Subgrade Modulus on the San Diego Test Road

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The San Diego County Experimental Base Project, which was constructed in 1966 and continued until 1973, was a test road consisting of 35 different sections designed to measure the performance of granular and asphalt-bound base courses. This paper presents an analysis of the test-road subgrade and its variability. Multiple regression analysis has shown that the subgrade resilient modulus can be correlated with the soil moisture and degree of saturation and that the ratio of the laboratory modulus to the field modulus is near unity for high degrees of saturation. For lower degrees of saturation (80 to 85 percent), laboratory-compacted samples tended to have higher values of the modulus than did field-compacted samples. On the average, for treated base sections, the in-place modulus predicted from regression analysis gave excellent agreement with values of the modulus derived from Benkelman-beam deflections in a multilayer elastic analysis. Because of an apparently stiffer in situ response of granular base material than was predicted in the laboratory, similar comparisons for granular base sections were not good. The modulus of granular base materials averaged six times greater in the field than in the laboratory. The test-road subgrade was not uniform, but was highly variable between and within test sections. Resilient-modulus tests of undisturbed field cores had a coefficient of variation of the modulus of approximately 97 percent between test sections. The coefficient of variation within test sections averaged approximately 40 percent.

The San Diego County Experimental Base Project was a test road consisting of 35 different sections designed to measure the performances of granular and asphalt-bound courses. The objects of the experiment were to determine the specific thicknesses of various types of base courses required to give a desired level of performance, to relate the properties of the pavement to its observed performance, and to study deflection and strain behavior for translating performance results to other environments. The test road was completed in August 1966, and performance observations were continued until 1973.

The test road was designed for six different types of base at four levels of thickness. The bases included asphalt concrete, asphaltic-cement treated, cutback treated, emulsion treated, and California classes 2 and 3 aggregate. One additional type of base at two levels of thickness and two sections of a standard California design were also included. The thickness and quality of the asphalt-concrete surface and the quality of the clay subgrade were designed to be uniform throughout the length of the project.

Detailed descriptions of the design and construction of the road have been reported by Riley and Shook (1) and by Kingham (2). The performance of the test sections have been given by Hicks and Finn (3) and by Shook and Lambrechts (4). Hicks and Finn (3), in their performance study, cited considerable ambiguity in defining the resilient modulus of the subgrade with time and between test sections. The absence of subgrade-soil test data was a major obstacle in the development of fatigue relations from the data.

This paper presents an analysis of the subgrade modulus, the methods of its measurement, and the correlations and associated variability between and within test sections. The methods of measurement included laboratory tests on compacted and undisturbed specimens and modulus estimates determined from Benkelman-beam deflections and multilayered elastic theory. It also includes an analysis of the in-situ response of the unbound, granular base-course materials.

SUBGRADE CONSTRUCTION

The subgrade soil for all test sections was an A-7-6. Special contract provisions required that there be a minimum of 0.6 m (2 ft) of the clay under each test section. The original ground was excavated to a depth of 0.6 m (2 ft) below subgrade and, if a suitable clay existed at the bottom of the excavation, the clay was backfilled and compacted. If material other than clay was found at the bottom of the excavation, an additional 0.3 m (1 ft) of material was excavated before backfilling. The contract specifications required that the clay backfill be compacted to at least 90 percent of its maximum density within a moisture-content range of -2 to +3 percent of optimum.

Subgrade compaction-test results for each of the test sections have been reported by Kingham (2). All of the test sections were compacted in excess of the minimum of 90 percent. The variations in dry density among the test sections ranged from 1672 to 1901 kg/m^3 (104.4 to 118.7 lb/ft^3). The moisture contents ranged from 11.4 to 18.0 percent.

SUBGRADE RESILIENT MODULUS

The background of the resilient modulus test has been given by Seed, Chan, and Lee (5). In this test, the modulus of resilient deformation ($\overline{M}_R$) is determined from repeated-load, triaxial compression tests. The modulus is computed as the ratio of the deviator stress to the resilient axial strain.

The modulus of fine-grained soils depends on both the deviator and the confining stresses, with the deviator stress being predominant. The modulus generally decreases with increasing deviator stress and decreasing confining pressure. The modulus is also significantly affected by the moisture content and the dry density of the soil.

