Stabilized Bases and Thin Asphalt Surfacings. Proc., 4th International Conference on the Structural Design of Asphalt Pavements, Univ. of Michigan, Ann Arbor, Vol. 1, 1977, p. 409.
12. P. E. Fossberg, J. K. Mitchell, and C. L. Monismith. Load-Deformation Characteristics of a Pavement with Cement-Stabilized Base and Asphalt Concrete Surfacing. Proc., 3rd International Conference on the Structural Design of Asphalt Pavements, Univ. of Michigan, Ann Arbor, Vol. 1, 1972, p. 795.
13. E. Otte and C. L. Monismith. Some Aspects of Upside-Down Pavement Design. Proc., 8th Australian Road Research Board Conference, Perth, 1976.
14. E. Otte. Factors Affecting the Behaviour of Cement-Treated Layers in Pavements. Proc., 9th Australian Road Research Board Conference, Brisbane, 1978.
15. G. M. Dormon and C. T. Metcalf. Design Curves for Flexible Pavements Based on Layered System Theory. HRB, Highway Research Record 71, 1965, pp. 69-84.

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Design of Pavements with Lean-Concrete Bases

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In the analytic approach to pavement design, structures with lean-concrete bases require certain distinctive considerations that arise from the incidence of cracks in the lean concrete attributable to thermal and shrinkage effects that influence stresses and strains in the rest of the structure. Details of a two-stage design process, for both conventional lean-concrete-base structures and for those that involve a "sandwich" layer of unbound granular material above the lean concrete, are presented. Both types of structures incorporate 100 mm of rolled-asphalt surfacing; the conventional structures use an additional thickness of dense bitumen macadam as part of a composite base. In stage 1 of the procedure, the full uncracked value of modulus is used for the lean concrete, and pavement life, in terms of numbers of standard axle loads, is calculated to the point at which secondary (traffic-induced) cracking occurs. In stage 2, the modulus of the lean concrete is substantially reduced as a consequence of this cracking, and additional life is calculated on the basis of asphalt fatigue cracking or permanent deformation, whichever is critical. The total life is the sum of stages 1 and 2. The results are compared with current design recommendations and with the results of full-scale trials. Since the granular material can only develop a modest value of modulus in sandwich construction, that technique does not appear to be very effective unless it can be shown to reduce the incidence of reflection cracking in the asphalt surface.

In applying the principles of analytic pavement design to structures that incorporate lean concrete layers, the occurrence of cracks, which develop in this material before traffic loading, introduces a problem that is not encountered with asphalt or unbound bases. If this "primary" cracking is augmented by the cracking caused by traffic, the resulting lean-concrete layer has an effective elastic modulus that is little better than that of granular material, and a figure of 500 MPa has been proposed (1). Alternatively, if this "secondary" (traffic-associated) cracking is avoided, then the full intact strength and stiffness of the lean concrete are available between cracks. Pell and Brown (2) have suggested, therefore, that this type of pavement should be designed in two stages, the first involving only primary cracking and the second involving secondary cracking. This approach has been developed by Walker and others (3), who produced analytical data related to the effects of primary cracking on the critical stresses and strains in the structure. This paper uses this and other relevant information in an analytically based design method for structures with lean-concrete bases, such as those used i Britain. It also considers the potential of "san construction, which involves a granular layer b the asphalt surfacing and the lean-concrete base

DESIGN CONSIDERATIONS

Lean concrete, as used in British pavements (4 quires good-quality aggregate to a fairly strict g and mix proportions that will produce a cube str of 10-20 MPa at 28 days. Although this is the m considered in this paper, the design principles well apply to lower-quality, cement-treated mat The primary cracking of lean concrete is conside Williams (5) to be caused by warping stresses an grade restraint stresses that arise during contr of the material under falling temperatures rathe by shrinkage in view of the good quality of aggre in lean concrete. Whatever the cause, these pri cracks are well spaced, and between them the v stiffness of lean concrete and its reasonable ten strength are fully available to the designer. How allowances must be made for the situation near t cracks; a procedure for estimating the critical st and strains in this location has been suggested b and others (3). Stage 1 of the process assumes t primary cracking has taken place, whereas stage lows consideration of an extensively cracked base occurrence (or otherwise) of this secondary crac which is largely caused by traffic-induced stress indicated by the results of the stage 1 analysis. total life of the pavement is simply the sum of the calculated for the two stages. Stage 2 will not oc those pavements that are strong enough to prevent traffic-induced cracking in the lean concrete.

During stage 1 of the design, the modulus of the concrete is much greater than that of the overlying tuminous material. The horizontal strains at the b tom of this bituminous material are, therefore, com pressive, and hence the question of fatigue cracki not arise. This statement should be qualified by n that tensile strain does develop in the vertical dire near the surface between dual wheels. Its magnitude was not large enough to be of importance in the designs considered in this paper, but it could be of some consequence in other situations and deserves further investigation. In stage 2, when the modulus of the lean concrete is substantially lower, the possibility of fatigue cracking in the bituminous material must be considered.

