THE UNIVERSITY OF MANITOBA

LABORATORY STRENGTH TESTS AND APPLICATIONS FOR WINNIPEG CLAY

by

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A dissertation submitted to the Faculty of Graduate Studies of the University of Manitoba in partial fulfillment of the requirements of the degree of

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ABSTRACT

This thesis deals with the careful evaluation in the laboratory of shear strength parameters of typical undisturbed Winnipeg clay samples. Included was the investigation of the shear strength parameters in terms of total and effective stresses. Triaxial tests using drained and undrained loading in both compression and extension were employed. Residual and peak effective stress parameters were obtained by direct shear tests. Procedures and results are included. The preconsolidation pressures were estimated by using consolidation test data, Mohr circle envelope, $\frac{1}{2}(\bar{\sigma}_1 - \bar{\sigma}_3)$ vs $\bar{\sigma}_3$, stress path and also from the A_f parameter.

Examples are given with comments on the application of these parameters to practical foundation and slope stability problems.

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LIST OF SYMBOLS

A	Pore pressure coefficient;
A_f	Pore pressure coefficient at failure;
B	Pore pressure coefficient
D	Depth factor;
D_f	Depth of foundation below grade;
L.L.	Liquid limit;
N_c	Bearing capacity coefficient;
N_q	Bearing capacity coefficient;
N_γ	Bearing capacity coefficient;
P.L.	Plastic limit;
S.F.	Safety factor;
U%	Degree of consolidation expressed as a percent;
b	Breadth of footing;
c	Cohesion;
c'	Effective cohesion;
c'_p	Peak effective cohesion;
c'_R	Residual effective cohesion;
c_u	Undrained shear strength
e_o	Initial void ratio;
P_c	Preconsolidation pressure;
q_u	Unconfined compressive strength;
$q_{net\ ult}$	Net ultimate bearing capacity;
u	Pore pressure;
Δu_a	Change in pore pressure due to all-around cell pressure;

ΔV	Change in volume;
ϕ	Internal friction angle;
ϕ'	Effective internal friction angle;
ϕ'_p	Peak effective internal friction angle;
ϕ'_R	Residual effective internal friction angle;
σ_1	Major principal stress;
σ_2	Intermediate principal stress;
σ_3	Minor principal stress;
$\bar{\sigma}_1$	Effective major principal stress;
$\bar{\sigma}_3$	Effective minor principal stress;
$\bar{\sigma}_f$	Effective normal stress at failure;
$\Delta\sigma_3$	Change in all-around cell pressure;
τ	Shear strength;
γ_m	Moist unit weight;
γ_1	Moist unit weight below base of footing;
γ_2	Moist unit weight above base of footing;
θ	Failure plane measured with respect to horizontal.

CHAPTER I

INTRODUCTION

Knowledge about the shear strength of soils is most important in foundation designs and soil stability problems. In order to apply the correct solutions to these problems, it is necessary to carry out laboratory tests on the soils under investigation.

Soils used in the study are the typical Winnipeg clays. Sourse of samples, site and sampling description are given in Chapter II. The soil classification test program, as well as the triaxial tests and the direct shear tests, have been done according to the standard procedures outlined by the A.S.T.M., and Bishop and Henkel[1]. The details of the experimental procedures are given in Chapter III and IV.

The main purposes of this thesis were as follows:

1. To perform routine laboratory tests which are needed for soil classification.

2. To conduct a variety of shear strength tests in order to determine to what extent shear strength parameters are a function of the type of test.

3. To investigate the application of the strength parameters obtained from the shear strength tests to some engineering problems, that are related to the bearing capacity of foundations and to slope stability problems.

Soil samples were tested and experimental investigations were carried out in the Soil Mechanics Testing Laboratory, Civil Engineering, University of Manitoba.

CHAPTER II

SITE AND SAMPLING DESCRIPTION

1 Soil Sampling and Preparation

Two test holes were drilled at the site of the Canada Cement Lafarge Plant at Fort Whyte in Winnipeg. The holes were drilled with a truck-mounted, 16-inch diameter power auger. Undistrubed, moisture content and bulk samples were obtained at depths indicated in Figures 1 and 2.

The large diameter of the test holes permitted the obtaining of a number of three-inch diameter Shelby tubes at each depth selected for sampling. This was considered necessary to provide sufficient material for the number of strength and consolidated test required.

The undisturbed samples were obtained by using the hydraulic-feed system of the drill to push the tubes into the ground. The tubes were then gently rotated to shear off the lower face of the soil. The tubes were then raised to the ground surface where they were prepared for taking to the laboratory. The tubes were cleaned, the ends sealed with wax and then labelled. Samples were also obtained from the auger for moisture content and identification tests.

In the laboratory the undisturbed samples were removed from the Shelby tubes, cut to convenient lengths, wrapped first in plastic film, and then with aluminium foil, completely coated with wax, labelled and stored in the moisture room. Material remaining from trimming the

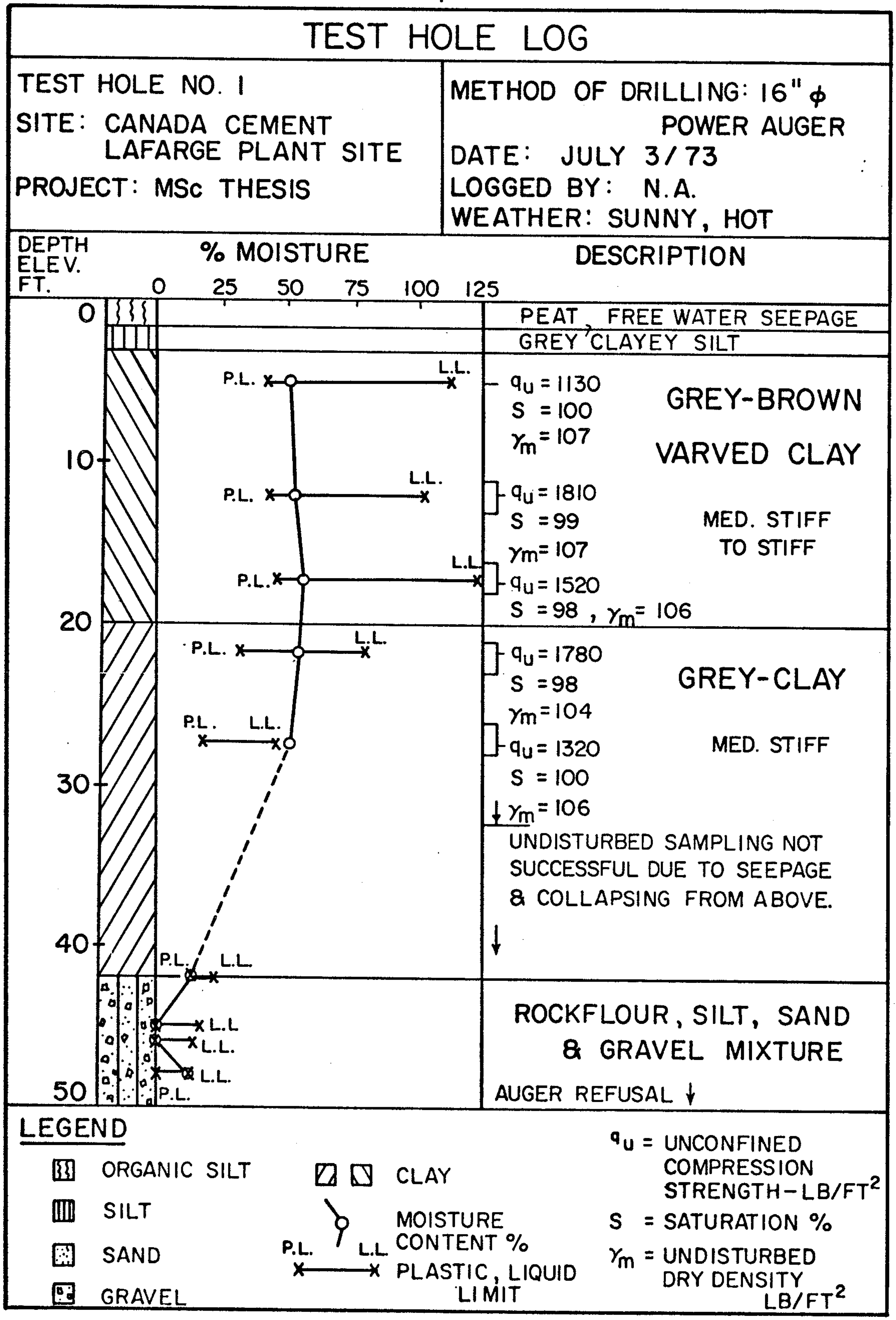

TEST HOLE LOG
TEST HOLE NO. 1
SITE: CANADA CEMENT LAFARGE PLANT SITE
PROJECT: MSc THESIS
METHOD OF DRILLING: 16" φ POWER AUGER
DATE: JULY 3/73
LOGGED BY: N.A.
WEATHER: SUNNY, HOT
DEPTH ELEV. FT.
% MOISTURE
0 25 50 75 100 125
DESCRIPTION
0
10
20
30
40
50
PEAT, FREE WATER SEEPAGE
GREY CLAYEY SILT
P.L.
L.L.
qu = 1130
S = 100
γm = 107
GREY-BROWN VARVED CLAY
qu = 1810
S = 99
γm = 107
MED. STIFF TO STIFF
qu = 1520
S = 98, γm = 106
qu = 1780
S = 98
γm = 104
GREY-CLAY
qu = 1320
S = 100
γm = 106
MED. STIFF
UNDISTURBED SAMPLING NOT SUCCESSFUL DUE TO SEEPAGE & COLLAPSING FROM ABOVE.
ROCKFLOUR, SILT, SAND & GRAVEL MIXTURE
AUGER REFUSAL
LEGEND
ORGANIC SILT
SILT
SAND
GRAVEL
CLAY
MOISTURE CONTENT %
PLASTIC, LIQUID LIMIT
qu = UNCONFINED COMPRESSION STRENGTH-LB/FT2
S = SATURATION %
γm = UNDISTURBED DRY DENSITY LB/FT2

FIGURE 1

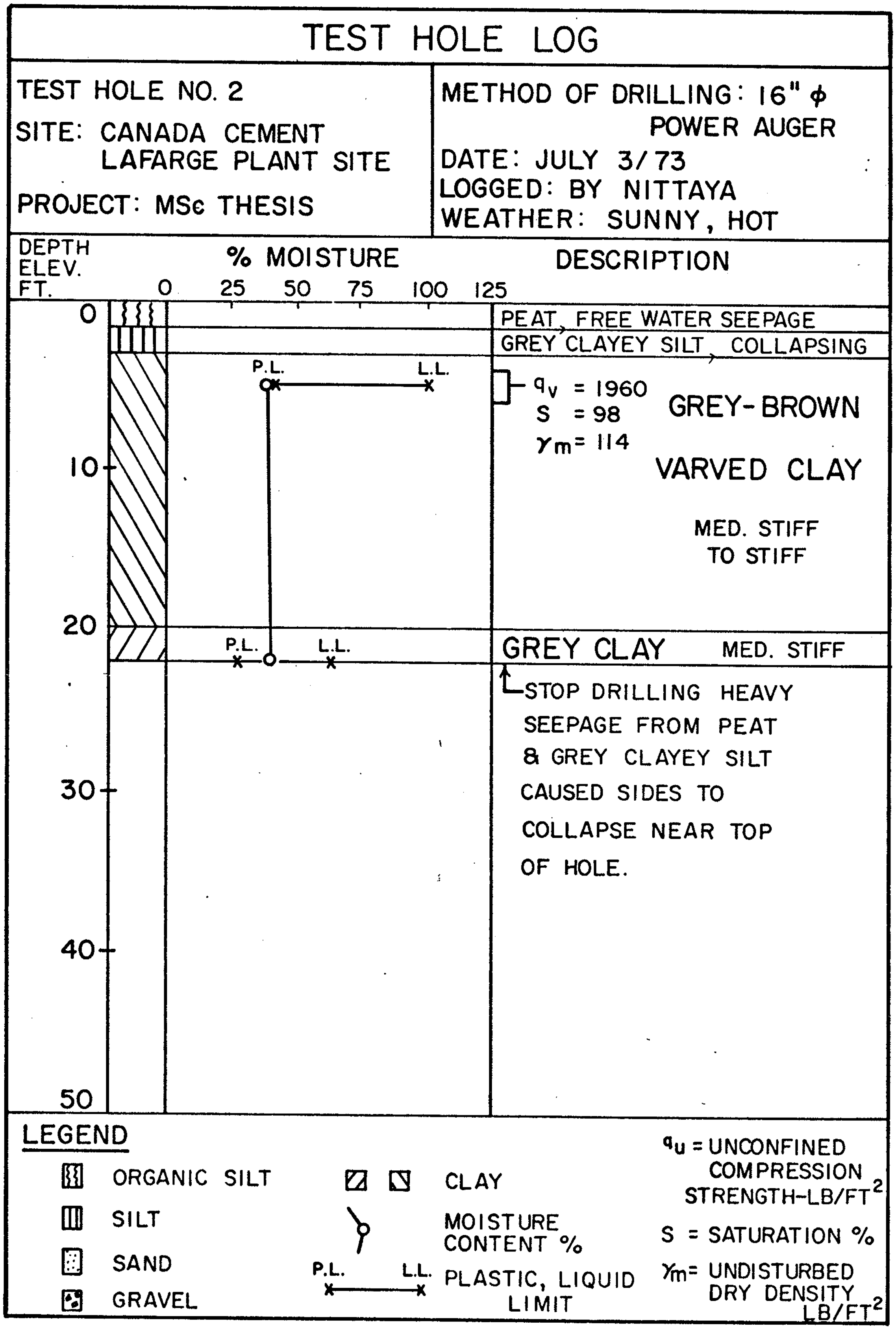

TEST HOLE LOG
TEST HOLE NO. 2
SITE: CANADA CEMENT LAFARGE PLANT SITE
PROJECT: MSc THESIS
METHOD OF DRILLING: 16" φ POWER AUGER
DATE: JULY 3/73
LOGGED: BY NITTAYA
WEATHER: SUNNY, HOT
DEPTH ELEV. FT.
% MOISTURE
0 25 50 75 100 125
DESCRIPTION
0
10
20
30
40
50
PEAT, FREE WATER SEEPAGE
GREY CLAYEY SILT, COLLAPSING
P.L.
L.L.
qv = 1960
S = 98
γm = 114
GREY-BROWN
VARVED CLAY
MED. STIFF
TO STIFF
P.L.
L.L.
GREY CLAY
MED. STIFF
STOP DRILLING HEAVY SEEPAGE FROM PEAT & GREY CLAYEY SILT CAUSED SIDES TO COLLAPSE NEAR TOP OF HOLE.
LEGEND
ORGANIC SILT
SILT
SAND
GRAVEL
CLAY
MOISTURE CONTENT %
P.L.
L.L.
PLASTIC, LIQUID LIMIT
qu = UNCONFINED COMPRESSION STRENGTH-LB/FT2
S = SATURATION %
γm = UNDISTURBED DRY DENSITY LB/FT2

FIGURE 2

sample was immediately used for moisture content tests. Grain size, plastic and liquid limit tests were done on representative bulk samples, and material remaining after the undisturbed tests were completed.

2 Results of Field Tests

The test holes show that the top approximate 1.5 feet consisted of peat. This was found to be underlain by about 1.5 feet of grey clay silt. The silt is stiff, fine-grained and non-plastic. No laboratory tests were performed on this material.

Underlying the silt, a grey-brown varved clay was found to a depth of about 20 feet. This material has a plasticity index ranging from 61 to 73, and liquid limit ranging from 103 to 123. The degree of saturation is very nearly 100% which can be assumed as fully saturated. The grain size test showed a predominance of clay, as high as 81% finer than 0.002 mm. The unconfined compression strengths of 1130 to 1810 psf indicate medium stiffness.

Below the 20-foot depth, grey clay was found extending to the 42-foot depth. It contained a fair amount of gypsum pockets and also silt varves and pockets. The clay content is less than for the grey-brown clay, and ranges from 24 to 62%. Towards the bottom it was sandy. The unconfined compression strengths of 1780 and 1320 psf indicated medium stiffness. It was of low permeability. The plastic index was in the range of 28 to 49 and the liquid limit was about 46 to 80. The degree of saturation was 99%, and the water content varied from 50 to 55%.

A rock flour, silt, sand, and gravel mixture layer extends below the 42-foot depth. Drilling was stopped because of auger refusal at the depth of 50 feet. The mixture contains about 23 to 30% sand and about 39 to 47% silt, and the rest is clay. Most of the gravel is crushed limestone in subangular shapes with diameter between 0.1 to 1.0 inch.

Seepage was encountered from the upper organic and grey silt layers. This interfered with undisturbed sampling below the 32-foot depth in test hole 1, and made sampling impossible below the 6-foot depth in test hole 2.

The test holes are shown in Figure 1 and 2. The soil properties are summerized in Table 1.

File No. ____________

TABLE 1 - LABORATORY TEST SUMMARY SHEET

Test Hole No	Depth ft	Sample No	Moisture Content %	Degree of Saturation %	Specific gravity	Moist Density lb/cu ft	Dry Density lb/cu ft	Strength Tests: Lateral Confining Pressure psi	Strength Tests: Deviator Stress at Failure psf	Strength Tests: Angle of Internal Friction Degrees	Strength Tests: Cohesion lb/sq ft	M I T Grain size Distribution: Clay %	M I T Grain size Distribution: Silt %	M I T Grain size Distribution: Sand %	M I T Grain size Distribution: Gravel %	Liquid Limit	Plastic Limit	Plasticity Index	Description Comments
1	6		48.50													114	41	73	
			51.80	100	2.75	108	71	-	1130	-	-	73	25	2	-				
1	11-13		56.80													103	42	61	
			48.00	99.0	2.75	107	72	-	1810	-	-	81	19	-	-				
1	17	A	51.90																
		B	61.20													123	44	79	
	16-18		54.60	98.2	2.75	106	68	-	1520	-	-	70	27	3	-				
1	21		54.90													80	31	49	
			55.20	98.3	2.75	104	67	-	1780	-	-	49	49	2	-				
1	26		50.25													46	18	28	
	28		53.90	99.5	2.75	106	70	-	1320	-	-	62	34	4	-				
1	42		14.30									24	47	23	6	23	14	9	
1	45		0.40									17	43	30	10	17	N.P.	17	
1	46		1.90									15	40	29	14	15	N.P.	15	
1	48		5.60									16	39	28	17	17	N.P.	17	
2	5		37.70													101	39	62	
			39.50	97.80	2.75	114	81	-	1960	-	-	83	14	3	-				
2	22		40.00									44	54	2	-	68	27	41	

Project M. Sc. Thesis

Tests Requested by Canada Cement Lafarge Oil Tank

Address Winnipeg, Canada

Date Submitted 21 July 1973 Checked by N.A.

SOIL MECHANICS LABORATORY
DEPARTMENT OF CIVIL ENGINEERING
UNIVERSITY OF MANITOBA
FORT GARRY MANITOBA

CHAPTER III

ROUTINE LABORATORY TESTS AND RESULTS

1 Classification and Moisture Content

Where possible standard test procedures were used for the laboratory tests as follows:

Liquid Limit	ASTM, D423-66
Plastic Limit	ASTM, D424-59
Moisture Content	ASTM, D2216-71
Grain Size	ASTM, D422-63

The results of these tests are summarized in Table 1. Grain size curves as shown in Figure A-1 and A-2 of Appendix A.

2 Consolidation Test

The consolidation tests on undisturbed samples were performed using 2.53-inch diameter floating ring consolidometers. Typical test data are shown in Appendix B. Testing was used by the procedure outlined in the University of Manitoba, Civil Engineering, Soil Testing Laboratory Manual, and calculations were performed using the laboratory computer program. It should be noted that the final void ratios were based on assumed 100% saturation at the end of the test, and all other void ratios referred to the final void ratios by calculation based on measured deflections.

Table 2 summarizes the results of the consolidation tests. Figure 3 shows the pressure vs void ratio relationship for the samples tested.

TABLE 2 - SUMMARY OF CONSOLIDATION TEST--CANADA CEMENT LAFARGE PLANT SITE

HOLE NO.	DEPTH--FT.	INITIAL MOISTURE CONTENT %	PRECONSOLIDATION PRESSURE, kg/cm^2	SWELLING PRESSURE kg/cm^2	COMPRESSION INDEX	COEFFICIENT OF CONSOLIDATION cm^2/sec FOR PRESSURE INCREMENT (kg/cm^2) OF:					COEFFICIENT OF PERMEABILITY cm/sec FOR PRESSURE INCREMENT (kg/cm^2) OF:				
						0.15 to 0.29	0.29 to 0.71	0.71 to 1.41	1.41 to 2.80	2.80 to 5.60	0.15 to 0.29	0.29 to 0.71	0.71 to 1.41	1.41 to 2.80	2.80 to 5.60
1	11-13	58.9	0.92	0.98	0.44	$5.55x10^{-5}$	$1.35x10^{-4}$	$4.79x10^{-5}$	$5.08x10^{-5}$	$4.95x10^{-5}$	$3.67x10^{-9}$	$8.56x10^{-9}$	$2.10x10^{-9}$	$1.44x10^{-9}$	$1.05x10^{-9}$
1	16-18	55.6	1.40	0.01	0.58	---	$8.17x10^{-5}$	$8.47x10^{-5}$	$9.10x10^{-5}$	$1.23x10^{-4}$	---	$4.62x10^{-9}$	$3.15x10^{-9}$	$2.89x10^{-9}$	$3.20x10^{-9}$
1	21-23	54.4	1.28	0.44	0.53	---	$1.66x10^{-4}$	$1.57x10^{-4}$	$1.66x10^{-4}$	$7.73x10^{-5}$	---	$9.30x10^{-9}$	$6.47x10^{-9}$	$5.22x10^{-9}$	$2.01x10^{-9}$
1	26-28	43.2	1.02	0.11	0.44	---	$1.96x10^{-4}$	$3.74x10^{-4}$	$4.69x10^{-4}$	$1.39x10^{-4}$	---	$9.77x10^{-9}$	$1.52x10^{-8}$	$1.38x10^{-8}$	$4.57x10^{-9}$
2	4-6	43.2	0.88	1.85	0.44	---	$4.83x10^{-5}$	$4.45x10^{-5}$	$4.77x10^{-5}$	$4.37x10^{-5}$	---	$2.69x10^{-9}$	$2.07x10^{-9}$	$1.24x10^{-9}$	$7.63x10^{-10}$

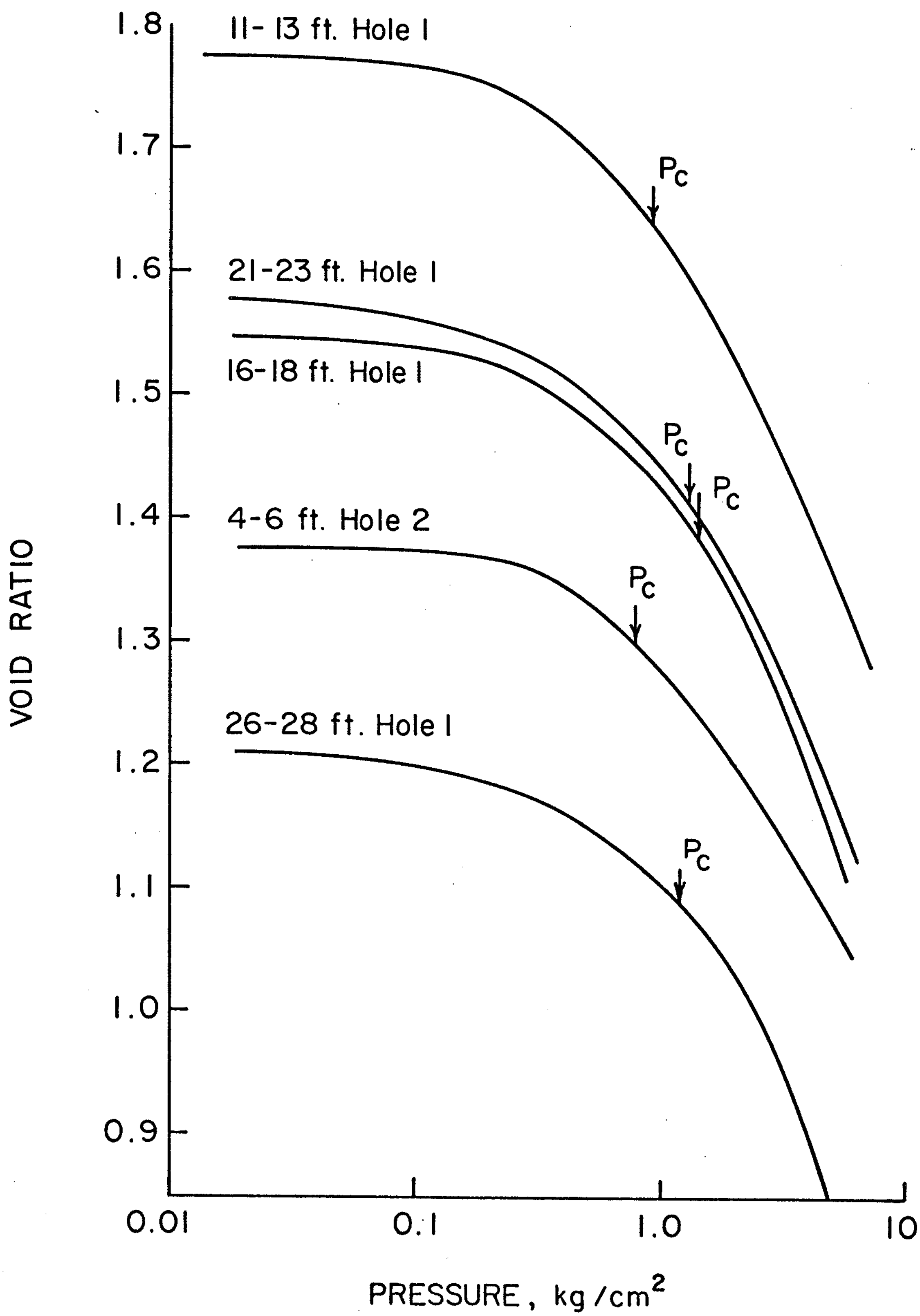


FIGURE 3. Consolidated test results; Pressure (log scale) void ratio curves (Canada Cement; Lafarge plant site; Winnipeg, Man.

3 Unconfined Compression Test

Unconfined compression tests were performed on the undisturbed samples extruded from the Shelby tubes. Typical test data are shown in Appendix C. The samples were measured and weighed prior to testing. They were then placed on an air-operated testing machine and strained at an approximate rate of 1% per minute until failure occurred. The sample was then placed in the oven. Moisture content, degree of saturation, void ratio and moist and dry densities were determined for all samples. The results are summarized in Table 1 and Figure 1. Stress-strain curves for all samples tested are shown in Figure 4.

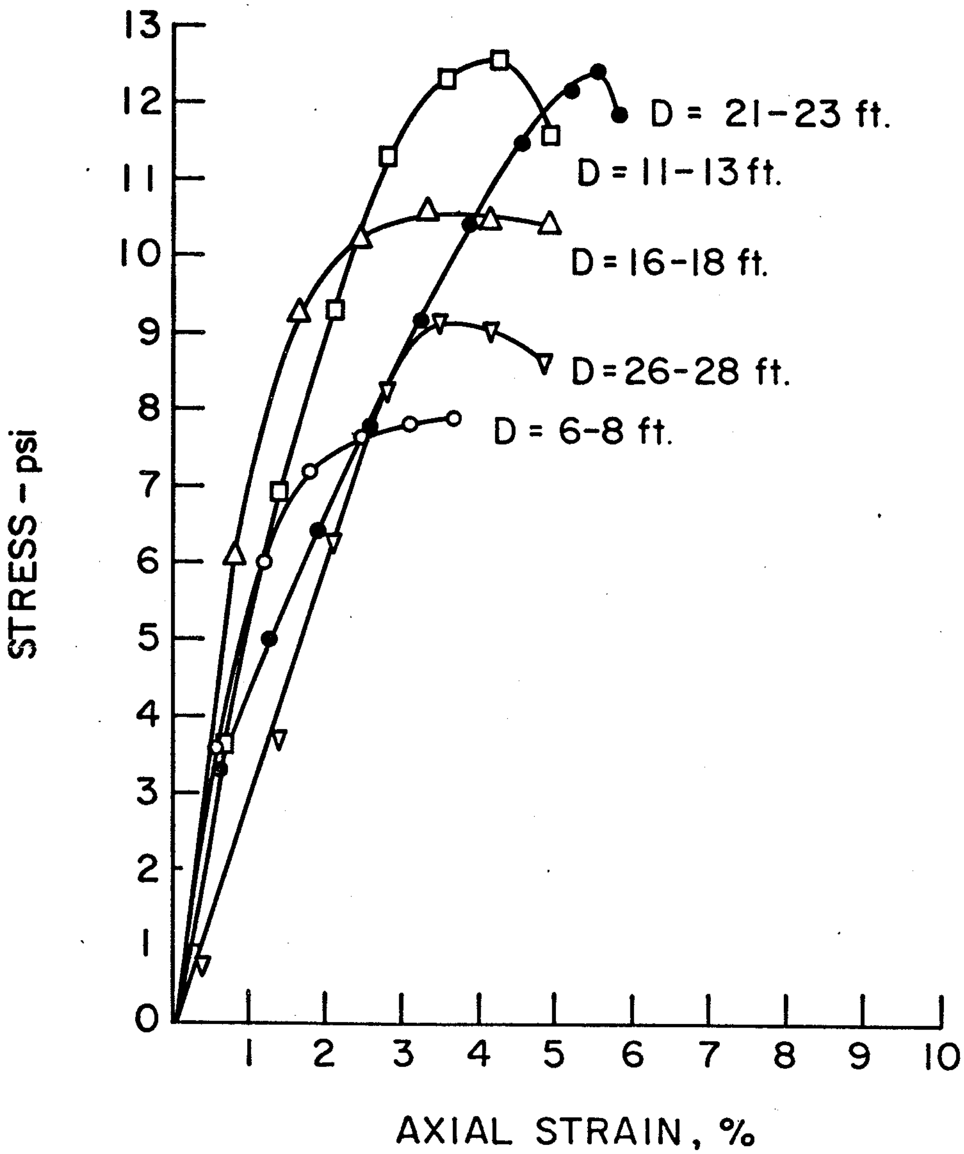


FIGURE 4. Stress vs. strain curves for unconfined compression test.

CHAPTER IV

TEST FOR SHEAR STRENGTH AND PORE PRESSURE PARAMETERS

1 Triaxial Test

A brief description is given here for the triaxial tests used in the study. Standard procedures used were those described in Bishop and Henkel[1].

1.1 Equipment

Standard 1.4-inch diameter triaxial cells were used as shown in Figure C-1 of Appendix C.

The systems of controlling the cell pressure were the air pressure system for undrained tests, and the self-compensating mercury control system as shown in Figure C-2 of Appendix C for drained tests. The disadvantage of using the air pressure control system was the possibility of dissolved air entering the sample through the membranes under high all-around cell pressure in tests lasting several days. Also, in long term tests, if the electricity was turned off accidentally, the cell pressure would drop and then the sample would fail.

