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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.

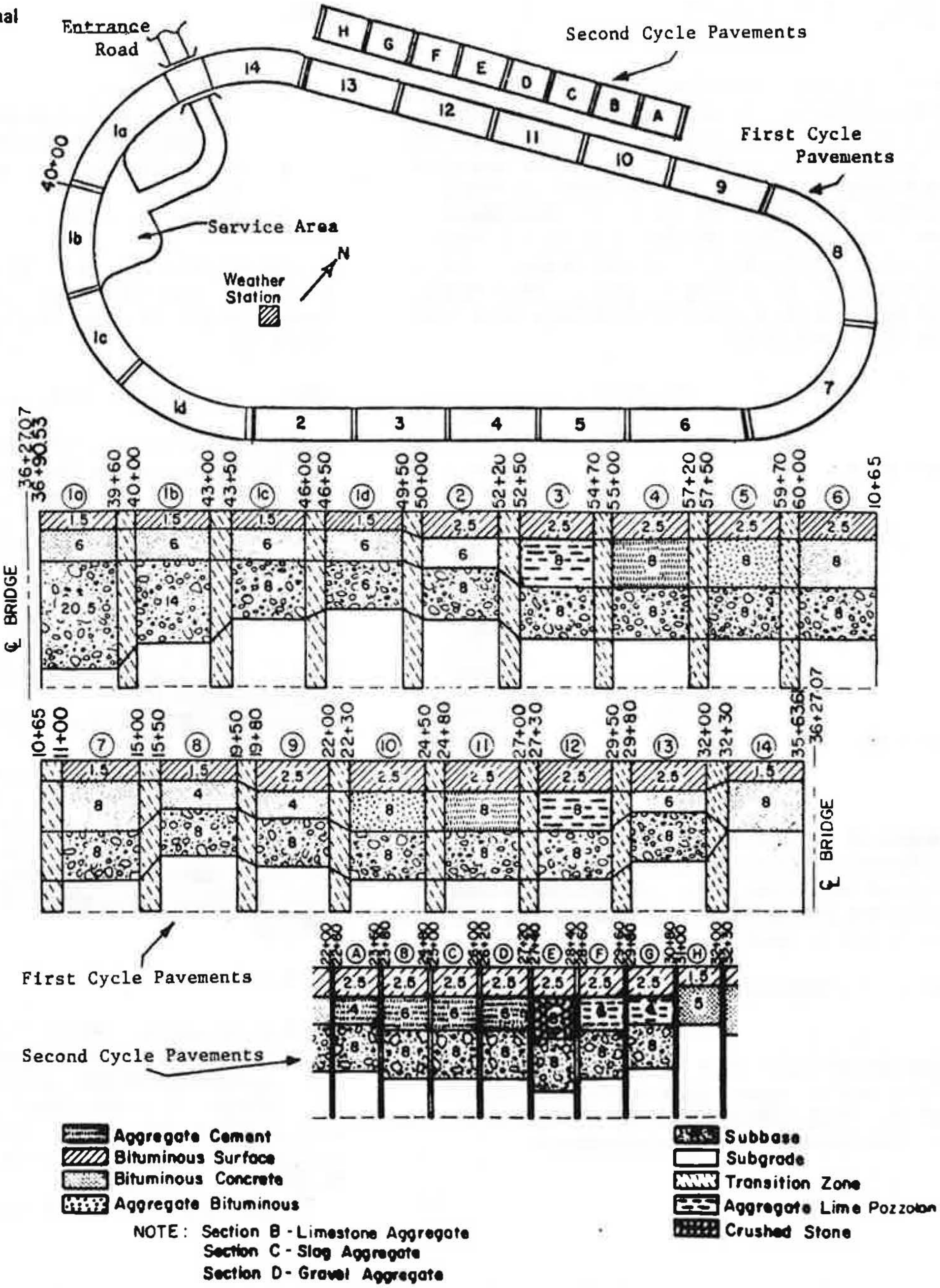
Two different approaches were taken in the evaluation of the structural coefficients of two stabilized base-course materials—namely, bituminous concrete and limestone aggregate cement. The first analysis was based on the use of performance data with analysis techniques similar to those used during the 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

Figure 1. Plan view and longitudinal profile of test track.



bituminous concrete and limestone aggregate-cement pavements are analyzed in this paper.

Field testing of pavement performance was conducted periodically. Rut depth was measured biweekly every 12.2 m (40 ft) in both wheel paths by using an A-frame that was attached to a 2.1-m (7-ft) long base channel. Surface cracking was surveyed and mapped biweekly. Surface roughness was measured in both wheel paths by using a MacBeth profilograph. The roughness factors obtained from the profilograph data were converted into present serviceability index (PSI) by using the following equations (since these equations are formulated in U.S. customary units, no SI equivalents are given):

$$PSI = 11.33 - 4.06 (\log RF) - 0.01 \sqrt{C+P} - 1.34 \overline{RD}^2 \quad (1)$$

$$RF = 63.267 + 1.0831 (R) \quad (2)$$

where

RF = Mays meter roughness factor,
C = area of cracking ($ft^2/1000\ ft^2$),
P = area of patching ($ft^2/1000\ ft^2$),
$\overline{RD}$ = average rut depth (in), and
R = profilograph readings (in/mile).

Equation 2 was developed by the Bureau of Materials, Testing, and Research of the Pennsylvania Department of Transportation (PennDOT).

In addition, surface deflections, pavement temperature, depth of frost penetration, weather data, and subgrade moisture were collected. Detailed information on field testing is available elsewhere (1).

