PERMANENT DEFORMATION CHARACTERISTICS OF SUBGRADE SOILS DUE TO REPEATED LOADING

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Various procedures are examined for controlling or estimating the contribution of the subgrade to the total permanent deformation that occurs in the pavement structure as a result of repeated traffic loads. The use of layered elastic analysis to estimate the vertical compressive strain at the subgrade surface is suggested as one procedure to control such deformation. A summary of available strain data is included. If these criteria are used for design purposes, the same range in stiffness for the asphalt-bound layer and the same values of Poisson's ratio must be used as were used to develop the criteria. Layered elastic analysis is also suggested for estimating the amount of permanent deformation caused by the subgrade. This analysis is based on relationships among permanent strain, applied stress, and number of stress applications based on repeated-load triaxial compression tests; and these relationships are discussed. Data also indicate the importance of stress history effects. Time hardening and strain hardening permit estimates of cumulative loading effects from the results of tests at single stress levels. Although neither method predicts precisely the accumulation of permanent strain under different stress sequences, both bound the measured data and therefore have the potential to assist in the estimation of pavement deformation or rutting from cumulative traffic loading.

PAVEMENT design may be considered as a process in which pavement structure selected by a particular method is checked, and modified if required, to ensure that either various forms of critical distress will be precluded or their effects will be reduced to tolerable levels for the selected design period. We will discuss aspects of the process concerned with minimizing or estimating the amount of permanent deformation (rutting) that occurs in the subgrade of the pavement structure as a result of repeated traffic loading.

At present, two approaches are available to examine this form of permanent deformation. In one method, the vertical compressive strain in the subgrade surface is limited to some tolerable amount associated with a specific number of load repetitions (1). Controlling the characteristics of the material in the pavement section through materials design and proper construction procedures (unit weight or relative compaction requirements) and using materials of adequate stiffness and sufficient thickness so that this strain level is not exceeded will ensure permanent deformation equal to or less than some prescribed amount. The other procedure involves using materials characterization data from laboratory tests and an appropriate analysis procedure to estimate the actual amount of rutting that might occur.

Both approaches are considered, although most attention is focused on test results

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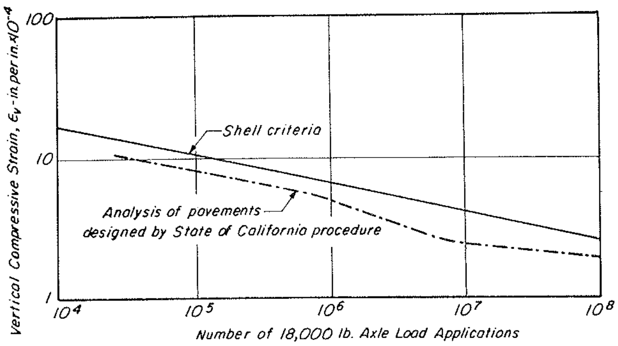
Figure 1. Criteria for vertical compressive strain at subgrade surface to minimize contribution of subgrade to pavement rutting.



from which suitable materials characteristics of subgrade soil can be ascertained for use in the second procedure.

LIMITING SUBGRADE STRAIN CRITERIA TO MINIMIZE PERMANENT DEFORMATION

Limiting subgrade strain criteria have been developed for both highway and airfield pavements (1, 2, 4).

Two such criteria will be discussed for highway pavements: one developed by Dorman and Metcalf (1) and the other developed by Monismith and McLean (2) from elastic analyses of pavements, thicknesses of which were selected by California pavement design procedure.

Criteria developed by Dorman and Metcalf (termed the Shell criteria) are shown in Figure 1 and can be used to ensure that permanent deformation in the subgrade will not lead to excessive rutting at the pavement surface. [These criteria have been developed from elastic analyses of pavements designed according to the California bearing ratio (CBR) procedure and for the AASHO Road Test.] Considering the actual performance results of the AASHO Road Test in terms of rut depth, the criteria in Figure 1 may be thought to be associated with ultimate rut depths of ¾ in. (19 mm) (3).

Alternatively, one could analyze structures designed according to other design procedures and establish different sets of criteria. The second set of criteria in Figure 1 has been developed by examining a series of pavement structures considered adequate according to the California pavement design procedure, and the strain values are less than those suggested by Dorman and Metcalf. It is possible, however, that smaller limiting values of permanent deformation are tolerated in pavements designed by the California procedure.

Both sets of criteria (Figure 1) were developed for stiffnesses of the asphalt-bound surface course in the range from 100,000 to 200,000 psi (689 to 1379 MPa) and for specific values of Poisson's ratio both in the subgrade and in the materials in the pavement structure.

For airfield pavements, the criteria developed by Witczak (4), based in part on the analysis of field trials conducted by the U.S. Army Corps of Engineers Waterways Experiment Station (5), appear appropriate; these strain criteria are as follows (1 in. = 25.4 mm) (4):

Load Applications	Vertical Compressive Strain on Subgrade (in./in. $\times 10^{-4}$)
10^3	19.2
10^4	16.8
10^5	15.2
10^6	14.6

These vertical compressive strains are substantially higher than those associated with highway pavements (Figure 1). Witczak's criteria for vertical strain are based on an analysis of a two-layered elastic pavement section in which the stiffness of the asphalt-bound layer was 100,000 psi (689 MPa) and Poisson's ratios for the upper and lower layers were 0.40 and 0.45 respectively.

When the strain values given previously are used for design purposes, the materials in the pavement structure under consideration must use the same values for asphalt concrete stiffness [i.e., from 100,000 to 200,000 psi (689 to 1379 MPa)] and for Poisson's ratio; otherwise, the resulting analyses will have little significance.

ESTIMATION OF AMOUNT OF RUTTING FROM REPEATED TRAFFIC LOADING

Procedures are available to estimate the amount of rutting from repeated traffic loading; however, none has been well-documented to date. They may be categorized as (a) use of elastic layered system to represent pavement structure and materials characterization by either repeated-load triaxial compression tests or creep tests (at least for the asphalt-bound layers), and (b) use of a viscoelastic layered system to represent pavement structure and materials characterization by creep tests. Of the two methodologies, using an elastic stress-strain analysis and either repeated-load or creep tests appears to be the more reasonable (6, 7, 8, 9) at this time and will be discussed.

