COMPUTER-AIDED DESIGN AND ANALYSIS OF CONCRETE SEGMENTAL PAVEMENTS

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ABSTRACT

This paper presents a computer-based methodology for the design of concrete block pavements for road and heavy industrial loadings. The paper begins by identifying the design criteria for segmental paving with respect to loading requirements, environmental factors and in-service performance. The characterisation of the pavement materials is then discussed and a simple damage model to relate the performance of the materials to the applied loadings is described. Methods for analysing block pavements are described and a design procedure is formulated. Details of the data input required by this procedure are given, together with an overview of the design methodology. Design curves are presented for typical road and industrial block pavements as illustrations of the methodology. Finally, the validity of the design procedure is assessed by comparing the output of the computer program with the thickness of a range of pavements already in service around the world.

INTRODUCTION

Increasingly, concrete block paving is being recognised as a valuable and versatile addition to modern paving techniques. During the last two decades there has been substantial research world-wide designed to define the technology of this new paving medium. Comprehensive data on the performance of block pavements under traffic are now available. This has been critically reviewed elsewhere (Shackel 1980a, 1984 and 1985a). Much of the research has been directed towards the development of design procedures for block pavements. The evolution of these methods has been described earlier (Shackel 1979a and 1980b).

In Australia, design methods for block road pavements have evolved from simple ad hoc interim procedures (Cruickshank 1976) to an empirically-based method (Shackel 1979b; Hodgkinson and Morrish 1982) derived from accelerated trafficking of prototype block pavements conducted both in Australia (Shackel 1979c) and in South Africa (Shackel 1979d and 1982a). Simultaneously, design procedures for industrial block pavements have been developed (Shackel 1982b and 1984). Initially, these used existing design procedures modified by experimentally-determined layer equivalencies relating block thickness to the equivalent thickness of conventional materials such as asphalt and crushed rock (Shackel 1979c). At first, both the road and industrial pavement design procedures were limited to pavements incorporating unbound bases. These were subsequently extended to embrace the use of both bound and unbound base and sub-base materials used either singly or in various combinations. This required the introduction of computer-based mechanistic procedures (Shackel 1985a and b). The purpose of this paper is to describe the methodology involved in these procedures.

DESIGN CRITERIA FOR BLOCK PAVEMENTS

The design of a pavement, whether for a road or an industrial hardstanding, requires that design criteria be established in each of the following areas:

(a) loading requirements,

(b) environmental factors, and

(c) in-service pavement performance.

Evidence summarised elsewhere (Shackel 1980a) shows that concrete block paving exhibits a number of unique response characteristics which distinguish it from other forms of pavement. This means that the design criteria that are routinely applied to conventional pavements are not always appropriate to block paving.

LOADING REQUIREMENTS

The loading requirements for road pavements and industrial yards are quite different from one another. Accordingly, for convenience, these requirements are considered separately.

Road Traffic Loading

The traffic running on a road typically embraces a wide spectrum of axle loads, ranging from those applied by cars and light commercial vehicles to those applied by heavy trucks and buses. In order to characterise a stream of mixed vehicles, engineers commonly employ the concept of the Equivalent Standard Axle (ESA). Usually this is chosen to correspond to the maximumn legally-permitted axle load. The actual traffic is then replaced for design purposes by an equivalent number of ESAs calculated on the basis that the damaging effect of a vehicle is proportional to the ratio of its axle load to the standard axle load raised to some power, n. In other words, N_1 repetitions of an axle load, W, is equivalent to N_2 equivalent standard axle loads where:

$$N_2 = \left(\frac{W}{ESA}\right)^n N_1 \qquad (1)$$

The power, n, in eqn (1) is commonly assumed to be 4, although accelerated trafficking tests of a variety of pavements conducted by the author and others show that, in practice, n ranges between 2 and 6 depending on the pavement materials and the sequence and thickness of courses (Freeme, Meyer and Shackel 1981).

The concept represented by eqn (1) is widely used in the design of asphalt pavements but must be treated with caution when designing block-paved roads. As noted elsewhere (e.g. Shackel 1979d), block pavements tend to stiffen under traffic. Consequently, they become less sensitive to load with the passage of time. Moreover, there is no evidence to suggest that the 'fourth power law' represented by eqn (1) or any similar damage concept applies to block paving. To the contrary, the existing data suggest that, at least for wheel loads up to 70 kN (7 t), neither the magnitudes nor numbers of vehicle loads have any substantial effect on the response of a block pavement once lock up is achieved (Shackel 1979d). In other words, the concepts of load equivalency do not apply to block pavements.

The preferred alternative to the use of axle load equivalencies is to design the pavement for the actual distributions of vehicle numbers and loads predicted from traffic studies. The disadvantage of this approach is that the effect of each of the axle loads making up the distribution must be calculated separately and then summed over the full spectrum of traffic. This greatly increases the amount of computation needed. However, calculations can be simplified by adopting an idealised distribution of axle loads based on traffic surveys. For example, Fig. 1 gives the distribution of commercial vehicle loads recommended for road pavement design in Australia (Potter and Donald 1984).

The approach just outlined has long been used in the design of rigid pavements, e.g. using the Portland Cement Association (1966) procedures. As noted below, the methodology can be readily adopted to the design of block pavements. It is then found that some of the lighter vehicles comprising a traffic stream have little or no effect on the thickness of pavement needed. However, if present in large numbers, such vehicles may significantly affect the calculation of the numbers of ESAs and the thicknesses based on these numbers. It therefore follows that the use of actual traffic distributions (e.g. as given in Fig. 1) tends to yield a more realistic, if more conservative, design than where ESAs derived from concepts of equivalency are used.

INDUSTRIAL LOADINGS

Industrial pavements must be designed to accept a wider range of vehicle types, axle configurations, wheel loads and tyre pressures than road

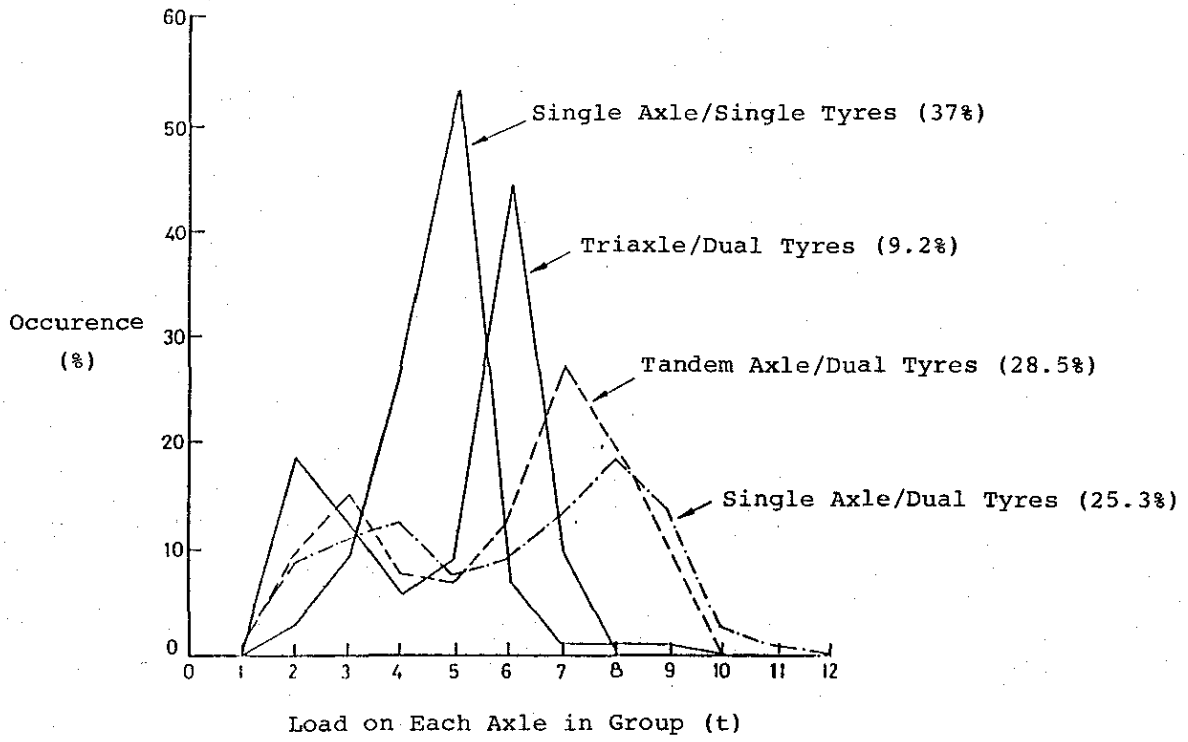

Fig. 1 - Distribution of truck axle loads in Australia (after Potter and Donald 1984)