The subgrade strain criterion used to deal with permanent deformation in pavements that have bituminous bases (6) is suitable for stage 2 calculations on lean-concrete bases but not for stage 1. Lower allowable strains should be associated with this more rigid construction, and an adjustment has been effected by using the deflection criteria developed by Lister (7). He found that the allowable deflection for a given pavement life with cemented bases is about 75 percent of that for fully flexible construction. The justification for use of this finding is that subgrade strains do correlate well with surface deflections (8).

A major problem in the use of lean concrete in practice is reflection cracking. Even in pavements in which the lean concrete shows only primary cracking, these cracks can propagate through the overlying bituminous material if it is too thin. Because analytic methods have not yet been devised to consider reflection cracking in any detail, the matter is not considered in this paper. However, the thinnest total thickness of bituminous cover used in the design calculations was 100 mm. This should provide some resistance to reflection cracks, particularly since the material involved was hot-rolled asphalt, which has good inherent crack resistance.

MATERIAL CHARACTERISTICS

The modulus of elasticity of lean concrete is related to its tensile strength. The following relation can be derived from the results presented by Williams (9):

$$E = 7.125f + 20\,500 \text{ MPa} \quad (1)$$

where f = flexural strength (MPa). It should be noted that distinctly different relations apply for other cement-bound materials.

Flexural strength f can be estimated from compressive cube strengths u_c by using

$$f = 0.1 u_c \quad (2)$$

Williams (9) has indicated this to be the lower-bound value, and it is therefore appropriate for design. Since current practice uses cube strength to specify the material quality, Equations 1 and 2 allow the elastic modulus to be estimated. The range of cube strengths allowed (4) is 10-20 MPa; these correspond approximately to moduli of 28 000-34 000 MPa, respectively. The other parameter required for analysis is Poisson's ratio. A study of relevant literature (10) indicates that 0.2 is an appropriate value to use. When a lean-concrete layer becomes extensively cracked as a result of traffic stresses, its effective modulus of elasticity is substantially reduced. It approaches a value similar to that for unbound granular material [a figure of 500 MPa has been suggested (1,3)]. The Poisson's ratio associated with this has been taken to be 0.3. The table below summarizes the elastic properties used for the lean concrete:

	E (MPa)		
Condition of Material	u_c = 10 MPa	u_c = 20 MPa	ν
Primary cracking	28 000	34 000	0.2
Secondary cracking	5 000	5 000	0.3

Lean concrete is susceptible to fatigue cracking. Figure 1 shows the results of some work carried out at the Transport and Road Research Laboratory (TRRL) that indicates that the criterion is the tensile stress expressed as a proportion of the tensile strength (f_c/f). Superimposed on Figure 1 are the results of tests by Kolias (10), who used a different test technique but whose results show general agreement. For the purposes of design, the endurance limit was taken as

$$f_c/f = 0.6 \quad (3)$$

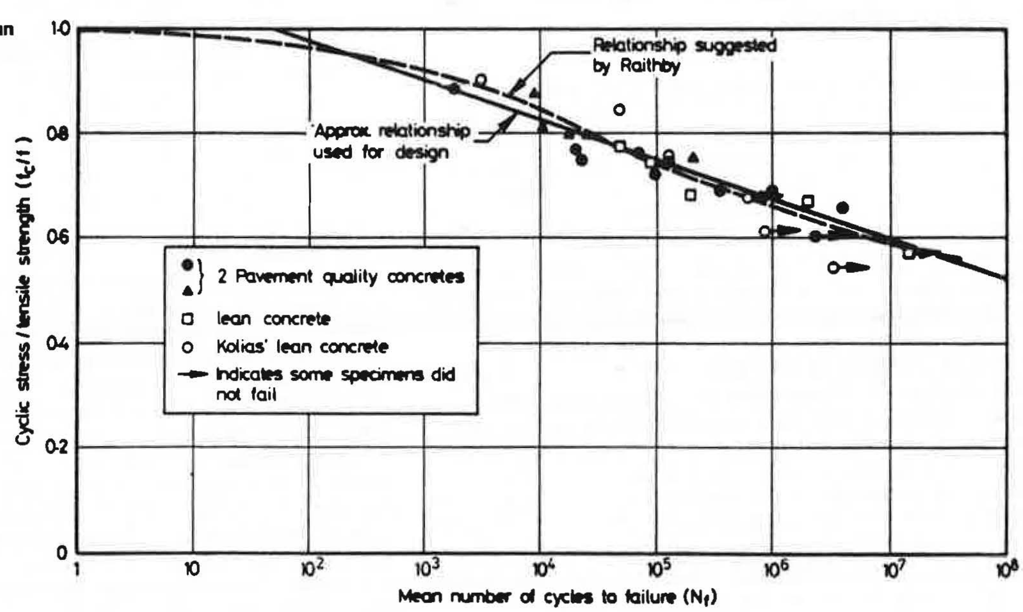
A straight-line relation was used to determine the fatigue lives N_f at higher stress levels, as follows:

$$N_f = 10 \exp(14.9 - 13.2 f_c/f) \quad (4)$$

Figure 1 shows that this approximation is inappropriate at stress levels above 0.95, but the corresponding lives are so short that this is of no consequence.

