rock aggregate management plan, the Gifford Pinchot National Forest, of the U.S. Department of Agriculture, Forest Service, used the available techniques from the Forest Service and developed a rock aggregate management planning process that can be applied to areas other than the forest land. The process could generate an optimal rock aggregate allocation pattern based on the least cost as well as least fuel consumption.

The planning process has been applied to the Mount St. Helens volcano timber salvage area to determine the rock aggregate needs and to allocate them for transportation system construction. The result of the application indicated that the process allows the examination of complex options and alternatives conveniently. The planner may use it to develop an aggregate management plan for a specific project or for a region under a pressing deadline. Its consideration of fuel consumption may increase the planner's confidence in optimal use of aggregate materials and energy.

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Soil Support Value–A New Horizon

GILBERT Y. BALADI AND TESFAI GOITOM

The soil support value of the American Association of State Highway Officials interim guide for the design of flexible pavement is given a new horizon. It is shown that the soil support scale can be expressed in terms of a normalized model. This model relates the dynamic load capacity of a subgrade soil to its static strength. The model was verified by using five different materials that ranged from gravel, sand, and clay to clayey silt.

The determination of a flexible pavement structural thickness (surface, base, and subbase) depends on two major factors--traffic and subgrade strength. Existing design procedures call for different subgrade strength parameters or strength-scaling factors [elastic modulus, resilient modulus, California bearing ratio (CBR), soil support value (SSV), etc.]. The American Association of State Highway Officials (AASHO) design method, in particular, uses a subgrade strength-scaling factor called an SSV. This factor was assigned an empirical scale with values from 3 to 10. Point 3.0 on the soil support scale represents the roadbed soils at the AASHO Road Test. As pointed out by the AASHO interim guide, the units of the SSV have no direct relationship to any procedure for testing soils. Therefore, it is necessary for each design agency to establish a correlation between SSV and some testing procedure before this guide can be used for flexible pavement design.

In this paper, it is shown that the empirical soil support scale is related to a significant physical property of the subgrade material in question. This relationship is independent of sample and test variables and it is unique in its nature for the particular subgrade material under consideration.

BACKGROUND

The basic design equation, developed from the results of the AASHO Road Test, is valid for one SSV, which represented the roadbed soils and the conditions that existed at the test site and during the time of test. Consequently, it was necessary to assume an SSV scale to accommodate the variety of soils that could be encountered at other sites (1,2). This led different highway engineers to assume different SSVs for the same subgrade materials. Further, the assumed SSV bore no relationship to any of the subgrade physical parameters. This problem has led researchers, highway engineers, and several agencies and highway departments to develop several correlations relating the SSV to different test results. These correlations include the following:

1. Correlation between CBR and SSV: The Utah State Department of Highways (1) conducted several CBR tests on compacted samples of the AASHO Road Test roadbed soils, the crushed-stone base materials, and other soil types. An empirical logarithmic scale, shown in Figure 1, was then assumed to relate the CBR and the estimated SSV of these materials. The same correlation plotted on arithmetic scales is also shown.
2. Correlation between modulus of deformation and SSV: Chou (3) presented a procedure for subgrade evaluation to estimate the SSV. He conducted triaxial tests on subgrade soil samples at field densities and moisture contents. The moduli of de-

Figure 1. Correlation between SSV and CBR.

