

Performance of Flexible Pavements

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An investigation was conducted to determine the life period of full-depth granular pavements in India. Benkelman beam deflections, rutting, cracking, and patching were measured on different test sections to calculate the performance of the pavements. A repeated-load-test apparatus was fabricated in the laboratory for testing the subgrade soils and water-bound macadam base to determine the elastic moduli under repeated triaxial stress conditions. A finite element computer program was developed to determine the stresses and strains in pavement layers and to correlate the pavement performance and the critical stress and strain values. The pavement surface deflections measured by Benkelman beam are in close agreement with the values predicted by the computer program. From the repeated-load-test results, relationships were found between (a) the resilient modulus and the lateral pressure and (b) the resilient modulus and the sum of principal stresses. The equation developed for predicting the rut depth from the Benkelman beam deflection data is in close agreement with that developed in the AASHTO road test. The rut depth was also related to the gravel surfacing depth and the cumulative axles. Finally, a regression equation for finding the lives of flexible pavements was established using the rut depth and subgrade vertical strain values.

Highway engineers have been constantly working toward developing a rational method to estimate the life of a given pavement through full-scale road tests, model studies, laboratory experiments, and theoretical analysis. Unlike steel and concrete, whose structural properties are well defined, the behavior of crushed rock, water-bound macadam (WBM), and other granular materials is extremely complex and was little understood until about 30 years ago. Since then slow progress has been made in the development of a rational method to find the relationship between the life period of pavements and the design of constituent layers.

Flexible pavement methods are still empirical or semiempirical, on the basis of past experience with similar subgrades, pavement materials, and traffic loads. These methods are satisfactory as long as the materials, traffic loading conditions, and layer thicknesses do not differ from those for which the methods were developed. Because of considerable increase in frequency and intensity of axle loads and the use of new pavement materials, the semiempirical design methods are no longer adequate.

The focus here is on the study of the performance of WBM pavements and the laboratory evaluation of properties of the

pavement materials to predict the serviceable life of a granular pavement in India.

DEVELOPMENT OF FINITE ELEMENT ANALYSIS

A computer program known as RAOPAVE was developed by idealizing the flexible pavement into a finite element continuum. The analysis was carried out for a single wheel load of 40,131 (9,000 lb) distributed over a circular area of 152-mm radius with a tire pressure of 0.55 MPa (80 psi).

Idealization

In this investigation a layered pavement system was idealized as an axisymmetric solid with finite boundaries in both radial and axial directions, as shown in Figure 1. The axisymmetric body was then divided into a set of ring elements, rectangular in section and connected along their nodal circles. The finite elements are actually complete rings in the circumferential direction, and the nodal points at which they are connected are circular lines in plan view. Because of axisymmetry, the three-dimensional problem reduces to a two-dimensional case similar to a plane strain problem. Tensile stresses and strains were taken to be positive, and compressive stresses and strains negative. For each element the four nodal points were numbered in the clockwise direction. Each node has two degrees of freedom.

Displacement Functions

The two displacement components in a solid continuum varied as complicated functions of position. A number of approaches, including power series and Fourier series expansions, have been proposed by several researchers (1,2) to represent the behavior of displacement components inside each element. Because of the assumptions made about these functions, the accuracy of the answer increases as the element size decreases. For this investigation, the displacement functions inside each element were approximated by the following:

$$u(r, z) = b_1 + b_2 r' + b_3 z' + b_4 r' z' \quad (1)$$

$$v(r, z) = b_5 + b_6 r' + b_7 z' + b_8 r' z' \quad (2)$$

where for each element the local coordinate system r', z' was used, which has its origin at the center of each element.

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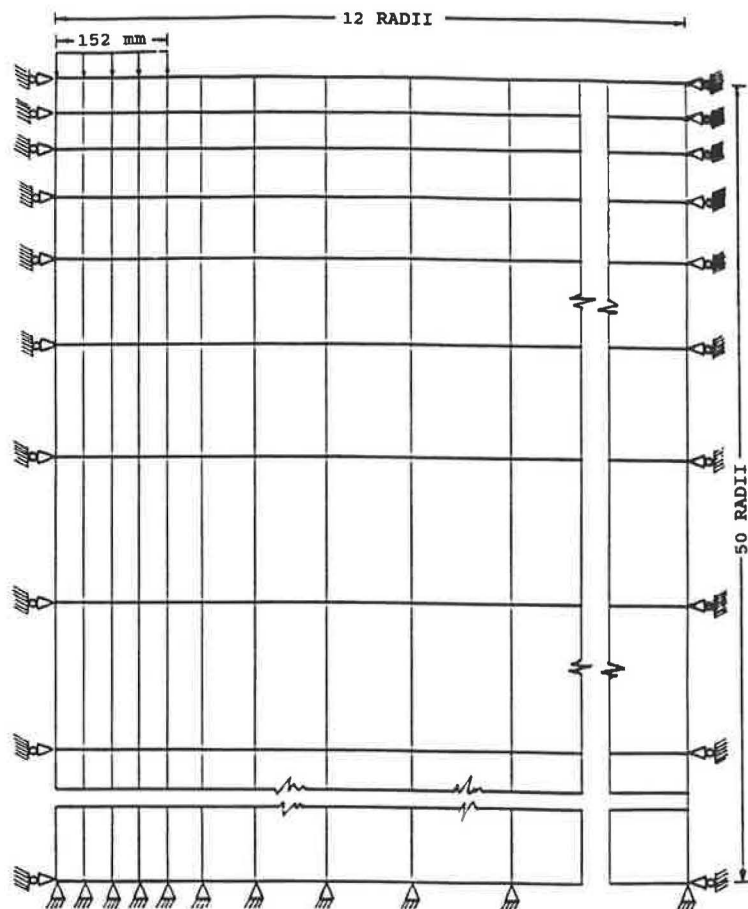


FIGURE 1 Finite element idealization.

