traffic engineers in gaining access to these records (6).

Further research into this area is suggested. Studies similar to the one described here should be performed in other regions of the country. Differences in laws from state to state probably require that each state develop its own background data.

The findings of this study are based principally on the West Virginia Court of Claims Reports. Although an unknown number of dubious claims are probably made, the results are believed to be reasonably accurate for the less serious accidents. On the other hand, the cases found in the South Eastern Reporter may be unrepresentative in that they refer only to the cases appealed from at least one (lower) court. Because this group of cases covers a fuller range, including the more serious accidents, and since the records contain much more detail, it would appear desirable to use the original court cases as a data source. Thus, the feasibility of using original court cases as a data source should also be examined.

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Determination of Structural Equivalency Factors of Recycled Layers By Using Field Data

ADRIAAN VAN WYK, ELDON J. YODER, AND LEONARD E. WOOD

The recycling of pavement materials is an effective cost- and energy-efficient method of reconstruction. Good results can and have been obtained with recycling. Since more than 90 percent of all hard surface roads and streets in the United States are composed of asphalt mixes, the use of recycled asphalt layers in reconstructed pavements can be extensive. Cold recycling is especially suitable for use on low-volume roads due to the lower cost and usually assumed lower strength compared with hot recycling. A value for the strength of the recycled layer or the structural coefficient is important in order to design the pavement. Underdesign can lead to premature failure and corresponding high maintenance costs. Overdesign, on the other hand, can lead to the ineffective use of available funds. Asphalt cement and emulsified asphalt have traditionally been used as a binder in recycling. Another promising binder is foamed asphalt. The latter has the advantage that it can be mixed at lower temperatures than asphalt cement and it does not need curing like emulsified asphalt. During the summer of 1981, the Indiana Department of Highways built an experimental section of 9 miles (15 km) by using asphalt emulsion and foamed asphalt in cold recycling on a low-volume road. Before, during, and after construction, various tests were conducted to determine the properties of the pavement layer to be used in the determination of the structural equivalency factors of the recycled layers. These included in situ California-bearing-ratio tests, the testing of 4-in (102-mm) recycled base-course cores taken from the pavement, and Dynaflect deflection measurements. Based on layer properties obtained in these tests and pavement deflections, an elastic-layer computer program was used to determine the structural equivalency factors. The elastic moduli of the pavement layers were determined from Dynaflect deflection measurements taken at three different times. The pavement sections determined were used to calculate structural coefficients. Various criteria were used. The methodology used to determine the elastic moduli of the pavement layers and to calculate the structural coefficients is described in this paper. A wide range of coefficient values was determined with the different criteria. A range of values was selected that can be used in design.

The energy crisis in 1973 stimulated the search for alternatives to conventional pavement construction methods. Although used since the 1930s, the recycling of pavements, especially asphalt pavements, became a feasible alternative to rehabilitation and reconstruction. During the past few years, emphasis has shifted from the saving of energy to the more effective use of highway funds. The construction costs of highways have more than doubled since 1973 (1), whereas the motor fuel tax revenue declined from 63 percent in 1970 to 55 percent in 1980 (2). Emphasis has moved further, from the construction of new highways to the maintenance of existing highways. More than 90 percent of all paved roads in the United States used bituminous surfaces (3). A large percentage of all the paved roads need resurfacing or reconstruction.

Recycling has the potential to help solve the problem by saving expensive energy and pavement materials and by reducing costs, if it performs as expected. Studies (4-6) have shown that asphalt pavements can be recycled and reach a strength of equal to or better than the conventional mixture in some instances. These studies were based primarily on laboratory test results. The performance of recycled layers has not been verified beyond doubt through field applications. This is one of the reasons why recycling has not been used more outside the United States (7).

The problem is even more profound with cold recycling and for low-volume roads. The control of the mixing process and the compaction procedure is usually less stringent for cold recycling on low-volume roads than that on high-volume roads. It is difficult to simulate these conditions in the laboratory. The recycled layer on low-volume roads further contributes to the largest portion of the strength of the pavement. The base course on low-volume roads is usually covered with thin asphalt-concrete or portland-cement layers or with another type of surface treatment. The selection of the structural coefficient is therefore of major concern in the design if the design method of the American Association of State Highway and Transportation Officials (AASHTO) is used. If a coefficient lower than the actual value is used in the design, unnecessary funds would have been spent in building a pavement thicker than necessary. On the other hand, if a coefficient higher than the actual value is used, the pavement will fail prematurely and require unscheduled maintenance. Both of these cases will nullify the benefits of recycling. The magnitude of change in the number of 18 000-lbf (80-kN) equivalent single axle loads (ESALs) to a terminal serviceability of 2.0 due to changes in the structural coefficient of the base course (or recycled layer) for this typical low-volume road pavement is shown below. The AASHTO design method is used by 33 states in the United States (8). The tabulation below is obtained from the AASHTO equation by using the following values: regional factor (R) = 1.1; soil-support value (S) = 5; and terminal serviceability value (pt) = 2.0. Surface layer: D_1 = 1 in; a_1 = 0.44. Base course: D_1 = 5 in; a_1 varies. Granular subbase: D_1 = 5 in; a_1 = 0.10. 1 lbf = 0.004 kN; 1 in = 25.4 mm.

a	SN	Wt (ESAL)
0.10	1.45	14 000
0.15	1.70	35 000
0.20	1.95	80 000
0.25	2.20	165 000
0.30	2.45	325 000
0.35	2.70	610 000

DEVELOPMENT IN STRUCTURAL COEFFICIENTS

The structural number (SN) has been defined by the AASHTO Interim Guide as an index number derived from an analysis of traffic roadbed soil conditions and regional factors that may be converted to the thickness of various flexible-pavement layers through the use of suitable layer coefficients related to the type of material being used in each layer of the pavement structure. The layer coefficient (designated a_1, a_2, and a_3 for surface, base, and subbase, respectively) is the empirical relationship between the SNs for a pavement structure and layer thickness, which expresses the relative ability of a material to function as a structural component (9). The relationship is

$$SN = a_1 D_1 + a_2 D_2 + a_3 D_3 \tag{1}$$

where D_1, D_2, and D_3 are the thicknesses of the surface, base, and subbase layers, respectively. Structural coefficients and equivalency factors are essentially the same. Both compare the performance of the material, based on some criterion, to the performance of a standard material, usually the asphalt-concrete surface used in the AASHO Road Test. The first is a ratio of the reference coefficient of 0.44 and the second is a thickness equivalent to 1 in (25 mm) of the reference material.