Laboratory-Molded Samples

The resilient modulus of the subgrade soil was determined on laboratory-molded samples by the Asphalt Institute. Specimens 15.2 cm (6 in) in diameter and 30.5 cm (12 in) high were compacted at varying moisture contents according to the American Society for Testing and Materials Test Method D-1557. Two compactive forces [862 and 584 kJ/m^3 (18 000 and 12 000 ft·lb/ft^3)] were used. The deviator-stress levels were 41.4, 82.7, and 124.1 kPa (6, 12, and 18 lbf/in^2), and the ratios of the deviator stress to the confining stress were 3.0 and 1.5. Additional tests were conducted in an unconfined state. A complete summary of these test results has been given by Jones (6) and Kallas and Shook (7). The relation is between the $\overline{M}_R$ and the dry density versus the moisture content at the two compactive forces are shown in Figure 1.

To consider the effect of the overall physical state of the soil on the modulus, the degree of saturation (S) was computed and used in a multiple regression analysis with the modulus and the moisture content (Figure 2). To remove the influence of the stress state, all of the values of M_R used in the analysis corresponded to a deviator stress of 41.4 kPa (6 lbf/in^2) and a confining stress of

Figure 1. Effect of compactive effort on subgrade M_R (laboratory-molded samples).

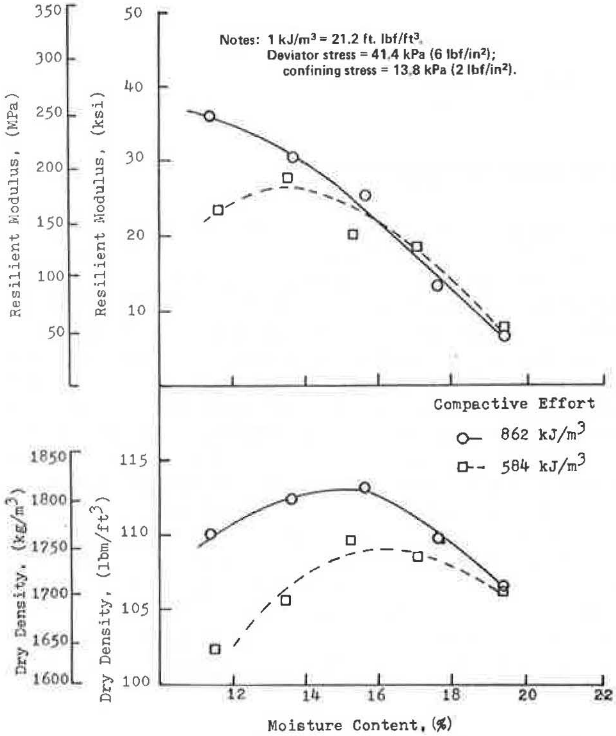
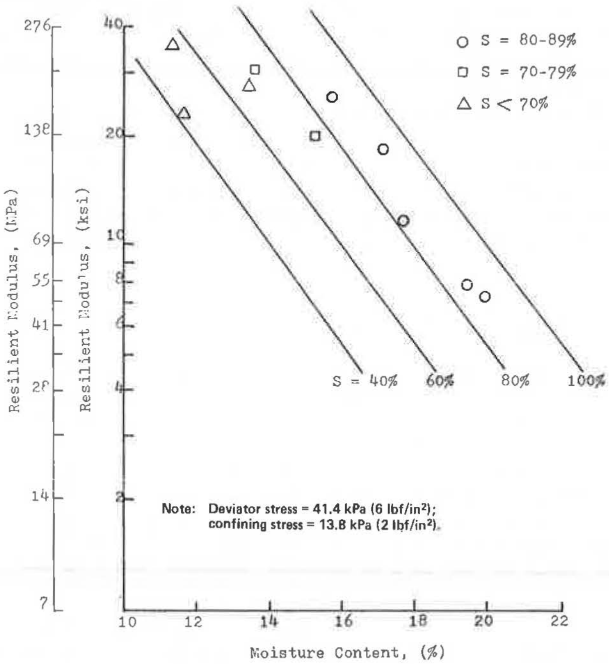


Figure 2. Relations between M_R, moisture content, and degree of saturation (laboratory-molded samples).