PAVEMENT PERFORMANCE DATA

Results of the crack survey for bituminous concrete and limestone-aggregate-cement pavements are summarized in Table 1 in terms of the number of 80-kN [18 000-lbf (18-kip)] equivalent axle-load (EAL) applications when significant fatigue cracking was observed, the total length of class 1 cracks, and the total area of class 2 and class 3 cracks. The table below gives rut-depth data at 1 million EALs and the number of 80-kN EALs when 6.4-mm (0.25-in) rutting occurred. Numbers of EALs for sections 13, 4, and B were extrapolated from field data (1 mm = 0.04 in):

Base	Section	Rut Depth at 1 Million EALs (mm)	Number of EALs at 6.4-mm Rutting
Bituminous concrete	1A	1.8	1 700 000
	1B	2.5	1 600 000
	1C	3.6	1 400 000
	1D	5.1	1 140 000
	2	3.1	1 650 000
	6	3.8	1 520 000
	7	4.1	1 420 000
	8	14.0	640 000
	9	12.5	570 000
	13	4.8	1 210 000
	14	4.3	1 180 000
	H	>50	270 000
Limestone aggregate cement	4	1.3	2 300 000
	A	9.4	760 000
	B	2.8	1 800 000

The variation of PSI with 80-kN EAL applications is shown in Figure 2. Note that each PSI value represents the average of both wheel paths. Performance data for other pavements appear in papers by Wang and Kilareski elsewhere in this Record.

PAVEMENT PERFORMANCE MODEL

The mathematical model used to describe the serviceability trends is the same as that used at the AASHO Road Test (3). It is assumed that the serviceability loss is a power function of axle-load applications:

$$c_0 - p = (c_0 - c_1)(W/\rho)^{\beta} \qquad (3)$$

where

c_0 = initial PSI,
p = PSI at time t,
c_1 = terminal serviceability index (TSI) = 2.5 for the models reported here,
ρ = pavement life expressed in terms of 80-kN EALs, and
β = rate of change of serviceability loss.

Fitting the model to the observed (PSI and EAL) data points was a straightforward application of simple linear regression with a transformed version of Equation 3 (3):

$$\log[(c_0 - p)/(c_0 - c_1)] = \beta(\log W - \log \rho) \qquad (4)$$

where the terms are as described for Equation 3.

Two parameters (β and log ρ) were estimated for each test section as a result of fitting Equation 4. These two parameters were interpreted as the indicators of pavement performance for each section. The two indicators of pavement performance were instrumental in the calculation of structural coefficients, in which the performance approach described below was used.

STRUCTURAL COEFFICIENTS DETERMINED BY PERFORMANCE APPROACH

For relating pavement performance to design and load variables, the AASHTO analysis procedure assumed mathematical models to relate β and log ρ to layer thickness, type of axle, and magnitude of axle loads. In these mathematical models, the layer thicknesses were contained in the structural number (SN), as follows (4):

$$SN = a_1 D_1 + a_2 D_2 + a_3 D_3$$

where a_1, a_2, and a_3 = structural coefficients for the surface, base, and subbase, respectively, and D_1, D_2, D_3 layer thicknesses of the surface, base, and subbase, respectively. The structural coefficients were averaged partial regression coefficients that resulted from examining the change in β and log ρ across levels of D_1, D_2, and D_3.

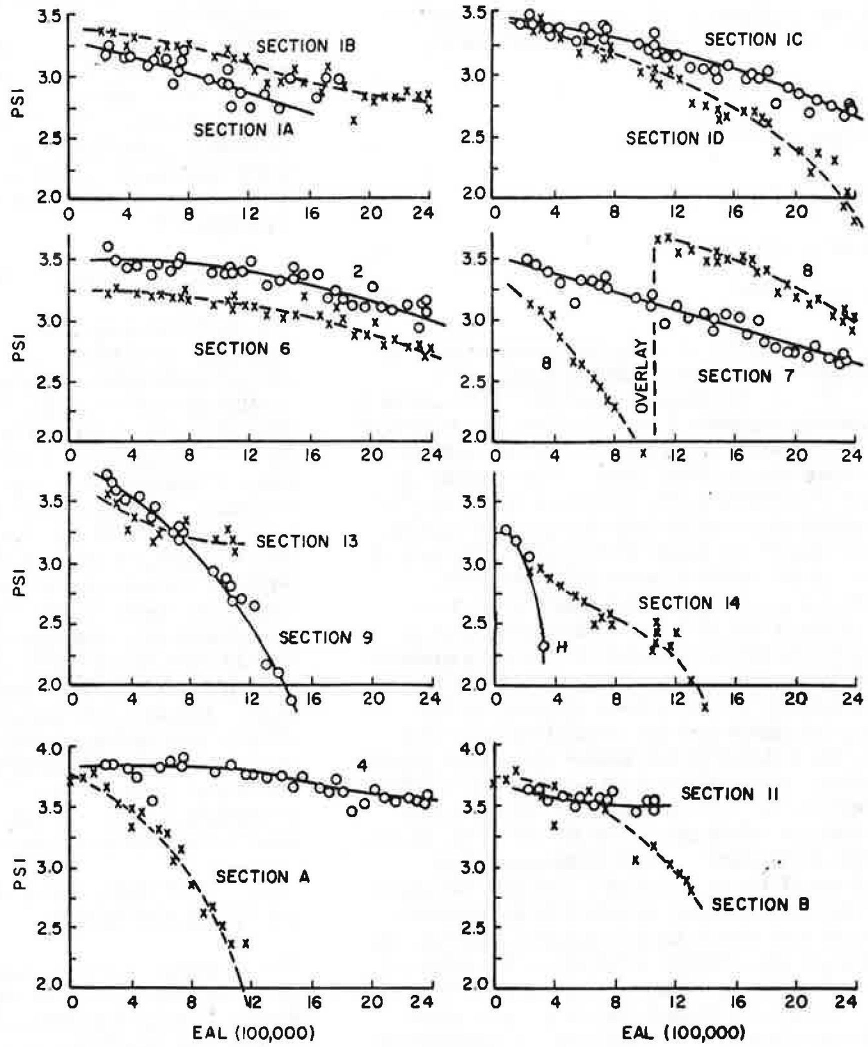
To determine structural coefficients in the current

Table 1. Results of crack survey.

