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SHEAR STRENGTH CHARACTERISTICS OF TWO CLAYS FROM WESTERN CANADA

by

E. L. Matyas

ANALYZED

Internal Report No. 345
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OTTAWA

July 1967

PREFACE

Stability analyses in the highly plastic clay and clay shale regions of Western Canada are quite unreliable because of uncertainties in interpreting laboratory test results. The large strains that occur in compression tests and the great differences between peak and residual strengths are factors that are not easily taken into account.

During a summer term as a visiting professor, Dr. E.L. Matyas of the University of Waterloo carried out a series of triaxial compression tests on samples of clay from Winnipeg, Manitoba and on samples of Bearpaw shale from the South Saskatchewan River Dam. The results of these tests are described in this report and their significance is discussed.

It is hoped that these results will add to the understanding of such soils and will lead to discussion of their properties.

Ottawa
July 1967

Robert F. Legget
Director

SHEAR STRENGTH CHARACTERISTICS OF TWO CLAYS FROM WESTERN CANADA

by

E.L. Matyas

SUMMARY

The results of drained triaxial compression tests on a highly plastic clay from Manitoba and a clay shale (Bearpaw shale) from Saskatchewan are reported. The shear strength parameters are expressed in terms of effective stresses for both peak and residual strengths. Both soils exhibited swelling characteristics. Some case histories cited in the references have indicated that the methods of analyzing slope failures in swelling clays are inadequate unless a swelling pressure term, p_s, is included in the effective stress equation for shear strength. This design expedient is questioned.

Hardy (1965) reported that many of the clays in the Prairie provinces and northwestern Canada in the area between the western fringe of the Precambrian shield and the Rocky Mountains exhibited severe swelling. This swelling introduced complications in a wide variety of engineering problems including the design of shallow structural foundations, slopes, earth dams, bridge abutments, hydraulic structures, and highway, railroad and runway embankments.

Considerable work has been done on these swelling clays in order to define their engineering characteristics (Peterson, 1960; Hardy, 1962; Crawford, 1964; Mishtak, 1964; Ringheim, 1964; and Hardy, 1965). Hardy (1965) has indicated that in practical engineering situations, conventional theories and methods of soil mechanics do not give results that can be used to predict accurately either the magnitude of the swelling forces, the volumetric changes they will produce, or the variation in the strength characteristics of the soil due to internal swelling forces.

To supplement existing data on swelling soils, two clays were selected for testing: a clay taken from a cut along the proposed route of the Red River Floodway in Winnipeg, Manitoba and a core sample of Bearpaw shale taken at the site of the South Saskatchewan Dam in Saskatchewan. This report* deals only with the strength characteristics of these clays.

SOURCE AND GENERAL DESCRIPTION OF SOIL SAMPLES

(a) Winnipeg Clay

A diversion channel known as the Red River Floodway is being constructed to bypass the Greater Winnipeg area in order to avoid the recurrence of serious flooding. During the investigation stage of this project in 1961, a test trench was excavated to a depth of about 45 feet. Details of the Floodway and the test trench were reported by Mishtak (1964) and Crawford (1964).

This report deals mainly with the shear strength properties of a block sample taken from a depth of 20 feet below the original ground level. The general soil properties and typical results from other depths are summarized in Table I.

(b) Bearpaw Shale

The Bearpaw shale used in this investigation was obtained from the site of the South Saskatchewan River Dam. The geology and the cross-sections of the site are in publications by Peterson (1960) and Ringheim (1964). In general, the soil profile consists of overburden underlain by soft, medium and hard shales in succession. This report will deal mainly with the strength characteristics of the hard shale. Typical soil properties are given in Table I.

SHEAR STRENGTH

(a) Undrained Strength of Saturated Fissured Clays

Skempton and LaRochelle (1965) have shown that in fissured clays the undrained strength as measured on small specimens is considerably greater than the strength of the clay in the mass. Thus, in stiff fissured clay, the fissures not only reduce the strength of the clay mass to below the strength of the intact material but also, when an excavation is made, the strength is locally reduced to zero at the points where fissures have been opened up.

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*Another report dealing with the magnitude of the swelling forces and volumetric changes has been submitted as NRC Internal Report No. 347.

(b) Peak and Residual Strengths

The "peak strength" and the "residual strength" of a soil have been dealt with in a paper by Skempton (1964)*. If an over-consolidated clay is sheared slowly under a given effective stress, the clay can offer limited resistance and this is the "peak strength", s_f. If the test is continued, the displacement increases and the strength of the clay decreases to a "residual strength", s_r, provided that the displacements are sufficiently large. If a number of tests are made at different effective stresses both the peak strengths and the residual strengths may be plotted against the effective stress. In terms of peak strengths:

$$s_f = c' + \sigma' \tan \phi' \qquad (1)$$

and in terms of residual strengths,

$$s_r = c'_r + \sigma' \tan \phi'_r . \qquad (2)$$

These terms will be used to define the strength parameters for Winnipeg clay and Bearpaw shale.

(c) The Effect of Swelling Pressure on Shear Strength

The classical expression for shear strength in terms of effective stresses is usually given in the form:

$$\tau = c' + (\sigma - u) \tan \phi' . \qquad (3)$$

Hardy (1962 and 1965) has modified this equation to the following form:

$$\tau = c' + (\sigma - u - p_s) \tan \phi' , \qquad (4)$$

where p_s is the magnitude of the swelling pressure. This equation will be discussed below.

STRENGTH TESTS

All strength tests were performed on cylindrical specimens approximately 8 cm in height and 3.6 cm in diameter. A nominal back pressure of 1 kg/cm^2 was used for the consolidation and the shearing stages. All specimens were allowed to consolidate or swell for a minimum of three days before shearing.