13.8 kPa (2 lbf/in²). The regression equation developed is given below.

$$\log M_R = -0.13\,282W + 0.013\,405S + 2.31\,909 \quad (n = 10 \text{ and } r = 0.97) \tag{1}$$

where W = percent moisture content. (SI units are not given for the variables in the equations in this paper because they were derived for U.S. customary units.)

A multiple variable model with limited test points may not, by itself, be statistically sound. However, the model selection was identical to the best-fit model developed from a larger data base of modulus tests on undisturbed specimens. The use of a similar model permitted comparisons between the two methods of prediction.

Undisturbed Field Samples

In 1970, undisturbed field samples from six test sections were tested for resilient modulus by the Asphalt Institute. In April and May of 1973, an extensive subgrade sampling program was conducted by San Diego County. The Asphalt Institute determined the M_R, moisture content, and dry density of undisturbed field samples from representative areas. These tests were conducted over a range of stress states; the complete test results have been given by Jones (6) and Kallas and Shook (7). Table 1 summarizes the test results for the subgrade for a deviator stress of 41.4 kPa (6 lbf/in²) and a confining stress of 13.8 kPa (2 lbf/in²). Figure 3 shows the range of test results at two different states of stress.

The effects of moisture and saturation on the M_R of the field samples were similarly examined. The values of the M_R from the first and second 0.3 m (1 ft) of the subgrade were combined for the 1970 and 1973 test series for the multiple regression analysis (Figure 4). The stress state chosen was the same as that chosen for the laboratory-molded samples. The derived relationship is shown below.

$$\log M_R = -0.111\,109W + 0.021\,699S + 1.17\,869 \quad (n = 97 \text{ and } r = 0.67) \tag{2}$$

The general pattern of Figure 4 is similar to the relationship developed from the laboratory-molded samples (Figure 2). A comparison of the two regression models is shown in Figure 5. There is good agreement at high degrees of saturation within the limits of moisture contents obtained (approximately 12 to 20 percent).

At high saturation levels, the modular ratio of laboratory-molded to field samples is near unity, which increases confidence in the use of laboratory-compacted samples for the evaluation of in situ modulus. However, for lower degrees of saturation, unconservative estimates of the field modulus may also be obtained with laboratory-compacted samples.

Specific reasons for this have been difficult to ascertain. However, Seed and others (5) have found that the method of compaction and the degree of saturation are only two of the many variables that affect the elastic modulus of clay. When cohesive soil samples are com-

Table 1. Tests on undisturbed field samples (April and May 1973).

Subgrade Depth[a] (m)	No. of Samples	Moisture Content (%) Mean	Moisture Content (%) Standard Deviation	Dry Density (kg/m³) Mean	Dry Density (kg/m³) Standard Deviation	Resilient Modulus (MPa) Mean	Resilient Modulus (MPa) Standard Deviation
0 to 0.30	44	16.9	2.1	1818.3	72.1	203.4	198.6
0.30 to 0.61	35	15.5	2.9	1792.6	73.7	182.7	160.6

Notes: 1 MPa = 145 lbf/in²; 1 m = 3.28 ft; 1 kg/m³ = 0.0624 lb/ft³.
Deviator stress = 41.4 kPa; confining stress = 13.8 kPa.

[a]Outer wheel path.

pacted at low degrees of saturation in the laboratory, little or no significant shearing strain is induced by any of the conventional methods of compaction, which results in the soil attaining a flocculated structure. When samples are compacted at a high degree of saturation (approximately greater than 80 percent), both kneading and impact compaction cause shear deformations that result in a more dispersed soil structure. For static compaction procedures, a flocculent structure occurs for all levels of saturation. Dispersed structures generally have higher deformations in the resilient-modulus test and consequently lower moduli.

The mean values of the degree of saturation for both the laboratory-molded samples and the initial undisturbed field samples were nearly identical (74 versus 75 percent), and one would not expect very large differences in the computed results with the two regression models.

Figure 3. Subgrade M_R test results (undisturbed specimens).