The pore pressure measuring system was the transducer type. It was considered as the most sensitive and accurate pore pressure measuring equipment giving accuracy in the order of 0.1 psi. Also, it permitted a wide range of pressures from 0 up to 100 psi. The transducer amplifier-indicator used in the experiments was the "Daytronic" Model 300 I.

1.2 General Procedure

The procedure for triaxial tests can be divided into a consolidation and shearing stage. In the consolidation stage, all types of the triaxial test were performed in the same way except that in undrained tests a back pressure was used to obtain 100% saturation in soil samples. The reason for using the back pressure was to ensure full saturation and to prevent dissolved air from coming out of solution which could interfere with the pore pressure readings. In the shearing stage, the procedures depended on how samples were brought to failure and will be discussed later.

1.3 Preparation of Samples

The triaxial test samples were prepared from the undisturbed soils kept in a moisture storage room. The three-inch diameter undisturbed soil samples were cut longitudinally into three specimens and trimmed into approximately 1.4 inches in diameter on a soil lathe. The ends were trimmed perpendicular to the axis to obtain a specimen length of approximately 1.5 to 2 times the diameter. Moisture contents of specimens were obtained using sample trimmings. Samples were then measured and weighed. The above were done rapidly to prevent drying.

1.4 Consolidation of Triaxial Test Specimens

The cell base was filled with de-aired water. A burette was connected to the pore pressure outlet on the base of the cell. De-aired water was allowed to flow back from the burette to cover the pedestal to ensure there was no air trapped in the lines. A saturated porous stone was slid onto the top of the pedestal. The sample was then placed on the porous stone. Another porous stone was placed on top of the sample. Then saturated 1/4-inch wide filter strips were placed vertically around the perimeter of the sample. Two to three rubber membranes were applied to the sample with a thin coating of grease between them to prevent leakage. A loading cap was put on top of the sample after ensuring that there was no air between the sample and rubber membranes. The membranes were sealed to the pedestal and capped by means of two or three O-rings.

The cell was assembled in the test frame and about three-quarters filled with de-aired water. Oil was then used to fill the cell. The oil acts as a piston lubricant as small amounts leak out along the piston. Finally, any remaining air was expelled through the air valve by admitting more de-aired water. The required cell pressure was then applied. Dial gauge was then attached. Initial burette and dial gauge readings were recorded and the starting time noted. The sample was now ready for testing. As soon as the consolidation started, the burette and dial readings were recorded as close as possible at the following elapsed times; 1, 2, 4, 8, 15 and 30 minutes and 1, 2, 4, and 8 hours. The consolidation was completed when the volume change and the strain dial readings were constant. In the tests, all samples took about 48 hours to complete the consolidation stage. In the consolidated drained tests, as well as in the consolidated undrained triaxial extension and constant mean normal stress tests, the consolidation stage was done on a platform scale load frame. The triaxial cell was first positioned on the platform scale, the cell pressure was set to the required pressure and the balance weight was adjusted to read exactly the same amount as the weight on the platform and the weight used for counteracting the upward pressure on the piston. The balance weight was recorded. Then the dial gauge was set. The crank of the platform scale load frame was adjusted in order to bring the piston in contact with the bearing ball which sat on the loading cap, and consolidation was started. In consolidated-undrained triaxial compression tests, all samples were set on the controlled strain testing machine which is electrically driven. First, the piston was brought to contact the bearing ball, then the dial gauge and strain dial were initially set. The consolidation was started.

1.5 Shearing Stage

In the shearing stage the samples were sheared by applying the axial stress or increasing the cell pressure, and the samples were brought to failure by either controlling strain or stress. If pore pressure was measured, the connection to pore pressure measuring system was made before shearing started. If the volume change was measured, shearing of the sample could be done immediately after the consolidation stage had finished. The details of procedure in shearing stage for each type of triaxial test are described as follows.

1.5.1 Consolidated-Undrained Triaxial Compression Test With Pore Pressure Measurement

The shearing stage of this test was done on the controlled strain testing machine. A back pressure of 10 psi was used to assure saturation. After consolidation stage finished, the transducer was connected through the pore pressure valve. Then the valve connected to the burette was closed and the burette was removed. Before shearing started, the pore pressure was checked to assure it read the same as the back pressure. If this was not the case the transducer was set to give a reading equal to the back pressure. The rate of strain was selected to be 0.0002 inches per minute. (This rate of strain is very slow and ensures the equilibrium of the pore pressure in the whole sample.) The strain dial readings and pore pressures were recorded at every 0.010 inch deflection until failure occurred. Two or three more readings were made after failure had taken place. Typical data are shown in Appendix D.

1.5.2 Consolidated Undrained Triaxial Extension Test With Pore Pressure Measurement

The tests were conducted the same way as were the compression triaxial tests, but instead of applying the deviator stress until samples failed, the all-around cell pressure was increased and the vertical pressure was kept constant by dead loading through a hanger. The shearing stage was performed on the platform scale. The base of the triaxial cell was clamped to the platform and also the top of the cell was clamped to the frame to prevent the cell from lifting up when the axial extension force was applied to maintain constant vertical pressure during the shearing stage. Before the stage was commenced, following the completion of the consolidation test, the loading cap was attached to the piston to take tension. For high initial all-around cell pressures, sufficient weights were placed on the platform to ensure that at high tensions, the platform scale readings remained greater than zero. The corrected area of the sample was determined after each load increment and the calculation was made to determine the tension force which was needed to keep the vertical stress constant. The dial gauge readings and the pore pressures were recorded in the same manner as they were for the compression tests. The loading cap used in extension tests is described by Bishop and Henkel[1]. Typical data are shown in Appendix D.

1.5.3 Pore Pressure Parameter B and A

The pore pressure parameters B and A were obtained using the undrained triaxial tests. In determining the pore pressure parameter B, the sample was prepared and set in the triaxial cell, the initial cell pressure was selected at 10 psi, the pore pressure was measured by using the transducer. Then the cell pressure was increased to 20, 30, 40, 50, 60, 70, 80, 90 and 100 psi. The pore pressure at each cell pressure was recorded after 15 minutes of elapsed time to permit equilibrium of the pore pressure in the sample. The cell pressure vs pore pressure curve was plotted to obtain the average value of B as shown in Figure 26.

The pore pressure parameter A can be obtained during shearing stage of consolidated undrained compression triaxial test. The value of this parameter at failure, A_f, was calculated for each confining pressure used in the standard tests. The pore pressure parameter, A_f, vs confining pressure for a series of undrained compression test was plotted as shown in Figure 27.

1.5.4 Consolidated Drained Test

After the consolidation of the triaxial samples was complete, shearing of the samples was done by applying the deviator stress until failure occurred. Using the platform scale load frame, the load increment was added by placing the required weight on the hanger. Then the load crank was adjusted. About a day was permitted for each sample to attain equilibrium before another load increment was applied. The dial and burette readings were recorded in the same manner used for the consolidation stage to ensure complete consolidation. As failure was approached, the load increments were reduced so that a more reliable determination of the failure stress would be made. Typical data are shown in Appendix D.

1.5.5 Consolidated-Drained With Constant Mean Normal Stress Test

In this test the soil samples were brought to failure by applying the deviator stress and decreasing the cell pressure at the same time, so that the mean normal stress at shearing stage was equal to the mean normal stress at consolidation stage. Since this test was also performed on the platform scale load frame, it was necessary to apply the deviator stress in the manner used for the standard drained tests. Again, dial and burette readings were recorded and the corrected area had to be determined before applying the next load increment. The amount of each load increment was based on the basis of maintaining the axial deflection to be about 0.015 inch each time until failure occurred. Typical data are shown in Appendix D.

1.6 Completion of Test

After completion of the shearing stage, the pore pressure measuring system for undrained test as well as the volume change system for drained test were disconnected. The strain dial was removed from the testing machine and the pressure valve was closed. The cell was brought to the suitable place and was disassembled. The sample was removed for weighing and obtaining the moisture contents.

2 Direct Shear Test

2.1 Equipment

The shear box used in the Soil Mechanics Testing Laboratory, University of Manitoba, is the constant rate of strain shear box. It is based on the design of A. W. Bishop, Imperial College of Science and Technology, London and made by Wykeham Farrance Engineering Limited. The shear box is a square box 5.52 inches square which is split in half horizontally. Normal loads are applied to the specimen by a load hanger. An additional lever load device can be fitted to this hanger. Shear force is applied by screw jack either hand operated or power driven. The shear box runs on ball tracks guided in hardened and ground slots. A proving ring is used to measure the applied shear load. The machine and motor unit are mounted on a stand. The equipment is shown in Figure E-1 of Appendix E.

2.2 Procedure

The top half of the shear box was screwed down on top of the bottom half by the locating screws. A porous stone placed in the bottom was followed by a serrated grid, set with its serrations at right angle to the direction of shear. The sample which was already trimmed to size in the trimmer was then carefully pushed down into the shear box. The upper serrated grid was placed on top of the sample and again its serrations were at right angle to the direction of the shearing action. Water was added into the shear box to maintain moisture content. The dial gauge for consolidation measurement was attached on top of the hanger. The shear box was brought to bear against driving mechanism by using the hand wheel. Contact was indicated by a slight movement of the proving ring. The strain dial was then attached. Dial gauge, strain dial and proving ring were set at zero. Only the drained direct shear tests were performed since the purpose of the test was only to determine the residual shear strength. The test was started by applying the load on the hanger and consolidation of the sample took place. The dial readings were recorded. Consolidation was continued until 100% consolidation was indicated by the deflection vs time plots. The electric motor was engaged to commence shear. The shearing speed used was 0.000096 inches per minute which has been shown to assure complete pore pressure dissipation. The proving ring and strain dial readings were recorded at every 0.020 inch. The test was continued until the shear force was constant for a few readings. A series of tests were performed in order to obtain the peak and residual shear strength parameters. Final moisture contents were obtained using the samples after testing. The typical test data are shown in Appendix E.

CHAPTER V

RESULTS OF SHEAR STRENGTH TESTS

1 Triaxial Test Stress, Pore Pressure and Volume Change Relation to Strain

For the consolidated-undrained triaxial compression tests, the deviator stress, $(\bar{\sigma}_1-\bar{\sigma}_3)$ vs axial strain curves are shown in Figure 5 for different confining pressures (cell pressure minus back pressure). It can be seen that the relationship is expressed by a family of curves with higher deviator stresses corresponding to higher confining pressures for a given magnitude of strain. Figure 6 shows the effective principal stress ratio, $(\bar{\sigma}_1/\bar{\sigma}_3)$ vs axial strain for different confining pressures. At low confining pressure, i.e. less than 14 psi, the curves reach a much higher principal stress ratio than is the case for the high confining pressures. At confining pressures of 5 and 14 psi, the stress ratios increase at a rapid rate and attain their maximum values at very low strains, i.e. about 1.5% strain for the confining pressure of 5 psi and 1.2% strain for the confining pressure of 14 psi. At confining pressures greater than 14 psi, but lower than or equal to 58.5 psi, the stress ratios increase at a slower rate and still attain the maximum value at low strains, i.e. about 2% strain for the confining pressure of 30 psi, and 3% for the confining pressure of 58.5 psi. When the confining pressure is greater than 58.5 psi, the stress ratios increase at a very slow rate and attain their maximum values at about 4.5% strain. However, it can be said that the strains at failure are low in undrained compression tests. The maximum values of $(\bar{\sigma}_1-\bar{\sigma}_3)$ and $\bar{\sigma}_1/\bar{\sigma}_3$ donot occur

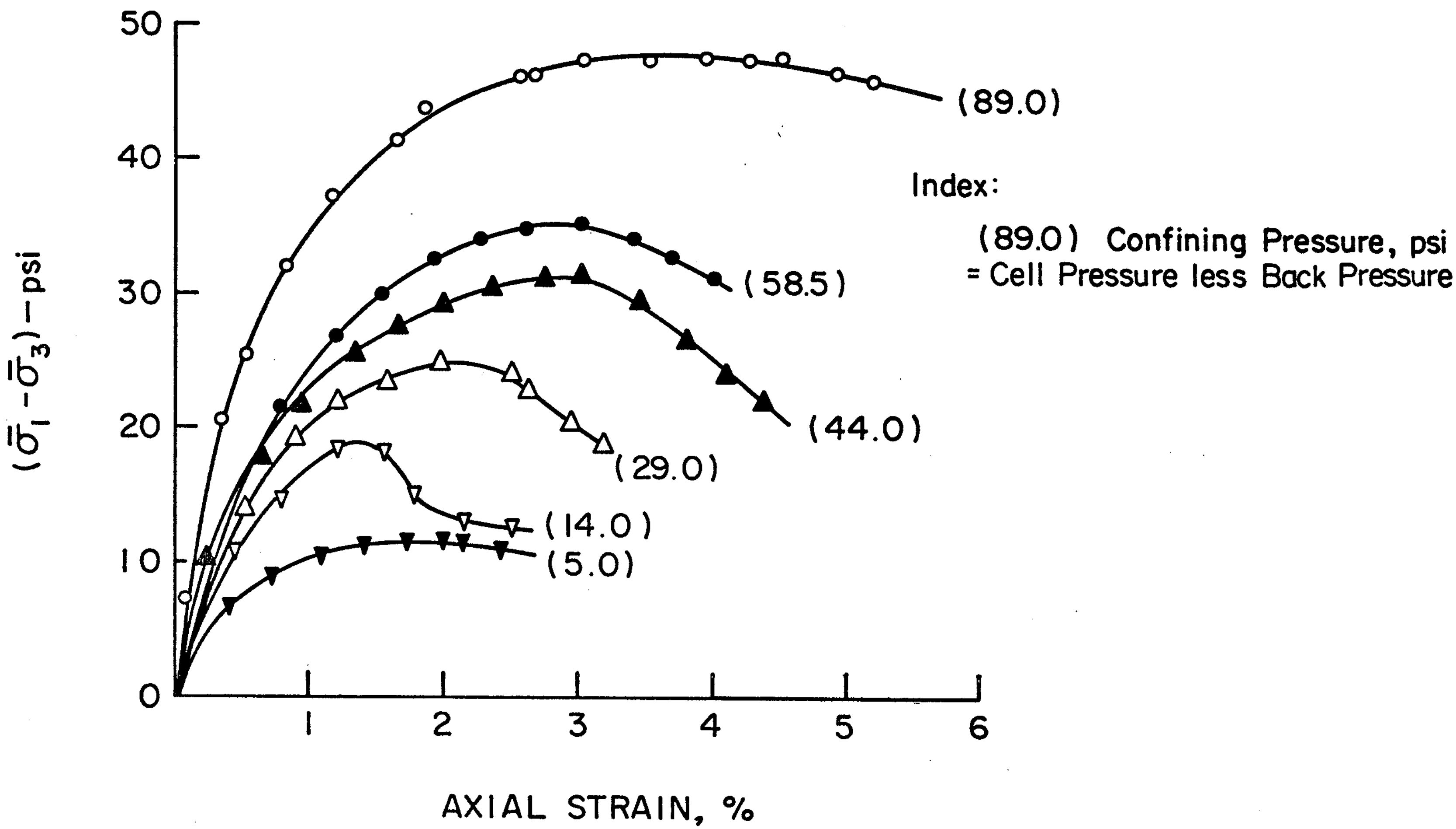


FIGURE 5. $(\bar{\sigma}_1 - \bar{\sigma}_3)$ vs strain curves for consolidated-undrained triaxial compression test.

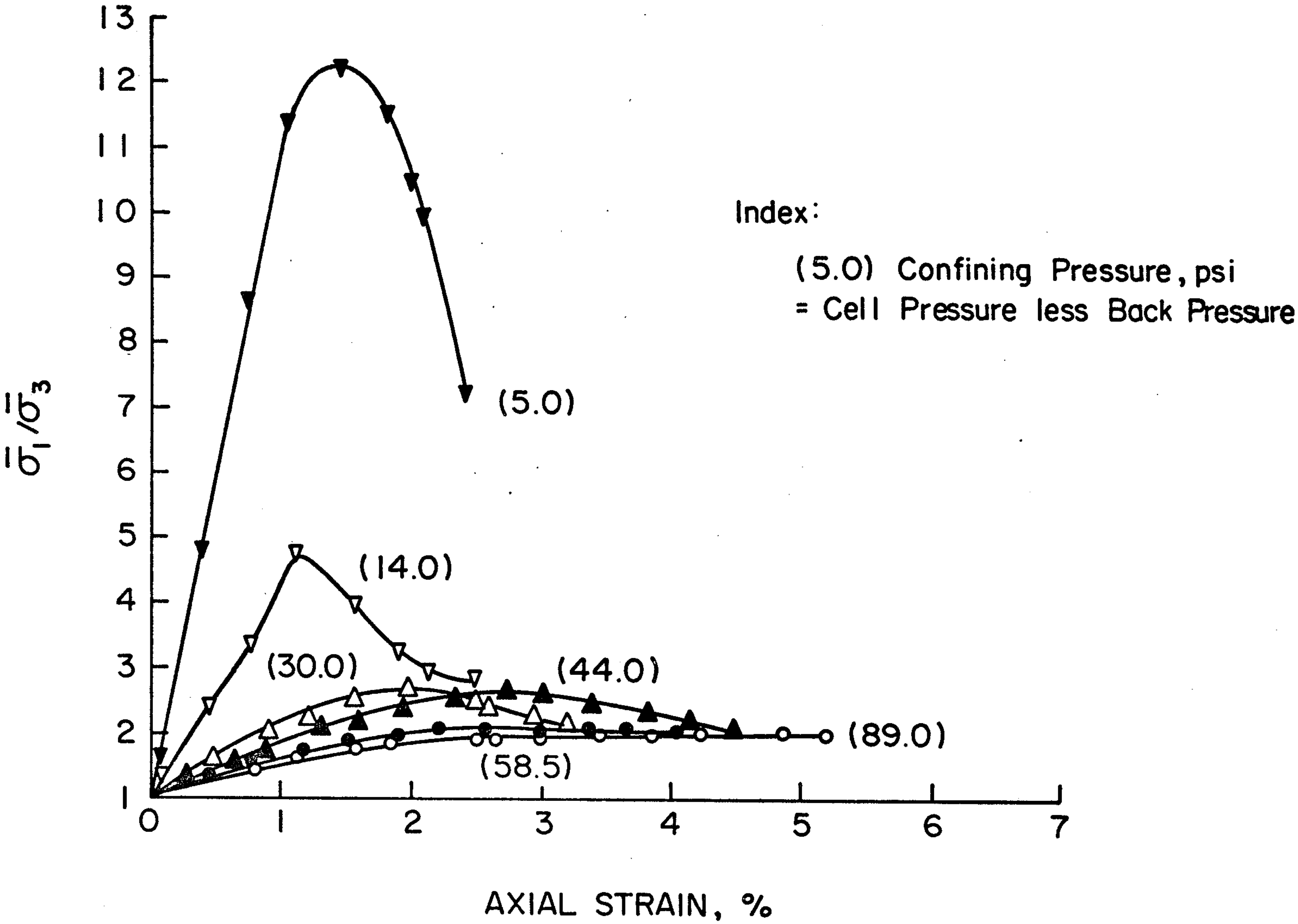


FIGURE 6. ($\bar{\sigma}_1/\bar{\sigma}_3$) vs. strain curves for consolidated-undrained triaxial compression test.

at the same axial strain in all the tests. At confining pressure of 5 and 14 psi, the maximum $\bar{\sigma}_1/\bar{\sigma}_3$ occurs before the maximum $(\bar{\sigma}_1-\bar{\sigma}_3)$ value. Also, samples tested at confining pressures greater than 30 psi, do not show the high $\bar{\sigma}_1/\bar{\sigma}_3$ peak. The pore pressure vs axial strain curves are shown in Figure 7. It can be noticed that for samples under confining pressure equal to or lower than about 44 psi, the maximum pore pressures are attained at the same strain as the maximum $(\bar{\sigma}_1-\bar{\sigma}_3)$. This is not the case when the confining pressure is greater than about 44 psi.

For consolidated-undrained triaxial extension tests, the deviator stress and the effective principal stress ratio vs axial strain curves are shown in Figure 8 and Figure 9 respectively. The curves show that the deviator stress increases in proportion to the confining pressure but this is not the case for the effective principal stress ratios. For the sample under an initial confining pressure of 59.5 psi the $\bar{\sigma}_1/\bar{\sigma}_3$ ratio decreases rapidly with increasing strain and reaches the same ratio as does the sample under initial confining pressure of 35 psi when failure plane was noticed. The pore pressure curves in Figure 10 show the increase in pore pressure with initial confining pressure. Therefore, it can be concluded from Figure 8 and Figure 10 that higher initial confining pressures correspond to higher values of deviator stress and pore pressure for all values of axial strain.

Figure 11 and Figure 12 show the curves of the deviator stress and effective principal stress ratio vs axial strain curves for the consolidated-drained compression tests. Again, as in the consolidated-undrained compression tests, the $\bar{\sigma}_1/\bar{\sigma}_3$ ratio for samples under confining pressure of 5 and 15 psi peak before the $(\bar{\sigma}_1-\bar{\sigma}_3)$ stress. The curves of

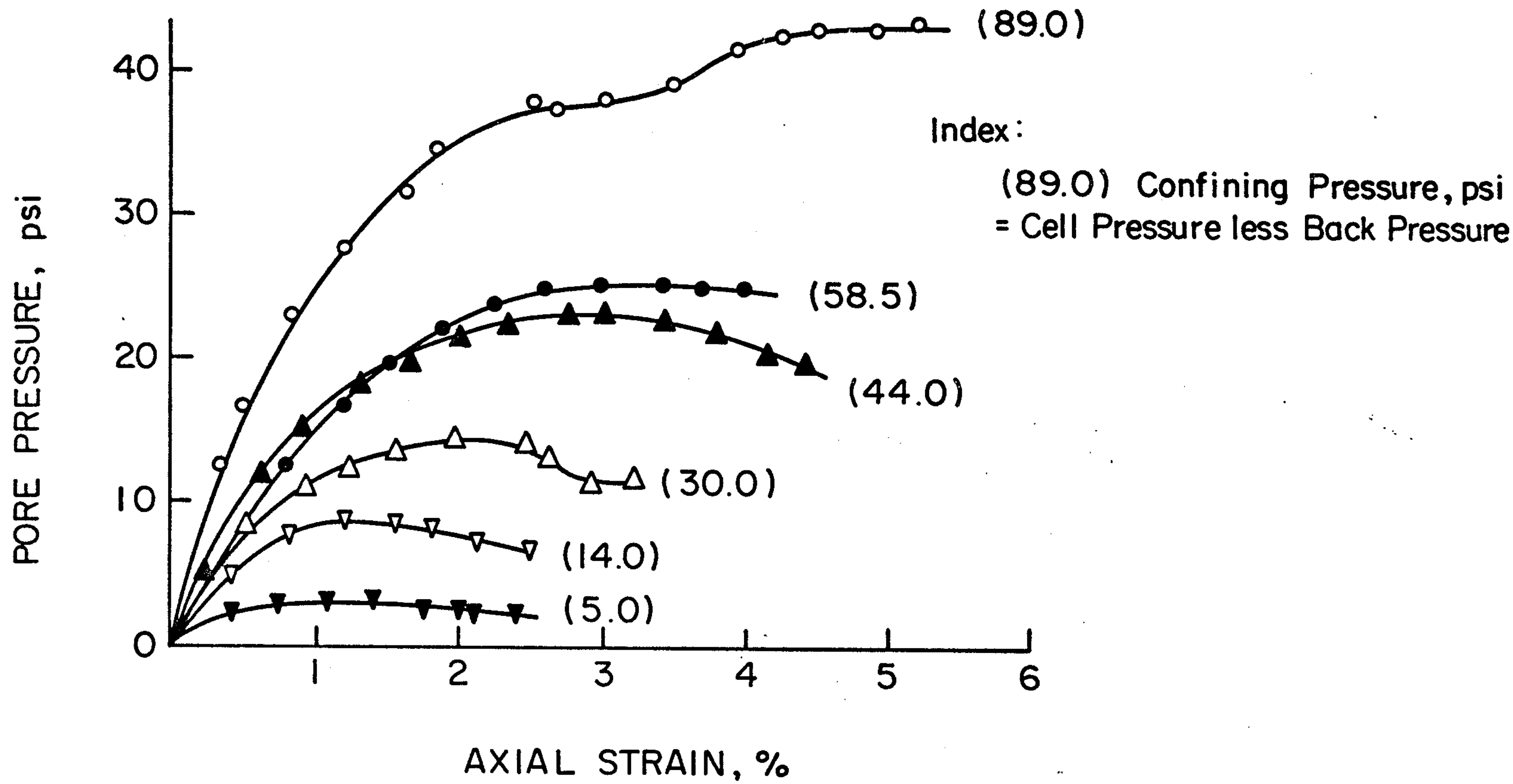


FIGURE 7. Pore pressure vs. strain curves for consolidated-undrained triaxial compression test.

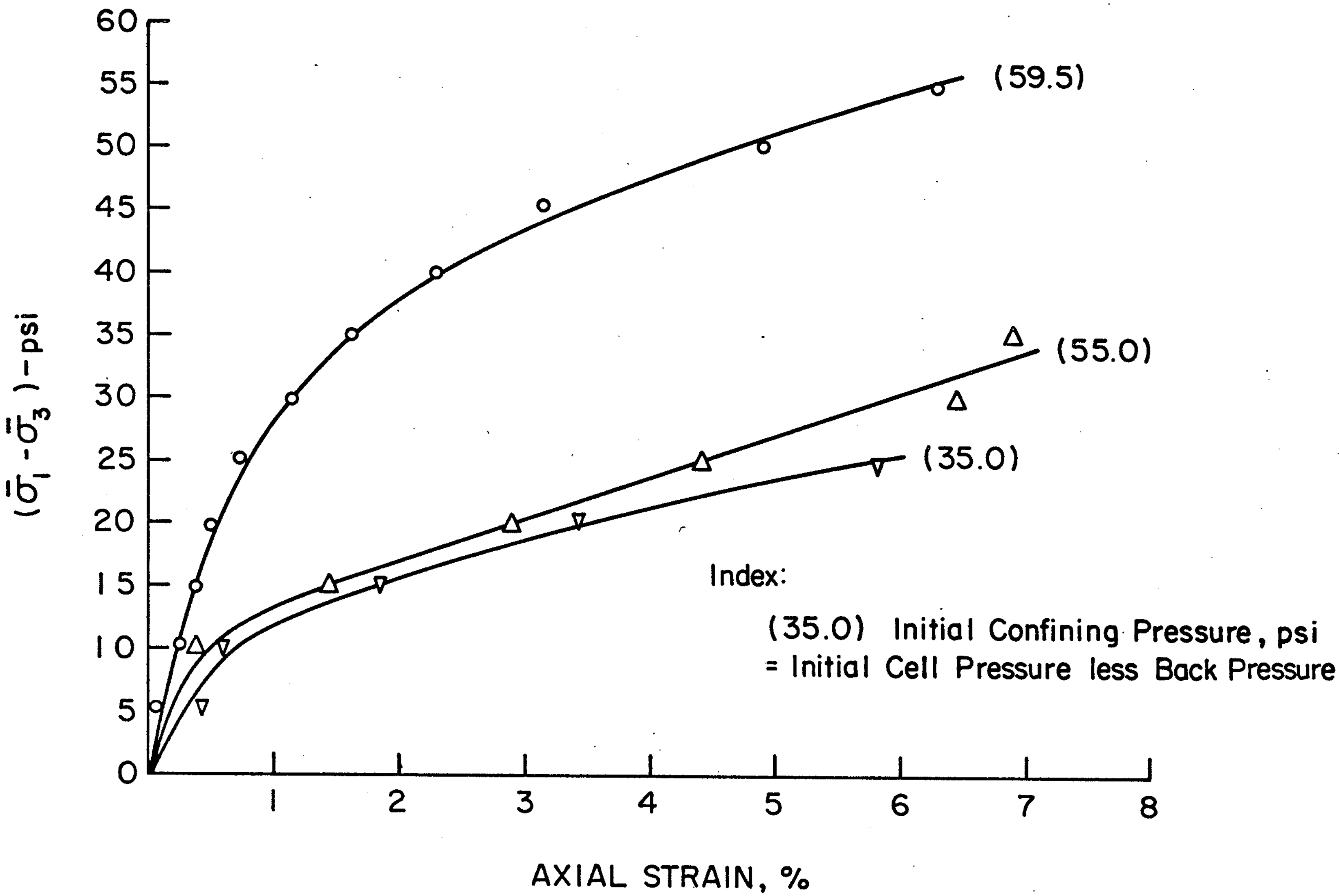


FIGURE 8. $(\bar{\sigma}_1 - \bar{\sigma}_3)$ vs strain curves for consolidated-undrained triaxial extension test

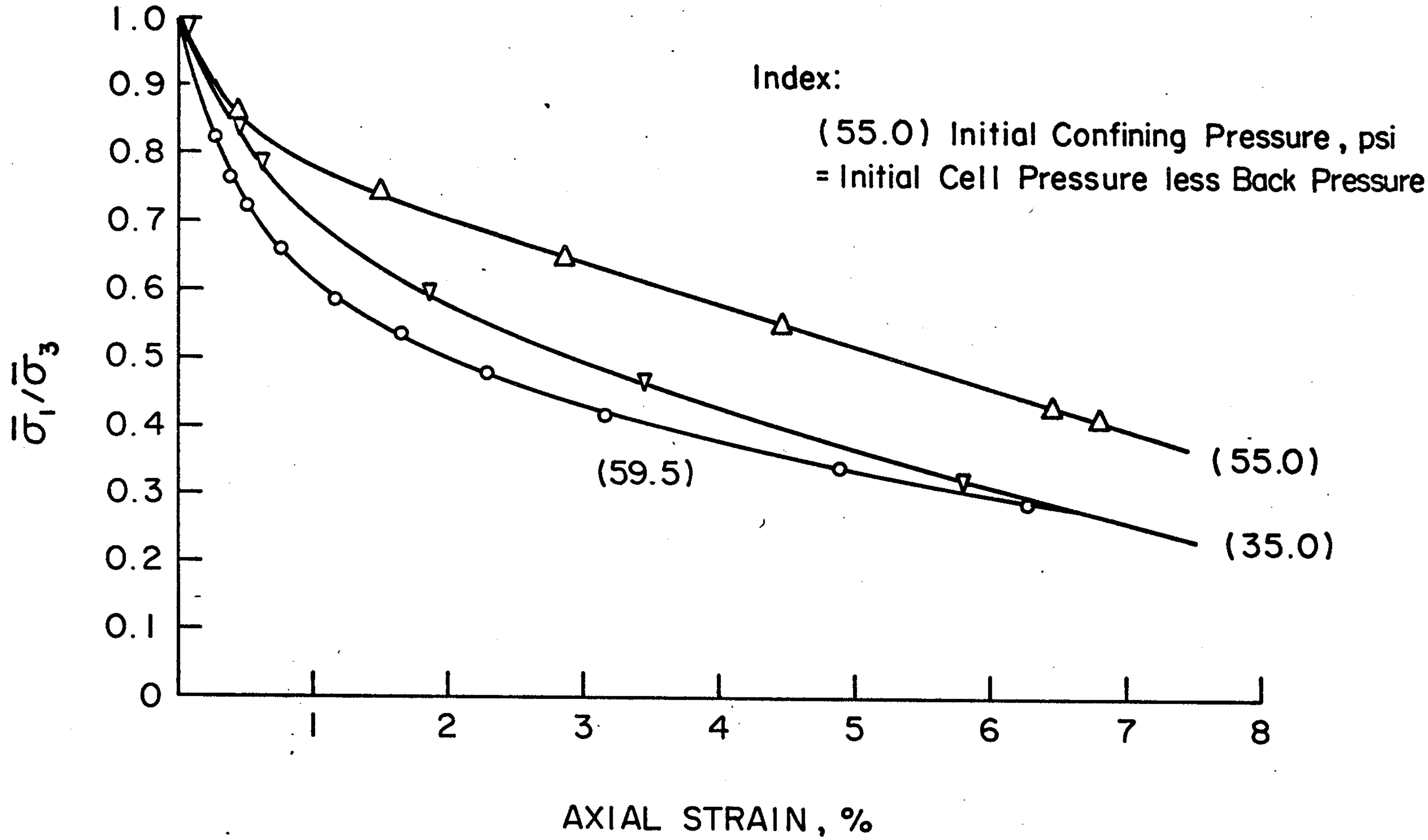


FIGURE 9. $(\bar{\sigma}_1/\bar{\sigma}_3)$ vs. strain curves for consolidated-undrained triaxial extension test