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*In his paper, Skempton credits the introduction of the term "residual strength" to a number of researchers.

(a) Winnipeg Clays

Consolidated drained tests were run on nine specimens which were trimmed from the undisturbed block sample. The specimens were loaded at a rate of strain sufficiently slow to ensure at least 95 per cent pore pressure dissipation.* Although a theoretical time of about 2.5 days to failure was required for 95 per cent dissipation, most of the specimens were loaded at a slower rate to ensure a dissipation approaching 100 per cent. Stress-strain curves and volume change curves are plotted on Figure 1. As expected, the failure strain was small (about 1.7 per cent) at low confining pressures and larger at the higher confining pressures.

In each case, the stress reached a peak value (maximum deviator stress) and then dropped off to a relatively constant value (residual strength). It may be noted that most specimens were dilating at failure. Figures 2 and 3 show a plot of $\frac{1}{2}(\sigma_1 - \sigma_3)$ against $\frac{1}{2}(\sigma_1' + \sigma_3')$ for peak and residual strengths respectively. These plots were used to calculate the failure envelope to the conventional Mohr circles of stress which are also shown on Figures 2 and 3; the values obtained were $c' = 0.31$ kg/cm^2, $\phi' = 16.8$ deg; $c_r' = 0.06$ kg/cm^2 and $\phi_r' = 15.1$ deg. Additional information is given in Table II.

(b) Bearpaw Shale

Four specimens were trimmed from a core sample taken from a depth of 115 to 116 feet. Owing to the hardness of the core sample, blocks measuring approximately two inches square by four inches long were cut on a band saw. These were waxed immediately and stored until needed. Since a wire saw could not be used, trimming was done with a heavy sharp knife. This method proved to be quite successful. Most of the specimens were consolidated for a minimum of one week and then loaded. Peak stresses occurred at least nine days after loading commenced and most specimens were loaded for about two weeks.

Stress-strain curves and volume change curves are plotted on Figure 4 and pertinent data are summarized in Table II. Failure strains varied from about three to four per cent. Mohr circle plots for peak stresses and residual stresses are shown in Figure 5. These indicate that

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*This was based on the consolidation stage of the test (Bishop and Henkel, 1957).

there is a curved failure envelope for peak stresses and therefore c' and ϕ' can only be given for a particular stress range. For the residual strengths $c'_r = 0.12$ kg/cm^2 and $\phi'_r = 21.5$ deg.

(c) Consolidated Drained Tests With Lateral Stress Decreasing

Only two specimens of Winnipeg clay were tested under conditions of controlled major principal stress. The results are plotted on Figures 2 and 6. Specimen 119-2-7 was tested with $\sigma' = 1.0$ kg/cm^2 which closely approximated the in situ effective stress. Specimen 119-2-10 was tested with $\sigma'_1 = 2.75$ kg/cm^2.

Since the tests were run concurrently with the consolidated drained tests, the failure envelope had not been established; consequently, the magnitude of the decrements in lateral stress were arbitrarily assumed. Each decrement, with the exception of the last, was removed about every 24 hours.

(d) Discussion of Results

(i) Winnipeg Clay

Figure 2 indicates that the Ψ - line, and, consequently, the failure envelope for both the peak and residual strengths were essentially straight throughout the range of consolidation pressures used in the test series. Crawford (1964) reported results for tests on Winnipeg clay taken at the same location from a depth of 30 feet. For peak stresses, values of $c' = 0.6$ kg/cm^2 and $\phi' = 9$ deg. were reported. These values differ considerably from the values reported here, that is, $c' = 0.31$ kg/cm^2 and $\phi' = 16.8$ deg. The soils were similar in each case except in water content (Table I) but these tests were strained at a much slower rate. This is thought to account for the differences.

Crawford made a few special tests in which specimens were immersed for several hours before testing and showed that this pretreatment caused a considerable reduction in strength under low stresses. Although very low (0.07 kg/cm^2) confining effective stresses were used in the present tests, this marked reduction in strength was not observed, indicating that immersion without confinement may be too severe.

The two tests made with constant σ_1 and decreasing σ_3 indicated failure stresses that agreed reasonably well with those obtained from standard tests with σ_1 increasing (see Figures 2 and 6).

These results confirmed observations made by Crawford (1964) and consequently no additional special tests were made.

All specimens were drained through the base and coarse porous stones were used at the top and the base of the specimens. Four equally spaced filter papers about 1/4 in. wide were used on the sides of the specimens and thin rubber membranes were used to jacket them. Because the rubber membranes were relatively thin and only a few filter drains were used, no corrections were applied. This is not entirely justified, however, particularly for post-peak conditions. Uncertainties in calculating post-peak stresses would also preclude the application of membrane and drain corrections.

Owing to the post-peak distortion of the specimens in the triaxial apparatus, large axial strains become unrealistic and it is unlikely that the residual strength that was recorded was a minimum. At substantially larger strains lower residual strength parameters would undoubtedly be determined. Skempton (1965) indicated that tests in direct shear boxes frequently gave $c'_r = 0$ and a ϕ'_r that was in the order of three degrees less than that given by tests in triaxial apparatus. On this basis, the results obtained from the triaxial tests have been reduced to modified residual strength parameters (c'_{mr} and ϕ'_{mr}) as shown in Table III.

(ii) Bearpaw Shale

Referring to Figure 5, it is seen that the failure envelope based on peak stresses is curved over the range of confining pressures used in the test series. This behaviour is typical of highly compacted soils (Matyas, 1963) and heavily overconsolidated soils (Bishop, 1965), and the shear strength parameters can only be expressed for a particular stress range. Although only three of the four tests were strained sufficiently to indicate residual strengths it appears that a straight-line envelope exists giving $c'_r = 0.12$ kg/cm^2 and $\phi'_r = 21.5$ deg. The modified shear strength parameters are $c'_{mr} = 0$ and $\phi'_{mr} = 18.5$ deg.