Walker and others (3) and Mitchell and Monismith (12)

Figure 1. Fatigue results for lean concrete.



both used tensile strain rather than stress as their design criterion. The choice of stress for this work was dictated by the nature of the rather limited data available on the material used in Great Britain.

DESIGN CRITERIA

The design calculations generally took place in two stages that corresponded to the situations of primary and secondary cracking in the lean concrete. In stage 1, the two design criteria were as follows:

1. Tensile stress at the bottom of the lean concrete (σ_θ), expressed as a proportion of the tensile strength (f)—i.e., σ_θ/f—to deal with fatigue cracking.
2. Maximum compressive strain in the subgrade (ϵ_z) to deal with permanent deformation.

The modified relation to give allowable strains that are 75 percent of those appropriate to bituminous and unbound bases is

$$N_f = (1.07 \times 10^{15})/\epsilon_z^{3.57} \qquad (5)$$

To take account of the intermittent primary cracks, stresses and strains calculated from pavement analysis were increased as follows (3): Tensile stress in the lean concrete × 1.25 and compressive strain in the subgrade × 2.5. These adjusted values were then used in assessing pavement lives.

In stage 2, the design criteria were those used in bituminous base constructions (6):

1. Tensile strain at the bottom of the bituminous layer to prevent fatigue cracking (the layer in question would be the one immediately above the lean concrete), and
2. Subgrade strain to limit permanent deformation.

DESIGN CALCULATIONS

Details of the structures that were considered are given in Table 1. A standard 40-kN dual wheel load was used in which each wheel had a contact pressure of 500 kPa. The space between contact areas was 110 mm.

Variability in the lean concrete was dealt with by considering the range of compressive strengths (10-20 MPa) allowed in current specifications (4). For each structure, the thickness of lean concrete was varied in 50-mm increments between 100 and 300 mm. A design life was then calculated for each.

Typical details for one of the design calculations are given in Table 2 and in the two text tables below (the lean-concrete specification is u_o = 10 MPa, f = 1050 kPa, and E = 28 000 MPa). The case considered in these tables is a design that has 75 mm of dense bitumen macadam (DBM) as the top of the composite base. The quality of the lean concrete is taken to be at the low bound of the specification (10 MPa).

For stage 1 (primary cracking only), the tensile stress in the lean concrete is multiplied by the factor 1.25 to allow for the cracks. This figure is then used to evaluate a life N_1, in millions of standard axle loads (msa), by using the fatigue relation of Equation 4 and the endurance limit. The latter implies that infinite lives result when the stress term 1.25 σ_θ/f is 0.6 or less. The stage 1 deformation analysis is considered on the right-hand side of Table 2. Here the design parameter is multiplied by a factor of 2.5, and a corresponding life N_1' is determined by using the modified relation of Equation 5.

The table below gives the stage 2 calculations (completely cracked):

Thickness of Lean Concrete (mm)	Tensile Strain in DBM × 10^{-6}	N_2 (msa)	Subgrade Strain × 10^{-6} (mm/mm)	N_2' (msa)
100	87	2.3	355	2.4
150	79	3.1	324	3.3
200	73	3.9	293	4.7

These calculations were only necessary for thicknesses of 100, 150, and 200 mm because the two larger thicknesses of lean concrete did not exhibit traffic-associated cracking. The analyses for stage 2 use the reduced modulus (500 MPa) for the lean concrete, and lives based on fatigue cracking in the DBM (N_2) and permanent deformation (N_2') are obtained. In this case, the fatigue lives (N_2) are slightly shorter and, therefore, critical.

Table 1. Details of analyzed structures.

Layer	Material	Stiffness (MPa)	Thickness (mm)
Wearing course	Rolled asphalt	6400	40
Base course	Rolled asphalt	7000	60
Composite road base	Dense bitumen macadam	9400	0, 75, 125
	Lean concrete, uncracked	28 000 or 34 000	100-300
	Lean concrete, cracked	500	
Subbase	Granular	75	200
Subgrade	3 percent California bearing ratio	30	-

Table 2. Details of typical design calculations: stage 1.

Thickness of Lean Concrete (mm)	Tensile Stress in Lean Concrete (σ_θ) (kPa)	1.25 σ_θ/f	N_1 (msa)	2.5 × Subgrade Strain × 10^{-6} (mm/mm)	N_1' (msa)
100	1110	1.32	0*	348	0.9
150	836	0.99	0*	248	3.0
200	645	0.77	0.05*	185	8.6
250	500	0.60	∞	140	23.0*
300	374	0.45	∞	113	50.0*

*Critical life.

