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SOME FINDINGS CONCERNING THE DETERMINATION OF GRANULAR-BASE MODULI FOR FLEXIBLE-PAVEMENT THICKNESS DESIGN

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Abstract

Modern flexible pavement design requires that the layer moduli, Poisson ratio, and thicknesses be known. While the moduli of subgrade and paving materials can be obtained from laboratory tests, unbound granular materials have been found to exhibit moduli that are nonlinear or stress dependent. As a result, various agencies and researchers have developed techniques to incorporate some aspects of this nonlinearity directly into elastic layered solutions. At present, the FAA's recent FAARFIELD software incorporates the 1977 USCOE method for computing the modulus of non-stabilized layers. Furthermore, the current Israeli Flex-Design software incorporates the earlier, 1975 USCOE method for computing the same modulus. In addition to these methods, the newly developed software of the Mechanistic-Empirical Pavement Design Guidelines (MEPDG) incorporates a non-linear constitutive model for granular materials along somewhat the same lines as the 1983 Asphalt Institute's DAMA software, from which the granular modulus calculated is found, in contrast to the above methods, (a) to decrease slightly with the increase in granular layer thickness; (b) to increase with the increase in subgrade modulus at a smaller rate than the linear. The same phenomenon was described in two recent Indian doctoral dissertations. This paper presents the effect of the latter phenomenon on the outputs of the Israeli Flex-Design software by analyzing several measured FWD surface-deflection basins.

Keywords: Back-calculation, design software, Falling Weight Deflectometer (FWD), flexible pavement, granular layer modulus, subgrade modulus.

1 Introduction

The mechanistic flexible-pavement design requires that the layer moduli, Poisson ratio, and thicknesses be known in order to compute the stress, strain, and displacement states of the pavement. Although subgrade and paving materials moduli can be obtained from laboratory tests, unbound granular materials have been found to exhibit moduli that are nonlinear or stress dependent. As a result, various agencies and researchers have developed techniques to incorporate some aspects of this nonlinearity directly into elastic layered solutions. In general, these procedures can be grouped into empirical relationships, iterative layered approaches, and finite elements solutions.

The various existing procedures yield a wide range of results, but they affect mainly the fatigue behavior of the asphalt layers. Thus, it is only logical to evaluate these procedures in the light of in-situ values in order to suggest a preferable procedure. With this background, the objectives of the present paper were formulated as follows:

- to summarize and compare the existing approaches for calculating the modular ratio (the ratio of granular layer modulus to subgrade modulus) in a given pavement structure, including the new Mechanistic-Empirical Pavement Design Guidelines (MEPDG);
- to evaluate and compare the in-situ modular ratio of existing pavements with the aid of in-situ Falling Weight Deflectometer (FWD) measurements;
- to present relevant recommendations for the calculation of modular ratio in Israeli and other design processes of new pavements.

The sections to follow will detail the process of attaining this paper's three objectives and their associated conclusions. The literature survey has accordingly been divided into three sections, each dealing with a specific subject.

2 Review of the simplified models

There are several approaches available for estimating the modulus of an unbound granular material in a flexible highway or airfield pavement. In this paper, they are grouped into (a) the simplified models group, which is utilized in several pavement-design procedures, including, as will be shown later in this section, the recent FAARFIELD program; (b) the layered developed models group, which is also utilized in other pavement-design procedures, including the recent Mechanistic-Empirical (M-E) design of flexible pavements (MEPDG), as will be detailed further on. This section describes the first group of models, and the next two sections describe the second group of models.

The first simplified model for predicting the modulus of an unbound granular course was published by the Shell Oil Company in 1965 [1]. Nowadays, it is also used for forwardcalculation of granular-layer moduli from FWD deflection measurements [2, 3, 4]. This model is defined by the following expression:

$$E_G = E_S \times 0.2 \times H_G^{0.45} \quad \text{and} \quad 2 < \frac{E_G}{E_S} < 4 \tag{1}$$

where E_G denotes the equivalent modulus of the granular layer, in MPa; E_S, the modulus of the subgrade, in MPa; H_G, the thickness of the granular layer, in mm.