It is interesting to note that the stress-strain curves shown on Figure 4 exhibited a straight-line relationship to almost maximum deviator stress. Since the specimens were drained, however, it is not possible to use this relationship to obtain the modulus of elasticity, E. No undrained tests were made to see whether the linearity would prevail for undrained conditions.

(iii) General

Equation (4) indicated that a swelling pressure term should be included in the expression for shear strength. Since all specimens were drained and distilled water was used in the drainage system it is reasonable to assume that the behaviour of in situ soil that is exposed to rainfall would be similar to the behaviour of laboratory specimens. The pore water of both clays contained limited quantities of salt and therefore chemical changes, osmotic pressures etc. would be insignificant. Indeed, even with high salt concentrations in the pore water, it is thought that the shear strength parameters as determined by conventional drained tests would still reflect the in situ behaviour and that the swelling pressure term need not be considered.* In addition, the acceptance of a swelling pressure term would indicate that two different pore water pressures exist simultaneously and this does not appear to be physically possible.

Infiltration may cause base exchange, osmotic pressures, etc. and these factors may change properties of the material such as swelling pressure and volume change characteristics. Rather than invalidate the accepted effective stress equations to accommodate these changes it would be more correct to consider it as an entirely different material.

STABILITY ANALYSIS

(a) General

In designing or analyzing clay slopes one must choose between a total stress analysis and an effective stress analysis. The choice of either analysis is dependent on the conditions to which the soil mass will be subjected. These conditions are described in detail by Bishop and Bjerrum (1960) and in general by Skempton (1964).

The total stress analysis ($\phi = 0$ condition) is only correct when the field condition corresponds to the laboratory test conditions, that is, when the shear stress tending to cause failure is applied under undrained conditions. The $\phi = 0$ method is restricted to saturated soils and to problems in which insufficient time has elapsed after the stress

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*Noble (1965) has also questioned Hardy's modification of the shear strength equation.

change considered for an increase or decrease in water content has occurred. Owing to the low permeability of clays, a change in water content is not likely to occur during the construction stage and therefore the $\phi = 0$ method is applicable to an "end of construction" condition.

When a cut is made in clay the pore pressure at any point depends partly on the position of the ground water level and partly on the response of the clay to the changes in stress that have taken place during excavation. Eventually, the pore pressures reach a state of hydrostatic equilibrium known as the "long-term" condition. The pore pressure may be determined by direct measurement with piezometers or by sketching a flow net to satisfy the boundary conditions.

In fissured clays the strength of a mass is lower than the intact strength of small specimens. This reduced strength can only be assessed from an analysis of full-scale slips. Of more importance is the fact that the intact strength cannot be relied on to design or analyze fissured clays.

For "long-term" conditions, Skempton (1964) has shown that the use of peak shear strength parameters in stability analyses may lead to disagreement between predicted and actual factors of safety. This is particularly true for fissured clays and Skempton has indicated that residual strengths must be considered if reasonable agreement between theory and practice is to be achieved.

(b) Winnipeg Clay

Mishtak (1964) reported the results of stability analyses which were made for the controlled full-scale slips in the 1 to 1 slope of the Red River Floodway Test Trench. Total stress analyses indicated that the average mobilized shear strength, $\bar{c}$, was about 4.3 lb/sq. in. Table II of Mishtak's paper shows that the average shear strength, c, from the surface to a depth of 45 feet is in the order of 8.8 lb/sq. in. It is apparent, therefore that 50 per cent or less of the measured strength was mobilized. This value is similar to that found by Skempton and LaRochelle (1965) for overconsolidated stiff fissured London clay. The Winnipeg clay did not appear to contain many fissures but it is evident that they were present in sufficient numbers to warrant a decrease in the over-all shear strength. For design, it is indicated that a ratio of $\bar{c}/c$ equal to about 0.4 should be considered.

Mishtak has also given results for effective stress analyses but he indicated that they may be unreliable owing to questionable piezometric levels. An effective stress analysis in terms of residual shear strength parameters was not carried out by the author since Mishtak did not include details of the proposed cross-section.

(c) Bearpaw Shale

Peterson (1960) and Hardy (1965) have reported that stability analyses in terms of effective stresses of natural slopes in Bearpaw shale were not satisfactory. Even the use of shear strength parameters obtained on completely remoulded samples has led to an overestimate of the factor of safety.

(d) Discussion

It is apparent that stability analyses of slopes in swelling clays are far from satisfactory. In the light of recent work by Skempton (1964) on residual strengths, Skempton (1965) on undrained strength of fissured clays and Morgenstern (1965) on non-circular slip surfaces it is thought that closer agreement can be obtained between theory and practice. These concepts have not been applied in this report to re-analyze reported slides because (1) residual shear strength parameters are not available for complete soil profiles and (2) published cross-sections are normally either incomplete or on such a small scale that it is difficult to reproduce the section accurately.

CONCLUSIONS

Consolidated drained triaxial compression tests have indicated that the residual strengths of both Winnipeg clay and Bearpaw shale are appreciably lower than the peak strengths. The lower strengths are recommended for use in analyzing the stability of slopes in these materials.

Large axial strains in samples tested in triaxial apparatus lead to sample distortion that makes the interpretation of results questionable. Alternative testing apparatus, therefore, should be used to investigate residual strengths such as direct shear box.

The properties and characteristics of in situ soils may change as a result of infiltration but this possibility should not invalidate the principle of effective stress. When changes do take place, a different material is evolved and it should be considered as a separate soil having its own properties.

