Material Layer Coefficients of Unbound Granular Materials from Resilient Modulus

GONZALO RADA AND MATTHEW W. WITCZAK

The overall objective of this study was to determine and evaluate AASHO-based material design parameters for unbound granular base/subbase materials from laboratory nonlinear resilient-modulus tests. A total of 101 nonlinear M_r-relationships were developed on six typical Maryland State Highway materials. Various levels of saturation and compactive effort were evaluated with each material. By using the M_r-θ (bulk-stress) relationships developed, a nonlinear elastic-layered analysis was undertaken on 40 different pavement cross sections to establish typical bulk-stress and resilient-modulus values. The correlations between modulus and layer coefficients (a_i) and composite modulus of subgrade reaction (k_c) given in National Cooperative Highway Research Program (NCHRP) Report 128 were used to evaluate the influence of subgrade support, material type, thickness of asphalt surfacing, compactive effort in the granular layer, and degree of saturation on the a_i- and k_c-values. Substitution ratios were also developed for flexible pavement design concepts by using a dense-graded aggregate base-course material as the reference. For each material investigated, predictive equations for the layer coefficient values (a_i) were determined as functions of the subgrade California bearing ratio, compaction, saturation, and thickness of asphalt. All these variables were found to be significant from a design viewpoint. Average reductions in the a_i-value of 0.065, 0.044, 0.041, and 0.029 were found for the range in subgrade support, saturation level, compactive effort, and asphalt thickness, respectively. In the study of the composite modulus of subgrade reaction k_c used in rigid pavement analysis, it was found that the greatest influence was exerted by the subgrade support and thickness of the granular subbase layer. The maximum influence of material type, compactive effort, and/or degree of saturation on the k_c-value was less than ±10 percent for all cases considered. Reliable predictive equations for k_c were developed in terms of the primary variables evaluated.

It is current Maryland State Highway Administration (MSHA) design practice to use modified forms of the Interim Guide of the American Association of State Highway Officials (AASHO) for design of rigid and flexible pavements within the state highway system (1). For flexible pavement design, the current MSHA practice is based on the AASHO analysis, combining the use of selected substitution ratios for various subbase and base materials (2).

One significant shortcoming of the rigid and flexible pavement design procedure pertains to the general characterization of the subbase and base materials from the performance of the AASHO Road Test. For rigid pavement design practice, Westergaard's modulus of subgrade reaction (k) is used in the MSHA analysis as a typical property of foundation substrata. The AASHO Interim Guide has one major advantage to this procedure in that it provides for the estimate of the composite modulus of subgrade reaction (k_c) based on subbase (base) thickness and the resilient modulus in stiffness of the subbase type used. This characterization is shown in Figure 1. As a method of using this plot, general ranges of stiffnesses are recommended for several subbase types. For granular materials, the recommended range, from the AASHO Interim Guide, is from 8000 to 28 000 psi.

For current flexible pavement design practice, the use of substitution ratios (SR) or layer equivalents is advocated by the MSHA design procedure (2). Values for the various MSHA granular material categories are SR = 1.0 for dense-graded aggregate (DGA), SR = 1.0 for sand aggregates, and SR = 0.75 for bank-run gravel.

Even though SRs have been (and are) used in flexible pavement design procedures, it should be emphasized that the basis for determining the magnitude of the ratio can also be explained and derived in terms of a_i-values (structural layer coefficients) directly used in the AASHO flexible design equation for the structural number (SN):

$$SN = a_1 D_1 + a_2 D_2 + \ldots + a_i D_i \tag{1}$$

where a_i are layer coefficients representative of each layer and D_i are layer thicknesses.

A general interpretation of the SRs used in the MSHA procedure (2) is as follows:

$$SR = (a_2/a_i)\,DGA \tag{2}$$

where a_2 is the layer coefficient for the MSHA DGA base course and a_i is the layer coefficient for any other material in the ith layer.

In Report 128 of the National Cooperative Highway Research Program (NCHRP), additional guidance concerning the selection of the specific a_i-value for a wider selection of material types and properties is presented (3). It is of special importance to note that among the correlative material properties suggested to determine the a_i-value of granular layers, the stiffness or resilient modulus of the material is recommended. The specific nomographs suggested for granular base and subbase materials are shown in Figure 2.

STUDY OBJECTIVES

Based on the general background information provided in the introduction, it is important to note that the resilient modulus or stiffness characterization of materials can be directly used in both rigid and flexible pavement design procedures used by MSHA as well as for pavement rehabilitation studies. However, one major limitation of the use of the modulus is that at present only suggested ranges of stiffnesses for a limited number of subbase and base types have been made. Thus, the design process for MSHA conditions must still allow for considerable engineering judgment for estimation of a design modulus of subgrade reaction (k_c) or SRs determined from an analysis of the structural layer coefficients (a_i). In view of this, an extensive laboratory study was initiated at the University of Maryland for MSHA to characterize the specific major types of base and subbase materials that are currently used in the state and to provide much-needed and important design input for both rigid and flexible pavement design systems.

