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Nonlinear Characterization of Granular Materials for Asphalt Pavement Design

A. F. STOCK AND S. F. BROWN

In view of the well-established nonlinear resilient properties of unbound granular materials, analytically based pavement-design procedures should take proper account of this characteristic. The importance of including a failure criterion in the nonlinear model is demonstrated; the potentially high modulus of granular materials is not being realized in situ because of the unfavorable stress conditions that develop. The nonlinear model that has been written into the pavement-design computer program ADEM is described, and typical results are presented and compared with those based entirely on linear-elastic analysis. The results show that the thickness and quality of granular material have only a minor influence on the required thickness of asphalt but that, for accurate design, nonlinear analysis is required.

The development of a computer program for the design of asphalt pavements by using analytical techniques has been reported (1). This program, ADEM, assumed all layers to be

linear elastic and estimated the modulus of the granular layer by assuming that the modular ratio between the granular material and the subgrade was 2.5. However, extensive evidence was available from materials testing that showed granular materials to be markedly nonlinear, and a nonlinear model was incorporated into the further development of ADEM. This allowed for a more-accurate representation of the granular layer so that its structural role could be better assessed under various conditions and also provided results that could be compared with those obtained by using simple linear elasticity.

The general conclusion that can be drawn from the literature on laboratory studies of unbound granular materials is that under load they exhibit stress-stiffening behavior. This conclusion has been reinforced by measurements in test pavements.

The most common form of relationship reported is

$$E = K_1\lambda^{K_2} \tag{1}$$

where

E = resilient modulus,
λ = stress function, and
K_1, K_2 = material constants.

Various stress functions have been used, including the normal stress ($\theta = \sigma_1 + \sigma_2 + \sigma_3$) as proposed by Hicks (2) and the mean value of the mean normal stress for any stress path as proposed by Boyce, Brown, and Pell (3) (mean normal stress $p = \theta/3$). Since Hicks (2) studied a variety of materials and his stress function is a simple one, the modulus relationship that he proposed was adopted for this work:

$$E = K_1\theta^{K_2} \tag{2}$$

Smith (4) used this model to derive equivalent moduli for the granular layer as functions of the asphalt thickness and stiffness, the subgrade modulus, and the constant K_1. These equivalent moduli give the same tensile strain in the asphalt and vertical strain on the subgrade as would be obtained in a nonlinear analysis. While this simplification is very useful, it was not adopted for this work since no failure criterion for the granular material was included in its derivation. Furthermore, use of such simplifications is only valid within the limits of their derivation and, since the ADEM design program is highly versatile, the restrictions imposed by Smith's simplification were considered unacceptable.

DEVELOPMENT OF THE NONLINEAR MODEL

The philosophy of the approach was similar to that originally proposed by Monismith and others (5), which involved used of linear-elastic-layer theory in an iterative procedure designed to obtain compatibility between the calculated stresses and the associated values of modulus for the nonlinear granular layer.

Initial attempts to produce a convergent iteration procedure by means of weightless layers failed; it was thus necessary to investigate methods for determining self-weight stresses.

Preliminary applications of the modulus iteration procedure indicated that unacceptably large tensile stresses were experienced within the granular layer, and further investigation was therefore undertaken with respect to development and incorporation of a failure criterion.

Determination of Self-Weight Stresses

Vertical stresses can be dealt with quite simply by considering the thickness and density of the layer. However, horizontal stresses are less straightforward; the usual method for their calculation is to multiply the vertical stresses by the coefficient of earth pressure at rest (K_0), which, for a linear-elastic system, is given by

$$K_0 = \nu/(1 - \nu) \tag{3}$$

where ν = vertical stress. However, it was felt that this approach was incompatible with a nonlinear system. Brooker and Ireland (6) relate K_0 to the results of materials testing by the equation

$$K_0 = 1 - \sin\phi' \tag{4}$$

where ϕ' = angle of shearing resistance for cohesionless soil. For overconsolidated clays, values greater than unity occur, and it was considered that the effect of compaction equipment on the granular layer produced the same type of effect. Hence, the procedure for estimating K_0 from the overconsolidation ratio (7) was adopted.

