EFFECT OF DEGREE OF SATURATION ON COMPRESSIBILITY OF SOILS FROM THE DEFENCE RESEARCH ESTABLISHMENT SUFFIELD

by

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by
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Champaign, Illinois

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FOREWORD

The soil properties presented herein were obtained for the purpose of clarifying the stress-strain relations which should be used in computer codes for predicting ground motions due to high pressure loading. This work is in conjunction with research on propagation of ground shock through soils being conducted by the Soils Division, U. S. Army Engineer Waterways Experiment Station, for the Defense Atomic Support Agency.

This report was requested and authorized by Mr. J. G. Jackson, Jr., Chief, Impulse Loads Section, Soil Dynamics Branch, under the direction of Mr. W. J. Turnbull, Chief of the Soils Division. The report was prepared under Purchase Order No. WESBPJ-68-67, dated 16 August 1967, issued to M. T. Davisson, Foundation Engineer, Champaign, Illinois.

Directors of the Waterways Experiment Station during the performance of this work and preparation and publication of this report were COL John R. Oswalt, Jr., CE, and COL Levi A. Brown, CE. Technical Directors were Mr. J. B. Tiffany and Mr. F. R. Brown.
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## Conversion Factors, British to Metric Units of Measurement

British units of measurement used in this report can be converted to metric units as follows:

<table>
<thead>
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<th>Multiply</th>
<th>By</th>
<th>To Obtain</th>
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<td>centimeters per second</td>
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SUMMARY

The soil tests reported herein were conducted to provide information on the influence of degree of saturation on high pressure stress-strain relations of undisturbed and remolded soils from the Defence Research Establishment, Suffield. These high-pressure one-dimensional tests were also to provide input data for computer codes concerning the relation between stress and strain invariants at high pressures. Some investigators were concerned that large strains might develop at high pressures in silt and clay as has been observed for sand due to grain crushing. As expected, the test results presented herein show that large strains do not develop at high pressures in fine-grained soils such as silt and clay.

The test program consisted of 12 one-dimensional tests on 4 specimens each of undisturbed and remolded silty clay, and 4 specimens of remolded sandy silt. In all tests the radial strain was essentially zero. Axial and radial stresses and axial strain were measured. The tests were carried to an axial stress of 20,000 psi unless soil extrusion occurred at a lower stress. The following conclusions were reached:

a. The degree of saturation and the initial void ratio are the most significant variables governing the one-dimensional stress-strain relations of soil at high pressures.

b. For pressures exceeding 3000 psi the compacted specimens and undisturbed specimens of Suffield soil yield the same relation if the initial degree of saturation and initial void ratio are identical before loading.

c. A lower bound to the secant modulus of deformation $M_s$ at a given level of axial stress $\sigma_a$ is given by

$$M_s = \frac{\sigma_a}{\left(1 - \frac{S_{null}}{100} e_i\right) + \frac{\sigma_a}{300,000 \text{ psi}}}$$

for both compacted and undisturbed samples of fine-grained soil subjected to pressures greater than 3000 psi.

d. The average unloading modulus of Suffield soils subjected to pressures greater than 3000 psi is approximately 10 times the loading secant modulus of deformation $M_s$.

e. It is probable that the stiffness of the Suffield soils when unsaturated will be greater under dynamic loading than the static values given herein. Previous comparisons of static and dynamic values of constrained moduli of Suffield soil have shown that the dynamic values are twice the static values. This observation is consistent with similar comparisons for NTS Frenchman Flat silt.
PART I: INTRODUCTION

OBJECT

1. The object of this study was to determine the high-pressure, static, one-dimensional stress-strain characteristics of compacted and undisturbed soils from the Defence Research Establishment, Suffield (DRES), in Alberta, Canada. Degree of saturation was the major variable investigated. These one-dimensional tests were also to provide input data for computer codes concerning the relation between stress and strain invariants at high pressures. These high-pressure relations were especially important since some investigators were concerned that large strains might develop at high pressures in silt and clay as has been observed for sand due to grain crushing.

SCOPE

2. Two 5-in. diam undisturbed Shelby tube samples and remolded samples of two different soils, which were air-dried and passed through a No. 10 sieve, were furnished by the U. S. Army Engineer Waterways Experiment Station (WES) for this study. A total of 12 static-undrained one-dimensional compression tests were performed.

3. Specifically, four tests were performed on the undisturbed samples, and four tests were performed on each of the two remolded soils, the remolded soils were compacted at predetermined water contents and dry densities. The results of the tests are presented in the form of plots of axial stress versus axial strain, secant modulus versus axial stress, and radial stress versus axial stress. The index properties for each of the different soil samples tested, and the test apparatus, experimental procedure, and an interpretation of the test results are also presented.

* A table of factors for converting British units of measurement to metric units is presented on page viii.
PART II: SOILS INVESTIGATED

SITE CONDITIONS

Location and Topography

4. The location of the site is within the DRES blast range at a location known as Watching Hill. The site is approximately 30 miles north of Medicine Hat, Alberta, Canada. Within the area of interest, the site is essentially level with a ground surface elevation of approximately 2164.0 ft msl.

Geology

5. A brief description of the geology of the site is available in reference 2 along with an estimate of the seismic velocities for the various layers. The site is in the southern end of the Ross Depression which, along with the areas to the south and west, has apparently been covered by a large lake. The soils to a depth of 200 ft are lacustrine deposits consisting of uniform beds of clay and silt with occasional sand lenses. However, glacio-fluvial processes and desiccation have altered approximately the upper 30 ft. In reference 2 a seismic velocity of 2200 fps has been assigned to the upper 30 ft, but indications are given that the upper 4 ft may have a velocity of 700 fps while the lower 26 ft has a velocity of 2550 fps. From 30 to 200 ft, a velocity of 5500 fps is indicated.