So that this type of analysis and the repeated-loading test data can be used, relationships between plastic strain and applied stress must be available for each of the pavement components, i.e.,

$$\epsilon^p = f(\sigma_{ij},\ C_{ij}) \tag{1}$$

where

ϵ^p = plastic or permanent strain,
σ_{ij} = stress state, and
C_{ij} = material properties.

For a particular layer it is then possible to estimate the permanent deformation occurring in that layer. This is done by computing the permanent strain at a number of points within the layer; the number is sufficient to reasonably define the strain variation with depth. Permanent deformation is then determined by summing the products of the average permanent strains and the corresponding difference in depths among the locations at which the strains were determined, i.e.,

$$\delta_i^p(x, y) = \sum_{i=1}^{n} (\epsilon_i^p \Delta z_i) \tag{2}$$

where

$\delta_i^p(x, y)$ = rut depth in the i th position at point (x, y) in the horizontal plane,

ϵ_i^p = average permanent strain at depth $z_i + \frac{\Delta z_i}{2}$, and

Δz_i = difference in depth.

Total rut depth may be estimated by summing the contributions from each layer. A schematic representation of the pavement system used is shown in Figure 2.

If the plastic strain at various numbers of load repetitions is known, the development of rutting with traffic applications can be estimated.

Materials characterization for this type of analysis for each pavement component is not well-defined at this time, although constitutive relationships for some of the materials in the pavement structure have been given.

Barksdale (6) has developed data for granular materials by using repeated-load triaxial compression tests. His data can be represented as follows:

$$\frac{\epsilon^p}{\sigma} = \frac{1/K\sigma_3^n}{1 - \frac{\bar{\sigma}R_f(1 - \sin\phi)}{2(c\cos\phi + \sigma_3\sin\phi)}} \cdot \left(\frac{N}{N_o}\right)^m \qquad (3)$$

where

$K\sigma_3^n$ = relationship defining the initial tangent modulus as a function of confining pressure,

c = cohesion,

ϕ = angle of internal friction,

R_f = constant relating compressive strength to an asymptotic stress difference in which $0.75 \le R_f \le 1$, and

m = experimentally determined coefficient.

These parameters are determined at a specific number of stress repetitions, N_o.

For asphalt concrete, the relationship is

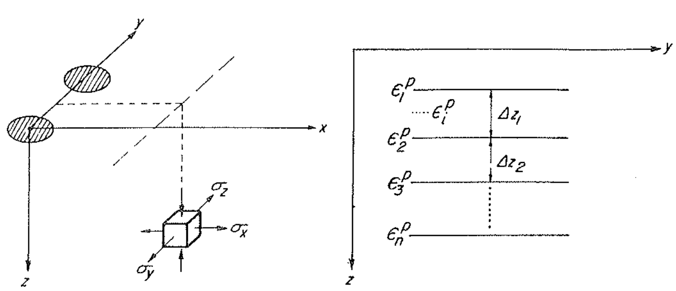
$$\epsilon_z^p = \delta(T)N^a\bar{\sigma}^{n-1}[\sigma_z - \tfrac{1}{2}(\sigma_x + \sigma_y)] \qquad (4)$$

where

σ_x, σ_y, and σ_z = components of stress at a point in the pavement layer and

$\bar{\sigma}$ = equivalent stress, which is defined as

Figure 2. Pavement system used to estimate permanent deformation.



$$\frac{1}{\sqrt{2}} \cdot \sqrt{(\sigma_1 - \sigma_2)^2 + (\sigma_2 - \sigma_3)^2 + (\sigma_3 - \sigma_1)^2}$$

This relationship permits estimation of permanent strain at a point in the pavement structure (18). Coefficients $\delta(T)$, a, and n must be determined experimentally. Although such an approach has not been used extensively, it does have the potential to predict the accumulation of rutting in asphalt-bound pavement layers.

The use of creep tests on asphalt concrete with elastic layer theory to represent the response of the pavement structure to load is an alternative approach proposed recently (10) to estimate the amount of rutting that occurs in the asphalt-bound layers of a pavement structure.

Observations of the development of rut depths in laboratory test tracks (two-layered pavements of asphalt concrete resting directly on the subgrade) as loads are repeated provide data that, when suitably transformed, exhibit the same shape as data from laboratory creep tests in uniaxial compression. The Dorman and Metcalf procedure (10) requires that the mixture stiffness and the viscous component of the asphalt stiffness be determined for use with parameters determined from an analysis of the pavement as an elastic layered system.

For fine-grained soils representative of subgrades, Seed et al. (11, 12) presented data on the influence of applied stress and load sequence on the accumulation of total strain in repeated-load triaxial compression tests performed in the laboratory. In these particular studies, no attempt was made to separate the permanent strain from the recoverable strain, and no constitutive relationships (e.g., equation 1) were developed. The results of the studies by Seed et al. have served, however, as basis for the test program discussed in the following sections. The purpose of this program was to develop constitutive relationships for use in layered system analysis and to permit estimation of the subgrade contribution to permanent deformation at the pavement surface.

REPEATED-LOAD TESTS ON SUBGRADE SOIL

To provide data that may eventually assist in estimating the subgrade contribution to permanent deformation, a series of repeated-load undrained triaxial compression tests were performed on a fine-grained subgrade soil. These tests were conducted to ascertain the influence of compaction conditions, stress magnitude, and stress sequence on the accumulation of permanent strain that occurs with repeated stress applications.

Equipment

Repeated-load triaxial compression tests similar to those used for determination of resilient moduli (2) were used to measure the accumulation of permanent deformation in the subgrade soil specimens. The triaxial cell (Figure 3) can accommodate specimens 2.8 in. (7.1 cm) wide by 6 in. (15.2 cm) high. Air was used for the confining pressure, and only 5 psi (3.5 kPa) was used. Repeated loads were applied pneumatically with a load duration of 0.1 sec and at a frequency of 20 repetitions per minute.