The total pavement lives are calculated below:

Thickness of Lean Concrete (mm)	Life (msa) Stage 1	Stage 2	Total (stage 1 + stage 2)
100	0	2.3	2.3
150	0	3.1	3.9
200	0.05	3.9	4.0
250	23.0	–	23.0
300	50.0	–	50.0

For the three smallest thicknesses of lean concrete, this involves summation of the lives from both stages, but for the 250- and 300-mm thicknesses, only stage 1 is involved.

The results of all calculations are summarized in Table 3.

A graphical representation of all the results is given in Figure 2, which shows the thicknesses of lean-concrete base for given pavement lives. A pair of curves is given for each of the three thicknesses of DBM used in the composite base. The asphalt thicknesses shown in the figure refer to the total thickness of bituminous material (including the surfacing). The shaded zones between each pair of lines show the effects of lean-concrete quality within the current specification. As may be expected, this effect decreases as the thickness of the DBM increases.

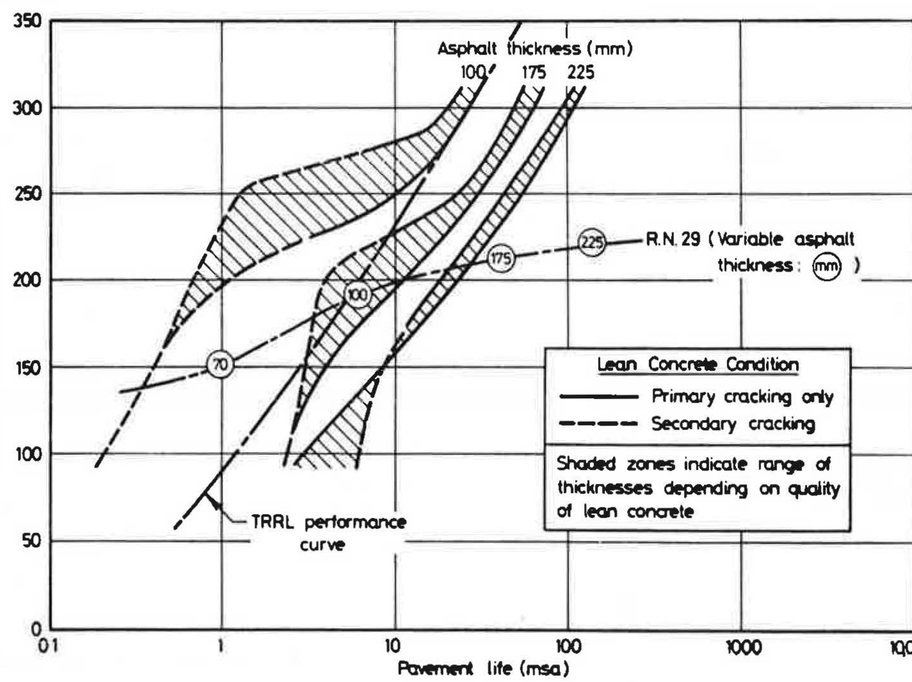
The recommended thicknesses from current British practice (13) are superimposed on these results for comparison. In Road Note 29, the thickness of bituminous cover varies continuously with traffic volume, and hence the thickness has been identified on the line in Figure 2 at various points to facilitate comparison with the analytic designs. This comparison is only possible at three discrete points and where the bituminous cover is of similar thickness. The analytic designs indicate between 60 and 100 mm more lean concrete for a given life.

The shape of the analytic curves in Figure 2 is of interest. The full lines indicate designs based on stage 1 only, which implies that traffic-associated cracking does not occur. The dotted lines are for designs that involve both stages. As DBM thickness increases, the lean-concrete stresses are reduced and less secondary cracking is apparent. The relation between the thickness of lean concrete and pavement life, when only primary cracking is involved, is a fairly steep one, best illustrated by the completely full line for the 225-mm bituminous thickness. The transition from primary to secondary cracking involves a shift to a lower life at the critical thickness, which explains the shape of the curves for the two smaller asphalt thicknesses. This shift is perhaps rather exaggerated, since the change in modulus from 28 000 or 34 000 MPa down to 500 MPa is applied suddenly and on the assumption that traffic-associated secondary cracking will occur rapidly once it begins. No reliable data on this transition are available, so the design calculations have erred on the safe side.

Table 3. Pavement lives at three DBM thicknesses.

Thickness of Lean Concrete (mm)	Pavement Life (msa) Zero DBM Thickness u_s = 10 MPa	Zero DBM Thickness u_s = 20 MPa	75-mm DBM Thickness u_s = 10 MPa	75-mm DBM Thickness u_s = 20 MPa	125-mm DBM Thickness u_s = 10 MPa	125-mm DBM Thickness u_s = 20 MPa
100	0.2	0.2	2.3	2.3	6.3	2.9
150	0.4	0.4	3.1	3.8	8.4	8.6
200	0.7	1.0	4.0	11.2	18.2	23.3
250	1.3	10.5	23.0	30.4	40.4	55.2
300	20.6	26.6	50.0	65.1	93.1	112.9

Figure 2. Life of pavements with lean-concrete bases.