Base	Section	Number of EALs at First Appearance of Significant Cracking (000 000s)	Amount of Cracking: Class 1 (m/km²)	Amount of Cracking: Classes 2 and 3 (m²/km²)
Bituminous concrete	1A	-	14 800	0[a]
	1B	-	13 200	0[a]
	1C	1.65	61 300	0[a]
	1D	1.45	85 900	26 000[a]
	2	2.35	56 400	0[a]
	6	-	4 900	0[a]
	7	2.38	8 900	0[a]
	8	0.386	0[b]	85 500[b]
	9	1.16	385 400	180 400[c]
	13	-	0[b]	0[b]
	14	0.906	47 600[d]	88 800[e]
	H	0.359	0[d]	133 300[d]
Limestone aggregate cement	4	2.3	195 800[a]	3 000[a]
	A	0.75	372 800[f]	94 200[f]
	B	-	32 900[f]	0[f]

Notes: 1 m/km² = 0.0003 ft/1000 ft²; 1 m²/km² = 0.001 ft²/1000 ft².
[a]As of May 31, 1978, EALs = 2 377 000.
[b]At the end of the first cycle of study, EALs = 1 082 000.
[c]Before overlay, EALs = 2 021 000.
[d]Before overlay, EALs = 405 000.
[e]Before overlay, EALs = 1 487 000.
[f]As of May 31, 1978, EALs = 1 303 000.

Figure 2. Performance data for pavements with bituminous concrete and limestone-aggregate-cement bases.



study, it was necessary to formulate a model that accommodated the differences between this study and the AASHO Road Test. The primary differences were (a) the use of two base-course materials; (b) the absence of load variables, since all axle loading was converted to 80-kN EAL; and (c) the need to determine if the base-course structural coefficients changed with base-course thickness.

The reduced model finally adopted to relate the pavement performance indicators to design variables (material type and thicknesses D_1 and D_2) was as follows (5):

$$Y_i = \lambda_0 + \lambda_1 M_i + \lambda_2 D_{1i} + \lambda_3 D_{2i}^2 + \lambda_4 (M_i \times D_{2i}) + \lambda_5 (M_i \times D_{2i}^2) + \epsilon_i \tag{6}$$

where

Y_i = β or $\log \rho$;
λ_i = partial regression coefficients;
M_i = base-course material type: 1 = bituminous concrete and 0 = aggregate cement;
D_{1i} and D_{2i} = thicknesses of wearing and base courses, respectively; and
ϵ_i = error terms ~ ind $N(0, \sigma^2)$.

Note that D_3 is constant and therefore was not used in the model and that the subbase structural coefficient was calculated in a separate analysis.

The detailed analysis procedure is outlined elsewhere (5). The analysis assumed that the effect of the design variables was the same (although opposite in algebraic sign) on both β and $\log \rho$. Accordingly, the reduced model (Equation 6) was fit alternately by using first β and then $\log \rho$ as the dependent variable in an iterative manner until the λ-coefficients closed to the same values when either β or $\log \rho$ was used as the dependent variable. All terms in Equation 6 were statistically significant at a 90 percent level of confidence except λ_2 (wearing-course structural coefficient). The term was left in the model because it is certainly reasonable to expect that D_1 has an effect on pavement performance. It was not possible to detect this effect as statistically significant because of the limited range of D_1 values [3.8-6.4 cm (1.5-2.5 in)] and the small number of observations in the data set (11). The presence of a significant nonlinear effect for D_2 (λ_3 and λ_5 in Equation 6) supports the conclusion that, for the materials and layer thicknesses included in this study, the base-course structural coefficient changes across levels of base-course layer thicknesses.

The results of fitting the model (Equation 6) yielded the following expression, which is referred to as KD only for identification purposes:

$$KD = -1.989M + 0.106D_1 + 0.020D_2^2 + 0.811 (M \times D_2) - 0.075 (M \times D_2^2)$$

where M = 1 for bituminous concrete base and 0 for aggregate cement base and D_1 and D_2 = thicknesses of surface and base courses, respectively. In addition, the two indicators of pavement performance were obtained as follows. For bituminous concrete base,

$$\beta = 1.0 + 8.954/(KD + 1)^{5.175} \tag{8a}$$

$$\rho = 1.262 \times 10^5 \, (KD + 1)^{4.279} \tag{8b}$$

For limestone-aggregate-cement base,

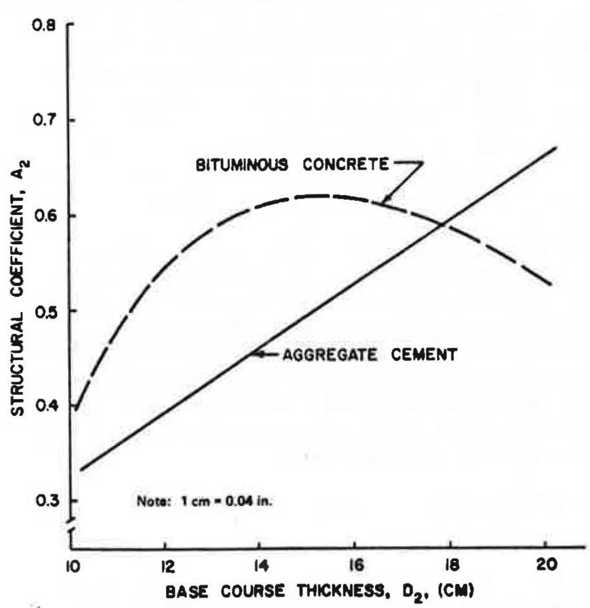
$$\beta = 1.0 + 9.204/(KD + 1)^{5.735} \tag{9a}$$

$$\rho = 1.018 \times 10^5 \, (KD + 1)^{4.668} \tag{9b}$$

Because, as mentioned before, the structural coefficient of the subbase was evaluated in a separate analysis, no subbase thickness is included in Equation 7. Thus, by equating Equation 7 with the first two terms of Equation 5, the structural coefficients of the surface and base courses can be determined. The results of the calculated structural coefficients are shown in Figure 3. The structural coefficient of aggregate cement increases throughout the range of the data while that of bituminous concrete peaks at about 15.2 cm (6 in).

