STRESS-STRAIN CHARACTERIZATION OF TWO COMPACTED SOILS

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This study defines a triaxial test method for controlling or measuring the variables that govern stress-strain behavior of partially saturated, compacted soil in compression. Triaxial data for Peorian loess and Hastings silt loam from Nebraska are combined into a useful, compact form by using exponential functions. Stress-strain curves representing a wide range of initial densities and moisture contents are reasonably well characterized by a hyperbolic stress-strain function. The two coefficients of this function are shown to be exponential functions of void ratio and degree of saturation. Techniques are given for determining elastic (secant) moduli as functions of the level of stress difference, degree saturation, and void ratio. These moduli can be used in flexible pavement design procedures requiring a subgrade stiffness for any reasonable assumed in situ subgrade condition of dry density and moisture content for the soils tested.

•THE primary cause of premature flexible pavement failure is cumulative plastic strain in the subgrade caused by repetitive, high-axle loads applied during periods of near saturation of the subgrade soil. Current concepts of flexible pavement design (1, 2, 3, 4, 5) predict the required depth of asphalt concrete to control these strains within acceptable limits. Designs are based on traffic and environmental conditions and laboratory tests of representative, compacted soil samples. Tests are conducted according to procedures purported to permit evaluation of subgrade strength and stiffness. Many of these tests neither measure nor control the known variables affecting the behavior of partially saturated, compacted soil (6, 7, 8). Current triaxial testing techniques can control or measure these variables. The triaxial test (9, 10, 11) is the most widely available test that is consistent with well-established principles of theoretical soil mechanics that have this capability. Currently, it appears to be the most appropriate test for routinely evaluating stress-strain behavior of compacted samples.

The specific test configuration for controlling both the relevant test variables and the synthesis of test data into a compact form is described in this paper. Possible applications of these data to flexible pavement design are shown.

STRESS-STRAIN CHARACTERISTICS OF COMPACTED SOILS

The physical state of a partially saturated soil depends on variables such as void ratio e, degree of saturation S_r, and soil structure λ. This indicates that it depends on the arrangement of soil particles and the electrical forces between adjacent particles (12) and invariants of the applied stress system (8).

Vertical strain ϵ in a partially saturated soil sample in triaxial compression ($\sigma_2 = \sigma_3$) can be given by a function of the form

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$$\epsilon = H(p_a, q, u_c, e, S_r) \tag{1}$$

where

σ_1 = major principal stress (compression +),
σ_3 = minor principal stress (confining pressure),
$p_a = \frac{\sigma_1 + 2\sigma_3}{3} - u_a$ = mean stress,
$2q = (\sigma_1 - \sigma_3)$ = stress difference,
$u_c = u_a - u_w$ = suction pressure,
u_a = pore air pressure, and
u_w = pore water pressure.

It can be shown that u_w at small strains and initial soil structure λ_i (of compacted samples having a mineralogical and grain size composition C and molding water content w) are equivalent considerations (13, 14, 15) under certain limiting conditions, namely, the compaction procedure used, time of aging, and specimen temperature during aging and testing. After large strains have been applied, the effects of initial structure are largely obliterated, and the maximum stress difference is principally a function of molding water content w (13). Therefore, from equation 1, for a given compaction procedure and a range of moisture contents, if σ_3, u_a, and C are held constant, for small strains

$$\epsilon = H_1(q, S_{r_i}, e_i) \tag{2}$$

and for large strains

$$\epsilon = H_2(q, S_{r_f}, e_f) \tag{3}$$

where S_{r_i}, e_i are initial values in the sample and S_{r_f}, e_f are so called ultimate values that are reached after large strains (15).

Nonlinear triaxial stress-strain curves for sands and clays have been successfully approximated by a hyperbolic function of the form

$$\sigma_1 - \sigma_3 = \frac{\epsilon}{a + b\epsilon} \tag{4}$$

where a and b are coefficients to be determined (16, 17, 18, 19, 20, 21). Coefficients a and b are geometric properties of the laboratory stress-strain curve. It has been shown (15) that

$$a = h_1(S_{r_i}, e_i) \tag{5}$$

and that

$$b = h_2(S_{r_f}, e_f) \tag{6}$$

A relationship similar to equation 5 can be found elsewhere (22).

EXPERIMENTAL DETERMINATION OF COEFFICIENTS a AND b

Sample Description and Testing

Two aeolian soils, Peorian loess and Hastings silt loam, were selected for study. Typical soil properties are given in Table 1, and compaction characteristics are shown in Figure 1.

Two types of triaxial test were used to control or to measure sample variables (23). A consolidated constant water content test was used to test samples with $S_{r_i} < 100$ percent. Sample volume was measured periodically during testing (6, 7, 8) so that S_r and e could be calculated from known values of initial water content and dry density. Saturated specimens were tested in a consolidated undrained saturated test. All data were reduced by using a FORTRAN program TRIAX (23).

