 (Fig. 4.12) though there is a reduction of between 35% and 43% in the modulus of the bottom sub-layer.

Finally, with the exception of the bottom sub-layer, structure 3 is unaffected (Fig. 4.13). Reference to the stress plot in Fig. 4.10 indicates that the bottom sub-layer of all three systems is within zone 2 and hence the moduli values are modified, but the modification

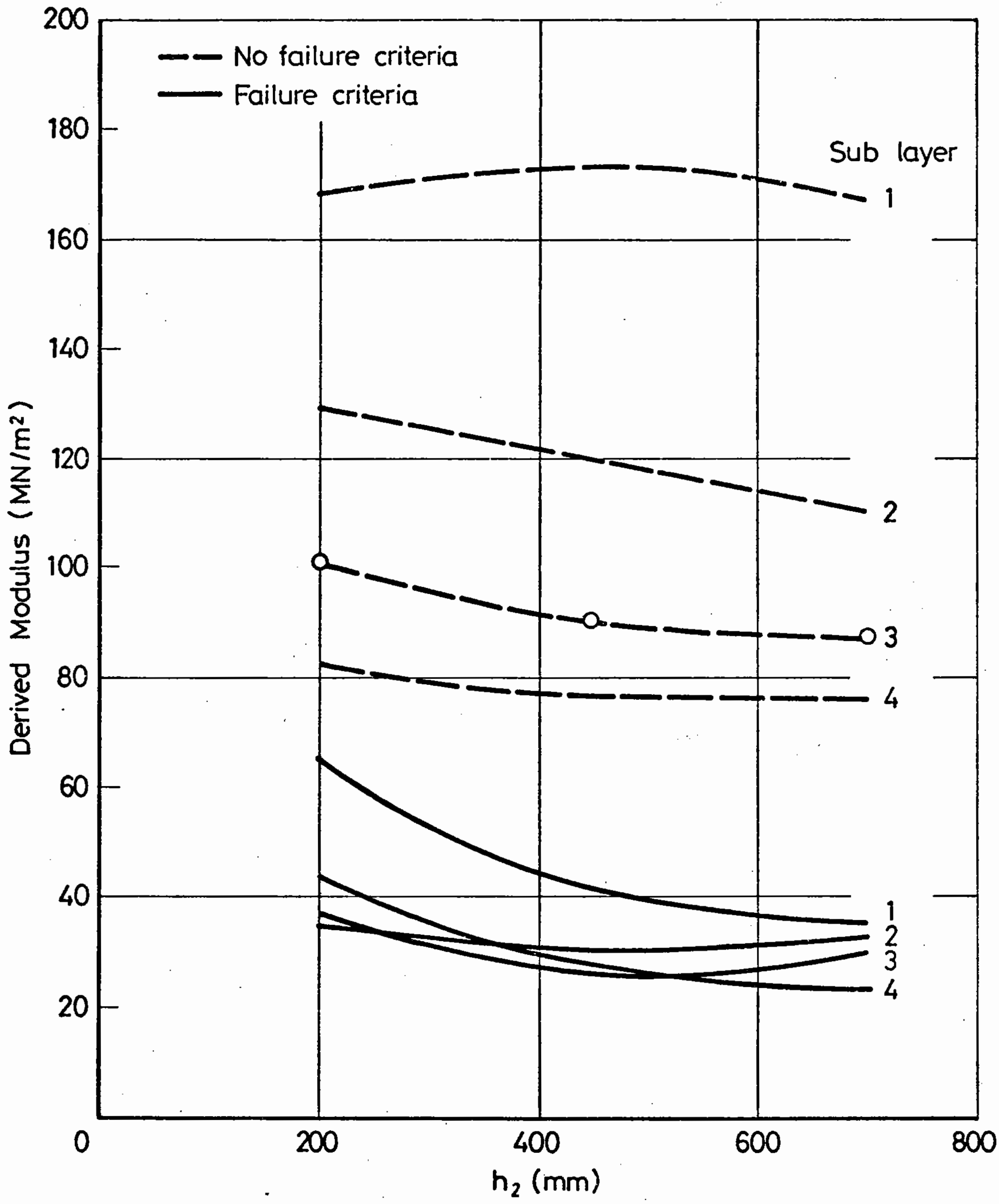


FIG. 4.11 COMPARISON OF DERIVED MODULI FOR STRUCTURE 1

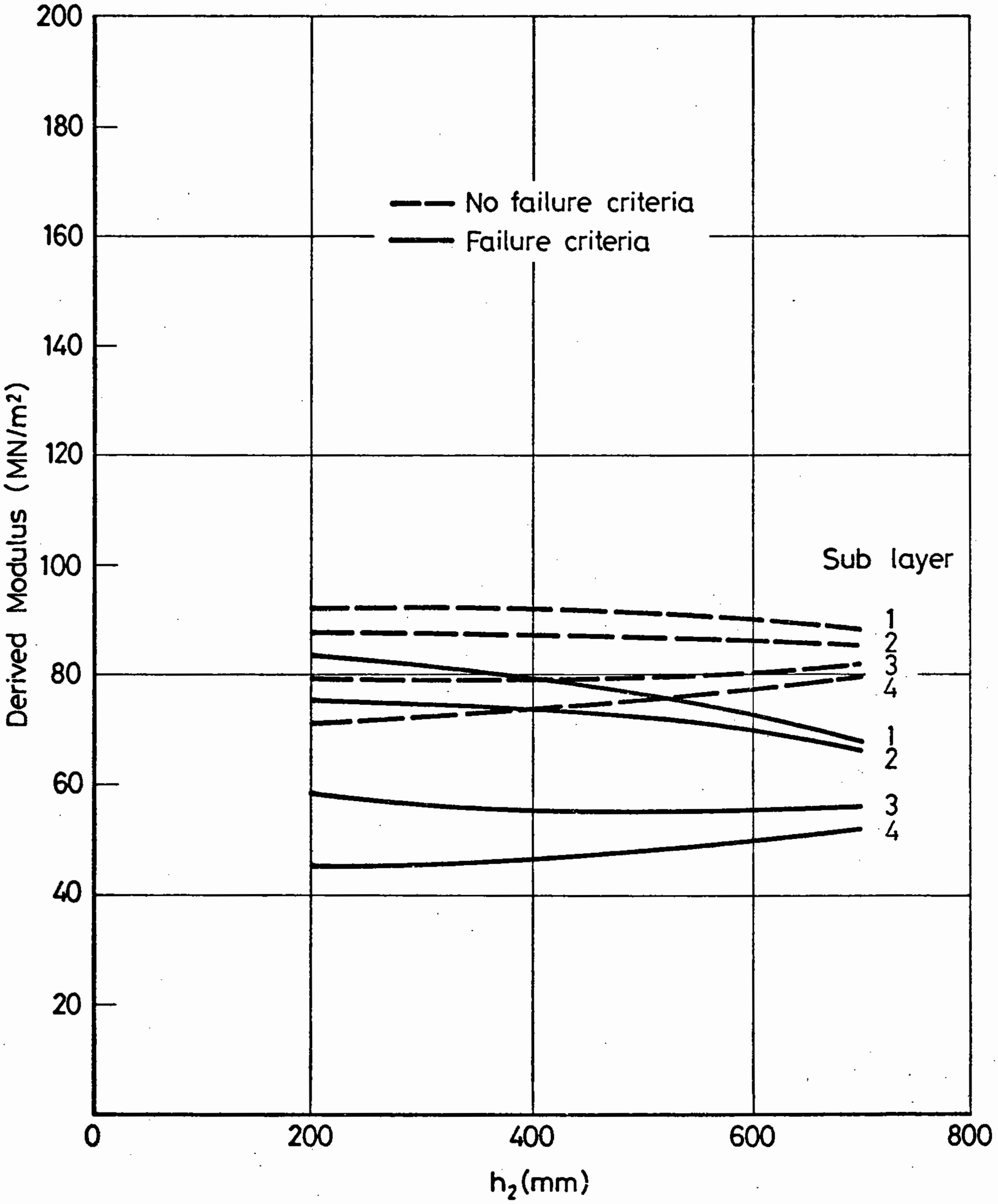


FIG. 4.12 COMPARISON OF DERIVED MODULI FOR STRUCTURE 2

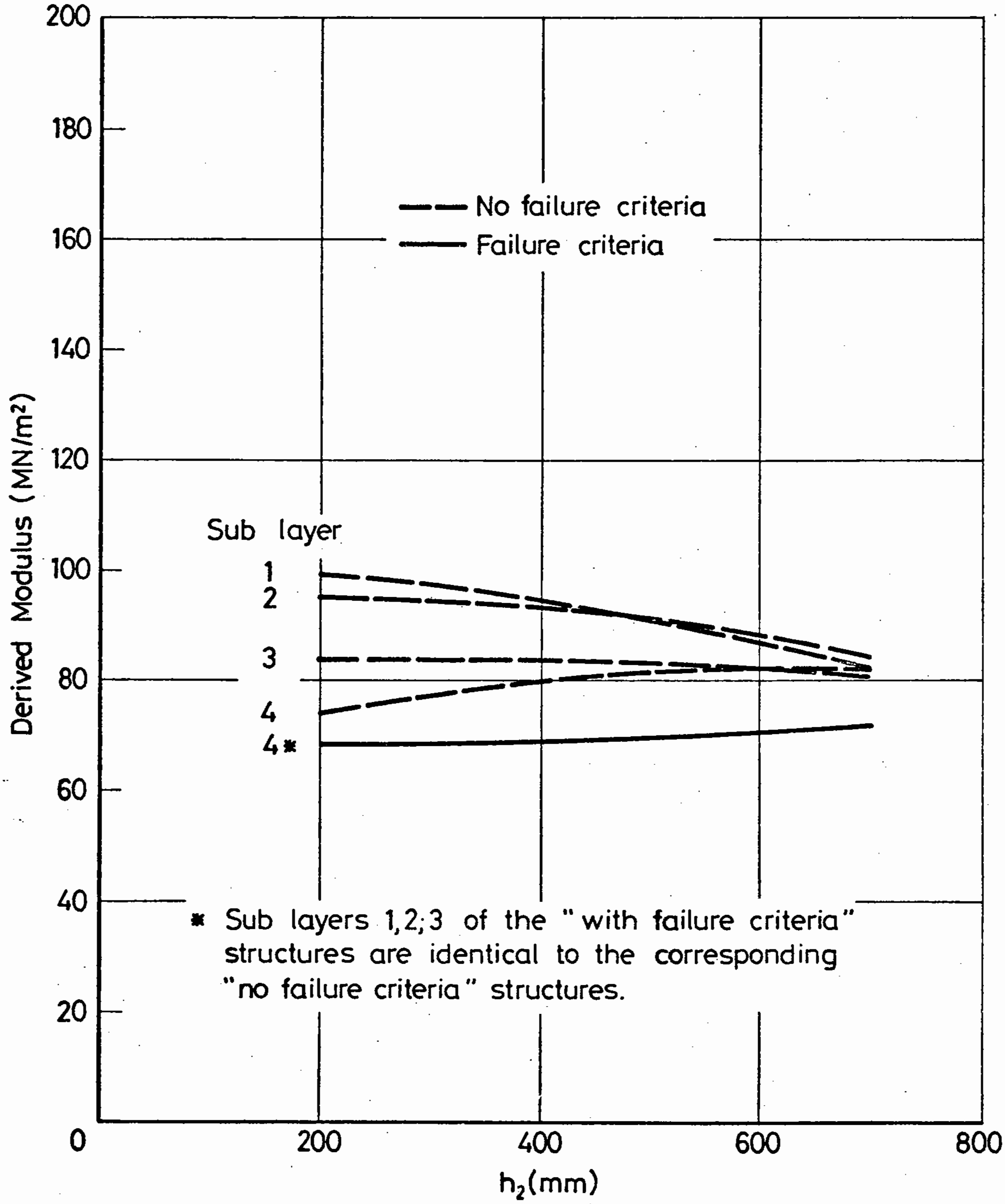


FIG. 4.13 COMPARISON OF DERIVED MODULI FOR STRUCTURE 3

is not large enough to cause significant change to the stresses within the granular layer.

4.4.3 The effect of the failure criterion on primary response parameters

The two primary response parameters chosen for this study were horizontal tensile strain in the asphalt and vertical strain on the subgrade. These parameters were chosen since they are commonly used in pavement design.

Figs 4.14 and 4.15 show the variations in subgrade strain and asphalt strain for the range of structures investigated. As would be expected from the preceeding sections, there is no effect in structure 3, since the unbound layer was largely unaffected by the failure criterion. Similarly, there is little effect in structure 2, except for asphalt strain over the 700 mm thick granular layer.

Structure 1 is most affected by the failure criterion. There is a marked difference at the subgrade level, the strains calculated on the basis of a failure criterion analysis being higher than those derived on the basis of no failure criterion. Fig. 4.15 shows that for asphalt strains, in structure 1, the difference between the two analyses is considerable. Opposite trends are observed, strain reducing with increasing granular layer thickness if the failure criterion is omitted but increasing when it is included.

Both primary response parameters are lower when failure is ignored. This will lead to an under-designed pavement, which is not acceptable. Hence, it has been concluded that a failure criterion must be included in the modulus iteration procedure proposed for the characterisation of granular materials and, accordingly, subsequent studies were undertaken on this basis.

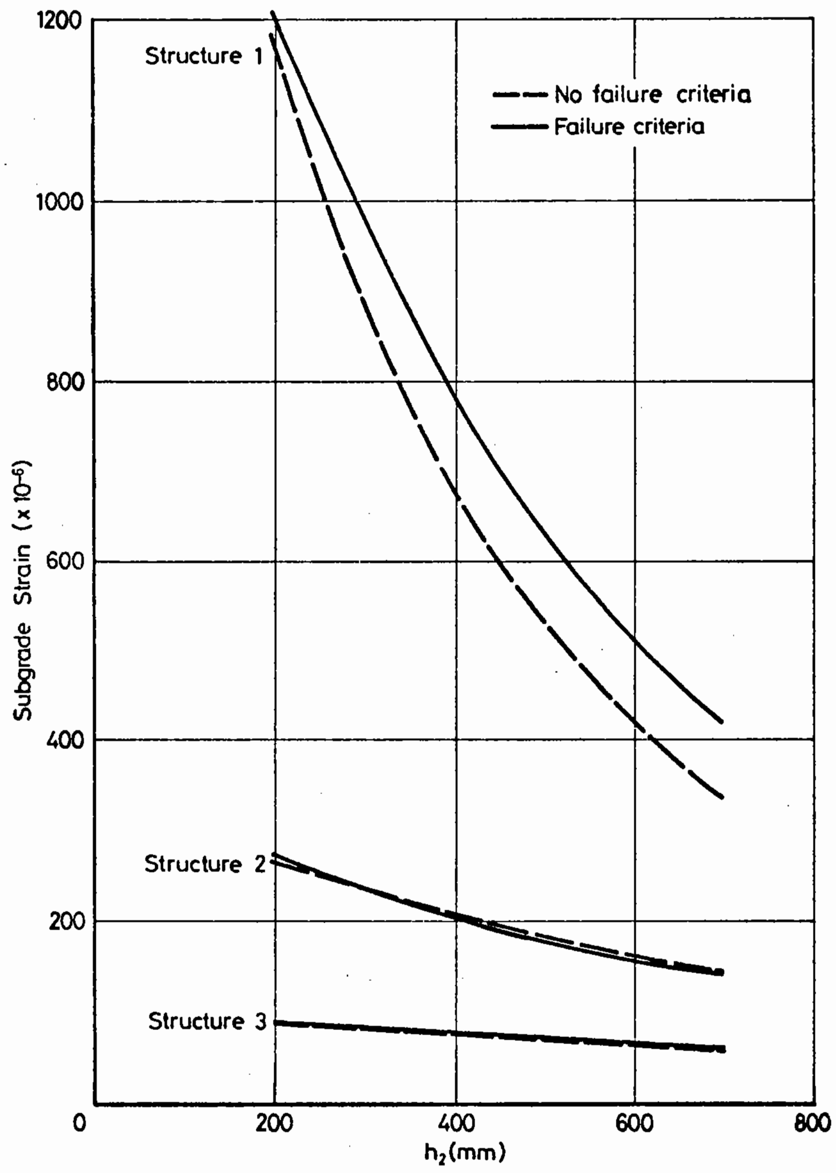


FIG. 4.14 COMPARISON OF SUBGRADE STRAIN FOR THE TWO METHODS OF ANALYSIS

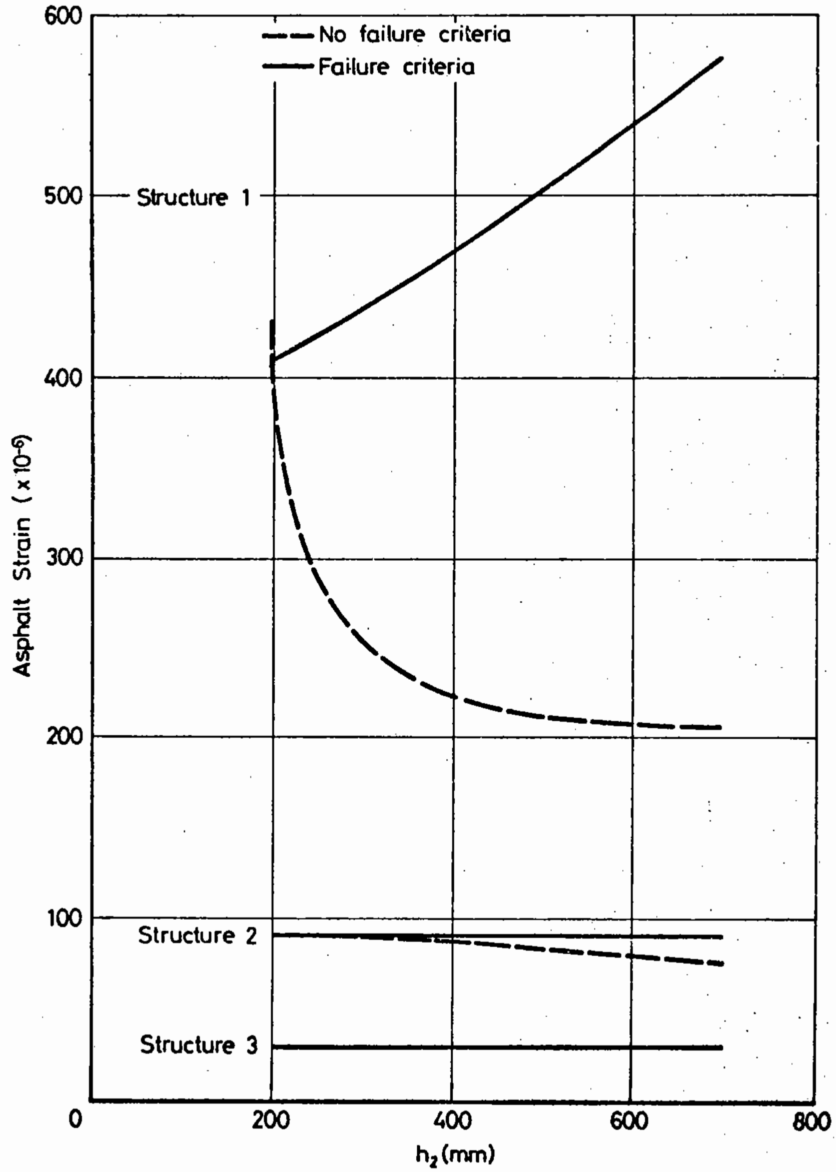


FIG. 4.15 COMPARISON OF ASPHALT STRAIN FOR THE TWO METHODS OF ANALYSIS

CHAPTER FIVE

THE STRUCTURAL BEHAVIOUR OF GRANULAR MATERIAL IN HIGHWAY PAVEMENTS

5.1 INTRODUCTION

Having developed a system for the non-linear elastic analysis of unbound granular material in pavements the structural behaviour of granular layers was investigated.

Before studying the behaviour of the granular layer, it was necessary to decide upon control parameters for the investigation. In a similar study with a less sophisticated material model, on three-layer pavement systems, Smith (72) chose material constant K_1, asphalt stiffness (E_A), subgrade modulus (E_S), asphalt thickness (h_1) and unbound layer thickness (h_2) as study parameters. It would seem appropriate to make a similar choice since K_2 (Equation 4.13) does not appear to vary greatly (60), and, whilst the failure criterion has a very important influence on the results, no information was available to indicate how it varies.

The most complete method of undertaking a parametric study is to vary each parameter in turn throughout the range of interest with all others held constant. However, this leads to a requirement to investigate a large number of structures, many of which will probably be outside the range covered by normal construction practice.

From a study of Road Note No. 29 (16), asphalt layer thicknesses vary from approximately 50 to 200 mm in unbound base structures and from 125 to 400 mm in bound base structures. Similarly, unbound layer thicknesses range from 400 to 750 mm for unbound base structures, and from 275 to 475 mm for bound base structures. Thus asphalt thicknesses ranging from 50 to 400 mm and unbound layer thicknesses ranging from 275 to 750 mm cover likely pavement structures. Accordingly, three basic structures were chosen for studies of the effects of K_1 and subgrade modulus, both indicated by Smith (72) as being parameters of major importance, the minimum thickness of granular material being reduced to 200 mm for comparison with previous work (41). These structures have 50 mm of asphalt on 200 mm of unbound material; 200 mm of asphalt on 400 mm of unbound material, and 400 mm asphalt on 700 mm of unbound material. The asphalt layer was assigned a stiffness of 7,000 MN/m^2 for all these structures. Table 5.1 indicates the parameter combinations studied.

Additional structures were investigated as detailed in Table 5.2 to assess the effects of asphalt modulus, asphalt thickness and granular layer thickness. These structures were studied with K_1 = 600 and on a subgrade of modulus 50 MN/m^2.

It will be noted from Table 5.2 that several structures are duplicated. They have, however, been included in the tabulation to give a clear indication of the comparative studies undertaken.

5.2 THE STRESS DISTRIBUTION WITHIN THE GRANULAR LAYER

5.2.1 The effect of K_1 and subgrade modulus

The effects of variation of K_1 and subgrade modulus are plotted in Figs 5.1, 5.2 and 5.3 for structures 1, 2 and 3 respectively.

The effect of E_s on the individual sublayers (Fig. 5.1) for structure 1, the most highly stressed of the three basic systems, will be discussed initially. For all K_1 the stresses in each sublayer decrease as E_s decreases, the q/p ratio at any point not showing a major change. As K_1 is reduced, the octahedral shear stress (q) decreases and the mean normal stress (p) increases, resulting in an increased margin between the existing stress state and the failure condition.

Structure No.	Subgrade Stiffness (MN/m^2)	K_1	Asphalt Thickness (mm)	Unbound Layer Thickness (mm)
1	30	600	50	200
		400	50	200
		200	50	200
	50	600	50	200
		400	50	200
		200	50	200
	70	600	50	200
		400	50	200
		200	50	200
2	30	600	200	450
		400	200	450
		200	200	450
	50	600	200	450
		400	200	450
		200	200	450
	70	600	200	450
		400	200	450
		200	200	450
3	30	600	400	700
		400	400	700
		200	400	700
	50	600	400	700
		400	400	700
		200	400	700
	70	600	400	700
		400	400	700
		200	400	700

Table 5.1 Parameters used in the initial study of the structural behaviour of granular layers

Parameter varied	Subgrade Stiffness (E_s)	Asphalt Modulus (E_A)	Asphalt Thickness (h_1)	Unbound Layer Thickness (h_2)
	(MN/m^2)	(MN/m^2)	(mm)	(mm)
h_1	50	7,000	50	200 *
	50	7,000	200	200
	50	7,000	400	200 *
h_2	50	7,000	50	200 *
	50	7,000	50	450
	50	7,000	50	700 *
E_A	50	3,000	50	200
	50	7,000	50	200 *
	50	12,000	50	200
h_1	50	7,000	50	700 *
	50	7,000	200	700
	50	7,000	400	700 *
h_2	50	7,000	400	200 *
	50	7,000	400	450
	50	7,000	400	700 *
E_A	50	3,000	400	700
	50	7,000	400	700 *
	50	12,000	400	700

* These structures are duplicated.

Table 5.2 Parameters used in the additional study of the structural behaviour of granular layers

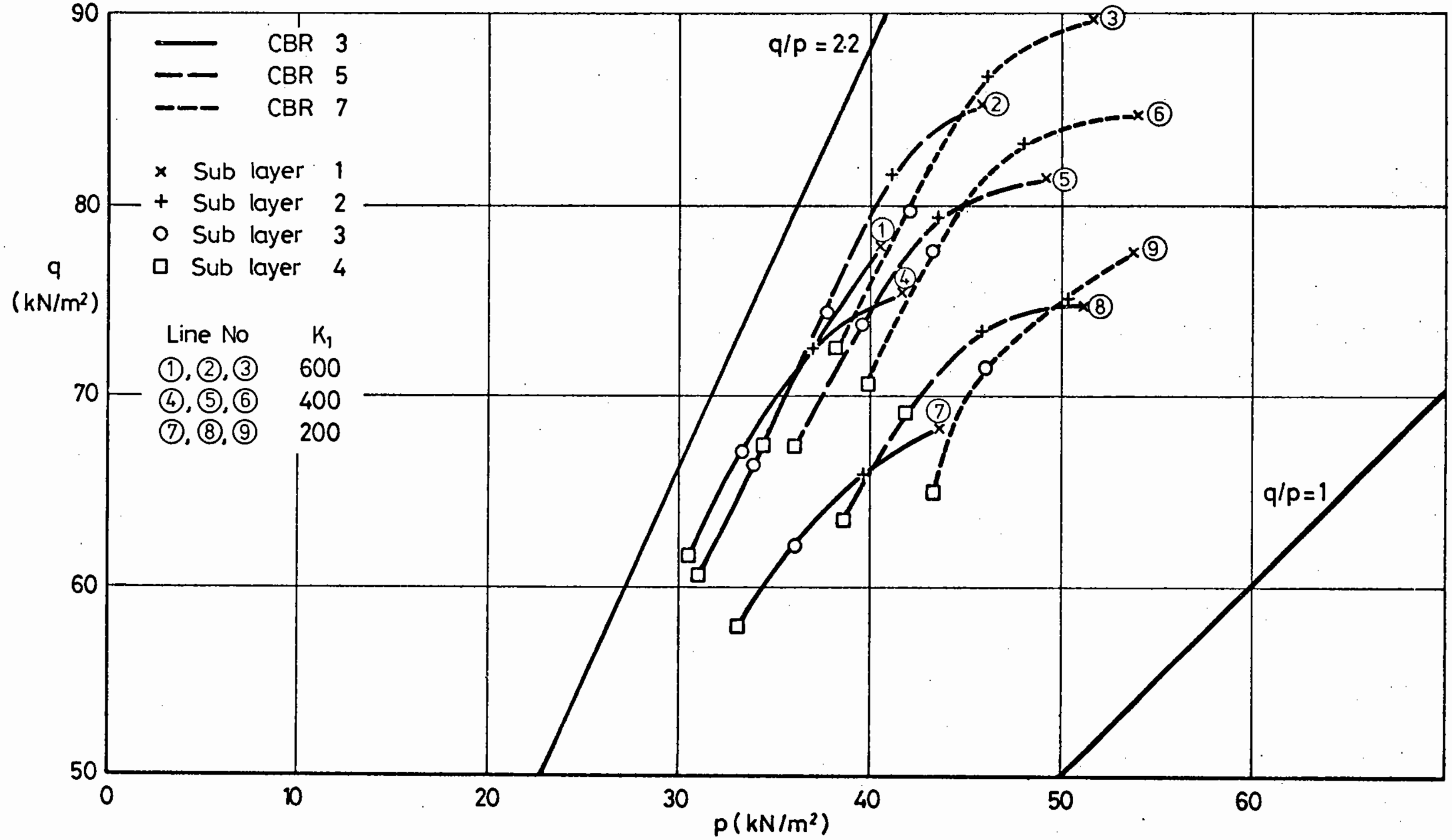


FIG. 5.1 STRESS DISTRIBUTION WITHIN THE GRANULAR LAYER FOR STRUCTURE 1

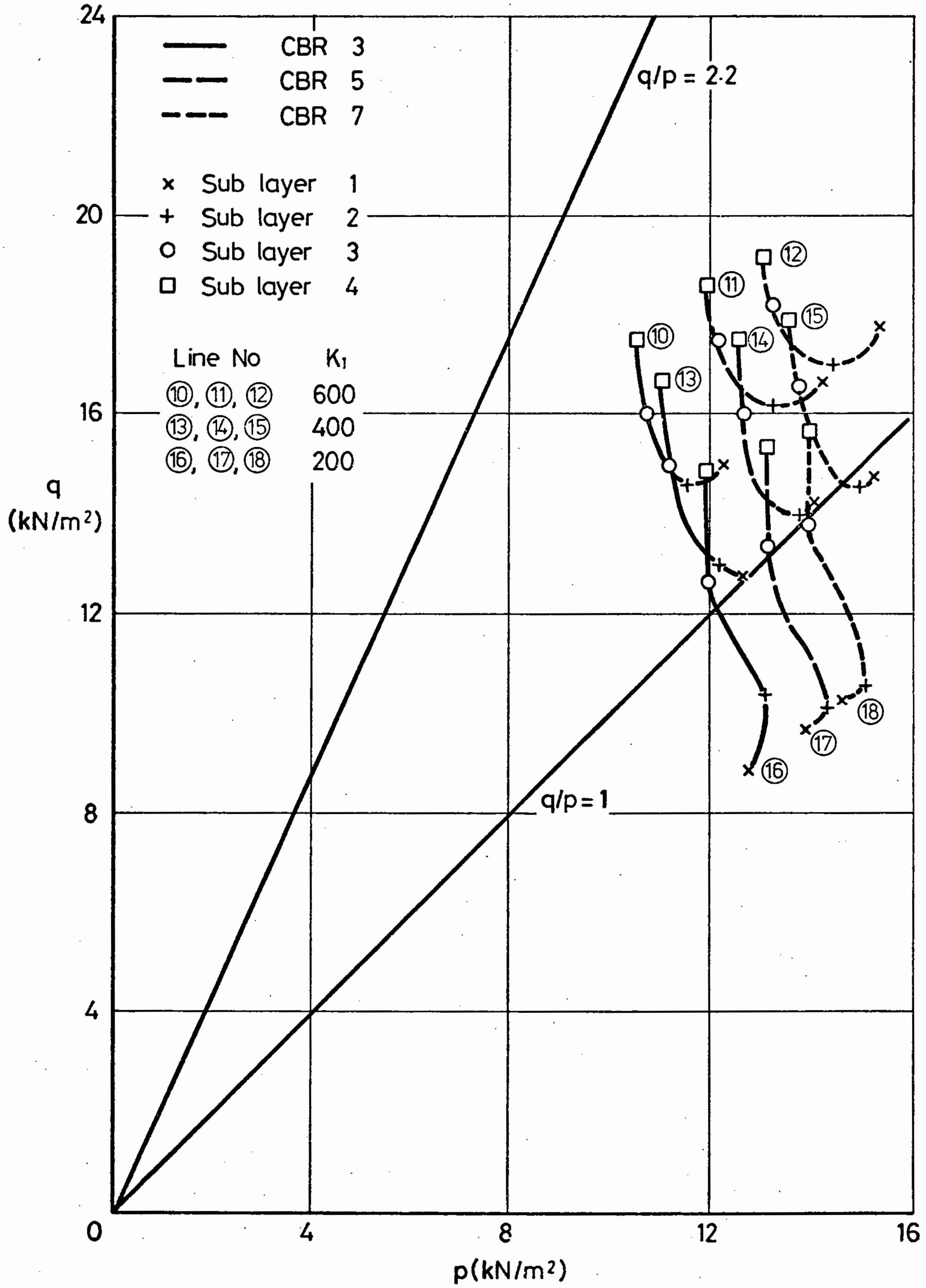


FIG. 5.2 STRESS DISTRIBUTION WITHIN THE GRANULAR LAYER FOR STRUCTURE 2

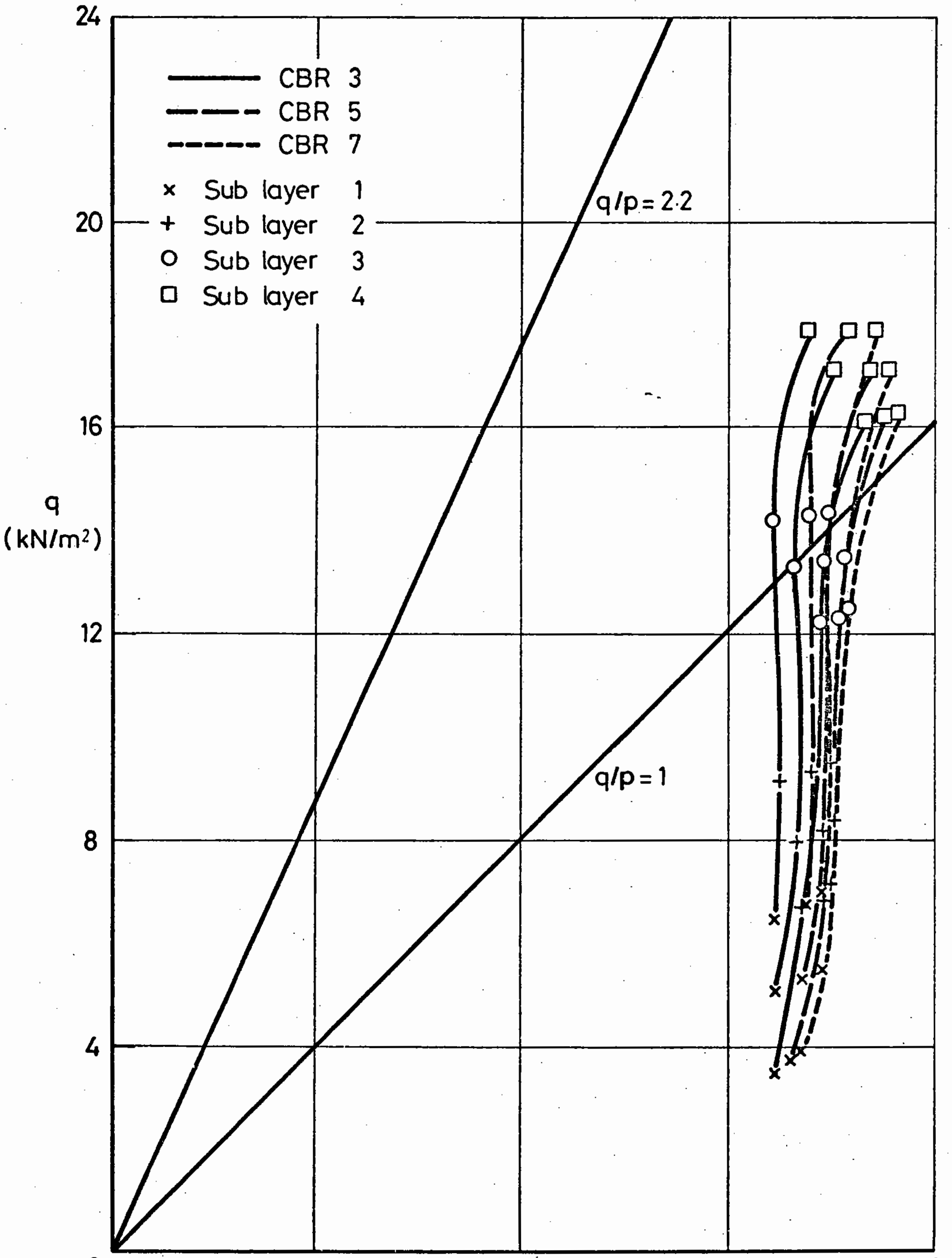


FIG. 5.3 STRESS DISTRIBUTION WITHIN THE GRANULAR LAYER FOR STRUCTURE 3

For the conditions examined in structure 1, the stresses decrease with depth. This indicates that wheel load stresses rather than the self-weight stresses, are most significant in determining the stress distribution. The stress distributions are broadly similar for all cases.

A much reduced level of stress is encountered in structure 2 (Fig. 5.2) and significantly different variation of stress with depth. At K_1 = 600, p decreases with depth, as for structure 1. However, q, whilst decreasing from sublayer 1 to sublayer 2, then increases, the highest q existing in the bottom sublayer. At K_1 = 400 a similar stress variation is observed, though at the lowest value of E_s (30 MN/m^2) there is no decrease in q from sublayer 1 to sublayer 2. The stress distribution with depth is distinctly changed when K_1 = 200. For this case, q increases with depth and p increases from sublayer 1 to sublayer 2, then decreases to below the sublayer 1 value at sublayer 3, remaining constant at this value in sublayer 4.

The variation in stress distribution with depth can be most clearly seen by examining the curves marked 10, 13 and 16 in Fig. 5.2.

The considerable change in stress distribution between structures 1 and 2 can be attributed to the increased significance of self-weight stresses in the thicker structure. In structure 1, the horizontal self-weight stresses are in the region of half those due to wheel loads, while in the vertical direction, they are approximately one-tenth. This situation changes dramatically for structure 2, when the horizontal self-weight stresses may be 10 times the wheel load stresses, the latter frequently being tensile, and the vertical self-weight stresses increase to approximately twice the wheel load stresses. Thus, the self-weight stresses dominate the analysis.

Having considered the dissimilarity between the stress distributions for structures 1 and 2, it should be noted that the effect of varying the subgrade modulus on the stress distribution is similar in both structures. Both p and q decrease by approximately the same amount with decreasing E_s, maintaining an approximately constant q/p ratio at each sublayer.

The very thick pavement systems represented by structure 3 (Fig. 5.3) are relatively insensitive to variations in either K_1 or subgrade modulus. It is interesting to note that p hardly varies at all through the granular layer. The octahedral shear stress q, however, varies considerably and in common with structure 2 is at a maximum in the bottom of the sublayer. The virtually constant value of p indicates that the wheel load stresses which decrease in the vertical direction and increase into the tensile region in the horizontal direction are being balanced by the change in self-weight stresses.

Comparing structures 2 and 3 it will be noted that q in the bottom sub-layer is approximately 18 kN/m^2 for both systems. This is due to the fact that the change in wheel load stress between the structures is balanced by the changes in self-weight.

5.2.2 The effect of varying the thickness of the asphalt layer

The effect of varying the thickness of the asphalt layer is shown in Figs 5.4 and 5.5. As indicated in Table 5.2, this study was undertaken on granular layers 200 and 700 mm thick, surfaced with 50, 200 and 400 mm of asphalt.

Increasing the thickness of the surfacing has a considerable effect on both structures. On the 200 mm granular layer (Fig. 5.4) increasing the asphalt layer thickness from 50 to 200 mm reduces the stress in the granular material dramatically and also reduces the stress

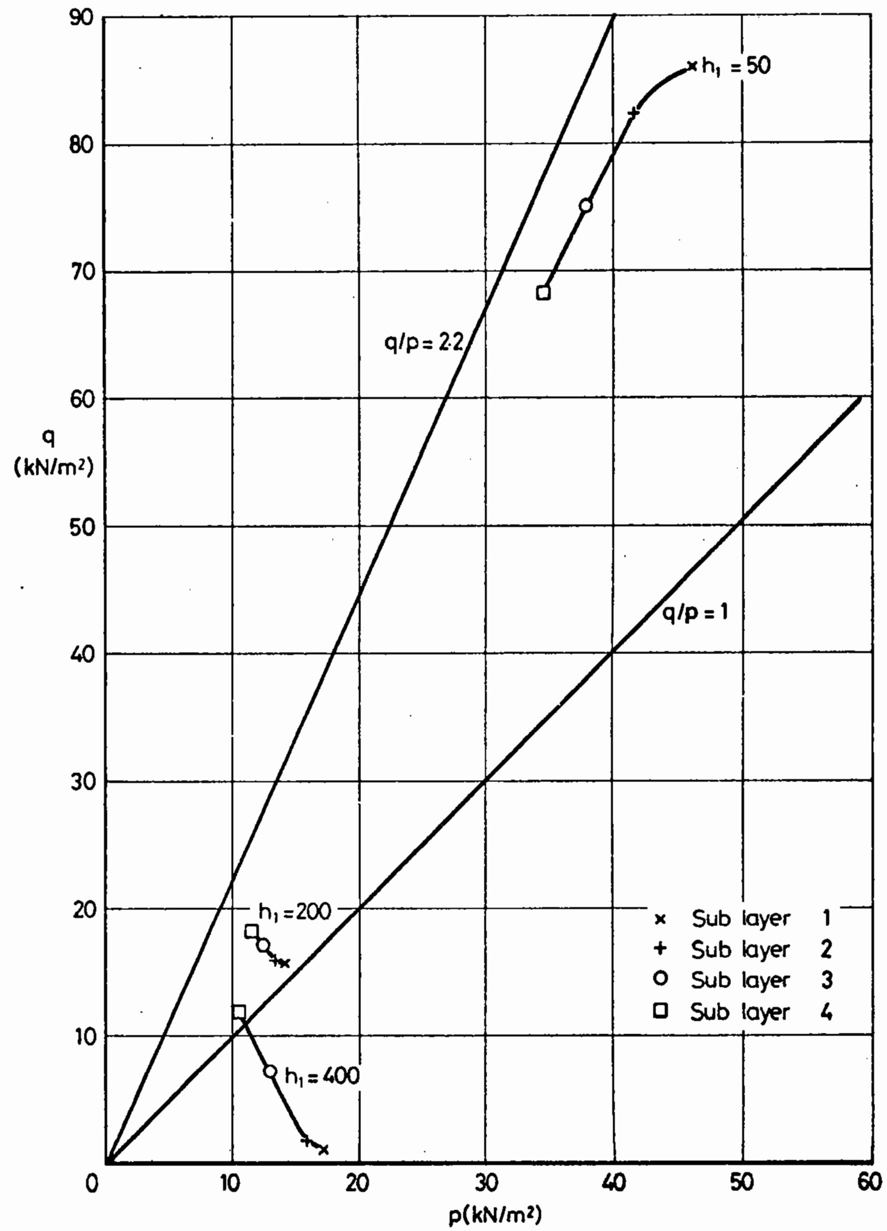


FIG. 5.4 STRESS DISTRIBUTION IN A 200 mm GRANULAR LAYER AS A FUNCTION OF ASPHALT THICKNESS (h_1)

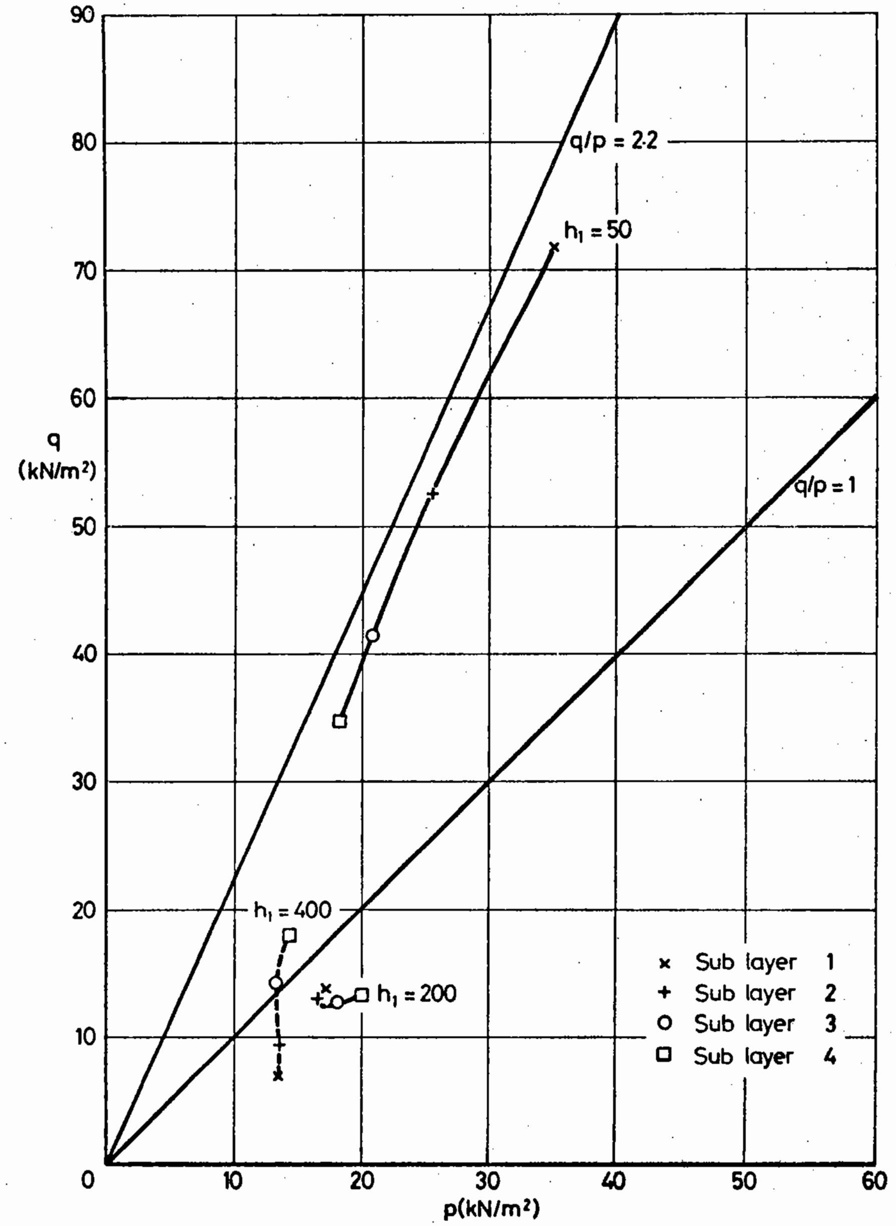


FIG. 5.5 STRESS DISTRIBUTION IN A 700 mm GRANULAR LAYER AS A FUNCTION OF ASPHALT THICKNESS (h_1)

gradient across the layer. A further increase in thickness, to 400 mm, reduces the octahedral shear stress but increases the mean normal stress in all but the bottom sublayer of the granular material. This increase in mean normal stress has the effect of keeping the stresses in most of the granular material clear of the transition zone.

The second structure, containing a 700 mm granular layer (Fig. 5.5) behaves in a generally similar manner. However, there is one important difference, which occurs between asphalt thicknesses of 200 and 400 mm. At 200 mm of surfacing the granular layer is clear of the transition zone, but increasing the surfacing to 400 mm reduces the mean normal stress whilst increasing the octahedral shear stress, thus placing the lower half of the layer inside the failure zone. This suggests that a thick layer of asphalt combined with a thick granular layer generates a more severe stress condition for the granular layer than when a thick layer is combined with an intermediate thickness of asphalt.

5.2.3 The effect of varying the granular layer thickness

Three granular layer thicknesses, 200, 450 and 700 mm, have been studied under 50 and 400 mm of asphalt. The results are plotted in Figs 5.6 and 5.7.

In comparison with the effect of asphalt thickness, the effect of unbound layer thickness is small. For the structure with 50 mm of asphalt the stresses decrease as the layer thickness increases, but they always stay close to the q/p = 2.2 line.

When the granular layer is surfaced with 400 mm asphalt, the octahedral shear stress increases with increasing thickness. The mean normal stress increases in the lower sublayers but decreases in the upper sublayers as the thickness of the layer increases.

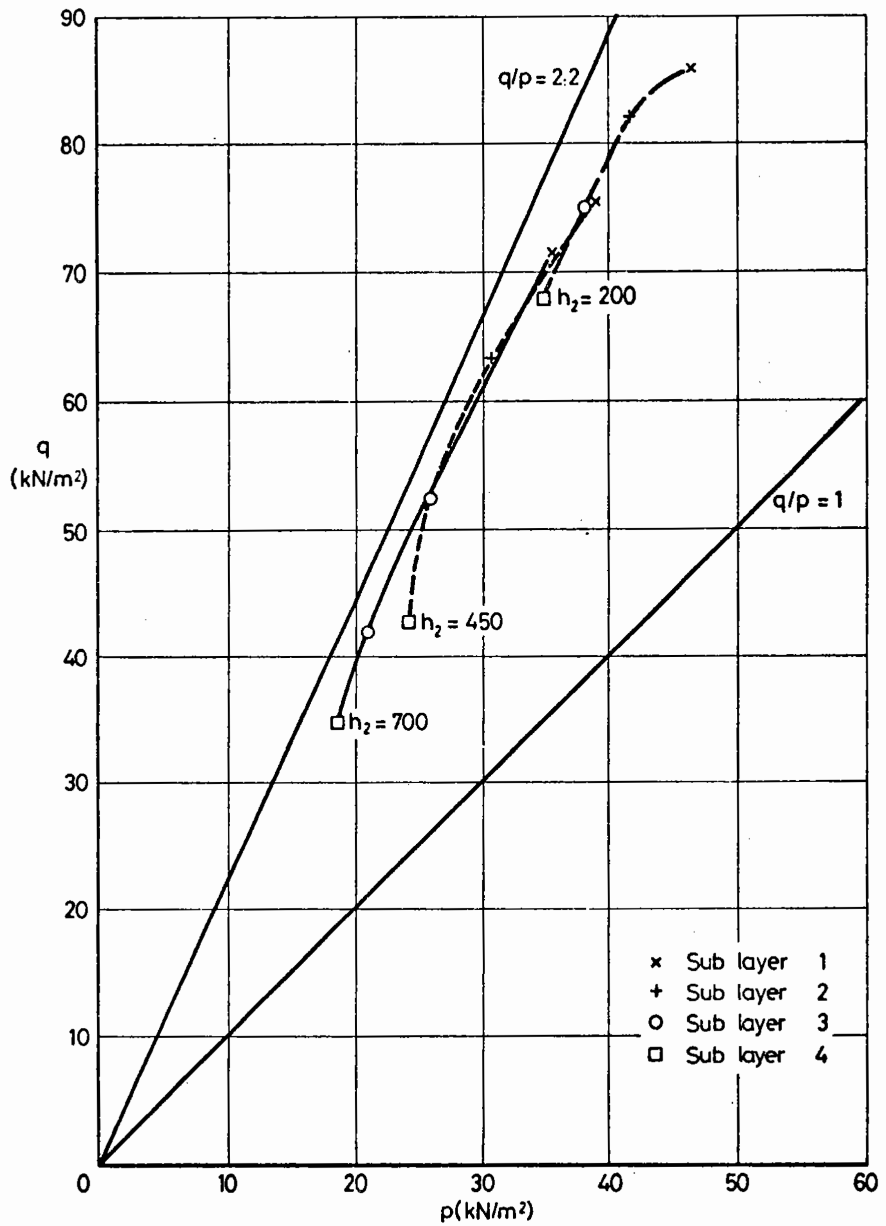


FIG. 5.6 STRESS DISTRIBUTION IN GRANULAR LAYERS OF VARYING THICKNESSES (h_2) FOR AN ASPHALT SURFACING 50 mm THICK

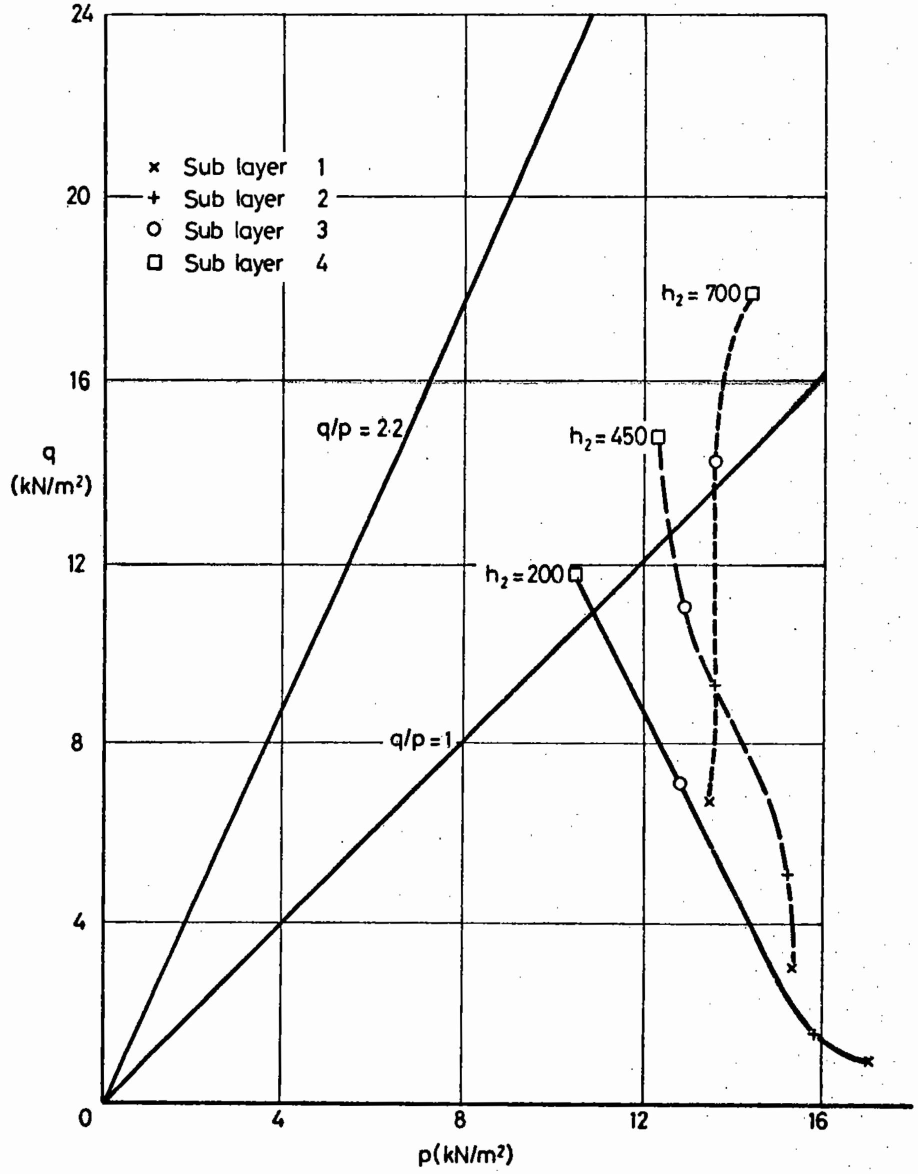


FIG. 5.7 STRESS DISTRIBUTION IN GRANULAR LAYERS OF VARYING THICKNESSES (h_2) FOR AN ASPHALT SURFACING 400 mm THICK

5.2.4 The effect of varying the asphalt stiffness

Two sets of structures were analysed for this study, one being 50 mm of asphalt on 200 mm of granular material, the other being 400 mm of asphalt on 700 mm of granular material. Both sets are on a subgrade with a modulus of 50 MN/m^2.

For the thin structure the stresses decrease with increasing stiffness (Fig. 5.8) and also move away from the limiting q/p = 2.2 condition though stresses in all systems are near the limit. However, for the thick structure (Fig. 5.9) p remains largely constant while q decreases with increasing modulus, thus ensuring a less severe stress distribution.

5.3 VARIATION IN DERIVED MODULUS

5.3.1 The effect of K_1 and subgrade modulus

Figs 5.10, 5.11 and 5.12 show the variation in derived moduli for the granular sub-layers as functions of K_1 and subgrade modulus (E_s). In these graphs the sub-layers have been labelled from 2 to 5 with No. 2 being nearest the surface of the structure.

For all conditions the derived modulus increases with increasing K_1. The effect of the failure criterion can be seen from the slopes of these plots. In Fig. 5.10, E_s = 30 MN/m^2, the increase in derived modulus with K_1 is small compared with that in Fig. 5.12, E_s = 30 MN/m^2. Reference to Figs 5.1 and 5.3 which plot the stress distributions for these structures indicates that in the first case, the stresses are near to failure, while in the second case, they are largely clear of the failure zone. Thus, if the stress conditions within a pavement are well away from failure, then as K_1 increases, the derived moduli will increase rapidly and approximately linearly. However, as failure is approached, with q/p reaching the limiting value of 2.2, the derived modulus - K_1

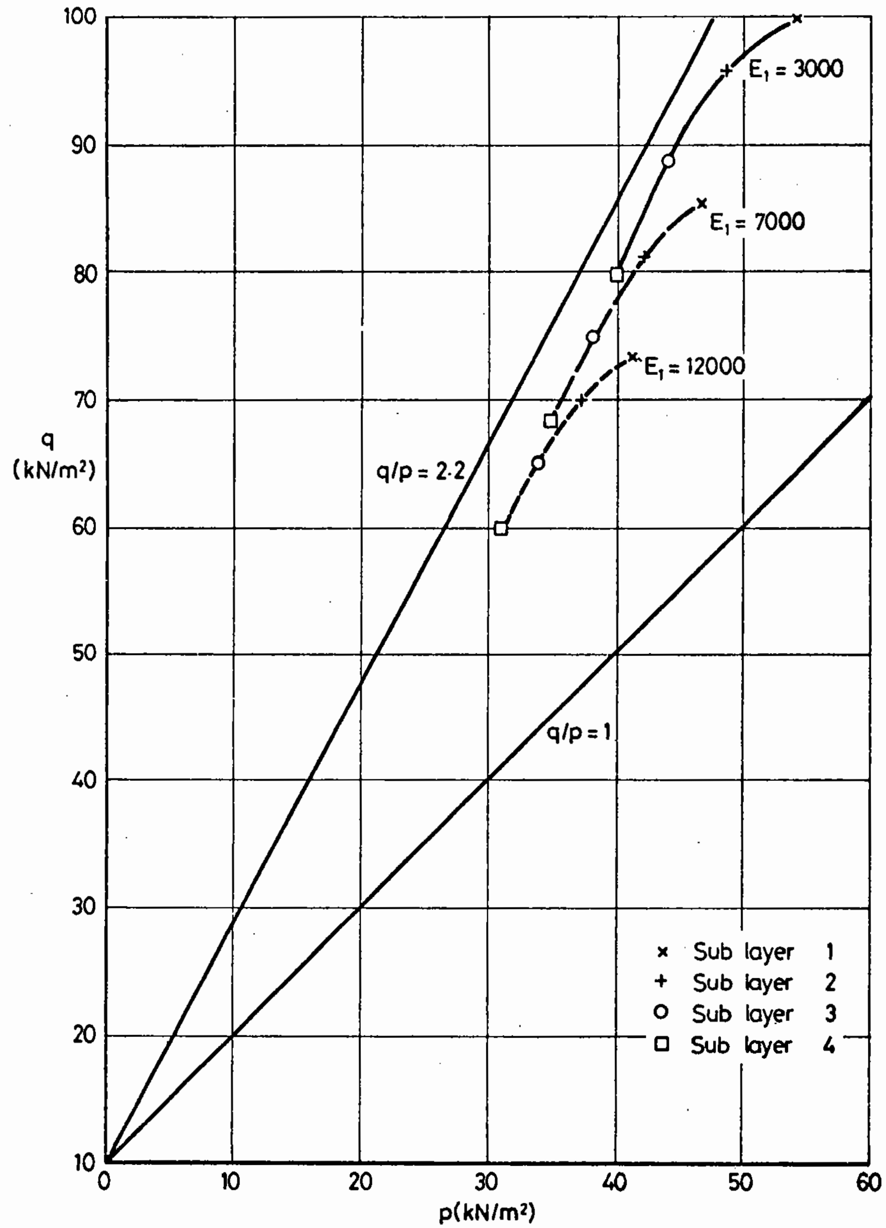


FIG. 5.8 STRESS DISTRIBUTION IN THE GRANULAR LAYER FOR THE THIN STRUCTURE WITH VARIOUS SURFACING STIFFNESSES (E_1)

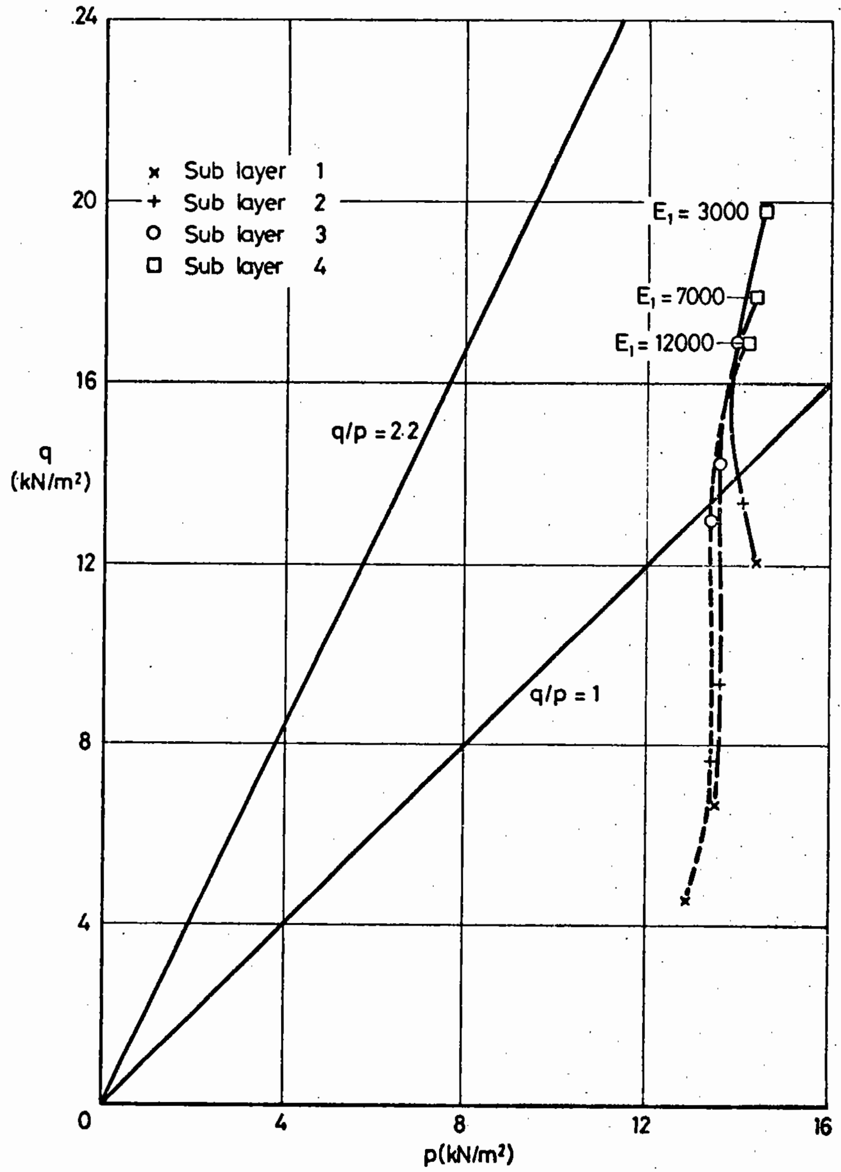


FIG. 5.9 STRESS DISTRIBUTION IN THE GRANULAR LAYER FOR THE THICK STRUCTURE FOR VARIOUS SURFACING STIFFNESSES (E_1)

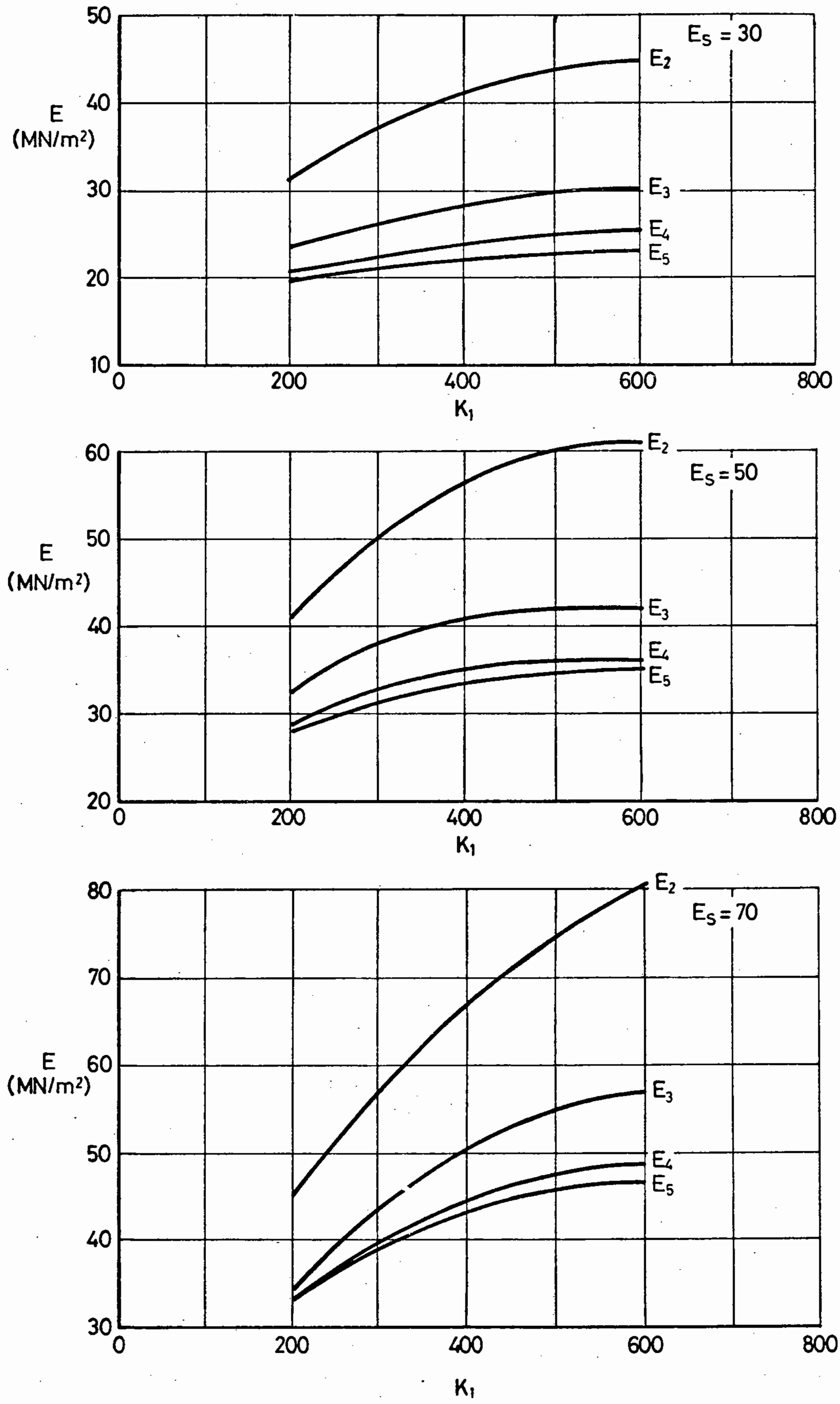


FIG. 5.10 DERIVED MODULI AS A FUNCTION OF K_1 AND SUBGRADE MODULUS FOR STRUCTURE 1

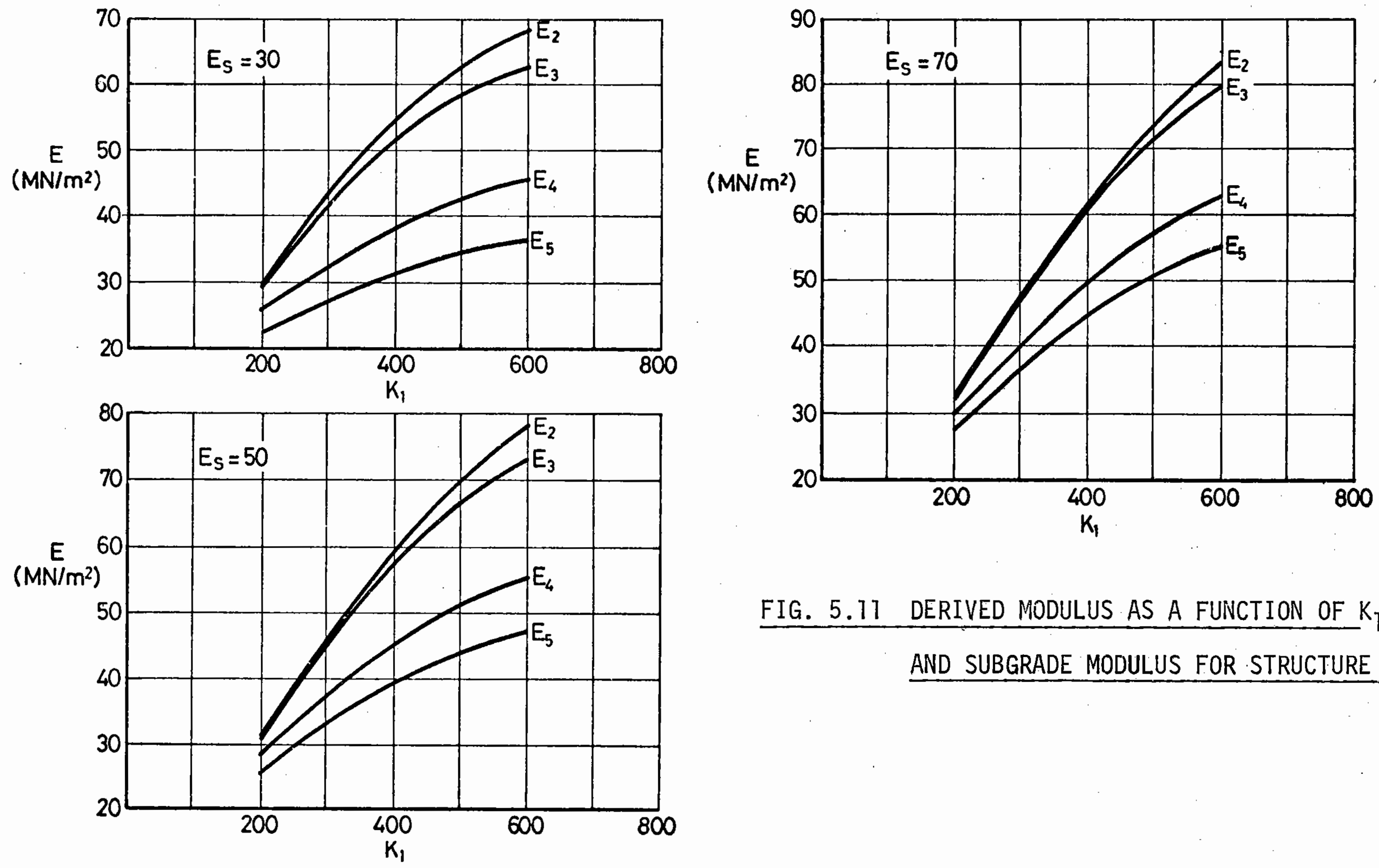


FIG. 5.11 DERIVED MODULUS AS A FUNCTION OF K_1 AND SUBGRADE MODULUS FOR STRUCTURE 2

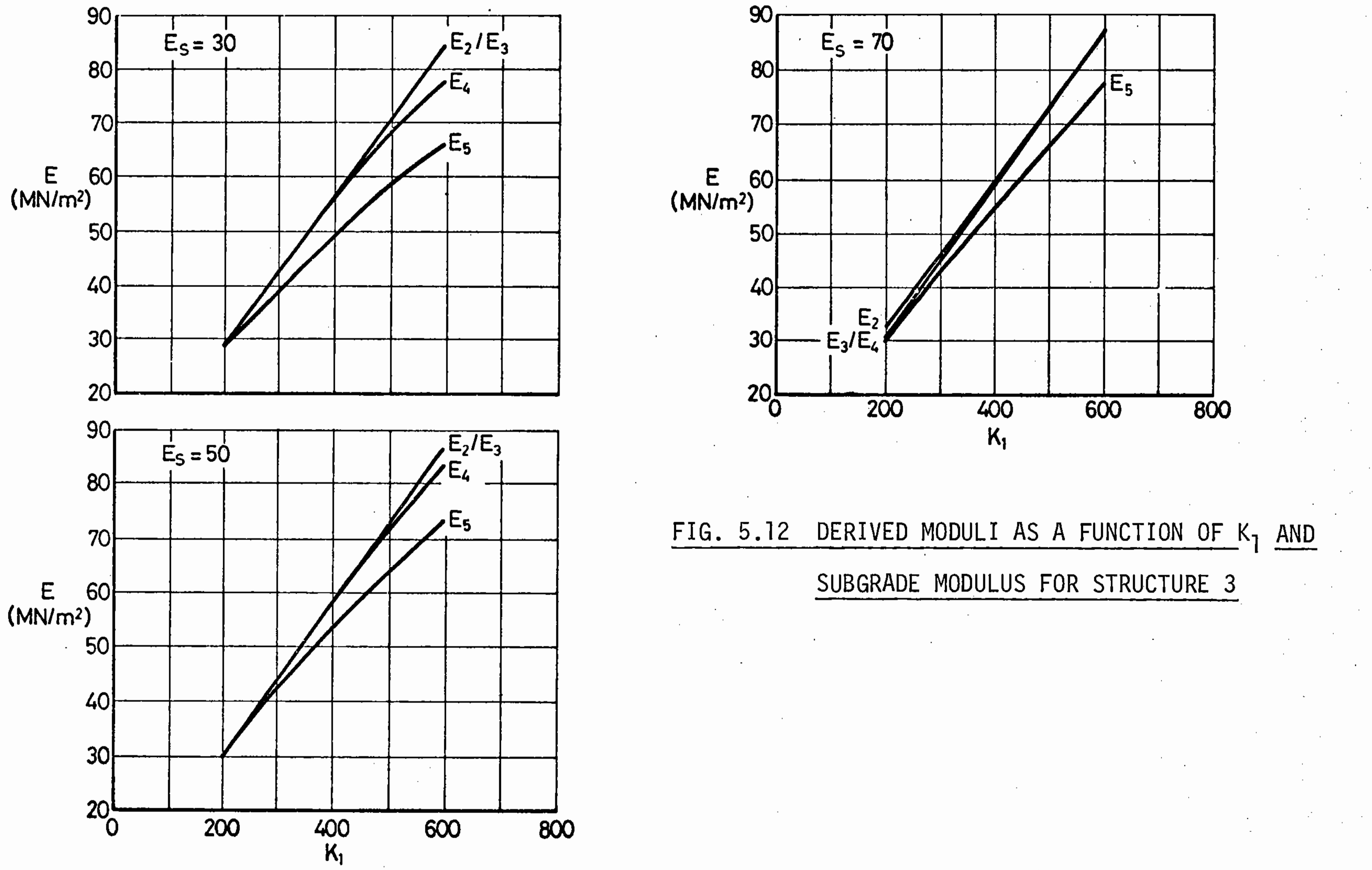


FIG. 5.12 DERIVED MODULI AS A FUNCTION OF K_1 AND SUBGRADE MODULUS FOR STRUCTURE 3

plots have reduced gradients, and become curved apparently tending to a limiting maximum value.