pavements, where load limits and vehicle configurations are controlled by legislation. As in the case of road pavements, it has been suggested that concepts of axle load equivalency can be applied to industrial pavements. For example, in Britain the concept of Port Area Wheel Loads (PAWL) has been introduced (Barber and Knapton 1979; Knapton 1986). Here, a PAWL may be regarded as an equivalent 12 t single wheel load applied via a tyre inflated to 0.8 MPa. The damaging effect, D, of a wheel of magnitude W (kg) running on a tyre inflated to p (MPa) is given by:

$$D = \left(\frac{W}{1200}\right)^{3.75} \times \left(\frac{p}{0.8}\right)^{1.25} \qquad (2)$$

Using this concept, the actual number of movements, N, of a given industrial vehicle traversing a site can be represented for design purposes by an equivalent number, N_e, of PAWLs calculated using the expression:

$$N_e = D.N \qquad (3) \qquad (3)$$

By summing the effects of all the vehicles which are planned to traverse a site, the total number of PAWLs may be obtained.

The alternative to the use of a standard wheel load such as the PAWL is to express the actual loading expected on a pavement in terms of an equivalent number of movements of a standard vehicle. For example, in the U.S., the National Concrete Masonry Association (1983) has selected a Hyster 620 forklift as the design vehicle for block pavements. This only applies a maximum wheel load of about 15 t.

Because, as noted above, concepts of axle load equivalency are not consistent with the observed performance of block pavements, the use of equivalent loads in the design of industrial block pavements needs to be treated cautiously, especially when the pavements are to incorporate bound materials such as lean concrete or soil-cement. These materials characteristically exhibit fatigue behaviour such that, within a very narrow range of working stresses or strains, a small change in the values of these parameters will typically have a major influence on the fatigue life. Stresses or strains lying above this region lead to early failure of the material and thereby, to predicting the service life of the pavement. Because of this, the pavement should be designed to resist the heaviest wheel load expected in service and the concept of equivalency should be restricted merely to

characterising the number of vehicle movements to be accepted by the pavement. This is, for example, the approach adopted in the British Ports Association (Barber and Knapton 1982) design method. Under no circumstances, however, should the design of the pavement be based on the stresses or strains generated by a standard vehicle or axle unless the working wheel loads are less than or equal to the equivalent standard load. For this reason, procedures utilising design curves derived for a particular vehicle (e.g. NCMA 1983) should be treated with caution.

One particular form of industrial loading deserves special consideration. This concerns the movement and storage of containers. Measurements show that the actual operating weight of containers follows distributions of loads which must be taken into account when calculating the operating loads of forklifts and other container-handling vehicles. It can be shown that it is inefficient to design container yards on the assumption that all containers are loaded to their rated maximum capacity. Rather, it is more appropriate to design the pavement to resist a spectrum of axle loads obtained by combining the load distributions given in Fig. 2 with the operating characteristics of the design vehicles. This means that, unlike road pavements which can be designed for a standard set of loading spectra (e.g. as given in Fig. 1), container areas will need to be designed for a series of loading spectra prepared for each of the vehicles that will use the facility. As noted below, mechanistic design procedures have been developed for accomplishing this.

Unlike roads, industrial pavements must accept heavy static loads in addition to vehicular traffic. These loads include bulk materials, manufactured components and, most importantly, containers, which are often stacked four or even five high. The weights of these container stacks are transferred to the pavement via corner blocks each approximately 170 mm square. Static loads of up to 22 t can be applied through each of these blocks. This means that, where containers are block-stacked so that four corner blocks are in close proximity, loads in excess of 80 t may be applied to the pavement surface.

Tests and experience have shown that block pavements are highly

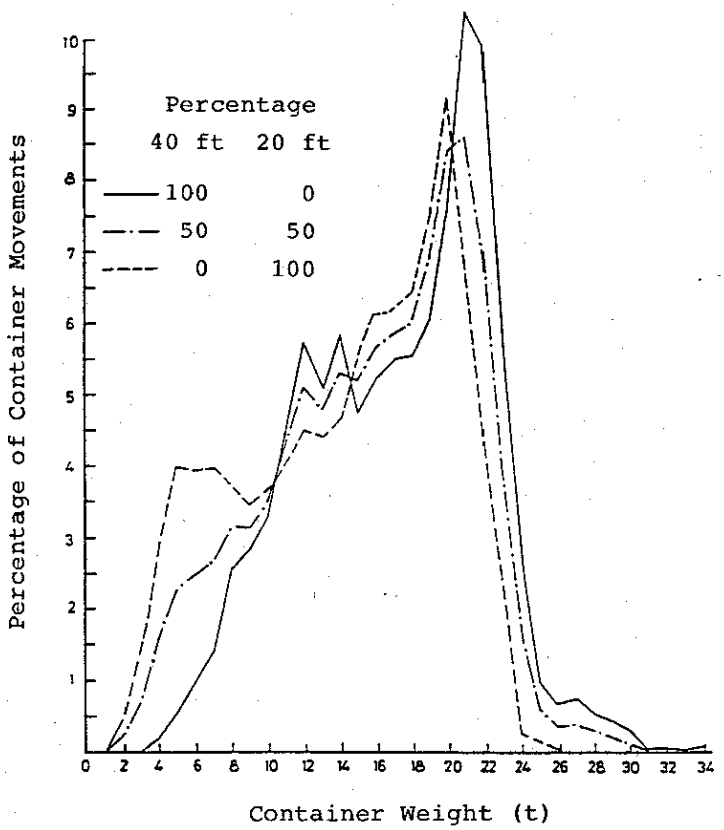

Fig. 2 - Distribution of container loads

resistant to punching by static vertical loads. Accordingly, for design purposes, it is normally sufficient to choose the pavement thickness to suit the container-handling vehicles rather than the static loads. However, the influence of container stack loads on such factors as the consolidation of compressible subgrades should be checked.

OVERLOADING AND FACTORS OF SAFETY

The tolerance of a pavement to occasional or continual overloads depends on the type of construction. For example, rigid concrete pavements are highly susceptible to damage by overloading. Indeed, a single vehicle movement may lead to cracking of a concrete slab if the wheel load sufficiently exceeds the strength of the material. A similar, albiet less severe, problem exists for asphalt pavements because the fatigue life of the surface decreases rapidly as the wheel load increases. Because of the disproportionate effect of changes in the wheel loads, it is customary to apply substantial factors of safety in the design calculations to ensure that the predicted in-service stresses and strains lie well below the theoretically tolerable values. By constrast, because block pavements tend to become less sensitive to load under traffic, the factor of safety tends to increase with time. For this reason, except in the very early stages of trafficking, block pavements are less likely to be damaged by overloading than either flexible or rigid pavements. This is of particular importance in the case of industrial pavements where unforseen changes in the loading requirements are common.

ENVIRONMENTAL FACTORS

Most conventional types of pavement are sensitive to environmental factors, of which temperature is the most important. For rigid concrete pavements, variations in temperature cause expansion and contraction as well as warping. To resist these effects requires the provision of suitable reinforcement and/or a system of joints. In the case of asphaltic pavements, changes in temperature cause marked alterations in the stiffness of the surfacing. For example, a change in pavement temperature from 15 to 40°C can reduce the modulus ten-fold from about 2800 MPa to as little as 280 MPa. This means that temperature must be taken into account in the design of asphalt pavements, either as a weighted mean annual average temperature (w_{WMAAT}) or, better, by modelling the actual variations in temperature in a mechanistic analysis (Shackel 1986a).