6. Bedrock at the site consists of Upper Cretaceous beds of the Foremost formation. These beds may be arenaceous shales and/or sandstones with many coal and carbonaceous beds. In many places the "Pale Beds" overlie the Foremost formation and consist of sandstone, shales, and sandy shales. The seismic velocity for these beds has been estimated as 7500 fps. At great depth, Mississippian limestone is found with a seismic velocity of approximately 20,000 fps.

SUBSURFACE INVESTIGATION

Field Data

7. Two undisturbed samples from boring 2-U and two remolded samples from boring 5-U, ranging in depth from 0 to 22.5 ft, were furnished for this study. The undisturbed samples were taken with a 5-in.-diam Shelby tube and extruded immediately into 6-in.-diam fiberboard containers. Wax was then used to fill the containers and seal the samples. The remolded samples were air-dried, mixed, and passed through a No. 10 sieve.

8. Additional information on the soil profile at the Watching Hill site can be obtained from references 4, 5, and 6.

Laboratory Testing

9. All soil samples received in the laboratory were subjected to routine identification and classification. The test number, sample depths, description, Unified classification, Atterberg limits, and specific gravities are listed in table 1. The gradation curves for the undisturbed and remolded samples are presented in figs. 1 and 2, respectively. All index properties for the soil samples were furnished by the WES.

10. The initial weight-volume data for each of the 12 static test specimens are listed in table 2.
Fig. 1. Gradation curves for undisturbed samples.
Fig 2. Gradation curves for remolded samples
PART III TEST PROCEDURES

DESCRIPTION OF APPARATUS

11. The apparatus used for the static one-dimensional tests, shown in fig. 3, consisted of a confining ring assembly which contained the soil specimen. The confining ring was centered on the baseplate with the aid of a lucite guide ring as shown in fig 3. The piston was also centered on the soil specimen with the aid of a second lucite guide ring. A split ring was mounted on the piston and furnished a reaction for the dial indicator which measured the axial deformation to the nearest 0.001 in. A 300-kip universal Riehle hydraulic testing machine was used to apply the axial stress to the soil specimen through the

---

Fig 3 Schematic of static loading machine showing axial strain instrumentation
loading piston. A photograph of this static test machine is shown in fig. 4a, a close-up of the confining ring assembly is shown in fig. 4b.

Fig. 4. Test apparatus
12. A steel ring 1.0 in. high with 4-in. inside diameter and a wall thickness of 1.0 in. was used to confine the test specimens. An attempt was made to limit the radial strains to the minimum value required to facilitate accurate recording by use of the SR-4 gages. The output of the SR-4 gages was monitored with an SR-4 indicator. Calibrations of the confining rings were performed previously as described in reference 4.

EXPERIMENTAL PROCEDURE

Preparation of Test Specimens

13. For the tests of undisturbed samples, it was mandatory to develop a trimming operation. The trimming procedure involved placing the waxed soil sample in the hydraulic press along with the sample trimming equipment. The important feature of the trimming equipment is the trimming ring. The trimming ring has a 4-in. inside diameter, equal to that of the confining ring, but the outside face is beveled to form a sharp cutting edge. The outer face of the ring has a shoulder that fits the outside diameter of the 1.0-in.-thick confining ring. When the confining ring and the trimming ring are pressed together, an integral unit is obtained that can be forced into a soil sample in a manner similar to the use of a thin-wall sampler in a field sampling operation. Excess soil and wax were trimmed away with a knife as the trimming ring was forced into the sample. When the trimming ring had penetrated the soil a sufficient distance, the ring was carefully removed and the soil specimen was trimmed level with the height of the confining ring. Sample trimmings from each test specimen were set aside for specific gravity, Atterberg limits, and grain-size determinations. Water content samples were also taken from the Shelby tube sections before and during the trimming process.

14. The tare weight of the confining ring is known along with its dimensions. Therefore, the weight of the ring and soil specimen furnishes sufficient data to calculate the initial density of the soil. With the specific gravity and water content data, complete weight-volume determinations can be made for the test specimen.

15. The final step is to place the confining ring on the baseplate and to assemble the confining ring assembly. A height determination for the assembly is made in a dial comparator to an accuracy of 0.001 in. Because the height of the assembly itself is known, the dial comparator reading furnishes a check on the initial height of the specimen.

16. The remolded specimen was compacted into the trimming ring with a Vicksburg tamper after soil batches were properly mixed to the desired water contents and allowed to equilibrate for 24 hr. The soil specimens were compacted in two layers with nine evenly distributed blows per layer. The height of fall of the 4-lb hammer was varied to obtain the predetermined dry densities.

17. The compaction energy varied from 0.2 ft-lb/in.$^3$ to 1.1 ft-lb/in.$^3$ of soil. All remolded specimens were prepared with the compaction tamper except the sandy silt specimen at a water content of 27 percent. In order to obtain the desired density this specimen was prepared by hand-placing the soil into the confining ring. After compaction, the trimming was carried out in the same manner used for the undisturbed specimens.

Test Procedure

18. The confining ring assembly was placed in the static test machine as shown in fig. 3. The dial indicators were set at zero under the load of the piston itself which corresponds to a stress of approximately 1 psi. Succeeding loads were applied in predetermined increments and held until the dial indicator and radial stress observations were made. A similar procedure was followed during unloading, however, at zero
applied load the soil specimen was allowed to rebound for approximately 5 min. whereas the load increments required approximately 1 min for completion. All tests were loaded to the 20,000 psi stress level or soil extrusion prior to the 20,000 psi stress.