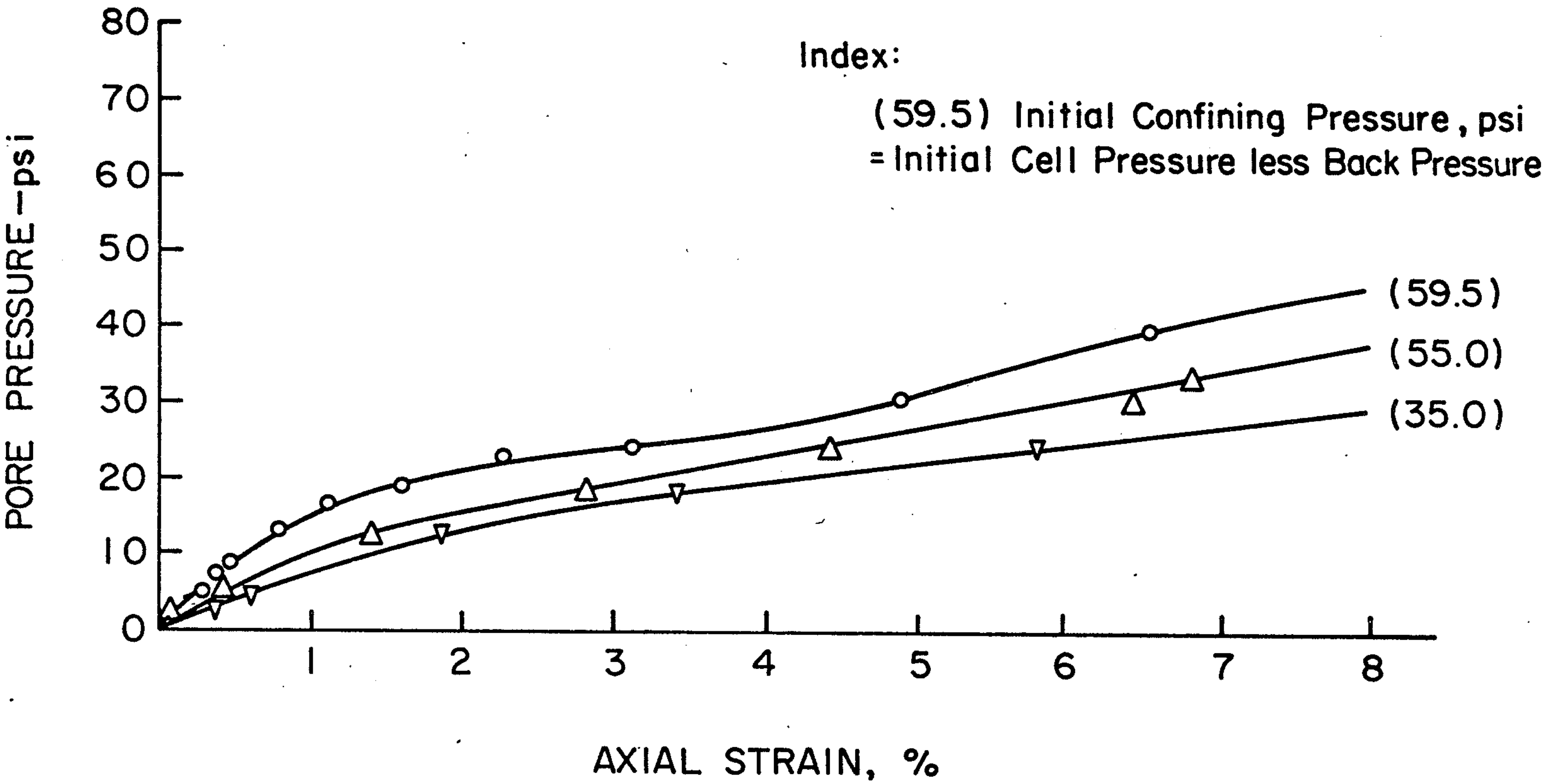


FIGURE 10. Pore pressure vs. strain curves for consolidated-undrained triaxial extension test

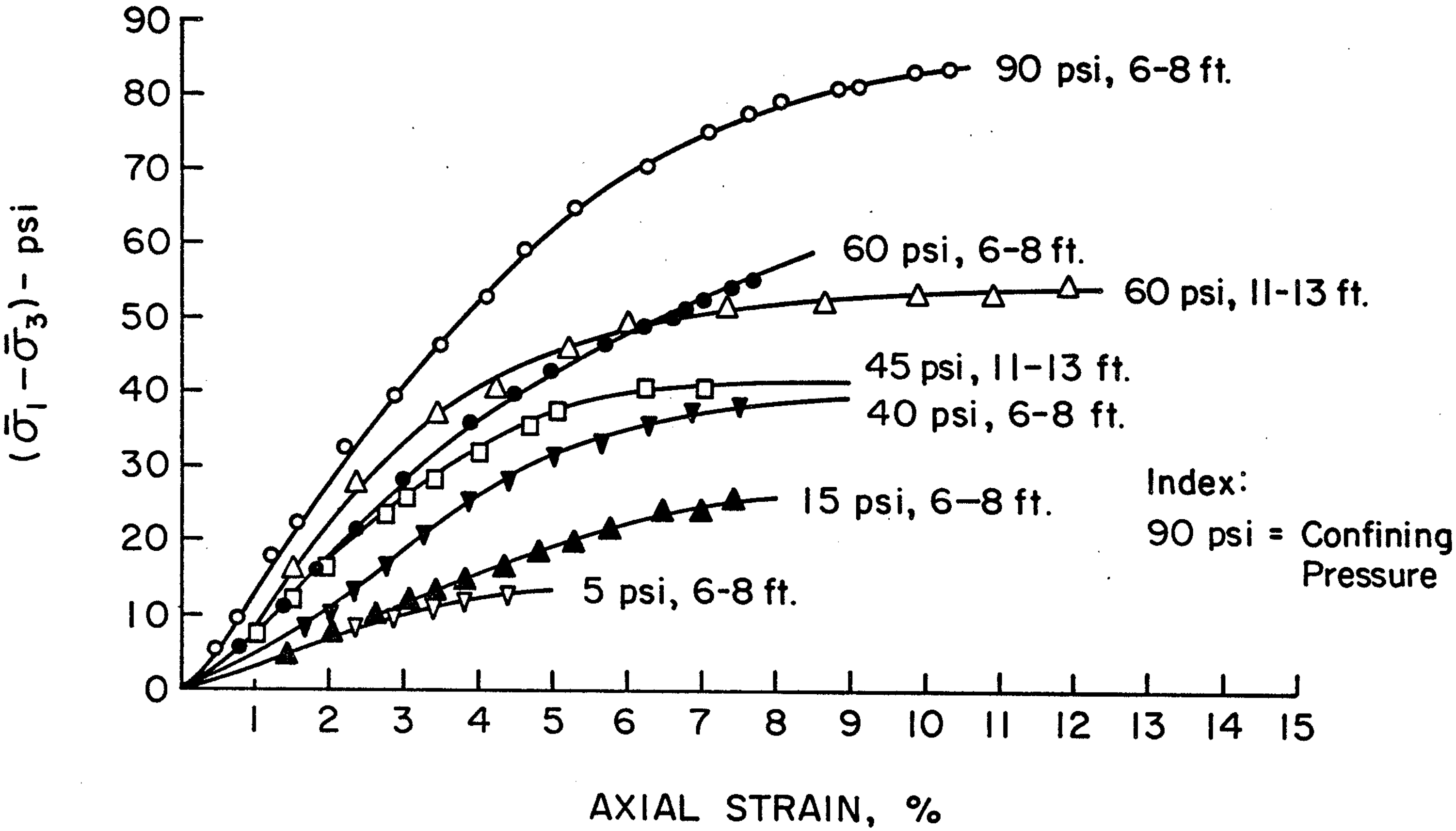


FIGURE 11. $(\bar{\sigma}_1 - \bar{\sigma}_3)$ vs. strain curves for consolidated-drained triaxial test

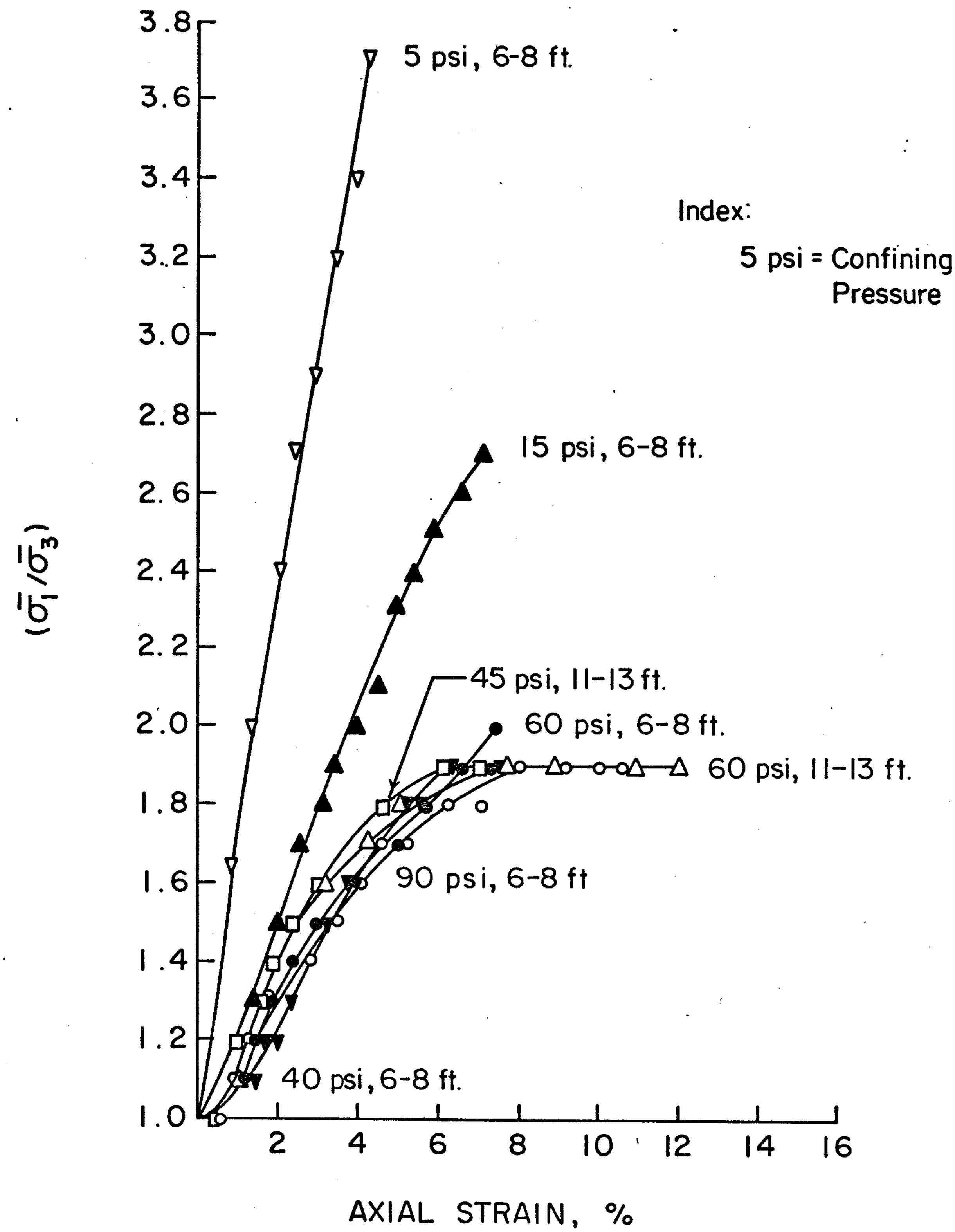


FIGURE 12. $(\bar{\sigma}_1/\bar{\sigma}_3)$ vs. strain curves for consolidated-drained triaxial test.

the $\bar{\sigma}_1/\bar{\sigma}_3$ vs axial strain for samples under confining pressures of 5 and 15 psi also have shapes different from the other curves. A similar difference is not seen in the $(\bar{\sigma}_1-\bar{\sigma}_3)$ vs axial strain curves. Samples in the consolidated-drained compression tests attain their maximum values of $(\bar{\sigma}_1-\bar{\sigma}_3)$ and $\bar{\sigma}_1/\bar{\sigma}_3$ at higher strains than do samples in the consolidated-undrained compression tests, i.e. at 4.2% and 1.5% axial strain respectively at confining pressures of 5 psi. The volume change vs strain curves are shown in Figure 13. The curves show the increase in volume at small strain and decrease in volume at large strain except for the samples obtained from the depth 6 to 8 feet tested at confining pressures of 5 and 60 psi. According to the theory (Bishop and Henkel[1]), a normally-consolidated clay will show a volume decrease during the shearing stage, and an over-consolidated clay will show a small volume decrease at small strain and dilation (volume increase) at a large strain. Therefore, the apparent dilation at small strain suggests the possibility of leakage in the drainage line between the cell and burette or possible evaporation from the burette during a long term test. Under the same conditions a normally-consolidated clay can show an incorrect dilation at small strain, and an apparently smaller than actual volume decrease at large strain. With this leakage or evaporation the over-consolidated clay can show incorrectly exaggerated dilation throughout the entire test. This has to be taken into account in interpreting the test results. From the volume change vs axial strain curves of the 11 to 13-foot depth samples, it appears that for a confining pressure of about 45 psi, the clay is over-consolidated, but at higher confining pressure the clay is normally-consolidated. The curves for the 6 to 8-foot depth samples suggest

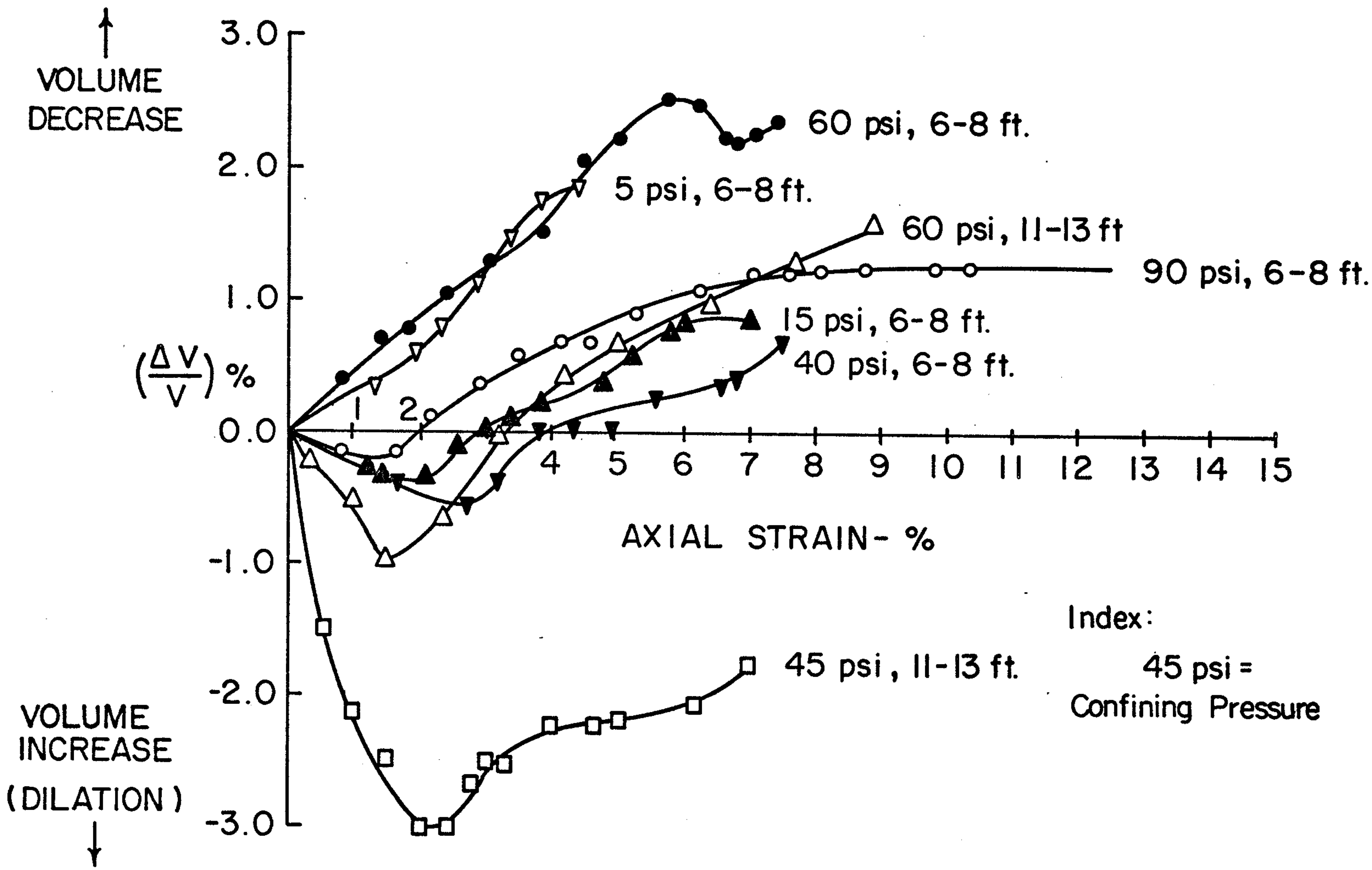


FIGURE 13. (ΔV/V) vs strain curves for consolidated-drained triaxial test.

possible membrane leakage for samples tested at confining pressures of 5 and 15 psi. At these low confining pressure, the samples should show dilation as expected for over-consolidated clays. Instead they show volume decrease explainable only by possible membrane leakage. It may be noted that the consolidation test shows a 13 psi preconsolidation pressure for the sample. So the sample at 5 psi confining pressure should definitely have behaved as an over-consolidated clay, which it does not.

Figure 14, 15 and 16 show the deviator stress, principal stress ratio and volume change vs axial strain curves for the consolidated drained with constant mean normal stress tests. It is shown that the deviator stress increases, the principal effective stress ratio increases, and the volume decreases for corresponding strain increases. At low confining pressure, the increasing of stress ratio and volume change are higher than under higher confining pressure for corresponding strain.

2 Effective Shear Strength Parameters From Triaxial Tests

The effective shear strength parameters c' and ϕ' are obtained by using the Mohr circle method, and by calculation using plots of $1/2(\bar{\sigma}_1-\bar{\sigma}_3)_f$ vs $\bar{\sigma}_3$ where the subscript "f" denotes failure. Values of c' and ϕ' obtained both ways for all types of triaxial tests are shown in Figure 17 to Figure 22 and the summary is shown in Table 3.

The Mohr circle method gives a close agreement to the $1/2(\bar{\sigma}_1-\bar{\sigma}_3)_f$ vs $\bar{\sigma}_3$ method except for the extension test. In both consolidated-undrained and consolidated drained tests the maximum difference in c'and ϕ' obtained from two methods are about 1.3 psi

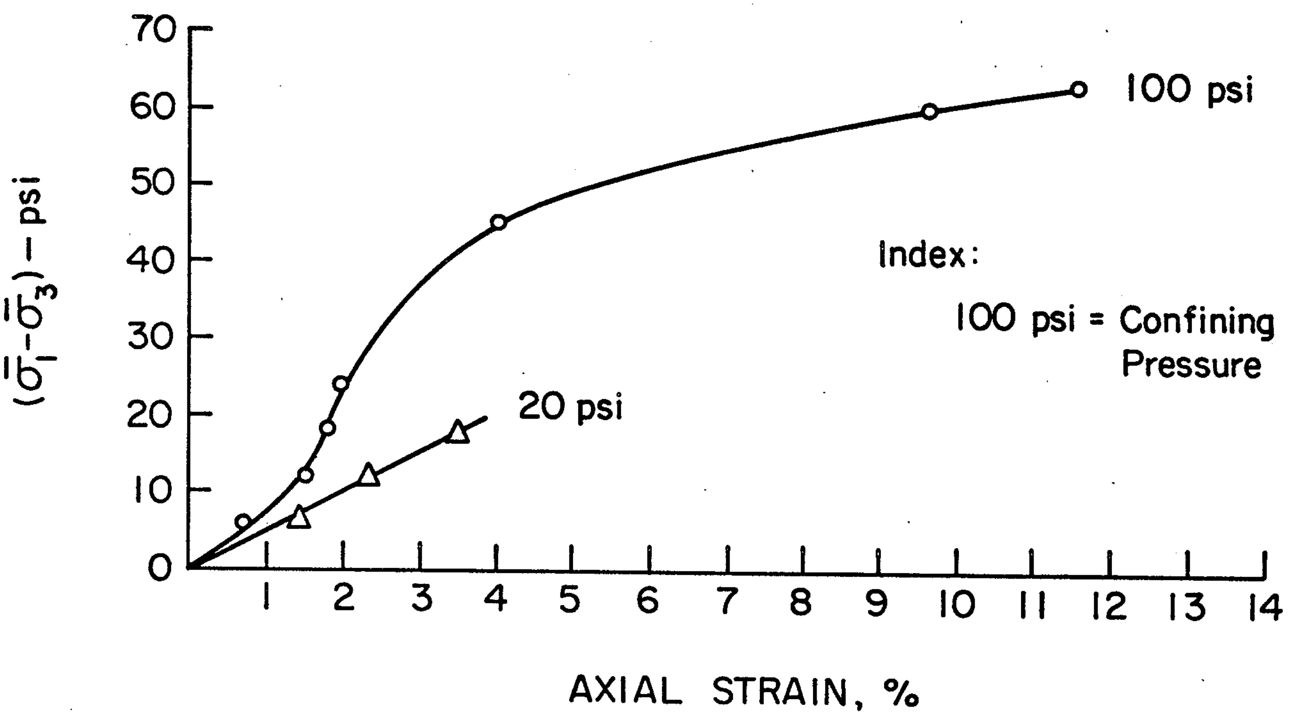


FIGURE 14. $(\bar{\sigma}_1 - \bar{\sigma}_3)$ vs. strain curves for consolidated-drained triaxial test with constant mean normal stress

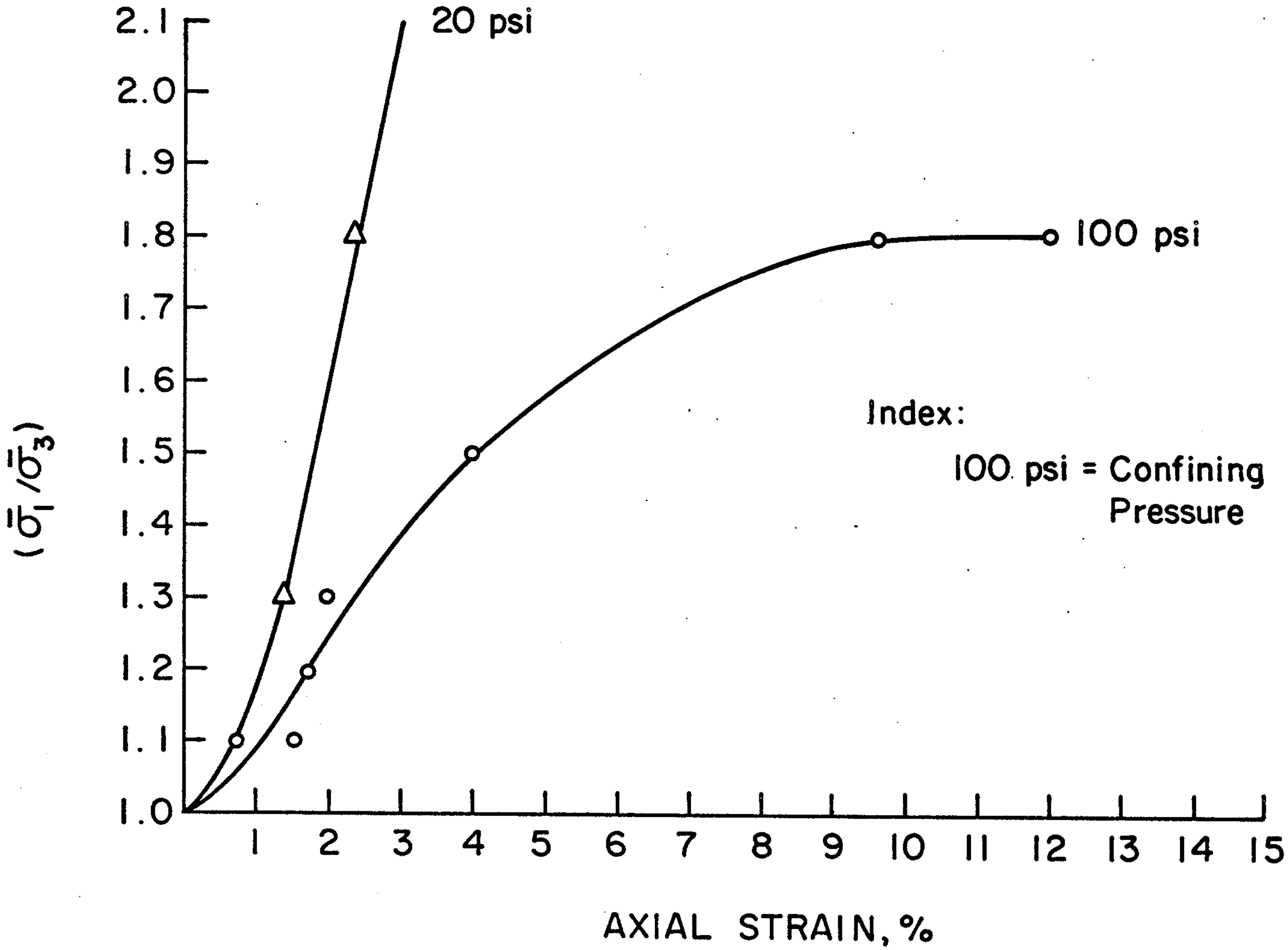


FIGURE 15. $(\bar{\sigma}_1/\bar{\sigma}_3)$ vs. strain curves for consolidated-drained triaxial test with constant mean normal stress.

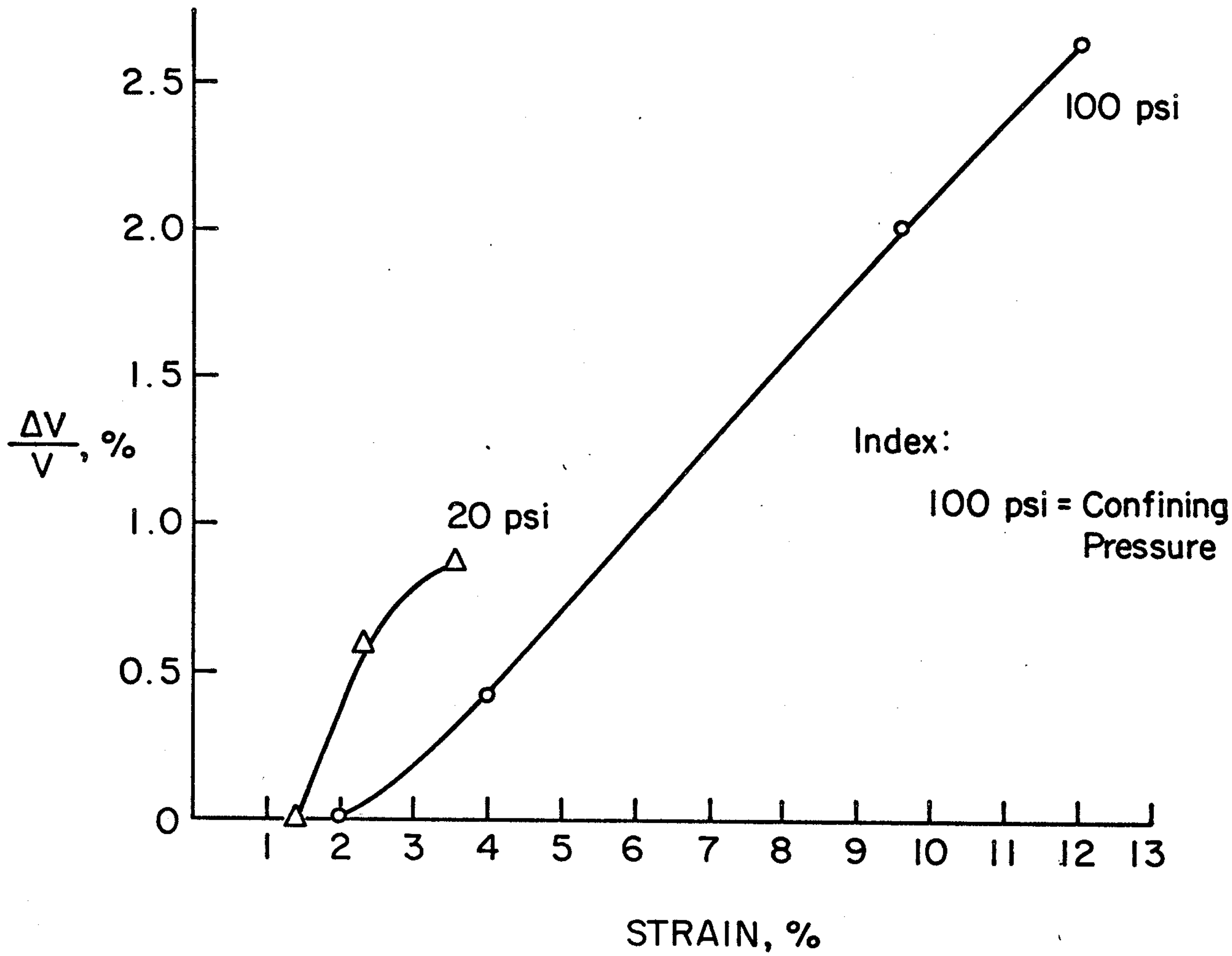


FIGURE 16. (ΔV/V) vs. strain curves for consolidated-drained triaxial test with constant mean normal stress.