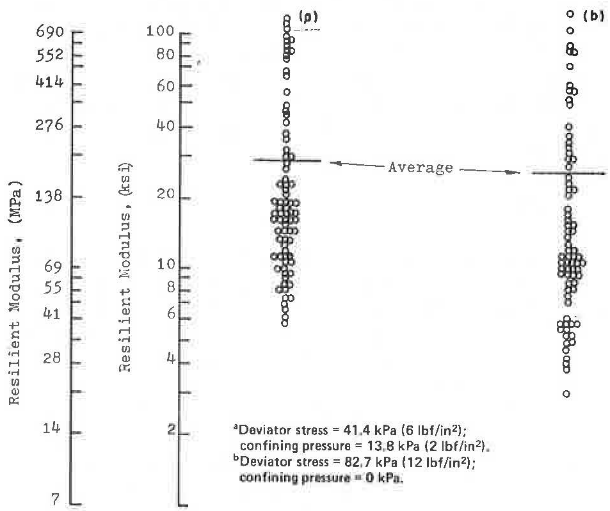
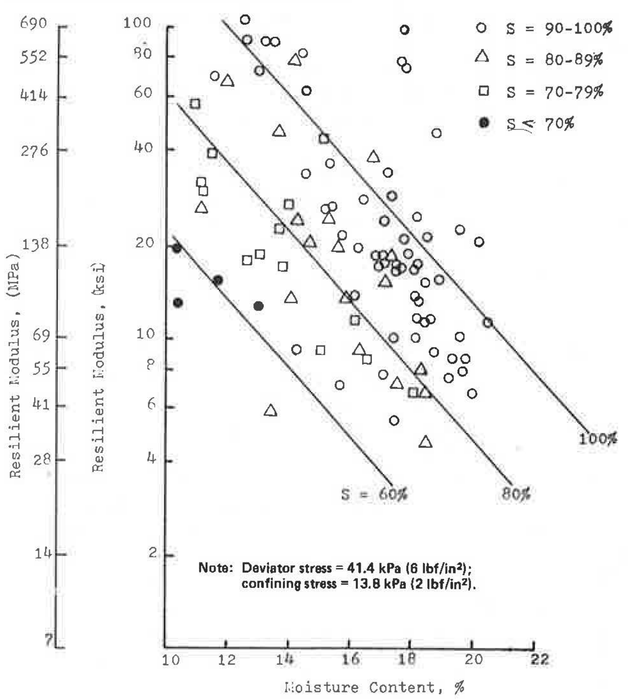


Figure 4. Relations between M_R, moisture content, and degree of saturation (undisturbed field samples).



However, the impact type of laboratory compaction does not precisely simulate the in-service condition. As was shown by Jones (6), the mean subgrade saturation increased with time from the as-built value (74 percent) to a value greater than 90 percent. The structure of the subgrade soil particles was probably more flocculent for the entire period of service. One-half of the laboratory-molded samples were impact-compacted at a saturation level greater than 80 percent and would therefore exhibit a dispersed structure. These differences in soil structure may somewhat explain the differences in the two regression models.

The initial subgrade-compaction data for each test section were input into the two regression equations to predict the initial, in-situ subgrade M_R in each test section. The predicted values derived from Equations 1 and 2 for the treated and granular base sections will be summarized.

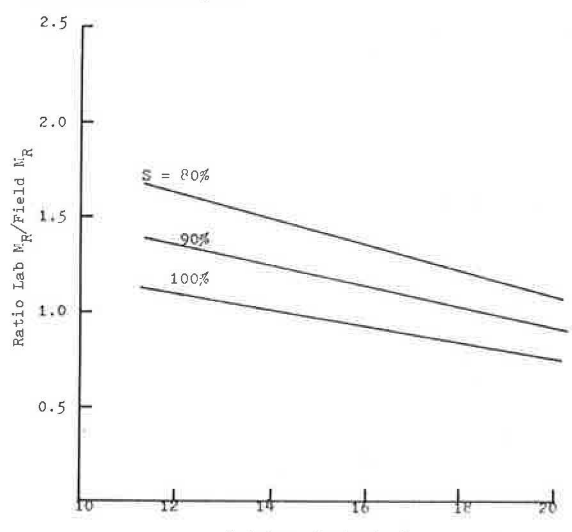
Derivation of Subgrade Moduli From Deflections

A major element in the San Diego measurements program was the measurement of the pavement-surface deflections with a Benkelman beam. The deflection measurements were made at least once a year over the entire test road and at more frequent intervals in test sections having high deflections. The measurements were obtained with a dual-tire-configuration, 80-kN (18 000-lbf/in²) axle load. The average pavement temperatures were also recorded.