19. Upon removing the confining ring assembly from the test machine, the height was determined with the dial comparator. This reading was compared with the initial dial comparator reading and served as a check on the residual deflection. The confining ring and specimen were removed from the assembly and a careful inspection was made for extrusion before a final water content determination was made.
20. The individual test results are tabulated in figs. A1-A12. The soil index properties are presented as well as the individual test data such as axial stress, axial strain, secant modulus, and corrected radial stress. An attempt to correct the measured radial stresses has been made by dividing the load determined from a hydraulic calibration on the full height of the ring (1 in.) by the actual height of the specimen.

21. These results have also been plotted in the form of axial stress versus corrected axial strain, constrained modulus versus axial stress, and corrected radial stress versus axial stress. The data points have not been shown on the plots because none of the points deviate from the curves. The axial stress versus axial strain plots for the 12 static tests are given in figs. B1-B12. Similarly, the constrained secant modulus versus axial stress plots are given in figs. C1-C12 and the radial stress versus axial stress plots in figs. D1-D12. The boxes in the upper left corner of the figures contain initial weight-volume data for the samples.

22. A summary of the static test data is presented in table 3. For each test the initial degree of saturation is given. At the maximum axial stress the corresponding values of axial strain and the ratio of radial stress to axial stress (denoted as $K_o$) are given. A pseudo-Poisson's ratio ($\mu$) has been calculated assuming that elastic theory is applicable. The residual axial strain and the ratio of residual to maximum axial strain are also presented. A notation is made in table 3 wherever soil extrusion occurred. Otherwise, the static test results can be interpreted in a straightforward manner.

STRESS-STRAIN RELATIONS

23. A summary of the axial stress-strain relations for the undisturbed samples of silty clay, compacted samples of silty clay, and compacted samples of sandy silt are shown in figs. 5, 6, and 7, respectively. The axial stress-strain curves for all static tests were concave toward the stress axis throughout the complete loading cycle; therefore, the compressibility decreases as the stress level is increased. The absence of the small initial concave downward curvature in the stress-strain diagram of the compacted samples is believed to be caused by the negligible preload effect because of the low compaction energy necessary to yield the desired dry densities of 80-88 pcf. The unloading portions of the stress-strain curves, which are shown on the individual test plots in Appendix B, are very steep at high stress ranges, but the slope decreases at a stress of approximately 500 psi. Table 3 is a list of the maximum axial strains and ratio of the residual strain to maximum strain for all tests. The maximum strain at the peak stress of 20,000 psi varied from 0.299 to 0.457 in./in. The ratio of residual to maximum strain varied from 0.85 to 0.96 for all test specimens.

24. The stress-strain relations of the compacted or remolded samples of silty clay given in fig. 6 show the effect of the initial degree of saturation for a dry density similar to field conditions. As the initial degree of saturation increases, the strain at which the stress-strain curve turns abruptly upward is reduced because of the amount of pore air decrease. However, the stress-strain curves shift downward toward the strain axis at low stress levels for samples with increasing degrees of saturation. Thus at low stress levels the samples with a high degree of saturation are more compressible than those with a low degree of saturation. However, the wetter specimens reach 100% saturation at lower strains and become stiffer than the dryer specimens at lower strains, resulting in a crossover of the stress-strain curves as illustrated by tests 7 and 8 in fig. 6. Also, variation of the dry density at a particular initial degree of saturation indicates a more compressible soil structure at a lower dry density as shown by tests 6 and 7, fig. 6. The behavior of the compacted specimens of sandy silt is similar to that of the silty clay;
Soil Extrusion

Test 1
0 - 0.5 ft
$S_r = 58\%$

Test No. 3
1.2 - 1.7 ft
$S_r = 19\%$

Test No. 4
1.7 - 2.3 ft
$S_r = 20\%$

Fig. 5. Summary of stress-strain curves for the undisturbed samples of silty clay.
Fig. 6. Summary of stress-strain curves for the compacted samples of silty clay
Fig. 7. Summary of stress-strain curves for the compacted samples of sandy silt
however, extrusion of the specimens with a high initial degree of saturation (tests 11 and 12) distorts the true confined stress-strain curves (fig. 7). The calculations of strain at pressure saturation indicate a modification of the stress-strain curves for tests 11 and 12. The strain at saturation can be readily calculated as

\[
\epsilon = \frac{3e}{1 + e_i} = \left( \frac{S_{nt}}{100} \right) e_i
\]

where \(e_i\) is the initial void ratio, \(S_{nt}\) is the initial degree of saturation, and \(\epsilon\) is the axial strain at saturation of the specimen.

25. The stress-strain curves for the remolded samples approximate the shape of the curves for the undisturbed samples as shown in fig. 5, but the behavior is different. Tests 3 and 4 were plotted as one curve because the difference between them was not distinguishable. In general, the behavior of the undisturbed samples is similar to that of the compacted samples of silty clay except that the undisturbed samples are less compressible in the low stress ranges (less than 2500 psi).

26. The shape of the stress-strain curves (concave upward) for the undisturbed samples at the shallow depths is indicative of uncedmented soils. This behavior is contrary to the behavior at greater depths as reported previously. The data in reference 4 indicate an initial concave downward stress-strain diagram and then a change in curvature as the stress level increases. With an increase in stress, the stress-strain diagram is concave upward as the initial stiffness due to preload is destroyed.

27. A typical variation of degree of saturation with depth for the Watching Hill site is presented in fig. 6 as determined from undisturbed specimens reported in references 4 and 5. These data are given for the purpose of enabling one to select the appropriate stress-strain curve from this report that is consistent with the degree of saturation at the particular depth for which the high-pressure moduli are desired.