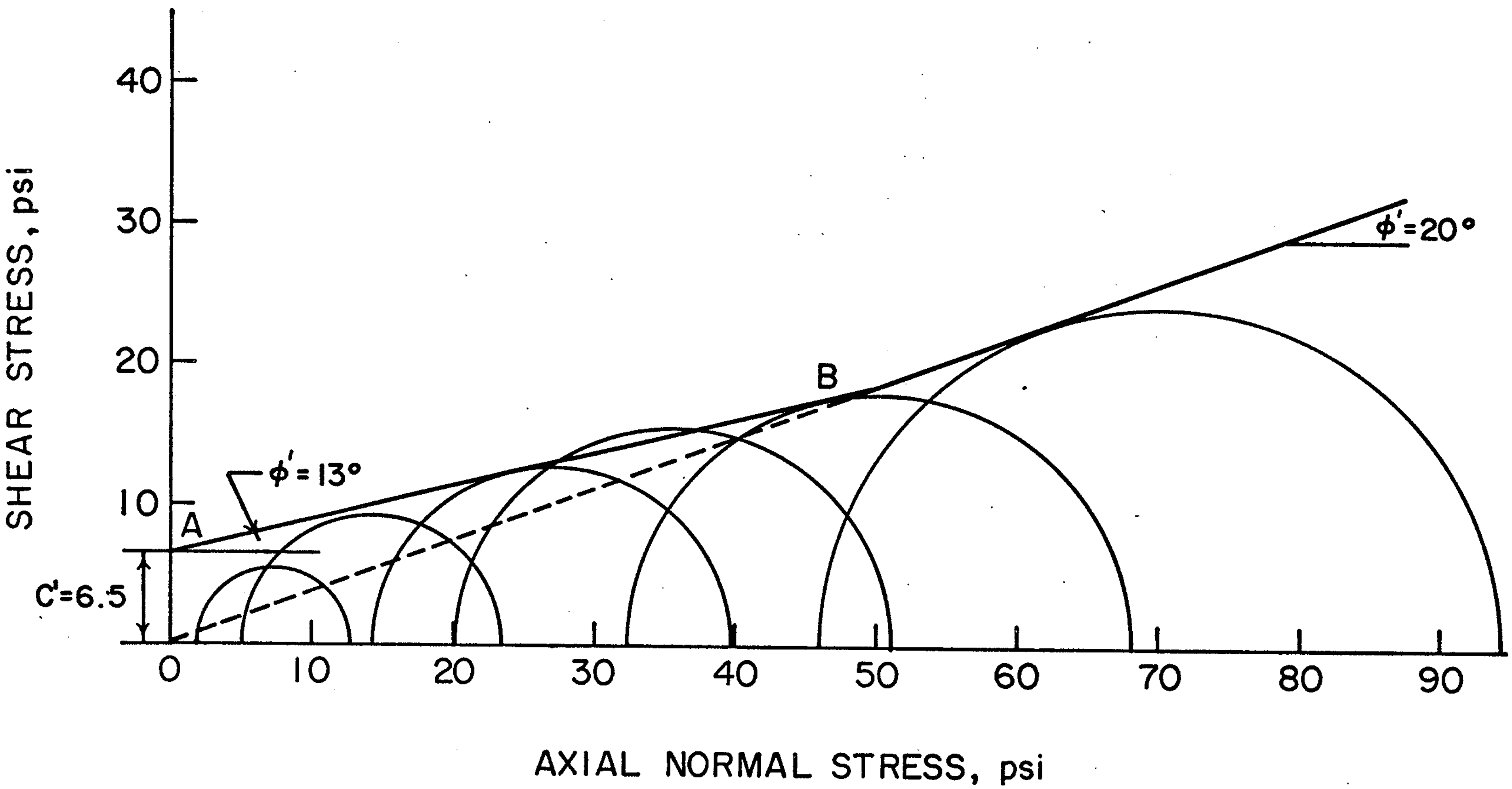
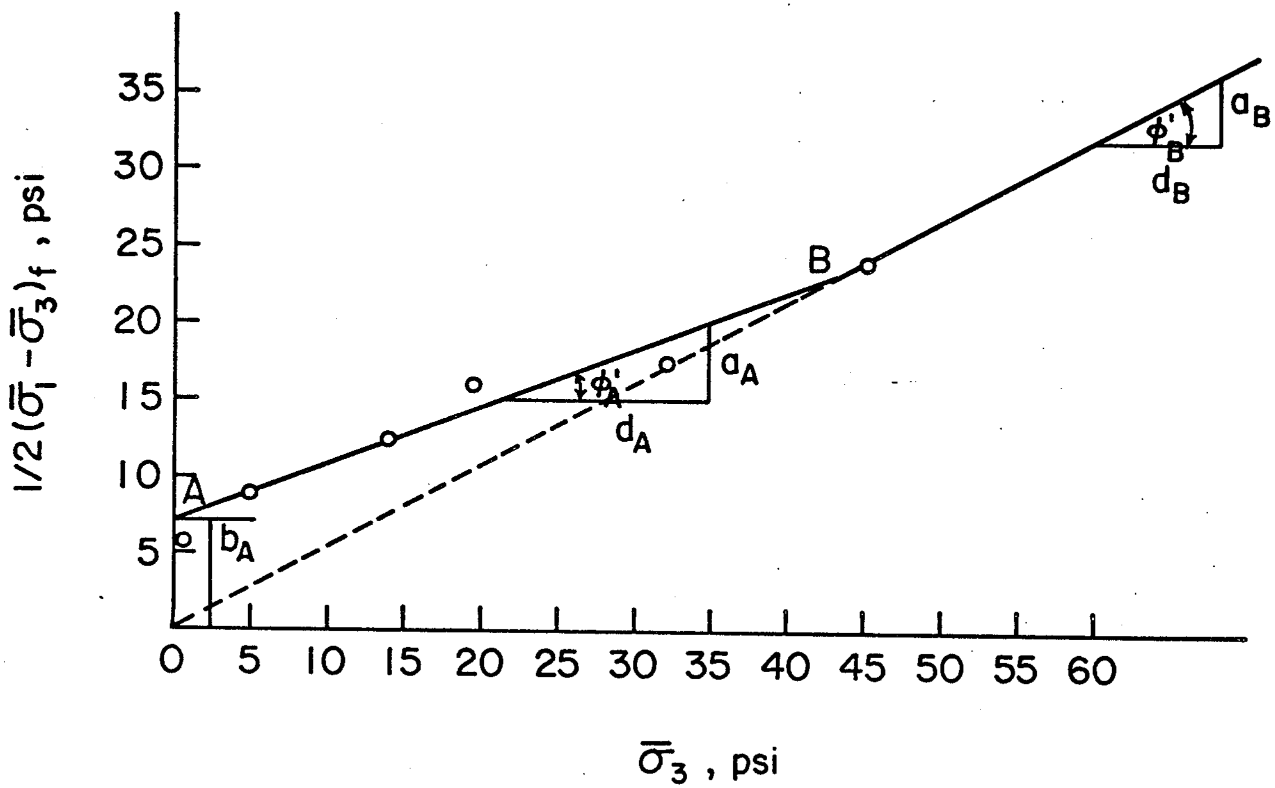


FIGURE 17. Mohr rupture envelope for consolidated-undrained triaxial compression test.



NOTE:

$\phi' = \sin^{-1} a/a+d$

$C' = b(1-\sin\phi')/\cos\phi'$

$\phi'_A = 15.6°$,
$C'_A = 5.2$ psi

$\phi'_B = 19.5°$

FIGURE 18. 1/2 $(\overline{\sigma}_1 - \overline{\sigma}_3)_f$ vs. $\overline{\sigma}_3$ curves for consolidated-undrained triaxial compression test.

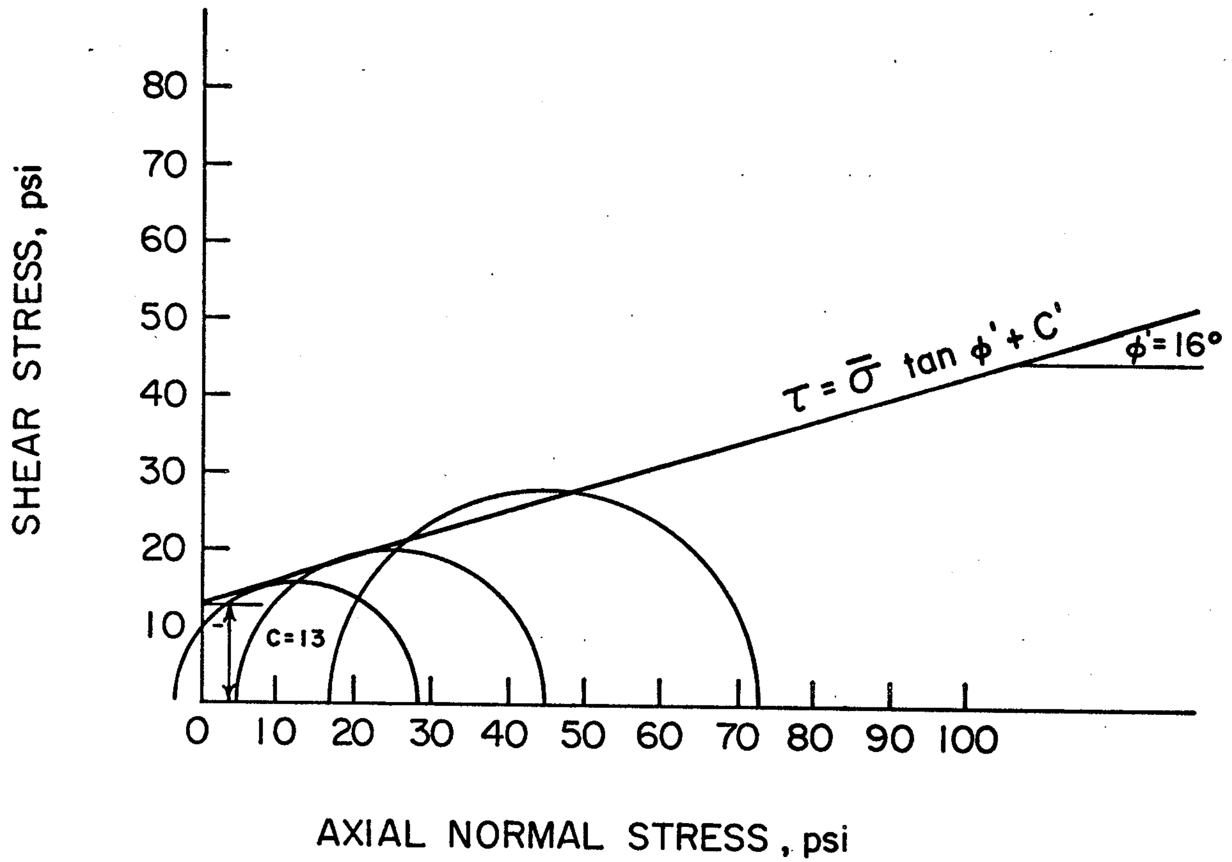


FIGURE 19. Mohr rupture envelope for consolidated-undrained triaxial extension test.

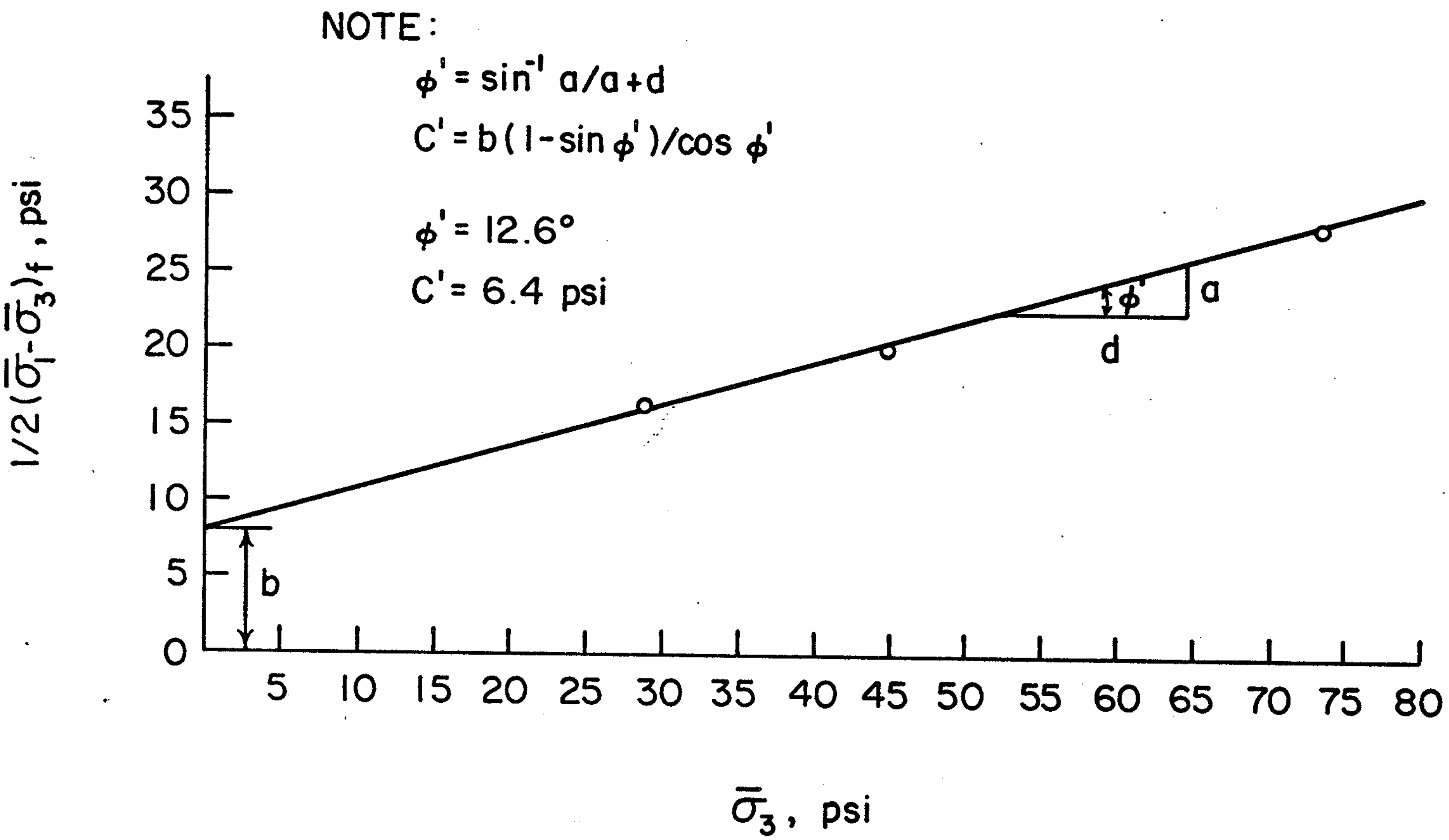


FIGURE 20. $1/2(\bar{\sigma}_1-\bar{\sigma}_3)_f$ vs. $\bar{\sigma}_3$ curves for consolidated-undrained triaxial extension test.

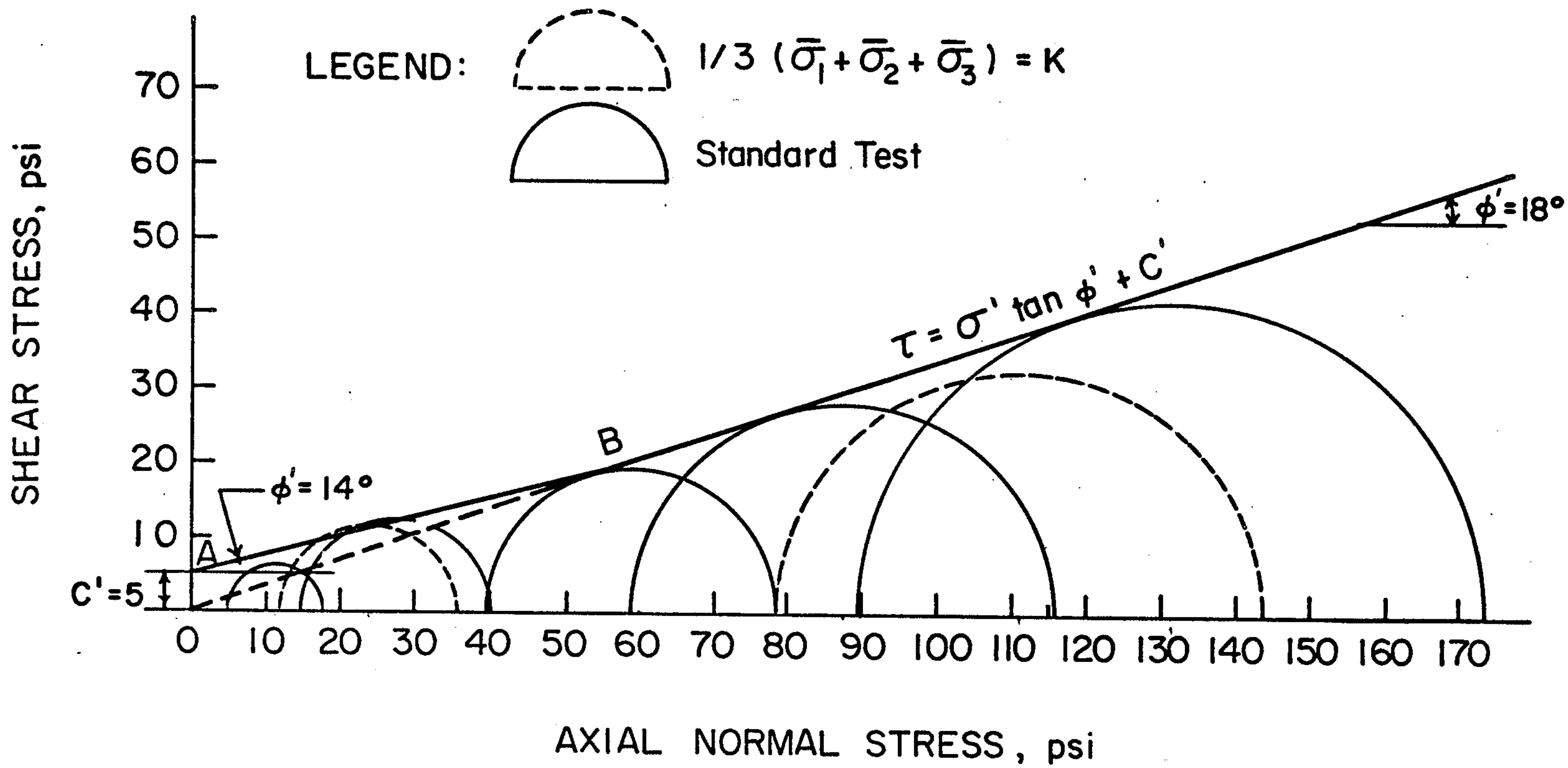


FIGURE 21. Mohr rupture envelope for consolidated-drained triaxial test.

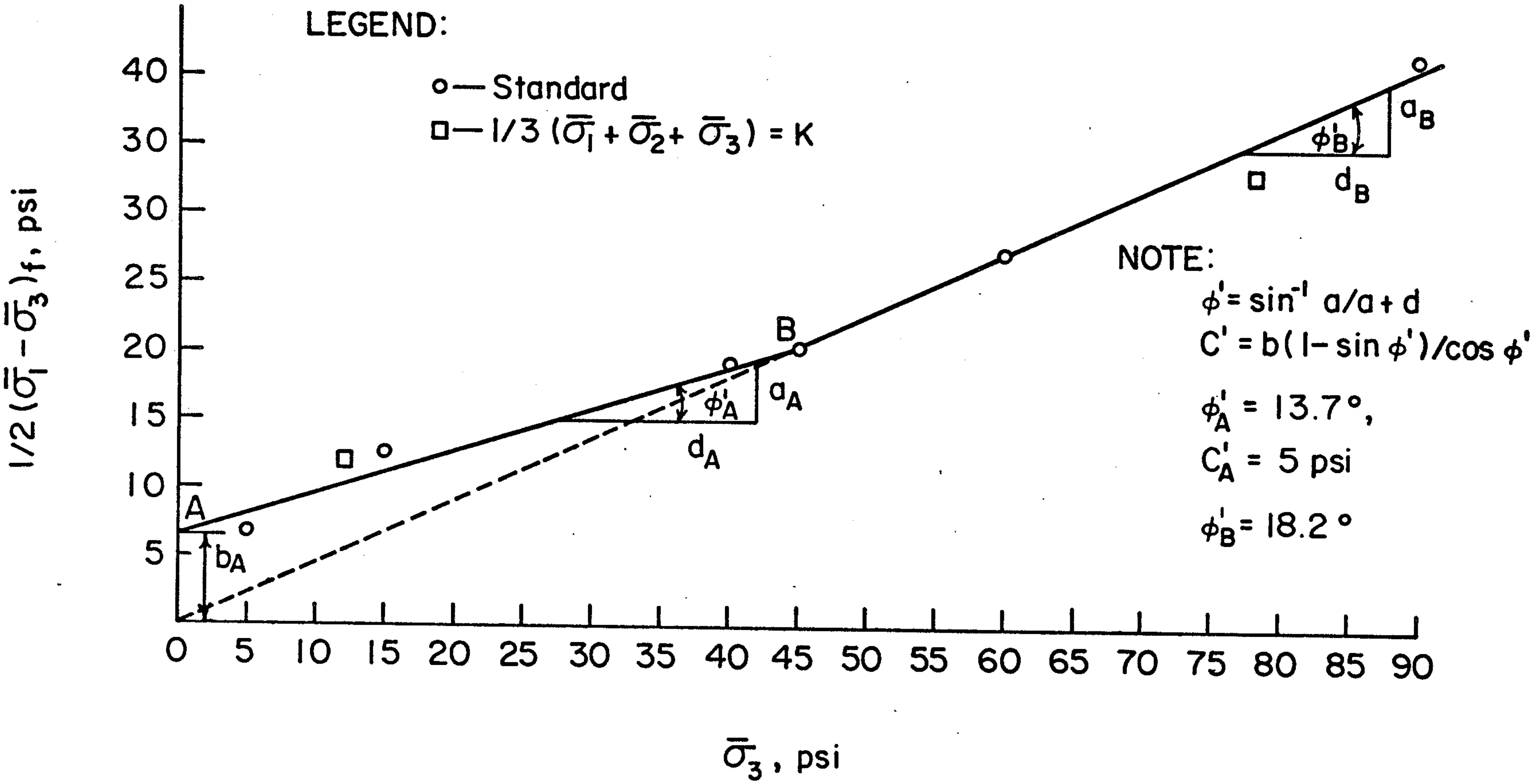


FIGURE 22. $1/2(\bar{\sigma}_1 - \bar{\sigma}_3)_f$ vs. $\bar{\sigma}_3$ curves for consolidated-drained triaxial test.

TABLE 3 - SUMMARY OF EFFECTIVE SHEAR STRENGTH PARAMETERS FROM VARIOUS TYPES OF TESTS

TYPE OF TEST	c' psi	ϕ' degree	COMMENTS
Consolidated-Undrained Triaxial Compression Test			L.L.=123, P.L.=44, q_u=1,520 psf
$0 \leq \bar{\sigma} \leq 44$ psi $\sigma \geq 44$ psi	6.5 0	13 20	Mohr Circle Method
$0 \leq \bar{\sigma} \leq 44$ psi $\sigma \geq 44$ psi	5.2 0	15.6 19.5	$\frac{1}{2}(\bar{\sigma}_1 - \bar{\sigma}_3)_f$ vs $\bar{\sigma}_3$ Method
Consolidated-Undrained Triaxial Extension Test			L.L.=123, P.L.=44, q_u=1,520 psf
$0 \leq \bar{\sigma} \leq 44$ psi $\sigma \geq 44$ psi	13 6.4	16 12.6	Mohr Circle Method $\frac{1}{2}(\bar{\sigma}_1 - \bar{\sigma}_3)_f$ vs $\bar{\sigma}_3$ Method
Consolidated-Drained Triaxial Test			L.L.=103-114, P.L.=41-42, q_u=1,810 psf
$0 \leq \bar{\sigma} \leq 44$ psi $\sigma \geq 44$ psi	5	14	Mohr Circle Method
$0 < \bar{\sigma} \leq 44$ psi $\sigma \geq 44$ psi	5 0	13.7 18.2	$\frac{1}{2}(\bar{\sigma}_1 - \bar{\sigma}_3)_f$ vs $\bar{\sigma}_3$ Method
Direct Shear Test			L.L.=48, P.L.=18, q_u=660 psf
Peak Strength Residual Strength	1 0	20 12.5	

and 2.6 degrees respectively. For the extension test the differences are respectively 6.6 psi, and 3.6 degrees which are considered large. The reason for these differences will be discussed later.

The soil samples used in the triaxial tests did not all come from the same depth. In order to have sufficient samples for the tests, materials were used from the 6 to 8, 11 to 13 and 16 to 18-foot depths. Similar plastic and liquid limits confirm the samples to be very similar highly plastic clays. A comparison between the consolidated-undrained and consolidated-drained tests can be made even though the tests were on samples from different depths with apparently small error.

It is shown from Figures 17, 18, 21 and 22 that the soil is under an over-consolidated condition within the stress range from 0 to about 44 psi which is shown by the portion AB of the curves. The way to interpret the over-consolidated and normally-consolidated conditions by plotting the Mohr circle is explained in all Soil Mechanics textbooks.

For consolidated-undrained and consolidated drained triaxial tests, when confining pressure is less than 44 psi, the value of ϕ' is 0.3 degrees greater to 2.6 degrees less obtained from the Mohr circle plots as compared to the value obtained from the $1/2(\bar{\sigma}_1-\bar{\sigma}_3)_f$ vs $\bar{\sigma}_3$ plots. Similarly, the value of c' is 1.3 psi greater. For confining pressures greater than 44 psi, both plots give ϕ' values 0.2 to 0.5 degrees greater. It is reported by Simons and Bjerrum[2] that for normally-consolidated clay the shear strength parameters c' and ϕ' obtained from consolidated-undrained triaxial tests are very close to the values obtained from consolidated drained triaxial tests. But there is some difference in c' and ϕ' values for over-consolidated clay obtained from both types of triaxial test (Simons[3]). For Winnipeg clay, Nalin P. Samarasingha[4] also observed that a higher cohesion intercept in terms of effective stress is obtained from the consolidated-undrained tests than from the consolidated-drained tests. Simons and Bjerrum[2] have reported that the reasons for the differences between consolidated-undrained and drained tests were also discussed by Bishop, Bjerrum, Casagrande and Wilson, Hirschfeld and Skempton and Bishop. In any comparison of the results between undrained and drained tests, it must be considered the rate of loading, saturation of the sample, work involved in changing volume (Bjerrum and Simons[2]). These are discussed as follows:

The rate of loading in undrained tests was 0.0002 inch per minute. It took on the average about 8 hours to load the samples to failure. In drained tests it took about 7 days by average. Since ϕ' is to some extent time dependent, it is necessary to use similar rates of testing in making an experimental comparison. The effect of time on shear strength was shown by Whitman[5]. For Winnipeg clays, it has been shown by Nalin P. Samarasingha[4] that with increasing the strain rate, ϕ' decreases and c' increases in consolidated-undrained tests. Thus, if the rate of strain in undrained tests are slower, there might be an agreement in the shear strength parameters obtained from both types of triaxial tests. However, the strain rate of 0.0002 inch per minute is generally slow enough to permit equalization of the pore pressure (Scott[6]).

Saturation of the samples also has an influence on the shear strength parameters because if a sample is only partially saturated, then measurements of pore pressure using an ordinary porous stone may result in significant error which is due to the dissolved gas in the soil volume entering the pore pressure measuring system. The use of extremely fine porous stone may be needed. Because of the high degree of saturation of the soil samples, i.e. at least about 98%, and the use of 10 psi back pressure is assumed to be sufficient to achieve full saturation. Lowe[7] has reported that this amount of back pressure results in full saturation. For different degrees of saturation, Lowe also has shown the amount of back pressure needed. Bishop and Henkel[1] have shown that in practical work the use of 30 psi back pressure is sufficient.

The last factor which involves in the comparison of shear strength parameters is the work done in changing volume. Theoretically, undrained and drained tests can only be compared if the drained test is corrected for the work to failure involved in changing volume. For normally-consolidated clays, Bishop and Bjerrum[8] have reported that Skempton and Bishop have shown theoretically using Hvorslev concepts of true cohesion and friction that there should be close agreement between the effective stress envelopes for consolidated-undrained and drained tests, more exactly the correction generally increases ϕ' obtained from the drained test. For over-consolidated clays the correction is important and the correction reduces the observed value of ϕ' obtained from the drained test (Bjerrum and Simons)[2].

Bishop and Bjerrum[8] have reported that for normally-consolidated clay the observed value of ϕ' from the consolidated-undrained tests is higher by 0 to 1 degree in typical cases, and for over-consolidated clay the drained test is usually found to give the higher value. The results obtained from Winnipeg clay do agree with Bishop and Bjerrum findings when the Mohr circle method is used. For the $1/2(\bar{\sigma}_1-\bar{\sigma}_3)_f$ vs $\bar{\sigma}_3$ method, the undrained test gave the higher ϕ' value for the over-consolidated condition contrary to Bishop and Bjerrum findings. The problem, however, is how to obtain the best fit lines given the scatter of data in both methods. The maximum difference in ϕ' of 2 degrees can be due to the difference in visually "fitting" the data in the two plots.

The failure theories used in practice generally assume the isotropic materials. Anisotropic materials would show directional properties. Therefore, the standard triaxial compression and extension tests on stratified soil have never been expected to give the same shear strength parameters. Kenny[9] has shown that two identical soil elements subjected to identical consolidation stress, but sheared to fail at different inclinations can exhibit different undrained strength confirming earlier work by Eden, Lo and Milligan. The latter had tested natural stratified and homogeneous clay in compression, the samples being cut at different inclinations. The result indicated that undrained strength was dependent on orientation of the sample when all other factors were equal. It is the same case when the standard compression and extension tests are compared. The failure planes in these cases are differently inclined. In the standard compression test it inclines at 45 + $\phi/2$ degrees to the horizontal where in the extension test it inclines at $45 + \phi/2$ to the vertical. Another reason for a difference may be that in the standard triaxial compression test, the intermediate principal stress remains constant, whereas in the extension test it is continually changing. This would be a factor if the strength depended on the intermediate principal stress. The c' and ϕ' obtained from undistrubed samples of Winnipeg clay are 6.4 psi and 12.6 degrees based on the $1/2(\bar{\sigma}_1-\bar{\sigma}_3)_f$ vs $\bar{\sigma}_3$, for the over-consolidated condition. The value of c' is about 28% greater than that for the compression tests, but ϕ' is about 19% smaller. The value of ϕ' which has been observed by other researchers, i.e. Johansen, Taylor, Taylor and Clough, Henkel, as reported by Hvorslev[10] was in some cases greater and in some cases smaller than those for compression tests in which the maximum difference is about 20%. These very substantial differences in results obtained from undisturbed Winnipeg clay would indicate a marked lack of isotropy. In this discussion the results based on the Mohr circle method is not mentioned because the tests have been done on only three samples and unfortunately only two samples are considered as being in the over-consolidated condition. It is very difficult to draw the best tangent to the closely spaced circles and especially where there are only two circles. Therefore, the $1/2(\bar{\sigma}_1-\bar{\sigma}_3)_f$ vs $\bar{\sigma}_3$ method is chosen to be the suitable method for this case.

