A Resilience Design Procedure for Flexible Pavements

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For several years the California Division of Highways has been engaged in a research project with the objectives of measuring and allowing for the resilient behavior of soils within flexible pavement systems. The ultimate objective of the study has been the incorporation of the resilience factor into pavement design for the purpose of eliminating or minimizing early "fatigue" failure of asphalt-concrete surfacing due to excessive transient deflection.

Design criteria in the form of maximum tolerable deflection were available from previous field studies. A laboratory testing device, the resiliometer, was developed to measure the resilience properties of various roadway materials. This report describes the apparatus itself, its development and the results of qualitative tests on the main types of soils encountered in roadway construction.

The incorporation of the resilience factor into pavement design required the establishment of the relationship between field deflection and laboratory resilience tests. The correlation program involved the sampling and testing of 20 different roadways. Field sampling and laboratory procedures are presented along with sample computations which illustrate the method of analysis of laboratory resilience data.

The report presents the results of a field trial of the resilience design procedure in which preliminary samples from a number of roadways were tested using resilience design criteria. Predicted deflections resulting from these preliminary resilience analyses are compared with those measured in the field following the completion of construction. The results indicate that the resilience design procedure is generally consistent and effective in isolating potential resilience problems.

•FATIGUE cracking of bituminous pavements has been recognized as a major problem with respect to flexible pavement performance. This is, of course, primarily because preliminary design procedures incorporating the factor of transient deflection have not been developed. During the past 20 years, however, several agencies have developed tolerable deflection criteria for in-place pavements. Middlebrooks (1) in 1943 stated, "Experience to date indicates that the critical deflection will vary from approximately 0.05 in. to 0.15 in. depending upon the type of subgrade, the type of base material, wheel load, and probably other factors."

The necessity of permanently installing electronic gage units for deflection measurement was eliminated during the operation of the WASHO Road Test (1953) with the development of the Benkelman beam. This device greatly simplified and speeded up the

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TABLE 1

TENTATIVE MAXIMUM PAVEMENT DEFLECTIONS

Thickness, In.	Type of Pavement	Max. Deflection for Design Purposes, In. (tentative)
8	Portland cement concrete	0.012
6	Cement-treated base (surface with bituminous pavement)	0.012
4	Asphalt concrete (plant mixed) on untreated aggregate base	0.017
3	Asphalt concrete (plant mixed) on untreated aggregate base	0.020
2	Asphalt concrete (plant mixed) on untreated aggregate base	0.025
1	Asphalt concrete (road mixed) on untreated aggregate base	0.036
½	Asphalt concrete surface treatment	0.050

measurement of transient pavement deflection. Utilizing this apparatus, approximately 60,000 individual readings were made on the WASHO test road. Analysis of these data revealed that this particular test pavement could withstand transient deflections of 0.045 in. in warm weather and 0.030 in. in cold weather (2) for a period of two years. It was emphasized in the report, however, that these values may not be applicable to older pavements or to those containing different types of asphalt or aggregate.

The results of a comprehensive statewide deflection study made by the California Division of Highways beginning in 1951 were made available in 1955 (3). The test data for this investigation were derived from readings of nearly 400 permanently installed General Electric travel gage units on 43 different projects.

Analysis of these data resulted in the safe limits (Table 1) for maximum deflection under a 15,000-lb single-axle load for several types of pavement and base construction necessary to preclude cracking after several millions of load repetitions.

These values were determined from tests on roadways with approximately 10 million equivalent 5000-lb wheel loads (EWL) and have been used as a guide criteria by the Division of Highways since 1955 for planning the reconstruction of existing roadways. Although no additional evidence has been found that would seriously invalidate these criteria, further adjustment may be warranted from experience gained from present-day construction.

In order to reevaluate these criteria in view of possible improvements in the fatigue resistance properties of current asphalt mixes and to make appropriate adjustments for variations in traffic volume, an investigation (4) is currently being made by the California Division of Highways in cooperation with the U.S. Bureau of Public Roads. The investigation will further determine the effect of varying amounts of traffic on tolerable deflection levels of various types of roadway structural sections.

As an interim method of adjustment for varying traffic volumes, a family of curves has been developed by the Materials and Research Department based on AC surfacing fatigue tests made by the department several years ago. The results of this work indicate that while fatigue life of individual AC specimens varied widely, presumably due to variation of mix design, age and number of previous traffic loadings, the slopes of their load repetition vs deflection lines were relatively uniform when plotted as logarithmic functions. By utilizing an average AC surfacing fatigue line slope and by pivoting lines through known deflection criteria at the traffic volume from which Table 1 was developed, Figure 1 was developed for the purpose of making "rule of thumb" adjustment in tolerable deflection for varying traffic volumes. Although these curves are based solely on laboratory surfacing fatigue data, they appear quite reasonable within the ranges of 6.0 to 10.0 TI. The traffic index (TI) is an exponential function of total 5000-lb EWL anticipated on the highway between the time construction is completed and the end of the design period, usually 20 years (see Test Method No. Calif. 301-B).

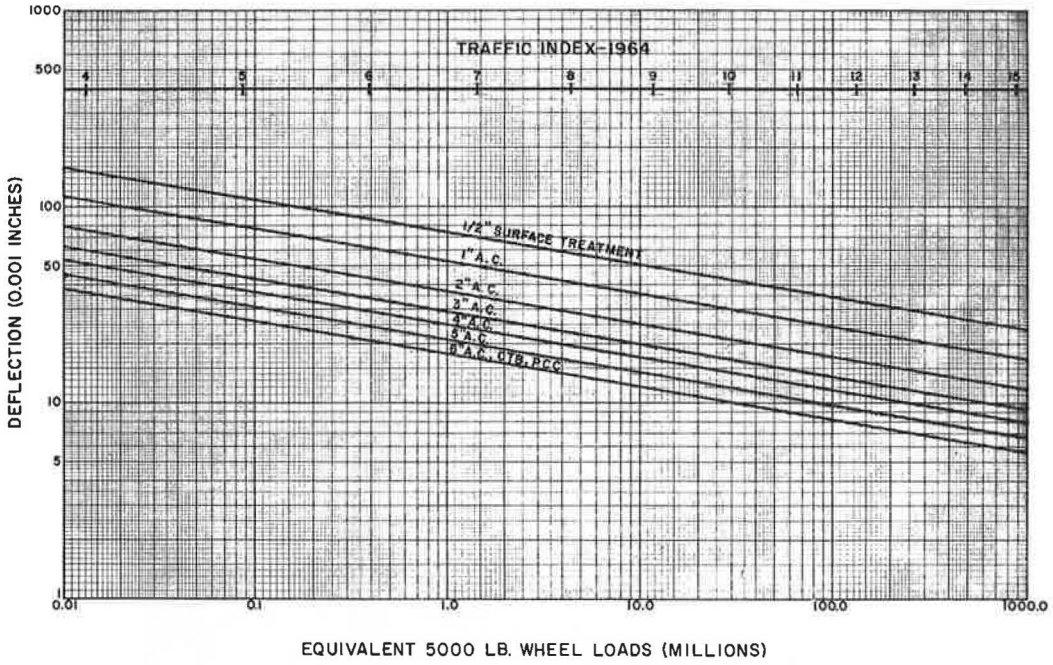


Figure 1. Variation in tolerable deflection based on AC fatigue tests.

The curves have, therefore, been utilized since 1964 in planning the reconstruction of lightly traveled roads based on deflection measurement.

Preliminary review of deflection data from the AASHO test road in Illinois indicates that the tested pavements tolerated deflections somewhat greater than those given in Table 1. However, differences in asphalt quality, design and control of the mixes, and duration of the test may have greatly influenced these values.

The results of a pavement deflection study of three years' duration in North Carolina were reported in 1960 by L. D. Hicks (5). In the course of this study, periodic deflection measurements were made over 4 projects with a Benkelman beam and a dump truck loaded to provide 7500 lb on each rear dual wheel assembly (15,000-lb axle load). This is the same arrangement as that employed by California.

Undoubtedly, the results of these and future deflection investigations, over a variety of pavement structural sections throughout the United States, will enable highway engineers to assign safe levels of deflection to pavements for a given traffic situation with reasonable certainty that the pavements will not be overly fatigued during their design life. These deflection levels will, of necessity, take into account local materials, weather, mixture design, and construction practices.

DEVELOPMENT OF THE RESILIENCE TEST

Pavement deflections in the past have been measured in situ. Therefore, the designer has had no basis from which to predict the probable deflection of a proposed structural section or to adjust the section so as to reduce an anticipated high deflection to within permissible limits. Over the past 20 years, the Materials and Research Department of the California Division of Highways has developed a testing device and procedure for the purpose of incorporating the deflection factor into pavement design by providing a definite measure of the compression and rebound of a soil specimen under dynamic loading. Since, for a given load and specimen size, this measurement is directly related to the recoverable strain energy of a deformed body when the load causing strain is removed, the instrument has been designated the resiliometer. The

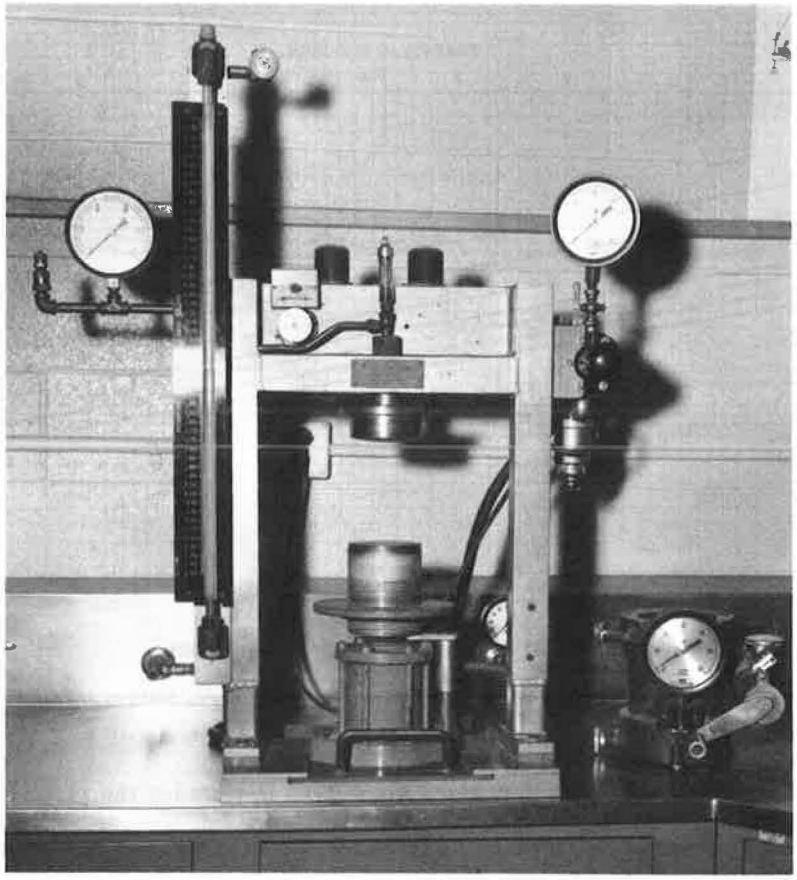


Figure 2. Resiliometer.

test will supplement existing tests because it measures a separate and distinct soil property (resilience) not measured by commonly used test methods.