From the relatively large data base that was generated from the study, the following specific objectives were studied in the project:

1. To determine from laboratory resilient-modulus test results, typical resilient-modulus relationships of granular base and subbase materials used by MSHA,
2. To determine typical structural layer coefficient (a_i) and SR-values for flexible pavement design based on an analysis of these modulus relationships by using Figure 2, and
3. To determine typical composite modulus of subgrade reaction values for rigid pavement design based on an analysis of the typical modulus relationships and Figure 1.

Figure 1. Modified AASHO chart for estimating k_c-values.

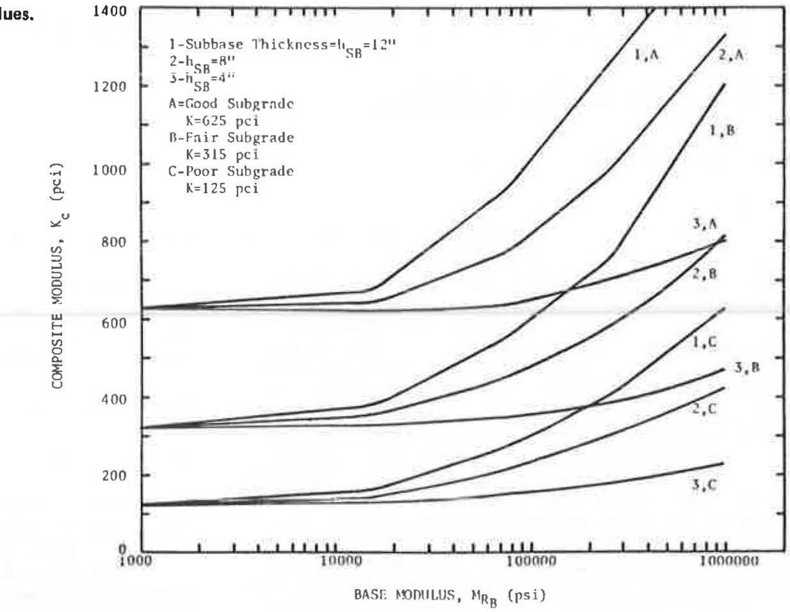
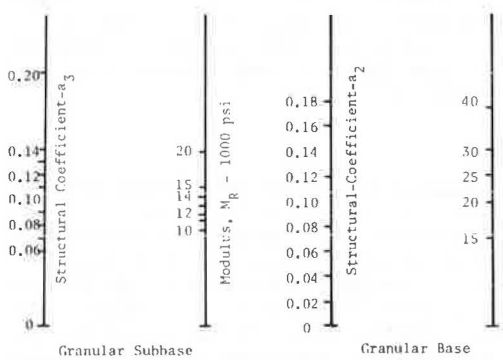


Figure 2. AASHO layer coefficient and modulus correlations for granular material.



SOURCE OF DATA

For the laboratory modulus study, 101 test results obtained at the University of Maryland were used as the data base. However, it should be noted that 170 other modulus results found in the literature were used as an aid in determining the effect of different factors (discussed in the next section) on M_r. The agencies, number of tests, material types studied, and references from which these results were obtained have been presented in a previous publication (4).

The University of Maryland study involved testing six different aggregate types (two limestones that met MSHA DGA specifications, a crushed stone and slag that met MSHA CR-6 specifications, a bank-run gravel that met MSHA GP specifications, and a sand-aggregate subbase blend). For each aggregate type investigated, three hand-blended gradations were used. On each aggregate-gradation combination, three compaction energies (low, standard, and modified) were used to develop moisture-density relationships.

The resilient-modulus phase of the test program involved testing 18 specimens per aggregate type. In general, for each aggregate-gradation combination, three M_r-tests were conducted at modified compaction effort (MCE) (optimum and ±2 percent optimum moisture), two tests at standard compaction effort (SCE) (optimum and +2 percent optimum), and one at optimum for a low compaction effort (LCE) (2200 ft•lb/ft³). The above test program should have yielded 108 data points, but seven specimens were unable to be tested.

RESULTS

Typical Mr-Values for MSHA Granular Materials

The initial objective of the overall study was to develop typical M_r-values for the MSHA unbound granular base and subbase materials. In order to do this, an investigation of the factors affecting M_r was undertaken. The details and results of this specific portion of the study have been the subject of a previous technical paper (4) and only a brief summary is presented here. The results of this effort indicated that the factors that most significantly affect M_r are the stress state, degree of saturation, and compactive effort. The amount of fines (percent passing the No. 200 sieve) and gradation were also found to affect M_r; however, their influence was very small when compared with the effect of the factors previously stated.