Figure 23 shows the results of the undrained triaxial tests plotted in a stress space. The results of the compression test are plotted above the space diagonal line and the results of the extension test are plotted below the space diagonal line. The stress paths for

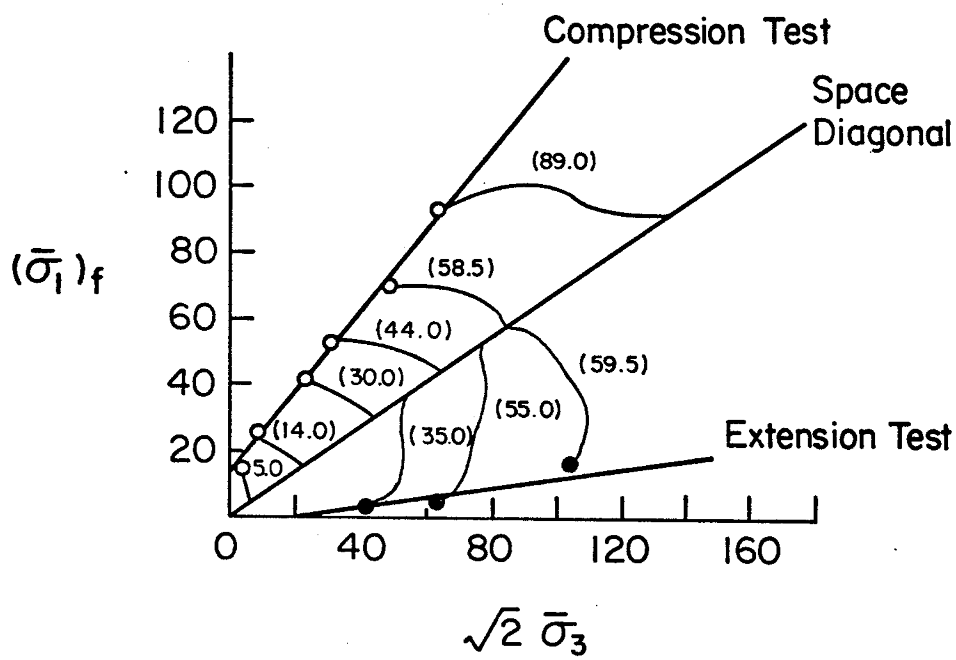


FIGURE 23. $(\sigma_1)_f$ vs. $\sqrt{2}\ \bar{\sigma}_3$ curves for consolidated-undrained triaxial test with pore pressure measurement.

EFFECTIVE STRESS AT FAILURE					
COMPRESSION TEST			EXTENSION TEST		
$\bar{\sigma}_{1f}$	$\bar{\sigma}_3$	$\sqrt{2}\ \bar{\sigma}_3$	$\bar{\sigma}_{1f}$	$\bar{\sigma}_3$	$\sqrt{2}\ \bar{\sigma}_3$
14.45	2.95	4.17	16.5	73.50	104.0
25.25	6.85	9.65	-3.0	29.00	41.0
41.10	16.25	23.00	5.0	45.00	63.5
53.22	22.00	31.00			
69.75	34.75	49.10			
92.90	45.30	64.00			

Index:

(89.0) Confining Pressure, psi = Cell Pressure less Back Pressure all samples are also shown in Figure 23. For the compression test, the stress paths for the samples consolidated at confining pressures of 5 and 14 psi have the shapes as expected for an over-consolidated clay. The samples consolidated at confining pressures of 44, 58.5 and 89 psi have the shapes as expected for a normally-consolidated clay. The sample consolidated at confining pressure 30 psi does not show clearly whether it is in an over-consolidated or normally-consolidated condition. Therefore, it can be considered to show the border between over-consolidated and normally-consolidated conditions or defining approximately the preconsolidation pressure. All results of the extension test show the shapes of stress paths expected for a normally-consolidated clay. This is to be expected as the confining pressures are greater than the apparent preconsolidation pressure.

It may be concluded from the stress paths obtained from the undrained compression and extension tests that preconsolidation pressure should be about 30 psi. Considering the results obtained from the consolidation test and triaxial tests, the preconsolidation pressure value ranges between about 20 psi and about 44 psi.

3 Direct Shear Test Results

Figure 24 shows the shear stress vs total strain for samples consolidated at normal pressures of 10, 20, 36.5 and 59 psi and tested in direct shear. Figure 25 shows the Mohr rupture envelopes from which the effective peak strength and effective residual strength parameters are obtained. The results are also summarized in Table 3. The peak shear strength parameters obtained in the direct shear test differ considerably from the parameters obtained in the triaxial tests.

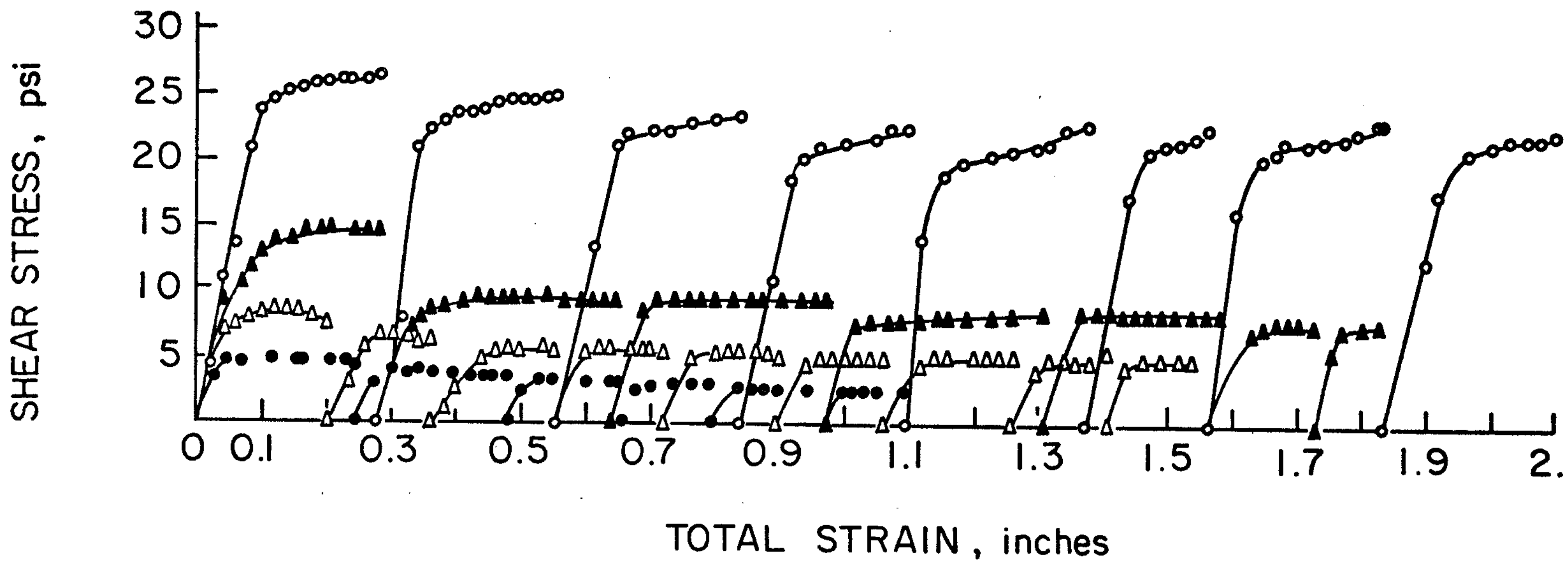


FIGURE 24. Shear stress vs. total strain curves for direct shear (drained) test.

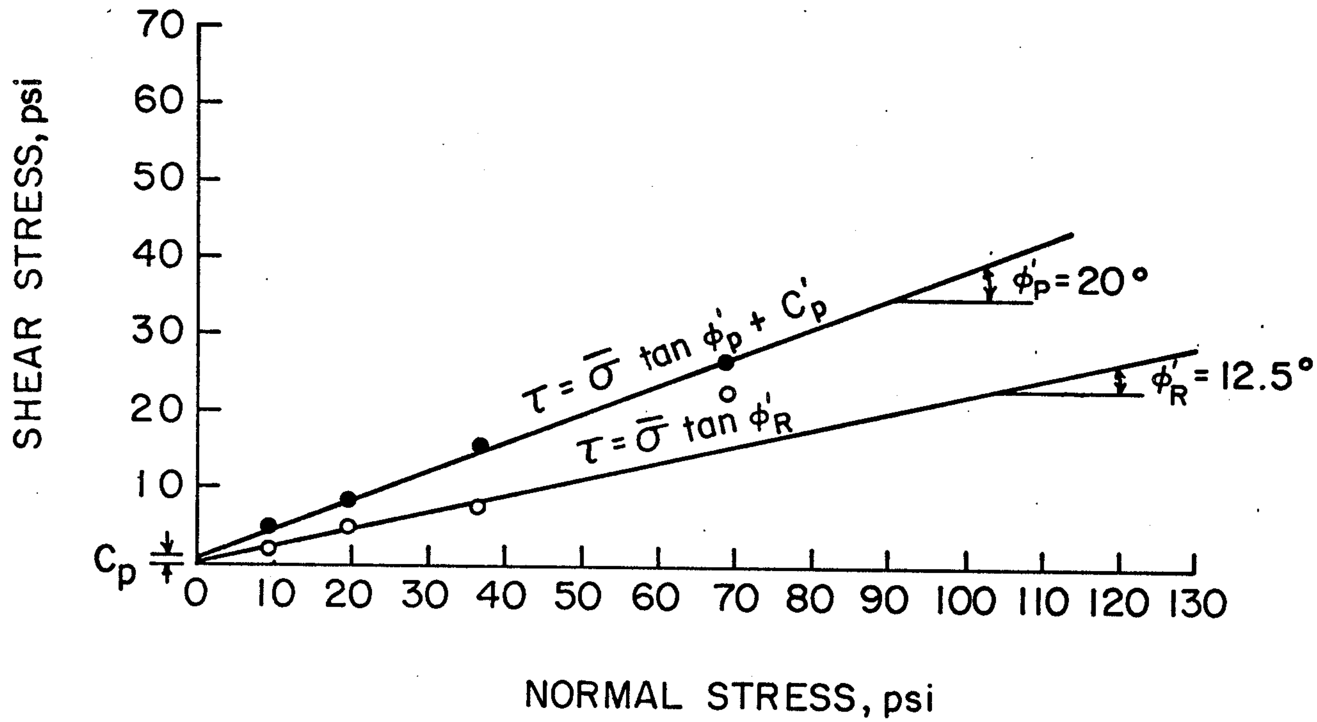


FIGURE 25. Mohr rupture envelopes for direct shear (drained) test.

This is to be expected since the samples are different. The soil is a highly plastic clay for the triaxial tests, and a silty clay for the direct shear test. The peak cohesion and friction angle are 1 psi and 20 degrees respectively. The residual cohesion is zero and the residual friction angle is 12.5 degrees. These low values can be expected for a silty clay.

4 Pore Pressure Parameter B and A_f

Figure 26 shows the linear relationship between the all-around cell pressure and pore pressure developed in the soil sample in an undrained test. Theoretically, for a fully saturated clay the pore pressure parameter B is equal to unity. It is less than unity for a partially saturated clay. The results of the test are shown in Table 4. The results are consistent with theory since B is nearly unity. The initial saturation of the sample used is 98%. In practical work this is considered as fully saturated. The negative intercept of the graph shown in Figure 26 may be explained as the result of an initial tension or a negative pore pressure in the pore water because the undisturbed sample has not been allowed to re-consolidate after it was taken from the ground.

Figure 27 shows the relationship between the pore pressure parameter A_f and the confining pressure. It is clearly shown that by increasing the confining pressure, A_f will increase. The values of A_f at confining pressures of 44, 58.5 and 89 psi range from 0.84 to 0.98, but the values of A_f at confining pressures of 5, 14 and 30 psi range from 0.21 to 0.63. According to Bishop and Henkel[1], when

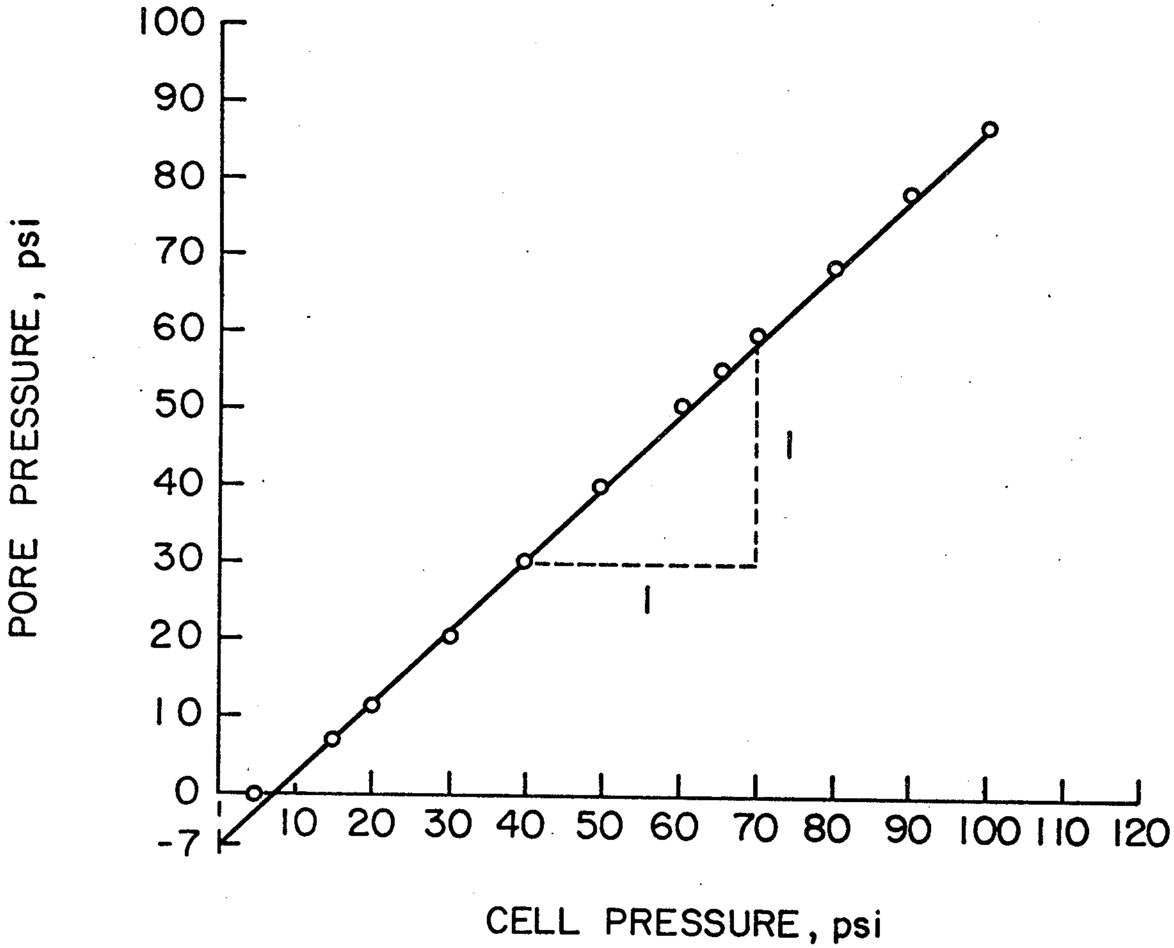


FIGURE 26. Pore pressure vs cell pressure curves for determination of pore pressure parameter B.

TABLE 4

PORE PRESSURE PARAMETER B

$\Delta\sigma_3$ psi	ΔU_a psi	B
10	3.5	---
20	12.0	0.85
30	21.5	0.95
40	31.0	0.95
50	40.5	0.95
60	50.0	0.95
70	60.0	1.00
80	69.5	0.95
90	79.0	0.95
100	88.5	0.95

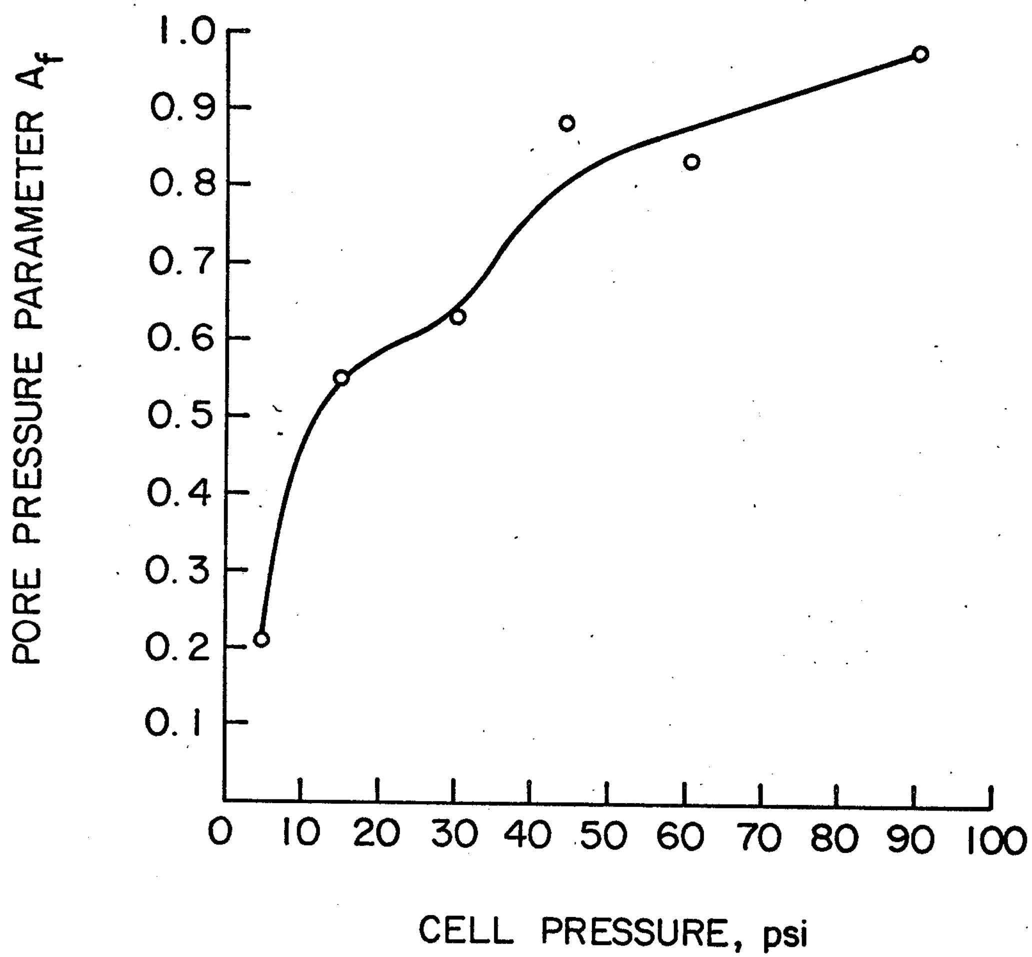


FIGURE 27. Pore pressure parameter A_f vs. cell pressure curve for consolidated-undrained triaxial compression test.

the over-consolidation pressure ratio equals one, the A_f values for typical cases is also close to one. Values of A_f decrease with increasing over-consolidation ratio, with a value $A_f = 0$, obtained for an over-consolidation ratio of about 4. Interpolation would suggest preconsolidation pressure somewhere between 20 to 45 psi for samples taken from the 16- to 18-foot depth.

CHAPTER VI

USE OF TEST DATA

1 Bearing Capacity of Foundations

1.1 Theory

The purpose of a structural foundation is to transfer the structural loads safely to the ground below. In general, the bearing capacity of the soil and the amount of differential settlement are the prime concern. The bearing capacity depends on the soil itself as well as the shape and size of the foundation. The solution for the bearing capacity has been developed first from Prandtl's theory of plastic failure for metals. Terzaghi[11] has presented a solution for the ultimate bearing capacity of long footing which is more general in nature than the others. According to Terzaghi, for a continuous footing of width b embedded a depth, D_f, in a soil with unit weight, γ, cohesion, c, and friction angle, ϕ, the bearing capacity for the undrained case may be expressed by the following:

$$q_{net\ ult} = cN_c + 1.0\ \gamma_1 \frac{b}{2} N_\gamma + \gamma_2\ D_f(N_q - 1) \quad \ldots\ldots\ (1)$$

where, $q_{net\ ult}$ = net ultimate bearing capacity;

c = cohesion;

γ_1 = unit weight of soil below elevation of base of footing;

γ_2 = unit weight of soil above elevation of base of footing; and

N_c, N_γ, N_q = coefficients depending only on the angle of internal friction, ϕ, as shown in Figure 28.

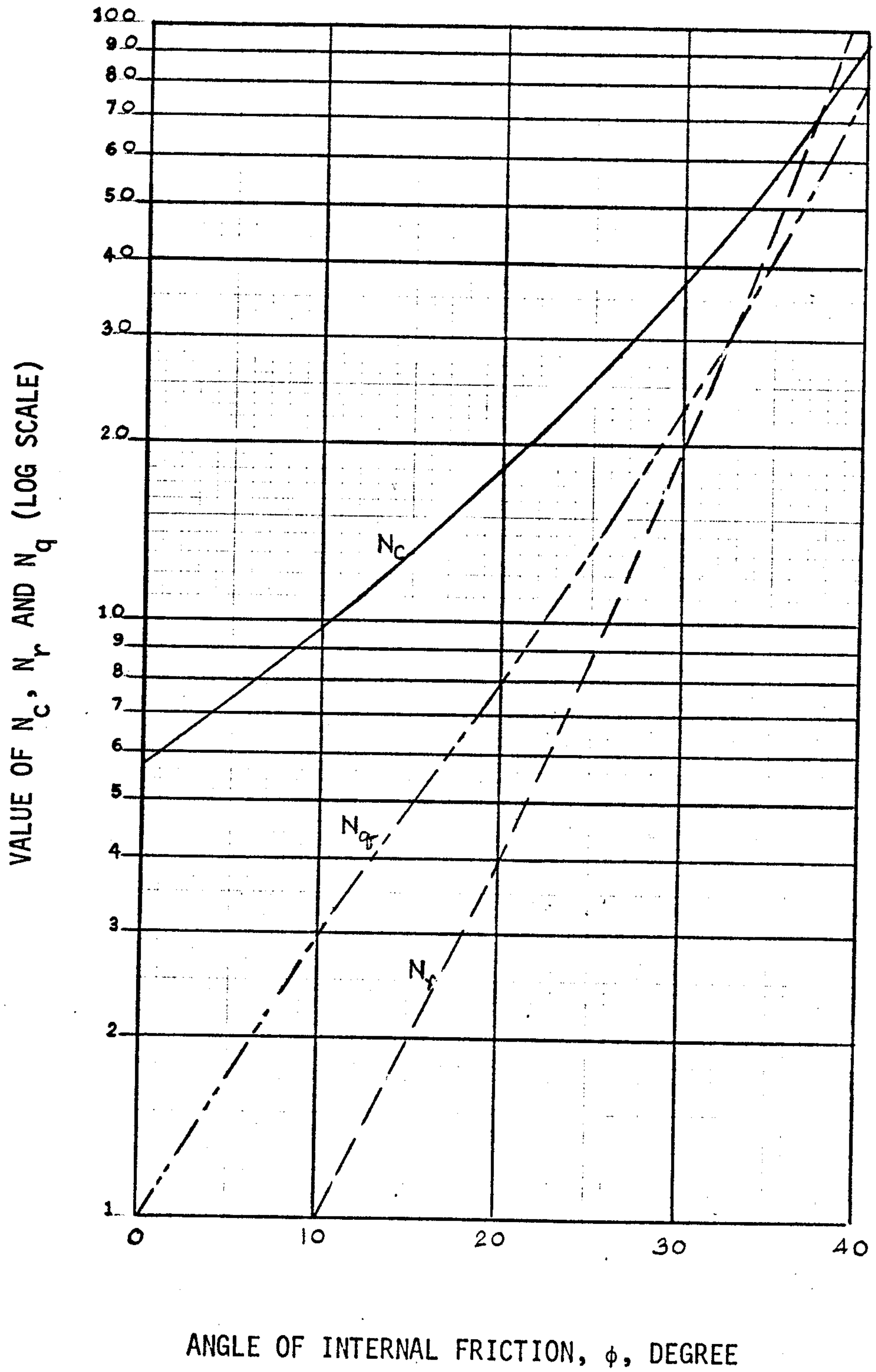


FIGURE 28 - RELATION BETWEEN ANGLE OF INTERNAL FRICTION AND THE TERZAGHI BEARING CAPACITY FACTORS FOR A ROUGH FOOTING

The bearing capacity of the soil for the drained case may be expressed in terms of net pressures as follows:

$$q_{net\ ult} = c'N_c + 1.0\ \gamma_1 \frac{b}{2} N_\gamma + \gamma_2 D_f(N_q - 1) \ldots\ldots\ldots (2)$$

where, c' = effective cohesion,

γ_1 = unit weight of soil below elevation of base of footing corrected for the position of the watertable;

γ_2 = unit weight of soil above elevation of base of footing corrected for the position of the watertable;

N_c, N_γ, N_q = coefficients depending on the effective angle of internal friction, ϕ', as shown in Figure 28.

The typical pattern of the rupture theoretical slip planes in the soil under a foundation at failure is shown in Figure 29.

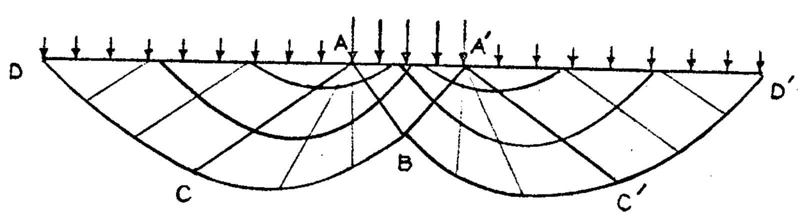


Figure 29 - TYPICAL RUPTURE SURFACES BENEATH A FOUNDATION AT FAILURE

The region ACD is a zone of Passive Rankine failure. The region ABC is a zone of radial shear. The soil in the region ABA' may or may not be in the state of plastic equilibrium depending on the roughness of the underside of the foundation. For rough footing, it is in a condition of elastic equilibrium, whereas for smooth footing it is in the active state.

1.2 Discussion

The bearing capacity of saturated clays is normally computed by using a total stress analysis based on no drainage taking place. This is the condition normally encountered where construction rates are relatively rapid, and because of low clay permeability virtually no drainage takes place during the time of construction and first loading. Since the foundation loads increase the stresses in the soil, the pore pressures are increased during loading, and with time subsequently reduce. Thus, the effective stresses have their least value at the end of the construction period, and increase as the soils consolidate. The foundation becomes more stable with time, and the long term stability need not generally be considered, if the foundation is shown to be safe initially. When the construction period is unusually long, or the load is applied in stages over a long period, some significant dissipation of the excess pore pressure may take place before loading is complete. A total stress analysis based on the undrained condition may be conservative. An analysis can be made in terms of effective stress, taking account of the dissipation of the pore pressure during loading. The shear strength parameters c' and ϕ' are normally obtained from undrained triaxial compression tests with pore pressure measurement, or from drained tests. The shear strength of clays has been a subject of discussion and disagreement ever since investigators began to think seriously about the subject. From Coulomb's original equation,

$$T_f = c + \sigma_f \tan \phi \quad \ldots\ldots \quad (3)$$

where, T_f = the maximum shear resistance;

σ_f = the normal stress on the failure plane;

$\sigma_f \tan \phi$ = the friction on the failure plane; and

c and ϕ = the cohesion and internal friction as defined before.

In a more fundamental form, Coulomb's equation is written as follows:

$$T_f = c' + (\sigma_f - u) \tan \phi' \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (4)$$

where, u = the pore pressure; and the other symbols are as previously defined.

According to Skempton and Bishop[12], in any isotropic soil the cohesion is a non-directional property and it may be regarded as the resultant of the physico-chemical forces acting between particles which is the important forces in clay soils.

Internal friction is derived principally from the actual friction of grain on grain. It is, however, also taken as including the resistance to shear developed as a result of the work which has to be done when the soil changes volume during shear. Internal friction for isotropic soils is not itself a directional property, but in the general case of an element under unequal principal stresses the shear resistance along different planes will vary in accordance with the variation in normal stress, σ, and hence the internal friction imparts directional properties to the soil.

There is evidence that the undrained shear strength of Winnipeg clays is distinctly anisotropic. For example, Loh and Holt[13] reported that the undrained shear strength of undisturbed samples cut with the axis 90^{o} from the horizontal is found to be about 2.2 times the value for that of samples cut 0° from the horizontal. The engineer is thus faced with several practical questions, such as the possibility of using c' and ϕ' obtained from consolidated undrained compression tests or drained tests, or perhaps the values obtained from some other tests, for example, extension tests. Also, c' and ϕ' can be obtained in terms of peak and residual strength in direct shear tests. The question arises which one is more applicable to foundation bearing capacity determination.

The calculated Terzaghi net ultimate bearing capacity for all values of c' and ϕ' are tabulated in Table 5 and Table 6. The calculations were done by considering a strip footing with 10 feet width and 6 feet depth, and a strip footing with 2.5 feet width and 1.5 feet depth, with ground watertable at the worst position, i.e. at ground level. These correspond to a typical large footing, and a small shallow footing in a Winnipeg building with a basement. The average unit weight of the soil both above and below the base of the footing were taken equal to 107 psf based on actual test values. The Winnipeg Building Code values are shown for comparison with ultimate values based on an assumed factor of safety of 2.5.