Structural coefficients were developed as a result of the AASHO Road Test conducted during 1958 to 1960. The coefficients were developed for pavement material, load, and environmental conditions used in the road test. Although these coefficients could be and have been used, the Interim Guide for design stated that they should be used with care for materials other than those used in the AASHO Road Test. The structural coefficients had to be expanded through theoretical studies, satellite studies, field tests, and laboratory tests (10). Various studies have been conducted since the road test to verify the coefficients or to develop new coefficients for different materials and conditions. The structural coefficients are not only dependent on the material and environmental conditions (e.g., moisture content), but also on the pavement temperature, surrounding layer thicknesses, loading intensities, material stiffnesses, and fatigue characteristics. It is not possible to use a single coefficient for a given material under all conditions (10).

DETERMINATION OF STRUCTURAL COEFFICIENTS

This paper describes an attempt that has been made to develop structural coefficients of a recycled layer, in this case a foamed-asphalt recycled layer, based on field data. The information was obtained from an experimental pavement section constructed during the summer and fall of 1981. The section, in which a foamed-asphalt recycled layer of 5.5 in (140 mm) was used as a base course, has a length of approximately 4.2 miles (6.72 km). The experimental section is located on State Road 16 approximately 40 miles (64 km) north of West Lafayette, Indiana. State Road 16 is a low-volume road that has a traffic volume of 550 vehicles/day in both directions and 18 percent trucks as counted in 1978. The foamed-asphalt recycled base course replaced the initial pavement asphalt layer of a little more than 5.8 in (145 mm) on average. The recycled layer was cold mixed at a central plant and placed in two layers. The base course was overlaid by a hot-mix asphalt-concrete layer of 1.25 in (32 mm). The construction of this experimental section is described in detail in another paper (11).

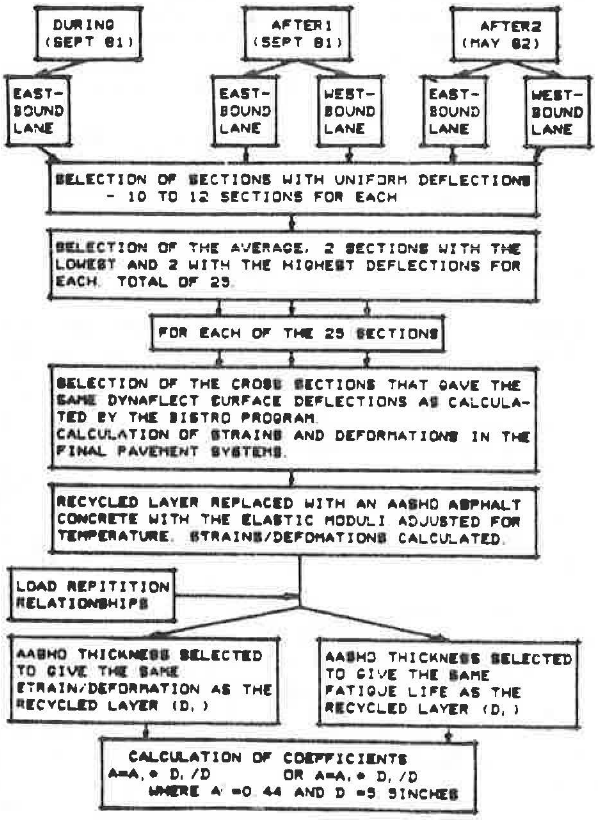
Tests were conducted before and during construction to determine the in situ strength of the subgrade by means of the dynamic cone penetrometer (DCP) and the thicknesses of the underlying layers. Laboratory specimens were also compacted from samples taken from the recycled layer during construction. These specimens and core specimens taken after construction were used to determine, among other things, the resilient modulus of the recycled layer and its behavior with curing time and temperature changes. The main source of information regarding the pavement characteristics was the Dynaflect deflection measurements taken during construction on one lane and twice after construction on both lanes.

The Dynaflect is a nondestructive testing device. An oscillating load with a peak of 1000 lb (455 kg) is applied through two steel wheels to the pavement. The deflections at five different positions on the pavement surface are measured by geophones. The geophones are oriented perpendicular to the axis of the wheels evenly spaced 12 in (304 mm) apart; the first one is located between the wheels.

The deflection measurements taken during construction were made after the compaction of the recycled base course and before the placement of the surface layer. These deflections were taken to represent the pavement one day after construction and will be designated as DURING. The first set of deflection measurements taken after construction was made approximately 12 days after construction and will be designated AFTER 1. Both the DURING and AFTER 1 measurements were taken in the fall. The second set of deflection measurements taken after construction, designated as AFTER 2, was made approximately 250 days (8 months) after construction in the spring.

The deflection measurements taken at the three different times were used to determine the struc-

Figure 1. Procedure for determination of structural coefficients.



tural coefficients as shown in Figure 1.

SELECTION OF PAVEMENT CROSS SECTIONS

For the granular layer, the thickness from the in situ DCP tests was used. A thin layer of the initial pavement remained on the granular subbase after the milling operation. The average thickness of this layer was approximately 1 in. The resilient moduli determined from the laboratory tests were used only as first-estimation values, since the curing times and compaction efforts did not compare exactly with those of the pavement layers during the deflection measurements. It was felt that the in situ conditions at the time of measurement could be better represented by the deflections. The resilient moduli determined with the deflections turned out to be compatible with those determined from laboratory tests for certain curing times.

The Dynaflect deflections taken at the three different times were used to divide each set of measurements on each lane into sections with uniform deflections. From each of these five sets of readings, five sections were chosen that represented the two sections with the highest deflections, the two with the lowest deflections, and the average. Each of these sections was at least 0.2 mile (320 m) long. As expected, the positions of the low- and high-deflection sections corresponded with the low- and high-deflection sections taken at different times but on the same lane.

These 25 different cross sections were simulated by pavement cross sections that would give the same deflections and deflection basin when analyzed by the BISTRO program, which is an elastic-layer program. It can be used to calculate stresses, strains, and deformations in the pavement for a large number of layers and different wheel configurations. The program does not take the stress sensitivity of the granular and subgrade material into consideration. Adjustments had to be made to the respective elastic moduli manually after each run. No relationships were available for Indiana pavement materials to be used to calculate the resilient modulus from the bulk or deviator stress. The following relationships were selected from the literature and adjusted slightly (5):

Granular subbase:

$$\text{Fall: } M_R = 5000\theta^{0.4}$$
$$\text{Spring: } M_R = 4500\theta^{0.4} \qquad (2)$$

Subgrade:

$$\text{Fall: } M_R = 27\,000\sigma_d^{-1.1}$$
$$\text{Spring: } M_R = 15\,000\sigma_d^{-1.1} \qquad (3)$$

where

M_R = resilient modulus,
θ = bulk stress, and
σ_d = deviator stress.