Both axial and radial deformations were measured by LVDTs attached to the soil specimen (Figure 3). Details of the clamps are shown in Figure 4. The axial strain device measured the accumulation of permanent deformation over the center 3 in. (7.6 cm) of the specimens. Measurement of the accumulation of permanent radial strain presented some difficulties initially because of soil creep at the clamps. To overcome this, the strain unit was suspended from springs (Figure 3) and bonded to the membrane by an epoxy resin. Repeated loads were applied over a range in axial stresses, and as many as 100,000 stress repetitions were applied to an individual specimen.

Figure 3. Apparatus for repeated-load test of soil.

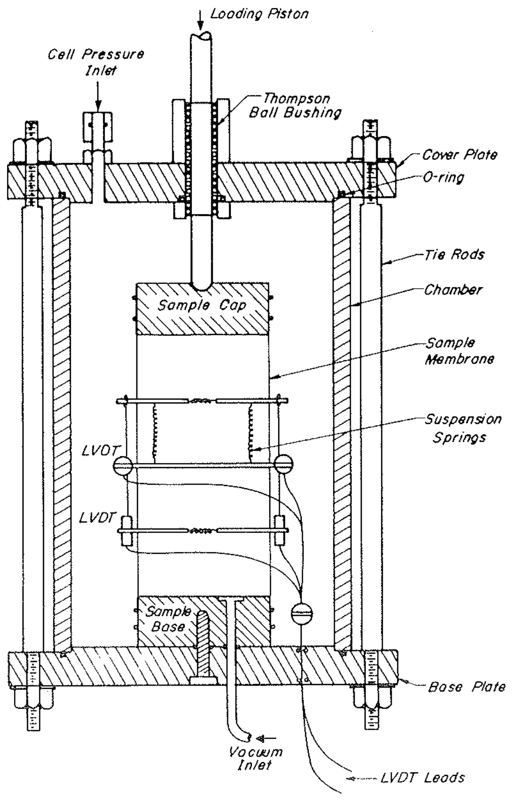


Figure 4. LVDT clamps.

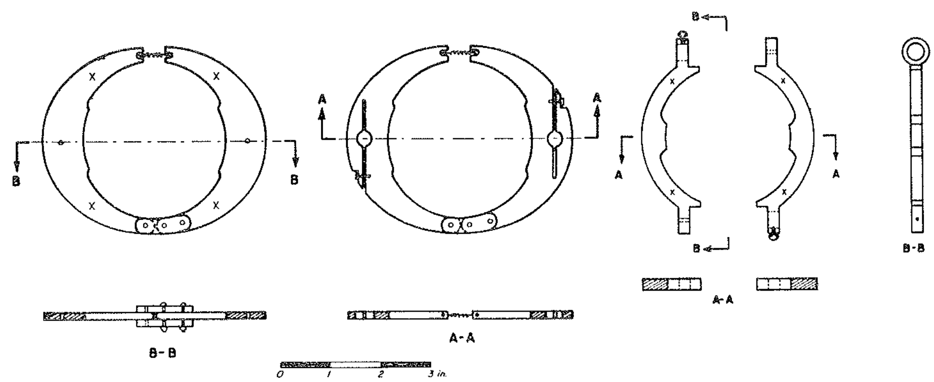
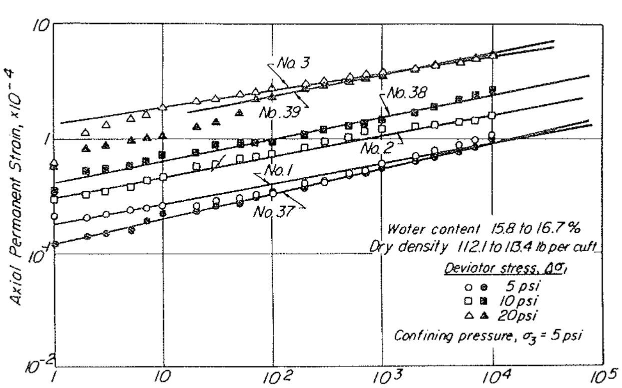


Figure 5. Reproducibility of axial permanent strain versus number of stress applications for Ygnacio Valley Road subgrade soil.



Reproducibility of test results is shown in Figure 5. Comparisons are shown for specimens whose water contents and dry densities were considered to be reasonably comparable. These data for the specimens in Figure 5 are given in Table 1.

Materials and Specimen Preparation

A silty clay (liquid limit = 35, plasticity index = 15), which is representative of the subgrade from a portion of Ygnacio Valley Road in Contra Costa County, California (13), was used.

Figure 6 shows the density versus water content for this material determined in the modified AASHO compaction test. Specimens were prepared at dry densities from 90 to 95 percent of the maximum value obtained in the modified AASHO compaction test and at water contents from 16 to 20 percent.

Static compaction was used in specimen preparation because it is thought to induce particle orientations at high degrees of saturation similar to those obtained under field conditions by compacting the specimen on the dry side of optimum and then subsequently soaking it (14).

Test Procedures

The test program consisted of (a) tests in which samples were subjected to single values of deviator stress [5 to 20 psi (34.5 to 137.8 kPa)] repeatedly applied for 10,000 stress applications generally but sometimes for 100,000 stress applications and (b) tests in which samples were subjected to sequential loading to ascertain the influence of stress history on the accumulation of permanent deformation.

In the stress history tests, two conditions were used:

1. Increasing sequence—10,000 applications each with 5, 10, and 20 psi (34.5, 68.9, and 138 kPa); and
2. Variable sequence—(a) 2,000 applications each with 3, 5, and 10 psi (20.7, 34.5, and 68.9 kPa), (b) 2,000 applications each with 5, 3, and 20 psi (34.5, 20.7, and 137.8 kPa), and (c) 2,000 applications each with 10, 5, and 3 psi (68.9, 34.5, and 20.7 kPa).

