SOIL MECHANICS AND BITUMINOUS MATERIALS
RESEARCH LABORATORY

ASPHALT MIXTURE BEHAVIOR IN REPEATED FLEXURE

by

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STATE OF CALIFORNIA

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THE UNITED STATES DEPARTMENT OF COMMERCE
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Soil Mechanics and Bituminous Materials

Research Laboratory

ASPHALT MIXTURE BEHAVIOR IN REPEATED FLEXURE

A report on an Investigation

by

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to

The Materials and Research Department
Division of Highways
State of California
Under

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INTRODUCTION

To better define the fatigue behavior of asphalt mixtures resulting from repeated flexure, a four-year program is currently underway at the Soil Mechanics and Bituminous Materials Laboratory of the University of California supported by a research grant from the Materials and Research Department of the California Division of Highways.

The initial phase of this project has been concerned with the definition of the influence, in controlled (or constant) strain tests, of asphalt source and hardness and aggregate type on the fatigue characteristics of asphalt concrete conforming to the current State of California specification requirements. This portion of the investigation has been substantially completed and the results are summarized in this report.

The second phase of the project is concerned with the possible application of laboratory-determined fatigue data to the performance of in-service pavements. To this end, a series of investigations have been instituted on the various components of the pavement section of the Gonzales By-Pass (Project V - Mon - 2 - C). These have included: (1) laboratory fatigue tests both on specimens sawed from the asphalt concrete courses and on laboratory-prepared specimens with the same components, and (2) repeated load tests on the base and subgrade materials.

In determining the laboratory fatigue characteristics of the asphalt concrete specimens, two modes of loading are available, controlled strain and controlled stress. Information is presented in the report to assist in the selection of a particular mode of loading depending on the pavement section under study. On the basis of this analysis, the controlled-stress test was used to define the fatigue behavior of the asphalt-treated materials in the Gonzales pavement, the section for which includes both an asphalt concrete surface and base course.

Since the stresses and/or strains which appear to be the fatigue determinant in the asphalt concrete are dependent on the resilient or elastic response of the underlying materials as well as the stiffness of the asphalt concrete course, test results are also reported for the resilient response of the untreated aggregate and subgrade soil in the pavement section.

Suggestions are also presented for various procedures by which the fatigue factor may eventually be included in the design of and specifications for asphalt concrete pavements.

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CONTROLLED-STRAIN TESTS

Materials

In the controlled-strain tests, five asphalts and two aggregates have been studied. For convenience the designation system for the asphalts is included in Table 1. Detailed test results on these asphalts have been included in Report No. TE 63-2 (1).

The two aggregates, Watsonville granite and Cache Creek uncrushed gravel, were separated into individual size fractions and recombined to meet the 1964 State of California specifications for a 1/2 in. maximum size, medium grading. Average grading curves for both materials are shown in Fig. 1.

Weight-volume relationships for the mixtures prepared from these aggregates were determined using specific gravity values shown in Table 2.

Specimens

Mixture specimens were prepared by kneading compaction using the Triaxial Institute kneading compactor and sawed with a diamond-tipped table saw to final dimensions of 2.0 × 2.7 × 12 in. A summary of the individual mix characteristics for the various specimens is shown in Table 3. As discussed in earlier reports (1, 2) the air void content is shown as a measure of mix uniformity.

Table 4 contains a summary of the modulus of rupture data for unflexed specimens tested under a uniform moment (third point loading) at a rate of deformation of 0.25 in. per min. at the particular test temperature. From the data presented in this table it will be noted that the specimens prepared for all test series had about the same degree of uniformity as far as flexural characteristics are concerned.

Test Procedure

Controlled-strain fatigue tests were performed by subjecting the beam specimens to constant-strain amplitudes monitored by two variable resistance bonded wire strain gages nominally one inch in length cemented with a thin film of epoxy cement to the lower surface of each beam in the region of maximum moment. The actual equipment to develop the fatigue data has been described in Reference (1).

The criterion for service (fatigue) life in these tests is defined as that number of

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repetitions corresponding to a reduction in modulus of rupture to 90 percent of the original unflexed value and is illustrated in Fig. 2 for tests at 68°F with specimens containing Watsonville granite and asphalt S-2.

It should again be emphasized that in these tests, as the name implies, strain was maintained constant throughout the repeated loading process. Generally, as damage from repeated flexing developed in the specimens, it was necessary to decrease the applied load as illustrated in Fig. 3, this decrease being more pronounced at the lower temperature. This is in contrast to the controlled-stress or controlled-load test where, as damage develops, the strain or deflection increases. An indication of the change in load during the tests is presented in Table 5 for tests at 40°F and in Table 6 for tests at 68 and 75°F.

By performing tests at a series of strains, relationships between strain and cycles to failure were established for the various mixes studied in this investigation.

Test Results

Strain vs. cycles to failure relationships for the Watsonville granite and asphalt S-2 are shown in Fig. 4 for tests at 40°F and 68°F. Similar relationships for the Cache Creek gravel and asphalt S-1 are shown in Fig. 5 for tests at the same temperatures. Also shown in Fig. 5 is a relationship between strain and cycles to failure at 68°F for the same material but with improper end clamping of the beam specimens, which emphasizes the importance of proper test conditions to insure the development of meaningful test results.

A comparison between test results for the specimens containing Cache Creek gravel and Watsonville granite with asphalt S-1 are presented in Fig. 6. For purposes of comparison, an estimated curve for the Watsonville granite specimens at 68°F is shown since limited data for these specimens were obtained at 75°F. This comparison demonstrates the importance of aggregate type on fatigue behavior, at least to the extent that this characteristic influences asphalt content. The specimens containing the crushed granite were tested at an asphalt content of 5.9 percent (by weight of aggregate) whereas those containing the uncrushed gravel were tested at an asphalt content of 4.5 percent, both asphalt contents having been selected in accordance with the State of California mix design procedure (1, 2).

At 40°F, the difference in behavior between the two materials may be even more

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pronounced than that shown because the average void content of the compacted granite specimens was about 5.5 percent whereas that of the gravel specimens was about 3.6 percent. According to data presented by Saal and Pell (Fig. 14, Reference (2)), this difference in void content could result in a factor of approximately two in cycles to failure. Thus the curve for the granite specimens would be displaced to the right in Fig. 6 if a comparison was to be made at the same void content as for the gravel specimens.

Fig. 7 contains a comparison between test results obtained at 40°F for the granite specimens containing the different asphalts. The strain vs. cycles to failure relationships for the specimens containing the California asphalts (G, S-1, S-2, and S-3) are all essentially parallel whereas that for the specimens containing asphalt E (prepared from midcontinent stocks*) has a slightly steeper slope. This relationship has been rechecked and apparently is not due to clamping difficulties such as that illustrated in Fig. 5 but is in all probability due to the nature of the material itself.

In general, in controlled-strain tests one would expect that the less stiff the material, the longer would be the fatigue life at a particular strain level. For the asphalts from the same source, i.e. S-1, S-2 and S-3, this appears to be the case since S-3 is 120-150 pen. asphalt, S-1 is an 85-100 pen. material and S-2 is a 40-50 pen. asphalt cement. The comparison between the specimens containing asphalts S-1 and S-3 may be slightly confounded, however, by the fact that the mixtures containing asphalt S-3 are slightly more dense (4.3 percent air voids) than those containing asphalt S-1 (5.5 percent air voids) and the load to produce a strain of 350×10^{-6} in. per in. is slightly less for the specimens containing asphalt S-1 than those with asphalt S-3 (Table 5).

From these data it also appears that asphalt source may have an influence since the data for specimens containing asphalt G lie to the left (fewer cycles at a particular strain level) of that for asphalt S-2 whereas the specimens containing asphalt S-2 are considerably stiffer than those prepared with asphalt G (Table 5).

Pell (3) has suggested a relationship between strain and cycles to failure of the form:

$$N = K\left(\frac{1}{\epsilon}\right)^{6} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (1)$$

*Asphalt prepared from stock which contained 50 percent West Texas crude and 50 percent Wyoming crude according to the report on the AASHO Road Test, HRB Special Report 61B.

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where:

N = cycles to failure

ϵ = bending strain

K = constant depending on mixture

for tests on mixtures subjected to constant strain amplitude fatigue tests. In Fig. 7, a line has been plotted in accordance with this relationship. It will be noted that the relationship between strain and cycles to failure for all of the asphalts produced from the California sources are parallel to this line, indicating that equation (1) could be used to define the behavior obtained in these tests with the value of K for these tests dependent on asphalt type, degree of compaction and asphalt content (e.g., specimens containing asphalt G were prepared at an asphalt content of 5.6 percent whereas those with the S-series asphalts at an asphalt content of 5.9 percent).

Similar data to those shown in Fig. 7 have been plotted in Fig. 8 for the tests at 75°F with specimens containing the granite aggregate and asphalts S-1, S-2, G and E. In this case the ordering appears in the direction of asphalt stiffness with a lower stiffness (as measured by the load to produce a strain of 500×10^{-6} in. per in., Table 6) resulting in a longer fatigue life at a particular strain level.

Also shown in this figure are the results of tests on laboratory-prepared specimens of the mixtures used in the Shell Avenue Test Road (4). While the relative stiffness of these mixtures is only slightly more than that of the S-1 mixtures (Table 6), there is a considerable difference in fatigue life as noted by the position of the curve. Thus these data would also indicate the possible influence of asphalt and aggregate type.

As in Fig. 7, a line with the slope defined by equation (1) has been plotted in Fig. 8. Since the various curves are essentially parallel to this line, equation (1) would appear to be a reasonable means to represent the fatigue data at this temperature also.

The data presented in Fig. 7 and 8 would suggest that for a specific mixture, the coefficient K in equation (1) is also dependent on temperature. For a particular mixture it is possible that K will have the same temperature dependence as the stiffness characteristics of the mixture. This is, of course, yet to be established.

While the tests which have been conducted thus far show an effect of temperature

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on the strain vs. cycles to failure relationships, the temperature range is fairly limited. Pell (4) has presented the results of constant-torsional strain tests on a sheet asphalt mixture over a more extended temperature range. These data are presented in Fig. 9. The results for tests below $0^{o}C$, as seen in this figure, indicate that below some specific temperature (in this case approximately $0^{o}C$) the stiffness characteristics are essentially constant (probably elastic). Pell (5) has suggested that the fatigue lives at the higher temperatures include a considerable crack propagation time which is not present at the low temperatures where cracks propagate quickly once they have formed, hence the displacement of the curves as shown.

It is possible that this type of behavior for a particular mix could also be deduced from stiffness measurements on the mix at a particular time of loading. In Fig 10 is shown a hypothetical relationship between flexural stiffness and temperature at a specific loading time. Above the temperature T_o, shown in this figure, the stiffness of the mixture decreases with increased temperature. As the temperature increases above this value, the time required for crack propagation in the controlled-strain tests will increase, thus resulting in a shift to the right of the strain vs. cycles to failure relationship as shown in Fig. 9. In controlled-stress tests, on the other hand, cracks in all probability propagate quickly over the range in temperatures of concern because of the nature of the test.

CONTROLLED-STRESS TESTS

Controlled-stress tests have been performed on sections of pavement from the Gonzales By-Pass, Highway 101 (Project V-Mon-2-C), and on laboratory specimens prepared from the same asphalts and aggregates with the fatigue equipment developed by Deacon (6) and reported in Report No. TE-64-2 (2). The purpose of the tests is to attempt to relate field performance to laboratory-measured fatigue characteristics. One of the ultimate objectives of this endeavor is to develop an additional test method which may assist the design engineer to preclude flexural fatigue cracking of asphalt concrete pavements on heavily traveled highways.

The structural section for this pavement (Fig. 11) consists of a type A asphalt concrete surface course and a type A asphalt concrete base course together with untreated Class 2 aggregate base and subbase, with a total thickness of the asphalt paving (exclusive of the open graded material) of 0.55 ft. An estimated traffic index for the period 1961 - 1971 is 9.1 (32.2×10^6 EWL).

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Mean Benkelman beam deflections obtained in February 1964 for sections from both northbound and southbound lanes are summarized in Table 7. As will be noted, the majority of the deflections are less than 0.015 in.

Materials

Asphalts from two sources were used in the pavement section, an 85-100 pen. asphalt cement produced by the Union Oil Company in the surface course and an 85-100 pen. asphalt cement produced by the Shell Oil Company in the base course. Tests on the materials used in the actual pavement section at the time of construction are shown in Table 8. To prepare the laboratory specimens used in this investigation, additional asphalt was required. Tests on materials obtained from the same sources in March 1965 are also shown in this table for comparison.

The aggregate for the surface course is essentially an all crushed material obtained from a source known as the Zaballa pit and was produced to meet the 3/4 in. medium grading specifications of the State of California Division of Highways. The results of control tests on the materials at the time of construction were used to determine the grading curve for the laboratory-prepared specimens. The average curve for 13 field samples as well as the range in values obtained is shown on the grading chart in Fig. 12. Also shown on this figure, for comparison, is the grading curve used for the laboratory-prepared specimens. Fig. 13 contains the grading curve for the laboratory specimens together with the specification limits for the 3/4 in. medium grading according to the 1964 Standard Specifications. Results of specific gravity determinations on this aggregate are shown in Table 9.

In the aggregate samples obtained from the original pit by the Materials and Research Department personnel for the laboratory specimens, some comparatively weak aggregate (possibly sandstone) was noted. This material appeared to be much more susceptible to fracture than the other portions, which appeared to be a mixture of granite and crushed gravel. This particular aspect may have some influence on the location of the fatigue results presented subsequently.

Aggregate for the asphalt concrete base course was obtained from the Granite Rock Company and is the familiar Watsonville granite. The majority of the material used in this course was produced to meet the grading specifications for a 1-1/2 in. maximum size asphalt concrete base. An average gradation for nine field samples

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obtained at the time of construction is shown in Fig. 14 together with the specifications limits.

The grading curve for the laboratory-prepared specimens was developed by maintaining the percent passing the no. 4 sieve about the same for laboratory specimens as for the field specimens, and by substituting material in the 3/4 in. by no. 4 size range for the material retained on the 3/4 in. sieve for the field specimens. The resulting grading curve for the laboratory-prepared specimens is also shown in Fig. 14.

Specimens

Block samples of the surface and base 15 in. × 15 in. in plan were obtained by the staff of the Materials and Research Department at the locations shown in Table 10 in March 1965 (approx. 2 years after the completion of the contract). These blocks were then sawed into 1.5 × 1.5 × 15 in. specimens in the laboratory with a diamond-tipped saw. Approximately 80 specimens of the surface course mixture and 40 specimens of the asphalt concrete base course were thus prepared for fatigue testing.

For the laboratory-prepared specimens the aggregate gradings shown in Figs. 13 and 14 were used. To estimate the asphalt contents to be used in these specimens, an analysis was made of the extracted asphalt content data (supplied by the Materials and Research Department) from the samples taken at the time of construction. The following variations in asphalt content were noted:

	No. of Samples	Range in Asphalt Content-percent	Average -percent
Surface Course	13	5.3-6.1	5.7
Base Course	9	4.3-5.0	4.6

On the basis of these data together with a consideration of the range noted for the project of 5.6 to 6.5 percent for the surface course and 4.3 to 5.0 percent for the base course, asphalt contents of 6.0 and 4.7 percent were selected for the laboratory-prepared specimens of surface and base respectively.

An attempt was made to reproduce the densities obtained in the field specimens through a compaction study in the laboratory. The procedure finally evolved from this study resulted in the preparation of 3.5 × 3.5 × 15 in. bars in the Triaxial Institute

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kneading compactor as follows:

Surface		Base	
No. of layers	Compactive effort per layer	No. of layers	Compactive effort per layer
3	20 tamps at 145 psi (leveling)	3	10 tamps at 145 psi (leveling)
	25 tamps at 290 psi		20 tamps at 290 psi

In addition, the surface of each beam for both surface and base course specimens received an additional 25 tamps at 40 psi pressure. All specimens were also subjected to a leveling pressure of 300 psi, this value being obtained at a rate of strain of 0.25 in. per min. and maintained for 30 sec.

Following compaction, each of the bars was sawed into four 1.5 × 1.5 × 15 in. specimens for the controlled-stress tests.

A comparison between the specific gravities of the field specimens and laboratory-prepared specimens is presented in Table 11. In general, it will be noted that the laboratory-prepared specimens exhibited higher specific gravities than those obtained from the field samples.

Stiffness moduli for both the field and laboratory-prepared specimens measured during the fatigue tests are presented in Table 12. These values were determined from measured loads and deflections after 200 repetitions of load from the relationship:

$$S = K \cdot \frac{P}{I \cdot \Delta} \quad \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \quad (2)$$

where

S = Flexural stiffness at 0.1 sec. loading time and temperature of test; psi

K = constant depending on the loading conditions (in this case uniform moment through central 4 in. of beam);

P = applied load; lb.

Δ = measured center of deflection of beam; in

I = moment of inertia of beam cross section; in^4

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The laboratory-prepared specimens, as noted in Table 12, exhibit stiffnesses of the same order or slightly less than the field specimens. This general agreement is due primarily to two offsetting characteristics. For the laboratory specimens, the increased density would tend to result in increased stiffness as compared to the field specimens; on the other hand this increased stiffness in the laboratory specimens would be offset somewhat by the fact that the asphalt in these specimens is not as stiff as that in the field specimens. Comparison of the properties of the recovered asphalts for both field and laboratory-prepared specimens presented in Table 13 tends to substantiate this latter point.

Test Procedures

The controlled-stress tests were performed with the equipment developed by Deacon (6) and described in TE-64-2 (2). Tests were performed at both 40°F and 68°F since this would appear to be a reasonable range in temperatures for the pavement section under investigation.

Essentially a constant moment and thus a constant stress was applied to the center 4 in. of each beam specimen with a duration of stress of approximately 0.1 sec. and a frequency of 100 applications per min.

In these tests, as compared with the controlled-strain tests, the cycles to failure represent the number of load applications at which each specimen completely fractured.

By subjecting the specimens to a range in stresses, the stress or strain vs. cycles to failure relationships could be developed for the various mixtures studied in this phase of the investigation.

Test Results

Results of controlled-stress fatigue tests at 40° and 68°F on the surface course mixture obtained from the field samples are presented in Fig. 15. Since there appear to be comparatively slight differences in characteristics between the various block samples, all of the results have been plotted together to produce representative curves at both of the test temperatures. The lines shown on the plot for both temperatures represent straight lines through the arithmetic means at each stress level. Also shown in this figure are the individual data points at each stress level so that some idea of the scatter in fatigue life can be obtained. In general it will be noted that a difference of

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at least a factor of 10 in life is obtained.

Similar relationships at both temperatures are shown for the base course specimens in Fig. 16. For the specimens used in these tests, it should be emphasized that comparatively large size aggregate was present. Moreover, some of the large particles were fractured, presumably during the rolling operation; this in turn could possibly contribute to an earlier failure in the test specimen than in the pavement. As will be seen subsequently, the curve for test results at 68°F for the base is approximately parallel to those obtained for the surface course specimens, whereas that for the base at 40°F has a flatter slope. At the present time, at least, there appears no explanation for this difference other than individual sample variation and an insufficient number of samples.

Test results on laboratory-prepared specimens are shown in Fig. 17 for specimens representative of the surface course and in Fig. 18 for specimens representative of the base course.

In general it will be noted that less variation is present in the test results on the surface course material than on those for the base material. This could in part be due to the differences in grading of both materials. For the base course specimens, since there is a comparatively large portion of the aggregate retained on the No. 4 sieve, it is possible that the behavior in fatigue is more sensitive to the distribution of aggregate and asphalt throughout the specimen, which in turn would have an effect on behavior.

Stress vs. cycles to failure relationships for field and laboratory specimens at both temperatures are summarized in Fig. 19. In general it will be noted that the curves for the laboratory-prepared specimens lie somewhat to the right of the corresponding field samples. In terms of stiffness alone one might infer that the curves for the field and laboratory specimens should coincide since both have about the same level of stiffness, as already noted in Table 12. In this instance, however, the laboratory specimens have been compacted to a higher density (lower air void content) and the asphalt content of the laboratory specimens may be slightly higher than the average value for the field specimens. Both of these factors may thus tend to influence the location of the fatigue relationships.

The results are encouraging, however, in that the relationships are essentially parallel for both field and laboratory specimens and the effects of temperature (i.e., a

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change from 40°F to 68°F) appear to be about the same for all test series.

Computed initial strain vs. cycles to failure data are shown for the field specimens in Fig. 20. These strains were determined from the stiffnesses measured after 200 repetitions of load, average values for which have already been presented in Table 12. Least square lines of best fit have been drawn through the individual data points for the tests at both 40°F and 68°F. Also shown in Fig. 20 are lines conforming to the relationships $N = K(1/\epsilon)^5$ and $(1/\epsilon)^6$.

Figs. 22 and 23 contain strain vs. cycles to failure data for the field and laboratory-prepared specimens of the base course, and Fig. 24 contains a summary of the strain vs. cycles to failure data for all of the mixes. As with the stress vs, cycles to failure data, the ordering of the curves indicates the laboratory-prepared specimens generally exhibit longer lives at a particular strain level and temperature.