With these factors in mind, typical $M_r = f(\theta)$ relationships for the six aggregate types studied were developed and are summarized in Table 1. As can be seen in this table, instead of a single M_r-relationship for each aggregate type, several relationships that reflected the relative influence of the significant variables--stress state, saturation, and compaction (density)--on the k_1- and k_2-constants in the equation $M_r = k_1\theta^{k_2}$ were developed.

A more comprehensive discussion of factors influencing M_r and typical M_r-values for the MSHA unbound granular materials shown in Table 1 is presented in papers by Rada and Witczak (4) and by Rada (5).

Layer Coefficients and SRs

The second objective of this study was to develop typical values for the structural layer coefficient (a_i) and the SR for the MSHA unbound granular materials under investigation based on the M_r (or k_1-k_2-relationships) results of the previous

Table 1. Typical M_r-relationships for MSHA unbound granular aggregates.

	Dry (S_r < 60 percent)				Wet (S_r > 85 percent)			
	SCE		MCE		SCE		MCE	
Aggregate	K_1	K_2	K_1	K_2	K_1	K_2	K_1	K_2
DGA-limestone-1[a]	8 500	0.5	10 500	0.5	7000	0.4	9000	0.4
DGA-limestone-2[a]	11 500	0.3	15 000	0.3	6000	0.5	7500	0.5
CR-6-crushed stone[a]	6 000	0.5	9 000	0.5	3500	0.7	5000	0.7
CR-6-slag[a]	12 500	0.35	20 000	0.35	5600	0.35	9000	0.35
Sand-aggregate blend[b]	3 800	0.5	6 000	0.5	1900	0.7	3000	0.7
Bank-run gravel[b]	5 000	0.4	8 000	0.4	1250	0.7	2000	0.7

[a] K_1- to K_2-values typical for fines percentage (No. 200) less than 15-18 percent.
[b] K_1-value should be decreased and K_2-value increased if fines percentage is greater than 10 percent.

section. In this study, layer coefficients (a_i) were calculated by using the NCHRP Report 128 (a_i-M_r) correlations presented in Figure 2 (3). SRs were also computed by using Equation 2. It is important for the reader to understand that new correlations were not developed in this study, but rather, the existing correlation found in NCHRP Report 128 was used to develop the typical a_i- and SR-values.

Base and Subbase Bulk-Stress Values

Before any investigation of the a_i- and SR-values could be undertaken, typical modulus M_r-values had to be calculated for the different material-property combinations investigated. Obviously, these modulus values should approximate those encountered in actual pavement systems and conditions.

Recalling the typical k_1-k_2 relationships previously developed for the six MSHA unbound granular materials (Table 1), which account for the physical properties influencing the resilient response of these materials (degree of saturation and compaction), the only remaining variable necessary to predict typical M_r-values for the unbound granular materials is the typical states of stress (θ) in the granular materials under different pavement conditions.

The different conditions used to estimate the θ-values for the different base course and subbase course materials were developed from a matrix of typical pavement structures designed in accordance with the MSHA flexible pavement design procedure.

The various pavement structure components were used in a multilayer elastic analysis to determine bulk-stress values. Elastic-layered input used in the study was as follows:

h_1 (AC thickness): 3, 4.5, 6, and 8 in;
h_2 (granular base): 6 in;
h_3 (subbase): varied by h_1- and E_4-value; five values per h_1; 0-27 in;
E_1 (AC modulus): $1.5\text{x}10^5$ psi and $1\text{x}10^6$ psi;
E_4 (subgrade modulus): California bearing ratio (CBR) = 3, 5, 10, 20, and 30 (E_4 = 1500 CBR);
$E_2 = M_{R_b} = 9100\theta^{0.45}$ psi; and
$E_3 = M_{R_{sb}} = 3870\theta^{0.58}$ psi.

This input represented pavement structures capable of handling critical lane traffic levels varying from 50 to 2600 equivalent 18-kip single-axle loads (ESALs) per day. In order to minimize the number of elastic-layered computer program solutions necessary, typical k_1-k_2-relationships for a base material and subbase material were used to estimate the bulk stress. Altogether, 40 different pavement structures were analyzed in this phase of the study.

Finally, for all pavement conditions evaluated, the bulk-stress values were computed by using the nonlinear resilient modulus (NLRM) computer program (5) developed at the University of Maryland. Because the program yields as output the modulus (M_r) value from nonlinear characterizations and not θ, the bulk-stress values were backcalculated for each solution by using the following equation:

$$\theta = (M_r/k_1)^{(1/k_2)} \tag{3}$$

where M_r, k_1, and k_2 are known values for both the base and subbase layers.

The resulting bulk-stress values ranged between 4 and 60 psi for the base layer and from 4 to 25 psi for the subbase materials. These values have been presented in detail by Rada (5).