For convenience, all specimens were tested at a confining pressure of 5 psi (34.5 kPa). [This pressure is higher than exists in situ; normally static confining pressures in the upper part of the subgrade will be from 1 to 1.5 psi (6.9 to 10.3 kPa)]. Repeated loads were applied at a frequency of 20 repetitions per minute and a duration of 0.1 sec. Time-of-loading discussions (15) would indicate this to be representative of traffic traveling at velocities from 20 to 40 mph (32.2 to 64.4 km/h).

Test Results

Relationships among axial, radial, and volumetric strain and number of stress applications for specific stress levels are shown on semilogarithmic plots in Figures 7, 8, and 9. (Volumetric strain was determined from

$$\epsilon_{vol} = \epsilon_1 - 2\epsilon_3$$

where ϵ_1 and ϵ_3 are the measured axial and radial strains respectively.) Some difficulties were experienced in measuring radial strains partly because of specimen creep. Thus, the values at larger stress repetitions may be slightly smaller than those experienced by the specimens.

Table 1. Water content and dry density for specimens in Figure 5.

Specimen Number	Water Content (percent)	Dry Density (pcf)
1	16.7	112
37	16.4	113
2	16.8	112
38	15.8	113
3	16.0	113
39	16.4	113

Note: 1 pcf = 16.02 kg/m³

Figure 6. Density versus water content for Ygnacio Valley Road subgrade soil conditions 1, 2, and 3.

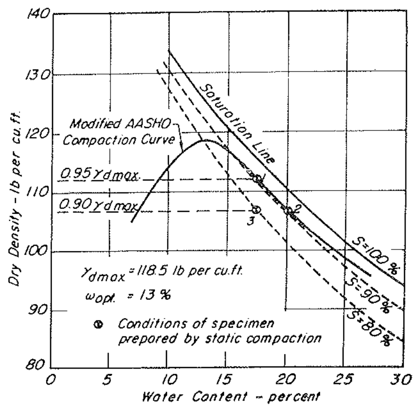


Figure 7. Axial, radial, and volumetric permanent strain versus number of stress applications, deviator stress = 5 psi (34.5 kPa) for condition 1.

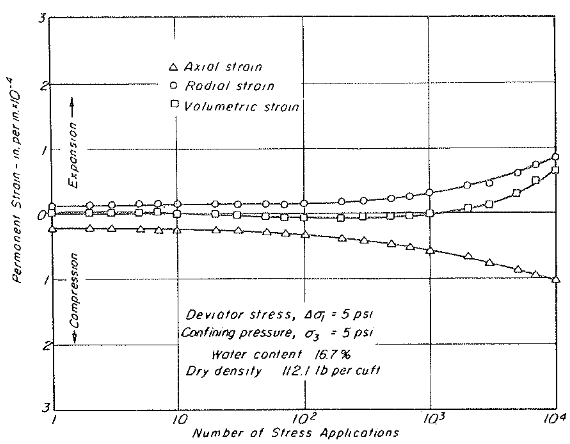
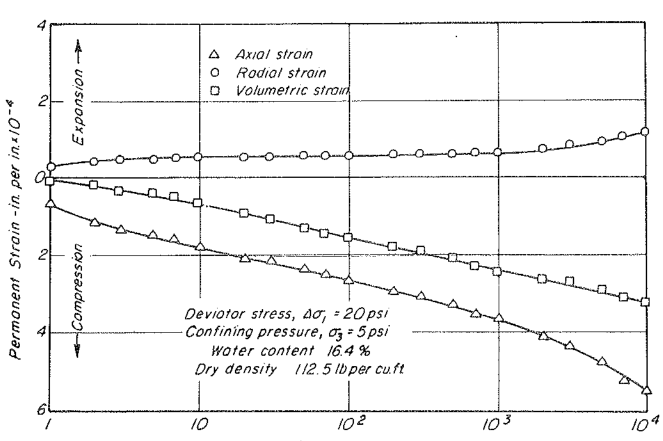


Figure 8. Axial, radial, and volumetric permanent strain versus number of stress applications, deviator stress = 20 psi (137.8 kPa) for condition 1.



To illustrate the change in accumulation of permanent deformation with load repetitions, plots were made of change in strain per cycle versus the number of stress applications. Typical results are shown in Figures 10 and 11.

The influence of stress history is shown in Figures 12 and 13 for specimens that were subjected to stress sequences in which the lower stresses were first applied. Results without conditioning at a lower stress level are also shown. These results are similar to those reported earlier by Seed et al. (11, 12) and illustrate the influence of stress history on the accumulation of permanent strain.

To ascertain whether such effects are predictable from the results of tests performed at particular stress levels, a few tests were performed in which the axial stress levels 3, 5, and 10 psi (20.7, 34.5, and 68.9 kPa) were varied in the sequences noted previously. These results are shown in Figure 14. Again, the influence of stress history is apparent because the three specimens were all subjected to a total of 6,000 stress applications.

RELATIONSHIPS AMONG PLASTIC STRAIN, APPLIED STRESS, AND STRESS APPLICATIONS

Plastic Strain Versus Stress Applications

To develop constitutive relationships that will be useful for analysis and design purposes, we plotted the data of the type shown in Figures 7 through 9 on log-log plots as shown in Figures 15 through 17. The following equation was used to fit the data by a least squares procedure:

$$\bar{\epsilon}^p = AN^b \qquad (5)$$

where

$\bar{\epsilon}^p$ = permanent strain,
N = number of stress applications, and
A, b = experimentally determined coefficients.

The data in these figures show that the linear relationship (on the log-log plot) can be used to represent the data reasonably well.

Table 2 gives the coefficients for a series of specimens tested for various water contents, dry densities, and deviator stresses. Although the data are comparatively few, it is possible that the exponent b depends only on soil type and that the coefficient A is a function of stress level, previous stress history, and placement conditions.