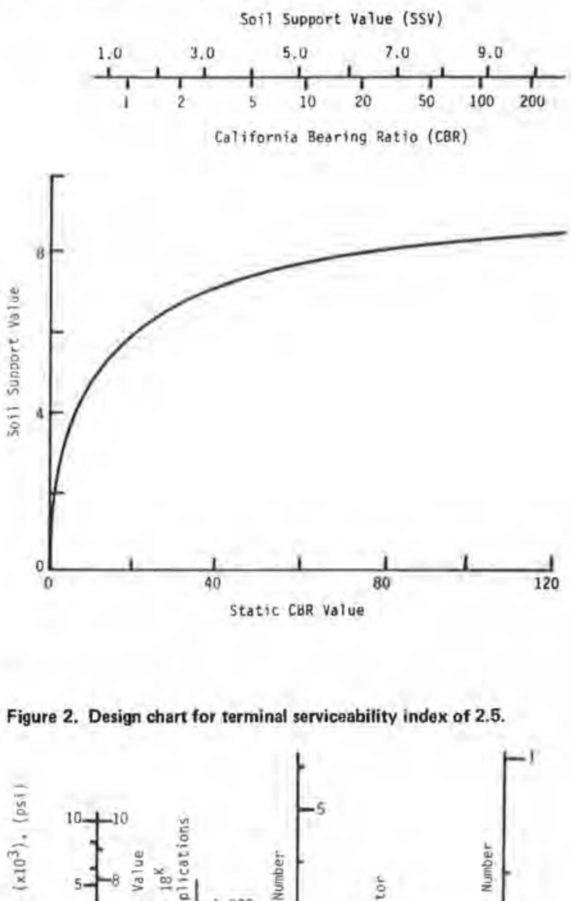


Figure 2. Design chart for terminal serviceability index of 2.5.

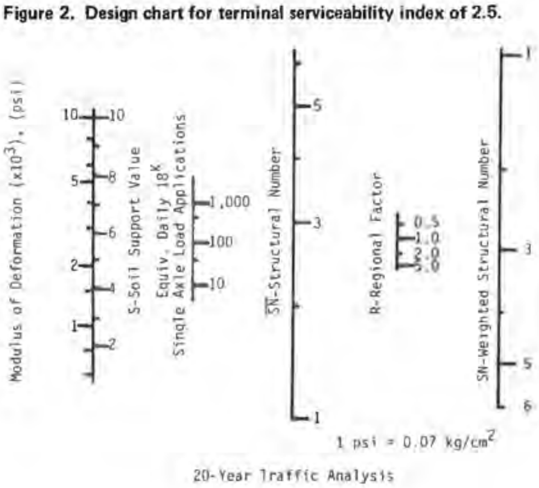


Figure 3. Correlation chart for estimating SSV.

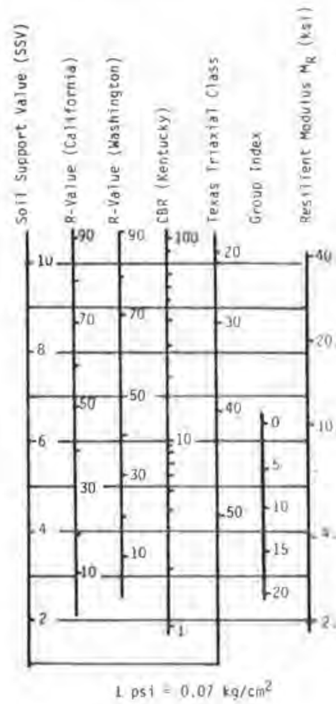


Figure 4. Resilient modulus versus SSV for recompacted and undisturbed cohesionless soils for first stress invariant of 15 psi.

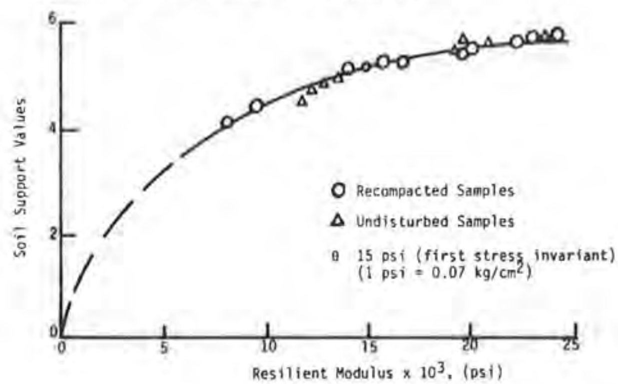
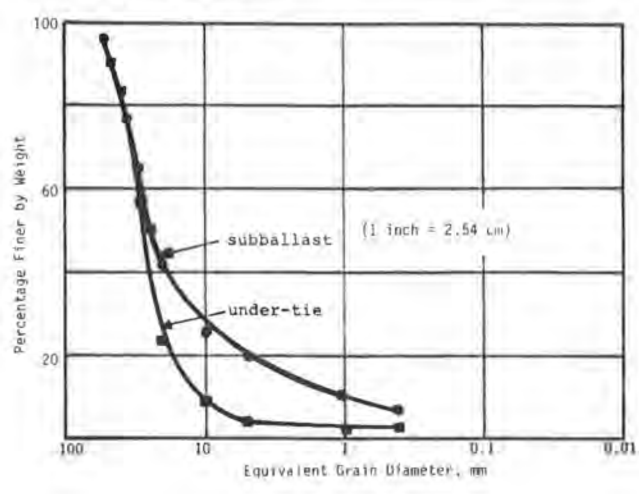


Figure 5. Average grain size distribution curves for Lorraine and Aberdeen subballast and under-tie materials.



formation were then calculated and correlated to an assumed SSV scale as shown in Figure 2 (based on the AASHO interim guide except for the addition of the modulus of deformation scale).