The resiliometer (Fig. 2) is an apparatus which measures the volumetric displacement resulting from repetitions of a cyclic dynamic load applied to soil specimens ranging from $2\frac{1}{2}$ to 4 in. in height and 4 in. in diameter. The load is applied to the top of the specimen through a rubber diaphragm associated with a pressure system containing ethylene glycol solution, the fluid being acted on by air pressures of from 0 to 60 psi. Volumetric displacement is measured by a manometer tube. Lateral pressures are applied and measured by the stabilometer. The apparatus, test method, and procedure are further described in another report (6).

During the period from 1954 to 1959, several modifications in equipment and technique were instituted which improved the instrument's sensitivity and test reproducibility. This period was also devoted to studies of the effect on resilience of specimen height, density, gradation, moisture content, and number of load repetitions. Qualitative resilience appraisals were made on roadway materials from locations throughout California. In addition, samples from Idaho and Washington, as well as from the WASHO and AASHO test roads, were tested.

The data assembled from these tests seem to warrant certain general observations concerning the resilient behavior of soils:

1. Resilience (internal compression and rebound) increases rapidly with increasing compaction moisture content and, to a lesser extent, with increasing void ratio. Resilience also increases with increasing post-compaction moisture content.

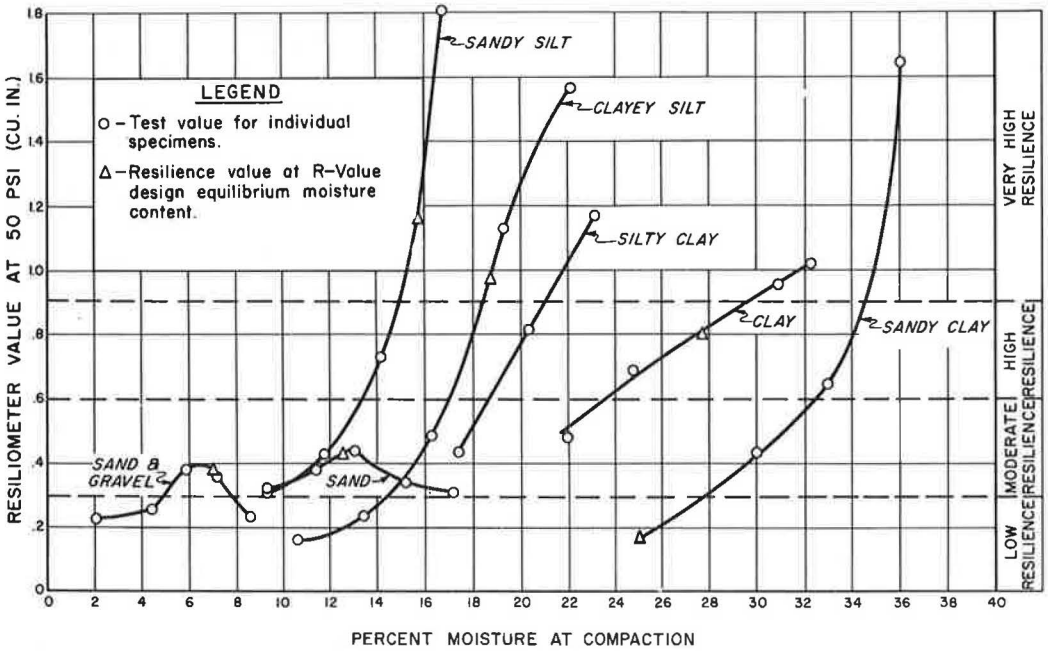


Figure 3. Typical resilience-moisture curves.

2. Although individual clay specimens have been found to be extremely resilient at elevated moisture contents, the greatest sensitivity to moisture, i.e., the largest variations in resilience for a given increase in moisture content, are consistently found in the soils classed as silts or silty types.

3. Sands and gravels are generally low in resilience. The resilience vs compaction moisture content plots shown in Figure 3 illustrate fairly typical behavior of several distinct types of soil. The general descriptive ratings on the right side of the chart are based on an evaluation of results from hundreds of individual tests.

4. As a general rule, the greatest soil "sensitivity" begins slightly on the wet side of optimum moisture content as determined by Test Method No. Calif. 301-F (7).

The accumulation of these data and the assignment of general resilience classifications proved beneficial for the qualitative appraisal of roadway materials for special projects and distress investigations. However, in order to introduce the resilience factor into the California flexible pavement structural design procedure on a rational basis, it was apparent that a relationship was required between laboratory resilience measurements and field performance as measured by pavement deflections.

RESILIENCE-DEFLECTION CORRELATION STUDY

In order to establish a relationship between pavement deflection and resilience values determined in the laboratory, a correlation study was initiated in the spring of 1959 at the Franklin Airport, southeast of Sacramento. This program consisted of measuring pavement deflections and taking undisturbed samples for resilience testing in the laboratory. Subsequent samplings were made at the California State Fair Grounds and in the Division of Highways Service and Supply Yard in Sacramento. The results of these early correlation samplings were beneficial primarily for development of a procedure for deflection measurements, sampling, and testing, specifically for the resilience-deflection correlation study. In addition, several basic changes in the method of analysis of data were made. The samplings which will be discussed in this report are from roadways

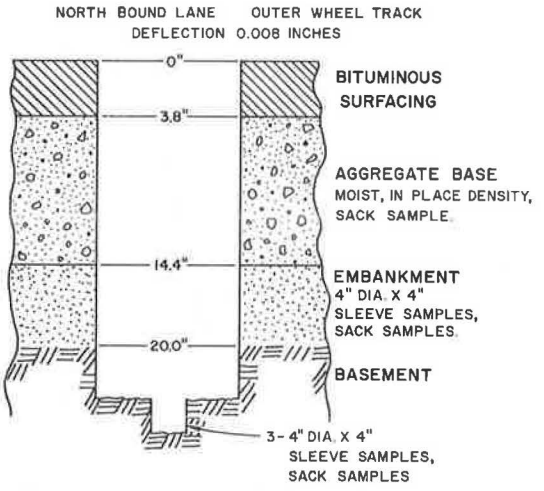


Figure 4. Structural section sampling diagram, Road 04-Mrn-Son-1.

throughout California and from I-15 in in Idaho. At present, the resilience-deflection correlation consists of 44 individual samplings from 20 different roadways.

Field Procedure

During the correlation study (1959-1963), a Benkelman beam was used to measure deflections produced by a dump truck with a rear-axle load of 15,000 lb. The load was supported by two dual wheel assemblies with 10:00 x 20 tires inflated to 70 psi. The test interval varied from 15 to 20 ft in each wheel track of a selected lane throughout a test area of 500 lineal ft. The temperature of the pavement surface was recorded at the time deflection measurement was made.

Locations from which samples were obtained were selected from within an area of relatively uniform deflection. Thus, samples representative of the general state of the roadway were obtained, thereby minimizing the effects of localized conditions. Areas with cracked surfacings were considered unsuitable for sampling since they yield abnormally high deflections due to the rocking action of individual blocks.

Sampling operations included taking moisture samples and a 40 to 50-lb disturbed sample from each different material to a depth of 30 in. from the surface. These samples were obtained from a 2 by 3-ft hole excavated to a depth of 30 to 36 in. In addition to thickness measurements, at least three undisturbed 4-in. high by 4-in. diameter samples were obtained from the basement or embankment soil and, in some cases, the subbase. Those materials from which undisturbed samples could not be taken, due to lack of cohesion, were tested for in-place density utilizing the sand-volume method (Test Method No. Calif. 216-F). A typical sampling diagram used for the sample taken on one of the roads is shown in Figure 4.

Laboratory Testing Parameters

In order to minimize the adverse effects of different loading variables, an attempt was made to reproduce in the laboratory the expected field conditions. The variables considered included lateral confining pressure, surcharge, vertical stress, distribution, effective depth of deformation, and rate of load application.

Lateral Confining Pressure—For correlation purposes, it was considered desirable to test specimens using a lateral confining pressure in the stabilometer comparable to that exerted on an in-place element of soil resulting from dynamic loading representative of the traffic using the highway. The in-place passive-active pressure state in soils covered with different pavements, however, cannot be duplicated with any known laboratory device. Therefore, a uniform lateral confining pressure of 3 psi was used for all tests. This value was selected as a result of a series of resilience tests in which several confining pressures were used with a variety of soils. The results indicated that, for the range of vertical pressures utilized, the least permanent lateral distortion occurred in the test specimens at 3 psi.

Vertical Surcharge—During the early developmental stages of the test procedure, varying vertical surcharge pressures representing different thicknesses of overburden were used. The varying surcharge pressure, however, was found to have no significant effect on the test data. It was, therefore, decided to adopt the more practical approach

of utilizing equal vertical and lateral confining pressures. The vertical pressure is composed of two parts, a constant surcharge component of 2.4 psi and an average component of 0.6 psi due to the weight of the manometer fluid. Thus, the test specimen has a uniform confining pressure of approximately 3 psi.

The vertical surcharge pressure is maintained by a spring-loaded "pop-off" valve on the air exhaust line which expels air at the end of each loading cycle until the pressure is reduced to 2.4 psi. This residual pressure is then maintained until the beginning of the next cycle.

Vertical Stress Distribution—Probably the most important single variable in the correlation of field deflection and laboratory resilience is the application of a vertical dynamic pressure with the resiliometer which corresponds to that absorbed by the soil as the result of a transient wheel loading. The more important variables which must be considered include stiffness and stratification of overlying material, wheel loading and spacing, and rate of load application.

A review of the literature on the subject of depth-vertical pressure relationships in soils must inevitably begin with the work of Boussinesq (8), who introduced in 1885 what is now a well-known mathematical expression for calculating the vertical pressure distribution pattern in a homogeneous, elastic, level and semi-infinite medium.

A great deal of productive research in recent years has been devoted to modifying the theory so that it may closely parallel experience with actual soil conditions. In 1936, a modification of the Boussinesq equation was proposed by A. E. Cummings (9). This involved the application of a concentration factor (n) as a parameter which could be adjusted to fit materials other than isotropic elastic solids. The concentration factor concept was empirical by nature and thus required verification by field data. The accelerated traffic test at Stockton Airfield (10) in 1948 was partially devoted to the comparison of recorded vertical pressures with theoretical values. These values were obtained using concentration factors ranging from 2 to 8 with varying wheel loads, temperatures, and structural sections. Examination of the resulting plots indicates that, for the range of variables included in the test, the concentration factor (n) fell generally between 2 and 4 (n = 3 for the theoretical equation). There appears to be a tendency toward larger concentration factors with heavier structural sections. The magnitude of the wheel loading, however, had no noticeable effect on the parameter.

In 1938, Westergaard (11) introduced a further modification of the Boussinesq equation, with the inclusion of Poisson's ratio. This change was based on the nonisotropic conditions found in sedimentary soils. These equations express relationships that are undoubtedly closer to conditions in sedimentary soils and are generally believed to be preferable for settlement predictions.