The reason for the crossing of the curves in the case of the 225-mm asphalt thickness is interesting. When lean-concrete thickness is 100 mm, secondary cracking occurs in the weaker lean concrete but not in the stronger. During stage 2, for the weaker material, the additional life of the pavement based on fatigue resistance of the DBM exceeds that based on permanent deformation for the stronger material that has only primary cracks. Thus, when the lean-concrete layer is relatively thin, there is some advantage to be gained from keeping its strength down.

In order to relate the analytic designs reported here to the performance of actual pavements that incorporate lean-concrete bases, a curve is superimposed on Figure 2 to indicate the results of TRRL experimental sections at Alconbury Hill (14). The relevant sections had 100 mm of hot-rolled asphalt surfacing, and the curve is seen to converge on the analytic designs at the heavy-traffic end. The divergence at lower traffic volumes indicates that secondary cracking apparently had less effect on-site than the analytic procedure indicates. The sharp transition from the primary to the secondary cracking situation referred to above tends to exaggerate the discrepancy between the site curve and the design calculations. As may be expected, the site curve intersects the recommendations of Road Note 29 (13) at the appropriate point in Figure 2.

SANDWICH CONSTRUCTION

Design Considerations

The idea of using an unbound granular layer above a cement-treated layer is, in principle, attractive for several reasons:

1. The granular material can be compacted against a stiff supporting layer so that it develops a higher modulus.
2. Because it is placed above a stiffer layer, the stresses induced in the granular material are largely compressive.
3. There exists the potential, at least, for protecting the asphalt to some degree from reflection cracking induced by the cement-treated layer.

The basic approach to design that was used followed that described for conventional lean-concrete bases. There was a major difference in the pavement analysis, however, in respect to the granular layer. The stress-strain relations for unbound granular material are markedly nonlinear, but this fact only becomes important when the granular layer assumes a significant role in the structure. Hence, nonlinearity in the subbase was ignored in detail in the analysis of the structures discussed above. For sandwich construction, however, it was thought necessary to model the granular material in the composite base more accurately.

The nonlinearity of granular material can be expressed by an equation of the following form:

$$E = Kp^n \tag{6}$$

where p = mean normal stress = 1/3 (sum of any three orthogonal stresses at a point) and K and n are constants that depend on the particular material and its grading, density, and moisture content.

For the calculations reported here, the actual relation used was

$$E = 25\,p_m^{0.67}\ \text{MPa} \tag{7}$$

where p_m = mean value of p caused by a combination of overburden stress and traffic-induced stress, in kilopascals. The constants were taken from work carried out at Nottingham on a crushed-limestone base material (15). These design considerations are similar to those described by Otte and Monismith (16), although they differ in detail. Otte and Monismith were concerned primarily with the analysis of structures that have thin asphalt surfacings, and they only considered a single traffic volume. Their cement-treated material was of lower quality than the lean concrete considered here, and the supporting subbase and subgrade were stiffer.

Details of the Analysis

Characteristics of the structures analyzed are given in Table 4. The quality of the lean concrete is taken to be at the low bound of the specification (10 MPa).

The choice of a stiffness (elastic modulus) of 200 MPa for the granular sandwich layer followed a number of preliminary analyses in which various values for this parameter were used. In each case, the stresses were calculated at the top, middle, and bottom of the layer at three horizontal locations: midway between the dual wheels, below the center of one wheel, and below the edge of one wheel. By using these stresses, as well as appropriate values for overburden pressure, the nonlinear modulus equation (Equation 7) was used to check the value of modulus. The results indicated that the values derived from the calculated stresses varied very little with the parameters investigated, which were (a) the initial modulus of the layer between 100 and 1000 MPa, (b) the modulus of the lean-concrete layer below (values of 28 000 and 500 MPa were used to represent the primary and secondary cracking situations), and (c) the thickness of the granular layer between 100 and 500 mm.

When the value of 200 MPa was used for the granular layer, the derived values at the various locations investigated varied between 130 and 300 MPa. Average values for the two extreme thicknesses and the two conditions

Table 4. Details of sandwich structures.

Layer	Material	Design Stiffness (MPa)	Thickness (mm)
Wearing course	Rolled asphalt	6400	40
Base course	Rolled asphalt	7000	60
Composite road base	Unbound granular	200	100-500
	Lean concrete, uncracked	28 000	150
	Lean concrete, cracked	500	
Subbase	Granular	75	200
Subgrade	3 percent California bearing ratio	30	-

of the lean-concrete base are given below:

Layer Thickness (mm)	Derived Modulus of Elasticity (MPa) Primary Cracking	Secondary Cracking
100	280	184
500	192	161

These values were considered sufficiently close to 200 MPa for this measurement to be taken as a representative value. The finding is interesting, since it demonstrates that some of the potential advantages of the stiffer support are not realized. For instance, the well-graded crushed-limestone material used in laboratory tests developed a modulus of 1000 MPa under appropriate stress conditions (15). When this value was used in analysis, the largest derived modulus was only 340 MPa. It is apparent, therefore, that the stress conditions in a sandwich layer are such that granular materials do not develop very high modulus values.