3. Correlation between SSV and resilient modulus: Van Til and others (4) were among the first researchers to establish a correlation between the SSV and the resilient modulus of the subgrade soil at the AASHO Road Test. They used 40 000 psi (2812 kg/cm²) (a maximum value) as the resilient modulus of the crushed-stone materials and 3000 psi (211 kg/cm²) (a minimum value) as the resilient modulus of the AASHO A-6 subgrade soils. These two values were the limiting resilient modulus values on their scale, as shown in Figure 3. Van Til and others recommended that an effort be made to strengthen the validity of the soil support scale as new analytical tools and methods of characterizing material properties become available.

Based on a recommendation by Van Til and others, Baladi and Boker (5) developed a relationship between SSV and the resilient modulus of Michigan cohesionless soil. This relationship was dependent on the stress intensity and is given by the following equation:

$$SSV = 1.96 \log M_R + (M_R/19\,750) - 3.98 \quad (1)$$

where M_R is the resilient modulus.

Figure 4 shows this relationship for recompacted and undisturbed Michigan cohesionless subgrade soil tested under the first stress invariant (θ) of 15 psi (1.05 kg/cm²). Similar correlations were obtained for first stress invariants of 20 and 30 psi (1.4 and 2.1 kg/cm²).

TEST MATERIALS

Five different types of materials that ranged from gravel to silty clay soils were used in this study. The gradation curves of the test materials are shown in Figures 5 through 8. It should be noted that types one and three materials (under tie and subballast) were tested by J.T. Siller at the University of Massachusetts, Amherst (6). The other three materials were tested at Michigan State University under the direction of Baladi.

TEST RESULTS

All five types of materials were tested by using static and repeated-load triaxial tests. Figure 9 shows a typical plot of the logarithm of accumulated axial permanent strain as a function of the logarithm of the number of load repetitions (N) for samples consolidated under a confining pressure of 5 psi (0.35 kg/cm²) and tested by using different cyclic stress ratios. Typical results of the static tests are shown in Figure 10.

DISCUSSION

Lentz (7) and Lentz and Baladi (8-10) provided the technical guidance for the early phase of this work. They reported that the plastic strain of sand subgrade materials could be predicted by using triaxial test results. They concluded that the predic-

Figure 6. Grain size distribution of highway subgrade sand.

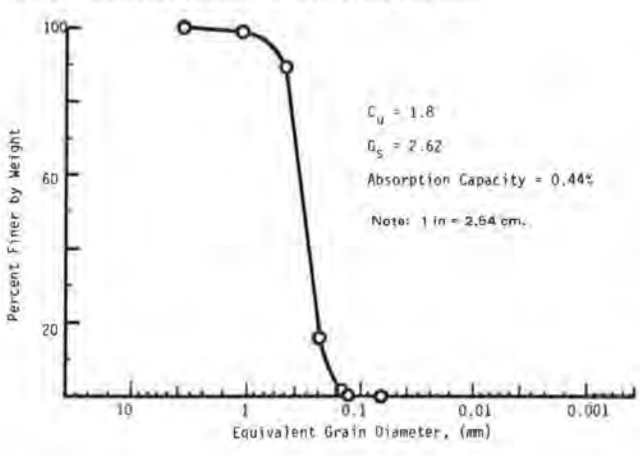


Figure 7. Grain size distribution curves for sites 1 and 2, Lower Peninsula.

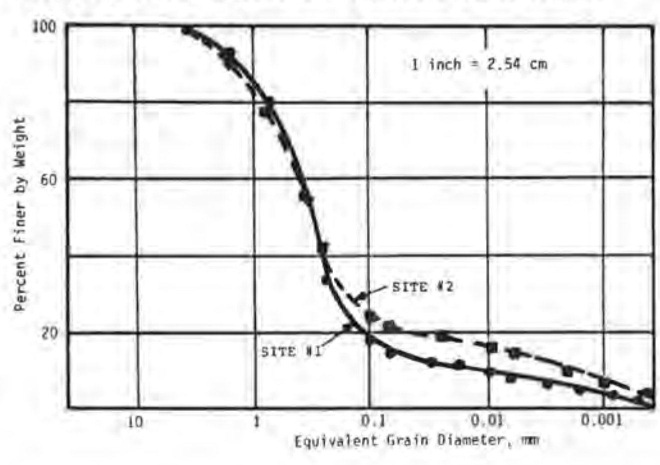


Figure 8. Grain size distribution curve for the AASHO roadbed soil.