These data suggest that a relationship similar to equation (2) may be applicable for defining the relationship between initial strain and cycles to failure, that is:

$$N = K\left(\frac{1}{\epsilon}\right)^n \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (3)$$

In the controlled-stress tests, n is less than 6, the value obtained for the controlled-strain tests. This difference in slope does not appear unreasonable, however, when one considers the mechanism of failure in both types of tests. At high stress or strain levels in both modes of loading. once cracks have been initiated they possibly will propagate quickly. On the other hand at the lower strain levels in controlled-strain tests, the fatigue life may include a considerable time for crack propagation, which will not occur to the same extent in the constant stress tests. Thus in the latter tests, a shorter life for a particular strain will be obtained and thus a steeper slope to strain vs. cycles to failure curve.

REPEATED LOAD TESTS - UNTREATED MATERIALS IN STRUCTURAL SECTION, GONZALES BY-PASS

In addition to the block samples of the paving materials, the staff of the Materials and Research Department also obtained disturbed samples of the untreated base and sub-base materials and both undisturbed and disturbed samples of the subgrade soil at Station 308+10 (corresponding to the location of Block No. 3).

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Repeated load triaxial compression tests were performed on the undisturbed samples of the subgrade and on remolded, recompacted samples of the base to obtain a measure of their resilient or elastic characteristics.

Subgrade

To estimate the stress conditions to be used in the repeated load triaxial compression tests on the undisturbed samples of the subgrade soil, the structural section was treated as a three-layer system (Fig. 25) and the tables for stresses developed by Jones (7) were used to estimate the stresses σ_{zz_2} (vertical compressive stress) and σ_{rr_3} (lateral or radial stress) at the surface of the subgrade. In these computations the asphalt concrete surface course and base course were treated as one layer and the modulus was assumed to vary within 100,000 to 1,000,000 psi to cover the range in stiffnesses which might be expected in service; the flexural stiffness measurements presented in Table 12 indicate this to be a reasonable assumption. The surface loading conditions approximate a 15,000 lb. wheel load on dual tires, conditions which are used to measure pavement deflections with the Traveling Deflectometer.

On the basis of data from tests on other granular materials (8), a modulus of 20,000 psi was assumed for the untreated granular material. In this case the base and subbase were considered together as one course.

The subgrade was assumed to have a modulus of 8,000 psi thus giving a modular ratio of E_2/E_3 of 2.5, which is in the range suggested by Heukelom and Klomp (9) and Dorman and Metcalf (10).

On these assumptions, it was determined that σ_{zz_2} would range from about 1.6 to 2.6 psi corresponding to the range in surface moduli shown. In addition, σ_{rr_3} due to the load was estimated to be approximately 0.3 psi, which when added to lateral stress due to the overburden* resulted in a total lateral pressure of about 1.4 psi. To cover the range in stresses it was thus decided to use a range in axial deviator stresses of 1 to 10 psi and a lateral pressure of 1.4 psi in the repeated load triaxial compression tests on the subgrade.

Materials and Specimens. The subgrade soil contained in the tube samples could be classified as a clayey sand by visual identification. (No classification tests were

*Based on a coefficient of earth pressure at rest, $K_o = 0.5$.

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performed). Water content and dry density data for this material obtained after repeated loading tests are shown in Table 14. These tabulated values of water content correlate well with the results of 14.1 and 11.4 percent reported by the Materials and Research Department staff determined at the time of sampling.

Initially it was planned to trim four 1.4 in. diameter × 3.5 in. high specimens from each of the three tubes supplied. Because of the sandy nature of the material, however, it was only possible to obtain a total of five specimens for test, those shown in Table 14.

Test Procedure. The repeated load triaxial test equipment developed by Seed and Fead (11) was used to test the undisturbed subgrade samples in accordance with the procedure described by Seed et al (12). The deviator stresses, summarized in Table 14, were applied at a frequency of 20 applications per min. and a duration of 0.1 sec.

Since the deformations corresponding to the range in stresses used were small, dual linear variable differential transformers were used to measure the resilient or recoverable deformations. In general the stresses were repeatedly applied for about 100,000 repetitions.

Resilient moduli were determined from the expression (12):

$$M_R = \frac{\sigma_d}{\epsilon_r} \quad \text{. . . . . .} \quad (4)$$

where

M_R = modulus of resilient deformation, psi
σ_d = repeated axial deviator stress, psi
ϵ_r = resilient axial strain, in. per in., corresponding to a specific number of load repetitions.

Test Results. Typical results for resilient axial deformation vs. number of load applications are shown in Fig. 26 for sample No. 3 subjected to a repeated deviator stress of 1.0 psi. The pattern of resilient deformation shown in this figure is typical of those obtained for the other specimens, generally decreasing with increased repetitions and leveling off after about 20,000 to 30,000 load applications.

Resilient moduli determined by using equation (4) at 1000, 10,000, and 100,000 repetitions are plotted in Fig. 27 as a function of deviator stress. In general, the

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influence of deviator stress on resilient modulus shown in this figure is the same as reported by Seed et al (12) and shown in Fig. 28 for tests on the AASHO subgrade soil.

In Fig. 27 it will be noted that the resilient modulus shows a marked decrease with increase in deviator stress in the low stress range. Beyond a stress of approximately 5 psi the modulus remains essentially constant or increases slightly with increase in deviator stress. As seen in this figure, however, the resilient modulus is not only dependent on stress level but also on the number of repetitions at which it is determined; with increase in number of repetitions the modulus increases. This increase in modulus follows from the pattern of resilient deformation shown in Fig. 26. In general it can be seen that, in order to use the modulus results to obtain a measure of pavement behavior, the stresses to which the subgrade will be subjected must be reasonably well established and, in the case of well-traveled highways, the modulus should correspond at least to that determined after 20,000 to 30,000 load applications.

Base Course.

The Class 2 aggregate base was tested in repeated load triaxial compression in accordance with a procedure developed by Mitry (13) and reported in Reference (8). Since it has been shown that the resilient modulus of untreated granular materials is dependent on confining pressure (13, 8), a range in confining pressures was used to establish the resilient characteristics of this material.

Material and Specimens. As noted above, the aggregate base conforms to the California Standard Specifications for a Class 2 aggregate base. Samples of the base were supplied from Station 308 + 10 (Block No. 3). In-place water contents obtained at the time of sampling varied from 5.1 to 7.5 percent. Two in-place density determinations indicated an average density of about 131 lb. per cu. ft. and an average water content of 5.9 percent. Based on a specific gravity of the fine material equal to 2.68 and of the coarse material equal to 2.61, the degree of saturation of the base at the time of sampling was around 60 percent. Using these data, representative specimens 3.9 in. in diameter by 7.8 in. in height were prepared in the laboratory to an average dry density of 130 lb. per cu. ft. (17 specimens) by vibratory compaction (13, 8) and with an average water content of 5.8 percent and degree of saturation of 57 percent.

Test Procedure. The compacted specimens were subjected to repeated load triaxial compression tests using lateral pressures varying from 1 to about 55 psi. Load was applied at the rate of 20 applications per min. and with a duration of 0.1 sec. Details of the test equipment are described in Reference (8).

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Resilient deformations were measured with a linear variable differential transformer and repeated loading was continued to at least 10,000 repetitions. The modulus of resilient deformation at a particular confining pressure, σ_3, was determined from equation (4) using the resilient axial strain obtained at 10,000 load applications.

Test Results. Resilient modulus as a function of confining pressure are shown in Fig. 29. These data can be approximated by an equation of the form suggested by Mitry (13):

$$M_R = K \cdot \sigma_3^n \quad \text{(5)}$$

where:

M_R = Modulus of resilient deformation, psi.

σ_3 = Effective lateral or confining pressure, psi.

K and n = Constants depending on the material under investigation.

For this material K is equal to 15,200 and n is equal to 0.482, as shown in Fig. 29. Similar tests performed by Mitry (13) on a uniform dry sand resulted in a value for K = 12,500 and n = 0.35 and for tests on a dry well-graded gravel K = 7,000 and n = 0.55. Equation (5) emphasizes the importance of confining pressure, and the data shown in Fig. 29 illustrate the wide range in resilient moduli obtainable depending on the pressure to which the material is subjected.

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APPLICABILITY OF VARIOUS TYPES OF FATIGUE TESTS

In the first part of this report, the results of two types of fatigue tests have been presented, namely constant-strain amplitude and constant-stress amplitude tests. Interpretation of the results of these types of tests may lead one to some conflicting conclusions. For example, in the controlled-strain tests, mixture stiffness, for a particular combination of asphalt and aggregate, appears to influence the strain vs. cycles to failure relationship in that the stiffer the mix, the shorter the fatigue life at a particular strain level (e.g. comparison of the results of tests on mixtures with asphalt S-1 and S-2). On the other hand, in the controlled-stress tests, the stiffer the mix, the longer is the fatigue life at a particular stress level.

Thus, if one were to base a mix design on conclusions drawn from the results of controlled-strain tests, one would tend toward the use of softer asphalts. On the basis of controlled-strain tests, however, the results would suggest the use of harder asphalts since these tend to increase mixture stiffness.

In an attempt to shed some light on this problem, an analysis of three-layer systems was performed by Hicks (14) in which the thickness and the stiffness of the asphalt concrete layer were varied to assess the influence of these variables on the computed stresses and strains in the asphalt concrete layer. Since the fatigue tests are also presented in these terms, this analysis should assist in determining the applicability of a specific mode of loading.

Three of the structural sections considered in this analysis are shown in Fig. 30. For both the thin and thick pavement sections shown in Figs. 30(a) and (b) respectively, thickness of the asphalt concrete layer was varied from 1 in. to 9 in. maintaining the total thickness of section at 13 in. and 26 in. respectively. In addition, the modulus of the asphalt concrete layer was varied from 100,000 to 4,000,000 psi for both sections. For the pavement section with the stiff base (Fig. 30(c)) the thickness of the asphalt concrete was varied from 2 in. to 9 in. and the modulus of this layer from 100,000 to 2,000,000 psi. The moduli of the base and subgrade were also varied as shown in Fig. 30.

Because of the number of variables involved, use was made of a computer solution developed by the Chevron Research Company, the staff of which also performed the necessary computations on their IBM 7094 computer.

Horizontal radial tensile strains at the underside of the pavement layer as a function

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of the stiffness of this layer are shown in Figs. 31, 32, and 35. Computations for the radial tensile stresses at the same location are presented in Figs. 33, 34, and 36. From an examination of Figs. 31 and 32 it will be noted that regardless of the stiffness of the asphalt concrete (for the range in moduli investigated), the tensile strains on the underside of the asphalt concrete of the 1 in. thick pavements are essentially constant. On the other hand for the thicker asphalt concrete layers, the tensile strain is reduced markedly as the stiffness of the asphalt concrete increases. This would imply that for thin surface layers, regardless of the stiffness of the asphalt concrete, its performance is governed primarily by the underlying materials. In thicker layers, however, the asphalt concrete will begin to contribute more to the behavior of the section and exert more influence on the strains to which it is subjected.

On the basis of these data it would thus appear that the results of constant-strain amplitude fatigue tests are applicable for predicting the performance of comparatively thin layers of asphalt concrete, layers approximately 2 in. or less in thickness.

In Figs. 33 and 34 it can be seen that for the thin layers of asphalt concrete, the tensile stress at the underside of the layer changes quite markedly with change in layer stiffness. As the thickness of the layer increases, however, the change in stress with layer stiffness is less marked; e.g., approximately a factor of 2 for the 9 in. thick layer with a change in stiffness by a factor of 40. Since the change in strain is much larger than this, at least a factor of 10, it would appear that a test which reflects mixture stiffness advantageously would be more appropriate to represent the fatigue behavior of asphalt concrete. Thus the controlled-stress test, at least on the basis of this analysis, would appear to be more representative. It is on this basis that this type of test was selected to determine the fatigue characteristics of the asphalt-bound materials used in the structural section of the Gonzales By-Pass.

The results presented in Figs. 31 through 34 have been developed from analysis of materials thought to be representative of untreated bases. When an asphalt concrete layer is placed on a base of relatively high stiffness (e.g. cement-treated base) different relationships between tensile stress and tensile strain as a function of surface stiffness were obtained as shown in Figs. 35 and 36. These data suggest that, over the practical range in stiffness as of asphalt concrete associated with moving wheel loads, fatigue in the upper layer may not be a problem so long as the lower layer maintains its high stiffness, since the stresses and strains in the upper layer are comparatively small.

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While it is difficult to precisely define a minimum pavement thickness for which the results of constant-stress tests would seem appropriate, it is suggested that a 4 in. thick layer of asphalt concrete be considered, at least at this time, as the lower limit. This implies that there is a range in asphalt concrete thickness for which neither the controlled-stress nor the controlled-strain mode of loading is strictly applicable; i.e., between 2 and 4 inches. Until this point can be clarified, both types of tests might be utilized and prediction of pavement behavior based on the results of both tests. Comparisons with pavement performance should in turn indicate which mode of loading is applicable.

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ANALYSIS OF STIFFNESS CONSIDERATIONS

In the discussion thus far, mixture stiffness has been shown to be an important factor in assessing the fatigue behavior of asphalt concrete in both types of tests. The Shell investigators (15, 16, and 17) have suggested a relatively simple procedure to estimate mixture stiffness at a particular time of loading and temperature. This estimate is based on the penetration and softening point of the asphalt contained in the mix and the volume concentration of the aggregate* and is applicable to mixtures with about 3 percent air voids.

With the characteristics of the recovered asphalt, the stiffness of the asphalt can be determined from the nomograph shown in Fig. 37, which is a modification by Heukelom and Klomp (17) of the original method proposed by Van der Poel (15). The stiffness of the mixture can then be estimated from the following equation:

$$\frac{S_{mix}}{S_{asphalt}} = \left(1 + \frac{2.5}{n} \cdot \frac{C_v}{1 - C_v}\right)^n \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (6)$$

where:

S_{mix} = Mixture stiffness, kg. per sq. cm.

$S_{asphalt}$ = Stiffness of recovered asphalt, kg. per sq. cm. and determined from Fig. 37

C_v = Volume concentration of aggregate

$n = 0.83 \log\left(\frac{4 \cdot 10^5}{S_{asphalt}}\right)$

Fig. 38 is a plot of this equation to simplify the stiffness determination (17).

Using this procedure, stiffnesses have been estimated for the various mixtures subjected to the controlled-stress tests. Properties of the recovered asphalts from the field

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*The volume concentration of aggregate is defined as:

$$C_v = \frac{\text{volume of aggregate}}{\text{volume of (asphalt + aggregate)}}$$

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and laboratory specimens for the Gonzales By-Pass were determined by the Materials and Research Department Laboratory. These computed stiffnesses are tabulated in Table 15 for temperatures of 68°F and 40°F and at a time of loading of 0.1 sec. Measured stiffness values are also shown in this table determined from equation (2) using the deflection measured after 200 repetitions of load. The computed stiffnesses are, in all cases, larger than the measured values. It will be noted, however, that the void contents of the specimens are higher than those for which the procedure was developed. Thus the differences are not unreasonable, at least for the surface course mixtures. For the base course mixtures it is possible that a difference in asphalt content as well as the difference in void content may contribute to the larger differences between the computed and measured stiffness.

Also shown in Table 15 are comparisons between measured and computed stiffnesses for a series of mixtures containing a dense graded granite and asphalts with varying degrees of stiffness. For these mixtures, the void content is closer to that for which the correlation was developed as indicated in the table. In general the comparison between computed and measured stiffnesses for these mixtures is encouraging. It should be noted at this point that Van der Poel (15) suggested that his original nomograph for asphalt stiffness prediction should give a difference between measured and computed values not larger than a factor of two. This would also affect the stiffness computations for the mixture as seen by equation (6).

Thus by this procedure a measure of in-place stiffness of the paving mixture could be obtained from a knowledge of the grading of the aggregate, the unit weight of the mixture, and the properties of the recovered asphalt, provided the mixes are well-compacted and have been designed in accordance with the stabilometer procedure. Otherwise, direct stiffness measurements on specimens sampled from the pavement would seem necessary to obtain a measure of the in-place characteristics of the paving material.

An attempt has been made to utilize the fatigue and stiffness data for the laboratory-prepared specimens of the surface course of the Gonzales By-Pass as well as fatigue and stiffness data for the other mixtures reported in Table 15 in a plot similar to that reported by Heukelom and Klomp (17). These data are plotted in Fig. 39. Lines representing the best fit for numbers of repetitions to failure corresponding to $N = 10^3$, 10^4, 10^5, and 10^6 applications are shown; also indicated (dashed lines) are the curves suggested by Heukelom and Klomp. Considerable scatter is indicated in the actual data. In

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addition, while the trends are similar to those presented by Heukelom and Klomp, the locations of corresponding curves differ even though the void contents of the mixtures are in a similar range.

As noted in Reference (2), it was hoped that such a plot might be used to represent mixtures typical of those conforming to the requirements for heavy duty pavements. The scatter in the data points suggests that this may not be possible. If all type A or type B mixes were to follow such a pattern, however, the design of pavements to incorporate the influence of fatigue would be simplified somewhat. Unfortunately, insufficient data are available at this time to either preclude or suggest the feasibility of such an approach.

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DESIGN CONSIDERATIONS

One of the objectives of this research program is to consider the possibility of using laboratory-measured fatigue characteristics on representative specimens of paving mixtures to assist in the design of asphalt concrete pavements. On the basis of a number of investigations (6, 8, and 10), it is now possible to at least suggest the outline of such an approach. In addition, it would also seem appropriate to briefly review other methods which have been or might be considered to minimize the incidence of fatigue cracking.

Of these latter procedures, the method suggested by Hveem et al (18) involves no testing of the asphalt concrete mixture. Rather, resilience tests are performed on representative specimens of the underlying paving materials. The results of these tests permit estimation of the deflection of the pavement structure under a 15,000-lb. axle load. By comparing the deflection so determined with deflections developed from in-service correlations, it is possible to estimate whether the particular design is adequate. If the deflection is higher than the permissible value, the design can be modified to reduce the expected deflection to a tolerable value.

A second procedure is that originally suggested by Monismith et al (19) and more recently advocated by Pell (5). Noting in general that when the magnitude of the tensile strain is 100×10^{-6} in. per in. in the asphalt concrete, the material is capable of withstanding large numbers of load repetitions before fracture (generally in excess of 10^7 load applications for surface course mixtures), these investigators suggest that the limiting tensile strain should not exceed this value for the design wheel or axle load. Analyses such as those presented in Figs. 31 through 36 or by Pell (5) should assist in assessing the adequacy of the design according to this criterion.

This procedure, however, requires that a measure of the elastic (or resilient) characteristics* of the underlying materials as well for the asphalt concrete be determined so that an estimate of the strain can be made within the framework of available theory. Recently Seed et al (8) have indicated that the three-layer theory of Burmister can be used to predict, at least approximately, pavement deflections provided the resilient characteristics of the components of the structural section are determined from tests in which the time of loading and numbers of repetitions are consistent with those to be expected in the field.

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*Elastic in the sense that the deformation under load is recoverable.

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Test results for granular base and compacted subgrade for the Gonzales By-pass pavement based on this procedure have already been presented in an earlier section of this report. These tests, together with stiffness measurements on the asphalt concrete, can be used to predict the deflections in a structure representative of the Gonzales By-Pass.

Computations of surface deflection have been made for the range of conditions shown in Table 16 by the procedure developed by Mitry (13) and detailed in Reference (8). As also seen in the table, four surface deflections have been computed. These deflections have been determined on the basis of two values of stiffness for the asphalt concrete, 125,000 and 1,000,000 psi, which should cover the range to be expected from the results reported in Table 15. Two different relationships for the resilient modulus of the subgrade soil were also used, the curves corresponding to N = 1,000 and 100,000 stress applications (Fig. 27). The resilient moduli for the granular base and subbase were assumed to be the same; specific values were determined from the expression shown in Fig. 39.

In the method suggested by Monismith et al (19) and Pell (5) the procedure described above (or some modification thereof) could thus be used to estimate the tensile strain in the asphalt concrete. If, for the conditions assumed, the strain exceeds the value of 100×10^{-6} in. per in., the pavement section could be modified to bring the tensile strain within the tolerable limit.

The difficulty with this procedure is to estimate the stiffness of the asphalt concrete, since as noted in Figs. 31 and 32, the stiffness of this layer influences the magnitude of strain developed. Pell (5) has suggested:

> "....These tensile strains (in the asphalt concrete) will be a maximum when the overall stiffness of the entire structure is a minimum. The stiffness of bituminous materials is dependent on temperature and the critical condition is therefore likely to arise at high temperatures during the summer months.
>
> "However, the fatigue tests at high temperatures show that although cracks initiate under these conditions, they propagate only slowly due to the lower stress, and thus failure will not necessarily be apparent at this time. But once the temperature falls and the stiffness of the bituminous layers increases, there will be an increase in stress, particularly at the tip of the crack, owing to the stress concentration effect. This will result in more

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rapid propagation of any fatigue cracks under winter conditions; but again, it will not necessarily lead to failure owing to the freezing of the sub-base and subgrade and the resultant increase in strength. During the thaw period, however, the fact that the surface layers are cracked increases greatly the likelihood of pavement deterioration from penetration of water and consequent local subgrade failure. ...."