In contrast to other types of surface, there is no evidence that the structural properties of interlocking concrete block pavements are affected by temperature. Here, not only do block pavements typically exhibit higher moduli than asphaltic concrete (see Table 1) but also they maintain these moduli irrespective of temperature. Moreover, block pavements appear to be free of the problems of warping or expension common in concrete slabs. In particular, experience shows that even where large pavement areas are involved, it is not necessary to provide expansion joints. This can be explained by the fact that typically between 1.5 and 2.5 per cent of a block pavement comprises joints filled with compressible material having the properties to accomodate expansion.

TABLE 1

CHARACTERISTICS OF PAVEMENT MATERIALS FOR ELASTIC LAYER MODELLING

Material	Modulus, E (MPa)		Poisson's Ratio (ν)	
	Range	Recommend	Range	Recommend
Blocks:				
rectangle	500-7000	2500	0.15-0.30	0.30
dentated	900-7500	3200	0.15-0.30	0.30
Base:				
crushed rock	200-800	350	0.10-0.50	0.35
stabilised	1000-30000	2300	0.10-0.50	0.35
Sub-base:				
gravel	150-450	225	0.10-0.50	0.35
stabilised	5000-7000	1500	0.10-0.50	0.35

PERFORMANCE CRITERIA

Two criteria are commonly used to assess the in-service performance of flexible pavements:

(a) the recoverable surface delections; and

(b) the permanent surface deformations or rutting.

These criteria are now discussed in detail.

DEFLECTIONS

For both rigid concrete and asphalt pavements it is commonly required that the surface deflections be strictly limited to values which are usually less than 0.5 mm. This is in order to avoid load-associated fatigue cracking of the pavement surface. However, block pavements can routinely tolerate much larger deflections than conventional pavements without becoming unserviceable. This means that the overall stiffness of a block pavement can often be much less than a conventional pavement. In this respect block pavements can consistently exhibit deflections of 2 mm or more without distress (Shackel 1979c and d). Accordingly, the thickness of base and sub-base needed in a block pavement is normally less than for alternative forms of construction. Moreover, it is not appropriate to adopt design methods developed for asphalt pavements to block pavements because such procedures are normally predicated on maintaining much smaller pavement deflections that can, in practice, be accepted by a block pavement.

It is not intended to imply that the deflections predicted in a block pavement are not a design consideration. Where the pavement incorporates bound base or sub-base layers such as cement-stabilised materials, it becomes necessary to limit the tensile strains in these layers to values that will ensure that fatigue cracking will not occur. This, in turn, implies that some restriction will need to be placed on the surface deflections. However, it is emphasised that the surface deflection per se is not the relevant design criterion.

DEFORMATION

A study of the literature shows that block pavements, like asphalt pavements, exhibit non-recoverable or rutting deformations under traffic. In the case of block paving, most of this deformation occurs early in the loading history prior to the development of lock up. The amount of rutting needs to be controlled to ensure that the riding quality of the pavement maintains an acceptable standard. Moreover, the rut depth should not be allowed to develop to the stage where water will pond in the ruts. Thus, the allowable rut depth depends on the crossfall or camber of the pavement.

For conventional asphalt pavements, the terminal levels of rutting at which maintenance is deemed to be necessary generally range between 10 mm for freeways up to 40 mm or more for lightly-trafficked rural roads. From consideration of serviceability, similar levels of rutting would be deemed acceptable for block pavements. In The Netherlands, a terminal rut depth of 25 mm is recommended as the limit of serviceability in a block pavement, whilst a rut depth of 35 mm is set as the structural limit (Molenaar, Moll and Houben 1984). Although these recommendations are consistent with the poor soil conditions common in The Netherlands, such values are unlikely to prove acceptable elsewhere. Indeed, experience shows that both pavement specifiers and road users generally expect higher standards in block pavements than in asphalt pavements. This is because, in contrast to the amorphous appearance of asphalt, the presence of joint lines in block paving makes it easy for the unaided eye to detect relatively small deformations. Consequently, there is a tendency for engineers to be more critical of block pavement performance than of asphalt. For this reason it is usually desirable to design block pavements so that, prior to lock up, they do not accumulate more than 5 or 10 mm terminal deformation in downtown applications and not more than 15 to 20 mm in rural or industrial applications. Overall, the control of rutting represents the principal design criterion for segmental pavements.

PAVEMENT CHARACTERISATION

Before any meaningful analysis of a block pavement can be made, it is necessary to first characterise each of the individual pavement materials. Once this has been accomplished the overall response of the pavement to traffic can be considered.

MATERIALS CHARACTERISATION

The material properties of the following pavement elements need to be considered:

(a) the block surface,
(b) the base and sub-base materials, and
(c) the subgrade.

Although some non-linear models based on finite element techniques are available, a study of the literature shows that in order to utilise existing data it is expedient to make the fundamental assumption that all of the materials can be characterised as being either linearly or non-linearly elastic. Accordingly, this is the approach adopted here. Typical values of the modulus, etc. are given in Table 1.

Block Layer

Several workers have characterised concrete block paving in terms of the performance of an equivalent elastic layer. In most cases it has been assumed that this layer is isotropic and linearly elastic. Accelerated trafficking tests conducted at the University of New South Wales have indicated that, early in the life of the pavement, the block layer may be characterised in terms of an equivalent elastic modulus of approximately 3200 MPa. This lies between the values of 900 MPa (Marais 1967) and 7500 MPa (British Ports Association 1982) used in earlier mechanistic analyses, but is less than values measured in Japan (Miura, Takaura and Tsuda 1984) and Europe (Kellersmann et al. 1984; Houben et al. 1982). Here, equivalent moduli ranging from about 250 to 170,000 MPa have been reported. Representative values are given in Table 1.

Base and Sub-Base

The exigencies of achieving rapid stress analyses by microcomputers make it highly desirable to select linear isotropic elastic deformation laws to characterise the pavement materials. This approach has been widely adopted in pavement analysis. Such a procedure provides acceptable modelling of stiff materials, such as concrete or stabilised layers, but is open to objection when modelling unbound materials such as crushed rocks and gravels. The stiffness of the unbound materials depends on the support provided by the underlying subgrade (the stronger the subgrade the stiffer the base material appears to be). More precisely, the stiffness of unbound materials depends on the stresses that they must carry. Consequently, because these stresses vary from point to point, the overall response of unbound materials needs to be characterised as being non-linear. This non-linear response can be modelled by dividing the layers of the material into a number of sub-layers to which moduli, etc. are then assigned on the basis of the stress conditions generated by the traffic loads. Because this is normally an iterative procedure it is not suited to manual calculations but is easily incorporated into a computer-based method such as that described shortly. Minimum values of the moduli of unbound materials have been reported to range between 150 and 700 MPa (Barker, Brabston and Chou 1982; Paterson and Maree 1978). Typical values are given in Table 1, together with representative values of the Poisson's ratio.

For cement-bound materials the reported moduli range between approximately 200 and 30,000 MPa (Paterson and Maree 1978). Cement-treated basecourses can be conservatively characterised in terms of a modulus of 3000 MPa or less. This value is close to that recommended by research workers at the Portland Cement Association (Larsen, Nussbaum and Colley 1969). For cement-treated sub-bases, a modulus of 15,000 MPa can be assumed. Again this value is believed to be conservative. Here, a Poisson's ratio of 0.35 is usually appropriate. Representative values are given in Table 1.

Subgrade

A crucial step in any pavement design procedure is to characterise the strength of the subgrade. In Australia, the subgrade material has naturally been characterised in terms of the California Bearing Ratio (CBR) of the soil. Normally the CBR value is determined by laboratory tests of compacted soil samples tested either dry (unsoaked CBR) or after immersion in water for several days (soaked CBR). The period of immersion is designed to ensure that the soil is tested in its weakest condition. Accordingly, the soaked CBR is the preferred measure of soil strength.