To implement this approach, the stress history of granular layers during compaction was estimated. From these estimates and the calculation of vertical self-weight stresses, a relationship between the overconsolidation ratio and depth was developed. This relationship in conjunction with that between K_0 and the overconsolidation ratio (7) provided the data from which K_0 could be expressed as a function of depth for various thicknesses of granular layer. For the purpose of the material model, the granular layer was divided into four sublayers. Average K_0 values for each of these sublayers in a granular layer, which varied in thickness between 100 mm and 700 mm, were estimated from the K_0-depth relationship and are shown in Figure 1.

Failure Criterion

Preliminary analyses with the iterative procedure indicated that unacceptably large tensile stresses could develop in the granular material. Barker of the U.S. Army Engineer Waterways Experiment Station, in attempting finite-element analyses of pavement systems, had observed a similar phenomenon and overcame it by including a failure criterion in the form of a limiting σ_1/σ_3 ratio (where σ_1 and σ_3 are the major and minor principal stresses, respectively). Boyce (8) undertook tests to failure by following several stress paths and noted that the σ_1/σ_3 ratio at which his specimens failed was a function of the stress path. However, when his results were expressed as a ratio of the stress invariants q/p, a value close to 2.2 applied for all cases. In general,

$$q = (1/\sqrt{2})[(\sigma_1 - \sigma_2)^2 + (\sigma_2 - \sigma_3)^2 + (\sigma_3 - \sigma_1)^2]^{1/2} \tag{5}$$

Results reported by Maree (9), when similarly expressed, produce a failure ratio of $q/p = 2.3$, which, in view of the fact that the two studies were totally independent and carried out on materials from widely different sources, adds considerably to confidence in applying this failure criterion.

For the purpose of pavement analysis, if the limiting q/p ratio of 2.2 was exceeded in the granular layer, an arbitrarily low value of modulus (3 MPa) was inserted into the calculation procedure. Since it was not considered to be physically possible for the stiffness of the granular material to reduce abruptly when failure occurs, the failure criterion included a modulus reduction routine as failure was approached. A lower limit of stress ratio $q/p = 1$ was set and, between this value and $q/p = 2.2$, the modulus was obtained by interpolating between the value appropriate to the stress conditions given by Equation 2 and the failure modulus, according to the distance across the zone bounded by $q/p = 1$ and $q/p = 2.2$. Figure 2 is a general plot in p-q stress space that shows the three zones in which the model operates and the appropriate formulas for modulus derivation. The lower limit of $q/p = 1$ for applicability of Equation 1 was selected after consideration of the experimental evidence, which suggests that it is only appropriate for stresses well below failure (2,3).

Figure 1. K_0 values as a function of granular layer thickness.

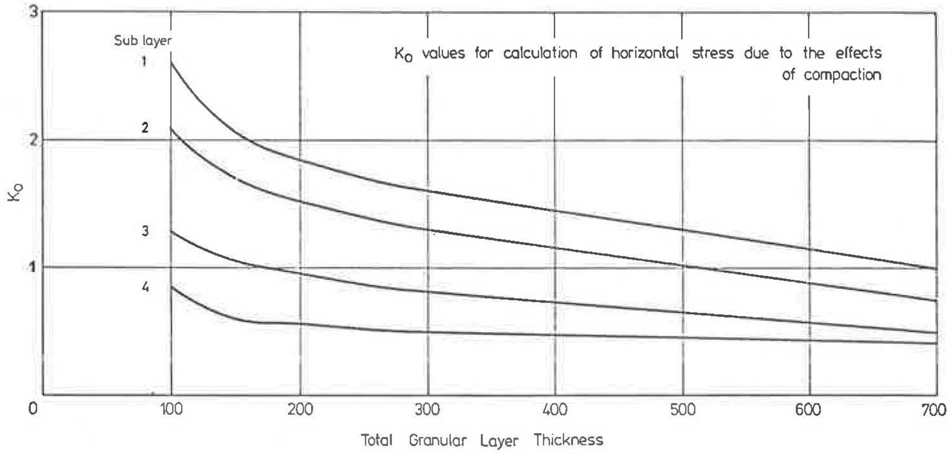


Figure 2. Derivation of granular material modulus for the three stress zones.