During this analysis, consideration was given to the possibility of failure conditions developing in the granular layer, since this would cause a reduction in modulus. By using the modulus of 200 MPa, failure conditions were approached at some locations in the layer, which indicates that the actual effective modulus may be slightly lower than this. Failure is expressed in terms of the ratio q/p, where q = deviator stress and p = mean normal stress, both values being the maximum reached under the application of a wheel load. Otte and Monismith (16) simply ensured in their analyses that p did not become tensile. This is not considered a sufficiently rigorous check and will lead to higher effective moduli for the granular layer.

Design Calculations

Some details of the design calculations are given in Table 5 and in the two text tables below. During stage 1 (Table 5), the lean concrete cracked very early for all but the thickest granular layers. In stage 2, permanent deformation was the critical parameter below 300 mm, but asphalt fatigue became critical at 400 mm [critical life (N_2) = 4.7]:

Thickness of Granular Layer (mm)	Tensile Strain in Asphalt $\times 10^{-6}$	N_2 (msa)	Subgrade Strain $\times 10^{-6}$	N_2' (msa)
100	148	4.5	484	0.8
300	148	4.5	310	3.9
400	147	4.7	255	7.7

The final design lives indicate a rather small range between 0.8 and 5.5 msa for thicknesses of granular "sandwich" between 100 and 500 mm:

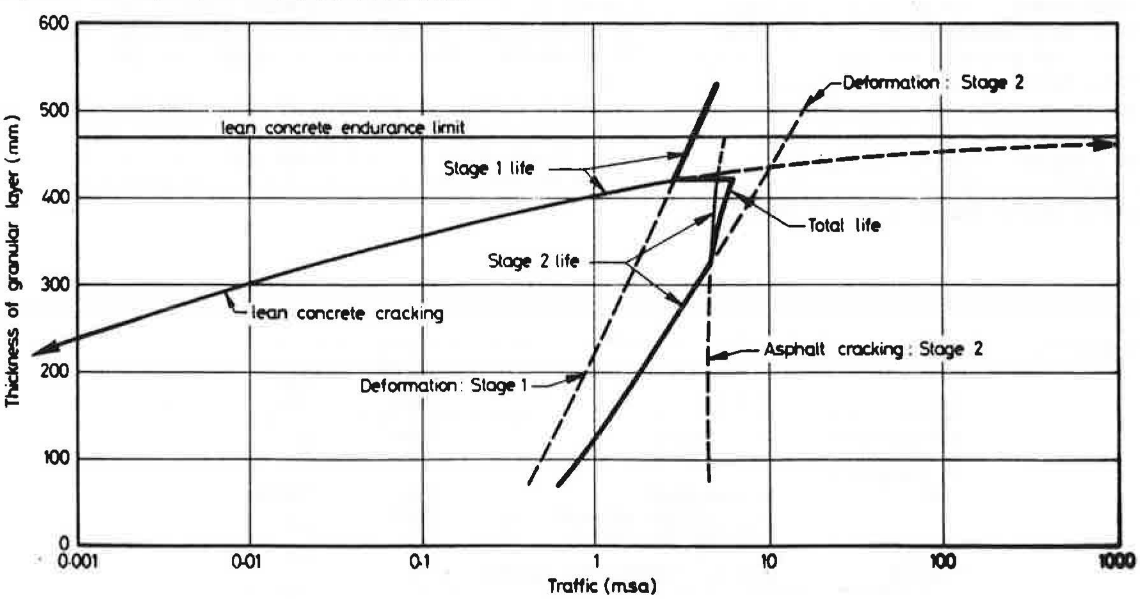
Thickness of Granular Layer (mm)	Life (msa) Stage 1	Stage 2	Total (stage 1 + stage 2)
100	0	0.8	0.8
300	0.01	3.9	3.9
400	0.8	4.7	5.5
500	4.3	–	4.3

Table 5. Details of typical design calculations for sandwich construction: stage 1.

Thickness of Granular Layer (mm)	Tensile Stress in Lean Concrete (σ_θ) (kPa)	1.25 σ_θ/f	N_1 (msa)	Tensile Strain in Asphalt $\times 10^{-6}$	N_1' (msa)	2.5 × Subgrade Strain $\times 10^{-6}$	N_1'' (msa)
100	1060	1.26	0*	102	27	405	0.5
300	700	0.83	0.01*	132	7.8	300	1.5
400	571	0.68	0.8*	137	6.5	260	2.6
500	480	0.57	∞	140	5.8	225	4.3*

*Critical life.

Figure 3. Life of pavement in sandwich construction.



The low modulus of the granular material means that it does not play a very significant structural role in the pavement.