In the case of a strip footing with 10 feet width and 6 feet depth, the calculated net ultimate bearing capacity based on c' = 6.5 psi, ϕ' = 13 degrees, is about 12,000 psf, and c' = 5.2 psi, ϕ' = 15.6 degrees, is about 11,700 psf. These shear strength parameters are obtained from the consolidated-undrained compression test. The values of net ultimate bearing capacity are very close and about 2.4 times the bearing capacity given by the Winnipeg Building Code for

TABLE 5 - BEARING CAPACITY FOR A 10-FOOT WIDE, STRIP FOOTING
D_f = 6 feet, B = 10 feet, L = ∞

TYPE OF TEST	c' psi	ϕ' degree	NET ULTIMATE BEARING CAPACITY (psf)	METHOD TO OBTAIN c', ϕ'
			Calculated After Consolidation Condition	
Consolidated-Undrained Triaxial Compression Test				
$0 \le \bar{\sigma} \le 44$ psi	6.5	13	12,000	Mohr Circle
$\bar{\sigma} \ge 44$ psi	0	20	2,700	
$0 \le \bar{\sigma} \le 44$ psi	5.2	15.6	11,700	$\frac{1}{2}(\bar{\sigma}_1 - \bar{\sigma}_3)$ vs σ_3
$\bar{\sigma} \ge 44$ psi	0	19.5	2,600	
Consolidated-Undrained Triaxial Extension Test				
$0 \le \bar{\sigma} \le 44$ psi	6.4	12.6	11,200	$\frac{1}{2}(\bar{\sigma}_1 - \bar{\sigma}_3)$ vs σ_3
Consolidated-Drained Triaxial Test				
$0 \le \bar{\sigma} \le 44$ psi	5	14	10,000	Mohr Circle
$\bar{\sigma} \ge 44$ psi	0	18	2,200	
$0 \le \bar{\sigma} \le 44$ psi	5	13.7	10,000	$\frac{1}{2}(\bar{\sigma}_1 - \bar{\sigma}_3)$ vs σ_3
$\bar{\sigma} \ge 44$ psi	0	18.2	2,200	
Direct Shear Test				
Peak Strength	1	20	6,000	
Residual Strength	0	12.5	1,300	
			Calculated Before Consolidation Condition	
Unconsolidated-Undrained Test, Assume $c_u = q_u/2$, $\phi=0$			4,700	
Winnipeg Building Code				
Firm Clay } Assume S.F.-2.5			5,000	
Soft Clay } Assume S.F.-2.5			2,500	

TABLE 6 - BEARING CAPACITY FOR A 2.5-FOOT WIDE, STRIP FOOTING
D_f = 1.5 feet, B = 2.5 feet, L = ∞

TYPE OF TEST	c' psi	ϕ' degree	NET ULTIMATE BEARING CAPACITY (psf)	METHOD TO OBTAIN c', ϕ'
			Calculated After Consolidation Condition	
Consolidated-Undrained Tri-axial Compression Test				
$0 \leq \bar{\sigma} \leq 44$ psi	6.5	13	11,100	Mohr Circle
$\bar{\sigma} \geq 44$ psi	0	20	685	
$0 \leq \bar{\sigma} \leq 44$ psi	5.2	15.6	10,500	$\frac{1}{2}(\bar{\sigma}_1 - \bar{\sigma}_3)$ vs $\bar{\sigma}_3$
$\bar{\sigma} \geq 44$ psi	0	19.5	655	
Consolidated-Undrained Tri-axial Extension Test				
$0 \leq \bar{\sigma} \leq 44$ psi	6.4	12.6	10,400	$\frac{1}{2}(\bar{\sigma}_1 - \bar{\sigma}_3)$ vs $\bar{\sigma}_3$
Consolidated-Drained Tri-axial Test				
$0 \leq \bar{\sigma} \leq 44$ psi	5	14	9,000	Mohr Circle
$\bar{\sigma} \geq 44$ psi	0	18	540	
$0 \leq \bar{\sigma} \geq 44$ psi	5	13.7	9,000	$\frac{1}{2}(\bar{\sigma}_1 - \bar{\sigma}_3)$ vs $\bar{\sigma}_3$
$\bar{\sigma} \geq 44$ psi	0	18.2	540	
Direct Shear Test				
Peak Strength	1	20	3,200	
Residual Strength	0	12.5	320	
			Calculated Before Consolidation Condition	
Unconsolidated-Undrained Test, Assume $c_u = q_u/2$, $\phi = 0$			4,700	
Winnipeg Building Code				
Firm Clay } Assume S.F.=2.5			5,000	
Soft Clay } Assume S.F.=2.5			2,500	

firm clay. The net ultimate bearing capacity based on c' = 6.4 psi, ϕ' = 12.6 degrees as obtained from the consolidated-undrained triaxial extension test is about 11,200 psf. This value is only a little smaller than that obtained from the consolidated-undrained compression test. In practical work that small difference is considered as insignificant. Therefore, it may be said that c' and ϕ' obtained from the extension test give the net ultimate bearing capacity about 2.4 times the code value. The net ultimate bearing capacity based on c' = 5 psi, ϕ' = 14 degrees, is the same as that based on c' = 5 psi, ϕ' = 13.7 degrees which is about 10,000 psf or 2 times the capacity obtained from the code. These are obtained from the consolidated-drained triaxial test. Hence, in the over-consolidated range the net ultimate bearing capacity based on the shear strength parameters obtained from the consolidated-drained test is less than that obtained from the consolidated-undrained triaxial compression and extension tests.

For the normally-consolidated range, the net ultimate bearing capacity based on c' = 0 psi, ϕ' = 20 degrees and c' = 0 psi, ϕ' = 19.5 degrees which obtained from the consolidated-undrained triaxial test are 2,700 and 2,600 psf respectively. Both bearing capacity values are about 0.5 times the bearing capacity given by the code. For the consolidated-drained test which c' = 0 psi, ϕ' = 18 degrees, and c' = 0 psi, ϕ' = 18.2 degrees, the net ultimate bearing capacity is equal in both cases and is about 2,200 psf or 0.4 times the code value. Therefore, the net ultimate bearing capacity obtained from the consolidated-drained triaxial test is also less than that obtained from the consolidated-undrained triaxial compression test for this pressure range.

In the case of a strip footing with 2.5 feet width and 1.5 feet depth, the values of net bearing capacity based on c' = 6.5 psi, ϕ' = 13 degrees and c' = 5.2 psi, ϕ' = 15.6 degrees, are about 11,100 psf and 10,500 psf respectively, or about 2.2 times the code value for firm clay. The net ultimate bearing capacity based on c' = 6.4 psi, ϕ' = 12.6 degrees, which obtained from the consolidated-undrained triaxial extension test is about 10,400 psf. Again, as in the case of the wider footing the net ultimate bearing capacity obtained from this test is very close to that obtained from the consolidated-undrained triaxial compression test. Therefore, it may be said that the bearing capacity obtained is about 2.2 times the code value. The net ultimate bearing capacity based on c' = 5 psi, ϕ' = 14 degrees is equal to that based on c' = 5 psi, ϕ' = 13.7 degrees which is about 9,000 psf or 1.8 times the code value. Thus, the net ultimate bearing capacity obtained from the consolidated-drained test is less than that obtained from the other two types of triaxial test by 0.4 times the code value as in the case of the 10 feet width and 6 feet depth footing for the over-consolidated range.

For the normally-consolidated range, the values of net ultimate bearing capacity obtained from the consolidated-undrained compression triaxial test and drained test range from about 540 psf to about 685 psf which are considered very low values. Normally, the allowable net bearing capacity used in Winnipeg is about 2,000 psf. This results in stresses not appreciably exceeding the lower preconsolidation pressure indicated by the tests, and generally less than the maximum indicated preconsolidation pressure. Consequently, the low bearing capacity value corresponding to the shear strength parameters beyond the preconsolidation pressure, has no practical meaning.

If the soil is homogeneous and isotropic, c' and ϕ' are constant for a given soil. But Winnipeg clay is laminated and anisotropic, therefore, the question arises whether or not the extension test may give a bearing capacity closer to an actual value in foundation problems since both of the standard compression and extension triaxial tests correspond to a passive earth pressure condition. In the standard compression test, the minor principal stress, σ_3, was equal to the intermediate principal stress, σ_2, and was equal to the all-around cell pressure. The sample was brought to failure by increasing σ_1 which was the vertical stress. In the extension test, σ_3 was equal to σ_2 and was also equal to all-around cell pressure. The sample was brought to failure by increasing σ_3 and σ_2 while σ_1 was kept constant. Consider the typical pattern of rupture surfaces in the soil under a foundation in Figure 29. When failure occurs, the soil in the passive zone will have an increased horizontal stress and constant vertical stress. This is simulated by the extension test in the laboratory. Therefore, for anisotropic soil it may be argued that the extension test is more applicable in the passive zone and should give the more reliable shear strength parameters for bearing capacity calculation. The calculated net bearing capacity values for a typical large and small footing show there is not much difference between the values based on the consolidated-undrained triaxial compression and extension tests. Therefore, for anisotropic soil it may be argued that the extension test is more applicable in the passive zone and should give the more reliable shear strength parameters for bearing capacity calculation. The calculated net bearing capacity values for a typical large and small footing show there is not much difference between the values based on the consolidated-undrained triaxial compression and extension tests. Therefore, for Winnipeg clay it does not appreciably matter whether the test is performed by using consolidated-undrained triaxial compression or extension test.

The long term net ultimate bearing capacity obtained from the consolidated-drained test is somewhat lower than that obtained from the consolidated-undrained triaxial test, and therefore more conservative. A lower factor of safety, for example, 2.5 as compared to 3.0 may be justified in the case of the drained test values.

The net ultimate bearing capacity values obtained from the peak and residual shear strength parameters are about 6,000 psf and 1,300 psf respectively for the 10 feet width and 6 feet depth, strip footing. For the narrower strip footing with 2.5 feet width and 1.5 feet depth, the net ultimate bearing capacity values corresponded to the peak and residual shear strength parameters are 3,200 psf and 320 psf respectively. These results of tests can be compared to the Winnipeg Building Code for soft clay value. When the comparison is made, the peak shear strength parameters give the bearing capacity about 2.4 times the code value for the wider footing, and about 1.3 times the code value for the narrower footing. The residual shear strength parameters give the bearing capacity about 0.5 times the code value for the wider footing, and about 0.13 times the code value for the narrower footing. The comparison between the net ultimate bearing capacity based on the triaxial test and the direct shear test is not made because of difference in the soil samples.

The residual shear strength parameters do not give a reasonable bearing capacity value. Settlement requirements and factor of safety preclude large strains, and the reduction of strength does not take place.

2 The Application of the Shear Strength Parameters to The Solution of Slope Stability Problems

2.1 Theory

If the undrained shear strength of a slope is measured by the consolidated undrained or unconfined compression test, the expression

$$\tau = C_u \quad \ldots \quad (5)$$

where, τ = shear strength; and

C_u = the apparent cohesion,

is inserted in the stability analysis which is called the $\phi = 0$ analysis. In this particular case

$$C_u = \tfrac{1}{2}(\sigma_1 - \sigma_3)_f \quad \ldots \quad (6)$$

where, $(\sigma_1 - \sigma_3)_f$ = the deviator stress at failure.

Since the unconfined compression test is a simple and economical test, it raises the question as to how applicable the $\phi = 0$ analysis is the stability problems in clay. L. Bjerrum and B. Kjaernsli[14] have found that for normally-consolidated clays, the stability analysis based on the undrained shear strength of the clay gives too low safety factors, and thus leads to unreliable results in the case of long term stability of natural slopes. The error in safety factor using the ϕ = 0 analysis for the long term stability of a natural slope of stiff clay was found to be over-estimated up to 2,000%. In soft clay, the stability tends to be under-estimated. In a heavily over-consolidated stiff clay, Henkel and Skempton[15] have reported that the ϕ = 0 analysis has led to an over-estimation of the factor of safety. The reason of the source of error in using the total stress analysis is explained as the dissipation of the negative pore pressure with time and the pore pressure acting on the failure plane will be determined solely by the ground water conditions. Therefore, the undrained tests, in general, cannot give reliable estimates or predictions of factor of safety for slope in over-consolidated clays either.

When the pore pressure is determined, the expression

$$\tau = c' + (\sigma - u)_f \tan \phi' \quad \ldots \quad (7)$$

where, $(\sigma-u)_f$ = the effective stress at failure;

c' = the effective cohesion; and

ϕ' = the effective angle of internal friction,

is used in the analysis. This is called "the effective stress analysis".

The studies of the slides indicate that the effective stress analysis yields satisfactory results for investigating the long term stability of slopes in normally consolidated and overconsolidated, intact clays (Bjerrum and Kjaernsli)[14]. Henkel and Skempton[16] have reported that in several analysis of very long term slips in over-consolidated fissured clays, the effective stress analysis gives over-estimated factors of safety. However, the error is found to be less than the total stress analysis. It is suggested by some field evidence that if the cohesion intercept of the failure envelope c' is neglected and the slope analysed in terms of the angle of shearing resistance ϕ' only, the good indication of stability is possible to be obtained. The reason of the reduction in apparent cohesion is probably due to a combination of factors as cyclical stress changes, local movements, and fissures (Henkel and Skempton)[15]. It is in constrast to the report of another slope analysis which is given by Skempton and Brown[16]. They reported that the full cohesion intercept, c', is operative on the actual slip surfaces in the lightly and heavily over-consolidated intact clay, where the c' is zero gave the values of factors of safety much less than one.

When the residual strength is determined, the shear strength can be expressed as the following:

$$\tau_R = c'_R + (\sigma-u)_f \tan \phi'_R$$

where, τ_R = the residual shear strength;

$(\sigma-u)_f$ = the effective stress at failure;

c'_R = the effective residual cohesion; and

ϕ'_R = the effective residual friction angle.

In most cases c'_R is almost zero, therefore, the residual shear strength may be written as the following:

$$\tau_R = (\sigma-u)_f \tan \phi'_R$$

Skempton[17] has explained that a fissured or jointed clay would not be able to develop a peak strength along the full length of the slip surface. Also, the cracks and holes can cause the peak to be crossed. By these reasons, when the fissured or jointed clay is concerned in stability analysis, the residual strength should be used.

In the case of a pre-sliding slope, the use of residual strength is valid since the shear strength has already been reduced to the residual value. In a slope that a progressive failure surface is expected, Bjerrum[18] reported that the residual shear strength must be used in slope stability analysis of progressive failures, and indeed, most of the slope failures in over-consolidated plastic clays and clay shales are proceeded by a mechanism of progressive failure.

2.2 Discussion

Experience elsewhere gives a guide to the appropriate shear strength parameters to be used in landslide, river bank, and embankment stability analyses. In the case of landslides, Thomson[19] investigated the Lesuerer landslide located on the outside of a bend of the North Saskatchewan River. The stratigraphic profile consisted of fine glacial lake sand, till, terrace sands and gravel overlying clay shales. The slide occurred in the clay shales. A series of first and second analyses indicated that the erosion of the terrace at the toe of the slope due to lateral migration of the river decreased the strength from the peak to the residual value over a long period of time possibly accompanied by creep movements. In the case of river banks, Thomson[20] did a stability study at the University of Alberta in Edmonton. The stratigraphic sections consisted of glacial lake sediments, till, preglacial sands and gravels and clay shales. He found that the river bank which had failed by uplift and erosion had a low factor of safety when residual strength parameters were used in an infinite slope analysis. The use of the residual strength parameters might have been somewhat conservative since the river bank had apparently never been subjected to movement. The use of peak strength parameters represented an upper bound of the slope stability. The lower bound may be derived using residual strength parameters.

Lo and Stermac[21] studied the failed roadway embankment at New Liskeard, Northern Ontario. The fill material was granular with a high percentage of pebbles and boulders placed in layers of 2 to 3 feet thick. The subsoil at the site was a layer of laminated silty clay approximately 8 feet thick lying on a varved clay stratum. The failure took place during construction period. The results indicated that using the average strength of all field and laboratory tests and taking tension crack and fill strength into account, the total stress (the $\phi = 0$) analysis led to an accuracy within 15% on the safe side of the factor of safety. The effective stress analysis yielded a factor of safety on the unsafe side by more than 20% unless the assumptions of no fill strength and zero cohesion intercept were made. Insley[22] also studied the failure of compacted clay embankment fill in the North Peace River area of Northern Alberta on the route of the Great Slave Lake Railway. The fill was done on a 10 foot brown clay underlain by a uniform grey stony clay. The total stress analysis yielded a factor of safety 1.01 when the mean unconfined compression strength for actual moisture contents, tension crack and the condition of failure passing just above the base of the fill were taken into account. The effective stress analysis yielded the minimum factor of safety of 1.24 which was not correct.

These selected case histories confirm the results of the theories observed by Skempton and others. Extending the principle involved to Winnipeg clays and using the test results obtained, a number of conclusions can be made.

For river banks and other slopes having long life, stability analyses based on residual effective shear strength parameters will give conservative results. Therefore, the peak parameters may be used as the upper limit and the residual parameters may be used as the lower limit. For the results obtained from the laboratory where:

c'_p = 144 psf, ϕ'_p = 20 degrees,

c'_R = 0 psf, ϕ'_R = 12.5 degrees,

and the moist unit weight, γ_m = 107 psf, the slope angle of a river bank with assumed 40 foot height and depth factor, D = 1, can be simply calculated according to Taylor[23].

The river banks in Winnipeg are subjected to changes in river level, and to seepage resulting from rainfall and snow melt. In either case, the condition can be for estimate purposes, represented by Taylor's rapid and complete drawdown case. These conditions would normally occur at least once a year. On this basis, the upper limit of the slope angle is 15 degrees corresponding to a short life, possibly no more than one year. For an assurance that sliding would never occur, the slope angle would have to be as low as 5 degrees. This latter figure presumes complete loss of cohesion. It is not known how quickly this loss occurs in the field. Indeed, it may never occur if the soil is not subjected to failure. It thus becomes very important to prevent failure if slopes steeper than 5 degrees are to be maintained. With even small cohesion, slopes of 10 to 12 degrees could be safe for a long time. It becomes most important, however, to avoid causing a failure in the first place. Adding of fill or construction on the top of a bank can cause failure, so can run-off, snow melt, etc. diverted to flow over the bank. This is to be avoided by providing quick adequate surface drainage. Erosion at the toe can cause sliding. Toe erosion protection is thus also very important.

Analysis of a 40 foot height bank using the $\phi = 0$ method would indicate that the critical slope angle is about 12 degrees using the lower unconfined compressive strength of 1130 psf and about 73 degrees for the higher value of 1810 psf obtained from the tests. The higher value would have a very short life. Tension cracks would soon form and reduce total strength available along the failure plane. The lower value of 12 degrees can be justified on the basis of effective stress parameters and assuming strength parameters having value between peak and residual values.

The conclusion follows that it is not possible on the basis of laboratory tests to obtain the safety factor of slopes except for extreme conditions. It is not known what value of effective strength parameters to be used when residual or peak strength are not applicable. The undrained strength parameters are only valid for a very short time after construction.

APPENDICES

APPENDIX A

GRAIN SIZE TEST RESULTS

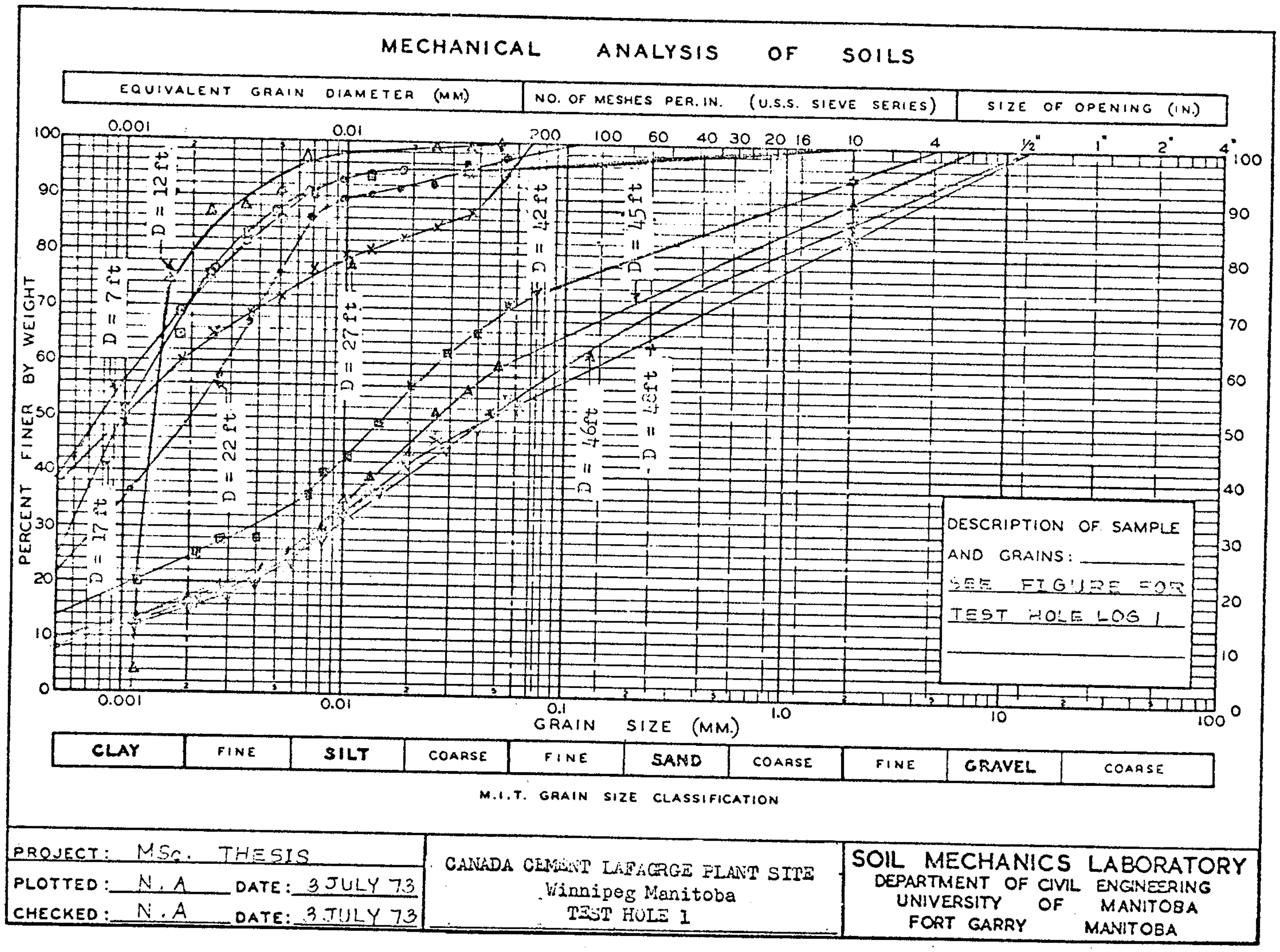

MECHANICAL ANALYSIS OF SOILS
EQUIVALENT GRAIN DIAMETER (MM)
NO. OF MESHES PER. IN. (U.S.S. SIEVE SERIES)
SIZE OF OPENING (IN.)
PERCENT FINER BY WEIGHT
GRAIN SIZE (MM.)
D = 7 ft
D = 12 ft
D = 17 ft
D = 22 ft
D = 27 ft
D = 42 ft
D = 45 ft
D = 46ft
D = 48ft
DESCRIPTION OF SAMPLE AND GRAINS:
SEE FIGURE FOR TEST HOLE LOG 1
CLAY
FINE
SILT
COARSE
FINE
SAND
COARSE
FINE
GRAVEL
COARSE
M.I.T. GRAIN SIZE CLASSIFICATION
PROJECT: MSc. THESIS
PLOTTED: N.A DATE: 3 JULY 73
CHECKED: N.A DATE: 3 JULY 73
CANADA CEMENT LAFARGE PLANT SITE
Winnipeg Manitoba
TEST HOLE 1
SOIL MECHANICS LABORATORY
DEPARTMENT OF CIVIL ENGINEERING
UNIVERSITY OF MANITOBA
FORT GARRY MANITOBA

FIGURE A-1

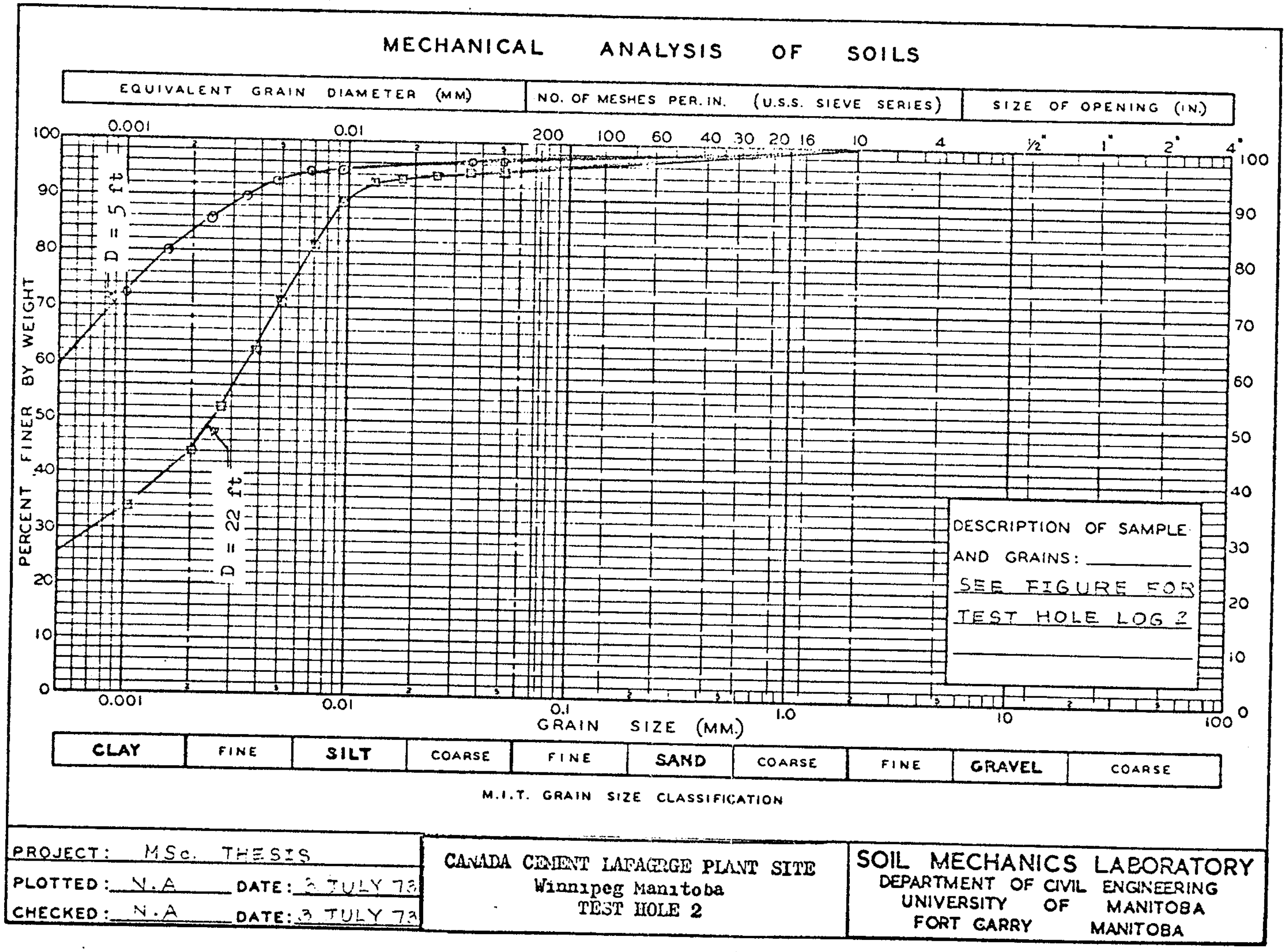

MECHANICAL ANALYSIS OF SOILS
EQUIVALENT GRAIN DIAMETER (MM)
NO. OF MESHES PER. IN. (U.S.S. SIEVE SERIES)
SIZE OF OPENING (IN.)
PERCENT FINER BY WEIGHT
D = 5 ft
D = 22 ft
GRAIN SIZE (MM.)
DESCRIPTION OF SAMPLE AND GRAINS:
SEE FIGURE FOR TEST HOLE LOG 2
CLAY
FINE
SILT
COARSE
FINE
SAND
COARSE
FINE
GRAVEL
COARSE
M.I.T. GRAIN SIZE CLASSIFICATION
PROJECT: MSc. THESIS
PLOTTED: N.A DATE: 3 JULY 73
CHECKED: N.A DATE: 3 JULY 73
CANADA CEMENT LAFARGE PLANT SITE
Winnipeg Manitoba
TEST HOLE 2
SOIL MECHANICS LABORATORY
DEPARTMENT OF CIVIL ENGINEERING
UNIVERSITY OF MANITOBA
FORT GARRY MANITOBA

FIGURE A-2

APPENDIX B

RESULTS AND TYPICAL DATA OF CONSOLIDATION TEST

CONSOLIDATION TEST DATA SHEET — Page 1 of 3

Project: MSC. THESIS
Sample no.: 3 Test hole no.: 1 Depth ft.: 16 - 18
Sample Description: GREY BROWN VARVED CLAY

SAMPLE MOISTURE CONTENTS

Tare + Ring + Soil + Water (End)	=	659.50	gm
Tare + Ring + Soil	=	637.31	gm
Tare (No. HXV)	=	287.00	gm
Ring + Soil + Water (Start)	=	380.90	gm
Ring + Soil	=	350.31	gm
Ring (No. 6)	=	295.27	gm
Water (Start)	=	30.59	gm
Water (End)	=	22.21	gm
Soil Solids	=	55.04	gm
Water (Start)	=	55.58	%
Water (End)	=	40.32	%

LOADING FRAME DATA

Section no.	=	4	
Load multipaction factor	=	10.00	
Weight of cap + ball	=	354.51	lb.

SAMPLE DIMENSIONS

Diameter	=	2.53	in
Thickness	=	0.62	in
Cross-Sectional Area	=	32.60	cm^2

COMMENTS

Elapsed time from start of new loading / Ames Dial Reading, 10^{-4} inches

Pan Load lb.	1/2		1		2		5		2		1		2	
Date (start)	7/JULY/73		9/JULY/73		10/JULY/73		11/JULY/73		13/JULY/73		14/JULY/73		16/JULY/73	
Time (start)	14.20		10.30		10.45		11.50		11.55		11.01		10.35	
	0	0.5000	0	0.4853	0	0.4869	0	0.4918	0	0.5061	0	0.4987	0	0.4929
	15^{s}	0.5008	15^{s}	0.4856	15^{s}	0.4877	15^{s}	0.4942	15^{s}	0.5052	15^{s}	0.4980	15^{s}	0.4931
	30^{s}	0.5008	30^{s}	0.4856	30^{s}	0.4878	30^{s}	0.4946	30^{s}	0.5051	30^{s}	0.4980	30^{s}	0.4932
	1^{M}	0.5009	1^{m}	0.4857	1^{m}	0.4879	1^{m}	0.4949	1^{m}	0.5050	1^{M}	0.4980	1^{m}	0.4932
	2^{M}	0.5010	2^{M}	0.4858	2^{m}	0.4880	2^{M}	0.4953	2^{m}	0.5048	2^{M}	0.4979	2^{M}	0.4933
	4^{M}	0.5009	4^{M}	0.4858	4^{M}	0.4882	4^{M}	0.4960	4^{M}	0.5042	4^{M}	0.4976	4^{M}	0.4937
	8^{m}	0.4998	8^{M}	0.4860	8^{M}	0.4886	8^{M}	0.4970	8^{M}	0.5039	8^{M}	0.4972	8^{M}	0.4939
	15^{M}	0.4982	15^{M}	0.4861	15^{M}	0.4890	15^{M}	0.4991	15^{M}	0.5030	15^{M}	0.4970	15^{M}	0.4940
	30^{M}	0.4968	30^{M}	0.4862	30^{M}	0.4894	30^{M}	0.5000	30^{M}	0.5022	30^{M}	0.4965	30^{M}	0.4942
	1^{h}	0.4948	1^{h}	0.4864	1^{h}	0.4900	1^{h}	0.5018	1^{h}	0.5010	1^{h}	0.4958	1^{h}	0.4948
	2^{h}	0.4925	2^{h}	0.4867	2^{h}	0.4908	2^{h}	0.5030	2^{h}	0.5000	2^{h}	0.4950	2^{h}	0.4950
	5^{h}	0.4891	4^{h}	0.4868	4^{h}	0.4911	4^{h}	0.5050	4^{h}	0.4994	4^{h}	0.4943	4^{h}	0.4952
	20^{h}	0.4865	6^{h}	0.4869	9^{h}	0.4913	8^{h}	0.5056	8^{h}	0.4991	8^{h}	0.4938	9^{h}	0.4954
	24^{h}	0.4860	10^{h}	0.4869	12^{h}	0.4917	24^{h}	0.5060	47^{h}	0.4987	$26\frac{1}{2}^{h}$	0.4930	25^{h}	0.4955
	44^{h}	0.4853	24^{h}	0.4869	24^{h}	0.4918	48^{h}	0.5061			$47\frac{1}{2}^{h}$	0.4923		

Tested for: CANADA CEMENT LAFARGE OIL TANK
Address: WINNIPEG, CANADA
Tested by: N.A
Date Completed: 30/JULY/1973

UNIVERSITY OF MANITOBA
Department of Civil Engineering
SOIL TESTING LABORATORY
Fort Garry   Manitoba

CONSOLIDATION TEST DATA SHEET

Project MSc. THESIS
Sample no. 3 Test hole no. 1 Depth ft. 16 - 18
Sample Description GREY BROWN VARVED CLAY

SAMPLE MOISTURE CONTENTS

Tare + Ring + Soil + Water (End)	=	659.50	gm
Tare + Ring + Soil	=	637.31	gm
Tare (No. HXV)	=	287.00	gm
Ring + Soil + Water (Start)	=	380.90	gm
Ring + Soil	=	350.31	gm
Ring (No. 6)	=	295.27	gm
Water (Start)	=	30.59	gm
Water (End)	=	22.21	gm
Soil Solids	=	55.04	gm
Water (Start)	=	55.58	%
Water (End)	=	40.32	%

LOADING FRAME DATA

Section no.	=	4	
Load multipaction factor	=	10.00	
Weight of cap + ball	=	354.51	lb.