In addition to this simplified model, the U.S. Army Corps of Engineers utilized two general estimation techniques to predict the equivalent modulus of a granular layer. The first of these is based on the following two expressions, published in 1975 [5]:

$$E_{GS} = E_S \times (1 + 0.0030) \times H_{GS} \tag{2a}$$

$$E_{GB} = E_{GS} \times (1 + 0.0067) \times H_{GB} \tag{2b}$$

where E_{GS} denotes the equivalent modulus of the granular subbase-course layer, in MPa; E_S, the modulus of the subgrade, in MPa; H_{GS}, the thickness of the granular subbase-course layer, in mm; E_{GB}, the equivalent modulus of the granular base-course layer, in MPa; H_{GB}, the thickness of the granular base-course layer, in mm.

Here it should be noted that the above two expressions have been incorporated into the Israeli extended CBR pavement-design procedure [6] with the aid of Flex-Design software. This procedure postulates the following upper bounds for the unbound granular moduli: 300 MPa for subbase materials and 700 MPa for base materials.

The second U.S. Army Corps of Engineers procedure is based on the following two expressions, published in 1977 [7]:

$$E_n = E_{n+1} \times \left[1 + 7.18 \times \log\left(\frac{h}{25.4}\right) - 1.56 \times \log\left(\frac{E_{n+1}}{0.006894}\right) \times \log\left(\frac{h}{25.4}\right) \right. \tag{3a}$$

$$E_n = E_{n+1} \times \left[1 + 10.52 \times \log\left(\frac{h}{25.4}\right) - 2.10 \times \log\left(\frac{E_{n+1}}{0.006894}\right) \times \log\left(\frac{h}{25.4}\right)\right] \tag{3b}$$

where n denotes a granular layer in the pavement system; E_n in Eq. (3a), the subbase-course resilient modulus of layer n, in MPa; E_n in Eq. (3b), the base-course resilient modulus of layer n, in MPa; $E_{n+1,}$ the resilient modulus of the layer beneath layer n, in MPa; h in Eq. (3a), the thickness of layer n for subbase-course materials, in mm; and h in Eq. (3b), the thickness of layer n for base-course materials, in mm.

Eqs (3) are used in the following way: (a) for subbase courses having a design thickness (termed the limiting thickness) of 200 millimeters or less, the subbase-course is treated as a single layer (i.e., n=1), whereas for a design subbase in excess of 200 millimeters, the subbase-course is divided into sub-layers of approximately equal thickness and the modulus of each sub-layer is determined individually; (b) for base courses, the base-course is treated as above but with a limiting thickness of 250mm instead of 200mm.

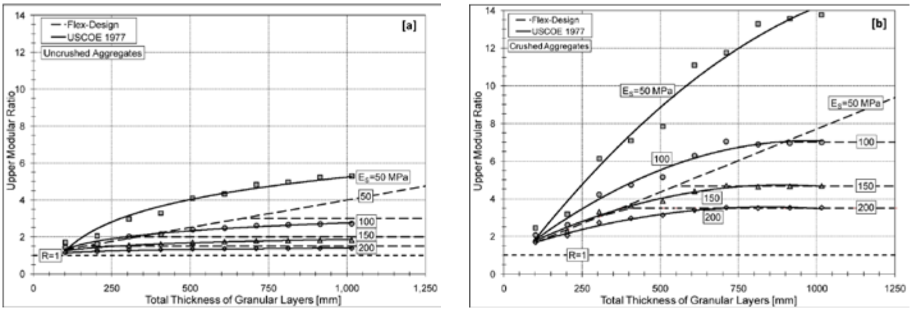

Figure 1 Comparison of the outputs of Eqs. (2) (Flex-Design) with the outputs of Eqs. (3) (USCOE 1977) for (a) subbase materials and (b) base materials.

The comparison of the outputs of Eqs. (2) (Flex-Design) with the outputs of Eqs. (3) (USCOE 1977) for both subbase and base materials is given in Fig. 1. This figure indicates that the modulus of the granular subbase and the base materials obtained by the USCOE 1977 procedure is considerably higher than that obtained by the Flex-Design procedure for low subgrade modulus and almost identical for a moderate or higher subgrade modulus. Thus, for practical reasons, the shift in the use of Eqs. (2) in the Flex-Design procedure toward Eqs. (3) seems unnecessary.

Here it might be noted that the above two expressions have recently been incorporated into the FAA software [8] developed for airport-pavement design and called FAARFIELD (FAA Rigid and Flexible Iterative Elastic Layer Design).