It is noticeable that for $K_1 = 200$, with only one minor exception, all sublayers in all structures have moduli values less than the subgrade. The phenomena of derived moduli being low in relation to the subgrade modulus can also be observed for $K_1 = 400$ and $K_1 = 600$, but the number of cases in which this is observed decreases as K_1 increases, and also as structures become thicker. It is implicit in the characterisation that as the stress condition in a granular layer approaches failure, its modulus will decrease to less than that of the subgrade. This analysis has shown that for a wide range of pavements, and particularly if low quality granular materials are used (as implied by $K_1 = 200$), then low granular moduli will result. This indicates that for these conditions the granular material contributes very little to the load spreading ability of the pavement.

Generally, for the structures investigated, the derived moduli increase as E_s increases. This increase is greatest at $K_1 = 600$ and becomes less significant as the structure becomes thicker. It is also noticeable for the thickest structure (Fig. 5.12) that the moduli for the various sublayers are similar. This is the result of the phenomenon observed in Fig. 5.3, that, for all conditions, p is effectively constant.

5.3.2 The effect of asphalt thickness

Figs 5.13 and 5.14 show the effect of variations in derived modulus as a function of the asphalt thickness. For both structures the derived moduli increase as the asphalt layer thickness increases, because the thicker layers create a less severe stress condition within the granular material. This may appear surprising since the thicker

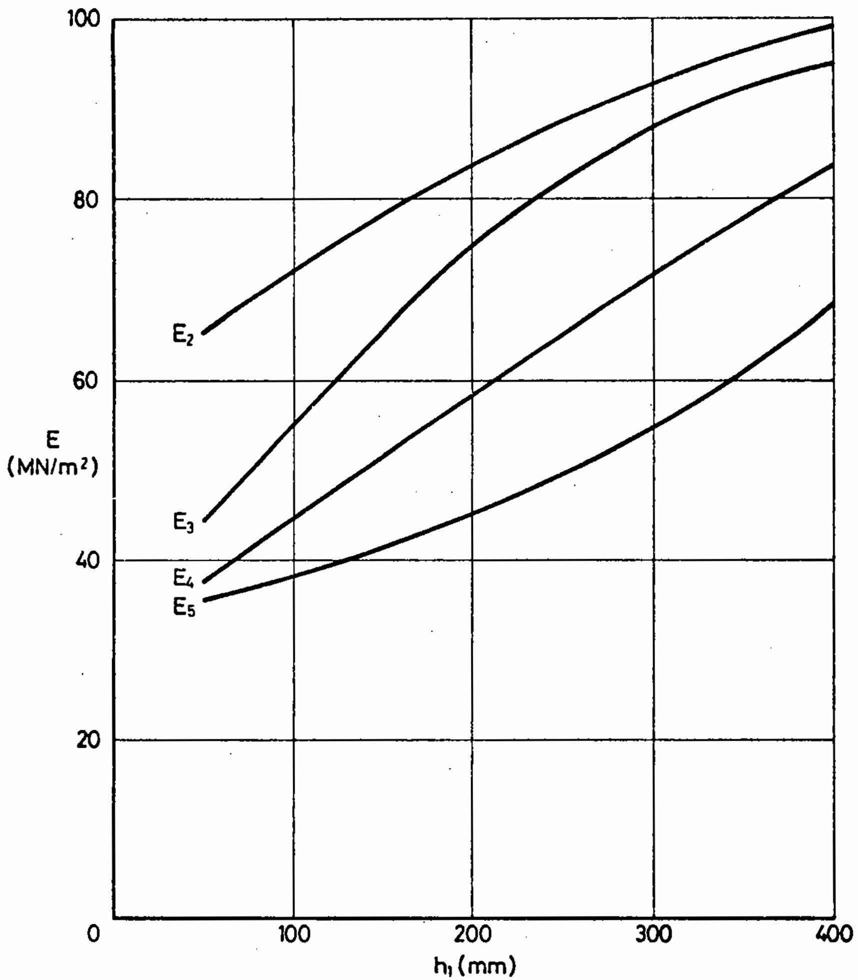


FIG. 5.13 VARIATION IN DERIVED MODULUS OF A 200 mm THICK GRANULAR LAYER AS A FUNCTION OF ASPHALT THICKNESS (h_1)

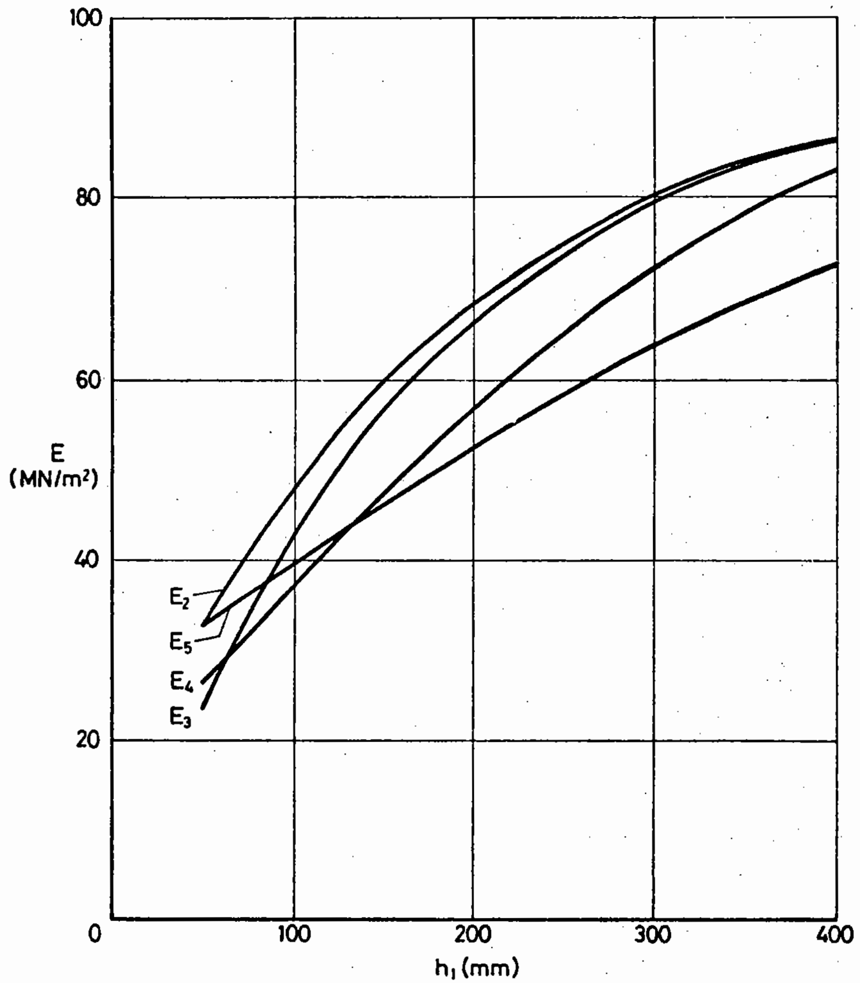


FIG. 5.14 VARIATION IN DERIVED MODULUS OF A 700 mm THICK GRANULAR LAYER AS A FUNCTION OF ASPHALT THICKNESS (h_1)

layers reduce the stress within the granular material and thus would be expected to reduce the derived modulus. However, the thicker asphalt layer causes a less severe stress condition, the q/p ratio reducing as asphalt layer thickness increases. Thus, whilst the modulus calculated from Equation 4.13 is reduced through decreasing stress, the reduction to this modulus, due to the effect of the failure criterion, decreases. Thus, for these structures it appears that the reduction in q/p ratio with increasing asphalt thickness is more significant with regard to the modulus derivation than the reduced stress in the granular material.

It will be noticed in Fig. 5.14 that for an asphalt thickness of 50 mm the sub-layer moduli do not decrease successively with depth as is more usual. Reference to Fig. 5.5 will indicate that this structure is very severely stressed with high q/p ratios, and that the upper and lower sublayers are further from the failure zone than the two central sublayers. Hence, the unusual variation in modulus with depth.

5.3.3 The effect of varying the granular layer thickness

The effect of different granular layer thicknesses changes significantly for different surfacing thicknesses, as shown by Figs 5.15 and 5.16. For thin surfacings (Fig. 5.15) the trend is for moduli to decrease with increasing granular layer thicknesses until the phenomenon of increased moduli in the centre sublayers, as discussed above, is encountered for the thick layer. This is due to the fact that the stress system becomes more severe as granular layer thickness increases, causing a drop in derived modulus despite the increasing mean normal stress.

For a thick surfacing the modulus of the upper two sublayers decreases as the layer thickness increases. Also for the thickest layer considered (h_2 = 700 mm) the top two sublayers have the same

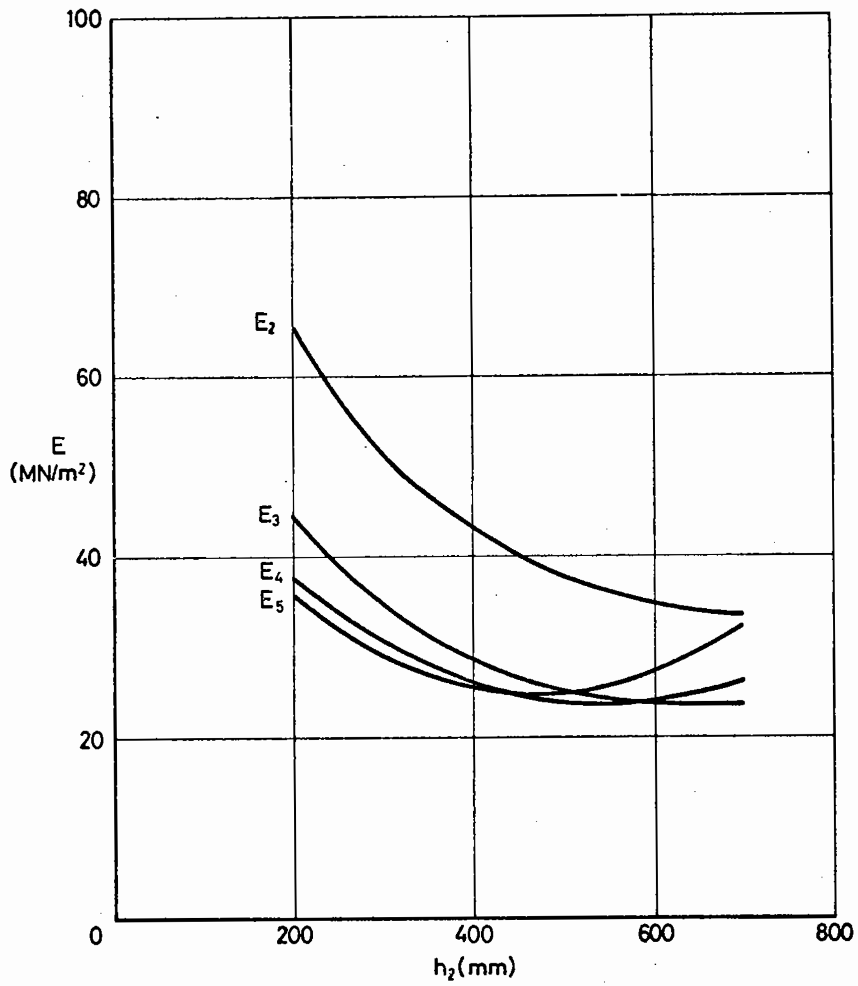


FIG. 5.15 DERIVED MODULI OF A GRANULAR LAYER OF VARYING THICKNESS (h_2) WITH A 50 mm ASPHALT SURFACING

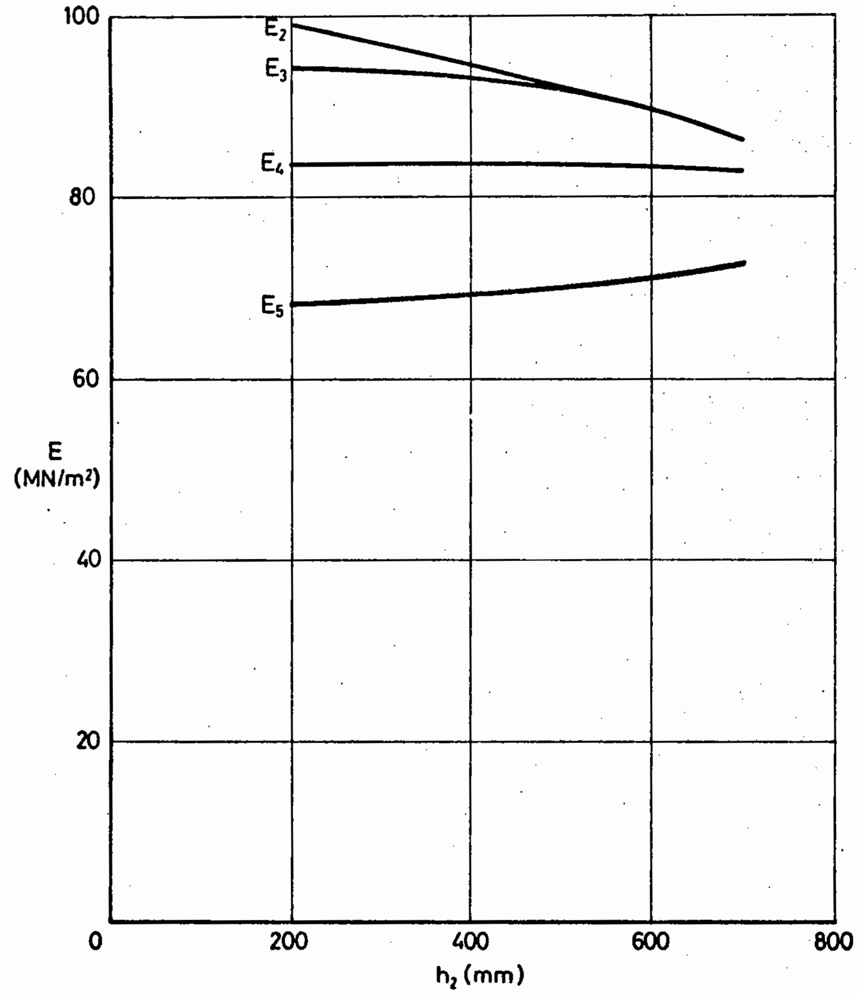


FIG. 5.16 DERIVED MODULI IN A GRANULAR LAYER OF VARYING THICKNESS (h_2) WITH A 400 mm ASPHALT SURFACING

modulus. In contrast, the modulus of the bottom sublayer increases as the thickness increases.

Reference to Fig. 5.7 indicates that as the granular layer thickness increases, the mean normal stress tends to become constant with depth. This implies that as the thickness of the structure increases, there will be a tendency for the derived moduli of the granular sublayers to be equal, provided that the stresses in the layer are not within the failure zone.

5.3.4 The effect of varying the modulus of the asphalt

Figs 5.17 and 5.18 show that varying the modulus of the asphalt does not have a great effect on the derived moduli of the granular sub-layers. For the thin structure (Fig. 5.17) the modulus increases slowly at all levels. In the thick structure a similar trend to that observed in Fig. 5.16 is evident. The moduli of the upper two layers decrease while that ofthe lower two increase, giving the appearance of an asymptotic approach to a common value.

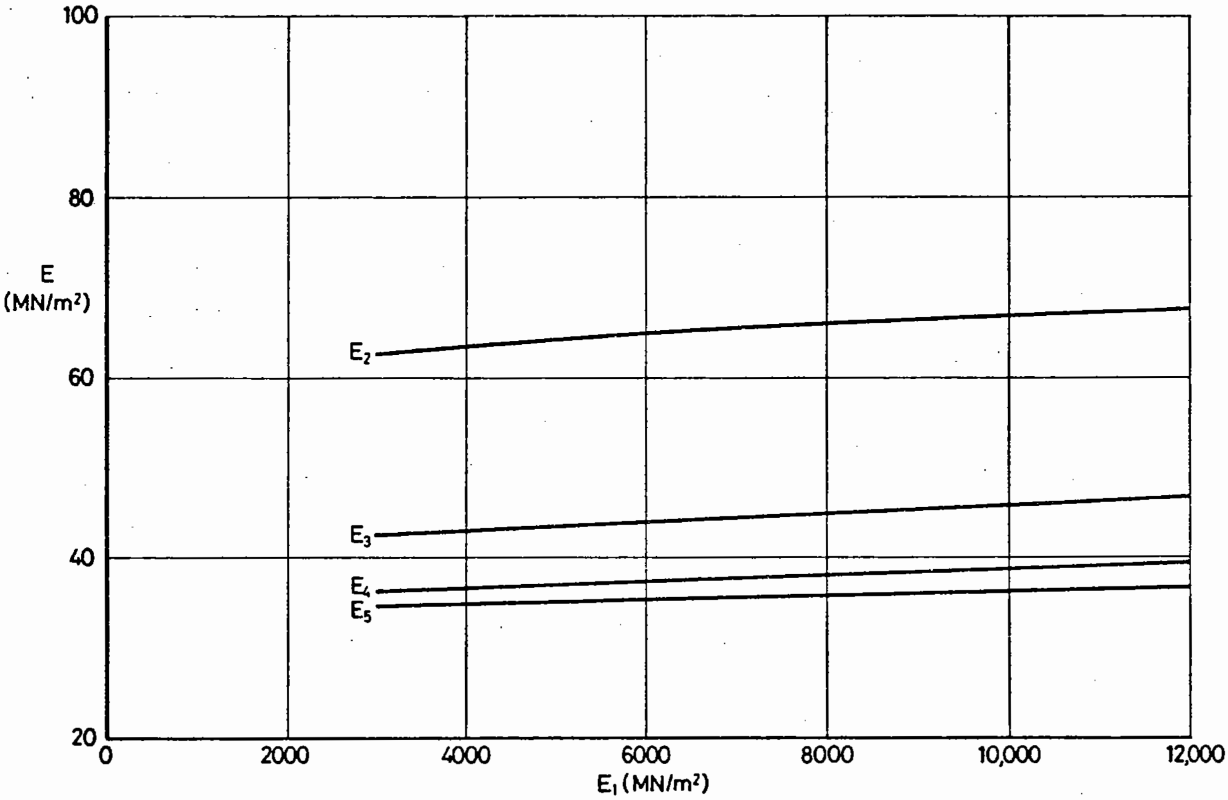


FIG. 5.17 DERIVED MODULI AS A FUNCTION OF SURFACING STIFFNESS (E_1) FOR THE THIN STRUCTURE

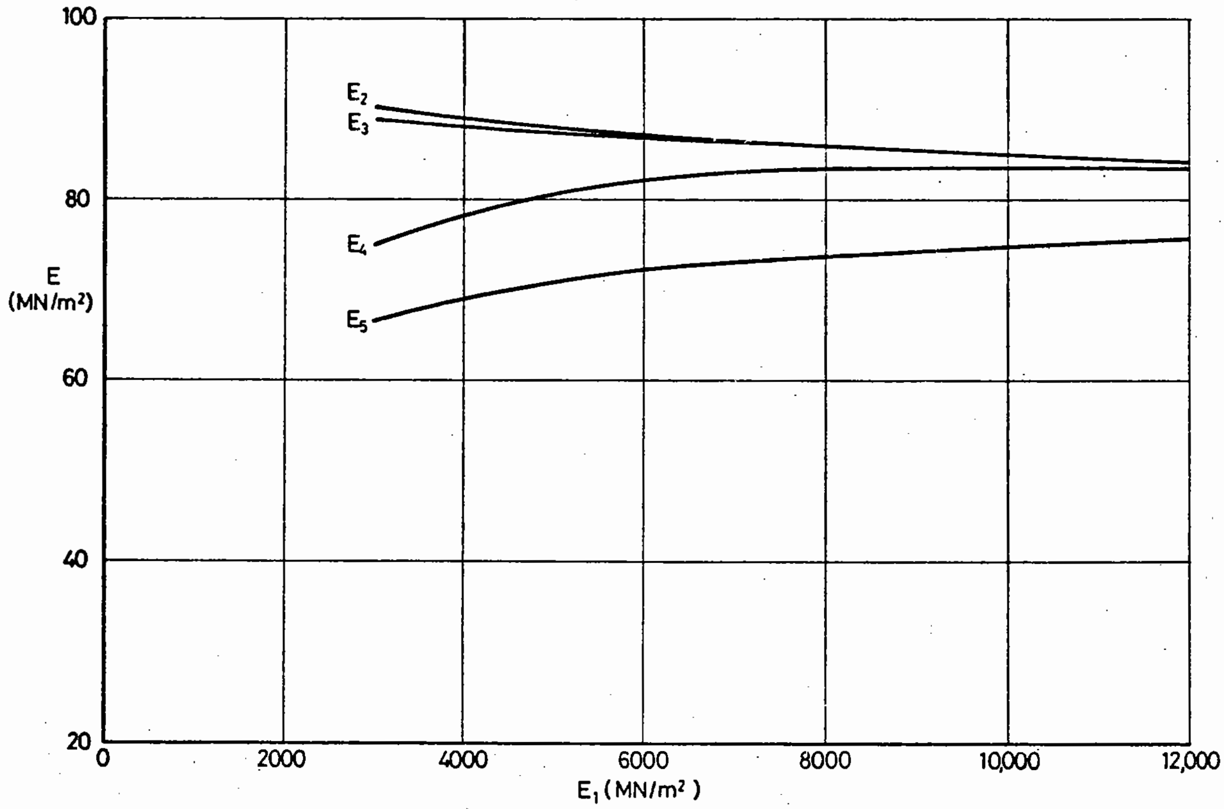


FIG. 5.18 DERIVED MODULUS AS A FUNCTION OF SURFACING STIFFNESS (E_1) FOR FOR THE THICK STRUCTURE

CHAPTER SIX

COMPARISON OF PRIMARY RESPONSE PARAMETERS USING LINEAR AND NON-LINEAR CHARACTERISATION OF THE UNBOUND LAYER

6.1 INTRODUCTION

The work reported in this chapter completes the analytical studies relating to the non-linear characterisation of unbound granular materials.

The object of this study was to assess the need for non-linear characterisation of unbound layers in a pavement design program, by a comparison between primary response parameters derived from linear and non-linear analyses. Two parameters were chosen which are commonly used for the assessment of pavement performance, namely tensile strain in the asphalt and vertical strain on the subgrade.

The analysis of the structures discussed in Chapter 5 was extended to include these two parameters, and for convenience the results will be discussed under the same headings.

6.2 THE EFFECTS OF VARYING K_1 AND SUBGRADE MODULUS

Figs 6.1, 6.2 and 6.3 plot primary response parameters as functions of K_1 and E_s for the structures detailed in Table 5.1.

Considering the thinnest structure (Fig. 6.1) the effect on asphalt strain can be seen to be highly significant, the non-linear analysis giving significantly higher strains than the linear analysis. It is also noticeable that as K_1 increases, the strain calculated by the non-linear system decreases. This is as would be expected since the moduli derived for the unbound sublayers of these structures increase with K_1. Subgrade strain is not greatly changed by using non-linear

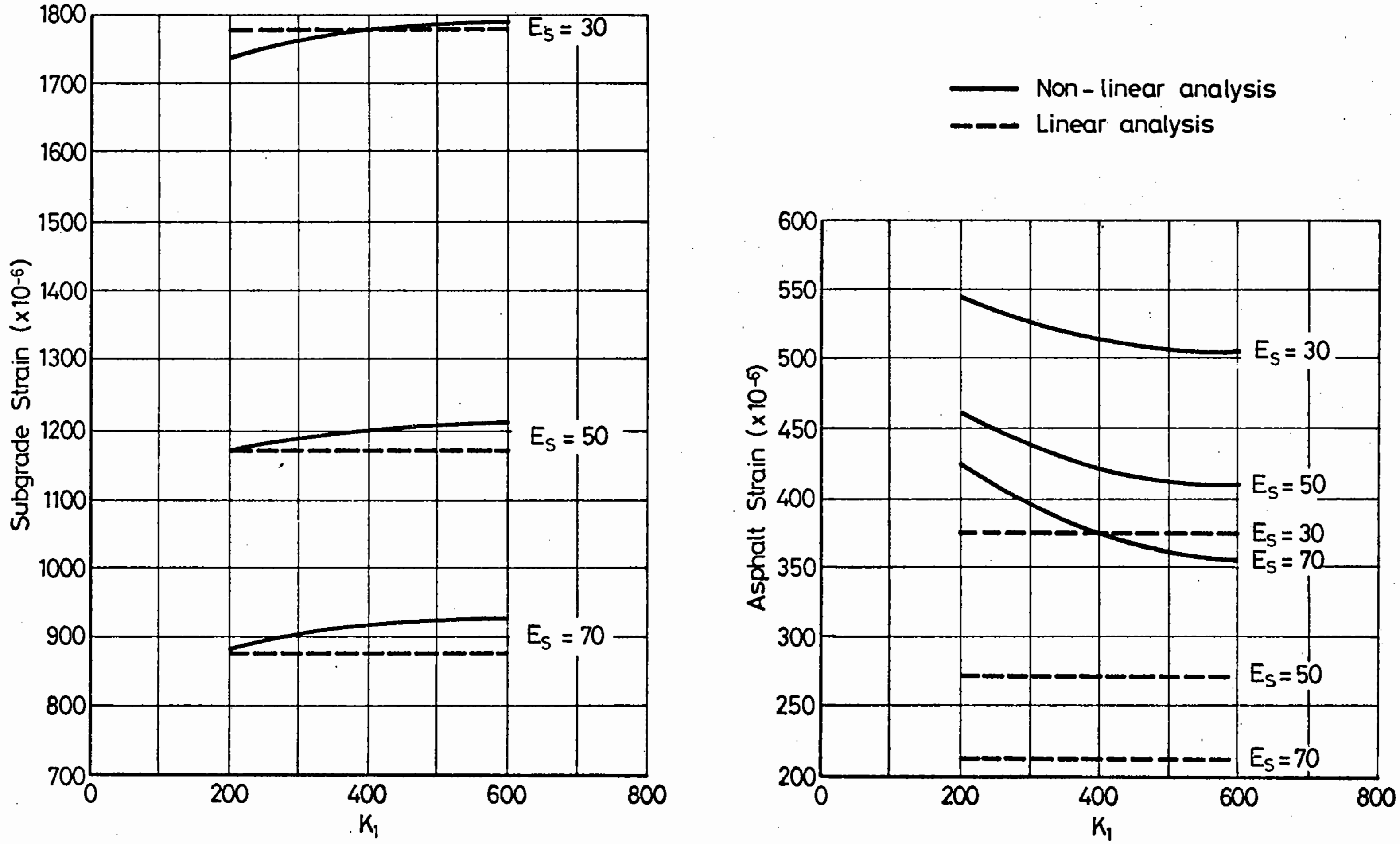


FIG. 6.1 COMPARISON OF PRIMARY RESPONSE PARAMETERS FOR LINEAR AND NON-LINEAR ANALYSIS FOR STRUCTURE 1, AS A FUNCTION OF K_1

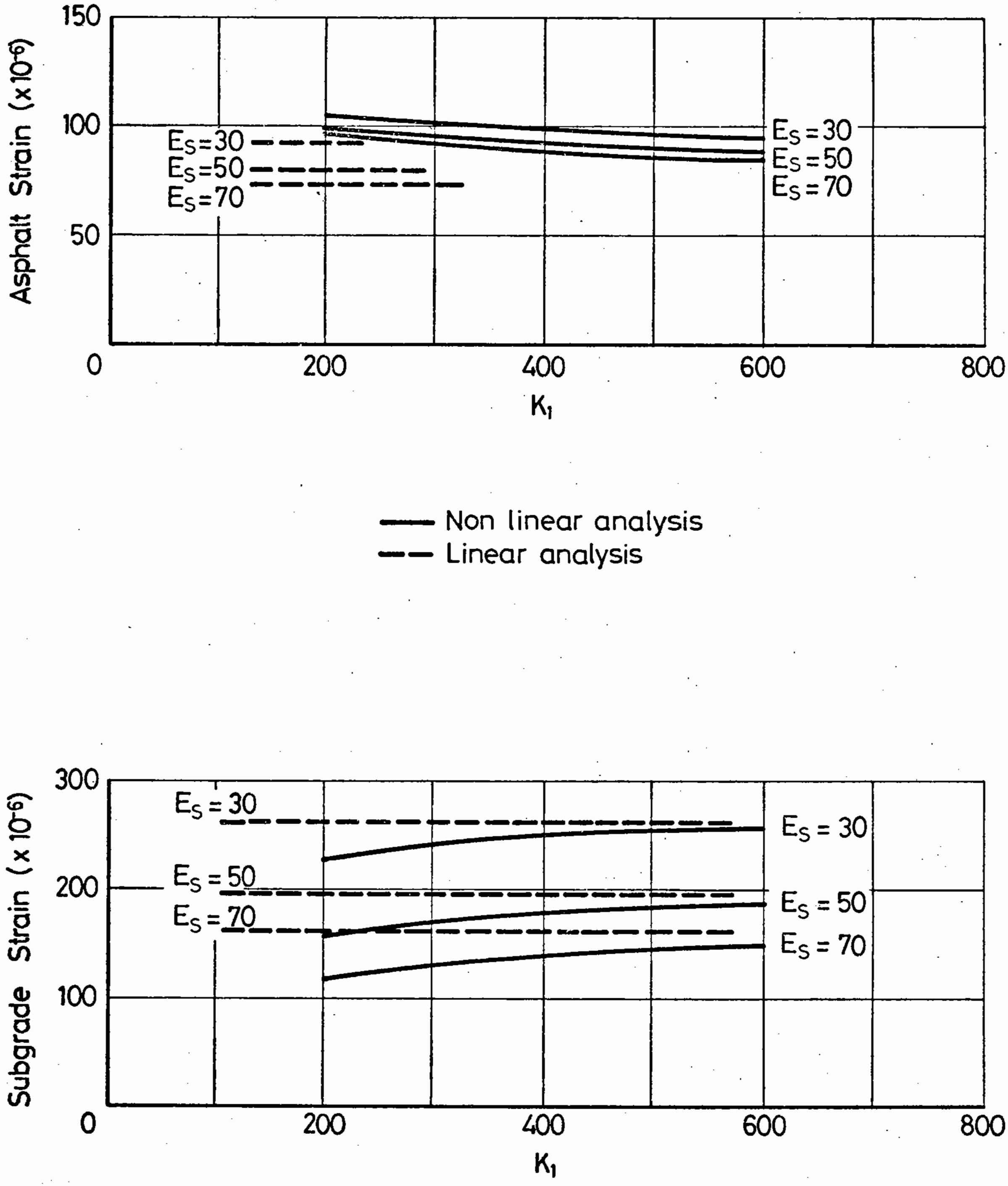


FIG. 6.2 COMPARISON OF PRIMARY RESPONSE PARAMETERS FOR LINEAR AND NON-LINEAR ANALYSIS FOR STRUCTURE 2, AS A FUNCTION OF K_1

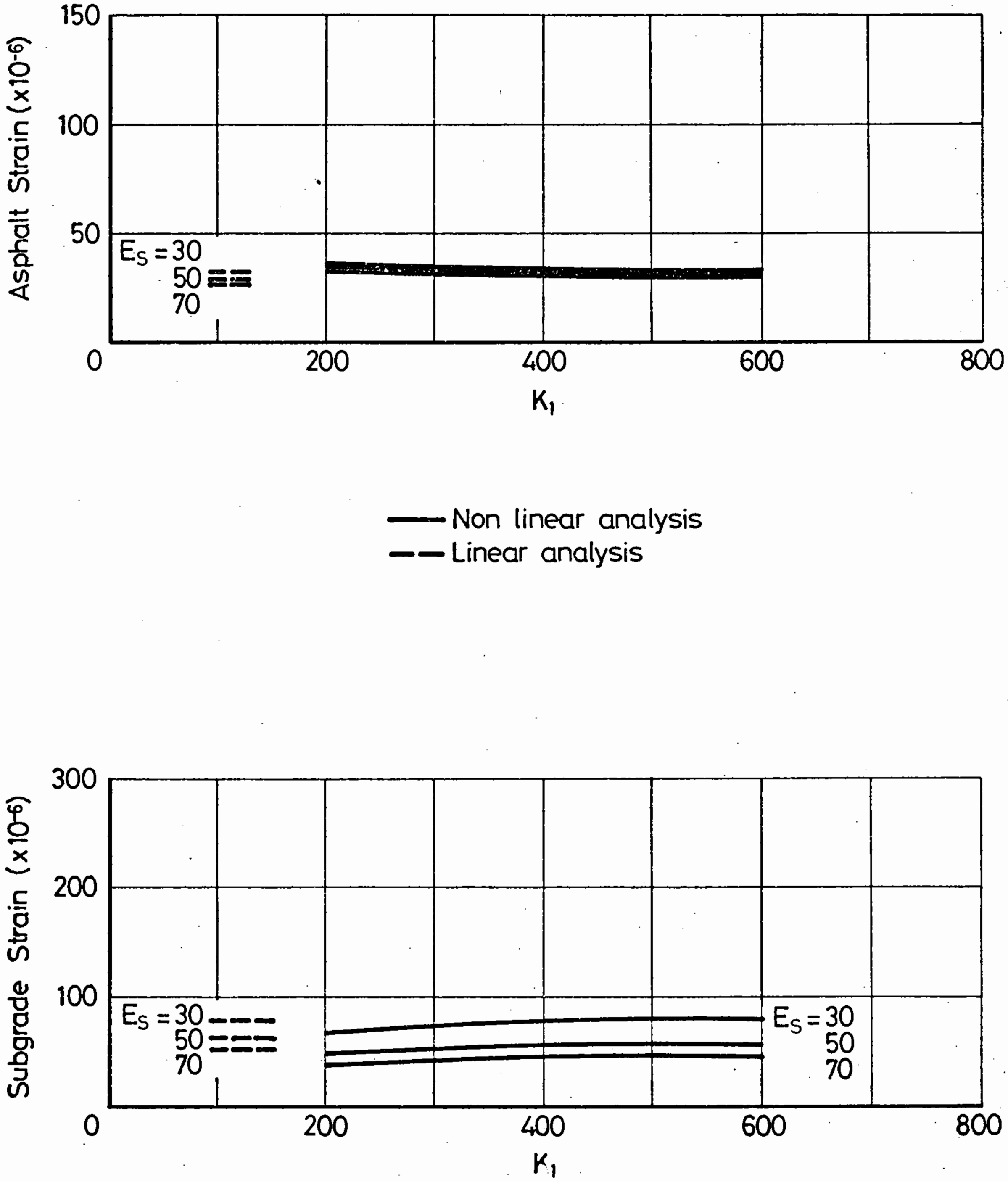


FIG. 6.3 COMPARISON OF PRIMARY RESPONSE PARAMETERS FOR LINEAR AND NON-LINEAR ANALYSIS FOR STRUCTURE 3, AS A FUNCTION OF K_1

analysis, the biggest difference being at $K_1 = 600$ and $E_s = 70$ MN/m^2, there being an increase of 50 microstrain ($\mu\varepsilon$) in 875 $\mu\varepsilon$. However, in contrast to asphalt strain the subgrade strain increases slightly with K_1.

The trends observed in Fig. 6.2 are similar to those in Fig. 6.1, though since the structure is much thicker the strains are much smaller. In addition, the difference between linear and non-linear systems is much reduced, and decreases as K_1 increases. The greatest difference is at $K_1 = 200$, with the asphalt strain being greater and subgrade strain being less in the non-linear system than in the corresponding linear system. Finally, for the very thick structure (Fig. 6.3) non-linear characterisation has virtually no discernable effect. Since, for this structure, variation in subgrade stiffness also has very little effect, it implies that the properties of the asphalt layer are structurally dominant.

6.3 THE EFFECT OF VARYING THE THICKNESS OF THE ASPHALT LAYER

The relationship between response parameter and asphalt thickness for the two structural systems considered is plotted in Figs 6.4 and 6.5.

Considering the thin structure (Fig. 6.4) the effect of the non-linear system on asphalt strain decreases as the asphalt thickness increases, though the strain calculated using linear characterisation is always less than that calculated using non-linear. However, the subgrade strain is hardly affected by the method of analysis.

Similar results are obtained with the thick structure (Fig. 6.5), but the increase in asphalt strain observed when using the non-linear system with thin asphalt layers is much greater with this structure. The decrease in subgrade strain observed for the structure with thin

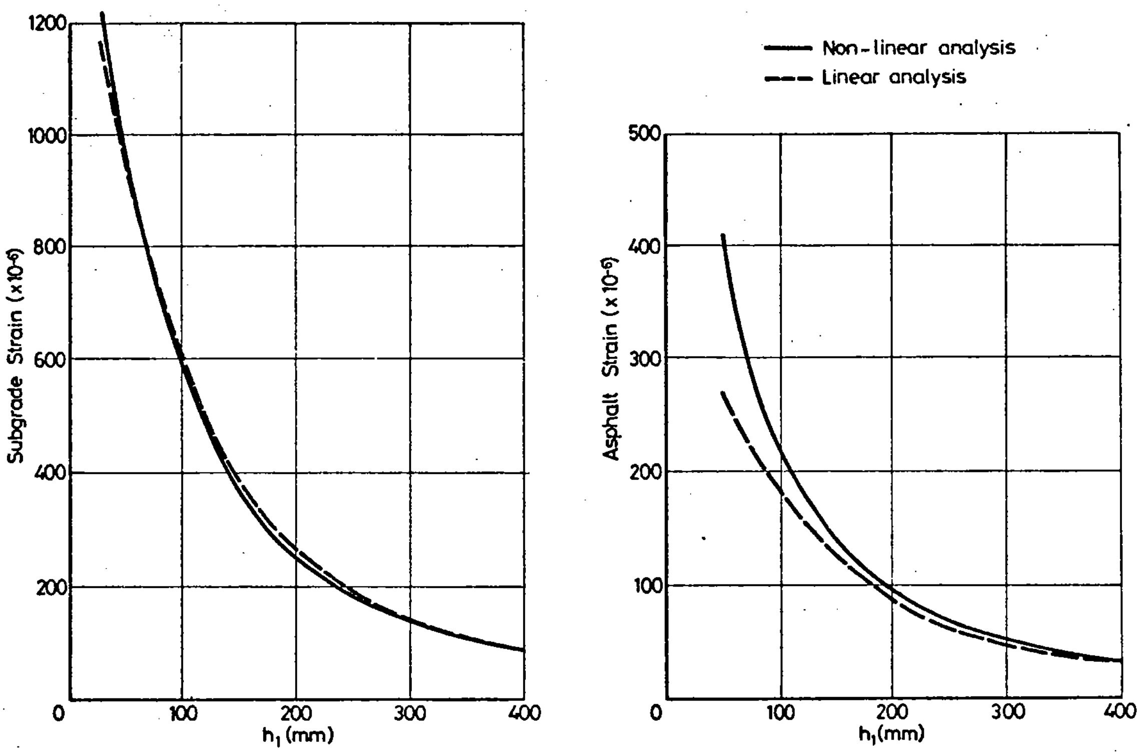


FIG. 6.4 COMPARISON OF PRIMARY RESPONSE PARAMETERS FOR LINEAR AND NON-LINEAR ANALYSIS AS A FUNCTION OF ASPHALT THICKNESS (h_1) FOR A 200 mm GRANULAR LAYER

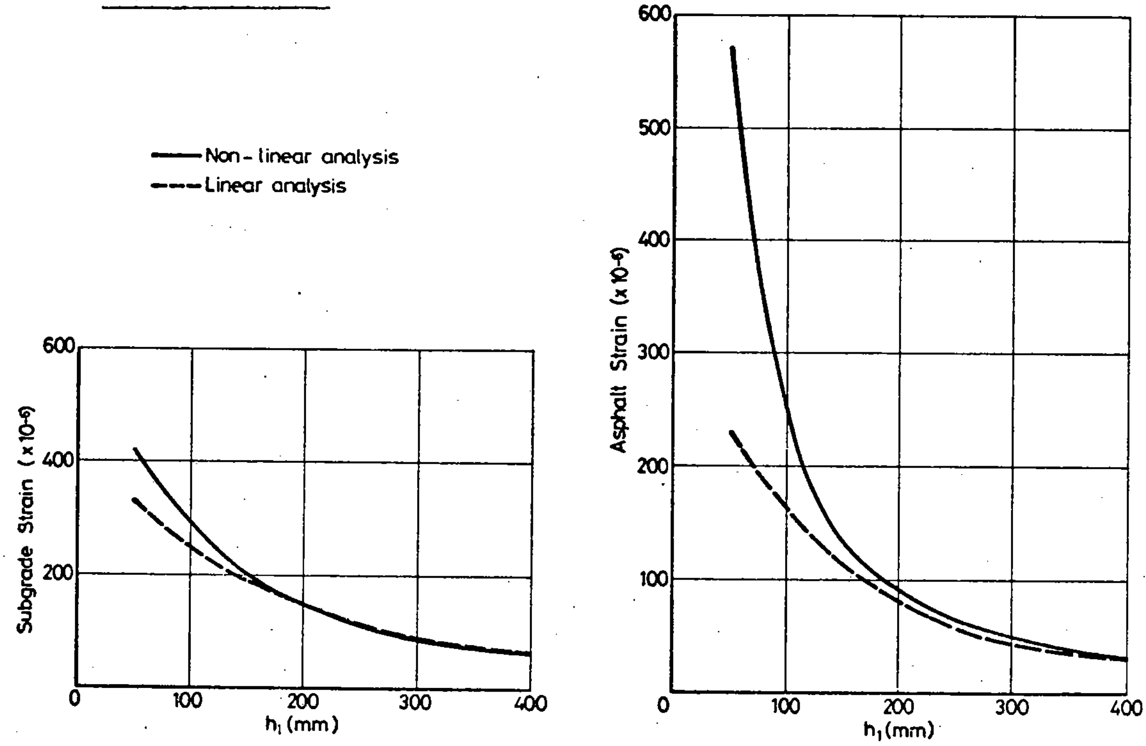


FIG. 6.5 COMPARISON OF PRIMARY RESPONSE PARAMETERS FOR LINEAR AND NON-LINEAR ANALYSIS AS A FUNCTION OF ASPHALT THICKNESS (h_1) FOR A 700 mm GRANULAR LAYER

asphalt layers is accentuated when it is analysed with the non-linear system.

6.4 THE EFFECT OF VARYING THE GRANULAR LAYER THICKNESS

Results for the thin structure (Fig. 6.6) indicate that the analysis method for the granular layer has considerable influence on the asphalt strain, which is significantly larger when the non-linear system is used. In addition, the non-linear system gives an increasing asphalt strain with increasing granular layer thickness whilst the linear system shows the opposite effect. Reference to Fig. 5.15, which shows the derived moduli for the unbound sub-layer decreasing as h_2 increases, will confirm that this trend is reasonable.

The variation of subgrade strain with granular layer thickness follows the same pattern for both analysis systems, but the non-linear one indicates less strain and as the thickness of the granular material increases, the difference between the two systems increases.

In contrast, for the thick structure (Fig. 6.7) there is no discernable difference in asphalt strain derived by either system and only a negligible difference in subgrade strain. This indicates, as observed previously in this structure, that the asphalt layer is structurally dominant.

6.5 THE EFFECT OF VARYING THE ASPHALT STIFFNESS

Figs 6.8 and 6.9 show the comparisons between the analysis systems for the response parameters in the two structures investigated, as functions of the asphalt modulus.

For the thin structure (Fig. 6.8) the effect of the analysis system with regard to asphalt strain is considerable, the non-linear characterisation indicating greatly increased strain over the linear

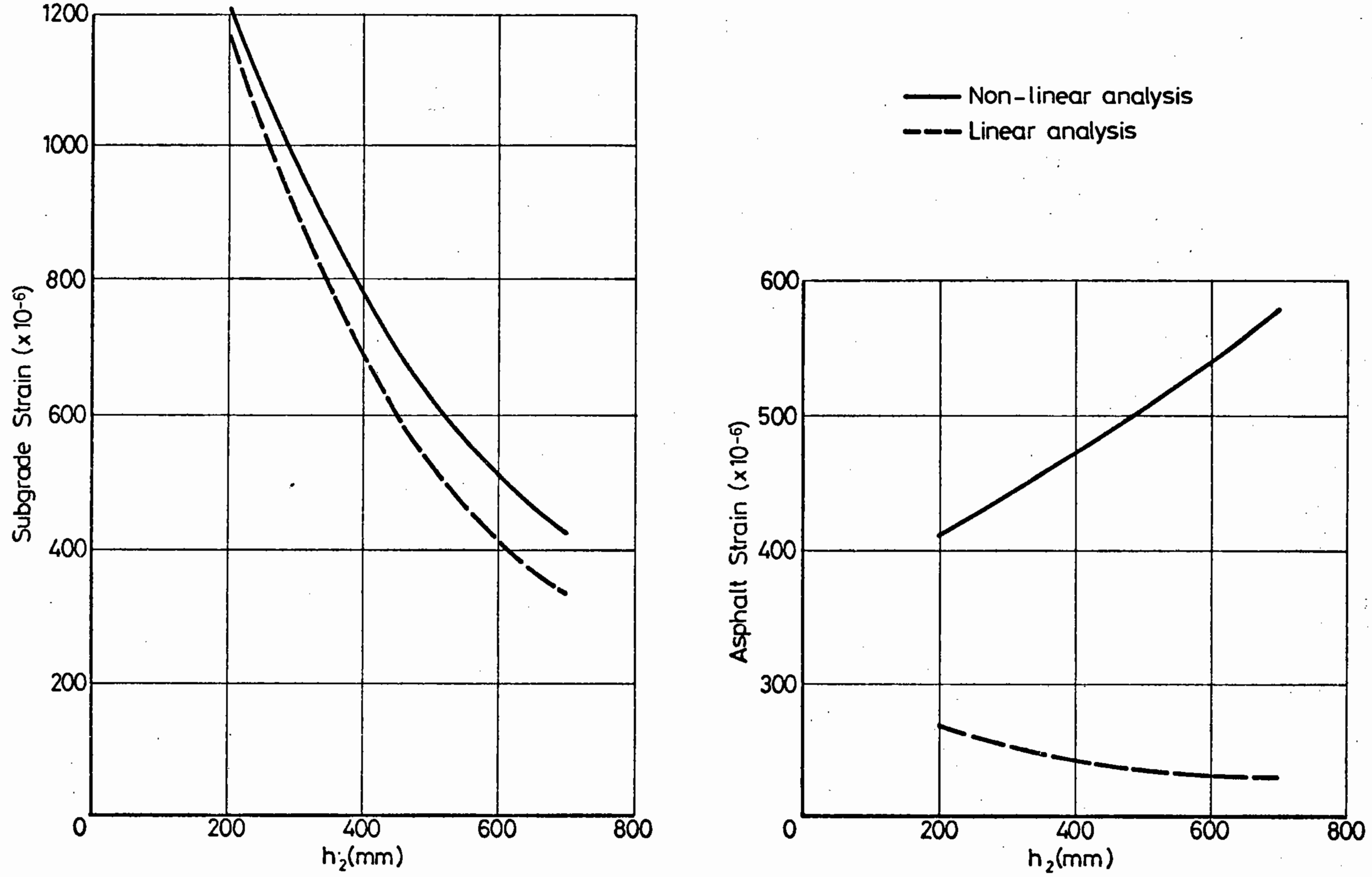


FIG. 6.6 COMPARISON OF PRIMARY RESPONSE PARAMETERS FOR LINEAR AND NON-LINEAR ANALYSIS AS A FUNCTION OF GRANULAR LAYER THICKNESS (h_2) FOR THE THIN STRUCTURE

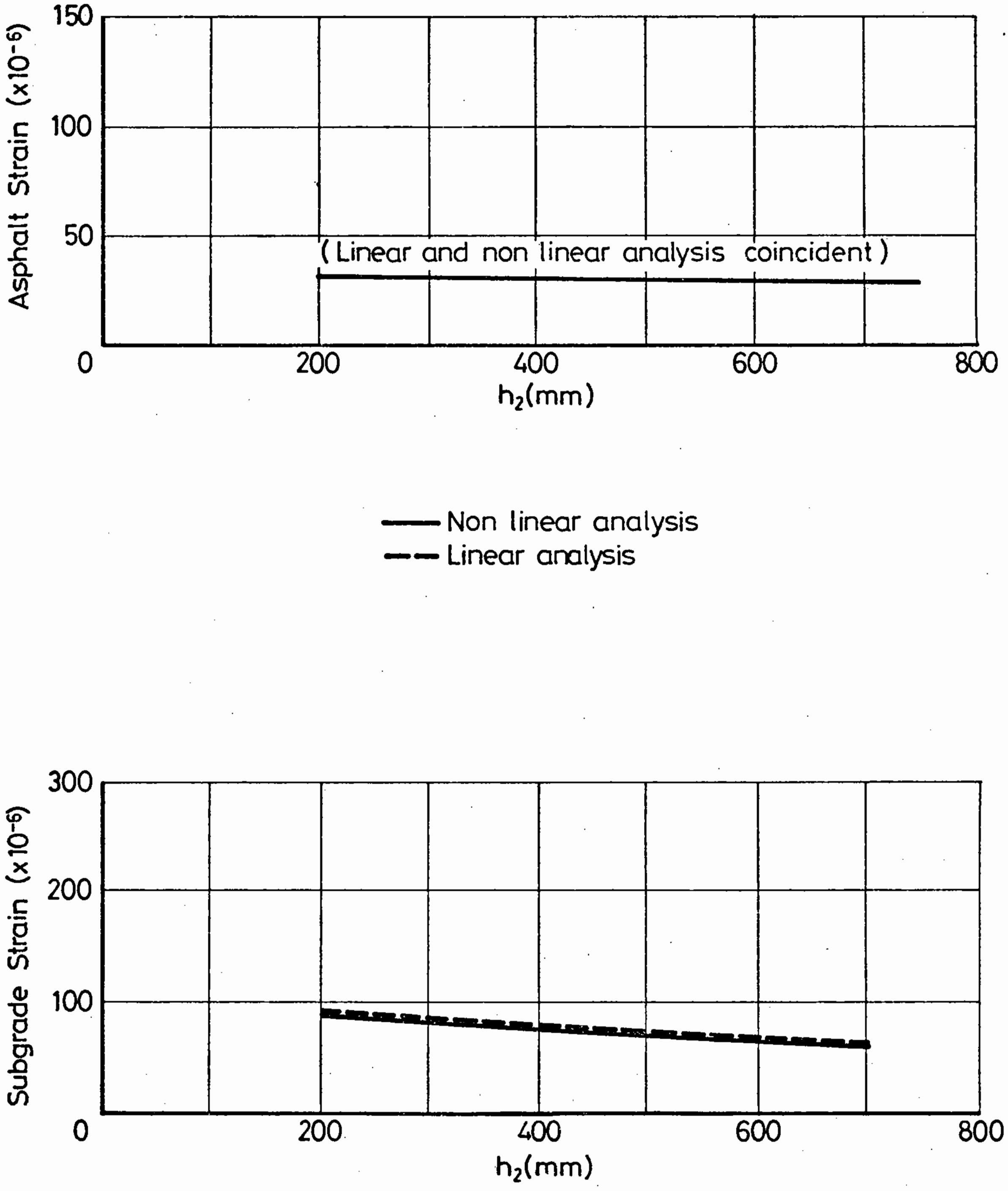


FIG. 6.7 COMPARISON OF PRIMARY RESPONSE PARAMETERS FOR LINEAR AND NON-LINEAR ANALYSIS AS A FUNCTION OF GRANULAR LAYER THICKNESS (h_2) FOR THE THICK STRUCTURE

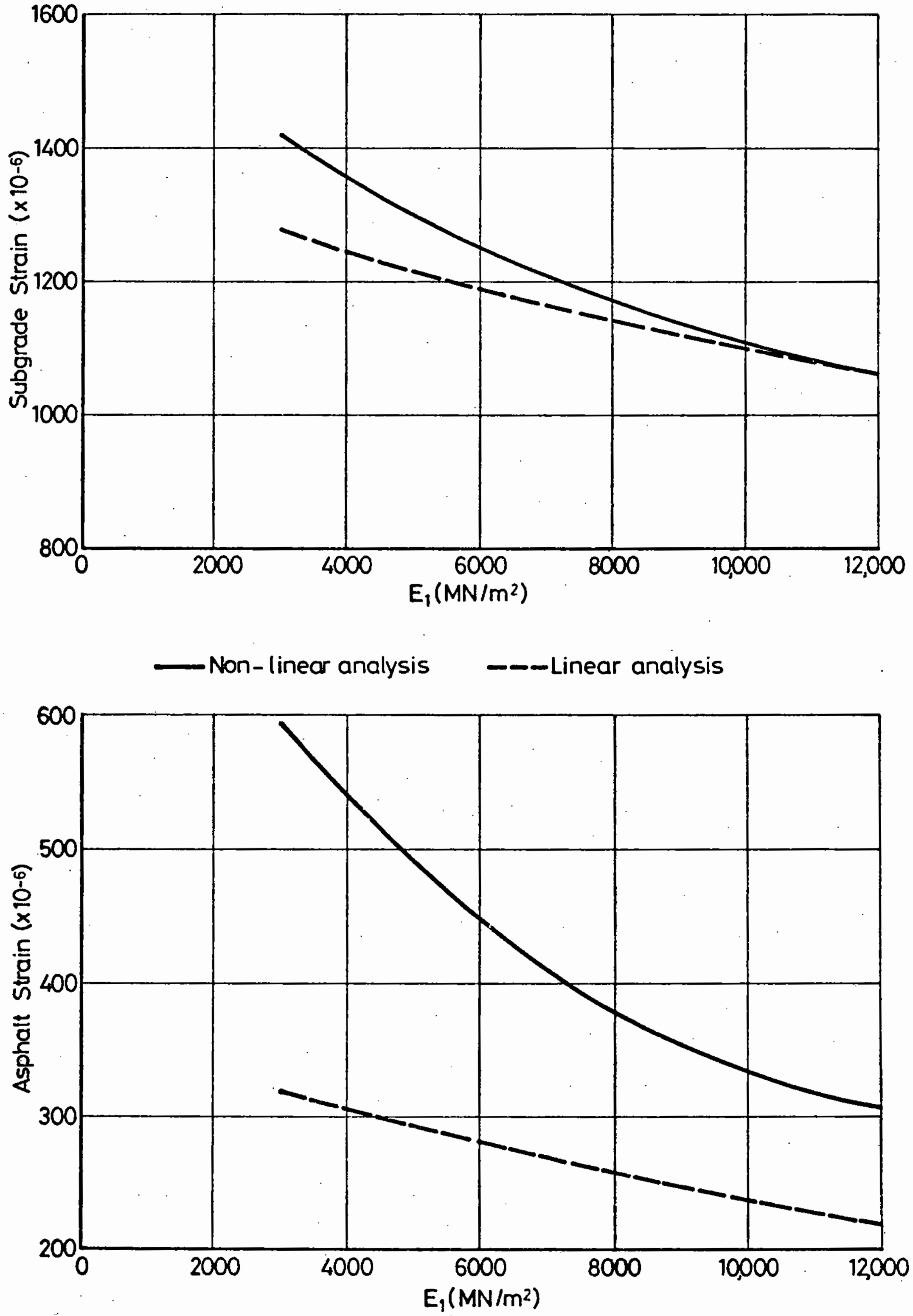


FIG. 6.8 COMPARISON OF PRIMARY RESPONSE PARAMETERS FOR LINEAR AND NON-LINEAR ANALYSIS AS A FUNCTION OF ASPHALT STIFFNESS (E_1) FOR THE THIN STRUCTURE

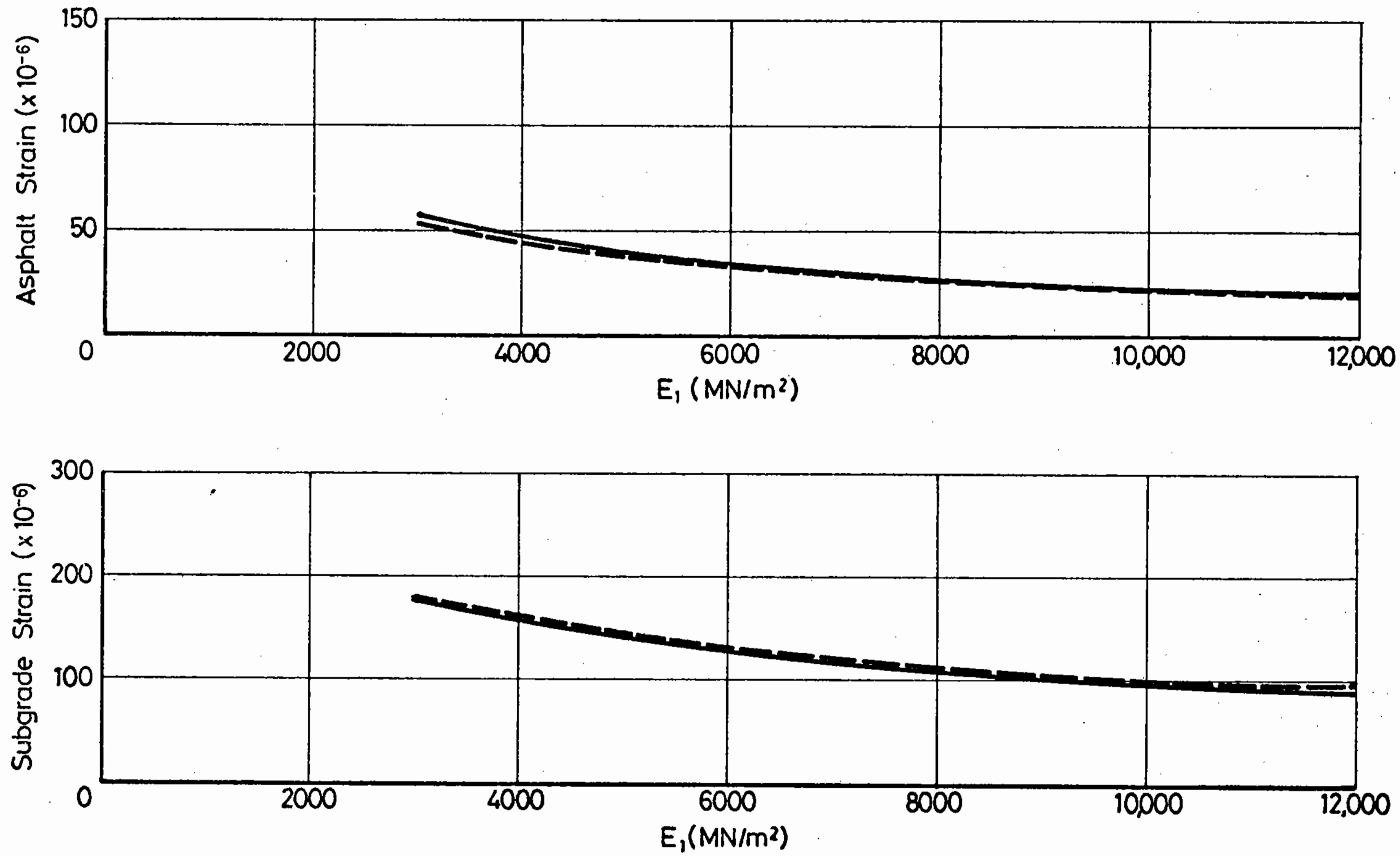


FIG. 6.9 COMPARISON OF PRIMARY RESPONSE PARAMETERS FOR LINEAR AND NON-LINEAR ANALYSIS AS A FUNCTION OF ASPHALT STIFFNESS (E_2) FOR THE THICK STRUCTURE

characterisation. Although the difference between the two systems decreases as the stiffness increases, it remains significant for all cases. Considering the subgrade strain for this structure, at low stiffnesses there is a significant difference between the two systems, with the non-linear analysis giving higher strains than the linear. However, the difference decreases with increasing stiffness until the same result is obtained at a stiffness of 12,000 MN/m^2.

Considering the thick structure (Fig. 6.9), for all practical purposes the two systems produce identical results for both asphalt and subgrade strain. It is also noticeable that the strains themselves do not change significantly with modulus. Since the ability of a pavement layer to withstand load is a function of its thickness and stiffness, it is evident that in structures with thick, relatively stiff asphalt layers, the layer thickness is the overriding consideration.

6.6 IMPLICATIONS REGARDING THE USE OF GRANULAR MATERIALS

The investigation reported above is by no means exhaustive, and the effect of non-linear characterisation on the result of a design calculation is discussed in Chapter 10. However, it is possible, at this stage, to make suggestions on how best to utilise the structural properties of unbound granular materials.

Figs 6.4 and 6.5 indicate that for a given thickness of unbound granular material, variation in asphalt thickness is not a significant factor until thin layers are encountered. This combined with the fact that variation in asphalt layer stiffness is most significant when it is used in thin layers, suggests that the granular material contributes most to the structure when used under a reasonably thick asphalt layer. There is not sufficient data in this study to indicate a minimum asphalt thickness for efficient use of granular material, with any confidence, though 150 mm seems appropriate from Figs. 6.4 and 6.5.

Since the derived modulus of the granular material is a function of the stress in that material, it may be expected that the higher the stress, the greater the modulus. In a highly stressed pavement system, however, the combination of stresses is such that the granular material is near failure which reduces its modulus, and in this condition the layer may have a detrimental effect on the performance of the pavement. A severe example of this is illustrated in Fig. 6.1. The non-linear analysis indicates that asphalt strains are greatly increased over the values predicted by linear analysis. Whilst this is an extreme situation and is probably not representative of practical structures it indicates that attempts to economise in pavement structures by increasing the thickness of the granular layer at the expense of the asphalt layer would lead to increased fatigue susceptibility in the asphalt.

An attempt to plot primary response parameters against a function of the granular material properties has been made in Fig. 6.10. The abscissa on these graphs is indicated as $\frac{h\overline{E}}{K_1}$ where:

h = thickness of granular sublayer,

$\overline{E}$ = mean value of derived modulus for the four sublayers,

K_1 = usual material parameter.