Thus within this framework, a stiffness representative of the pavement under comparatively high temperatures and the speed of loading associated with moving traffic should be considered. This, on the basis of available data, would appear to be in the range of 100,000 to 150,000 psi. It should be noted that this value would not correspond to a measurement at 140°F since, although the surface of the pavement may reach this level, the average temperature within the asphalt concrete will be less than this value, being lower as the thickness of asphalt concrete increases (e.g., Dormon and Metcalf (10)).

This approach, from a design standpoint, would require a specification for a minimum mixture stiffness at a temperature depending on the climatic conditions and asphalt concrete thickness and also a requirement that the fatigue life at 100×10^{-6} in. per in. exceed some specific number of repetitions, e.g., 10^6 or 10^7 applications for the design load.

The other procedure referred to in the initial paragraph of this section may permit the design of a pavement which will perform adequately for a specific time period, at least from the cracking standpoint. As with the method described above, the resilient or elastic characteristics of the materials comprising the pavement section would have to be defined so that appropriate theory can be utilized to determine the stresses and strains in the asphalt concrete course associated with the different traffic loads applied to the pavement.

In addition, the stress vs. cycles to failure data in simple loading must be defined for a range of stresses consistent with those which will be developed in the field (e.g., data such as that shown in Figs. 15 through 18).

With these data it may be possible to estimate whether a pavement will crack during the design life under the wheel loadings to which it is subjected. The procedure is described in the following paragraphs. To simplify the discussion, a constant stiffness will be assumed for the asphalt concrete layer.

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1. From a knowledge of the resilient properties of the compacted subgrade (in a condition representative of that which occurs during the life of the structure, e.g., Fig. 27), the resilient characteristics of the aggregate components (e.g., Fig. 29), and the stiffness characteristics of the asphalt concrete, stresses associated with the various wheel loads can be estimated from appropriate theory such as that developed by Burmister and utilized by Seed et al (8).

2. Corresponding to a particular computed stress, an estimate of the cycles to failure can be determined from laboratory measured fatigue data. This procedure is illustrated schematically in Fig. 40.

3. Using the equation developed by Deacon (6), which is simply a modification of the linear summation of cycle ratios procedure (referred to as the Miner criterion), an estimate of the fracture life under the combination of stresses can be ascertained. Deacon's formulation may be stated as:

$$Y_f = N_f^k / \sum_i p_i \left(\frac{\sigma^k}{\sigma_i} \right)^b \quad \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \quad (7)$$

where:

Y_f = predicted, arithmetic-mean fracture life, compound loading

N_f^k = average fracture life at standard stress level, σ^k, simple loading
(would correspond to σ' and N_f', etc., shown in Fig. 40)

p_1 = applied percentage of load condition i (based on traffic estimates)

σ^k = any standard stress level

σ^i = various computed stress levels associated with range of vehicles to which the pavement is subjected.

b = a constant taken to be the slope of the linear log $\overline{N}_f$ - log σ relation for simple loading.

Since it has been demonstrated quite conclusively that fatigue is a stochastic process (e.g., Ref. (6)), some measure of the variability in the predicted fracture life by this procedure would be desirable. Deacon (6) has also demonstrated that the standard deviation in compound loading fracture life can be estimated from:

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$$s[Y_f] = s[N_f^k] / \sum_i p_i \left(\frac{\sigma^k}{\sigma^i}\right)^c \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots (8)$$

where:

$s[Y_f]$ = predicted standard deviation of the compound-loading fracture life

$s[N_f^k]$ = measured standard deviation of the simple-loading fracture life at standard stress level, σ^k

c = a constant taken to be the slope of the linear log $s[N_f]$

4. By comparing the number of wheel loadings determined from equation (7) with those anticipated during the design life, a measure of the adequacy of the design with respect to fatigue is obtained. For example, should Y_f be substantially less (statistically speaking) than the actual number anticipated, redesign of the section should be considered.

It should be emphasized that a number of simplifying assumptions have been made in developing this approach. The stiffness of the asphalt concrete was assumed constant in order to compute a set of stresses associated with a range in wheel loads. Moreover, one stiffness was considered to define the stress vs. fracture life relationship for the asphalt concrete. In reality, the stiffness of the asphalt concrete will vary in service because of daily and seasonal temperature variations. As indicated in Fig. 15, for example, the stiffness of the asphalt concrete as affected by temperature also influences the relationship between stress and cycles to failure. Thus, compound loading in practice not only involves the superposition of various loads but also the effects of temperature.

In addition, the influence of aging of the mixture has yet to be ascertained. Hardening of the asphalt would tend to stiffen the mixture. This is turn would shift the stress vs. cycles to failure relationship to the right in controlled-stress tests; i.e., at a given stress level the number of cycles to failure should be increased. Thus in pavements containing comparatively thick asphalt concrete layers this may prove advantageous, as illustrated in Fig. 32, <u>from a fatigue standpoint</u>. In comparatively thin sections, on the other hand, where the results of controlled-strain tests would appear to be applicable, this hardening would be detrimental from a fatigue standpoint, since it would tend to increase the stiffness of the mixture (e.g., Fig. 7). In addition, crack propagation would occur more quickly because of the increased stiffness.

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The procedure described above in which compound loading is considered appears somewhat more gratifying from an engineering standpoint than the procedure suggested by Monismith et al and Pell. From a practical standpoint, however, this latter procedure has certain advantages, particularly in terms of simplicity from a specification and testing standpoint. In addition, the uncertainties of temperature effects are minimized. Thus, as a first step in the incorporation of fatigue considerations into design and specifications, this may be the more desirable of the two procedures. It must be emphasized, however, that before either approach (or other methods which might evolve as more insight is obtained) can be utilized, extensive field and laboratory comparisons are required.

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RECOMMENDATIONS FOR RESEARCH FOR 1965-66

As noted in Reference (2), one of the objectives of this research program should be the initiation of studies of in-service pavements for which performance data are available. To this end, the studies of the Gonzales By-Pass have already been undertaken. During this next year an extensive study of the second project recommended in Reference (2) V-SLO-56-C, will be undertaken.

The structural section for this pavement consists of 2.5 in. of asphalt concrete (Type A), 8 in. of Class 2 aggregate base, and 12 in. of Class 2 aggregate subbase over the compacted subgrade. Arrangements have been made with the staff of the Materials and Research Department to obtain samples of all of the components so that fatigue tests on the asphalt concrete and resilience tests on the underlying materials can be performed. These, together with those already developed for the Gonzales By-Pass will be used to predict the possibility of cracking on these projects. Traffic data have recently been supplied for both sections to assist in this endeavor. The analysis outlined in the previous section will be used to predict the approximate time, if any, at which cracking may develop.

It should be noted that Project V-SLO-56-C, D may offer a better opportunity to meet these objectives than V-Mon-2-C, since deflections on the former project in some instances exceed 0.030 in.

Since the asphalt concrete surface for V-SLO-56-C, D has a thickness of about 2.5 in., the pavement is in the range of thickness where analyses such as those shown earlier indicate that neither controlled-stress nor controlled-strain tests may strictly be applicable to define the performance in the laboratory. As a result, more extensive studies will be performed on the laboratory-prepared specimens for this project, in that both controlled-stress and controlled-strain tests will be conducted. This should permit, at least in one instance, a comparison between the two types of tests to be established for an asphalt mixture representative of California practice, while at the same time permitting the possibility of fatigue cracking to be analyzed using the results of both tests.

In the light of data such as those presented in Figs. 35 and 36, it would also appear desirable to consider testing the Project IV-Nap-8, 49-Nap A, D with emphasis placed on tests to ascertain the fatigue characteristics of cement-treated base. Shen (20) has

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already developed techniques for measuring such characteristics in another investigation. Stiffness measurements on the asphalt concrete as well as resilience measurements on the underlying materials would permit an extension of the deflection prediction and fatigue analyses presented in the previous section.

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SUMMARY AND CONCLUSIONS

In the test program completed to date, two types of fatigue tests have been conducted on a series of asphalt concrete specimens, namely controlled-strain and controlled-stress.

Controlled-strain tests have been performed on mixes containing five different asphalts and two different aggregates at temperatures of 40°F and either 68 or 75F. The general objectives of these tests have been to define the influence of asphalt source and hardness and aggregate type on the fatigue characteristics of asphalt concrete conforming to current State of California specification requirements.

Controlled-stress tests have been performed on specimens obtained from both the surface and asphalt concrete base courses of the pavement section in the Gonzales By-Pass (V-Mon-2-C). In addition, the same type of test has been performed on laboratory-prepared specimens of the same materials to develop a comparison between the performance of the field specimens and laboratory specimens with a representative aggregate grading, asphalt content, and unit weight. The eventual objective of this investigation is to develop information to permit incorporation of fatigue considerations in both the design of and specifications for asphalt concrete pavements.

Since it has been demonstrated that the stresses or strains which result in the fatigue of asphalt concrete are dependent not only on the asphalt concrete itself but also on the resilience characteristics of the underlying materials, some repeated load triaxial compression tests have been performed on representative specimens of the base and subgrade materials from the Gonzales By-Pass. These tests as well as the fatigue tests on the asphalt concrete are part of the development of the design objectives noted in the preceding paragraph.

One of the important mix variables which may be useful in assessing the fatigue behavior of asphalt concrete regardless of the test method employed is mixture stiffness, which is simply:

$$S = \frac{\sigma}{\epsilon} (t, T)$$

that is, stress divided by strain at a particular time of loading and temperature. For asphalts from a particular source, this mix property may be useful in indicating, at least in a qualitative fashion, fatigue behavior, regardless of the mode of loading.

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For conditions of controlled-strain, the data presented suggest that the less stiff the mix at a particular temperature, the longer the fatigue life, so long as asphalts from the same source are compared, (e.g., comparison of the data for mixtures with asphalts S-1, S-2, and S-3 and the granite aggregate).

When making this comparison, however, one cannot always use the properties of the original asphalt. Rather, the influence of the asphalt after it has been mixed with the aggregate must be considered. This is illustrated by the data presented for the fatigue tests on the laboratory-prepared specimens used in the Shell Avenue Test Road. While the original asphalts were classed as 40-50 and 85-100 penetration asphalt cements on the basis of original viscosity measurements, their contribution to mixture stiffness appeared to be about the same (based on load to cause a specific strain level). Moreover the fatigue data for these mixtures indicated essentially the same performance.

While mixture stiffness appears to be a significant property in being able to classify the performance of mixtures in controlled-strain tests, the data also suggest that asphalt source and aggregate characteristics may have an effect. In comparing the results of tests at 40°F on specimens prepared with asphalts G and S-2 and the granite aggregate, the mixtures containing asphalt S-2 are stiffer than those with asphalt G. The fatigue life at a particular strain level is longer, however, for the specimens prepared with asphalt S-2.

Aggregate type would appear to influence the results of controlled-strain tests with specimens prepared from asphalt S-1 and the uncrushed gravel having a shorter fatigue life at a particular strain level than specimens prepared with the same asphalt and the crushed granite. Part of this difference may be due to asphalt content, however; the specimens prepared with the gravel could only tolerate 4.5 percent asphalt content whereas those with the granite were prepared at 5.9 percent asphalt for essentially the same design requirements.

In general the results of the controlled-strain tests appear to follow a relation between strain and cycles to failure of the form

$$N = K\left(\frac{1}{\epsilon}\right)^n$$

where:

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N = service life (number of load applications to reach a particular level of damage)

ϵ = magnitude of the tensile strain repeatedly applied

n = constant (approximately 6)

K = constant dependent on:

(a) temperature
(b) asphalt stiffness
(c) asphalt source
(d) aggregate characteristics

While not developed to any extent from data in this investigation, K in this expression also appears to be dependent on asphalt content and density, for a particular mixture, with an increase in asphalt content and/or density resulting in increased service life at a particular level of strain.

In the controlled-stress mode of loading, stiffness also influences the fatigue life at a particular stress or strain level. For these conditions, an increase in stiffness causes an increase in the fatigue life at a particular level of stress (e.g., comparison of fatigue tests for either the field or laboratory-prepared specimens of the Gonzales By-Pass materials at 40°F and 68°F).

As with the controlled-strain test method, both asphalt content and density also influence the fatigue behavior, with an increase in either characteristics resulting in an increase in fatigue life.

From an analysis of three-layer systems using Jones'(7) developments from Burmister's theory, it would appear that controlled-strain testing is applicable to assess the performance of mixtures in pavements consisting of comparatively thin asphalt concrete sections, whereas a controlled-stress mode of loading seems more suitable for sections consisting of comparatively thick asphalt concrete courses. On the basis of this analysis, the controlled-stress tests were used to define the fatigue characteristics of the Gonzales By-Pass pavement specimens.

While considerable scatter is evident in the data obtained for the Gonzales specimens, the same general trends are observed in both the field and laboratory-prepared specimens for this pavement section. Some difference in life at a particular stress is evident,

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however, between the field and laboratory-prepared specimens. On the basis of stiffness alone, one might expect slightly longer life for the field specimens. Actually, the reverse was obtained. This trend could be in part due to the difference in densities of the field and laboratory specimens and in part due to the fact that the field specimens have been in service for at least one year and may have been damaged an indeterminate amount because of traffic. This latter point requires considerable additional study, however, before any definite conclusions can be drawn.

The fatigue data for these specimens should permit some estimate of the potential for crack development to be made from available traffic data. To this end, the resilient moduli determined for both the base and subgrade are necessary since the stresses developed in the surface course are a function of the stress-deformation characteristics of the underlying materials as well as the stiffness of the course itself.

Results of resilience tests on the base material at a water content and dry density representative of the in-situ condition at the time of sampling indicate that the expression suggested by Mitry, namely:

$$M_R = K \cdot \sigma_3^n$$

is applicable to define the influence of stress conditions on the modulus of resilient deformation of this material. For the particular conditions of test $K = 15,200$ and $n = 0.482$ in the above expression.

For the subgrade soil, the modulus of resilient deformation determined in specimens trimmed from undisturbed tube samples was noted to be dependent both on stress intensity and number of load applications, with the modulus increasing as the number of stress applications increased and decreasing with increase in stress intensity.

The moduli so determined together with stiffness measurements on the asphalt concrete provide data for use in the three-layer elastic theory to estimate pavement deflection. For the range in values considered, the computation procedure has been shown to provide an estimate of deflection which is within the range measured in the pavement section of the Gonzales By-Pass with the Traveling Deflectometer.

Since the deflection estimates are reasonable, the cumulative damage procedure suggested for combining the effects of different stress intensities may have some potential in estimating the onset of cracking. This is, of course, yet to be determined and is compounded by the effects of aging and temperature to mention a few of the variables.

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TABLE 1 - ASPHALT DESIGNATION

Sample	U. C. Designation	Materials and Research Designation	Pen. at 77°F Materials and Research Lab.
Standard 40-50	S-2	R-3655	39
Standard 85-100	S-1	R-3656	96
Standard 120-150	S-3	R-3657	119
Golden Bear 85-100	G	R-3618	85
AASHO 85-100	E	R-3722	90

TABLE 2 - AGGREGATE SPECIFIC GRAVITIES (ASTM C-127, 128)

Aggregate	Fraction	Bulk Specific Gravity	Apparent Specific Gravity	Absorption Percent
Watsonville	All	---	2.92	---
Cache Creek	Coarse	2.64	2.74	1.4
	Fine	2.59	2.63	0.6

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TABLE 3 - SUMMARY OF MIX CHARACTERISTICS PRIOR TO FATIGUE TESTING

Test Series Aggregate	Test Series Asphalt	Test Temperature °F	Total No. of Specimens	Percent air voids Mean	Percent air voids Standard Deviation	Percent air voids Coefficient of Variation
Watsonville	E	75	33	4.4	0.36	8.1
"	E	40	37	5.1	0.37	7.1
"	G	75	27	3.9	0.37	9.4
"	G	40	26	5.4	0.48	10.7
"	S-1	75	10	4.8	0.24	5.0
"	S-1	40	26	5.5	0.40	7.4
"	S-2	68	53	5.2	0.32	6.1
"	S-2	40	97	5.1	0.80	15.2
"	S-3	40	50	4.3	0.50	11.1
Cache Creek	S-1	68	21	4.3	0.17	4.5
"	S-1	40	33	3.6	0.50	12.6

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TABLE 4 - MODULUS OF RUPTURE OF UNFLEXED SPECIMENS

Test Series Aggregate	Asphalt	Test Temperature °F	Total No. of Specimens	Modulus of rupture - psi Mean	Standard Deviation	Coefficient of Variation
Watsonville	E	75	12	102	10.3	10.0
"	E	40	14	553	39.1	7.1
"	G	75	12	80.8	10.8	13.4
"	G	40	9	936	88.6	9.5
"	S-1	75	4	110	3.0	2.7
"	S-1	40	10	690	67.0	9.7
"	S-2	68	14	291	20.7	7.1
"	S-2	40	18	984	111	11.3
"	S-3	40	17	572	45.0	7.9
Cache Creek	S-1	68	6	195	20.4	10.5
"	S-1	40	12	807	106	13.1

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TABLE 5 - LOAD TO PRODUCE STRAIN AMPLITUDE OF 350×10^{-6} IN. PER IN. AT $40^{o}F$

Test Series Aggregate	Asphalt	Asphalt Content - Percent	Percent Air Voids (mean)	Start of Test Ave.	Start of Test Range	At Estimated Fatigue Life Ave.
Watsonville	E	5.8	5.1	295	270-345	205
"	G	5.6	5.4	490	365-540	380
"	S-1	5.9	5.5	387	310-460	236
"	S-2	5.9	5.1	807	710-855	516
"	S-3	5.9	4.3	417	360-470	310
Cache Creek	S-1	4.5	3.6	575	440-660	450

TABLE 6 - LOAD TO PRODUCE STRAIN AMPLITUDE OF 500×10^{-6} IN. PER IN.

Test Series Aggregate	Asphalt	Asphalt Content - Percent	Percent Air Voids (mean)	Temperature ^{o}F	Load - Lb. Start of Test-Ave.	Load - Lb. At Estimated Fatigue Life Ave.
Watsonville	E	5.8	4.4	75	60	52
"	G	5.6	3.9	75	95	76
"	S-1	5.9	4.8	75	65[a]	--
"	S-2	5.9	5.2	68	388	200
Cache Creek	S-1	5.9	4.3	68	177	121
Basalt	B	5.7	4.9	75	100	--

(a) Estimated

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TABLE 7 - SUMMARY OF DEFLECTION MEASUREMENTS

2/27/64

Station	Direction	Lane	Mean Deflection - in. $\times 10^{-3}$ Inner Wheel Track	Outer Wheel Track
246 to 256	Northbound	Travel	11.8	13.5
311 to 301	Southbound	Travel	13.6	15.0
311 to 301	Southbound	Passing	13.8	15.6
354 to 344	Southbound	Travel	11.1	14.3
354 to 344	Southbound	Passing	12.2	13.9

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TABLE 8 - PHYSICAL PROPERTIES OF ASPHALT CEMENTS[1] USED IN SURFACE AND BASE COURSES - GONZALES BYPASS

	Surface Course		Base Course	
	Original[2]	New	Original[3]	New
Penetration at 77°F, 100 gm, 5 sec., dmn	94[(a)]	84	91[(d)]	87
Penetration Ratio	34[(b)]	39	39[(e)]	38
Flash Point, P.M.C.T., °F	490[(b)]	485	466[(e)]	450
Viscosity at 275° F, SSF	185[(c)]	186	177[(e)]	172
Softening Point, Ring and Ball, °F	---	117	---	116.5
Heptane Xylene Equivalent	-35[(b)]	-35	-35[(f)]	-35
Thin Film Oven Test, 325°F, 5 hrs.:				
Percent weight loss	0.33[(c)]	0.25	.57[(f)]	0.54
Percent penetration retained	57[(c)]	65	54[(f)]	62
Ductility of residue, 77°F, cm	75+[(c)]	100+	75+[(f)]	100+

[1]Tests performed at the laboratories of the Materials and Research Department.

[2]Tests from samples obtained during January through June 1963.

(a) 176 Samples
(b) 23 Samples
(c) 13 Samples

[3]Tests from samples obtained during December 1962 through January 1963.

(d) 48 Samples
(e) 10 Samples
(f) 5 Samples

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TABLE 9 - AGGREGATE SPECIFIC GRAVITIES (ASTM C-127, 128)

Aggregate	Fraction	Bulk Specific Gravity	Apparent Specific Gravity	Absorption (percent)
Zaballa pit	Coarse	2.58	2.68	1.3
	Fine	----	2.62	---
Watsonville Granite (Granite Rock-Aromos)	All	----	2.92	---

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TABLE 10 - PAVEMENT SAMPLE[1] SUMMARY

U. C. Designation	Station and Location	Thickness - ft		
		Course	Plan	Actual
Block No. 1	308 + 15 Southbound travel lane; between wheel track	Open Graded	0.06	0.05
		Surface	0.30	0.29
		Base	0.25	0.23
Block No. 2	350 + 10 Southbound travel lane; between wheel track	Open graded	0.06	0.04
		Surface	0.30	0.29
		Base	0.25	0.25
Block No. 3	308 + 10 Southbound travel lane; outer wheel track	Open graded	0.06	0.05
		Surface	0.30	0.27
		Base	0.25	0.27
Block No. 4	350 + 15 Southbound travel lane; between wheel track	Open graded	0.06	0.04
		Surface	0.30	0.30
		Base	0.25	0.27

[1]Samples taken March 2-4, 1965.