A comparatively recent and comprehensive physical pressure-depth investigation was conducted by the Civil Aeronautics Administration Technical Development and Evaluation Center under the direction of Raymond C. Herner (12). Herner utilized a "mechanical subgrade" with which it was possible to measure, on a plane, the vertical pressures induced by a variety of static aircraft and truck wheel loadings through asphaltic concrete surfacing and flexible bases of differing thickness and quality. Herner's study provided a great deal of useful physical data. The tests indicated wide variation in maximum vertical pressure with varying pavement and base thickness and quality, wheel load, and subgrade reaction. Although the tests were primarily concerned with aircraft-tire loadings, a number of readings were made utilizing 8.25 × 20 and 10:00 × 20 dual truck-tire loadings over a weak subgrade (modulus of subgrade reaction = 82). These data for the 7 and 8-kip loadings and 70-psi inflation pressure are shown in Figure 5 along with the theoretical Boussinesq curve utilizing the loading and configuration (twin circular discs at a uniform pressure of 70 psi with 5½ in. between inside edges) most representative of tire print of California's Benkelman beam truck. Since the beam truck wheel load is 7.5 kips and utilizes 10:00 × 20 tires, the data from the load transmission test is approximately applicable. It is interesting to note that these points are in comparatively good agreement with the theoretical curve.

In 1943, Burmister (13) introduced a rigorous mathematical development of the elastic theory for the general case of a two-layer system for the determination of

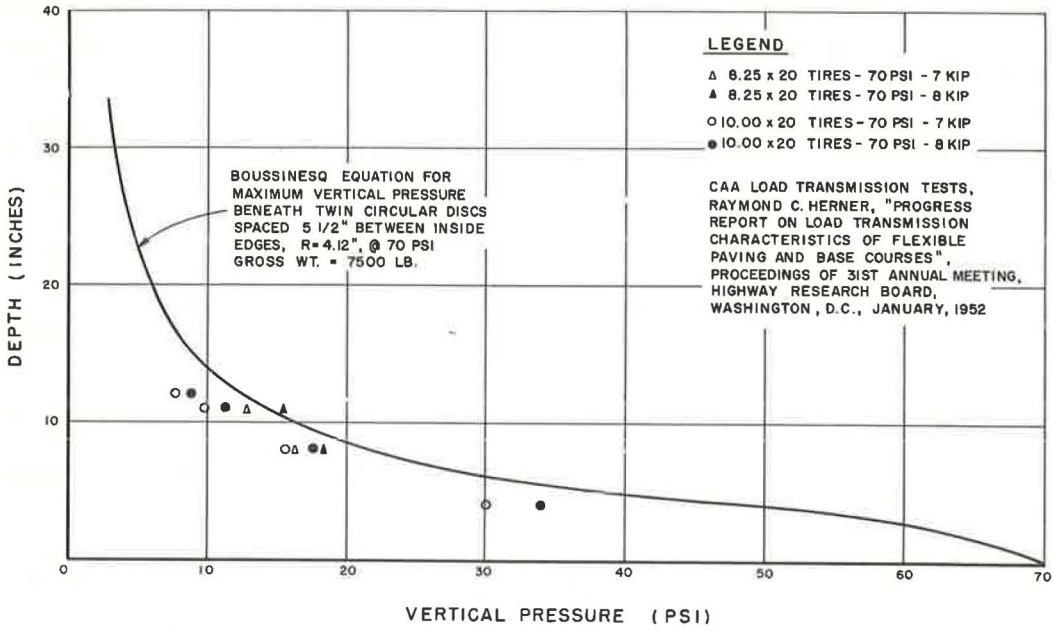


Figure 5. Vertical pressure vs depth.

stresses in layered soil deposits. Burmister enlarged his original analysis in 1945 to include a three-layer system (14).

In 1951, W. E. A. Acum and L. Fox (15) published a series of tables for the three-layer system in which stresses were numerically presented for a series of specific conditions. These computations were based on the following assumptions:

1. All materials involved behave elastically.
2. Perfect continuity (or friction) exists at each interface.
3. Poisson's ratio equals 0.5 for all elements of the structural section.

Whether these assumptions are valid for highway design will have to be determined by further physical measurements.

The evolution of influence diagrams or equations for application of the elastic theory for the more complex structural sections is continuing through efforts of Burmister and others. At the present time, however, the material available for convenient application of the elastic theory to the present day multilayered sections is still inadequate. The results of the accelerated traffic test at Stockton and the work of the CAA with the mechanical subgrade agree well enough with the theoretical Boussinesq equation for flexible pavement systems so that its use for assumed variation of pressure with depth was adopted with reasonable confidence.

Further justification for the use of the Boussinesq pressure distribution curve for flexible sections was provided by Vesic and Domaschuk (16), who, in a recent study of the deflection data from the AASHO Road Test, found the Boussinesq theory to fit the AASHO data more adequately than the layered-solid theory.

Effective Depth of Deformation—The electronic gage units referred to previously were installed throughout California from 1951 to 1955 and provided not only the data on total pavement deflection but also some idea of the amounts contributed by individual layers of strata. Examination of these data indicated that compression and rebound is developed in measurable amounts to depths of 21 ft. However, computations for flexible sections revealed that, on the average, approximately 86 percent of the deflection in the upper 8 ft of material occurred in the upper 2 ft and that 82 percent of that in the top 18 ft depth was in the top 3 ft. A typical example of this phenomenon is shown in Figure 6.

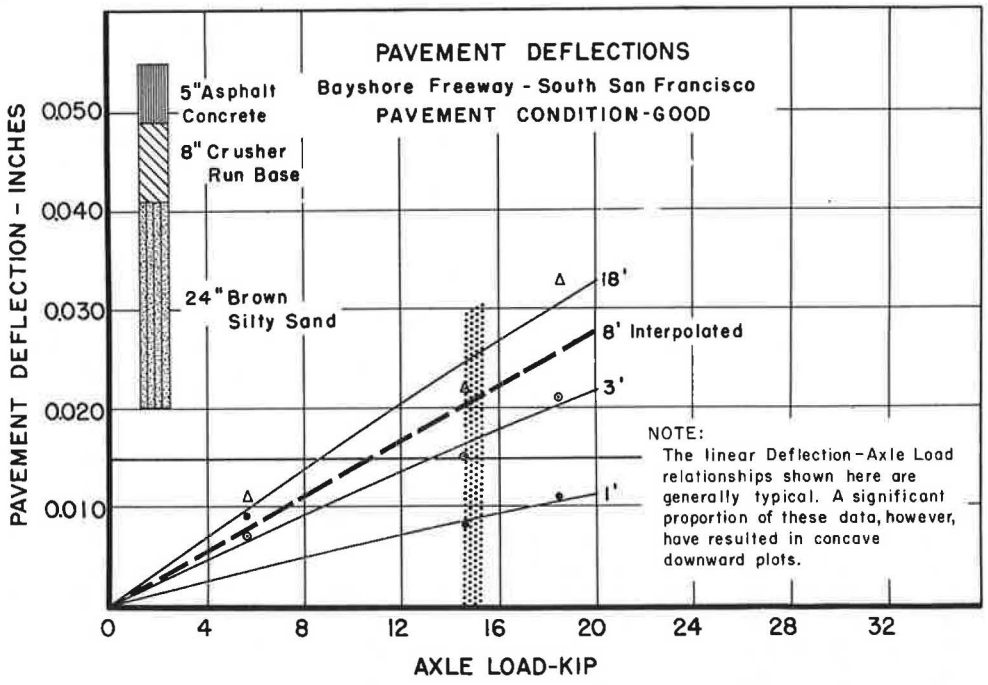


Figure 6. Typical pavement deflections.

It was apparent that either sampling or taking into account the effects of depths below 2 to 3 ft would be unrealistic. In addition, pressures occurring at depths below 2 ft are so low that experimental errors begin to mask out the significance of the resulting resilience data (Fig. 5). Accordingly, the limiting depth for sampling and consideration in computations was set at 30 in.

For purposes of computation, the test resilience value for any stratum is obtained by determining the average pressure at the depth the material exists. This can be conveniently done by using the depth-vertical pressure curve shown in Figure 5. Since this curve is not linear, calculations are made in 4-in. increments of depth. The pressure so determined is adjusted by adding 10 psi and the test resilience value is determined at the adjusted pressure reading. The addition of the 10 psi is a deviation from the theoretical curve and is utilized because it distorts the depth-pressure curve in favor of those materials appearing at lower depths and, therefore, tends to compensate for the 30-in. depth limitation.

Rate of Load Application—The rate of load application has been held to a constant 8 cycles per minute, the minimum period found necessary for full rebound of the specimen. The 7.5-sec cycle is divided so that pressure is applied to the specimen for 0.75 sec, the minimum period of time needed to secure an accurate reading of the manometer tube. A typical Benkelman beam deflection vs time plot is shown in Figure 7 superimposed on a Brush Analyzer record chart of the resiliometer test in which the dynamic load is plotted against time. The deflection trace indicates the rate of load application at a given point at the approximate operating speed of the Benkelman beam truck.

The number of load repetitions applied at each increment of pressure is dependent on the nature and state of the material being tested. The volumetric displacement is recorded only when the rebound reading is within 0.02 cu in. of the initial reading, so that the data reflect, almost entirely, resilient deformation of the specimen.

Thus, with sands and silts, readings can be taken almost immediately while clays require a large number of repetitions at each increment of pressure in order to reduce

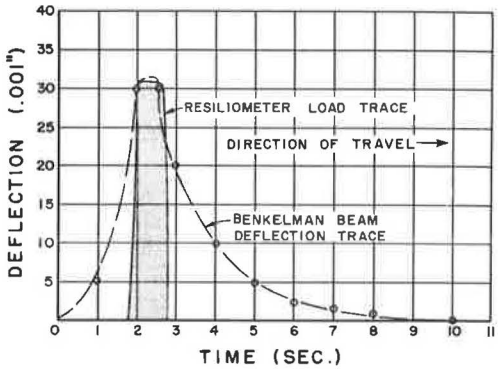


Figure 7. Deflection-resiliometer loading trace.

the plastic deformation to an acceptable minimum. Although plastic deformation is cumulative throughout the test, resilient deformation remains virtually constant for each applied pressure after the initial period of preliminary consolidation. This behavior is illustrated graphically by the compression and rebound history of an undistrubed clay specimen (Fig. 8).

Analysis of Resiliometer Data

Method of Analysis—The method of analyzing resilience test data now differs from that reported previously (6, 17). At present, the characteristic slope of the individual plotted points for each specimen is projected through the "no load" pressure (confining pressure) on the pressure-resilience plot rather than using the curve of best fit through the actual plotted points as was the previous practice. This revised procedure was adopted because samples prepared in an identical manner frequently produce widely dispersed pressure-resilience curves even though each curve has approximately the same slope (Fig. 9). The scatter in absolute resilience values is possibly due to individual specimen end effects and variations in specimen densification. These differences tend to offset the curves in varying amounts laterally. The use of a curve having the characteristic slope of all the individual samples projected through the ordinate confining pressure value appears to have eliminated this problem and has produced a better correlation between resilience summation and field deflection.