SAMPLE DIMENSIONS

Diameter	=	2.53	in
Thickness	=	0.62	in
Cross-Sectional Area	=	32.60	cm^2

COMMENTS

Pan Load lb.	5		10		20		10		5		2		5	
Date (start)	17/JULY/73		19/JULY/73		20/JULY/73		21/JULY/73		24/JULY/73		25/JULY/73		26/JULY/73	
Time (start)	13.30		14.11		14.22		14.15		12.55		12.55		12.57	
Elapsed time from start of new loading / Ames Dial Reading, 10^{-4} inches	0	0.4955	0	0.5060	0	0.5213	0	0.5468	0	0.5342	0	0.5223	0	0.5091
	15s	0.4970	15s	0.5080	15s	0.5269	15s	0.5450	15s	0.5332	15s	0.5214	15s	0.5106
	30s	0.4972	30s	0.5083	30s	0.5272	30s	0.5448	30s	0.5330	30s	0.5212	30s	0.5108
	1m	0.4976	1m	0.5089	1m	0.5230	1m	0.5443	1m	0.5328	1m	0.5210	1m	0.5110
	2m	0.4980	2m	0.5093	2m	0.5290	2m	0.5439	2m	0.5324	2m	0.5207	2m	0.5116
	4m	0.4985	4m	0.5101	4m	0.5301	4m	0.5432	4m	0.5320	4m	0.5200	6m	0.5120
	8m	0.4992	8m	0.5111	8m	0.5319	8m	0.5423	8m	0.5311	8m	0.5192	8m	0.5125
	15m	0.5000	15m	0.5127	15m	0.5340	15m	0.5410	15m	0.5308	15m	0.5183	15m	0.5132
	30m	0.5014	30m	0.5146	30m	0.5372	30m	0.5390	30m	0.5287	30m	0.5170	30m	0.5142
	1h	0.5030	1h	0.5160	1h	0.5404	1h	0.5372	1h	0.5269	1h	0.5160	1h	0.5157
	2h	0.5042	2h	0.5188	2h	0.5431	2h	0.5358	2½h	0.5247	2h	0.5132	2h	0.5168
	6h	0.5052	5h	0.5202	4h	0.5448	5h	0.5350	6h	0.5236	5h	0.5110	5h	0.5175
	8h	0.5054	8h	0.5206	6h	0.5477	24h	0.5343	3h	0.5230	24h	0.5031	25h	0.5179
	24h	0.5058	24h	0.5213	24h	0.5465	50h	0.5343	24h	0.5223				
	48h	0.5060					72h	0.5342						

Tested for CANADA CEMENT LAFARGE OIL TANK
Address WINNIPEG, CANADA
Tested by N.A
Date Completed 30/JULY/1973

UNIVERSITY OF MANITOBA
Department of Civil Engineering
SOIL TESTING LABORATORY
Fort Garry Manitoba

CONSOLIDATION TEST DATA SHEET

Project MSc. THESIS
Sample no. 3 Test hole no. 1 Depth ft. 16 - 18
Sample Description GREY BROWN VARVED CLAY

SAMPLE MOISTURE CONTENTS

Tare + Ring + Soil + Water (End)	=	659 . 50	gm
Tare + Ring + Soil	=	637 . 31	gm
Tare (No. HXV)	=	287 . 00	gm
Ring + Soil + Water (Start)	=	380 . 90	gm
Ring + Soil	=	350 . 31	gm
Ring (No. 6)	=	295 . 27	gm
Water (Start)	=	30 . 59	gm
Water (End)	=	22 . 21	gm
Soil Solids	=	55 . 04	gm
Water (Start)	=	55 . 58	%
Water (End)	=	40 . 32	%

LOADING FRAME DATA

Section no.	=	4	
Load multipaction factor	=	10 . 00	
Weight of cap + ball	=	354 . 51	lb.

SAMPLE DIMENSIONS

Diameter	=	2 . 53	in
Thickness	=	0 . 62	in
Cross-Sectional Area	=	32 . 60	cm^2

COMMENTS

Pan Load lb.	10		20		40	
Date (start)	27/JULY/73		28/JULY/73		29/JULY/73	
Time (start)	12.55		14.20		14.20	
	Elapsed time from start of new loading	Ames Dial Reading, 10^{-4} inches	Elapsed time from start of new loading	Ames Dial Reading, 10^{-4} inches	Elapsed time from start of new loading	Ames Dial Reading, 10^{-4} inches
	0	0.5179	0	0.5307	0	0.5509
	15^s	0.5195	15^s	0.5345	15^s	0.5610
	30^s	0.5198	30^s	0.5350	30^s	0.5620
	1^m	0.5200	1^m	0.5354	1^m	0.5630
	2^m	0.5207	2^m	0.5361	2^m	0.5647
	4^m	0.5212	4^m	0.5373	4^m	0.5675
	8^m	0.5222	8^m	0.5388	8^m	0.5721
	15^m	0.5232	15^m	0.5404	15^m	0.5727
	30^m	0.5250	30^m	0.5439	30^m	0.5738
	1^h	0.5271	1^h	0.5460	1^h	0.5787
	2^h	0.5289	2^h	0.5483	2^h	0.5830
	8^h	0.5303	7^h	0.5500	6^h	0.5857
	24^h	0.5307	24^h	0.5503	20^h	0.5880
					24^h	0.5895

Tested for CANADA CEMENT LAFARGE OIL TANK
Address WINNIPEG , CANADA
Tested by N.A
Date Completed 30 / JULY / 1973

UNIVERSITY OF MANITOBA
Department of Civil Engineering
SOIL TESTING LABORATORY
Fort Garry Manitoba

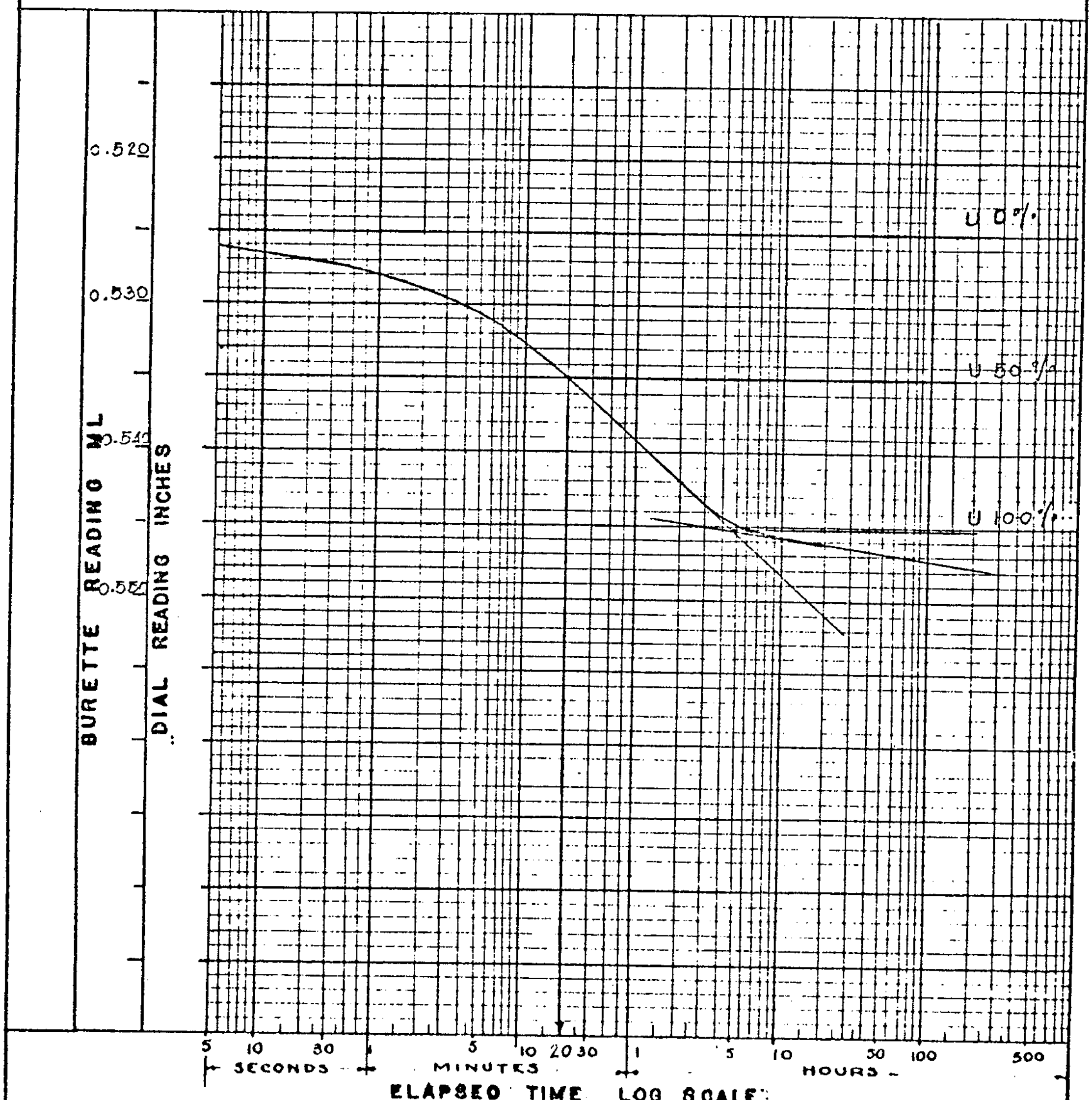
CONSOLIDATION TIME CURVES

JOB NO. ________
TEST NO. ________

TYPE OF TEST __ CONSOLIDATION ☒, TRIAXIAL COMPRESSION ☐.
TEST FRAME NO. __4__, LOAD MULTIPLICATION FACTOR __10.00__.
PAN LOAD INCREMENT FROM __10__ LB. TO LB. __20__

FOR TRIAXIAL TEST ONLY:

TYPE _ CONSOLIDATED UNDRAINED ☐ , CONSOLIDATED DRAINED ☐ .
CELL PRESSURE INCREMENT FROM ______ PSI. TO ______ PSI.

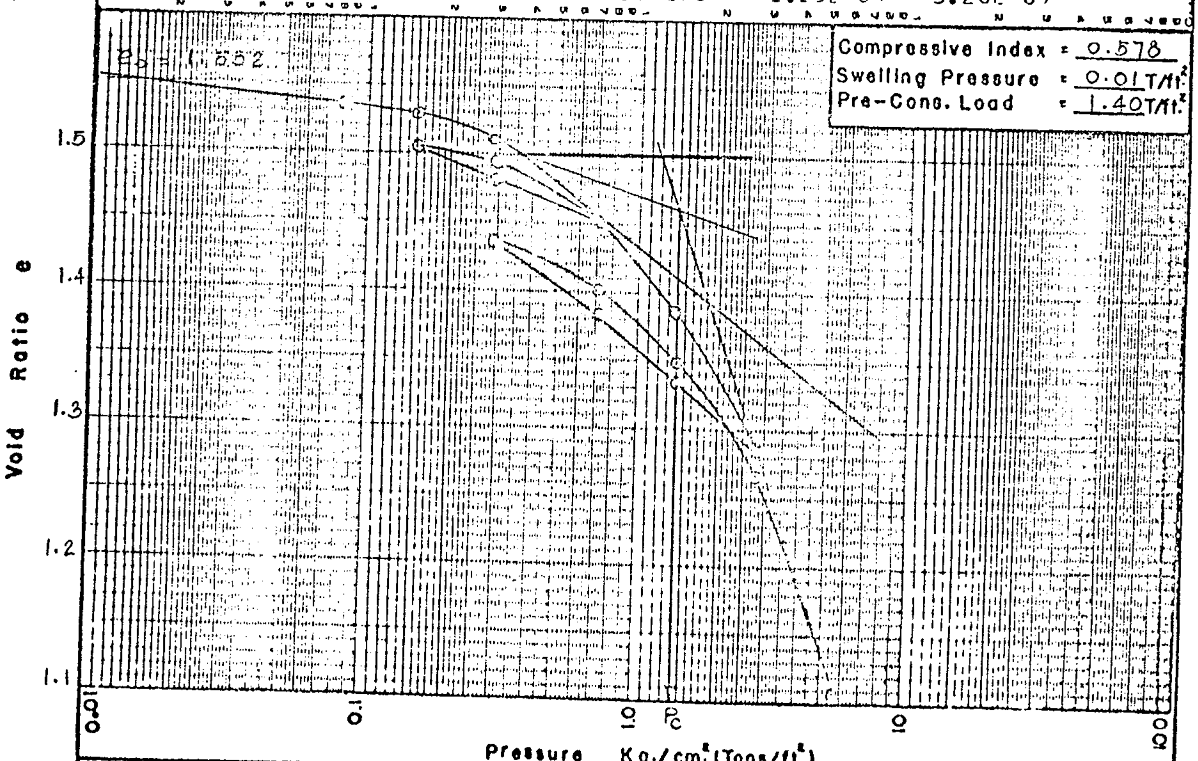


PROJECT __MSc. THESIS__
SAMPLE __2__ HOLE __1__ DEPTH __16-18__
LOCATION __CANADA CEMENT OIL TANK__
TECHNICIAN __N. A__ DATE __20 / JULY / 73__

SOIL MECHANICS LABORATORY
DEPARTMENT OF CIVIL ENGINEERING
UNIVERSITY OF MANITOBA
FORT GARRY MANITOBA

PERCENT MOISTURE, START= 55.58 END= 40.32
INITIAL VOID RATIO MEASURED=1.552
DEGREE OF SATURATION, PERCENT= 98.5
UNIT WT START PCF, MOIST= 104.6 DRY= 67.2

PAN LOAD LB	DIAL READING INCHES	VOID RATIO	STRESS KG PER SQCM	COEF.OF CONSOL. SQCM/SEC	PERM. COEF. CM/SEC.
0.0	0.5000	1.4771	—		
0.5	0.4853	1.5376	0.081		
1.0	0.4869	1.5310	0.151		
2.0	0.4918	1.5108	0.291		
5.0	0.5061	1.4520	0.7104	8.17E-05	4.62E-09
2.0	0.4987	1.4824	0.291		
1.0	0.4929	1.5063	0.151		
2.0	0.4955	1.4956	0.291		
5.0	0.5060	1.4524	0.710		
10.0	0.5213	1.3894	1.4098	8.47E-05	3.15E-09
20.0	0.5465	1.2857	2.8087	9.10E-05	2.89E-09
10.0	0.5342	1.3363	1.4098	1.02E-04	1.59E-09
5.0	0.5223	1.3853	0.710		
2.0	0.5091	1.4396	0.291		
5.0	0.5179	1.4034	0.710		
10.0	0.5307	1.3507	1.4098	8.35E-05	2.65E-09
20.0	0.5509	1.2676	2.8087	9.35E-05	2.41E-09
40.0	0.5895	1.1087	5.6066	1.23E-04	3.20E-09



PROJECT MSc. THESIS SAMPLE 3 HOLE 1 DEPTH 16-18 FT. TESTED N.A DATE 30/JULY/73	SOIL MECHANICS LABORATORY DEPARTMENT OF CIVIL ENGINEERING UNIVERSITY OF MANITOBA FORT GARRY MANITOBA

APPENDIX C

RESULTS AND TYPICAL DATA OF UNCONFINED COMPRESSION TEST

UNCONFINED COMPRESSION TEST

LAB ORDER NO

PROJECT MSc. THESIS

SAMPLE NO 3 TEST HOLE NO 1 DEPTH FT 21 - 23

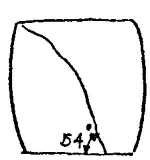
SAMPLE DESCRIPTION GRAY, STIFF CLAY WITH GRAVEL ½" ϕ (ONE GRAVEL FOUND)

Load Dial	Strain Dial	Total Strain In	Unit Strain	1 - Unit Strain	Corrected Area	Load In lb	Vertical Stress psi
0	0.800	0.000	0.0000	1.0000	7.650	0	0
81	0.775	0.025	0.0064	0.9936	7.700	25.2	3.25
125	0.750	0.050	0.0127	0.9873	7.750	39.0	5.00
161	0.725	0.075	0.0191	0.9809	7.800	50.2	6.40
197	0.700	0.100	0.0254	0.9746	7.860	61.8	7.80
234	0.675	0.125	0.0319	0.9681	7.900	73.0	9.15
268	0.650	0.150	0.0383	0.9617	7.950	83.5	10.41
298	0.625	0.175	0.0446	0.9554	8.010	93.0	11.50
319	0.600	0.200	0.0510	0.9490	8.075	99.0	12.20
321	0.587	0.213	0.0545	0.9455	8.075	100.0	12.38
310	0.575	0.225	0.0575	0.9425	8.100	96.5	11.80

SAMPLE DIMENSIONS

Diameter Top In	3.110
Middle In	3.125
Bottom In	3.110
Average In.	3.115
Average Height In	3.917

SKETCH AT FAILURE



SAMPLE MOISTURE CONTENTS

		START	END
Container No		41 B	
Wt Container + Moist Sample	gm	1035.50	1034.00
Wt Container + Dry Sample	gm		738.00
Wt Container	gm	205.40	205.40
Wt Moist Sample	gm		828.60
Wt Moisture	gm		296.00
Wt. Dry Sample	gm		532.60
Moisture Content	%		55.20

Specific Gravity 2.75 From Test ______ Assumed ✓

Volume of Sample cc 495.0 cu in 30.2

Volume of Soil Solids cc 194.0 Void

Volume of Voids cc 301.0 Ratio 1.48

Degree of Saturation % 98.3

Density lb per cu ft Dry 67.0 Moist 104.0

Tested For CANADA CEMENT LAFARGE OIL TANK

Address WINNIPEG, CANADA

Tested By N.A Date 5 JULY 1973

Calculated By N.A Date 21 JULY 1973

SOIL MECHANICS LABORATORY
DEPARTMENT OF CIVIL ENGINEERING
UNIVERSITY OF MANITOBA
FORT GARRY MANITOBA

APPENDIX D

RESULTS AND TYPICAL DATA OF TRIAXIAL TESTS

1. Consolidated Undrained Triaxial Compression Test With Pore Pressure Measurement
2. Consolidated Undrained Triaxial Extension Test With Pore Pressure Measurement
3. Consolidated Drained Triaxial Test
4. Constant $\left(\frac{\bar{\sigma}_1+\bar{\sigma}_2+\bar{\sigma}_3}{3}\right)$ Test

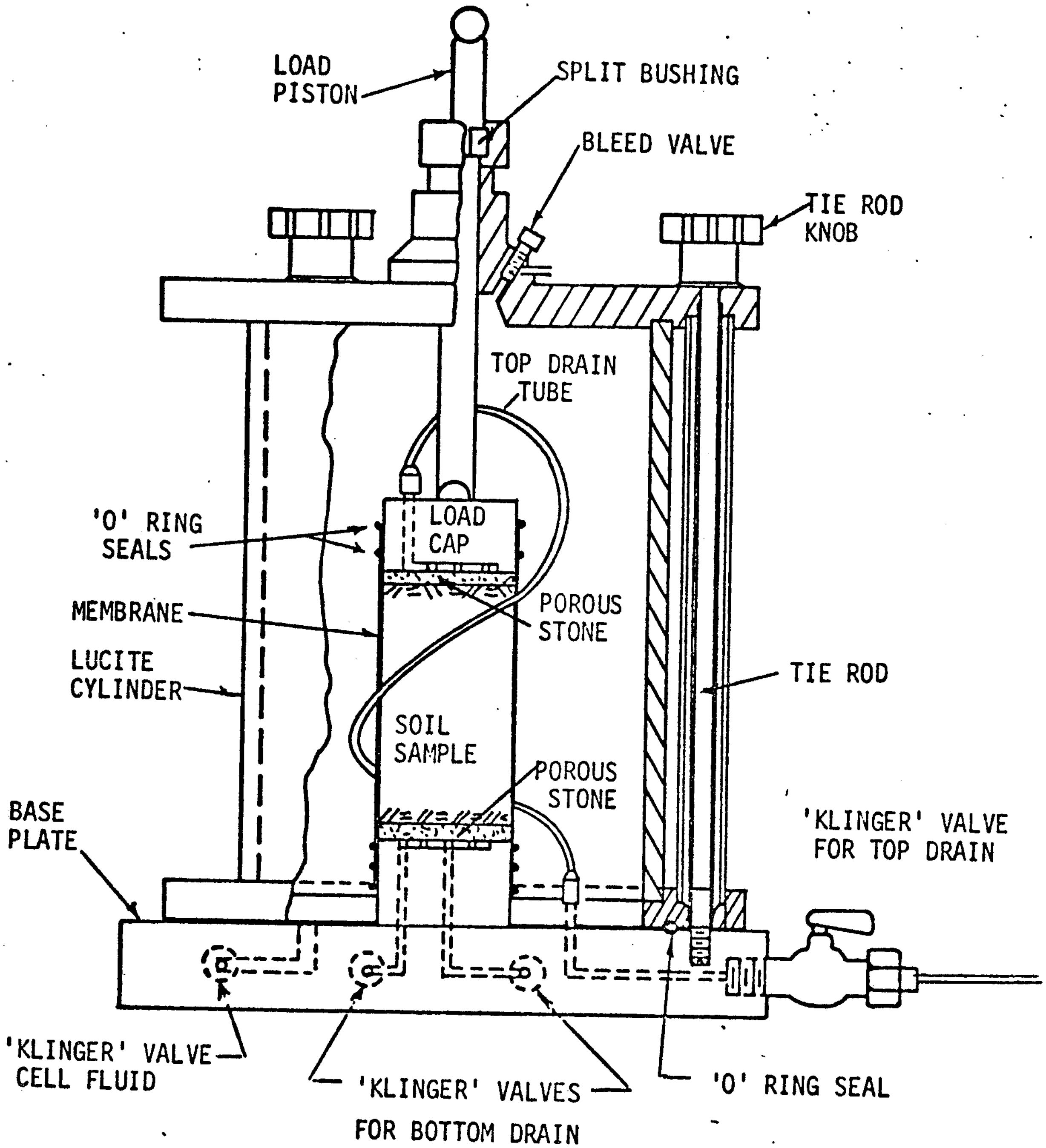


FIGURE C-1 - TRIAXIAL CELL, (PARTIAL SECTION SHOWING INTERIOR)

UNIVERSITY OF MANITOBA
CIVIL ENGINEERING DEPARTMENT
SOIL MECHANICS LABORATORY

NOTES

1. For pressure range 0 to 62 psi, close valves G_1 and G_2, open valve G_3.
2. For pressure range 62 to 124 psi, close valve G_3, open valve G_1 and G_2.
3. Close valve G_4 when making adjustment to pressure or when filling upper reservoirs with mercury.

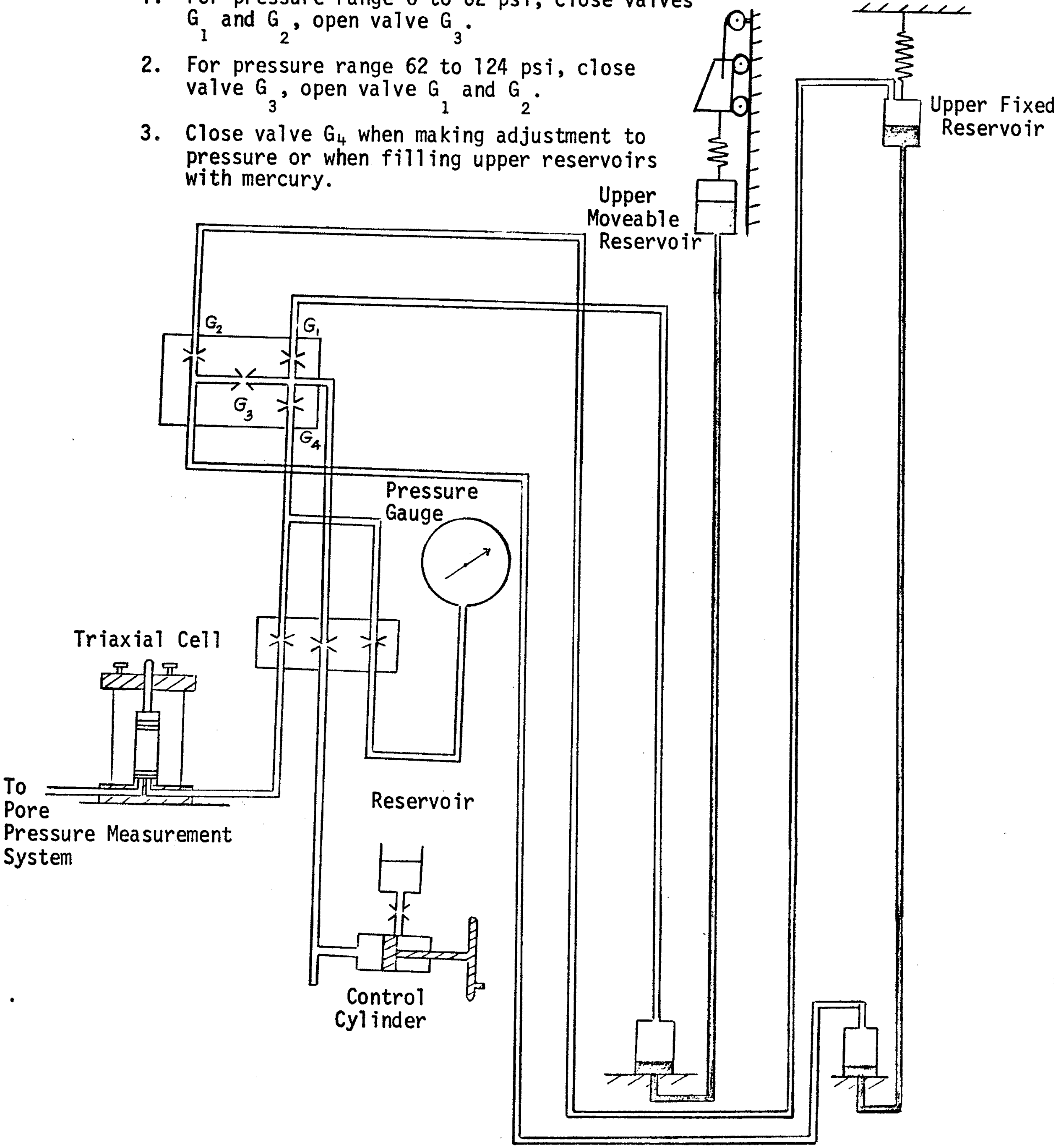


FIGURE C-2

ANCILLARY DATA FOR TRIAXIAL OR DIRECT SHEAR TEST

PAGE 1 OF 2

PROJECT MSc. THESIS

JOB NO. ____ TEST HOLE NO. 1 SAMPLE NO. 4 DEPTH 16 - 18 FT.