SECANT MODULI-STRESS RELATIONS

28. Many of the ground motion problems in protective construction can be approximated by assuming that the displacements occur in the direction of the stress-wave propagation. Under these imposed strain conditions, the constrained modulus is the significant property of the soil controlling the ground motions. A constrained secant modulus of deformation \(M_s\) is by definition the ratio of the axial stress to the axial strain under conditions of zero radial strain.

29. All the graphs of the secant modulus versus axial stress are shown in figs 11-12. The shapes of the modulus-stress curves follow directly from the changes of the stress-strain curves just examined, and thus require little additional discussion. The secant moduli vary linearly with axial stress for both the loading and unloading curve. The secant moduli are dependent on the initial degree of saturation, initial void ratio and the axial stress level. The lower bound secant modulus of deformation for stress levels above 3000 psi can be approximated by

\[
M_s = \frac{S_{nt}}{1 + e_i + \frac{a}{100}}
\]

above 3000 psi.
Fig. 8. Soil profile data from referenced reports
for the soils tested in this study. The average unloading secant modulus from the residual strain intercept is given by

\[ M_u = 10M_s \]

**AXIAL STRESS-RADIAL STRESS RELATIONS**

30. At any given stress level, the ratio of radial stress to axial stress is denoted as \( K_\theta \). In this series of one-dimensional static tests full drainage could not occur; therefore, the ratio of radial stress to axial stress determined for these tests is essentially in terms of total stresses.

31. In general, the value of \( K_\theta \) is closely related to the degree of saturation. As the degree of saturation increases, the value of \( K_\theta \) increases and approaches a value of unity for saturated soils. Because the degree of saturation depends on the axial strain, the value of \( K_\theta \) can vary continuously throughout the test. The values of \( K_\theta \) presented in table 3 vary from 0.38 to 1.00, and a pseudo-Poisson's ratio varied from 0.28 to 0.50.

32. In situ, \( K_\theta \) may be considered as unity for soils below the water table. For soils above the water table having high degrees of saturation, by capillarity or otherwise, the value of \( K_\theta \) will be nearly unity. Where the water table fluctuates, as it does at the Dror, the values of \( K_\theta \) (and secant modulus) will depend on the applied stress and the degree of saturation existing at the time a field test is performed.

33. During the unloading cycle, the radial stresses are reduced at a slower rate than the axial stress, this causes a concave downward curve that lies above the loading curve. Therefore, values of \( K_\theta \) often exceed unity during unloading.

**FORMULATION OF THREE-DIMENSIONAL STRESS-STRAIN RELATIONS**

34. The test data given in Appendix A have been used to compute: (a) octahedral shearing stress, (b) octahedral normal stress, and (c) octahedral linear strain; tabulated data for each test are presented in Appendix E. From these data, graphs have been prepared of octahedral normal stress versus octahedral linear strain; the graphs are presented in Appendix F. The relations between octahedral shearing stress and octahedral normal stress are given in Appendix G in the form of graphs.

35. A detailed discussion of the data in Appendixes F and G is beyond the scope of this report; only general comments will be made. Such data are useful in the formulation of generalized stress-strain relations for soils. For instance, the graphs shown in Appendix F show the average principal strain or octahedral linear strain which results from the average principal stress or octahedral normal stress. The slope of these curves is equal to three times that of the bulk modulus of the specimen. Note that in all instances the curves in Appendix F become very steep at some value of the strain. This value of strain is essentially that required for the soil to become fully saturated. The numerical value of the strain at this point is dependent upon the initial degree of saturation and the initial void ratio. At higher strains the bulk modulus is equal to or greater than that of water.

36. The data presented graphically in Appendix G are useful in establishing yield criteria to be used in multidimensional computer programs. Note the curves for tests 9, 10, 11, and 12 which show a nearly linear relation between octahedral shearing stress and octahedral normal stress during loading. This is not surprising since these samples were silty sand, and the shearing resistance of sand increases linearly with
normal pressure. Note that the specimens of relatively dry silt, clay, tests 3, 4, and 5, showed the same behavior because the initial degree of saturation was low and the specimens probably never became saturated. Thus the shear strength increased with pressure throughout the entire test. Note the results of test 6, 7, and 8, however, where the octahedral shearing stress approaches a constant as the octahedral normal stress increases. In each case the specimen has become saturated and is behaving as if $\sigma = 0$ beyond the pressure at which the curve turns horizontal. In each of these cases the soil is a silty clay with a relatively high initial degree of saturation.
PART V CONCLUSIONS

The following conclusions were drawn from the investigation:

1. The degree of saturation and the initial void ratio are the most significant variables governing the one-dimensional stress-strain relations of soil at high pressures.

2. For pressures exceeding 3000 psi, the compacted specimens and undisturbed specimens of Suffield soil yield the same relation if the initial degree of saturation and initial void ratio are identical before loading.

3. A lower bound to the secant modulus of deformation, $M_s$, at a given level of axial stress $\sigma_a$ is given by:

$$
M_s = \frac{\sigma_a}{1 - \frac{S_{\infty}}{100} \frac{1}{1 - e_1} - \frac{\sigma_a}{500,000 \text{ psi}}}
$$

for both compacted and undisturbed samples of fine-grained soil subjected to pressures greater than 3000 psi.

4. The average unloading modulus of Suffield soils subjected to pressures greater than 3000 psi is approximately 10 times the loading secant modulus of deformation $M_s$.