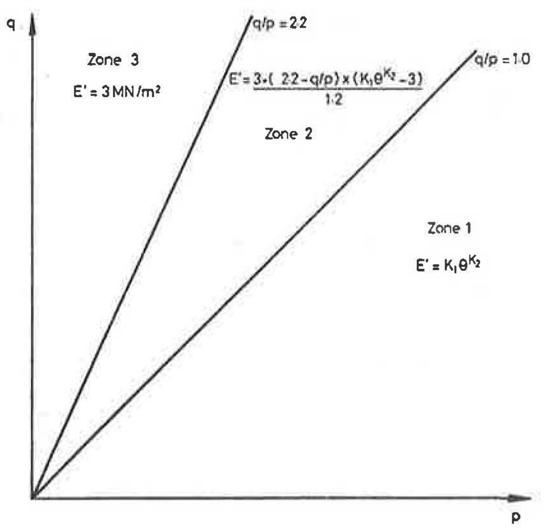
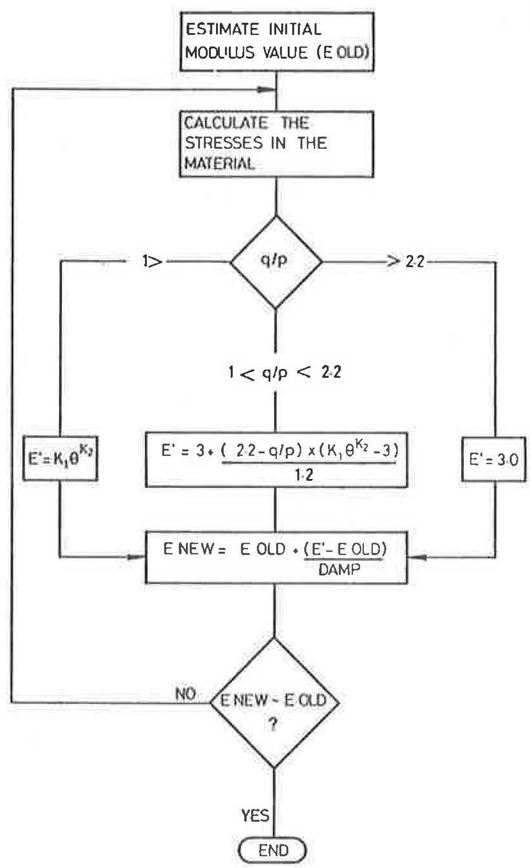


Figure 3. Flowchart for the modulus iteration procedure.



The Nonlinear Model

The nonlinear model finally incorporated into the ADEM pavement-design program was developed around the BISTRO (10) program produced by Shell. In this system, the full vehicle load is applied to the structure through dual wheels. The granular layer is divided into four sublayers, and an iteration procedure that starts with assumed moduli for each sublayer is followed until two consecutive modulus iterations agree to within 5 percent or 3 MPa, whichever is greater, for all four layers.

Stresses for the iterative procedure are calculated on the axis of symmetry of the dual-wheel-load configuration at the center of each sublayer. No attempt is made to consider horizontal variation in the modulus. To speed convergence, the iteration procedure is damped, as indicated in the flowchart of Figure 3; the damping factor ranges from 2 to 5.

EFFECT OF FAILURE CRITERIA

Stress Distribution

Figures 4 and 5 are p–q plots of the stress distribution within the granular layer for structures with 50 mm and 200 mm of asphalt surfacing that has a stiffness of 7000 MPa. Three thicknesses of granular layer (h_2) have been considered--200, 450, and 700 mm. As would be expected, the most highly stressed granular layer is in the structure that has only 50 mm of asphalt surfacing. It can be seen clearly that, in this structure (Figure 4), q is high and p is low; this combination gives a failure condition throughout the layer. Within this structure the failure criterion has the effect of both reducing q and increasing p to bring the layer within zone 2 of the model (Figure 2), although it remains close to failure.