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TABLE 11 - SPECIFIC GRAVITIES[1] OF FIELD AND LABORATORY PREPARED SPECIMENS - GONZALES BYPASS

Material	No. of specimens	Specific Gravity		
		Mean	Standard Deviation	Coefficient of variation (percent)
Field surface	78	2.23	0.02	0.92
Lab. surface (series 1)	24	2.28	0.014	0.60
Lab. surface (series 2)	10	2.30	0.019	0.85
Field Base	44	2.46	0.02	0.93
Lab. Base	32	2.54	0.02	0.78

[1]Based on weight in air and weight of uncoated specimen in water.

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TABLE 12 - MEASURED STIFFNESS MODULI OF BEAM SPECIMENS IN CONTROLLED-STRESS TESTS

Stiffness - psi × 10^5

	@ 68°F				@ 40°F			
	No. of Spec.	Mean	Standard Deviation	Coef. of Variation (percent)	No. of Spec.	Mean	Standard Deviation	Coef. of Variation (percent)
Specimens for Surface Course:								
Field: Block No. 1	11	1.79	0.44	24.6	8	6.80	1.96	28.8
Block No. 2	12	1.65	0.39	23.7	8	7.03	1.91	27.2
Block No. 3	13	1.52	0.41	27.1	7	7.12	1.52	21.4
Block No. 4	12	1.35	0.34	25.0	7	5.90	1.10	18.6
Laboratory Prepared	12	1.29	0.22	18.6	15	5.76	0.75	13.0
Specimens for base course:								
Field: Block No. 1	6	1.57	0.42	26.6	6	5.95	1.54	25.8
Block No. 2	5	1.39	0.26	18.7	7	5.66	3.14	55.5
Block No. 3	6	1.47	0.41	28.0	4	4.44	0.90	20.5
Block No. 4	6	1.42	0.42	29.9	4	4.96	1.22	24.6
Laboratory prepared	13	1.19	0.19	15.7	16	5.31	1.50	28.4

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TABLE 13 - RECOVERED ASPHALT PROPERTIES[1]

Layer	Block No.	Test Results on Recovered Asphalt		
		Pen. at 77°F dmm	Ring and Ball Soft. Pt. °F	Ductility at 77°F cm
Surface	1	46	128	100+
Surface	2	36	138	100+
Surface	3	52	133	100+
Surface	4	59	126	100+
Surface	Laboratory	57	128	100+
Base	1	36	130	100+
Base	2	35	138	100+
Base	3	33	140	75
Base	4	48	126	100+
Base	Laboratory	45	132	100+

[1]Tests performed at Materials and Research Department Laboratories.

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TABLE 14 - SUMMARY OF CHARACTERISTICS OF SUBGRADE SOIL

Sample	Deviator stress, σ_d psi	Water content percent	Dry density lb. per cu. ft.
3	1	10.7	121
4	3	11.3	120
5	10	10.7	117
6	5	10.7	118
1	5	11.6	---

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TABLE 15 - STIFFNESS DETERMINATIONS, ASPHALT CONCRETE TEST SPECIMENS

Sample Group	No. of Specimens	Ave. asphalt Content	Ave. C_v	Percent air voids	Recovered Asphalt		Computed S[a] psi × 10^5		Measured S[b], psi × 10^5			
									60°F		40°F	
					Pen @77°F dmm	R&B Soft pt-°F	68°F	40°F	Mean	Std. Dev.	Mean	Std. Dev.
Specimens from surface course:												
1	19	5.9[c]	0.87	8.0	46	128	3.3	15.1	1.79	0.42	6.80	1.53
2	20	5.9[c]	0.87	8.9	36	138	5.0	14.7	1.65	0.39	7.03	1.91
3	20	5.9[c]	0.87	7.6	52	133	3.2	11.8	1.52	0.41	7.12	1.41
4	19	5.9[c]	0.87	8.1	59	126	3.1	12.7	1.34	0.37	5.90	1.11
Lab. Compacted	26	6.0	0.86	5.0	57	128	2.8	10.5	1.29	0.22	5.76	0.74
Specimens from base course:												
1	12	4.6[c]	0.87	8.8	36	130	7.4	23.2	1.57	0.42	5.95	1.54
2	12	4.6[c]	0.88	8.0	35	130.5	8.9	27.4	1.37	0.26	5.66	3.14
3	8	4.6[c]	0.88	9.4	33	140	5.8	21.0	1.47	0.41	4.40	0.90
4	10	4.6[c]	0.88	9.1	48	129	5.8	20.1	1.42	0.42	4.96	1.22
Lab. Compacted	29	4.7	0.89	5.7	45	132	7.6	27.8	1.19	0.19	5.31	1.50
Granite + 85-100 pen. asp.[d]	76	6.0	0.85	4.5	40	121	2.8[e]	--	2.62[e]	0.32[e]	--	--
Granite + 85-100 pen. asp.[d]	14	6.0	0.85	3.6	52	128	2.7	11.5	2.06	0.58	12.6	1.82
Granite + 85-100 pen. asp.[d]	12	6.0	8.85	3.0	52	123	3.6	--	2.10	0.42	--	--
Granite + 15 pen. asp.[d]	12	6.0	0.85	3.6	15	161	7.8	--	5.60	0.92	--	--

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Footnotes to Table 15

(a) Stiffness determined at a time of loading of 0.1 sec. (from Figs. 37 and 38).

(b) Flexural stiffness measured after 200 repetitions of bending stress at 0.1 sec. loading time in flexural fatigue apparatus developed by Deacon (6).

(c) Average asphalt content based on analysis of mixture samples obtained at central plant at the time of construction.

(d) Laboratory prepared specimens with a gradation conforming to the 1960 State of California Division of Highways for 3/8 in. maximum size from an investigation by Deacon (6).

(e) Stiffness at 75°F.

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TABLE 16 - SURFACE DEFLECTION COMPUTATIONS

Asphalt Concrete Stiffness, psi	125,000	125,000	1,000,000	1,000,000
Resilient modulus of subgrade, psi				
at N = 1,000 stress applications	16,000	---	23,500	---
at N = 100,000 stress applications	---	27,500	---	60,000
Average resilient modulus of untreated aggregate, psi	21,000	26,000	23,500	28,600
Computed surface deflection, in $\times 10^{-3}$	18.0	14.2	8.0	5.3

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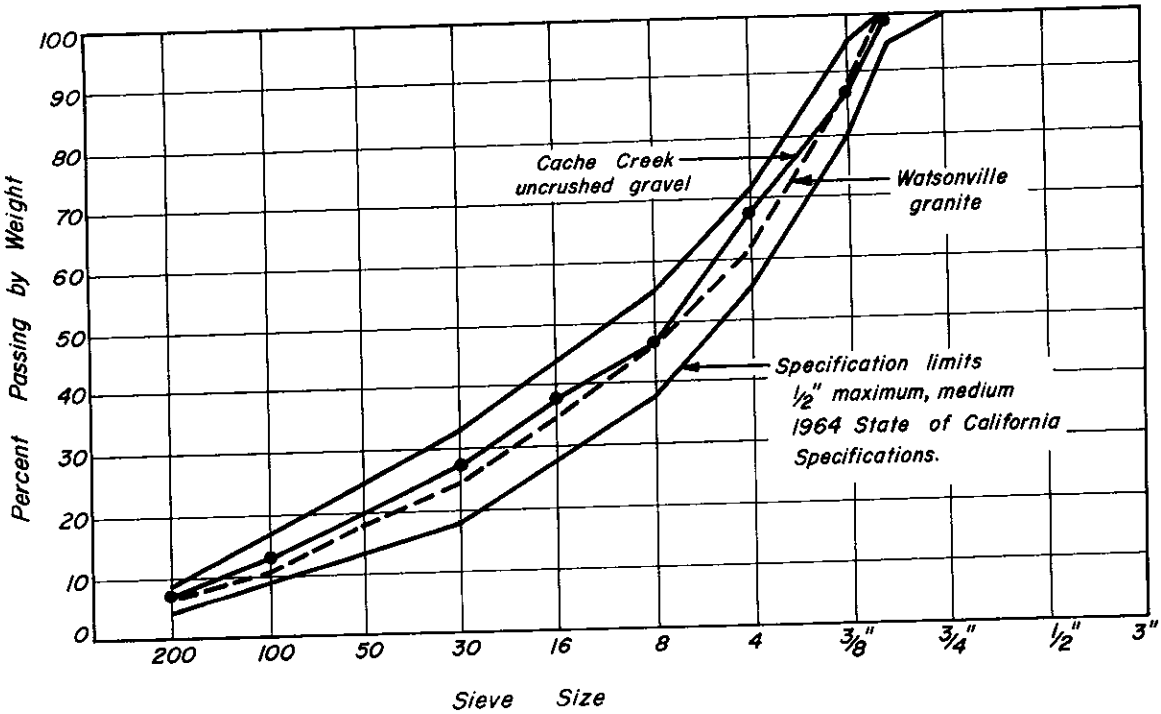

Fig. 1 — Grading curves, Watsonville granite and Cache Creek gravel; constant-strain tests.

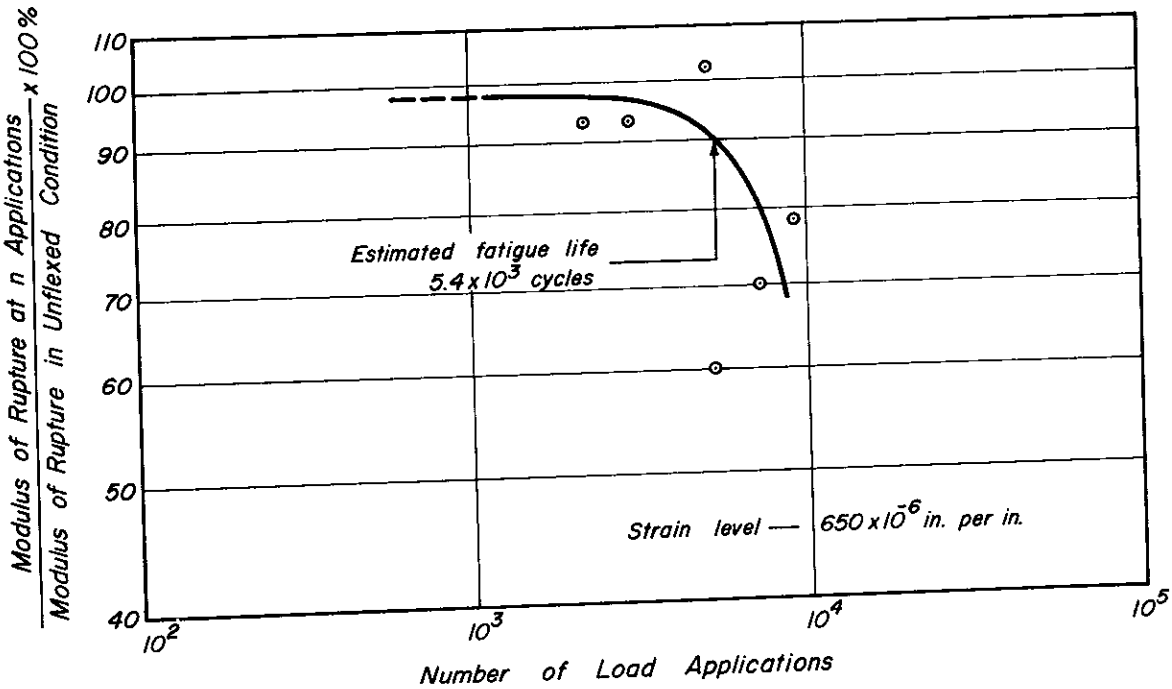

Fig. 2 - Modulus of rupture vs. number of load applications for specimens prepared with Watsonville granite and asphalt sample S-2 at 68°F.

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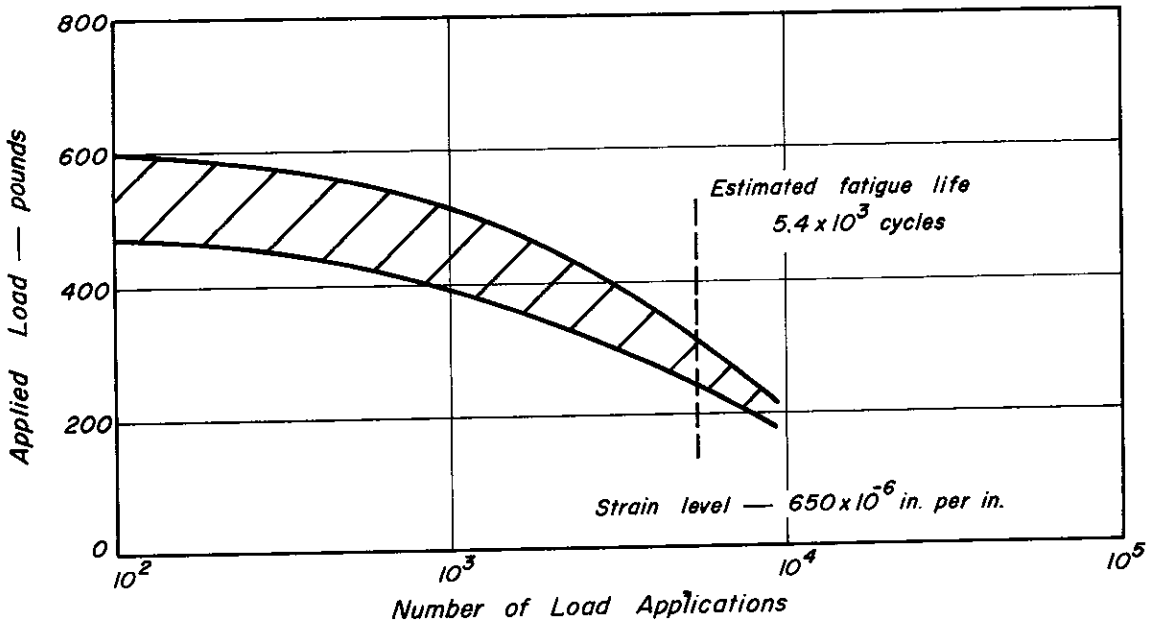

Fig. 3 — Load to produce strain of 650 x 10^{-6} in. per in. vs. number of load applications for specimens containing Watsonville granite and asphalt sample S-2 at 68°F.

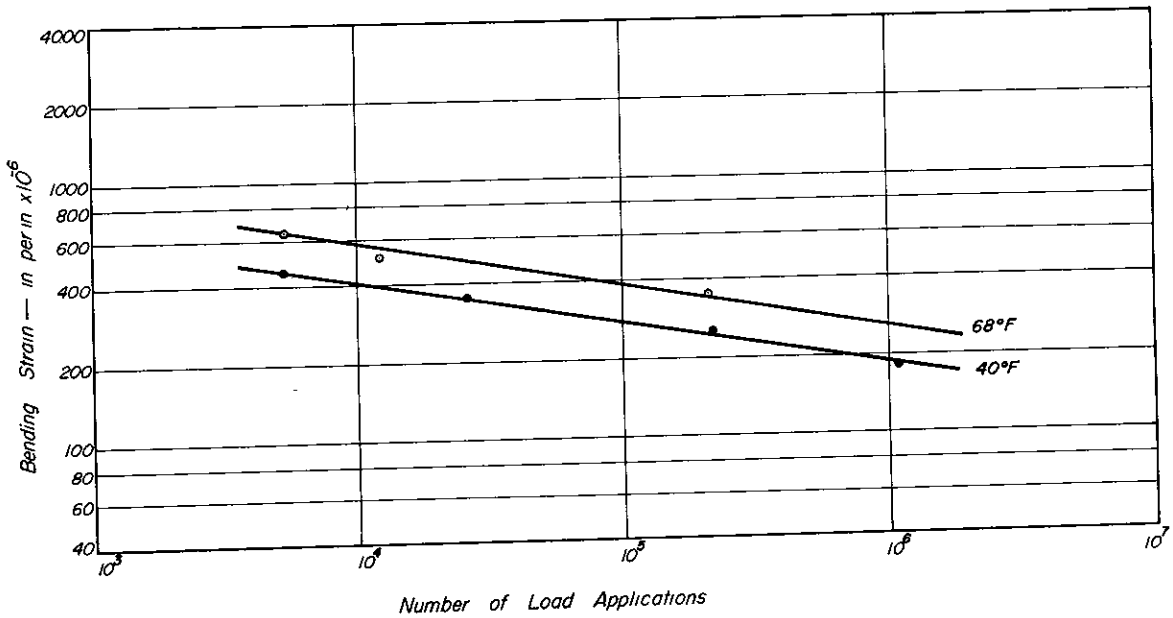

Fig. 4 - Results of constant-strain amplitude fatigue tests at 40°F and 68°F for specimens containing Watsonville granite and asphalt sample S-2.

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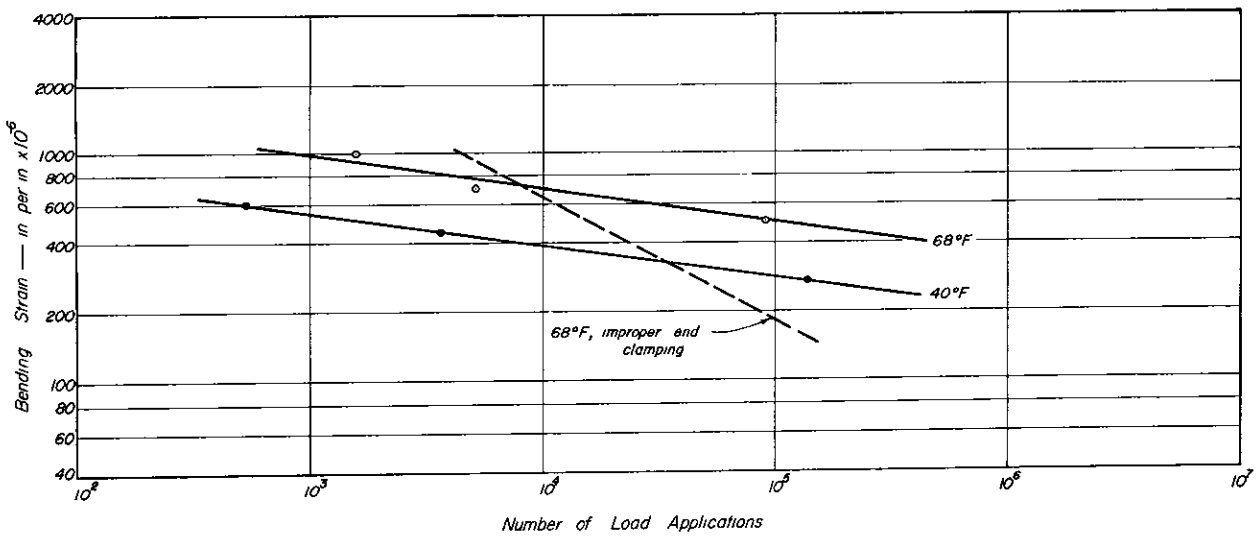

Fig. 5 — Results of constant-strain amplitude fatigue tests at 40°F and 68°F for specimens containing Cache Creek gravel and asphalt sample S-1.

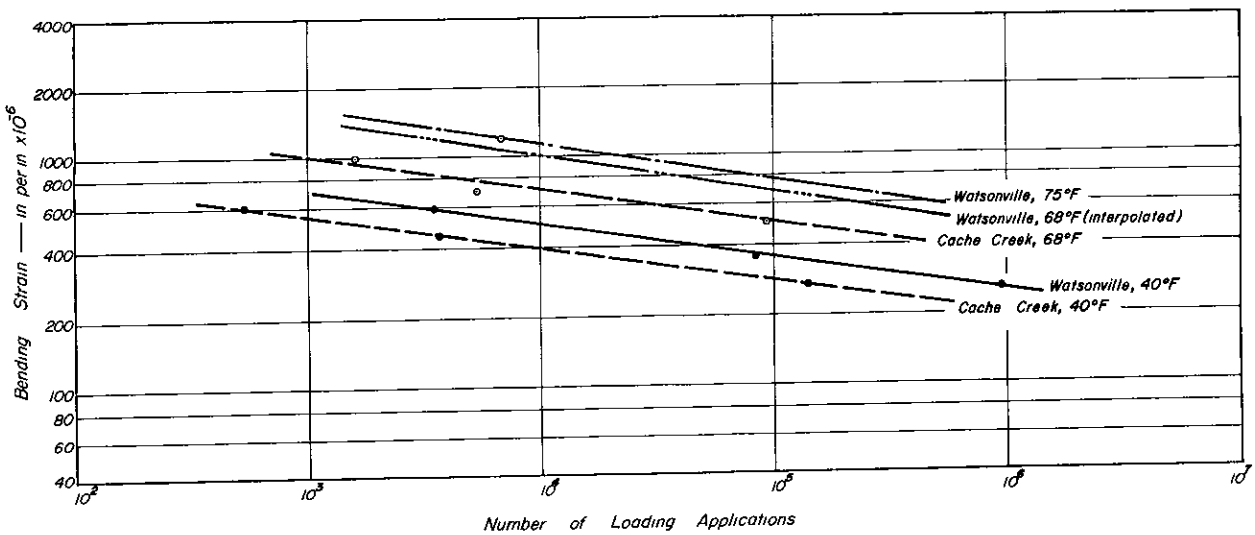

Fig. 6 — Comparison of constant-strain amplitude fatigue test results for specimens containing Watsonville granite and Cache Creek gravel and asphalt sample S-1.

