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EFFECTS OF SOIL SUCTION ON TRIAXIAL SHEAR TESTS OF CLAY

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INTRODUCTION

Construction of the Grand Coulee Third Powerplant required considerable alteration of existing slopes on the right (east) bank of the Columbia River just downstream from the original concrete dam (fig. 1).

Overburden soils in this area are in the Nespelem formation [1, 2]*; and the clay strata of this formation are preconsolidated, varved, and relatively dense. The clay strata are overlain and underlain by sand and gravel and also contain lenses of sand and gravel [3].

Changing pore-water pressure in the clays was the source of slope instability, with the most recent extensive slope instability following high flooding in 1948. At that time, stability was restored by drilling drainage wells and placing riprap on a large area at the toe of the slope near the river's edge.

An additional problem anticipated from construction of the third powerplant was the expected fluctuation of the river level, which was scheduled to be as much as 20 feet (6.1 m) during power generation operations.

Laboratory studies were performed to determine the strength of the highly plastic clays in an undisturbed state. This report discusses one typical clay sample. The preconsolidated state of the clays presented unusual problems in selecting testing procedures appropriate to test specimen conditions caused by unavoidable sample disturbance. Stress and volume change that occurred during sampling was also considered in interpreting test data. These factors and moisture content fluctuations due to generator operation may be different than conditions during laboratory testing because of the availability of water to the soil and the difference in time factors.

The overconsolidated, saturated, highly plastic varved clays encountered in the area of the Grand Coulee Third Powerplant presented challenges in triaxial shear testing that were related to stress relief and subsequent development of soil suction in test specimens, often of a magnitude of several atmospheres. Although the 6-inch (15-cm) diameter Pitcher-type samples available for testing were as nearly undisturbed as possible, stress relief began at the instant of sampling in the field; and the soil suction increased as smaller hand-cut specimens were prepared in the laboratory. Thus, the specimens tested were not under the same stress conditions as the in-place clays, nor is it probable that the in-place clays below the water table would become similarly

*Numbers in brackets refer to entries in the bibliography.

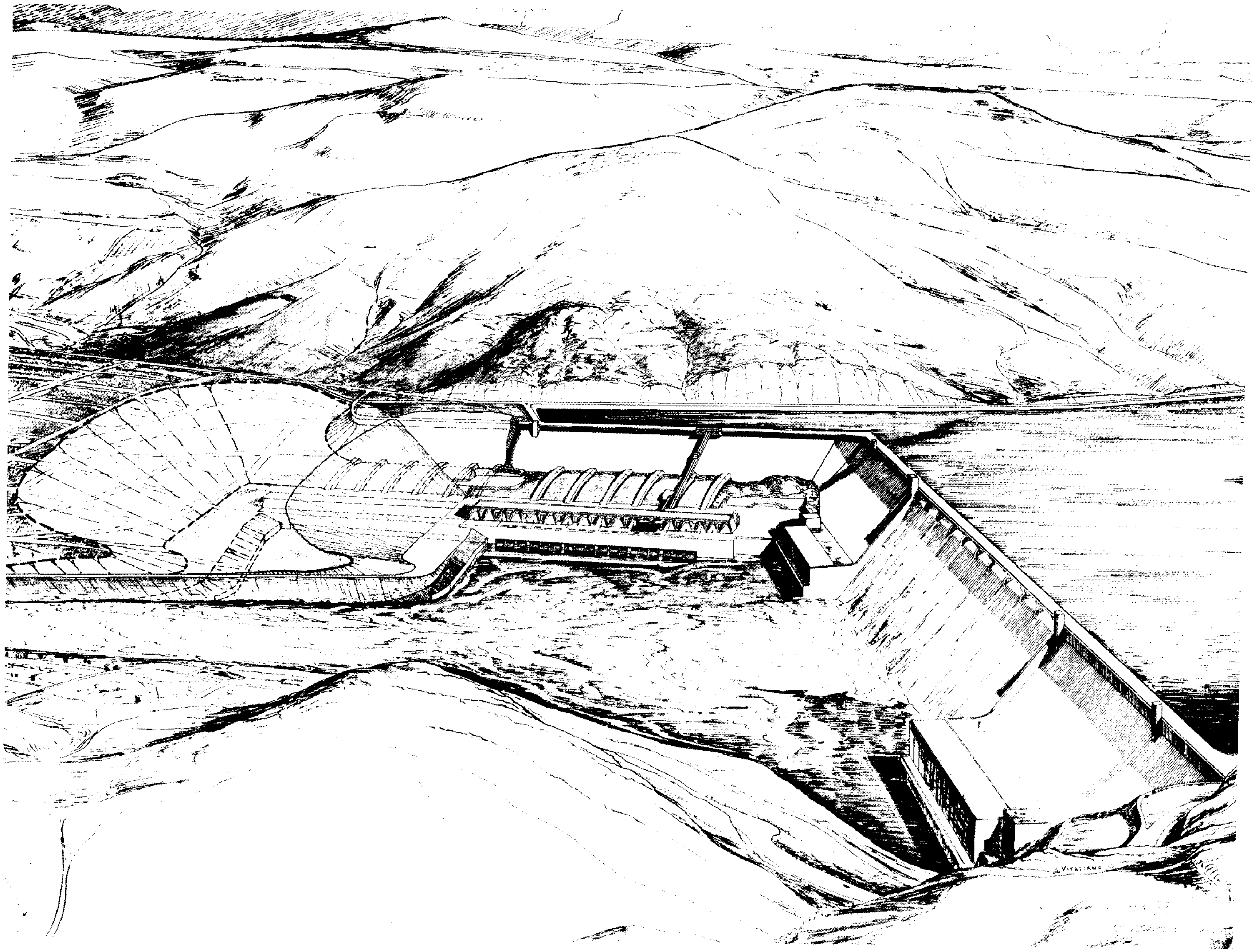


Figure 1.—Grand Coulee Third Powerplant, Columbia Basin Project, Washington (P1222-D-63800AA).

stressed upon unloading during excavation since the much slower reduction in overburden load would be accompanied by a moisture content increase in the soil.

A study of the relationship of soil suction, volume change, and water intake with changes in stress was made using a sealed specimen in a one-dimensional consolidometer equipped to measure soil suction. Two series of triaxial shear tests were also performed to evaluate soil suction effects on the soil strength parameters: (1) series 1, regular CU (consolidated-undrained) tests performed on specimens at their moisture content as received in the laboratory; and (2) series 4, special test specimen preparation by reloading to approximate preconsolidation loads, then unloading in increments while allowing an increase in moisture content before CU triaxial shear testing. Tests were continued to high strains to provide a basis for evaluation of residual strength values.

TESTING PROGRAM

Soil identification tests were performed on each undisturbed 6-inch (15-cm) diameter sample received from an extensive field investigation program. Selection of samples for the detailed one-dimensional consolidation, triaxial shear, and other related tests was made on the basis of soil location and classification. The critical characteristics of low density, high moisture content, and high plasticity were emphasized in sample selection for detailed studies. The samples available for testing were very near saturation; therefore, CU triaxial shear tests were considered appropriate to determine the soil shear strength [3]. These tests conducted with moisture contents as received in the laboratory are referred to as series 1. The clays were petrographically examined to provide a basis for evaluating swelling potential.