Another existing procedure used to predict the equivalent modulus of a granular layer is given by AUSTROADS [9], which recommended in 1992 sub-layering the granular material placed directly on the subgrade. In this procedure, it is constrained that (a) the sub-layer thickness lies approximately in the range of 50-150mm and (b) the ratio of moduli of adjacent sub-layers does not exceed 2. On the basis of (a) the findings described in [10] and (b) the sub-layering of the granular material into five layers of equal thickness, the following expression was suggested in 2007 [11]:

$$E_n = E_{n+1} \times 2^{[(d_{n+1})/125]} \tag{4}$$

where n denotes a granular layer in the pavement system; E_n, the resilient modulus of granular layer n, in MPa, E_{n+1}, the resilient modulus of the granular layer beneath layer n, in MPa; and

d_{n+1}, the distance between the mid-heights of sub-layer n and n+1, in mm. Note: The mid-height of a subgrade is taken to be the top of the subgrade.

The above expression (Eq. 4) applies to both base and subbase materials. It is very similar to that given in Eq. (2b) for granular layer thicknesses higher than 150mm, and thus it yields higher values than those obtained from Eq. (2a). Furthermore, this procedure postulates the following upper bounds for the unbound granular moduli: 350 (instead of 300) MPa for subbase materials and 500 (instead of 700) MPa for base materials.

All the above simplified procedures show these two characteristics: (a) the modular ratio is always greater than 1 or even 2; (b) the modular ratio always increases with the increase in granular thickness up to the predefined limiting values.

3 Review of layered developed models

As mentioned before, unbound granular materials have been found to exhibit moduli that are nonlinear or stress dependent. One widely used form of expressing this nonlinearity is the following very simple expression:

$$M_R = K_1 \times \Theta^{K_2} \tag{5}$$

where M_R denotes the laboratory resilient modulus of a granular material, in MPa; Θ, the bulk stress (first stress invariant), in MPa; K_1 and K_2, the regression constants reflecting material type and physical state of the M_R-Θ results, both in MPa.

From the literature survey, such as [12, 13, 14], It may be concluded that the characteristic values for K_1 in an MPa unit-system are (a) 21 for the granular base layer and (b) 10.5 for the granular subbase layer. For both layer types, the characteristic value for K_2 is 0.5. Accordingly, new equations for calculating the equivalent modulus of the granular layer in the pavement system were developed in 1981 [12]. This study utilizes the following layered-iteration approach:

Sub-layers of approximately 50mm are developed within the granular layer, and assumed moduli values are assigned initially to each. Layered solutions are then calculated by means of the CHEV computer program (the Chervon N-layered elastic theory computer program) in order to obtain states of stress within each sub-layer. These stress results are substituted into the modulus expression of Eq. (5) to yield predicted moduli values. Comparisons are made between the assumed and predicted sub-layer moduli. Iteration is pursued until tolerable error differences between these moduli are reached. For this final state of sub-layer division and their iterated moduli, critical strains (vertical strain at the subgrade surface and horizontal strain at the asphalt-layer bottom) are calculated for a 40-kN single-wheel load. Finally, equivalent (unique) granular moduli are determined that yield identical strains for each of these two specified strains.

Utilizing this procedure leads to the determination of unique E_{G1} (Version 1 of the equivalent granular moduli for the vertical strain equivalency) and E_{G2} (Version 2 of the equivalent granular moduli for the horizontal strain equivalency) values for various predefined pavement structures, various predefined K_1 values, and K_2=0.5 [13]. Finally, the multiple regression techniques applied to these values yielded these two expressions:

$$\log\left(\frac{E_{G1}}{6.894}\right) = 0.959 - 0.430 \times \log\left(\frac{H_A}{25.4}\right) - 0.073 \times \log\left(\frac{H_G}{25.4}\right) - \\ -0.122 \times \log\left(\frac{E_A}{6.894}\right) + 0.294 \times \log\left(\frac{E_S}{6.894}\right) + 0.848 \times \log\left(\frac{K_1}{6.894^{0.5}}\right) \tag{6a}$$

$$\log\left(\frac{E_{G2}}{6.894}\right)=1.079-0.511\times\log\left(\frac{H_A}{25.4}\right)-0.008\times\log\left(\frac{H_G}{25.4}\right)-$$
$$-0.155\times\log\left(\frac{E_A}{6.894}\right)+0.279\times\log\left(\frac{E_S}{6.894}\right)+0.888\times\log\left(\frac{K_1}{6.894^{0.5}}\right) \tag{6b}$$