5. It is probable that the stiffness of the Suffield soils when unsaturated will be greater under dynamic loading than the static values given in this report. Previous comparisons of static and dynamic values of constrained moduli of Suffield soils (reference 4) have shown that the dynamic values are twice the static values. This observation is consistent with similar comparisons for NTS Frenchman Flat silt (reference 7).
LITERATURE CITED


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<th>Sample from Distant Plain 6</th>
<th>Depth ft</th>
<th>Description</th>
<th>Unified Classification</th>
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<th>% LL</th>
<th>% PL</th>
<th>G_s</th>
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<td>1</td>
<td>Undisturbed sample 1, boring 2-U</td>
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<td>19.9</td>
<td>37</td>
<td>19</td>
<td>2.63</td>
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<td>2</td>
<td>Undisturbed sample 1, boring 2-U</td>
<td>0.5-1.1</td>
<td>Brown silty clay with trace of sand</td>
<td>CL</td>
<td>19.1</td>
<td>38</td>
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<td>Undisturbed sample 2, boring 2-U</td>
<td>1.2-1.7</td>
<td>Brown silty clay</td>
<td>CL</td>
<td>8.3</td>
<td>42</td>
<td>21</td>
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<td>23</td>
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<td>Brown silty clay</td>
<td>CL</td>
<td>-</td>
<td>34</td>
<td>16</td>
<td>2.69</td>
</tr>
<tr>
<td>6</td>
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<td>5-22</td>
<td>Brown sandy silt</td>
<td>ML</td>
<td>-</td>
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## Table 2
Initial Specimen Data

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<th>Test No.</th>
<th>Type of Sample</th>
<th>Dry Density $\gamma_d$, dens.</th>
<th>Water Content $w_t$, %</th>
<th>Degree of Saturation $S_n$, %</th>
<th>Void Ratio $e_1$</th>
<th>Specific Gravity $G_s$</th>
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<td>86.2</td>
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<td>58.0</td>
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<td>Undisturbed, 0.5-1.1 ft</td>
<td>83.3</td>
<td>19.1</td>
<td>51.2</td>
<td>0.993</td>
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Table 3
Summary of Static Test Data

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<th>Initial Value</th>
<th>Initial Peak</th>
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<th>Residual Strain</th>
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* Soil extrusion at initial peak axial stress.
** Water extrusion.
APPENDIX A: TABULATED TEST DATA
### One-Dimensional Static Test Data

**Test No.** 1  
**Soil Type:** Undisturbed  
**Soil Location:** Parking 211, Distant Plane 6  
**w:** 19.9 %  
**S:** 58.0 %  
**Y:** 103.2 psi  
**G:** 26.2 psi  
**s:** 2.43  
**e:** 0.902  
**L:** 37 %  
**P:** 19 %  
**Classification:** Brown, Silty Clay, w/ sand & trace of organic matter (CL)

**Axial Stress** $\sigma_a$  
**Axial Strain** $e_0$  
**Secant Modulus** $M_s$  
**Radial Stress** $\sigma_r$  
**Degree of Saturation** $S_d$

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**Note:** Water Extrusion at 21,000 psi  
**Radial Stress was not recorded for this test.**

**About 69% of water was extruded during the test.**

---

**Fig. A1**
ONE-DIMENSIONAL STATIC TEST DATA

Test No ______ 2 ______ Soil Type Undisturbed Soil Location Depth 0.5 - 1.5 ft. 
Silty Clay Baring R.U. Distant Plain 6

\[ Y_1 = 191 \% S, 512 \% Y, 99.1 \text{lpcf} Y_o, 33.3 \text{lpcf} S_o, 2.6 \text{ e}, 0.973 \]
\[ L = 28 \% P_1, 18 \% \text{ Classification Brown Silty Clay w/ trace of} \]
\[ \text{Sand (CL)} \]

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<th>Secant Modulus ( M_s )</th>
<th>Axial Strain ( \sigma_r )</th>
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Axial Stress \( \sigma_a \) | Axial Strain \( e_a \) | Secant Modulus \( M_s \) | Axial Strain \( \sigma_r \) | Degree of Saturation \( S_o \)

\[ \text{Axial Stress} \sigma_a \quad \text{Axial Strain} e_a \quad \text{Secant Modulus} M_s \quad \text{Axial Strain} \sigma_r \quad \text{Degree of Saturation} S_o \]

Fig. A2
### ONE-DIMENSIONAL STATIC TEST DATA

**Test No.** 3  **Soil Type.** Undisturbed  **Soil Location.** Depth 1.2-1.7 ft

**Silty Clay.**  **Boring Z-W, Distant Plains.**

**w.** 83.7% S., 19.1% Y.. **R.** 89.2 pcf  **γ.** 77.5 pcf  **S.** 2.5  **σ.** 11.71

**L.** 42%  **P.** 21%  **Classification.** Brown Silty Clay (CL)

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Fig 13
ONE-DIMENSIONAL STATIC TEST DATA

Test No 4 Soil Type Undisturbed Soil Location Depth 1.7 - 3.3 ft.

Soil Type: Silty Clay

Soil Location: 2.4, Distant Plains

Material: 90% S., 20.1% Y, 83.1 pcf Y., 76.2 pcf S., 2.47 e., 1.280

Classification: Brown Silty Clay (Cc)

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Fig. A4
### ONE-DIMENSIONAL STATIC TEST DATA

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- **Material Properties:**
  - % S: 6.8
  - % Y: 12.5
  - % T: 700.0 pcf
  - % E: 2.5 pcf

#### Soil Sample, Distinct Plane

- **Location:**
  - Ave: Asl Scnt. Radial
  - Secant Rod: 1.2

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**Fig A5**
# ONE-DIMENSIONAL STATIC TEST DATA

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Silty Clay  Bag Samples, Distant Plain 6  

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| L | 36% P, 16% Classification Brown Silty Clay (Cc)  

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Note: Strain gages on Hg  
Confining ring yielded at an axial stress of 18,000 psi.  