Early in the test program, preconsolidation of the clays was evident from one-dimensional consolidation test data. Preconsolidation pressures, P_c, of up to 450 lbf/in^2 (3103 kPa) were indicated when analyzed using Schmertmann's procedure [4]. The clays in situ still remain under considerable overburden, over 250 feet deep [76 m] (P_o = 130 lbf/in^2 [896 kPa]) in some locations. During initial triaxial tests, it was noted that these clays tended to expand (largely a result of stress relief), and to develop large soil suction pressures (about 5 atmospheres [507 kPa]) even though the clays were at a high degree of saturation, 90 percent or more [5, 6]. The soil suction influences the soil strength, as determined by the triaxial shear test, by indicating a sizable cohesion intercept. In an effort to minimize the soil suction effects, a second series of triaxial shear tests was conducted on test specimens that were wetted. For the second series of tests, the triaxial shear specimens were loaded in increments under drained conditions until the confining pressure, σ_3, was 200 lbf/in^2

(1379 kPa), approximately the preconsolidation pressure, P_c. After consolidation was complete, the specimens were unloaded in increments with free water available to the specimen. The unloading increments were adjusted to maintain the net suction pressure to less than 5 to 7 lbf/in^2 (34.5 to 48.3 kPa) so that cavitation would be prevented in the water supply lines. When the water intake and soil expansion were essentially complete, either CU or UU (unconsolidated-undrained) triaxial shear tests were performed. These tests are referred to as series 4. About 20 specimens were tested in triaxial shear. Sample No. 48B-336 is typical of the Nespelem clays and is discussed in detail.

Further studies were made to provide an understanding of the stress relief, suction, and swell characteristics of these clays. The apparatus used was the floating ring consolidometer [3], which was designed to minimize the effects of temperature variation on volume-change measurements. The high air-entry value, saturated, ceramic disk set in the bottom end plate with the small air source offset to one side provided a capability for measuring negative pore-water pressure, u_w, of −10 atmospheres (−1013 kPa) or more. The test procedure was an adaptation of that outlined for isotropic compression by Gibbs [5], Knodel and Coffey [6], and Gibbs and Coffey [7]. The top end plate was a coarse, low air-entry (less than 5 lbf/in^2 [34.5 kPa]) ceramic disk, dry, and under the same air pressure as the bottom interface to minimize consolidation of the soil specimen during this initial phase of the test. The same apparatus and test specimen were used to determine the water intake (phase 2) and specimen swell (phase 3) resulting from unloading.

The "undisturbed" triaxial shear test specimens were resaturated by back pressuring in a triaxial cell; but even under anisotropic loading conditions, the horizontally varved structure of these clays appeared to be disturbed by this procedure.

FLOATING RING CONSOLIDOMETER

Original work with the floating ring consolidometer was reported in the Bureau's contribution to the 1960 Shear Conference at Boulder, Colorado [3]. This apparatus (fig. 2) is similar to a standard one-dimensional consolidation apparatus except that pore-pressure measuring systems can be controlled to measure the pore-air and pore-water pressures. The latter included capillary effects that can range in value to −10 atmospheres (−1013 kPa). Since the test specimen was sealed, soil specimen drainage or water inflow could also be controlled. The consolidometer specimen container as sketched and pictured on figures 3 and 4 has three parts: base, top, and sealing collar. The surface of the base on which the soil rested had a fine ceramic disk connected by a

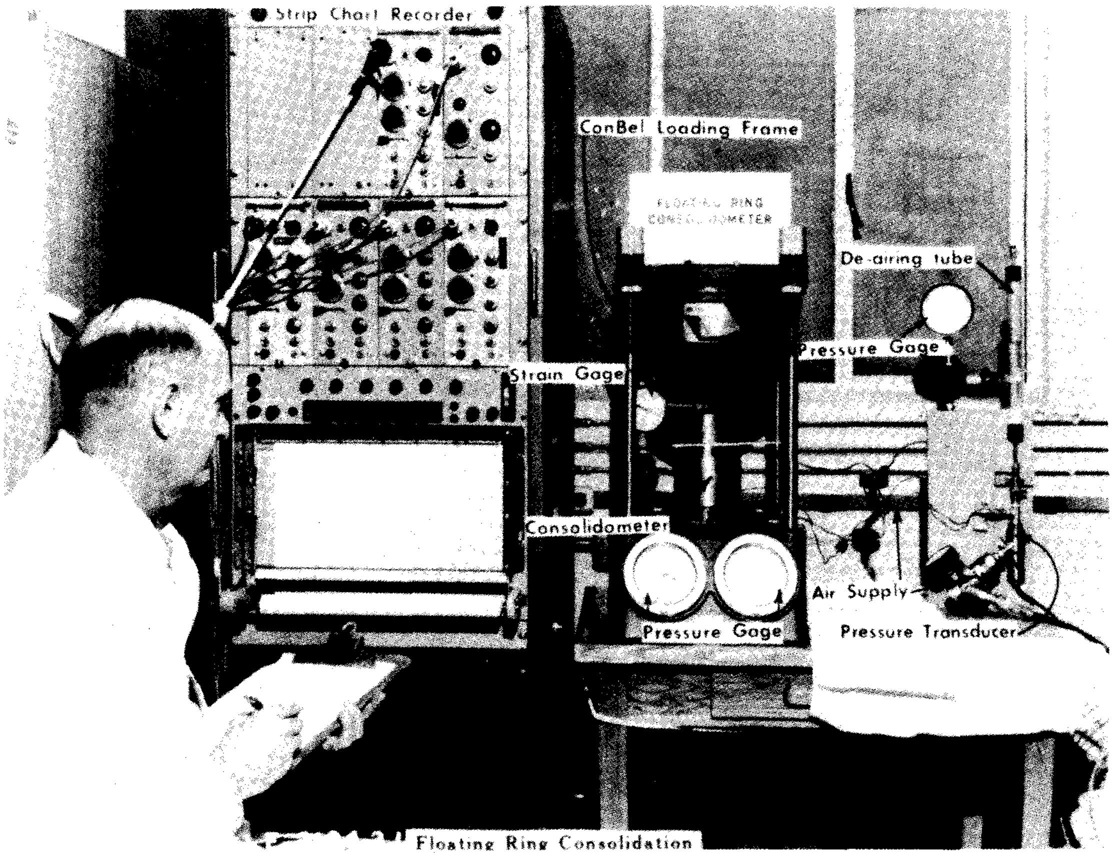


Figure 2.—Floating ring consolidometer test apparatus.

de-aired water source to an absolute pressure transducer. Air pressure, to control the measuring system, was induced through a 1/32-inch (0.08-cm) diameter hole set to the side and isolated from the ceramic disk. The top had a relatively coarse ceramic disk next to the soil with connections suitable to either (1) apply an air pressure equal to those applied to the base, or (2) bleed off air pressure during some phases of the test. The test assembly was accomplished by placing the soil specimen on the base, and positioning the top and collar. The seal was effected by O-rings between the collar and both the base and top. This assembly was then placed in a standard one-dimensional consolidation loading frame. During the assembly, nearly continuous readings were made of the suction developed in the base ceramic disk; and precautions were taken to keep the suction to less than -10 lbf/in^2 (-69 kPa). A suction greater than this caused cavitation of the water column to the pressure transducer. The effect of friction on the O-ring seals was directly determined by introducing air pressure into the sealed part of the apparatus and measuring the load on the consolidometer load measuring cell.