Figure 5 shows that with a 200-mm asphalt surfacing the stress in the granular layer is much reduced. With the exception of the bottom sublayer in the 200-mm granular layer, the q/p stress ratio does not exceed 2.2. However, in this structure the failure criterion still has a significant

Figure 4. Stress distribution within the granular layer for a structure with 50 mm of asphalt surfacing.

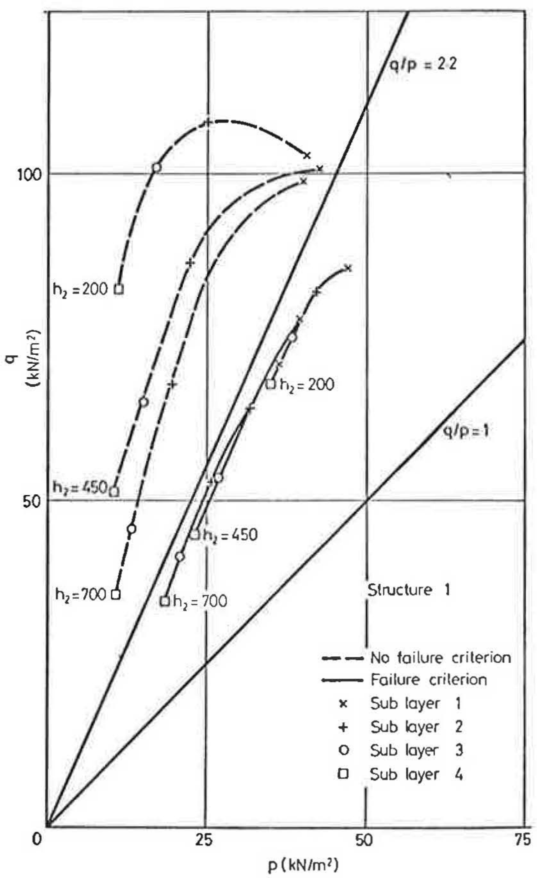
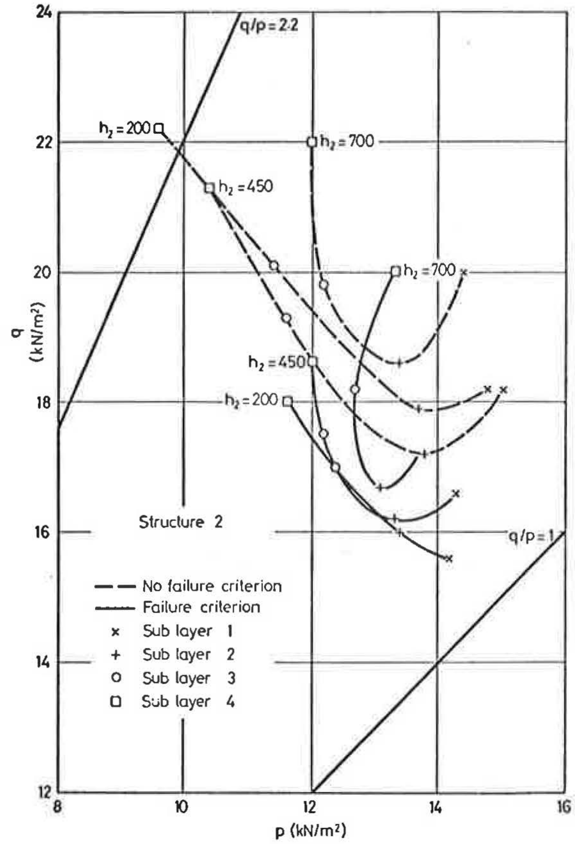


Figure 5. Stress distribution within the granular layer for a structure with 200 mm of asphalt surfacing.



effect on the stress distribution, mainly by reducing q.

It should be noted that the stress distributions in the granular layer for the two surfacing thicknesses are quite different. For 50-mm surfacing both p and q decrease from the top sublayer (sublayer 1) to the bottom (Figure 4). However, when the layer is surfaced with 200 mm of asphalt (Figure 5), the variation in stress with depth is less consistent, although generally p decreases and q increases. This results in a stress distribution that becomes more severe with depth, the q/p ratio generally increasing with depth. Thus, although under 50-mm surfacing the entire granular layer is near failure because it lies along a line of approximately constant q/p, for 200-mm surfacing the stress distribution (q/p ratio) becomes more severe with depth.