where E_{G1} denotes the equivalent granular moduli for the vertical strain equivalency (Version 1), in MPa; E_{G2}, the equivalent granular moduli for the horizontal strain equivalency (Version 2), in MPa; E_S, the modulus of the subgrade; E_A, the modulus of the asphalt layer, in MPa; H_G, the thickness of the granular subbase-course layer, in mm; H_A, the thickness of the asphalt layer, in mm; K_1,the Eq. (5) regression constant of the granular material, in an MPa unit-system. It should be noted here that the Asphalt Institute's DAMA program [15] utilizes a similar expression to Eq. (6a) or Eq. (6b) for calculating the equivalent granular moduli. In its equation, the regression coefficients are defined by the average of those given in Eq. (6a) and Eq. (6b). The graphical presentations of Eq. (6a) and Eq. (6b) are given in Fig. 2 for both subbase (K_1=10.5) and base (K_1=21) for a constant asphalt modulus of 3,000 MPa, and for a constant asphalt thickness of 100mm. This figure indicates the following: (a) contrary to the simplified models, the modular ratio values derived from Eqs. (6) decrease slightly with the increase in the granular layer; (b) the equivalent granular moduli for the horizontal strain equivalency (granular modulus of Version 2) is generally greater than the equivalent granular moduli for the vertical strain equivalency (granular modulus of Version 1); (c) the difference between the above-mentioned moduli increases for pavement structures with thinner surface layers and higher quality, thick, granular base layers; (d) for moderate and high values of subgrade modulus, the modular ratio of the subbase material decreases below the 1.0 value; (e) for only high values of subgrade modulus, the modular ratio of the base material decreases below the 1.0 value.

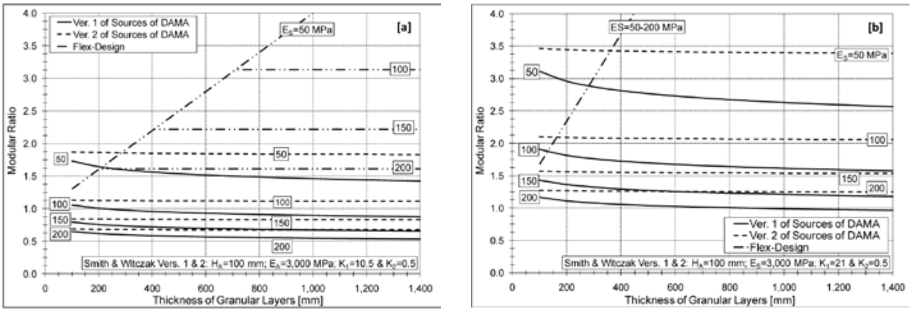

Figure 2 Comparison of the outputs of Eqs. (2) (Flex-Design) with the outputs of Eqs. (6) (Sources of DAMA) for (a) subbase material with K_1=10.5 and (b) base material with K_1=21.

Fig. 2 also includes a comparison of the outputs of Eqs. (2) (Flex-Design) with the outputs of Eqs. (6) (termed the Sources of DAMA in the figure) for both subbase and base materials. This comparison shows that the modulus of the granular subbase and base materials obtained by Eq. (6b) is considerably lower than that obtained by the Flex-Design procedure for all thickness and subgrade modulus ranges, with the following exception: small granular-layer thickness at low subgrade modulus. Thus, it seems that in the light of these findings the use of Eqs. (2) in the Flex-Design procedure is questionable.

In addition to the above findings, it can be seen directly from Eq. (6a) that an increase in the asphalt modulus (E_A) or the asphalt thickness (H_A) decreases the modular ratio. This behavior is compatible with the fact that stresses distributed in the granular layer are reduced when E_A or H_A are increased. This leads to a lesser granular modulus as indicated by Eq. (5). It can further be seen directly from Eqs. (6) that an increase in the subgrade modulus (E_S) leads to a lower rate of increase in the granular modulus than that of all the models described in the previous section, namely, a power of 0.294 as given by Eq. (6a) or 0.279 as given by Eq. (6b) instead of a power of 1.0.