A test in the floating ring consolidometer included several parts or phases. Phase 1 was the determination of the initial soil suction pressure. This was done by applying air back pressure to

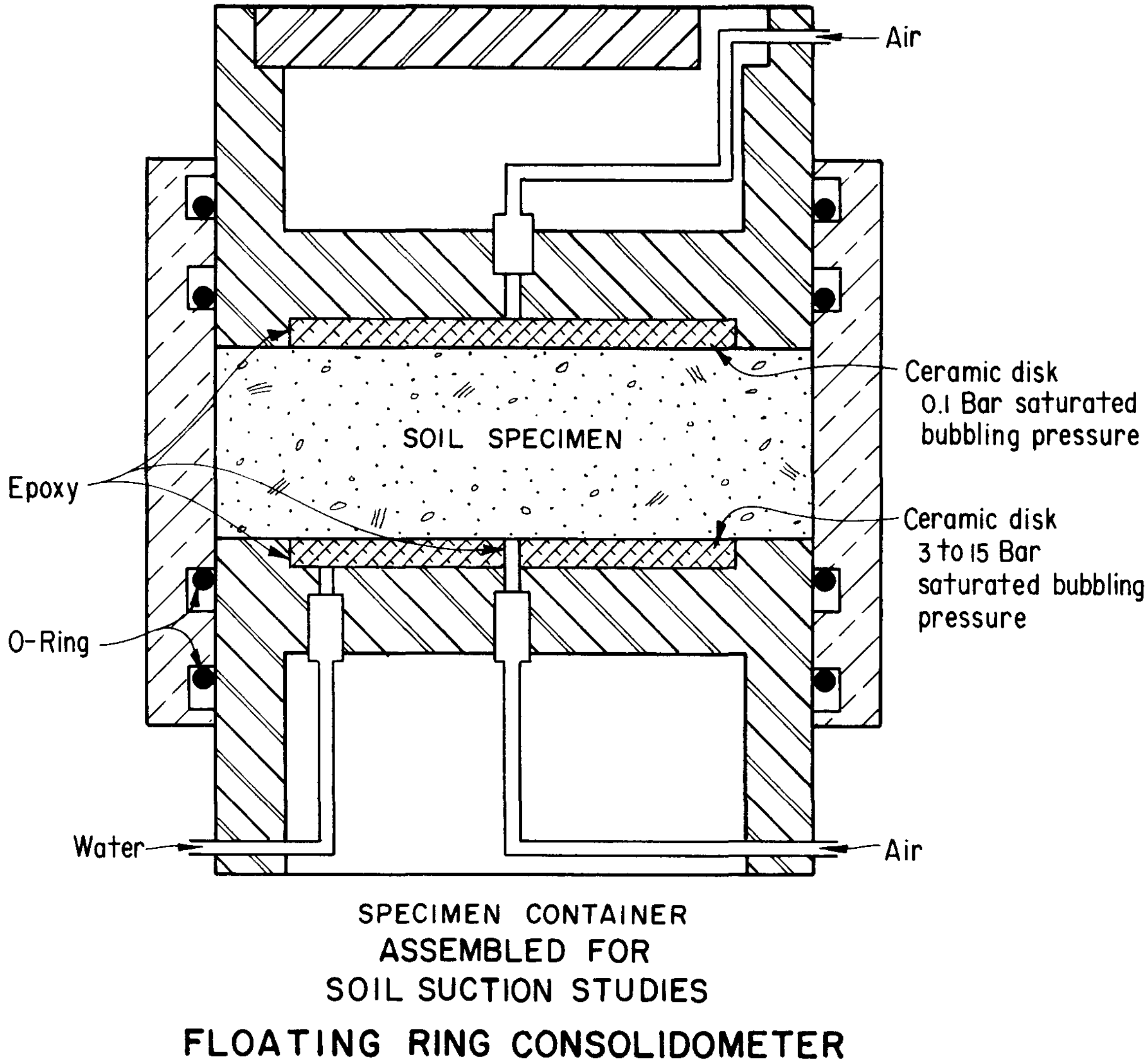


Figure 3.—Floating ring consolidometer container assembly sketch.

the ceramic disk-soil interfaces, usually in increments of about 5 lbf/in² (34.5 kPa) as the menisci developed. A plot of this test phase is shown on figure 5a.

The second phase of the test was to determine the pore-water pressure change with volume change of the test specimen induced by an increase in the loading of the specimen. This phase was accomplished by sealing the specimen (with the elevated air pressure) and increasing the vertical load. This increase resulted in an increase in the pore-water and pore-air pressure and a decrease in the specimen volume. Direct measurement of the pore-air pressure, u_a, and the pore-water pressure, u_w, allowed computation of the equivalent capillary pressure, u_c, according to the equation:

$$u_c = -(u_a - u_w) \tag{1}$$

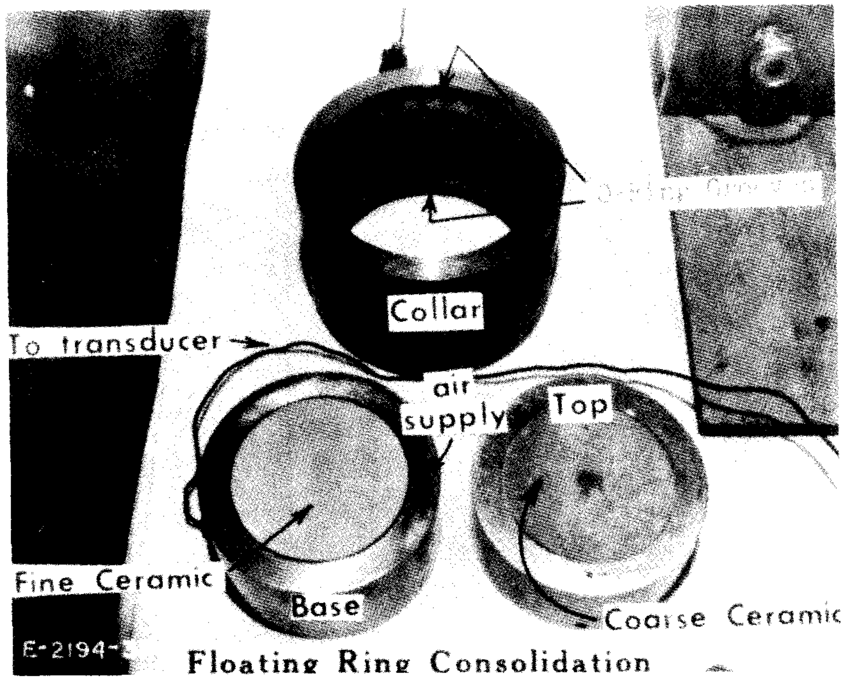


Figure 4.—Floating ring consolidometer test specimen container.