Layer Coefficient Values (a_i)

By using the k_1-k_2 relationships shown in Table 1 and the bulk-stress values previously developed, resilient moduli were calculated for the six MSHA unbound granular materials prior to computing the a_i-values. Since for each of the six MSHA base and subbase materials investigated, four k_1-k_2-relationships were developed (combinations of saturation and compactive effort) as well as 40 different pavement systems (combinations of E_i or M_{r_i}, h_i, and CBR_{s_g}), a total of 960 (640 for base, 320 for subbase) M_r-values were computed. These values have been summarized by Rada (5, Chap. 4).

In Figure 2, the NCHRP nomographs relating the a_i-value for granular base and subbase material to the resilient modulus were presented. Mathematically, these relationships can be defined as follows:

Base layer:

$$a_{2b} = 0.249 \log M_r - 0.977 \tag{4}$$

Subbase layer:

$$a_{2sb} = 0.227 \log M_r - 0.839 \tag{5}$$

By using these relationships and the M_r-values computed by the previous bulk-stress (k_1-k_2) study, a_{2i}-values were determined for all 960 combinations investigated.

Table 2 represents a condensed summary of the a_{2i}-values as a function of material types, asphalt-layer thickness, subgrade support, saturation level, and compactive effort. In this table, it can be observed that the values shown are based on an average temperature value (average or typical AC modulus). For the range of AC modulus studied, the influence of this factor, especially for levels of asphalt thickness used in practice, was not very sensitive to a_{2i}-changes. In addition, average layer coefficient values for the two DGA aggregates are shown along with average values for the two

Table 2. Typical layer coefficient values.

Material	Asphalt Thickness (in)	Subgrade CBR	Dry (S_r < 60 percent) SCE	Dry (S_r < 60 percent) MCE	Wet (S_r > 85 percent) SCE	Wet (S_r > 85 percent) MCE
Base						
DGA	<5	3	0.124	0.150	0.092	0.117
		5	0.130	0.157	0.098	0.125
		10	0.144	0.170	0.113	0.139
		25	0.171	0.197	0.145	0.171
	≥5	3	0.100	0.126	0.065	0.090
		5	0.104	0.130	0.069	0.095
		10	0.110	0.137	0.076	0.102
		25	0.139	0.165	0.108	0.134
Crusher run	<5	3	0.096	0.140	0.091	0.129
		5	0.103	0.147	0.100	0.138
		10	0.120	0.164	0.124	0.163
		25	0.155	0.199	0.174	0.212
	≥5	3	0.066	0.110	0.048	0.087
		5	0.071	0.115	0.055	0.093
		10	0.079	0.123	0.067	0.105
		25	0.115	0.159	0.116	0.155
Slag	<5	3	0.137	0.187	0.050	0.101
		5	0.141	0.192	0.054	0.105
		10	0.154	0 204	0.067	0.118
		25	0.178	0.229	0.091	0.142
	≥5	3	0.115	0.166	0.028	0.080
		5	0.119	0.170	0.032	0.083
		10	0.124	0.175	0.037	0.089
		25	0.149	0.200	0.062	0.114
Subbase						
Sand/gravel	<5	3	0.060	0.100	0.024	0.042
		5	0.071	0.117	0.035	0.060
		10	0.100	0.145	0.054	0.102
		25	-	-	-	-
	≥5	3	0.060	0.100	0.024	0.042
		5	0.068	0.113	0.029	0.054
		10	0.082	0.128	0.033	0.078
		25	0.103	0.148	0.064	0.110

Table 3. Layer coefficient equations.

Layer	Material	Equation
Base	DGA	$a_i = (0.140 + 0.0029CBR_{sg}) + f_c + f_s + f_t$
	Crusher run	$a_i = (0.130 + 0.0035CBR_{sg}) + f_c + f_s + f_t$
	Slag	$a_i = (0.180 + 0.0024CBR_{sg}) + f_c + f_s + f_t$
Subbase	Sand/gravel	
	Thin AC	$a_i = (0.080 + 0.0064CBR_{sg}) + f_c + f_s$
	Thick AC	$a_i = (0.100 + 0.0021CBR_{sg}) + f_c + f_s$

Notes: Above equations based on following conditions: dry (S_r <60 percent), modified compactive effort, and thin asphalt surface (h_1 < 5 in). Equations valid for CBR_{sg} < 15 - 20.

subbase materials investigated (bank-run gravel and sand-aggregate blends).

From Table 2, it can be observed that the a_{2i}-values increase with an increase in subgrade support and decrease as compactive effort is reduced from modified to standard; the materials become more saturated and the thickness of the asphalt layer is increased. For one well-versed in pavement stress-distribution effects and nonlinear modulus characterization, these results are all very logical.

Relative to the influence of subgrade support, as stronger foundation soils are encountered, the stress state (θ) in the granular layer is increased. This increase in the bulk stress with increasing support will tend to increase the granular modulus and hence structural layer coefficients. Previous studies have shown that a reduced level of compactive effort tends to decrease the k_1-term in the expression $M_r = f(\theta)$. As such, this change in reduced compactive effort results in a lower

Table 4. Correction factors.