As stated earlier, most of the specimens were subjected to 10,000 stress applications. So that the appropriateness of equation 5 for predicting responses at larger numbers of stress repetitions could be ascertained, a few specimens were subjected to 100,000 applications. Comparisons between measured deformations and estimated values are shown in Figure 18, and reasonable agreement is indicated.

Plastic Strain Versus Applied Stress

Subgrades of well-designed pavements are subjected to comparatively small stresses from conventional traffic loads. Figure 19 shows the results of computations for a two-layer pavement of asphalt concrete resting directly on the subgrade and subjected to a 9,000-lbf (40 000-N) load on dual tires. At stiffnesses in the asphalt-bound layer larger than 200,000 psi (1379 MPa), the vertical compressive stresses in the subgrade are less than 5 psi (34.5 kPa) and approach 0.5 psi (3.5 kPa) at high asphalt concrete

Figure 9. Axial, radial, and volumetric permanent strain versus number of stress applications, deviator stress = 20 psi (137.8 kPa) for condition 2.

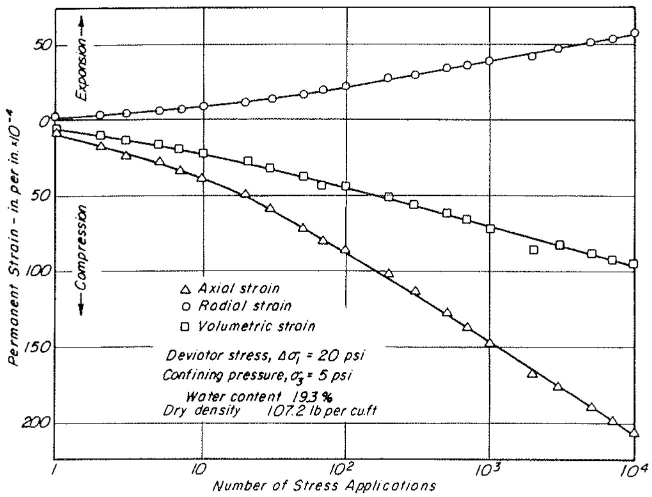


Figure 10. Strain increment versus number of stress applications for condition 1.

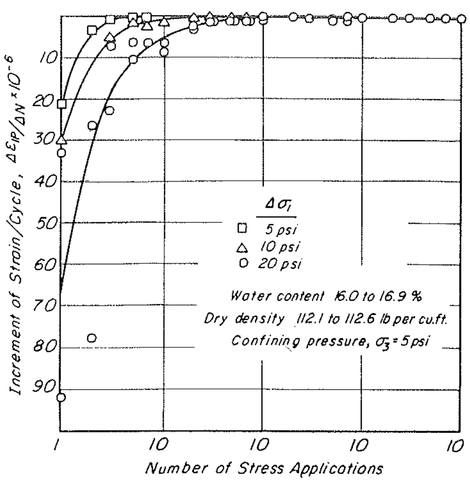


Figure 11. Strain increment versus number of stress applications for condition 2.

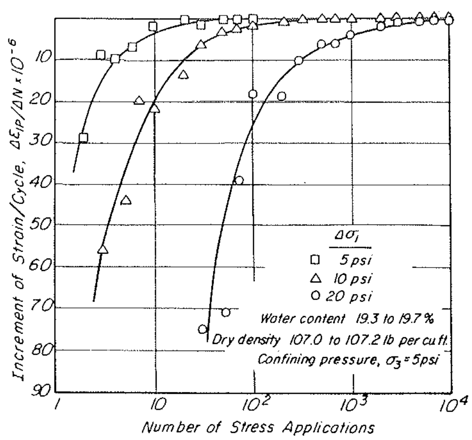


Figure 12. Influence of stress history on permanent strain accumulation for condition 1.

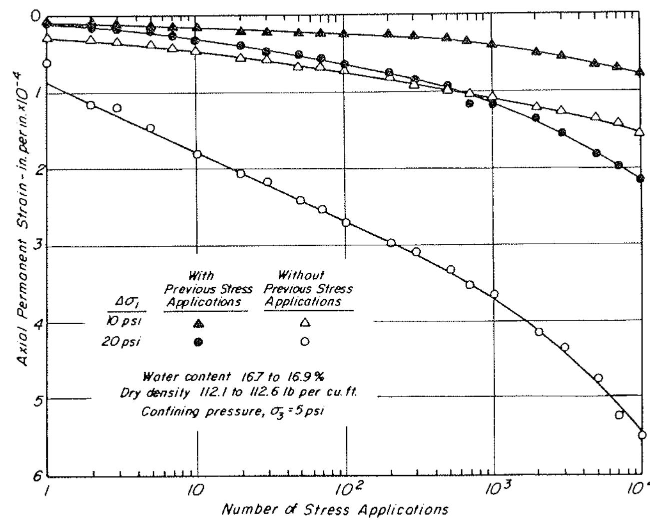


Figure 13. Influence of stress history on permanent strain accumulation for condition 2.

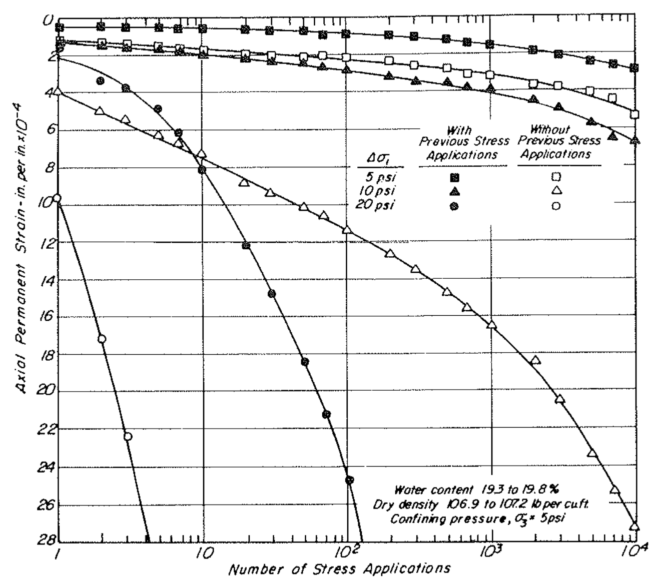


Figure 14. Influence of stress sequences on permanent strain accumulation.