Global Stiffness

The stiffnesses of all elements were assembled to obtain the total stiffness for the system. The required assembly was accomplished by using the element-node table and displacement-code number array. Because the system stiffness matrix comes out to be banded and symmetric, it is assembled in half-band form. A consistent load vector was formulated using the principle of virtual work (3).

Boundary Conditions

Nodal points on the vertical boundary at a distance of 12 radii from the center and on the centerline were constrained from radial movement; those on the bottom boundary were not allowed to move vertically or horizontally. The bottom boundary was fixed at a depth of 50 radii in accordance with the findings of Duncan et al. (4).

Verification of Elastic Finite Element Analysis

The validity of the finite element computer program was established by comparing the results with those of Ahlvin and

Ulery (5). The vertical and radial stresses along the centerline of the load, as shown in Figures 2 and 3, respectively, and at a radial distance of 0.752 radii (from the centerline of the load) obtained by the program, were compared with those obtained by the elastic half-space analysis after Ahlvin and Ulery. The deflections and stresses computed by this program agree closely with those obtained from the elastic half-space analysis. A detailed printout of the computer program is provided by Tangella (6). Several other elastic multilayered computer programs (e.g., ELSYM, BISAR, ILLIPAVE, and FEPAVE) are available for the structural analysis of a pavement, and any of them also can be used. However, the finite element method using the principle of stress transfer developed by Zienkiewicz et al. (2) is suitable for eliminating the tensile stresses in unbound granular layers. This work is beyond the scope of the present study and has been done elsewhere (7).

EXPERIMENTAL SETUP

The schematic diagram of the experimental setup for repeated load triaxial tests is shown in Figure 4. The details of this setup and its mode of working are given by Tangella (6) and Tangella and Pandey (8).

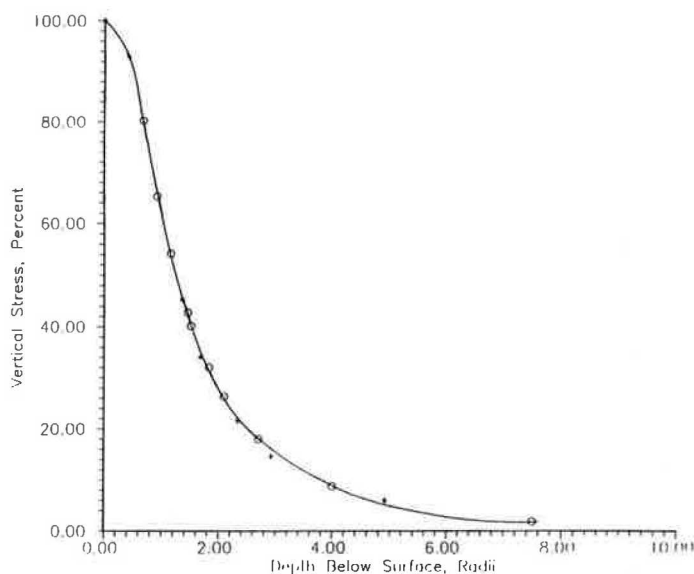


FIGURE 2 Comparison between vertical stresses of RAOPAVE (open circles) and elastic half-space analysis (at centerline of load) (closed circles).

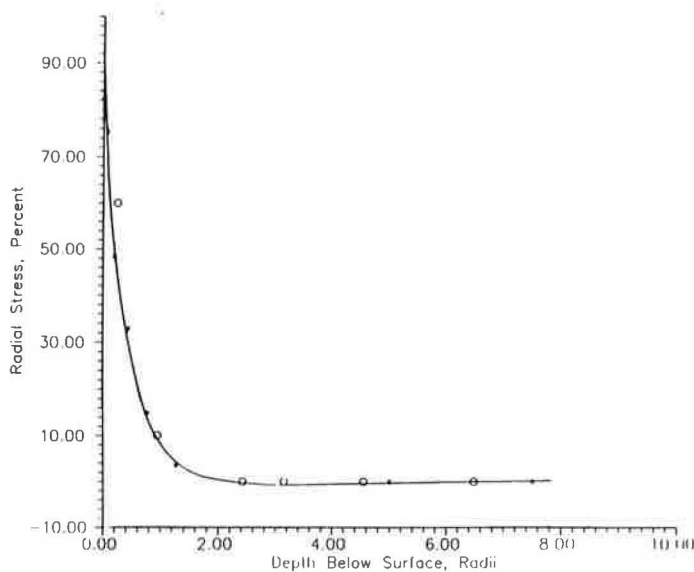


FIGURE 3 Comparison between radial stresses of RAOPAVE (open circles) and elastic half-space analysis (at centerline of load) (closed circles).