Typical Computation—The mechanics of sampling, testing, and analysis of data for the resilience-deflection correlation study can best be illustrated by a typical example, in this case, from Project 04-Mrn-Son-1 near Bodega Bay, California. A series of four deflection measurements taken on November 29, 1960, on the northbound outer wheel track from Station 0+00 to Station 8+00 were found to range from 0.004 in. to 0.016 in. Station 6+00, with a deflection of 0.008 in. was selected for sampling. The structural section and sampling pattern are shown in Figure 4. Disturbed samples and

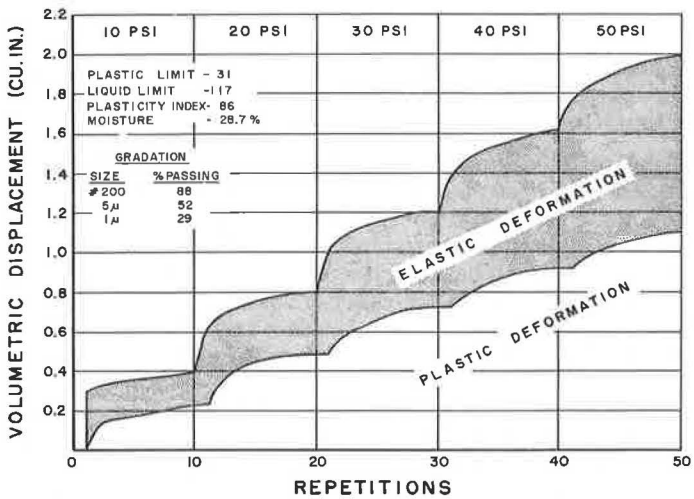


Figure 8. Compression and rebound history of undisturbed clay specimen.

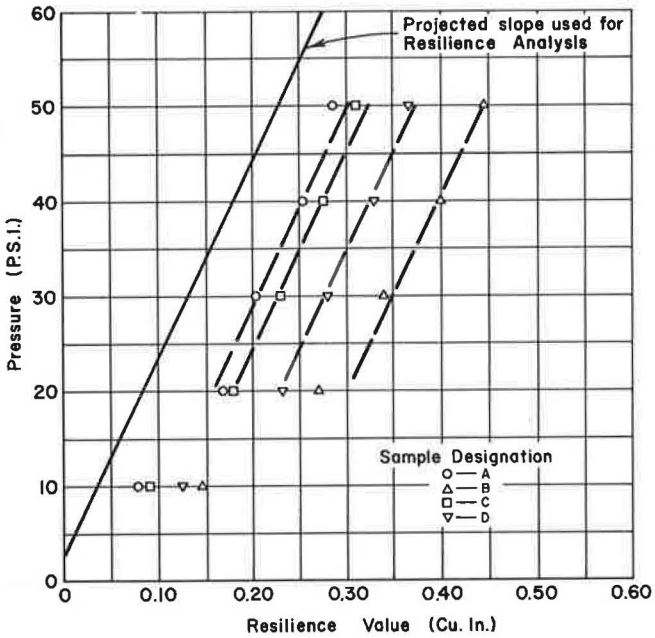


Figure 9. Resilience test results illustrating slope method of analysis, Road 10-Cal-12, 49 (aggregate base).

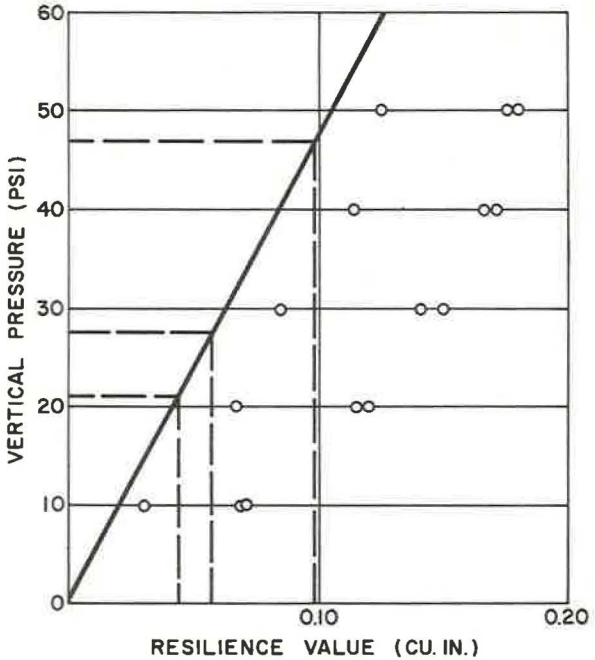
moisture samples of all elements of the structural section were taken. In-place density measurement of the base was made using Test Method No. Calif. 215-F (sand volume). Three 4-in. diameter by 4-in. high undisturbed samples were obtained from the embankment and basement soil at depths ranging from 14.4 to 32 in.

Samples of base and embankment were compacted in the laboratory at field moisture and density and tested in the resiliometer under pressures ranging from 10 to 50 psi in 10-psi increments.

Undisturbed samples which were taken by driving a brass sleeve into the embankment and basement soil were trimmed to proper length and tested in a like manner. The results are shown in Figures 10 to 12. The pressure-resilience relationship is obtained by drawing the slope of the plotted points through the confining pressure value on the ordinate scale. In establishing the representative resilience slope of basement or borrow samples, greater weight is given to data taken at lower pressures (10 to 30 psi); similarly, the resilience slope of base and subbase materials is largely determined from the resilience readings at high pressures (30 to 50 psi).

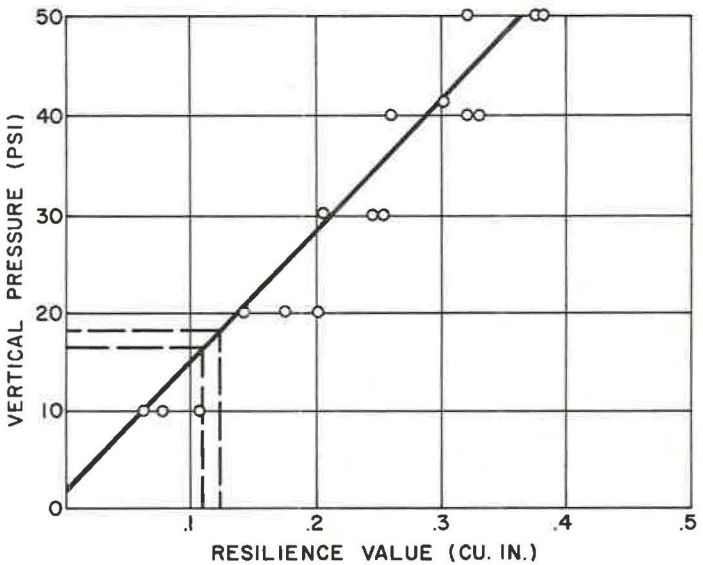
From Figure 5 it may be seen that the first 4-in. increment of depth (3.8 to 7.8 in.) has an average pressure of 37.0 psi. Adding 10 psi, the resilience at 47.0 psi is observed to be 0.099 cu in. from Figure 10. The average pressure for the second 4-in. increment of the base (7.8 to 11.8 in.) is 17.4 + 10 = 27.4 psi (Fig. 5). The resilience for the remainder of the base (11.8 to 14.4 in.) at 11.2 + 10 = 21.2 psi is 0.044 cu in.

However, since the resilience data applied only to tests on specimens 4 in. in height, a correction is made on the last increment by multiplying the resilience value by a ratio of the actual thickness of the increment to 4 in., in this case 0.044 (2.6/4.0) = 0.029 cu in. The total contribution to resilience by the layer of the base material is, therefore, 0.099 + 0.057 + 0.029 = 0.185 cu in. The vertical pressure for the first 4-in. increment of the embankment (14.4 to 18.4 in.) was found to average 8.1 psi + 10 psi = 18.1 psi. From the resilience pressure plot, for a pressure of 18.1 psi, the resilience was found to be 0.120 cu in. The resilience for the increment of embankment from 18.4 to 20.0 in. corrected for height was found to be 0.044 cu in., thus making the total embankment resilience equal to 0.164 cu in.



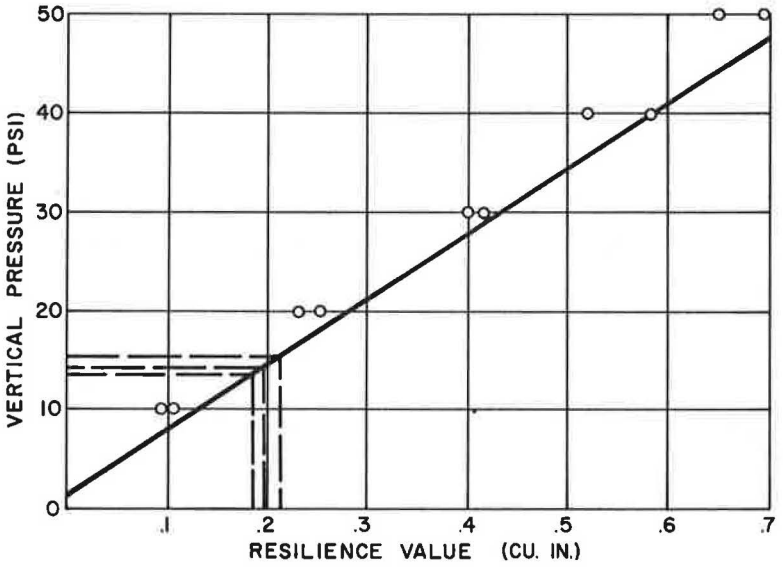
DEPTH (IN)	AV PRESSURE (PSI)	RESILIENCE VALUE (CU. IN.)
3.8 - 7.8	37 + 10	.099
7.8 - 11.8	17.4 + 10	.057
11.8 - 14.4	11.2 + 10	$.044 \times \frac{2.6}{4} = .029$
		TOTAL .185

Figure 10. Pressure-resilience plot of aggregate base, Road 04-Mrn-Son-1.



DEPTH (IN.)	AV PRESSURE (PSI)	RESILIENCE VALUE (CU. IN.)
14.4 - 18.4	8.1 + 10	0.120
18.4 - 20.0	6.5 + 10	$110 \times \frac{1.6}{4} = 0.044$
		TOTAL 0.164

Figure 11. Pressure-resilience plot of embankment, Road 04-Mrn-Son-1.