A multilayer, linear-elastic system and the computer program, NLAYER, described by Michelow (8) were used to compute subgrade modulus values from the deflections. The elastic properties of all other pavement materials, determined from laboratory testing, were used as input to the analysis.

The basic approach used to develop a history of the subgrade modulus versus time for each test section was to develop a family of curves for each test section that related the computed deflection from the NLAYER program to the subgrade modulus and the temperature corresponding to an elastic modulus of the bound materials. By using these curves with the measured deflections and temperatures, an effective subgrade modulus was determined at which the measured deflections equaled the predicted deflections.

Figure 5. Relation between M_R of laboratory-molded and undisturbed field samples.



For pavement base-course materials having stress-dependent moduli, a layered-stress iteration procedure was used. The modulus of granular material and of emulsion and cutback-treated base in an uncured state can be expressed by the equation $M_R = k_1\theta^{no}$; where k_1 and no. are experimentally derived constants and θ is the first stress invariant.

The laboratory-derived values of k_1 and no. for unbound granular materials when tested at optimum moisture are given below (6, 7).

Aggregate Material	k_1	no.
Class 2 base	1880	0.64
Class 3 base	2101	0.57
Class 4 subbase	1944	0.59

The initial series of computer solutions for the class 2 and class 3 granular-base sections using the values of k_1 and no. given above gave exceedingly high values of deflection within the base. The ranges of computed deflections varied from the measured field deflections, particularly in the earlier stage of the project, leading to the conclusion that the base moduli values for granular materials in the field were higher than those determined from laboratory testing. A new M_R versus θ relationship, $M_R = 4000\ \theta^{0.5}$, was then selected for the analysis. This relationship differed only slightly from that used by Hicks and Finn (3) in their analysis of the San Diego data. (Later in this paper the actual in-situ response of the granular materials will be discussed more fully.)

Partial results of the deflection versus modulus analysis are given in Table 2. The derived moduli for the untreated base sections are significantly higher than those for the treated base sections, even though a higher M_R versus θ relationship than that predicted in the laboratory was used in the analysis.

Table 3 shows that the subgrade modulus values for the granular base sections do not compare well with the values predicted from the previously given regression models and the subgrade compaction tests. The high values predicted from the deflection analysis may be the result of the NLAYER-program prediction of high deflections within the base. It appears, therefore, that the granular base exhibits a stronger response in the field than was indicated by laboratory tests or the assigned modulus function ($M_R = 4000\ \theta^{0.5}$).

The values of the mean initial modulus given in Table 2 for the treated base sections also show some variance between the asphalt-concrete and asphalt-cement-treated bases versus those composed of the emulsion and cutback-treated materials. However, the differences among the treated bases are much smaller than the differences between the treated and untreated bases. The asphalt-concrete and asphalt-cement-treated bases are less complex and can be analyzed as two-layer systems in which the percentage of cure and the stress state are insignificant. However, an accurate measure of the in-situ modulus of the emulsion and cutback-treated bases is dependent on reliable estimates of both the percentage of cure and the in-situ stress state. Consequently, these sections would probably have the most reliable measure of subgrade M_R, which would be (initially) about 172 400 kPa (25 000 lbf/in^2). The difference between this value and the average modulus for all treated sections [138 600 kPa (20 100 lbf/in^2)] is relatively small and may be within the range of error introduced in the deflection-derived modulus study.

Table 2. Predicted initial subgrade modulus from deflections.

Type of Base	Derived Subgrade Modulus (MPa) Mean	Standard Deviation
Asphalt concrete	183.4	40.7
Asphalt-cement treated	166.9	57.2
Emulsion treated	116.5	40.7
Cutback treated	100.7	11.7
Class 2	326.8	175.1
Class 3	370.9	75.2
Standard California design	362.0	24.1
All treated	138.6	51.0
All granular	350.9	117.2

Note: 1 MPa = 145 lbf/in^2.

Table 3. Initial subgrade M_R from three methods of measurement.