It is interesting to note that the calculated life in the final design table above for the 500-mm thickness is less than that with 400 mm of granular material. This arises from the fact that between these two thicknesses there is a change from stage 1 + stage 2 behavior to stage 1 only for the 500-mm situation. This, together with other interactions of the design parameters, is shown in Figure 3. Here the effect of the thickness of the granular layer on the critical design parameters for both stage 1 and stage 2 is shown and the total life is indicated by the thickest line. The endurance limit for the lean concrete in stage 1 comes in when the granular layer is 470 mm thick. The effect of this is to reduce the pavement life, since the permanent deformation criterion under stage 1 leads to a shorter life than the combined stage 1 and stage 2 lives at slightly lower thicknesses.

The detail shown in Figure 3 should not be allowed to lead to a false sense of accuracy in these designs, although it is helpful in explaining the interactions in this particular example. The final life may be influenced by several factors. In particular, use of a DBM base course instead of rolled asphalt would move the line that represents stage asphalt cracking to a position around 1 msa.

The effect of the thickness of the granular layer on some of the design parameters is interesting. During stage 1, when the lean-concrete modulus is high, increasing the thickness of granular material actually causes an increase in tensile strain in the asphalt surfacing. This parameter was not critical and is therefore excluded from Figure 3, but the values are given in Table 5. This effect results from the low modulus of the granular layer so that when it is thick it allows greater flexing of the surface material. During stage 2, when the lean-concrete modulus is low, the thickness of the granular layer has a negligible effect on the tensile asphalt strain, as shown by the nearly vertical line for this parameter in Figure 3.

The sandwich construction can be compared with the conventional lean-concrete base. For a surface thickness of 100 mm and a lean-concrete base of 150 mm—the conditions used for the sandwich structures—a life of 0.4 msa was obtained. This corresponds to the point in Figure 3 where the line for total life would intersect the horizontal axis, i.e., zero granular-layer thickness. The maximum benefit to be obtained by adding the granular layer is to increase the life to 6 msa when the layer thickness is 420 mm. Figure 2 indicates that 240 mm of lean-concrete base would be required to produce this same life. It is possible to extend this and compare equivalent thicknesses of lean concrete, greater than 150 mm, with those of granular sandwich to produce the same life, as follows:

Pavement Life (msa)	Equivalent Thickness (mm)	
	Granular	Lean Concrete
0.4	0	0
0.8	100	40
1.8	200	65
3.7	300	75
5.6	400	85

For granular layers as thick as about 200 mm, the lean concrete has an equivalence, in terms of thickness, of approximately 3 to 1. Above this, the equivalence increases considerably.

CONCLUSIONS

1. The analytic approach can be used to design pavements with lean-concrete bases.
2. The analytic designs indicate the need for greater thicknesses in lean-concrete bases than those currently specified in British practice.
3. The quality of lean concrete has an increasingly important influence on design thickness as the thickness of bituminous cover is decreased.
4. The analyses indicate that in sandwich construction the modulus of the granular layer can reach only a modest value, a fact that lessens the potential advantages of this type of construction. In view of this, the thickness of granular material in sandwich construction does not have a major effect on pavement life.

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REFERENCES

1. P. S. Pell and S. F. Brown. The Characteristics of Materials for the Design of Flexible Pavement Structures. Proc., 3rd International Conference on the Structural Design of Asphalt Pavements, London, Vol. 1, 1972, pp. 326-342.
2. P. S. Pell and S. F. Brown. Authors' reply to discussion, Proc., 3rd International Conference on the Structural Design of Asphalt Pavements, London, Vol. 2, 1972, pp. 132-134.
3. R. N. Walker, W. D. O. Paterson, C. R. Freeme, and C. P. Marais. The South African Mechanistic Pavement Design Procedure. Presented at 4th International Conference on the Structural Design of Asphalt Pavements, Univ. of Michigan, Ann Arbor, 1977.
4. British Department of Transport. Specification for Road and Bridge Works. Her Majesty's Stationery Office, London, 1976.
5. R. I. T. Williams. Cement-Stabilised Materials. In Developments in Highway Pavement Engineering (P. S. Pell, ed.), Applied Science Publishers, Ltd., London, 1978.
6. S. F. Brown, P. S. Pell, and A. F. Stock. The Application of Simplified, Fundamental Design Procedures for Flexible Pavements. Proc., 4th International Conference on the Structural Design of Asphalt Pavements, Univ. of Michigan, Ann Arbor, Vol. 1, 1977, pp. 327-341.
7. N. W. Lister. Deflection Criteria for Flexible Pavements. Transport and Road Research Laboratory, Crowthorne, Berkshire, England, Rept. LR 375, 1972.
8. C. L. Saraf, W. S. Smith, and F. N. Finn. Rut Depth Prediction. TRB, Transportation Research Record 616, 1976, pp. 9-14.
9. R. I. T. Williams. Properties of Cement-Stabilized Materials. Journal of the Institute of Highway Engineers, Vol. 14, No. 2, 1972, pp. 5-19.
10. S. Kolias and R. I. T. Williams. Cement-Bound Road Materials: Strength and Elastic Properties Measured in the Laboratory. Transport and Road Research Laboratory, Crowthorne, Berkshire, England, Rept. SR 344, 1978.
11. K. D. Raithby. Some Flexural Fatigue Properties of Concrete: Effects of Age and Method of Curing.