Repeated-Load Experiments

Specimens 100 mm in diameter and 200 mm in height were cast using the soil samples brought from the subgrades of the field test sections. The field bulk density was calculated for each test section at the maximum in-situ moisture content for each of the four seasons. The samples were cast to the field density at the highest value of the in situ moisture content. Trials were made, varying the number of layers and the number of blows, using the standard Proctor hammer (of 24.44 N weight and 300-mm fall) to compact the soil to the fixed

volume. The variation in the moisture content of the specimen was kept within 0.2 percent.

Initially, 10 load repetitions were applied to condition the specimen. After 1,000 repetitions, for a given deviatoric stress and for each set of lateral pressures, the load cell and linear variable differential transformer (LVDT) pulses were recorded for about 30 load repetitions. The process was continued until all sets of deviatoric stresses were recorded. Strict control was exercised in casting the specimens at the in situ moisture content and field density. Figure 5 shows the repeatability of two experiments. Deviatoric stresses ranging

from 15 to 150 kN/m² were applied to samples of 15 test sections. Lateral pressures of 13.8 kN/m² (2 psi) and 20.7 kN/m² (3 psi) were applied, which are very close to the subgrade lateral pressures found by the computer program. Because there is no significant change in results under the cyclic lateral pressure or the constant lateral pressure (equivalent to the mean of the cyclic lateral pressure) (9), a constant lateral pressure is used in the laboratory study even though cyclic lateral pressure occurs under actual conditions. The duration of the loading pulse was 0.12 sec (10).

Tests for Other Soil Properties

The laboratory California bearing ratio (CBR) tests at field densities and in-situ moisture contents, liquid limit, plastic limit, and Proctor compaction tests were conducted on soil samples of the subgrades of all the test sections of national highways.

Repeated-Load Tests on Granular Materials

Two types of aggregate gradations were adopted for repeated-load triaxial tests. One gradation adopted is the same as that of Monismith et al. (11,12), and the other corresponds to that of the Indian Roads Congress (IRC-19-1977). The maximum size of the aggregate is 20 mm. Hicks and Monismith (13) found that the influence of grading on the resilient modulus for crushed rock is negligible. Hence, the results of repeated-load tests on IRC B-type screenings were used to analyze all layers of the WBM portion of the test sections. Dolerite chips were crushed to the required gradations, and 10 kg of material was prepared for each specimen. Specimens of 100-mm (4-in.) diameter and 200-mm (8-in.) height were cast using a split mold. With the help of a needle vibrator, dry densities of 2224 kg/m³ (139 lb/ft³) for the gradation given by Monismith et al. (11,12) and 2127 kg/m³ (133 lb/ft³) for the gradation

given by IRC for B-type screenings were obtained. A 5-psi suction was applied during casting of the sample and was continued until the sample was fixed in the triaxial cell.

The procedures used to fix the sample and conduct the test are the same as those followed for soil samples, except for the following details.

Three sets of deviatoric stresses were applied for all samples. The values were 122.6 kN/m², 147.1 kN/m², and 196.1 kN/m². For each of the deviatoric stresses, seven sets of lateral pressures were applied (14): 69 kN/m² (10 psi), 138 kN/m² (20 psi), 207 kN/m² (30 psi), 276 kN/m² (40 psi), 345 kN/m² (50 psi), 414 kN/m² (60 psi), and 483 kN/m² (70 psi). All of these stresses are found in the field pavement structures (6,10,11). Using the results, relationships were found between (a) the resilient modulus and the lateral pressure, as shown in Figures 6 and 7, and (b) the resilient modulus and the sum of principal stresses, as shown in Figures 8 and 9. The results of this investigation are close to those of Monismith et al. (11).

EXPERIMENTS ON IN-SERVICE PAVEMENTS

Selection of Test Sections

Fifteen test sections were selected in the eastern parts of India, covering about 250 000 km² between the latitudes 19°N and 27°N and the longitudes 83°E and 88°E. For uniformity and quality control in the test sections, the following criteria were used:

1. The preferred length of the test section is 300 m.
2. The section should be a straight stretch with uniform soil characteristics and pavement structural composition. Steep longitudinal gradients and curves must be avoided. The stretch should be continuous with no major repairs and with no culverts in between.