Figures 6 and 7 show the derived moduli for the four sublayers for each structure. The effect of the failure criterion is particularly significant for pavements with only 50 mm of surfacing (Figure 6), in which the derived moduli for nearly all sublayers is less than the subgrade modulus and in all cases less than half the modulus derived without regard to failure. For the structures surfaced with 200 mm of asphalt (Figure 7), the effect of the failure criterion is much reduced, as would be expected from the stress distribution (Figure 5), but the use of the failure criterion still indicates a reduction in the derived modulus.

Primary Response Parameters

The effect of the failure criterion on primary response parameters is shown in Figures 8 and 9 as a function of granular layer thickness for the two asphalt-surfacing thicknesses. Structure 1 is surfaced with 50 mm of asphalt, and structure 2 is surfaced with 200 mm. The two parameters chosen—vertical strain on the subgrade and tensile strain at the bottom of the asphalt layer—are commonly used as design criteria in flexible-pavement design and are included as such in ADEM.

It is clear that for a thin asphalt surfacing there is a significant difference between the results obtained by the linear and nonlinear systems; the nonlinear analysis, as would be expected from the lower derived moduli, always gives the greater strain. It is also important to note that, although asphalt strain decreases as the granular layer thickness increases if potential failure is ignored, it increases as the granular layer thickness increases when failure is considered.

It was therefore concluded that it is essential to consider material failure in a nonlinear model for granular layers. Accordingly, the failure criterion was included in the model for granular layers incorporated in the ADEM design program used to produce the results discussed in the following section.

EFFECT OF NONLINEAR MODEL ON DESIGN THICKNESS

Having developed a system for nonlinear characterization of granular layers, we investigated the effect of using this model on the design thickness of an asphalt pavement. To do this, the nonlinear model was incorporated as an option in the ADEM design program, so that pavements could be designed on the basis of linear or nonlinear characterization of the granular layer. For the linear designs a modular ratio of 2.5 was used between the granular layer and subgrade.

It is not the purpose of this paper to discuss ADEM, but a brief description of the program is given since it was used to derive these results. ADEM produces the required thickness of the asphalt layer for a given design life by analyzing a trial structure and adjusting the asphalt-layer thickness to ensure that the design criteria are satisfied. Two performance parameters are used: (a) tensile strain in the

Figure 6. Derived moduli for the granular sublayers for a structure with 50 mm of asphalt surfacing.

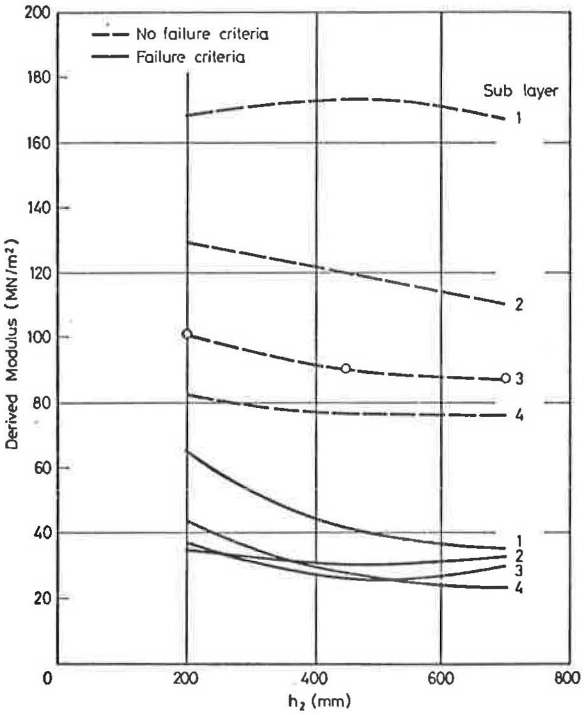
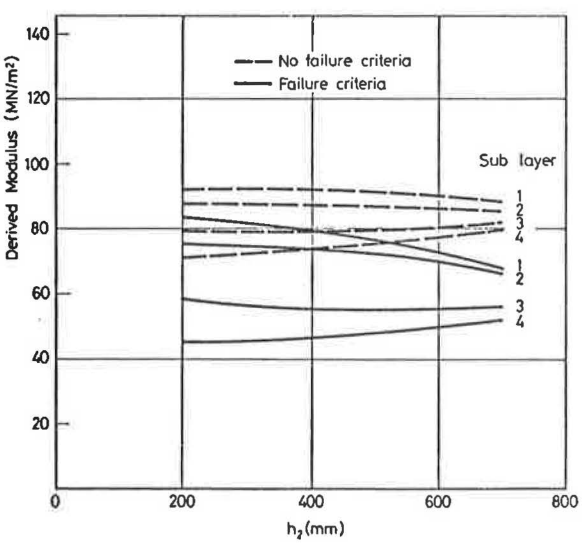