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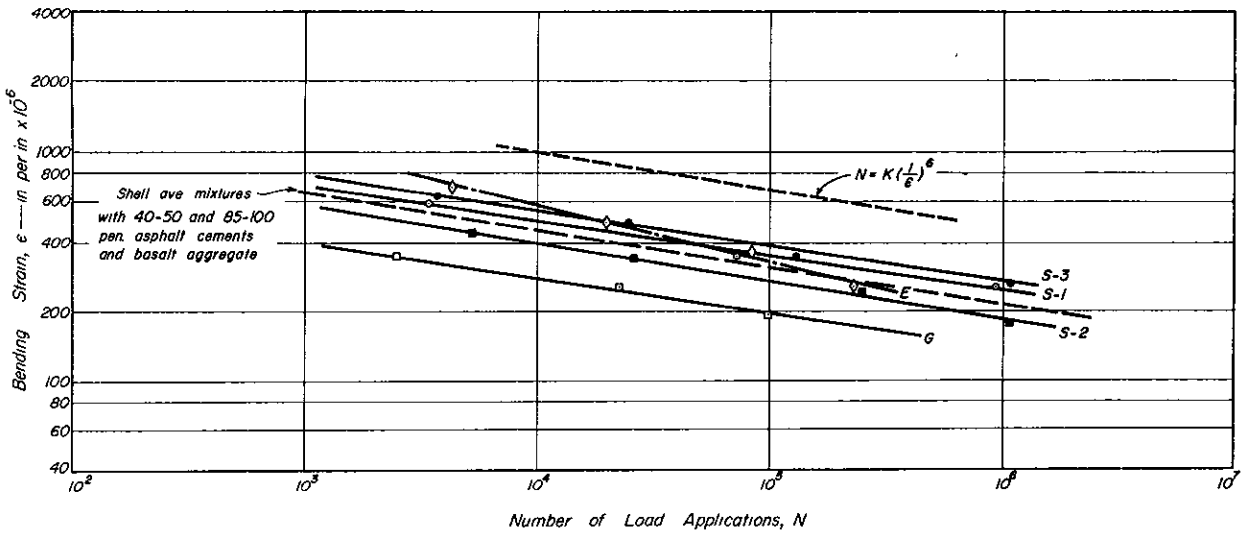

Fig. 7 — Summary of constant-strain amplitude fatigue test results at 40°F.

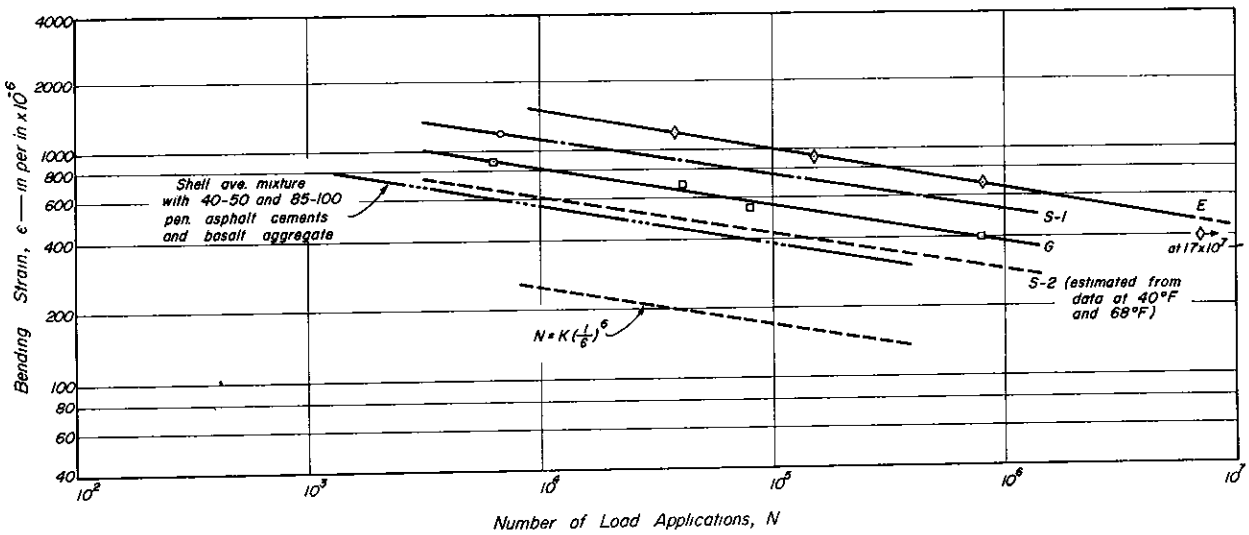

Fig. 8 — Summary of constant-strain amplitude fatigue test results at 75°F.

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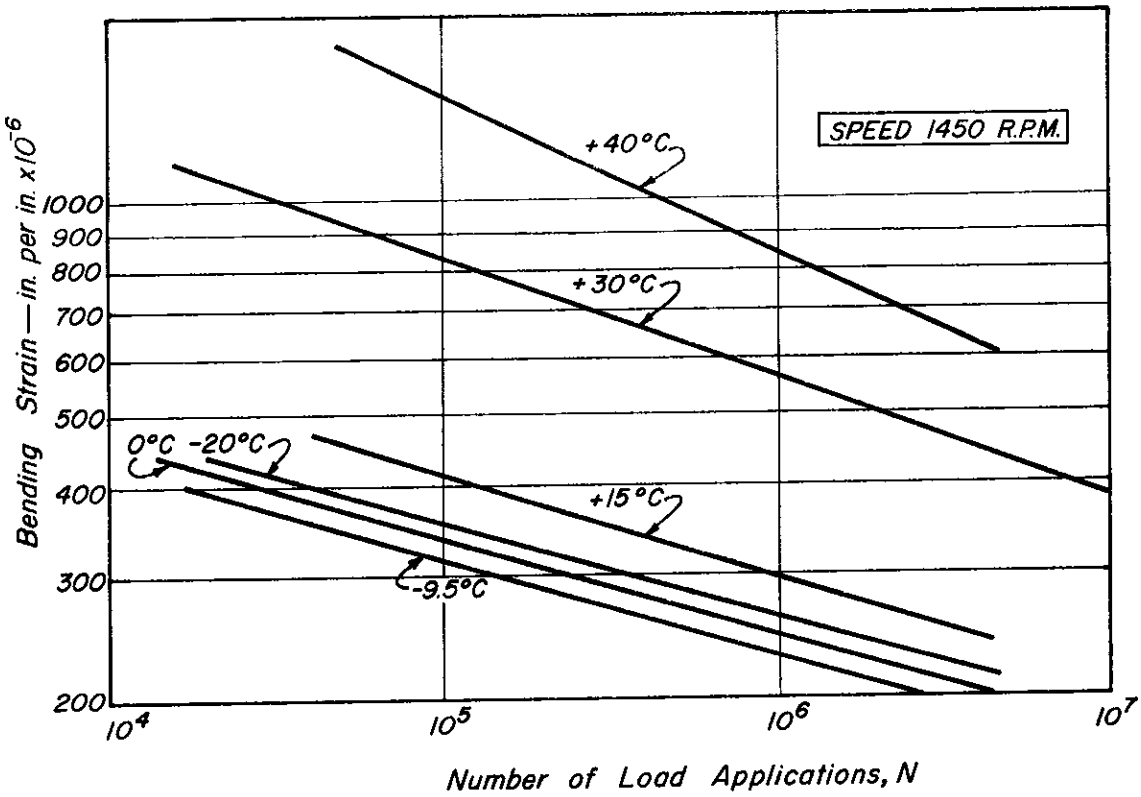

Fig. 9 — Fatigue results for sheet asphalt specimens at various temperatures under constant torsional strain. (After Pell.)

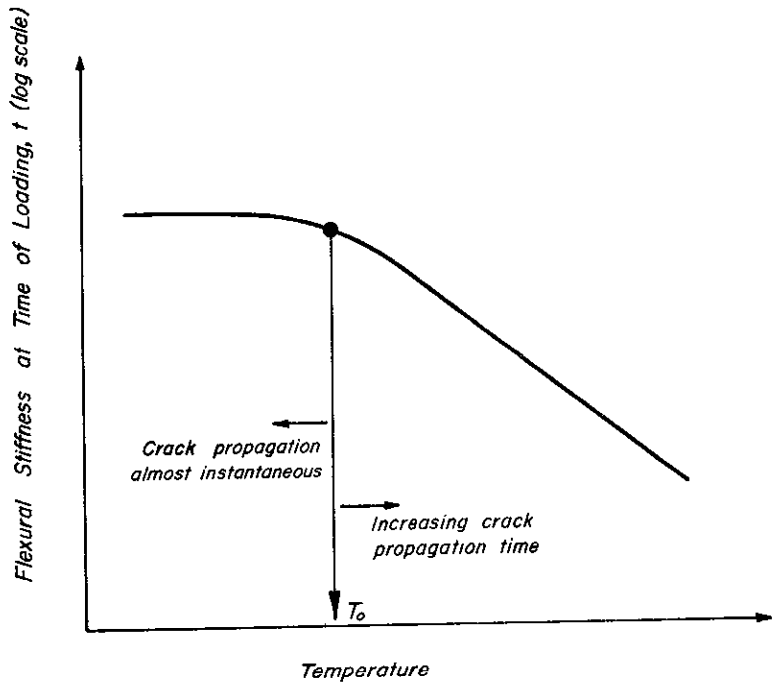

Fig. 10 — Flexural stiffness of asphalt concrete at a particular time of loading vs. temperature.

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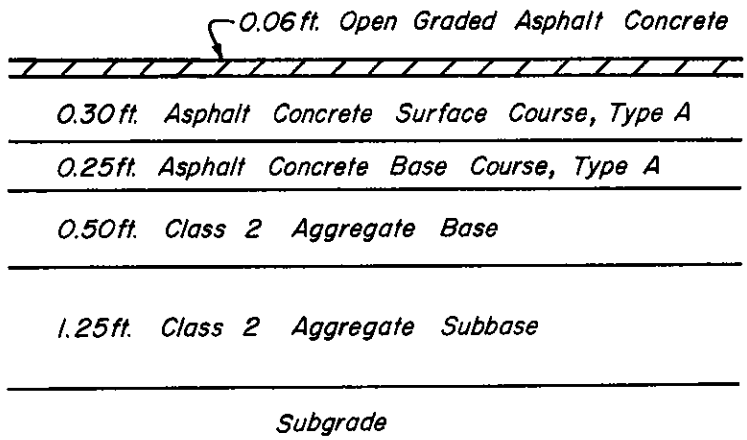

Fig. 11 — Structural section, Gonzales By-Pass (V-Mon-2-C).

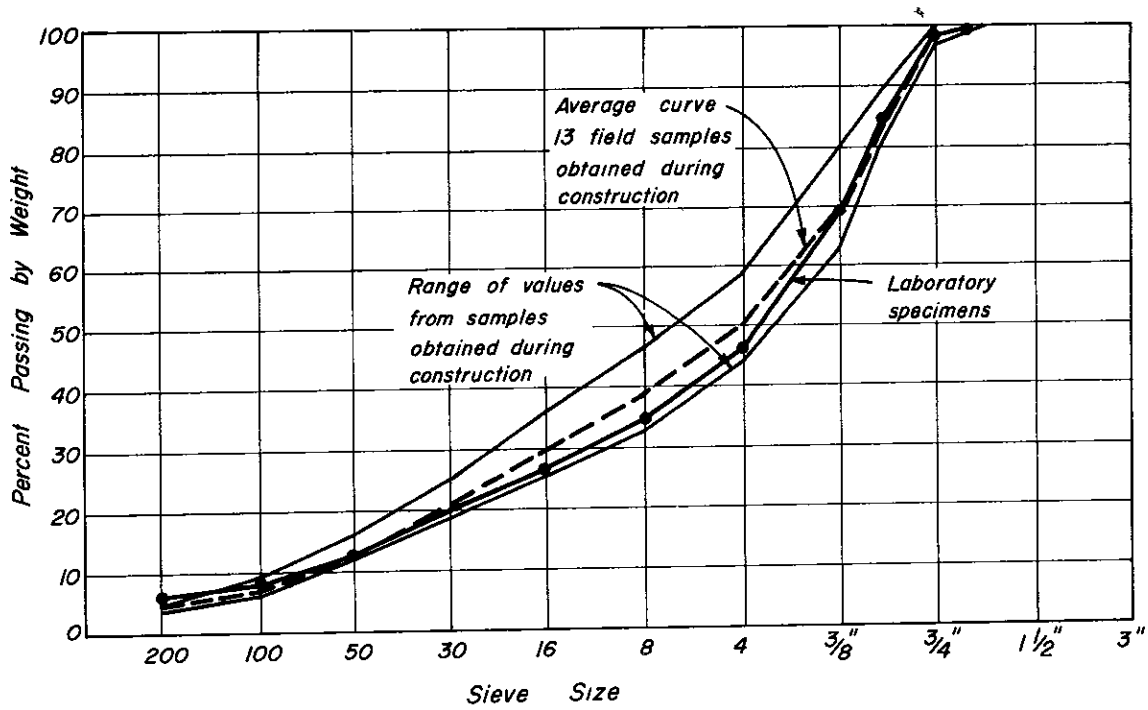

Fig. 12 — Aggregate grading curves, surface course; Gonzales By-Pass.

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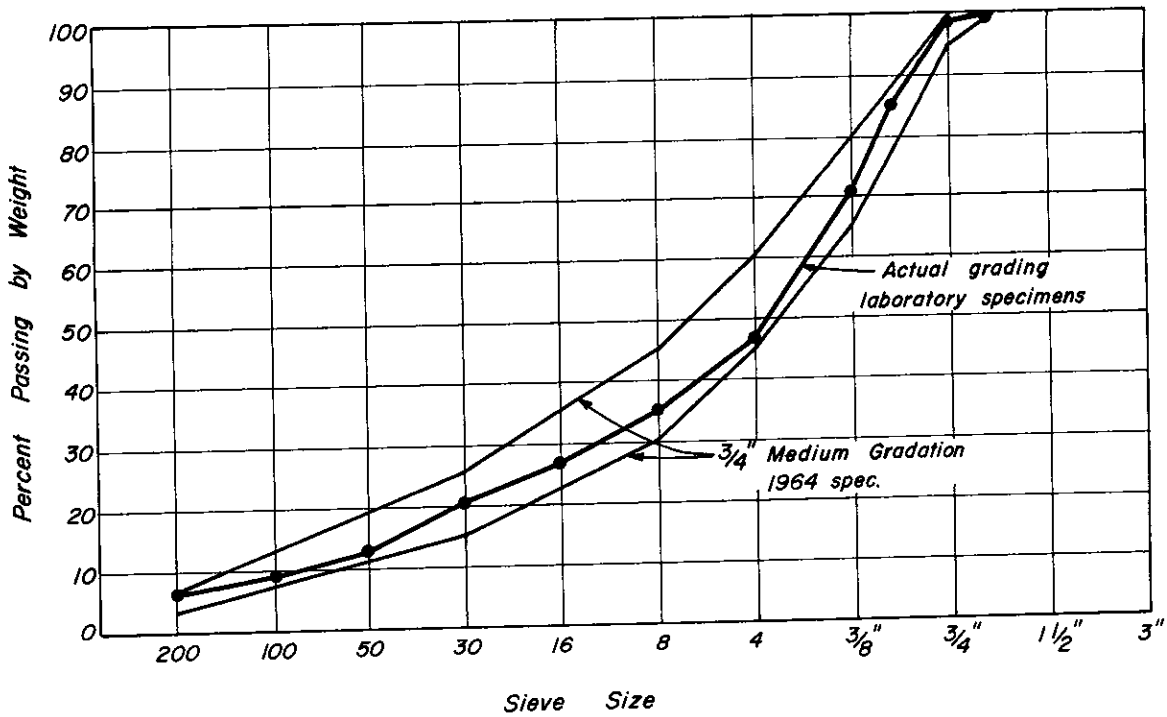

Fig. 13 — Aggregate grading curve, laboratory prepared specimens of surface course; Gonzales By-Pass.

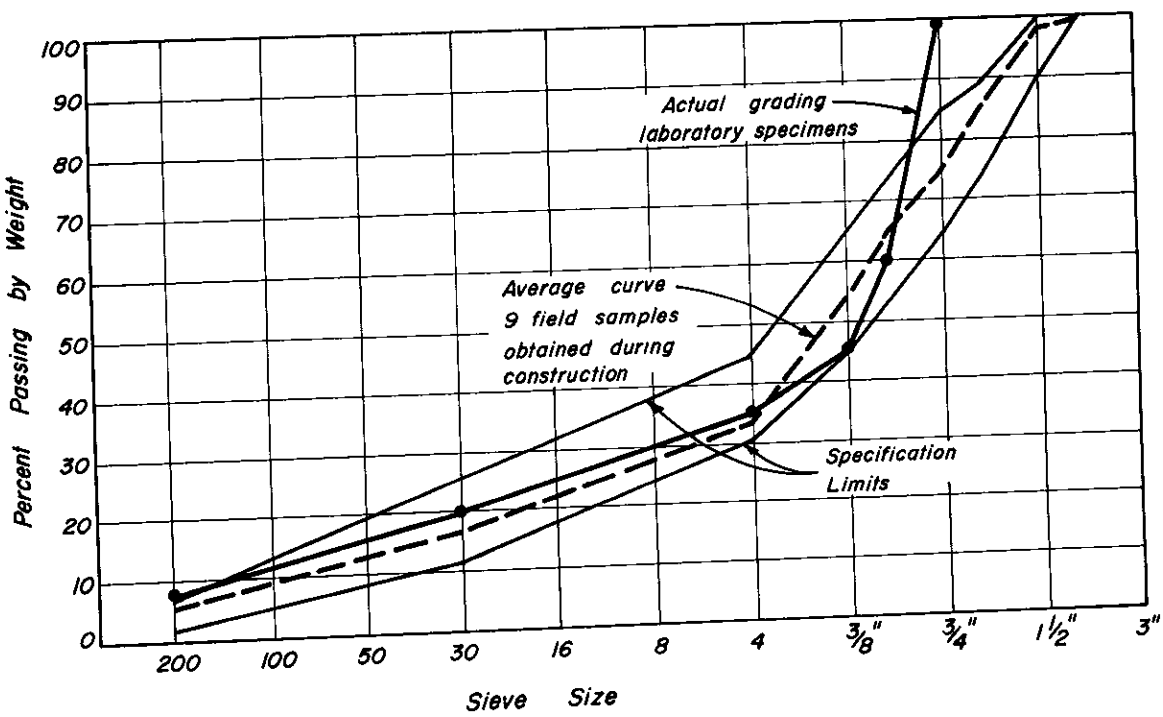

Fig. 14 — Aggregate grading curves, asphalt concrete base course; Gonzales By-Pass.

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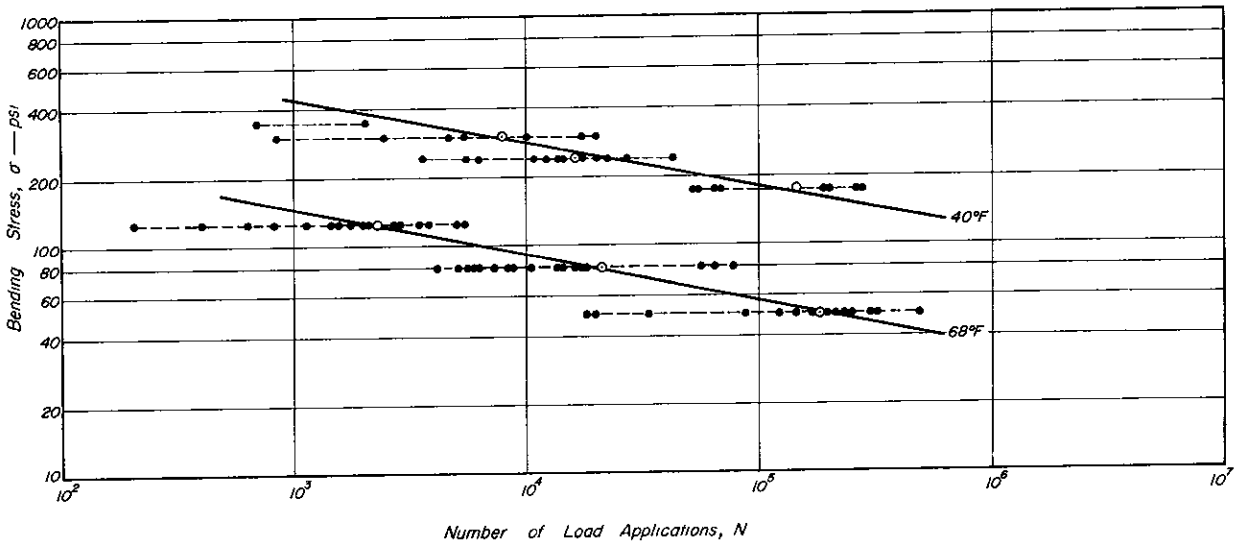

Fig. 15 — Results of constant-stress fatigue tests on field specimens of surface course; Gonzales By-Pass; 40°F and 68°F.