A plot of u_c values as ordinate and percent volume change as abscissa is shown on figure 5b.

The third phase of the test was to determine the amount of water taken into the specimen upon rebounding from the saturated condition at the end of phase 2. At the end of phase 2 of the test, the air pressure was elevated. But since the soil specimen was loaded and the soil suction was near zero, it was possible to bleed off the air pressure without danger of cavitating the measuring system. When this was accomplished, water was allowed to enter the specimen and the load was released in increments. The water intake, expressed as a percentage of the initial specimen volume, was plotted with relation to the specimen swell (fig. 5c). With total specimen swell, final specimen volume exceeded initial specimen volume. This shows that soil suction acts as an effective stress and inhibits the stress relief and swell of the laboratory test specimen.

TEST RESULTS

Sample No. 48B-336 was a varved, highly plastic, overconsolidated clay; its physical properties are summarized in table 1. The clays represented by this sample, although very fine grained with nearly 65 percent finer than 0.002 mm (78.7×10^{-6} in), showed activities of greater than 0.75

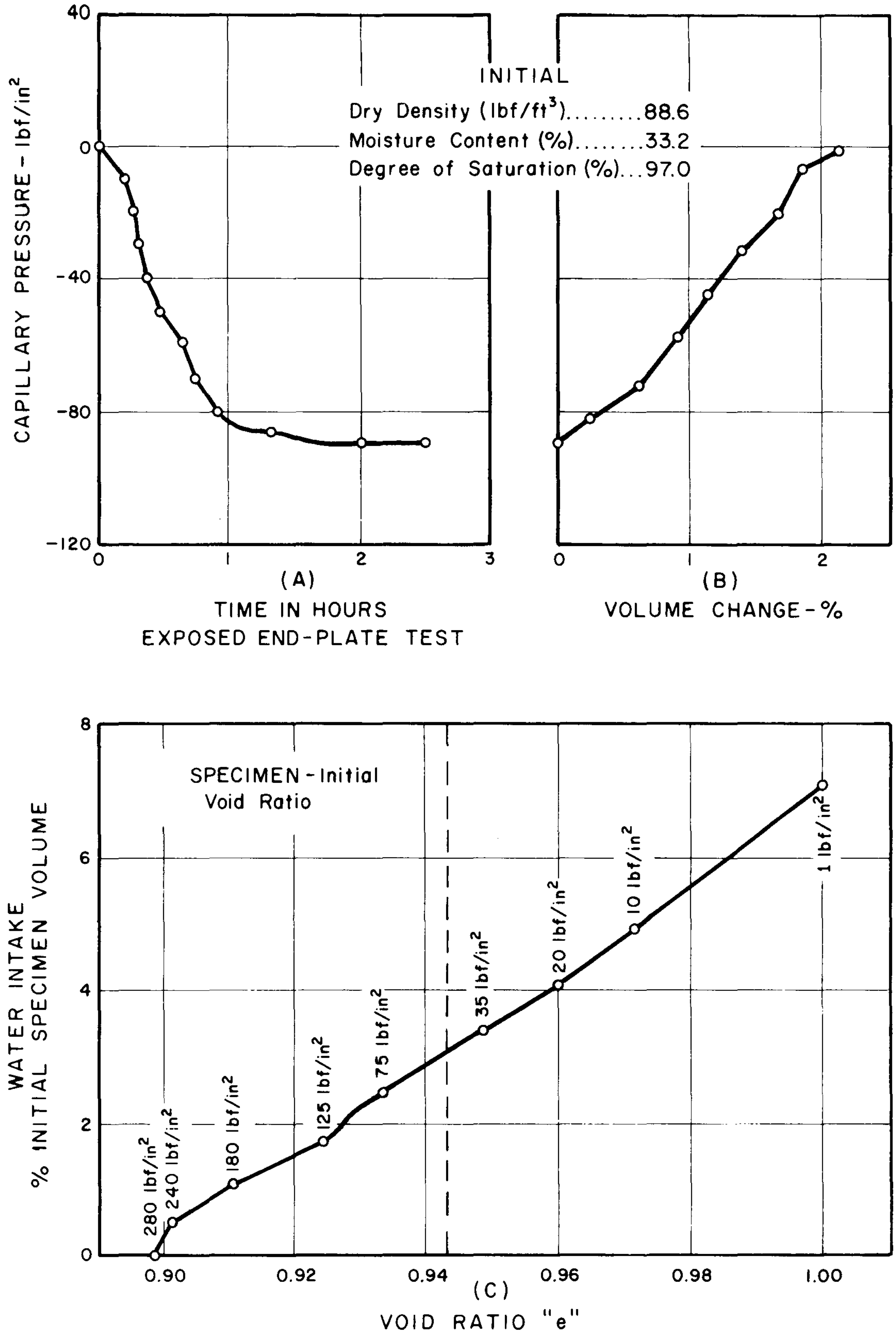


Figure 5.—Water intake and volume change. [To convert from lbf/in² to kPa, multiply lbf/in² × 6.894].

Table 1.–Test values for overconsolidated, varved clay – sample number 48B-336.

Sample depth	220 feet (67 m)
Water table depth	90 feet (27 m)
Liquid limit	72 percent
Plastic limit	24 percent
Plasticity index	48
Shrinkage limit	20 percent
Relative consistency	83 percent
−0.001 mm	45 percent
−0.002 mm	64 percent
Activity	0.75 percent
Initial dry density*	85.8 lbf/ft^3 (1374 kg/m^3)
Initial moisture content*	35.5 percent
Initial void ratio	1.0174
Present overburden pressure	150 lbf/in^2 (1034 kPa)
Preconsolidating pressure	360 lbf/in^2 (2482 kPa)
Overconsolidation ratio	1.5
Specific gravity	2.77
Degree of saturation*	97.2
Initial equivalent capillary pressure	−89.6 lbf/in^2 (−617.8 kPa)

	Triaxial shear test values					
	Maximum stress			Residual		
	Φ' (degrees)	Cohesion (lbf/in^2)	Cohesion (kPa)	Φ' (degrees)	Cohesion (lbf/in^2)	Cohesion (kPa)
Series 1	13.5	18.1	124.8	8.0	12.0	82.7
Series 4	20.8	6.3	43.4	18.2	3.2	22.1

*Average of eight test specimens.