ACKNOWLEDGEMENTS

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TABLE I

AVERAGE SOIL PROPERTIES

Block No.	Color	Depth ft	γ lb per cu ft	w %	G	S_r %	Clay Size $< 2\mu$ %	w_L %	w_p %	I_p %	Reference
				Winnipeg Clay							
119-1	Brown	10	107.2	52.4	2.74	95.6	94	109	35	74	
119-2	Brown	20	110.7	45.2	2.77	97.8	61	80	30	50	
		20	105.3	56.0							Mishtak (1964)
119-3	Brown	30	104.9	53.9	2.76		83	94	34	60	Crawford (1964)
		30	105.8	56.1							Mishtak (1964)
119-4	Grey	40	101.6	59.4	2.75	96.6	82	99	33	66	
		40	102.0	64.3							Mishtak (1964)
				Bearpaw Shale							
146-1*	Grey	42-43	115.1	35.3	2.76	94.6	56	114	34	80	
146-3**	Grey	115-116	128.5	21.7	2.76	94.7	45	87	26	61	

* Soft shale
** Hard shale

DRAINED COMPRESSION TEST RESULTS

Specimen	Water Content % Initial	Water Content % End of Consol	Water Content % At Peak Stress	Water Content % At Residual Stress	Consol Pressure kg/sq cm	$(\sigma_1 - \sigma_3)_f$ kg/sq cm	σ'_{1f} kg/sq cm	$(\sigma_1 - \sigma_3)_r$ kg/sq cm	ϵ_f %	t_f days
Winnipeg Clay										
Consolidated - drained triaxial with axial stress increasing										
119-2-1	42.9	43.6	43.3	44.4	0.25	1.25	1.50	0.26	1.7	3.5
119-2-2	44.4	43.9	43.8	43.6	1.00	2.05	3.05	0.41	2.5	2.1
119-2-3	44.5	44.1	43.4	43.8	0.50	1.00	1.50	0.22	1.7	1.5
119-2-4	44.9	44.9	43.9	44.2	0.77	1.48	2.25	0.42	2.6	4.9
119-2-5	45.5	43.7	42.5	42.9	2.00	2.18	4.18	0.75	3.8	4.3
119-2-6	44.4	43.8	42.9	43.1	1.40	1.90	3.30	0.60	2.8	4.1
119-2-8	44.4	46.1	46.1	49.4	0.07	0.75	0.82	0.07	1.5	2.1
119-2-9B	44.7	42.3	40.8	40.6	3.00	3.48	6.48	1.07	4.3	5.0
119-2-15					4.50	4.41	8.91		6.5	8.0
Consolidated - drained triaxial with lateral stress decreasing										
119-2-7	44.6	43.3	44.0		1.00	0.89	1.00		0.5	8.0
119-2-10	43.6	41.5	42.4		2.75	1.97	2.72		1.8	22.0
Bearpaw Shale										
Consolidated - drained triaxial with axial stress increasing										
146-3-1	21.6	23.2	22.6	23.4	2.00	11.98	13.98		3.1	12.0
146-3-2	21.4	22.7	21.7	23.2	1.00	7.60	18.60	1.83	3.0	9.0
146-3-3	21.7	21.7	21.2	22.2	4.50	18.90	23.40	5.55	3.2	9.0
146-3-5	22.1	28.8	37.2	30.6	0.09	3.00	3.09	0.25	4.3	10.0

TABLE III

SUMMARY OF SHEAR STRENGTH PARAMETERS

Soil	Depth ft	Peak c' kg/cm^2	Peak ϕ' deg	Residual c'_r kg/cm^2	Residual ϕ'_r deg	Residual c'_{mr} kg/cm^2	Residual ϕ'_{mr} deg	Remarks
Winnipeg Clay	30	0.31	16.8	0.06	15.1	0	12	Block Sample
Bearpaw Shale	115	curved failure envelope		0.12	21.5	0	18.5	Core Sample