Fundamental subgrade properties such as the modulus may also be estimated from the CBR value. A variety of correlations have been proposed. Of these, the most widely accepted relationship can be written as:

$$E = 10 \times CBR \qquad (4)$$

where E is the modulus (MPa) and the CBR is in per cent. In the case of the Poisson's ratio, suitable values may be selected from Table 1. For Australia (and the U.K.) an alternative form of the modulus/CBR relationship has been proposed for

roads (Powell et al. 1984). This takes the form:

$$E = 17.6\ CBR^{0.64} \quad (5)$$

and has been incorporated into the mechanistic road design procedure described below.

CHARACTERISATION OF PAVEMENT RESPONSE

Having defined the characteristics of the individual pavement materials it becomes necessary to examine the overall response of a pavement to traffic. In practice, this requires that the cumulative effects of a series of traffic loads be assessed. One means that has been suggested for accomplishing this is to adopt Miner's linear cumulative damage hypothesis (Shackel 1985a).

The Miner hypothesis states that, irrespective of the magnitude of the stress, each stress repetition is responsible for a certain amount of fatigue damage. It is assumed that there is a linear rate of fatigue damage irrespective of the order of load application and that fatigue occurs when the sum of the damage increments at each level of stress accumulates to unity. The law can be expressed in the form:

$$\sum_{i=1}^{n} \frac{n_i}{N_i} = 1 \quad (6)$$

where N_i is the number of cycles to failure at stress level i, and n_i is the number of cycles adopted at stress level i.

Having decided to adopt Miner's hypothesis it remains only to postulate the mechanisms of failure (i.e. to predict N_i in eqn (6)) in order to estimate the service life of any pavement layer. Broadly, two criteria of failure can be adopted for unbound and cement-treated bases and sub-bases respectively. Where unbound materials such as crushed rocks or gravels are used, the pavement is assumed to fail by the gradual accumulation of permanent rutting deformation. In flexible pavement design, it has been commonly accepted that the rutting deformation can be related to the vertical compressive strain at the top of the subgrade. Various criteria have been proposed to relate the magnitude of these strains to the number of strain repetitions that the pavement can carry before developing unacceptable rutting. These have been reviewed elsewhere (Barker et al. 1982). The criterion most widely used in block pavement design has been that developed by Claessen et al. (1977) as the basis of the Shell design procedure for flexible asphalt pavements. This can be written as:

$$\epsilon = \frac{2800}{N^{0.25}} \quad (7)$$

where ϵ is the permissible subgrade compressive strain (microstrain) and N is the number of strain repetitions.

In the case of cement-bound base or sub-base materials, a fatigue failure criterion must be adopted. Several different fatigue criteria have been used in the design of block pavements (Shackel 1982b; Houben et al. 1982). For example, in the case of lean concrete bases, the following criterion has been recommended:

$$\epsilon_T = \frac{993500\ f'_c}{E_b^{1.022}\ N^{0.0502}} \quad (8)$$

where ϵ_T is the tensile strain at the bottom of the bound layer, f'_c is the characteristic compressive strength of the base material, E_b is the modulus of the base, and N is the number of load repetitions.

Similarly, for cement-stabilised (soil-cement) bases, a fatigue criterion developed at the Portland Cement Association in the U.S. by Larsen et al. (1978) has been used successfully. This may be written as:

$$R = \frac{k\ N^{0.025}\ h^3}{(2.1h - 1)^2} \quad (9)$$

where R is the permissible radius of curvature, h is the layer thickness, N is the number of repetitions and k is a constant characteristic of the soil.

ANALYSIS OF CONCRETE BLOCK PAVEMENTS

Several approaches to the analysis of block pavements have been described:

(a) modified slab analyses (Potter and Donald 1984; Barker et al. 1982);

(b) layered elastic analysis (Shackel 1982b; Houben et al. 1982); and

(c) finite element analyses (Houben et al. 1982; Nishizawa et al. 1984).

Of these, layered elastic theory represents the most widely used procedure for analysing pavements. Here, the pavement is modelled as a succession of layers each having linear elastic properties. The stresses and strains throughout the pavement may then be calculated as functions of the load magnitudes and placement and the layer thickness and properties.

Prior to the advent of high-speed digital computers in the 1960s, little progress had made made in the analysis of layered systems. However, since that time a variety of elastic solutions have been published including such widely known programs as CHEVRON, ELSYM, BISTRO and CIRCLY. Most of these programs require mainframe computers and are relatively cumbersome. This makes them unsuitable for pavement design procedures which, typically, entail hundreds of iterative analyses before a solution can be obtained. For this reason, pavement engineers frequently resort to the use of simpler, albiet approximate, methods of pavement analysis.

At least two approximate methods for the analysis of layered systems deserve consideration as the basis for an iterative design procedure. The first of these is to use existing mainframe programs to derive a series of statistically-based regression equations to link the critical stresses and strains in a pavement to such factors as layer thickness, layer moduli, fatigue relationships, applied loads and subgrade properties (Shackel 1985a). Although the method is capable of yielding equations of high predictive value, it usually requires a major computing and analysis effort to derive the design equation. However, the method yields a very compact module of computer code for determining the stresses and strains. This advantage is offset by the need to ensure that the design conditions fall wholly within the normally very limited domain of the regression analyses. This severely limits the usefulness of the regression approach.

An alternative to the regression method is to adopt approximate methods of stress analysis. Of these, the Method of Equivalent Thicknesses (MET) originally developed by Odemark (1949) and further developed by Ullidtz (1974) and others offers a number of practical advantages. Firstly, subject to certain restrictions (Ullidtz 1974) on the thicknesses and modular ratios of successive layers, it can be used to analyse a wide range of practical pavement situations. Secondly, it yields very compact computer code. Here, the amount of code needed is typically about three to five times that needed for the regression method described above but this only represents a fraction (typically less than 10 per cent) of the code needed for more exact programs such as CHEVRON. This means that it is ideally suited to iterative design procedures implemented on microcomputers.

Details of the MET have been given elsewhere. Essentially, the method involves replacing the actual thickness of each layer by the equivalent thickness so that the pavement may be progressively modelled in terms of an equivalent elastic half space. Because the method is approximate, Odemark (1949) originally noted the need to incorporate correction factors in the calculation of the equivalent thickness. Typically such factors range between about 0.75 and 1.0 (e.g. Barber and Knapton 1982). Although some authorities such as the British Ports Association have apparently ignored the need for correction factors, in general it is prudent to incorporate them into the design analysis. The factors vary depending on the materials used but can be determined by trial by comparing MET results with those obtained by more exact analyses such as, for example, CHEVRON. Provided suitable calibration factors are determined, then good correspondence can be obtained between the MET analysis and other methods. This is illustrated, using typical data obtained by the author, in Fig. 3 for the critical strain components in a three-layer block pavement.

The method of equivalent thicknesses has been successfully applied in the analysis both of flexible pavements (Molenaar et al. 1984) and of interlocking concrete block pavements (Shackel 1980b; NCMA 1983). To date, such analyses have assumed that the pavement materials are isotropic. However, it is worth noting that the MET procedures can, if required, be extended to embrace limited cases of anisotropy.