and would be classed as inactive [4, 8]. Shrinkage limit values ranged from 20 to 22 percent, and petrographic examination indicated a soil with very low expansive potential. The relative consistency of the undisturbed clays was in the range of 80 percent or more, indicating a nonsensitive condition. The one-dimensional consolidation test data (fig. 6) indicated the preconsolidated condition of these clays. The preconsolidation load, P_c, for the sample discussed was 360 lbf/in^2 (2482 kPa). The range of P_c for all samples tested was from 250 to 450 lbf/in^2 (1724 to 3103 kPa) with the larger number of samples showing data that would favor the higher values and subsequent higher overconsolidation ratios than the 1.5 to 3.5 range. This range appears to have been influenced more by the degree of unavoidable disturbance during sampling than by depth of individual samples. The significance of the preconsolidation was evident in the resulting stress relief at

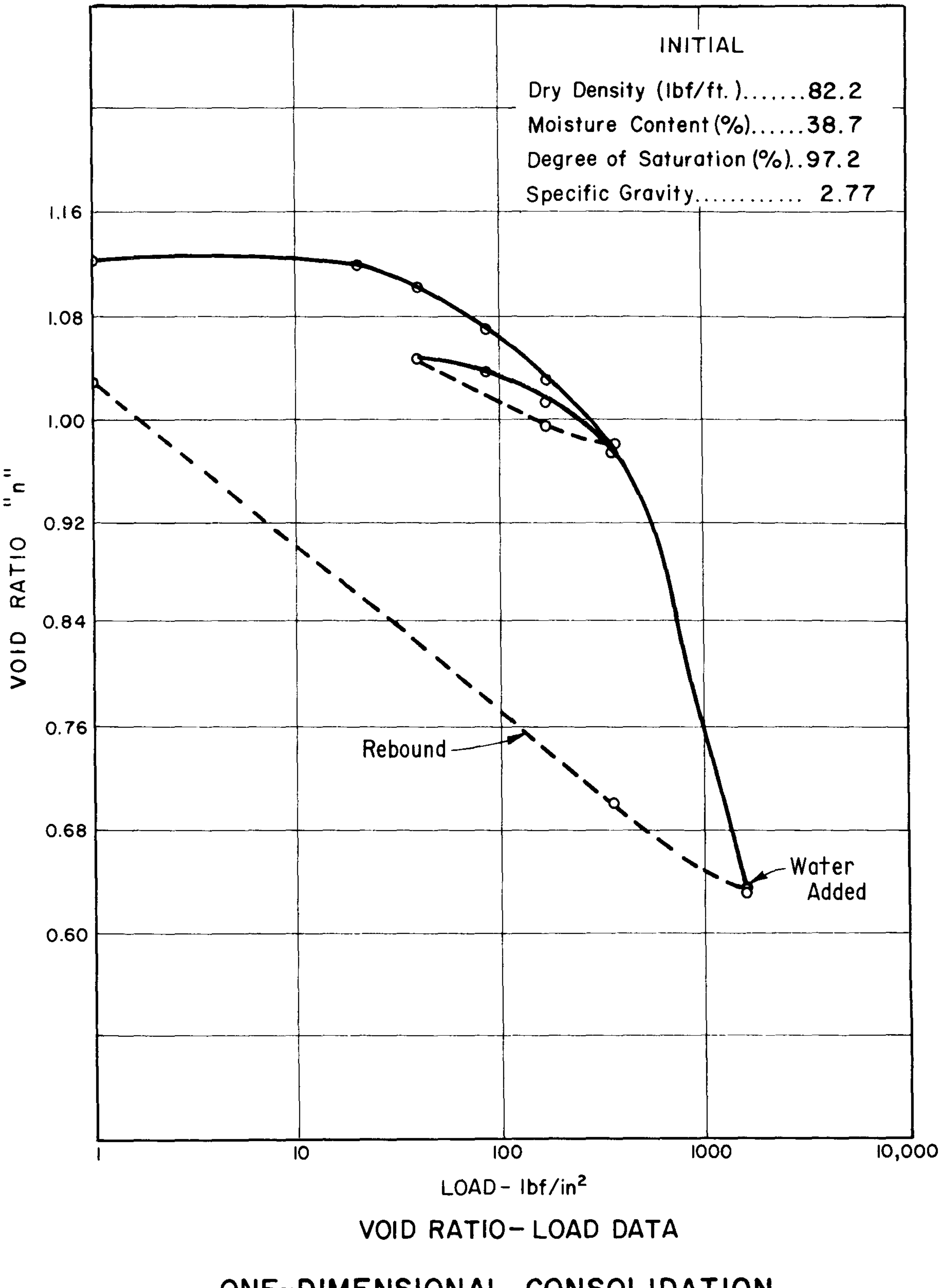


Figure 6.—One-dimensional consolidation void ratio-load data. [To convert from lbf/in^2 to kPa, multiply lbf/in^2 × 6.894].

constant moisture content, which was accompanied by the development of soil suction of several atmospheres in the laboratory triaxial shear test specimens. These factors were, however, considered during interpretation of laboratory shear strength. The interpretation was further complicated because the soil in-place rebounds during excavation with free water available, which limits suction development in the pore water.

Initial triaxial shear tests on these soils in which the average lateral strain was held to zero (K_o tests [9]), resulted in values of K_o of about 0.5. These values were in the range expected from the overconsolidation ratios estimated from consolidation tests [10, 11].