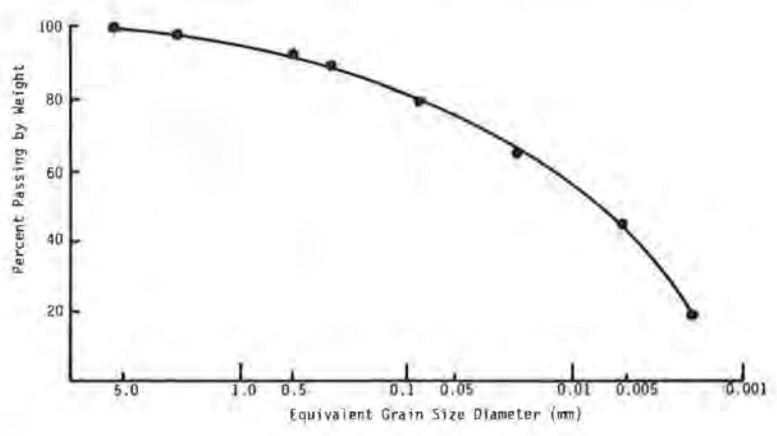


Figure 9. Typical axial permanent strain versus number of load applications (site 2, Lower Peninsula).

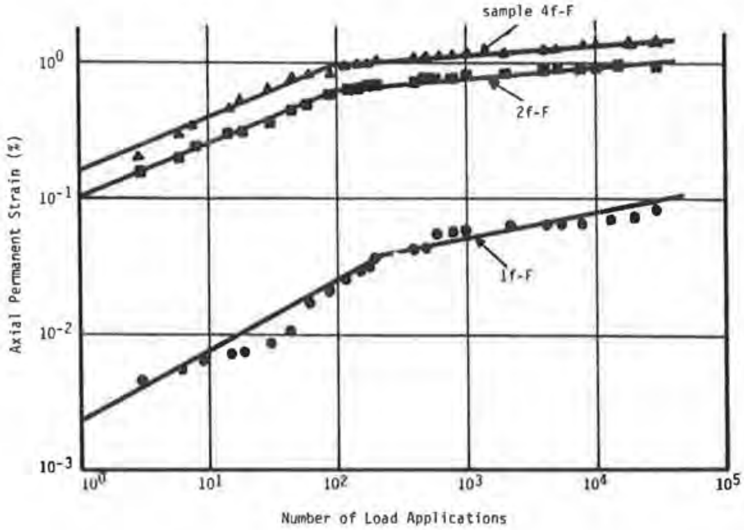


Figure 10. Principal stress difference versus total axial strain from incremental creep tests (site 2, Lower Peninsula).

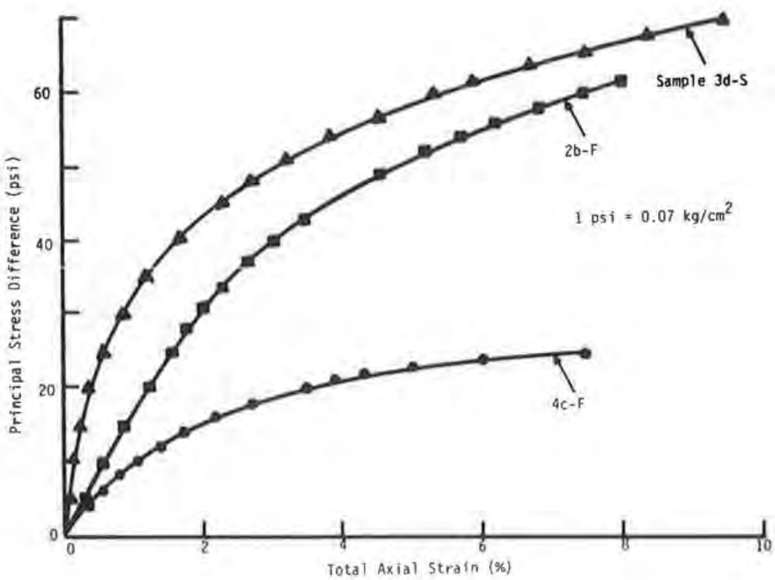


Figure 11. Normalized cyclic principal stress difference versus normalized permanent strain.