Recently, similar calculations were conducted in a doctorial thesis [16] using the KENLAYER program for wet-mix macadam material, for which K_1=16 and K_2=0.748 in an MPa unit-system. The loading considered in that analysis was a dual wheel load of 102-kN (51-kN on each wheel load) acting over a circular contact area at a contact pressure of 700 kPa. These calculations led to the following expressions for the equivalent granular moduli that resulted only from the vertical strain equivalency (see also Fig. 3a):

$$E_{G1} = 80.12 - 3.8310 \times 10^{-3} \times E_A - 0.15 \times H_A - 0.03 \times H_G + 0.42 \times E_S \tag{7a}$$

$$E_{G1} = 122.627 - 0.105 \times H_G + 0.38 \times E_S \Rightarrow \quad \text{for the} \quad H_A = 0 \quad \text{case} \tag{7b}$$

where the notation of these equations is the same as for Eqs. (6).

Another doctorial thesis [17] used the three-dimensional finite element model, which accounts for nonlinear behavior of unbound pavement materials. For the case of HA=0, K1=21.3, and K2=0.737 (in an MPa unit-system), the following expression for the equivalent granular moduli resulting from the vertical strain equivalency was obtained (see also Fig. 3b):

$$E_{G1} = 243.64 - 0.21 \times H_G + 0.22 \times E_S \Rightarrow \quad \text{for the} \quad H_A = 0 \quad \text{case} \tag{8}$$

where the notation of this equation is, again, the same as for Eqs. (6).

The graphical presentation of Eq. (7a) for H_A=100mm, Eq. (7b) for H_A=0mm, and Eq. (8) also for H_A=0mm is given in Fig. 3. This figure shows the following: (a) for the H_A=100mm case, the values of the base-modular ratio of Eq. (7a) are extremely low, descending to values below 1.0; (b) for the H_A=0mm case, the values of the base-modular ratio of Eq. (7b) are lower than those of Eq. (8); (c) as in the Sources of DAMA case, all values of the base-modular ratio also decrease with an increase in the granular layer; (d) for the range of 100 to 1,000mm thickness, an increase in the subgrade modulus (E_S) leads to a lower rate of increase in the granular modulus than that of all the models described in the previous section: a power of 0.26 to 0.67 as can be calculated by Eq. (7a), 0.45 to 0.63 as can be calculated by Eq. (7b), or 0.09 to 0.39 as can be calculated by Eq. (8).

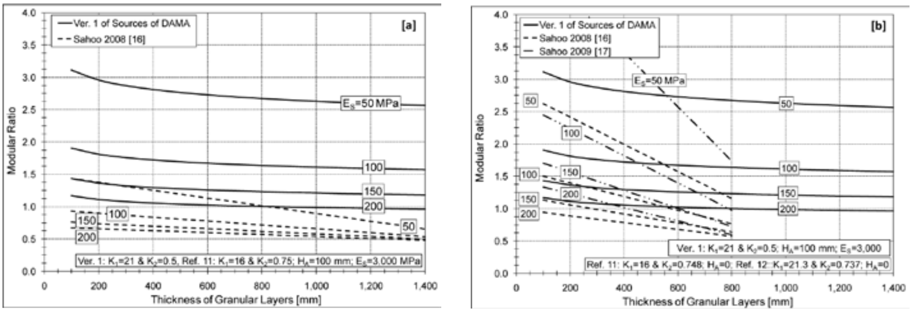

Figure 3 Comparison of the outputs of Eq. (6a), (Version 1 of Sources of DAMA) with the outputs of (a) Eq. (7a) with H_A=100mm and (b) Eq. (7b) and Eq. (8) both with H_A=0, for base materials.

Fig. 3 also includes a comparison of the outputs of Eq. (6a) (Version 1 of Sources of DAMA) with the outputs of Eq. (7a) for base materials. This comparison indicates that the modulus of the granular base materials obtained by Eq. (7a) is considerably lower than that obtained by

Version 1 of the Sources of DAMA procedure for all ranges of thickness and subgrade modulus. In light of the findings shown in Fig. 3b, it is possible that this discrepancy can be reduced if a similar procedure to that which led to Eq. (8) is applied to the $H_A > 0$ case.

4 New Mechanistic-Empirical model

In order to update the models reviewed, it is necessary to describe the procedure that takes place in the new Mechanistic-Empirical (M-E) design of flexible pavements, termed MEPDG [18]. In this M-E procedure, the design inputs that will characterize the material are introduced from one of three levels of data quality: (a) Level 1 (Laboratory Testing), which consists of specific site and/or material inputs for the project obtained through direct testing or measurements; (b) Level 2 (Correlation with Other Material Properties), which consists of the use of correlations to establish or determine the required inputs; (c) Level 3 (Typical Values), which consists of the use of national or regional values to define inputs.