DEPTH (IN)	AV PRESSURE (PSI)	RESILIENCE VALUE (CU. IN.)
20.0-24.0	5.4+10	0.215
24.0-28.0	4.3+10	0.200
28.0-30	3.6+10	0.190 X $\frac{2.0}{4}$ = 0.095
		TOTAL 0.510

RECAPITULATION

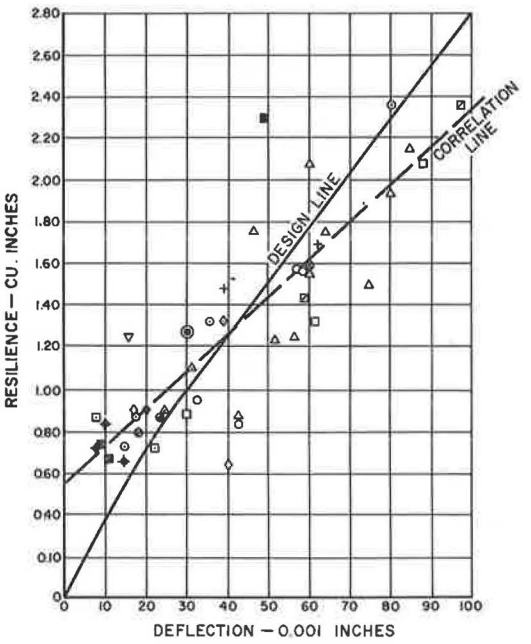
MATERIAL	TOTAL RESILIENCE VALUE
AGGREGATE BASE	0.185
EMBANKMENT	0.164
BASEMENT SOIL	0.510
GRAND TOTAL	0.859

Figure 12. Pressure-resilience plot of basement soil, Road 04-Mrn-Son-1.

This procedure was repeated for the basement or embankment soil (Fig. 12) to a depth of 30 in. The total resilience for a depth of 30 in. totaled 0.859 cu in. This was plotted against the field deflection at that point (0.008 in.) in Figure 13.

It may be noted from the foregoing example that no resilience value was computed for the AC surface. This is based on the assumption that the resilience of AC is negligible compared to that of the underlying soils.

Discussion of Correlation Plot—The results of 44 samplings from 20 projects are shown in Figure 13 with a regression line of correlation. Although there is considerable scatter, a fairly well-defined pattern emerges. These data produced a coefficient of correlation of 0.85 with a 95 percent confidence bank of 0.74 to 0.92. It is interesting to note that samples from the same project usually check each other, i.e., the lower deflections result in the lower summations of resilience. The average deflections

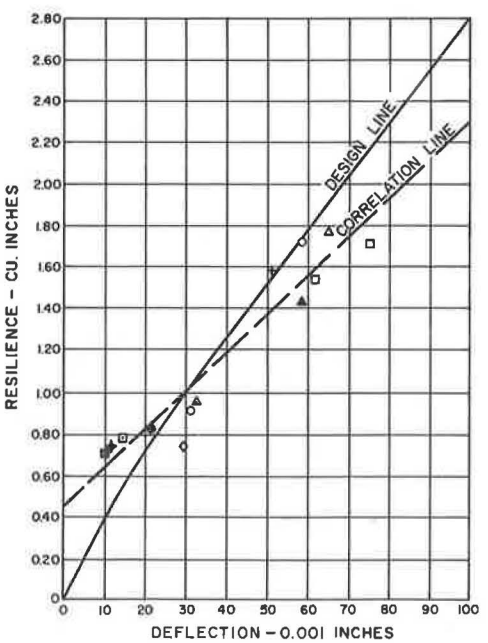


LEGEND	
ROAD LOCATION	
⊡ 04-Mrn,Son-1	⊠ City of Woodland
○ 03-But-32	■ City of Woodland
△ 03-Sac-99	▽ 02-Teh-FAS-1078
□ City of Hawthorne	◉ 03-Col-45
⊙ 05-SBt-156	▲ 03-Sut-99
⧋ 10-Mer-FAS-914	+ 03-Yol-16
◈ 06-Sac-16	◆ 04-Son-FAS-780
◇ 04-SCr-FAS-1270	⊕ 04-Son-FAS-787
● Striplin Road (County)	⊛ 05-SBt-156
✦ INTERSTATE 15-IDAHO	⊠ 06-Tul-FAS-1143

COEFFICIENT OF CORRELATION = 0.85
STD. ERROR OF ESTIMATE = 0.269 CU. IN.

Figure 13. Resilience summary vs field deflection, 44 individual samples from 20 road locations.

and resilience summations of individual projects are shown in Figure 14. Averaging results from the same projects increased the coefficient of correlation to 0.95 with a 95 percent confidence band of 0.82 to 0.98. Considering the variables which are not controlled in the study, the trend toward correlation is gratifying. The following factors are believed to be responsible for the scatter in results:



LEGEND
ROAD LOCATION
⊡ 04-Mrn-Son-1
○ 03-But-32
△ 03-Sac-99
□ City of Hawthorne
⊙ 05-SBt-156
⧋ 10-Mer-FAS-914
◇ 04-SCr-FAS-1270
✦ INTERSTATE 15-IDAHO
⊠ City of Woodland
▲ 03-Sut-99
+ 03-Yol-16
● 05-SBt-156
⊠ 06-Tul-FAS-1143

COEFFICIENT OF CORRELATION = 0.95
STD. ERROR OF ESTIMATE = 0.127 CU. IN.

Figure 14. Resilience summary vs field deflection, averages of samples from 13 road locations.

1. Variations in density and moisture content in the materials as they exist in the field;
2. Deviation from the assumed depth-pressure distribution due primarily to varying states of hardness of the asphalt surfacing and, to a lesser extent, its temperature; and
3. The inability to reproduce in the testing apparatus the in-place lateral active-passive pressure state of the surrounding soil.

APPLICATION OF LABORATORY RESILIENCE-FIELD DEFLECTION CORRELATION TO PRELIMINARY PAVEMENT DESIGN

The primary purpose of the correlation experiment described was to determine whether a relationship existed between the laboratory resilience measurements and pavement deflection as measured on the roadway. In order to prove the existence of such a relationship, every effort was made to reproduce, under laboratory conditions, those which existed in place when the deflection measurement was made. Thus, wherever possible, soil specimens were subjected to resilience tests in an undisturbed state. For coarse-grain materials, individual specimens were fabricated and tested at field moisture content and density. Having produced what is believed to be a satisfactory relationship between laboratory resilience and field deflection, the final phase of the investigation was begun, i.e., the application of the correlation to a preliminary design situation. Here, of course, it was necessary to assign design moisture and density criteria in the fabrication of specimens from preliminary samples and to take into consideration possible thixotropic effects which will have existed in the original undisturbed samples. In subsequent sections, the evolution of resilience design criteria for preparation of preliminary specimens will be described.

Resilience Design Moisture Content

Of primary concern in the adoption of the resilience-field deflection correlation to preliminary design was the selection of design moisture content. Because it has long been observed that moisture content is the prime variable with respect to resilience

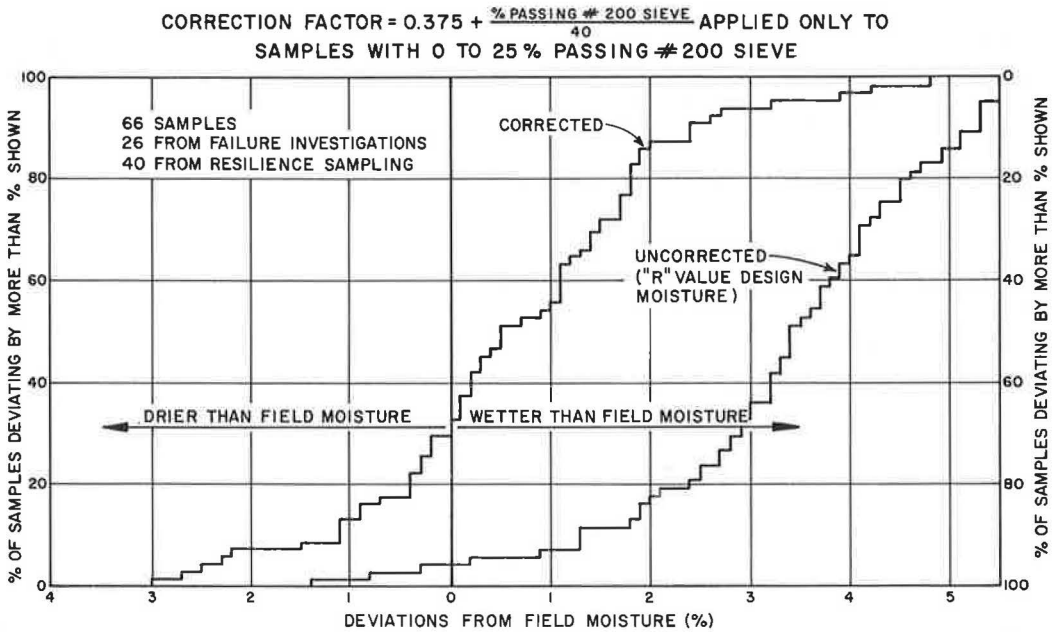


Figure 15. Ogive curve showing deviation of "R" value design moisture from field moisture.

even in coarse-grained materials, it was decided to compare in-place field moisture data from various distress investigations and resilience samplings with the current 300-psi exudation pressure R-value design moisture criteria. Exudation pressure is the moisture condition at which a soil specimen will be saturated under a static load of 300 psi. This represents the worst condition the roadway is expected to attain in its design life (12).

The results of this investigation for coarse materials (0 to 25% passing the No. 200 sieve) are shown in Figure 15. As shown by the ogive curve, approximately 95 percent of the samples were wetter at R-value design moisture criteria than at field moisture content.

A similar plot for fine-grained materials (25% plus passing the No. 200 sieve) revealed that only 70 percent of the samples by the R-value moisture criteria were wetter than field moisture (Fig. 16). Although excess moisture has relatively little effect on the results of stability tests on coarse-grained materials, deviations between field and laboratory moisture content can induce large increases in resilience. It was, therefore, decided to adjust the R-value criteria (300-psi exudation pressure) for coarse-grained material by a factor sufficient to bring it into line with that for fine-grained materials. The following corrective equation is simply a straight-line function which accomplishes this adjustment:

$$C_f = 0.375 + \frac{x}{40}$$

where

C_f = a correction factor to be multiplied by the 300-psi exudation pressure moisture content for soils with 0 to 25 percent passing the No. 200 sieve; and

x = percent passing the No. 200 sieve.

As shown by Figure 15, the corrected values for the coarse-grained soils result in an ogive curve which is quite similar to the uncorrected curve for fine-grained soils (Fig. 16), which, in effect, brings the corrected coarse-grained design moisture con-

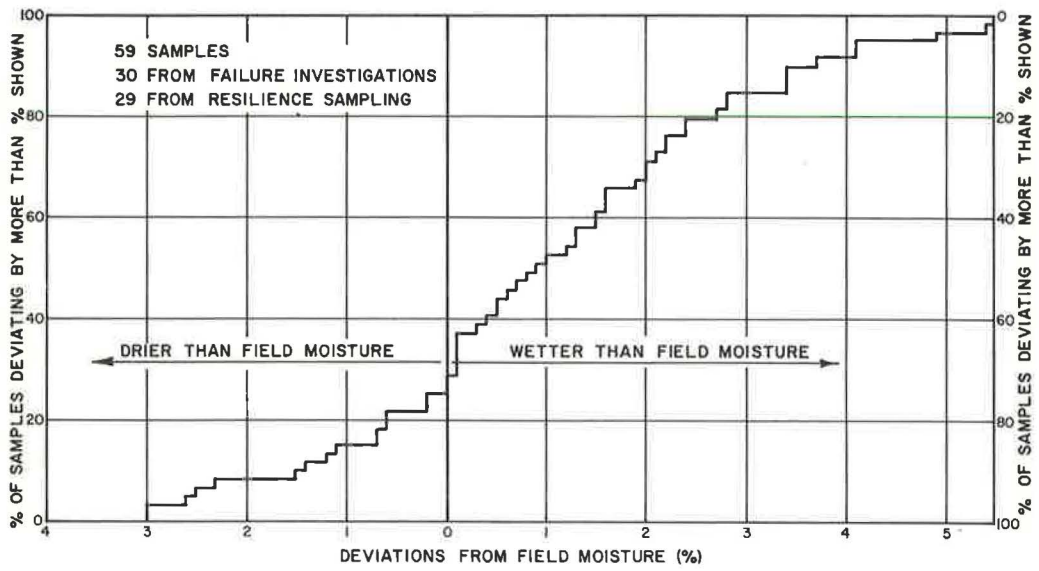


Figure 16. Ogive curve showing deviation of "R" value design moisture from field moisture, for materials having greater than 25 percent passing No. 200 sieve.

tents to a state of accuracy commensurate with that being used at present in R-value design procedure for fine-grained materials. A correction of this nature has not been necessary for the R-value design procedure since the R-value of coarse-grained soils is not generally sensitive to moisture variation within the range under consideration. This correction, therefore, is applied for all resilience design testing.