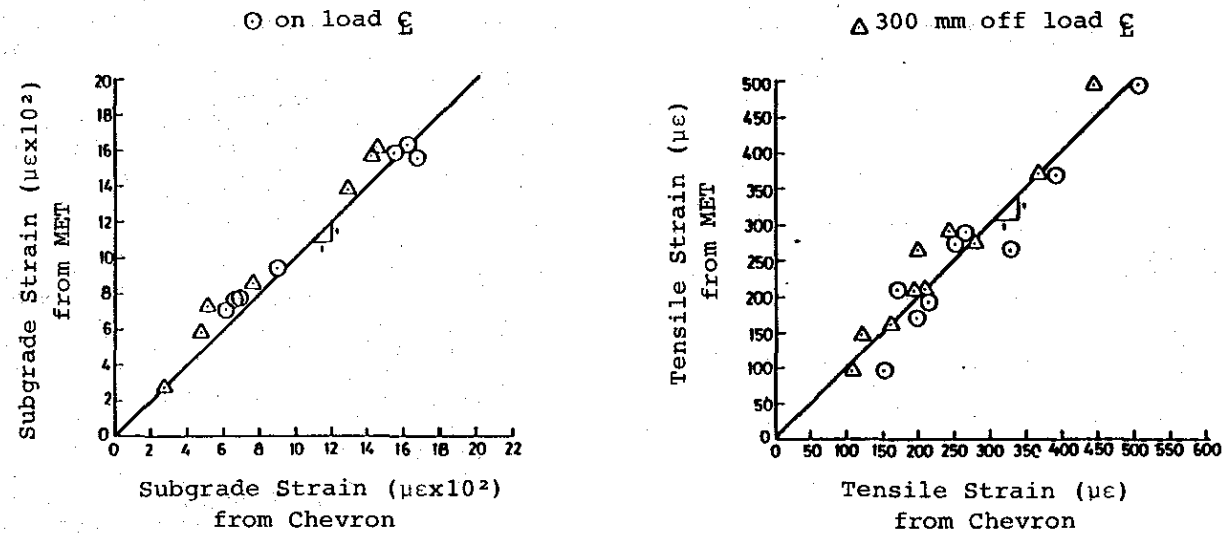

Fig. 3 - Comparison of results obtained using the method of equivalent thicknesses with results from Chevron analysis

DESIGN PROCEDURES FOR BLOCK PAVEMENTS

The methods that are available for the design of concrete block pavements can be divided into four categories:

(1) design on the basis of experience;

(2) ad hoc modifications of existing design procedures for conventional flexible pavements;

(3) empirical designs based on full-scale trafficking tests; and

(4) mechanistic designs based on the determination of design parameters from field and laboratory tests.

These methods have been comprehensively reviewed and evaluated elsewhere (Shackel 1979a, 1980b, 1984 and 1985a). Of the various approaches listed above, mechanistic procedures are the most versatile and comprehensive. Details of such procedures developed in Australia are now given.

METHODOLOGY OF MECHANISTIC DESIGN

Mechanistic pavement design involves the formulation and solution of a boundary value problem. This requires the determination of the stresses and strains at critical locations through the pavement. These are compared with the values that would, theoretically, be predicted to cause failure of the pavement. As noted above, failure may be the result of either excessive rutting or, in the case of bound materials, of extensive cracking leading to a loss in both strength and stiffness. Typically, as shown in Fig. 4, the critical stresses and strains occur either on or near the vertical load axis at the bottom of all bound (brittle) layers and at the top of the subgrade. The fatigue life of the bound layers can be related to the repeated tensile strains or stresses whilst the permanent deformation or rutting of the pavement is normally assumed to be a function of the repeated vertical compressive strains at the top of the subgrade. Provided the critical stresses and strains can be calculated, it is possible to predict the service life of the pavement. Conversely, for a designated service life, it is possible to calculate what values of stresses and strains can be tolerated and, by trial and error, to determine what combination of layer thicknesses

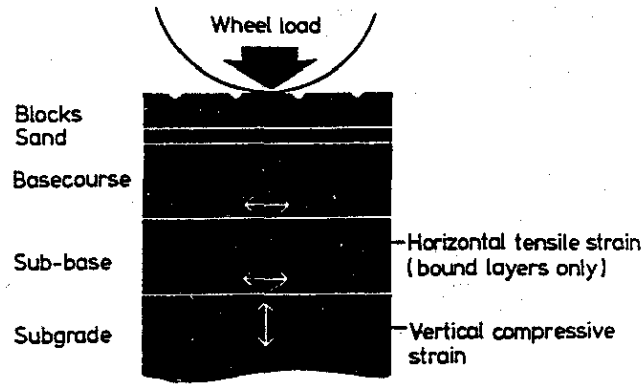

Fig. 4 - Location of critical strains

are required if these stresses and strains are not to be exceeded. Thus mechanistic pavement design involves, first, the analysis of the stress and strain distributions caused by some designated loading and, second, the progressive adjustment of layer thicknesses until the predicted stresses and strains can be deemed to be insufficient to cause failure within the required service life of the pavement. The adjustment of layer thicknesses must be done iteratively. This implies that a large number of calculations will be necessary. Because of this, and because of the mathematical complexity of analysing layered systems, mechanistic analyses normally require the use of computers.

In Australia, the author has developed mechanistic design procedures for rigid pavements (Shackel 1986b), asphalt pavements (Shackel 1986a) and concrete block pavements (Shackel 1985a). These methods are capable of implementation on 16-bit microcomputers (Shackel 1985b). Details of the block pavement design procedures now follow.

MECHANISTIC PROCEDURES FOR DESIGN OF BLOCK PAVEMENTS

The structure of the computer-based design procedure is illustrated schematically in Fig. 5. From this Figure, it may be seen that the method is made up of two sequential procedures comprising:

(a) data input - this comprises the choice of pavement materials and the assembling of those design data needed by the program; and

(b) pavement design - this comprises the systematic calculation of layer thicknesses for the various materials chosen to make up the pavement.

Each of these procedures is now considered in more detail.

DATA INPUT

The data required for the program comprise:

(a) the properties of the surface, base and sub-base materials;

(b) the strength of the subgrade; and

(c) the design loadings.

Materials Properties

In the case of the pavement methods the designer must first choose the type, thickness and laying pattern of the paver. For road and industrial pavements, accelerated trafficking tests (Shackel 1979d) have demonstrated that it is prudent to select blocks having a category A shape as defined by the Cement and Concrete Association of Australia (Hodgkinson and Morrish 1982) laid in herringbone bond. Usually a thickness of 80 mm will prove adequate for all types of loading. The properties of the block surfacing can then be chosen using typical published values such as those given in Table 1, or can be measured directly by Falling Weight Deflectometer or other tests.

Once the block surface has been characterised, the designer must choose the type of base and, if relevant, sub-base material. Normally the choice of material will be dictated by economic consideration. However, it is worth noting that comparative trafficking tests of unbound and cement-stabilised basecourses in block pavements have, in all cases, demonstrated that the performance of pavements incorporating cement-stabilised bases is superior to that of pavements incorporating crushed rock or gravel bases and sub-bases (Shackel 1980c and 1982a). Moreover, it should be noted that the design thicknesses of a cement-bound layer are less sensitive to changes in traffic or subgrade strength than where unbound materials are used.

If the designer elects to use an unbound crushed rock or gravel as the base or sub-base, he need not determine the elastic properties of the material because, as noted above, the design program models such materials as stress-dependent and will automatically assign moduli to the materials depending on the support of the underlying layer by using methods developed earlier for flexible pavement design (Barker et al. 1982). Here the program limits the maximum modulus to 350 MPa for base and 225 MPa for sub-base materials. Comparison with Table 1 shows these values to be conservative.

In the case of bound materials the designer may either choose conservative values of moduli, etc. from typical published data (e.g. as summarised in Table 1). Alternatively, data obtained from laboratory tests on the particular materials selected may be used.