Correlating Functions

Hyperbolic stress-strain coefficients a and b were determined for the 17 Peorian and 12 Hastings samples tested. Semilog plots of a and b as functions of $e_i\sqrt{S_{r_i}}$ and $e_f\sqrt{S_{r_f}}$ respectively were linear as shown for a in Figure 2. Three different curves representing different compaction energies are shown in Figure 2. The slope of these curves was an exponential function of the degree of saturation at optimum moisture content S_{r_o} for each compactive effort (15). A composite equation for the a coefficient is given by

$$a = C_1 \exp (2.303\ m_1\ e_i\sqrt{S_{r_i}}) \tag{7}$$

where

$$m_1 = C_2 \exp (2.303\ m_2\ S_{r_o}) \text{ and}$$
$$C_1 = C_3 \exp (2.303\ m_3\ S_{r_o}).$$

Numerical values of C_2, C_3, m_2, and m_3 are given elsewhere (15). It was also found that $e_i\sqrt{S_{r_i}}$ and $e_f\sqrt{S_{r_f}}$ are linearly related (15).

DATA USE

Two examples of possible ways in which these correlating functions can be used are given below. When equations 4 and 7 are used together with the equation for the b parameter, triaxial stress-strain curves can be computed for the soils tested for any impact compacted sample (Proctor to modified Proctor energy). Initial and final values of e and S_r must be known for the sample.

Five identical samples of Peorian loess have initial dry densities of 100 lb/ft^3 (1.60 g/cm^3), w_i = 17.0 percent, and S_{r_o} = 76.8 percent (Figure 1). The secant modulus of each sample at $\epsilon = 0.002$ must be determined if four samples increase in water content at constant dry density as shown in Figure 1. The a and b parameters for each sample can be computed as described above (15). A plot of the secant modulus versus S_r at the final water content is shown for these samples and for five Hastings samples in Figure 3.

Subgrade moduli (secant moduli) for various strains can be generated as shown above

Table 1. Sample description.

Sample Name	Classification	Sand (percent)	Silt (percent)	Clay (percent)	ω_ℓ	I_p	G_s
Peorian loess	CH	0	72	28	50	24	2.73
Hastings silt loam	CH	1	66	33	60	38	2.68

Figure 1. Compaction curves for Peorian loess.

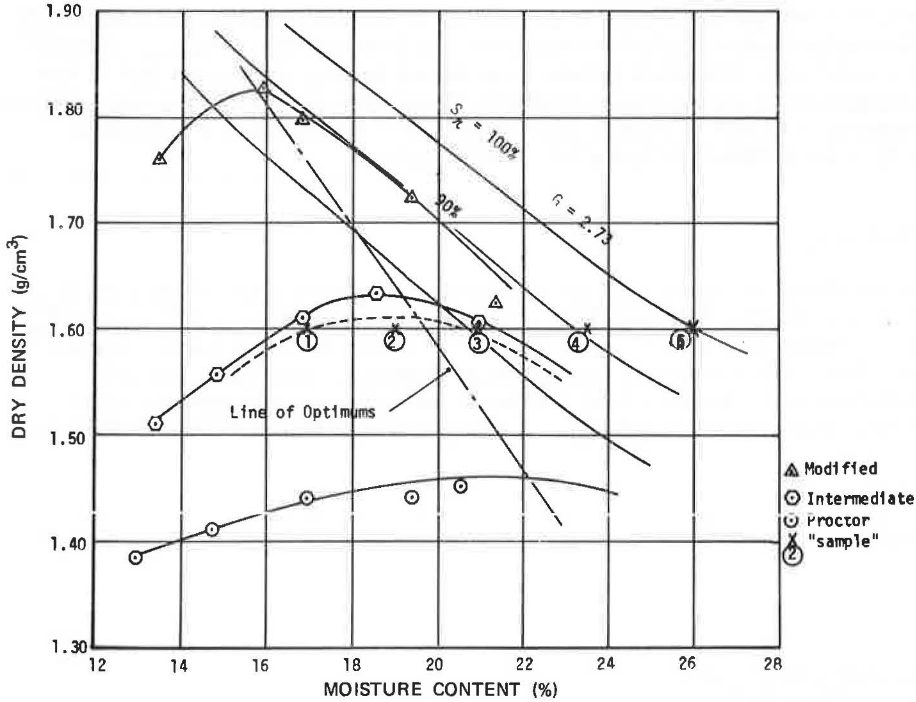


Figure 2. Correlation of coefficient a for Peorian loess.