The in situ California-bearing-ratio (CBR) values could not be used since they have not been correlated with stress-sensitive resilient moduli. The bulk stress was only calculated in the center of the granular layer, although the elasticity is nonlinear. The differences between moduli calculated in this manner and moduli calculated at different positions and then averaged were small. The same conclusion was reached in an independent study (12). The elastic moduli of the remaining initial pavement layer were assumed to be between 10 000 and 50 000 psi (69 and 345 MPa). The condition of this layer was not known after the milling operation. The elastic modulus of the surface layer from laboratory-compacted samples was 300 000 psi (2070 MPa). The elastic moduli of the surface layer were expected to be lower than that value due to the short curing time and high pavement temperatures in the appropriate cases. The ranges of elastic moduli, layer thicknesses, and Poisson's ratios used are summarized in Table 1. Figure 2 displays the layer and load configurations.

The load applied in the BISTRO program is the load used for standard Benkelman-beam deflections, which are the same as those calculated by the BISTRO program.

Various methods have been suggested to calculate material properties, mostly the subgrade elastic moduli, from deflection measurements. None of these were particularly useful in the study, although they all have their particular application. The procedures were as follows:

1. Overlays were prepared by plotting the deflections of the 25 selected pavement sections on transparent paper.

2. A large number of deflection basins were plotted from the BISTRO results for low, intermediate, and high elastic moduli for all the layers. Figure 3 depicts some of these basins. The deflections at the third, fourth, and fifth sensor positions were virtually insensitive to changes in the elastic moduli of the overlying layers for the ranges used in this study and subgrade moduli of more than 5000 psi (34.5 MPa).

3. Initial pavement sections were then selected by comparing the deflection basins of various combinations of elastic moduli with the actual deflection basins. The subgrade moduli could be chosen almost

Table 1. Pavement cross-section characteristics.

Layer	Poisson's Ratio	Elastic Modulus (psi)	Thickness (in)	Description of Layer
1	0.35	1 200 000–250 000[a]	1.25	Asphalt-concrete surface
2	0.35	14 000–13 000	5.50	Foamed-asphalt recycled
3	0.35	10 000–50 000[a]	1.00	Remaining initial pavement
4	0.40	4700–12 200	3,4,5	Granular subbase
5	0.50	2350–9600	100–450	Sandy silt subgrade
6	0.50	Varied	Varied	Extra layer where necessary
7	0.50	>1 000 000		Stiff subgrade layer at 350-in minimum

Note: 1 in = 25.4 mm; 1 psi = 6.89 kPa.
[a]E-values arbitrarily chosen in this range.

Figure 2. Typical pavement cross section used in analysis.

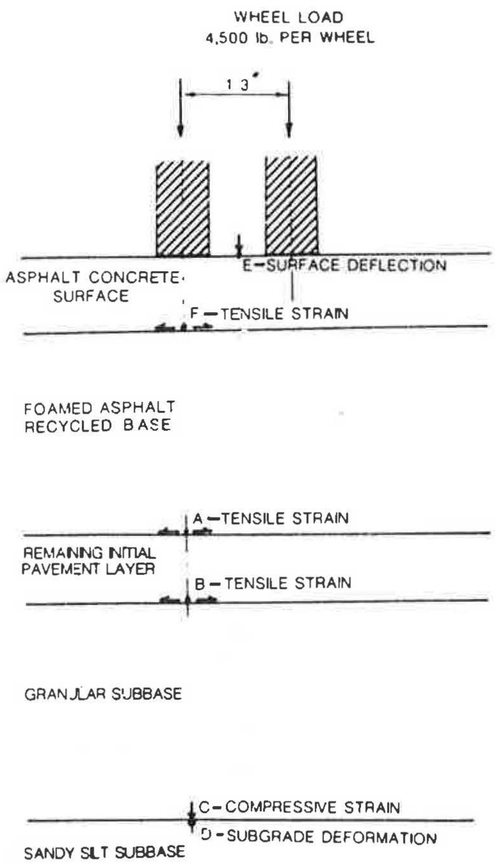
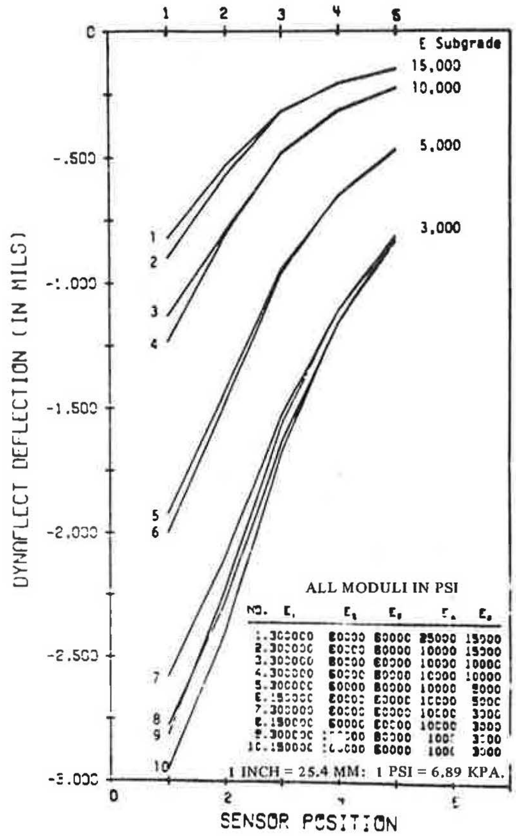


Figure 3. Influence of layer moduli on deflection basin.



uniquely by looking at the deflections at sensors 3, 4, and 5. The moduli of the overlying layers could not be identified uniquely. The elastic moduli of the remaining initial pavement layer (E_3) were arbitrarily chosen, and then the M_R-θ relationship for the granular layer could be used through iterations to determine an elastic modulus for the recycled layer in the DURING case (without the surface layer). These values of the elastic moduli of the recycled layer were then used as an indicator of values in the AFTER 1 case.

Ranges for elastic-modulus values were assumed for the surface layer in the AFTER 2 case and to some extent for the AFTER 1 case (Table 1). At the same time that the iterations were made to determine the M_R of the granular material and the surface layers, the subgrade moduli were refined.