The structural coefficients shown in Figure 3 are those obtained for a TSI of 2.5 and after normalizing the raw analysis results to a wearing-course structural coefficient of 0.44. This was necessary because any analysis will determine coefficients specific to that analysis; e.g., consider that the coefficient indicates the change in log ρ (logarithmic scale) with a unit change in layer thickness and that the ρ-values for the AASHO Road Test were in the EAL range of 10^5-10^6 whereas those in the current study are in the 10^6-10^7 EAL range. Therefore, the coefficients would differ even if an additional 2.5 cm (1 in) of material increased the pavement life by the same number of EALs in both studies. The coefficients that result from a specific analysis do, however, indicate the relative abilities of the different layers to contribute to the structural adequacy of the pavement. The relative magnitudes of the raw coefficients were maintained by multiplying the coefficients by the ratio of 0.44 to the raw a_1 value. This normalized the coefficients to a value in common use (a_1 = 0.44) and

Figure 3. Structural coefficients of base-course materials determined by performance approach.



facilitated interpretation of the base-course structural coefficients.

The subbase structural coefficient was calculated by using a similar but separate analysis of the performance data of three test sections whose only variable was subbase thickness [15.2, 20.3, and 35.6 cm (6.0, 8.0, and 14.0 in)]. Note that these three test sections contained base courses of bituminous concrete. The normalized value for a_3 supports the 0.11 value currently being used by PennDOT.

MATERIAL PROPERTIES

The material properties needed to analyze pavement response were modulus of elasticity and Poisson's ratio of each pavement constituent material. The elastic moduli of surface, bituminous concrete base, subbase, and subgrade materials have been reported elsewhere (6). The elastic modulus of limestone-aggregate-cement material was determined by using the same method as that used to determine bituminous concrete base material. It was found that the modulus of limestone aggregate cement did not vary significantly with the curing ages of the specimen, which ranged from two months to one year. In addition, the modulus was practically independent of confining pressures up to 0.21 MPa (30 lbf/in²), deviatoric stress up to 0.41 MPa (60 lbf/in²), and number of load applications up to 2000. The average modulus of limestone aggregate cement was approximately equal to 25 000 MPa (3.6 million lbf/in²). Poisson's ratio of aggregate cement was found to range from 0.15 to 0.22 according to other researchers (7, 8). Thus, a Poisson's ratio of 0.20 was used in the following response analysis.

PAVEMENT RESPONSE AND LIMITING CRITERIA

The response of the test pavements to traffic loading was analyzed for the climate condition that is most critical to pavement performance. The analysis was made by using an elastic-layer computer program and appropriate material properties. The computer program adopted was the BISAR program developed at Koninklijke Shell Laboratorium in Amsterdam.

The traffic loading used was an 80-kN EAL on dual wheels that had a tire pressure of 552 kPa (80 lbf/in²). The critical responses analyzed were maximum radial tensile strain in the surface and the base layers, maximum vertical compressive strain in the subgrade, and maximum surface deflection. These critical responses were considered because maximum tensile strain and maximum surface deflection are related to fatigue cracking whereas maximum compressive strain is associated with rutting. The pavement response analyzed was related to pavement performance to establish limiting strain criteria.

Since a rut of 6.4 mm (0.25 in) has been widely used for developing limiting strain criteria (9-11), the 80-kN EAL required to produce 6.4-mm rutting for each section concerned is related to the maximum compressive strain at the top of the subgrade (see Figure 4). Note that the relation for the bituminous concrete base material has been given elsewhere (6). Also shown are the results determined at the San Diego test road (9) and the criteria developed by Monismith and McLean (10) and Dormon and Metcalf (11).

Figure 4 demonstrates that, for the same subgrade compressive strain, the pavement that contains bituminous concrete base requires a greater number of EALs than that with aggregate-cement base in order to reach the same rutting of 6.4 mm. This is rather

Figure 4. Maximum compressive strain versus EAL at 6.4-mm rut depth.