Figure 7. Derived moduli for the granular sublayers for a structure with 200 mm of asphalt surfacing.



asphalt layer to design against asphalt fatigue failure and (b) vertical strain on the subgrade to design against overall deformation failure. Limiting values of these strains are obtained within the program from knowledge of the strain-life relationships and are used as criteria for the design process. Designs have been produced for assessing the effect of nonlinear modeling of the granular layer for a wide range of parameters. The variables studied were subgrade modulus (E_S), granular material quality characterized by variation in K_1 (Equation 1), granular layer thickness (h_2), and bitumen content and void content of the asphalt mix. Design life, traffic speed, bitumen grade, and temperature have been held constant throughout

Figure 8. Effect of the failure criterion on asphalt strain.

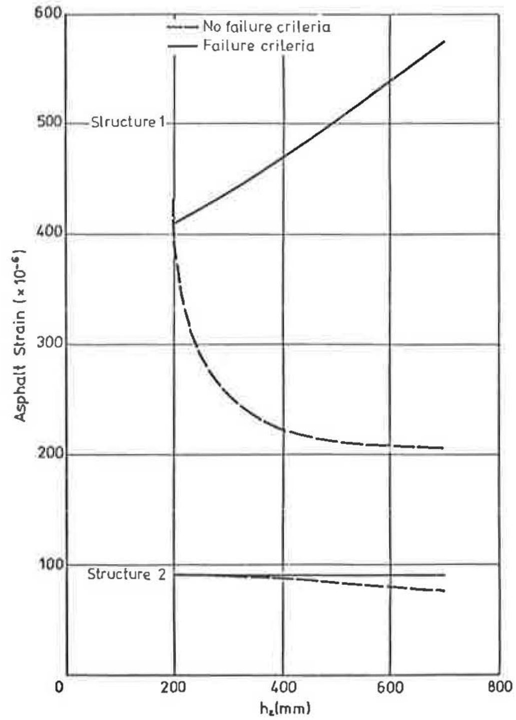
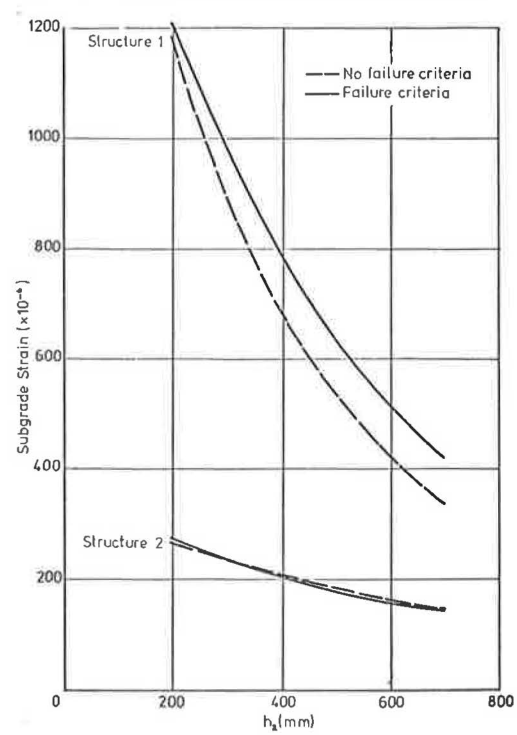


Figure 9. Effect of the failure criterion on subgrade strain.



the study at 50 million standard axles, 80 km/h, penetration value of 100, and 15°C, respectively.