The inclusion of only one subgrade layer in the pavement section gave, in most cases, deflections well above those measured (13). The inclusion of a second or sometimes third stiff subgrade layer solved this problem. The inclusion of a second stiffer subgrade layer decreased the deflections, with a constant at all five sensor positions, without changing the deflection basin. The inclusion of a stiff subgrade layer is theoretically sound, since the M_R increases as the deviator stress decreases. The elastic modulus calculated under the pavement layer combinations investigated was approximately 1 000 000 psi (6900 MPa) at 350 in (890 mm). The subgrade was therefore divided into a stiff layer at 350 in and a soft layer on top of that. In some cases another layer with an appropriate modulus was inserted between the two layers. The elastic moduli of the subgrade layers were also adjusted during the iteration process. The iteration was stopped when the deflections at all five sensors were within at least 10 percent of the actual values.

4. The BISTRO program was again used to calculate strains at the bottom of the asphalt layers and the subgrade compressive strain and deformation under one wheel for the final selected pavements.

Table 2. Sensitivity analysis.

			Percent Change In[a]					
Symbol	Condition	Amount	1	2	3	4	5	6
E	Upper limit	250 000	−1	+48	−1	−2	−2	−2
	Reference	180 000	0	0	0	0	0	0
	Lower limit	120 000	+2	−32	0	+2	+3	+3
E	+20 percent	48 000	−2	−40	−2	−4	−4	−2
	Reference	40 000	0	0	0	0	0	0
	−20 percent	32 000	+4	+64	+1	+4	+6	+1
E	Upper limit	50 000	−3	−3	−17	−8	−3	−1
	Reference	30 000	0	0	0	0	0	0
	Lower limit	10 000	+4	+3	+29	−1	+3	+2
E	+20 percent	12 000	−2	0	−6	−7	0	−1
	Reference	10 000	0	0	0	0	0	0
	−20 percent	8000	+3	0	+7	+8	0	+1
E	+20 percent	7080	−11	+7	−3	−3	−11	−13
	Reference	5900	0	0	0	0	0	0
	−20 percent	4720	+14	−9	+4	+4	+13	+17
E	Reference[b]	2.8×10^{-6}	NC	NC	NC	NC	NC	NC
	Reference	1.4×10^{-6}	0	0	0	0	0	0
	Reference[c]	0.7×10^{-6}	NC	NC	NC	NC	NC	NC
E	Reference[b]	2.0×10^{-6}	NC	NC	NC	NC	NC	NC
	Reference	1.0×10^{-6}	0	0	0	0	0	0
	Reference[c]	0.5×10^{-6}	NC	NC	NC	NC	NC	NC
h	+20 percent	265	+1	0	0	0	0	+2
	Reference	220	0	0	0	0	0	0
	−20 percent	175	−2	+1	0	0	0	−3

Notes: All elastic moduli (E) in psi; height (h) in inches.
NC = no change.
[a]1 = maximum surface deflection; 2 = strain at bottom of surface layer; 3 = strain at bottom of foamed-asphalt recycled layer; 4 = strain at bottom of remaining initial pavement layer; 5 = maximum compressive strain on subgrade; 6 = maximum subgrade deflection.
[b]Times 2.
[c]Divided by 2.

SENSITIVITY ANALYSIS

A sensitivity analysis was conducted on one of the cross sections, an average section, to investigate the effect of changes in the elastic moduli and thicknesses on strains and deformations. Table 2 summarizes the results of the sensitivity analysis. The only changes that had an influence of more than 10 percent on the strain and deformation values were the elastic moduli of the top five layers for the ranges analyzed. These ranges coincide with the upper and lower limits arbitrarily chosen for the first- and the third-layer elastic moduli and 20 percent for the other layers. An accuracy of 20 percent was considered to have been easily obtained during the selection process. All the thicknesses except for the thickness of the subgrade layers were determined through actual measurement and testing and therefore were fixed.

The elastic modulus of the subgrade (E_5) was the most important single parameter. A change of 10 percent in E_5 produced strains and deformations within 10 percent of the average values. An accuracy of at least 10 percent was secured during the selection of the subgrade elastic moduli, since the deflections of the Dynaflect sensors 3, 4, and 5 served as good indicators of the subgrade modulus. The elastic modulus of the recycled layer had a large effect on the strain below the surface layer only. Changes in the elastic moduli of the surface layer also had a large effect on the strain below this layer only. These radial strains were not used in the determination of the coefficients. Changes in the elastic modulus of the remaining initial pavement layer (E_3) induced large changes in strains at the bottom of the recycled layer. The selection of a wrong E_3-value would not influence the calculation of the structural coefficient, since the ratio of strains remained approximately the same in the case where the recycled layer is replaced with an AASHTO layer. It must be taken into consideration, however, when strains from cross sections with different E_3-values are compared. The sensitivity showed that although the values selected for the pavement sections might not be the exact values, they would predict strains and deformations within acceptable ranges.

DETERMINATION OF EQUIVALENT AASHTO ASPHALT-CONCRETE SURFACE THICKNESSES

The temperatures when the deflection measurements were taken were not the same for all times of measurement. By a procedure developed by Southgate and Deen (14) that used the mean ambient temperature of the five days preceding the measurement and the pavement surface temperature during testing, pavement temperature at a depth of 3 in (75 mm) was determined for each case. Instead of a constant elastic modulus for the standard AASHTO surface layer of 450 000 psi (3105 MPa), the elastic modulus was adjusted for each temperature. A relationship between elastic modulus and temperature developed for the standard AASHO Road Test mixture from laboratory-compacted samples (15) and interpolations among the elastic moduli and temperatures of the AASHTO surface concrete layer measured at four different seasons during the road test (16) were used. Four different temperatures were recorded. These temperatures and the corresponding AASHTO-layer elastic moduli are given in Table 3.

The standard AASHTO material was used in each of the 25 sections in place of the recycled layer to calculate stresses, strains, and deformations by means of the BISTRO program. This was done for the AASHTO material layer thicknesses of between 0.5 in (12 mm) and 5 in (125 mm). Nothing in the pavement section was changed except the resilient modulus of the granular material, which was changed slightly to adjust for the different stresses in the layer. The strains and subgrade deformations were again calculated for positions under one wheel (Figure 2).

SELECTION OF DISTRESS CRITERIA

The use of distress criteria is an essential part of the process of determining the structural coefficient. The distress criteria are used to compare the performance of a layer, in this case a recycled layer, with that of a standard AASHTO asphalt-concrete layer. Relationships have been developed in which the performance, in terms of load repetitions, has been correlated with a strain or deformation in the pavement system. Two general types of relationships are available, namely, those that correlate subgrade compressive strain, subgrade deformation, or maximum deflection with load repetitions to failure (N_f) and those that relate the tensile strain in the asphalt layers to N_f. The first set of relationships predicts rutting or excessive deformation of the pavement. The second set predicts when fatigue cracking in the asphalt layers will occur. The performance, as measured by N_f, is a function of only the SN for a given type of loading, terminal serviceability, and environmental conditions. Pavements with equal SN-values will perform similarly under similar conditions. Either the number of load repetitions to some type of failure (N_f) or the SN can therefore be used to compare pavement sections.