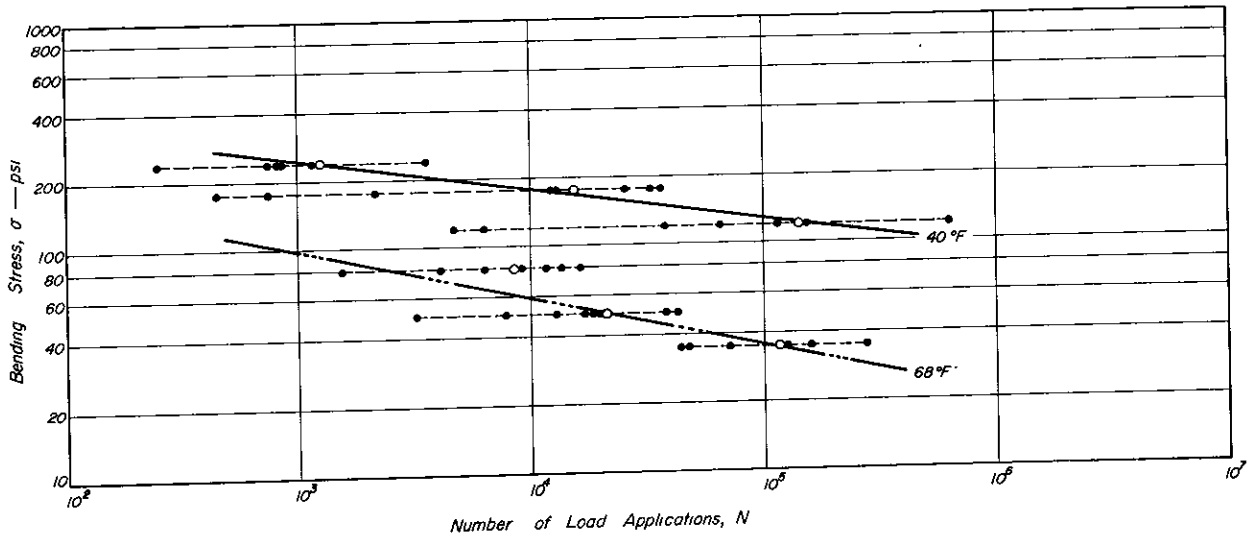

Fig. 16 — Results of constant-stress fatigue tests on field specimens of asphalt concrete base course, Gonzales By-Pass; 40°F and 68°F.

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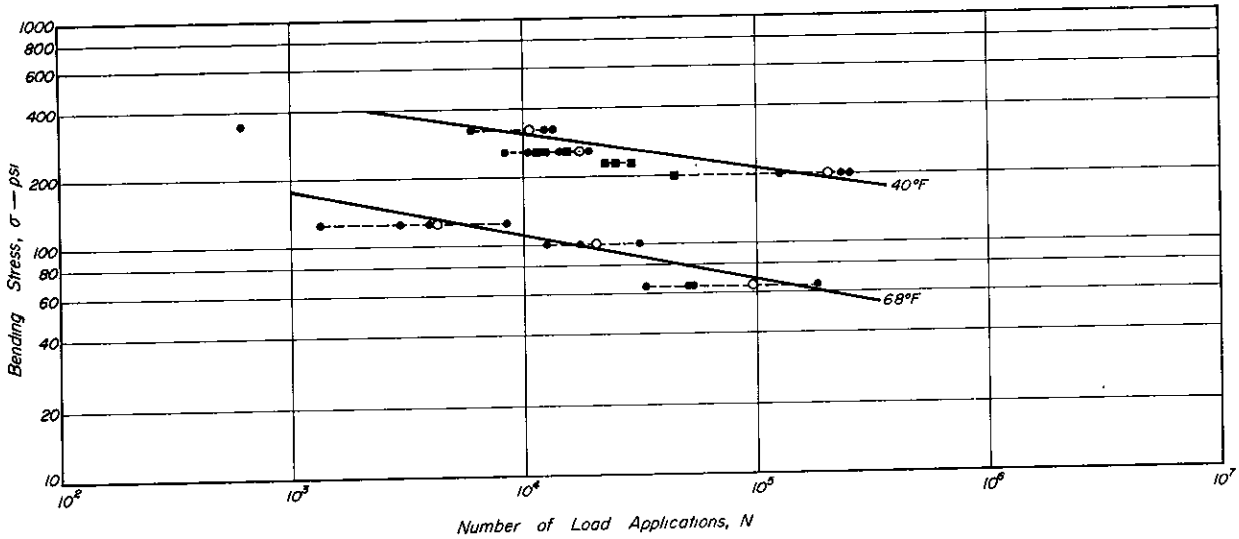

Fig. 17 — Results of constant-stress fatigue tests on laboratory prepared specimens of surface course, Gonzales By-Pass; 40°F and 68°F.

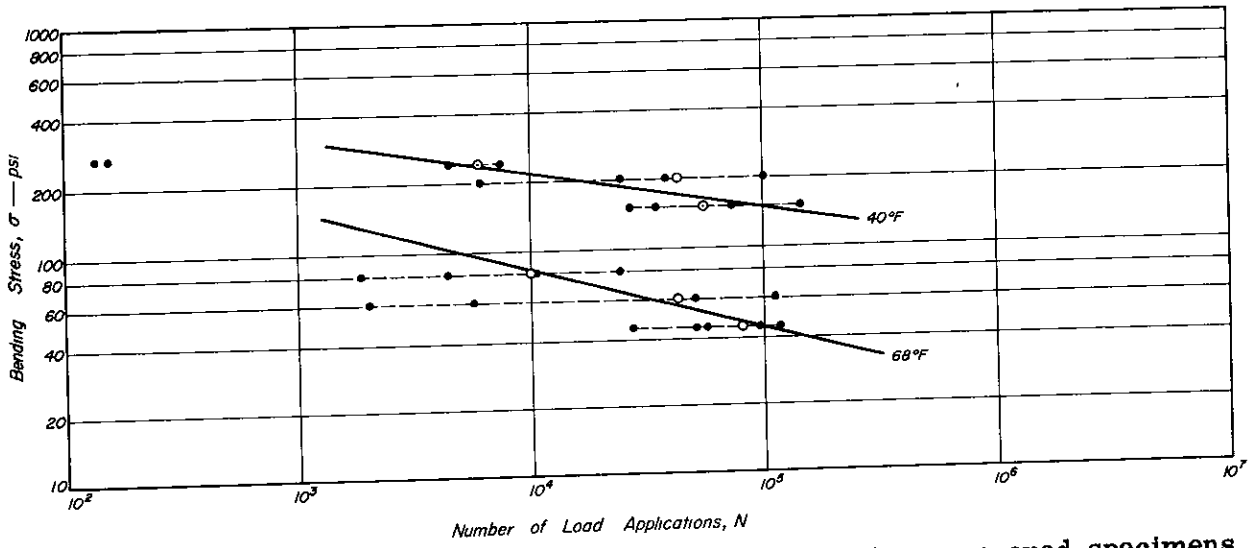

Fig. 18 — Results of constant-stress fatigue tests on laboratory prepared specimens at 40°F and 68°F; Gonzales By-Pass.

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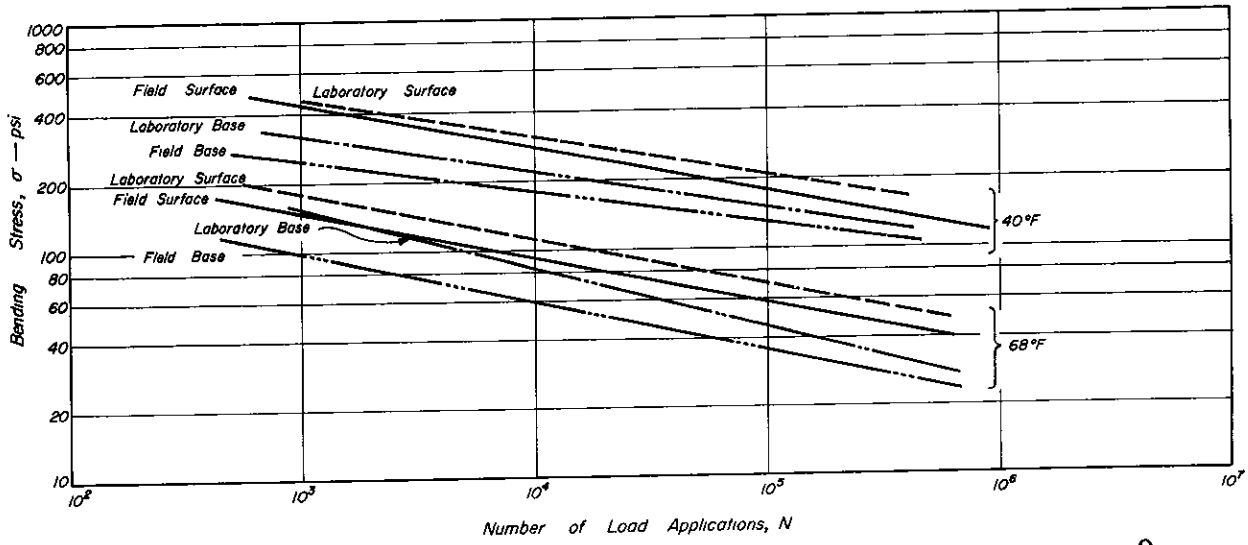

Fig. 19 — Comparison between field and laboratory prepared specimens at 40°F and 68°F; Gonzales By-Pass.

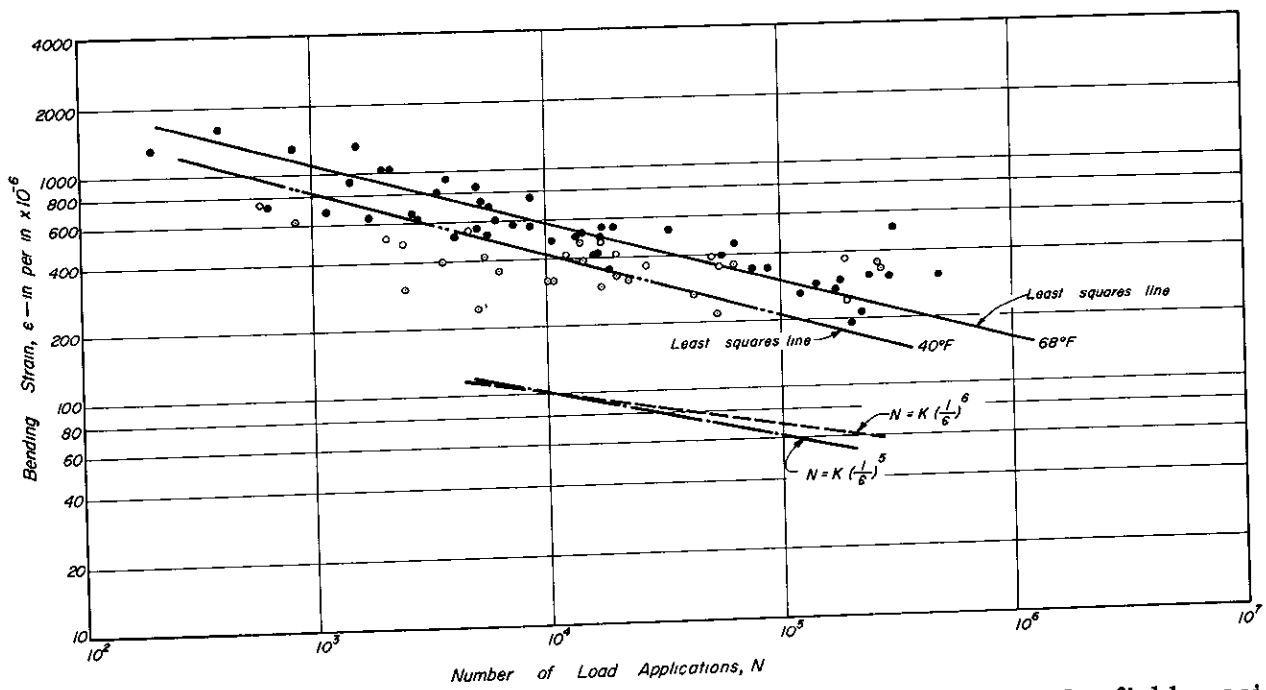

Fig. 20 — Computed initial strain vs. number of applications to failure for field specimens of surface course; Gonzales By-Pass.

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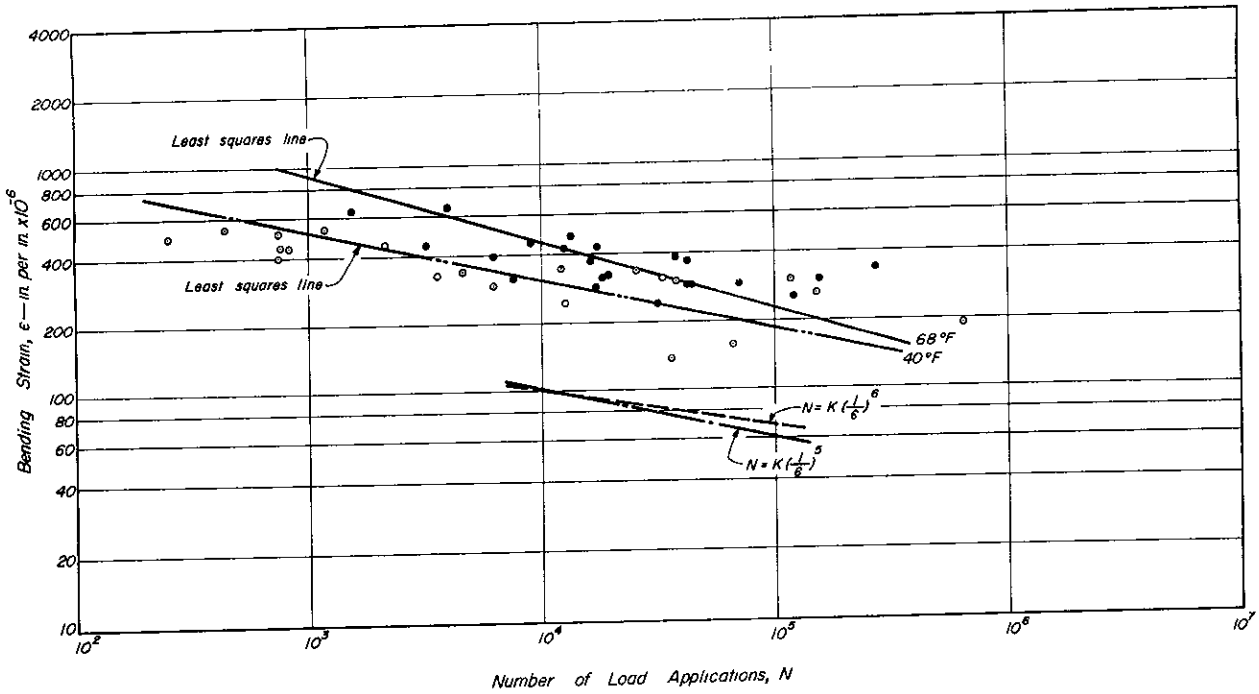

Fig. 21 — Computed initial strain vs. number of applications to failure for field specimens of asphalt concrete base course; Gonzales By-Pass.

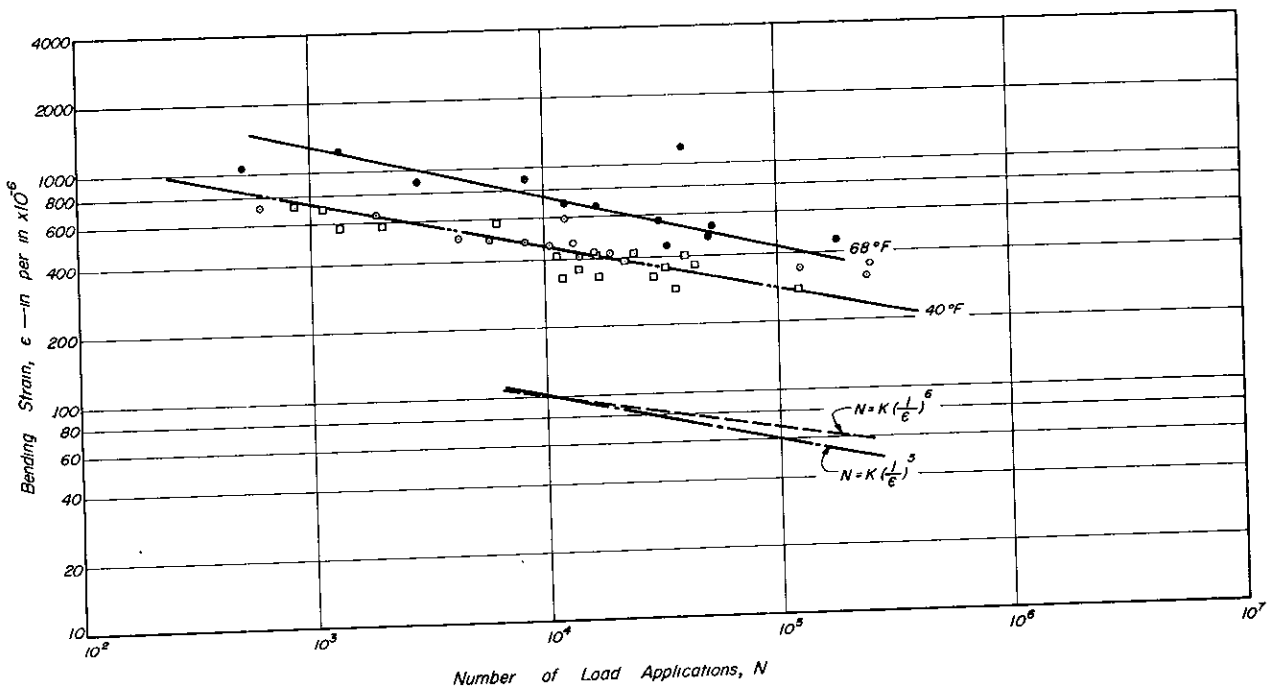

Fig. 22 — Computed initial strain vs. number of applications to failure for laboratory prepared specimens of surface course; Gonzales By-Pass.

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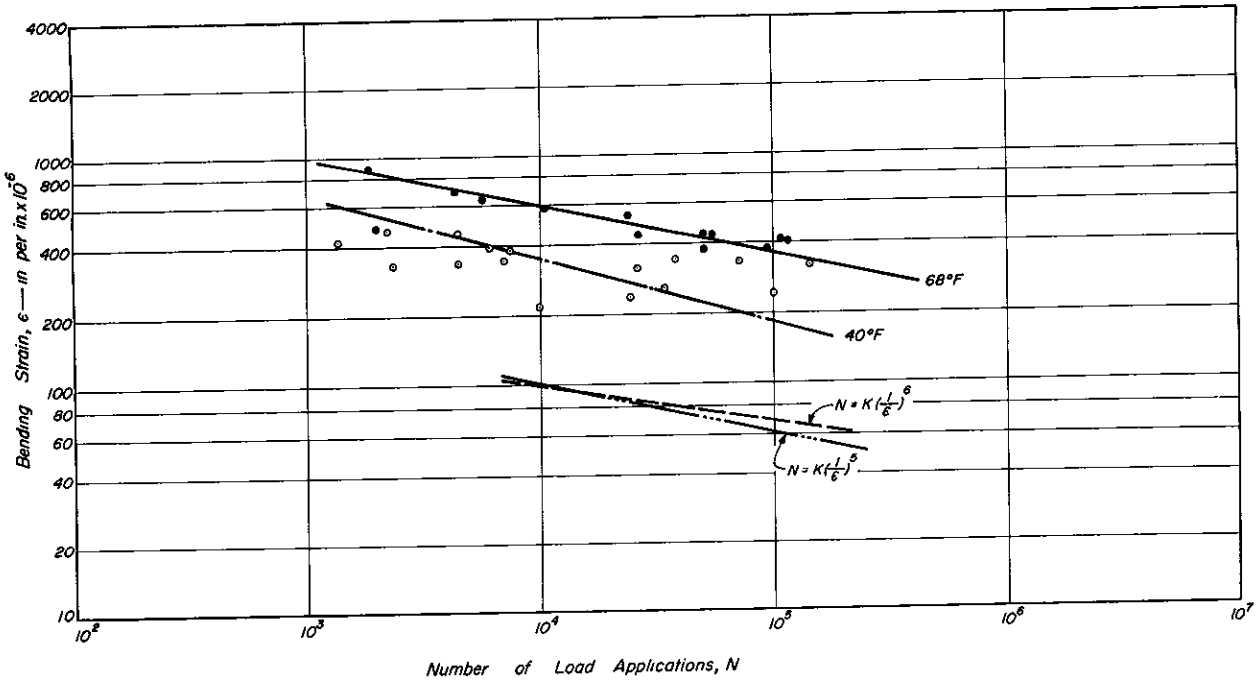

Fig. 23 — Computed initial strain vs. number of applications to failure for laboratory prepared specimens of base course; Gonzales By-Pass.