Subgrade Strength

The strength of the subgrade should normally be specified in terms of a soaked CBR value determined by laboratory tests. Values estimated from soil classification data or from in situ penetrometer tests can also be

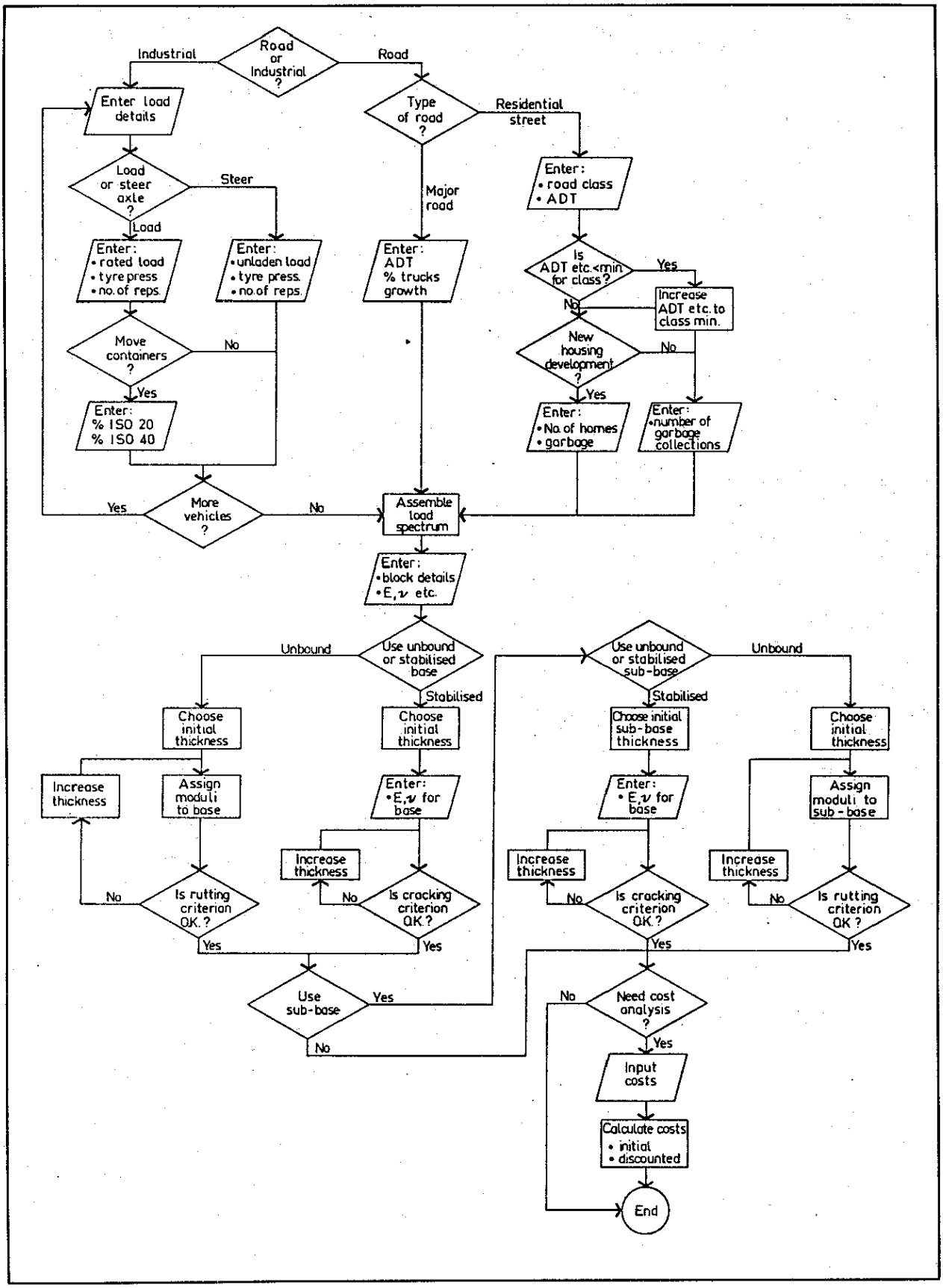

Fig. 5 - Flow chart of computer program

used but are far less reliable as indicators of the subgrade strength. Most road and industrial pavement sites will exhibit variability in the soil properties from place to place. Accordingly, it is important to select a representative CBR value for design purposes. Normally this implies that a series of CBR determinations representating different site areas will be needed. This enables the mean CBR and standard deviation to be calculated. It is then prudent to design the pavement in terms of the lower 10th percentile CBR (mean - 1.3 standard deviation). This implies that there will be only a 10 per cent chance of encountering less than the design CBR. Thus, the chances of a pavement failure attributable to the use of a non-representative CBR value can be minimised.

Traffic Loadings

For convenience, the design loadings for roads and industrial hardstands will be considered separately.

Road Traffic

As noted earlier, trafficking tests of prototype block pavements (Shackel 1979c) have shown that, once lock up is achieved, they are relatively insensitive to either the numbers or magnitudes of the wheel loads (for loads < 70 kN). In practice, the effects of light vehicles such as cars and trucks having a gross weight less than 3 t can be disregarded for design purposes. Rather, design should be based solely on the number of heavy vehicles to be carried by the pavement. In Australia, NAASRA loadometer surveys have established the distributions of axle type and load. For design purposes, Potter and Donald (1984) have recommended the distributions shown in Fig. 1 as a basis for pavement design. Accordingly, this has been incorporated into the design methodology described here. In general, the designer need only obtain an estimate of the average daily traffic (ADT) to be carried on this pavement, the traffic growth rate and the percentage of vehicles heavier than 3 t gross weight. The program then automatically calculates the distribution and numbers of axle loads according to Fig. 1.

In the case of residential streets, studies supervised by the Australian Road Research Board (Schofield 1985) have established that the percentage of heavy vehicles is typically less than 2 per cent of the ADT. Here, other factors need to be included in the design input. First, in the case of a new residential subdivision, it is important to recognise that the greatest intensity of truck traffic is generated very early in the pavement life in the form of construction traffic bringing in building materials. The average numbers and axle loads of trucks per home site have been estimated for typical Australian conditions (Schofield 1985). These estimates are incorporated into the design program. This enables the designer to specify the number of home sites to be served by the road. The program will then automatically calculate the total number of construction vehicles that will use the street. Conservatively it is assumed that all of these vehicles will traverse the same spot in the pavement. It is further assumed that, during this early construction trafficking, the block surface will initially be no stiffer than the underlying basecourse and will only develop the full specified modulus during the passage of the first 10,000 vehicles. This assumption is conservative but recognises the important point that residential street pavements carry the heaviest traffic they will ever experience during the construction of the houses, i.e. at a time when the blocks have not received sufficient traffic to develop interlock.

The second refinement of the specification of traffic loads for residential streets lies in the ability of the designer to specify the number and type of garbage compactor trucks that will service the street. Here the program uses data obtained in Melbourne by ARRB. This design feature recognises that the passage of heavily-laden garbage vehicles often represents the heaviest axle loads to be regularly carried by a residential street pavement. These may comprise a significant percentage of the total number of design loads to be carried over the design life of the pavement (typically assumed to be 20 years).

Industrial Traffic

In the case of industrial block pavements, the designer should assemble a schedule of all vehicles that will use the pavement. For each axle of each vehicle the following information is required:

(a) axle load,
(b) number of wheels,
(c) tyre pressure,
(d) number of vehicle movements,
(e) type of axle (for container-handling vehicles only), and
(f) proportions of 40 ft and 20 ft containers.

In the case of container-handling vehicles it has been assumed that the loads applied by the load axle will vary according to the distribution of container weights set out in Fig. 2. The loads applied by steer axles are assumed, conservatively, to remain constant at their maximum values (i.e. for the unladen vehicles).

From the data given above, a distribution of the magnitudes and numbers of axle loads to be carried by the pavement is automatically assembled. By combining the distribution with the damage concept represented by eqn (6), it becomes possible to model accurately the true service history of the pavement without recourse to the dubious notions of axle load equivalency or the use of so-called 'design vehicles' that form the basis of most earlier design procedures for industrial block pavements (e.g. NCMA 1983).

PAVEMENT DESIGN METHODOLOGY

Once the input data have been assembled, the designer must decide whether or not to use a sub-base in addition to a base. Essentially this is an economic decision that can be easily made by running a series of comparative designs. However, as a broad generalisation, the decision to use a sub-base will only be justified where:

(a) the subgrade is weak (CBR < 5 per cent, say),

(b) the axle loads are high, or

(c) the available basecourse materials are unusually expensive.

The design program executes the following steps.

(a) The loads to be carried by the pavement are expressed in terms of a spectrum of design axle loadings. As noted above, distributions such as those given in Fig. 1 may be used for roads. Alternatively, industrial loading spectra incorporating the data shown in Fig. 2 may be chosen.

(b) The base and sub-base layers are specified in terms of being either unbound granular or cement-stabilised materials. In the case of cement-stabilised materials, they are further classified as being either coarse-grained or fine-grained. This recognises that the fatigue characteristics of these two categories of material are quite different from one another (Paterson and Maree 1978).