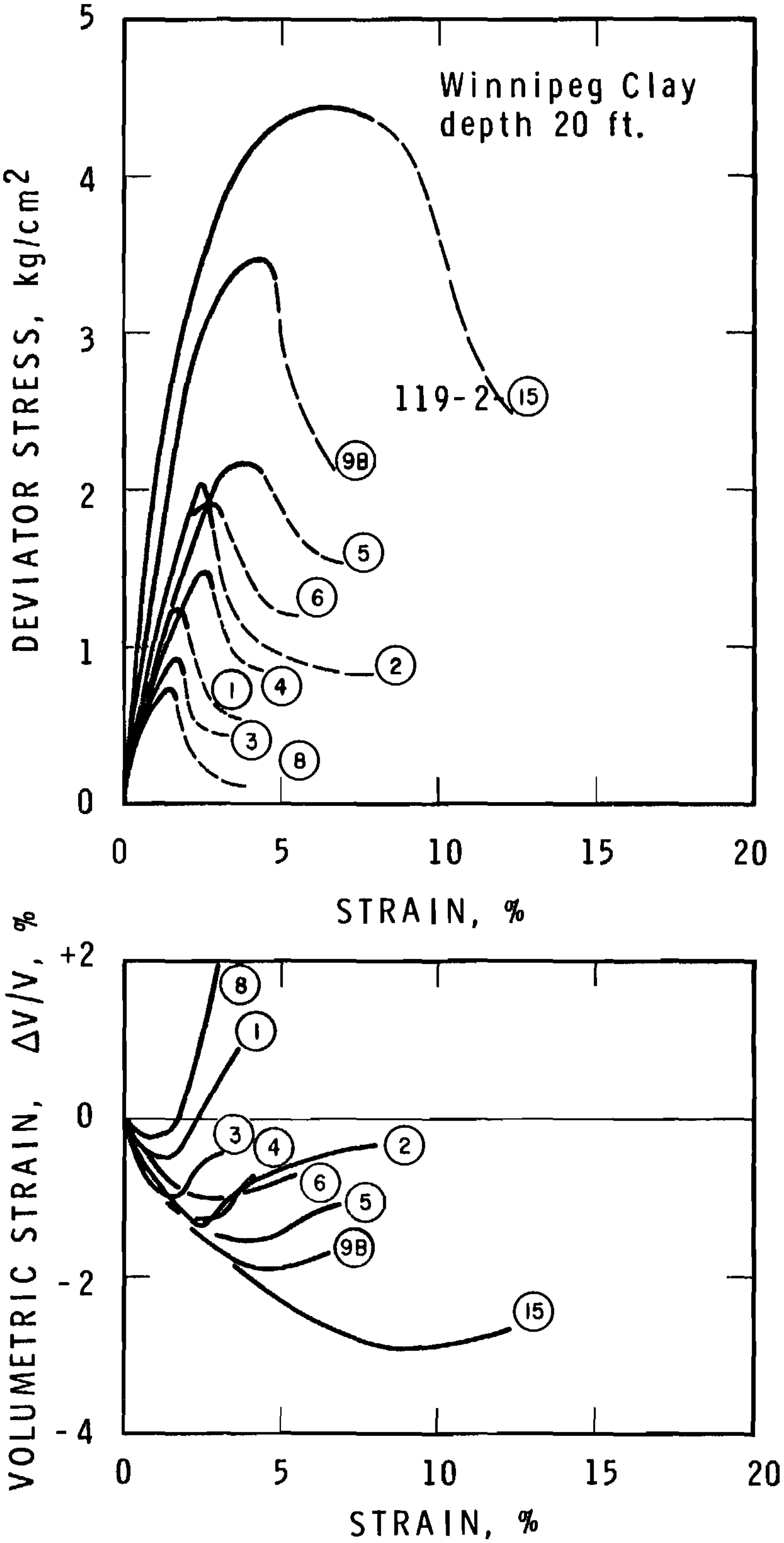


FIGURE 1

CONSOLIDATED DRAINED TRIAXIAL TEST RESULTS - AXIAL STRESS INCREASING

BR 3944-1

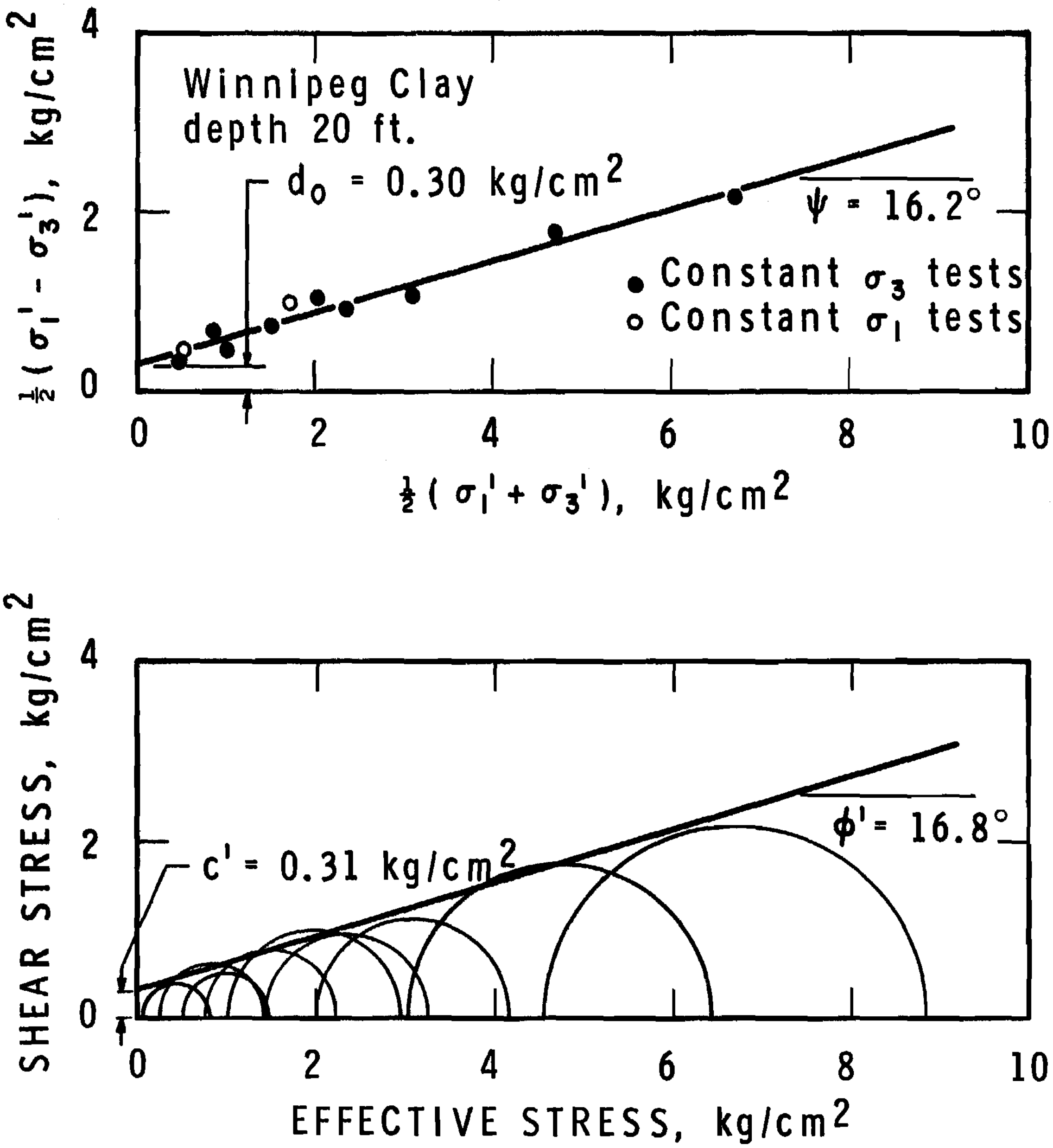