Proc., 1st Australian Conference on Engineering Materials, Sydney, 1974.
12. J. K. Mitchell and C. L. Monismith. A Thickness Design Procedure for Pavements with Cement-Stabilised Bases and Thin Asphalt Surfacings. Proc., 4th International Conference on the Structural Design of Asphalt Pavements, Univ. of Michigan, Ann Arbor, Vol. 1, 1977, pp. 409-416.
13. British Road Research Laboratory. A Guide to the Structural Design of Pavements for New Roads, 3rd Ed. Her Majesty's Stationery Office, London, Road Note 29, 1970.
14. P. D. Thompson, D. Croney, and E. W. H. Currer. The Alconbury Hill Experiment and Its Relation to Flexible Pavement Design. Proc., 3rd International Conference on the Structural Design of Asphalt Pavements, London, 1972, pp. 920-937.
15. J. R. Boyce, S. F. Brown, and P. S. Pell. The Resilient Behaviour of a Granular Material Under Repeated Loading. Proc., Australian Road Research Board, Vol. 8, 1976, pp. 8-19.
16. E. Otte and C. L. Monismith. Some Aspects of Upside-Down Pavement Design. Proc., Australian Road Research Board, Vol. 8, 1976.

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Evaluation of Structural Coefficients of Stabilized Base-Course Materials

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The structural coefficients of two stabilized base-course materials—bituminous concrete and aggregate cement—are evaluated by using two different methods of analysis: the American Association of State Highway and Transportation Officials (AASHTO) performance analysis and the limiting-criteria approach. The AASHTO performance analysis is based on the field performance of 11 bituminous concrete pavements and three aggregate cement pavements; the limiting-criteria approach is based on maximum tensile strain at the bottom of the base course, maximum compressive strain at the top of the subgrade, and maximum pavement surface deflection. The test pavements were constructed at the Pennsylvania Transportation Research Facility. The field performance data collected were rutting, cracking, and present serviceability index. Limiting criteria were developed by using the BISAR computer program and the rutting and cracking data for the test pavements. Results of the evaluation show good agreement between the two methods of analysis. The structural coefficients of the base-course materials were found to vary with many factors, such as the thickness and stiffness of each pavement constituent layer, structural coefficients of other pavement layers, and pavement life. It is concluded that it is very difficult to assign a constant value to the structural coefficient of a base-course material.

The design procedure of the American Association of State Highway and Transportation Officials (AASHTO) serves as the basis for the design of flexible pavements for many highway agencies. One of the requirements of the design procedure is that structural coefficients be assigned to all materials used above the subgrade. Structural coefficients of some pavement materials were determined at the AASHO Road Test; it was recommended, however, that the coefficients be refined to reflect local material properties and climatic conditions. The refinement of the structural coefficients for materials used in Pennsylvania was one of the principal objectives of research at the Pennsylvania Transportation Research Facility at Pennsylvania State University. The specific objective of the study was to determine structural coefficients for the various stabilized base-course materials used in Pennsylvania.

Two different approaches were taken in the evaluation of the structural coefficients of two stabilized base-course materials—namely, bituminous concrete and limestone aggregate cement. The first analysis was based on the use of performance data with analysis techniques similar to those used during the AASHO Road Test. The second approach was based on limiting criteria so that pavement deflection, tensile strain at the bottom of the stabilized base, and compressive strain at the top of the subgrade could be limited within permissible values. This paper presents the results of the analysis.

RESEARCH FACILITY AND FIELD TESTING

The Pennsylvania Transportation Research Facility was constructed in the summer of 1972. The original facility was a 1.6-km (1-mile), one-lane test road composed of sections with different base-course materials and different layer thicknesses. In the fall of 1975, four sections were replaced by eight shorter sections. The plan view and longitudinal profile of the facility are shown in Figure 1. More detailed information on the design, construction, and traffic operations of the facility are available elsewhere (1, 2).

The base-course materials studied were bituminous concrete, aggregate cement, aggregate-lime-pozzolan, and aggregate-bituminous. Three types of aggregate were used in the aggregate-cement base: limestone, slag, and gravel. Among these base-course materials, there was only one base thickness for the aggregate-bituminous material and for the slag and gravel aggregate-cement material. Although three different base thicknesses were available for the aggregate-lime-pozzolan, the pavements that had 10.1- and 15.2-cm (4- and 6-in) thick base—i.e., sections F and G—did not cure properly because of cold weather during construction. Thus, only bituminous concrete and limestone aggregate cement had three levels of base-course thickness. Since the calculation of structural coefficients using the AASHTO performance approach requires examining the change in the indicators of pavement performance across levels of layer thicknesses, only