In more detail, the resilient modulus values of Level 1 for unbound materials are determined from cyclic triaxial tests on prepared representative samples. The nonlinear elastic coefficients and exponents of the constitutive model are determined from these tests. The constitutive model, which is different from the one given in Eq. (5), is defined by the following expression:

$$M_R = k_1 \times P_a \times \left(\frac{\Theta}{P_a}\right)^{k_2} \times \left(\frac{\tau_{oct}}{P_a} + 1\right)^{k_3} \qquad (9)$$

where M_R denotes the laboratory resilient modulus of a granular material, in MPa; P_a, the normalizing stress (atmospheric pressure), in MPa; Θ, the bulk stress (first stress invariant), in MPa; τ_{oct}, the octahedral shear stress $(1/3)\times[(\sigma_1-\sigma_2)^2+(\sigma_1-\sigma_3)^2+(\sigma_2-\sigma_3)^2]^{0.5}$, in MPa; k_1, k_2, and k_3, the regression constants of the M_R-Θ-τ_{oct} results, reflecting material type and physical state.

It should be noted here that for the M-E design of Level 1, the input data is not the actual M_R test data but, rather, the coefficients k_1, k_2, and k_3.These coefficients must therefore be determined outside the Design Guide software. Their representative values are given in the technical literature [19].

Level 2 resilient modulus values for unbound materials are estimated from predefined general correlations that describe the relationship among soil index, strength properties, and resilient modulus. For example, the resilient modulus of the subgrade or unbound base materials is estimated from CBR or R-values, using empirical correlations. Finally, Level 3 resilient modulus values for unbound materials are estimated from predefined values for any given material classification. For example, for a GW material, a maximum M_R value is equal to 290 MPa, and a typical M_R value is equal to 283 MPa. In the same manner, for a GP material, a maximum M_R value is equal to 276 MPa, and a typical M_R value is equal to 262 MPa. Note that in contrast to the models described in the previous sections, the values of Level 2 and Level 3 are not a function of either the subgrade modulus or the thickness of the granular layer. Moreover, the predefined M_R values are rather low in comparison with the limiting values incorporated in the Flex-Design software (300 MPa for subbase material and 700 MPa for base material) in the Australian practice (350 MPa for subbase material and 500 MPa for base material) or as reported by others (600 MPa in [20] and 1,000 MPa in [21], both for limestone base-course material). Finally, as in the previous section, here, too, the sub-layering method for the Level 1 case is applied in the following way: an unbound base layer thicker than 150mm and a subbase layer thicker than 200mm are sub-layered internally in the Design Guide software for analysis purposes. For the base layer (first unbound layer), the first sublayer is always 5 mm; the remaining thickness of the base layer and any subbase layers that are sub-layered are divided into sub-layers having a minimum thickness of 100mm.

To conclude, the MEPDG procedure described leads to modular ratio values smaller that those associated with the Flex-Design software. In addition, for input data for Level 2 and Level 3, these modular ratio values are not a function of either the subgrade modulus or the thickness of the granular layer. As for Level 1, the same pattern of Eqs. (6) or Eq. (7a) may apply.

5 Validation with FWD measurements

In order to determine the model for calculating the modular ratio from the models described in the previous sections, this section presents a comparison study of several sets of FWD backcalculated ratios with the aid of MODULUS 6 software. Fig. 4 displays the variation of backcalculated modular ratios (for the subbase and base layers combined as one layer) with backcalculated subgrade moduli as obtained from FWD deflection measurements at five different sites. This figure shows that contrary to the Flex-Design software or the FAARFIELD software and others, a modular ratio of values below 1.0 may exist. Here it should be added that backcalculated granular moduli that exceed the value of 1,500 MPa were excluded from Fig. 4 as were backcalculated granular modulus values associated with unrealistic backcalculated depths to bedrock. The 1,500 MPa value, which is higher than the limiting values associated with the Flex-Design software, was chosen to include the proven fact that FWD backcalculated granular moduli are in many cases higher than the average corresponding laboratory resilient moduli; according to [22], about 1.8 times higher.
Here it is worthwhile noting that recent LTPP studies [4] suggest a screening procedure for the backcalculation outputs to conform to what may be considered a reasonable, predefined broad range of modulus values for various pavement materials. For the granular materials--namely uncrushed gravel, crushed gravel, and crushed stone--these reasonable maximum values are 750, 1,000, and 1,500 MPa, respectively. In the same manner, the maximum reasonable backcalculated value for all granular materials according to [2] is 1043 MPa.
Fig. 4 also shows that the variation in the backcalculated modular ratio (R) with the backcalculated subgrade modulus (E_S) is of a negative power nature (i.e., $R=\alpha\times E_S^{-\beta}$, and thus $E_G=\alpha\times E_S^{1-\beta}$). For Sites Nos. 2 to 5, the power values (β) obtained range from 0.63 to 1.17; and for Site No. 1, it is equal to 0.17. As a power value of $\beta=1.0$ yields granular moduli that are independent of the subgrade moduli, it may be concluded that except for Site No. 1, all the backcalculated granular moduli shown in Figure 4 are almost independent of their corresponding subgrade moduli. It should be noted that according to Eqs. (6), β is equal approximately to 1-0.3=0.7.