FIGURE 2

CONSOLIDATED DRAINED TRIAXIAL TEST RESULTS AT PEAK STRESSES

BR 3944-2

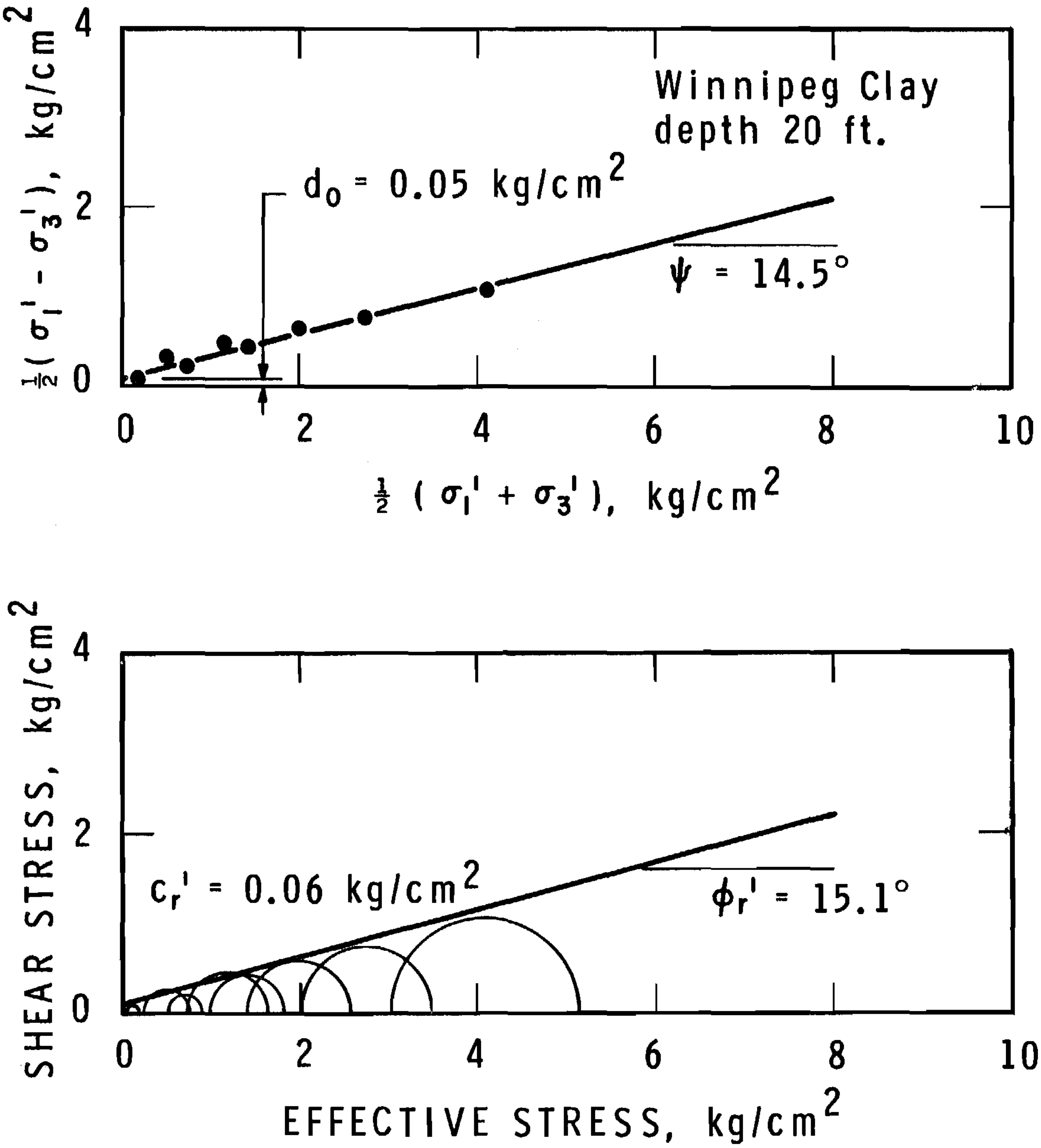


FIGURE 3