Section and Method of Measurement	M_R (MPa) Mean	Standard Deviation
Treated base		
Benkelman-beam deflections	138.6	51.0
Regression model from undisturbed field samples[a]	138.6	53.1
Regression model from laboratory-molded samples[a]	224.1	64.1
Granular base		
Benkelman-beam deflections	350.9	117.2
Regression model from undisturbed field samples[a]	121.3	22.1
Regression model from laboratory-molded samples[a]	217.2	65.5

Note: 1 MPa = 145 lbf/in^2.
[a]All regression models based on a deviator stress of 41.4 kPa and a confining stress of 13.8 kPa.

Comparisons

Table 3 gives a comparison of the predicted initial M_R values and the standard deviations computed for each of the prediction methods for both treated and unbound base sections. For the treated base sections, the mean values of M_R computed from the deflection measurements are in excellent agreement with the mean values predicted by the regression model from the field cores. However, the mean values of the deflection-derived M_R for granular sections do not compare well, and these values are not considered representative of the actual subgrade stiffness.

For the treated base sections, the mean values of the initial M_R determined from the laboratory-molded samples are about 50 percent higher than those predicted by deflections or by the field cores (Figure 5). The mean subgrade saturation at the time of compaction was about 75 percent, and the mean moisture content was about 14 percent. Thus, from Figure 5, one would expect higher predicted values from the laboratory samples. However, cohesive subgrades tend to gain moisture with time, and their saturation levels approach 90 to 100 percent. This was true on the San Diego Test Road. Consequently, in terms of the longer term or ultimate subgrade modulus in the field, tests on laboratory-molded samples should give reasonable estimates.

SUBGRADE VARIABILITY

Analysis of the M_R values from tests on field cores and those values derived from deflections gave results indicative of the large variations in the test-road subgrade. For example, the results of the 1973 field-sampling program, summarized in Table 1, showed the coefficient of variation (CV) of the modulus in the top of the subgrade, between test sections, to be 97 percent. When analyzed for variability within test sections, the field cores had a mean CV of the M_R of 40 percent in the top half of the subgrade.

The standard deviations of the deflection measurements within test sections were also used to estimate the magnitude of the subgrade variability. From the family of curves used to develop moduli from deflections, the range in M_R corresponding to the range of plus and minus one standard deviation of deflection was determined. From this analysis, a mean standard deviation and CV of the M_R was computed for the initial deflection-measurement period and for each deflection measurement in the pavement history. The computed average values of the CV within test sections for the entire length of the test road are given below.

At	Average Values (%) From Deflections	From Field Cores
Project initiation	17	–
Project termination	38	40

(The value given from the deflection measurement at project termination represents the average for all deflection measurement periods, and the value given from the field cores at termination is from the 1973 sampling program.)

A number of inferences can be drawn from this table. The variation of the initial subgrade M_R as measured by deflections is within reasonable limits. Although a small number of test sections showed high variations, the overall mean CV of 17 percent is within the normal limits of control of pavement construction. The values of CV at the time of the project termination indicate that the variability has increased with time. This high degree of variability has undoubtedly occurred because of nonuniform changes in the physical state of the clay soil. The mean CV of the modulus as determined from field samples agrees well with the mean value computed from deflections, i.e., 40 percent from field cores versus 38 percent from deflections. The values computed from deflections represent the average variation for all periods of the deflection measurement. Thus, the high variability in the subgrade probably existed earlier in the project history than is indicated by the 1973 coring program.

The variations of the in-situ subgrade M_R are generally much greater than the variations created by differences in stress state. Test data for eight of the subgrade cores taken in 1973 were randomly chosen and their CV of M_R computed over five different states of stress. The average for the eight cores was 18 percent. Thus, the influence of stress state in the determination of the in-situ modulus is secondary when compared to the physical variations in the soil. This can also be seen in Figure 3, where the range of core-test results is significantly greater than the range in average results due to differing states of stress.