Since:

$$\overline{E} = \sum_{1}^{N} E_{n/N} \tag{6.1}$$

where E_n is derived modulus for the individual sublayers and

$$E_n = K_1 \theta_n^{K_2} \tag{6.2}$$

The abscissa may be rewritten as:

$$h \sum_{1}^{4} \left(\frac{\theta_n}{4}\right)^{K_2}$$

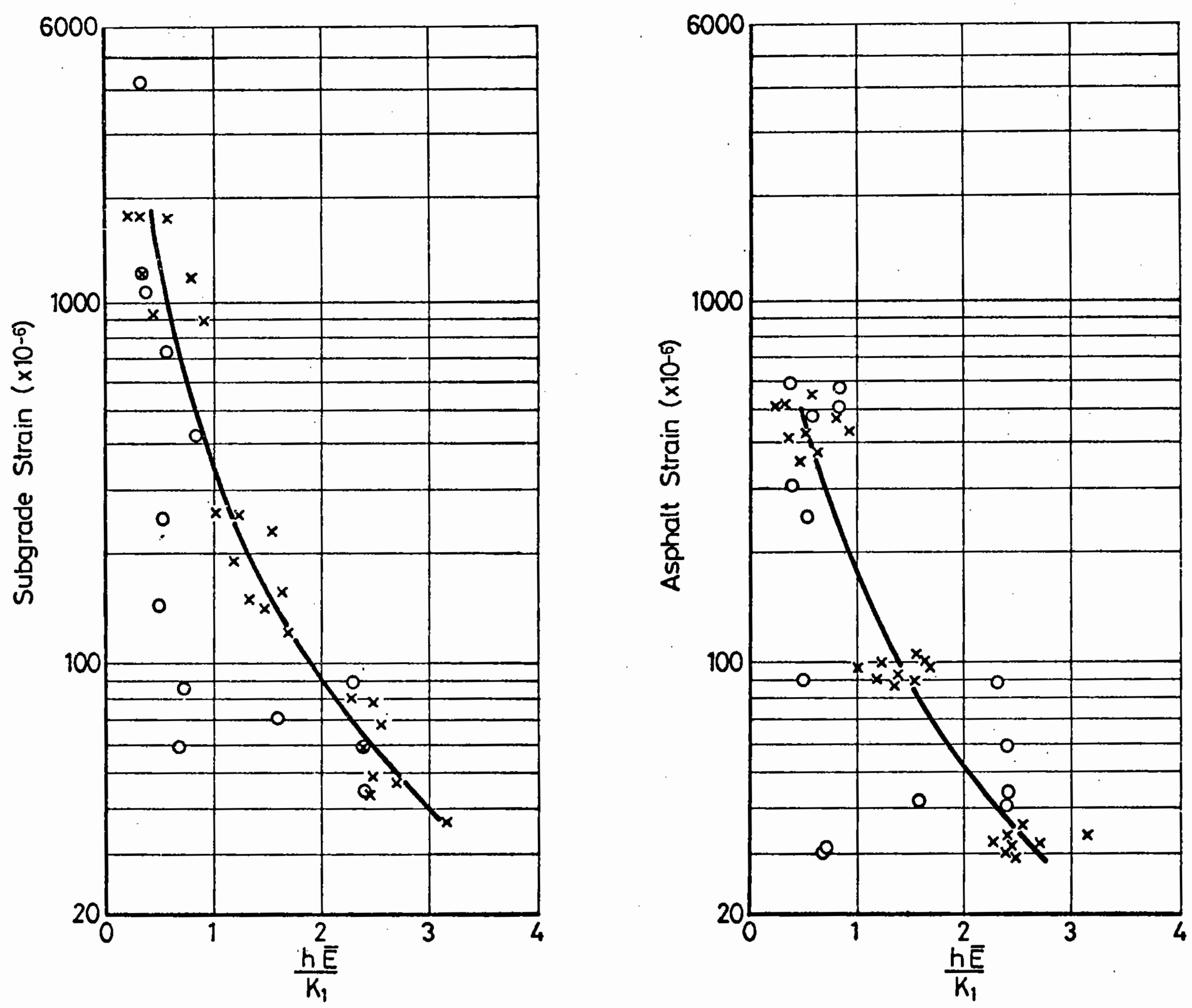


FIG. 6.10 PRIMARY RESPONSE PARAMETER AS A FUNCTION OF THE GRANULAR MATERIAL PROPERTIES

so the log of the primary response parameter is plotted as a function of the thickness of the granular sublayer (h) and the stress within that layer (θ_n), which excludes the quality of the material, but includes consideration of the other layers in the structure. These plots were constructed from the results obtained with the structures listed in Table 5.1 (plotted x) and a best fit line was superimposed by eye. To test the validity of the simplified presentation the results from the structures in Table 5.2 (plotted o) were added. In general, they follow the same pattern, but there are exceptions which do not follow any obvious subsidiary pattern. These figures indicate that reliance on trends when dealing with a highly complex system such as a highway pavement can lead to erroneous conclusions since many parameters must be considered and the interaction between these parameters is complex.

In summary, it may be stated that a granular layer needs to be protected by a stiff layer to function efficiently. Whilst the material has potentially considerable structural stiffness, in conventional British pavements this cannot be realised since the stress system is generally unfavourable for the generation of high moduli.

6.7 CONCLUSIONS

1. A usable model for non-linear elastic characterisation of unbound granular material has been developed. It is essential that this model includes consideration of the behaviour of the material as it approaches failure.
2. The model indicates that under certain conditions the performance of a pavement will be significantly different from that predicted using the basic linear elastic simplification.
3. Investment in improving unbound granular material quality will not be repaid with improved pavement performance.

4. False economies will result from substituting unbound material for bound material in conventional pavement structures.

CHAPTER SEVEN

THE CHARACTERISATION OF SUBGRADES FOR DESIGN

7.1 INTRODUCTION

There is a considerable volume of literature in the field of soil mechanics relating to the behaviour of cohesive soils. Unfortunately, it has not been possible to review all this information. The publications which have been examined relate specifically to pavement studies, or are from sources which relate to the general topic of pavement design. As will be shown, this literature is lacking in data on various points which could possibly be obtained from a wider study.

As with the unbound granular materials, there are two basic approaches to the characterisation of subgrades; the simple linear elastic approach, and the non-linear stress dependent approach. Thus the literature will be reviewed under these two headings and appropriate recommendations made.

A computer program has been produced to include non-linear analysis of the subgrade and a brief study undertaken to compare the results of analysis with and without non-linear characterisation.

Although the study concluded that non-linear characterisation is currently inappropriate for design calculations, the work in this area is considered to be of value in providing a platform for future work. In addition, information has been obtained on the importance of subgrade non-linearity in relation to pavement design as a whole.

7.2 LITERATURE REVIEW

7.2.1 Simple elastic characterisation

The relation used most frequently for characterisation of subgrades is that proposed by Heukelom and Foster (86):

$$E = 10 \times CBR \quad MN/m^2 \tag{7.1}$$

This relation was derived from wave velocity measurements and CBR tests carried out in the field.

Tests reported by Croney (87) on the subgrade at the Alconbury Hill trial agree with the results of Heukelom and Foster (86), suggesting that the factor of 10 defines a lower bound to the scatter of the experimental results.

This relationship has been widely used and is incorporated in the design procedures outlined by Finn et al (88), Claessen et al (44), Celard (89), Southgate et al (90) and Verstraeten et al (91) at the 1977 International Conference on the Structural Design of Asphalt Pavements.

In view of the wide acceptance of Equation 7.1 and the lack of any other guide, this relationship is recommended for use in the linear elastic characterisation of subgrades.

7.2.2 Non-linear characterisation

Considerable research has been reported in relation to non-linear subgrade properties, which is reviewed under several headings as follows.

Effect of moisture on resilient modulus: There is general agreement in the literature (92-95) that increasing moisture content decreases the resilient modulus (E_R) of a soil. Similar results were obtained by Croney and Bulman (96) though their research did not consider non-linear soil behaviour.

Thompson and Robnett (95) have attempted to quantify the moisture effect, and recommend that the resilient modulus be reduced by 2.3 MN/m^2

for each 1% increase in degree of saturation from the optimum, obtained under the conditions of 95% AASHTO T-99 compaction.

Seasonal moisture variation could be significant with respect to the determination of the appropriate resilient modulus for pavement subgrades, particularly if a detailed cumulative damage approach to the design of a pavement is considered. However, before this work can be utilised, information regarding moisture variation in pavement subgrades as a function of time is necessary. Croney and Bullman (96) indicate approximate variation in the level of the water table, but this does not help greatly in determining the degree of saturation.

Effect of compaction on resilient modulus: Nielsen (97) carried out tests on virgin and compacted soils on an airfield project and observed that whilst there was a variation in resilient modulus for the virgin soil it was much smaller than that observed for the compacted soil.

Both Seed et al (92) and Thompson and Robnett (95) have observed differences in measured properties under static and gyratory compaction, Seed et al (92) indicating a closer correlation between laboratory and field results with gyratory compaction. For tests carried out under kneading compaction Seed et al (92) indicate that at a given water content the resilient modulus increases slightly up to a limiting dry density, but further densification can cause a sharp decrease in resilient modulus. They (92) also indicated that such conclusions may be invalid for a series of tests on specimens compacted by a different method.

Age at initial loading: Strength gain has been observed in specimens allowed to rest between compaction and loading (98-100). Seed et al (92) investigated this phenomenon and found a significant reduction in resilient deformation for stored specimens, which was dependent on the number of load repetitions. Above about 40,000 repetitions the resilient behaviour of stored and non-stored specimens was effectively identical.

Effect of soil type on resilient modulus: Seed et al (92) compared three types of soil on the basis of their individual optimum moisture contents (AASHTO tests). These tests indicated that for compaction water contents from 4% below laboratory optimum to 2.5% above the subgrade soils from the AASHO (101) and WASHO (102) road tests and a Vicksburg silty clay all had similar resilient properties. However, at more than 2.5% above optimum there is considerable variation between the three soils.

The work of Thompson and Robnett (95) examined 50 different soils from the State of Illinois, USA, and concluded that current classification procedures do not collect fine-grained soils into groups with distinctive resilient properties. This view was endorsed by Moossazadeh (103) when reviewing resilient modulus relationships for soils.

Regression equations were developed by Thompson and Robnett (95) for estimating resilient properties based on soil characteristics, degree of saturation and volumetric water content. Typically, the standard error (α) of these predictions lies in the range of 10-24 MN/m^2. This means that to be sure that the modulus of the soil in question is correctly predicted 95% of the time, limits of $\pm 2\alpha$ (i.e. ± 20 - 48 MN/m^2) must be applied to the value obtained from the regression equation. Thus the margin for error appears to be large, and the significance of this error must be judged with regard to its effect on the particular pavement structure under consideration.

It is evident that soil type has a significant effect on the resilient behaviour and if accurate modelling is required, with respect to pavement analysis, then the soil in question must be tested to obtain appropriate resilient properties.

Seasonal effects on resilient modulus: Probably the most significant seasonal effect on a pavement subgrade is caused by variations in water table, and by, if relevant, freezing of the subgrade. Part of a table by Croney and Bulman (96), reproduced in Table 7.1, gives information on the depth below pavement surface of the water table for a temperate climate. Estimated CBR values are also given. However, this information does not help to determine the moisture changes in the pavement unless some assumptions are made regarding the degree of saturation.

Thus, it is evident that, as pointed out by Thompson and Robnett (95), current technology is unable to predict subgrade moisture contents and moisture changes as a function of time. This represents a major drawback to efforts to implement a representative model for the resilient behaviour of subgrades, and attention should be focussed on providing this information.

7.2.3 Models for resilient behaviour

There is general accord in the literature (61,92,93,95,97,104,105) indicating that resilient modulus is a function of deviator stress, although Coffman (94), when carrying out unconfined tests, did not observe any effect of stress on resilient modulus. There is also general agreement that the resilient modulus decreases as deviator stress increases. Seed et al (92) observed the modulus reduction with increasing deviator stress, but also that at deviator stresses above 100 kN/m^2 there was a small increase in modulus. The general pattern

Description of Climate	Probable depth of water-table below road surface (m)	Road Surface temperature (^{o}C)	Subgrade							
			Heavy clay (PI = 50%)		Silty Clay (PI = 25%)		Sandy Clay (PI = 10%)		Non-plastic Sand	
			CBR (%)	E (MN/m^2)	CBR (%)	E (MN/m^2)	CBR (%)	E (MN/m^2)	CBR (%)	E (MN/m^2)
<u>Temperate</u>										
With rainfall in excess of 500 mm:										
(a) Winter Condition	0.8	-5 to +5	2	21	3	32	5	39	10	84
(b) Spring Condition	1.0	20 to 25	2.5	26	5	39	7	60	20	126
(c) Summer Condition	1.2	30 to 35	3	32	6	53	12	105	30	160

Table 7.1 The effect of climate on pavements (after Croney and Bullman, 96)

of behaviour determined from specimen testing has been confirmed by in situ observations from instruments in pavement structures (61). This agreement gives confidence in the use of triaxial testing as a method for determining material parameters for use in pavement analyses.

In their comprehensive study, Thompson and Robnett (95) produced a relationship of the form shown in Fig. 7.1. In discussing this work and with reference to Fig. 7.1, the following points should be noted:

(1) Moisture contents are expressed relative to the optimum for the soil.

(2) D_1 is the slope for the left part of the plot.

(3) D_2 is the slope for the right part of the plot.

(4) E_{R_i} is the resilient modulus at the intersection point.

(5) σ_{d_i} is the deviator stress at the intersection point.

From their wide ranging experimental study, Thompson and Robnett (95) concluded that E_{R_i} is the most significant parameter relative to predicting the structural response of a flexible pavement. For practical purposes a D_1 value of 1.1 $kN/mm^2/N/mm^2$ may be used, and σ_{d_i} may be taken as 41 kN/m^2. Several procedures were reported for predicting E_{R_i} depending on the data available, regression equations being reported for various combinations of soil property, degree of saturation, volumetric water content and soil classification data. Since over 40 regression equations were presented, and some of the soil classification data requires detailed explanation, the reader is advised to consult the original document (95) for detailed information.

Brown et al (105) introduced overconsolidation ratio as a variable into resilient modulus studies on saturated, reconstituted specimens of a silty clay (Keuper Marl). In this work, it was found that the

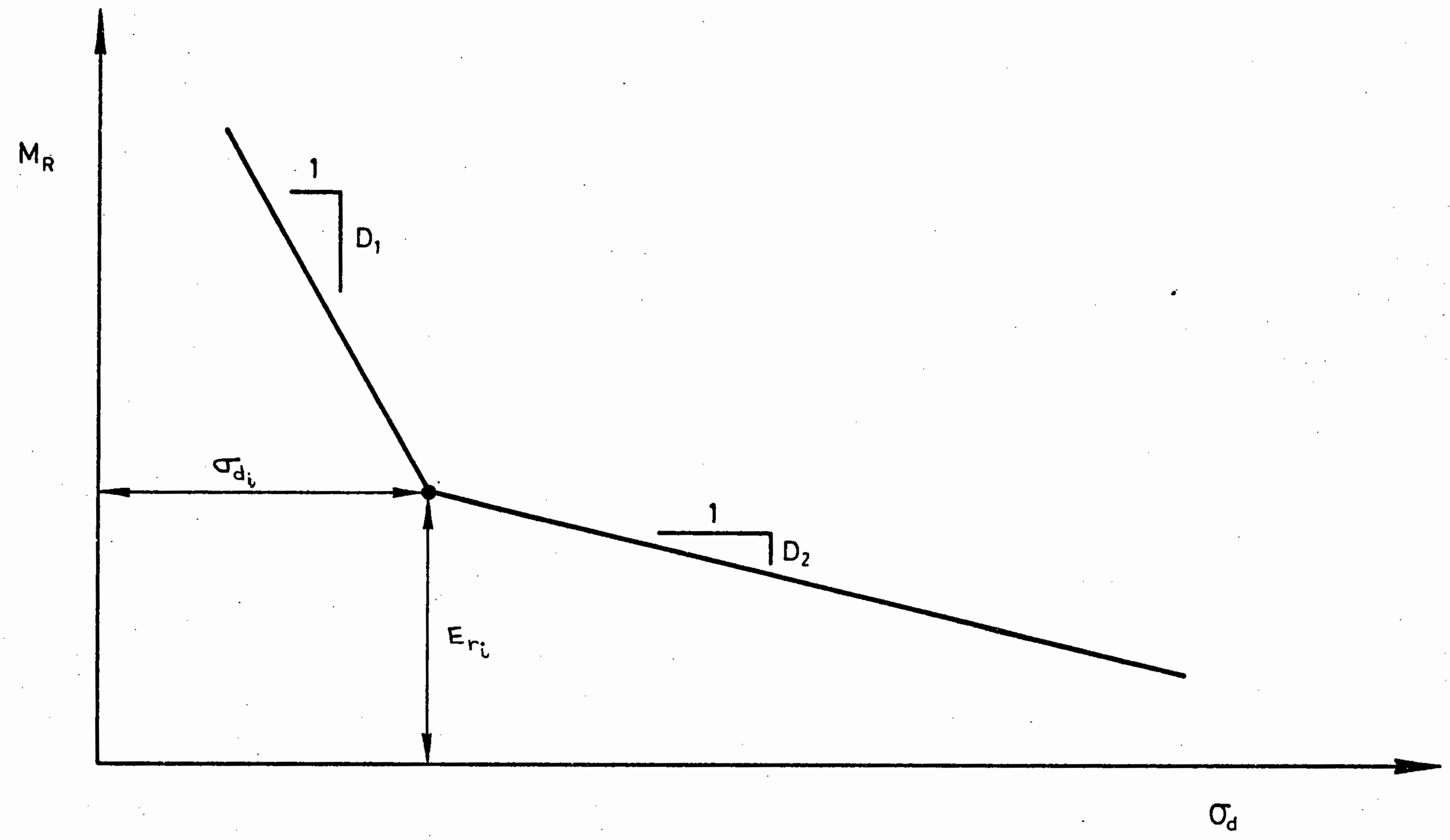


FIG. 7.1 GENERAL MODULUS-STRESS RELATIONSHIP FOR COHESIVE SUBGRADES

modulus-stress relationship was affected by the overconsolidation ratio. However, plotting $E_R = f(q_r/\sigma_3')$ where $q_r = \sigma_1' - \sigma_3'$ (the prime indicating effective stresses) eliminated the effect of overconsolidation ratio.

In order to apply the results of material testing to a pavement, careful consideration of the test condition is necessary to determine how they relate to a pavement structure. This is particularly so with the work of Brown et al (105) since in a pavement system under generalised conditions $\sigma_2 \neq \sigma_3$, but in the triaxial test $\sigma_3' = p'$, the mean normal effective stress.

To implement the model of Brown et al (105) it is necessary to calculate q_r from wheel load stresses only, since in the pavement this is effectively the load pulse. p' must be calculated from body forces only. Pore water pressures and hence the level of the water table must be considered since the effective stresses are required.

7.2.4 Implementation of stress dependent soil characterisation

A study by Moossazadeh (103) assumed a stress dependent relationship of the form:

$$E_R = D_1 \sigma_d^{D_2} \tag{7.2}$$

where D_1 and D_2 are material constants, and σ_d is deviator stress.

When D_2 is given a -ve value, Equation 7.2 gives a stress softening relationship of the same general form as that shown in Fig. 7.1.

This relationship was used (103) to develop equivalent single moduli for a non-linear soil for two conditions in a three-layer pavement structure. One condition required a single modulus to give the same vertical strain on the subgrade as the non-linear analysis, and the other required the same vertical deflection at the base/subgrade interface.

One conclusion of this study was that self-weight stresses in the pavement do not have any significant effect, which is surprising since, in general, they will be of the same order of magnitude as the wheel load stresses. It is also unfortunate that this study did not comment on the significance of changing from linear to non-linear equivalent characterisation of the subgrade in terms of pavement performance.

7.3 ANALYSIS TO COMPARE LINEAR AND NON-LINEAR CHARACTERISATION OF SUBGRADES

7.3.1 Introduction

In order to assess the effect of non-linearity of the subgrade on design parameters for this research a modified version of the BISTRO (14) program was developed to incorporate the model proposed by Brown et al (105). Since, in general, the stress in the soil does not vary greatly with depth within 500 mm or so of the subgrade/sub-base interface, it was considered unnecessary to sub-divide the soil layer. Fig. 7.2 is a flow chart outlining the modulus iteration procedure incorporated into BISTRO (14) to allow use of this model.

It should be noted that non-linear characterisation of the unbound layer, as outlined in the previous chapter, was included in this study.

7.3.2 Results of the study

Two structures, shown in Fig. 7.3, were analysed for asphalt and subgrade strain, using the full non-linear elastic characterisation, non-linear characterisation of the unbound layer only, and all layers linear. For these calculations, the water table was assumed to be 1 m below formation level and the soil above the water table was assumed to be saturated. Table 7.2 gives the results of the analysis.

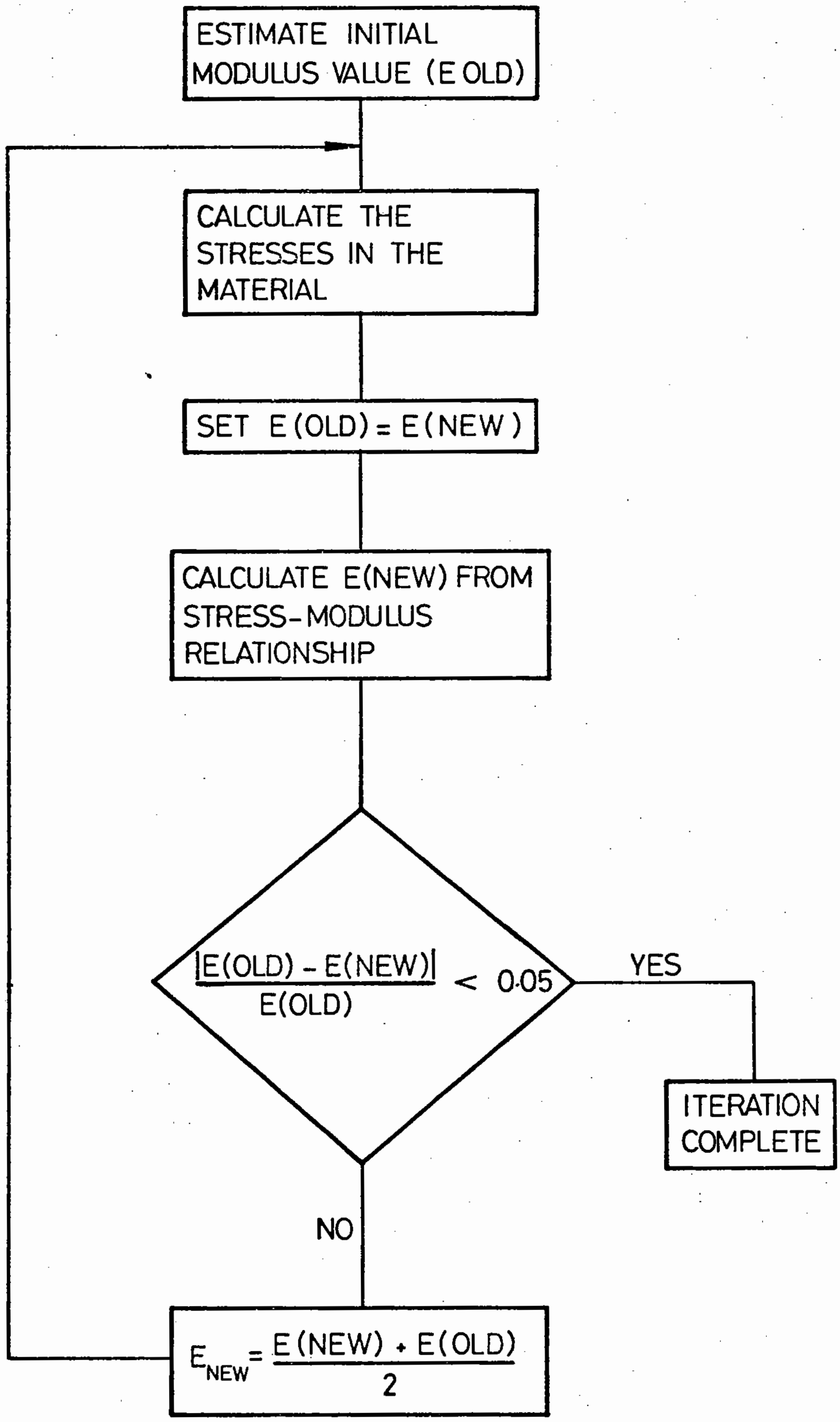


FIG. 7.2 FLOW CHART FOR SUBGRADE MODULUS ITERATION PROCEDURE

Structure 1

$E_1 = 7{,}000\ MN/m^2$

$h_1 = 200$ mm

$E_2 = 125\ MN/m^2$

or

$E_2 = f(\theta)$

$h_2 = 200$ mm

$E_3 = 50\ MN/m^2$

or

$E_3 = f(q_r/\sigma_3')$

Structure 2

$E_1 = 7{,}000\ MN/m^2$

$h_1 = 200$ mm

$E_2 = 125\ MN/m^2$

or

$E_2 = f(\theta)$

$h_2 = 750$ mm

$E_3 = 50\ MN/m^2$

or

$E_3 = f(q_r/\sigma_3')$

FIG. 7.3 STRUCTURES ANALYSED TO COMPARE LINEAR AND NON-LINEAR SUBGRADE CHARACTERISATION

Structure	Parameter	Simple linear	Non-linear base only	Full non-linear
1	ε_A	86	94	86
	ε_z	269	248	208
2	ε_A	78	89	80
	ε_z	144	143	42

Table 7.2 Asphalt and subgrade strains (ε_A and ε_z) for various analytical treatments

It is interesting to note that for structure 1 the full non-linear analysis and simple linear analysis give the same asphalt strain, the non-linear unbound layer plus linear subgrade giving a higher strain. This is probably due to the effect of the subgrade modulus on the derived granular sublayer moduli. In this case, the change in granular modulus has fortuitiously combined with the subgrade modulus to give a similar asphalt strain to that indicated by the simple linear characterisation. A similar effect is noticeable with structure 2, though the asphalt strains are not quite the same for linear and full non-linear analysis in this case.

The effect of full non-linear analysis on subgrade strain is very marked, causing a significant reduction in both cases, reducing the strain to less than one-third of the value obtained by the other two analysis methods for this structure. This is undoubtedly due to the low stress on the subgrade causing the derivation of a very high modulus of 238 MN/m^2.

7.4 FAILURE CRITERIA FOR SUBGRADES

Whilst in the general literature of soil mechanics there is considerable information with regard to the failure of soils, this is invariably not correlated with the CBR test. Since the information generally available for characterisation of pavement subgrades is its

CBR, the carefully derived failure theories cannot be applied without correlation factors, which do not appear to be available.

Heukelom and Klomp (46) presented an expression for the permissible vertical stress (σ_v) on the subgrade for a given number of stress applications (N) as follows:

$$\sigma_v = \frac{C.E}{1 + 0.7 \log N} \tag{7.3}$$

where E = subgrade modulus and C = 0.008 or 0.006.

The value of C is dependent on the analysis system used, since this relationship (Equation 7.3) was derived empirically from measurements on pavement structures. Hence, this is not a material related criterion, but a performance criterion and as such provides no guidance with regard to distress within the subgrade itself.

It is therefore clear that no convenient design criterion to protect the subgrade from failure due to unacceptable stress conditions exists.

7.5 RECOMMENDATIONS FOR SUBGRADE CHARACTERISATION

It was concluded from the brief analytical study that non-linear characterisation of the soil can have a significant effect on structural calculations in pavement systems. However, since Brown et al (105) did not include the CBR value of their soil in their results, any comparison with CBR derived moduli is arbitrary. Also, since no data is available regarding moisture conditions within a pavement it is not considered appropriate to include non-linear subgrade characterisation in a pavement design procedure. However, if further information becomes available, including appropriate design criteria, an analysis tool has been made available and a detailed study should be undertaken to assess the significance of non-linear subgrade characterisation as a part of a pavement design procedure.

Until such a study is completed, Equation 7.1, which relates modulus to subgrade CBR, should be used to determine appropriate subgrade properties.

CHAPTER EIGHT

PERMANENT DEFORMATION

8.1 INTRODUCTION

The most common mode of distress in British pavements is that of excessive permanent deformation under the action of traffic. Hence, the information currently available relating to the prediction of rut depths in pavements was studied with a view to improving the way in which deformation is handled in the ADEM design program.

Considerable progress has been made towards developing a truly fundamental approach based on measured deformability of materials and a knowledge of the stress conditions in a pavement. However, these methods have not been developed to a point which permits simple and reliable application.

The alternative approach, based on analysing pavements of known performance has been applied to the limited data available from the Alconbury Hill (87) pavement design experiment. This study, which is reported in full in this chapter, involved the development of a regression equation relating the rate of rutting per standard axle to a linear combination of calculated parameters within the pavement.

8.2 LITERATURE REVIEW

8.2.1 Introduction

There are basically three approaches to the problem of limiting permanent deformation in a highway pavement: (a) the simple approach of limiting the vertical strain on the subgrade, (b) a slightly more sophisticated approach for predicting a rate of rutting as a function of primary response parameters, the functional relationship being derived from regression analyses of pavements of known behaviour, and (c) a fundamental approach, in which the development of deformation is related to material properties and the stress conditions within the pavement. The literature relating to these approaches is reviewed under separate headings.

The International Conference on the Structural Design of Asphalt Pavements in 1977 included a full session on permanent deformation. The moderators of this session, R.D. Barksdale and R.G. Hicks, produced an excellent report (106), which provided a state-of-the-art review as well as summarising the eight papers presented for the session.

8.2.2 Simple deformation criterion

This is the simplest system for dealing with permanent deformation; simply restricting the vertical strain on the subgrade. It is used in the design systems proposed by many authorities, which include the Asphalt Institute in its manual for the design of airports (107) and its latest version of the design manual for full depth asphalt pavements (108); the Shell Oil Company in their design procedure (44,109); Barker et al (49), Southgate et al (90), Jimenez (110), and Santucci (111).

Peattie (43) stated "The primary function of a road structure is to protect the underlying soil from excessive stresses produced by traffic loads". This premise was developed into a design recommendation (43) requiring that the vertical compressive stress on the soil be limited. Dormon (112) used the limiting stress concept of Peattie (43), analysed pavements designed according to the CBR design charts (113), and developed a design criterion which required a limiting strain of 6.5×10^{-4}. Dormon and Metcalf (48) undertook further analyses, obtaining an empirical relationship between limiting compressive strain and number of applications of a standard axle, for pavements with satisfactory overall deformational characteristics.

It should be noted that this type of criterion applies to the entire pavement structure, and does not necessarily relate in any manner whatsoever to the deformational properties of the subgrade. Neither does it imply that deformation of a pavement is concentrated in the subgrade. As with all empirically derived relationships, it is, strictly speaking, only applicable within the range of parameters used for its derivation.

8.2.3 Improved deformation criterion

Since many analytically based design methods involve taking a trial structure and predicting its performance, there has been increased interest in a deformation sub-system capable of predicting rut depth at various stages in the life of the pavement.

Saraf et al (114) developed two regression equations from analysis of the AASHO road test data (101), producing equations dependent on the thickness of asphalt concrete.

For less than 6 inches (15 cm) of asphalt concrete:

$$\log RR = -5.617 + 4.343 \log d - 0.167 \log (N_{18}) - 1.118 \log \sigma_c \quad (8.1)$$

$R = 0.98$, $\alpha = 0.316$

For over 6 inches (15 cm) of asphalt concrete:

$$\log RR = -1.173 + 0.717 \log d - 0.658 \log (N_{18}) + 0.666 \log \sigma_c \quad (8.2)$$

$R = 0.957$, $\alpha = 0.174$

where RR = rate of rutting, microinches per repetition (25.4×10^{-6} mm/rep.)

d = surface deflection, 10^{-3} inches

σ_c = vertical compressive stress in asphalt concrete, $\mathrm{lb/in^2}$

N_{18} = total number of 18,000 lb single axle load up to and including the season for which the rate of rutting is to be calculated in 10^5 cycles

R = multiple regression coefficient

α = standard error of estimate

Carmichael et al (115) also using regression techniques and the AASHO road test data (102) have produced an equation for predicting the number of applications (N) of an 80 kN equivalent load from an allowable rut depth.

$$\log N = 7.51475 + 0.96831\ (R) + 9.01173\ (\varepsilon_{1z} \times 10^5/\sigma_{1z})$$
$$+ 0.04322\ (\sigma_{2z}) - 0.01687\ (\sigma_{2x}) + 0.05608\ (\sigma_{3z})$$
$$+ 0.10803\ (\varepsilon_{4z} \times 10^4) + 0.18032\ (\sigma_{5z})$$
$$+ 0.10226\ (\varepsilon_{5z} \times 10^4) + \log\ (365/d_T) \qquad (8.3)$$

where R = allowable rut depth, inches

ε_{1z} = vertical strain at the bottom of the top layer $\times 10^4$

σ_{1z} = vertical stress at the bottom of the top layer

σ_{2z} = vertical stress at the bottom of the second layer

σ_{2x} = horizontal stress, parallel to the load axle at the bottom of the second layer

σ_{3z} = vertical stress at the bottom of the third layer

ε_{4z} = vertical strain at the bottom of the fourth layer $\times 10^4$

σ_{5z} = vertical stress at the top of the fifth (subgrade) layer

ε_{5z} = vertical strain at the top of the fifth (subgrade) layer $\times 10^4$

d_T = number of days per year when the average daily temperature is equal to or greater than 18°C (should be a five-year average)

No correlation coefficient or standard error was quoted for this equation, though, as indicated, coefficients were quoted to a high order of accuracy.

Potter (116) used data from five experimental sites in the UK comprising 25 sections in all to develop a regression equation for the overall rate of rutting. The plots of rut development with time which generally show considerable fluctuation between measurement points were smoothed and a constant rate of rutting, reached after the initial settling down period, was used for the regression calculations.

The equation was:

$$\ln(D) = 0.22 - 0.008a_1 + 0.02a_2 + 0.03a_3 \qquad (8.4)$$

where D = deformation rate (mm/million cumulative standard axles)

a_1 = computed transient change in asphalt thickness (μm)

a_2 = computed mean isotropic stress at the top of the sub-base (kN/m^2)

a_3 = computed mean isotropic stress at the top of the subgrade (kN/m^2)

This work is of considerable interest, since a comparison of prediction with this equation and measurements at a site not used in the regression showed very good agreement. Unfortunately, this work used linear elastic characterisation of the unbound layer and the asphalt stiffnesses used in the analysis were not compatible with those obtained from the Shell method used in this research. These differences would cause considerable discrepancy in the predictions and, since the basic deformation data is not available, the regression equations cannot be re-evaluated on a compatible basis.

8.2.4 Fundamental prediction methods

These methods are the most complex and recently developed of those presented in the literature. Barksdale (117) and Romain (118) outlined a procedure for sub-dividing a pavement into elements, calculating the stresses in these elements using elastic theory, then determining the elemental deformations from the stress-deformation law for the material and finally summing to produce the overall vertical deformation in the pavement. Thus these procedures require materials testing, analysis and accumulation of damage and the relevant literature will be reviewed with these requirements in mind.

Materials characterisation: The ideal material characterisation will come from a test which duplicates precisely the stress state in a highway pavement. Whilst this is not possible in any one test, Brown and Bell (119) suggest the use of stress invariants as the most appropriate method of representing pavement stress states for characterisation purposes.

The simplest test used for determining the deformational properties of a bituminous material is the creep test. Van de Loo (120), Barksdale and Miller (121), Huschek (122), Kirwan et al (123), Lai and Hufferd (124) and Battiato et al (125) have all used this test to determine the deformational characteristics of pavement materials. However, before the results of a creep test can be used for the calculation of deformation it is necessary to relate the stress conditions of the test to those in the pavement.

Van de Loo (120) solves the problem of relating creep test and in situ behaviour by use of empirical factors applied to the results of tests undertaken at a stress of 0.10 MN/m^2. Barksdale and Miller (121) use a similar approach developed to calculate an average stress to apply to creep test specimen to give the same elastic strain as that calculated in the asphalt layer.

Repeated load triaxial tests have been used by Monismith et al (126), Francken (127), Brown and Bell (119), Kirwan et al (123), and

Meyer et al (128) for their material characterisation.

Barksdale and Hicks (106) pointed out in their Moderators' Report, that material characterisation methods requiring expensive and complex equipment and numerous tests will not be used by pavement designers and that a relatively simple procedure is required. The work by Kirwan et al (123), indicated that if the root mean square value of the stress pulse from a dynamic triaxial test is used in a creep test, similar results are obtained. This represents a useful step in the direction of simplification.

Brown and Cooper (129) have compared triaxial loading under sinusoidal and square shaped loading pulses with creep tests. They found that the creep test produced similar results to sinusoidally loaded triaxial specimens when the creep stress is about 60% of the peak sinusoidal stress. This compares favourably with the r.m.s. value of Kirwan et al (123). A procedure for evaluating the equivalence in a particular case was proposed by Brown and Cooper (129) based on the principle that equivalent permanent strain will result from tests wherein the moment of the area below the stress-time relationship is the same, which was found satisfactory for uniaxial tests with sinusoidal loading but inapplicable for square wave loading.

It is evident that a procedure for characterising materials that would be accurate and acceptable to design engineers and supported by a majority of research workers is not currently available. This is not surprising since it is only in recent years that attention has been focussed on this problem even though for UK conditions it is and always has been the prevalent mode of distress.

Rut depth prediction systems: Prediction systems generally require a method for accumulating deformation from the results of simple loading tests. The two methods which have been suggested are a time-hardening procedure and a strain-hardening procedure (130), both illustrated in Fig. 8.1.

The most detailed study in this field of rut depth prediction is reported by Brown and Bell (119), who used the time-hardening cumulative damage approach. An important discovery by these authors was that, whilst surface deformation may be adequately predicted, the distribution of deformation with depth may be vastly different to that actually occurring. This has very important implications and explains why some prediction models work for one structure, but when applied to a structure with different layer interactions can be grossly in error.

The Shell prediction system described by Van de Loo (120) and based on the previously mentioned creep test, is quite involved. In this method the stiffness of the mix is defined as the stress in the creep test specimen divided by the total strain, and is time dependent. The stiffness of the bitumen (S_{bit}) is obtained from the Van der Poel Nomograph (3) or it can be measured, and is plotted as a creep curve against the S_{mix} obtained from the creep test. The rut depth prediction model is as follows:

$$\Delta H = C_m \; H_o \left(\frac{\sigma_{avg}}{S_{mix}}\right) \qquad (8.5)$$

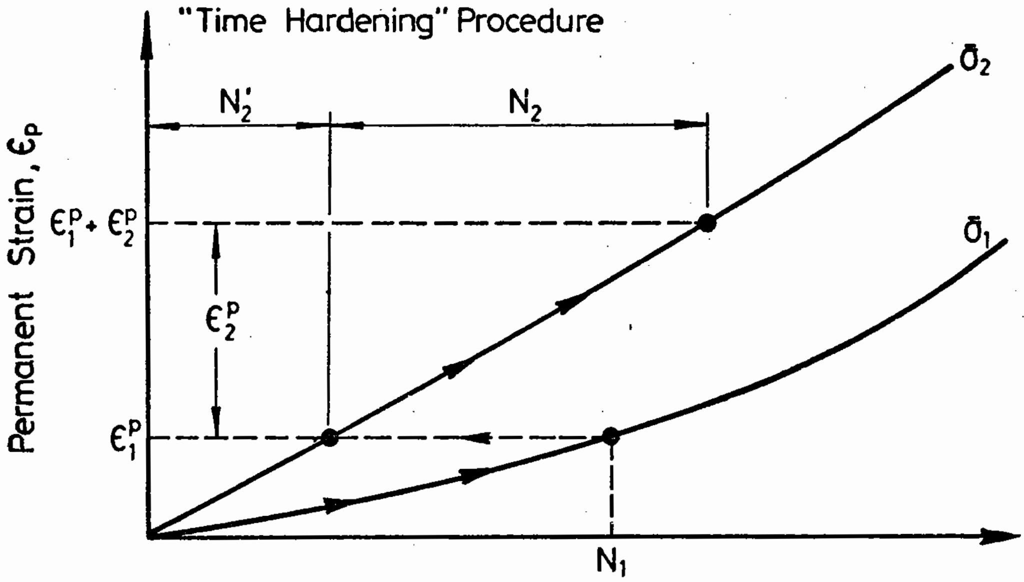
where ΔH = estimated rut depth in layer of thickness H_o

S_{mix} = stiffness of the mix obtained from the creep curve at the value of the stiffness of the bitumen corresponding to the design life of the pavement

H_o = asphalt layer thickness

C_m = correction factor which includes dynamic effects. This factor depends on the type of mix and must be empirically determined.

σ_{avg} = average stress due to moving wheel load in the layer in which rutting is being calculated.



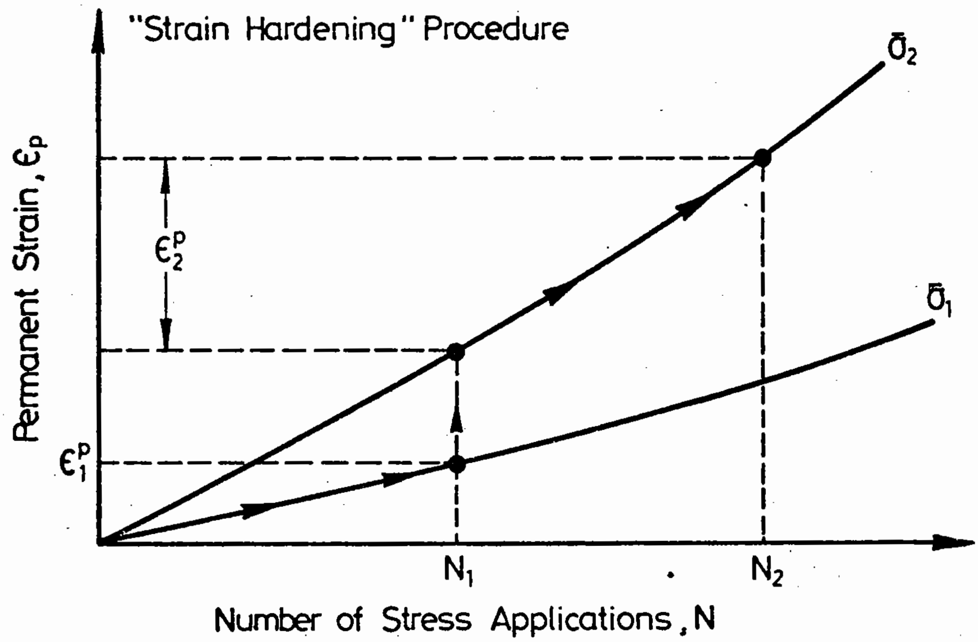


FIG. 8.1 METHODS FOR ACCUMULATING PERMANENT STRAIN (AFTER MONISMITH, 130)

$$\sigma_{avg} = Z.\sigma_o \tag{8.6}$$

where σ_o = contact stress

Z = factor relating the elastic strain which would occur in the asphalt layer in situ to that which would occur in an unconfined specimen of the asphalt subject to the stress σ_o.

To determine the stiffness of the bitumen appropriate for the design life, the following expression can be used:

$$S_{bit} = \frac{3\eta}{N.t_o} \tag{8.7}$$

where t_o = loading time of the wheel pass

η = bitumen viscosity obtained from nomograph

N = total number of wheel passes

Kirwan et al (123) recommend using a creep test on bituminous material with the root mean square of the dynamic pulse load applied to a test specimen. Subgrade properties are obtained from a simple chart and the time-hardening cumulative damage approach is used together with a finite element analysis of the pavement.

Monismith et al (126) consider traffic details in their method including axle loads and lateral distribution. They also consider temperature gradients in the pavement. Material characterisation is by triaxial testing, and permanent strain is estimated from the equation:

$$\varepsilon_z^p = R\left[\sigma_z - \tfrac{1}{2}(\sigma_x + \sigma_y)\right] \tag{8.8}$$

where σ_z = stress in vertical direction

σ_y = stress in radial direction

σ_x = stress in tangential direction

$R = \dfrac{\bar{\varepsilon}^p}{\bar{v}}$, the ratio of total "effective" strain to the "equivalent" stress where, for triaxial conditions,

$\bar{v} = (\sigma_1 - \sigma_3)$ and $\bar{\varepsilon}^p = 2/3(\sigma_1 - \sigma_3)$.

Monismith et al (126) concluded that the method gave reasonable agreement and recommended the use of the technique for special situations as a check on the development of deformation.

Meyer and Hass (131) present a working sub-system for estimating rutting. Observations from the Brampton road test were used in conjunction with triaxial testing for material characterisation. In order to simplify rutting calculations, sensitivity analyses of potentially significant variables were undertaken. From the data on rut depths and significant variables, a regression equation was developed for predicting rut depth:

$$\begin{aligned} D = &- 1.0318 + 1.2067\ AT + 0.0803\ N - 2.3684\ \ln AT \\ &+ 0.1896\ \ln(AT.N) + 1.1639\ E_1 \ln AT - 0.0216\ E_s N \\ &- 0.4114\ E_1 N \ln AT + 0.0456\ E_s N \ln AT \end{aligned} \tag{8.9}$$

where D = permanent deformation in inches

AT = equivalent asphalt thickness/10, inches

E_1 = modulus of the asphalt layer/10^6 psi

E_s = modulus of subgrade/10^4 psi

N = number of equivalent 18 kip load applications/10^5

The equivalent asphalt thickness is for use in pavements with unbound bases, when 25 mm of asphalt mix is equivalent to 50 mm of granular base and 75 mm of sub-base. Barksdale and Hicks (106) wisely suggest caution when converting from one type of material to another since equivalency ratios can vary greatly with material quality.

Viscoelastic systems have been suggested by Battiato et al (125), Huschek (122) and Thrower (132). These methods do not appear to be very popular, possibly because of their apparent mathematical complexity. The advantage of using this approach is not clear since in general they do not appear to offer improved accuracy of prediction. One advantage claimed is their ability to deal with the swelling of material on either side of the rut. However, the surface profiles measured by Brown and Bell (119) generally do not indicate the formation of a shoulder, and when they do it is very small. Also the profiles reported by Croney (87) do not show this phenomenon. Thus, when the aim is to produce a simple engineering system, the complexities of the viscoelastic approach seem to be difficult to justify.

The current state-of-the-art of the mechanistic approach suggests that it has great potential, but that currently it must be used with caution. Whilst for limited conditions good agreement can be obtained between prediction and measurement, its general applicability cannot be guaranteed.

8.3 INVESTIGATIONS TO IMPROVE THE DEFORMATION SUBSYSTEM

8.3.1 Introduction

In developing the design system over the first two years of the project, the simple deformation criterion of subgrade strain was used. Whilst this approach has the merit of simplicity, and as implemented in ADEM, has been specifically developed for British conditions, it does not provide any indication of the development of deformation with time. An attempt was, therefore, made to develop an improved deformation system which would allow prediction of rut depths, and permit investigation of the effects on design of varying the allowable deformation.

The most attractive system is the mechanistic one, as described above. Unfortunately, the method requires extensive material testing, which was outside the scope of this investigation. Furthermore, in spite of the sound fundamental principles embodied in its approach, it does not appear to be able to offer, as yet, a real degree of reliable precision. Thus an improved deformation subsystem based on this approach was ruled out.

The work reported by Potter (116) provides a useful simple method for predicting deformation. This method is somewhat empirical and can therefore only be applied within a framework compatible to its derivation. As indicated previously, the stiffness values used by Potter (116) are not compatible with those used in the ADEM system, and therefore its application would lead to incorrect rut depth predictions. Unfortunately, the basic data used by Potter (116) was not presented and was not available to the author. Therefore, the only remaining source of information is the Alconbury Hill full scale design experiment. Regression equations were obtained from a detailed study of some sections from Alconbury Hill, which predicted the rate of rutting as a function of certain primary response parameters in the pavement. Evaluation of these equations for a range of structures outside that for which they were derived indicated that they did not give reasonable predictions.

A further exercise was therefore undertaken to re-evaluate the simple subgrade strain deformation criterion in the light of non-linear analysis techniques.

8.3.2 The Alconbury Hill trial

Information on the Alconbury Hill trial is available from numerous sources (133 - 136) but the most complete information is given by Croney (87) and has been used for this study.

A range of variables was studied in the Alconbury Hill experiment with data available for sections 51 to 54 with rolled asphalt bases, and sections 38, 60 and 61 with wet mix bases. Since all these sections were constructed with a sand sub-base, the thickness of which varied from one end to the other, the average thickness for each length was taken for analysis purposes. This is justifiable since the deformation measurements reported are the average of three locations along the section. Fig. 8.2 shows the assumed geometry of the various sections and Figs 8.3 to 8.6 show the development of deformation with time in these sections.

Since a detailed analysis of the pavement at monthly intervals was undertaken, careful consideration of traffic, environmental and material variation was necessary. Information on these various input parameters is given in the sections below.

Traffic: The initial traffic at Alconbury Hill was 1400 c.v.d. with an estimated initial growth rate of 5% (87). A study in 1961 reported by Croney and Loe (134) indicated 3900 c.v.d. in both directions, i.e. 1950 c.v.d. in one direction, with a 6% growth rate. It was found that in order to achieve the appropriate 1961 traffic density a growth rate of 7% (rather than the estimated 5%) was necessary, when the 1400 c.v.d. of 1957 was used as a starting point. Therefore, 7% was assumed for the 1957-1961 period. Since measurements were taken in 1961 it was assumed that the growth rate of 6% given by Croney and Loe (134) was accurate and this was used for traffic calculations in the post-1962 period. Monthly totals for commercial vehicles were therefore calculated using these growth rates. Considering inter-lane distribution as proposed in Appendix A, indicates that approximately 92% of this traffic will be in the nearside lane, and the traffic count has been adjusted accordingly. Since the traffic volume does not vary greatly with time a constant value of 92% has been used for ease of computation. The seasonal reductions in traffic during the month of August, and over the Christmas period have also been allowed for in the traffic computations.

Section 51

Rolled Asphalt Surfacing	100 mm
Rolled Asphalt Base	75 mm
Sand Sub-base	300 mm
Subgrade CBR = 4.5	

Section 52

Rolled Asphalt Surfacing	100 mm
Rolled Asphalt Base	150 mm
Sand Sub-base	225 mm
Subgrade CBR = 4.0	

Section 53

Rolled Asphalt Surfacing	70 mm
Rolled Asphalt Base	150 mm
Sand Sub-base	260 mm
Subgrade CBR = 4.0	

Section 54

Rolled Asphalt Surfacing	37 mm
Rolled Asphalt Base	150 mm
Sand Sub-base	290 mm
Subgrade CBR = 4.6	

Section 38

Rolled Asphalt Surfacing	100 mm
Wet-mix Base	230 mm
Sand Sub-base	140 mm
Subgrade CBR = 4.5	

Section 60

Rolled Asphalt Surfacing	100 mm
Wet-mix Base	150 mm
Sand Sub-base	230 mm
Subgrade CBR = 4.5	

Section 61

Rolled Asphalt Surfacing	70 mm
Wet-mix Base	150 mm
Sand Sub-base	260 mm
Subgrade CBR = 4.0	

FIG. 8.2 ALCONBURY HILL TRIAL PAVEMENT SECTIONS

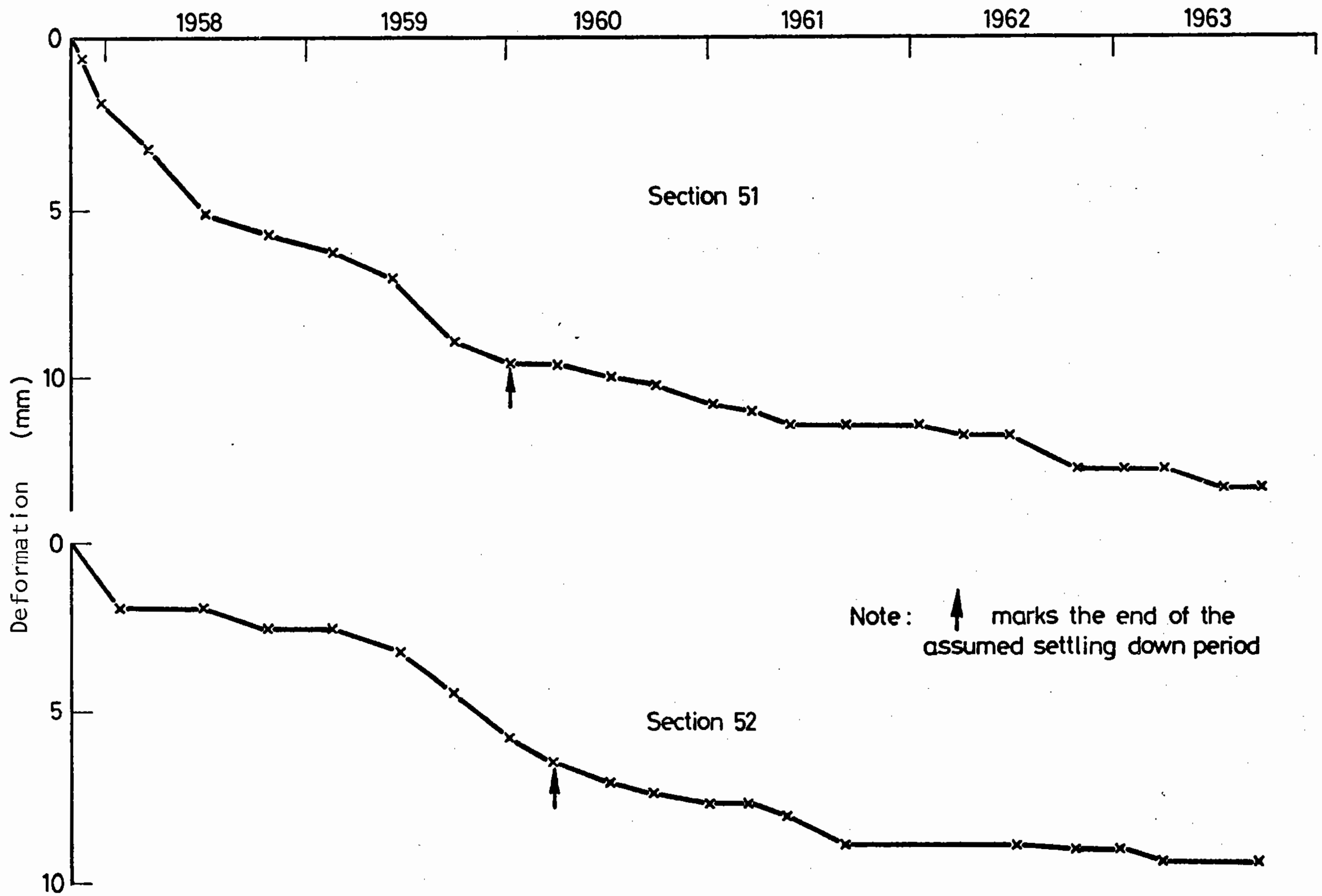


FIG. 8.3 DEFORMATION DEVELOPMENT FOR ALCONBURY HILL TRIAL SECTIONS 51 AND 52

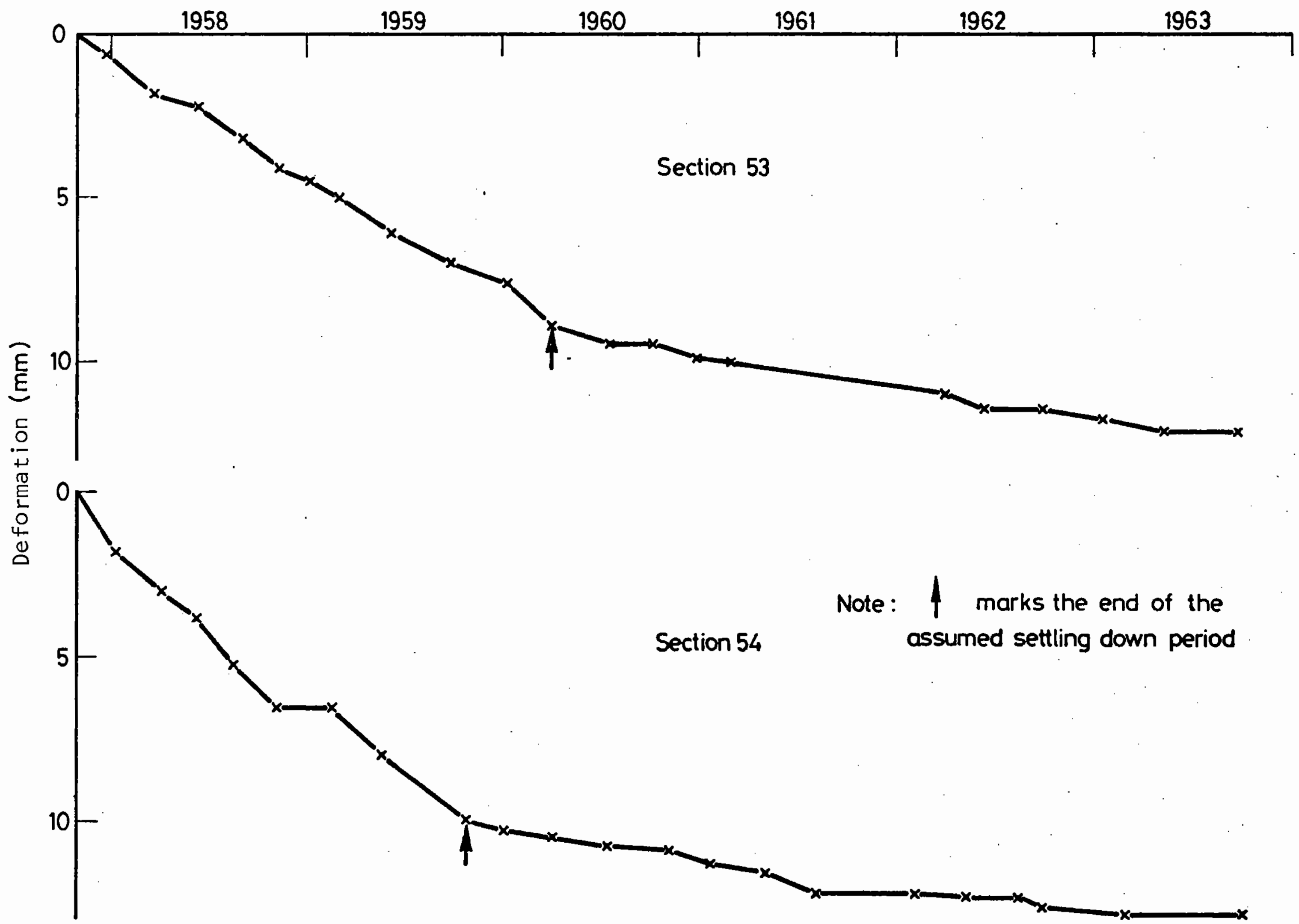


FIG. 8.4 DEFORMATION DEVELOPMENT FOR ALCONBURY HILL TRIAL SECTIONS 53 AND 54

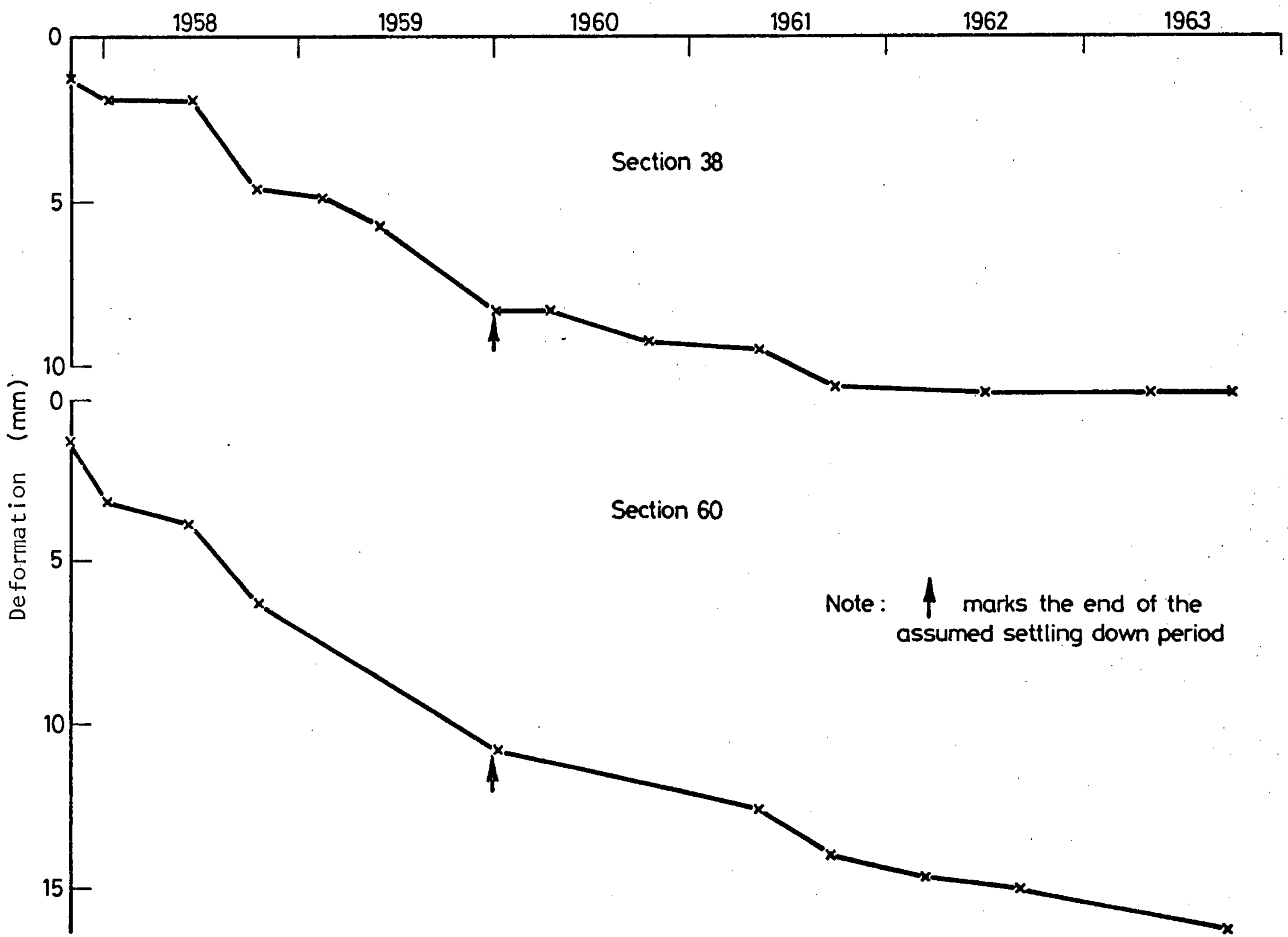


FIG. 8.5 DEFORMATION DEVELOPMENT FOR ALCONBURY HILL TRIAL SECTIONS 38 AND 60

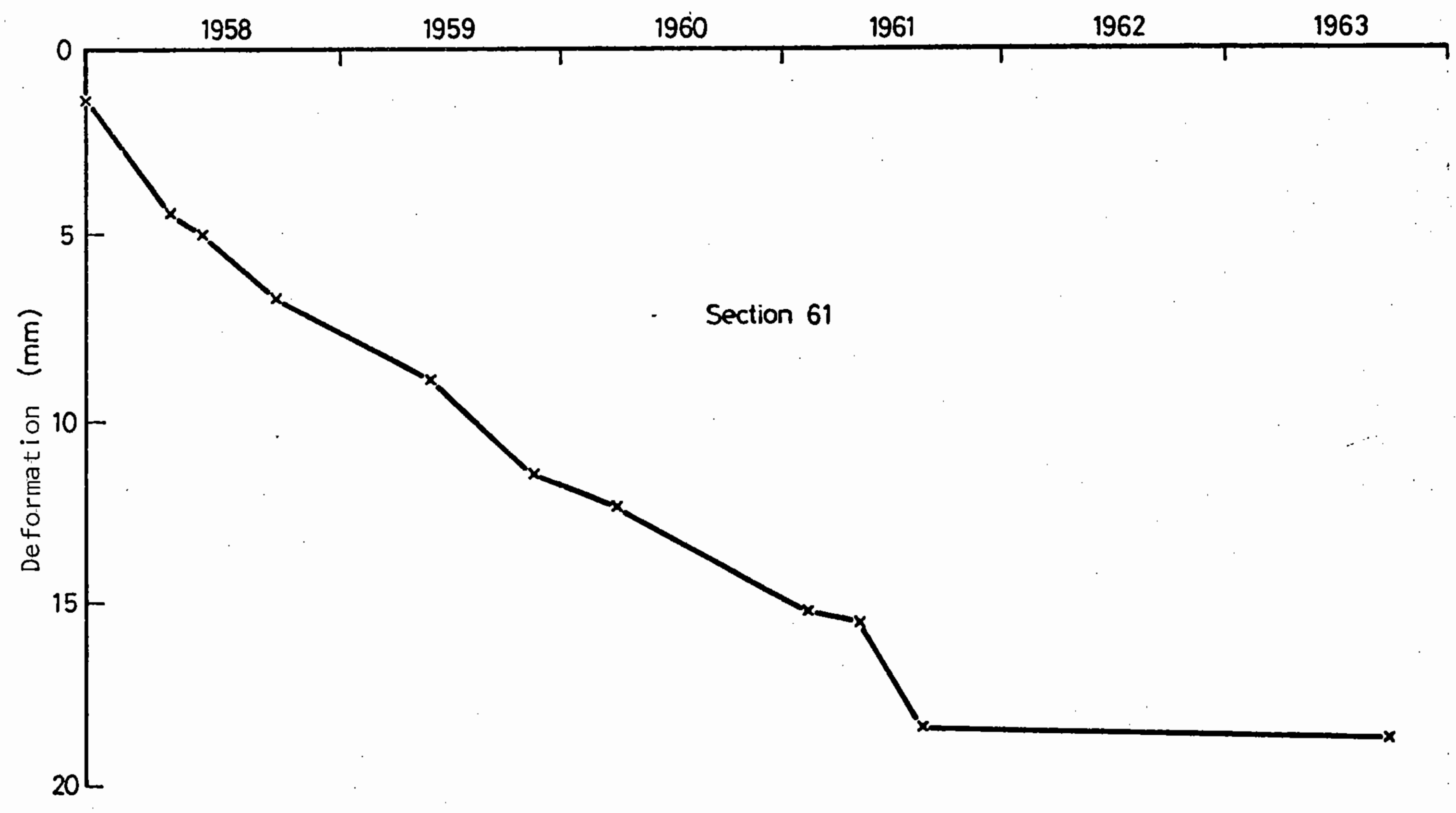


FIG. 8.6 DEFORMATION DEVELOPMENT FOR ALCONBURY HILL TRIAL SECTION 61

Finally, the commercial vehicle count was converted to standard axles using the factor suggested by Croney (87), of 25 standard axles/100 commercial vehicle. (It is interesting to contrast this with the figure of 225 standard axles/100 commercial vehicles currently recommended for use in design (137).)

Temperature: Temperature records for the Alconbury Hill site were not available. However, the Metereological Office produces monthly weather surveys and prospects (138) for numerous weather stations. Since, according to Croney's classification (87), Alconbury Hill lies in the central climatic zone, ten weather stations within this area were selected amd mean monthly temperatures for each extracted from the monthly weather reports. This data was then averaged to give the information in Table 8.1. Unfortunately, records for 1958 and 1959 were incomplete, and for those months starred, the long term means (87) have been used.

For the purpose of the analysis these air temperatures were then converted to pavement temperatures using the procedure outlined in Appendix A.

Asphalt: In order to estimate the stiffness of the asphalt layers it is necessary to know the mix proportions. For the Hot Rolled Asphalt wearing course, a 50 pen binder was assumed, the mix having 7.9% bitumen by weight and a compacted void content of 4%. The Hot Rolled Asphalt basecourse was assumed to have 5.7% by weight of binder and a void content of 6%. Recovered properties of the binder were estimated as previously described (Chapter 3) and the mix stiffnesses determined for a range of temperatures using the PONOS computer program (25).