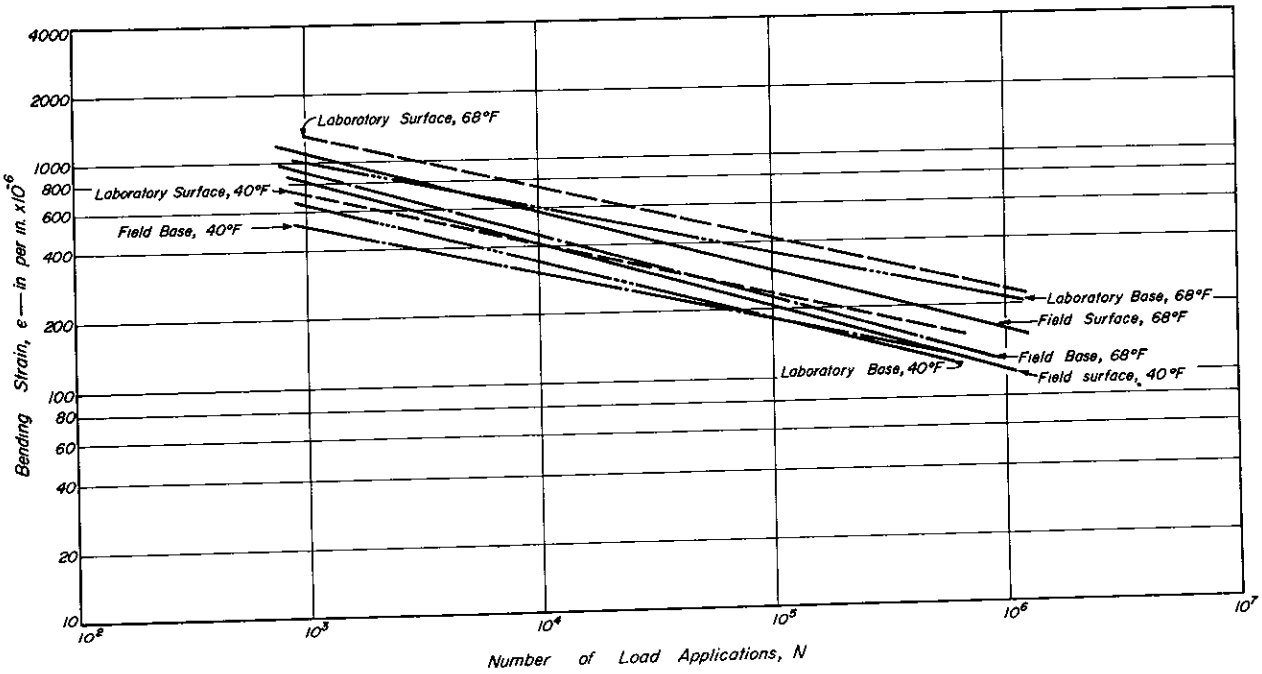

Fig. 24 — Comparison between computed initial strain and number of load applications to failure, all specimens from Gonzales By-Pass; 40°F and 68°F.

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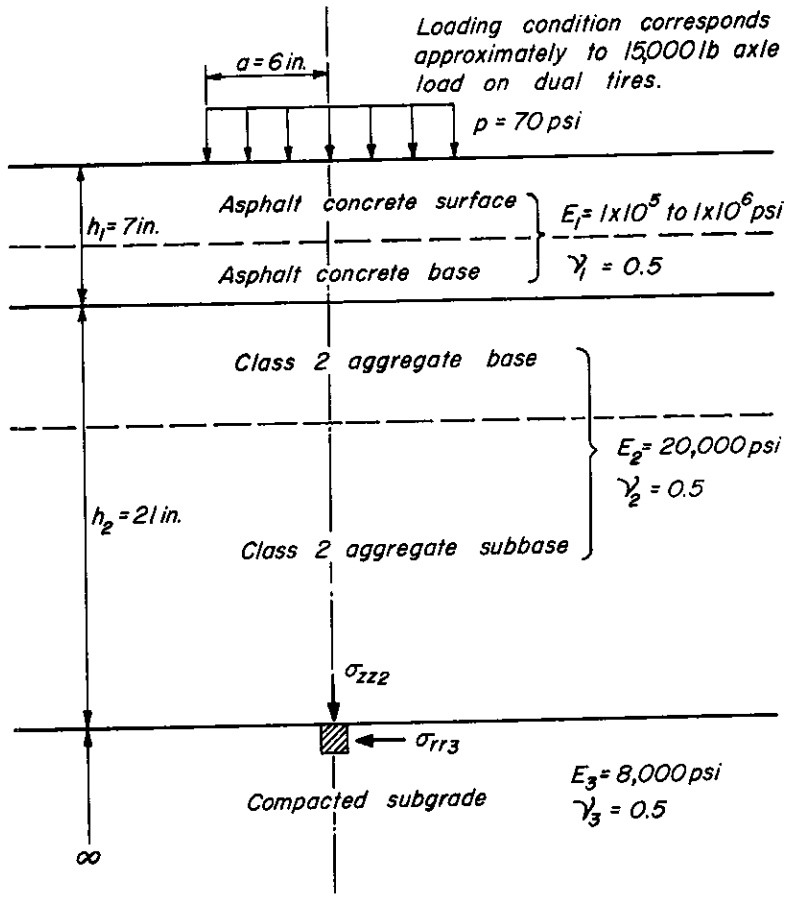

Fig. 25 — Conditions assumed to determine range of subgrade stresses for repeated load tests.

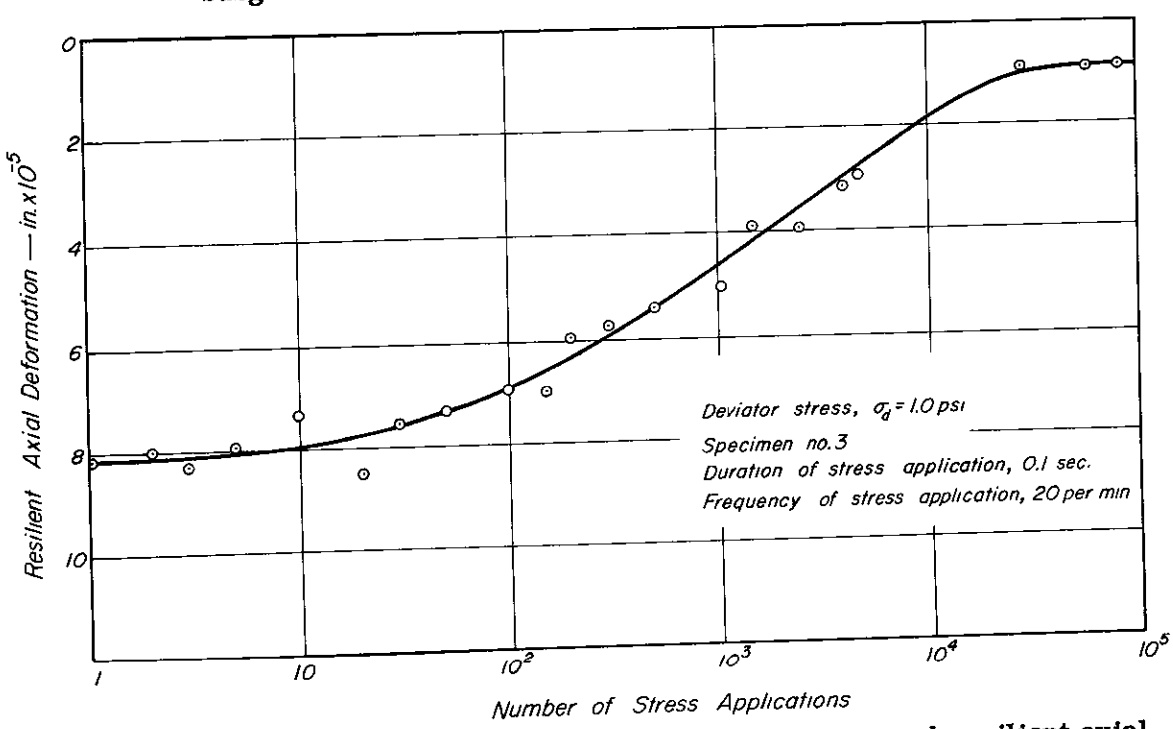

Fig. 26 — Relationship between number of load applications and resilient axial strain for undisturbed sample of subgrade soil from Gonzales By-Pass subjected to repeated deviator stress of 1. 0 psi.

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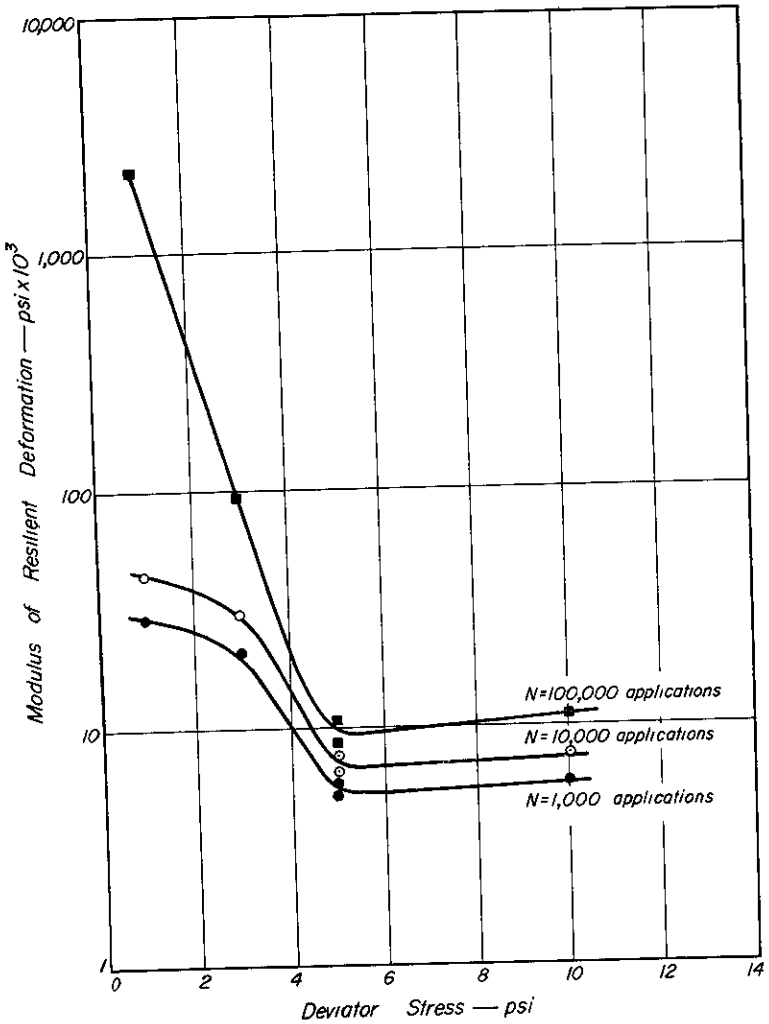

Fig. 27 — Effects of stress intensity and number of load applications on resilient modulus for undisturbed specimens of subgrade soil; Gonzales By-Pass.

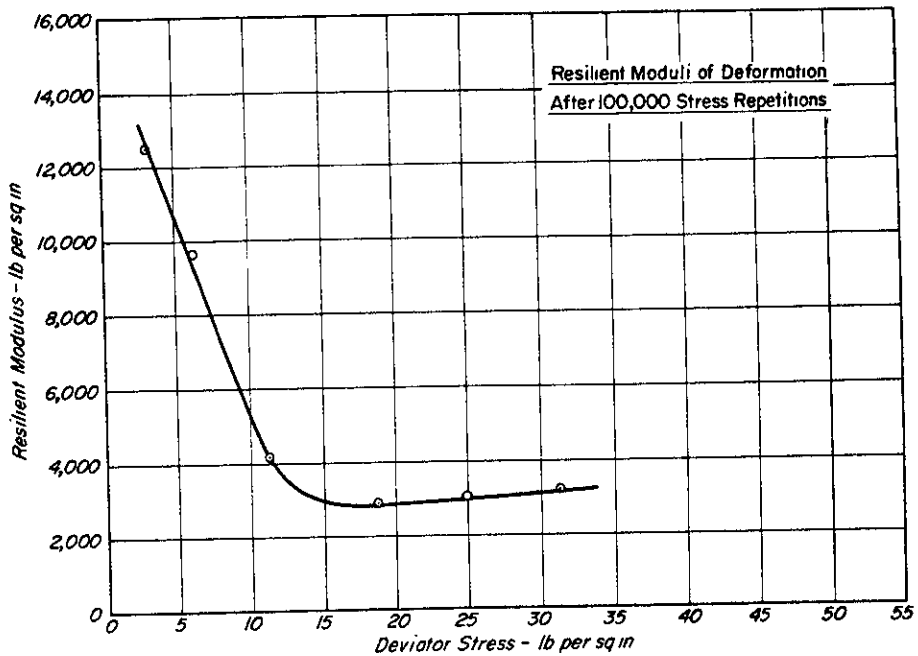

Fig. 28 — Effect of stress intensity on resilient modulus for AASHO subgrade soil.

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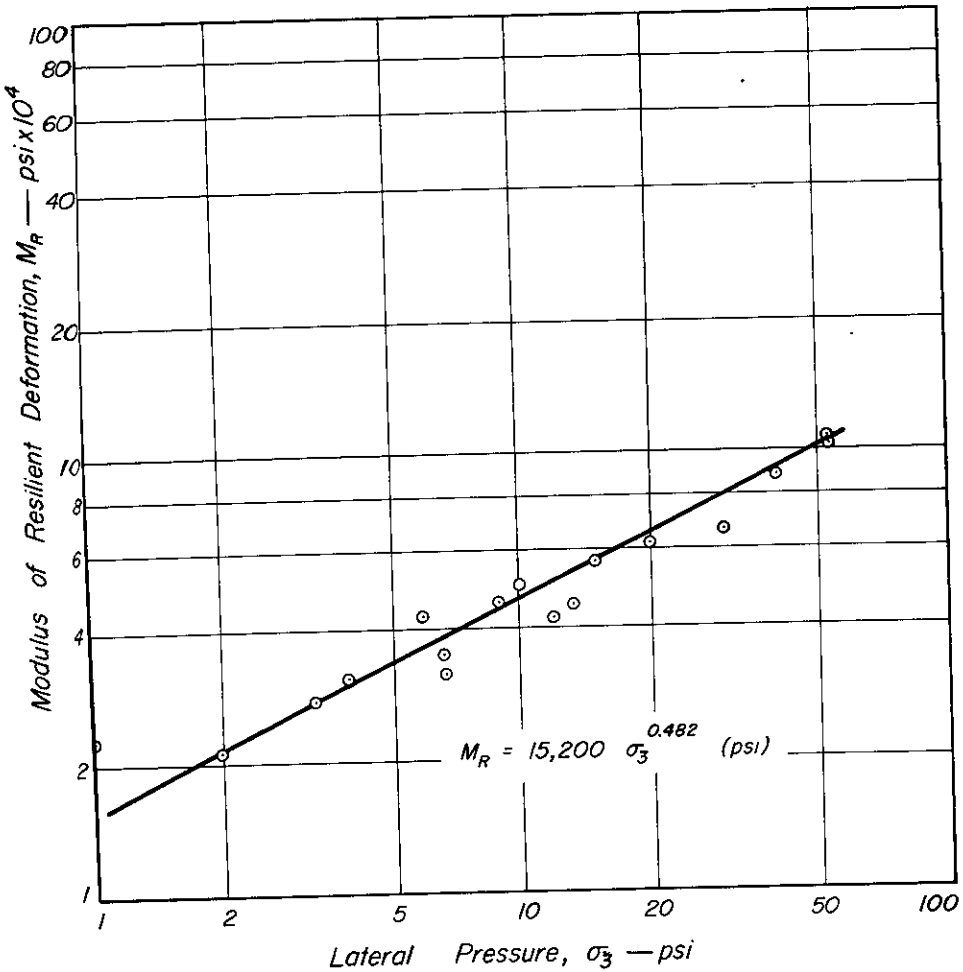

Fig. 29 — Relationship between modulus of resilient deformation and confining pressure, laboratory prepared specimens of untreated base course; Gonzales By-Pass.

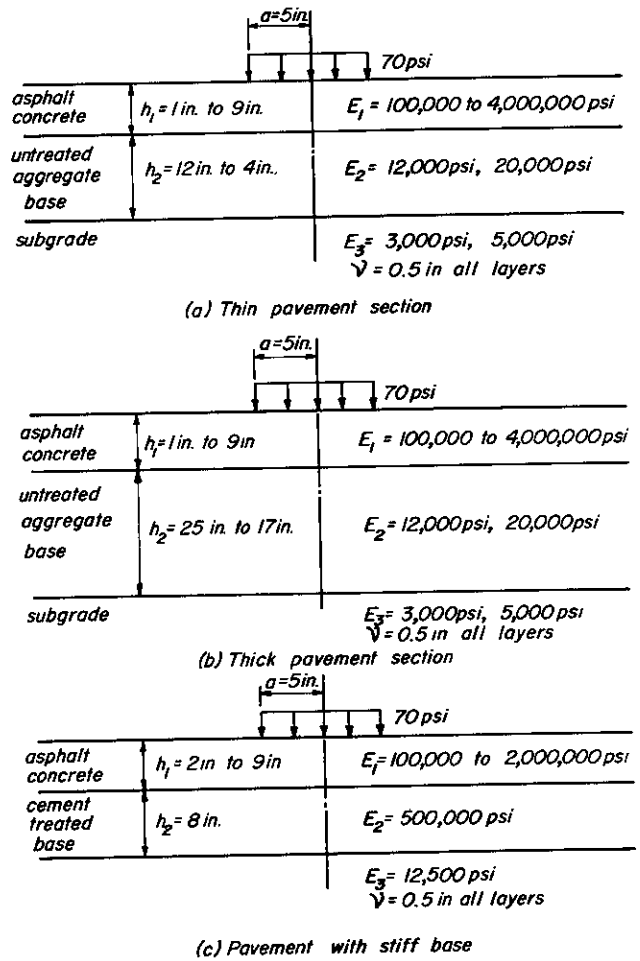

Fig. 30 — Pavement sections analyzed according to three-layer elastic theory.

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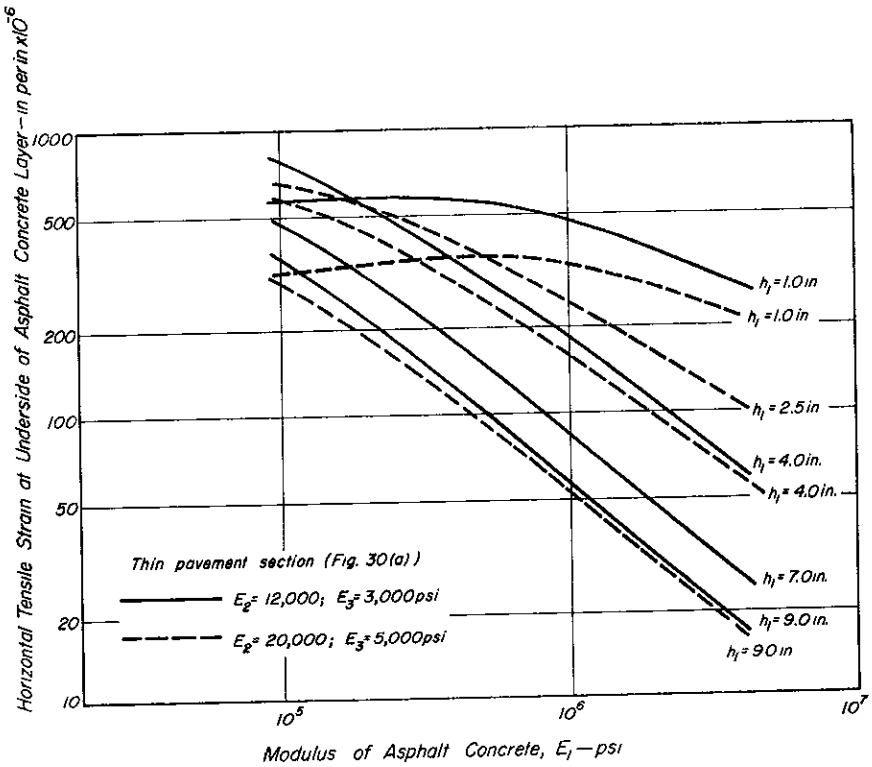

Fig. 31 — Influence of stiffness of asphalt concrete, layer thickness, and modulus of base and subgrade on tensile strain on the underside of the asphalt concrete layer (thin pavement section).