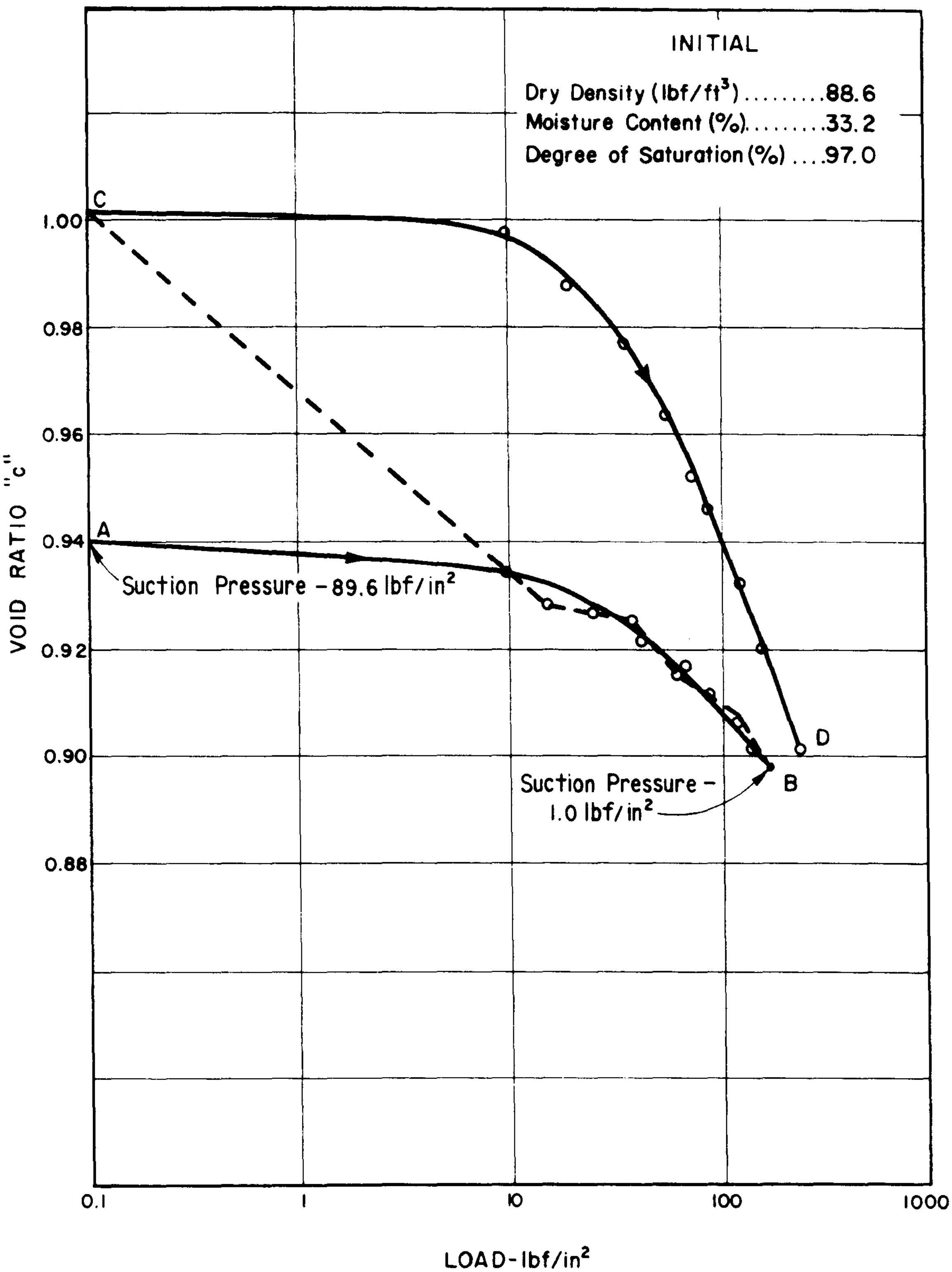
The soil suction pressures, soil stress relief, and volume-change characteristics were investigated using the floating ring consolidation apparatus described earlier. The largest initial soil suction pressure was 90 lbf/in^2 (620 kPa). Other suction values were in the range of 40 lbf/in^2 (276 kPa). These values were rather large in relation to the low overconsolidation ratios [12].

The variation in the suction values may be a result of the soil's varved structure and the moisture content changes that occurred in the samples during the interval between sampling and laboratory testing.

Stress relief occurred until the soil suction, acting as an effective stress, developed sufficiently to prevent additional swelling of the soil. The magnitude of the volume swell, as indicated by reloading the soil until the suction pressures were returned to zero (solid line AB, figs. 6 and 5b), ranged from 0.95 to 2.1 percent, shown in this typical sample. A maximum load of 280 lbf/in^2 (1930 kPa) was applied during the reloading cycle, which was the second phase of the test. Other samples required loads of only 50 lbf/in^2 (345 kPa). These values were considerably less than the preconsolidation loads indicated from the one-dimensional consolidation test. The difference between the loads was probably the result of stress relief and soil expansion which occurred during and immediately following sampling while moisture content increased an unknown amount. The water intake from the reloaded condition of zero soil suction to an unloaded condition (dashed line BC, figs. 7 and 5c) was measured and was expressed in terms of the initial specimen volume. The volume increase indicated by the water intake was 3 percent of the initial test specimen volume. An additional 4 percent volume increase was measured as the load was reduced to 1 lbf/in^2 (6.9 kPa) (fig. 5c).

Other samples swelled less than 1.0 percent as a result of similar moisture content changes. The solid line CD on figure 7 shows the path of reloading to the initial test specimen volume.

As outlined above, the soil suction pressures that developed after the soils were taken from the ground were of sufficient magnitude to affect the soil strength values determined in the laboratory



VOID RATIO-LOAD DATA

FLOATING RING CONSOLIDATION

Figure 7.—Floating ring consolidation void ratio-load data. [To convert from lbf/in² to kPa, multiply lbf/in² × 6.894].

[13]. The water intake studies suggested that unloading the soils, which were located below the water table, would result in swelling of the soil with a concurrent loss of strength. The triaxial shear tests were performed in two series to better define these factors: (1) series 1 CU tests on specimens cut from undisturbed samples at the moisture content as received in the laboratory samples, and (2) series 4 tests on similar hand-cut specimens reloaded with a 200-lbf/in² (1379 kPa) lateral pressure (σ_3 = 200 lbf/in² [1379 kPa]) and then unloaded with free water available, before the CU triaxial shear tests were performed.

The results of series 1 tests show a range in value of friction angle Φ' from 13 to 15 degrees with the sample discussed indicating a value of 13.5 degrees (table 2 and fig. 8). The corresponding cohesion values ranged from 18.1 lbf/in² (124.8 kPa) for the sample discussed to a maximum value of 39.4 lbf/in² (271.7 kPa). The plotted stress paths clearly show the effects of preconsolidation in that the effective stress was constant while the applied stress was also constant. This indicates that essentially no volume change or pore-pressure increase occurred as the total stress increased.

Stress paths for series 4 tests on specimens 44 and 65 of the same soil show that the stress relief that occurred during the unloading cycle with a moisture content increase reduced the overconsolidation and soil suction effects. With the applied chamber pressure held constant, the total effective stress decreased as the local stress increased. The pore pressures increased during shear to failure, which occurred after 3.2 to 6.6 percent axial strain. Test specimen 41, although tested under the same procedures, was apparently restricted during rebound and was, therefore, much stronger than the other specimens. A probable cause for the lesser volume increase was a separation (cavitation) in the water intake system.

The series 4 test results, including specimen 41, show an increase in Φ' to 20.8 degrees and a two-thirds decrease in cohesion to 5.9 lbf/in² (40.7 kPa).

The change in soil strength characteristics with increased moisture content and volume increase is evident in the stress versus strain plots on figure 9. The series 1 tests having significant soil suction pressures also showed a tendency toward a brittle-type failure. The series 4 tests showed a failure more typical of softer clay soils. Additionally, the residual strengths at high strains in the series 1 tests were approximately one-half the peak strengths while the series 4 residual strengths at high strains were only slightly less than the peak strengths, as shown on figure 9. The pore-air pressure and volume change data are also plotted on figure 9. In the series 1 tests, the pore pressures that developed were small during the sealed part of the test as shearing loads were applied. These results were further indication of the preconsolidation of the clay. The A_f ($\bar{A}$ at

Table 2.—Summary of triaxial specimen data.