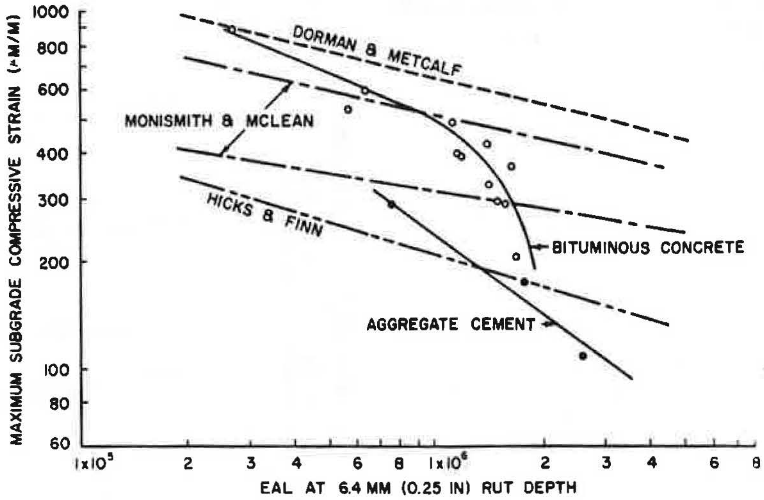
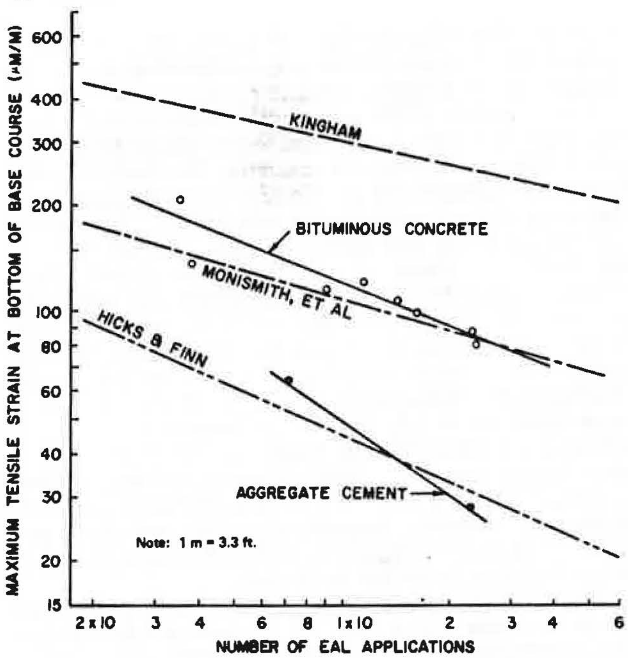


Figure 5. Maximum tensile strain at bottom of base course versus EAL applications.



unexpected, for it is generally believed that under the same loading condition bituminous concrete base material will undergo more plastic deformation than aggregate-cement material, so that the pavement that contains bituminous concrete base course will display greater rutting. The possible reasons for this effect are yet to be investigated. Figure 4 indicates that the limiting compressive strain at 1 million EALs equals 450 μm/m (450 $\times 10^{-6}$ in/in) and 230 μm/m (230 $\times 10^{-6}$ in/in) for the bituminous concrete and limestone aggregate cement, respectively. One million EALs was adopted in the development of limiting criteria because that figure is widely associated with 20-year pavement life.

In Figure 5, the maximum tensile strain at the bottom of the base course is related to EAL based on the number of axle loadings at the first appearance of significant surface cracking. Note again that the relation for bituminous concrete base material has been established for field data collected up to July 1976. Results obtained by other researchers (9, 12, 13) are also included in the figure. The figure shows that the limiting tensile strain at 1 million EALs equals 120 μm/m (120 $\times 10^{-6}$ in/in) for the pavements that contain bituminous concrete base and 45 μm/m (45 $\times 10^{-6}$ in/in) for the limestone-aggregate-cement base.

Figure 6 summarizes the relation between the number of EALs at the first appearance of significant surface cracking and the maximum pavement surface deflection. Results obtained by others (9, 13, 14) are also included in the figure for comparison. Figure 6 indicates that the limiting maximum surface deflection at 1 million EALs equals 0.51 mm (0.020 in) for the pavements that contain bituminous concrete base and 0.30 mm (0.012 in) for the limestone-aggregate-cement base.

STRUCTURAL COEFFICIENTS DETERMINED BY LIMITING CRITERIA

The structural coefficients of the bituminous concrete and limestone-aggregate-cement base materials were computed by using the following basic equation developed from the AASHO Road Test (3):

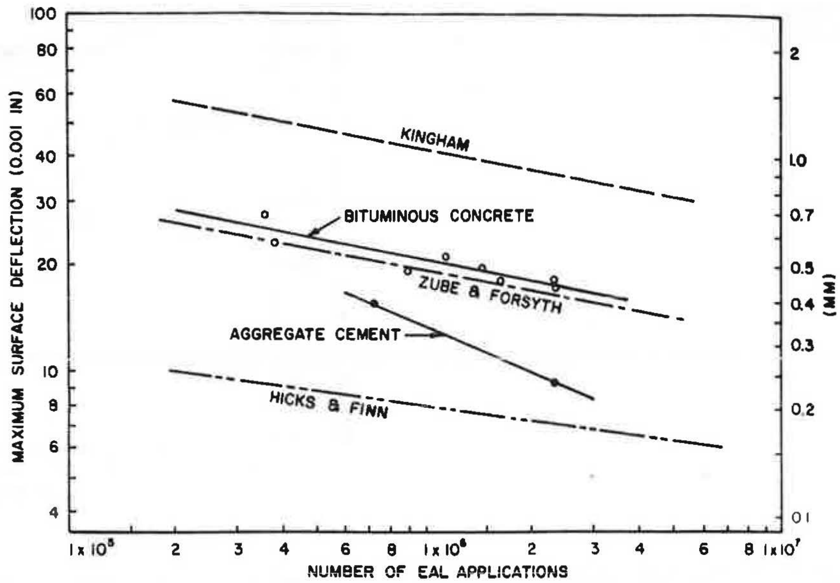
$$\rho = 0.64\,(SN + 1)^{9.36}$$

where ρ = EALs at failure and SN = structural number as defined in Equation 5.

In the computation, the structural coefficient of 0.44, which was originally developed from the AASHO Road Test, was used for the bituminous concrete surface material. Details of the computation procedure are available elsewhere (6). The structural coefficients determined are shown in Figure 7 (6) for the bituminous concrete base and in Figure 8 for the aggregate-cement base material. The figures also show some results obtained from the performance approach; these are included for later discussion.