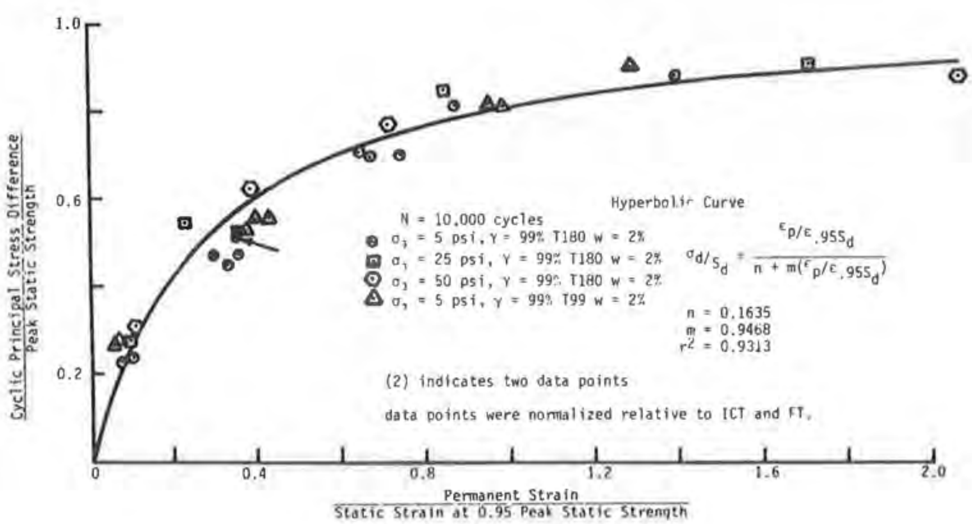
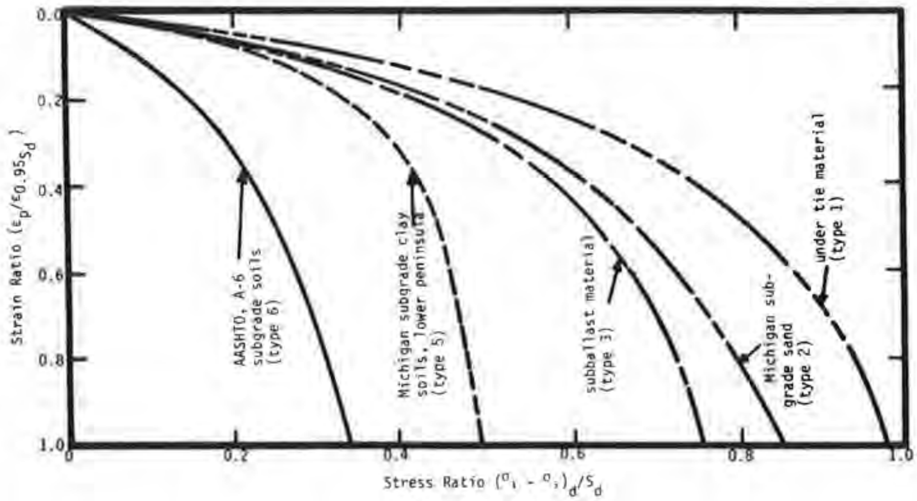


Figure 12. Normalized cyclic stress-strain ratio for five different materials subjected to 1 000 000 load repetitions.



tion model is dependent on the number of load applications and independent of the test variables (confining pressure, stress level) and sample variables (compaction effort and moisture content). They observed that the cyclic and static tests are highly dependent on the same test and sample variables. Consequently, they rationalized that the data from both tests could be normalized to minimize the effects of the sample and test variables. Their normalization process could be summarized as follows:

1. The cyclic principal stress difference $(\sigma_1 - \sigma_3)_d$ was expressed in terms of the peak static strength (S_d) of an identical soil sample tested under the same confining pressure by using static triaxial tests. [The term "static triaxial tests" indicates either incremental creep test, ramp test, or conventional triaxial test (11).]
2. The cumulative permanent strain (ϵ_p) at the desired number of load repetitions (N) was normalized relative to the static axial strain at 95 percent of the sample strength ($\epsilon_{0.95S_d}$) of an identical sample tested under the same confining pressure by using the incremental creep test. Figure 11 shows their normalized data for a natural-sand deposit as well as that for manufactured sand (12). For more information on their data and normalization procedure, the reader is referred to a paper by Lentz and Baladi (9).