Year	January	February	March	April	May	June	July	August	September	October	November	December
1957	-	-	-	-	-	-	-	-	-	-	6.7	4.6
1958	3.2	4.6	5.7*	8.5*	11.3*	14.4*	16.0*	15.8	14.0*	10.8	6.6*	4.5*
1959	3.3*	3.7*	5.7*	9.2	11.3*	14.4*	16.0*	15.6*	14.0*	12.5	7.0	5.9
1960	4.2	3.9	5.9	8.8	12.4	15.7	15.1	14.9	13.2	10.3	7.2	4.2
1961	3.5	6.7	9.3	9.6	10.6	14.4	14.8	15.4	15.0	10.9	6.4	2.7
1962	4.4	4.4	2.6	7.4	9.8	13.3	14.3	14.3	12.7	10.4	5.7	2.3
1963	-0.1	-0.6	5.7	8.0	11.2	14.2	14.9	14.1	13.0*	10.8	7.9	3.16

* Temperatures from the average year (28)

Table 8.1 Mean monthly air temperature

Wet mix base material: No tests have been reported on this material. It was therefore assumed to be a high grade granular material whose resilient behaviour could be represented by a non-linear model as outlined in Chapter 4. Since this material was expected to be of excellent quality, having been chosen for an experimental pavement, a K_1 value of 600 was used with a K_2 value of 0.6 (Equation 4.13).

Sand sub-base: Croney (139) has presented the results of some repeated load triaxial tests on the Alconbury Hill sand sub-base. From these results it was possible to develop a non-linear model for this material in which K_1 = 335 and K_2 = 0.52.

Subgrade: It is evident that some difficulty was experienced with the Alconbury Hill subgrade (134), due to a high water table and although Croney (87) reported the results of some triaxial testing on the soil, this cannot readily be related to the subgrade conditions of any particular section.

Thus, the commonly used relationship of

$$E = 10 \times CBR \quad (MN/m^2) \tag{8.10}$$

was used with the estimated final CBRs (136). Some support for this approach is given by Croney (87), reporting the relationship between in situ CBR and wave velocity measurements on the exposed subgrade at Alconbury Hill. The results suggest that the factor of 10 seems to give a reasonable lower limit for the modulus of the clay subgrades.

8.3.3 Analysis

The tool used for analysis of the test sections was a modified version of the BISTRO (14) program. This modified program was used so that full advantage of improved non-linear characterisation of both the unbound base and sand sub-base could be incorporated. For the purpose of the non-linear characterisation both the base and the sub-base were subdivided into two sub-layers.

From consideration of the literature, current practice and intuition, the following parameters were chosen for derivation from the analysis, which was undertaken for the usual 80 kN dual wheel load:

(a) Surface deflection on the axis of symmetry of the two wheels and under the centre line of one wheel.

(b) The first strain invariant $(\varepsilon_1+\varepsilon_2+\varepsilon_3)$, the octahedral shear strain $\left\{\frac{1}{\sqrt{2}}\sqrt{(\varepsilon_1-\varepsilon_2)^2+(\varepsilon_2-\varepsilon_3)^2+(\varepsilon_1-\varepsilon_3)^2}\right\}$, the tangential strain (ε_T) and the difference between the average horizontal $\left\{\frac{\varepsilon_T+\varepsilon_R}{2}\right\}$ and vertical (ε_z) strain at the bottom of the asphalt on the axis of symmetry of the dual wheel load.

(c) The first stress invariant $(\sigma_1+\sigma_2+\sigma_3)$, the octahedral shear stress $\left\{\frac{1}{\sqrt{2}}\sqrt{(\sigma_1-\sigma_2)^2+(\sigma_2-\sigma_3)^2+(\sigma_3-\sigma_1)^2}\right\}$ and their ratio at the centre of the lowest sub-layer in the sub-base, and when appropriate the unbound base on the axis of symmetry of the load.

(d) The first strain invariants, the octahedral shear strain, the vertical strain and the difference between the average horizontal and vertical strain at the top of the subgrade, on the axis of symmetry of the load.

(e) Transient thickness change in the asphalt on the axis of symmetry of the dual wheels and under the centre of one of the wheels.

For the sake of convenience, the absolute values of the parameters listed above were used in the regression analysis. This may appear ambiguous in relation to the parameter calculated from the difference between the average horizontal strain and the vertical strain. This parameter, calculated with due regard for the sign of the individual parameters, may be expected to change sign depending on the relative magnitude of the component strains. However, there was no sign change in this calculated parameter for either the asphalt layer or the subgrade and since it showed no sign of approaching zero use of its absolute value was considered justifiable.

The output from the modified BISTRO did not contain sufficient significant figures to show up variations in surface deflections or transient thickness changes. This was not discovered until all calculations had been completed, by which time it was too late to further modify the program and recalculate these parameters. The regression analysis, therefore, neglected these parameters and they are not reproduced in Tables 8.2 to 8.6.

8.3.4 Regression analysis

Initially, a regression analysis was attempted using data from every month of the period for which data was available. It was, however, impossible to obtain a reasonable correlation with this data set. The reason for this quickly became apparent when temperature and rate of rutting were plotted together as a function of time as in Fig. 8.7 for section 53. The hypothesis for the development of the deformation model was that deformation is a function of temperature. Fig. 8.7 shows clearly that this hypothesis is not generally ture for section 53 of the Alconbury Hill experiment and a similar conclusion can be drawn for other sections. In an attempt to improve the correlation the subgrade and sub-base properties were altered for the winter months on the basis that primary response parameters should be similar for similar rates of rutting and subgrade support could well be reduced during the winter.

This approach proved to be unsuccessful and suggests that the difference between the deformation mechanisms in operation during the

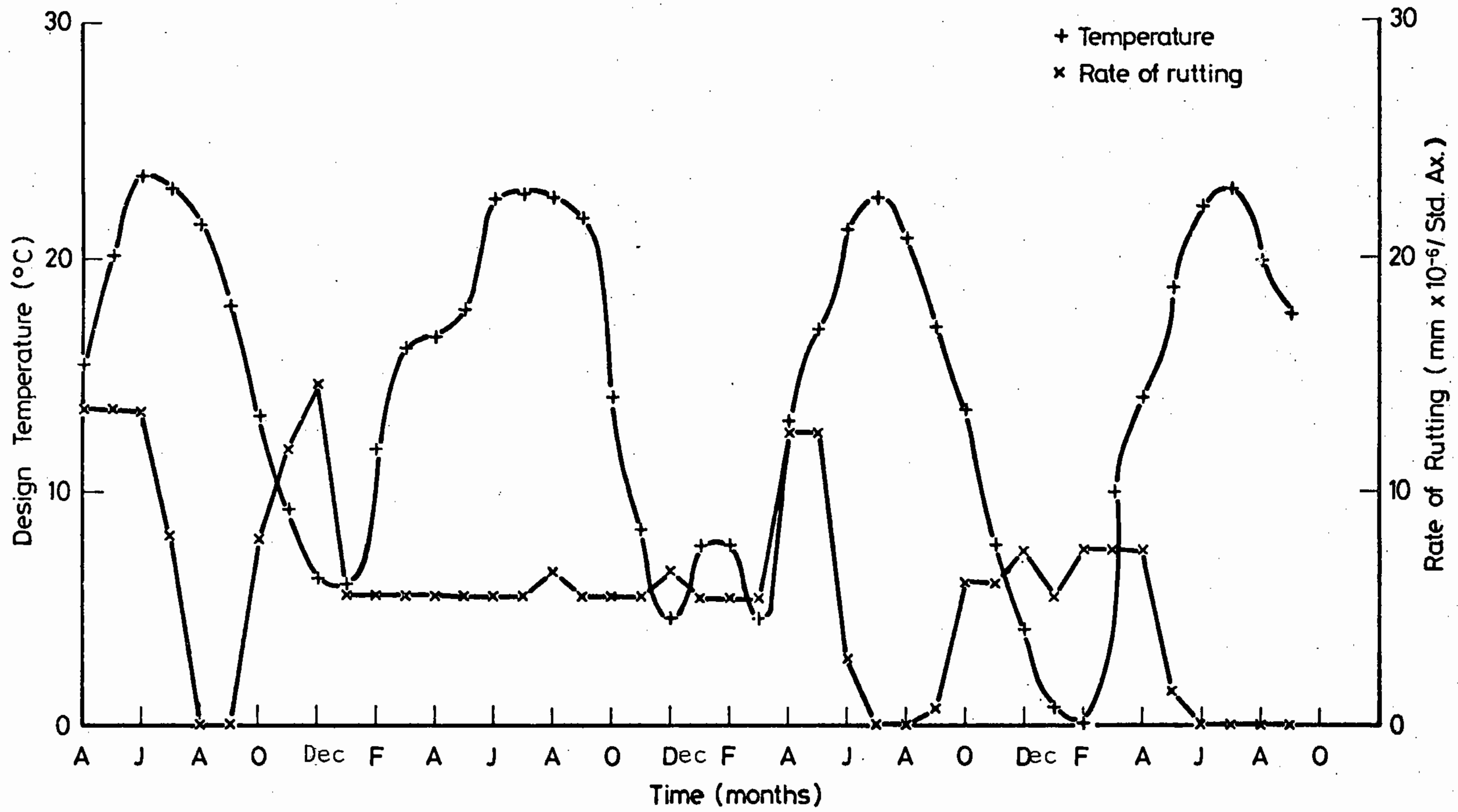


FIG. 8.7 TEMPERATURE AND RATE OF RUTTING AS FUNCTIONS OF TIME

summer and winter months are such that a single regression equation from back analysis cannot reasonably be expected to cope with them both.

In order to proceed with the regression analysis, months in which the temperature-deformation hypothesis appeared to be valid were selected by inspection of plots similar to Fig. 8.7 for all the sections, except No. 53, which showed particularly poor correlation with the hypothesis. This section has been omitted completely from the regression. Similarly, section 61, an unbound base section, was omitted from the analysis. The data thus selected from the remaining sections was used for the final regression analysis and is presented in Tables 8.2 to 8.6.

Initially, the standard system Statistical Package for Social Scientists (S.P.S.S. (140)) was used for stepwise regression. This package produces an optimum regression equation from several independent variables which are added one at a time provided that they contribute to the statistical significance of the equation. With this package, highly significant regressions were obtained, but when the standard error of the equation was examined it was found to be unacceptably large.

A second statistical package (GENSTAT (141), was tried and this provided statistically significant relationships and a much reduced standard error. The final equation for pavements with rolled asphalt bases was:

$$RR = 34.111 + 0.1199\ X_1 + 0.1525\ X_2 - 5.9916\ X_3 \qquad (8.11)$$

where RR = rate of rutting (mmx10^{-6}/standard axle)

X_1 = the octahedral shear strain at the bottom of the asphalt layer

X_2 = difference between the average horizontal strain and the vertical strain at the bottom of the asphalt layer

X_3 = the first stress invariant at the centre of the lowest unbound sub-layer.

Year	Month	Temperature (^{o}C)		Rate of Rutting mm×10^{-6} per std axle	Bottom of Asphalt				Sub-base			Subgrade			
		Air	Design		ε_T	Invariants 1st	Invariants 2nd	$\frac{\varepsilon_T+\varepsilon_R}{2} - \varepsilon_V$	p	q	q/p	ε_V	Invariants 1st	Invariants 2nd	$\frac{\varepsilon_T+\varepsilon_R}{2} - \varepsilon_V$
					(microstrain)				(kN/m^2)			(microstrain)			
1960	February	3.9	6.5	0	78	39	96	138	10	15.8	1.465	224	72	198	300
	March	5.9	10.5	0	95	45	114	154	11.4	17.1	1.5	252	78	223	336
	April	8.8	15.4	8.21	119	55	140	204	12.2	18.7	1.55	282	87	258	385
	May	12.4	20.2	9.80	155	69	178	211	13.2	20.9	1.59	333	102	298	447
	September	13.2	18.2	9.48	135	62	158	229	12.6	19.8	1.573	308	93	278	415
1961	February	6.7	11.8	7.72	100	47	120	147	11.6	17.5	1.51	261	81	232	348
	March	9.3	16.2	8.45	124	57	147	212	12.2	19.1	1.55	293	89	263	393
	May	10.5	17.8	11.37	134	62	162	227	12.6	19.7	1.57	306	92	276	412
	November	10.4	8.3	0	85	42	103	148	11.1	16.4	1.476	236	75	210	316
	December	2.7	4.5	0	72	36	89	128	10.6	15.2	1.440	211	69	187	283
1962	April	7.4	13.0	0.72	107	50	127	183	11.8	17.9	1.524	268	83	234	359
	July	14.3	22.5	19.68	185	85	212	313	14.8	22.2	1.62	365	111	325	495
	August	14.3	20.8	20.35	161	72	186	271	13.3	29.2	1.60	339	104	304	456

Table 8.2 Regression analysis data, section 51

/contd

Year	Month	Temperature (^oC)		Rate of Rutting mm×10^{-6} per std axle	Bottom of Asphalt				Sub-base			Subgrade			
		Air	Design		ε_T	Invariants 1st	Invariants 2nd	$\frac{\varepsilon_T+\varepsilon_R}{2} - \varepsilon_V$	p	q	q/p	ε_V	Invariants 1st	Invariants 2nd	$\frac{\varepsilon_T+\varepsilon_R}{2} - \varepsilon_V$
						(microstrain)			(kN/m^2)			(microstrain)			
1962	September	12.7	17.0	16.77	127	59	151	218	12.5	19.4	1.56	298	90	268	401
	October	10.4	13.4	16.71	118	51	128	237	11.8	18.0	1.525	272	83	234	363
	November	5.7	7.6	0	82	41	101	144	11.1	16.2	1.475	230	73	204	308
	December	2.3	4.0	0	71	36	87	126	10.5	15.1	1.43	208	68	184	278
1963	January	-0.1	0.7	0	61	31	75	110	10.2	14.2	1.405	188	63	167	250
	February	-0.6	0	0	60	30	74	106	10.0	14.0	1.4	183	62	163	244
	March	5.8	10.0	0	93	44	112	161	11.4	17.0	1.49	244	78	220	332
	April	8.0	14.0	10.14	112	52	132	192	11.9	18.2	1.53	276	84	247	369
	May	11.2	18.7	11.43	140	63	164	236	12.9	20.2	1.58	316	97	284	425

Table 8.2 contd

Year	Month	Temperature (^oC) Air	Temperature (^oC) Design	Rate of Rutting mm×10^{-6} per std axle	Bottom of Asphalt ε_T (microstrain)	Bottom of Asphalt Invariants 1st	Bottom of Asphalt Invariants 2nd	Bottom of Asphalt $\frac{\varepsilon_T+\varepsilon_R}{2} - \varepsilon_V$	Sub-base p (kN/m^2)	Sub-base q (kN/m^2)	Sub-base q/p	Subgrade ε_V (microstrain)	Subgrade Invariants 1st	Subgrade Invariants 2nd	Subgrade $\frac{\varepsilon_T+\varepsilon_R}{2} - \varepsilon_V$
1960	June	15.7	23.6	14.0	116	58	146	213	11.5	16.9	1.5	257	78.5	231	341
	July	15.1	23.0	12.84	112	56	140	205	11.4	16.7	1.475	273	81.5	246	360
	August	14.9	21.5	11.55	102	51	128	186	11.0	16.0	1.45	256	78.0	230	340
	September	13.2	18.0	8.73	82	42	101	150	10.4	14.6	1.398	222	69.5	198	299
	October	10.3	13.2	6.82	63	33	80	122	9.9	13.1	1.327	178	62.0	166	252
	November	7.2	9.2	7.33	65	36	89	132	9.1	7.86	1.06	213	80.0	186	279
	December	4.2	6.2	9.11	58	31	79	116	8.9	8.78	1.08	193	75.0	168	249
1961	January	3.5	5.9	0.77	47	24	60	91	9.3	11.4	1.28	145	50.0	128	192
	February	6.7	11.8	0	48	25	62	94	9.8	12.8	1.305	178	59.0	157	237.5
	March	9.3	16.2	6.911	74	38	91	138	10.1	14.0	1.369	208	66.0	185	279
	April	9.6	16.6	10.71	76	39	103	141	10.2	14.1	1.372	211	67.0	188	283
	May	10.5	17.8	13.65	81	42	99	148	10.4	14.5	1.395	220	69.0	196	296
	June	14.4	22.5	16.61	108	54	132	198	11.2	16.5	1.465	268	81.5	242	353
	July	14.8	22.7	16.5	110	55	138	202	11.3	16.6	1.47	270	82.0	243.5	356
	August	15.4	22.6	19.8	109	55	137	200	11.3	16.6	1.47	269	82.0	243	355
	September	15.0	21.7	6.68	103	52	130	189	11.0	16.1	1.455	259	79.0	233	343
	October	10.9	14.0	0	66	35	82	126	10.0	13.4	1.339	193	62.5	170	257
	November	6.4	8.3	0	51	27	66	101	9.4	11.8	1.255	157	54.0	139	208
	December	2.7	4.5	0	44	23	57	86	9.1	11.0	1.201	138	48.1	122	181

Table 8.3 Regression analysis data, section 52

/contd

Year	Month	Temperature (°C)		Rate of Rutting mm×10⁻⁶ per std axle	Bottom of Asphalt				Sub-base			Subgrade			
		Air	Design		ε_T	Invariants 1st	Invariants 2nd	$\frac{\varepsilon_T+\varepsilon_R}{2} - \varepsilon_V$	p	q	q/p	ε_V	Invariants 1st	Invariants 2nd	$\frac{\varepsilon_T+\varepsilon_R}{2} - \varepsilon_V$
					(microstrain)				(kN/m²)			(microstrain)			
1962	January	4.4	7.7	0	50	26	64	98	9.4	11.7	1.33	154.5	53	136.5	204
	February	4.4	7.7	0	50	26	64	98	9.4	11.7	1.33	154.5	53	136	205
	March	2.6	4.5	0	44	23	57	86	9.1	11.0	1.201	138	48.1	122	181
	April	7.4	13.0	0	63	34	84	122	9.9	13.1	1.321	185	60.5	163	246
1963	January	-0.1	0.7	5.48	50	26	63	92	8.5	7.85	0.96	160	63	139	206
	March	5.7	10.0	10.88	46	38	90	136	9.2	9.47	1.06	220	82	191	286
	April	8.0	14.0	1.35	66	35	84	127	10.0	13.4	1.339	193	62.5	170	257

Table 8.3 contd

Year	Month	Temperature (°C)		Rate of Rutting mm×10^{-6} per std axle	Bottom of Asphalt				Sub-base			Subgrade			
		Air	Design		ε_T	Invariants 1st	Invariants 2nd	$\frac{\varepsilon_T+\varepsilon_R}{2} - \varepsilon_V$	p	q	q/p	ε_V	Invariants 1st	Invariants 2nd	$\frac{\varepsilon_T+\varepsilon_R}{2} - \varepsilon_V$
					(microstrain)				(kN/m²)			(microstrain)			
1959	November	7.0	9.0	6.3	97	53	123	182	9.8	10.43	1.02	270	113	228	343
	December	5.9	7.8	7.84	91	50	116	173	9.7	10.22	1.02	259	110	220	330
1960	January	4.2	7.0	5.82	89	49	126	168	9.7	10.08	1.02	253	108	215	322
	February	3.9	6.5	4.96	86	47	110	164	9.5	9.94	1.02	250	102	212	319
	March	5.9	10.5	4.92	82	42	100	148	11.0	15.3	1.4	218	71	194	292
	April	8.8	15.4	8.17	101	50	124	181	11.7	16.75	1.425	255	79.5	229	345
	October	10.3	13.2	2.37	92	46	113	166	11.4	16.1	1.405	239	76	214	319
	November	7.2	9.2	7.85	99	54	124	185	10.0	10.5	1.02	271	114	229	344
1961	February	6.7	11.8	6.17	87	44	106	156	11.2	15.7	1.405	228	73	202	304
	March	9.3	16.2	6.91	106	52	129	190	11.9	17.1	1.435	262	82	236	354
	April	9.6	16.6	6.88	108	53	132	193	11.95	17.2	1.44	264	82.5	238	357
	May	10.5	17.8	11.38	115	56	140	206	12.1	17.6	1.453	270	86	244	367
	June	14.4	22.5	14.35	154	74	184	265	13.2	19.8	1.505	317	98	284	426
	July	14.8	22.7	14.25	156	75	186	268	13.25	2.0	1.507	321	98.5	288	430
	November	6.4	8.3	0	74	38	91	135	10.5	14.8	1.4	204	68	182	272
	December	2.7	4.5	0	64	32	78	116	10.0	13.7	1.38	183	61	164	245

Table 8.4 Regression analysis data, section 54

/contd

Year	Month	Temperature (^oC)		Rate of Rutting mm×10^{-6} per std axle	Bottom of Asphalt				Sub-base			Subgrade			
		Air	Design		ε_T	Invariants 1st	Invariants 2nd	$\frac{\varepsilon_T+\varepsilon_R}{2} - \varepsilon_V$	p	q	q/p	ε_V	Invariants 1st	Invariants 2nd	$\frac{\varepsilon_T+\varepsilon_R}{2} - \varepsilon_V$
					(microstrain)				(kN/m^2)			(microstrain)			
1962	January	4.4	7.7	0	72	37	89	132	10.5	12.0	1.4	201	67	179	268
	February	4.4	7.7	1.46	72	37	89	132	10.5	12.0	1.4	201	67	179	268
	March	2.6	4.5	2.9	81	43	102	152				236	101	202	301
	April	7.4	13.0	2.16	92	45	112	164	11.4	13.5	1.4	226	73	201	301
	September	12.7	17.0	11.88	110	54	134	197	12.0	17.3	1.44	266	84	240	360
	October	10.4	13.4	4.17	88	44	108	158	11.4	16.1	1.405	240	76	214	321
	November	5.7	7.6	3.12	72	37	88	131	10.6	14.9	1.4	200	66	178	267
	December	2.3	4.0	3.87	79	42	100	148				232	99	199	296
1963	March	5.7	10.0	0.68	81	41	98	146	10.9	15.2	1.4	215	70	191	288

Table 8.4 contd

Year	Month	Temperature (^oC) Air	Temperature (^oC) Design	Rate of Rutting mm×10^{-6} per std axle	Bottom of Asphalt ϵ_T (microstrain)	Bottom of Asphalt Invariants 1st	Bottom of Asphalt Invariants 2nd	Bottom of Asphalt $\frac{\epsilon_T+\epsilon_R}{2} - \epsilon_V$	Base p (kN/m^2)	Base q (kN/m^2)	Base q/p	Sub-base p (kN/m^2)	Sub-base q (kN/m^2)	Sub-base q/p	Subgrade ϵ_V (microstrain)	Subgrade Invariants 1st	Subgrade Invariants 2nd	Subgrade $\frac{\epsilon_T+\epsilon_R}{2} - \epsilon_V$
1960	January	4.2	7.0	0	202	84	220	307	18.0	31.7	1.746	14.7	26.9	1.816	432	119	385	589
	February	3.9	6.5	0	198	83	216	301	17.9	31.4	1.739	14.6	26.7	1.814	428	118	381	583
	March	5.9	10.5	0	236	95.5	254	352	18.9	34.2	1.796	15.5	28.4	1.831	465	127	412	632
	April	8.8	15.4	4.9	297	115	312	423	20.7	38.8	1.857	16.5	30.6	1.853	513	138	459	698.5
	May	12.4	20.2	14.61	374	136	388	512	22.9	42.4	1.900	17.5	33.0	1.879	562	151	512	763
	June	15.7	23.6	14.53	441	153	456	585	24.1	46.2	1.924	18.5	35.0	1.898	597	162	548	818
	July	15.1	23.0	14.44	432	151	448	576	23.8	45.5	1.921	18.3	34.7	1.895	584	167	544	812
	August	14.9	21.5	17.33	399	143	413	540	23.0	43.8	1.911	17.9	33.75	1.887	589	155	520	779
	September	13.2	18.0	14.28	334	126	349	466	21.5	40.4	1.882	17.0	31.9	1.867	536	144	484	731.5
	October	10.3	13.2	16.32	268	106	289	389	19.9	36.4	1.832	16.1	29.7	1.843	488	138	433	663
	November	7.2	9.2	3.14	223	91	241	335	18.6	33.2	1.776	15.2	27.9	1.826	453	124	401	616
	December	4.2	6.2	3.91	195	82	214	298	17.7	31.2	1.734	14.6	26.6	1.813	426	117	379	579
1961	January	3.5	5.9	3.11	192	80	210	294	17.6	31.0	1.731	14.5	26.4	1.812	423	117	377	576
	February	6.7	11.8	3.09	250	100	268	368	19.4	35.2	1.813	15.7	29.1	1.837	478	129	423	649
	March	9.3	16.2	3.07	308	118	323	436	20.9	38.9	1.865	16.6	31.0	1.857	520	140	467	709
	April	9.6	16.6	3.06	313	120	328	442	21.1	39.2	1.868	16.7	31.1	1.858	524	141	470	714
	May	10.5	17.8	15.16	329	125	345	462	21.4	40.2	1.880	17.0	31.8	1.866	536	484	144	732
	June	14.4	22.5	19.63	421	148	436	564	23.5	44.9	1.918	18.3	34.6	1.895	591	158	542	808
	July	14.8	22.7	19.51	427	150	442	570	23.6	45.1	1.919	18.4	34.8	1.896	593	159	544	812

Table 8.5 Regression analysis data, section 38

/contd

Year	Month	Temperature (°C)		Rate of Rutting mm×10^{-6} per std axle	Bottom of Asphalt				Base			Sub-base			Subgrade			
		Air	Design		ε_T	Invariants 1st	Invariants 2nd	$\frac{\varepsilon_T+\varepsilon_R}{2} - \varepsilon_V$	p	q	q/p	p	q	q/p	ε_V	Invariants 1st	Invariants 2nd	$\frac{\varepsilon_T+\varepsilon_R}{2} - \varepsilon_V$
					(microstrain)				(kN/m^2)			(kN/m^2)			(microstrain)			
1961	August	15.4	22.6	23.4	424	149	439	567	23.5	45.0	1.918	18.35	34.7	1.895	592	158.5	543	810
	September	15.0	21.7	1.48	404	144	419	546	23.1	44.0	1.912	17.9	33.9	1.888	581	155	532	795
	October	10.9	14.0	2.22	277	109	293	399	20.2	37.2	1.841	16.2	30.0	1.846	499	135	444	678
	November	6.4	8.3	2.20	215	88	233	324	18.3	32.5	1.764	14.9	27.4	1.822	445	124	394	605
	December	2.7	4.5	2.74	181	77	199	279	15.0	30.1	1.711	14.25	25.9	1.860	412	114	368	560
1962	January	4.4	7.7	2.18	208	86	227	316	18.2	32.2	1.756	14.9	27.2	1.820	439	121	390	598
	February	4.4	7.7	2.17	208	86	227	316	18.2	32.2	1.756	14.9	27.2	1.820	439	121	390	598
	March	2.6	4.5	2.16	181	77	199	279	15.0	30.1	1.711	14.25	25.9	1.860	412	114	368	560
	April	7.4	13.0	2.15	265	195	281	386	19.8	36.2	1.828	16.0	29.6	1.841	489	132	434	665
	May	9.8	16.9	2.14	316	121	332	446	21.2	39.5	1.872	16.8	31.4	1.861	527	474	141	700

Table 8.5 contd

Year	Month	Temperature (^oC)		Rate of Rutting mm×10^{-6} per std axle	Bottom of Asphalt				Base			Sub-base			Subgrade			
		Air	Design		ε_T	Invariants 1st	Invariants 2nd	$\frac{\varepsilon_T+\varepsilon_R}{2} - \varepsilon_V$	p	q	q/p	p	q	q/p	ε_V	Invariants 1st	Invariants 2nd	$\frac{\varepsilon_T+\varepsilon_R}{2} - \varepsilon_V$
					(microstrain)				(kN/m^2)			(kN/m^2)			(microstrain)			
1960	January	4.2	7.0	12.5	202	85	223	310	19.4	32.2	1.772	15.6	28.6	1.8275	488	144	440	660
	February	3.9	6.5	9.09	197	84	218	304	19.2	33.4	1.766	15.5	28.4	1.825	483	142	436	653
	March	5.9	10.5	9.03	238	96	258	354	20.4	37.0	1.805	16.5	30.4	1.843	527	153	474	713
	April	8.8	15.4	8.99	301	115	317	430	22.4	41.4	1.843	18.0	33.3	1.867	584	169	527	793
	October	10.3	13.2	8.69	269	106	288	392	21.5	39.4	1.827	17.4	31.75	1.861	559	161	503	756
	November	7.2	9.2	8.63	223	92	243	336	20.0	36.0	1.793	16.2	29.6	1.837	513	150	462	694
	December	4.2	6.2	10.74	194	82	215	299	19.1	33.5	1.762	15.5	28.3	1.825	480	141	433	650
1961	January	3.5	5.9	8.54	192	81	213	297	18.8	32.5	1.746	15.4	28.2	1.823	477	140.5	429	644
	February	6.7	11.8	8.48	253	101	273	373	21.0	38.2	1.816	16.9	31.0	1.849	541	157	487	732
	March	9.3	16.2	8.45	314	119	329	444	22.7	42.1	1.848	18.2	33.7	1.870	595	171.5	537	808
	April	9.6	16.6	8.41	319	120	334	450	22.9	42.5	1.851	18.4	33.9	1.873	600	173	542	814
	May	10.5	17.8	21.23	336	126	351	470	23.5	43.7	1.858	18.6	34.7	1.879	615	177	555	834
	June	14.4	22.5	23.41	419	150	434	566	26.0	49.3	1.888	20.3	38.4	1.904	684	195	618	929
	July	14.8	22.7	23.25	424	151	439	571	26.2	49.6	1.891	20.4	38.6	1.905	688	196	621	935
	August	15.4	22.6	27.9	421	150	436	568	26.1	49.4	1.889	20.3	38.5	1.904	686	195	619	932
	September	15.0	21.7	22.27	404	146	420	549	25.6	48.2	1.882	20.0	37.7	1.899	673	192	606	912
	October	10.9	14.0	8.13	281	109	299	406	20.8	37.9	1.813	17.6	32.3	1.860	568	164	512	769
	November	6.4	8.3	8.08	214	89	234	313	19.8	35.3	1.785	16.0	29.2	1.834	503	147	453	680
	December	2.7	4.5	10.06	181	78	201	281	18.7	32.3	1.742	15.1	27.5	1.818	462	137	417	624

Table 8.6 Regression analysis data, section 60

/contd

Year	Month	Temperature (°C)		Rate of Rutting mm×10^{-6} per std axle	Bottom of Asphalt				Base			Sub-base			Subgrade			
		Air	Design		ε_T	Invariants 1st	Invariants 2nd	$\frac{\varepsilon_T+\varepsilon_R}{2} - \varepsilon_V$	p	q	q/p	p	q	q/p	ε_V	Invariants 1st	Invariants 2nd	$\frac{\varepsilon_T+\varepsilon_R}{2} - \varepsilon_V$
					(microstrain)				(kN/m^2)			(kN/m^2)			(microstrain)			
1962	January	4.4	7.7	8.00	209	87	230	319	19.5	34.7	1.779	15.8	28.9	1.83	496	145	448	670
	February	4.4	7.7	7.97	209	87	230	319	19.5	34.7	1.779	15.8	28.9	1.83	496	145	448	670
	March	2.7	4.5	7.21	181	78	201	281	18.7	32.3	1.742	15.1	27.5	1.818	462	137	417	624
	November	5.7	7.6	6.71	208	87	200	280	19.5	39.7	1.779	15.8	28.8	1.83	495	144	446	669
	December	2.3	4.05	4.0	177	76	198	276	18.7	32.3	1.742	15.0	27.2	1.815	457	156	412	617
1963	January	-0.1	0.7	6.65	154	68	173	243	17.5	28.2	1.683	14.3	25.7	1.803	425	127	384	574
	February	-0.6	0	6.62	150	65	168	238	17.3	28.7	1.671	14.2	25.4	1.800	420	125	378	566
	March	5.7	10.0	6.60	233	95	252	347	20.3	36.6	1.800	11.4	30.1	1.841	521	152	469	706
	April	8.0	14.0	6.56	281	109	299	406	21.8	40.1	1.833	17.6	32.3	1.860	568	264	516	769

Table 8.6 contd

This equation accounts for 76.8% of the variance in the data and is statistically significant at all tabulated levels. The standard error in the predictor lies in the range 0.5 to 0.95 and does not vary greatly with magnitude of the predicter. A typical standard error of 0.8 could be applied to this equation.

For unbound base sections:

$$RR = -10.976 + 0.5675\,Z_1 - 0.4719\,Z_2 + 0.0862\,Z_3 \qquad (8.12)$$

where Z_1 = the octahedral shear strain at the bottom of the asphalt layer

Z_2 = the difference between the average horizontal strain and the vertical strain at the bottom of the asphalt layer

Z_3 = the second strain invariant at the top of the subgrade.

This equation accounts for 65.6% of the variance in the data and is statistically significant at all tabulated levels. There is a definite tendency for the standard error of the predicter to increase as the predicted value increases, its mean value is 1.07.

8.3.5 Implementation of the regression equation

It will be noticed that the form of Equations 8.11 and 8.12 would allow the prediction of a negative rate of rutting. Should this occur in a calculation it was decided to set the rate of rutting for that calculation to zero.

The two equations were applied to a wide range of structures designed by the computer program (see Chapter 10). For the majority of these structures, designed for a life of 50 million standard axles, unrealistic predictions of rut depths at the end of the pavement's life were obtained, predictions frequently being in terms of metres. This poor result is probably due to the attempted extrapolation of the regression equations to design calculations, the total traffic on the Alconbury Hill experiment during the period for which data was available being only about 1.3 million standard axles. In addition, the selection of the parameters for the regression analysis could be at fault since the various combinations of stress and strain within the individual layers are not truly independent of each other.

Therefore, it was decided that these regression equations could not be used for rut depth prediction in the ADEM design program.

8.3.6 Development of a simple deformation criterion

Since the attempt to develop a deformation prediction system had not produced a satisfactory result, analyses were undertaken to develop a simple deformation criterion based on allowable subgrade strains.

The structures analysed as for the development of this criterion are shown in Fig. 8.8 and were analysed with both linear and non-linear systems.

The stiffness of the bitumen bound materials in the structures analysed were derived on the assumption that they were of average mix proportions according to the appropriate specification and that the temperature of the material was 15^{o}C.

The results of this study are plotted in Fig. 8.9, together with the line suggested by Brown (41), which was derived on the basis of linear analysis. For pavements with bound bases adoption of non-linear analysis for the unbound sub-base layer causes a small downward change for the criterion, implying that if pavements are designed to Brown's deformation criterion (41) by a system which includes non-linear base characterisation, they will be slightly under-designed.

From Fig. 8.9 it is clearly inappropriate to use Brown's (41) design criterion for pavements with unbound bases. However, a

HRA Base

	1 million standard axles		10 million standard axles		100 million standard axles
(1)	$E_1 = 7746\ MN/m^2$, $h_1 = 145$ mm	(2)	$E_1 = 7589\ MN/m^2$, $h_1 = 220$ mm	(3)	$E_1 = 7395\ MN/m^2$, $h_1 = 315$ mm
	$\nu_1 = 0.42$		$\nu_1 = 0.42$		$\nu_1 = 0.42$
	$E_2 = 75\ MN/m^2$, $h_2 = 330$ mm		$E_2 = 75\ MN/m^2$, $h_2 = 390$ mm		$E_2 = 75\ MN/m^2$, $h_2 = 480$ mm
	$\nu_2 = 0.3$		$\nu_2 = 0.3$		$\nu_2 = 0.3$
	$E_3 = 30\ MN/m^2$		$E_3 = 30\ MN/m^2$		$E_3 = 30\ MN/m^2$
	$\nu_3 = 0.4$		$\nu_3 = 0.3$		$\nu_3 = 0.4$

DBM Base

	1 million standard axles		10 million standard axles		100 million standard axles
(4)	$E_1 = 7811\ MN/m^2$, $h_1 = 155$ mm	(5)	$E_1 = 7610\ MN/m^2$, $h_1 = 240$ mm	(6)	$E_1 = 7312\ MN/m^2$, $h_1 = 370$ mm
	$\nu_1 = 0.42$		$\nu_1 = 0.42$		$\nu_1 = 0.42$
	$E_2 = 75\ MN/m^2$, $h_2 = 330$ mm		$E_2 = 75\ MN/m^2$, $h_2 = 390$ mm		$E_2 = 75\ MN/m^2$, $h_2 = 480$ mm
	$\nu_2 = 0.3$		$\nu_2 = 0.3$		$\nu_2 = 0.3$
	$E_3 = 30\ MN/m^2$		$E_3 = 30\ MN/m^2$		$E_3 = 30\ MN/m^2$
	$\nu_3 = 0.4$		$\nu_3 = 0.4$		$\nu_3 = 0.4$

FIG. 8.8 STRUCTURES ANALYSED FOR THE DEVELOPMENT OF LIMITING SUBGRADE STRAINS

Unbound Base

1 million standard axles		10 million standard axles		100 million standard axles	
Surfacing $E = 7918\ MN/m^2$ $\nu = 0.42$	65 mm	Surfacing $E = 7810\ MN/m^2$ $\nu = 0.42$	115 mm	Surfacing $E = 7609\ MN/m^2$ $\nu = 0.42$	210 mm
Base	150 mm	Base	220 mm	Base	260 mm
Sub-base	315 mm	Sub-base	400 mm	Sub-base	480 mm
Subgrade $E = 30\ MN/m^2$ $\nu = 0.4$		Subgrade $E = 30\ MN/m^2$ $\nu = 0.4$		Subgrade $E = 30\ MN/m^2$ $\nu = 0.4$	

FIG. 8.8 contd

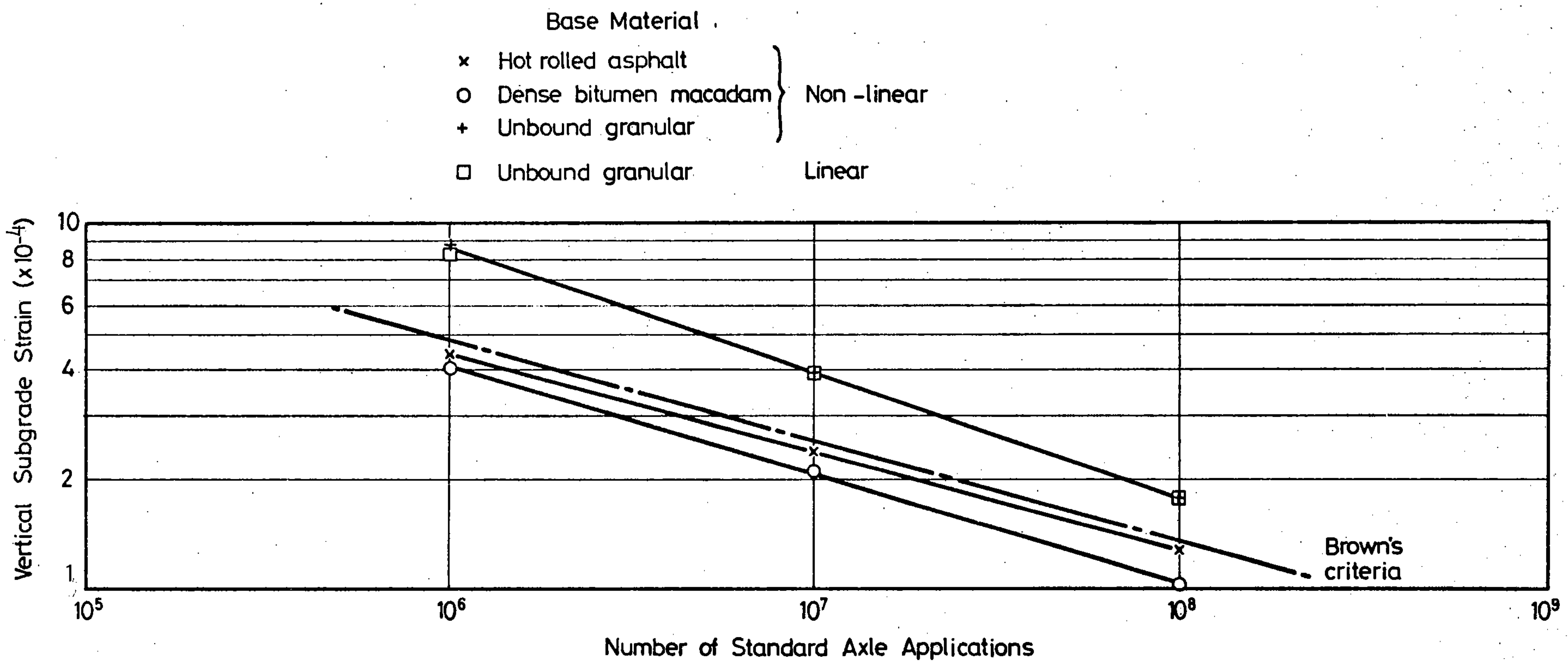


FIG. 8.9 SUBGRADE STRAIN LINES FOR LIMITING PERMANENT DEFORMATION

simplification introduced by Brown (41) was to limit the unbound layer thickness to 200 mm and therefore to effectively consider only pavements with bound bases. In order to extend the design method, it is, strictly speaking, necessary to define the relative layer thicknesses at which a pavement with an unbound base becomes one with a bound base. Further consideration along these lines will indicate that the distinction between the bound and unbound base pavements is based on the contribution of the layer to the overall structure and is dependent on relative stiffnesses (and therefore temperature) as well as thickness. Thus the transition point between one design criterion and the other becomes very difficult to identify.

It is evident that use of a simple deformation criterion based on subgrade strain for design, is becoming progressively less satisfactory as improvements are made in the overall characterisation of highway pavements. It is necessary to stress that empirically derived criteria, such as the simple deformation criterion, are strictly speaking, only valid for the precise conditions of their derivation. It is, however, believed that use of the simple deformation criterion provides realistic designs for the conditions under which it was derived and these are selected specifically for the design of British pavements. Outside of these conditions, the simple deformation criterion provides a basis for the assessment of the probable performance of structural solutions, which is a considerable improvement over the currently available design method. Therefore, the continued use of the simple design criterion is acceptable, provided that the user is fully aware of its limitations and the implications thereof.

Since the line proposed by Brown (41) provides a reasonable average for pavements analysed, assuming simple elastic behaviour for the two bound base pavements it is recommended for incorporation in a design package.

This same design criterion should be used in the design options that incorporate non-linear characterisation of unbound layers to ensure a consistent framework for assessing the effect of changing from linear to non-linear analysis tools on design results.

CHAPTER NINE

TEMPERATURE AND TRAFFIC CONSIDERATIONS FOR PAVEMENT DESIGN

9.1 INTRODUCTION

It is possible to consider temperature and traffic variation in considerable detail when assessing the performance of a pavement. Variation of temperature with both time and depth can be considered and appropriate material properties assigned, also consideration of the full spectrum of axle loads is possible, as well as the distribution with respect to time. If the two factors of temperature and load are combined, a detailed analysis can be undertaken to determine the behaviour of the pavement.

Preliminary investigations with respect to incorporating this level of detailed analysis as part of a cumulative damage procedure into the design package are reported in Appendix A. It became clear from this work that it is inappropriate to attempt to use detailed cumulative damage analysis within the design system because of the inadequacies of the design criteria. These criteria have been derived on the basis of simple analyses, and the application of more detailed traffic and temperature parameters would require a re-evaluation of the design criteria. Since this was not possible within the time available for this work, and would in itself introduce additional problems, the simple approach outlined below has been utilised.

9.2 TRAFFIC

9.2.1 Wheel loading

The current British design procedure (16) converts the axle load spectrum to an equivalent number of standard axles. Since this procedure is both convenient and simple to use, it is recommended for the design method reported in this thesis. It should be noted that the method detailed in Road Note No. 29 (16) has recently been changed (137) and this modified method should be used for design.

The standard axle applies a load of 80 kN to the pavement. A dual wheel configuration was chosen as being representative of commercial vehicles with a tyre contact pressure of 500 kN/m^2. A limitation of the BISTRO (14) program is that loads must be circular, which with due consideration for the total load and contact pressure gives the loaded area a radius of 113 mm.

9.2.2 Wheel spacing

The effect of wheel spacing upon the two design parameters (maximum tensile strain in the bottom of the asphalt, and vertical strain on the subgrade) has been investigated by analysing three-layer structures under a dual wheel standard axle load, the wheel spacing (gap between wheels) being 50, 100, 150 and 200 mm. The three structures considered all had a bituminous layer of stiffness 6,000 MN/m^2 with a layer thickness of 65, 110 and 210 mm for structures 1, 2 and 3 respectively, as shown in Fig. 9.1.

The results of these analyses are plotted in Figs 9.2 and 9.3 as functions of wheel spacing. These graphs indicate that wheel spacing has its most significant effect in structure 1, which is to be expected since this structure contains only 65 mm of bituminous material.

Further consideration of these plots in conjunction with those of critical parameters as functions of layer thickness and stiffness (Appendix B) indicate that the difference in the strain in the asphalt between 50 and 200 mm wheel spacing for structure 1 may be equated with a change in layer thickness of approximately 35 mm. When this is

Structure 1

Thickness	Modulus
65 mm	$E_1 = 6,000$ MN/m^2
150 mm	$E_2 = 75$ MN/m^2
	$E_3 = 30$ MN/m^2

Structure 2

Thickness	Modulus
110 mm	$E_1 = 6,000$ MN/m^2
220 mm	$E_2 = 75$ MN/m^2
	$E_3 = 30$ MN/m^2

Structure 3

Thickness	Modulus
210 mm	$E_1 = 6,000$ MN/m^2
260 mm	$E_2 = 75$ MN/m^2
	$E_3 = 30$ MN/m^2

FIG. 9.1

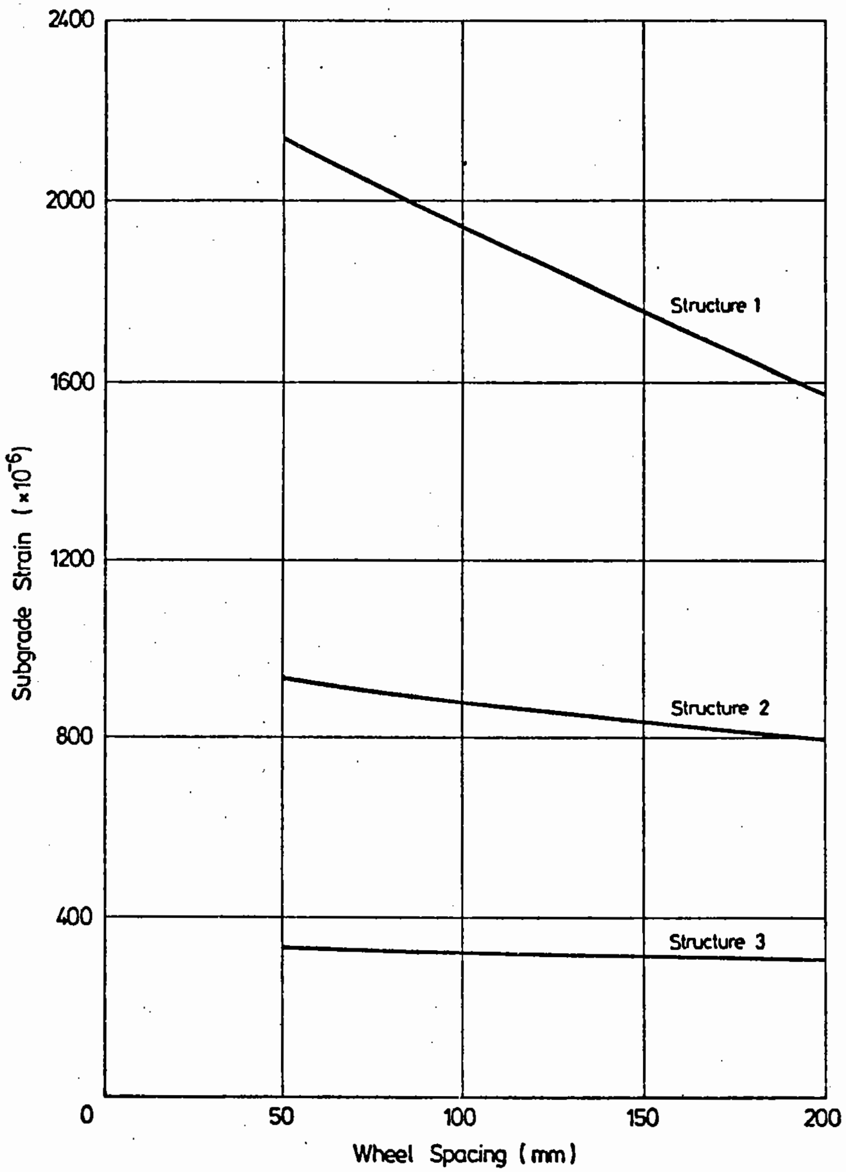


FIG. 9.2 THE EFFECT OF WHEEL SPACING ON SUBGRADE STRAIN

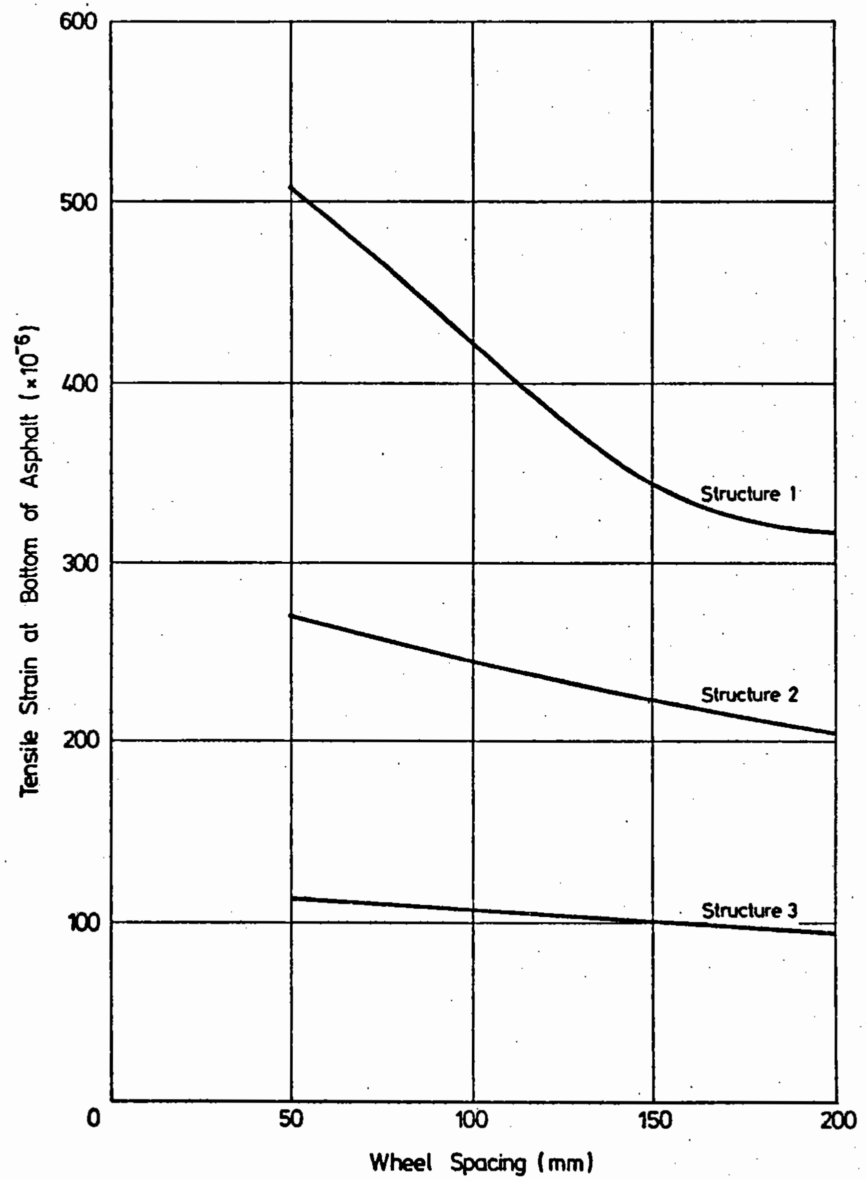


FIG. 9.3 THE EFFECT OF WHEEL SPACING ON ASPHALT STRAIN

considered in relation to the accuracy obtainable from road construction procedures, it is not excessive. The thickness change necessary for the other criteria is in all cases less than 35 mm.

Thus, since wheel spacing does not appear to be highly significant, a spacing of 150 mm was chosen for the design system, since it appeared to relate to commercial vehicles.

9.3 TEMPERATURE

The long term average air temperature for the months June, July, August and September is $15^{o}C$. This temperature was chosen for the calculation of stiffnesses when design criteria were derived, and is therefore recommended for use in the design program to ensure compatibility.

CHAPTER TEN

A COMPUTER PROGRAM FOR PAVEMENT DESIGN

10.1 INTRODUCTION

Several authors (41,44,49,90,91,143) have presented design systems for pavements. The majority of these procedures (49,90,91,143) are in effect checking procedures, the performance of a trial structure being estimated. If this performance was considered unsatisfactory, then a new structure was assumed and tested again. This type of procedure generally involves the use of a computer for part of the checking procedure, but decisions regarding the modification of the structure have to be made by the design agency external to the checking operation. This procedure is both inefficient and time consuming unless the designer has uninterrupted use of a computer, and even then it requires continual re-entry of data.

It was therefore decided to produce a design package such that any required structural modifications were taken within the program itself, so requiring only one entry to the computer and one set of input data. This has been achieved by developing a system for varying the thickness of the bitumen bound base layer in a pavement so that the final structure is neither under- or over-designed.

A further important requirement for the design program was that it should analyse the trial structures as accurately as possible. This leads to the necessity of incorporating an analysis tool into the design program and, as already indicated, the BISTRO program (14) was selected for this purpose.

In order to make the program as attractive as possible to offices regularly engaged in the design of pavements, the input scheme has been designed to use, as far as possible, only the data which is acquired by the current Road Note No. 29 (16) design procedure. Some additional data is, of course, required but this has been tailored to be no more than a designer may reasonably be expected to know. Measured material properties are to be preferred if available, but it has been recognised in the development of the design program that very few designers have the facilities for the determination of dynamic material properties. Thus estimation procedures have been incorporated into the design system.

Since it is currently in vogue to give computer programs names, the program described herein has been named ADEM, after *A*nalytical *DE*sign *M*ethod and as recognition of the fact that it is the first of its kind.

10.2 BASIC ASSUMPTIONS FOR THE PROGRAM

10.2.1 Introduction

In order to develop ADEM, it was necessary to make some assumptions regarding the characterisation of pavements and the design requirements. The need to retain maximum versatility within ADEM conflicts with the requirement to keep the input scheme as simple as possible. Thus, several limitations have been placed on the system as discussed in the following sections.

10.2.2 The pavement structure

In general, British pavements are five-layer structures, consisting of a subgrade, a sub-base, a base, a basecourse and a wearing course. Frequently, the base, basecourse and wearing course are bitumen bound materials with similar properties and can be combined without loss of accuracy.

Thus the ADEM program gives the designer the choice of either five-, four- or three-layer structures. In all cases, the subgrade is overlain by an unbound layer, the upper layers being combined at the discretion of the designer, and in all cases it is the lowest bitumen bound layer in the pavement which is treated as the design layer.

10.2.3 Traffic

The design life of a pavement is expressed as a number of standard axles, and this figure should be used for input to the ADEM program. Estimates for the design life of the pavement should be made using the currently accepted methods (16,137).

10.2.4 Wheel load

The standard axle load is 80 kN. For the purpose of this design program, a dual wheel configuration was chosen with a wheel spacing of 150 mm. It is a requirement of the BISTRO analysis program (14) that the wheel load be circular, therefore a radius of 113 mm was used with a contact stress of 500 kN/m^2.

10.3 MATERIAL CHARACTERISATION

10.3.1 Introduction

Pavements are commonly constructed from bitumen bound materials, cement bound materials, and unbound granular materials. Pavements constructed with pavement quality concrete have not been considered in the development of the ADEM program. A literature review and brief analytical study of cement treated base materials reported in Appendix D indicated that further work was necessary before cement treated bases could be considered as part of the design procedure. Thus ADEM considers only pavements constructed from bitumen bound material on unbound granular material and so this section will be concerned with these materials and the subgrade.

10.3.2 Bitumen bound materials

Ideally, the stiffness of the bitumen bound material should be measured by the designer. Since few laboratories in the UK have facilities for the measurement of material properties, estimation procedures have been included in the design program.

The Shell procedure (3,4,5) has been used to determine the stiffness of the bituminous mix. This procedure had been computerised (23,25) and the program was incorporated in ADEM.

Estimation of stiffness requires the recovered penetration index and recovered softening point of the binder, which should, if possible, be measured. However, if this is not possible, use can be made of estimation procedures to determine these properties. Recovered penetration is assumed to be 65% of the initial penetration (P), and softening point (S.P.) can be estimated from the equation:

$$S.P. = 99.13 - 26.35 \log P \qquad (10.1)$$

It should be noted that this relationship is valid for both initial and recovered softening point derivation and that the result relates to the ASTM method of testing (28), which is appropriate to the bituminous stiffness nomograph (3). Having determined the stiffness of the asphalt mix it is treated as a linear elastic material.

10.3.3 Unbound granular materials

Granular materials can be characterised either by a linear elastic system, or by a stress dependent modulus system, as required by the designer.

The linear elastic system is based on the work of Heukelom and

Klomp (4) and assumes that the stiffness of the unbound layer is 2.5 times the stiffness of the subgrade.

The stress dependent modulus system is based on the work of Hicks (45) in which the stiffness (E) of the material is a function of the sum of the principal stresses (θ):

$$E = K_1 \theta^{K_2} \qquad (10.2)$$

where K_1 and K_2 are material constants.

A literature review indicated that most measured stiffness-stress relationships fell within the bounds set by $K_1 = 600$, $K_2 = 0.6$ and $K_1 = 200$, $K_2 = 0.6$. A K_1 value of 600 may be said to represent very high quality granular material, and $K_1 = 200$ represents poor quality material.

An iterative procedure for calculating the moduli of the unbound material has been incorporated into the ADEM design program. This system, which incorporates failure criterion for the material, is fully described in Chapter 4 of this thesis.

10.3.4 Subgrade

Characterisation of the subgrade is based on the work of Heukelom and Foster (86):

$$E = 10 \times CBR \qquad (10.3)$$

and is linear elastic.

Studies related to the use of stress dependent characterisation indicated that whilst a program for undertaking this type of analysis has been developed, there is insufficient data relating simply determined soil properties such as CBR to a stress dependent model.

10.4 DESIGN CRITERIA

10.4.1 Criteria for repeated loading

Research has shown that bitumen bound materials fracture under conditions of repeated load. The tendency for the material to fracture has been shown (7) to be a function of the applied tensile strain.

A procedure for predicting the fatigue life of British mixes has been published by Cooper and Pell (35), which predicts the laboratory fatigue life of the mix and therefore must be modified before it can be realistically applied to the pavement design system. Studies of the literature and consideration of the performance of typical British pavements (see Chapter 3) have indicated that increasing the predicted fatigue life by a factor of 100 provides realistic lives for use in the design calculation. Thus, the ADEM program incorporates the procedure proposed by Cooper and Pell (35), together with the factor of 100. (See page 20.)

10.4.2 Criteria for deformation

The deformation criterion follows from the early work reported by Peattie (43), and developed by Dormon (112) and Dormon and Metcalf (48). This approach is based on an analysis of pavements which are known to perform satisfactorily, and provides a relationship between the allowable vertical strain on the subgrade and the design life.

Analysis of pavement structures derived from Road Note No. 29 (16), as reported in Chapter 8, produced a strain life line tailored to British conditions.

$$\log \varepsilon_z = -1.67 - 0.28 \log N$$

10.4.3 Location of critical parameters

In order to simplify the pavement analysis as far as possible it was decided to calculate design parameters at critical locations only. Analytical studies of a range of pavement structures indicated that the maximum vertical strain on the subgrade was always on the axis of symmetry of the dual wheel load.

However, the maximum principal tensile strain in the bituminous layer is not always at the bottom of the layer on the axis of symmetry of the wheels. Fortunately, the conditions under which this occurs are rare and so the assumption that the critical strain is at the bottom of the layer on the axis of symmetry is generally true. Fig. 3.2 indicates the limits of validity of the assumption and should be used to check the validity of designs.

10.5 THE ADEM PROGRAM

Fig. 10.1 is a simplified flow diagram of ADEM, indicating the major operations undertaken within the program.

Seven options are available within ADEM, according to the required combination of design criteria.

Linear elastic analysis of the unbound layer is incorporated in Options 1-4 which have the following combinations of design criteria:

Option 1: asphalt fatigue criterion

Option 2: simple deformation criterion

Option 3: selects the most critical condition from fatigue or deformation and designs to satisfy it.

Option 4: selects the most critical condition from fatigue, deformation or sub-base stress* and designs to satisfy it.

Options 5-7 use non-linear elastic characterisation of the unbound layers in the pavement with the following combinations of design criteria:

Option 5: asphalt fatigue criterion

Option 6: simple deformation criterion

Option 7: selects the most critical condition from fatigue and deformation and designs to satisfy it.

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* See page 28.

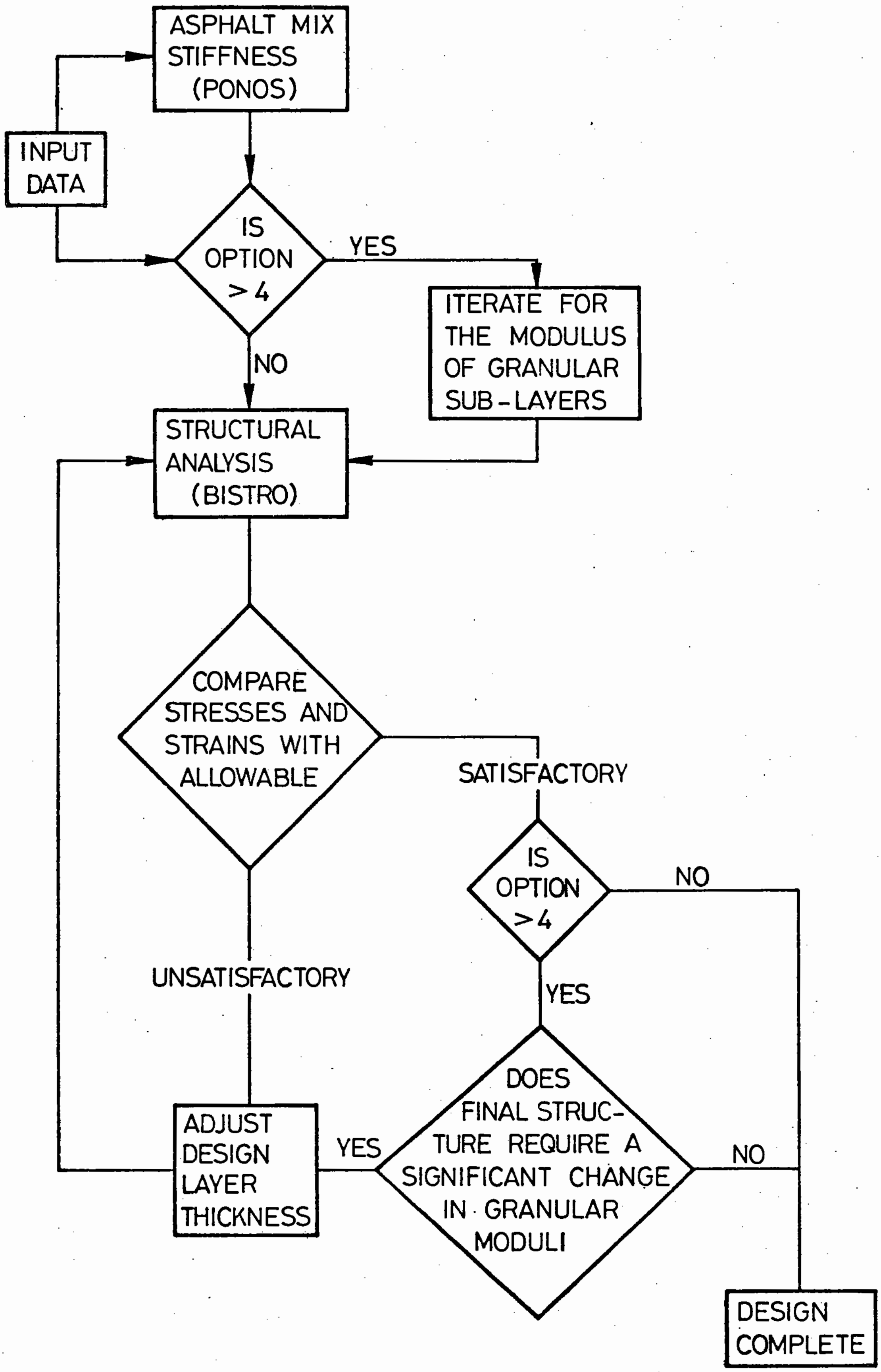

FIG. 10.1 FLOW CHART OF ADEM3

The various forms of material input data and the ability to design three-, four- or five-layer pavement structures are common to all options. Detailed instructions with regard to input requirements for ADEM3 are given in Appendix C which should be regarded as an operating manual for the design program.

One of the unique features of ADEM is the internal adjustment of asphalt layer thickness to satisfy the design criterion. An initial estimate of 160 mm for design thickness is used by the program. This is then checked and, if necessary, the layer thickness is increased or decreased, as required. Current construction procedures are unlikely to produce a finished structure with an asphalt layer thickness closer than 10 mm to that specified, therefore the basic thickness change made by ADEM is 10 mm. However, continual 10 mm steps in thickness adjustment are wasteful and so after 3 steps the average change in design parameter is calculated, then on the assumption of linear change in design parameter with thickness, the number of 10 mm steps necessary to satisfy the design criterion are calculated. Since design parameters and thickness are not linearly related, it is often necessary to further modify the layer thickness and the adjustment procedure consisting of three 10 mm steps followed by a calculated correction is continued until a satisfactory design is obtained.

The conclusion that a design is satisfactory is facilitated by the use of tolerance bands for the design criteria. It is impossible to obtain a layer thickness which precisely matches the design criteria when thickness changes are in terms of 10 mm steps, and in any case, such a requirement is unreasonable, when considered in practical road building terms. Thus a tolerance appropriate to a change in layer thickness of approximately 10 mm for average conditions was set for the design criteria. These tolerances are:

Asphalt strain	±5 microstrain
Sub-base stress	±0.3 kN/m^2
Subgrade strain	±10 microstrain

This in general gives the design thickness limits of ±10 mm on the value derived by ADEM, which is considered to be perfectly adequate for all purposes. Fig. 10.1A shows typical input and output from ADEM.

10.6 VALIDATION OF ADEM

Any engineering design method must be shown to produce a satisfactory solution, or "validated". However, some thought must be given as to the true meaning of validation. Strictly speaking, to validate a pavement design method, a pavement structure must be built for a specific design requirement, perform satisfactorily until that requirement has been met and then fail completely. To be fully satisfactory, the validation exercise should be carried out on several pavements under actual traffic conditions. The obvious problems of time and finance, besides the availability of sites, puts this approach outside the scope of this project. It is, in fact, questionable as to whether this approach is workable at all. Since pavement failure is progressive rather than catastrophic, determination of a failure life is very difficult, and since the time scale of such an experiment is likely to be in terms of several years, it is probable that unforeseen changes will occur with respect to design parameters such as traffic. A pavement designed according to analytical principles could be reassessed with respect to such changes, but careful and detailed monitoring of the pavement would be necessary in order to accomplish this. The result would also be clouded by the inherent variability of road building materials and construction, which could cause one part of a test

TYPICAL INPUT DATA

Initial binder penetration	= 50.0
Binder content	= 4%
Void content	= 6.5%
Binder S.G.	= 1.02
Aggregate S.G.	= 2.70
Mean pavement temperature	= 15°C
Mean traffic speed	= 80 km/hr
Design life	= 20 msa
Subgrade CBR	= 3%
Sub-base thickness	= 200 mm

OUTPUT

DESIGN CRITERIA

Asphalt fatigue strain = 75.1 microstrain
Subgrade strain = 193 microstrain

State of Parameters

Iteration Number	Layer Thickness (mm)	Mix Stiffness (MN/m^2)	Asphalt Strain (microstrain)	Subgrade Strain (microstrain)	
1	250	11739	51.1	178	Design conservative
2	240	11767	54.4	190	Design conservative
3	230	11796	58.1	205	Subgrade strain critical
NSTP = 1					
4	240	11767	54.4	190	

DESIGN COMPLETE

Asphalt layer thickness = 0.2400 metres

↓↓↓↓↓ ↓↓↓↓↓

ASPHALT

Thickness = 0.24 m
Stiffness = 11767 MN/m^2
Poisson's ratio = 0.4

SUB-BASE

Thickness = 0.20 m
Stiffness = 75 MN/m^2
Poisson's ratio = 0.3

SUBGRADE

CBR = 3%
Poisson's ratio = 0.4

FIG. 10.1A TYPICAL INPUT AND OUTPUT FROM ADEM section to fail and another part to perform satisfactorily. It is suggested that responding to the call for validation would lead, at best, to no more than a grudging acceptance of the method. Perhaps a solution to the problem lies in the use of devices such as the heavy vehicle simulator as reported by Walker et al (143) which can provide a rapid assessment of the performance of a structure. However, failure still needs careful definition and the test conditions are artificial, since the full range of climatic conditions will not be encountered during the experiment. It is therefore suggested that whilst validation is frequently called for, the authorities who do this have given little thought to how it may be achieved. To the author's knowledge there is no published validation for the Road Note No. 29 (16) design method.