Figures 7 and 8 demonstrate that the structural coefficients of the base and subbase materials vary with the thicknesses of surface, base, and subbase layers for the conditions studied. The structural coefficient of aggregate cement, however, does not vary with the base-course thickness as much as that of the bituminous concrete base material. The structural coefficient of the subbase material determined from the pavements that contain bituminous concrete base course fluctuates around 0.10. This value is very close to that originally developed at the AASHO Road Test (3) for a sandy gravel

Figure 6. Maximum surface deflection versus EAL applications.



subbase. The structural coefficient of the subbase material determined from the pavements that contain aggregate-cement base course, however, fluctuates around 0.04. This deviation could be attributed to the difference in the relative stiffness between the base and subbase layers. The resilient moduli given earlier for the base and subbase materials indicate that the relative stiffness between the aggregate cement base and the subbase is considerably greater than that between the bituminous concrete base and the subbase materials. Since the layer-thickness combinations that satisfy the limiting criteria depend on the relative stiffness of the constituent layers, the effect on the base thickness of a unit change in the subbase thickness is less significant for the system that contains a base course much stiffer than the subbase course. As a consequence, the structural coefficient of the subbase material evaluated with the aggregate-cement base was lower than that evaluated with the bituminous concrete base. A similar effect of the stiffness of one layer on the structural coefficient of another layer has also been observed by VanTil and others (15).

DISCUSSION OF RESULTS

According to the findings of the AASHO Road Test and Equation 10, pavement life in terms of 80-kN EAL can be expressed as a function of a structural number. Based on this equation, the SNs required for pavement lives of 1 million, 2 million, and 3 million EALs are 3.59, 3.94, and 4.16, respectively. The performance analysis for pavements at the Pennsylvania Transportation Research Facility resulted in two different equations for the two stabilized base-course materials: Equation 8b for bituminous concrete and Equation 9b for aggregate cement. Note that in this analysis the SN is equal to the sum of KD and a_3D_3 (the subbase structural coefficient times the subbase layer thickness). According to these equations, the SNs required for pavement lives of 1 million, 2 million, and 3 million EALs are 3.45, 4.66, and 5.45, respectively, for bituminous concrete and 3.50, 4.57, and 5.28, respectively, for aggregate cement. The results for 1 million EALs are in good agreement with the AASHTO result, whereas those for 2 million and 3 million EALs are slightly higher. Note that the SNs required for different EALs for the two stabilized base-course materials studied are nearly the same; this suggests that for a given pavement life the SNs required are practically independent of the type of base-course material used in the pavement.

A comparison of the structural coefficients of the base-course materials determined by using the two different analysis approaches—namely, the AASHTO performance approach and the limiting-criteria approach—can be made for a common pavement life. The limiting-criteria approach was used for a pavement life of 1 million EALs. From the performance approach, the SN required for the bituminous concrete pavement with a pavement life of 1 million EALs is 3.45, as mentioned earlier. Assuming this SN, a relation between the layer thicknesses of surface and base is given by Equation 7. The base layer thicknesses required for each surface layer thickness of 3.8, 6.4, and 8.9 cm (1.5, 2.5, and 3.5 in) were determined by using this equation. The base-course structural coefficients were then calculated by using 0.44 and 0.11 as the structural coefficients of surface and subbase, respectively. The calculated results are plotted in Figure 7, and a good agreement between the two approaches is shown.

For the aggregate-cement pavement with a pavement life of 1 million EALs, the SN required is 3.50. Base-course structural coefficients were calculated for the same three surface layer thicknesses mentioned above by using Equation 7. In the calculation, a surface-layer structural coefficient of 0.44 was used. However, two different values were used for the subbase structural coefficient: 0.11 and 0.04. The 0.11 value was determined in the performance approach from three pavement sections that contained bituminous concrete base. This value was also produced in the evaluation of bituminous concrete pavement by use of the limiting-criteria approach. As mentioned before, because of the considerable difference in the base-course stiffness of the aggregate cement and the bituminous concrete, the subbase structural coefficient found in the evaluation of aggregate-cement pavements by use of the limiting-criteria approach was about 0.04 rather than 0.11. Because a similar effect has been reported by VanTil and others (15), it is reasonable to use 0.04 as the subbase structural coefficient for this comparison. The calculated results for a_3 = 0.04 are shown in Figure 8. Although the data points fall outside the range of the limiting approach for base layer thickness, a fairly good agreement between the two approaches is obtained.

Figure 7. Structural coefficients of bituminous concrete base and limestone subbase.