Figure 10 shows design thicknesses as a function of granular layer thickness for three values of subgrade modulus and granular material quality (K_1). The effect of variation in K_1 is very small, so a constant value of 400 was used for subsequent studies. For the lowest subgrade modulus (E_S = 20 MPa), linear and nonlinear characterization

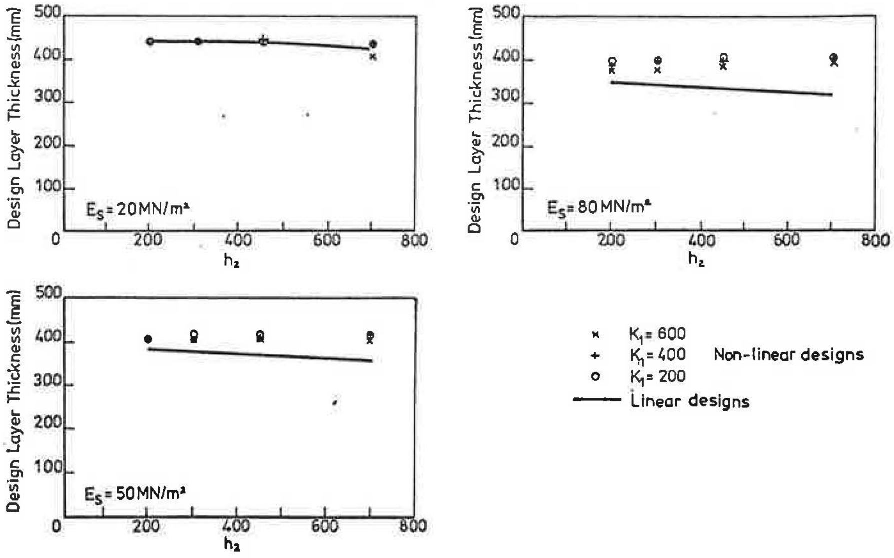


Figure 10. Design thicknesses as a function of granular layer thickness.

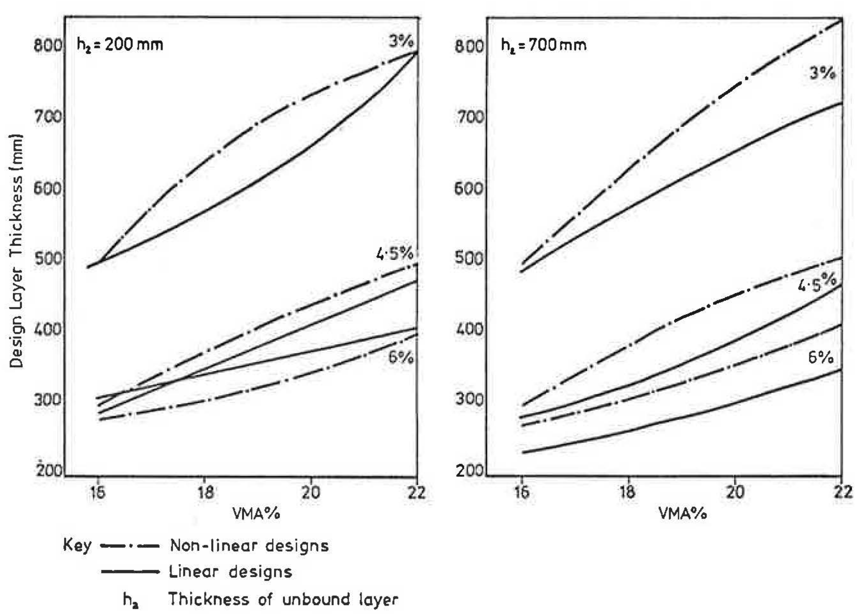


Figure 11. Design thicknesses as a function of VMA and binder content.

produced identical design thicknesses. As the subgrade modulus increases, the two methods of treating the granular layer start to give different answers; the thickness requirement by using the nonlinear system is greater than that for the linear system. The differences between the two systems are greatest on the stiffest subgrade (E_S = 80 MPa) and for thick granular layers (h_2 = 700 mm), but the differences are still not large. It is interesting to note that the granular layer thickness does not greatly affect the asphalt thickness, but designs produced with the nonlinear option are less sensitive to variation in granular layer thickness than those produced by means of the simple linear option.