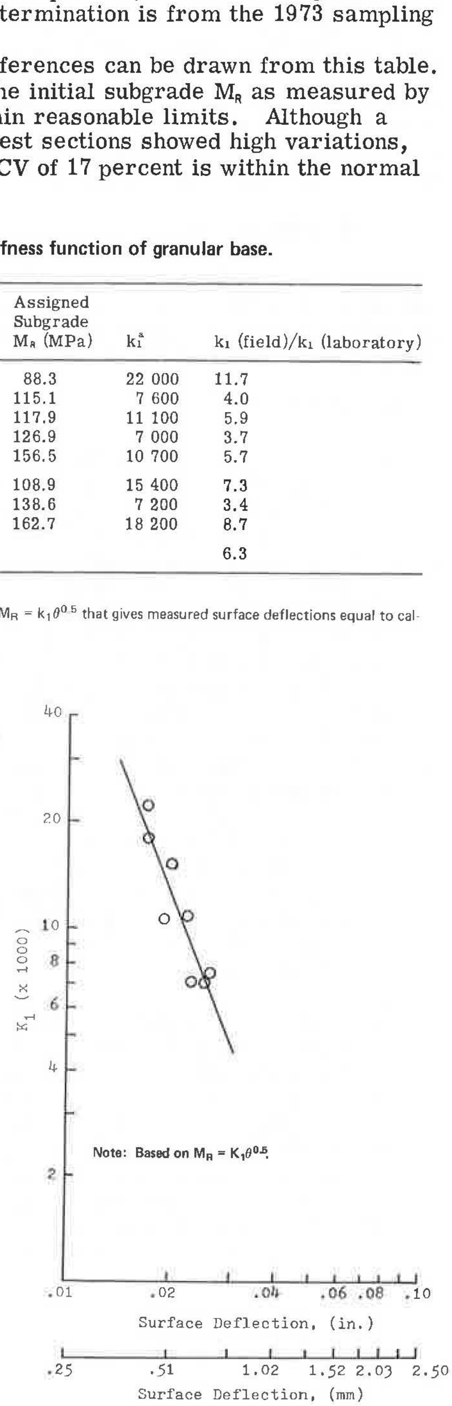
The San Diego Test Road was a controlled experiment, and the subgrade was designed and constructed to be a constant in the experiment. The wide variations that have, in fact, existed must cast doubt on the meaning of the relative performance observations and records. Only through an extensive analysis of the performance, properly weighted against the subgrade stiffness and its variation, can any meaningful conclusions relative to type of base be drawn.

Table 4. Estimated stiffness function of granular base.

Type of Base	Section	Assigned Subgrade M_R (MPa)	k_1^a	k_1 (field)/k_1 (laboratory)
Class 2 aggregate	1	88.3	22 000	11.7
	10	115.1	7 600	4.0
	11	117.9	11 100	5.9
	14	126.9	7 000	3.7
	24	156.5	10 700	5.7
Class 3 aggregate	8	108.9	15 400	7.3
	18	138.6	7 200	3.4
	26	162.7	18 200	8.7
Avg				6.3

Note: 1 MPa = 145 lbf/in^2.
[a]Value of k_1 in the expression $M_R = k_1\theta^{0.5}$ that gives measured surface deflections equal to calculated surface deflections.

Figure 6. Variation of computed granular-base M_R with deflections.



ESTIMATED MODULUS OF GRANULAR BASE SECTIONS

The results of the derivation of subgrade modulus values from deflections indicate that granular base material exhibits a stronger response in the field than is indicated by laboratory testing. The initial modulus values of the subgrade derived from deflections averaged 136 MPa (20 100 lbf/in^2) for treated base sections and 346 MPa (50 900 lbf/in^2) for granular base sections. The function $M_R = 4000\,\theta^{0.5}$ was used in the analysis for all granular materials.

To ascertain the modulus of the granular base sections that was being exhibited in the field, new values of subgrade M_R were assigned to eight of them. The object of the analysis was to determine the value of k_1 in the expression $M_R = k_1\theta^{0.5}$ such that the computed deflections were equal to the measured deflections. By using the NLAYER program and a trial-and-error procedure, the analysis was conducted for the initial deflection-measurement period.