No fatigue relationships were developed for the recycled material or the pavement under consideration. Relationships had to be selected from the literature. These relationships were developed for different materials and under different conditions. They are not always fully applicable to all materials under all conditions. The asphalt layers were not at a standard temperature during the testing. Temperature adjustments were made to the fatigue-life relationships of the asphalt layers according to a method proposed by Rauhut and Kennedy (17). The fatigue-life relationships used are summarized in Tables 4 and 5 and the other relationships in Table 6.

Table 3. Temperature and AASHTO-layer elastic moduli.

Designation	Time After Construction (days)	Lane Direction	Pavement Temperature (°F)	Adjusted Elastic Modulus of AASHTO Layer
DURING	1	East	77	350 000
AFTER 1	12	East	72	400 000
AFTER 1	12	West	80	300 000
AFTER 2	250	East	85	200 000
AFTER 2	250	West	85	200 000

Note: $t°F = (t°C \div 0.55) + 32$.

Table 4. Fatigue relationships used for asphalt concretes.

Designation	k_1	k_2	Temperature (°F)	Reference	Description
Asphalt concrete (experimental road)	4.997×10^{-14}	5.10	85	18	From indirect tensile tests of laboratory specimen and of strength after eight months
	1.83×10^{-6}	3.20	85		
Asphalt concrete (AASHTO material)					
X	7.87×10^{-7}	3.29	70	17	Developed from laboratory beam tests on AASHO Road Test materials for <10 percent cracking
	3.11×10^{-6}	3.14	72		
	1.15×10^{-6}	3.25	77		
	5.83×10^{-6}	3.07	80		
	1.76×10^{-5}	2.95	85		
Y	2.16×10^{-13}	4.995	70	17	Based on tensile strain at bottom of AASHO Road Test pavements (elastic-layer theory and psi = 2.5 used)
	8.54×10^{-13}	4.79	72		
	3.16×10^{-13}	4.90	77		
	1.60×10^{-12}	4.72	80		
	4.82×10^{-12}	4.60	85		

Notes: $N_f = k_1 (1/\epsilon_r)^{k_2}$.
$t°F = (t°C \div 0.55) + 32$.

Table 5. Fatigue relationships used for recycled material.

Designation	k_1	k_2	Temperature (°F)	Reference	Description
1	4.36×10^{-8}	3.47	70	17	For asphalt-treated bases
	6.37×10^{-8}	3.56	72		
	1.72×10^{-7}	3.45	77		
	3.23×10^{-7}	3.39	80		
	9.73×10^{-7}	3.26	85		
2	2.20×10^{-6}	3.29	70	17	From laboratory fatigue tests on AASHTO material with E = 150 000 psi
	5.95×10^{-6}	3.17	72		
	8.69×10^{-6}	3.12	77		
	1.63×10^{-5}	3.07	80		
	4.91×10^{-5}	2.95	85		
3	4.476×10^{-15}	4.83	68	15	From laboratory fatigue tests on foamed-asphalt stabilized material, E = 190 000 psi
	1.21×10^{-15}	4.73	72		
	1.77×10^{-14}	4.69	77		
	3.32×10^{-14}	4.62	80		
	9.99×10^{-14}	4.50	85		
4	9.02×10^{-6}	2.29	68	15	Same as 3 but a different mixture, E = 160 000 psi
	1.32×10^{-5}	2.25	72		
	3.57×10^{-5}	2.14	77		
	6.70×10^{-5}	2.07	80		
	1.95×10^{-4}	1.95	85		

Notes: $N_f = k_1 (1/\epsilon_r)^{k_2}$.
$t°F = (t°C \div 0.55) + 32$.

Table 6. Failure relationships based on strains or deformations.

Designation	Relationship	Reference	Description
A	Tensile strain at bottom of recycled layer (ϵ_r)		
B	Tensile strain at bottom of remaining initial layer (ϵ_r)		
C	Compressive strain on subgrade (ϵ_r)		
	$\log \epsilon_v = -1.9765 - 0.2008 \log (N)$	16	For AASHTO materials
	$\log N_f = 2.050\,76 - 597.662\ (\epsilon_v)$	16	Regression on AASHO Road Test results
	$\epsilon_v = 2.8e\text{-}2\ (N) - 0.25$	16	Used by Shell in their design
	$\epsilon_v = 14.81 + 14\,805.05\ (SN + 1)$	20	From analysis of AASHTO satellite test road
D	Deformation of subgrade (δ_s)		
	$N_f = 0.98 - 3.39 \ln \delta_s$	5	From results of analysis on loop 4 of AASHO Road Test
E	Maximum deformation at surface (δ_m)		
	$\log RR = -6.866 + 4.325 \log \delta_m$	17	From AASHO Road Test results
	$\delta_m = 0.85 + 1287.30\ (SN + 1)$	20	From analysis of AASHTO satellite test road

CALCULATION OF STRUCTURAL COEFFICIENTS

Two pavement sections will give the same performance with the same predicted life before failure or the same SN under similar conditions. The SNs can be compared to determine an unknown structural coefficient. In this case, SN = SN' or

$$a_1D_1 + a_2D_2 + a_3D_3 + a_4D_4 = a_1'D_1' + a_2'D_2' + a_3'D_3' + a_4'D_4' \quad (4)$$

where

D_1 to D_4 = thicknesses of surface layer, AASHTO layer, remaining initial pavement layer, and granular subbase;
a_1 to a_4 = corresponding coefficients;
SN' = SN of pavement with recycled layer;
D_1' to D_4' = thicknesses of surface layer, recycled layer, remaining initial pavement layer, and granular subbase; and
a_1' to a_4' = corresponding coefficients.

However, $a_1D_1 = a_1'D_1'$, $a_3D_3 = a_3'D_3'$, and $_4D_4 \approx a_4'D_4'$.

The two pavement sections have slightly different elastic moduli for the subbase material but the same for the subgrade. Rada and Witczak (19) concluded that these coefficients depend on the strength of the subgrade and that therefore no adjustments were needed. The differences in M_R were small. Even if the relationships in NCHRP 128 (10) were used, the difference in a_4's would be negligible in view of some of the other assumptions already made. Therefore,

$$a_2' = a_2D_2/D_2' \quad (5)$$

where

a_2' = structural coefficient of recycled layer,
a_2 = 0.44,
D_2' = 5.5 in (145 mm), and
D_2 = thickness of equivalent AASHTO layer.