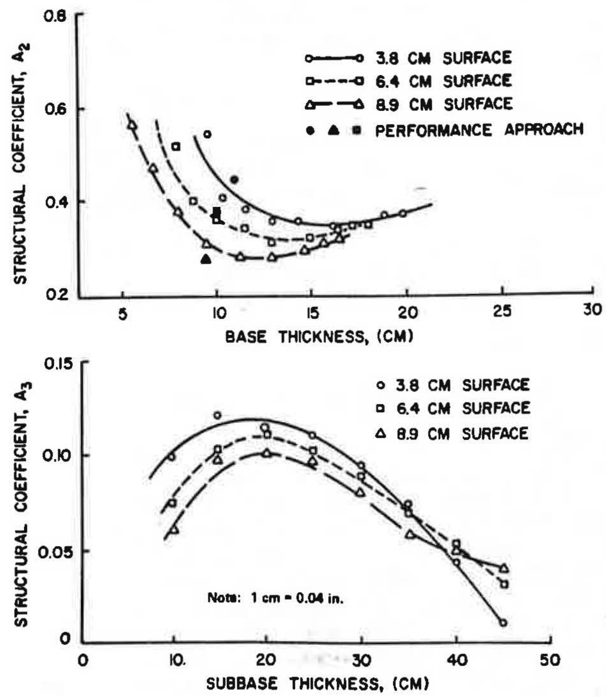
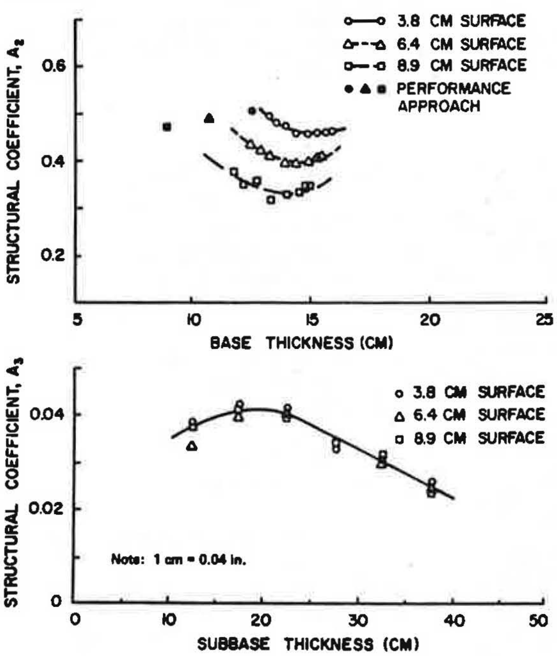
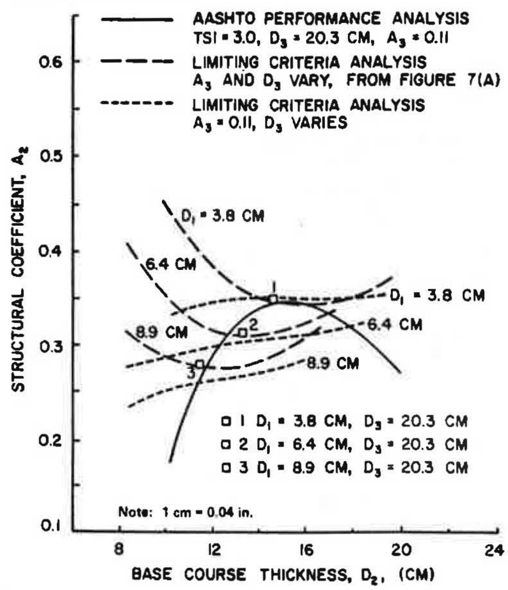


Figure 8. Structural coefficients of aggregate cement base and limestone subbase.



The performance approach can also roughly control the estimated pavement lives by specifying the TSI. For the pavement sections at the research facility, the TSI level that resulted in pavement lives closest to 1 million EALs was about 3.0. The structural coefficients calculated by using this TSI level are compared with the results of the limiting-criteria analysis in Figures 9 and 10. One important point that should be reiterated here is that the performance analysis was performed

Figure 9. Comparison of structural coefficients for bituminous concrete.



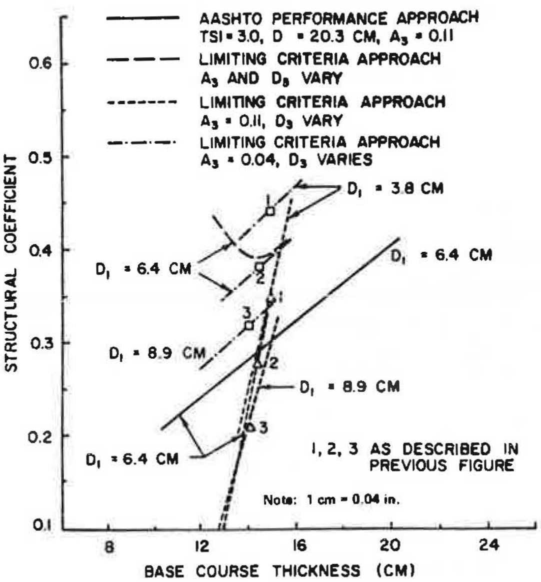
for a constant subbase layer thickness [D_3 = 20.3 cm (8 in)] and a constant subbase structural coefficient (a_3 = 0.11), whereas the limiting-criteria approach allowed both a_3 and D_3 to vary. Because of this basic difference, a direct comparison requires common a_3 and D_3 values.

For the bituminous concrete sections, three data points that have conditions common to both methods of analysis are shown in Figure 9. A very good agreement between the two different approaches is seen. It was valid to plot the results of the limiting-criteria approach for all three wearing thicknesses because no surface - base-course thickness interaction could be detected in the performance analysis. This implied that the base-course structural coefficient is independent of the surface layer thickness (within the range of the field data). This contradicts the findings of the limiting-criteria approach, which shows base-course structural coefficients to vary with surface layer thickness. The interpretation is that this effect was not sufficiently pronounced in the field data to be detected as statistically significant since the surface layer thickness only occurred at two levels in the field and the field data composed a very limited data set.

For aggregate cement, two values of the subbase structural coefficient a_3 were used for the reason stated earlier. Figure 10 shows that, when a constant value of a_3 is used, the structural coefficient of the aggregate cement (a_2) increases linearly with increasing thickness of the base course (D_2) regardless of the method of analysis used. The results of the limiting-criteria analysis indicate that, although the rate of increase of a_2 with D_2 for a_3 = 0.04 is almost equal to that obtained from the performance approach, the rate of increase for a_3 = 0.11 is much greater. Furthermore, a_2 varies considerably with surface layer thickness D_1 when a_3 = 0.04. When a_3 = 0.11, however, the effect of D_1 on a_2 is not as significant.