(c) The subgrade is characterised in terms of a soaked CBR value which is then related to a subgrade modulus using eqn (4) for hardstands or eqn (5) for roads.

(d) The thickness of the block layer is specified. (As noted above, for most practical cases, a thickness of 80 mm will be suitable provided dentated blocks are used).

(e) The program selects trial thicknesses (> 100 mm) of the base or sub-base.

(f) For each of the design axles the distribution of stresses and strains is computed. In the case of unbound materials the base or sub-base layers are automatically subdivided into three thinner layers which are assigned values of moduli depending on the stiffness of the underlying layers according to the criteria developed earlier for conventional flexible pavements (Paterson and Maree 1978). In this manner the granular materials are treated as being stress dependent (non-linear).

(g) Having computed the distribution of stress and strain, the number of repetitions of those stress and strain magnitudes that the pavement can withstand prior to failure are calculated in accordance with the failure/damage criteria detailed above.

(h) Steps (f) and (g) are then repeated for the remaining design axles and the cumulative damage factor is calculated using Miner's law (eqn (6)). If the factor exceeds unity the trial thicknesses are progressively incremented (i.e. steps (e), (f) and (g) are repeated) until the pavement can be shown to have sufficient fatigue and damage capacity to withstand the entire planned spectrumn of loading. Where both a base and sub-base is to be designed, the program first assumes a value of the support to be provided by the sub-base and determines an appropriate base thickness. The corresponding sub-base thickness is then calculated. In estimating the sub-base support it is assumed, conservatively, that, estimating the sub-base support it is assumed, conservatively, that, irrespective of the type of sub-base material, it behaves as a granular material with a stress-dependent modulus which will not exceed 225 MPa for gravel or 400 MPa for cement-treated sub-base.

ILLUSTRATION OF DESIGN METHODOLOGY

Using the methodology just described, design curves have been prepared for

both road pavements and industrial hardstandings. For roads the traffic is assumed to follow the loading spectrum given in Fig. 1. In all cases the tyre pressure has been assumed to be 700 kPa. The resultant design curves are given in Fig. 6.

Because industrial pavements typically must accept a mix of heavy vehicles, each having different loading characteristics and each applying different numbers of load repetitions, it is not practical to prepare design nomographs to cover the full range of industrial loadings. Nevertheless, as an illustration of the mechanistic design procedure just described, sets of design curves have been prepared and are presented in Fig. 7 for vehicles handling containers, the weights of which are distributed according to Fig. 2. Strictly these curves will only yield optimal designs where it can be assumed that only one type of vehicle will operate on the pavement. Where the pavement is to carry a range of vehicle types and weights then, ideally, the full computer-based methodology should be employed. However, the nomographs given in Fig. 7 can still be used for design by selecting the curve appropriate to the heaviest axle to be carried by the pavement and by assuming that all vehicle movements will be at this axle load. This will yield a conservative but non-optimal design.

In practice, Fig. 7 represents just a subset of the design possibilities. For example, for 10^6 repetitions of an axle load of 100 t there are at least six economically feasible pavement alternatives. These are shown in Fig. 8. This further emphasises the ease with which computer-based mechanistic procedures can compare different types of pavement.

The curves plotted in Figs 6 to 8 are given merely as illustrations of the mechanistic design procedure. In this respect it is important to note that the curves are specific to sets of conditions representing particular combinations of wheel loads, block type and thickness, and base types. A major advantage of mechanistic design procedures is that they can rapidly provide designs appropriate to any designated combination of service conditions. However, the implementation of such anapproach requires the use of computers. Fortunately, the design procedures lie within the capabilities of relatively inexpensive microcomputers.

VERIFICATION OF DESIGN PROCEDURES

The procedures just described have been used in the design of a wide range of roads, bus termini and port and container facilities in Australia,

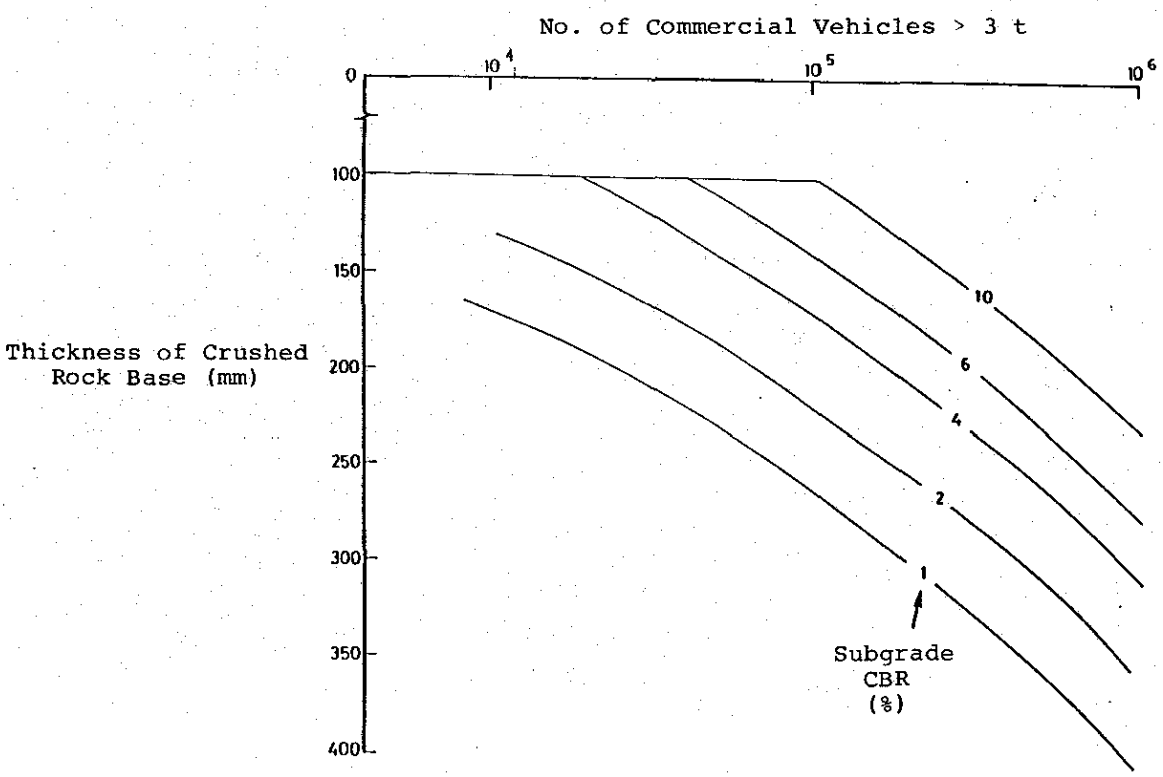

Fig. 6 - Typical road pavement design curves

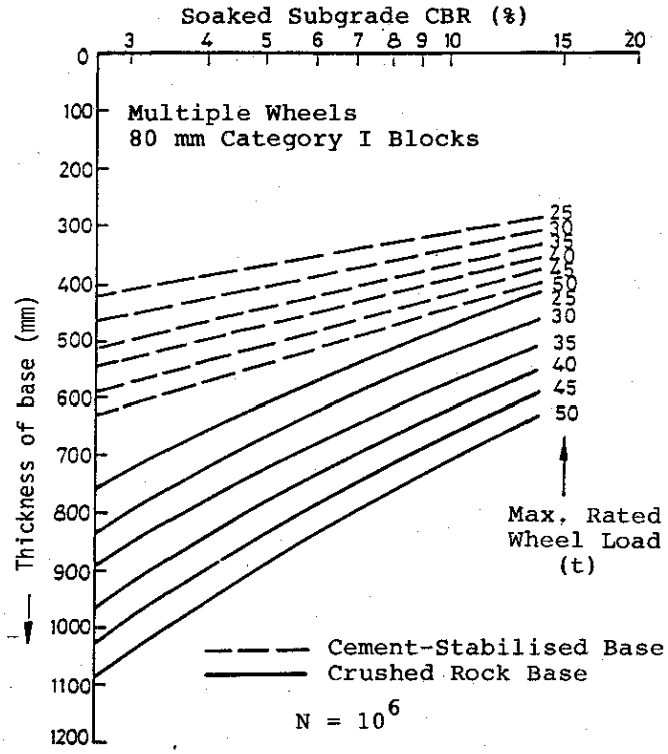

Fig. 7 - Design curves for industrial pavements subject to a single design vehicle