Fig. A6
## ONE-DIMENSIONAL STATIC TEST DATA

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**Soil Type:** Remolded  
**Soil Location:** Depth 5 ft  
**Remark:** Bag Sample, Dusty Plain  
**Soil Sample:**  

### Test Data Table

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### Notes
- Axial load was stopped at 18,000 psi because of capacity of strain gages on confining ring.
## ONE-DIMENSIONAL STATIC TEST DATA

**Test No.**

**Soil Type** Remolded

**Soil Location** Depth 2-5 ft

**Silty Clay** Bay Sample Distance Blaine

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**Note:** Water extraction at $\epsilon = 400$ psi, a 2 gals of water per day.

Loading was stopped at 10,000 psi because of the capacity of the strong arm on the testing rig.

---

Fig. A8
# ONE-DIMENSIONAL STATIC TEST DATA

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<td>Baring Silt Distal Plain</td>
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- 21% S, 94% Y, 320 pcf, 48.2 ft, 24 ft, 2.87
- 20% P, 19% Classification: Brown Sandy Silt (MU) 

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Fig A9
**ONE-DIMENSIONAL STATIC TEST DATA**

Test No | Soil Type | Depth | Soil Location | Classification
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10 | Recalde | 5.2 | Sandy Silt | Brown Sandy Silt (mi)

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Fig. A10
## ONE-DIMENSIONAL STATIC TEST DATA

**Test No**: 11  
**Soil Type**: Remolded  
**Soil Location**: Depth 5-22 ft.  
**Sandy Silt**  
**Barry S.U. Distinct Planck**  

**Classification**: Brown Sandy Silt (Mo)

<table>
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<th>Axial Stress $\sigma_a$</th>
<th>Axial Strain $e_a$</th>
<th>Secant Modulus $M_s$</th>
<th>Radial Stress $\sigma_r$</th>
<th>Degree of Saturation $S_r$</th>
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**Axial Stress $\sigma_a$**: psi  
**Axial Strain $e_a$**: in/ln  
**Secant Modulus $M_s$**: psi  
**Radial Stress $\sigma_r$**: psi  
**Degree of Saturation $S_r$**: %

**Notes**:  
- Water extraction at $\sigma_a = 2500$ psi.  
- Degree of water extracted during test.

---

Fig. All
### ONE-DIMENSIONAL STATIC TEST DATA

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<td>Y, = 120pcf</td>
<td>$S_0 = 88.0pcf$</td>
<td>$e_0 = 0.098$</td>
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<td>L = 20% P, 19%</td>
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#### Axial Stress (psi) vs. Axial Strain (in/in) vs. Secant Modulus (psi) vs. Radial Stress (psi) vs. Degree of Saturation (%)

<table>
<thead>
<tr>
<th>Axial Stress (psi)</th>
<th>Axial Strain (in/in)</th>
<th>Secant Modulus (psi)</th>
<th>Radial Stress (psi)</th>
<th>Degree of Saturation (%)</th>
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<td>0</td>
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#### Notes
- Water extraction at...
- $f_r = 10%$: $e = 0.08$ gals of water
- Extruded during test.

---

Fig. A12
APPENDIX B  AXIAL STRESS VERSUS AXIAL STRAIN
Fig. B1. Stress-strain relation in one-dimensional compression, test 1.
Fig. B2. Stress-strain relation in one-dimensional compression, test 2
Fig B3. Stress-strain relation in one-dimensional compression, test 3
Test No. = 4
Sample = Undisturbed
Depth = 17 to 2.3 ft
$S_{s1} = 20.1\%$
$\gamma_1 = 83.1\text{ pcf}$
$\gamma_{dl} = 76.2\text{ pcf}$
$\omega_1 = 9.0\%$
$e_1 = 1.20$
$S_b = 2.69$

Fig B4. Stress-strain relation in one-dimensional compression test 4
Fig 85 Stress-strain relation in one-dimensional compression test 5
Fig B6 Stress-strain relation in one dimensional compression test.
Fig. B6 Stress-strain relation in one dimensional compression test.
<table>
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<td>$Y_{ss}$</td>
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<td>$e_i$</td>
<td>0.972</td>
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<td>$S_b$</td>
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![Graph](image)

**Fig. 5** Stress-strain relation in one dimensional compression, test 7
Fig B8 Stress-strain relation in one-dimensional compression, test 8
Fig. B9. Stress-strain relation in one-dimensional compression test 9
Fig B10 Stress-strain relation in one dimensional compression, test 10
Fig B11  Stress-strain relation in one-dimensional compression, test 11
Fig. B12. Stress-strain relation in one-dimensional compression, test 12
APPENDIX C. CONSTRUCTED MODULUS VERSUS AXIAL STRESS.
Fig C1 Relation between secant modulus and axial stress, test 1

- Test No. = 1
- Sample = Undisturbed
- Depth = 0 to 0.5 ft
- $S_{dr} = 58.0\%$
- $\gamma = 103.2$ pcf
- $\gamma_{sat} = 86.2$ pcf
- $w = 19.9\%$
- $e_s = 0.902$
- $S_s = 2.63$
Secant Modulus, $M_s$, 1,000 psi vs Axial Stress, $\sigma_0$, psi

Fig. C2. Relation between secant modulus and axial stress, test 2
Fig. C4. Relation between secant modulus and axial stress, test 4
Fig. C5. Relation between secant modulus and axial stress, test 5
Fig. C6. Relation between secant modulus and axial stress, test 6
Fig. C9. Relation between secant modulus and axial stress, test 9