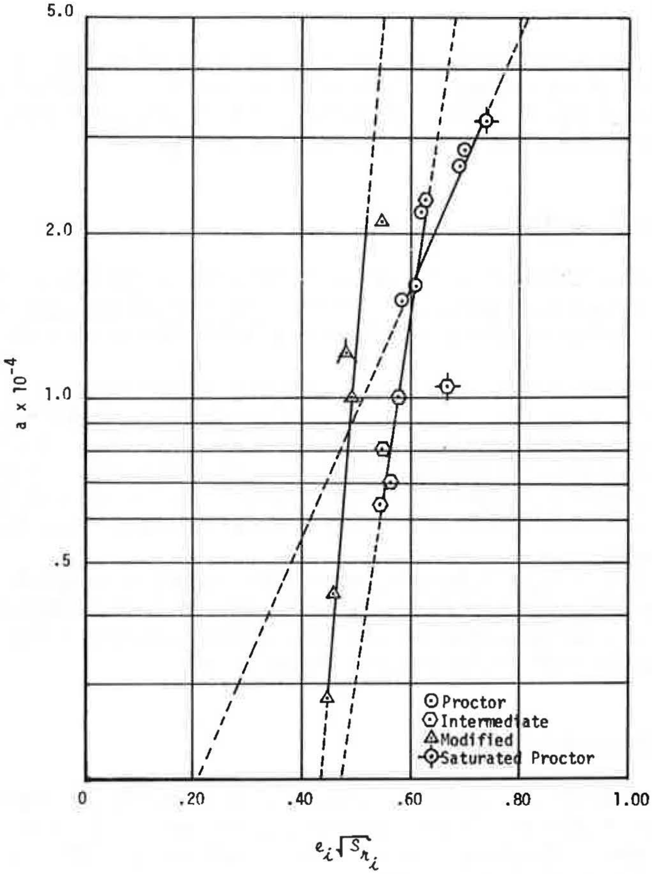
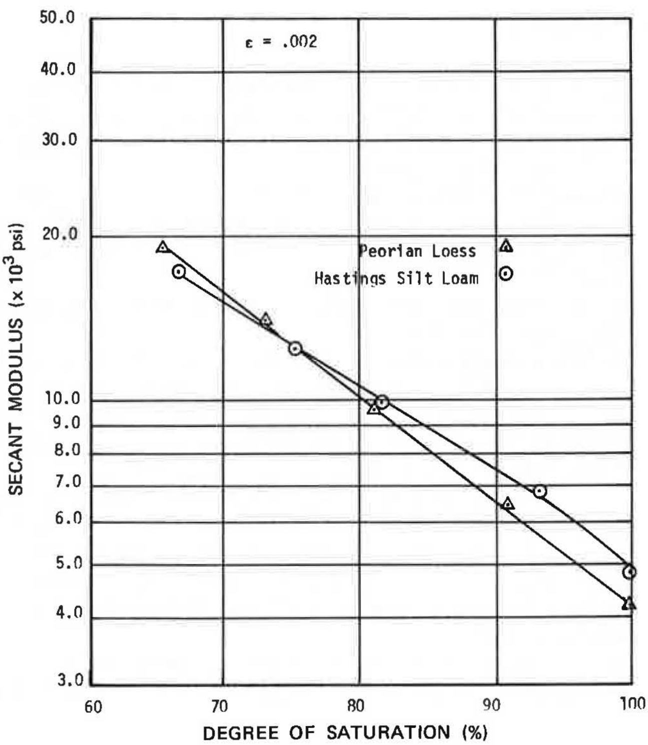


Figure 3. Secant modulus versus S_r calculated at 0.002 strain.



for use in a flexible pavement design procedure similar to the Kansas method (3). The value of the secant modulus depends on the assumed strain since soil has a nonlinear stress-strain curve. Therefore, a trial and error procedure is required (3, 24). Such a procedure has been given elsewhere (15).

CONCLUSIONS

A triaxial testing method was demonstrated for determining the stress-strain curves of compacted, partially saturated, or saturated subgrade soil capable of controlling or measuring those variables that determine static compression stress-strain behavior.

Stress-strain curves representing a wide range of densities and moisture contents for Peorian loess and Hastings silt loam from Nebraska were reasonably approximated by a hyperbolic stress-strain function. The two coefficients of this function, a and b, were nonlinear functions of S_{r_o}, $e_i\sqrt{S_{r_i}}$ and $e_f\sqrt{S_{r_f}}$. Use of these nonlinear functions produced a compact form for determining a and b and, therefore, for generating stress-strain curves over wide ranges of initial density and degree of saturation for the soils tested.

The design of flexible pavements, based on subgrade stiffness criteria, can be accomplished for a broad range of assumed in situ subgrade conditions of density and degree of saturation for the soils tested, and there is no need to test samples that have been brought to the assumed design state.

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