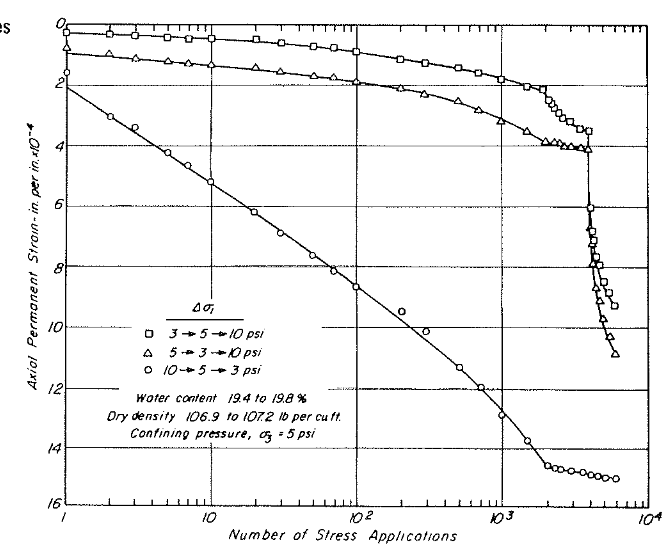


Figure 15. Axial permanent strain versus number of stress applications for condition 1.

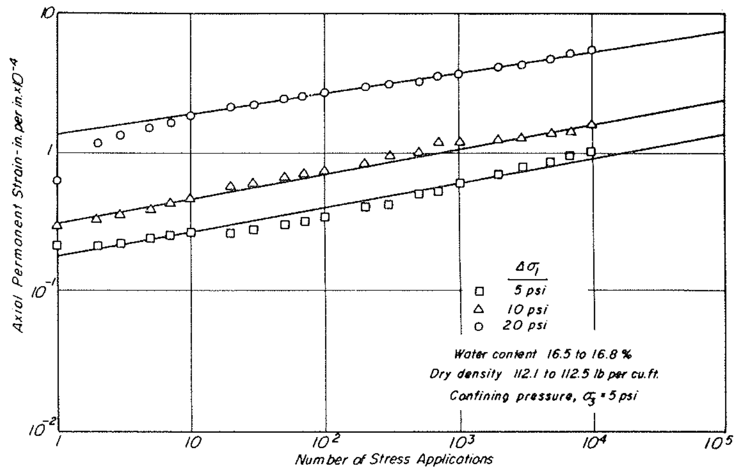


Figure 16. Axial permanent strain versus number of stress applications for condition 2.

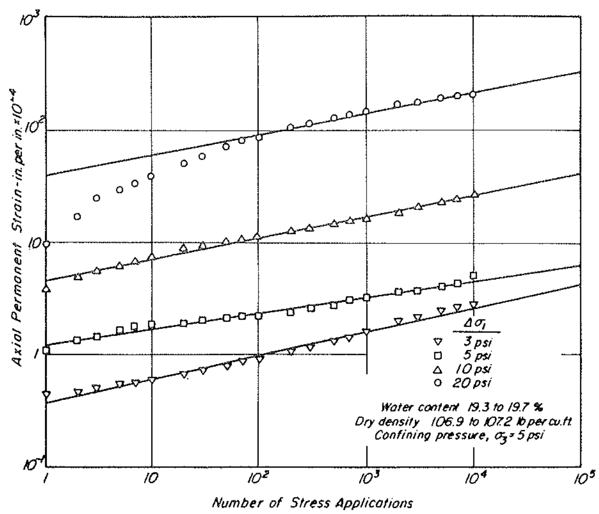


Figure 17. Axial permanent strain versus number of stress applications for condition 3.

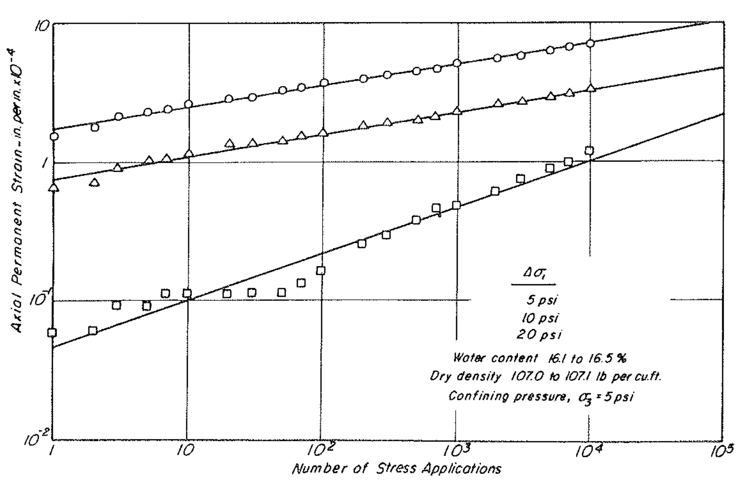


Table 2. Permanent deformation determinants for Ygnacio Valley Road subgrade soil.

Specimen Conditions				
Water Content (percent)	Dry Density (pcf)	Repeated Deviator Stress (psi)	Coefficient[a] A (× 10^{-4})	b
16.7	112	5	0.168	0.184
16.8	112	10	0.306	0.185
16.5	112	20	1.28	0.156
19.8	107	3	0.378	0.212
19.3	107	5	1.22	0.145
19.7	107	10	4.57	0.193
19.3	107	20	39.5	0.185
16.4	107	5	0.0467	0.332
16.5	107	10	0.746	0.163
16.1	107	20	1.73	0.154

Note: 1 pcf = 16.02 kg/m³. 1 psi = 6.89 kPa.
[a]For $\epsilon^p = AN^b$.