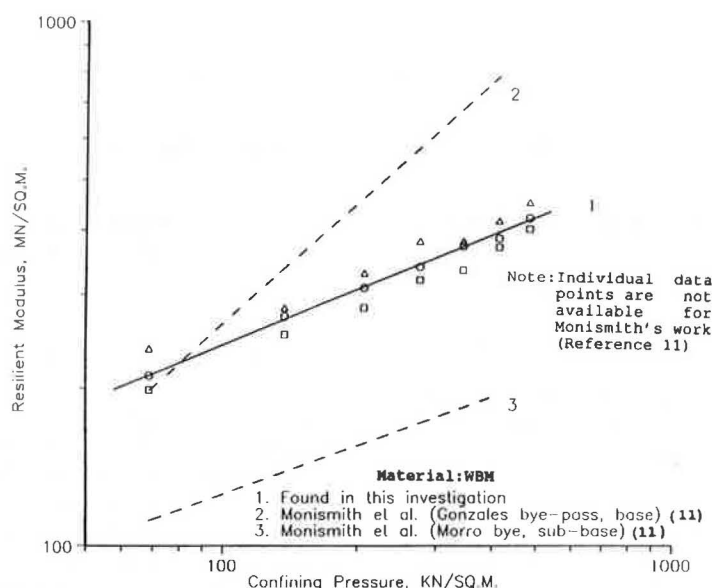


FIGURE 6 Confining pressure versus resilient modulus (11).

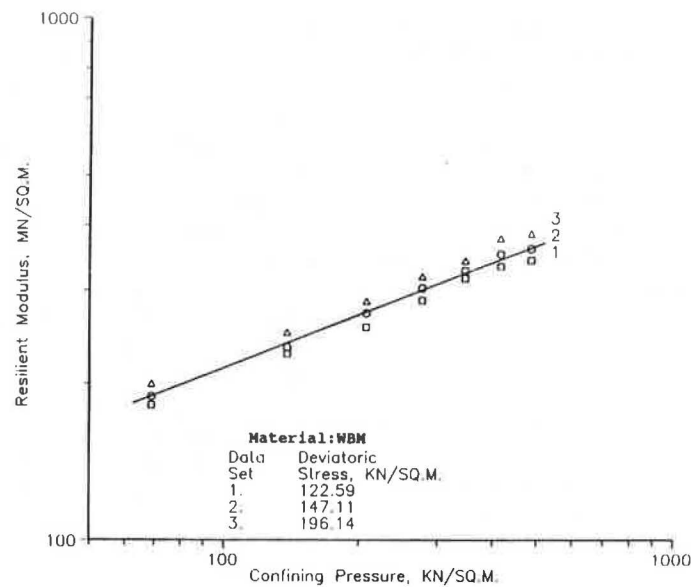


FIGURE 7 Confining pressure versus resilient modulus for granular materials (IRC-19-1977).

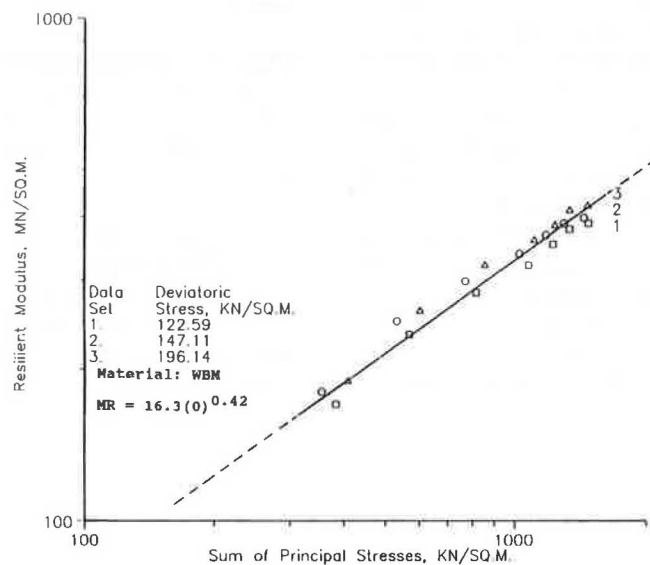


FIGURE 8 Sum of principal stresses versus resilient modulus (11).

3. The maximum height of the embankment should be less than 6 m.

4. The section should not be located in areas likely to be flooded.

5. The section should not be part of a newly constructed road.

6. The coefficient of variation of the Benkelman beam deflection data should be less than 33 percent to ensure the uniform strength of the test section.

Benkelman Beam Deflection Tests

The Benkelman beam test was conducted using the Canadian Good Roads Association's procedure (15) on the outer wheel-

paths in both directions at 10-m intervals. For a 300-m test section, a total of 62 points were tested.

Measurement of Rutting, Cracking, and Patching

Rutting in wheelpaths was measured by a 3-m straightedge placed transversely on the carriageway surface. The observations were taken on both sides of the pavement. The maximum value of rutting in the wheel track was measured at 10-m intervals. The percentage of cracked and patched areas of the pavement was measured for each test section. The structural geometries, field densities, and moisture contents were recorded.

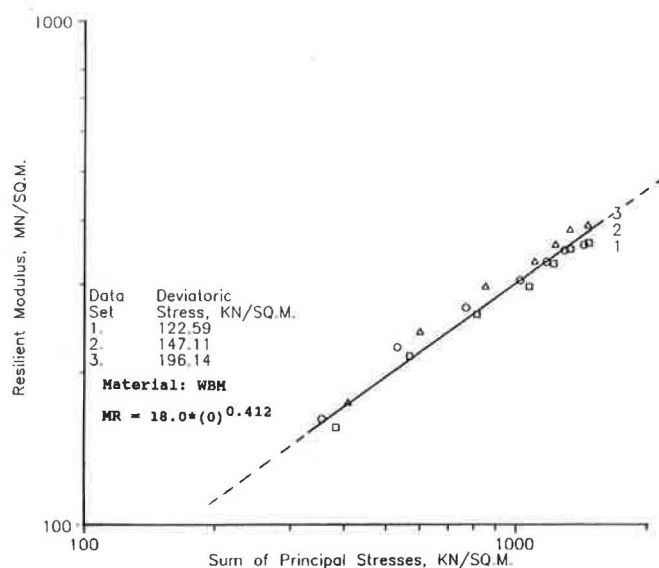


FIGURE 9 Sum of principal stresses versus resilient modulus.