Layer	Material	f_c[a] (compactive effort)	f_s[b] (saturation)	f_t[c] (AC thickness)
Base	DGA	-0.026	-0.033	-0.035
	Crusher run	-0.040	-0.010	-0.045
	Slag	-0.051	-0.087	-0.025
Subbase	Sand/gravel	-0.045	-0.046	NA

Note: f-values vary ±0.005 with subgrade CBR_{sg}; average values shown.
[a] Used when going from MCE to SCE.
[b] Used when going from dry (S_R < 60 percent) to wet (S_R > 85 percent).
[c] Used when going from thin AC surface (<5 in) to thick AC surface (≥5 in).

modulus and hence decreased a_{2i}-value. It is also known that the influence of moisture in granular materials is a significant factor (especially for S_r-values greater than 85 percent). When high levels of degree of saturation occur, a reduction in the k_1-value and increase in k_2 occur, which have the net effect of a reduced modulus and a_{2i}-value. Finally, the thickness of asphalt is an important factor because it directly interfaces with the stress distribution within the granular layers. From basic slab rigidity concepts, stress attenuation is directly proportioned to the third power of layer thickness. As a result, when the thickness of the asphalt layer is increased, stress levels (i.e., θ) in the underlying layers are reduced. This results in a reduction in the M_r (a_{2i})-value as the thickness is increased. In Table 2, it can be observed that two levels of asphalt thickness have been selected. An analysis of the results indicated that for asphalt thicknesses of 5 in or more, there are only minor changes in the a_i-coefficient.

Although each of the four materials shown in Table 2 has its own unique relative ranking of factors regarding their sensitivity toward changing the a_{2i}-magnitude, the asphalt thickness had the least effect, whereas the subgrade support (over a range in CBR from 2 to 20) exhibited the greatest. The other two factors (compactive effort and degree of saturation) were intermediate and generally of the same order of magnitude. The average change in the a_{2i}-coefficient values over the range of parameters investigated in the study was found to be Δa_{2i} = 0.029 (asphalt thickness); Δa_{2i} = 0.041 (compactive effort); Δa_{2i} = 0.044 (saturation); and Δa_{2i} = 0.065 (subgrade support).

In analyzing the a_{2i}-values, it was found that for a given material, all the Δa_{2i}-values for a given parameter were independent (noninteracting) of the other factors. This was true for all cases except the thickness effect of the subbase (sand/gravel) material. This important conclusion allowed for the development of very simple and practical predictive equations (by material type). Table 3 summarizes these equations. In general, they are considered very accurate and applicable for subgrade CBR values less then 15-20.

The equations shown are all based on granular material at a modified compaction effort; dry (S_r <60 percent), and a thin asphalt (<5-in) pavement layer. For conditions other than these, simple correction factors (f_c, f_s, f_t) must be applied to the a_i-values. These factors are shown in Table 4. As a simple example, consider the a_{2b}-value for a slag base course used with a 3-in asphalt layer on a CBR = 10 subgrade. If the slag is assumed to exist in an in situ condition of 90 percent saturation at 100 percent standard compaction, the a_i-value would be, from Table 2,

$$a_{2b} = (0.180 + 0.0024CBR_{sg}) + f_c + f_s + f_t \quad (6)$$

Figure 3. Influence of parameters investigated on layer coefficients (base course, DGA).

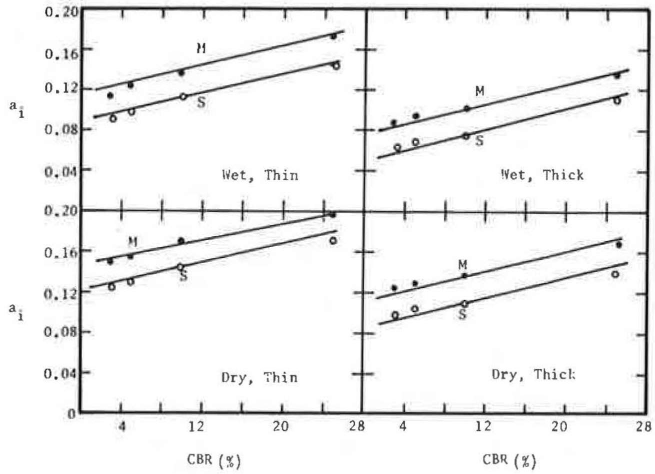


Figure 4. Influence of parameters investigated on layer coefficients (base course, crusher run).