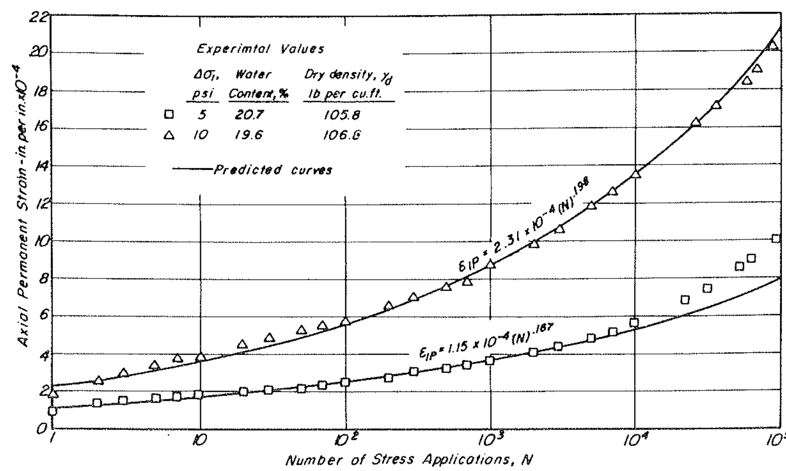


Figure 18. Comparisons of predicted and measured permanent strains versus stress applications.

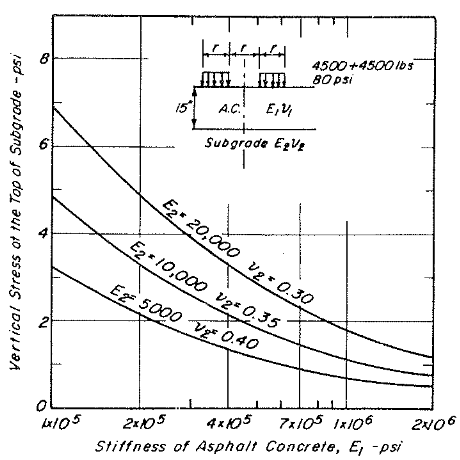


Figure 19. Influence of asphalt concrete and subgrade stiffness on vertical compressive stress at subgrade surface; υ_1 = 0.40.

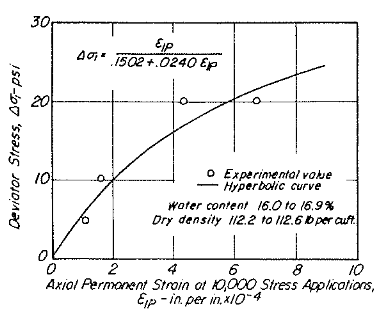


Figure 20. Axial permanent strain versus repeatedly applied deviator stress for condition 1.

stiffnesses for a comparatively weak subgrade. At these stress levels, measurement of permanent deformation is not so precise as at higher stresses. Accordingly, if the permanent deformation at lower stresses could be deduced from measurements at higher stresses, such measurement difficulties could be minimized.

Kondner (16) and Duncan and Chang (17) have suggested that stress versus strain obtained from static triaxial tests can be approximated by a hyperbola. Similarly, Barksdale (6) has presented data indicating the applicability of such a relationship to the data for plastic strain versus stress at a specific number of load applications for granular materials.

Based on this concept, an equation of the following form was used to develop a relationship between applied stress and plastic strain at a particular number of stress applications for the fine-grained soil studied:

$$\Delta\sigma_a = \frac{\epsilon_a^p}{\ell + m\epsilon_a^p} \tag{6}$$

where

σ_a = repeated axial stress,
ϵ_a^p = cumulative permanent axial strain at a specific number of stress applications, and
ℓ, m = experimentally determined coefficients.

Equation 6 may also be expressed as

$$\frac{\epsilon_a^p}{\Delta\sigma_a} = \ell + m\epsilon_a^p \tag{7}$$

If $(\epsilon_a^p/\Delta\sigma_a)$ is plotted as a function of ϵ_a^p, a straight line is obtained. The intercept of this line with the $(\epsilon_a^p/\Delta\sigma_a)$ axis yields the value of ℓ, and the value of m corresponds to the slope of the line.

Using least squares techniques, we analyzed the experimental data according to equation 7. After coefficients ℓ and m were deduced by this procedure, the equations were then plotted in the form of equation 6. Comparisons of actual data at 10,000 stress applications and the hyperbolic relationship are shown in Figures 20, 21, and 22 respectively for the data shown in Figures 15, 16, and 17. It would appear that such a relationship is a useful way of representing the data for permanent strain versus applied stress and that permanent strains at small stress levels can be deduced from tests at larger stress levels for which the strains can be more precisely measured with relatively simple equipment of the type used in this investigation.

The relationships (Figures 20, 21, 22) can also be used to estimate the accumulation of permanent deformations for different stresses resulting from wheel loads of varying intensities.

CUMULATIVE LOADING CONSIDERATIONS

Data that were obtained by applying stresses of different magnitudes in different sequences have already been shown in Figures 12 through 14. Inasmuch as the actual stress sequence is not known in the field, it would appear desirable to have some way in which the results of such cumulative loading could be predicted from the results of simple loading tests (e.g., Figures 15 through 19).

At present, at least two methods are available to obtain the cumulative permanent strain from the results of simple loading tests: (a) a time-hardening procedure and (b) a strain-hardening procedure. Both are shown schematically in Figure 23.

Figure 21. Axial permanent strain versus repeatedly applied deviator stress for condition 2.

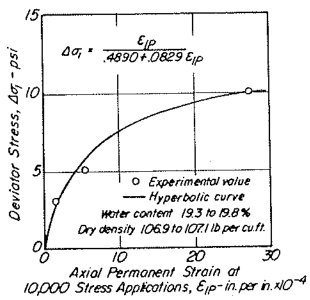


Figure 22. Axial permanent strain versus repeatedly applied deviator stress for condition 3.

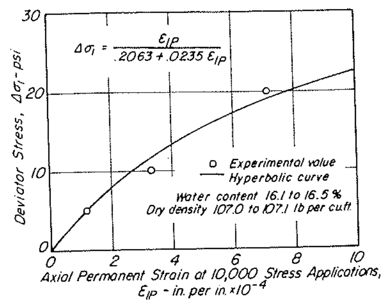


Figure 23. Procedures to predict cumulative loading from results of simple loading tests.