Specimen Number	Placement				Void ratio		
	Dry density (lbf/ft^3)	Dry density (kg/m^3)	Moisture content (%)	Degree of saturation (%)	Placement	Consolidation	Failure
Series 1							
11	85.1	1363	35.9	96.5	1.0324	1.0536	1.0505
2	84.8	1358	37.0	98.8	1.0397	0.9445	0.9367
9	84.6	1355	36.2	96.3	1.0429	0.9164	0.9112
3	85.3	1366	35.1	95.0	1.0264	0.8815	0.8708
Series 4							
44	87.7	1405	33.7	96.7	0.9649	0.9336	0.9381
65	84.1	1347	37.5	99.1	1.0481	0.9814	0.9791
41	88.4	1416	33.5	97.4	0.9503	0.8126	0.8139

Table 2.—Summary of triaxial specimen data.—Continued

Specimen number	Test value at failure											Shear value corrected for pore pressure		
	Applied lateral pressure (lbf/in^2)	Applied lateral pressure (kPa)	Pore pressure (lbf/in^2)	Pore pressure (kPa)	Effective lateral pressure (lbf/in^2)	Effective lateral pressure (kPa)	Volume change (% of initial)	Axial strain (%)	Deviator stress (lbf/in^2)	Deviator stress (kPa)	$\tan\Phi$	c' (lbf/in^2)	c' (kPa)
Series 1													
11	10.0	68.9	1.5	10.3	8.5	58.6	−0.89	2.07	45.2	311.6			
2	50.0	344.7	2.2	15.2	47.8	329.6	5.05	6.76	72.0	496.4			
9	100.0	689.5	2.0	13.8	98.0	675.7	6.44	2.17	123.7	852.9			
3	200.0	1378.9	5.7	39.3	194.3	1339.7	7.68	3.43	155.8	1074.2	0.24	18.1	124.8
Series 4													
44	50.0	344.7	50.0	344.7	0.0	0.0	1.37	4.71	15.8	108.9			
65	30.0	206.8	14.1	97.2	15.9	109.6	3.37	6.57	36.4	251.0			
41	100.0	689.5	8.0	55.2	92.0	634.3	6.99	3.22	118.2	815.0	0.38	5.9	40.7

failure) values were from 0.02 to 0.03 for the series 1 tests and 0.03 to 1.3 for the series 4 tests. These values were to be expected in clays with overconsolidation ratios of 1 to 3 [14]. Photographs of three failed test specimens on figure 10 show that shear occurred across the varved structure in some areas while there was evidence of slippage along the silty planes in others.

SUMMARY

The varved clays encountered in the excavated slopes for Grand Coulee Third Powerplant were overconsolidated. While the overconsolidation ratios ranged between 1 and 3, the magnitude of

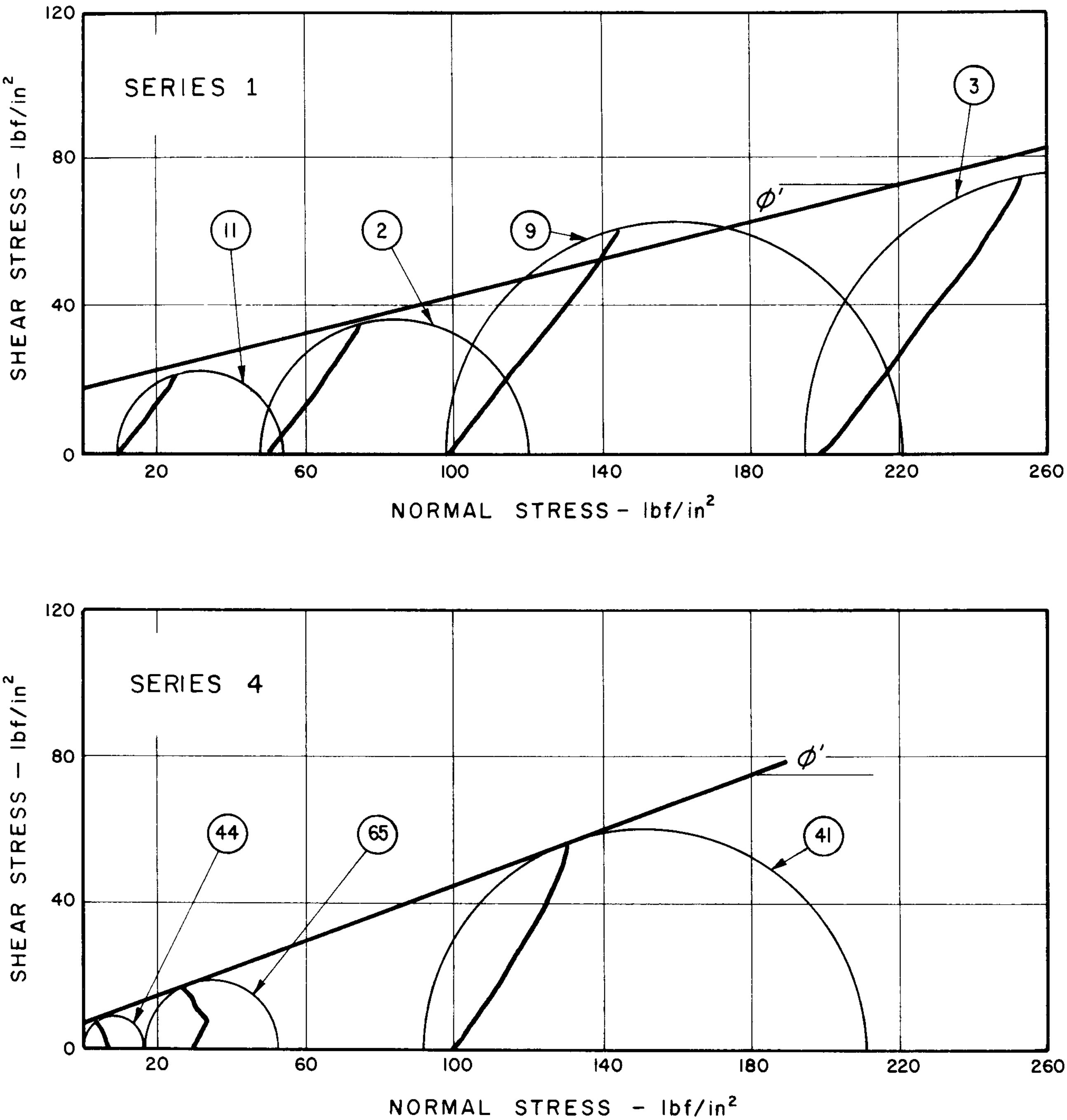


Figure 8.—Triaxial shear data. [To convert lbf/in² to kPa, multiply by 6.894].

the preconsolidating loads was large. The preconsolidating loads, P_c, indicated from the one-dimensional consolidation testing using Schmertmann's [4] graphic analysis, were in the range of 250 to 400 lbf/in² (1724 to 2758 kPa).