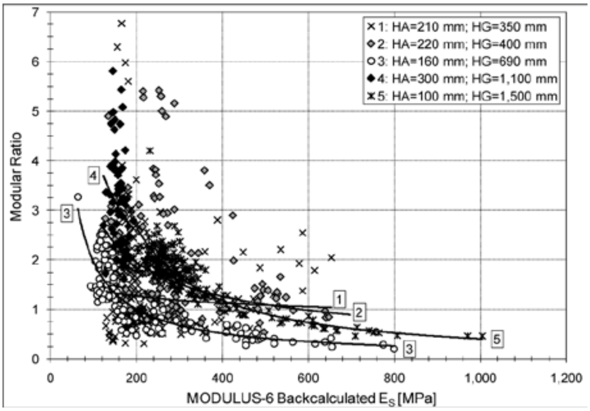

Figure 4 Display of backcalculated modular ratios with backcalculated subgrade modulus as obtained from FWD measurements.

In addition to Fig. 4, Table 1 presents the following backcalculated values for each of the five measured sites: (a) average and standard deviation values of the modular ratio; (b) average and standard deviation of both the granular and subbase moduli. Again, the given values of the granular materials are for subbase and base layers combined as one layer. The values in parenthesis in the table denote the combined modular ratio (R_C) calculated from Eqs. (2), utilizing the AASHTO SN equivalency for H_{GB}=150mm and H_{GS}=(H_G-150)mm. In both the table and Fig. 4, H_G denotes the total thickness of the granular layers, and H_A the asphalt-layer thickness. H_{GB} denotes the granular base thickness, and H_{GS} the granular subbase thickness.

Table 1 shows that the values given in parenthesis are higher than both the average backcalculated modular ratios and, obviously, the average minus standard deviation (i.e., the 15th percentile) of the backcalculated modular ratios. Thus, the use of Eqs. (2) in the Flex-Design software may lead to overestimated values in cases in which the subgrade modulus is rather high (222 MPa and above) or when the granular thickness is rather high.

The average backcalculated data in Table 1 enables conducting a multiple regression analysis for the following independent variables: H_G, E_G, and E_S. The outcome of this analysis is Eq. (10a). Another outcome is Eq. (10b), for which a predefined β of 1-0.3=0.7, taken from Eqs. (6), was adopted in order to be compatible with the Sources of DAMA.

$$E_G = 85.1859 \times H_G^{0.1946} \times E_S^{0.0432} \tag{10a}$$

$$E_G = 21.0130 \times H_G^{0.1931} \times E_S^{0.3000} \tag{10b}$$

In Eqs. (10), the coefficient of determination is significantly low: 0.095 and 0.100, respectively, for Eq. (10a) and Eq. (10b).

Table 1 FWD backcalculated granular moduli and modular ratios measured at five sites

Properties Group	Properties Details	Site No.				
		1	2	3	4	5
Structure Characteristics	State	New	New	Old	Rehab.	Old
	H_G [mm]	350	400	690	1,100	1,500
	H_A [mm]	210	220	160	300	100
Modular Ratio	Average	1.50	1.89	1.18	3.19	1.70
	S. Deviation	1.00	1.12	0.68	0.97	0.55
	Flex-Design	(2.20)	(2.32)	(3.09)	(4.28)	(5.46)
Granular Modulus [MPa]	Average	325	494	210	505	500
	S. Deviation	244	284	62	148	86
Subgrade Modulus [MPa]	Average	222	287	246	159	329
	S. Deviation	98	119	168	14	156