Relationship Between CBR and Subgrade Modulus

The relationship between the laboratory CBR (at field density and in situ moisture content) and the resilient modulus was developed for various deviatoric stresses on the 15 test sections. The deviatoric stress was varied from 15 to 150 kN/m². Figure 10 shows that the resilient modulus (MR) values are between the lines E (MN/m²) = 19.6 * CBR and E (MN/m²) = 7.55 * CBR with a mean line as follows:

$$E = 12.48 * \text{CBR} \quad (3)$$

Similar work was carried out on the subgrade of the Alconbury Hill experimental road (16). The relationship between the in situ CBR value and the modulus of elasticity determined from the Rayleigh wave velocity, shown in Figure 11, was as follows:

$$E \text{ (MN/m}^2\text{)} = 13.3 * \text{CBR} \quad (4)$$

PREDICTION OF BENKELMAN BEAM DEFLECTION AND VERTICAL SUBGRADE STRAINS

Most roads in developing countries (including large portions of the national highways) have WBM bases with a thin bituminous surfacing in the form of premix carpet or surface dressing. Development of a suitable method for predicting surface deflections and stresses and strains for these pavement structures would be a valuable step in solving the problem of estimating the life period for a given thickness of WBM layer on a chosen subgrade soil. In this study the methodology adopted for stress analysis is similar to that used by Monismith et al. (11).

The resilient modulus of the subgrade soil was estimated from Equation 3. Subbase and base courses consist of untreated granular materials. Because these layers cannot with-

stand tensile stresses, twice the value of subgrade modulus was assigned (17-19). The vertical and radial stresses were obtained at different points using the finite element computer program. To these stresses the effect of self weight of the road structure was added. For each layer a point is selected at the center of the layer on the axis of the wheel load. For these points the sum of the principal stresses was calculated in the WBM layers, and deviatoric stresses were calculated in the subgrade soil. Using these values from the repeated-load experimental graphs (e.g., Figures 5 and 9), the moduli of the different layers were found. These modified stiffness values were then processed by the computer program. The iteration process was continued until the moduli obtained from the computer program coincided with those from the experimental data. Thus, the nonlinearity in the stiffness characteristics of the pavement materials is accounted for in the finite element computer program.

Table 1 presents a comparison of the predicted surface deflections obtained from the computer program and the measured deflections obtained by the Benkelman beam method for the test sections. The average difference in the results of the two methods is 5 percent, with a range of 1.5 to 13.1 percent. Most of the test sections had granular base topped with a thin bituminous surfacing in the form of premix carpet or surface dressing. The contribution of thin, open-graded, bituminous surfacings to the structural strength is negligible, and these highways were considered as full-depth granular highways. Although the bituminous surfacings of the five test sections (Sections 1, 9, 11, 12, and 14) had considerable thicknesses, severe grid pattern cracking leading to the breakout of the bituminous surfacings was found on the wheel tracks (because of the large volume of truck traffic). Because of this grid pattern cracking, the load-spreading capacity of the bituminous surfacing reduces considerably. Hence, for each of the five test sections the bituminous surfacing was considered part of the granular layer. The total thickness of WBM and surfacing (equivalent WBM) is presented in Table 1. The deflections can also be measured using a suitable measuring

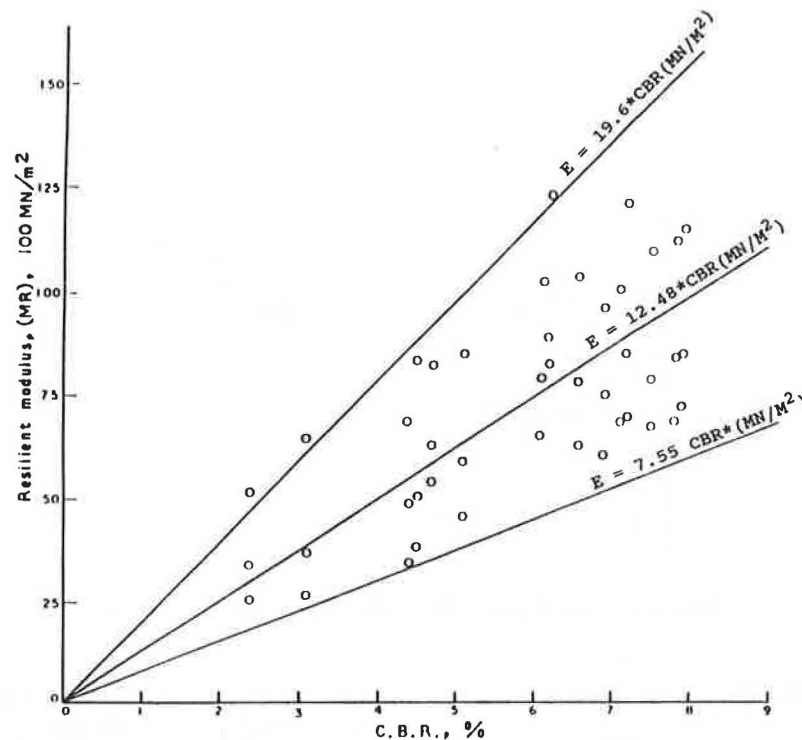


FIGURE 10 CBR versus resilient modulus.