The computed values of k_1 are given in Table 4. Analysis of these values indicates no evident dependence on the type of base, thickness, or location in the test road. The derived values were, however, related to the surface deflections, as shown by Figure 6 and Equation 3.

$$\log k_1 = -2.4529 \log \Delta + 7.2931 \quad (r = 0.91) \tag{3}$$

This figure, which indicates decreasing k_1 with increasing deflection, suggests that the modulus function of the base cannot be taken as a unique value, but varies with the pavement section. Each of the derived values exceeds the laboratory values by a substantial factor. The average of the derived values of k_1 for the eight sections analyzed was 12 400. This compares with the 1880 determined in the laboratory for class 2 material and the 4000 used in the previous derivation of subgrade modulus values. As shown in Table 4, the ratio of k_1 for field to laboratory conditions ranged from 3 to 12, with an average near 6.

The high computed k_1-values help to explain why the computed subgrade modulus values in granular base sections are considerably higher than those in the treated base sections. The use of a higher k_1-value in the previous derivations would have reduced the magnitude of the subgrade modulus to values consistent with those derived for the other test sections.

It has not been the intent of this study to ascertain the reasons for the seemingly high values of k_1. Laboratory M_R tests approximate as closely as possible the loading conditions that are believed to exist in the field. These results are important, however, in that they suggest that the field behavior of granular base pavements is more complex than previously believed and that multilayer elastic analysis underpredicts the in-situ state of stress. This is an area that needs further research. The implications of the high response of granular bases is extremely important in the use of rational design procedures based on limiting values of tensile and compressive strains.

CONCLUSIONS

The following conclusions are presented.

1. The M_R of a cohesive subgrade soil is dependent on the physical soil state. Multiple regression models using moisture content and degree of saturation were developed to predict the modulus for both laboratory compacted and undisturbed field samples. A comparison of the two regression models indicates that the ratio of laboratory M_R to field M_R is generally equal to unity for high degrees of saturation (S > 90 percent) for the range of moisture contents obtained.
2. For lower degrees of saturation (S < 80 to 85 percent), the laboratory-compacted samples may have higher values of M_R than do undisturbed field samples having identical physical soil states. Although more research is needed, it is believed that variations in the clay structure, produced by differing methods of compaction, may cause the variations in the measured modulus.
3. The best estimate of the mean initial subgrade modulus for the entire test road is about 138 000 to 172 500 kPa (20 000 to 25 000 lbf/in^2).
4. For test sections made of asphalt-treated base materials, the mean deflection-derived subgrade modulus of 138 000 kPa (20 000 lbf/in^2) agreed well with the mean value of modulus predicted from the regression models developed from direct laboratory testing of field samples.
5. In general, the deflection-derived values of the initial subgrade modulus for both emulsion and cutback-treated base sections averaged about 69 000 kPa (10 000 lbf/in^2) lower than those derived for asphalt-concrete and asphalt-cement-treated base sections. This indicates that, for bases having temperature and stress dependency and state-of-curing effects on the modulus, the treated bases may have been exhibiting a lower response than that determined from laboratory modulus evaluation.
6. Resilient-modulus testing of undisturbed cores taken 7 years after the test road was opened to traffic showed that the CV of the modulus between test sections was 97 percent.
7. The CV of the modulus within test sections averaged 17 percent initially, but increased with time. The average CV over the project life (approximately 7 years), as determined from surface deflections, was 38 percent. This value is in excellent agreement with the CV of 40 percent determined from laboratory testing of undisturbed cores.
8. Variations of the subgrade modulus due to the physical state of the soil are of much greater significance in analyzing pavement performance than is the range in modulus values due to differing stress states normally imposed in the laboratory.
9. In the derivation of the subgrade modulus from deflection measurements, for the granular-base test sections, the comparison between the derived moduli and those determined from direct laboratory testing on field cores was very poor. For these types of bases, the stress-dependent function for modulus of the base, determined from laboratory tests, tended to underpredict the in-situ response of the granular material. When the apparent in-situ granular moduli were recomputed from the initial surface deflections, an adjusted value of k_1 ($M_R = k_1\theta^{0.5}$) ranged from 7000 to 22 000 with a mean of 12 400. In general, the ratio of the k_1-term derived from the field to that found in the laboratory ranged from about 3 to 12 with an average near 6.

ACKNOWLEDGMENTS

We wish to acknowledge the financial and technical support of the Asphalt Institute, which made this research possible. Particular gratitude is expressed to James Shook and Bernard Kallas of the institute staff.

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Publication of this paper sponsored by Committee on Strength and Deformation Characteristics of Pavement Sections.