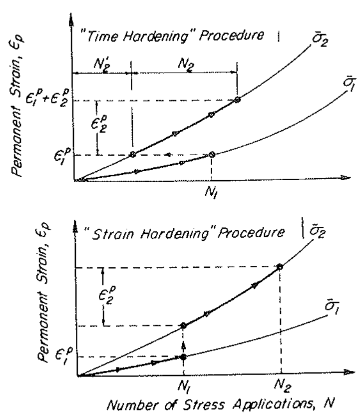


Figure 24. Comparison of predicted and measured cumulative permanent strain, sequence of 3 to 5 to 10 psi (20.7 to 34.5 to 68.9 kPa).

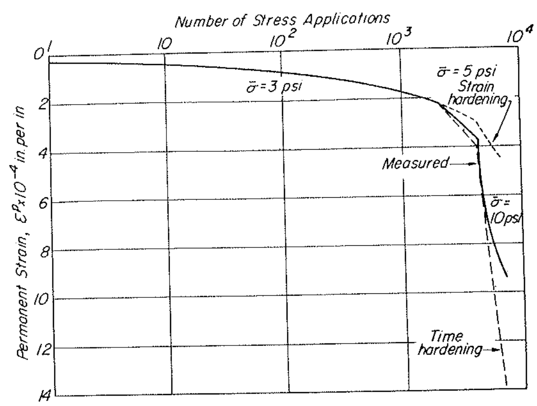


Figure 25. Comparison of predicted and measured cumulative permanent strain, sequence of 5 to 3 to 10 psi (34.5 to 20.7 to 68.9 kPa).

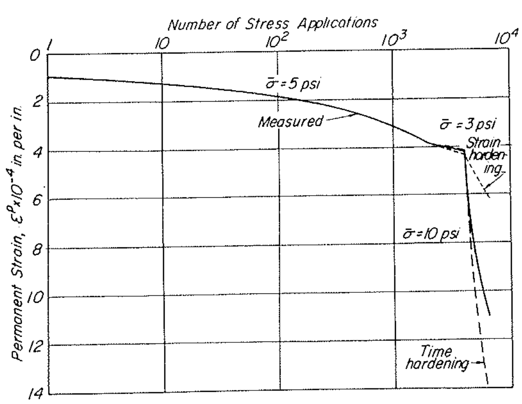
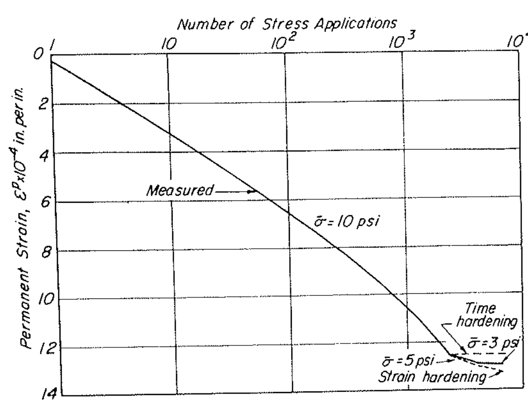


Figure 26. Comparison of predicted and measured cumulative permanent strain, sequence of 10 to 5 to 3 psi (68.9 to 34.5 to 20.7 kPa).



In the time-hardening method, if the specimen is loaded for N_1 repetitions of stress state $\bar{\sigma}_1$, the resulting permanent strain will be $\epsilon_1^p(N)$. The equivalent number of repetitions, N_2', at stress state $\bar{\sigma}_2$, which would have given the same permanent strain, is obtained as shown in Figure 23. If further N_2 applications of load $\bar{\sigma}_2$ are applied, the total strain will follow the path as shown in Figure 23.

The strain-hardening procedure shown requires determination of ϵ_1^p after N_1 repetitions of stress $\bar{\sigma}_1$. The number of repetitions at stress $\bar{\sigma}_2$ is then taken to be equal to N_1, and a further N_2 application is applied. Total permanent strain is the sum ϵ_1^p and ϵ_2^p.

Both approaches were used to predict the responses shown in Figure 14 from test data at single stress levels. Figures 24, 25, and 26 show comparisons of the experimental results with those predicted by both methodologies for 2,000 applications at each stress level. In these figures, neither method provides a solution that agrees quantitatively with the experimental results. Interestingly, however, the predicted results are in qualitative agreement; furthermore, the predicted results bracket the actual data. More detailed examination indicates that the time-hardening procedure provides closer agreement if the stress levels are successively increased and the strain-hardening method provides closer agreement if the loads are successively decreased.

SUMMARY AND CONCLUSIONS

This paper discussed procedures for controlling or estimating the contribution of permanent deformation in the subgrade to the total deformation occurring in the pavement structure. If a design methodology is used in which the vertical compressive strain at the surface of the subgrade is limited to some specific value (e.g., the criteria used by Dorman and Metcalf), the same stiffness values for the bound paving materials and the same Poisson's ratio for the subgrade must be used in the estimation process as were used to develop the criteria.

Contribution of the subgrade to the total permanent deformation occurring at the pavement surface can be estimated by using elastic theory and constitutive equations of the form developed in equations 1 and 6. Equation 1 relates permanent strain to the number of stress applications, and equation 6 relates applied stress and permanent strain.

The coefficients of equations 1 and 6 must be determined experimentally because no general guidelines are available as yet to permit these estimations.

If the effects of cumulative loading are considered, it would seem possible only to bound the subgrade contribution to permanent deformation because the influence of stress history is not well-defined. The time-hardening and strain-hardening procedures described, which make use of data for permanent strain versus number of stress applications at single stress levels, have the potential to provide these bounds.

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The contents of this paper reflect the views of the authors, who are responsible for the facts and the accuracy of the data presented. The contents do not necessarily reflect the official views or policies of the state of California or the Federal Highway Administration. This paper does not constitute a standard, specification, or regulation.

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