In fact, Road Note No. 29 (16) is validated by its derivation, which is from full scale road trials. Table 10.1 is a comparison between designs produced by ADEM and designs produced by Road Note No. 29 (16). The agreement is good, there being no more than 20 mm difference between the two methods. Indeed, it would be surprising if there was, since the design criteria incorporated within ADEM are derived on the basis of past experience with British roads, the same information that was used for Road Note No. 29 (16). In addition to using this experience, the literature relating to pavement design and pavement material behaviour has been examined and the usable information contained therein has also been incorporated in ADEM. Thus, past experience has been augmented by published research which itself has been subjected to the critical scrutiny of those engaged in pavement studies. It is therefore suggested that the ADEM design program is at least as valid as the existing design procedure (16).

Acceptance of this point will allow pavement designers to use

ADEM, thereby proving that it produces practical structures. This will allow observation of these structures under traffic conditions, and permit designers to decide as to whether or not the design approach is truly valid.

10.7 INTERPRETATION OF THE RESULTS OBTAINED WITH ADEM

In order to economise on the number of iterations required to complete the design, a system for jumping several thickness change steps has been incorporated in the program as described previously.

Since tolerances are applied to the design criterion, this process may bring the calculated strain anywhere within the tolerance band. For typical British pavements, in which bitumen bound layers rarely exceed 350 mm in thickness, this provides a unique solution to the design, there being only one layer thickness that will give a solution within the tolerance band. However, some input parameters require very thick asphalt layers for a satisfactory design and in this case the change in strain associated with a change of 10 mm in the asphalt thickness is small, and can be less than 1×10^{-6}. Graphs which illustrate this phenomenon have been plotted in Appendix B. Thus, it is possible for a design just inside the lower end of the tolerance band to require perhaps 100 mm less asphalt than one just inside the upper limit of the tolerance band. Hence, the system used for speeding convergence may arrive anywhere within this tolerance band and the design solution may, to some extent, be dependent on the starting point. This could be overcome by producing a design tolerance which decreases as the change in strain for each 10 mm step reduces. This, however, is not truly justifiable since a false sense of accuracy would be presented. Both currently used design criteria are insensitive when thick asphalt layers are considered, which suggests that their use under these conditions is inappropriate.

Base Material	Required thickness of bituminous material (mm)					
	$N = 10^6$ CBR = 5%		$N = 10^7$ CBR = 6%		$N = 5\times10^7$ CBR = 7%	
	ADEM	RN 29	ADEM	RN 29	ADEM	RN 29
Rolled asphalt	200	210	220	220	270	280
DBM (100 pen)	200	220	230	240	310	330

Table 10.1 Comparison between ADEM and RN 29

CHAPTER ELEVEN

THE EFFECT ON DESIGN THICKNESS OF NON-LINEAR CHARACTERISATION OF UNBOUND LAYERS

11.1 INTRODUCTION

This chapter discusses the results of a study to compare designs using linear and non-linear characterisation of the unbound layer in a pavement.

In order to reduce the number of variables in this study, it was undertaken on a three-layer system. Whilst a five-layer system is more truly representative of British pavements, design studies on five-layer pavements introduce four additional variables, the thickness and stiffness of the upper two layers. Since the purpose of this study was to compare linear and non-linear analytical tools in the design process a three-layer system was regarded as adequate. Furthermore, with fewer variables involved, more attention can be given to variables appropriate to the characterisation of unbound layers.

The principal variables chosen for this study were the K_1 material constant in the elastic model (Equation 4.13), the thickness of the unbound layer, the subgrade modulus, binder content, binder type, and compaction of the bituminous mix.

K_1 values of 600, 400 and 200 were selected as these have previously been found to represent the range of unbound granular materials.

Unbound layer thicknesses of 200, 300, 450 and 700 mm were chosen, 300 to 700 mm representing the range in British pavements, and 200 mm representing previous assumptions by Brown (41).

Subgrade moduli of 20, 50 and 80 MN/m^2 were chosen, corresponding to subgrade CBR values of 2, 5 and 8% which represent the range generally encountered in the British Isles.

Binder contents of 3, 4.5 and 6% by weight were chosen as representative of the range allowed within the current British specifications (144, 145) for base mixes with 50 or 100 pen binder. It is appreciated that not all combinations of binder content and grade exist within the specification, but the strength of the design methodology embodied in the ADEM program is that it is not limited to mixes which appear in current specifications.

Recent research (30) indicates that VMA is a good indicator of the state of compaction of an asphalt mix. Hence, VMA values of 16, 19 and 22% have been taken as representative of good, average, and poor compaction respectively. The ADEM program requires void content as an input parameter, hence void contents have been calculated for the appropriate binder contents, the values being shown in Table 11.1.

VMA (%)	Void contents (%) for binder contents of:		
	3	4.5	6
16	9.1	5.5	1.8
19	12.4	10.0	5.3
22	15.6	12.3	8.8

Table 11.1 Void contents for given VMA and binder contents

Whilst the primary purpose of this study was to assess the effect of using a non-linear analysis tool in the design process, and the results are discussed on this basis, it also allowed some general comments to be made on design trends and these are added in the sections below as appropriate. Traffic speed = 80 km/h. Design temperature = 15°C.

11.2 THE EFFECT OF K_1 AND THE THICKNESS OF THE UNBOUND LAYER

Mix properties were held constant for this study, a binder content of 4.5% by weight being chosen and a VMA of 19%. This led to a void content of 10% by volume.

The comparative study was then undertaken for all values of K_1, unbound layer thickness (h_2), subgrade modulus, and both grades of binder.

Figs 11.1 and 11.2 are plots of design thickness as a function of K_1 and h_2 for the 50 pen and 100 pen mixes respectively, indicating design thickness as a function of the granular layer thickness for both linear and non-linear analysis systems. These plots also show the effect of K_1 on design thickness and indicate the change from fatigue to deformation critical design for the various K_1 values. Considering firstly the 50 pen mixes (Fig. 11.1), for low stiffness subgrades there is very little to chose between designs produced by the linear and non-linear systems, except that in the middle of the h_2 range, low quality granular material ($K_1 = 200$) requires slightly less thickness. However, as the subgrade modulus increases, a difference between the two approaches can be detected. For subgrade moduli of 50 and 80 MN/m^2 the linear approach requires increased thickness of asphalt for thin granular layers, and decreased asphalt thickness for thick granular layers. These differences are greater for the stiffer, 80 MN/m^2, subgrade. It is noticeable that linear characterisation of the sub-base indicates that variation in h_2 has an effect on design thickness requirements, but non-linear characterisation suggests that design is independent of h_2.

The results obtained for designs using a 100 pen asphalt mix (Fig. 11.2) show that at low subgrade stiffness, the linear and non-linear approaches produce essentially similar results. However, for the two stiffer subgrades, the difference between the approaches

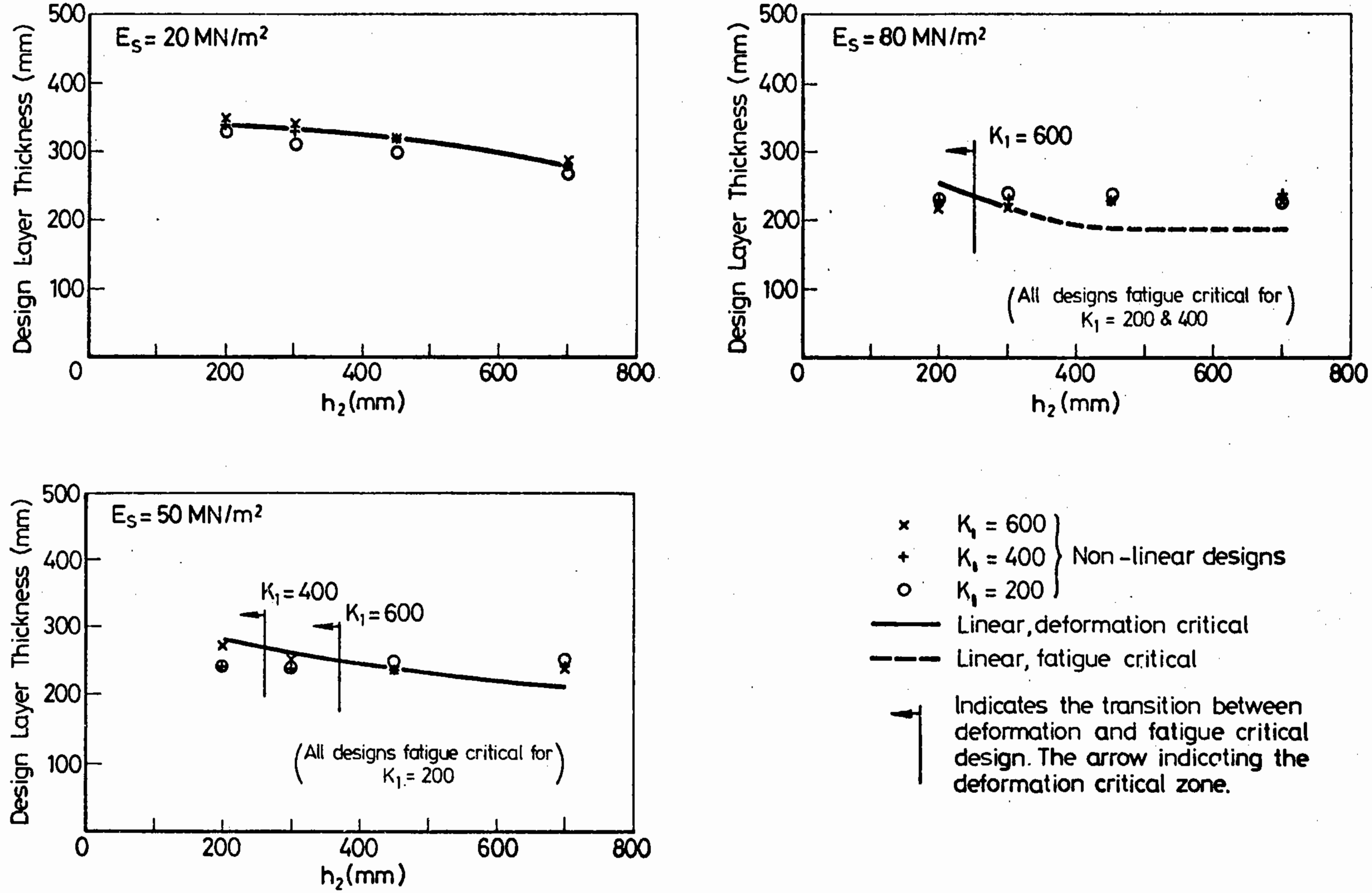

FIG. 11.1 DESIGN THICKNESS AS A FUNCTION OF SUBGRADE MODULUS AND GRANULAR LAYER THICKNESS FOR 50 PEN MIXES

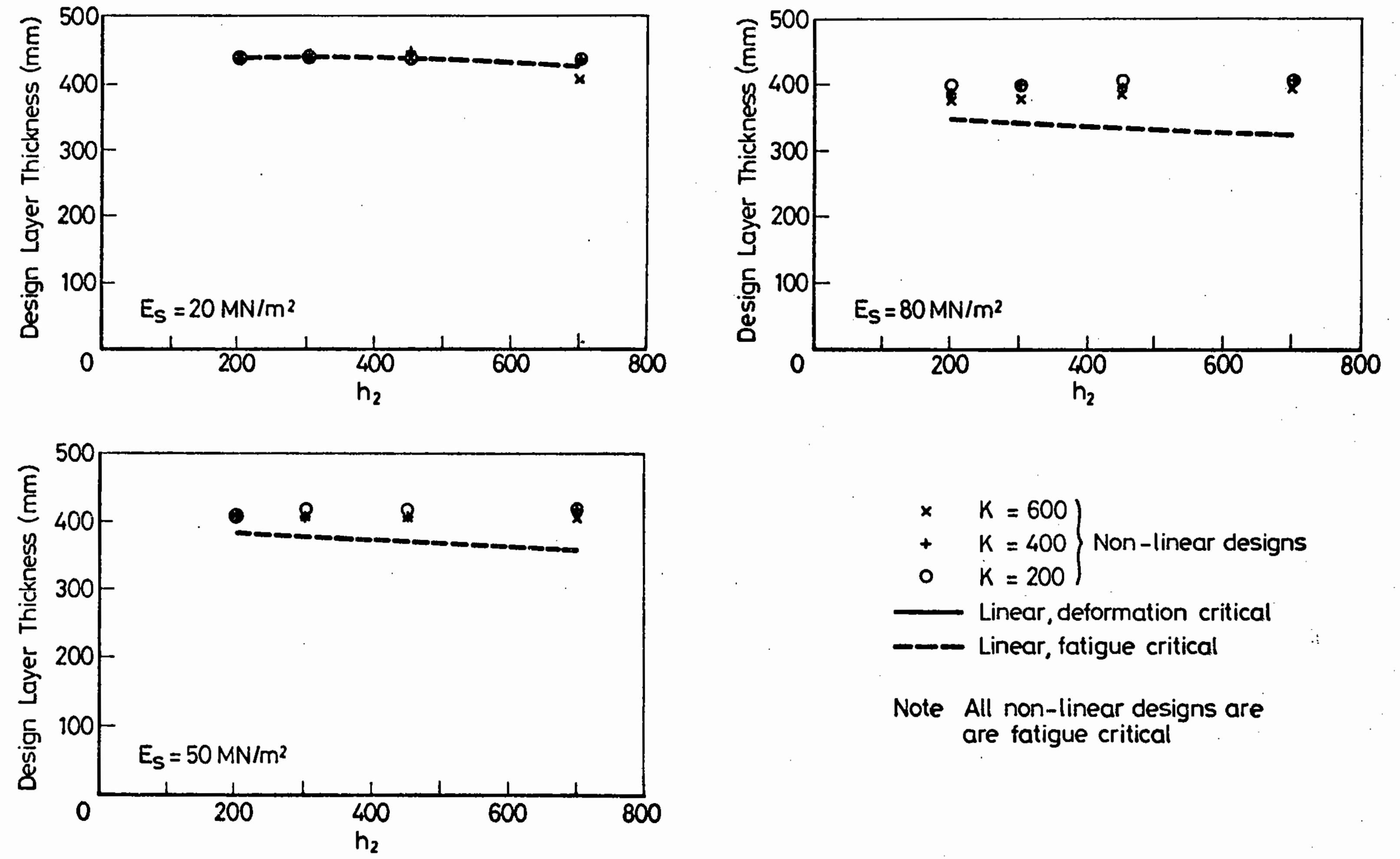

FIG. 11.2 DESIGN THICKNESS AS A FUNCTION OF SUBGRADE MODULUS AND GRANULAR LAYER THICKNESS FOR 100 PEN MIXES

becomes more significant, the linear system always producing reduced thickness requirements. The two approaches indicate opposing effects in relation to increasing h_2, the linear giving a decreasing asphalt thickness requirement and the non-linear giving a barely discernable increasing thickness requirement. This may seem surprising, but references to Fig. 6.7 of Chapter 6 will confirm that for very thick asphalt layers, variation in granular layer thickness has very little effect on the design criteria.

Figs 11.1 and 11.2 show that variation in the quality of granular material, as modelled by varying K_1, has, in general, very little effect on the required design thickness. However, material quality has a definite effect on the point of transition between deformation and fatigue critical designs (Fig. 11.1), reducing material quality (decreasing K_1) increasing the fatigue susceptibility of the pavement. (This is only evident for the designs with 50 pen mixes since for both analysis systems all designs with 100 pen binder are fatigue susceptible.) This indicates that the poorer quality granular material, which is generally not able to develop such a high modulus, provides less support in its upper sub-layers for the asphalt whilst the strain reaching the subgrade is not substantially changed.

11.3 THE EFFECT OF VARYING THE SUBGRADE MODULUS

The results of Figs 11.1 and 11.2 have been replotted as functions of subgrade modulus for each of the four granular layer thicknesses in Figs 11.3 and 11.4. In these figures the non-linear results have been represented as a band, covering the variation caused by K_1 rather than individual points.

For the 50 pen mix and a thin (200 mm) unbound layer, the linear

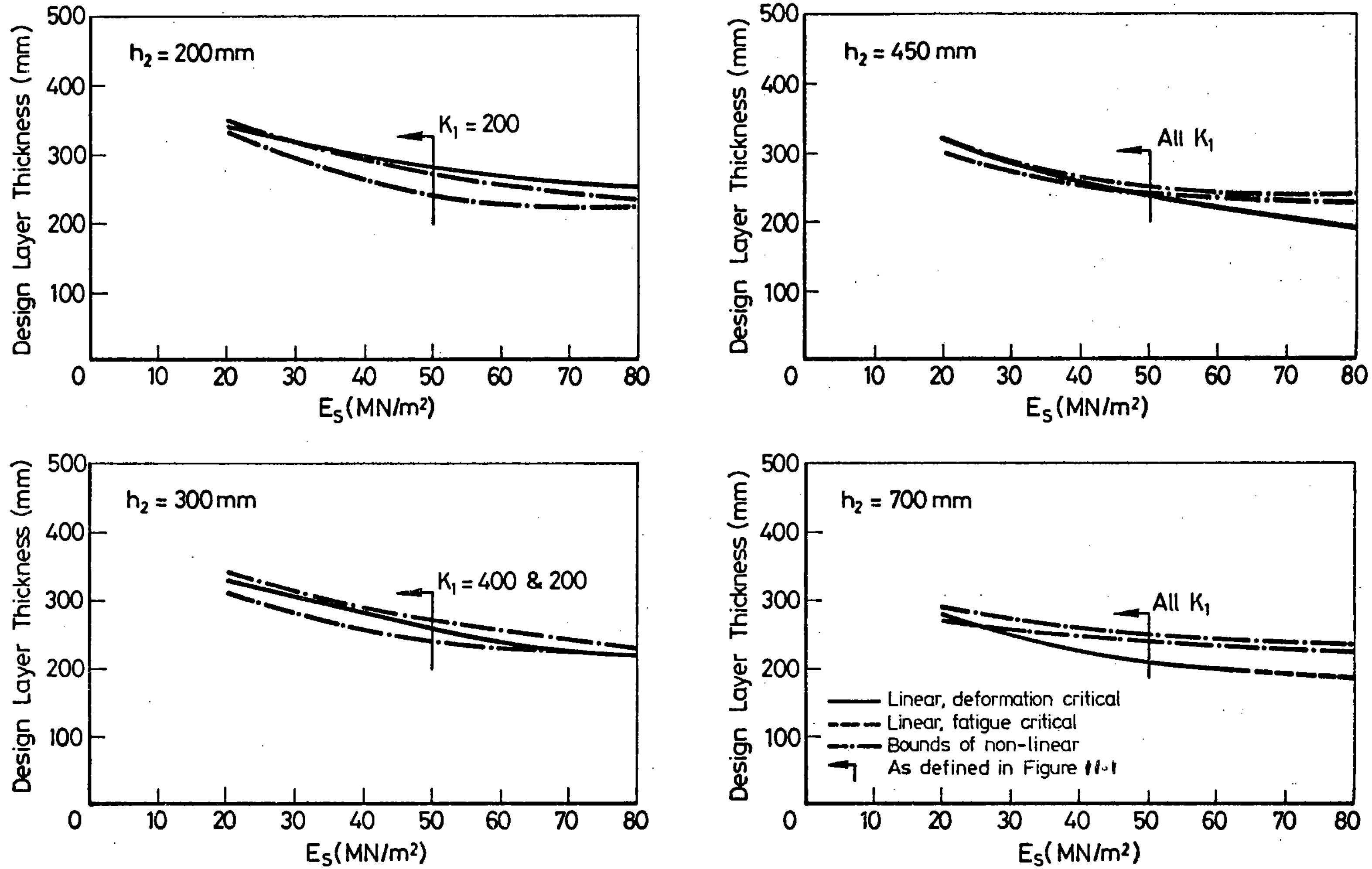

FIG. 11.3 DESIGN THICKNESS AS A FUNCTION OF SUBGRADE MODULUS AND GRANULAR LAYER THICKNESS FOR 50 PEN MIXES

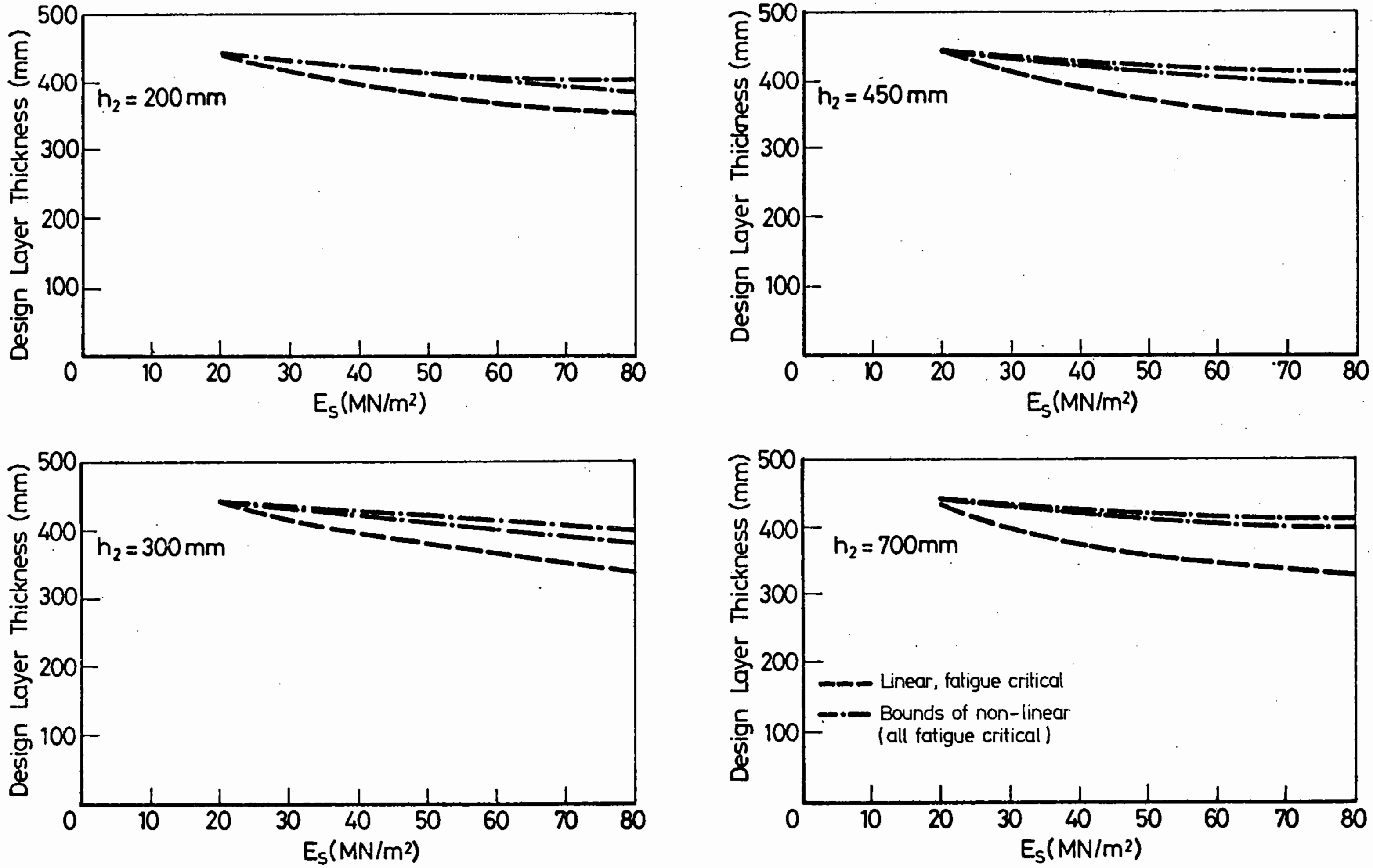

FIG. 11.4 DESIGN THICKNESS AS A FUNCTION OF SUBGRADE MODULUS AND GRANULAR LAYER THICKNESS FOR 100 PEN MIXES

approach gives a greater thickness requirement, indicating that for this condition, which is the one adopted by Brown (41), the error is on the safe side. At an unbound layer thickness of 300 mm, the two systems give similar results and it is only for a subgrade with a high modulus that there is any difference at 450 mm thickness. However, for the 700 mm granular layer and all designs with 100 pen mixes, the non-linear system consistently gives a larger design thickness than the linear system. In addition, the difference between the linear and non-linear systems increases as the stiffness of the subgrade increases.

11.4 THE EFFECT OF ASPHALT COMPACTION

Having decided, from the previous study, that granular material quality has little significant effect on design thickness, the intermediate value of $K_1 = 400$ was chosen for this study, together with the intermediate value of subgrade stiffness, $E_s = 50$ MN/m^2. The variables considered were the full range of binder contents, the full range of VMA values, both grades of bitumen, and the two extremes of unbound layer thickness.

Fig. 11.5 plots these results as a function of VMA at each binder content, for the different binder grades and unbound layer thicknesses.

For the 50 pen mixes on a 200 mm unbound layer at 3% binder content, the non-linear system requires thicker layers. However, at 4½% and 6% binder contents the linear system requires a thicker structure. It will be noticed that for 3% binder content, fatigue is the critical design parameter. The increased thickness requirement for the non-linear system under these conditions is consistent with the observation made in the previous section that it increases the fatigue susceptibility of the pavement.

However, at 4½% binder content, when both analysis systems show a transition from fatigue to deformation critical designs with increasing

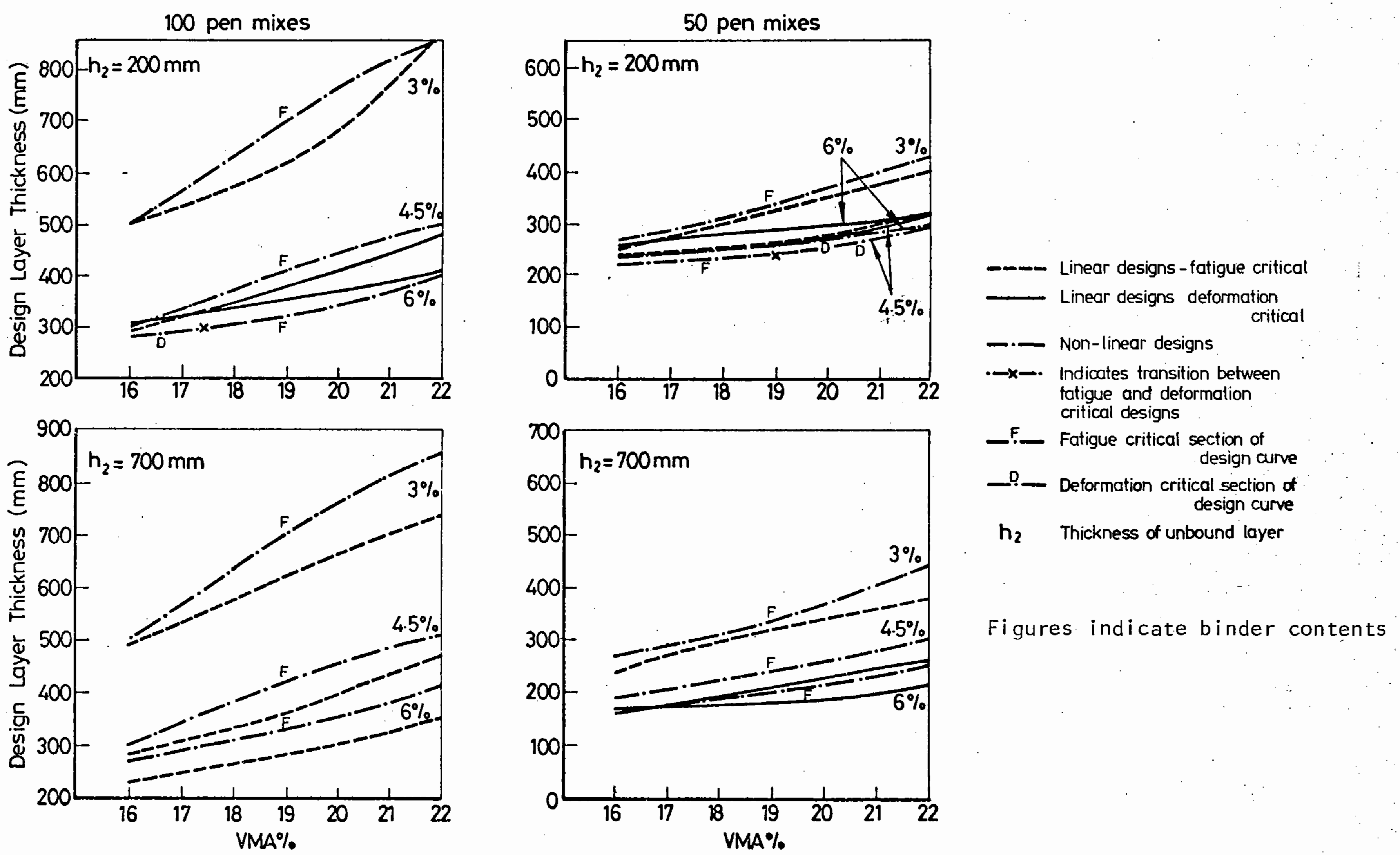

FIG. 11.5 DESIGN THICKNESS AS A FUNCTION OF VMA AND BITUMEN CONTENT FOR BOTH 50 AND 100 PEN MIXES

VMA, the non-linear system produces a reduced design requirement. It should be noted that for this design curve, the linear system produces a balanced design, both fatigue and deformation being critical for VMA's of 19% and 22%, and this may be the reason for the inconsistency in the apparent increase in fatigue susceptibility of designs produced by non-linear analysis.

At 6% binder content in which both systems produce deformation critical designs the thickness requirement for the non-linear system is less than that for the linear system, and is once more consistent with the observed tendency for the non-linear characterisation to produce enhanced fatigue susceptibility and reduced deformation susceptibility in a pavement.

For the 50 pen mixes on a 700 mm thick unbound layer the designs produced by the non-linear system are all fatigue critical whereas those produced by the linear system are only fatigue critical for 3% binder content. However, in all cases, the non-linear system produces the greatest thickness requirement.

For the 100 pen mixes virtually all designs are fatigue critical, with the exception of the well compacted mixes (VMA = 16) at 6% binder content on 200 mm of unbound material. For all cases, the non-linear system gives designs which are thicker than the linear system.

A general trend for non-linear analysis to require thicker asphalt layers than linear analysis is observed if both systems produce fatigue susceptible design curves for the complete range of VMA values at any given binder content. For the one design curve in which both systems give deformation critical designs (50 pen, 6% binder on h_2 = 200 mm) the non-linear system requires a reduced design thickness. However, when a change in design criteria occurs along a design curve, or when the two systems use differing criteria, then no simple pattern is available for indicating which design approach produces thicker asphalt layers.

11.5 THE EFFECT OF BINDER CONTENT

It will be noticed that the design curves for mixes at a specific VMA do not always indicate decreasing design thickness with increasing binder content. This suggests that an optimum binder content for minimum design thickness may exist.

In order to examine this possibility more closely, and to compare linear and non-linear systems as a function of binder content, the results from Fig. 11.5 have been replotted in Fig. 11.6 as a function of binder content.

For the 50 pen mixes, the design curves for an unbound layer thickness of 200 mm shed some light on the relationship between the two methods for design curves in which the critical criterion changes. At low binder content all designs are fatigue critical with the non-linear system indicating thicker layers. As the binder content increases, designs change to deformation critical and curves produced by the different methods cross, until at high binder contents with all designs being deformation critical, the non-linear system indicates a smaller thickness requirement than the linear. It should also be noted that for these conditions, the non-linear system shows greater sensitivity to changes in binder content than the linear system. With regard to optimum binder contents for minimum thickness design, both methods indicate similar values for the poor and average compaction condition, but for good compaction the non-linear system indicates a higher binder content.

Considering the 50 pen mixes on a 700 mm thickness of unbound material, the non-linear system once more produces thicker designs and for VMA's of 19% and 22% the two methods produce essentially parallel curves,

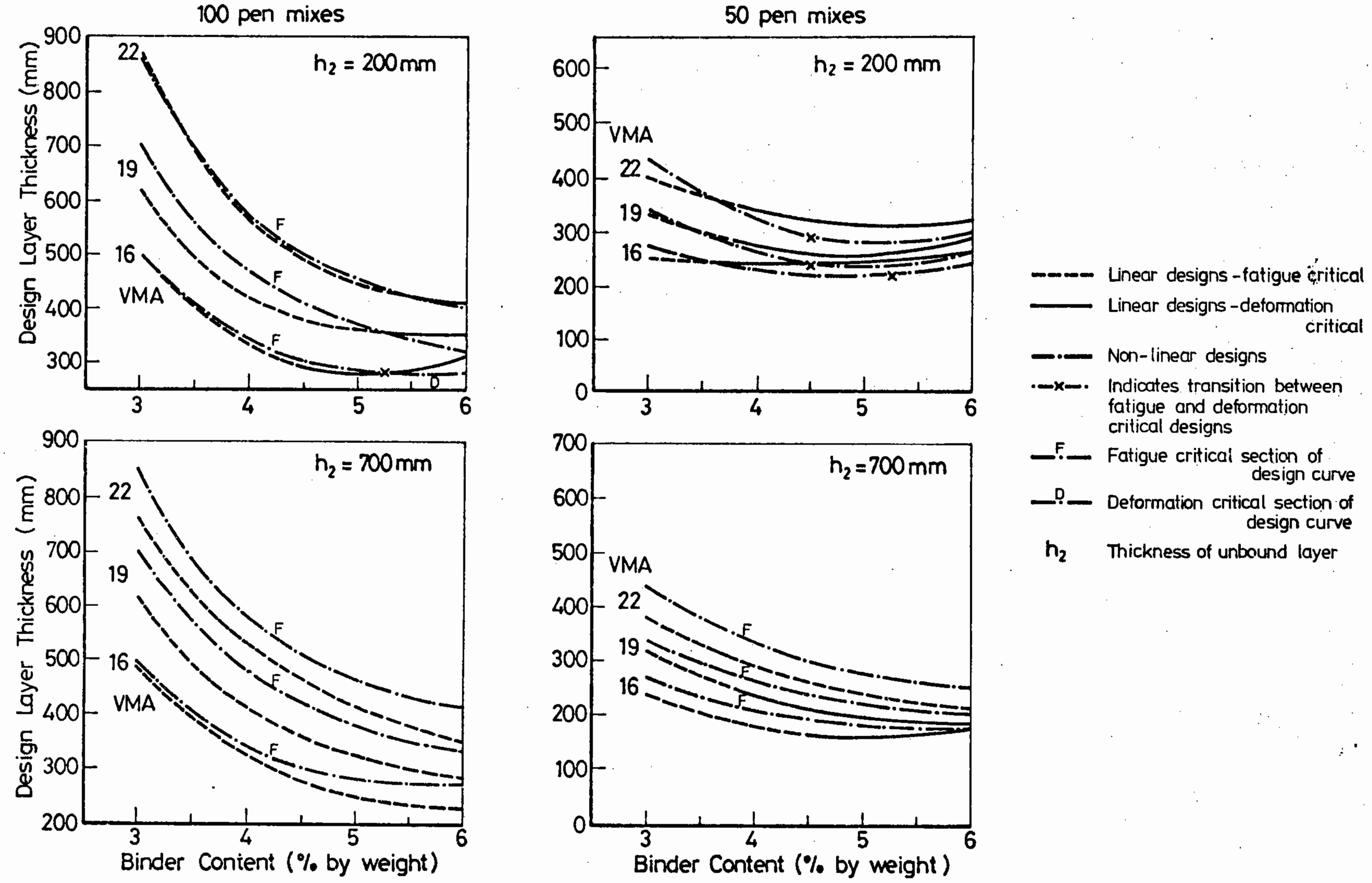

FIG. 11.6 DESIGN THICKNESS AS A FUNCTION OF BINDER CONTENT AND VMA FOR BOTH 50 AND 100 PEN MIXES

despite changes in critical criteria. However, at a VMA of 16% the linear system passes through a minimum whilst the non-linear system continues to decrease.

The 100 pen mix design curves show far greater sensitivity to binder content than the 50 pen curves. For 200 mm bases, the two systems produce similar results for the good and poor compaction conditions, but for the intermediate condition, the non-linear system requires thicker layers. The design curves under these conditions for the two different systems cross, as with the 50 pen, but the point of intersection has changed to the high binder content end of the curve from the low binder content end.

For 100 pen mixes on 700 mm of unbound material in which all the designs are fatigue critical the non-linear system, as usual, requires a greater thickness of asphalt.

11.6 THE EFFECT OF VARYING THE SUBGRADE/SUB-BASE MODULAR RATIO

Since none of the designs so far derived on the basis of linear analysis were designed against the sub-base stress criterion a brief design study was undertaken to compare the behaviour of the linear and non-linear analysis systems when this criterion was the design determinant.

It was found that designs dependent on sub-base stress could be produced by increasing the sub-base to subgrade modular ratio from 2.5 to 3.75. Fig. 11.7 is a plot of the results for a 100 pen mix on a 200 mm unbound layer for the range of binder contents and VMA's used previously. The non-linear system designs plotted are those from Fig. 11.6.

Fig. 11.7 shows that the required design thicknesses increase when the sub-base stress criterion comes into operation, whereas the non-linear

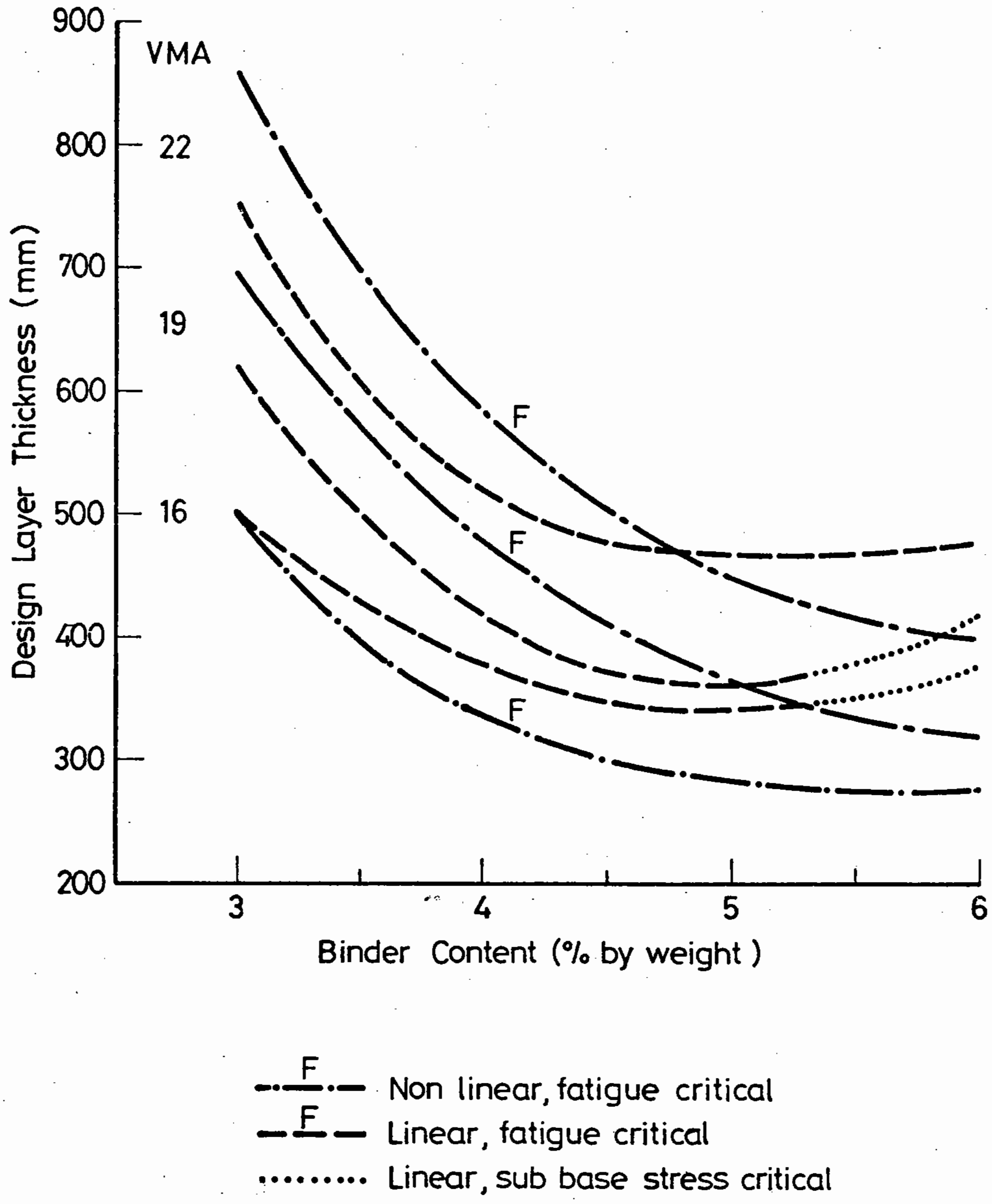

FIG. 11.7 COMPARISON OF LINEAR DESIGNS WHICH INCLUDE SUB-BASE STRESS DESIGN CRITERIA WITH NON-LINEAR DESIGNS

system still indicates decreasing thicknesses. Although this type of trend is shown in Fig. 11.6, the differences between the two systems are very small. Hence, the use of varying modular ratios to model granular material quality in conjunction with linear elastic theory is not recommended.

11.7 GENERAL DISCUSSION OF THE COMPARATIVE STUDY

It has been observed that when both analysis systems produce fatigue critical designs for all values of the control parameter, the non-linear approach always gives a larger layer thickness. The increased thickness requirement varies according to the structure and the asphalt mix properties, the maximum increase recorded being 70 mm on the 340 mm required by the linear analysis. On the other hand, for the one case in which both systems give deformation critical designs for all values of the control parameter, the non-linear approach always gives a smaller layer thickness. These observations have to be considered with due regard for the limitations of the design criteria.

Reference to Fig. 6.8 indicates that for bitumen bound base pavements, use of non-linear analysis reduces the allowable subgrade strain in the pavement and would therefore increase the thickness requirement for any pavement designed to this criterion. However, the difference between these two lines is small, being a maximum of 10×10^{-6} strain at one million standard axles, which is unlikely to result in a design changed by more than 10 mm.

For unbound bases, the subgrade strain-life lines for both analysis systems are the same (Fig. 8.9). Whilst this line was not used due to the difficulty of defining which pavements have unbound bases, it is clear that use of this line would not affect the comparison between linear and non-linear analysis. However, it would affect the relationship between the fatigue and deformation susceptibility of pavements. Since the strain-life line for unbound base pavements is higher than the currently used line, it will have the effect of reducing the required pavement thickness for both systems and also of making designs generally more fatigue susceptible.

The asphalt fatigue criterion is derived from laboratory studies which have had a correction factor applied to account for the difference between laboratory and field performance. This factor is partially based on information from the literature and partially on the empirical requirement that pavements designed to average British conditions are generally not fatigue susceptible. Thus, whilst the change to a non-linear design tool certainly alters the relationship between tensile strain at the bottom of the asphalt layer and vertical strain at the top of the subgrade, its effect on the derivation of the asphalt fatigue criterion is likely to be small.

In general, the two systems predict broadly similar results and trends. The exception is when the subgrade to sub-base modular ratio in the linear system is used in an attempt to model variations in the quality of the unbound granular layer. When this is attempted the sub-base stress criterion becomes the design determinant and appears to give an increasing design thickness requirement which is not mirrored by the non-linear system. Since the non-linear system contains the more accurate method of characterising the granular layer, designs obtained with this system are considered to be the more reliable.

11.8 CONCLUSIONS

1. An improved design system, ADEM3, is available that incorporates non-linear characterisation of the unbound granular layer, as an option.

2. The linear system will give a satisfactory indication of design trends as a function of the variables under the designer's control.
3. If a working design is required, it should be obtained using the non-linear system as somewhat greater thicknesses result from its use and because of its greater accuracy.
4. Any designs obtained using the linear system in which the sub-base stress criterion is the critical parameter should be regarded as unreliable.

CHAPTER TWELVE

PAVEMENT DESIGN STUDIES WITH ADEM

12.1 INTRODUCTION

Several investigations were undertaken with ADEM, the study being to determine the effect of mix variables on the design of a pavement. Three grades of binder were considered, of 50, 100 and 200 penetration. Mixes were studied at binder contents varying from 3% to 6% by mass at void contents (V_V) of 2%, 6% and 10%. Pavement designs for these mixes were produced at lives of 10^6, 10^7 and 10^8 standard axles at a temperature of 15°C. Subsidiary studies at 6% void content were undertaken covering the above range of binder contents, with 100 pen binder at a design life of 10^7 standard axles. These studies included variations in temperature to 10 and 20°C, varying the Poisson's ratio of the asphalt to 0.35 and 0.45, and increasing the subgrade CBR from 3% to 6%.

The results of these studies will be presented and discussed individually in the following sections.

12.2 DESIGNS FOR MIXES WITH 50 PEN BINDER

Fig. 12.1 plots design thickness as a function of binder and void content at design lives (N) of 10^6, 10^7 and 10^8 standard axles. From these graphs, two definite trends appear. Design thickness increases with increasing binder content and with increasing void content, while the rate of increase in design thickness is independent of the void content. There is one departure from the pattern described above, at a design life of 10^8 standard axles, a binder content of 3% and a void content of 10%. It will be noticed that there is no change in layer thickness as the binder content increases from 3% to 4%. It should also

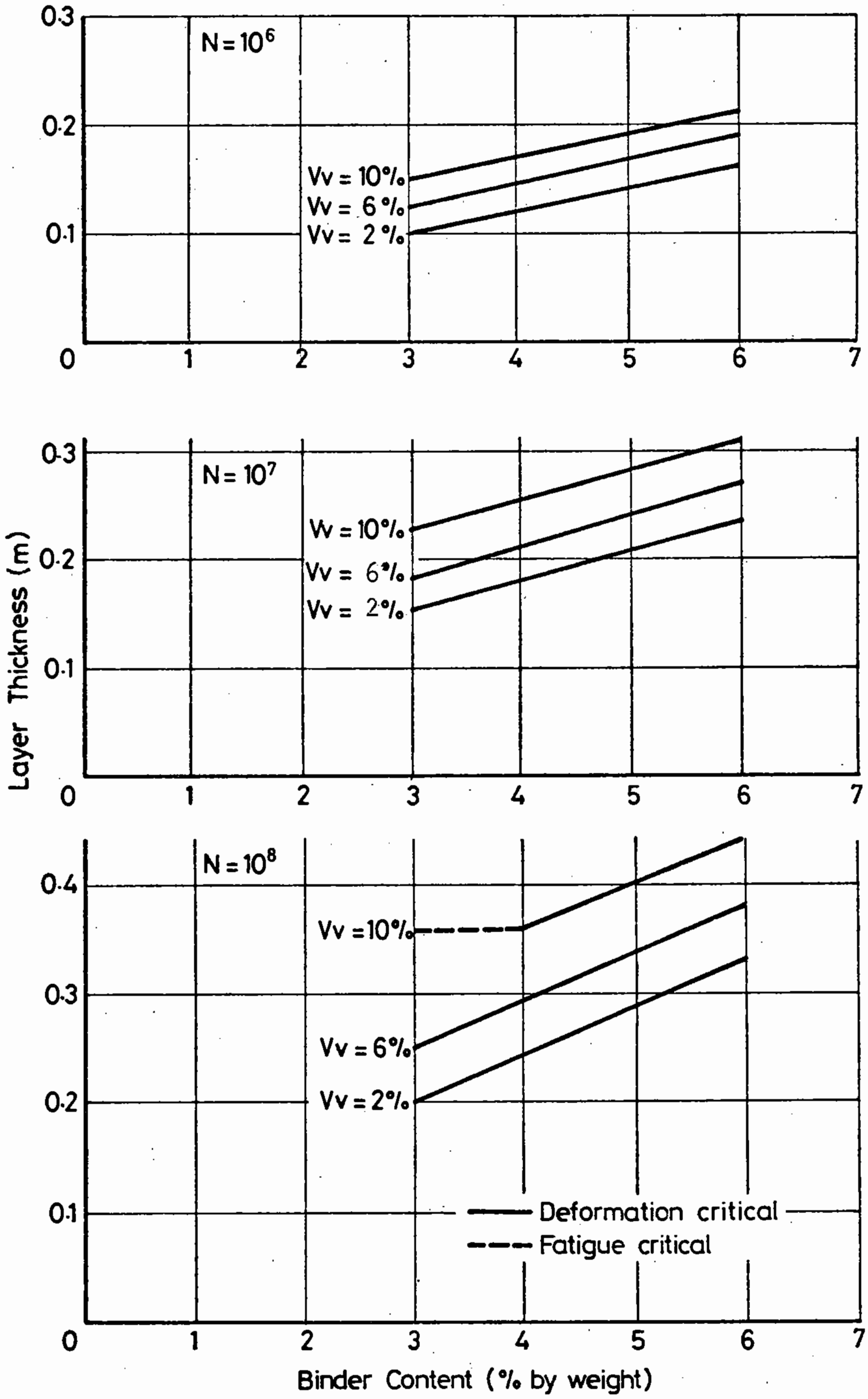

FIG. 12.1 DESIGN THICKNESS AS A FUNCTION OF BINDER CONTENT AND VOID CONTENT FOR 50 PEN MIXES

be noted that the critical design parameter has changed from subgrade strain, which was critical in all the other structures, to the tensile strain in the asphalt layer, i.e. fatigue rather than deformation determines the layer thickness.

Fig. 12.2 is a plot of design thickness as a function of binder content and design life for a void content of 6%. This indicates that increasing the design life causes a higher rate of increase of layer thickness with increasing binder content.

Leech and Powell (30) have suggested that the voids in mineral aggregate (VMA) is a useful parameter for describing a mix, and so design thickness has been plotted as a function of VMA in Fig. 12.3 for each design life. The relationship is largely independent of void content suggesting a close relationship between design thickness and a single mix parameter.

12.3 DESIGNS FOR MIXES WITH 100 PEN BINDER

At a life of 10^6 standard axles and for 2% and 6% void contents, Fig. 12.4 shows a similar pattern of thickness against binder content to that in Fig. 12.1 for 50 pen mixes. However, as mix void content and design life increase, the nature of the relationship changes through initially constant design thickness at void content of 10% and a design life of 10^6 standard axles to initially decreasing design thickness, followed by the familiar increase. Fig. 12.4 also indicates that fatigue has become the critical criterion in many designs and is the critical condition in designs exhibiting a decrease in layer thickness with increasing binder content.

Fig. 12.5, which plots design thickness as a function of binder content and life, indicates that as design life increases, all trends, whether they be to increasing or decreasing thickness, are accentuated.

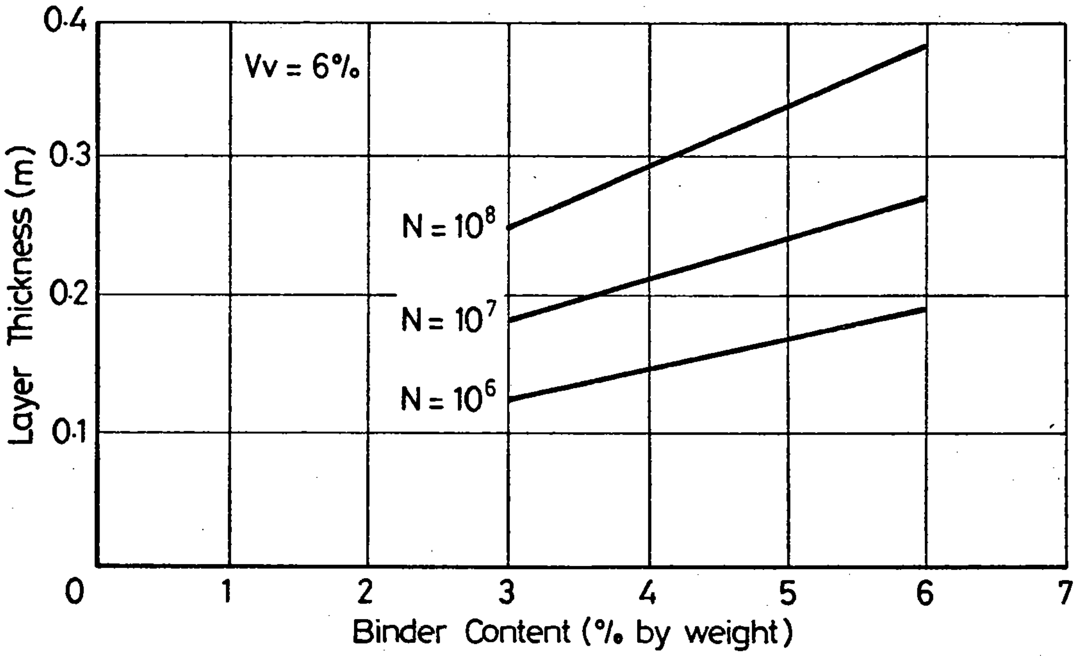

FIG. 12.2 DESIGN THICKNESS AS A FUNCTION OF BINDER CONTENT AND DESIGN LIFE FOR 50 PEN MIXES

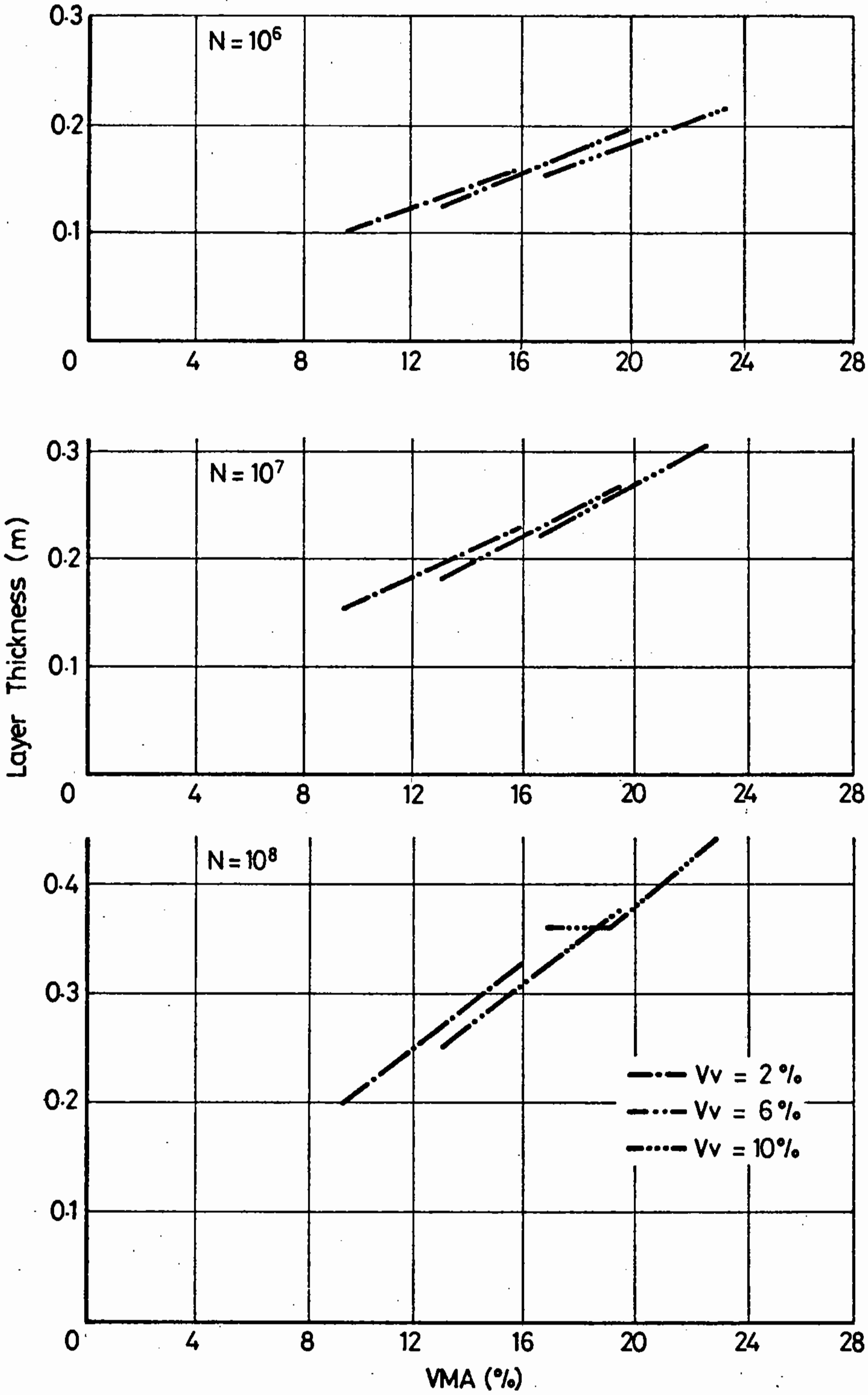

FIG. 12.3 DESIGN THICKNESS AS A FUNCTION OF VOID CONTENT AND VMA FOR 50 PEN MIXES

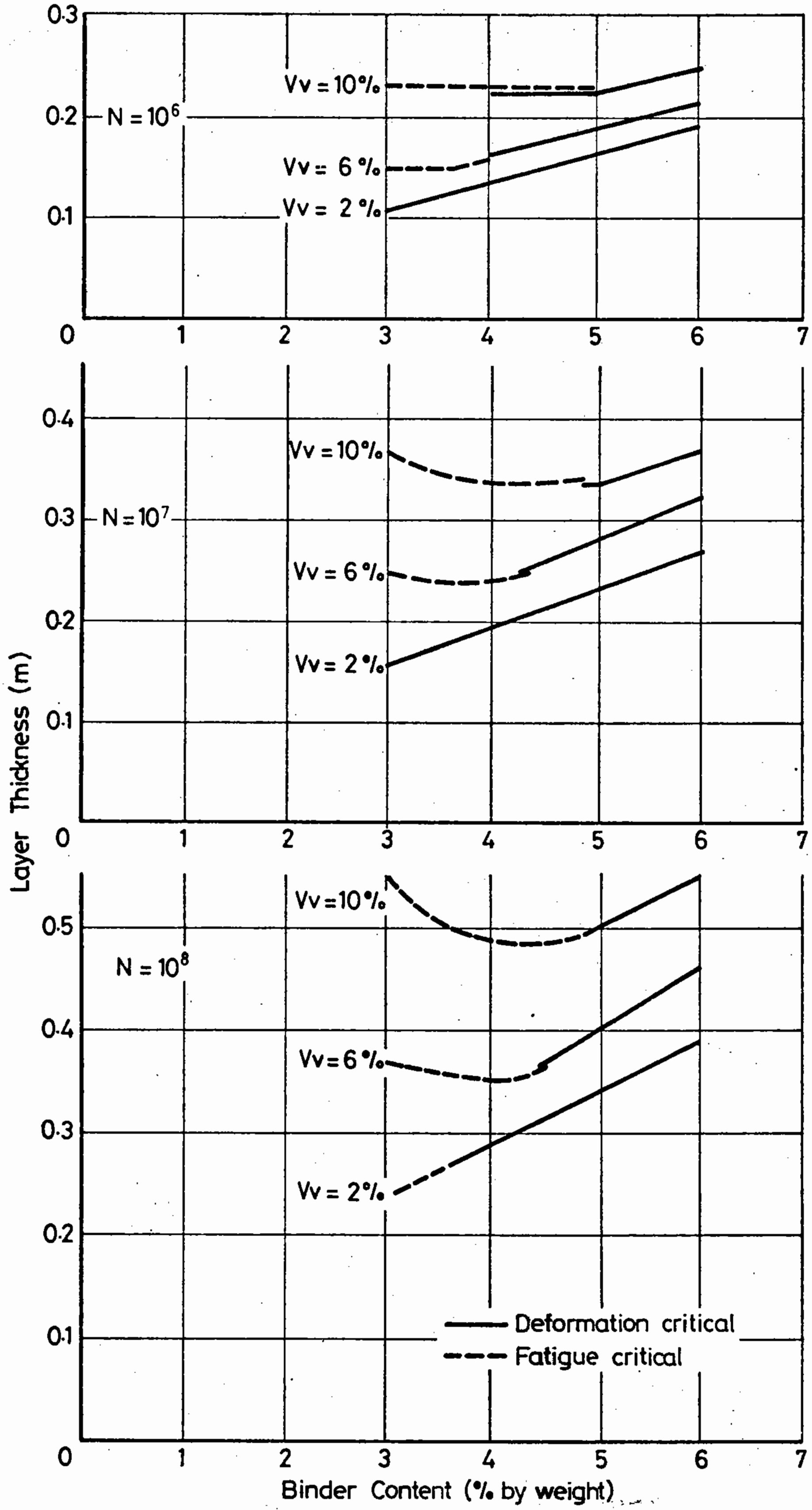

FIG. 12.4 DESIGN THICKNESS AS A FUNCTION OF BINDER CONTENT AND VOID CONTENT FOR 100 PEN MIXES

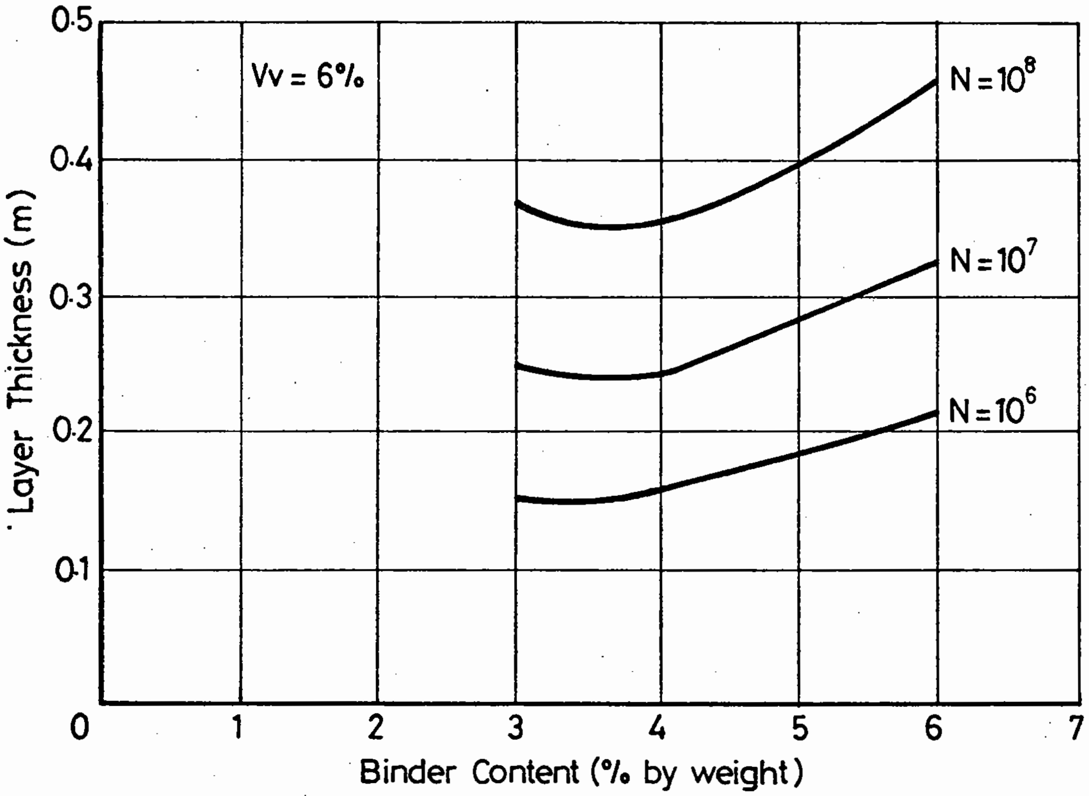

FIG. 12.5 DESIGN THICKNESS AS A FUNCTION OF BINDER CONTENT AND DESIGN LIFE FOR 100 PEN MIXES

Fig. 12.6 indicates that the relationship between VMA and design thickness is no longer conveniently unique. It should be noted that it is the designs dependent on the fatigue criterion which depart from a simple VMA - thickness relationship.

The behaviour of the structures designed with 100 pen binder can be explained as follows. Lean mixes are fatigue susceptible. As their binder content increases, their fatigue resistance improves. However, increasing binder content also reduces the mix stiffness; hence, whilst a mix is able to withstand a higher strain level, because its stiffness is reduced, an increased strain will be induced in the structure. The relative rates of change of these two factors will determine whether the design thickness decreases, remains constant or increases, and thus gives rise to the pattern of results observed with the 100 pen binders.

12.4 DESIGNS FOR MIXES WITH 200 PEN BINDER

Fig. 12.7 indicates a similar form to that observed for the 100 pen mixes, although almost all structures are designed against the fatigue criterion. No designs have been produced for mixes having 10% voids and a life of 10^8 standard axles, because these mixes are so highly fatigue susceptible that the allowable tensile strain is of the order of a few microstrain, rendering the design exercise meaningless.