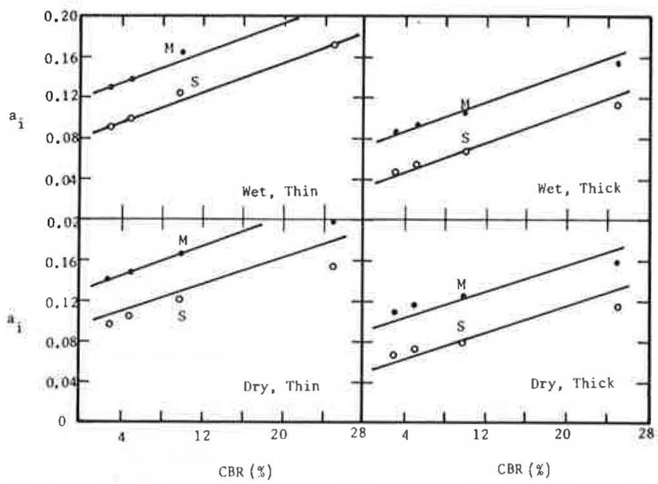


Figure 5. Influence of parameters investigated on layer coefficients (base course, slag).

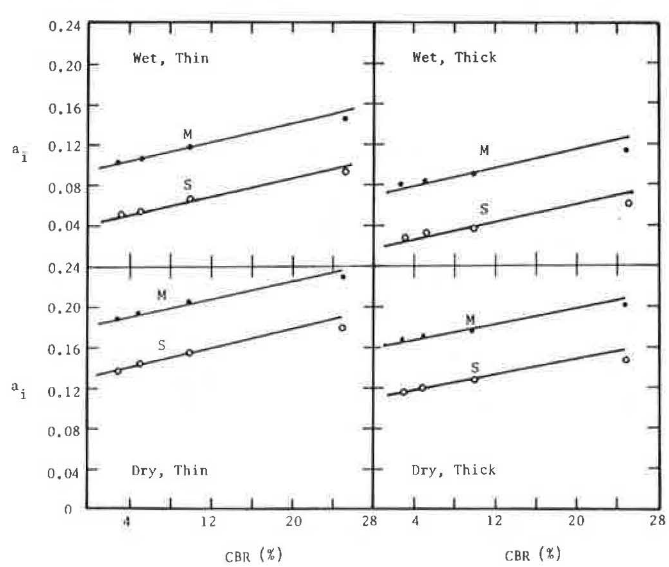
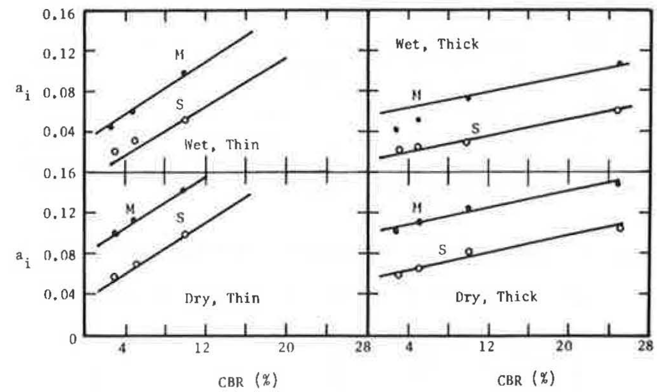


Figure 6. Influence of parameters investigated on layer coefficients (subbase course, sand gravel).



where

f_c = -0.051,
f_s = -0.087, and
f_t = 0 (no thickness correction necessary since thin category is present);

or

$$a_{2b} = 0.180 + 0.024 - 0.051 - 0.087 = 0.066 \qquad (7)$$

(a_2 is 0.067 from Table 2.)

Although the a_{2b}-value of 0.066 may at first appear to be low, the influence of saturation on this material (as reflected by f_s = -0.087) was quite large in the laboratory M_r test program. This can be observed by viewing the reduction in the k_1-values in Table 1 for the CR-6-slag material between the dry and the wet conditions.

In order to help visualize the accuracy of the predictive equations noted in Table 3, Figures 3 through 6 show the predicted relationships compared with the individual data points noted in Table 2.

SR-Values

The development of typical SRs for unbound granular base and subbase materials was simply a continuation of the a_i-calculations and predictive equations developed. SR-values were developed by using Equation 1 and the predictive a_i-equations summarized in Table 3. The basic a_2-value selected for the computations was the DGA base material, compacted at modified compaction, dry (S_r < 60 percent), and with a thin asphalt layer (<5 in).

Because the a_i-values are functions of the subgrade CBR, the SR-values calculated were all evaluated at the same CBR value for the DGA and ith material-property combination. Figures 7 and 8 summarize the results of this analysis for the base and subbase materials, respectively. In these figures, only the extreme combinations are plotted (dry, modified to wet, standard). The range between these two combinations reflects the general variation in the SR-value due to variable compaction-moisture conditions that would probably be expected to occur in the field.

As can be noted, each combination (of compaction and saturation) is a function of the subgrade CBR. From a practical viewpoint, the SR-value decreases with an increase in subgrade support. However, with the exception of several wet-standard plots, the practical effect of subgrade is not significant, especially as the compactive effort is increased and low saturation levels are encountered.

Table 5 is a general summary of typical SR-values (at a subgrade CBR ≈ 10) for the factors evaluated in the study. Obviously, with the exception of subgrade support (normalized in the SR-computations), the same parameters and their relative ranking influencing the a_i-value affect the SR-values. The table clearly indicates the influence of compactive effort, moisture, and thickness of asphalt on the resultant values.