It should be noted that the only variable associated with the aggregate-cement base course in the field experimental design was the thickness of the aggregate-cement base course. The varying subbase thickness and varying surface layer thicknesses all occurred in sections that contained bituminous concrete. Accordingly, all conclusions of the performance analysis concerning

Figure 10. Comparison of structural coefficients for aggregate cement.



the surface course, surface × base interactions, and subbase were generated from the data of the bituminous concrete base. Of necessity, these results were "borrowed" by the analysis of the aggregate cement to identify surface and subbase structural coefficients to be used with the aggregate-cement structural coefficients. As noted before, in the performance analysis the thicknesses of the surface and subbase layers used with the aggregate-cement base course were 6.4 and 20.3 cm (2.5 and 8 in), respectively. The limiting-criteria data points that correspond to these layer thicknesses are identified by the number 2 in Figure 10. It is seen that a better agreement between the two approaches was obtained for $a_3 = 0.11$ simply because it was used in the performance analysis.

Both Figures 9 and 10 demonstrate that, for the two types of base-course materials studied, the base-course structural coefficient a_2 first decreases and then increases with increasing base-course thickness D_2, when both subbase structural coefficient a_3 and subbase layer thickness D_3 are allowed to vary. When a_3 is kept constant, however, a_2 increases with an increase in D_2. Furthermore, when both a_3 and D_3 are kept constant, a_2 also increases with D_2 for the aggregate-cement base material whereas, for the bituminous concrete base material, a_2 first increases and then decreases with increasing D_2 within the range of conditions studied.

The preceding results indicate that a_2 varies with many factors, such as the thickness and stiffness of each pavement constituent layer, structural coefficients of other pavement layers, and pavement life. This complex dependency of a_2 not only makes it difficult to assign a constant and unique value of a_2 for a base-course material but also complicates practical applications of a_2 in pavement design.

SUMMARY AND CONCLUSIONS

The structural coefficients of two stabilized base-course materials—bituminous concrete and aggregate cement—were evaluated. Two different methods of analysis were used: the AASHTO performance analysis and the limiting-criteria approach. The AASHTO performance analysis was based on the field performance of 11 bituminous concrete pavements and three aggregate-cement pavements, whereas the limiting-criteria approach was based on the maximum tensile strain at the bottom of the base course, the maximum compressive strain at the top of the subgrade, and the maximum pavement surface deflection.

The results of the evaluation showed good agreement between the two methods of analysis. It was found that the structural coefficients of the base-course materials varied with many factors, such as the thickness and stiffness of each pavement constituent layer, structural coefficients of other pavement layers, and pavement life. This complex dependency makes it very difficult to assign a constant and unique value to the structural coefficient of a base-course material.

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Behavior and Performance of Aggregate-Cement Pavements

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Field performance of six aggregate-cement pavements at the Pennsylvania Transportation Research Facility was evaluated based on their rutting and cracking behavior and values of present serviceability index. Three types of aggregate were used in the aggregate-cement bases: limestone, slag, and gravel. The results of an analysis of relative performance among the three types of aggregate-cement materials are presented. The pavement response to an 80-kN [18 000-lbf (18-kip)] equivalent single-axle load was analyzed by using an elastic-layer computer program. The pavement response was related with the field performance data to establish limiting criteria. Among the three types of aggregate studied, limestone possesses the greatest strength and performs best in terms of rutting, cracking, and change in serviceability index. Gravel possesses greater compressive strength but smaller resilient modulus and fatigue strength than slag. The pavement with a base of slag aggregate cement performs better than that with a base of gravel aggregate cement. The limiting criteria developed were a maximum compressive strain of 230 μm/m (0.000 230 in/in) for limestone aggregate and 180 μm/m (0.000 180 in/in) for both slag and gravel aggregates, a maximum tensile strain of 45 μm/m (0.000 45 in/in), and a maximum pavement surface deflection of 0.30 mm (0.012 in) for all three types of aggregate studied. With these limiting criteria, it would be possible to design aggregate-cement pavements to withstand 1 million 80-kN equivalent axle-load applications without significant surface cracking or excessive rutting.

The use of cement-stabilized material in pavement structures has increased steadily over the past decades. Most available procedures for thickness design of cement-stabilized layers are largely based on empirical rules. Recognizing the need for developing an improved method of thickness design, the Committee on Structural Design of Roadways of the American Society of Civil Engineers identified steps required for achieving this goal (1). Among the steps identified are the establishment of failure criteria and performance studies in the field. A number of studies have provided information relative to these steps, including those by Bofinger (2), Shen and Mitchell (3), Larsen and Nussbaum (4), Larsen (5), Mitchell and Freitag (6), and Nussbaum and Larsen (7). However, most of these studies dealt with cement-stabilized soils; very few studies on cement-stabilized aggregates are currently available.

An investigation of the field performance of various stabilized base-course materials was conducted at the Pennsylvania Transportation Research Facility at Pennsylvania State University. The stabilized materials studied were aggregate cement, bituminous concrete, aggregate-lime-pozzolan, and aggregate-bituminous. Three types of aggregate were used in the aggregate-cement material. The performance of bituminous concrete and aggregate-lime-pozzolan pavements has been discussed elsewhere (8, 9). This paper presents the results of a performance evaluation for pavements that contain aggregate-cement base courses. Limiting strain and limiting deflection criteria are developed from field performance data and pavement response. The results provide information that is useful in the steps identified above.

AGGREGATE-CEMENT MATERIAL

The aggregate-cement base material was composed of six percent by weight of type 1 portland cement and 94 percent by weight of aggregate. The mix design was determined by the Bureau of Materials, Testing, and Research of the Pennsylvania Department of Transportation (PennDOT).

Three types of aggregate were used: crushed limestone, gravel, and slag. The limestone and gravel are natural to central Pennsylvania. The slag was a blast-furnace slag obtained from Johnstown, Pennsylvania. Some basic characteristics of the slag are summarized