Once more, the mixes at different void contents produce structures that have different thickness VMA relationships (Fig. 12.8). With these mixes, the stage has once more been reached at which the reduction in stiffness accompanying increases in binder content dominates the designs, requiring additional layer thickness to reduce strains to an acceptable level. In contrast to the designs for 50 pen mixes, instead of extra thickness being required to protect the subgrade, in this case it is

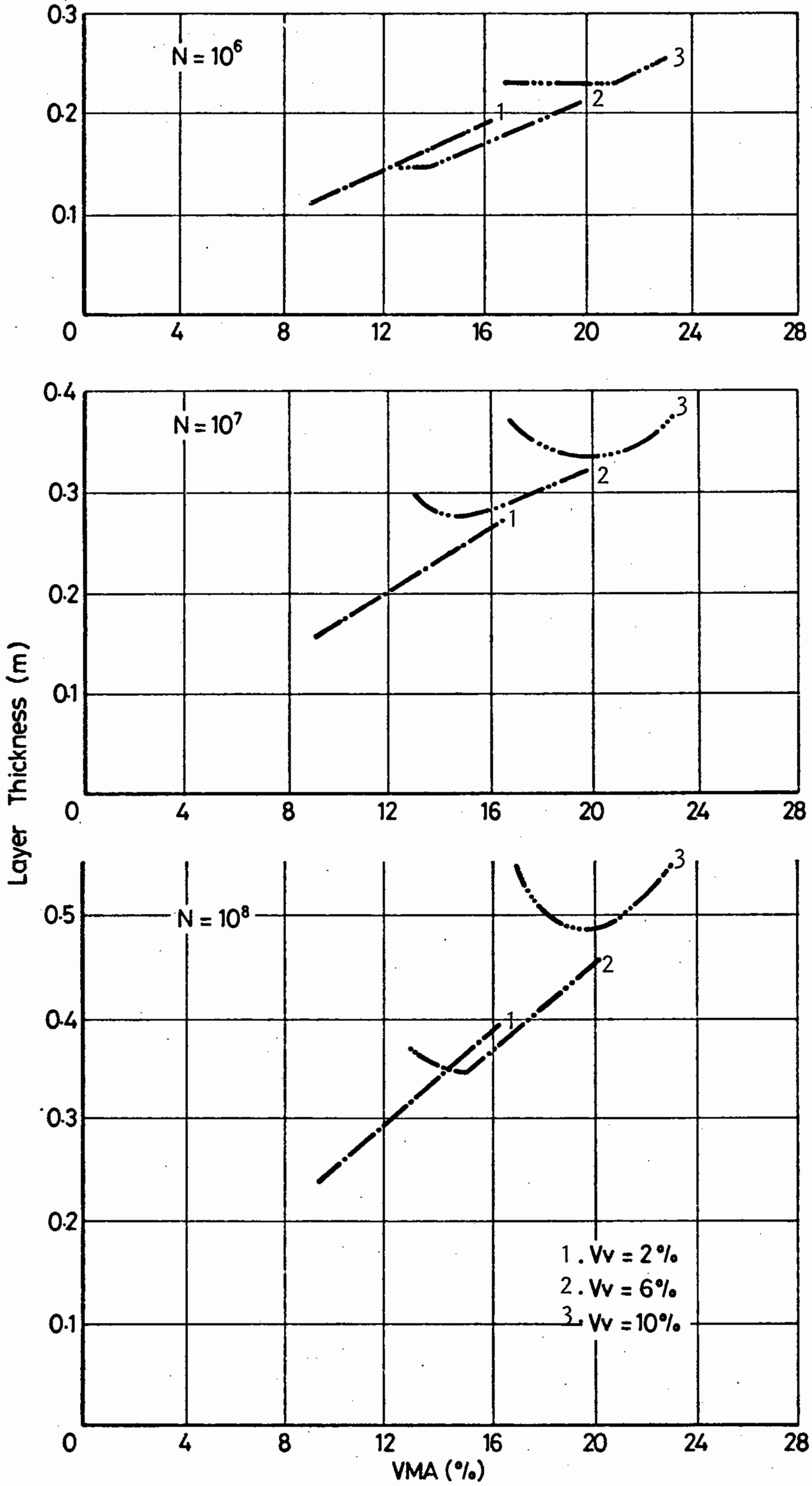

FIG. 12.6 DESIGN THICKNESS AS A FUNCTION OF VOID CONTENT AND VMA FOR 100 PEN MIXES

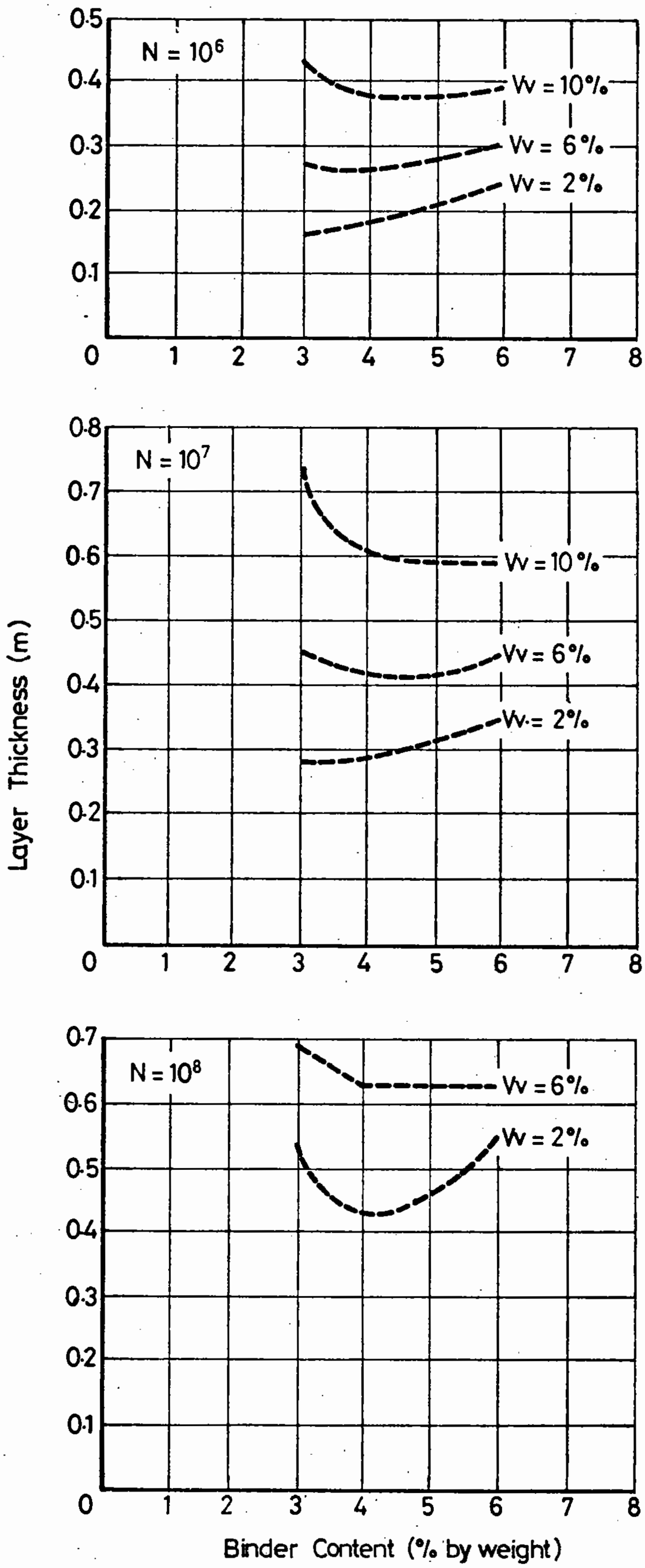

FIG. 12.7 DESIGN THICKNESS AS A FUNCTION OF BINDER CONTENT AND VOID CONTENT FOR 200 PEN MIXES

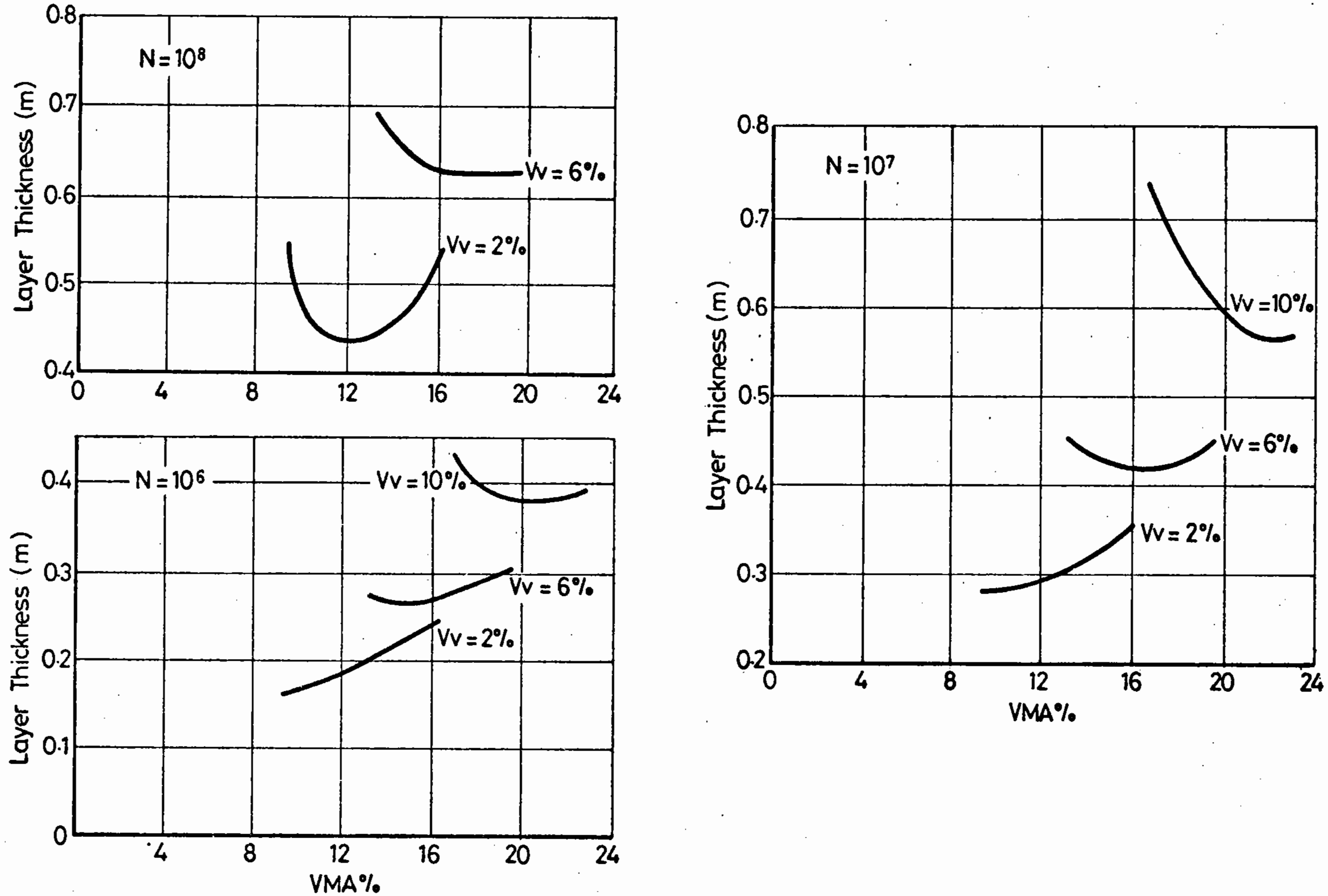

FIG. 12.8 DESIGN THICKNESS AS A FUNCTION OF VMA AND VOID CONTENT FOR 200 PEN MIXES

necessary for the asphalt layer to protect itself.

12.5 THE EFFECT OF BINDER GRADE

Some of the results from the proceding discussion have been re-plotted as functions of binder grade at a design life of 10^7 standard axles to indicate the effect on design of the three different binders. Fig. 12.9 shows design thickness as a function of binder content and grade. This clearly indicates the much greater thicknesses required when using a 200 pen binder. Since the 50 and 100 pen binders produce designs which are relatively close to each other, the saving in terms of reduced mixing and laying temperatures etc. for the 100 pen mix, may offset the cost of the additional 40 to 50 mm of layer thickness required.

12.6 THE EFFECT OF TEMPERATURE

Fig. 12.10 plots design thickness as a function of binder content and temperature for a 100 pen binder at a design life of 10^7 standard axles. This confirms the expected trend, since increasing temperature reduces stiffness, which in turn increases the strain induced in the structure at any given layer thickness. Thus, increased layer thicknesses are required as temperature increases. Considering the transition between the two design criteria, it is noticeable that as the temperature increases, progressively higher binder contents are necessary before designs revert to the subgrade strain criterion.

12.7 THE EFFECT OF POISSON'S RATIO FOR THE ASPHALT LAYER

Poisson's ratio was taken as 0.42 for the main part of this study, this being the value suggested by such laboratory data as is available (147,148). The effect on design of varying this parameter between 0.35 and 0.45 is shown in Fig. 12.11, from which it will be noted that

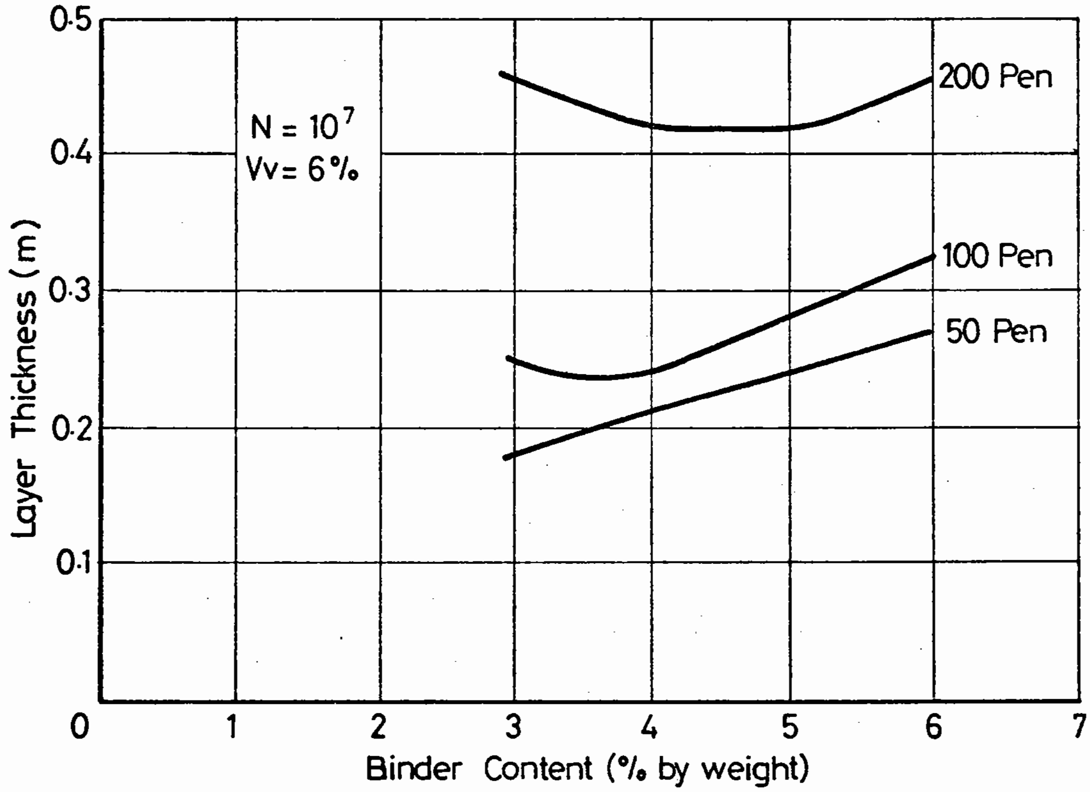

FIG. 12.9 DESIGN THICKNESS AS A FUNCTION OF BINDER CONTENT AND BINDER GRADE

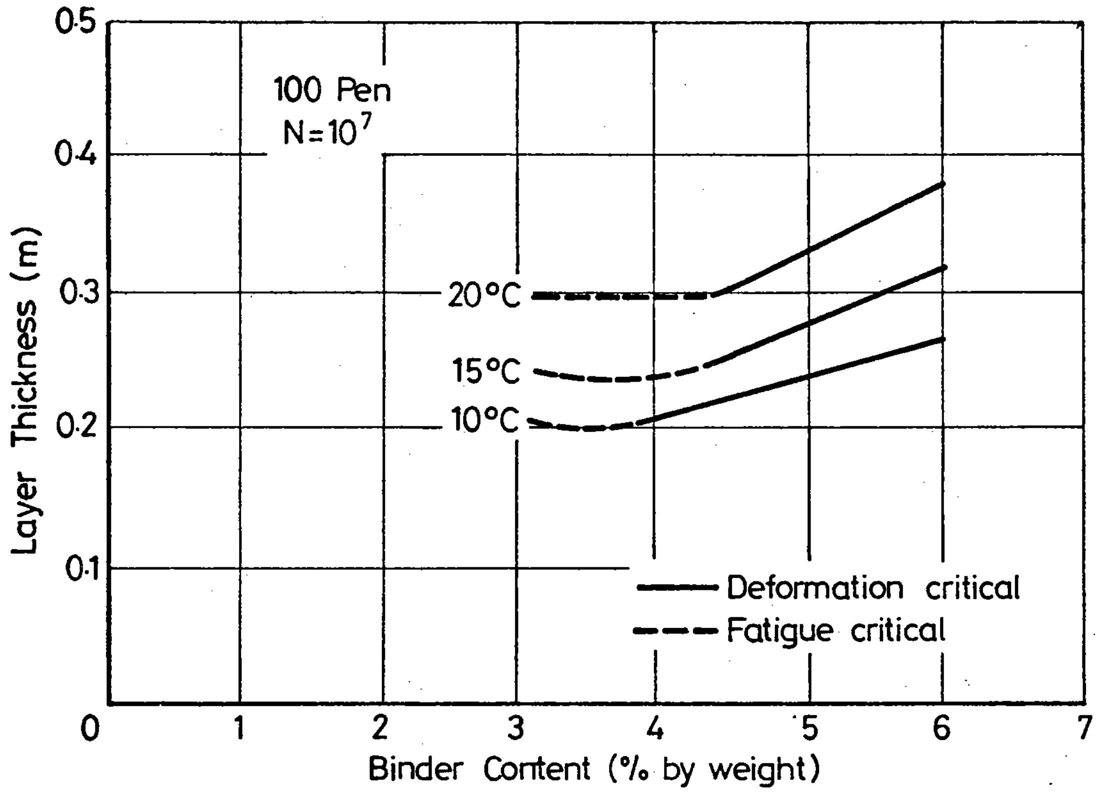

FIG. 12.10 DESIGN THICKNESS AS A FUNCTION OF BINDER CONTENT AND TEMPERATURE FOR 100 PEN MIXES

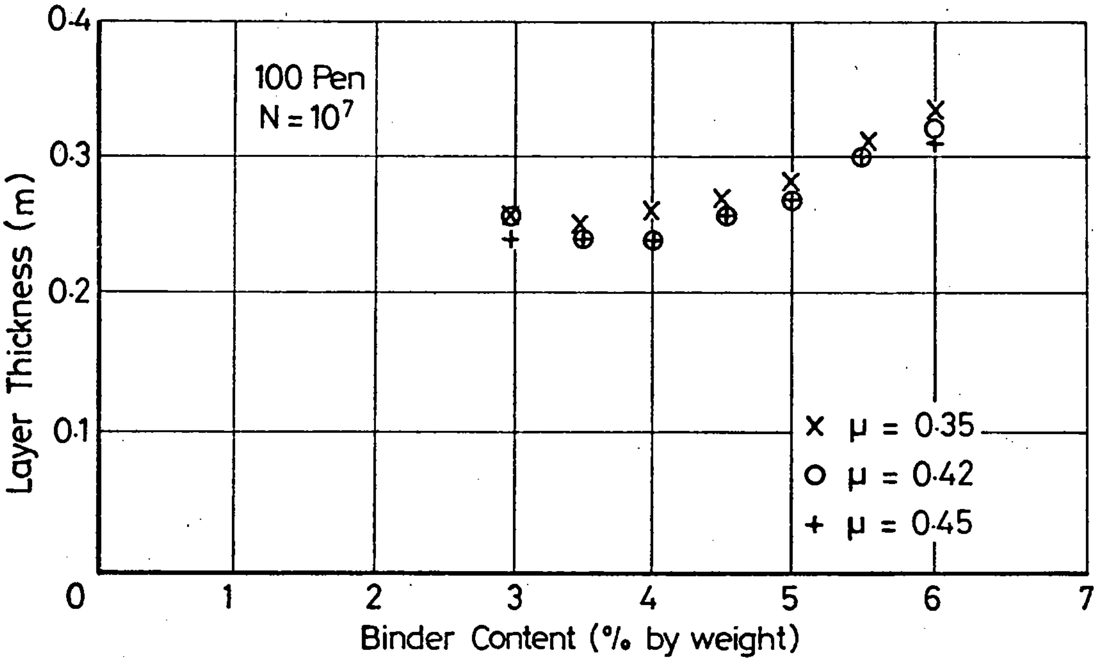

FIG. 12.11 DESIGN THICKNESS AS A FUNCTION OF BINDER CONTENT AND POISSON'S RATIO FOR 100 PEN MIXES

relatively little change in layer thickness results. Following this study, Poisson's ratio for bituminous material has been fixed at 0.4 within ADEM.

12.8 THE EFFECT OF SUBGRADE CBR

Fig. 12.12 shows that an improvement in the quality of the subgrade from a CBR of 3% to 6% reduces the required design thickness by 20 to 40 mm.

12.9 GENERAL DISCUSSION OF RESULTS

It is clear from the design thickness plots presented in this chapter that compaction of the asphalt mix is very important, and that improved compaction always leads to reduced layer thickness for a given performance requirement. Alternatively, it may be said that increased life can be obtained from a given pavement structure by improving the compaction.

Very few of the structures in this design study have a balance of fatigue and deformation performance. The structures are critical with respect to one criterion and have a significant residual life with respect to the other. This type of solution does not make most effective use of the materials and is therefore not the most economic. It also indicates that mix design should be undertaken as part of the pavement design procedure. The method suggested by Brown (17) could be developed for more general application to do this.

The results reported have deliberately excluded plots of design thickness as a function of mix stiffness. Plots of this type tend to suggest that design thickness and mix stiffness are closely related, and that required layer thickness could be predicted from mix stiffness, but to conclude this is incorrect. A design results from calculations that include performance criteria as well as material stiffnesses. Factors

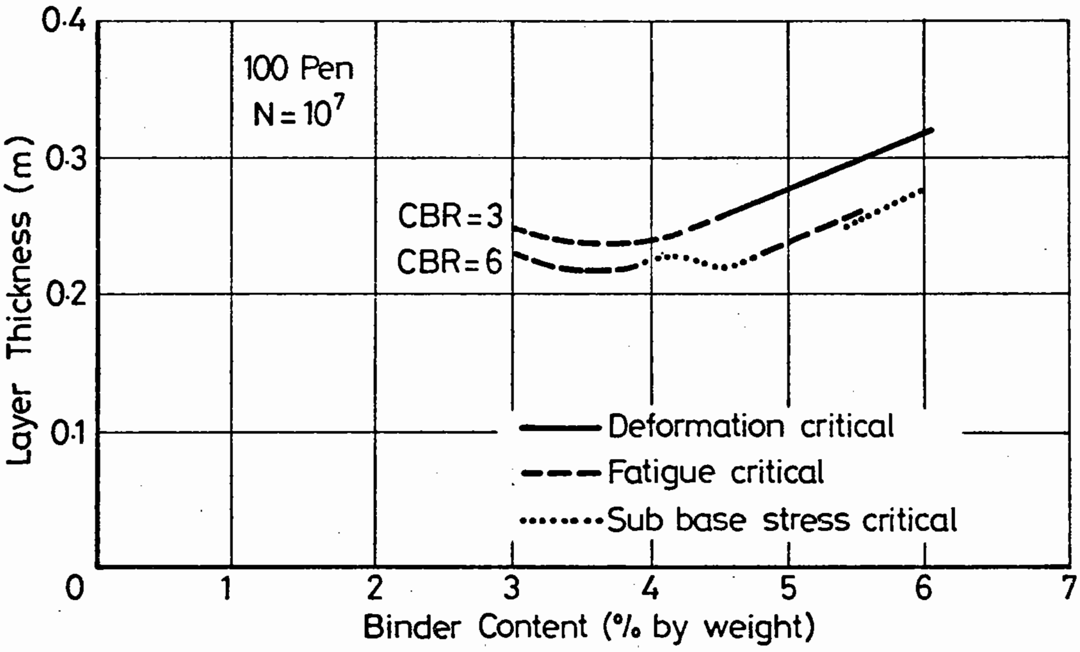

FIG. 12.12 DESIGN THICKNESS AS A FUNCTION OF BINDER CONTENT AND CBR FOR 100 PEN MIXES

which change the stiffness of an asphalt mix may not change its fatigue resistance, and vice versa. Thus, two mixes may have the same stiffness but very different performance characteristics and therefore will require different structures to satisfy given design requirements. Reference to a stiffness-thickness plot in this instance would give incorrect results.

Due to the complexity of the design calculation, the result of which is dependent on the interaction between many design parameters, it is recommended that design calculations are always carried out, using the ADEM program, rather than attempting to guess an answer from previous studies. This will ensure that appropriate consideration is given to the major parameters which affect design and ensure a reliable result.

CHAPTER THIRTEEN

SUMMARY DISCUSSION OF THIS RESEARCH

The purpose of this chapter is to discuss briefly the major points arising from this research.

Considering material characterisation first, linear elastic representation of asphaltic materials is perfectly acceptable. Non-linear characterisation of the subgrade, using a stress-softening relationship is desirable, and an analytical tool with the capability to do this has been developed. However, many factors affect the resilient behaviour of soils, and there is insufficient evidence regarding the variation of these parameters in the field to permit practical application of soil non-linearity to design.

Sufficient information was available to permit the development of a non-linear analysis system for unbound materials which could be used for design purposes. Analytical investigations underlined the importance of including failure criteria in the analysis procedure and the implementation of the non-linear model in analysis and design programs incorporates consideration of failure. It was noticed that the stress condition within a pavement does not favour the development of high moduli within the granular material, since as the normal stress (θ) increases, the ratio of the two stress invariants which define failure also tends to increase, overriding the effect of the normal stress. Thus, it appears that the most structurally efficient use of unbound material in conventional pavements is under a structurally significant asphalt layer.

Since this work is concerned with pavement design, consideration of design criteria was an important part of the work. Two modes of failure are considered in the design program, fatigue in the asphalt, and permanent deformation. Considerable research has been undertaken on the fatigue behaviour of asphalt mixes, and this has led to a system for predicting fatigue behaviour (34,35). However, this system predicts the life of a mix made in a laboratory under continuous cyclic loading, and direct application to pavement design was not possible. A multiplication factor of 100 was derived to convert laboratory fatigue lives to pavement fatigue lives, from consideration of the literature and the known performance of British pavements. Whilst use of this factor is quite reasonable and provides realistic pavement designs, further evidence for its use would considerably strengthen the design procedure.

It is evident that the problem of permanent deformation in pavements is complex since it embraces all layers within the pavement. It is only relatively recently that attention has been focussed on the problem of permanent deformation in pavement structures; hence, the techniques available are not as well developed as those relating to fatigue behaviour. The most fundamentally sound and thereforeattractive method involves use of stress-deformation relationships for paving materials in conjunction with analysis of the pavement structure. Several systems have been developed based on this approach but they do not offer an acceptable degree of accuracy when applied to pavement structures other than those used in the derivation of the procedure.

The alternative approach to limiting permanent deformation is to analyse pavements of known performance and derive a regression equation from the analysis. The equations can take several forms, the most complex attempting to predict a rate of rutting from calculated primary response parameters, and the simplest indicating a limiting vertical strain on the subgrade for a given design life. This type of approach is purely empirical, and as such is not universally applicable. However, having derived a regression equation with respect to particular conditions, then provided the constraints of these conditions are respected, the design criterion is perfectly valid and usable. It was impossible to obtain a usable regression equation which related rate of rutting to primary response parameters for British pavements through lack of data. Hence, a simple deformation criterion relating a limiting vertical subgrade strain to the design life was developed from the structures derived according to current British practice, and incorporated in the ADEM design procedure.

Design studies to investigate the relative effects of non-linear and linear analysis of unbound granular layers in ADEM have indicated that similar general trends are produced by both systems. However, the two systems do not always produce identical design thicknesses. Therefore, since non-linear characterisation of unbound material is the more accurate method, designs produced using the non-linear options must be regarded as more accurate. However, the non-linear options for pavement design use significantly more computer time than linear options, hence it will save computing costs if preliminary design studies are undertaken using linear options and then probable solutions checked using the non-linear options.

Design studies undertaken (using linear options) to evaluate the effect of various mix parameters on the required layer thickness for three-layer pavements have underlined the importance of good compaction of the asphalt layers, considerable economies being possible if a reduced void content can be achieved. This point is of considerable importance with regard to road building practices since currently they do not necessarily lead to the most efficient compaction of asphaltic materials.

During the course of this research considerable mental effort has been expended in attempting to explain design trends. It has been concluded that this is not a worthwhile occupation since there are a large number of factors which contribute to a design calculation. The interaction between these various factors is highly complex and it is easy to find apparently rational explanations for incorrect results, as has been experienced on several occasions during the course of this research. Also, since the latest version of the design program, ADEM3, has become available, it is easier to calculate than speculate.

Caution must be expercised in applying results obtained from one design study to different conditions, even if they appear to be similar. It is much safer to undertake a second study, which will cost relatively little and could save a costly mistake.

Having discussed this research at some length and pointed out the many limitations of ADEM it must be emphasised that ADEM is a unique, sophisticated, easy to use, implementable design program. Close scrutiny will indeed show areas of weakness; however, similar scrutiny of empirical methods will show that they are at least as fallible, and indeed more so. Thus, when the flexibility of the ADEM program with regard to designing pavements for a variety of conditions is considered, it is evident that in using ADEM, designers have "nothing to loose but their problems" (149).

CHAPTER FOURTEEN

CONCLUSIONS

1. The primary objective of this research, to produce a pavement design method based on structural analysis that can be used in practice, has been achieved. The design method has been produced as a computer program, ADEM3, which calculates the thickness of an asphalt layer necessary to satisfy specific design requirements. ADEM3 is highly versatile, permitting consideration of a wide range of mix formulation and offering two degrees of analytical sophistication.
2. Non-linear characterisation of the unbound granular layers within a pavement structure is necessary for accurate design.
3. Failure criteria are essential to the realistic application of non-linear characterisation of unbound materials. It is also essential to consider self-weight stresses in the analysis.
4. A program has been developed which can consider non-linear characterisation of subgrades in the analysis of a pavement structure.
5. Non-linear characterisation of subgrades cannot be included in the design procedure since insufficient data is available concerning the effect on the non-linear model of soil variables.
6. Design studies have shown that improved compaction of the asphalt layer in a pavement structure can lead to significant reduction in layer thickness.
7. Economy in layer thickness may be achieved by increasing binder contents if the critical parameter within the pavement structure is fatigue.

8. Variations in Poisson's ratio for the asphalt and aggregate specific gravity have no significant effect on required design thicknesses.

CHAPTER FIFTEEN

RECOMMENDATIONS FOR FURTHER WORK

1. Experimental studies to relate the fatigue performance of asphalt mixes measured in the laboratory with their performance in a pavement structure are necessary.

2. The effect of variation in material quality on the K-θ model and failure criteria for unbound granular materials should be investigated.

3. Further research to develop a fundamental method for predicting the deformational behaviour of pavements is essential to the further development of analytical pavement design. When accurate prediction of deformations becomes a reality, then detailed cumulative damage calculations will be possible and will result in much greater understanding of the probable performance of a pavement structure.

5. Further analytical studies of the behaviour of cement treated layers should lead to an implementable system for considering them in an improved version of ADEM.

6. Research to develop mix design method compatible with the analytical approach to pavement design would significantly improve the capability of designers to provide economic pavement structures.

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APPENDIX A

CUMULATIVE DAMAGE CONSIDERATIONS

A.1 INTRODUCTION

Throughout this research it has been assumed that traffic can be reduced to an equivalent number of standard axles, and that seasonal temperature changes can be reduced to a mean annual air temperature. However, pavements are subjected to considerable variation in both axle loads and temperature, and accumulate damage according the combination of axle loads and the physical state of the pavement.

The most commonly used method for computing cumulative damage is the linear summation of cycles, generally referred to as Miner's rule,

$$D = \sum_{i=1}^{i} \frac{n_i}{N_i} \qquad (A.1)$$

where D = total cumulative damage having a maximum value of 1 at failure

n_i = number of applications at level i

N_i = number of applications to cause failure at level i.

In the literature, Miner's rule is usually applied to asphalt mixes and so the parameters n_i and N_i relate to tensile strains in the asphalt layer. Its use in this context has been verified by Deacon (A1) and McElvaney (A2). O'Neill (A3) concluded from a review of some general cumulative damage theories that there was no hypothesis which showed a clear general superiority to Miner's law, and so it appears to be reasonable to apply the law to other performance parameters relevant to pavement systems.

Details of a procedure for predicting asphalt fatigue life on the basis of cumulative damage have been reported by Deacon (A4). This procedure considers the physical state of a pavement, which corresponds to given moisture and temperature profiles, traffic, and levels of critical stress and strain in the structure.

It is the purpose of this Appendix to examine the traffic and temperature conditions appropriate to UK pavements and assess the applicability of a cumulative damage approach to the analytical design procedure.

It should be noted that no effort has been made to consider pavement moisture conditions within this study since, as indicated in Chapter 7, there is no usable data available on moisture movements in UK pavements.

A.2 TRAFFIC

A.2.1 Growth with time

Croney (A5) analysed vehicle licencing records which indicated that there had been a rapid rise in number of registered commercial vehicles up to the mid-1960s, when growth had levelled off. This study ended in 1973 with no sign of a return to a rapid growth rate. However, whilst the number of vehicles has remained relatively constant, the number of axles per commercial vehicle has continued to increase, as has vehicle laden weight. This has had a dramatic effect on the factor applied in calculating the number of standard axles from number of commercial vehicles. Table A.1 gives a comparison between the factors recommended in the 1970 revision of Road Note No. 29 (A6) and the revision of these factors according to the latest Technical Memorandum (A7) issued by the Engineering Intelligence Division of the Department of Transport.

A.2.2 Traffic distribution

Variation in a 24-hour period: Fig. A.1, taken from Croney (A5), shows the 24-hour variation in commercial traffic observed on the west bound

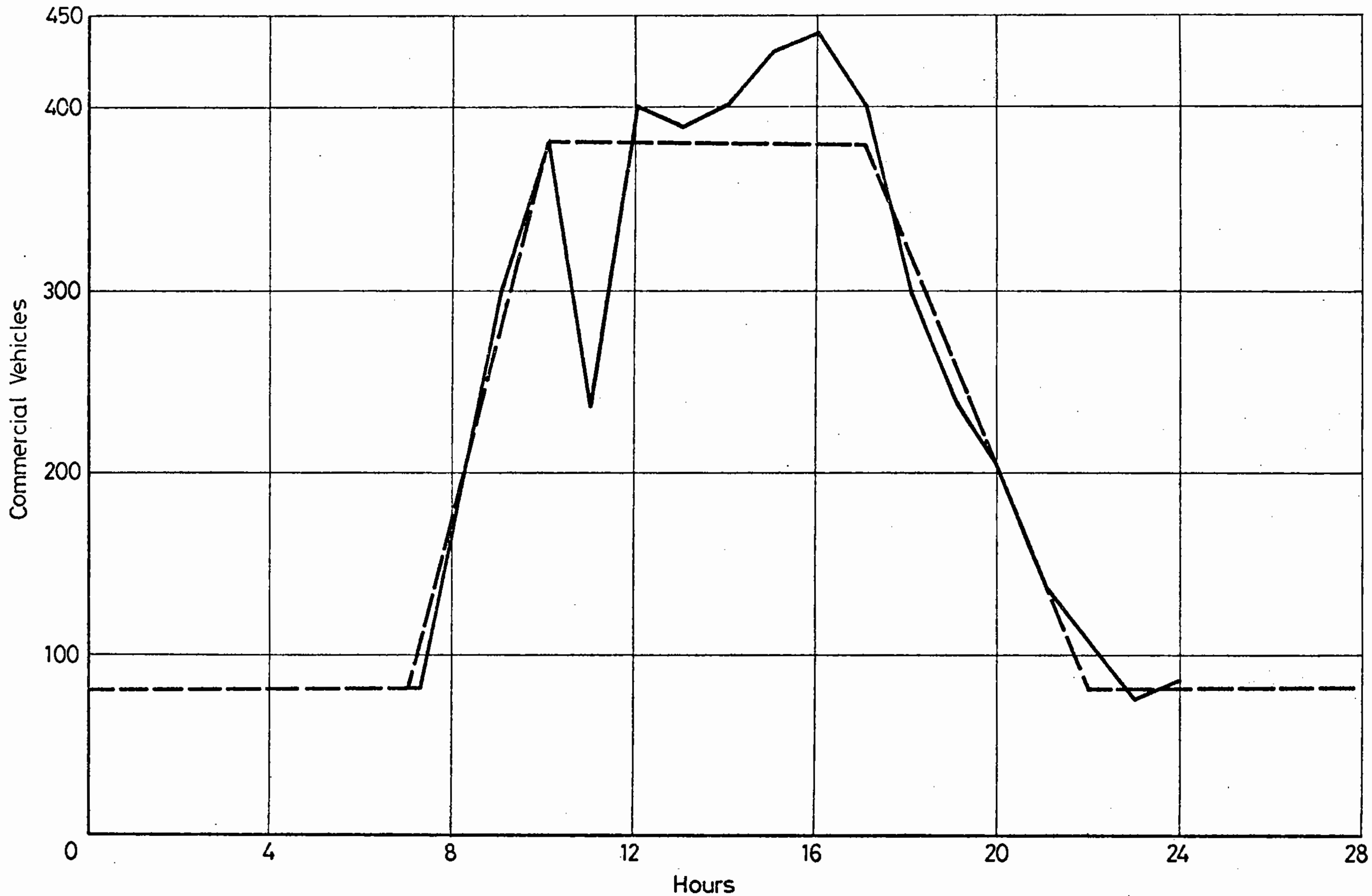

FIG. A.1 24-HOUR TRAFFIC VARIATION ON THE M6 (AFTER CRONEY, A5)

Traffic loading at time of construction	Equivalent number of standard axles/commercial vehicle	
	RN 29 (1970)	H6/78
> 2000 commercial vehicles in each direction	*	2.75
1000-2000 commercial vehicles in each direction	1.08	2.25
250-1000 commercial vehicles in each direction	0.72	1.25
All other public roads	0.45	0.45

* The category of over 2000 c.v.d. did not exist in the 1970 edition of Road Note No. 29 (A6).

Table A.1 Factors for converting commercial vehicles to standard axles

lanes of the M6 Motorway at Perry Barr, July 1972. Broadly speaking, the distribution is trapezoidal, and inspection of plots from other sites (A5) indicates a similar pattern. If a precise evaluation of pavement damage during any 24-hour period is required, then detailed traffic data should be used, and this can only be gained by vehicle count after construction of the pavement. However, for design purposes, a simplified distribution was considered adequate and the trapezoidal distribution shown in Fig. A.1 was used. A period of low volume from midnight to 0700 hrs is followed by a rapid linear rise period from 0700 to 1000 hrs to a maximum volume. A constant level is then maintained until 1700 hrs, when the volume decreases linearly to the minimum by 2200 hrs. The ratio of maximum to minimum number of commercial vehicles in a 24-hour period using the simplified distribution gave 5220 vehicles which compares favourably with the 5250 computed from the actual distribution.

By using the simplified distribution, and maintaining the 4.75:1 ratio between maximum and minimum traffic levels the traffic levels can be calculated from consideration of the areas of plane figures and the 24-hour total commercial vehicle count (C),

$$\text{minimum traffic} = 0.0168C \tag{A.2}$$

$$\text{maximum traffic} = 4.75 \times \text{min. traffic} \tag{A.3}$$

Thus, considerable economy can be made in the calculation required for an assessment of the cumulative effect of traffic in a 24-hour period.

Distribution in lanes: Fig. A.2 (after Croney, A5) plots the percentage of commercial vehicles in the left-hand and overtaking lanes of a two-lane highway. The data used for this plot terminated at just under 8000 c.v.d. and has been extrapolated as shown by the dotted line. This linear extrapolation is not really justified, it being probable that the lines will be asymptotic to the 50% condition.

Important information is provided by Fig. A.2, since on a heavily trafficked multi-lane road, a relatively high proportion of vehicles will be moved across to a second lane, thus significantly reducing the loading on the nearside lane, which could permit economy in design.

Seasonal variation in traffic: Studies undertaken by the Transport and Road Research Laboratory (A5) have shown that commercial vehicle movements do not vary greatly throughout the year. The major variations are a 17% reduction in the month of August, and 40% reduction during the week before and the week after Christmas. The reduction during August could be of great significance since at this time of year pavement temperatures are generally high, asphalt stiffnesses low, and therefore susceptibility to damage high (see next section).

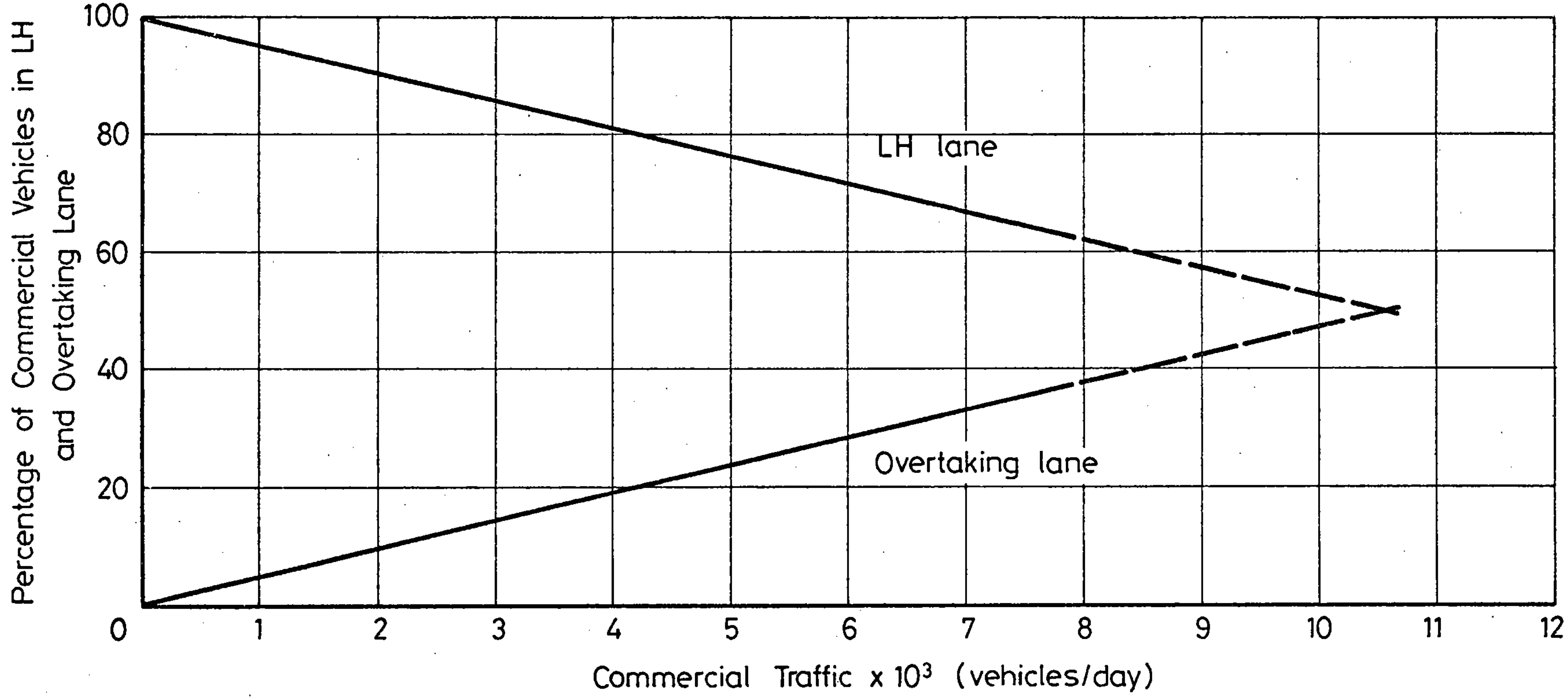

FIG. A.2 DISTRIBUTION OF COMMERCIAL VEHICLES BETWEEN LANES (AFTER CRONEY, A5)

A.3 TEMPERATURE

A.3.1 Introduction

The stiffness of asphalt is highly dependent on its temperature. Thus, since for the majority of the pavement structures considered in this work, the most structurally significant layer is the asphalt one, it is important to define the temperature within that layer.

Temperature data has been published for British pavements by Galloway (A8) and Forsgate (A9). However, this data is presented in terms of frequency distribution of observation and are not related to the time of day. The time-temperature relationship is important since it permits correlation with traffic and thus allows damage calculations to be undertaken on the basis of periodic variation in both parameters.

Edwards and Valkering (A10) used temperature data from Kallas (A11) and Forsgate (A9) to develop a weighted mean annual air temperature for design purposes from mean monthly air temperatures. This was achieved by analysing pavements with due consideration to temperature gradients, calculating by Miner's rule the damage for a year and then determining the temperature (weighted mean annual air temperature) which gave equivalent damage for that year.

Since data has become available (A5) regarding temperature variation in British pavements, these structures have been analysed in a manner similar to that used by Edwards and Valkering (A10), to develop a relationship between air and pavement temperatures. This relationship has then been used for a study of British pavements with a view to reducing the number of periods to be considered for cumulative damage calculations.

A.3.2 Analysis for 24-hour periods

Croney (A5) has produced data showing the 24-hour temperature variation in the structures shown in Fig. A.3 for the months April, July, October and January during the period March 1969 to February 1970. The selection of this particular period was justified (A5) on the basis that the mean monthly air temperatures were close to the long-term means.

Of the two structures for which data is available (Fig. A.3) only structure 1 was analysed. This is because it has an asphalt base layer and is generally more typical of British pavements than structure 2. Two damage determinants were chosen for this study, tensile strain in the asphalt and vertical strain on the subgrade since these two parameters are commonly used for design purposes.

A subgrade modulus of 40 MN/m^2 was assumed, and load applied through the usual 80 kN dual wheel standard. The temperatures for 38 mm and 203 mm depths were assumed to be the average temperatures for each layer, and corresponding asphalt stiffnesses for typical rolled asphalt and bitumen macadam mixes were determined. Details of the calculations are presented in Tables A.2 to A.5. The damage index reported in these tables has been calculated according to Miner's rule (Equation A.1) using the simplified traffic distribution developed in this Appendix. To reduce the amount of computation, the number of commercial vehicles rather than standard axles was used, with the result being multiplied by 10^6 for numerical convenience. This gives an accurate representation of relative damage which is satisfactory for this study. If actual damage is required, the damage index must be divided by 10^6 and by the appropriate factor to convert commercial vehicles to standard axles.

Structure 1

101 mm rolled asphalt	x d_1 = 38 mm
254 mm bitumen macadam	x d_2 = 203 mm
152 mm sub-base	
Subgrade	

Structure 2

203 mm sealed bitumen macadam	x d_1 = 38 mm x d_2 = 102 mm x d_3 = 203 mm
203 mm wet mix road base	
152 mm sub-base	
Subgrade	

x Depths at which temperatures were measured.

FIG. A.3 STRUCTURES USED BY CRONEY (A5) TO MEASURE TEMPERATURE VARIATION

38 mm below surface		203 mm below surface		ε_A	ε_s	No. of commercial vehicles	Asphalt		Subgrade	
T (^oC)	E_1 (MN/m^2)	T (^oC)	E_2 (MN/m^2)	($x10^{-6}$)	($x10^{-6}$)		N	$\frac{n}{N}$*	N	$\frac{n}{N}$*
10.25	9689	12.0	13305	25	87	80	222.5	0.360	345.71	0.231
9.25	10230	11.3	13841	24	84	80	270.8	0.295	391.96	0.204
8.5	10669	11.0	14071	24	83	80	270.8	0.295	409.12	0.196
7.5	11285	10.5	14476	23	81	80	332.4	0.241	446.43	0.179
7.3	11411	10.3	14637	23	80	80	332.4	0.241	466.72	0.171
7.4	11348	9.5	15393	22	79	80	411.7	0.194	488.20	0.164
7.75	11128	9.3	15472	22	79	80	411.7	0.194	488.20	0.164
8.2	10488	9.5	15393	22	80	130	411.7	0.316	466.72	0.279
11.25	9689	10.0	14880	23	83	230	332.4	0.692	409.12	0.562
14.0	7864	10.2	14718	24	87	330	270.8	1.219	345.71	0.955
16.0	7012	10.5	14475	25	90	380	222.5	1.708	306.22	1.241
17.52	6332	11.8	13450	27	95	380	153.7	2.472	252.36	1.506
19.5	5510	12.5	12916	28	100	380	127.0	2.946	210.05	1.809
21.25	4854	13.3	12299	29	105	380	108.9	3.489	176.41	2.154
22.3	4483	14.2	11624	31	110	380	79.0	4.810	149.36	2.544
22.3	4483	15.2	10885	32	113	380	67.8	5.604	135.56	2.803
21.5	4767	15.7	10501	33	113	380	58.5	6.496	135.56	2.803
20.0	5316	15.8	10424	32	113	350	67.8	5.162	135.56	2.582
18.0	6120	15.7	10501	31	107	290	79.0	3.671	164.89	1.759
16.2	6922	15.5	10655	31	104	230	79.0	2.911	182.55	1.260
14.75	7544	14.5	11405	29	99	170	108.9	1.561	217.74	0.781
13.5	8094	14.5	11405	28	97	110	129.0	0.853	234.34	0.469
12.5	8564	13.0	12526	26	92	80	184.3	0.434	283.06	0.283
11.5	9055	12.5	12916	26	90	80	184.3	0.434	306.22	0.261
					Totals	5220		46.596		25.360

* $\frac{n}{N}$ is the damage index. N is the relative life of the pavement.

Table A.2 Cumulative damage calculations for April

38 mm below surface		203 mm below surface		ε_A	ε_s	No. of commercial vehicles	Asphalt		Subgrade	
T (^oC)	E_1 (MN/m^2)	T (^oC)	E_2 (MN/m^2)	($x10^{-6}$)	($x10^{-6}$)		N	$\frac{n}{N}$*	N	$\frac{n}{N}$*
18.25	6000	21.2	7060	41	128	80	20.58	3.887	86.84	0.921
18.5	5950	20.4	7482	39	125	80	26.18	3.056	94.54	0.846
17.75	6200	19.5	7990	38	121	80	29.67	2.696	106.20	0.753
16.75	6674	18.6	8528	35	112	80	44.07	1.815	140.03	0.571
15.5	7200	18.7	8466	35	114	80	44.07	1.815	131.44	0.609
15.5	7200	18.4	8654	35	113	80	44.07	1.815	135.65	0.590
16.2	6900	18.4	8654	35	114	80	44.07	1.815	131.44	0.609
17.75	6200	18.5	8592	35	111	130	44.07	2.950	144.60	0.899
19.2	5700	18.7	8466	37	120	230	33.73	6.819	109.40	2.102
21.2	4900	19.3	8106	39	126	330	26.18	12.605	91.88	3.592
23.5	4000	19.5	7990	40	133	380	23.18	16.393	75.72	5.018
26.0	3261	20.0	7700	42	141	380	18.33	20.731	61.44	6.185
28.1	2700	21.0	7155	46	151	380	11.83	32.121	48.08	7.903
29.75	2300	21.8	6757	49	159	380	8.73	43.528	39.97	9.507
31.0	2030	22.7	6288	52	168	380	6.56	57.927	32.82	11.578
31.3	2000	23.7	5786	55	175	380	5.01	75.848	28.36	13.399
31.0	2030	24.5	5417	57	180	380	4.22	90.047	25.65	14.815
29.3	2380	24.7	5320	56	176	350	4.59	76.252	27.79	12.594
27.1	2963	24.7	5320	55	169	290	5.01	57.884	32.14	9.023
25.3	3460	24.5	5417	53	162	230	5.98	38.462	37.39	6.151
23.6	4095	23.6	5835	49	152	170	8.73	19.473	46.96	3.620
21.75	4650	22.7	6288	46	143	110	11.83	9.298	58.42	0.531
20.9	4979	21.7	6807	43	136	80	16.37	4.887	69.91	0.874
20.0	5316	21.0	7155	41	131	80	20.58	3.887	79.94	0.999
						Totals		586.011		111.818

* $\frac{n}{N}$ is the damage index. N is the relative life of the pavement.

Table A.3 Cumulative damage calculations for July

38 mm below surface		203 mm below surface		ε_A	ε_s	No. of commercial vehicles	Asphalt		Subgrade	
T (°C)	E_1 (MN/m²)	T (°C)	E_2 (MN/m²)	(x10⁻⁶)	(x10⁻⁶)		N	$\frac{n}{N}$*	N	$\frac{n}{N}$*
8.8	10488	10.3	14637	23	82	80	332.4	0.241	427.25	0.187
9.0	10368	10.2	14718	23	82	80	332.4	0.241	427.25	0.187
9.0	10368	10.3	14637	23	82	80	332.4	0.241	427.25	0.187
9.0	10368	10.3	14637	23	82	80	332.4	0.241	427.25	0.187
8.8	10488	10.3	14637	23	81	80	332.4	0.241	446.43	0.179
8.3	10789	10.0	14880	23	80	80	332.4	0.241	466.72	0.171
8.0	10970	10.0	14880	23	80	80	332.4	0.241	466.72	0.171
8.0	10970	10.0	14880	23	81	130	332.4	0.391	446.43	0.291
8.5	10669	9.8	15049	23	82	230	332.4	0.692	427.25	0.538
9.4	10148	9.8	15049	23	84	330	332.4	0.993	391.96	0.842
11.0	9304	10.2	14718	24	86	380	270.8	1.403	360.31	1.055
12.5	8564	10.5	14476	25	88	380	222.5	1.708	331.86	1.145
13.0	8323	10.9	14152	25	89	380	222.5	1.708	318.71	1.192
13.7	8007	11.3	13838	26	94	380	184.3	1.062	262.10	1.450
15.1	7286	12.3	13071	27	95	380	153.7	2.472	252.36	1.506
15.8	7095	12.4	12993	27	95	380	153.7	2.472	252.36	1.506
15.6	7178	12.6	12838	27	94	380	153.7	2.472	262.10	1.450
14.3	7733	12.8	12682	27	93	350	153.7	2.277	272.32	1.285
13.75	7979	12.8	12682	26	91	290	184.3	1.574	294.35	0.985
12.5	8564	12.5	12916	25	88	230	222.5	1.034	331.86	0.693
11.5	9080	11.9	13373	25	87	170	322.5	0.754	345.71	0.492
10.7	9458	11.9	13373	24	84	110	270.8	0.406	391.96	0.281
10.0	9817	10.8	14233	24	84	80	270.8	0.295	391.96	0.204
9.7	9982	10.6	14395	24	83	80	270.8	0.295	409.12	0.196
						Totals		24.695		16.380

* $\frac{n}{N}$ is the damage index. N is the relative life of the pavement.

Table A.4 Cumulative damage calculations for October

38 mm below surface		203 mm below surface		ε_A	ε_s	No. of commercial vehicles	Asphalt		Subgrade	
T (^oC)	E_1 (MN/m^2)	T (^oC)	E_2 (MN/m^2)	($\times 10^{-6}$)	($\times 10^{-6}$)		N	$\frac{n}{N}$*	N	$\frac{n}{N}$*
5.0	12948	5.0	18962	19	69	80	833.6	0.095	792.31	0.101
5.2	12810	5.0	18962	19	70	80	833.6	0.096	752.55	0.106
5.4	12672	5.1	18608	19	70	80	833.6	0.096	752.55	0.106
5.4	12672	5.2	18523	19	70	80	833.6	0.096	752.55	0.106
5.4	12672	4.8	19136	19	69	80	833.6	0.096	792.31	0.101
5.1	12879	5.0	18962	19	69	80	833.6	0.096	792.31	0.101
4.8	13092	5.0	18962	19	69	80	833.6	0.096	792.31	0.101
4.5	13309	4.8	19136	18	69	130	1081.4	0.120	792.31	0.164
4.5	13309	4.7	19224	18	68	230	1081.4	0.213	834.79	0.276
4.5	13309	4.7	19224	18	68	330	1081.4	0.305	834.79	0.395
4.7	13164	4.9	19049	18	69	380	1081.4	0.351	792.31	0.480
5.0	12948	5.0	18962	19	69	380	833.6	0.456	792.31	0.480
5.2	12810	5.0	18962	19	70	380	833.6	0.456	752.55	0.505
5.4	12672	5.2	18523	19	70	380	833.6	0.456	752.55	0.505
5.5	12604	5.2	18523	19	71	380	833.6	0.456	715.31	0.531
5.4	12672	5.2	18523	19	70	380	833.6	0.456	752.55	0.505
5.0	12948	5.3	18439	19	70	380	833.6	0.456	752.55	0.505
5.0	12948	5.6	18186	19	71	350	833.6	0.420	715.31	0.489
4.7	13164	5.4	18354	19	70	290	833.6	0.348	752.55	0.385
4.5	13309	5.8	18017	19	71	230	833.6	0.276	715.31	0.322
4.5	13309	5.7	18101	19	70	170	833.6	0.204	752.55	0.226
4.0	12948	5.6	18186	19	70	110	833.6	0.132	752.55	0.146
5.2	12810	5.4	18354	20	71	80	651.3	0.123	715.31	0.112
5.2	12810	5.0	18962	19	70	80	833.6	0.096	752.55	0.106
						Totals		6.872		6.854

* $\frac{n}{N}$ is the damage index. N is the relative life of the pavement.

Table A.5 Cumulative damage calculations for January

Data from Tables A.2 to A.5 has been plotted in Figs A.4 to A.7 in order to graphically represent the development of repeated load damage in a pavement from temperature and traffic variations. Also by comparing Figs A.4 to A.7, the vast difference in behaviour of the pavement from winter to summer conditions becomes apparent. The hot summer months are obviously very much more damaging than the cold winter months. During a 24-hour period in July the damage starts to increase from about 0300 hrs reaching a peak during the 1600-1700 hr period, when both pavement base temperature and vehicle numbers start to decrease. It is noticeable that despite the decrease in temperature 38 mm down from the surface starting at about 1430, the base temperature continues to rise until 1700, and the damage calculation confirms that this is the more significant factor.

Turning to a 24-hour period in January, the damage, by comparison with the summer months, is very small and spread over a wider period of time. It should be noted that in these calculations no attempt has been made to model possible reductions in subgrade support during the winter period, since no data is available that indicates how this may be accomplished. It is thought that in a well constructed pavement in the British climate the variation in subgrade support is negligible, but if this is not true the damaging effect of cold months could increase.

The months of April and October appear to be intermediate between January and July, but the temperature fluctuations during the Spring appear to be more significant than those during the Autumn.

A.3.3 Determination of equivalent pavement temperatures

Table A.6 has been produced from analyses of pavement structure 1 (Fig. A.3), for a range of temperatures for tensile strain in the

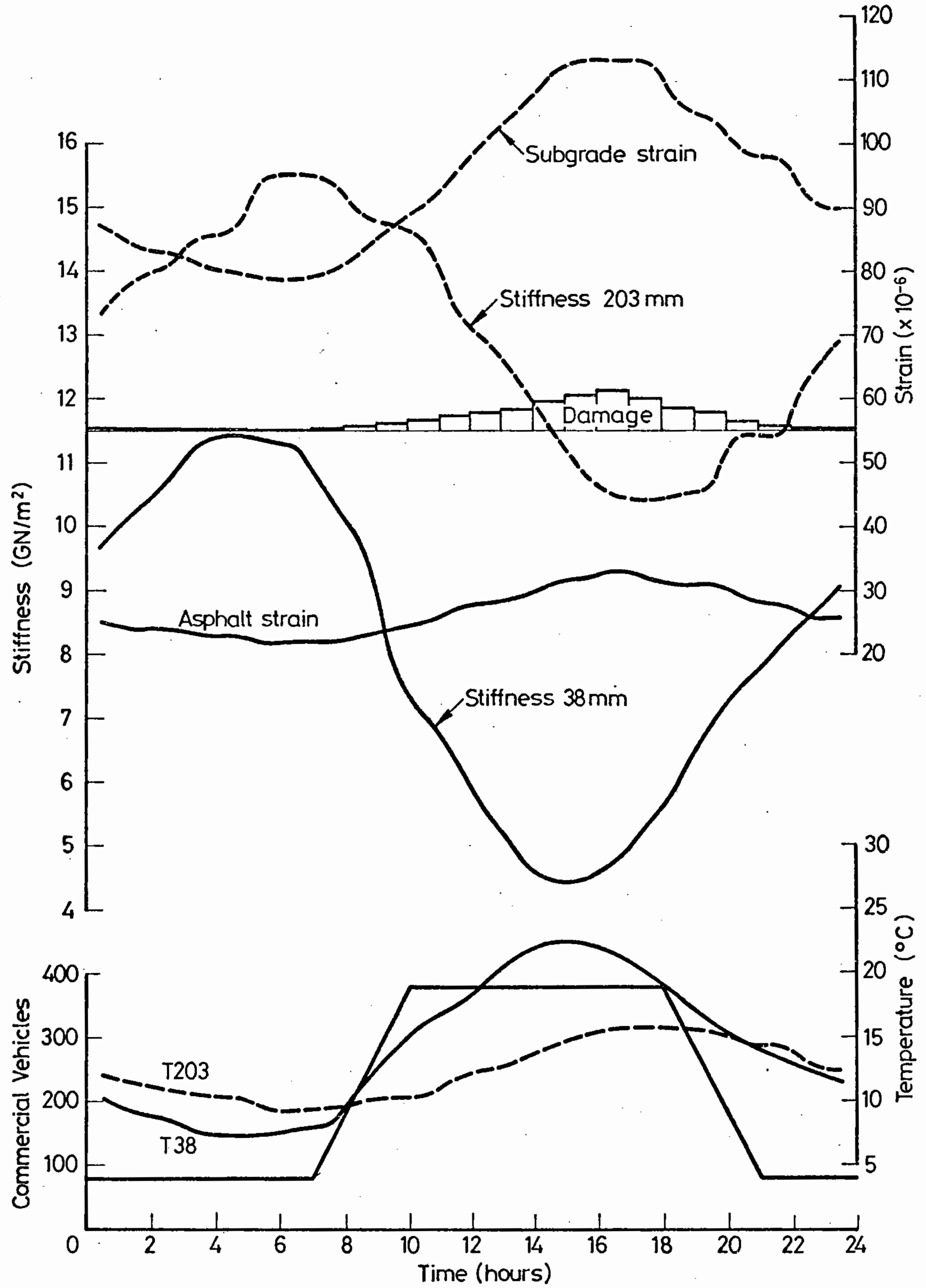

FIG. A.4 STRUCTURAL PARAMETERS FOR A 24-HOUR CYCLE IN APRIL

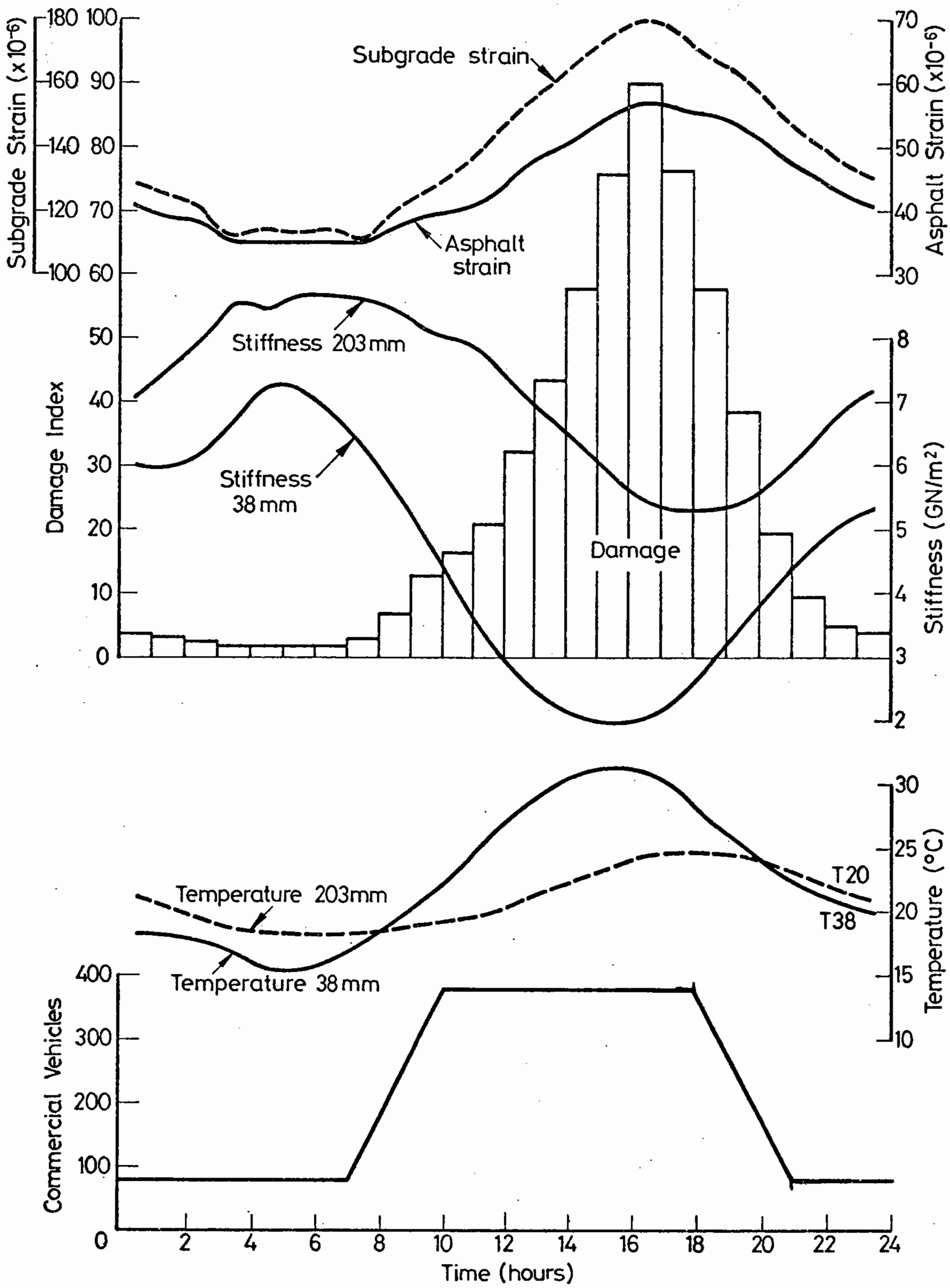

FIG. A.5 STRUCTURAL PARAMETERS FOR A 24-HOUR CYCLE IN JULY

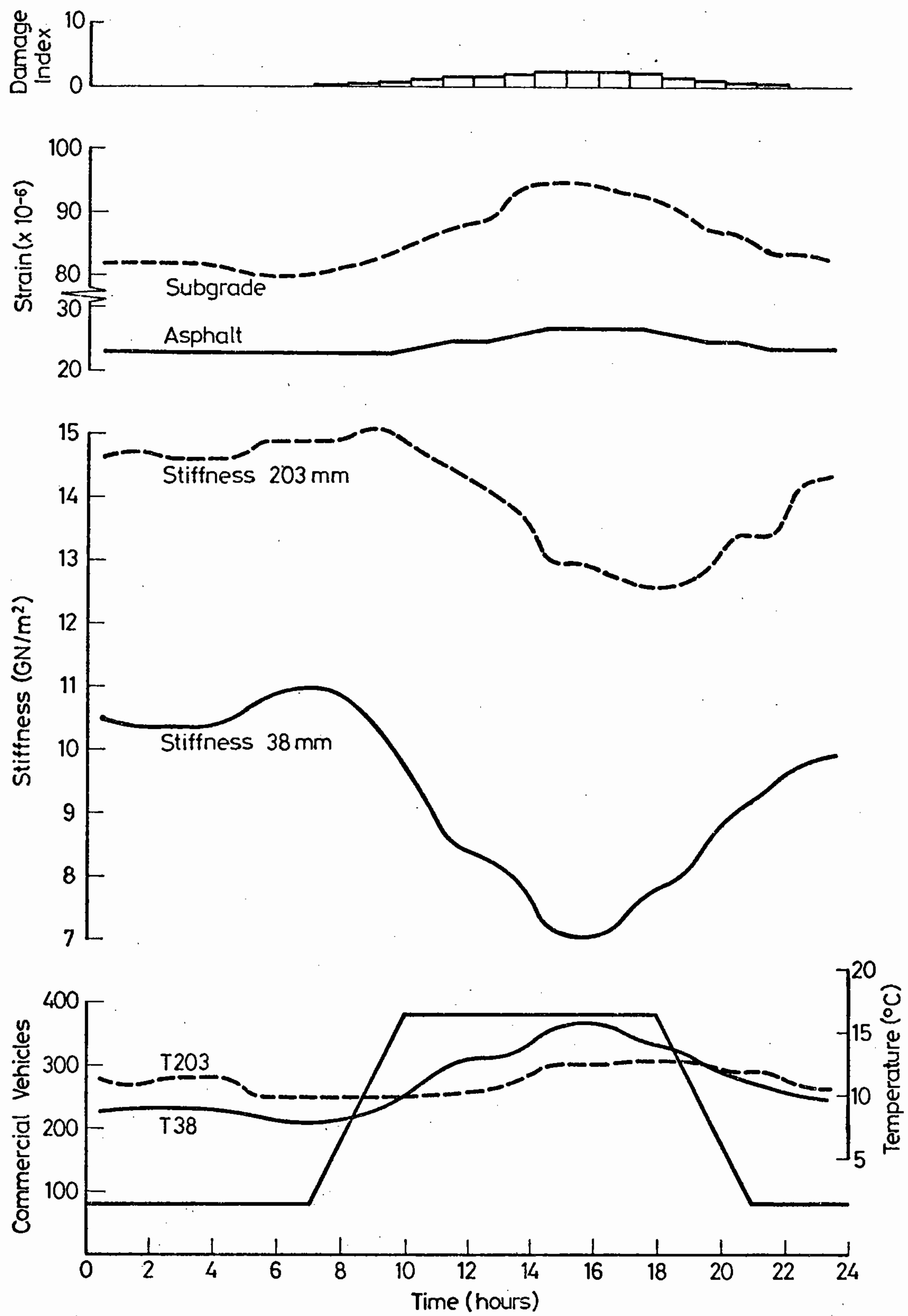

FIG. A.6 STRUCTURAL PARAMETERS FOR A 24-HOUR CYCLE IN OCTOBER

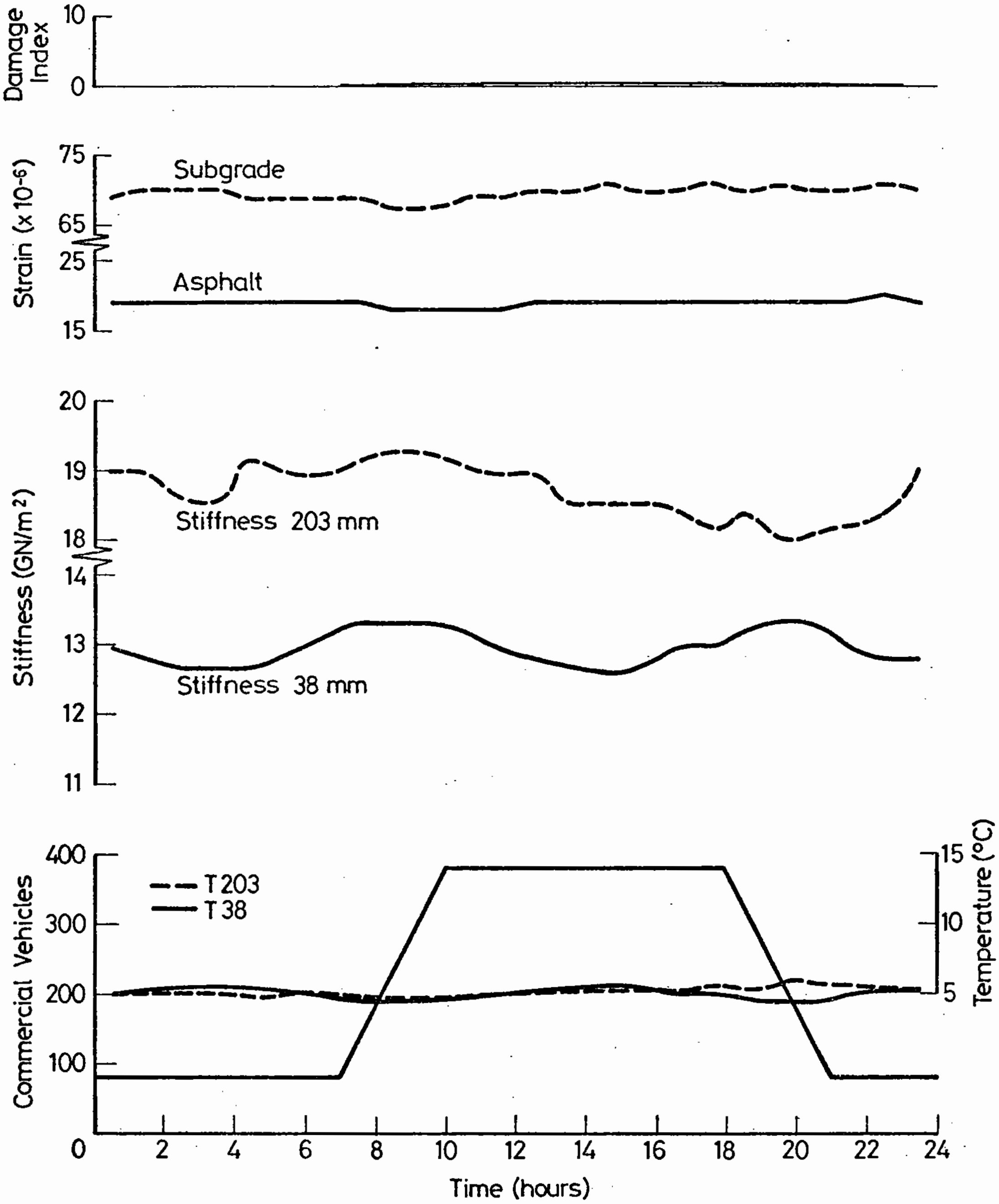

FIG. A.7 STRUCTURAL PARAMETERS FOR A 24-HOUR CYCLE IN JANUARY

Temperature (^{o}C)	E_1 (MN/m^2)	E_2 (MN/m^2)	ε_A ($x10^{-6}$)	ε_s ($x10^{-6}$)	N		Damage index ($^n/_N$)	
					Asphalt	Subgrade	Asphalt	Subgrade
5	12948	18962	19	69	833.7	792.3	6.261	6.588
6	12259	18118	19	72	833.7	680.4	6.261	7.672
7	11600	17303	20	74	651.3	616.9	8.015	8.462
8	10970	16517	21	77	515.0	535.1	10.14	9.755
9	10368	15726	22	80	411.7	466.7	12.68	11.18
10	9817	14880	23	83	332.4	409.1	15.70	12.76
11	9304	14071	24	86	270.9	360.3	19.27	14.49
12	8805	13305	25	89	212.5	318.7	24.56	16.38
13	8323	12526	27	93	153.7	272.3	33.96	19.17
14	7864	11770	28	96	129.0	243.1	40.47	21.47
15	7428	11039	29	101	108.9	202.7	47.93	25.75
16	7012	10270	31	105	79.0	176.4	66.08	29.59
17	6561	9556	33	110	58.5	149.4	89.23	34.94
18	6120	8899	35	116	44.1	123.5	118.4	42.27
19	5705	8280	37	121	33.7	106.2	154.9	49.15
20	5316	7700	39	127	26.2	89.3	199.2	58.45
21	4942	7155	42	133	18.3	75.7	285.2	68.96
22	4591	6658	44	140	14.7	63.0	355.1	82.86
23	4230	6130	47	147	10.7	52.9	487.9	98.68
24	3878	5639	51	156	7.2	42.8	725.0	122.0
25	3667	5184	54	164	5.47	35.78	954.3	145.9
26	3261	4761	58	174	3.88	28.95	1345.4	180.3
27	2988	4363	62	183	2.81	24.17	1857.7	216.0
28	2736	3985	66	194	2.08	19.62	2509.6	266.1
29	2504	3620	71	206	1.46	15.83	3575.3	329.8
30	2273	3286	77	218	0.99	12.92	5272.7	405.0

Table A.6 Calculations to derive equivalent temperature asphalt (ε_A) and vertical strain in the subgrade (ε_S). Damage calculations were carried out for 5220 commercial vehicles, the total number for a 24-hour period, and assumed that a single temperature could be used to characterise all bitumen bound materials.