This preconsolidating load had a significant effect on the triaxial shear testing. The moment sampling was begun in the drill hole, the clay tended to expand as a result of stress relief. This expansion

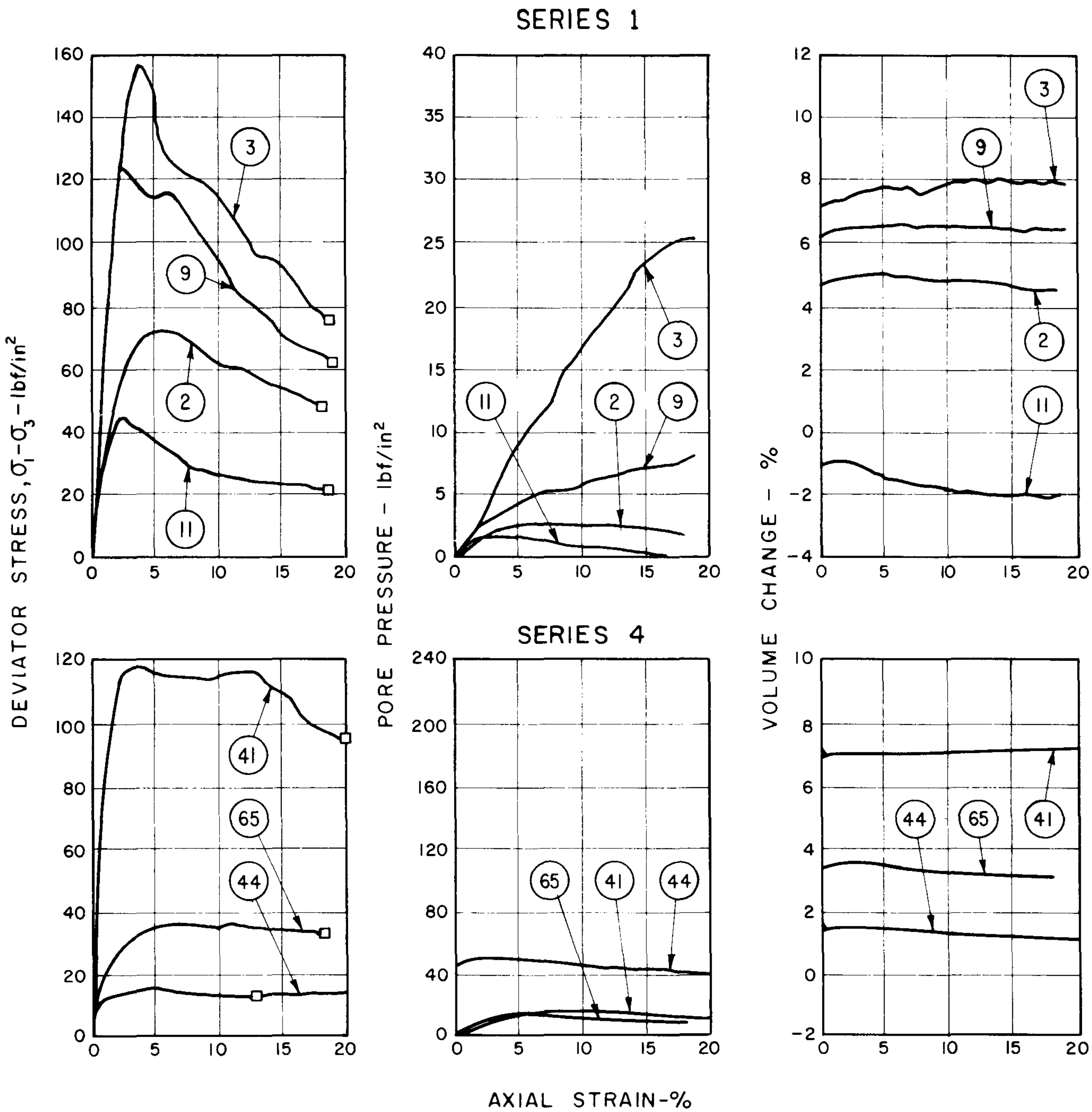


Figure 9.—Stress, pore-pressure, and volume change relationship with strain. [To convert from lbf/in² to kPa, multiply by 6.894].

occurred as long as excess water was available and continued until specimens were reloaded in the triaxial shear test. When the initial excess water was absorbed, further expansion tended to be limited by the development of suction pressures acting as an effective stress to as great as 90-lbf/in² (621-kPa) negative. Floating ring consolidation test results indicated that loads approximately equivalent to the present overburden pressure were needed to overcome these suction pressures.

Figure 10.—Test specimens after failure: specimen 3 (Photo E-2185-290), specimen 11 (E-2185-294), and specimen 44 (E-2185-300).

A method to relieve the suction pressures was to provide excess water continuously during an unloading cycle. This was done in the series 4 triaxial shear tests. This procedure induced a strength loss, which was probably due to the combined result of reduced influence of the suction pressures and relief of the preconsolidating stresses during the triaxial test.

CONCLUSIONS

These varved clays were preconsolidated by loads in the order of 350 lbf/in^2 (2413 kPa) or more. The present overburden loads are on the order of 130 lbf/in^2 (896 kPa) or more.

Sampling permitted stress relief with a volume increase in the soil. The data from the floating ring consolidometer test indicated the volume change to have been in the order of 3 percent by the time specimens were ready for triaxial testing. This volume increase was accompanied by the development of soil suction pressures as great as the 90-lbf/in^2 (621 kPa) negative measured for the sample discussed. Additional water intake resulted in a further volume increase of about 4 percent for a total swell of 7 percent from original volume. The test apparatus confined the clay horizontally forcing the volume change to a vertical direction which was frequently perpendicular to the varved structure.

The soil strength in-place is greater than shown by the laboratory tests because of the increase in moisture content and soil volume during sampling and test specimen preparation.

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APPENDIX

Symbols

LL = Liquid limit

PL = Plastic limit

PI = Plasticity index

SL = Shrinkage limit

C_R = Relative consistency

a = Activity

$$\bar{A} = \frac{\Delta u}{(\sigma'_1 - \sigma'_3)}$$

γ_d = Dry density, lbf/ft^3

W_o = Initial moisture content

e_o = Initial void ratio

e_c = Void ratio after consolidation

e_f = Void ratio at failure

ε = Strain, in/in

σ_3 = Confining pressure, lbf/in^2

$\sigma_1' - \sigma_3'$ = Deviator stress (effective stress, lbf/in^2)

σ_1' = Major principal effective stress, lbf/in^2

σ_1'/σ_3' = Stress ratio

τ = Shear stress, lbf/in^2

u_a = Pore-air pressure, lbf/in^2

u_c = Equivalent capillary pressure, lbf/in^2 (u_c always <0.0)

u_w = Pore-water pressure, lbf/in^2; $u_w = u_a + u_c$

P_o = Present overburden pressure, lbf/in^2

P_c = Preconsolidating pressure, lbf/in^2

$\dfrac{P_c - P_o}{P_o}$ = Overconsolidation ratio

Sp. Gr. = Specific gravity

S_r = Degree of saturation

APPENDIX

Symbols

Φ'	=	Angle of internal friction, on an effective stress basis
c'	=	Cohesion intercept, effective stress basis
Residual strength	=	The fairly constant lesser value in deviator stress at axial strains greater than the strain at peak deviator stress

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