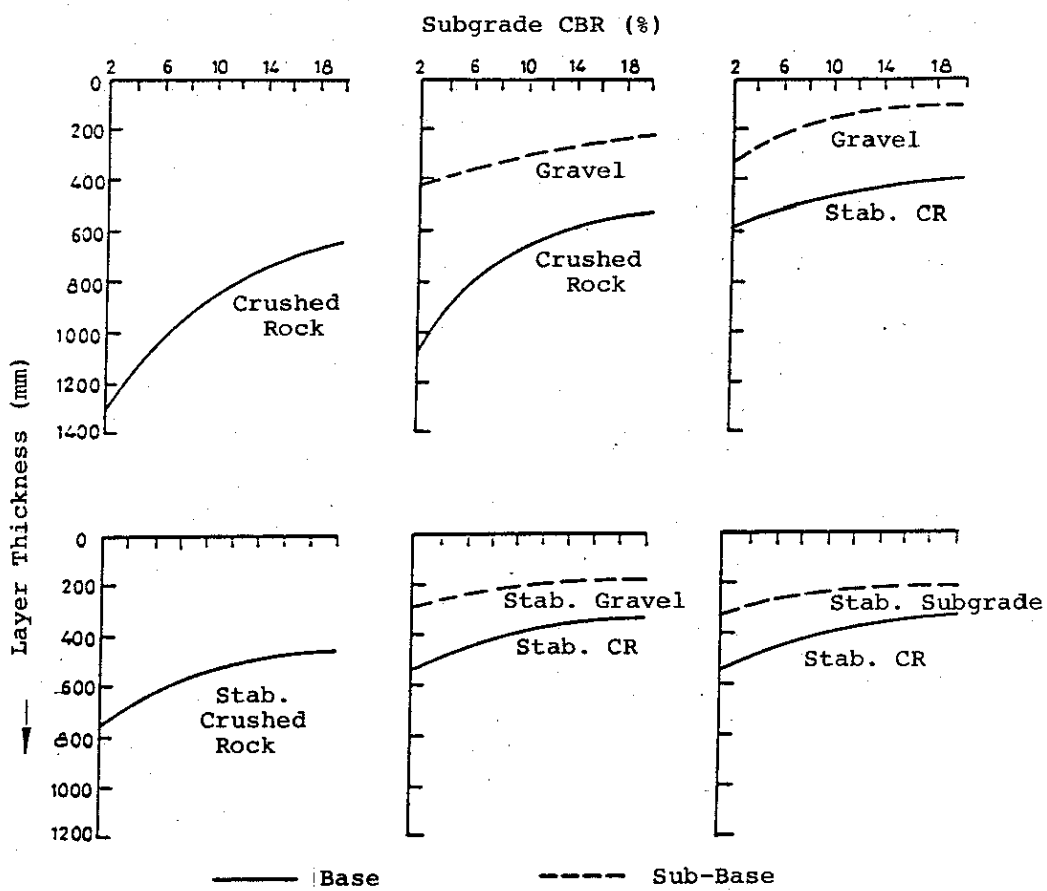

Fig. 8 - pavement design alternatives for vehicle having a 100 t axle load

Canada and the U.S., including such major projects as the Massey Coal Terminal, Newport News, Virginia (Brunston 1984) and the Canadian Pacific Intermodal Facility, Edmonton. This widespread experience suggests that the methodology is conservative. This conclusion is suggested by comparisons of the thickness indicated by the mechanistic method with those actually chosen in a number of block pavements serving satisfactorily around the world for which reliable data are available. Figs 9 and 10 compare the theoretical and as-constructed base thicknesses for roads and industrial pavements. It may be seen that the mechanistic design procedures tend to require slightly greater thicknesses than those found in practice to be satisfactory. In other words, the design method appears to be conservative.

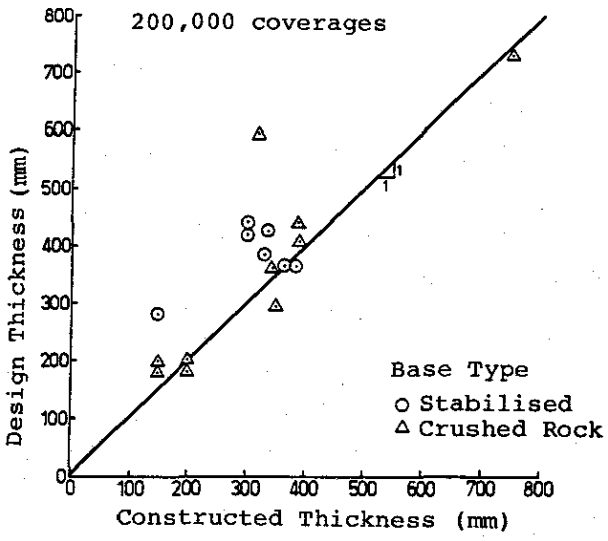

Fig. 9 - Comparison of thickness given by the computer program with those actually constructed (industrial pavements)

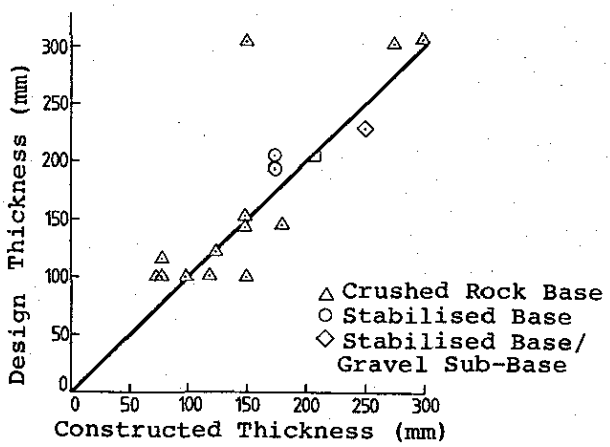

Fig. 10 - Comparison of thickness given by the computer program with those actually constructed (road pavements)

ADVANTAGES OF A COMPUTER-BASED METHODOLOGY

The mechanistic procedures described in this paper offer the following advantages over earlier manual design procedures.

(a) They predict more rigorous modelling of both the loading conditions and the materials characteristics than can be achieved using earlier procedures.

(b) It is feasible to consider a far wider range of potential material combinations than is possible by using manual procedures. This can aid the designer to achieve a more cost-effective solution than where he is restricted to consideration of just a single type of pavement or a single type of material as is the case with most design procedures published to date.

(c) The methods are systematic; they lead the user through a systematic design procedure which ensures that all essential design requirements are considered and that the designer's attention is drawn to the consequences of poor or incorrect decisions. For example, it is possible to warn against the use of materials which do not meet the requirements implicit in a particular design method. In this way it is possible to assist the tyro engineer.

(d) The methods concentrate on essentials. By isolating the designer from the minutia of complex design calculations he is left free to concentrate on the important objective of optimising his design.

(e) The methods provide a rapid design solution. By yielding a fast means of obtaining design solutions, computer-based methodology encourages the designer to examine the relevance and importance of the design criteria and to refine progressively the design assumptions (e.g. by progresssively choosing different combinations (and costs) of materials).

CONCLUDING COMMENTS

A wide variety of methods is available for the design of concrete block pavements. However, many of these are specific to particular types of base materials, to very narrow ranges of

subgrade conditions or to simplistic concepts of traffic loadings. Fortunately, the emergence of sophisticated mechanistic design procedures have provided engineers with powerful new tools for studying a wide range of practical pavement materials and subgrades. A particular advantage of such methods is that they are capable of accurately modelling complex loading conditions such as those common in industrial paving applications. Moreover, the methods allow the designer to study rapidly a wide range of materials used either singly or in combination with other materials. This facilitates the achievement of an optimal economic design. For this reason, mechanistic design procedures are likely to be used increasingly in pavement engineering in the future.

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