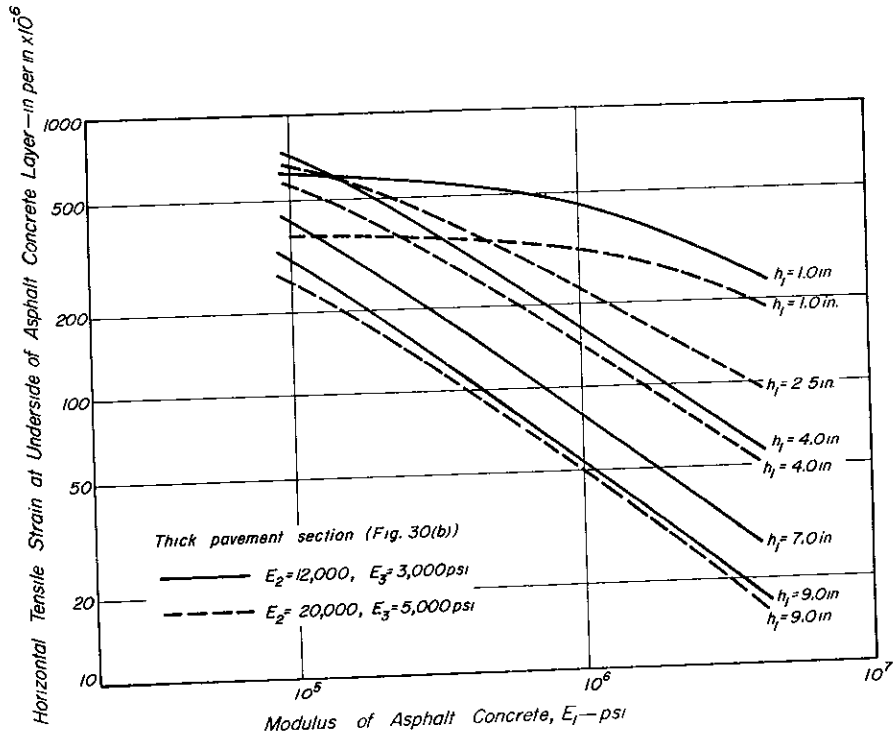

Fig. 32 — Influence of stiffness of asphalt concrete, layer thickness, and modulus of base and subgrade on tensile strain on the underside of the asphalt concrete layer (thick pavement section).

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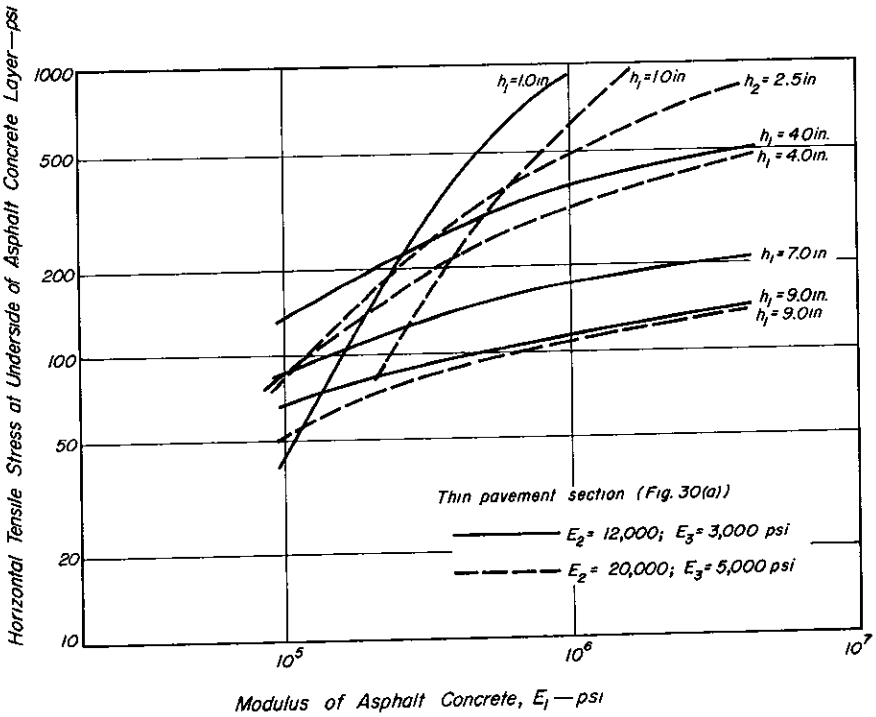

Fig. 33 — Influence of stiffness of asphalt concrete, layer thickness, and modulus of base and subgrade on tensile stress on the underside of the asphalt concrete layer (thin pavement section).

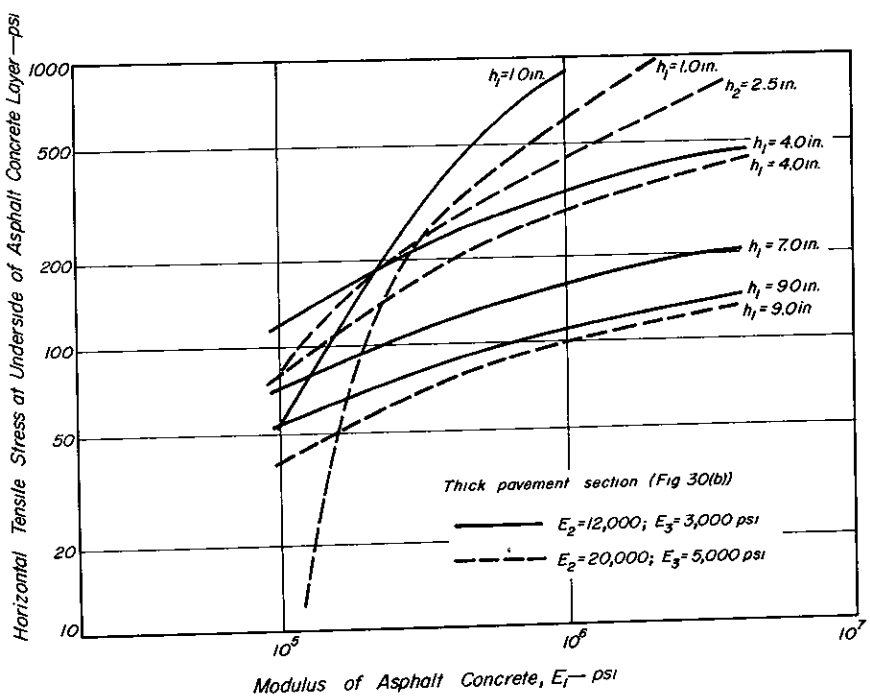

Fig. 34 — Influence of stiffness of asphalt concrete, layer thickness, and modulus of base and subgrade on tensile stress on the underside of the asphalt concrete layer (thick pavement section).

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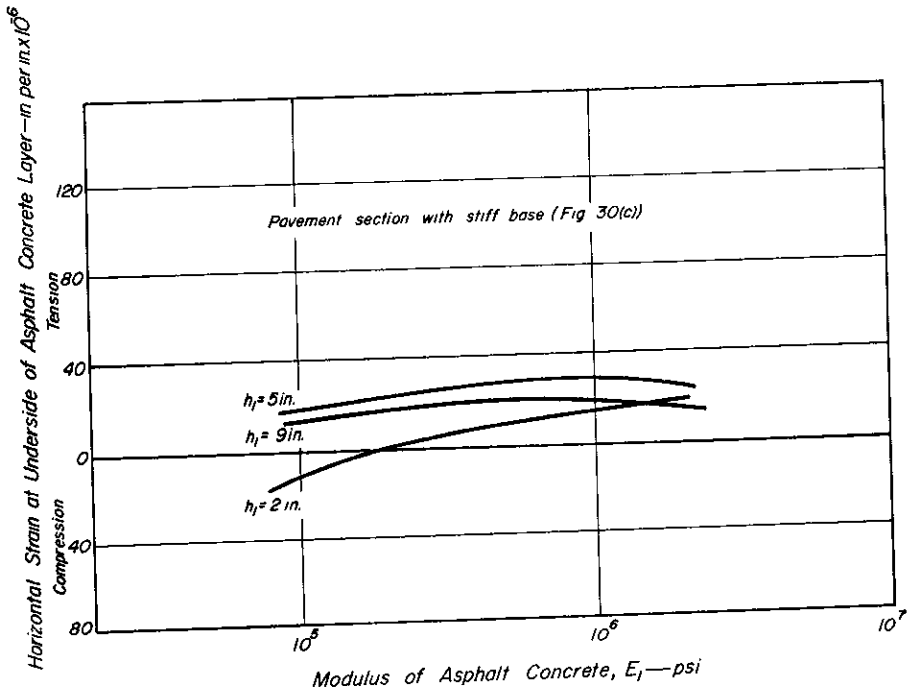

Fig. 35 — Influence of stiffness of asphalt concrete and layer thickness on tensile strain on underside of asphalt concrete layer for pavement section with stiff base.

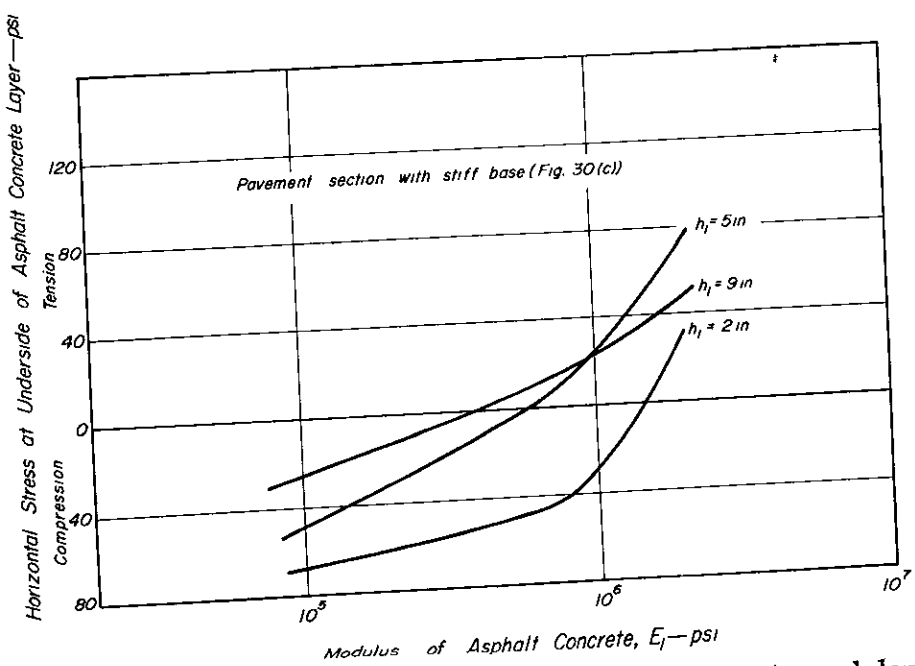

Fig. 36 — Influence of stiffness of asphalt concrete and layer thickness on tensile stress on underside of asphalt concrete layer for pavement section with stiff base.

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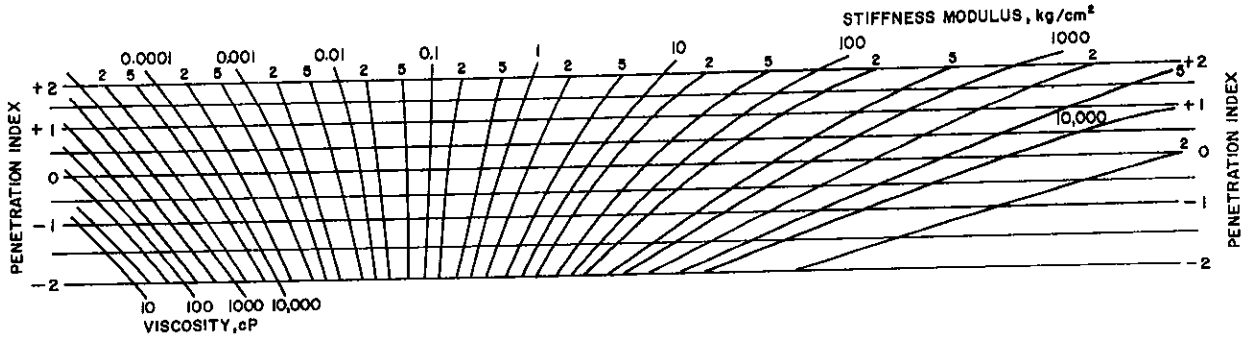

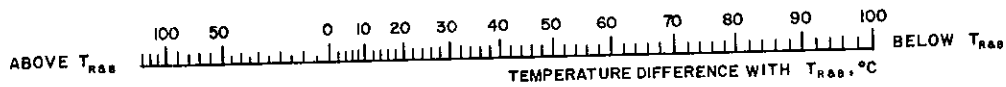

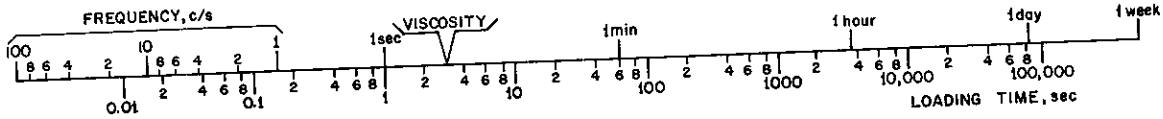

Fig. 37 — Nomograph for determining the stiffness modulus of asphalts. (After Heukelom and Klomp.)

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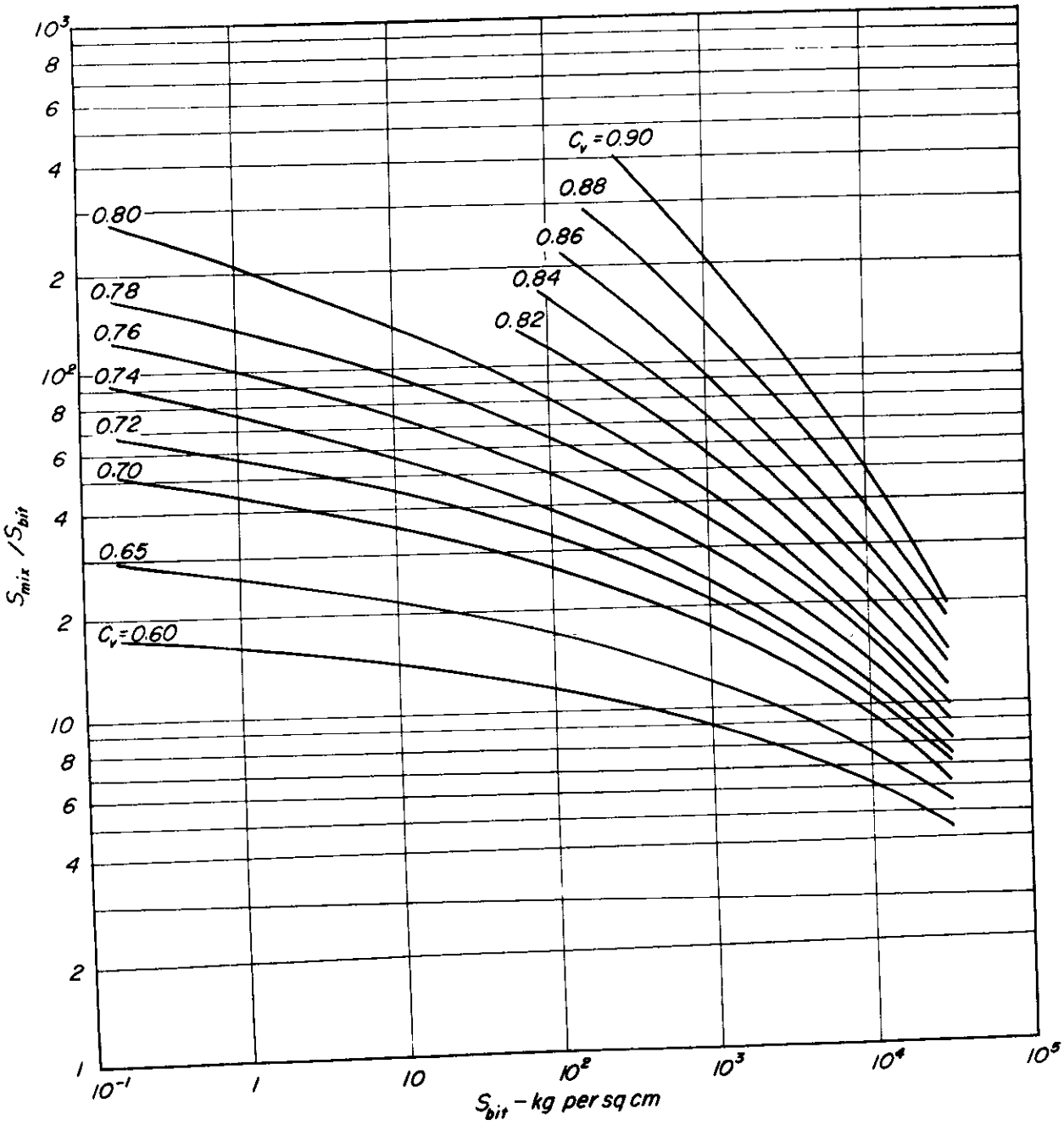

Fig. 38 — Ratio of the stiffness of the mix to the stiffness of the asphalt, S_{mix}/S_{bit}, as a function of S_{bit} and C_v. (After Heukelom and Klomp.)

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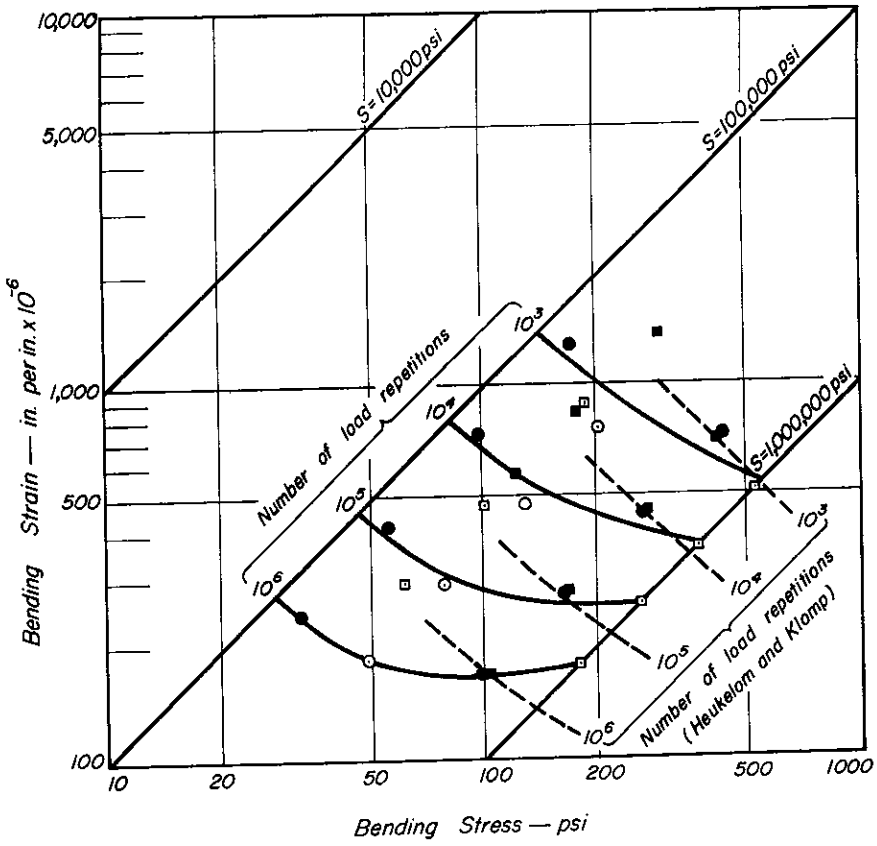

Fig. 39 — Relationship between bending strain, strain, stiffness, and number of applications to failure in constant-stress fatigue tests.

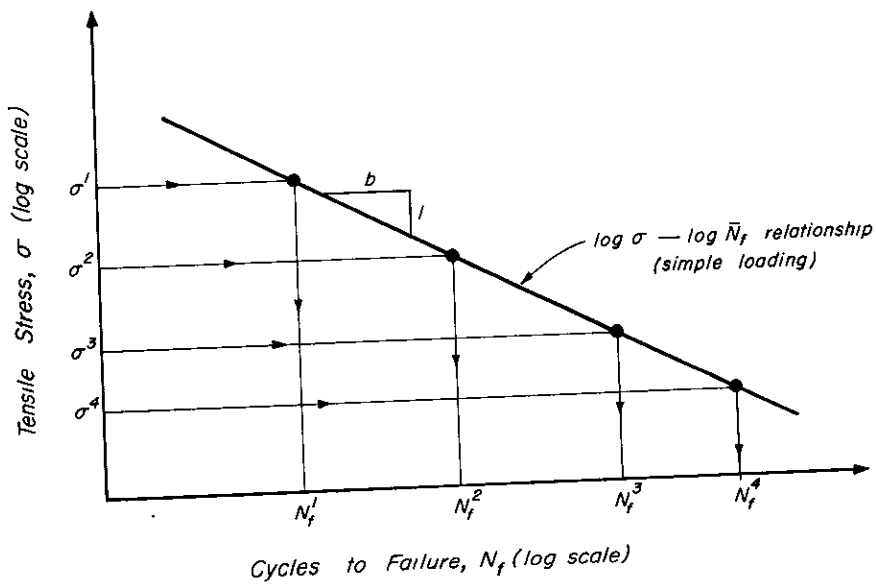

Fig. 40 — Schematic representation of data required for analysis of asphalt concrete in compound loading.

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