Structural coefficients were calculated with the following distress modes for all 25 selected sections (Figure 2):

1. Tensile strain at the bottom of the recycled layer: The tensile strains were used directly to determine the equivalent AASHTO layer thickness and subsequently the structural coefficient of the recycled layer. This is not a very reliable method, since the materials will most likely have different fatigue characteristics. Structural coefficients were therefore also calculated for different combinations of fatigue characteristics of the AASHTO layer and the recycled layer. Different relationships were used, since it was not possible to select one representative relationship for each material. Eight combinations, denoted X1, X2, X3, X4, Y1, Y2, Y3, and Y4 (Table 4), were used. Only five gave coefficients between 0.05 and 0.44. The other three combinations were considered to misrepresent the fatigue characteristics of the materials used. For the DURING condition, an equivalent AASHTO layer thickness could not be determined, since the tensile strain fluctuates with thickness for thin AASHTO material layers. Figure 4 depicts the changes over time (AFTER 1, AFTER 2) in the average coefficient values for the five fatigue-life combinations.
2. Tensile strain at the bottom of the remaining initial pavement layer: The maximum tensile strain in the asphalt layers occurred at the bottom of the remaining initial pavement layer. Since the material in the layer remained the same with and without the recycled layer, the tensile strains were compared. Identical tensile strains would have given identical fatigue lives.
3. Compressive subgrade strain: The maximum compressive strain on top of the subgrade is usually used to predict pavement rutting. It is also widely used to compare different pavement sections.
4. Subgrade deformation: The subgrade deformation has been correlated with the number of ESALs to cause failure (psi = 2.5) (16).
5. Maximum deformation: Wang (20) correlated the performance in equivalent axle loads (EALs) of flexible pavements with the maximum surface deflection.

Figure 5 illustrates the changes in structural coefficients of the average sections over time.

CONTROLLING CRITERION

Structural coefficients can be determined from various criteria, but only one will control the performance of the pavement. This will be the criterion that predicts the shortest service life. In the case of this recycled pavement, the controlling criterion was either the subgrade deformation or the tensile strain at the bottom of the recycled layer, depending on which fatigue-life relationship was used. Fatigue-life relationship 3 (as designated in Table 3) predicted the shortest service life in all cases. Structural coefficients calculated with this relationship were less than 0.05 for the AFTER 1 case but had acceptable values for the AFTER 2 case. This is not unreasonable, since it is unlikely that newly constructed pavement layers will have the same fatigue characteristics 12 and 250 days after construction. This relationship can therefore apparently not be used for the AFTER 1 case. The relationship for the prediction of the service life based on subgrade deformation was also

Figure 4. Effect of time after construction on structural coefficients: asphalt fatigue.

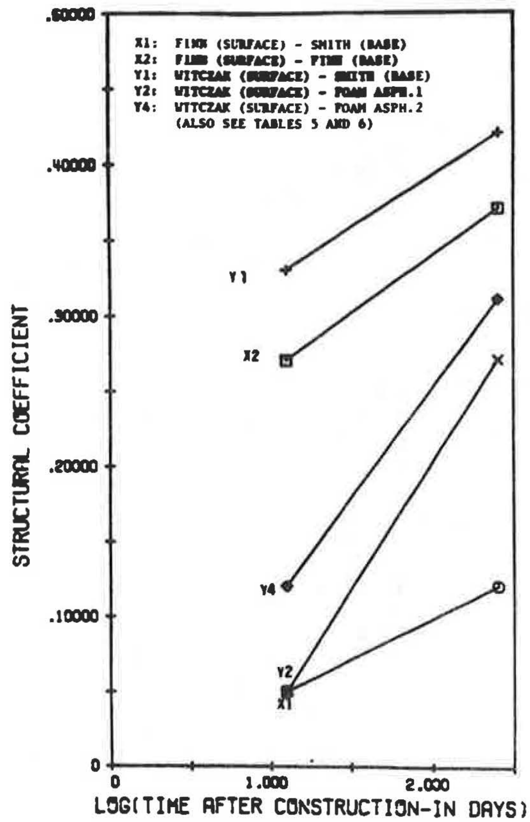
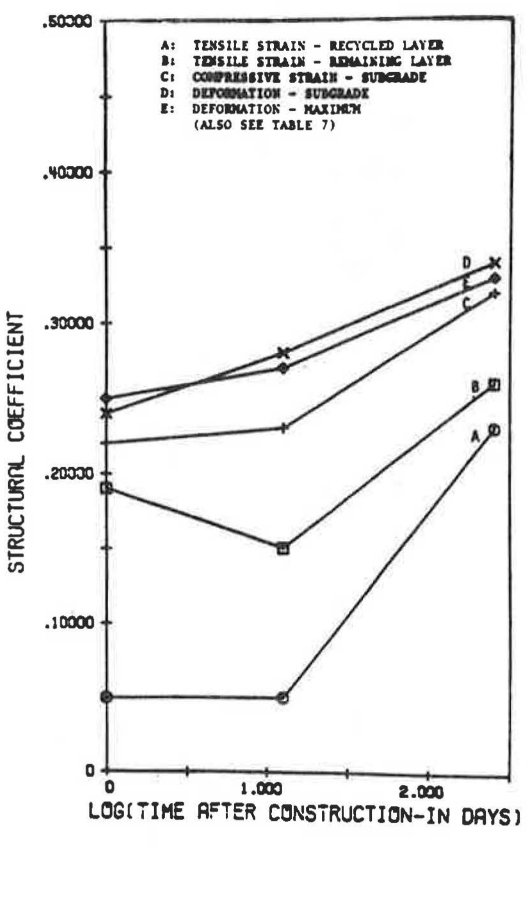


Figure 5. Effect of time after construction on structural coefficients: strains and deformations.