CONSOLIDATED DRAINED TRIAXIAL TEST RESULTS FOR RESIDUAL STRESSES

BR 3944 - 3

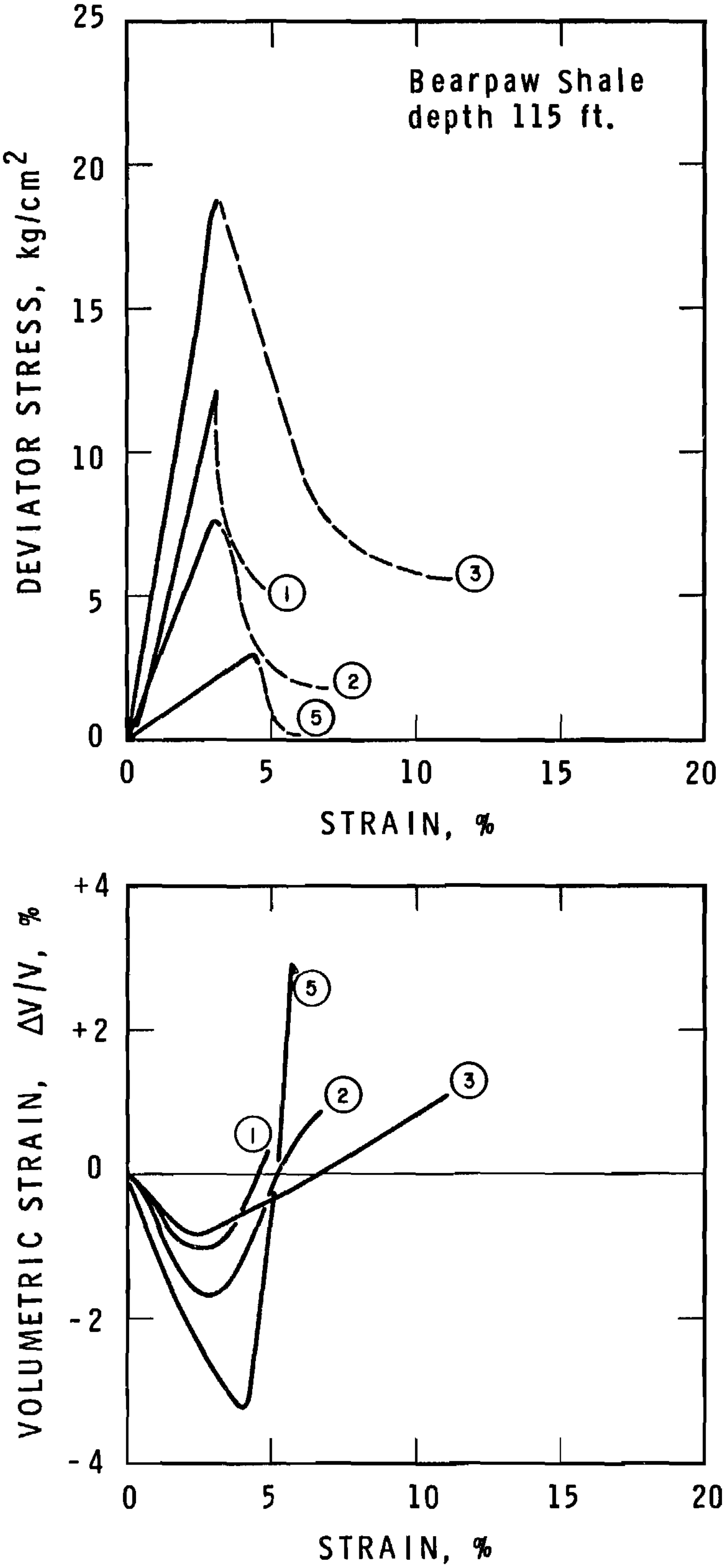
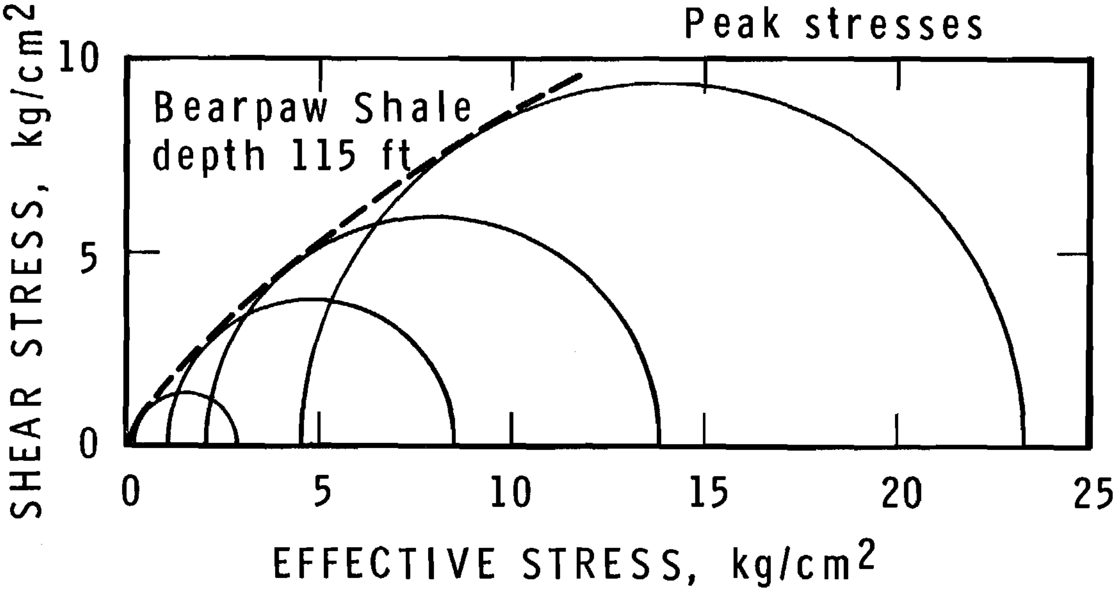