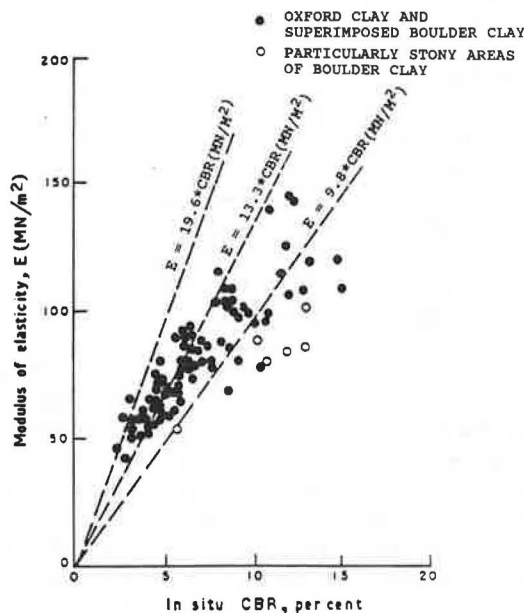


FIGURE 11 In situ CBR versus modulus of elasticity for subgrade of Alconbury Hill Experiment road (16).

device (e.g., the Benkelman beam). However, field measurements are costly and time-consuming, requiring interruption of the traffic (which may be inconvenient, especially on a busy freeway). Therefore, predicting the deflection would likely be useful to the field engineer in evaluating the structural condition of the pavement (20).

On average, the test truck stands on a test point for 20 sec during the Benkelman beam deflection test. From the test

results given by Tangella (6), it can be observed that the effect of loading on resilient modulus is negligible (for a time range of 0.12 to 20 sec). Figure 12 shows the results of a typical subgrade soil of a test section. Hence, moduli values pertaining to 0.12-sec loading time were used to analyze the pavement structure. The loading time due to traffic is about 0.12 sec for a speed of 30 km/hr (10) for the test sections under consideration. The use of resilient moduli pertaining to 0.12-sec loading duration is suitable to predict the subgrade vertical strains for developing relationships among rut depth, subgrade vertical strain, and cumulative traffic in terms of equivalent single-axle loads (ESALs), as shown in the following section.

RESULTS AND CONCLUSIONS

Relationships were developed for 15 test sections on the national highways by carrying out regression analysis between (a) the laboratory CBR and the resilient modulus, (b) the average rut depth and the Benkelman beam deflection, (c) the average rut depth and the cracking and patching area, (d) the average rut depth and the vertical subgrade strain and cumulative ESALs, and (e) the average rut depth and WBM thickness and cumulative ESALs. Goodness of fit of the models, represented by the equations presented previously, to the observed data was evaluated by studying the correlation coefficients (R), the significance of each independent variable, the standard error of estimate, and the residuals. For this purpose, t -tests and f -tests were done. A summary of the characteristics of the test sections is presented in Table 2.

From the traffic data, including the axle equivalency factors and the traffic growth rate (21), the cumulative ESALs that

TABLE 1 COMPARISON OF PREDICTED AND MEASURED (BENKELMAN BEAM) DEFLECTIONS OF TEST SECTIONS

Test section no.	Location	Equivalent WBM thickness (mm)	Deflections		
			Computed	Measured	Difference %
1.	Km 36.477 to 36.777, NH6	310	1.152	1.060	8.0
2.	Km 139.7 to 140.0, NH32	285	0.965	0.838	13.1
3.	Km 256.7 to 257.0, NH5	325	1.124	1.033	8.1
4.	Km 334.6 to 334.9, NH5	290	1.053	0.999	5.1
5.	Km 83.6 to 83.9, NH5	320	1.027	0.992	3.4
6.	Km 274.5 to 274.8, NH5	310	1.049	1.005	4.2
7.	Km 100.0 to 100.2, NH5	385	1.690	1.665	1.5
8.	Km 137.56 to 137.86, NH32	265	1.065	0.931	12.6
9.	Km 134.7 to 135.0, NH33	230	1.166	1.099	4.0
10.	Km 371.0 to 371.3, NH5	255	1.276	1.250	2.0
11.	Km 595.075 to 595.375, NH2	260	1.549	1.480	4.5
12.	Km 607.0 to 607.3, NH2	300	2.019	1.914	5.2
13.	Km 81.4 to 81.7, NH5	215	1.461	1.430	2.1
14.	Km 657.475 to 657.725, NH2	205	1.786	1.680	5.9
15.	Km 134.73 to 135.03, NH6	150	1.373	1.318	4.0

passed on each test section were found. The regression equation is expressed as follows:

$$RD = -0.265 + 6.79 * SVS + 3.08 * ESAL \quad (5)$$

where

RD = average rut depth (mm),

SVS = subgrade vertical strain ($\times 10^{-3}$) obtained from the computer output (RAOPAVE), and

ESAL = cumulative equivalent single-axle loads ($\times 10^7$).