Secant Modulus, $M^g$, 1000 psi

Axial Stress, $\sigma$, psi

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Fig. C10. Relation between secant modulus and axial stress test 10
Fig. C11. Relation between secant modulus and axial stress, test 11
Fig C12. Relation between secant modulus and axial stress, test 12.
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<td>$\gamma_1$</td>
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<td>86.2 pcf</td>
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**Fig. D1.** Relation between radial and axial stress, test 1.
Fig. D5. Relation between radial and axial stress, test 5

Test No. = 5
Sample = Remolded Silty Clay
$S_{ss}$ = 13.5 %
$\gamma_1$ = 90.0 pcf
$\gamma_{ss}$ = 85.8 pcf
$w_1$ = 48 %
$\phi_1$ = 0.960
$S_b$ = 2.69
Fig. D7. Relation between radial and axial stress, test 7
Fig. D8. Relation between radial and axial stress, test 6
Fig. D9. Relation between radial and axial stress, test 9
Fig. D10  Relation between radial and axial stress, test 10
Fig. D11. Relation between radial and axial stress, test 11

Sample: Remolded Sandy Silt

$S_i$: 45.0 %
$Y_i$: 101.8 pcf
$Y_{d1}$: 68.5 pcf
$w_i$: 15.0 %
$e_i$: 0.888
$S_i$: 2.67
APPENDIX E: TABULATED DATA FOR THREE-DIMENSIONAL STRESS-STRAIN RELATIONS
### ONE-DIMENSIONAL STATIC TEST DATA

**Test No.** 1  
**Soil Type:** Undisturbed  
**Soil Location:** Depth 0.00 ft  
**Silty Clay**  
**Barry 2U, Distant Plan 6**

- **Silt %:** 18.9  
- **Sand %:** 38.0  
- **Clay %:** 42.2  
- **Soil:** 0.36  
- **m.s.:** 1.43  
- **e:** 0.902  
- **Dense:** 37  
- **Sand:** 19  
- **Classification:** Brown Silty Clay w/ sand & trace of organic matter (CL)

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*Note: Radial Stresses were not recorded for this test.*
ONE-DIMENSIONAL STATIC TEST DATA

Test No 2  Soil Type Undisturbed Soil Location Depth 0.5 - 1.1 ft

Silty Clay  Sawing 2-V, Distant Plan B

Z  19.1 % S.  81.2 % 7.  99.1 pcf  $0.83.3 pcf  $ 2.66  e  0.993

L  38 % P  18 % Classification Brown Silty Clay w/ trace of sand (CL)

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<th>$\sigma_0 + 2\sigma_0$</th>
<th>$\sigma_0$</th>
<th>$\sqrt{\frac{\sigma_0 - \sigma_1}{3}}$</th>
<th>Radial Stress $\sigma_r$</th>
<th>Degree of Saturation S</th>
<th>$\sigma_0 + 2\sigma_0$</th>
<th>$\sigma_0$</th>
<th>$\sqrt{\frac{\sigma_0 - \sigma_1}{3}}$</th>
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<td>psi</td>
<td>psi</td>
<td>%</td>
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Fig. E2
# One-Dimensional Static Test Data

**Test No:** 3  
**Soil Type:** Undisplaced  
**Soil Location:** Depth 1.2-1.7 ft  
**Silty Clay**  
**Bearing K-V, Distinct Plane**

- ** Moisture:** 9.3\%  
- **S:** 19.1\%  
- **R:** 2.4\%  
- **K:** 9.75\%  
- **S:** 2.69\%  
- **e:** 1.170

**Classification:** Brown Silty Clay (CL)

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Fig. E3
ONE-DIMENSIONAL STATIC TEST DATA

Test Nr: 4  Soil Type: Undisturbed  Soil Location: Depth 17-23 ft
             Silty Clay  Boring 24, Distant Plain

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L = 44 % P, 23 % Classification: Brown Silty Clay (C2)

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Fig. E4
### One-Dimensional Static Test Data

**Test No:** 5  
**Soil Type:** Remolded  
**Soil Location:** Depth 0 - 5 ft  
**Classification:** Brown Silty Clay (CS)

#### Silt Clay Bag Sample, Distinct Plain

- **p<sub>r</sub> = 4.9**  
- **s<sub>r</sub> = 12.5**  
- **7.9**%  
- **85.8**%  
- **s<sub=r</sub> = 2.69**  
- **e = 0.260**

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| 81 | 0.021 | 14 | 7%
| 137 | 0.038 | 49 | 106%
| 187 | 0.048 | 80 | 31%
| 275 | 0.057 | 130 | 225%
| 391 | 0.065 | 147 | 297%
| 570 | 0.063 | 205 | 365%
| 663 | 0.068 | 238 | 495%
| 1000 | 0.078 | 353 | 750%
| 134 | 0.080 | 410 | 1001%
| 673 | 0.089 | 585 | 260%
| 980 | 0.095 | 722 | 1470%
| 2300 | 0.099 | 849 | 700%
| 2633 | 0.102 | 967 | 1950%
| 3280 | 0.108 | 117 | 2420%
| 3913 | 0.111 | 147 | 2870%
| 4572 | 0.114 | 178 | 3360%
| 5213 | 0.117 | 197 | 3820%
| 5867 | 0.119 | 2220 | 4300%
| 6580 | 0.121 | 2420 | 4810%
| 713 | 0.123 | 2750 | 5710%
| 7753 | 0.125 | 3005 | 6630%
| 8360 | 0.126 | 3280 | 5040%
| 9000 | 0.127 | 3540 | 5500%
| 9667 | 0.128 | 3770 | 1090%
| 10333 | 0.130 | 4000 | 7500%