Figure 12 shows a plot of the normalized stress ratio as a function of the normalized strain ratio for all five materials. These normalized curves were modeled by using a hyperbolic function (Equation 1). As was expected, this function was found to be independent of confining pressure, principal stress difference, and water content:

$$\epsilon_p/\epsilon_{0.95S_d} = n/[S_d/(\sigma_1 - \sigma_3)_d] - m \qquad (2)$$

where

ϵ_p = cumulative permanent strain at desired number of load repetitions;
$\epsilon_{0.95S_d}$ = total axial strain at 95 percent of static strength;
S_d = static strength from incremental creep test, ramp test, or conventional triaxial test (5);
n,m = regression parameters; and
$(\sigma_1 - \sigma_3)_d$ = cyclic principal stress difference, density and water content.

The value of parameters n and m of Equation 2 is dependent only on material type and number of load repetitions.

Study of the AASHO interim guide has indicated that the SSV of one particular subgrade material is independent of lateral stress, stress level, water content, regional factor, and method of compaction. The SSV, however, is dependent only on the soil type. This could be restated as follows: the SSV of one material is fixed and constant unless some stabilizing agent is introduced and thus the soil type is changed. Indeed, the A-6 material, according to the AASHO soil classification, is assigned an SSV of 3.0. An SSV of 10.0 was assigned to the A-1 materials. These observations suggested that the physical parameter of the subgrade material to be related to the SSV should possess the following properties: (a) be independent of lateral stress; (b) be independent of stress level; (c) be independent of ambient and moisture conditions; (d) be independent of density, void ratio, and consolidation; and (e) be dependent only on soil type. To the best of our knowledge, such a physical parameter does not exist. Consequently, a new search to explain the SSV and relate it to a mathematical and/or physical model rather than one single parameter was initiated. The requirements of the model should be the same as those of the physical parameter.

At this time, the normalization process discussed above was finalized and proved to be valid for a wide range of materials. Recall that the normalized curve (stress ratio versus strain ratio) was found to be independent of (a) confining pressure; (b) stress level; (c) moisture content; and (d) density, void ratio, and consolidation; it was dependent on soil type and the number of load applications. These requirements appeared to be adequate except for the dependency of the normalized model on the number of load applications. These observations suggested that if the normalized model is fixed at a certain number of load repetitions (N = 1 000 000), it could be used to examine its relation to the SSV.

In reference to Figure 12, if it is assumed that for each soil type, failure occurs when the strain ratio reaches 100 percent, it follows that for the same number of load repetitions, a different stress ratio is required for different materials to fail. These stress ratios for the five materials in Figure 12 are 0.33 for A-6 subgrade soils, 0.49 for Michigan clay subgrade, 0.76 for the subballast materials, 0.85 for Michigan sand subgrade, and 0.98 for under-tie materials. These values could be obtained from Equation 2 by using a strain ratio of 1.0 and solving for the stress ratio:

$$(\sigma_1 - \sigma_3)_d/S_d = 1/(n + m) \quad (3)$$

The value of the stress ratio of Equation 3 represents the dynamic load capacity that the soil in question can be subjected to so that failure will occur at 1 000 000 load repetitions. If the predesignated SSV is superimposed on the above data (SSV = 3.0 for A-6 soils and SSV = 10.0 for the under-tie materials), it follows that the SSV can be expressed by using the following equation:

$$SSV = 10\ [(\sigma_1 - \sigma_3)_d/S_d]_{(f,\ N=10^6)} = [10/(n + m)]_{(f,\ N=10^6)} \quad (4)$$

where the subscript $(f,\ N = 10^6)$ indicates failure at 1 000 000 load applications. Equation 4 can be generalized as follows:

$$SSV = \alpha\ [(\sigma_1 - \sigma_3)_d/S_d]_{(f,\ N)} = [\alpha/(n + m)]_{(f,\ N)} \quad (5)$$

where α is constant depending on the number of load repetitions (N) and the subscript (f, N) indicates failure at N load applications.

CONCLUSIONS

A normalized and predictive model of the plastic strain of pavement materials was developed. The ability of the model to evaluate and predict the plastic behavior of several materials subjected to cyclic loading had previously been demonstrated. In this paper, a new understanding of the AASHO SSV scale is presented and it is based on the developed normalized and predictive model. A correlation between the SSV and the ratio of dynamic stress to static strength of the soil in question is shown.

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