Effects of Thixotropy

Preliminary data on the effect of thixotropy with respect to clay soils were presented in 1962 by Hveem et al (17). It was found that freshly remolded AASHO embankment soils resulted in resilience values up to three times as great as those for the undisturbed samples of this material at the same moisture and density. It was decided therefore, to investigate various "sensitive" California clays in order to compare remolded with in-place resilience characteristics and possibly to determine a suitable curing period for remolded clay soils to allow for a thixotropic regain of strength which would be comparable to that eventually attained in the finished roadway. This problem was not as serious as originally envisioned since only 3 of the 16 soils tested established a remolding loss and thixotropic regain of strength to a significant degree. The test data for the most sensitive material (Road 03-Sut-99) will be discussed further.

Approximately 200 lb of clay basement soil and five undisturbed samples from Project 03-Sut-99 (El Centro Road) were obtained. The undisturbed samples were tested for resilience upon receipt by the laboratory. Subsequently, a number of specimens were compacted in sets of four each at the average moisture and density of the undisturbed samples. The first set of specimens was tested immediately after compaction and succeeding sets were tested after moist-curing 1 day, 6 days, 14 days, 57 days and 76 days. Moisture loss during the curing was prevented by placing these specimens, while still in the mold, in metal cans, sealing them in plastic bags, and curing in a 100 percent humidity cabinet.

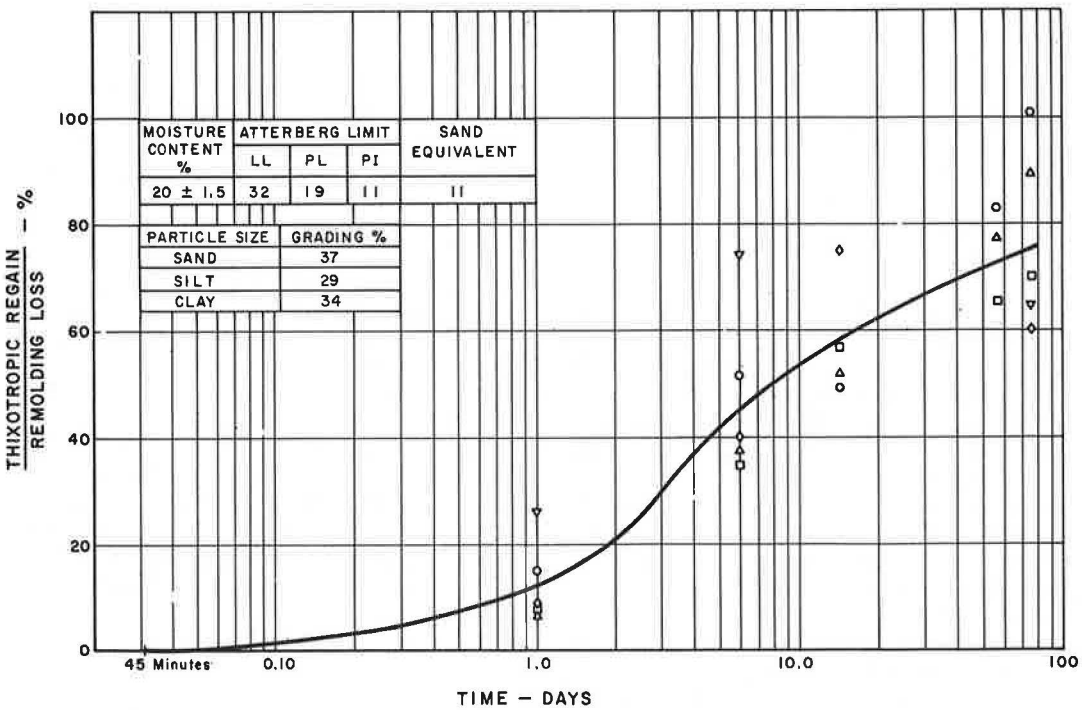


Figure 17. Thixotropic regain of a sandy silty clay from Project 03-Sut-232-A.

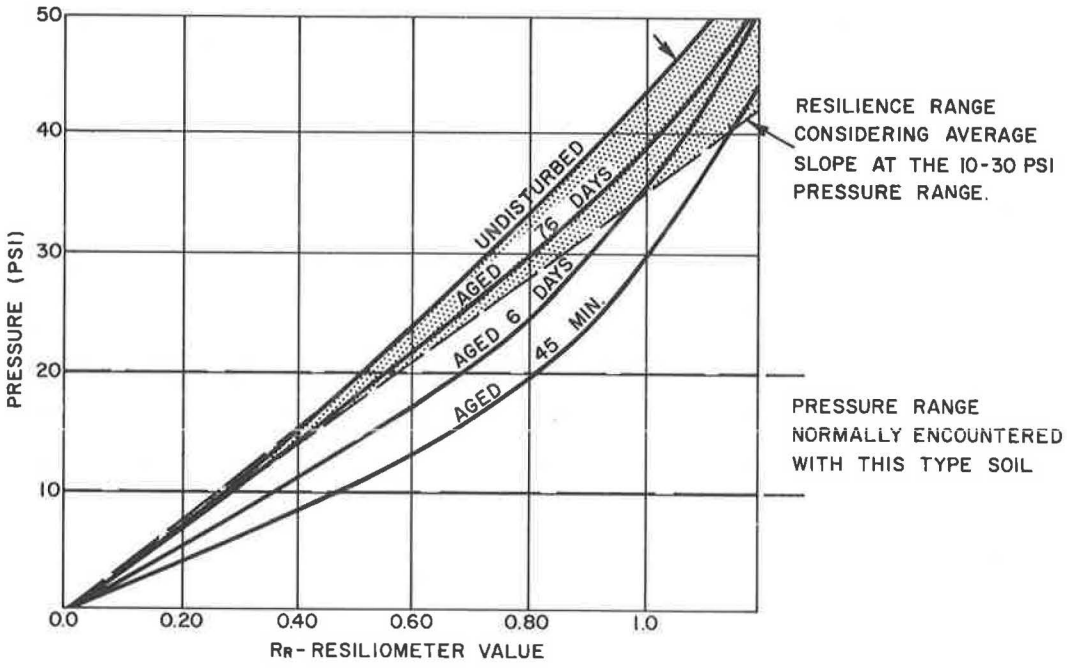


Figure 18. Change in resilience due to thixotropy of a sandy silty clay from Project 03-Sut-99.

Plots of thixotropic strength regain vs curing time are shown in Figures 17 and 18. The thixotropic strength regain was calculated as a ratio of the decrease in resiliometer value at a given curing time to the increase in resiliometer value immediately after compaction.

Curing 7 days resulted in recovery of half the strength lost by disturbing the material (Fig. 17). After 7 more days of curing, approximately 10 percent additional strength was regained. In the next 7 days, only 4 percent additional strength was regained.

For this material, a point of diminishing return is reached after approximately 15 days of curing. Based on the results of this study, a 2-week curing period was initially established as standard procedure for preliminary resilience samples containing a significant clay percentage.

With the adoption of the present procedure for the analysis of resilience data, i.e., the utilization of the slopes of plotted values rather than absolute resilience measurements, a 2-week curing period did not appear to be justified by the relatively slight increase in test accuracy. As shown in Figure 18, the shaded zone represents the range in resilience, considering average slopes at the 10- to 30-psi pressure range. Between the 10- and 20-psi pressure increments which represent the usual range under consideration for basement soil, the shaded band indicates a deviation range from the undisturbed resilience curve of only approximately one-fourth of that shown by the various solid curves representing individual plotted values. In this particular case, we note a deviation of 0.02 to 0.07 cu in. in resilience which would represent a small portion of the resilience summation for an entire structural section. It should also be emphasized that the material represented by Figure 18 is an extreme case and is, therefore, not representative of most clayey soils.

Although the potential problem of thixotropy has not been entirely discounted, the use of an extended curing period for fine-grain soils has been, for the present, discontinued. Unless further study reveals the need for a longer curing period, or a modification in the method of analysis of tests of clayey soils, a one-day cure between the time of compaction and the time of testing will be used.

Application to Design

The relationship obtained from the correlation study between field deflection and laboratory resilience determination for various pavement systems provides a basis for the design of flexible pavement structural sections. An example of the resilience analysis applied to a design situation is included herein utilizing the resilience-deflection design curve (Fig. 13). The design curve provides a factor of safety to the resulting design and, therefore, is used instead of the resilience-deflection correlation line.

The basis of this analysis is the determination of a resilience summation for the proposed structural section to a depth of 30 in. with which it is possible to predict a deflection from the design curve. When a predicted deflection exceeds the tentative criteria given in Table 1 and shown in Figure 1, adjustment of the structural section is required to reduce the summation of resilience and thus bring the predicted field deflection within tolerable limits. Examination of resilience data will indicate which element of the structural section is critical or may be adjusted most economically. Alternative solutions for excessive resilience would include:

1. Reduction of surfacing thickness, thereby increasing the allowable deflection;
2. Utilizing a composite, i.e., semirigid, structural section (for example, AC surfacing over a cement-treated base) to reduce deflection by increasing stiffness; or
3. Increasing the thickness of subbase, base or surface layers.

Design Example No. 1

The design application just described is illustrated by the following example. A roadway pavement design is proposed with a structural section consisting of 3 in. of AC surfacing, 8 in. of aggregate base and 10 in. of aggregate subbase for a county road with a traffic index of 8.0. Resilience test results on preliminary samples compacted at resiliometer design moisture content are shown in Figure 19. The test values used in the illustration are considered generally representative for these materials.

The calculation of the summation of individual layer resilience is also shown in Figure 19. The average vertical pressure in the upper 4 in. of the base layer (3 to 7 in.) is found from the Boussinesq equation (Fig. 5) to be 41.9 psi plus 10 = 51.9 psi. The resilience from this curve at 51.9 psi equals 0.210 cu in. Similarly, for the next 4-in. increment of the base layer a resilience of 0.115 cu in. is obtained. The resilience contribution for the base layer will, therefore, equal 0.325 cu in.