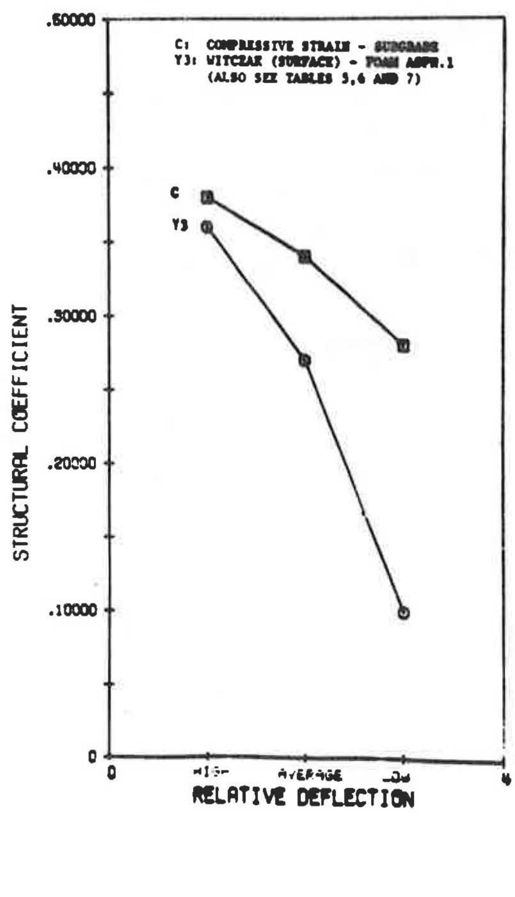
developed for conditions not necessarily similar to those in this study. The coefficients calculated from both the subgrade deformations and the fatigue-life relationship 3 are given in Figure 6.

DIFFERENT RECYCLED-LAYER THICKNESSES

The constructed recycled layer was 5.5 in, and all the structural coefficients were calculated for such a thickness. Since the coefficients are also a function of the thickness of the layer, the structural coefficients were also calculated for recycled-layer thicknesses of 3 and 8 in (75 and 200 mm). The 5.5-in layer was replaced with the 3- and 8-in layers in the sections with average deflections in the AFTER 2 case. The coefficients were calculated as previously described and are displayed in Figure 7. Only two combinations of fatigue-life relationships, namely, X2 and Y3, gave realistic coefficients. The coefficients seem to increase in general from layers with a thickness of 5.5 in to layers of 8 in, independent of which criterion is used. The increases are small. There are in general large differences between the coefficients of layers of 3 in and 5.5 in, but this is dependent on which criterion is used. Coefficients calculated from tensile strains in the asphalt layers are substantially lower than the coefficients calculated from other criteria for a thickness of 3 in. The tensile strains at the bottom of the asphalt layers are sensitive to changes in the elastic modulus of thin layers. Figure 7 gives an indication of how the thickness of the recycled layer influences the structural coefficients for a recycled-layer modulus of 75 000 psi (517.5 MPa) at 85°F (29°C) only.

The resilient modulus of the recycled material is the most important single contributor to the struc-

Figure 6. Coefficients based on controlling criteria.



tural coefficients, as can be seen in Figure 8. The relationships in Figure 8 were developed strictly for the ranges of pavement material properties and layer thicknesses present in the experimental road section. The resilient moduli were determined at actual pavement temperatures and not adjusted to a standard temperature. The average M_R in the AFTER 2 case was approximately 140 000 to 150 000 psi (960-1035 MPa) at 70°F (21°C).

SUMMARY OF RESULTS

The structural coefficients calculated covered a very wide range due to the many different criteria used to calculate them. Since reliable fatigue characteristics of the recycled layer were not available to calculate exact coefficients for this recycled layer, a combination of criteria was used to establish a range of representative structural coefficients. The subgrade strain and deformation were selected as criteria because the first is widely used to establish structural coefficients and the second to predict the shortest service life for the pavement. The tensile strain at the bottom of the remaining initial pavement layer was also used, since it represented the position where the maximum strain in the asphalt layers occurred. It is further independent of the fatigue characteristics of the layer. Finally, three fatigue-life combinations were selected, one (X2) for both the AFTER 1 and AFTER 2 cases and one each (Y1 and Y3) for the two AFTER cases. The selection of these combinations was based on combinations that gave coefficient values between 0.05 and 0.44.

The assumptions made in the determination process are summarized below (E_1 = elastic modulus of surface layer; 1 psi = 0.69 kPa):

Procedure	Assumption
Selection of cross sections	M_R-θ relationships for granular material were valid for fall and spring conditions
	θ calculated at center of granular layer represented θ of layer
	M_R-σ_d relationships for subgrade were valid for fall and spring conditions
	Conversion factor to convert Dynaflect deflections to Benkelman-beam deflections was valid for deflections at all five sensors
	Range of E_1 was 120 000-250 000 psi
	Range of E_3 was 10 000-50 000 psi
AASHTO layer	Pavement temperatures were accurately measured during deflection measurements
	AASHTO-layer E-temperature relationships used to adjust E_2-values are applicable
Coefficient determination	Fatigue-life relationships used were applicable to materials in this study (for asphalt materials as well as for subgrade)
General	BISTRO elastic-layer program is good representation of actual conditions in all cases investigated

Figure 7. Effect of layer thickness on structural coefficients.

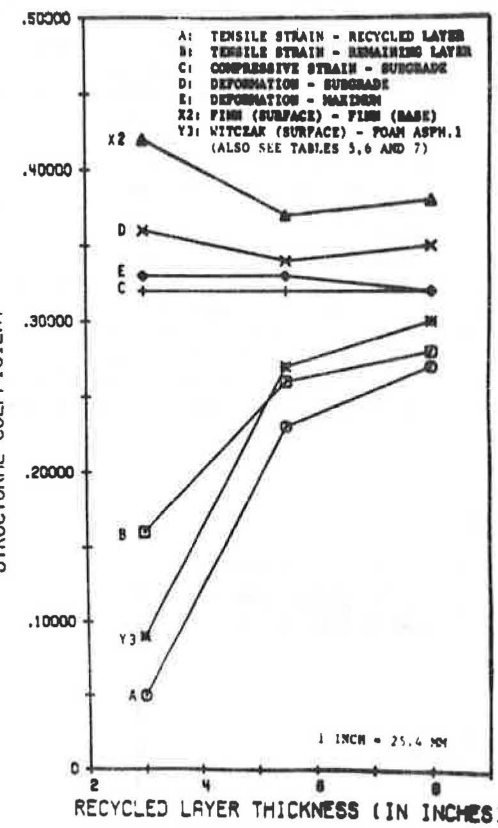
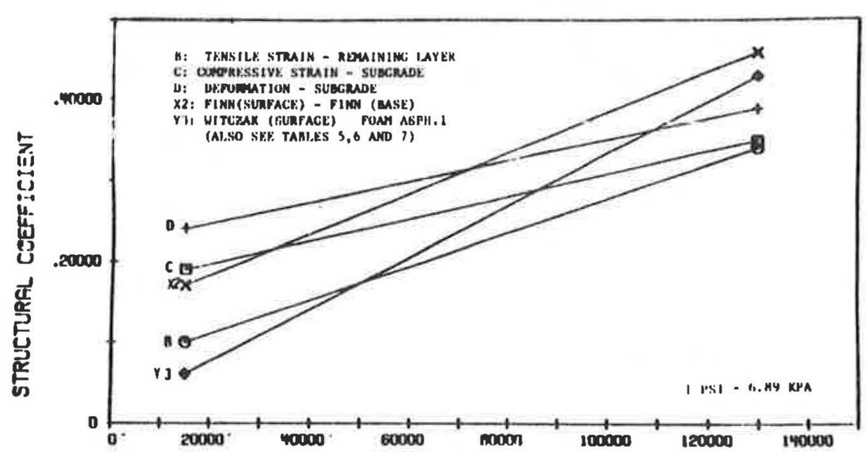


Figure 8. Relations between resilient modulus and structural coefficient.