For the same assumption for which the thickness of H_{GB} comes from the data in Table 1 (i.e., 150mm) and for the assumption for which the ratio of the granular base material (E_{GB}) to the granular subbase material (E_{GS}) equals 2.0, the following two expressions can be derived, again by utilizing the AASHTO SN equivalency:

$$E_{GS} = \frac{E_G}{\left(\frac{H_G + 39}{H_G}\right)^3} \tag{11a}$$

$$E_{GB} = 2 \times E_{GS} \tag{11b}$$

The utilization of Eqs. (11) leads to the plot presented in Fig. 5. In other words, this figure displays the variation in the estimated base-course modulus (EGB) with the variation in the total subbase and base-course thickness for (a) the regression models of the measured data in Table 1, with (1-β) equals 0.0432 and 0.3000; (b) the Flex-Design model. Fig. 5 points out that for low and moderate subgrade moduli, the Flex-design model leads to lower base-course moduli for both (1-β) values. This conclusion supplements that given earlier with regard to the Table 1 data. It is also compatible with the FWD measurements presented in [11] for low and moderate subgrade moduli.

The above findings lead to uncertainty in adopting any model for calculating the granular modulus. Furthermore, it seems that the change in the granular modulus in the Flex-Design model with the variation in subgrade modulus and granular thickness may be unrealistic, since it yields underestimated values for low subgrade moduli or overestimated values for high granular thicknesses.

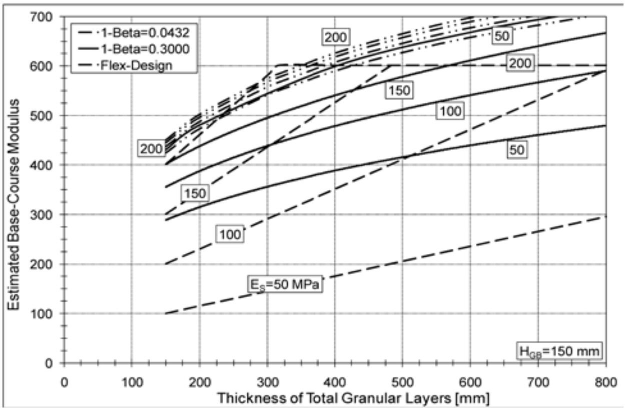

Figure 5 Variation in estimated base-course modulus with thickness of total granular layers and a fixed base-course layer of 150mm according to (a) Eqs. (2) and (b) Eqs. (11) and Eqs. (12).

Thus, it seems that at this stage of our knowledge, the Level 3 method of the M-E procedure, with somewhat higher values for its granular moduli (say, 300 MPa for subbase material and 400 MPa for base-course material), is more suitable for application in the Flex-Design software and other software that calculate the required asphalt layer according to the maximum tensile (horizontal) strain occurring at the bottom of the asphalt layer. Here, it should be emphasized that according to the Flex-Design outputs, a higher granular modulus requires a thinner asphaltic layer and vice versa. For example, in the case of 20×10^6 ESALs, an increase of 50% in the granular modulus decreases the required asphalt layer by about 20%, while a decrease of the same rate increases the required asphalt layer by about 25%.

6 Conclusions and recommendations

The findings of the present study of the various models for calculating the granular modulus and the accompanying FWD measurements lead to the following conclusions and recommendations:

- The wide range of results obtained from all models reviewed in the present paper leads to uncertainty in adopting any model as preferable for calculating the granular modulus;
- The change in granular modulus in the Flex-Design model or the USCOE model with the variation in subgrade modulus and granular thickness seems to be unrealistic, as it yields either underestimated values for low subgrade moduli or overestimated values for high granular thicknesses;
- It seems that at this stage of knowledge, the Level 3 method of the new Mechanistic-Empirical (M-E) procedure, with somewhat higher values for its suggested granular moduli (say, 300 MPa for subbase material and 400 MPa for base-course material), is more suitable for the Flex-Design software and other software that calculate the required asphalt layer according to the maximum tensile (horizontal) strain occurring at the bottom of the asphalt layer.

Obviously, the best procedure for calculating granular moduli is to insert the layered elastic procedure into the Flex-Design and other design software as suggested in Level 1 of the new Mechanistic-Empirical (M-E) software, MEPDG, for the design of flexible pavements.

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