Figure 7. Influence of parameters investigated on base material SR-values.

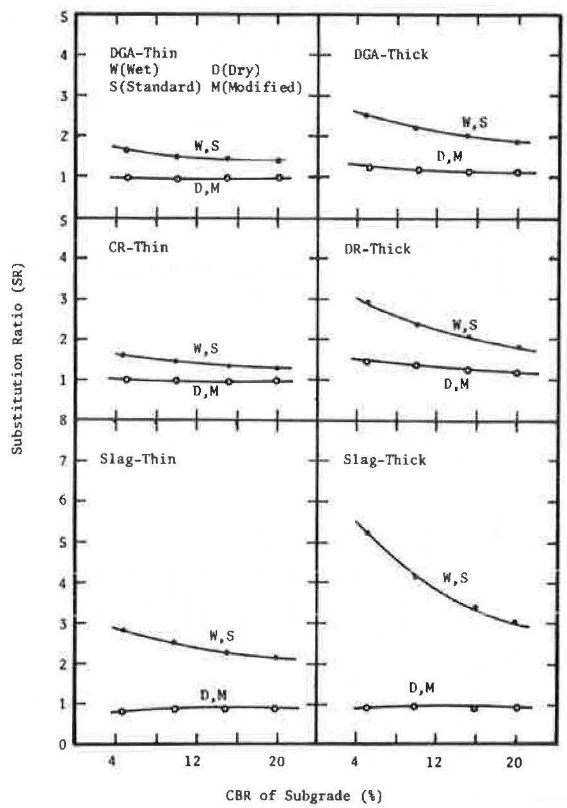
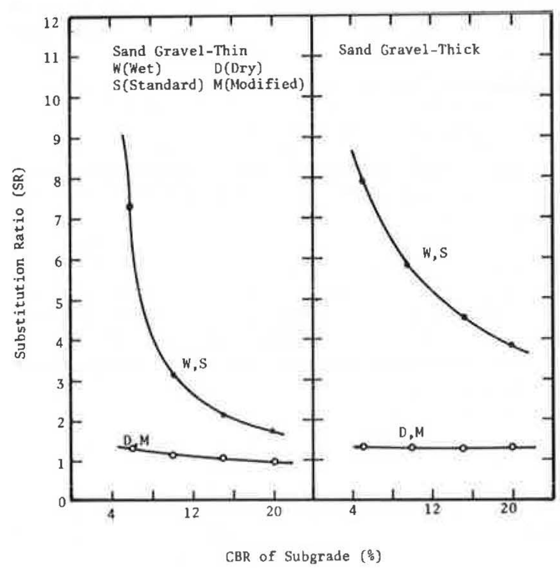


Figure 8. Influence of parameters investigated on subbase material SR-values.



Composite Modulus of Subgrade Reaction

The other major objective of this study was to develop composite modulus of subgrade reaction values k_c for the six MSHA materials investigated, based on the resilient-modulus (k_1-k_2) relationships previously developed. As stated in this report, these k_c-values were to be based on the relationships presented in NCHRP Report 128 (3).

Bulk-Stress Values of Subbase Materials

Like the layer coefficient and SR study for flexible pavements, the only unknown parameter needed to predict the resilient moduli for granular subbase layers beneath the rigid pavement is the bulk-stress value. In this analysis, a value of 4.2×10^6 psi for the modulus of the concrete layer and a linear relationship between log E_1 (modulus of surface layer) and θ (bulk-stress value) were assumed. Bulk-stress values were then found for the subbase materials under rigid pavement (8-10 in thick) by extrapolating the NLRM computer program results.

Based on this study, an average bulk-stress value of θ = 4 psi was selected as the value that defined the typical state of stress in the subbase layer for all subbase conditions.

Composite Modulus (k_c) Results

Once the typical bulk-stress value of 4 psi was selected, the M_r-values for the six aggregate types in this study were computed by using the following equation:

$$M_r = k_1 \theta^{k_2} = k_1 (4)^{k_2} \text{ psi} \quad (8)$$

where k_1 and k_2 are the values (for combinations of saturation and compaction) found in the first part of the study. The M_r-values calculated for the different combinations of saturation and compaction are presented in Table 6.

By using these M_r-values for each of the aggregates and in situ conditions previously described, the existing NCHRP nomograph for the composite modulus of subgrade reaction value was used to determine k_c-values at three levels of subbase thickness (h_{sb} = 4, 8, and 12 in) and three levels of native subgrade support as follows:

Support	k (pci)	CBR (%)	Mr (psi)
Poor	125	2	3 000
Fair	315	5	7 500
Good	625	10	15 000

This study resulted in k_c-values for all possi-

Table 5. Summary of typical material SRs.