Equation 5 has a squared multiple regression coefficient of 0.951, a calculated F value of 116.0, and a critical F value of 6.93 at 0.01 significance level. Because the calculated F value is greater than the critical F value, the null hypothesis is rejected, leaving the linear terms in the model. It would be useful, and seems a natural extension, to relate rut depth to gravel surfacing depth and cumulative axles. This relationship is shown by the following equation:

$$RD = 3.878 * ESAL - 0.0459 * TWBM \quad (6)$$

where TWBM equals the thickness of WBM, in millimeters.

Equation 6 has a squared multiple regression coefficient of 0.727, a calculated F value of 32.0, and a critical F value of 6.93 at 0.01 significance level.

Comparison of Rutting Model

For granular pavements, the loss of serviceability is caused mostly by excessive permanent deformation along the wheel-

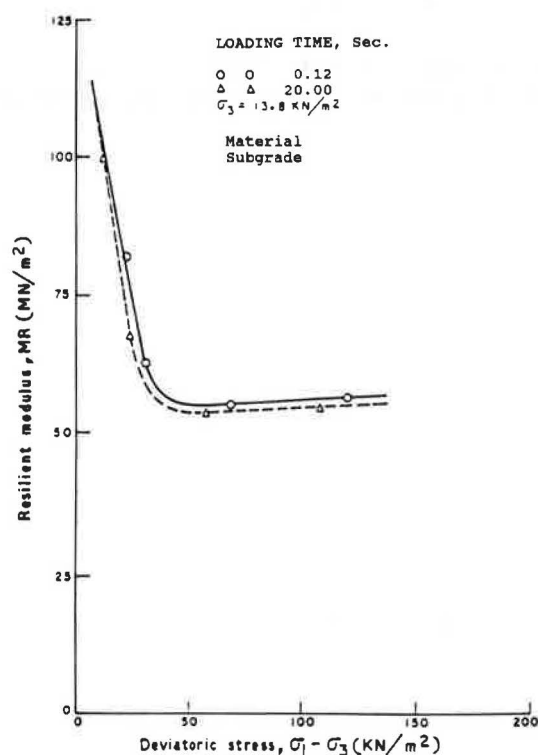


FIGURE 12 Resilient modulus versus deviatoric stress for subgrade soil of Test Section 10.

TABLE 2 SUMMARY OF CHARACTERISTICS OF TEST SECTIONS

Test section no.	Rut depth (mm)	Total area of cracking and patching (%)	Cumulative no. of std. axles (x10 millions)	Subgrade vertical strain (x 10 ⁻³)	Lab. CBR (%)
1.	11.6	1.4	3.303	0.977	6.9
2.	9.4	0.2	0.485	1.059	6.2
3.	13.6	11.6	2.282	1.156	7.1
4.	16.5	13.8	1.446	1.111	7.9
5.	11.2	9.1	1.183	1.147	7.8
6.	13.8	2.3	2.327	1.191	7.5
7.	22.6	16.7	3.822	1.317	3.1
8.	11.9	0.4	2.154	1.350	7.2
9.	21.0	17.9	2.550	1.592	6.6
10.	13.7	1.1	1.446	1.595	4.7
11.	22.8	11.8	3.284	1.659	4.5
12.	25.3	10.1	3.714	2.004	2.4
13.	18.4	4.4	1.183	2.019	5.1
14.	29.8	25.8	4.922	2.458	4.4
15.	25.8	21.0	2.697	2.626	6.1

path. Hence, in this study rutting was adopted as the index for judging pavement performance, which is in agreement with British practice (16,20,22). An average rut depth of 15 mm is considered adequate for an overlay done at the optimal time (20). Putting this value of rut depth in Equation 5, the following rutting model was developed for the traffic range covered in this study:

$$SVS = 4.93 * 10^5 * ESAL^{-1.22} \quad (7)$$

where SVS equals the subgrade vertical strain in 0.001 mm/mm.

Equation 7 was compared with that of Shell (23), as shown in Figure 13:

$$SVS = 2.8 * 10^{-2} * ESAL^{-0.25} \quad (8)$$

The difference in the coefficients of these equations is caused by the difference in the material characteristics. Although the