The parameter termed "voids in mixed aggregates" (VMA) has been introduced to study the effect of mix variables since it is a useful parameter for describing a bituminous mix, particularly with regard to its state of compaction (11). Recent research has indicated that for U.K. mixes and conditions, VMA values of 16, 19, and 22 percent may be taken as approximately representative of good, average, and poor compaction, respectively. In order to examine a range of mixes, bitumen contents of 3, 4.5, and 6 percent by mass were assumed; the calculated void contents for the various combinations of VMA and bitumen content are given below:

	Binder Contents (%)		
VMA (%)	3	4.5	6
16	9.1	5.5	1.8
19	12.4	10.0	5.3
22	15.6	12.3	8.8

Figure 11 shows design thicknesses as a function of VMA and binder content; designs produced by both linear and nonlinear options are compared for two thicknesses of granular layer and a subgrade modulus of 50 MPa.

In all but one case, the design thicknesses of asphalt based on nonlinear analysis are greater than those based on linear analysis. It is therefore evident that significant differences in design-thickness requirements can be obtained if the granular layer is characterized as nonlinear rather than linear elastic.

It was concluded from this design study that linear-elastic characterization of granular layers may be

adequate for comparative design studies but, if a pavement design is required for a highway project, nonlinear characterization of the granular layer is highly desirable. The findings generally confirm those obtained by Dehlen and Monismith (12).

It is not the purpose here to discuss the implications for asphalt-mix design of the results plotted in Figure 11, but we feel that an explanation for the decrease in design thickness with increase in bitumen content is required. Increasing the bitumen content in a mix decreases the stiffness of that mix and increases its fatigue resistance. Reduced mix stiffness means that the tensile strain developed in the bottom of the asphalt will increase, and an increased design thickness may therefore be expected in order to compensate for this. However, for the case illustrated in Figure 11, the improvement in fatigue performance and thus the increased allowable strain is much more significant than is the stiffness loss; hence a reduced layer thickness results. It must be emphasized that reduction in thickness with increased binder content will only be achieved for structures in which fatigue is the critical parameter and will not necessarily be achieved for all these structures.

SUMMARY AND CONCLUSIONS

1. It is essential to include a failure criterion in a system for nonlinear analysis of granular materials.
2. In conventional pavement structures the potentially high modulus of a granular material generally cannot be realized because, when it is highly stressed, the material approaches a failure stress state with a consequent reduction in modulus.
3. Nonlinear characterization of unbound granular layers is required for accurate designs, particularly when thin asphalt surfacings are contemplated.
4. In pavement structures that have asphalt layers of about 300 mm, the required thickness of the asphalt layer is not greatly affected by the thickness of the granular layer.

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Analysis of In Situ Granular-Layer Modulus from Dynamic Road-Rater Deflections

PAUL A. D'AMATO AND MATTHEW W. WITCZAK

The major objective of this study was to investigate the ability of elastic-layer theory coupled with nonlinear dynamic modulus tests to predict pavement deflections in a way comparable to dynamic road-rater deflection measurements on three highway sections in Maryland. It was found that theoretically predicted deflections were two to four times the measured road-rater deflections for all three pavement sections studied at all four road-rater sensor locations and for all times throughout the year in which road-rater deflections were made. To obtain equality between predicted and measured deflections, the granular-layer modulus was adjusted by using a K_1-factor. A linear log-log relationship was evident when the K_1-adjustment factor was plotted versus measured surface deflections. It was concluded that the current laboratory method of modulus characterization underestimates the modulus of a granu-