Material	Asphalt Thickness (in)	Dry (S_r < 60 percent) SCE	Dry (S_r < 60 percent) MCE	Wet (S_r > 85 percent) SCE	Wet (S_r > 85 percent) MCE
Base					
DGA	Thin (<5)	1.20	1.00	1.55	1.25
	Thick (≥5)	1.60	1.25	2.25	1.65
Crusher run	Thin (<5)	1.35	1.00	1.50	1.10
	Thick (≥5)	2.10	1.40	2.40	1.55
Slag	Thin (<5)	1.10	0.85	2.55	1.45
	Thick (≥5)	1.30	0.95	4.10	1.85
Subbase					
Sand/gravel	Thin (<5)	1.70	1.15	3.20	1.70
	Thick (≥5)	2.20	1.40	5.60	2.25

Notes: Values shown reflect SR at CBR_{sg} = 10 percent. Some material-physical condition combinations may vary significantly with CBR value.

Table 6. Typical M_r-values for subbase layer of rigid pavements.

Material	Dry (S_r < 60 percent)		Wet (S_r > 85 percent)	
	SCE	MCE	SCE	MCE
DGA-limestone	17 000	21 000	12 188	15 670
DGA-limestone-2	17 430	22 736	12 000	15 000
CR-6-crushed stone	12 000	18 000	9 237	13 195
CR-6-slag	20 306	32 490	9 097	14 621
Sand-aggregate blend	7 600	12 000	5 014	7 917
Bank-run gravel	8 705	13 930	3 300	5 778

ble combinations of material type (six), degree of saturation (two), percentage of compaction (two), subbase thickness (three), and subgrade support (three). These values have been summarized in both tabular and graphical form elsewhere (5).

Discussion of Results

By referring to Figure 1 and Table 6, it can be observed that the sensitivity of the subbase modulus for unbound granular materials is very slight on the final composite k_c-value (on the type of subbase layer). This fact was obviously reflected by the results obtained in this study. From a practical design viewpoint, a simple but reliable predictive equation of the k_c-value for unbound granular subbase materials was found to be as follows:

$$k_c\ (\text{pci}) = f_o\,(k_{sg} + 7.5h_{sb} - 20) \qquad \text{for } h_{sb} \geq 4.0 \text{ in} \tag{9}$$

where

k_{sg} = modulus of subgrade reaction (pci),
k_c = composite granular subbase-subgrade reaction value (pci),
h_{sb} = thickness of granular subbase (in), and
f_o = adjustment factor reflecting material type and in situ conditions.

In this equation, the typical f_o-values are shown below:

Subbase Material	fo-Value Dry-Modified	Wet-Standard
Crushed stone	1.00	0.95
Sand/gravel	0.95	0.90
Slag	1.08	0.92

In essence, the greatest influence on k_c is reflected by the foundation support value and thickness of the subbase layer. The maximum influence of material type, compactive effort, and/or degree of saturation on the k_c-value appears to be less than 10 percent for all cases considered.

SUMMARY AND CONCLUSIONS

The study presented in this report was based on resilient-modulus test results of granular materials obtained from an extensive laboratory study. Based on this analysis, the following conclusions were obtained.

1. An analysis of the University of Maryland M_r-test results indicated that the primary variables influencing the M_r-response of granular materials are the stress state, degree of saturation, and degree of compaction. The amount of fines (percent passing the No. 200 sieve) or gradation was also found to have an effect on M_r; however, it was relatively small and not considered as a primary variable in the study.

2. Several relationships (instead of a single M_r-relationship for each aggregate) that reflected the relative influence of the significant variables--stress state, saturation, and compaction--on the k_1 and k_2 (constants in $M_r = k_1\theta^{k_2}$) values were developed for six MSHA unbound granular materials investigated. These relationships were presented in Table 1.

3. The study of the factors influencing the structural layer coefficients a_i and SRs showed that degree of saturation (S_r), percent compaction (PC), subgrade CBR, and asphalt layer thickness were important parameters affecting their magnitude.

4. By using the $M_r = f(\theta)$ relationships and bulk-stress (θ) values developed in this study and the a_i and M_r correlations from NCHRP Report 128, typical layer coefficients (a_i) and SRs were developed for the MSHA granular materials investigated. Predictive equations for the structural layer coefficient values of these materials were developed in terms of the primary variables studied.

5. The final study phase dealt with determining composite modulus of subgrade reaction values (k_c) for the MSHA materials studied. A typical bulk-stress value of θ = 4 psi was found to be applicable based on elastic-layered studies for rigid pavement subbase layers. By using this, typical M_r-values were determined and the relationship between k_c and M_r found in NCHRP Report 128 was used to investigate the influence of variables on the composite modulus of subgrade reaction. It was found that the effect of material type and subbase modulus (for all combinations of saturation and compaction) on k_c is small when compared with the influence of the subgrade modulus and subbase layer thickness. As in the a_i study, simple predictive equations were developed in terms of the variables considered.

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