Similar computations for subbase and basement soil (to a depth of 30 in.) result in the resilience increments shown in the tabular material in Figure 19 for a total of 0.970 cu in. for the proposed section. From the design line in Figure 13, the predicted equivalent transient deflection of this section equals 0.029 in. Tentative criteria indicate that the 3-in. AC pavement will not tolerate deflections in excess of 0.023 in. for a TI of 8.0 (Table 1 and Fig. 1). Therefore, a design adjustment is required.

For a second trial (Fig. 20), the thickness of surfacing was increased to 4 in. which decreased the allowable deflection to 0.020 in. In addition, the thickness of the least resilient material, aggregate base, was increased from 8 to 10 in. and the aggregate subbase was increased from 10 to 12 in.

As shown by Figure 20, this manipulation reduced the resilience summation to 0.740 cu in. with a new predicted deflection of 0.020 in., just equal to the tolerable limit. The redesign, therefore, meets the criteria for both stability and resilience. An alternate solution would involve the utilization of a cement-treated base (CTB) with a 3-in. AC surfacing.

Studies on the deflection damping characteristics of various roadway materials indicate that reductions of from 0.002 in. to 0.0035 in. of deflection per inch of thickness are possible with a CTB over and above that resulting from an equivalent layer of gravel base. Thus, utilizing the original design with a 6-in. CTB instead of the gravel base, the predicted deflection of this roadway could reasonably be expected to be reduced from 0.029 in. to 0.014 in. or less than the tolerable limit for a CTB section at a TI of 8.0.

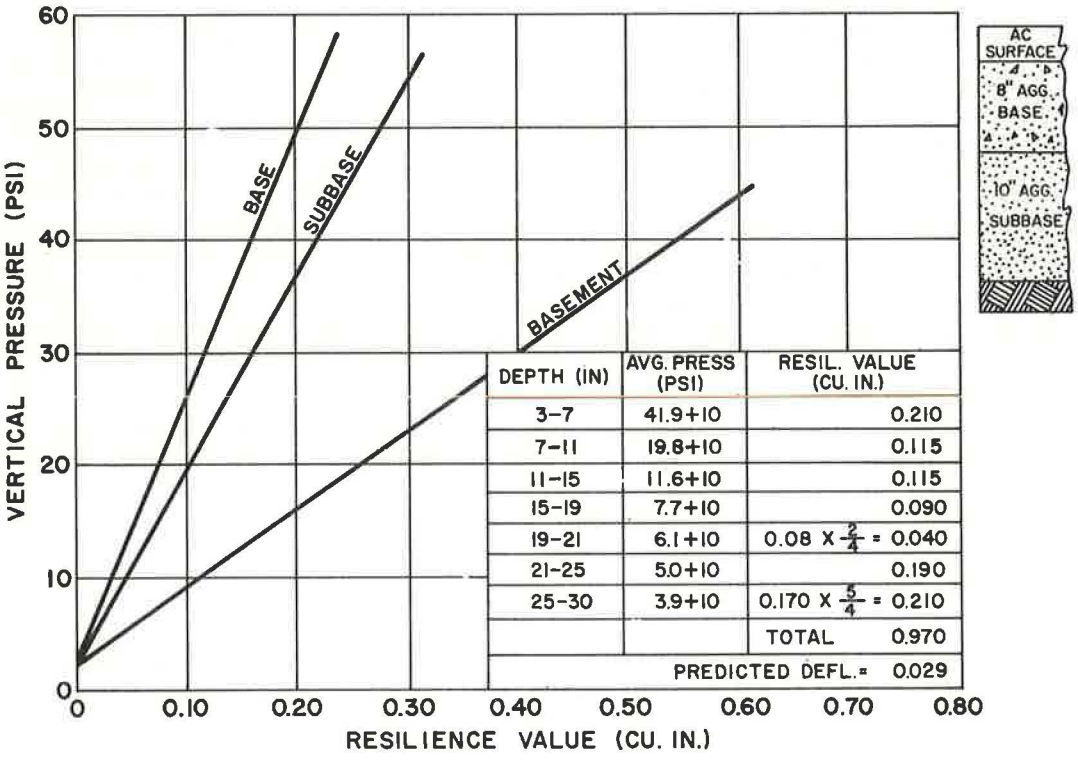


Figure 19. Design trial No. 1.

The decision as to whether the 4-in. AC surfacing and gravel base or 3-in. AC surfacing with cement-treated base should be used would depend on construction and economic considerations.

Design Example No. 2

As a second example of the application of the resilience design method, a proposed structural section consisting of 2 in. of AC surfacing and 6 in. of aggregate base for a municipal street with a TI of 6.5 will be checked for compliance with the deflection criteria of Table 1 and Figure 1.

The first step, as in design example No. 1, is to compute the resilience summation for a depth of 30 in. The base and basement soil are treated separately and divided into 4-in. increments (or portions thereof) in order to determine from Figure 5 the vertical pressures at various depths below the surface. These average pressures are shown with the constant 10 psi added to each in Table 2. For purposes of this example, it will be assumed that Figures 10 and 12 represent the results of laboratory resilience tests on the aggregate base and basement soil, respectively. From Figure 10, therefore, the resilience of the upper 4 in. of base is found to be 0.120 cu in. The resilience of the remaining increments of base are found to be 0.076 and 0.061 cu in. after the necessary height correction.

Similarly, the resilience for each increment of basement soil is determined from Figure 12. The total resilience to a depth of 30 in. is found to be 1.15 cu in. (Table 2).

A deflection for the proposed structural section can now be predicted from the design line of Figure 13. The predicted deflection for a resilience of 1.085 cu in. is approximately 0.034 in. This predicted deflection must now be checked against the maxi-

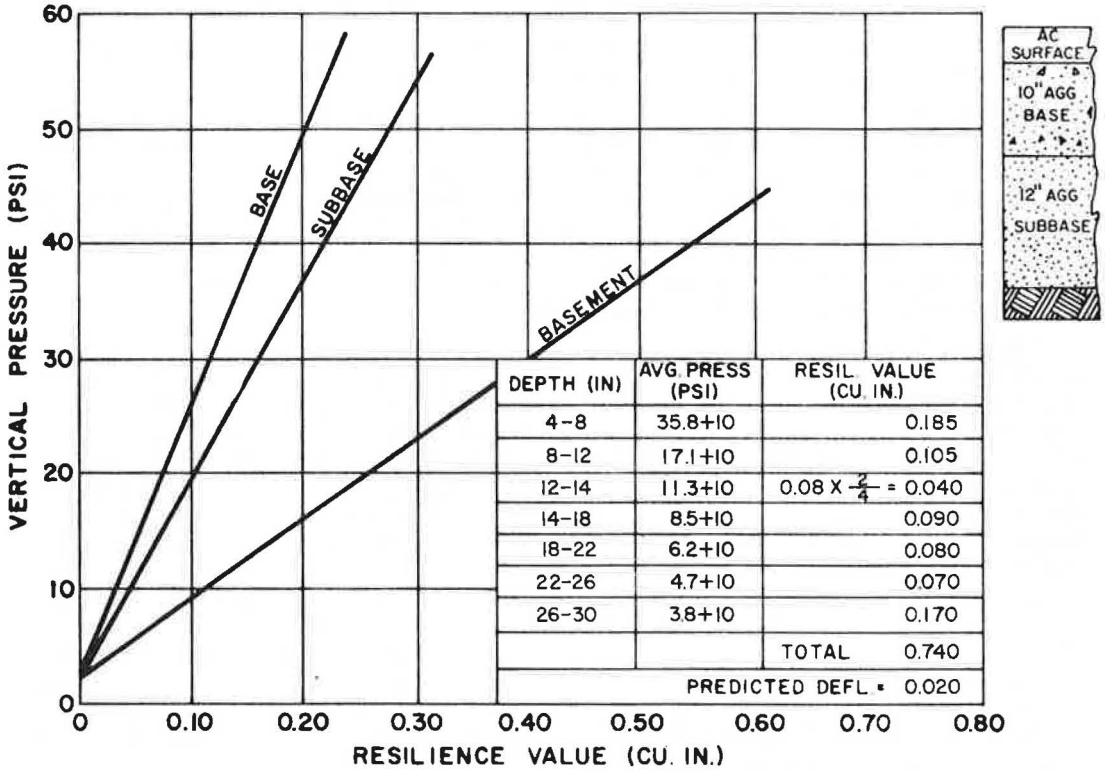


Figure 20. Design trial No. 2.

mum allowable deflection for a 2-in. AC surfacing for a roadway with a TI of 6.5. From Figure 1, the maximum allowable deflection for the stated conditions is found to be approximately 0.038 in. Since the predicted deflection is less than the maximum allowable deflection, the proposed structural section meets the resilience criteria.

TABLE 2

INCREMENTAL RESILIENCE VALUES FOR DESIGN EXAMPLE NO. 2

Depth (in.)	Material	Average Pressure (psi)	Resilience (cu. in.)
2.0- 6.0	Base	47.0 + 10	0.120
6.0-10.0	Base	23.5 + 10	0.070
10.0-14.4	Base	12.8 + 10	0.050 × 4.4/4 = 0.055
14.4-18.4	Basement	8.2 + 10	0.250
18.4-22.4	Basement	6.0 + 10	0.215
22.4-26.4	Basement	4.7 + 10	0.200
26.4-30.0	Basement	3.8 + 10	0.200 × 3.6/4 = 0.175
			Total = 1.085

FIELD TRIAL OF THE RESILIENCE DESIGN METHOD

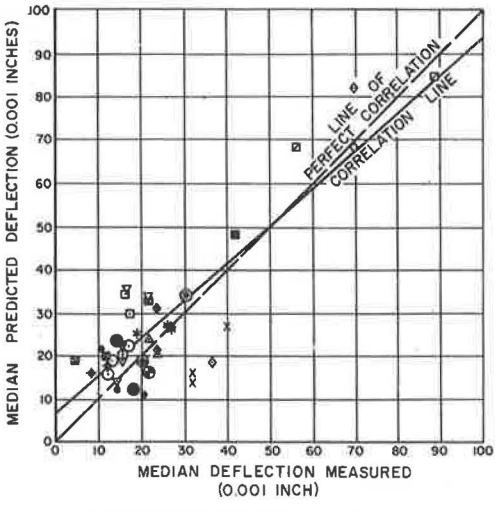
The development of the resilience design procedure was accomplished in four distinct phases. Three of these have been described. They are:

1. Qualitative resilience tests on a variety of soil types for the purpose of evolving a reproducible and practical testing device and procedure for the determination of the general resilience characteristics of various soil types;
2. The establishment of a satisfactory correlation between measured field deflection and laboratory resilience tests on samples of roadway structural sections which were either in an undisturbed state or tested at field moisture and density; and
3. The establishment of suitable test criteria for the extension of the resilience design procedure to preliminary design situations.

The fourth and final phase involved field trial of the resilience design procedure under conditions which would prevail in a preliminary design situation. For this phase, soil samples were tested utilizing the newly established design criteria rather than in the undisturbed state or at field moisture or density. To determine the confidence with which field deflection could be predicted, 40 test locations from 21 different projects were analyzed. In several instances, excess material from the correlation phase of the study was retested utilizing the design criteria. For the most part, however, embankment or basement soils were recovered from construction projects along with pit samples of aggregate base and subbase.