Fig. A.8 plots some of the data from Table A.6 to a natural scale. It should be noted that fatigue damage in the asphalt layer increases exponentially with temperature to the extent that it is not possible to continue the plot on a natural scale and cover the required temperature range. Thus the full results of Table A.6 have been plotted to a logarithmic scale in Fig. A.9. Close inspection of this figure will show that the log damage-temperature relationship for the asphalt is still one which increases with temperature. It is evident from both Figs A.8 and A.9 that vertical subgrade strain is much less sensitive to temperature than asphalt tensile strain.

The equivalent temperature and factors for converting mean monthly air temperature (obtained from metereological data) to equivalent pavement temperature, summarised in Table A.7, have been obtained as follows.

The total damage for the 24-hour period for the months of April, July, October and January were obtained from Tables A.2 to A.5. The pavement temperature required to produce the same damage in a 24-hour period was then read off from the plots of damage as a function of temperature (Figs A.8 and A.9) and conversion factors calculated for these four months. The conversion factors were then plotted as a function of mean monthly air temperatures (Fig. A.10), thus allowing the conversion factor for other months to be interpolated. The temperatures and factors in Table A.7 are for long term mean temperatures and are appropriate for design purposes. Fig. A.10 is also valid for

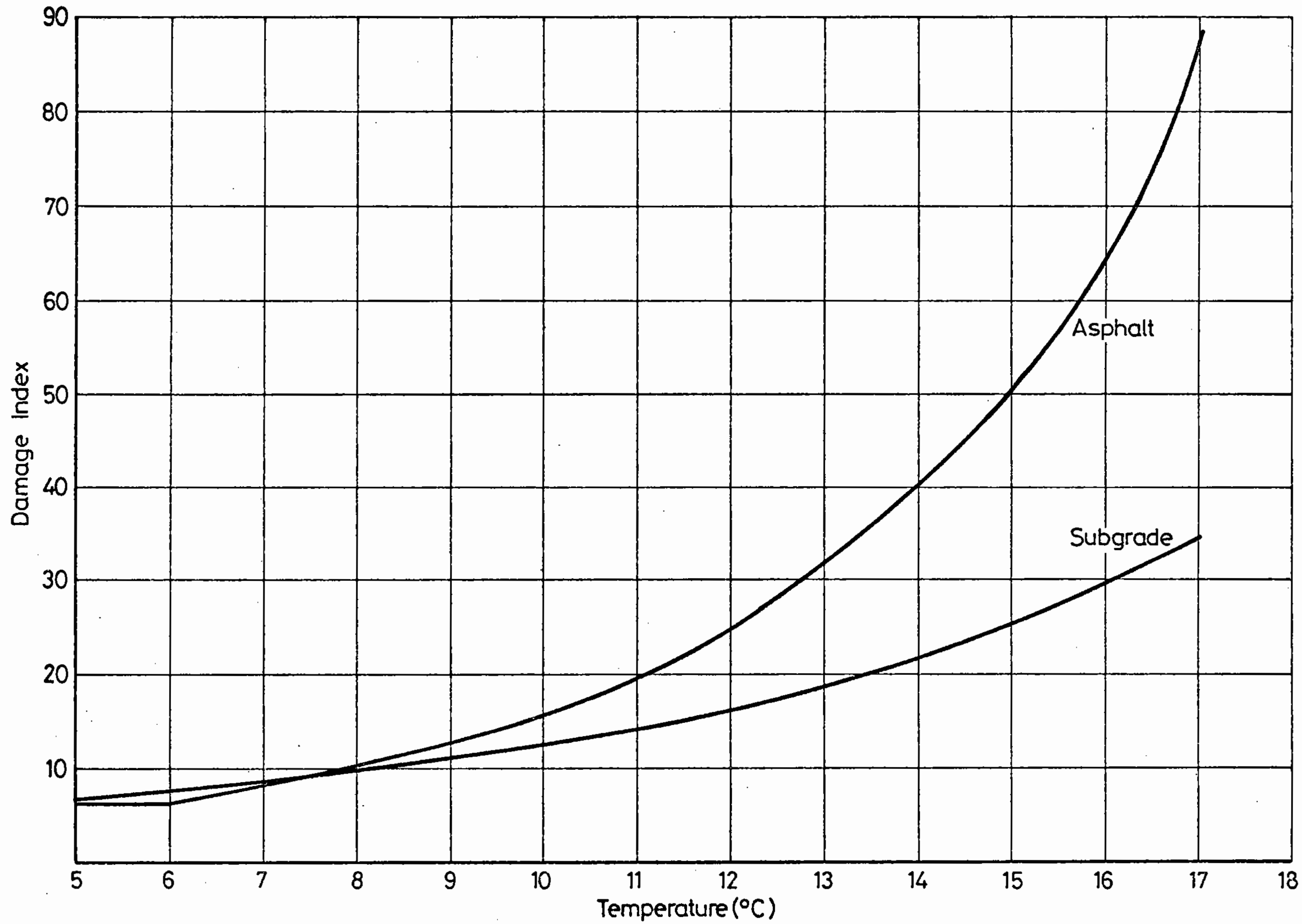

FIG. A.8 DAMAGE INDEX AS A FUNCTION OF TEMPERATURE TO NATURAL SCALES

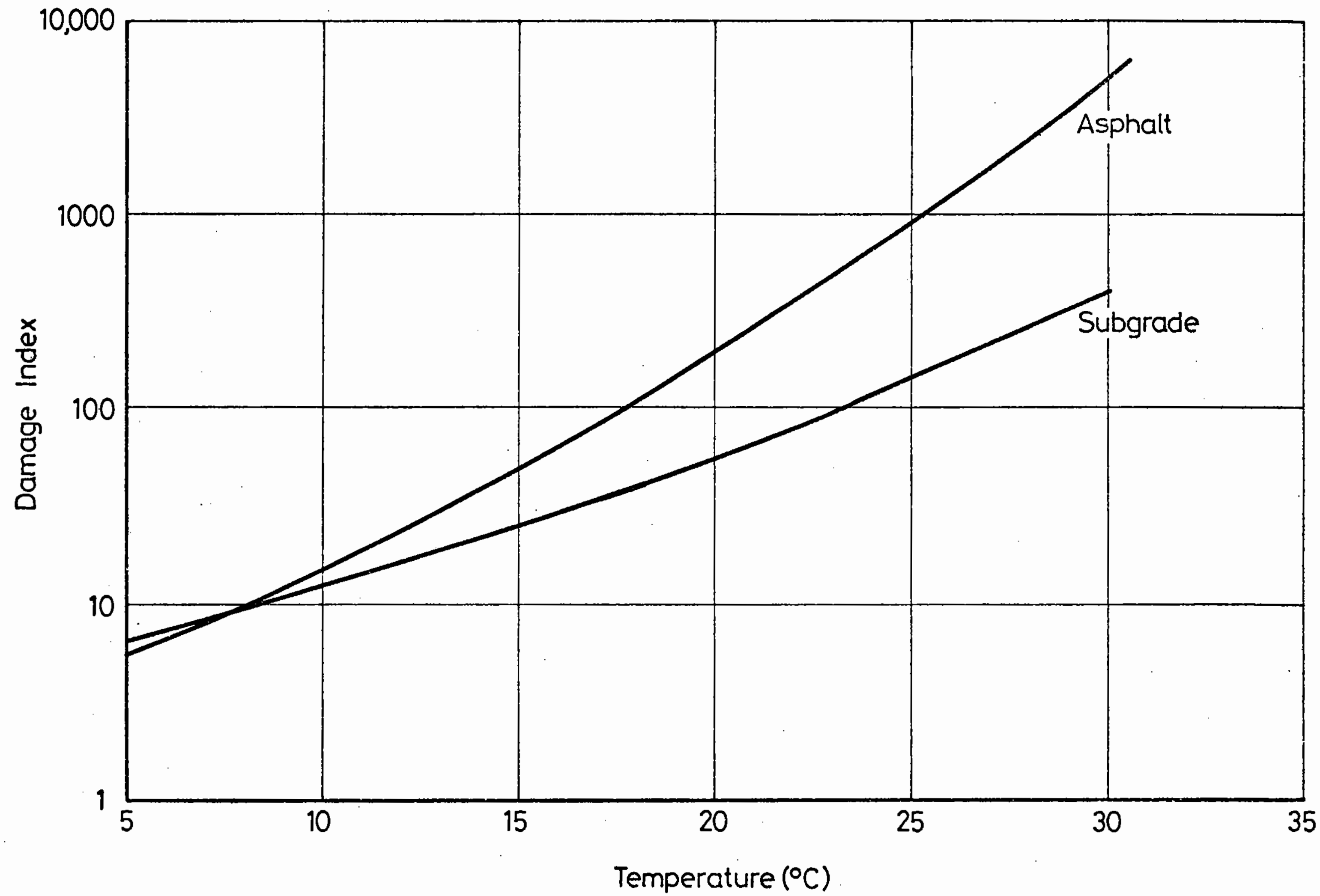

FIG. A.9 DAMAGE INDEX AS A LOGARITHMIC FUNCTION OF TEMPERATURE

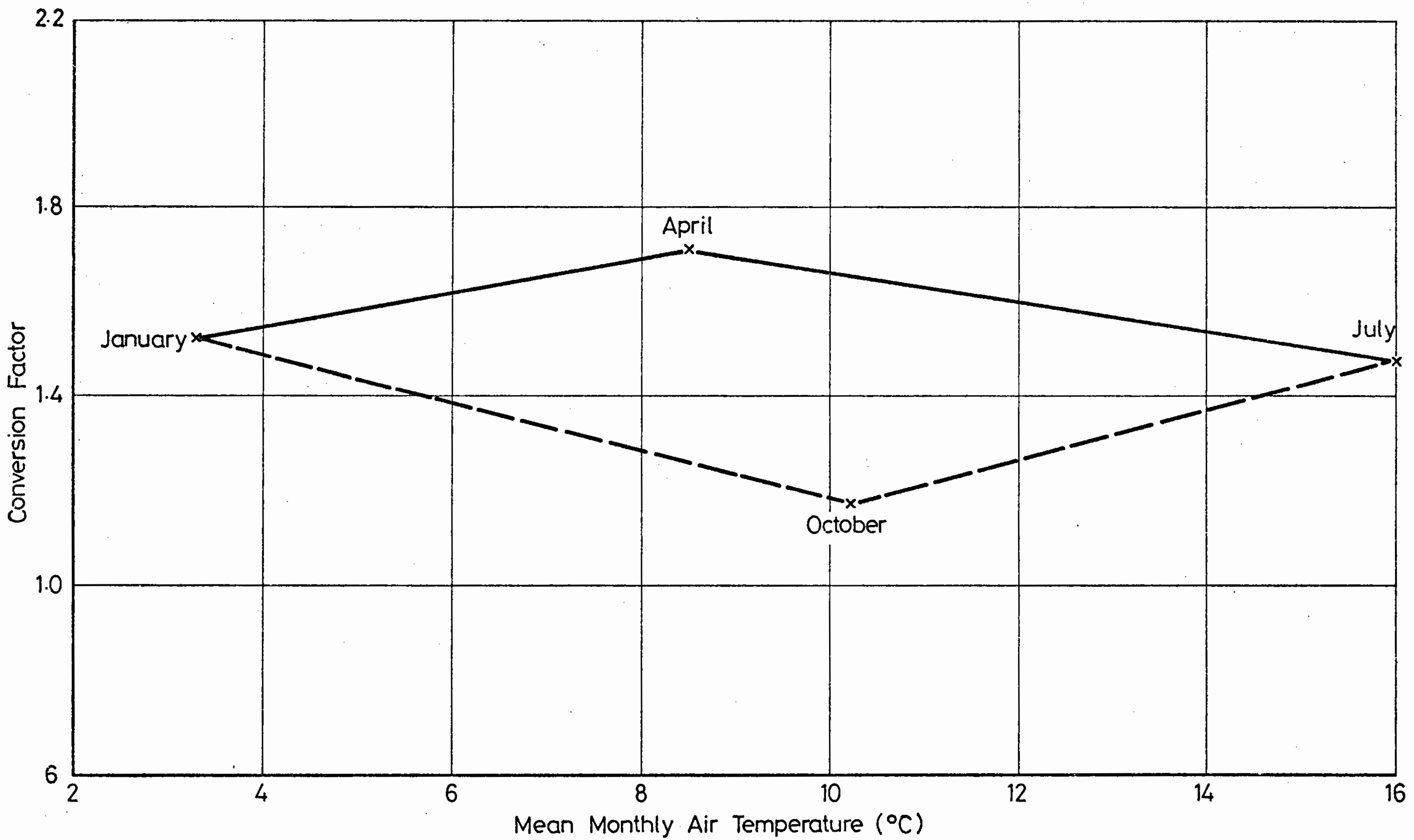

FIG. A.10 TEMPERATURE CONVERSION FACTORS AS A FUNCTION OF MEAN MONTHLY AIR TEMPERATURE

Month	Mean air temperature (^{o}C)	Conversion factor	Design temperature (^{o}C)	Relative damage (fatigue)
January	3.3	1.52	5.0	5.5
February	3.7	1.58	5.8	6.0
March	5.7	1.60	9.1	13.0
April	8.5	1.71	14.5	45.0
May	11.3	1.62	18.3	129.0
June	14.4	1.53	22.0	365.0
July	16.0	1.47	23.5	584.0
August	15.6	1.45	22.6	436.0
September	14.0	1.37	19.2	163.0
October	10.2	1.17	11.95	24.5
November	6.6	1.35	8.9	12.5
December	4.5	1.46	6.6	8.0

Table A.7 Temperature conversion factors and damage for an average year

mean temperatures during a pavement's life, if required for back analysis. Should these mean monthly temperatures fall outside the range 3.3 to 16^{o}C of Fig. A.10 extrapolation parallel to the temperature axis is recommended. This exercise was undertaken for both damage determinants, the results being, for all practical purposes, identical. Thus, in subsequent discussions, only asphalt fatigue is used.

It will be noticed that Fig. A.10 indicates a hysteresis effect on the temperature conversion factors, which can be justified from measurements of temperatures in pavements. Fig. A.11, reproduced from Croney (A5) plots the relationship between mean monthly air and pavement

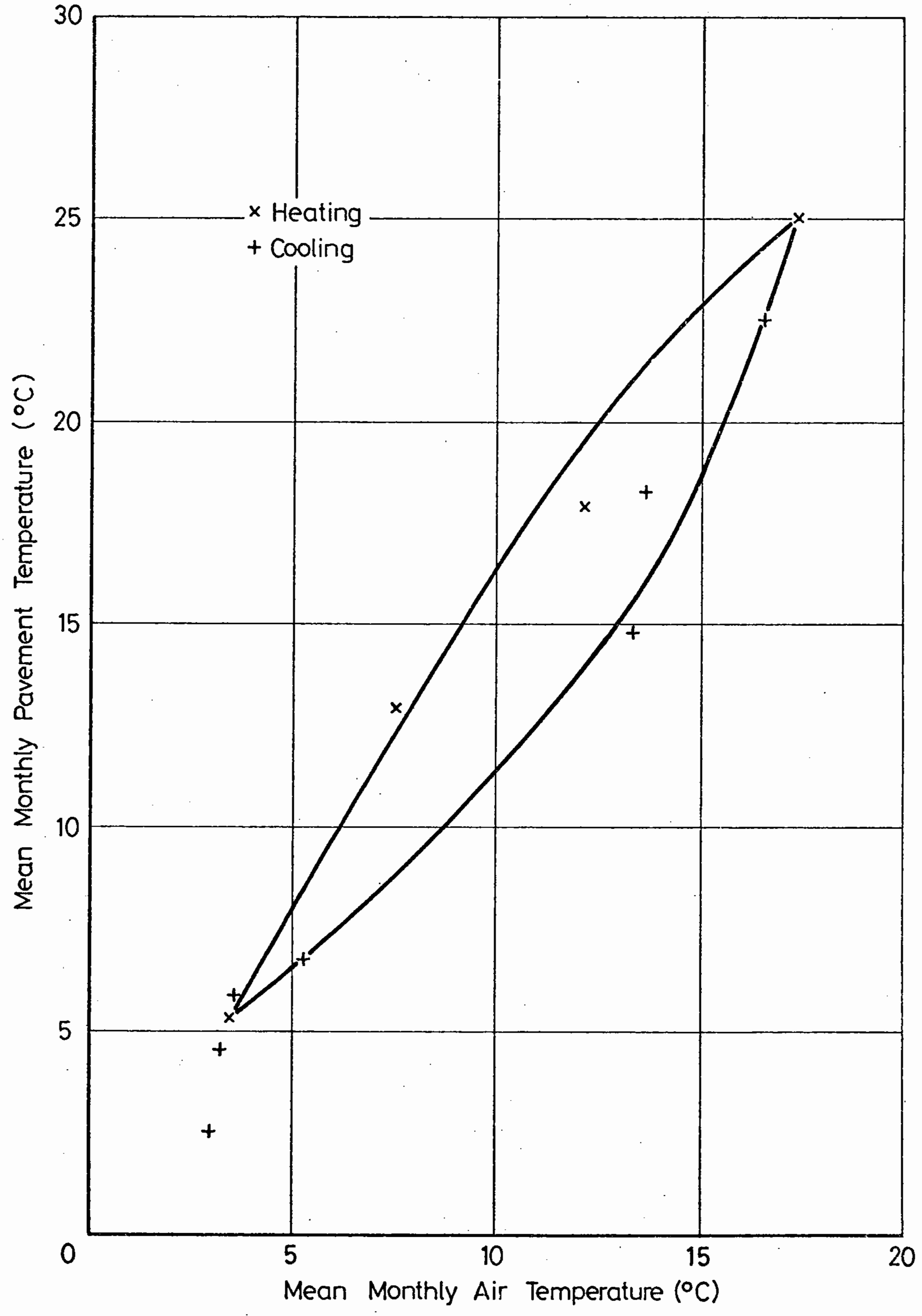

FIG. A.11 MEAN MONTHLY PAVEMENT TEMPERATURE AS A FUNCTION OF AIR TEMPERATURE (AFTER CRONEY, A5) temperatures 38 mm below the surface of a pavement similar to the structure analysed for this work and indicates significant hysteresis. Fig. A.12 plots the equivalent temperature for an average year as a function of mean monthly temperature and shows close similarity with the measurements plotted in Fig. A.10. Hence the temperature hysteresis shown in Fig. A.10 is considered reasonable.

Fig. A.13 is a histogram of the monthly fatigue damage tabulated in Table A.7 for a year assumed to have the long term mean monthly temperatures. In order to economise on cumulative damage calculations for a design year, it is convenient to combine months involving similar damage into climatic periods. It will be noticed that the months October to April contribute relatively little damage to the pavement. In fact, the relative damage for this 7-month period, 114.5 is only 6.4% of the total for the year and the month to month variation is small. These months were, therefore, grouped into a winter period, in which each month has the same temperature. The average mean monthly temperature of 9^{o}C gives a monthly relative damage of 13, and seven months at this relative damage gives a total relative damage of 91. Whilst this under-estimates the actual damage by 20%, with regard to the total damage for the year this error is only 1% and is therefore considered negligible. It will also be noticed that the months of May and September contribute similar levels of damage. Grouping these two months together gives an average monthly design temperature of 18.75^{o}C. This gives a relative damage of 145 per month, a total damage of 290. Since the cumulative damage for this period is 292, the average is regarded as a satisfactory approximation. Because the damage occurring during June, July and August constitutes the major proportion of the damage for the year, and also because of the high sensitivity of the pavement to damage during these months, it is recommended that they be treated separately.

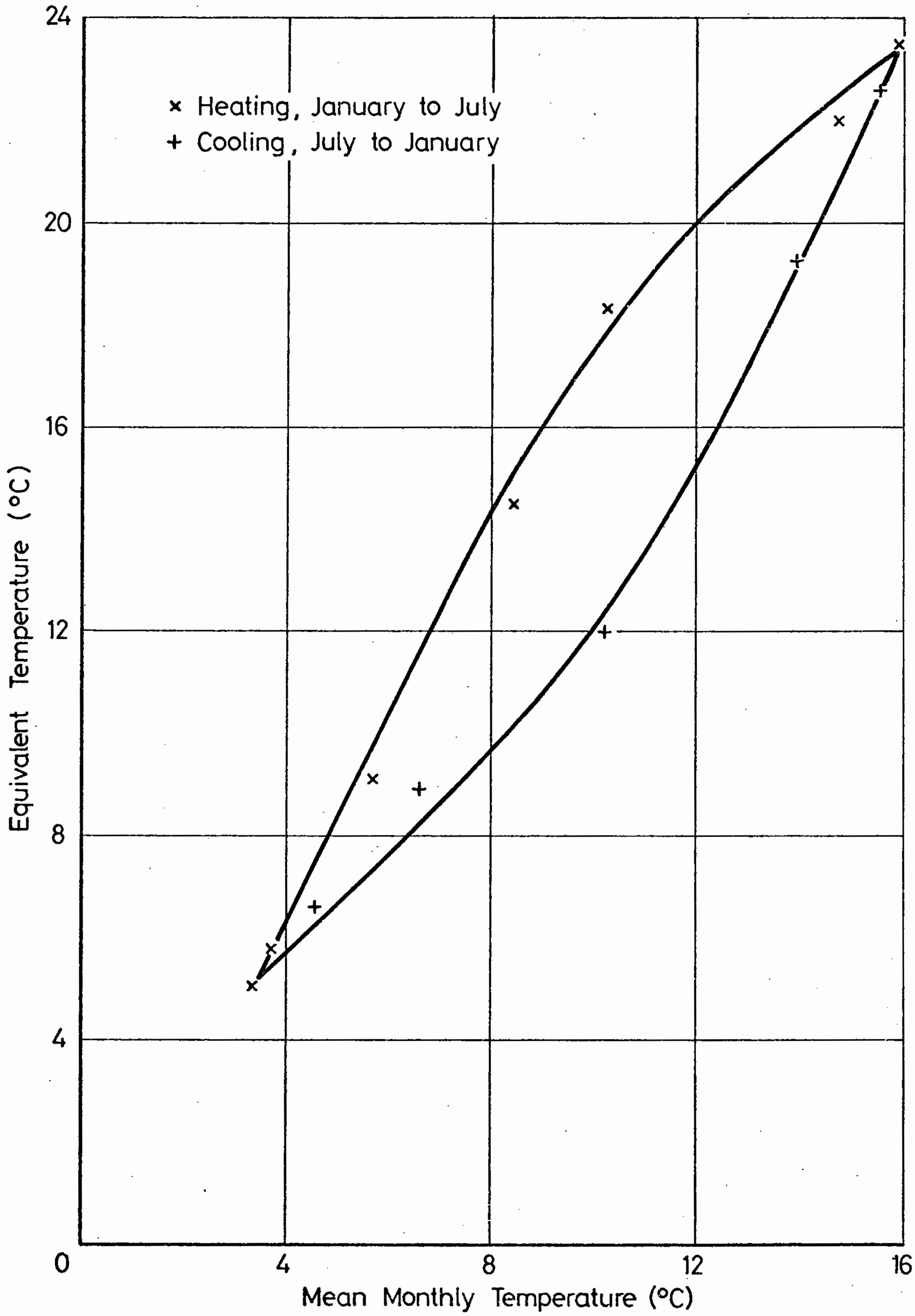

FIG. A.12 EQUIVALENT TEMPERATURE AS A FUNCTION OF MEAN MONTHLY AIR TEMPERATURE

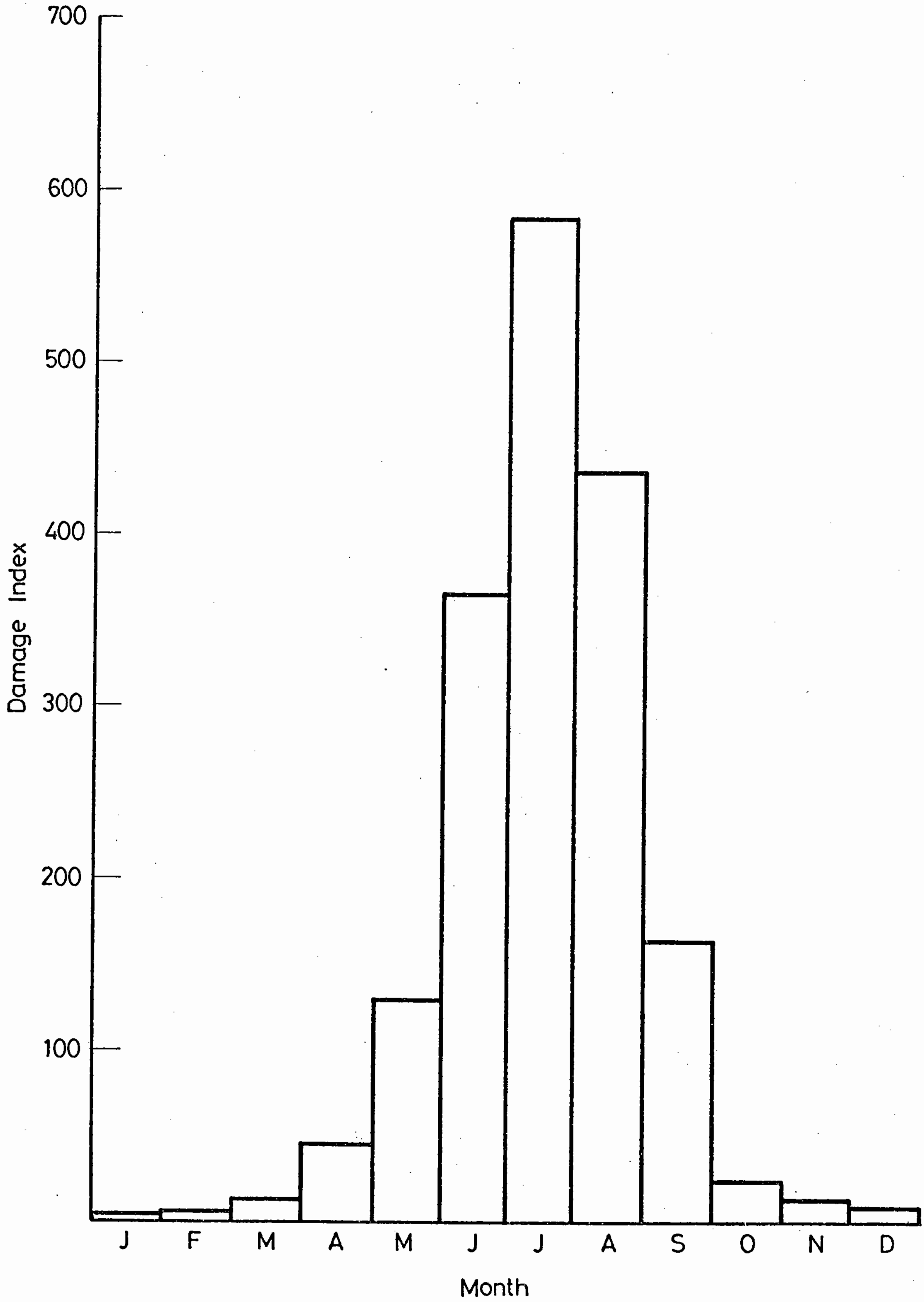

FIG. A.13 HISTOGRAM OF MONTHLY FATIGUE DAMAGE FOR AN AVERAGE YEAR

The cumulative damage computations for a year were therefore reduced to five periods:

(a) Winter - October to April - Mean monthly temperature = $9^{o}C$.

(b) Seasonal change - May and September - Mean temperature = $18.75^{o}C$.

(c) June - Mean temperature = $22^{o}C$.

(d) July - Mean temperature = $23.5^{o}C$.

(e) August - Mean temperature = $22.6^{o}C$.

Another possible simplification is to consider the year to be made up of twelve periods (months) each of the same temperature. The average monthly design temperature from Table A.7 is $13.2^{o}C$ which gives a monthly damage index of 33.8 and a total damage index of 405.6, which is much less than the accumulated damage index for the year. In order to derive the appropriate damage index for the year, the monthly design temperature must be $18.85^{o}C$ which can only be related to the air temperature by an additional empirical factor. This is not considered desirable as it makes the relationship between design temperature and air temperature vague.

The final simplification is to consider one damage period only, for the year, analysing the pavement at this temperature and deriving the damage directly. To do this the design temperature required is $27^{o}C$. It is worth noting that this is close to the temperature of $30^{o}C$ derived by Potter (A12), but increasing the temperature from $27^{o}C$ to $30^{o}C$ represents a 2.5 times increase in damage index, since, as shown in Fig. A.8, the temperature-damage relationship is exponential.

A.4 IMPLEMENTATION

The simplifications in traffic distribution and temperature variation discussed in this Appendix can readily be implemented for studies of highway pavements.

If analysis of an existing structure is required, traffic and temperature data obtained on site, if available should always be used in preference to estimates. However, such data is usually scarce, and when this is so the application of the procedures outlined will give a reasonable approximation in absolute terms and provide a reliable framework for comparative studies. Temperature data is available from the Metereological Office for a large number of weather stations and it should be possible to obtain data from a station or group of stations close to the area of interest. This data is usually available in the form of mean monthly temperatures and the simplification into five climatic periods can then be used.

Application of a cumulative damage approach to design is not recommended at the present time. This is because of the limitations in design criteria currently in use.

The asphalt fatigue strain criterion comes basically from laboratory testing, which has been adjusted to account for the difference between laboratory and site conditions. This adjustment has been made partially on the basis of experimental observations, and in part empirically. The empirical adjustment has been made on the basis of analysis of pavements at a fixed temperature of 15^{o}C and as such is invalid for a system which includes seasonal temperature variation.

The simple deformation criterion, subgrade strain, is totally empirical in its derivation, and as for the asphalt fatigue strain criterion this is based on analyses at a unique annual temperature of 15^{o}C. Application of this criterion within a system which considers seasonal temperature variation is likely to give rise to major errors, and is, in any case, fundamentally incorrect.

Thus, until improved design criteria are available, and in particular with respect to permanent deformation, it is incorrect to attempt to include considerations of cumulative damage in an overall design procedure.

There is some attraction in changing the unique temperature currently used for design to 27°C indicated as a possible simplification in Section A.3.3. This is not considered desirable since the relationship between this design temperature and air temperature has been derived for one condition and locality only. Since the pavement is very sensitive to damage at high temperature, the relationship between design and air temperatures must be well substantiated if it is to be used with confidence for local climatic conditions.

A.5 REFERENCES

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A12. Potter, J.F., "Deformation of road pavements: correlation between elastic theory and measured behaviour of rolled asphalt road-bases", TRRL Report, LR 784, 1977.

APPENDIX B

PAVEMENT PRIMARY RESPONSE PARAMETERS

The figures in this Appendix plot primary response parameter for three-layer structures as functions of the stiffness of the asphalt layer and its thickness, calculated for a standard axle with a dual wheel configuration. Subgrade modulus = 30 MN/m^2.

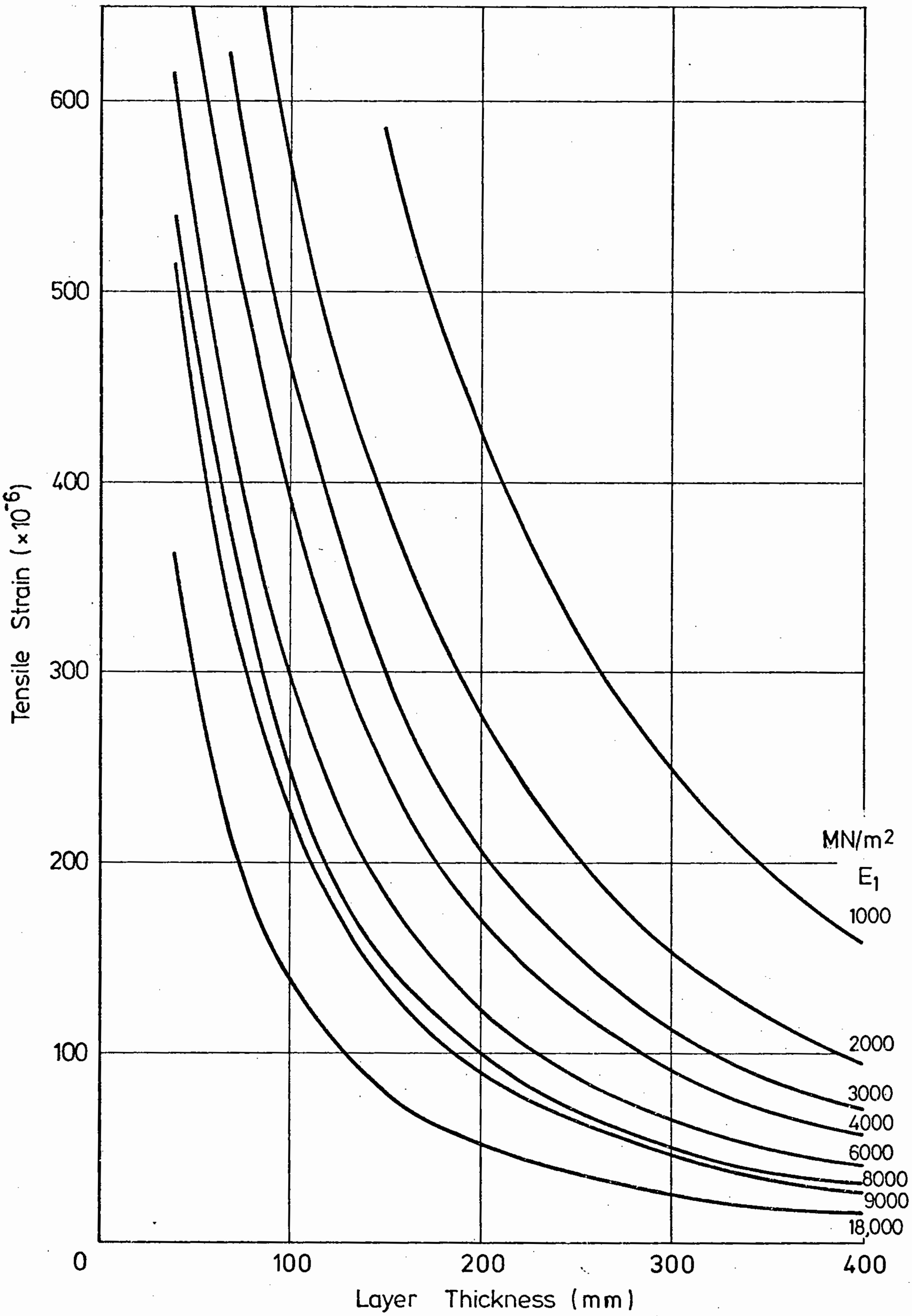

FIG. B.1 PRINCIPAL TENSILE STRAIN AT THE BOTTOM OF THE ASPHALT

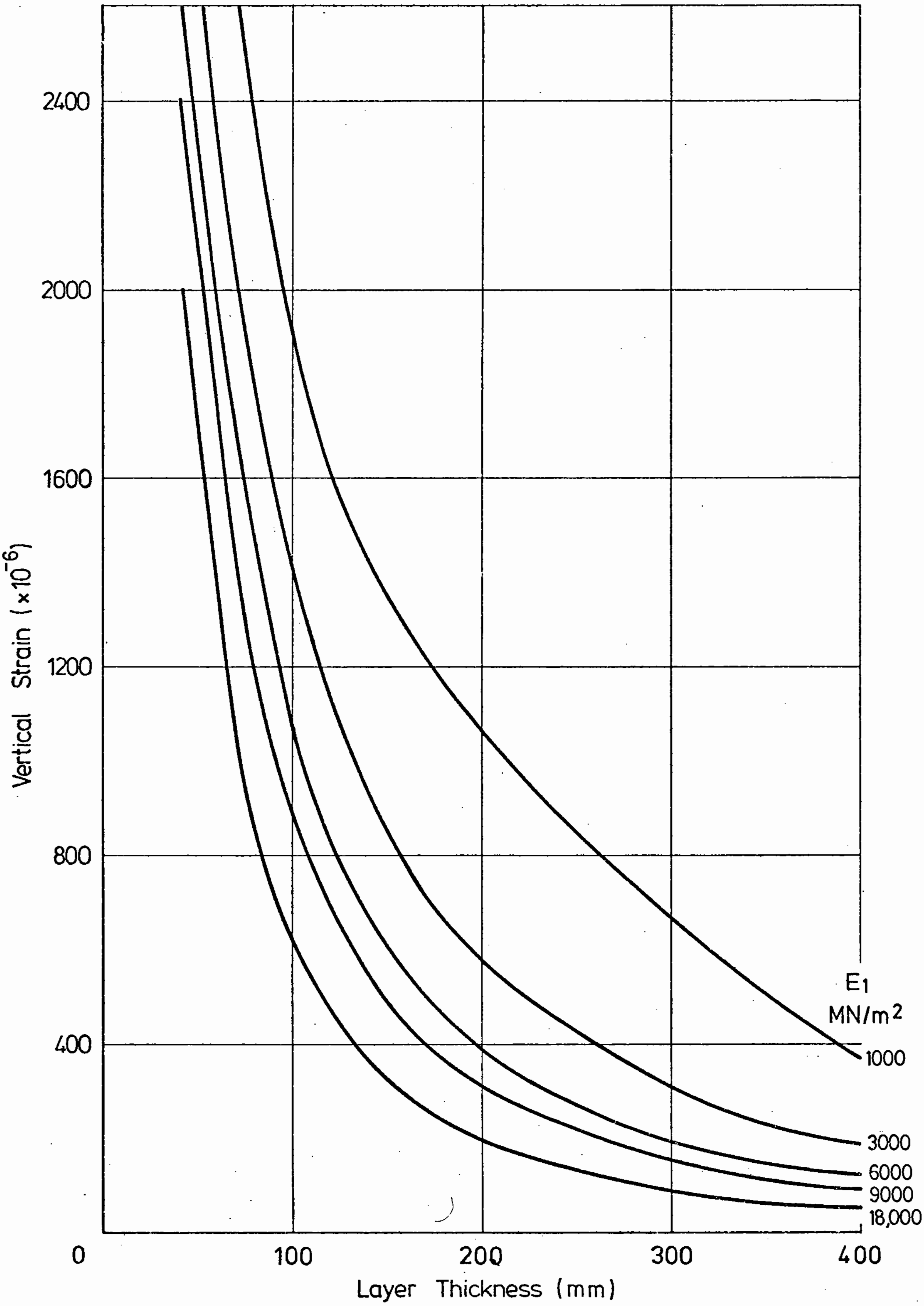

FIG. B.2 VERTICAL STRAIN ON THE SUBGRADE

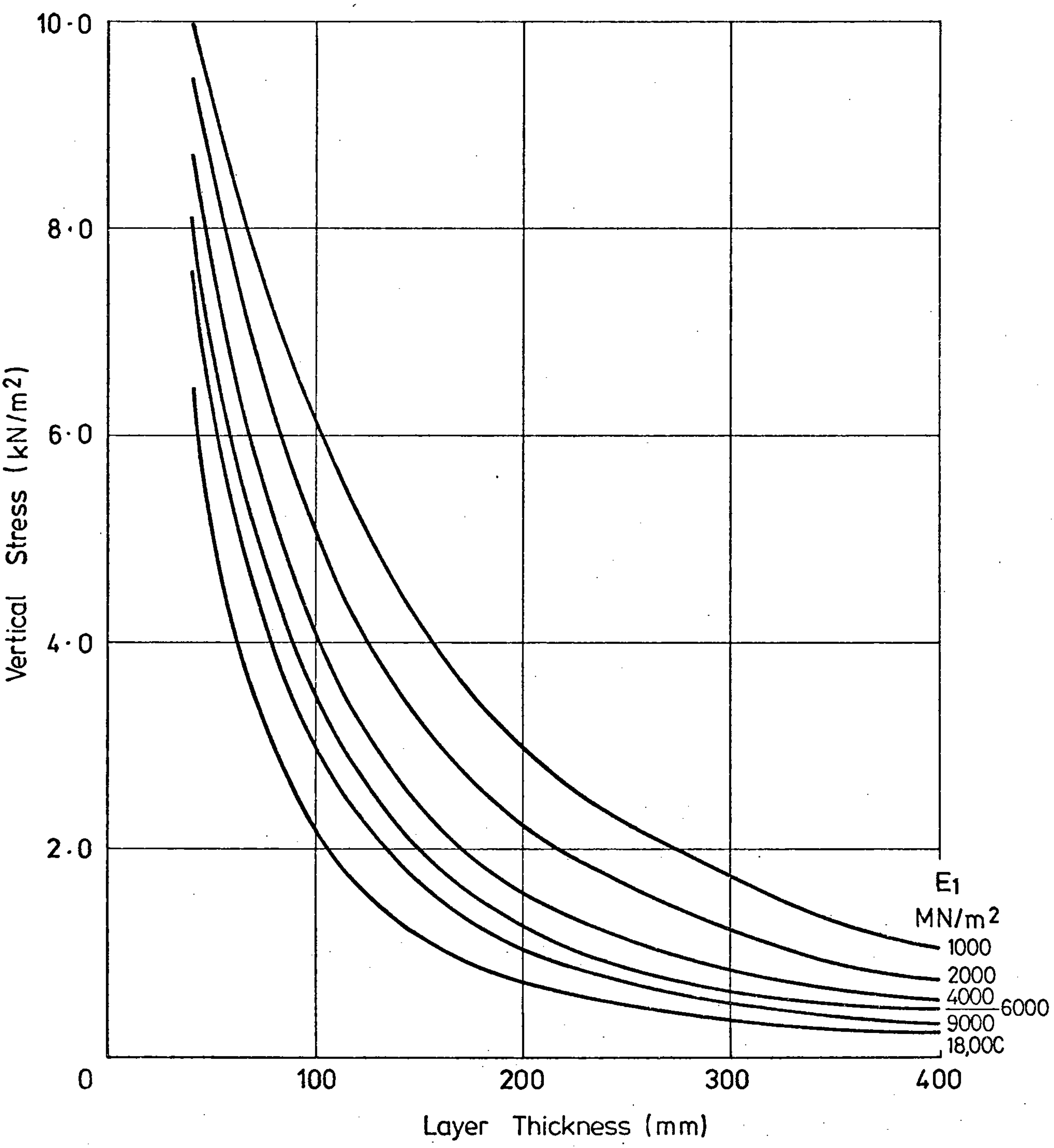

FIG. B.3 VERTICAL STRESS AT THE BOTTOM OF THE SUB-BASE

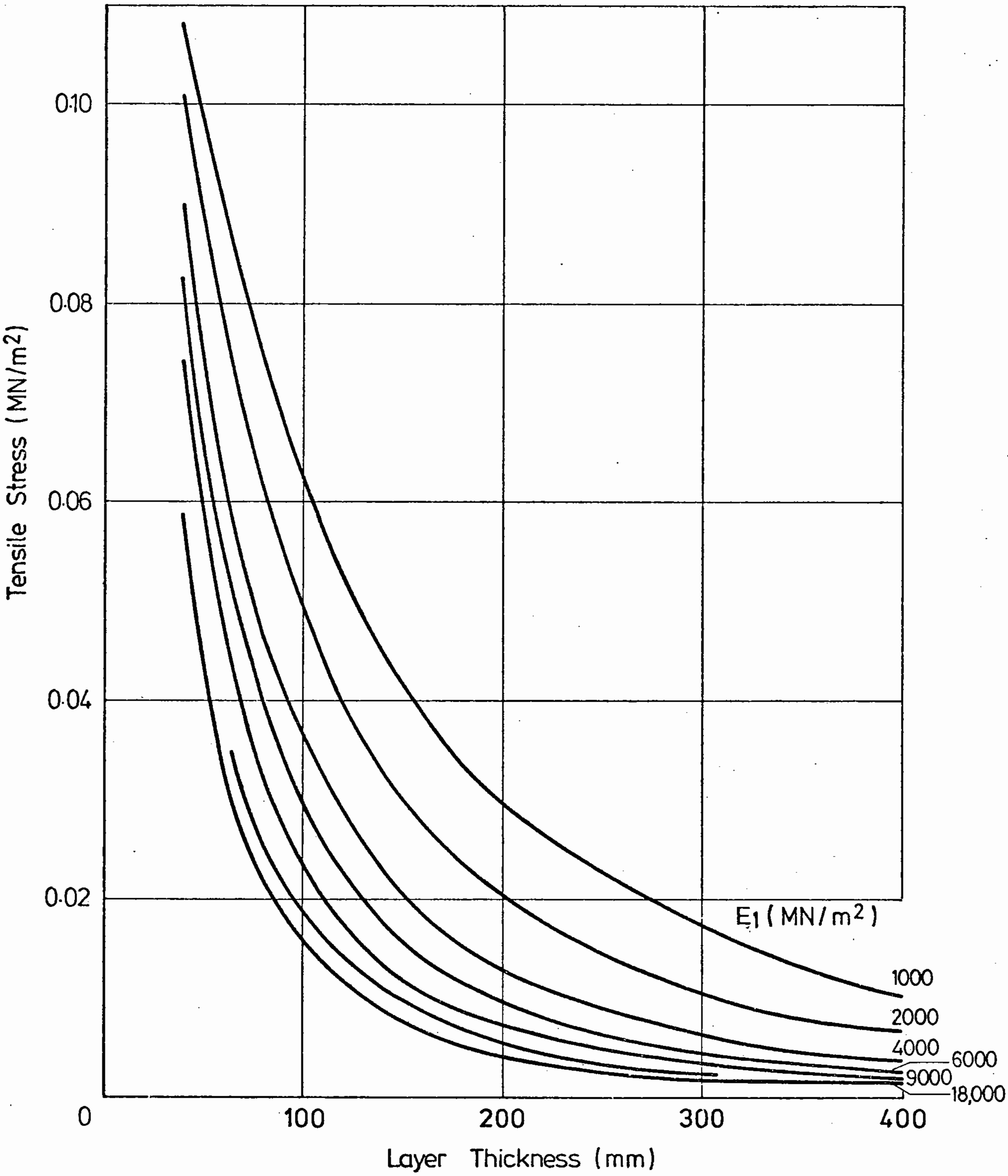

FIG. B.4 TENSILE STRESS AT THE BOTTOM OF THE SUB-BASE

APPENDIX C

USER MANUAL FOR ADEM3

C.1 INTRODUCTION

ADEM3 is a computer program for the analytical design of highway pavements. Two failure modes are considered: fatigue in the asphalt layer, characterised by the horizontal tensile strain at the bottom of that layer and overall deformation of the pavement, characterised by the vertical strain at the top of the subgrade. A further criterion, to limit stress in the sub-base, is available in one of the options but experience with the program suggests that results produced in which this parameter is the design determinant may not be reliable.

The program offers considerable versatility with regard to number of layers in the pavement, combinations of design criteria, and variations in material properties. Seven options are available according to the selection of design criteria and analytical method employed for the unbound granular material as follows:

Options 1-4 consider the unbound material as linear elastic, Options 5-7 treat it as non-linear elastic.

Option 1 - asphalt fatigue criterion.

Option 2 - simple deformation criterion.

Option 3 - selects the most critical condition from fatigue or deformation and designs to satisfy it.

Option 4 - selects the most critical condition from fatigue, deformation or sub-base stress and designs to satisfy it.

Options 5-7 - as for 1-3 but using non-linear elastic characterisation of the unbound layer.

The number of cards for input are dependent on the option chosen, number of layers in the pavement and combination of materials data available. The following list gives a brief description of the input requirements card by card, indicating the format used, whether the information is essential and for which option it is appropriate. All punching starts in column 1 of each card. The units adopted for the system are:

Metres for layer thickness

Meganewtons/square metre for elastic modulus

Kilogrammes/cubic metre for density

<u>CARD GROUP A</u> - provides essential information required for all options

TEXT and DATE

The first 60 characters may be text indicating the purpose of the design, followed by the date.

TEMPERATURE

The design temperature in decimal form (^{o}C) (e.g. 15.0).

VELOCITY

Average vehicle speed in km/hr for the proposed pavement, in decimal form (e.g. 80.0).

NUMBER OF LAYERS (n)

Will be 3, 4 or 5 depending on the desired pavement configuration, in integer form (e.g. .03).

BINDER CONTENT

Entered in decimal form as a proportion by mass of total mix (i.e. 4% of binder content punched as 0.04).

VOID CONTENT

Entered in decimal form as a proportion (i.e. 6% voids punched as 0.06).

SPECIFIC GRAVITY OF AGGREGATE

Entered in decimal form (e.g. 2.70).

SPECIFIC GRAVITY OF BITUMEN

Entered in decimal form (e.g. 1.02).

DESIGN LIFE

Entered in decimal form as millions of standard axles (i.e. 1,000,000 standard axles punched as 1.0).

SUBGRADE CBR

Entered in decimal form (e.g. 5.0).

CARD GROUP B - this group of cards is required for all options but depends upon the number of layers (n) input previously.

If n = 3:

THICKNESS OF UNBOUND LAYER

Entered as a decimal (e.g. 0.2).

If n = 4:

THICKNESS OF SURFACING

Thickness of the top layer in the pavement, entered as a decimal (e.g. 0.075).

STIFFNESS OF SURFACING

Stiffness of the above layer, entered as a decimal (e.g. 6000.0).

THICKNESS OF UNBOUND LAYER

Entered as a decimal (e.g. 0.2).

If n = 5:

THICKNESS OF WEARING COURSE

Thickness of the top layer in the pavement, entered as a decimal (e.g. 0.05).

STIFFNESS OF WEARING COURSE

Stiffness of the above layer, entered as a decimal (e.g. 7500.0).

THICKNESS OF BASE COURSE

Thickness of the second layer down into the pavement, entered as a decimal (e.g. 0.075).

STIFFNESS OF BASE COURSE

Stiffness of the above layer, entered as a decimal (e.g. 6500.0).

THICKNESS OF UNBOUND LAYER

Entered as a decimal (e.g. 0.2).

CARD GROUP C - required for all options.

INITIAL PENETRATION

Penetration of the binder prior to mixing, entered as a decimal (e.g. 100.0).

INITIAL SOFTENING POINT

Softening point of the binder prior to mixing, entered as a decimal. This should be determined according to the method of BS 4692 (e.g. 44.1).

RECOVERED PENETRATION

Penetration grade of the binder after mixing and laying, entered as a decimal (e.g. 65.0).

RECOVERED SOFTENING POINT

Softening point of the binder after mixing and laying, entered as a decimal. This should be determined according to the ASTM test specification (e.g. 50.6).

MIX STIFFNESS

Stiffness of the asphalt mix used in the design layer, in Meganewtons/square metre, entered as a decimal (e.g. 6250.0).

Considerable variation is possible among this card group. If only the initial penetration is known, a common situation, all the subsequent parameters will be calculated within the program. However, if data is available for the other parameters, it should be included. With regard to specifying mix stiffness, care should be taken to ensure that it relates to the appropriate temperature and loading time. If data is not available for any of the above parameters, the card should be left blank.

CARD GROUP D:

OPTION

1-7 depending on the required set of design criteria, entered as an integer, for all options.

NUMBER OF DESIGN ITERATIONS

This is a safeguard against the program continuing in an endless loop, the design iteration being stopped after the specified number of steps. It has been found that if a design is not complete after 15 iterations, there is a high probability of data error. Enter as an integer. To be supplied for all options (e.g. 15).

SUBGRADE/SUB-BASE MODULAR RATIO

To be supplied only for options 1-4. A value of 2.5 should be used unless there is very strong evidence to indicate otherwise.

CARD GROUP E - this group completes the input requirement for the program and is different for each option. The detailed requirements are therefore listed for each option in turn.

Option 1

INDEPENDENT FATIGUE CRITERION

This parameter is set to 0 (zero) if the internal system for determining the allowable fatigue strain is to be used and to 1 (one) if a fatigue life-strain line appropriate to the mix concerned and suitable for application to a full-scale pavement is available. Entered as an integer.

If fatigue data is available, i.e. the above parameter is set to 1, five cards are necessary to input the fatigue data, each having the following format.

STRAIN, LIFE

Strain (in microstrain) is punched as a decimal number starting in the first column. It is the limiting strain for a given life. Life is punched after strain with a gap of two spaces as a decimal number in units of millions (i.e. 1 million is punched as 1.0). It is the life corresponding to the previously punched strain. Each of the five cards contains two parameters, strain and life starting with the shortest life and increasing in order to the longest life. The design life should lie within the range of the input values (e.g. 30.0␣␣0.4, 20.0␣␣3.0, 15.0␣␣20.0, 10.0␣␣80.0, 7.0␣␣950.0 - Note: ␣ = blank space).

Option 2

No data card required.

Option 3

As for Option 1.

Option 4

As for Option 1, followed by:

LAYER DENSITIES

The densities of each layer in the pavement in kilogrammes per cubic metre entered as decimals with two spaces between individual numbers (e.g. 2240.0␣␣2000.0␣␣1800.0).

Option 5

COEFFICIENTS OF HORIZONTAL PRESSURE

Coefficients for calculating horizontal stresses from vertical self weight stresses for the non-linear treatment of the granular layer. These coefficients are a function of the total thickness of the granular layer and can be obtained from Fig. C.1. Four coefficients are required,

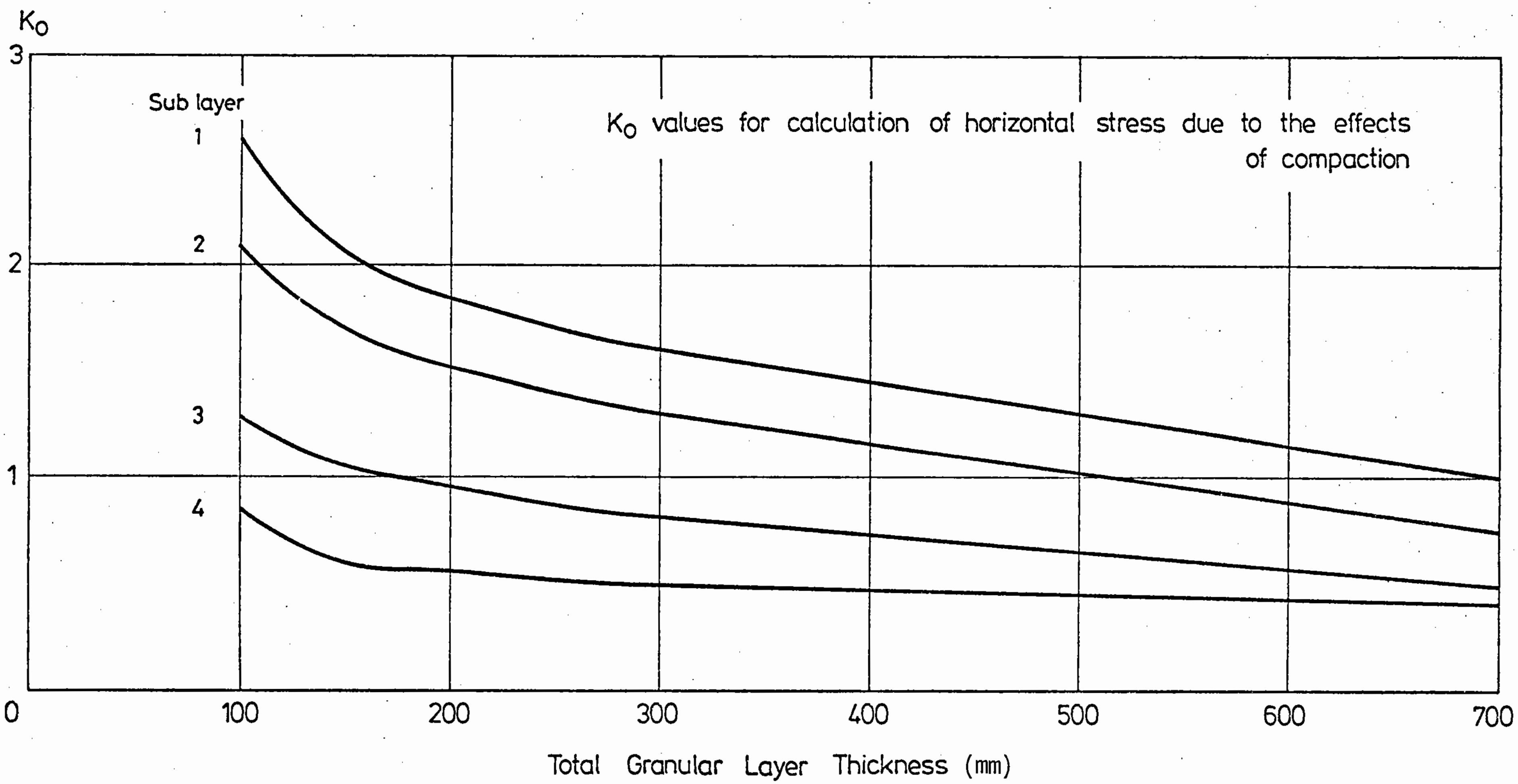

FIG. C.1 K_o VALUES FOR A RANGE OF GRANULAR LAYER THICKNESSES CONSIDERED AS FOUR SUB-LAYERS one for each granular sub-layer, entered on one card as decimals, with two spaces between each (e.g. 1.5␣␣1.3␣␣1.0␣␣0.85).

K_1:

First constant in the granular material model. Four values are required, one for each sub-layer, entered on one card as decimals, with two spaces between each number. Unless material tests indicate otherwise, a value of 400 is generally suitable for each sub-layer (e.g. 600.0␣␣600.0␣␣600.0␣␣400.0).

K_2:

Second constant in the granular material model. Four values are required, one for each sub-layer entered on one card as decimals, with two spaces between each number. Unless material tests indicate otherwise, a value of 0.6 is generally suitable for each sub-layer (e.g. 0.6␣␣0.6␣␣0.6␣␣0.6).

INITIAL MODULI

Starting values of modulus for the iteration procedure used to derive the granular moduli. Four values are required, one for each sub-layer. Accurate estimates will reduce computing time taken and therefore computing cost. Values of 80.0, 60.0, 40.0 and 30.0 will be adequate until experience in estimating is obtained. Entered as decimals on one card, with two spaces between each number (e.g. 80.0␣␣60.0␣␣40.0␣␣30.0).

LAYER DENSITIES

As for Option 4.

DAMPING FACTOR AND GRANULAR MATERIAL FAILURE CRITERION

Three numbers are needed which should be 2.0, -1.0 and -2.2, entered as decimals with two spaces between each number. The first number, the damping factor, may be increased if difficulty is experienced in obtaining a convergent modulus iteration. The other two numbers must <u>NOT</u> be changed without careful reference to a description of the material model used in the program for granular materials, and sound experimental data.

NUMBER OF MODULUS ITERATIONS

This is a safeguard against the program continuing in an endless loop within the modulus iteration procedure. If this procedure has not converged within 15 iterations it is unlikely to do so.

The last cards are as for Option 1.

Option 6

As for Option 5 omitting the Option 1 cards from the end.

Option 7

As for Option 5.

C.2 NOTES ON PARAMETERS

K_1:

This is the first constant in the stress-modulus relationship used for unbound granular materials ($E = K_1\theta^{K_2}$). Values range from approximately 200.0 for poor low grade material to 600.0 for good quality well compacted material.

K_2:

This is the second constant in the stress-modulus relationship used for unbound granular materials ($E = K_1\theta^{K_2}$). Its value does not vary greatly with material quality, 0.6 being a usable average.

Alternative values for K_1 and K_2 can be obtained from references C1 and C2.

COEFFICIENT OF HORIZONTAL PRESSURE

As indicated, these coefficients are necessary to calculate the horizontal self-weight stresses. The coefficients are a function of the thickness of the granular layer and can be obtained from Fig. C.1 for layers from 100 to 700 mm thickness.

GRANULAR MATERIAL FAILURE CRITERION

The numbers -1.0 and -2.2 are limiting q/p ratios for unbound granular material. -2.2 is the ratio at which failure occurs and -1.0 is the ratio at which the stress-modulus relationship starts to be modified. Unless the user has undertaken a significant study of the behaviour of unbound granular materials which indicates different values of the limiting q/p ratios, -1.0 and -2.2 must be used.

SOFTENING POINTS

Initial softening point is used for the determination of fatigue life. In order to be compatible with the research which produced this prediction system, the initial softening point must be determined by the British Standard Method.

Recovered softening point is used for the determination of asphalt stiffness. In order to be compatible with the stiffness estimation system the recovered softening point must be determined by the ASTM method.

It is possible to convert between ASTM to British Standard softening points by use of the equation SP(ASTM) = SP(BS) + 1.5.

C.3 COST OF USING THE PROGRAM

Since the speed at which the program will run and therefore its cost depends on details of the computer system, this will vary from place to place. The figures given are for the University of Nottingham's ICL 1906S computer using a pre-compiled program and are according to the charging algorithm used on this machine.

Cost is a function of the number of design iterations, design criteria and the use of linear or non-linear analysis.

A linear system using one design criterion (Option 1 or 2) will usually cost 2-3 units; one using 3 design criteria, 8 units.

For non-linear systems, Options 5-7, the difference between using 1 or 2 criteria is small, the chief cost being in the modulus iteration subsystem. The total cost may exceed 40 units, depending on the details. Generally, the stiffer the pavement and the more accurate the initial estimate of the modulus values for the unbound layer, the cheaper the design calculation.

C.4 PROGRAM ERRORS

The most common error, particularly when the user is unfamiliar with the input format of the program, is incorrectly formatted data. Data must be checked very carefully before a program error can be suspected.

If it is still impossible to obtain a satisfactory result, the output obtained, and a copy of the input data must be sent to:

Professor P.S. Pell
Department of Civil Engineering
University of Nottingham
Nottingham NG7 2RD.