Fig. E5
ONE-DIMENSIONAL STATIC TEST DATA

Test No. 6  Soil Type Remolded  Soil Location Depth 0.5 ft
Silty Clay  Bag Sample Dist.  Plain E
Wt. 2.7 % S., 25.3 % Y, 66.8 % Y_o 77.2 % Y_s 3.6 % a. 1.120
L. 34 % P. 14 % Classification Brown Silty Clay (6)

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*Fig. E6*
### ONE-DIMENSIONAL STATIC TEST DATA

**Test No.** 7  **Soil Type:** Remolded  **Soil Location:** Depth 0.5ft

**Silty Clay**  **Big Sample, Distant Plume**

| L | 98% | S | 27% | Y | 98.5% | D | 88% | P | 14% | Classification: Brown Silty Clay (CL) |

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### ONE-DIMENSIONAL STATIC TEST DATA

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**Soil Location:** Depth 0-5 ft  
**Classification:** Silty Clay  
**Bag Sample:** Disturb Plain

- **Soil:** 14% G.  
- **Percent:** 24% P.  
- **Classification:** Brown Silty Clay (C.)

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Fig. E8
### ONE-DIMENSIONAL STATIC TEST DATA

**Test No:** 9  
**Soil Type:** Remolded  
**Soil Location:** Depth 5-22 ft  

**Soil Characteristics:**  
- Cross Sections: 31% S, 94% Y, 92% def. %, 890% def. %, 2.67% e, 0.67%  
- Classification: Brown Sandy Silt (ML)

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Fig E9
**ONE-DIMENSIONAL STATIC TEST DATA**

Test No 10  
Soil Type: *Beach Sand*  
Soil Location: *Depth: 5-62 ft*  
Classification: *Brown Sandy Silt (ML)*

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Fig. E10
# One-Dimensional Static Test Data

Test No: 11  Soil Type: Tonnalced  Soil Location: Depth: 5.27 ft

- Sandy Silt  Barren, S-H, Tidewater Plain

- w: 15.0 %  S: 45.0 %  Y: 126.5 psi  S: 8.55  e: 0.80
- L: 20 %  P: 10 %  Classification: Brown Sandy Silt (ML)

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Fig E11
### ONE-DIMENSIONAL STATIC TEST DATA

**Test No.** 17  
**Soil Type:** Amended  
**Soil Location:** Depth 5-23 ft  
**Sandy Silt**  
**Boeing 34 Drilled Danilo**  

- 57" S., 823" T., 17.6% S.  
- 8800 ft, 59% S, 2.4"  
- 0.898  
- 20% P., 19% Classification: Brown Sandy Silt (M.D.)

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Fig. E12
APPENDIX F  OCTAHEDRAL NORMAL STRESS VERSUS OCTAHEDRAL LINEAR STRAIN
NOTE RADIAL STRESSES WERE NOT RECORDED FOR TEST NO. 1

TEST NO 3
12-17 FT
$S_r = 19\%$

TEST NO 4
17-23 FT
$S_r = 20\%$

TEST NO 2
0.5-11 FT
$S_r = 31\%$

Fig. F1. Octahedral normal stress versus octahedral linear strain for the undisturbed samples of silty clay
Fig. F2. Octahedral normal stress versus octahedral linear strain for the compacted samples of silty clay
Fig. 17: Octahedral normal stress versus octahedral linear strain for the compacted samples of sandy silt
APPENDIX G: OCTAHEDRAL SHEARING STRESS VERSUS OCTAHEDRAL NORMAL STRESS
NOTE: RADIAL STRAINS WERE NOT RECORDED FOR TEST NO 1.

Fig G1: Octahedral shearing stress versus octahedral normal stress for the undisturbed samples of soil clay.
Fig. G2. Octahedral shearing stress versus octahedral normal stress for the compacted samples of silty clay.
Fig G3. Octahedral shearing stress versus octahedral normal stress for the compacted samples of sandy silt.
EFFECT OF DEGREE OF SATURATION ON COMPRESSIBILITY OF SOILS FROM THE DEFENCE RESEARCH ESTABLISHMENT, SUFFIELD

Soil tests were conducted to provide information on the influence of degree of saturation on high-pressure stress-strain relations of undisturbed and remolded soils from the Defence Research Establishment, Suffield, and to provide input data for computer codes concerning the relation between stress and strain invariants at high pressures. As expected, the test results presented herein show that large strains do not develop at high pressures in fine-grained soils such as silt and clay. The test program consisted of 12 one-dimensional tests on 4 specimens each of undisturbed and remolded silty clay, and 4 specimens of remolded sandy silt. In all tests the radial strain was essentially zero. Axial and radial stresses and axial strain were measured. The tests were carried to an axial stress of 20,000 psi unless soil extrusion occurred at a lower stress. The following conclusions were reached: The degree of saturation and the initial void ratio are the most significant variables governing the one-dimensional stress-strain relations of soil at high pressures. For pressures exceeding 3000 psi the compacted specimens and undisturbed specimens of Suffield soil yield the same relation if the initial degree of saturation and initial void ratio are identical before loading. A lower bound to the secant modulus of deformation $M_s$ at a given level of axial stress $p_a$ is given for both compacted and undisturbed samples of fine-grained soil subjected to pressures greater than 3000 psi. The average unloading modulus of Suffield soils subjected to pressures greater than 3000 psi is approximately 10 times the loading secant modulus of deformation $M_s$. It is probable that the stiffness of the Suffield soils when unsaturated will be greater under dynamic loading than the static values given herein. Previous comparisons of static and dynamic values of constrained moduli of Suffield soils have shown that the dynamic values are twice the static values. This observation is consistent with similar comparisons for NTS Frenchman Flat silt.
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