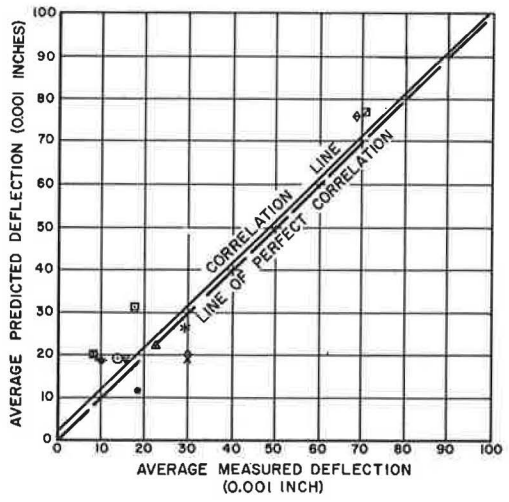
Deflection measurements were then made during the first spring season after the completion of construction for the purpose of comparing predicted vs actual deflection. These results are presented in Figure 21, which is a plot of median predicted vs median measured deflection. The plot reveals the design with few exceptions to be conservative since only approximately one-fourth of the predicted deflections are larger than those actually measured. The data in Figure 21 produce a coefficient of correlation of 0.86 with a 95 percent confidence band of 0.73 to 0.92.

In Figure 22, the average predicted deflection from two or more sampling locations on the same project is plotted against the average measured deflection for that particular



LEGEND	
ROAD LOCATION	
• 03-But-33	■ 06-Tul-FAS-1143
⊙ 03-Sac-FAS-1223	◐ 04-Son-787
△ 04-Nap-FAS-607	□ City of Woodland
⊡ 04-SCl-FAS-1015	■ City of Woodland
x 05-Mon-FAS-652,593	⊕ 05-SBT-156
▽ 05-SB-246	◆ 04-Son-FAS-780
◇ 10-Cal-12, 49	▽ 02-Teh-FAS-1078
✱ 10-Mpa-49	■ 04-Mrn-Son-1
⊕ 10-Mpa-132	◉ 03-Col-45
● 10-Sta-FAS-913,904	◆ 03-Yol-16
	✦ INTERSTATE 15-IDAHO

Figure 21. Predicted deflection vs median measured deflection, 40 individual samples from 21 road locations.



COEFFICIENT OF CORRELATION = 0.93
STD. ERROR OF ESTIMATE = 0.008 IN.

Figure 22. Average predicted deflection from several similar sample locations vs average measured deflection (see Figure 21 for legend).

project. Consideration of averages rather than individual sampling locations improved the coefficient of correlation to 0.93 with a 95 percent confidence band range from 0.75 to 0.98.

Here, the results of 9 of 12 projects indicate that the resilience procedure would have resulted in the selection of a design in which measured deflections were either less or very close to the predicted values. For the remaining three projects, the resilience design could have resulted in measured field deflections slightly in excess of those predicted and designed for. The measured range of deflection on all three of these projects, however, is probably too low for these deviations to have a significant effect on ultimate pavement performance.

The field-trial phase of the development of resilience design procedure will be continued for a period of from one to three years in order to further verify this procedure and to make necessary modification of the method of analysis, test equipment and procedure.

SUMMARY AND CONCLUSIONS

Since 1946 the Materials and Research Department of the California Division of Highways has been engaged in the development of a testing device which applies a dynamic repetitive load to soil specimens. During the early years of this project, primary attention was given to the development of the apparatus itself and the modifications necessary to improve test reproducibility. Subsequently, the resiliometer was utilized for qualitative resilience evaluations of many different soil types.

Beginning in 1960, a correlation program was initiated to relate the resilience properties of individual elements of the various roadway structural sections to pavement deflection measurement. This phase of the investigation was carried out utilizing soil specimens in an undisturbed state or compacted at field moisture and density. Subsequently, design criteria were established for the purpose of using resilience test data in a preliminary pavement design situation in order to predict the deflection of a proposed structural section. The results of these phases of investigation permit the following general conclusions:

1. The relationship between the resilient characteristics of the structural section components and pavement deflections appears to be a realistic and effective basis for a flexible pavement design method.
2. Pavement deflections can be predicted from laboratory resilience values with sufficient accuracy for design purposes. This accuracy is substantiated by the correlation plots of predicted and measured deflections.
3. Our present resilience design method tends to produce conservative structural sections since the majority of predicted deflections are somewhat larger than the measured deflections (Fig. 22).
4. The effectiveness of the resilience design method in relieving fatigue cracking in asphaltic concrete pavements should prove to be as reliable as the deflection criteria on which it is based. Refinements in criteria and design method may be expected when more information regarding effects of different traffic conditions and variables in materials, construction practices and environment is obtained.

ACKNOWLEDGMENTS

The results of the Resilience Research Program and the preparation of the report were accomplished under the general direction of John L. Beaton, Materials and Research Engineer of the California Division of Highways. The authors wish to acknowledge the contribution of many individuals who have participated directly in obtaining, tabulating and analyzing the resilience test data utilized for this report. We are particularly grateful to Harold Munday and Rogel Prysock of this department.

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Discussion

W. H. CAMPEN, Omaha Testing Laboratories, Inc.—Mr. Zube and his co-workers have done a tremendous amount of work in developing a method for thickness design based on rebound or elastic deformation. They have apparently succeeded in determining

in the laboratory the rebound value of the components of a layered system and then in correlating total rebound with actual field rebound on the same system. Finally, a procedure was established for determining thicknesses of base, subbase and select material required on a given basement soil in order to adequately carry a 15,000-lb axle load. In my opinion, the method is very complicated and it may be that the answer could be obtained more directly by constructing a small test section and testing it with the Benkelman beam.

Based on my own research and that of my associates in the evaluation of flexible pavement test sections for airport runways, I can say that the use of elastic deformation as a criterion for the determination of thickness is sound. In 1944, we presented a paper entitled "Analyses of Field Load Bearing Tests Using Plates" (HRB Proc., Vol. 24, p. 87, 1944). The data in the paper reveal three principal reasons for the use of the elastic deformation criterion. They are as follows:

1. With a given plate and given load, the rebound or elastic deformation decreases as the thickness of superimposed layers increases.
2. With a given thickness and a given plate, the elastic deformation increases as the load increases and the load-deflection relationship is a straight line.
3. With a given thickness and a given plate, the elastic deformation remains constant with the repetition of load.

Before asking a few questions, I wish to point out that, in addition to restricting elastic deformation, the layered system as a whole must not show any appreciable permanent consolidation or lateral displacement. I am sure Mr. Zube will agree with me on this.

In addition to the foregoing comments, I wish to ask a few questions.

1. Under general conclusions concerning the behavior of soils, it is stated that resilience increases rapidly with increase in moisture content above optimum moisture. This statement gives the impression that the rebound is due to moisture itself, whereas it is fairly well known that rebound is due to the decompression of trapped air or other gases. I am wondering therefore if the increase in moisture content is the cause of lateral flow of the soil against the air sack surrounding the sample under test. If this is so, the rebound would be a measure of the backflow of the soil-water mixture rather than the increase in volume of the trapped gases in the soil.
2. I am of the opinion that well-graded, highly densified bases and subbases do not possess any rebound quality when loaded normally. For that reason, I ask the following: Would a soil-aggregate mixture having a maximum density of 140 pci, an optimum moisture of 5 percent, an air content of 4.7 percent and CBR of about 100 show any significant resilience in your test?
3. You say that bituminous mixtures have no resilience. How can this be since they may contain 5 percent or more air?
4. You have established a correlation between rebound results obtained in a 4-in. diameter mold and a 15,000-lb axle load. How would you determine thickness for a smaller or a larger wheel load?

ERNEST ZUBE AND RAYMOND FORSYTH, Closure—Before going on to Mr. Campen's specific questions, it would be appropriate to comment on his statement to the effect that in addition to restricting the elastic deformation, the layered system as a whole must not show any appreciable permanent consolidation or lateral displacement.

We agree with this statement wholeheartedly. The resilience design procedure was originated, in fact, to measure the propensity of and eliminate excessive transient deflection and thus possible early fatigue cracking of asphalt concrete pavement designs which satisfied the R-value design criteria. It has been our experience that the R-value test effectively eliminates plastic deformation within a flexible structural section.

It would seem reasonable, however, that a pavement design procedure which satisfied certain elastic criteria would automatically preclude failure due to plastic deformation. Put another way, if a maximum pavement deflection were built into the design, is it likely that the system would suffer any appreciable permanent distortion? In the authors' opinion, this possibility is remote. Therefore, future consideration should be given to the possible elimination of conventional static tests once effective procedures have been established which satisfactorily control the elastic properties of the pavement section.

The following are answers to the four specific questions posed by Mr. Campen:

1. The nature of the mechanics of resilient deformation of soils is beyond the intended scope of this project, although we would agree that this deformation is probably associated with compression of trapped air within the system. It seems likely that increased moisture content tends to lubricate individual soil particles, thus reducing the amount of energy required to effect a given volumetric compression. During the resilience test, the soil specimen is enclosed in a stabilometer with which it is possible to obtain a measurement of the horizontal pressure induced within the soil specimen by the vertical dynamic load. This, of course, is possible through the compression of a known volume of air within the stabilometer system. It is, therefore, possible to convert the lateral pressure measurement to lateral volumetric displacement. This is subtracted from the total measurement obtained by reading the resiliometer manometer tube. Therefore, the resilience value for a given soil reflects net internal compression and rebound only.

2. There can be little doubt that a well-graded densified base and subbase would not manifest a high degree of resilience when loaded normally. Even for these materials, however, there is a very definitely measurable and significant amount of resilience. From a quantitative standpoint, the resilience in a base or subbase material would be substantially lower than that in a basement or embankment material. However, when considering the fact that the pressures applied to a base or even a subbase may be five to ten times greater than that applied to the basement or embankment soil, the resilience contributions of these materials may be equal in importance or even exceed that of the basement and embankment material.

3. Our tests indicate that bituminous mixes have a measurable, though relatively small, amount of resilience. This and the relative uniformity of newly constructed AC surfacing resilience measurements are such that the inclusion of AC resilience testing as part of the test procedure does not appear to be justified at this time.

4. Critical deflection levels for various types of pavement structural sections have been determined for California highways utilizing a 15,000-lb test axle load (3). Thus the performance of several pavement types had been related reasonably well to deflection measurements at this test load, even though these pavements are subject to a wide variety of wheel loadings throughout their service life. This being the case, it does not appear to be necessary to correlate resilience laboratory test data with deflection measurements obtained at other than the standard 15,000-lb single-axle loading. Such a conversion would not be difficult, however, since lineal deflection has been found to be almost directly proportional to test axle load. Thus, deflection measurements obtained with a different axle loading can be conveniently corrected to the 15,000-lb standard. Another approach would involve simply utilizing a revised pressure distribution curve appropriate to test loading and wheel configuration of interest in the analysis of resilience test data.