C.5 REFERENCES

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APPENDIX D

CEMENT TREATED HIGHWAY MATERIALS

D.1 INTRODUCTION

A detailed review of the literature relating to the behaviour of cement treated materials has been reported elsewhere (D1). This Appendix reports the investigations which arose from that review and gives a summary of the principal findings. The literature examined is listed and for more information readers are referred either to the review (D1) or to the articles listed.

D.2 ANALYSIS OF SOIL-CEMENT PAVEMENTS

A pavement section was obtained from Road Note No. 29 (D2) for analysis in order to investigate the sensitivity of the stress and strain distribution through the pavement to variations in the elastic properties of the soil cement. The structure chosen, detailed in Fig. D.1, is for 1.5 million standard axles, and is the most heavily trafficked pavement in which a soil cement road base can be used.

These analyses were carried out in two stages. The first stage involved calculating the moduli in the granular sub-base by means of the iterative program described in Chapter 4 but not including failure criteria, using an equivalent stiffness for the combined roadbase and surfacing. This was followed by a detailed analysis of the pavement structure, the base and surfacing being considered as separate layers. Stresses and strains were calculated on the vertical axis of symmetry of the dual wheel load. The first stage of this analysis indicated that variations in surfacing stiffness had little effect on the derived moduli for the granular layer, and so constant moduli were used while the effects of variation of E and ν in soil cement were investigated.

Surfacing	$E = 5{,}000\ MN/m^2$ $\nu = 0.4$	70 mm
Base	$E = 6{,}000\ MN/m^2$ $10{,}000\ MN/m^2$ $17{,}000\ MN/m^2$ $\nu = 0.1$ 0.15 0.3	150 mm
Sub-base	$E = 75\ MN/m^2$ $\nu = 0.3$	330 mm
Subgrade	CBR = 3%	

FIG. D.1 PAVEMENT SYSTEMS ANALYSED TO INVESTIGATE THE EFFECT OF MATERIAL VARIABILITY IN THE CEMENT STABILISED BASE

Some of the results from this investigation are presented in Tables D.1 and D.2 and plots of typical stress and strain distribution through the pavement are plotted in Figs D.2 and D.3. The tabulated results prompt the following observations.

(1) Varying Poisson's ratio of the cement treated layer does not have a substantial effect upon the calculated stresses and strains in the other layers of the pavement.

(2) Increasing Poisson's ratio does have a significant effect on the maximum tensile stress in the bottom of the cement treated layer. These results agree with Mitchell et al (D14) who cite work by Hadley et al (D54) and Fossberg (D50) which indicates changes of 15% or less in maximum tensile stresses for a Poisson's ratio changed from 0.125 to 0.375 though from Table D.2 the effect of increasing ν from 0.1 to 0.3 is to increase the maximum tensile stress by approximately 15%. However, Mitchell et al (D15) consider this change to be insignificant, although it can represent a significant change in stress level on a low strength material.

(3) Increasing the modulus of the cement treated layer causes a significant increase in the tensile stress in the bottom of this layer.

(4) The horizontal strain at the base of the asphalt is compressive, but large tensile strains are developed in the vertical direction, towards the surface of the layer. This situation requires investigation and consideration should be given to the type of fatigue model applicable to this case, i.e. whether a constant stress model or a constant strain model is applicable.

A further observation, not evident from Tables D.1 and D.2, is that the tabulated horizontal stress, σ_{yy} (tangential), is not always a

System number			1 E = 6,000				2 E = 10,000				3 E = 17.000			
Position number	Layer number	Depth (mm)	σ_{zz}	σ_{yy}	ε_{zz}	ε_{yy}	σ_{zz}	σ_{yy}	ε_{zz}	ε_{yy}	σ_{zz}	σ_{yy}	ε_{zz}	ε_{yy}
			(kN/m^2)		(x10^{-6})		(kN/m^2)		(x10^{-6})		(kN/m^2)		(x10^{-6})	
1	1	35	1.4	468	-75	56	2.0	439	-72	52	-4.5	384	-65	45
2	1	70	22.0	271	-41	28	16.0	283	-42	33	13.7	264	-38	32
3	2	70	22.0	212	-8	28	16.0	388	-10	33	13.7	628	-10	32
4	2	145	39.0	-185	13	-30	36.0	-181	7	-17	35.0	-166	4	-9
5	2	220	28.0	-637	30	-97	24.0	-800	22	-72	20.0	-1000	16	-53
6	3	220	28.0	2.5	201	-97	24.0	5.0	163	-72	20.0	6.0	130	-53
7	4	385	22.5	1.3	170	-103	21.0	3.0	144	183	19.0	5.0	119	-66
8	6	550	21.7	-4.4	188	-125	21.0	-3.0	164	-103	20.0	-1.0	139	-88
9	7	550	21.7	8.4	309	-125	21.0	9.0	269	-106	20.0	9.0	229	-88
10	7	650	22.7	9.9	269	-103	22.0	10.0	237	-93	21.0	10.0	205	-78

Table D.1 Stresses and strains in soil cement pavements - variation due to E

System number			1 $\nu = 0.10$				2 $\nu = 0.15$				3 $\nu = 0.30$			
Position number	Layer number	Depth (mm)	σ_{zz}	σ_{yy}	ε_{zz}	ε_{yy}	σ_{zz}	σ_{yy}	ε_{zz}	ε_{yy}	σ_{zz}	σ_{yy}	ε_{zz}	ε_{yy}
			(kN/m^2)		($\times 10^{-6}$)		(kN/m^2)		($\times 10^{-6}$)		(kN/m^2)		($\times 10^{-6}$)	
1	1	35	-2.5	441	-72	52	-2.0	439	-72	52	-1.8	428	-72	50
2	1	70	16.3	285	-42	33	16.0	283	-42	33	16.6	274	-42	31
3	2	70	16.3	372	-6	33	16.0	388	-10	33	16.6	455	-27	31
4	2	145	36.5	-177	6	-17	36.0	-181	7	-17	35.8	-193	12	-17
5	2	220	23.9	-774	15	-73	24.0	-800	22	-72	23.5	-895	48	-70
6	3	220	23.9	4.5	164	-73	24.0	5.0	163	-72	23.5	4.8	159	-70
7	4	385	20.6	3.3	144	-83	21.0	3.0	144	-83	20.4	3.0	141	-81
8	6	550	20.7	-2.6	164	-107	21.0	-3.0	164	-103	20.6	-2.4	161	-104
9	7	550	20.7	8.7	270	-107	21.0	9.0	269	-106	20.6	8.7	265	-104
10	7	650	21.9	10.1	238	-93	22.0	10.0	237	-93	21.8	10.1	234	-91

Note: Compressive stress and strain are positive.

Table D.2 Stresses and strains in soil cement pavements - variation due to Poisson's ratio

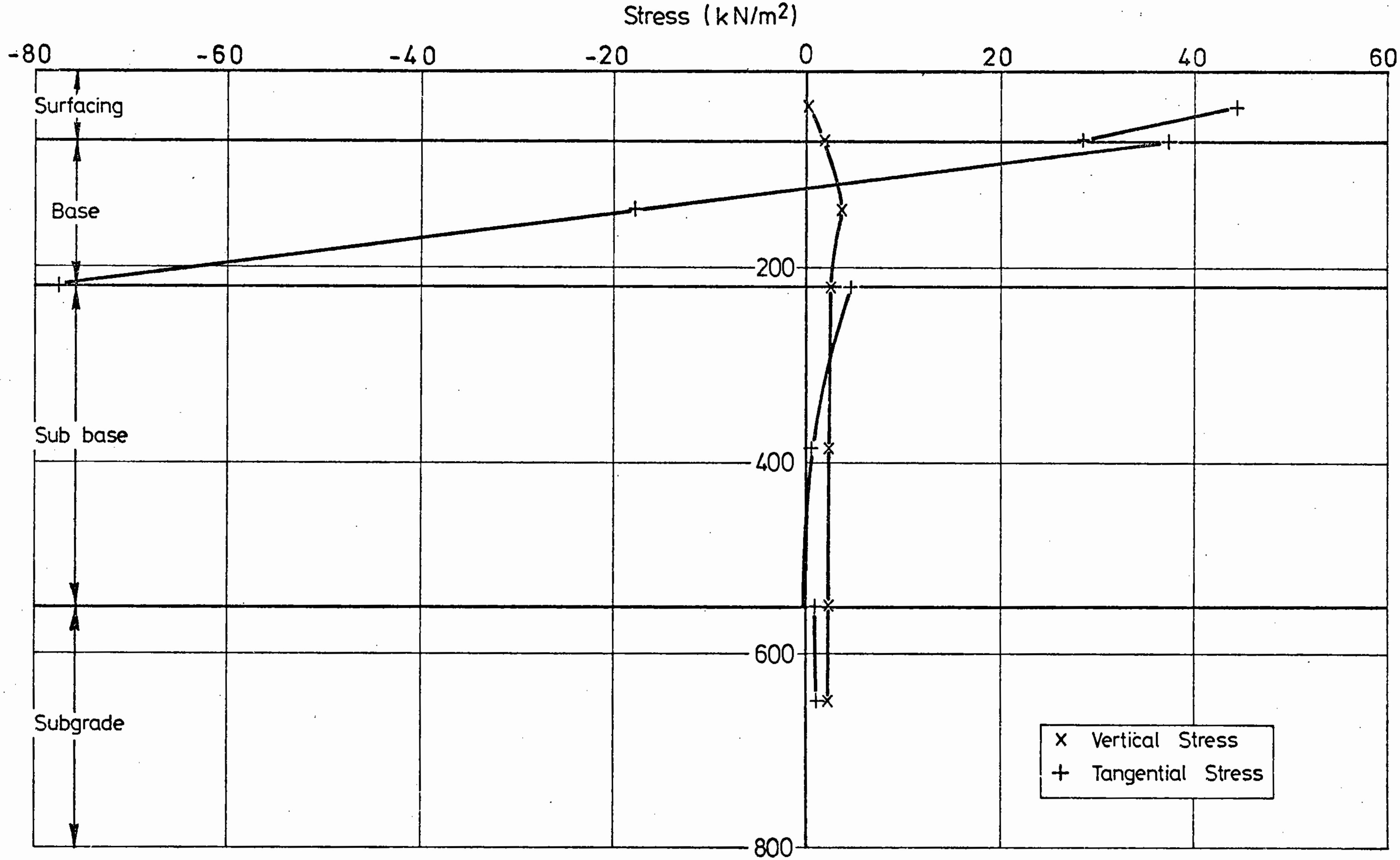

FIG. D.2 STRESS DISTRIBUTION IN A PAVEMENT WITH A SOIL CEMENT BASE

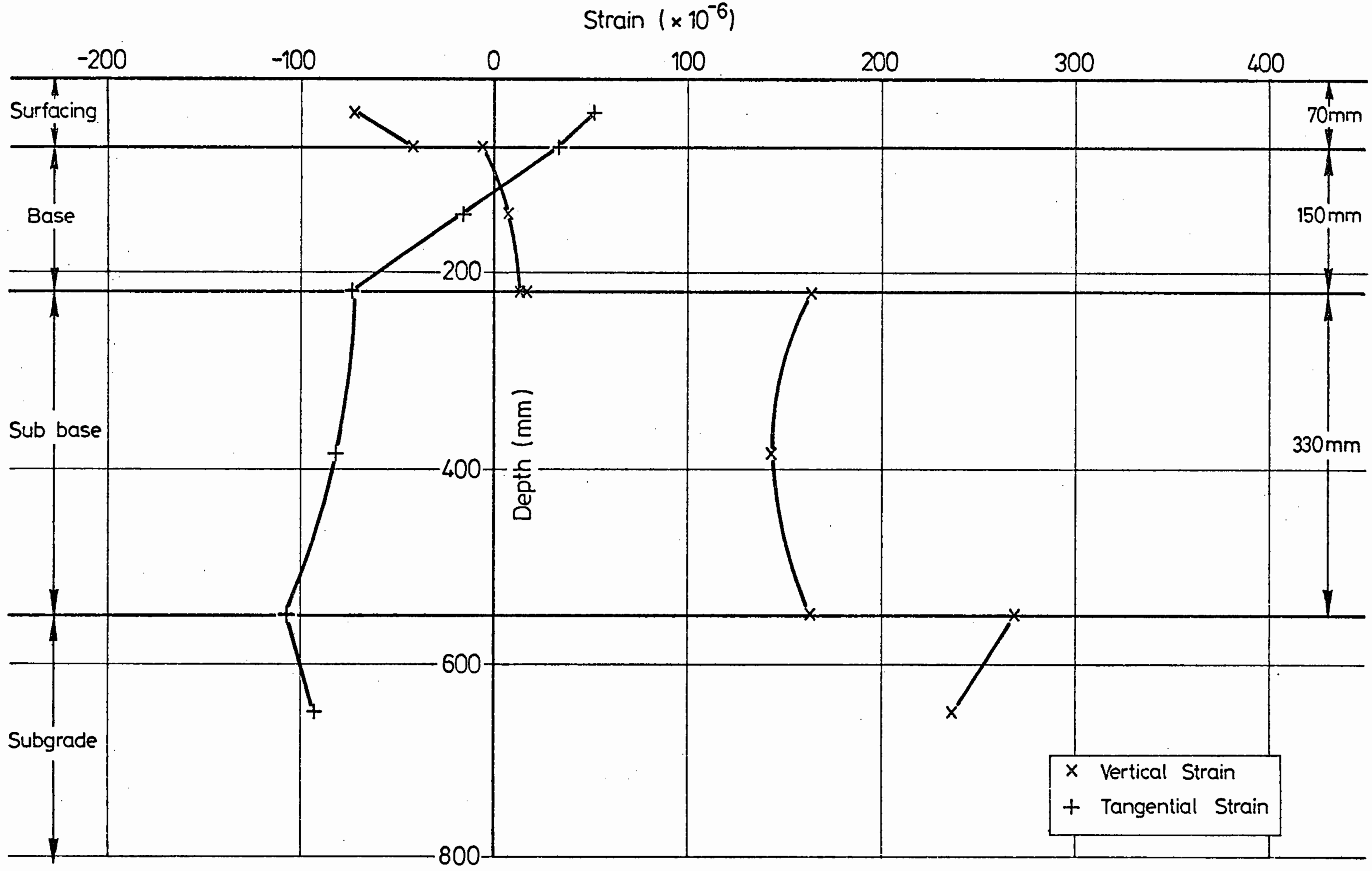

FIG. D.3 STRAIN DISTRIBUTION IN A PAVEMENT WITH A SOIL CEMENT BASE principal stress. For all the cases examined, it was only the principal value for position 5 and below, i.e. at the bottom of the cement treated layer and below. Above this location, σ_{xx} (radial) is usually the principal value.

D.3 APPLICATION OF FACTORS TO ACCOUNT FOR CRACKING

Factors of increasing stress and strain in both the cracked layer and the two underlying layers as reported by Walker et al (D32) derived from the work by Otte (D33) have been presented in Table D.3. The effect of applying these factors to the pavements analysed previously has been considered.

Application of the factor to increase stress in the cement treated layer appears to be straightforward, the tensile stress simply being appropriately increased. Application of the factor to underlying layers is, however, more complex. The modulus of the granular material depends on the state of stress in that material in three-dimensional stress space. Hence, it is necessary to know the change in horizontal stress as well as vertical stress. Calculation of this horizontal stress is complicated by the fact that the horizontal stress could increase in the tensile direction depending on the relative stiffnesses of the component layers.

A few pavement analyses in which the stiffness of the surfacing layer was decreased (thus increasing the vertical stress in the granular layer) were carried out. These analyses indicated that as the vertical stress increased the additional horizontal stress was tensile, and led eventually to a net tensile stress in the bottom of the granular layer. It was, therefore, concluded that further consideration of the model used for the granular layer is necessary in order to eliminate the tensile stresses which the analysis predicts, since these stresses could not be sustained in practice.

Type of cracking expected, and typical situations	Total thickness of cementitious material (mm)	Maximum horizontal tensile stress or strain*	Maximum vertical stress or strain	
			First underlying layer	Second underlying layer
No cracking expected (e.g. less than 2% lime or cement).		1.00	1.0	1.0
Moderate cracking; crack width less than 2 mm (e.g. natural materials with lime or 2-3% cement).	<200	1.10	2.5	1.5
	>200	1.20	7.0	3.5
Extensive cracking; crack width more than 2 mm (e.g. high quality gravels and crushed stone with 4-6% cement).	<200	1.25	5.0	2.5
	>200	1.40	14.0	7.0

* Parallel and adjacent to crack.

Table D.3 Factors for increasing stresses and strains to be applied in respect of initial cracking (after Walker et al, D32)

Thus, it was decided that the factors presented in Table D.3 could not be applied to the ADEM system. However, it should be emphasised that this approach marks a considerable advance in attempts to provide a simple model for a cracked layer and is therefore worthy of development.

D.4 SUMMARY AND RECOMMENDATIONS

(1) Values of elastic parameters for cement treated materials should be measured.

(2) The equations derived from the work of Williams (D27) will estimate electro-dynamic moduli to within a factor of approximately 2 for soil cement, 1.25 for CBGM and 1.2 for lean concrete. These relationships, listed below, are derived from a very small set of data, but one relevant to British conditions:

Soil Cement: $E = 3.28f + 4.25$ GN/m^2

CBGM: $E = 5.47f + 10.2$ GN/m^2

Lean Concrete: $E = 7.125f + 20.5$ GN/m^2

where E = electro-dynamic modulus (GN/m^2)

f = flexural strength (MN/m^2)

(3) The electro-dynamically derived modulus is recommended since it correlates well with compressive and tensile moduli, and because the wide variety of methods used for measuring strain in flexural tests can lead to considerable variation in results.

(4) A Poisson's ratio of 0.15 to 0.2 is appropriate to the three cement treated materials.

(5) A study of the effect of possible errors in material properties on the analysis of a pavement with a soil cement base indicates that the tensile stresses in the lower surface of the cement treated layer are significantly increased by increasing the modulus and

that the strains are decreased. (A modulus increase from 6 to 17 GN/m^2 increases the tensile stress from 0.63 to 1.0 MN/m^2.) Similar behaviour is exhibited in relation to Poisson's ratio variation, but the changes are smaller. (Increasing ν from 0.1 to 0.3 increases the tensile stress from 0.74 MN/m^2 to 0.90 MN/m^2.) These changes may be important, depending on the fatigue characteristics of the material.

(6) Large vertical tensile strains occur in the asphalt surfacing on the axis of symmetry of the loading system when using the uncracked modulus of soil cement. Investigation is required to ascertain if a critical location for this strain exists, and if a controlled stress or controlled strain derived fatigue law is applicable.

(7) The application of the stress factors presented by Walker et al (D32) to the stabilised layer appears to be straightforward.

(8) The application of these factors to the layers below the cement treated layer poses problems. Investigation has shown that increasing the vertical stress increases the tendency for tensile stresses to appear in the granular material. The modified stress state requires further modification of the modulus used for the granular material and the program currently in use does not have the facility to do this.

(9) The significance of thermal stresses has not be investigated, but should be considered in any future work.

(10) The failure criterion for cement treated material must be investigated by formulating a preliminary design procedure and correlating this with experimental data or existing design recommendations.

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APPENDIX E

COMPARISON OF THREE AND FOUR LAYER PAVEMENT SYSTEMS

E.1 INTRODUCTION

The validity of the assumption that all the bitumen bound material in a pavement could be considered as one layer made at an early stage in this research required validation. This initiated an investigation that included comparative analyses of three- and four-layer systems and, when the four-layer version of ADEM became available, comparison of designs obtained with three- and four-layer systems.

The results indicated that the three-layer system was perfectly adequate for use as originally intended, that is for comparing designs under various conditions, but the four-layer system, with its more realistic modelling of a pavement, would produce more realistic designs in absolute terms.

E.2 COMPARATIVE ANALYSIS OF THREE AND FOUR LAYER PAVEMENTS

To complement the analysis of three-layer systems and to assist in verification of the four-layer design system, a series of four-layer structures were analysed for principal tensile strain at the bottom of the asphalt, vertical strain on the subgrade, and vertical and horizontal stress in the sub-base. The basic structures analysed are shown in Fig. E.1 and the results of these analyses have been plotted in Figs. E.2 to E.5, with the stress or strain being shown as a function of base stiffness (E_2) and with two abscissae; base thickness (h_2), and base thickness plus surfacing thickness (h_1+h_2). The second abscissa is based on a constant value for h_1. The surfacing stiffness was also held constant at 4,000 MN/m^2. The results of the previous three-layer

$h_1 = 40$ mm $\quad E_1 = 4{,}000\ MN/m^2$

$h_2 = 50\text{-}300$ mm

$E_2 = 1{,}000\text{-}15{,}000\ MN/m^2$

$h_3 = 200$ mm

$E_3 = 75\ MN/m^2$

CBR = 3

FIG. E.1 BASIC STRUCTURE FOR ANALYSIS OF EFFECT OF MATERIAL PROPERTY VARIATION ON DESIGN CRITERION

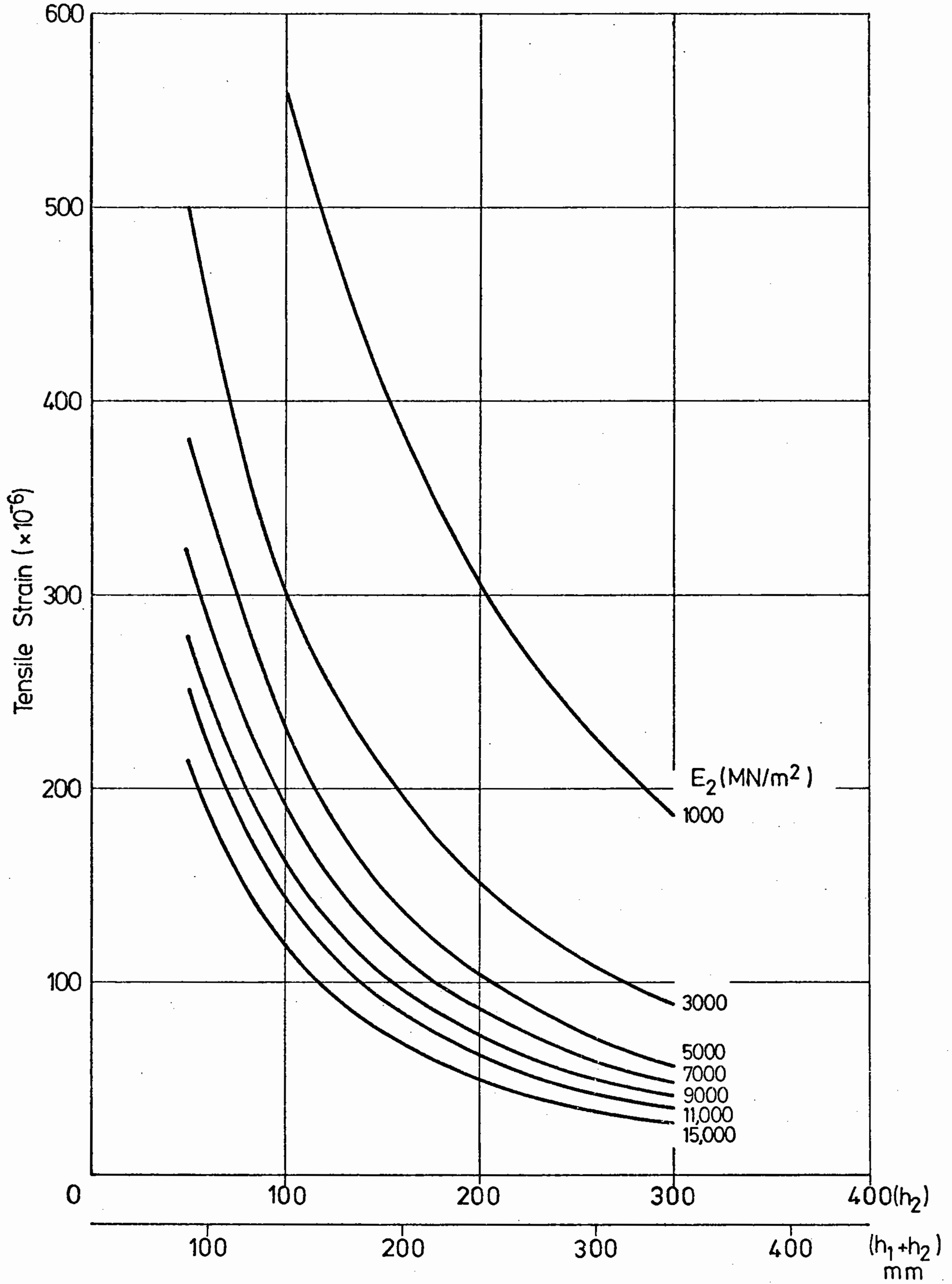

FIG. E.2 PRINCIPAL TENSILE STRAIN AT THE BOTTOM OF THE BITUMINOUS LAYER AS A FUNCTION OF BASE THICKNESS (h_2), TOTAL THICKNESS OF THE BITUMINOUS LAYER (h_1+h_2) AND BASE STIFFNESS, FOR A 4-LAYER STRUCTURE

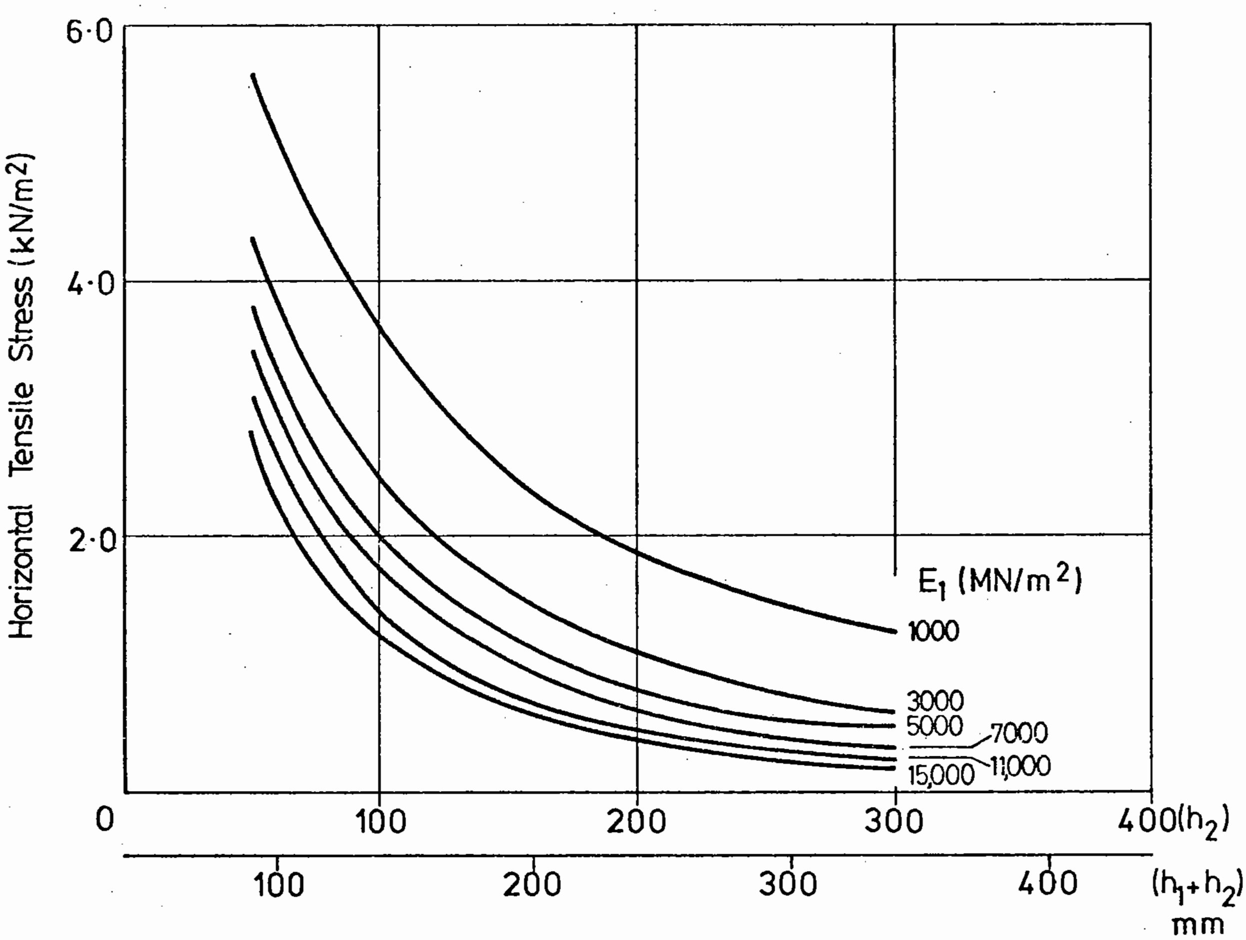

FIG. E.3 PERMANENT TENSILE STRESS AT THE BOTTOM OF THE SUB-BASE AS A FUNCTION OF BASE THICKNESS (h_1), TOTAL THICKNESS OF THE BITUMINOUS LAYER (h_1+h_2) AND BASE STIFFNESS FOR A 4-LAYER STRUCTURE

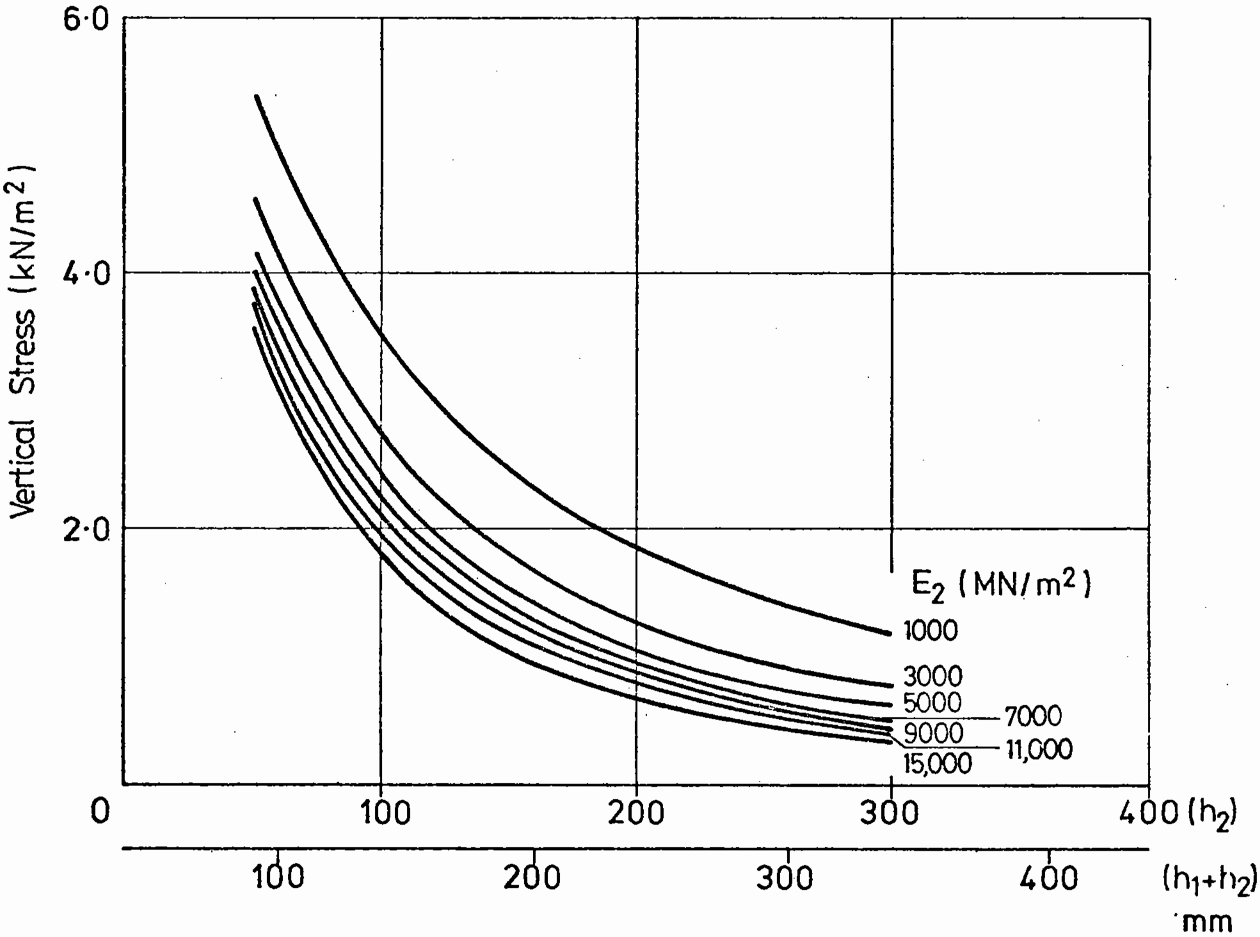

FIG. E.4 VERTICAL STRESS AT THE BOTTOM OF THE SUB-BASE AS A FUNCTION OF BASE THICKNESS (h_2), TOTAL THICKNESS OF THE BITUMINOUS LAYER (h_1+h_2) AND BASE STIFFNESS, FOR A 4-LAYER STRUCTURE

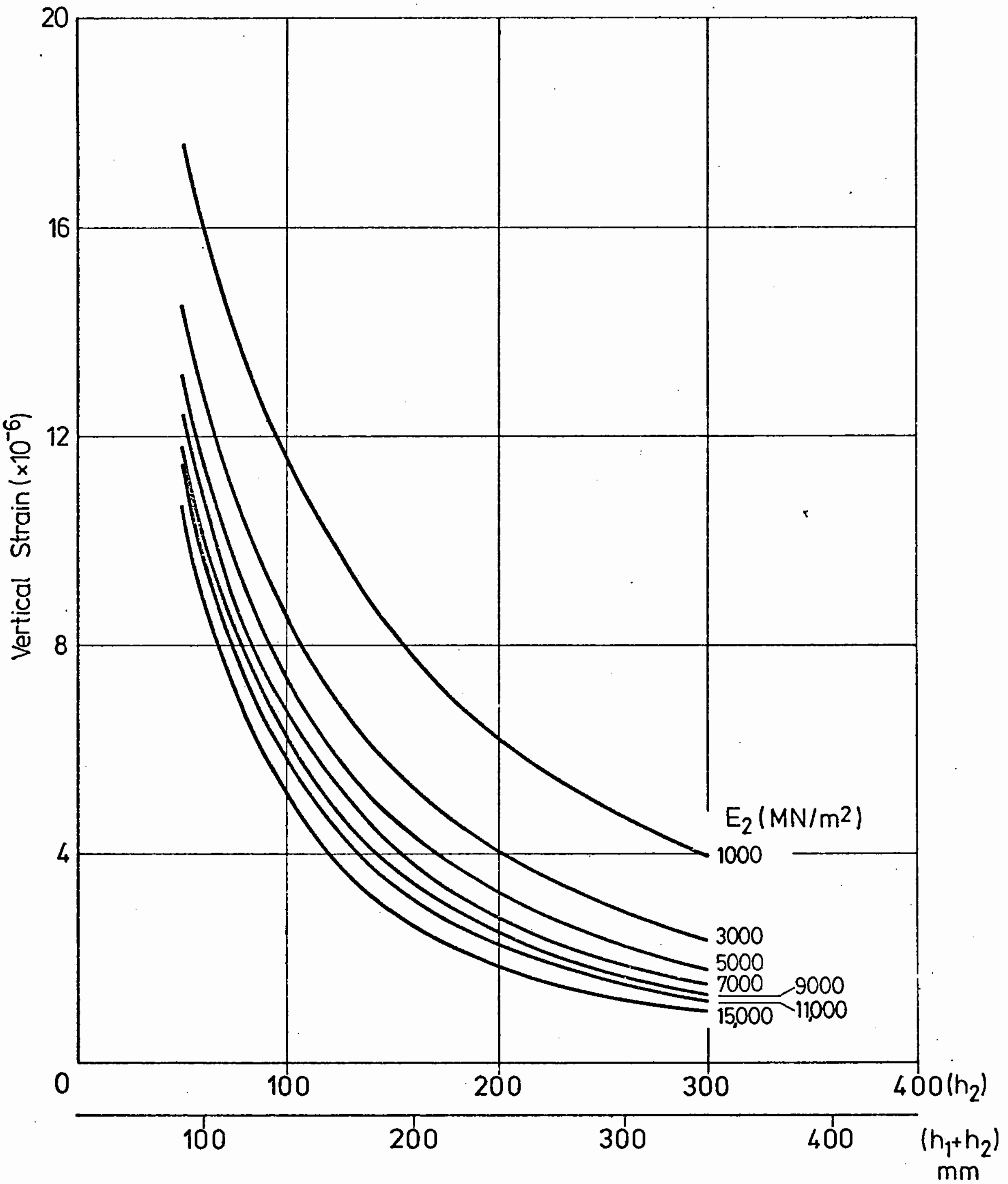

FIG. E.5 VERTICAL STRAIN ON THE SUBGRADE AS A FUNCTION OF BASE THICKNESS (h_2), TOTAL THICKNESS OF THE BITUMINOUS LAYER (h_1+h_2) AND BASE STIFFNESS, FOR A 4-LAYER STRUCTURE analyses can be found in Appendix B. As would be expected, the three- and four-layer systems exhibit similar behaviour.

Table E.1 has been produced as a comparison between the three- and four-layer systems in terms of the design parameters. An attempt was also made to assess the influence of the difference between the calculated parameters in terms of layer thickness by deriving the parameter Δh as follows. A design parameter was chosen, say asphalt strain. A four-layer structure with a given base stiffness and a givel total thickness of asphalt (h_1+h_2) was chosen and the strain corresponding to this structure read from the appropriate graph. This strain was used to enter the plots for three-layer systems and the layer thickness corresponding to this strain for the same layer stiffness read from the abscissa. The difference between layer thickness in the three-layer system and the thickness used initially in the four-layer system gives the value of Δh, which was assigned a positive sign if the three-layer thickness was greater than that of the four-layer system. For the sake of convenience, the results in Table E.1 will be discussed in relation to the three design criteria.

E.2.1 Strain in the asphalt

There does not appear to be any systematic variation in the difference between the two systems with respect to stiffness or total asphalt layer thickness.

The maximum value of Δh is 20 mm which may be regarded as small when considered in relation to the ±10 mm limits applied to the program. When the E_1/E_2 ratio in the four-layer system is greater than 1 (i.e. base modulus = 1,000 MN/m^2), the three-layer system overestimated the design thickness, the converse also being true for E_1/E_2 less than 1.

E_{base} (MN/m^2)	Total thickness (mm)	Strain in Asphalt ($\times 10^{-6}$)			Δh (mm)	Stress in Sub-base (kN/m^2)			Δh (mm)	Strain on Subgrade ($\times 10^{-6}$)			Δh (mm)
		3-layer	4-layer	$\frac{4-3}{3}$ %		3-layer	4-layer	$\frac{4-3}{3}$ %		3-layer	4-layer	$\frac{4-3}{3}$ %	
1000	300	250	225	-10	+20	17.5	14.5	-17	+40	670	470	-30	+60
1000	200	430	395	-8	+20	30.0	23.5	-25	+40	1060	770	-27	+70
1000	100	-	-	-	-	62.8	51.0	-19	+20	1900	1610	-15	+20
9000	300	45	50	+11	-10	3.5	3.5	0	0	150	160	+7	-10
9000	200	87	97	+11	-10	7.5	8.0	+7	-10	300	340	+13	-10
9000	100	250	250	0	0	23.0	27.5	+20	-10	880	1000	+14	-10
15000	300	33	33	0	0	2.5	2.5	0	0	115	120	+4	-10
15000	200	63	68	+8	-10	3.5	6.0	+9	-10	230	260	+13	-20
15000	100	170	190	+12	-10	19.0	22.5	+18	-10	700	880	+26	-20

Note: Δh is positive if the three-layer analysis indicates a thicker layer requirement than the four-layer analysis.

Table E.1 Comparison between three- and four-layer systems

This is to be expected since, at a given stiffness, the only variable in the analysis is the thickness and it is usual in this type of structure for the strains to decrease as the thickness increases.

There is evidence to suggest that the effect of considering the surfacing separately decreases as the stiffness of the base increases. This also appears to be logical since the stiffer the base the more significant its effect on the analysis. However, the evidence is inconclusive. It should be borne in mind that the surfacing thickness will play an important role in determining its effect on the structure and that these analyses have only been undertaken for a very thin (40 mm) surfacing.

E.2.2 Stress in the sub-base

The difference between the two systems is most pronounced at the lowest value of base layer stiffness, 1,000 MN/m^2 and the 40 mm differences in layer thickness are quite significant in relation to the design tolerance. However, in almost all the design exercises undertaken the sub-base stress was not a critical design parameter, and since a refined approach to the treatment of the sub-base is available which does incude this sub-base stress as a design parameter, the large difference exhibited in respect of this criterion is not considered to be a cause for concern.

E.2.3 Vertical strain on the subgrade

For the two systems with E_1/E_2 less than 1 (i.e. base moduli of 9,000 and 15,000 MN/m^2) the effect of a four-layer representation is small, Δh not exceeding 20 mm and in all cases being negative. This indicates that the three-layer approximation is reasonable.

However, when the base stiffness is small (1,000 MN/m^2), the values of Δh suggest that the three-layer approximation could lead to serious over-design of the structure, particularly with thicker layers. However, a bituminous base layer with a stiffness as low as 1,000 MN/m^2 should not be encountered very often, so whilst this is a situation in which the three-layer assumption causes error, it is also a situation which should not be common.

E.2.4 General discussion

From the results of these analyses it is felt that the three-layer simplification is adequate for most practical situations. However, the results also show that relatively thin surfacings can become structurally significant under certain conditions. The implication of this could be far-reaching in that relatively thin overlays to existing pavements which have deteriorated to a condition of low stiffness could greatly benefit the structure provided failure is not induced in the surfacing itself.

E.3 COMPARISON OF THREE AND FOUR LAYER DESIGNS

E.3.1 Introduction

A series of designs were undertaken with the four-layer system covering binder contents from 3 to 6%, at a void content of 6%. Designs were produced for these mixes at surfacing stiffness of 1,000, 4,000 and 10,000 MN/m^2 for comparison with designs obtained with the three-layer system.

Fig. E.6 shows the results of this study. Considering the four-layer systems only, the required layer thickness decreases as the surfacing stiffness increases, which is as expected. The three-layer system produces designs which in most instances are thicker than the four-layer system, confirming the data given in Table E.1.

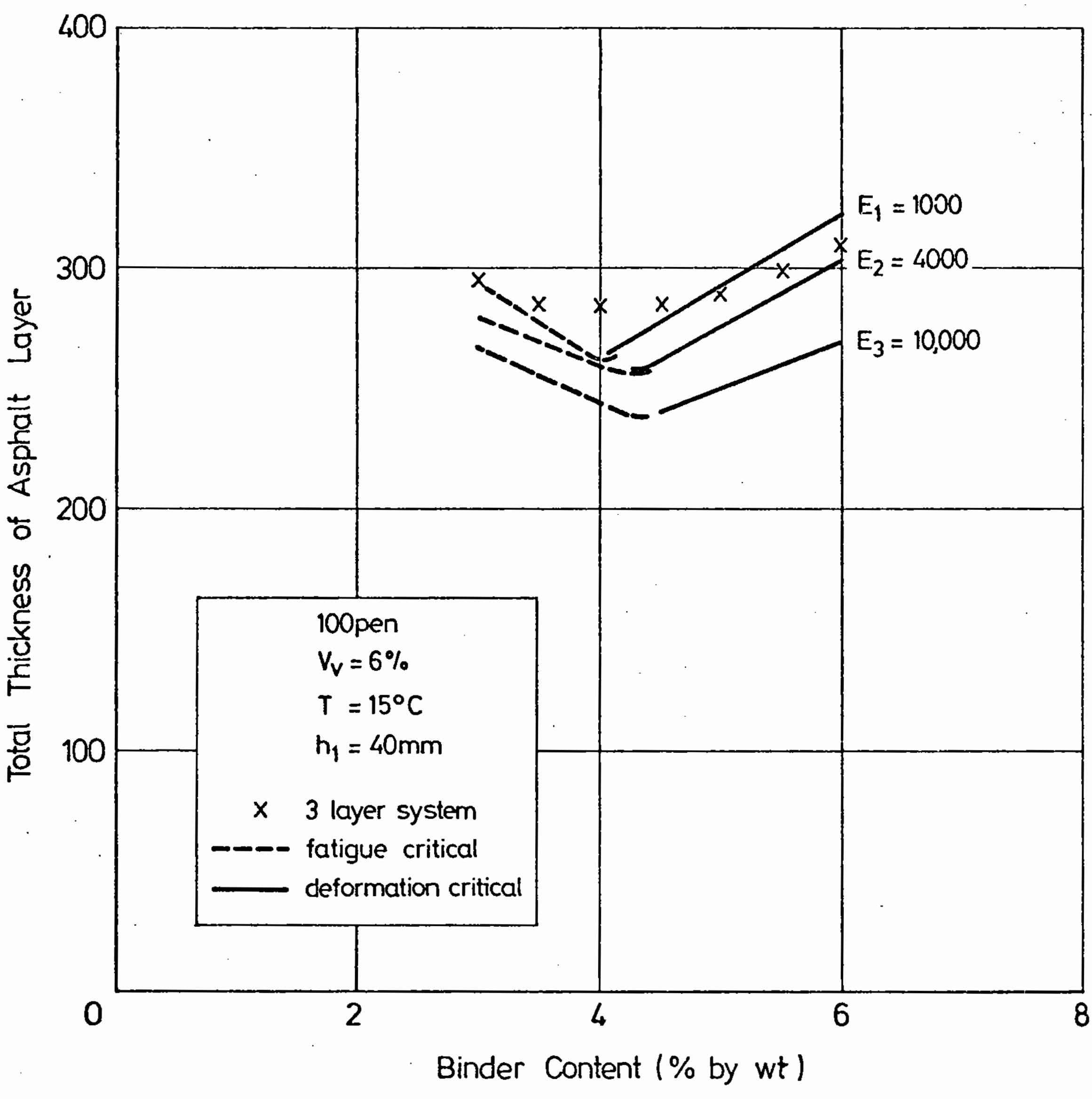

FIG. E.6 COMPARISON BETWEEN DESIGNS OBTAINED WITH THREE AND FOUR LAYER OPTIONS

E.3.2 The concept of effective stiffness

In an attempt to explain the reason for the difference between the results obtained with three- and four-layer design systems, an attempt was made to find a parameter which would have the effect of reducing a four-layer structure to one of three layers. Since layer thickness and stiffness have major effects on the structural analysis, possible combinations of these two parameters were considered.

A simple E_1/E_2 ratio was tried, and a thickness weighted ratio, h_1E_1/h_2E_2, without success. However, results obtained from the use of an effective stiffness (S_E) derived from the equation:

$$S_E = \frac{h_1E_1 + h_2E_2}{h_1 + h_2} \qquad (E.1)$$

were encouraging. The results of the comparative design study (Fig. E.6) have been replotted in Fig. E.7 using effective stiffness derived from Equation E.1 as the abscissa. This has the effect of slightly reducing the overall difference between the lines for the three different four-layer structures. Designs from the three-layer system for the same range of mixes have also been plotted on this graph, an effective stiffness for these designs being the actual stiffness derived for the layer. The agreement between the systems is not greatly improved by this manipulation. However, the area of greatest disparity is that part of the curve which covers the transition from designs with asphalt fatigue as the critical parameter to those with subgrade strain as the critical parameter.

It is reasonable to expect that different layer equivalence parameters will be necessary for the two different design criteria. When designing to the asphalt fatigue criterion the interaction between changes in stiffness and allowable fatigue strain is the overriding parameter. It is evident that when the changes in layer stiffness and fatigue criterion balance to give a constant design thickness (3.5 to 4.5% binder, Fig. E.5)

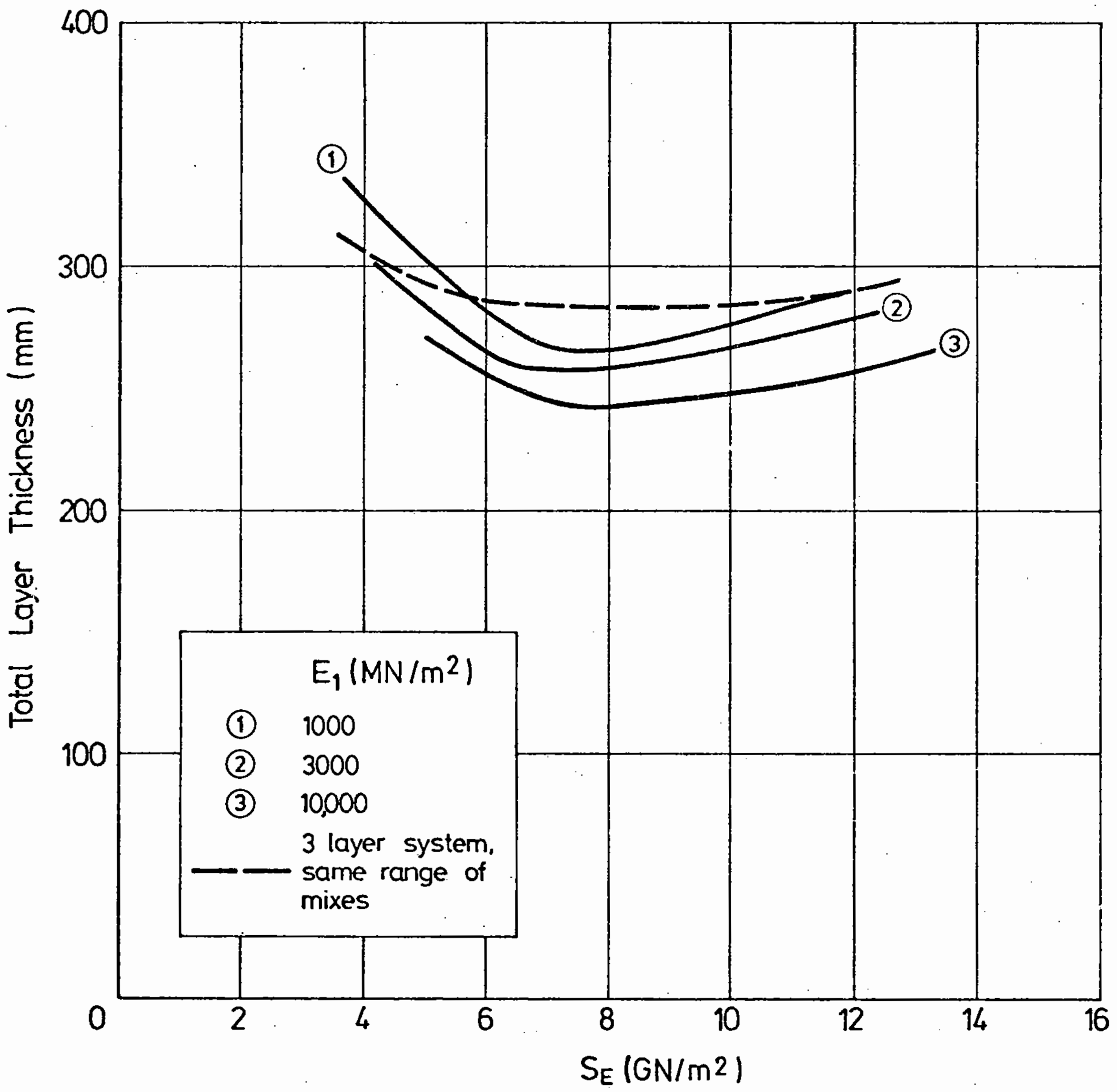

FIG. E.7 COMPARISON OF THREE AND FOUR LAYER DESIGNS USING EQUIVALENT STIFFNESS (S_E)

the concept of effective stiffness as defined herein is not totally satisfactory.

E.3.3 Effect of layer stiffness ratio

A series of designs was undertaken with E_1/E_2 values of 0.5 and 2, for design lives of 10^7 standard axles and with a surfacing thickness (h_1) of 40 mm. All mixes involved 100 pen bitumen and a void content of 6%. Three different binder contents were used: 3, 4.5 and 6% by mass. The facility to supply the program with a mix stiffness rather than derive one internally was used for this study to ensure a controlled stiffness ratio. Hence, at each binder content, since life and mix proportions are held constant, the criteria are the same in all cases.

The designs resulting from this data are plotted in Figs E.8, E.9 and E.10. For the 3% binder content the mix is highly fatigue susceptible, and all designs are on the basis of the fatigue criterion. There is a constant 20 mm difference between the designs for the two modular ratios, the thinner base being required when the surfacing is the stiffer of the two layers (i.e. $E_1/E_2 = 2$).

At a binder content of 4.5% by weight (Fig. E.9) and at low stiffnesses, the designs are again controlled by the fatigue criterion, and the same difference of 20 mm is observed as with the 3% binder content mixes. However, the transition to deformation controlled designs, marked by the 'elbow' in the curve, changes the spacing to 40 mm. This confirms the view that different layer equivalences are necessary for different design criteria.

Fig. E.10 for the 6% binder contents shows curves similar in shape to those in Fig. E.9. However, all designs in Fig. E.8 have been derived on the basis of subgrade strain. Further examination of this graph indicates that the difference between the two lines is constant at

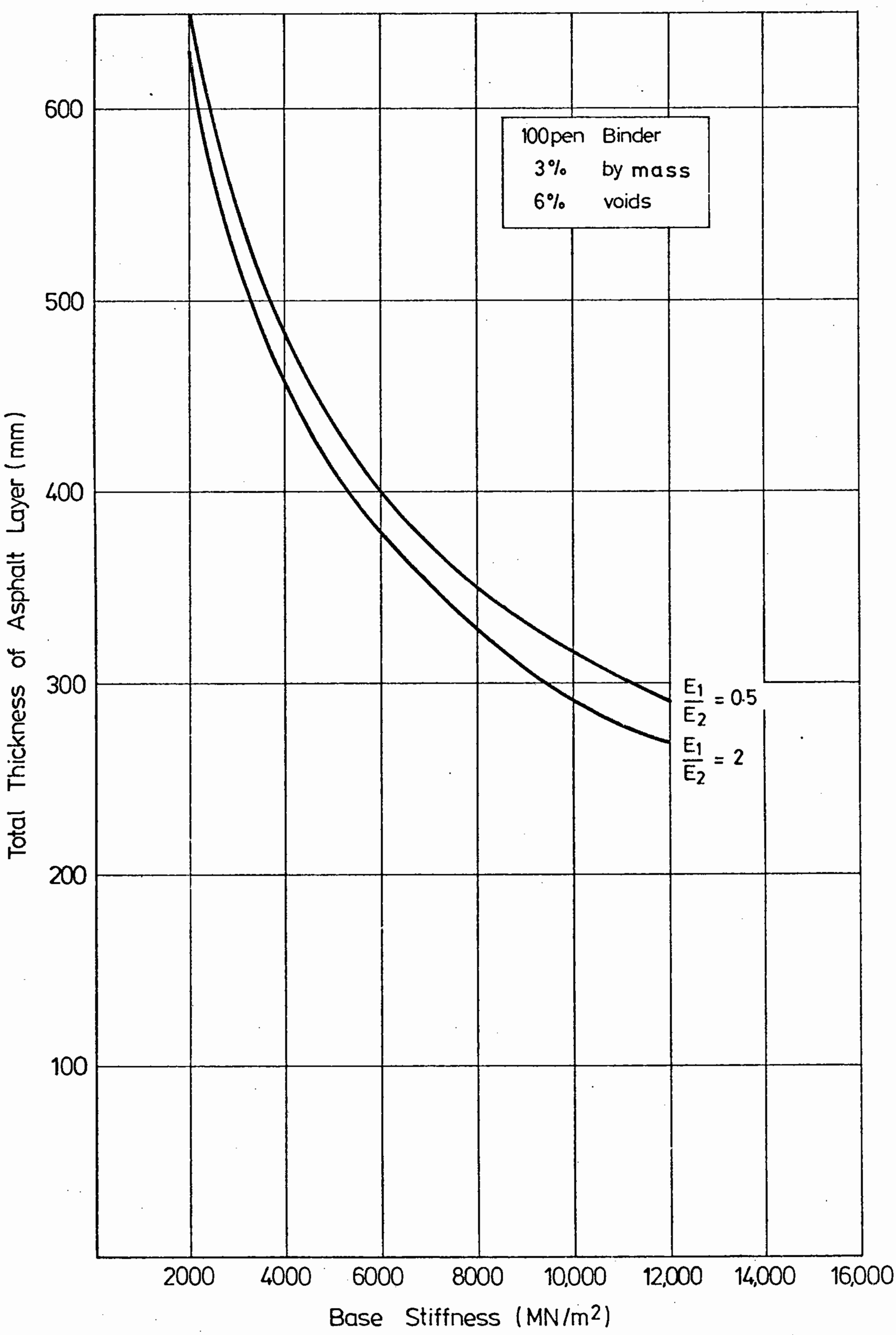

FIG. E.8 TOTAL THICKNESS OF ASPHALT AS A FUNCTION OF BASE STIFFNESS AND SURFACING/BASE STIFFNESS RATIO FOR 3% BINDER CONTENT

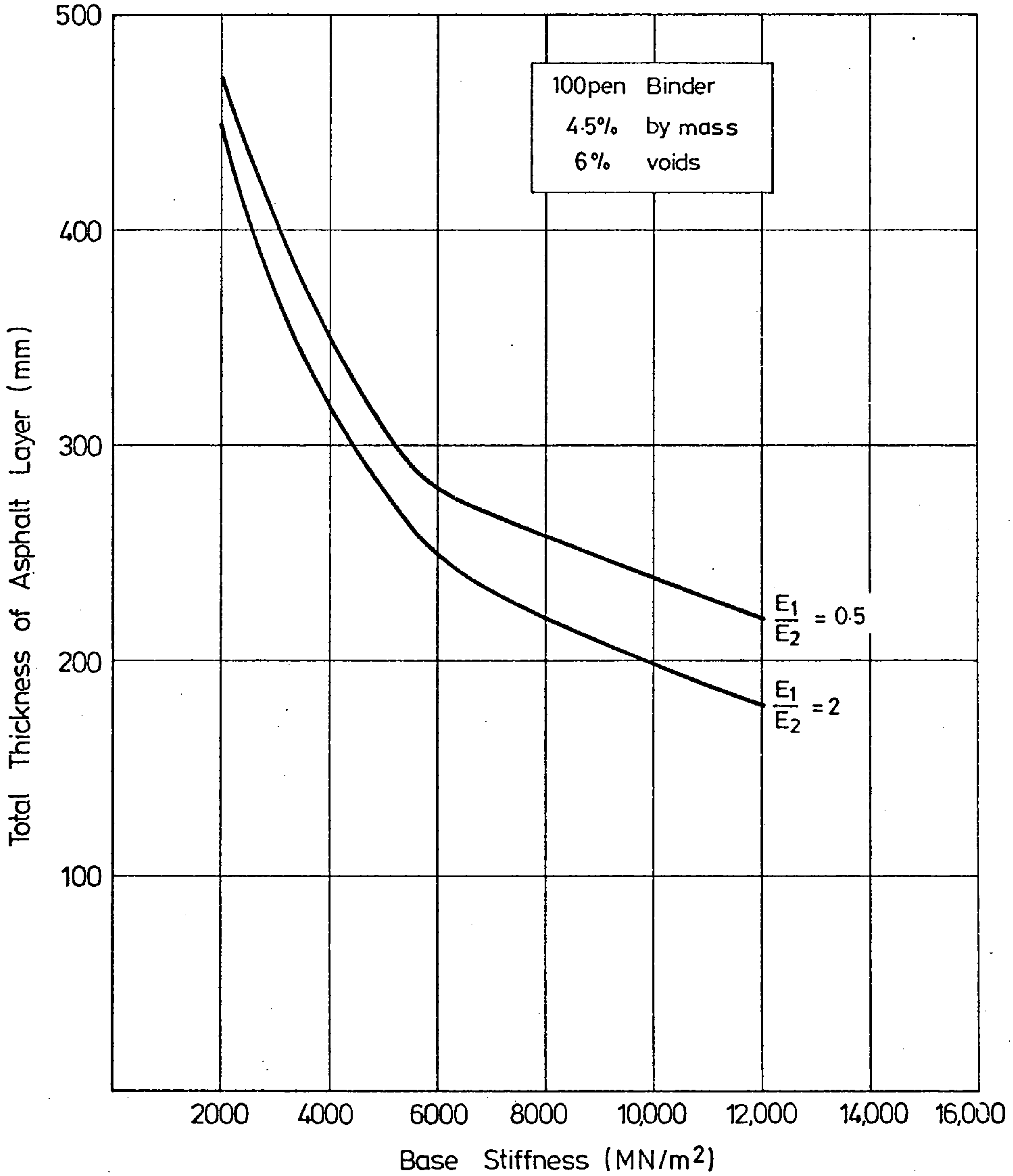

FIG. E.9 TOTAL THICKNESS OF ASPHALT AS A FUNCTION OF BASE STIFFNESS AND SURFACING/BASE STIFFNESS RATIO FOR 4.5% BINDER CONTENT

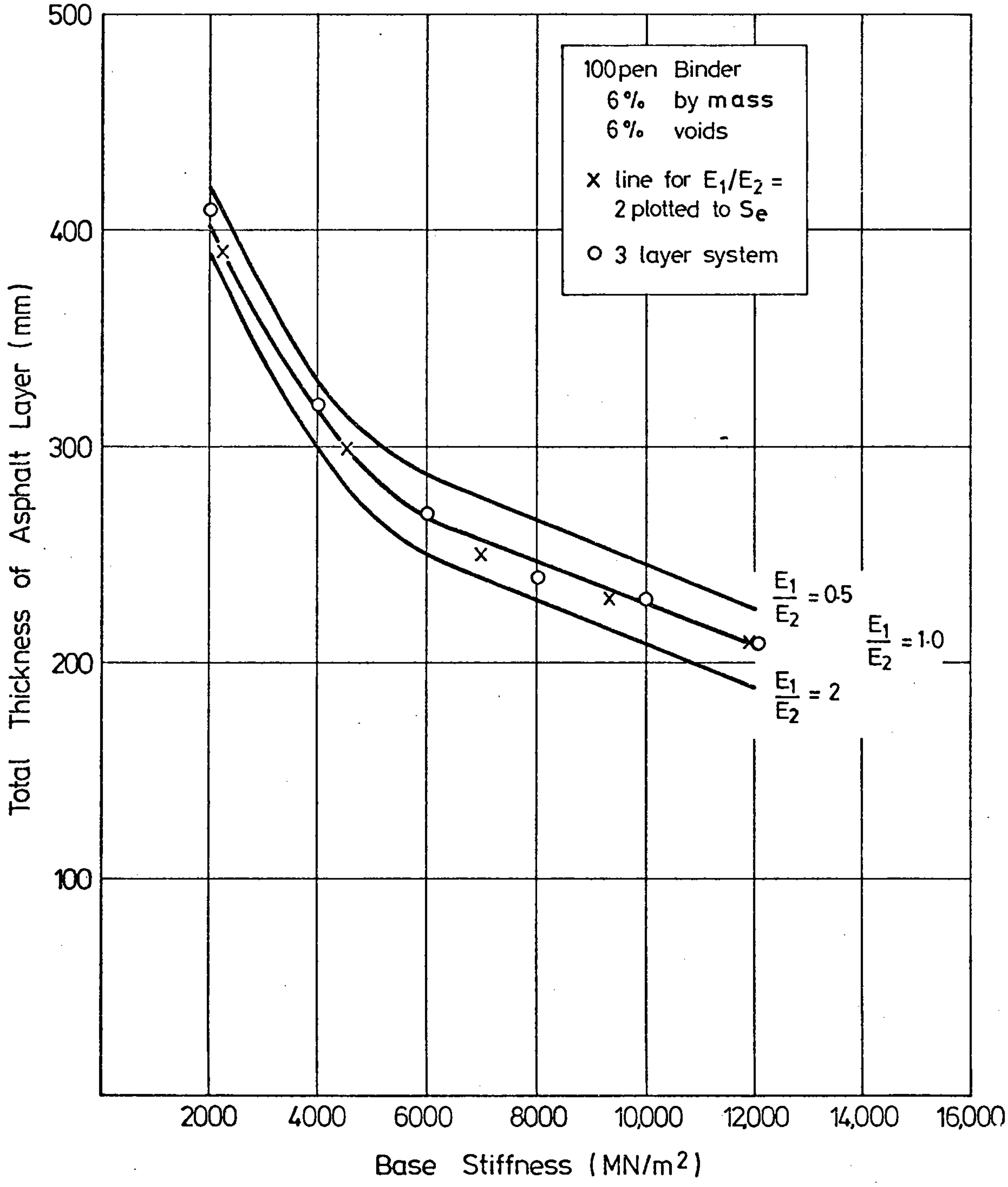

FIG. E.10 TOTAL THICKNESS OF ASPHALT AS A FUNCTION OF BASE STIFFNESS AND SURFACING/BASE STIFFNESS RATIO FOR 6% BINDER CONTENT

40 mm, and so the change in shape of the stiffness-thickness plot in this case is considered to be due to the fact that subgrade strain does not vary linearly with either mix stiffness or layer thickness (see Fig. E.5).

Also plotted in Fig. E.10 are designs with a modular ratio of 1, achieved by using the same stiffness for the surfacing and the base. This line lies midway between the lines for ratios of 0.5 and 2.0. The designs for a modular ratio of 2.0 have been converted to an effective stiffness using Equation E.1 and plotted in Fig. E.10 with crosses. For all practical purposes, this stiffness transformation brings the designs to the line corresponding to a modular ratio of 1.0.

It is appropriate at this point to add a cautionary note regarding the interpretation of this last study and also regarding other data plotted with stiffness as the abscissa. The exercise discussed in this section is rather artificial. For each series of designs the mix proportions have been maintained constant, to keep a constant fatigue criterion. However, the stiffness of this mix has been varied through wide limits. Thus, it is inferred that some external condition, such as temperature, must have changed to cause this stiffness variation, and in some cases the mix considered is unlikely to achieve, in any pavement, the stiffness value assumed. Thus, when considering any plot of thickness against stiffness, it must be remembered that the graph is for a specific mix, which therefore has specific fatigue properties, for which a stiffness has been derived, at a specific temperature and time of loading. Erroneous results will be obtained if an attempt is made to derive a design thickness from a stiffness for a mix other than that used to derive the particular stiffness-thickness plot available.

It is also necessary to bear in mind, when using the facility to provide a mix stiffness for ADEM rather than use the built-in stiffness calculation that, in order to obtain a realistic design, the stiffness provided must be the stiffness of the mix used in calculating the fatigue criterion at a realistic temperature and loading time. This stiffness must also be similar to that which would be derived by the procedure within ADEM. If these conditions are not satisfied, ADEM will produce a design but it may relate to some unusual temperature and loading time conditions, and whilst being faithful to the data supplied, may appear to be quite unrealistic and cause the user to doubt the accuracy of the system. This is because ADEM was calibrated against Road Note No. 29 designs on the basis of various assumptions of properties for these designs. Thus ADEM is only valid as a design system if the constraints imposed by those assumptions are honoured.

E.4 CONCLUDING REMARKS

The three-layer approximation used in the first version of ADEM provides perfectly adequate results when used as a means for considering the relative effects of various parameters on design.

The four-layer program produces a more accurate model of the pavement, and therefore calculates design thickness more accurately. However, the surfacing thickness and stiffness are variables in this system and they can both have significant effects on the behaviour of the structure. To produce a detailed analysis of the effect of the four-layer refinement, it would be necessary to repeat the studies reported herein for a range of surfacing stiffnesses and thicknesses.

Generally, since it provides a greater degree of sophistication in modelling the pavement the four-layer system is recommended for use, but the three-layer system is perfectly adequate for comparative studies.

Because of the need for wide ranging parameter studies regarding the effects of surfacing and basecourse layers, the five-layer version of

ADEM has been verified but no design studies have been undertaken.

The concept of equivalent stiffness has some value, particularly when the design is controlled by a single criterion and by the stiffness of the asphalt layer. When the changes in stiffness and fatigue criterion start to balance each other the function suggested is less efficient. It is believed that such a relationship is of most use in deriving values of stiffness for the combination of layers other than the design layer, e.g. in simplifying a five-layer system to a four-layer one by combining the wearing and base course. However, this has not been thoroughly investigated.