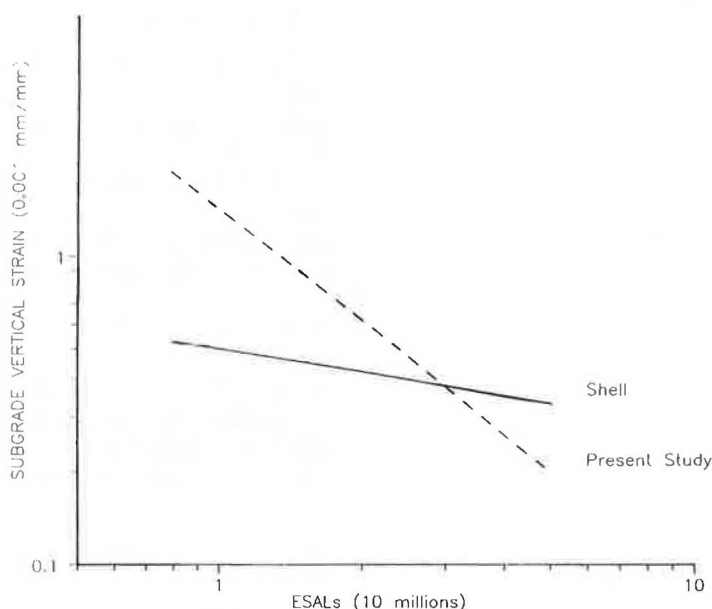


FIGURE 13 Comparison of rutting models.

difference of the corresponding coefficients in the two equations is considerable, the curves are close to one another when plotted. For an ESAL value of 30 million (a typical value—the ESAL range in this study is 8 to 45 million), both equations yielded a subgrade vertical strain of 0.00038 mm/mm, as shown in Figure 13. An average rut depth of 20 mm is considered adequate for the complete failure of a pavement structure (6,19,20). For this rut depth, both equations gave the same subgrade vertical strain (0.000348 mm/mm), for an ESAL value of 42 million.

Determination of Pavement Lives

The cumulative ESALs were found using the value of 20 mm for rut depth (considering a complete failure of pavement) and the subgrade vertical strain (obtained from the computer program) of each pavement section in Equation 5. The life period of the pavement structure was calculated from the current traffic data. The present age was subtracted from the life period of the pavement structure to obtain the remaining life (in years). The results are presented in Table 3. This table shows that, for five test sections (Sections 7, 11, 12, 14, and 15), the pavement life was exhausted because of the high magnitude of subgrade vertical strain and the large number of standard axles passed. For four test sections (Sections 1, 3, 6, and 9), the remaining life period is almost exhausted (less than 3.5 years). For the rest of the six test sections, the remaining life is longer (4.7 through 15 years) because of the low magnitude of subgrade vertical strain and the smaller number of standard axles passed.

Other Relationships

Regression analysis was also carried out between the average surface deflection (d) and the average rut depth (RD), both in millimeters. The equation developed is as follows:

$$d = 0.4838 + 0.0428 * RD \quad (9)$$

For Equation 9, the squared coefficient of correlation is 0.717, the calculated t value is 5.74, and the critical t value is 3.01 (for a significance level of 0.01). This equation was compared with that developed in the AASHO road test (24), which is given as follows:

$$d = 0.0096 + 0.0617 * RD \quad (10)$$

where d and RD are in inches.

For Equation 10, the squared coefficient of correlation is 0.6. The trend and the magnitude of variables are similar in Equations 9 and 10. From the equation developed in this investigation, the surface deflection found is 1.34 mm, whereas in the AASHO road test equation the surface deflection is 1.48 mm, for the rut depth of 20 mm.

The following equation was developed between the total cracking and patching area (CPA), in percent, and the average rut depth (RD), in millimeters:

$$CPA = -9.05 + 1.06 * RD \quad (11)$$

For Equation 11, the squared coefficient of correlation is 0.678, the calculated t value is 5.23, and the critical t value is

TABLE 3 CALCULATED LIVES OF PAVEMENT STRUCTURES

Test section no.	Av. no. of commercial vehicles per day	Present age (years)	Total no. of std. axles for full life period (x10 millions)	Remaining no. of std. axles (x10 millions)	Remaining life (years)
1.	3029	18	4.426	1.123	3.2
2.	1412	7	4.450	3.965	15.8
3.	3906	7	4.031	1.749	3.7
4.	2048	9	4.130	2.684	9.2
5.	1554	10	4.051	2.868	12.9
6.	3983	7	3.954	1.627	3.5
7.	3328	20	3.676	NIL	NIL
8.	1412	27	3.603	1.449	6.6
9.	2276	19	3.070	0.520	2.1
10.	2048	9	3.063	1.617	6.2
11.	3207	16	2.922	NIL	NIL
12.	3627	16	2.162	NIL	NIL
13.	1554	10	2.129	0.946	4.9
14.	4652	17	1.161	NIL	NIL
15.	2349	20	0.790	NIL	NIL

3.012 (for a significance level of 0.01). Equation 11 yields a CPA value of 12 percent when compared with the total pavement area, or 33 percent when compared with the wheelpath area, for a rut depth of 20 mm. For thin asphalt pavements, Craus et al. (25) suggest 30 percent cracking under the wheelpaths when limiting conditions are approached.

Limitations and Conclusion

The equations developed here are valid only for the range of conditions evaluated in the study (e.g., traffic is 8 to 45 million ESALs, CBR of soil is 3 to 8 percent, and surface deflection is 0.8 to 1.9 mm). The purpose of Equations 9 and 11 is to compare the results with those of the AASHO road test and of Craus et al. (25). For designing the overlays and to ensure longer serviceability, Equation 5 can be used to monitor the performance at any stage during the life period of a pavement.

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