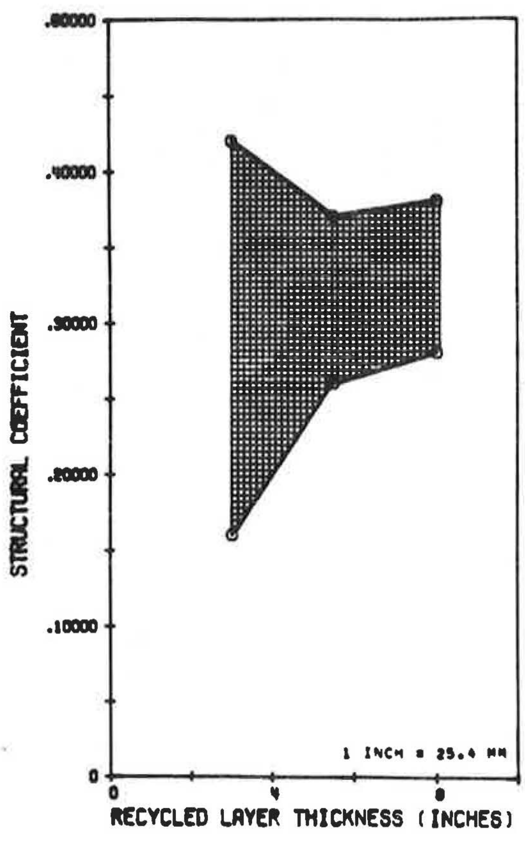


Although the assumptions are valid, they should be kept in mind in the evaluation of the results, since they could have influenced the results.

The most important conclusions from this study are the following:

1. The structural coefficients increased over time after construction, as expected.
2. The elastic modulus was the most important single parameter that contributed to the structural coefficient.
3. The structural coefficients changed with changes in recycled-layer thickness. The change depended on the criteria used but was fairly small for thicknesses from 5 to 8 in for the ranges of pavement properties and thicknesses used.
4. The structural coefficients of the recycled layer one day after construction varied from 0.13 to 0.30; those of the section with average deflection were between 0.19 and 0.24.
5. The structural coefficients of the recycled layer 12 days after construction ranged from 0.11 to 0.39. The average value ranged from 0.17 to 0.33. Without the 0.17 value, the average ranges from 0.23 to 0.33.
6. The condition of the pavement 250 days after construction is a good representation of the average conditions during the life of the pavement in terms of stiffness and density. Deflection measurements were made during the spring season, the worst subgrade conditions. Structural-coefficient values determined for these conditions will therefore be a good indicator of the structural coefficients to be used in design. The coefficient values ranged from 0.10 to 0.43. The coefficients of the average deflection sections ranged from 0.26 to 0.37. They are valid for the approximately 5- to 8-in foamed-asphalt recycled layer studied, which had a resilient modulus of approximately 140 000-150 000 psi (960-1035 MPa) at 70°F. Figure 9 depicts the recommended range.

Figure 9. Recommended ranges of structural coefficients.



The structural coefficient is a function of many factors and, in this study, mainly of the criteria used. They vary over a wide range. They should not be applied blindly to any type and thickness of foamed-asphalt recycled layer, since they were developed for a specific foamed-asphalt recycled mixture in a specific pavement section.

EXAMPLE

The section of foamed-asphalt recycled pavement used in the analysis has the following characteristics pertaining to the AASHTO flexible-pavement design method:

Characteristic	Amount
ESALs	190 000
S_i	4.8
R	1.1
Terminal present serviceability index	2.0

When this information is used in the AASHTO equation, the SN should be at least 2.33 to sustain the traffic for 20 years:

$$SN = a_1 D_1 + a_2 D_2 + a_3 D_3 + a_4 D_4 \quad (6)$$

where

D_1 = asphalt surface thickness = 1.25 in (32 mm),
a_1 = 0.44,
D_2 = foamed-asphalt recycled-layer thickness = 5.5 in,
a_2 = between 0.26 and 0.37,
D_3 = remaining initial pavement-layer thickness = 1 in (25 mm),
a_3 = 0.10 (low value assumed since the layer can be broken up and cracked),
D_4 = granular-layer thickness = 4.5 in (115 mm), and
a_4 = 0.11.

Consider the following values of a_2:

1. a_2 = 0.26: The SN is 2.58. The current SN of the pavement can be reduced with 0.25 and the pavement will still sustain the loads. This corresponds to a reduction of more than 0.5 in (0.25/0.44) in the thickness of the asphalt surface layer and a subsequent savings of $40 000, or 11 percent of the construction cost.
2. a_2 = 0.32 (center of the recommended range): The SN is 2.36. The asphalt surface layer would be unnecessary. Some type of surface protection, e.g., a seal or a surface treatment, is necessary to protect the water-sensitive foamed-asphalt recycled mixture. The savings would still have been at least $75 000 (with a surface treatment), or 20 percent of the total cost.

At least 10 percent of the initial construction cost could have been saved had the structural coefficients been known during the design. The pavement is overdesigned, even if the lower value of the range is used.

CONCLUSIONS

The structural coefficients are influenced by many factors. The criterion used in their analysis has a significant effect on the coefficient values. A single structural coefficient cannot be determined for a recycled material layer without reliable fatigue characteristics available for all the pavement layers. This, unfortunately, requires extensive additional testing. A range of structural coefficients can be determined from in situ pavement-layer properties and carefully selected available fatigue characteristics for the layer. Reliable structural coefficients, even a range, can be used in pavement thickness design and induce saving by eliminating overdesign or underdesign. The selection of a particular structural coefficient from a range is still the responsibility of the designer, but the selection is limited and the consequences are known. Structural coefficients describe the performance of a particular layer, e.g., a foamed-asphalt layer with specific properties and thickness, and should not be used without thorough consideration for any other layer.

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