FIGURE 4

CONSOLIDATED DRAINED TRIAXIAL TEST RESULTS - AXIAL STRESS INCREASING

BR 3944-4



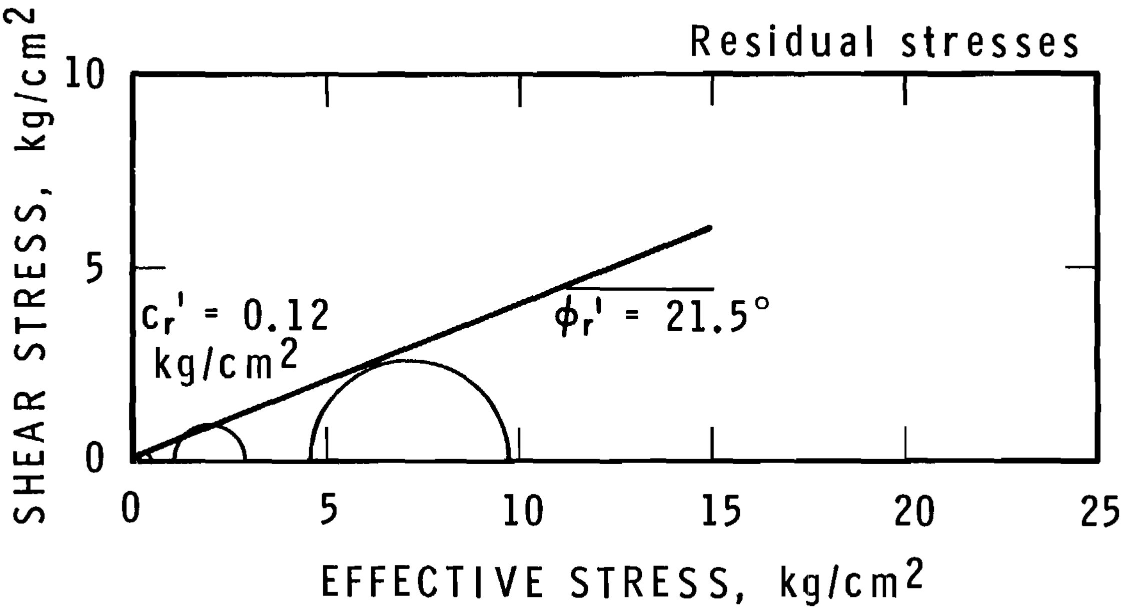


FIGURE 5

CONSOLIDATED DRAINED TRIAXIAL TEST RESULTS AT PEAK AND RESIDUAL STRESSES

BR 3944-5

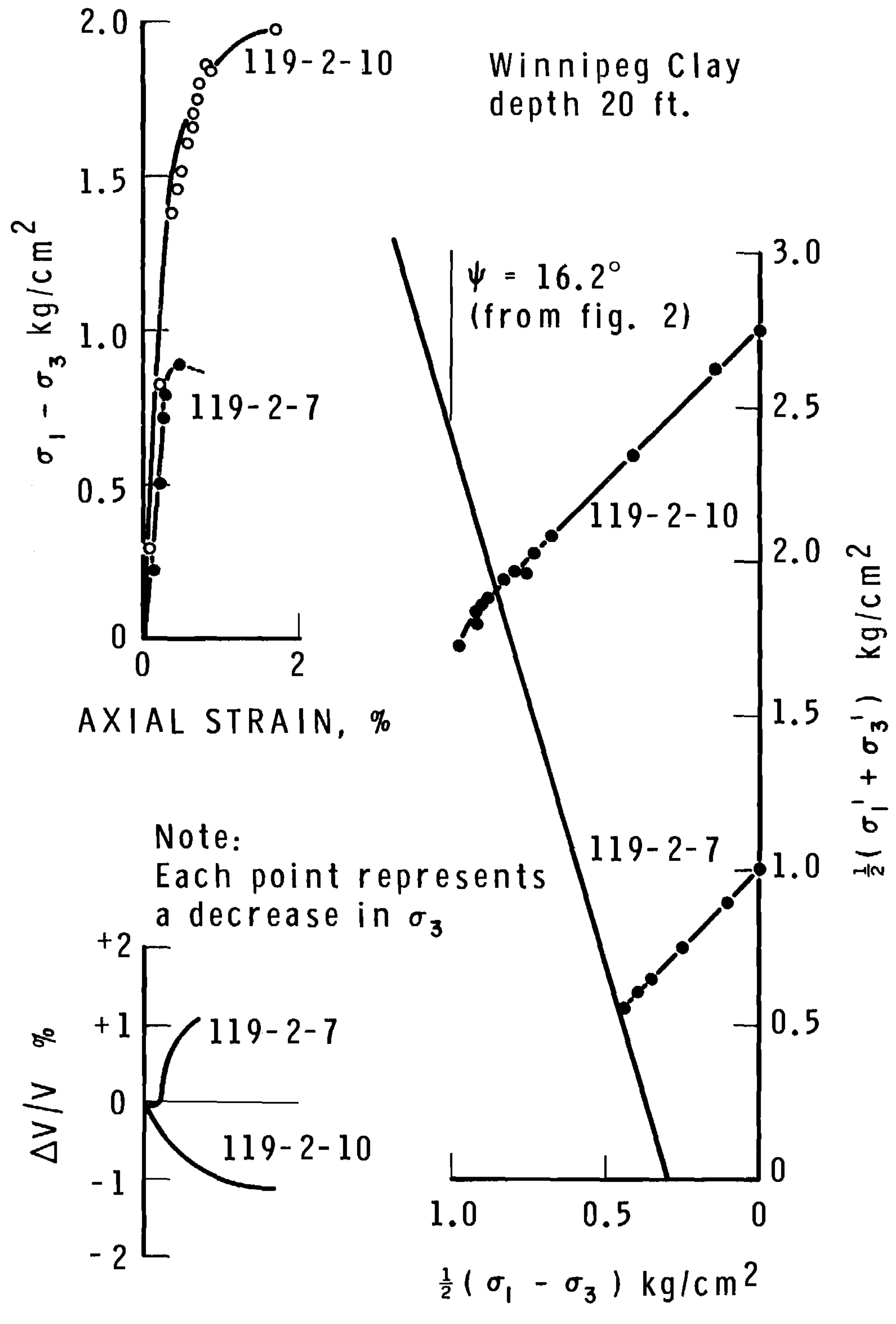


FIGURE 6

CONSOLIDATED DRAINED TRIAXIAL TEST RESULTS LATERAL STRESS DECREASED IN STAGES

BR 3944-6