DESCRIPTION OF SAMPLE GREY BROWN VARVED CLAY

METHOD OF PREPARATION TRIMMING

TYPE OF TEST CONSOLIDATED UNDRAINED TRIAXIAL COMPRESSION TEST

CELL NO. 9 DIRECT SHEAR BOX NO. ____ MACHINE NO. 1

COMMENTS

SPECIFIC GRAVITY OF SOLIDS 2.75 ☐ FROM TEST, ☒ ASSUMED

SAMPLE DIMENSIONS

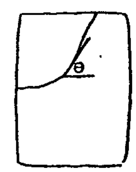
DIAMETER INCHES		X	Y		LENGTH INCH	WIDTH INCH
TOP	=	1.395		OR		
MIDDLE	=	1.395				
BOTTOM	=	1.397				
AVERAGE	=	1.395				

HEIGHT = 2.930 INCH = 7.36 CM

X-AREA = 1.525 SQ IN = 9.85 SQ CM

VOLUME = 4.470 CU IN = 73.50 CU CM

SKETCH OF FAILURE



ANGLE OF SHEAR PLANE = 45°

SAMPLE MOIST WEIGHTS

		BEFORE PLACING CELL	AFTER REMOVING FROM CELL
TARE NO.		D	D
WT. SAMPLE + TARE, GM	=	220.22	216.01
WT. TARE, GM	=	95.97	95.97
WT OF SAMPLE, GM	=	124.25	119.04
CHANGE OF WEIGHT DURING TEST, GM	=		5.21

MOISTURE CONTENTS

		INITIAL		AFTER COMPLETION OF TEST			
		TRIMMING	TRIMMING	TOP	FAILURE PLANE	BOTTOM	ENTIRE SAMPLE
TARE NO.		AR		U 51	U 56	U 63	D
WT. SOIL+WATER+TARE, GM	=	79.87		43.22	61.29	42.93	157.13
WT. SOIL + TARE, GM	=	63.05		39.22	51.38	37.34	130.21
WT. TARE, GM	=	31.92		31.72	31.63	26.39	95.97
WT. WATER, GM	=	16.62		4.00	9.91	5.59	15.92
WT. SOIL, GM	=	31.12		7.50	19.75	10.95	42.24
PERCENT MOISTURE	=	54.00		53.40	50.20	51.00	44.75

WEIGHT - VOLUMETRIC RELATIONSHIP

INITIAL CONDITIONS			AFTER ISOTROPIC CONSOLIDATION		
VOLUME OF SAMPLE,	=	73.50	VOLUME CHANGE FROM BURETTE, CU CM	=	4.90
WEIGHT OF SOLIDS, GM	=	80.60	WEIGHT OF WATER, GM	=	38.75
VOLUME OF SOLIDS, CU CM	=	29.35	CHANGE IN HEIGHT, CM	=	0.42
VOLUME OF WATER, CU CM	=	43.65	HEIGHT AFTER CONSOLIDATION, CM	=	6.93
VOLUME OF VOIDS, CU CM	=	44.15	VOLUME AFTER CONSOLIDATION, CU CM	=	68.60
DEGREE OF SATURATION	=	98.80	X-AREA, SQ CM = 9.90 , SQ IN	=	1.53
			VOLUME OF VOIDS, CU CM	=	39.25
VOID RATIO	=	1.50	VOID RATIO	=	1.34
MOISTURE CONTENT, %	=	54.00	MOISTURE CONTENT, %	=	40.00
UNIT WEIGHT DRY, PCF	=	[illegible]	UNIT WEIGHT DRY, PCF	=	73.50
UNIT WEIGHT MOIST, PCF	=	105.50	UNIT WEIGHT MOIST, PCF	=	109.00

Tested by N.A Date 5 Dec. 73

Calculated by N.A Date 5 Dec. 73

Checked by N.A Date 5 Dec. 73

SOIL MECHANICS LABORATORY
Department of Civil Engineering
University of Manitoba
Fort Garry Manitoba

TRIAXIAL TEST WITH PORE WATER PRESSURE MEASUREMENT

JOB NO = M.Sc. THESIS DATE 5 Dec. 73
SAMPLE NO = 4 PAGE 2 OF 2

TIME	ELAPSED TIME	LOAD DIAL DIVISIONS or PAN LOAD lb	CELL PRESSURE psi	PORE WATER PRESSURE GAUGE psi		u psi	STRAIN DIAL inch	ΔL inch	$\frac{\Delta L \times 100\%}{L_1}$	$1 - \frac{\Delta L}{L_1}$	AREA sq in	PROVING RING	AXIAL LOAD lb	$\bar{\sigma}_3$ psi	$\bar{\sigma}_1 - \bar{\sigma}_3$ psi	$\bar{\sigma}_1$ psi	$\frac{\bar{\sigma}_1}{\bar{\sigma}_3}$
10.30			100	9.20		0	0.4670	0	0	1.0000	1.520	0	0	90.80	0	90.80	1.00
11.30				12.75		3.55	0.4700	0.0030	0.1100	0.9989	1.520	48.0	11.04	87.25	7.25	94.50	1.08
13.00				25.75		16.55	0.4810	0.0140	0.5125	0.9949	1.530	164.0	37.70	74.25	24.65	98.90	1.33
14.00				32.20		23.00	0.4900	0.0230	0.8420	0.9916	1.534	213.5	49.05	67.80	32.00	99.80	1.47
15.00				36.80		27.60	0.5000	0.0330	1.2100	0.9879	1.538	250.0	57.50	63.20	37.40	100.60	1.59
16.10				41.20		32.00	0.5117	0.0447	1.6380	0.9836	1.546	277.8	63.90	58.60	41.30	100.10	1.72
17.50				44.00		34.80	0.5222	0.0552	1.8500	0.9815	1.550	295.0	67.60	56.00	43.75	99.75	1.78
18.10				47.00		37.80	0.5360	0.0690	2.5300	0.9747	1.560	311.2	71.50	53.00	45.90	98.90	1.86
18.33				47.00		37.80	0.5400	0.0730	2.6750	0.9732	1.562	314.5	72.30	53.00	46.30	99.30	1.87
19.20				48.60		38.00	0.5500	0.0830	3.0400	0.9696	1.567	321.5	74.00	51.40	47.25	98.65	1.92
20.15				50.20		39.00	0.5620	0.0950	3.4800	0.9652	1.575	325.5	74.80	49.80	47.50	97.30	1.95
21.07				51.50		42.30	0.5740	0.1070	3.9200	0.9608	1.580	327.7	75.35	48.50	47.60	96.10	1.98
21.47				52.20		43.00	0.5830	0.1160	4.2500	0.9575	1.588	328.0	75.40	47.80	47.50	95.30	1.99
21.50				52.00		42.80	0.5840	0.1170	4.2900	0.9571	1.588	327.9	75.40	48.00	47.60	95.50	1.99
22.15				52.40		43.20	0.5900	0.1230	4.5000	0.9550	1.591	327.0	75.20	47.60	47.25	94.85	1.99
23.05				52.80		43.60	0.6010	0.1340	4.9050	0.9509	1.598	322.0	74.00	47.20	46.25	93.45	1.97
23.35				52.60		43.40	0.6100	0.1430	5.2000	0.9480	1.602	312.0	71.70	47.40	44.75	92.15	1.94

TEST	APPROPRIATE DATA FROM PAGE 1
☒ $\bar{CU}$	After isotropic consolidation: L_1 = 2.73 inch. A_1 = 1.52 sq in.
☐ $\bar{UU}$	Without isotropic consolidation: $L_0 = L_1$ = ______ inch. $A_0 = A_1$ = ______ sq in.

Back pressure, u_b = 9.20 psi
Wt. ball + cap = ______ lb
P.P index correction = ______ psi
Load = (Dial Div.) x (0.23) lb, or:
= (Pan Load) x (______).

SOIL MECHANICS LABORATORY
DEPARTMENT OF CIVIL ENGINEERING
UNIVERSITY OF MANITOBA
FORT GARRY MANITOBA

ANCILLARY DATA FOR TRIAXIAL OR DIRECT SHEAR TEST — PAGE 1 OF 1

PROJECT MSc. THESIS
JOB NO. ______ TEST HOLE NO. 1 SAMPLE NO. 2 DEPTH 16 - 18 FT.
DESCRIPTION OF SAMPLE GREY BROWN VARVED CLAY

METHOD OF PREPARATION TRIMMING
TYPE OF TEST CONSOLIDATED UNDRAINED TRIAXIAL EXTENSION TEST
CELL NO. 11 DIRECT SHEAR BOX NO. ______ MACHINE NO. 2
COMMENTS

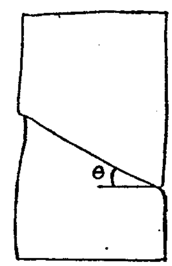
SPECIFIC GRAVITY OF SOLIDS 2.75 ☐ FROM TEST, ☒ ASSUMED

SAMPLE DIMENSIONS

DIAMETER INCHES		X	Y		LENGTH INCH	WIDTH INCH
TOP	=	1.405		OR		
MIDDLE	=	1.406				
BOTTOM	=	1.407				
AVERAGE	=	1.406				

HEIGHT = 2.902 INCH = 7.37 CM
X-AREA = 1.55 SQ IN = 10.00 SQ CM
VOLUME = 4.50 CU IN = 73.90 CU CM

SKETCH OF FAILURE



ANGLE OF SHEAR PLANE = 42°

SAMPLE MOIST WEIGHTS

		BEFORE PLACING CELL	AFTER REMOVING FROM CELL
TARE NO.		S	D
WT. SAMPLE + TARE, GM	=	164.11	218.17
WT. TARE, GM	=	38.42	96.08
WT OF SAMPLE, GM	=	125.69	121.09
CHANGE OF WEIGHT DURING TEST, GM	=	4.60	

MOISTURE CONTENTS

		INITIAL		AFTER COMPLETION OF TEST			
		TRIMMING	TRIMMING	TOP	FAILURE PLANE	BOTTOM	ENTIRE SAMPLE
TARE NO.		AV		U 29	AR	U 64	D
WT. SOIL+WATER+TARE, GM	=	80.03		46.37	67.57	49.57	145.77
WT. SOIL + TARE, GM	=	63.26		41.11	55.21	43.76	123.39
WT. TARE, GM	=	32.04		27.73	31.98	32.12	96.08
WT. WATER, GM	=	16.77		5.56	12.36	5.81	16.38
WT. SOIL, GM	=	31.22		13.38	23.23	11.64	33.31
PERCENT MOISTURE	=	53.60		43.75	53.10	50.00	49.20

WEIGHT - VOLUMETRIC RELATIONSHIP

INITIAL CONDITIONS			AFTER ISOTROPIC CONSOLIDATION		
VOLUME OF SAMPLE,	=	73.90	VOLUME CHANGE FROM BURETTE, CU CM	=	4.40
WEIGHT OF SOLIDS, GM	=	81.70	WEIGHT OF WATER, GM	=	39.59
VOLUME OF SOLIDS, CU CM	=	29.70	CHANGE IN HEIGHT, CM	=	0.53
VOLUME OF WATER, CU CM	=	43.99	HEIGHT AFTER CONSOLIDATION, CM	=	6.84
VOLUME OF VOIDS, CU CM	=	44.20	VOLUME AFTER CONSOLIDATION, CU CM	=	69.50
DEGREE OF SATURATION	=	99.40	X-AREA, SQ CM = , SQ IN	=	1.58
			VOLUME OF VOIDS, CU CM	=	39.80
VOID RATIO	=	1.49	VOID RATIO	=	1.34
MOISTURE CONTENT, %	=	53.60	MOISTURE CONTENT, %	=	48.50
UNIT WEIGHT DRY, PCF	=	69.10	UNIT WEIGHT DRY, PCF	=	73.50
UNIT WEIGHT MOIST, PCF	=	106.00	UNIT WEIGHT MOIST, PCF	=	109.00

Tested by N.A Date 15 FEB 73
Calculated by N.A Date 15 FEB 73
Checked by N.A Date 15 FEB 73

SOIL MECHANICS LABORATORY
Department of Civil Engineering
University of Manitoba
Fort Garry Manitoba

TRIAXIAL TEST WITH PORE WATER PRESSURE MEASUREMENT

JOB NO = MSc THESIS DATE 15 FEB 73
SAMPLE NO = 2 PAGE 2 OF 2

TIME	ELAPSED TIME	LOAD DIAL DIVISIONS or PAN LOAD lb	CELL PRESSURE psi	PORE WATER PRESSURE GAUGE psi		u psi	STRAIN DIAL inch	ΔL inch	$\frac{\Delta L \times 100\%}{L_1}$	$1 - \frac{\Delta L}{L_1}$	AREA sq in		AXIAL LOAD lb	$\bar{\sigma}_3$ psi	$\bar{\sigma}_1 - \bar{\sigma}_3$ psi	$\bar{\sigma}_1$ psi	$\frac{\bar{\sigma}_1}{\bar{\sigma}_3}$
13.30	0	132.25	70	10.25		0	0.8000	0	0	1.0000	1.580		0	59.75	0	59.75	1.00
13.45		125.25	75	11.05		0.80	0.7977	0.0023	0.085	0.9991	1.580		-7.9	63.95	-5	58.95	0.92
14.10		117.45	80	12.30		2.05	0.7930	0.0070	0.2590	0.9974	1.575		-15.8	67.70	-10	57.70	0.85
15.00		109.65	85	17.20		6.95	0.7900	0.0100	0.3700	0.9963	1.576		-23.6	67.80	-15	52.80	0.78
15.45		101.75	90	19.20		8.95	0.7860	0.0140	0.5180	0.9948	1.570		-31.5	70.80	-20	50.80	0.72
16.45		93.95	95	23.05		12.95	0.7790	0.0210	0.7770	0.9922	1.567		-39.3	71.95	-25	46.95	0.65
17.35		86.25	100	26.75		16.50	0.7680	0.0320	1.1850	0.9881	1.560		-47.0	73.25	-30	43.25	0.59
18.20		78.65	105	29.90		19.65	0.7560	0.0440	1.6300	0.9837	1.550		-54.6	75.10	-35	40.10	0.53
19.05		71.15	110	32.90		22.65	0.7385	0.0615	2.2800	0.9772	1.542		-62.1	77.10	-40	37.10	0.48
19.50		63.35	115	36.80		26.55	0.7155	0.0845	3.1500	0.9685	1.530		-69.4	78.20	-45	33.20	0.42
20.25		56.75	120	40.25		30.00	0.6680	0.1320	4.9000	0.9510	1.500		-76.5	79.75	-50	29.75	0.37
21.25		50.65	125	47.25		37.00	0.6300	0.1700	6.3000	0.9370	1.480		-82.6	77.75	-55	22.75	0.29
22.10	8⅔ h	43.95	127	53.50		43.25	FAILED						-84.3	73.50	-57	16.50	0.22

TEST ☐ $\overline{CU}$ ☐ $\overline{UU}$

APPROPRIATE DATA FROM PAGE 1

After isotropic consolidation:
L_1 = 2.70 inch. A_1 = 1.58 sq in.

Without isotropic consolidation:
$L_0 = L_1$ = ______ inch. $A_0 = A_1$ = ______ sq in.

Back pressure, u_b = 10.25 psi
Wt. ball + cap = ______ lb
P.P index correction = ______ psi
Load = (Dial Div.) x (______) lb, or:
= (Pan Load) x (______).

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DEPARTMENT OF CIVIL ENGINEERING
UNIVERSITY OF MANITOBA
FORT GARRY MANITOBA

ANCILLARY DATA FOR TRIAXIAL OR DIRECT SHEAR TEST

PAGE 1 OF 2

PROJECT M.Sc. THESIS
JOB NO. ______ TEST HOLE NO. 1 SAMPLE NO. ____ DEPTH 6 - 8 FT.
DESCRIPTION OF SAMPLE UNDISTURBED, 15 P.S.I CONFINING PRESSURE

METHOD OF PREPARATION TRIMMING
TYPE OF TEST CONSOLIDATED DRAINED TEST
CELL NO. 9 DIRECT SHEAR BOX NO. ______ MACHINE NO. 1
COMMENTS

SPECIFIC GRAVITY OF SOLIDS 2.75 ☐ FROM TEST, ☒ ASSUMED

SAMPLE DIMENSIONS

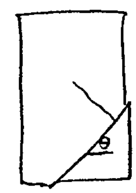
DIAMETER INCHES		X	Y		LENGTH INCH	WIDTH INCH
TOP	=	1.400		OR		
MIDDLE	=	1.406				
BOTTOM	=	1.420				
AVERAGE	=	1.410				

HEIGHT = 3.015 INCH = 7.64 CM
X-AREA = 1.552 SQ IN = 10.00 SQ CM
VOLUME = 4.710 CU IN = 77.20 CU CM

SAMPLE MOIST WEIGHTS

		BEFORE PLACING CELL	AFTER REMOVING FROM CELL
TARE NO.		D	D
WT. SAMPLE + TARE, GM	=	228.16	226.00
WT. TARE, GM	=	95.97	95.97
WT OF SAMPLE, GM	=	132.19	130.03
CHANGE OF WEIGHT DURING TEST, GM	=	2.16	

SKETCH OF FAILURE



ANGLE OF SHEAR PLANE = 52°

MOISTURE CONTENTS

		INITIAL		AFTER COMPLETION OF TEST			
		TRIMMING	TRIMMING	TOP	FAILURE PLANE	BOTTOM	ENTIRE SAMPLE
TARE NO.		U53		U88	AB	U64	D
WT. SOIL+WATER+TARE, GM	=	53.34		47.38	57.43	44.62	172.65
WT. SOIL + TARE, GM	=	44.55		42.52	49.15	40.75	148.13
WT. TARE, GM	=	27.95		32.77	32.50	32.06	95.97
WT. WATER, GM	=	8.79		4.86	8.28	3.87	24.52
WT. SOIL, GM	=	16.60		9.75	16.65	8.69	52.15
PERCENT MOISTURE	=	53.00		49.80	49.75	44.50	47.00

WEIGHT - VOLUMETRIC RELATIONSHIP

INITIAL CONDITIONS		AFTER ISOTROPIC CONSOLIDATION	
VOLUME OF SAMPLE,	= 77.20	VOLUME CHANGE FROM BURETTE, CU CM	= 1.20
WEIGHT OF SOLIDS, GM	= 66.50	WEIGHT OF WATER, GM	= 44.50
VOLUME OF SOLIDS, CU CM	= 31.42	CHANGE IN HEIGHT, CM	= 0.11
VOLUME OF WATER, CU CM	= 47.75	HEIGHT AFTER CONSOLIDATION, CM	= 7.53
VOLUME OF VOIDS, CU CM	= 45.78	VOLUME AFTER CONSOLIDATION, CU CM	= 76.00
DEGREE OF SATURATION	= 100.00	X-AREA, SQ CM = 10.10, SQ IN	= 1.565
		VOLUME OF VOIDS, CU CM	= 44.58
VOID RATIO	= 1.46	VOID RATIO	= 1.42
MOISTURE CONTENT, %	= 53.00	MOISTURE CONTENT, %	= 51.15
UNIT WEIGHT DRY, PCF	= 70.00	UNIT WEIGHT DRY, PCF	= 71.00
UNIT WEIGHT MOIST, PCF	= 107.10	UNIT WEIGHT MOIST, PCF	= 107.20

Tested by N.A Date 27 Sept 73
Calculated by N.A Date 14 Oct. 73
Checked by N.A Date 14 Oct. 73

SOIL MECHANICS LABORATORY
Department of Civil Engineering
University of Manitoba
Fort Garry Manitoba

TRIAXIAL TEST WITH DRAINAGE PERMITTED

JOB NO. = M.Sc. THESIS PAGE 2 of 2
SAMPLE NO. = ______

DATE	TIME	ELAPSED TIME	CELL PRESSURE psi	BURETTE READING ml	ΔV_w ml	$\frac{\Delta V_w \times 100\%}{V_o}$	STRAIN DIAL inch	ΔL inch	$\frac{\Delta L \times 100\%}{L_o}$	AREA FACTOR F_a	AREA sq in	LOAD DIAL DIVISIONS or PAN LOAD lb		AXIAL LOAD lb	$\bar{\sigma}_1 - \bar{\sigma}_3$ psi	$\bar{\sigma}_1$ psi	$\frac{\bar{\sigma}_1}{\bar{\sigma}_3}$
27 Sept 73		0	15	15.30	0	0	0.9550	0	0	1.000	1.565	31.50		0	0	15.00	1.0
29 "		44^h	15	15.55	-0.25	-0.329	0.9138	0.0422	1.420	1.020	1.595	38.50		7	4.40	19.40	1.3
1 Oct 73		47^h	15	15.60	-0.30	-0.395	0.8937	0.0613	2.065	1.025	1.602	43.50		12	7.48	22.50	1.5
2 "		24^h	15	15.35	-0.05	-0.066	0.8787	0.0763	2.570	1.030	1.611	47.50		16	9.93	25.00	1.7
3 "		27^h	15	15.25	+0.05	+0.066	0.8641	0.0909	3.060	1.031	1.611	50.50		19	11.80	26.80	1.8
4 "		21^h	15	15.20	+0.10	+0.132	0.8523	0.1017	3.421	1.034	1.617	53.50		22	13.62	28.60	1.9
5 "		24^h	15	15.15	+0.15	+0.197	0.8416	0.1134	3.820	1.040	1.626	56.50		25	15.39	30.40	2.0
8 "		72^h	15	15.30	0.00	0.000	0.8269	0.1281	4.315	1.045	1.633	59.50		28	17.14	32.10	2.1
9 "		24^h	15	14.95	+0.35	+0.461	0.8118	0.1432	4.820	1.048	1.640	62.50		31	19.20	34.00	2.3
10 "		26^h	15	14.80	+0.50	+0.658	0.7977	0.1573	5.290	1.050	1.642	65.50		34	20.70	35.70	2.4
11 "		$21\frac{1}{2}^h$	15	14.70	+0.60	+0.790	0.7830	0.1720	5.790	1.053	1.648	68.50		37	22.41	37.40	2.5
12 "		24^h	15	14.60	+0.70	+0.921	0.7622	0.1928	6.490	1.060	1.660	71.50		40	24.10	39.10	2.6
13 "		28^h	15	14.65	+0.65	+0.855	0.7467	0.2082	7.010	1.065	1.665	73.50		42	25.21	40.20	2.7
13 "		2^h	15	14.65	+0.65	+0.855	0.7341	0.2209	7.440	1.084	1.650	74.50		43	25.30	40.30	2.7

FROM PAGE 1
L_o = 2.97 inch
A_o = 1.565 sq in
V_o = 76.00 cu cm

Back pressure u_b = 0 psi.
$F_a = (1 - \Delta V/V_o) / (1 - \Delta L/L_o)$.
Load, lb = (Dial Divisions) x (______) or (Pan Load) x (______)

Wt. ball + cap = 0.0724 lb
Area = $F_a \times A_o$.

SOIL MECHANICS LABORATORY
DEPARTMENT OF CIVIL ENGINEERING
UNIVERSITY OF MANITOBA
FORT GARRY MANITOBA

ANCILLARY DATA FOR TRIAXIAL OR DIRECT SHEAR TEST

PAGE 1 OF 2

PROJECT MSc. THESIS
JOB NO. ______ TEST HOLE NO. 1 SAMPLE NO. 1 DEPTH 11 - 13 FT
DESCRIPTION OF SAMPLE UNDISTURBED GREY BROWN VARVED CLAY

METHOD OF PREPARATION TRIMMING
TYPE OF TEST CONSTANT MEAN NORMAL STRESS $\frac{1}{3}(\bar{\sigma}_1+\bar{\sigma}_2+\bar{\sigma}_3)$ = 100 PSI
CELL NO. 9 DIRECT SHEAR BOX NO. ______ MACHINE NO. 3 1
COMMENTS

SPECIFIC GRAVITY OF SOLIDS 2.75 ☐ FROM TEST, ☒ ASSUMED

SAMPLE DIMENSIONS

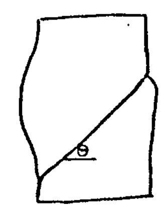
DIAMETER INCHES		X	Y		LENGTH INCH	WIDTH INCH
TOP	=	1.400		OR		
MIDDLE	=	1.400				
BOTTOM	=	1.404				
AVERAGE	=	1.401				

HEIGHT = 2.975 INCH = 7.55 CM
X-AREA = 1.540 SQ IN = 9.91 SQ CM
VOLUME = 4.580 CU IN = 75.30 CU CM

SAMPLE MOIST WEIGHTS

		BEFORE PLACING CELL	AFTER REMOVING FROM CELL
TARE NO.		D	D
WT. SAMPLE + TARE, GM	=	222.87	211.93
WT. TARE, GM	=	95.95	95.95
WT OF SAMPLE, GM	=	126.92	125.98
CHANGE OF WEIGHT DURING TEST, GM	=		

SKETCH OF FAILURE



ANGLE OF SHEAR PLANE = 55°

MOISTURE CONTENTS

		INITIAL		AFTER COMPLETION OF TEST			
		TRIMMING	TRIMMING	TOP	FAILURE PLANE	BOTTOM	ENTIRE SAMPLE
TARE NO.		AT		AZ	1108	AR	D
WT. SOIL+WATER+TARE, GM	=	86.43		46.31	79.60	42.44	140.31
WT. SOIL + TARE, GM	=	67.19		42.25	66.24	39.14	127.98
WT. TARE, GM	=	32.22		32.08	32.83	31.98	95.95
WT. WATER, GM	=	19.24		4.06	13.36	3.30	12.33
WT. SOIL, GM	=	34.97		10.17	33.41	7.16	32.03
PERCENT MOISTURE	=	55.00		40.00	40.00	46.00	38.50

WEIGHT - VOLUMETRIC RELATIONSHIP

INITIAL CONDITIONS		AFTER ISOTROPIC CONSOLIDATION	
VOLUME OF SAMPLE,	= 75.30	VOLUME CHANGE FROM BURETTE, CU CM	= 7.40
WEIGHT OF SOLIDS, GM	= 81.50	WEIGHT OF WATER, GM	= 37.62
VOLUME OF SOLIDS, CU CM	= 29.60	CHANGE IN HEIGHT, CM	= 0.45
VOLUME OF WATER, CU CM	= 45.12	HEIGHT AFTER CONSOLIDATION, CM	= 7.10
VOLUME OF VOIDS, CU CM	= 45.60	VOLUME AFTER CONSOLIDATION, CU CM	= 67.90
DEGREE OF SATURATION	= 99.00	X-AREA, SQ CM = 9.55, SQ IN	= 1.48
		VOLUME OF VOIDS, CU CM	= 38.10
VOID RATIO	= 1.53	VOID RATIO	= 1.28
MOISTURE CONTENT, %	= 55.00	MOISTURE CONTENT, %	= 46.00
UNIT WEIGHT DRY, PCF	= 67.70	UNIT WEIGHT DRY, PCF	= 75.20
UNIT WEIGHT MOIST, PCF	= 105.00	UNIT WEIGHT MOIST, PCF	= 109.80

Tested by N.A Date 16 Jan.74
Calculated by N.A Date 26 Jan.74
Checked by N.A Date 26 Jan.74

SOIL MECHANICS LABORATORY
Department of Civil Engineering
University of Manitoba
Fort Garry Manitoba

TRIAXIAL TEST WITH DRAINAGE PERMITTED

JOB NO. = MSc.THESIS
SAMPLE NO. = 1

DATE	TIME	ELAPSED TIME	CELL PRESSURE psi	BURETTE READING ml	ΔV_w ml	$\frac{\Delta V_w \times 100\%}{V_o}$	STRAIN DIAL inch	ΔL inch	$\frac{\Delta L \times 100\%}{L_o}$	AREA FACTOR F_a	AREA sq in	LOAD DIAL DIVISIONS or PAN LOAD lb		AXIAL LOAD lb	$\bar{\sigma}_1 - \bar{\sigma}_3$ psi	$\bar{\sigma}_1$ psi	$\frac{\bar{\sigma}_1}{\bar{\sigma}_3}$
16 Jan 73		0	100	70.4	0	0	0.5215	0	0	1.000	1.480	49.00		0	0	100	1.0
16 "		24^h	98	70.4	0	0	0.4990	0.0225	0.760	1.002	1.480	57.90		8.9	6	104	1.1
17 "		24^h	96	70.4	0	0	0.4780	0.0435	1.560	1.018	1.497	67.00		18.0	12	108	1.1
18 "		24^h	94	70.4	0	0	0.4730	0.0485	1.730	1.016	1.540	76.70		27.7	18	112	1.2
19 "		24^h	92	70.4	0	0	0.4645	0.0570	2.040	1.020	1.560	86.00		37.1	24	116	1.3
20 "		24^h	85	70.1	0.30	0.442	0.4094	0.1121	4.020	1.036	1.535	119.00		70.1	45	130	1.5
21 "		24^h	80	68.9	1.50	2.210	0.2530	0.2685	9.600	1.070	1.585	141.20		92.2	60	140	1.8
23 "		48^h	79	68.6	1.80	2.650	0.1860	0.3355	12.000	1.105	1.637	149.00		100.0	63	142	1.8
25 "		10^h	78				FAILED					157.00		108.0	66	144	

FROM PAGE 1
L_o = 2.736 inch
A_o = 1.482 sq in
V_o = 67.900 cu cm

Back pressure u_b = 0 psi.
Wt. ball + cap = 0.072 lb
$F_a = (1 - \Delta V/V_o) / (1 - \Delta L/L_o)$.
Area = $F_a \times A_o$.
Load, lb = (Dial Divisions) x (______) or (Pan Load) x (______)

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APPENDIX E

RESULTS AND TYPICAL DATA OF DIRECT SHEAR TEST

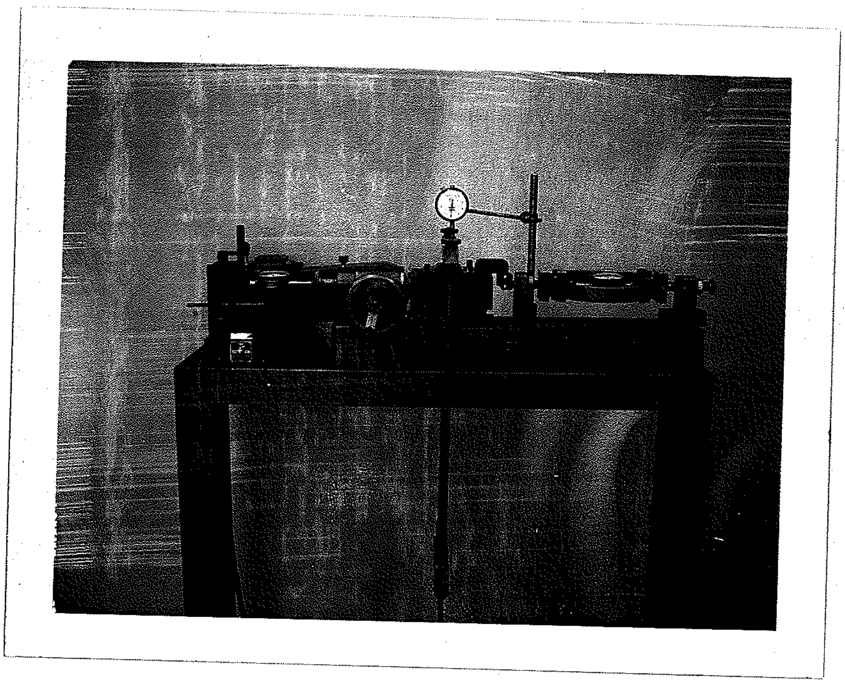

FIGURE E-1 - DIRECT SHEAR MACHINE

ANCILLARY DATA FOR TRIAXIAL OR DIRECT SHEAR TEST	PAGE 1 OF ______

PROJECT M.Sc. THESIS
JOB NO. ______ TEST HOLE NO. 1 SAMPLE NO. ______ DEPTH 26 - 28 FT.
DESCRIPTION OF SAMPLE UNDISTURBED GREY CLAY

METHOD OF PREPARATION TRIMMING
TYPE OF TEST DIRECT SHEAR TEST
CELL NO. ______ DIRECT SHEAR BOX NO. 01 MACHINE NO. 1
COMMENTS A LOT OF SMALL GRAVELS, DIAMETER 3-5 MM.

SPECIFIC GRAVITY OF SOLIDS 2.75 ☐ FROM TEST, ☒ ASSUMED

SAMPLE DIMENSIONS

DIAMETER INCHES	X	Y		LENGTH INCH	WIDTH INCH
TOP =					
MIDDLE =			OR		
BOTTOM =					
AVERAGE =				2.35	2.35

HEIGHT = 0.990 INCH = ______ CM
X-AREA = 5.520 SQ IN = ______ SQ CM
VOLUME = ______ CU IN = ______ CU CM

SKETCH OF FAILURE

ANGLE OF SHEAR PLANE = ______

SAMPLE MOIST WEIGHTS

	BEFORE PLACING CELL	AFTER REMOVING FROM CELL
TARE NO.	–	
WT. SAMPLE + TARE, GM =	329.61	260.53
WT. TARE, GM =	158.30	108.72
WT OF SAMPLE, GM =	171.31	151.81
CHANGE OF WEIGHT DURING TEST, GM =	19.50	

MOISTURE CONTENTS

	INITIAL		AFTER COMPLETION OF TEST			
	TRIMMING	TRIMMING	TOP	FAILURE PLANE	BOTTOM	ENTIRE SAMPLE
TARE NO.	AD		U73	U26	U103	
WT. SOIL+WATER+TARE, GM =	107.44		79.06	65.00	72.77	
WT. SOIL + TARE, GM =	87.62		71.34	60.32	64.52	
WT. TARE, GM =	31.59		32.78	28.03	32.60	
WT. WATER, GM =	19.82		7.72	7.68	8.25	
WT. SOIL, GM =	56.03		38.56	32.29	31.92	
PERCENT MOISTURE =	35.40		20.00	23.80	25.85	

WEIGHT - VOLUMETRIC RELATIONSHIP

INITIAL CONDITIONS		AFTER ISOTROPIC CONSOLIDATION	
VOLUME OF SAMPLE	=	VOLUME CHANGE FROM BURETTE, CU CM	=
WEIGHT OF SOLIDS, GM	=	WEIGHT OF WATER, GM	=
VOLUME OF SOLIDS, CU CM	=	CHANGE IN HEIGHT, CM	=
VOLUME OF WATER, CU CM	=	HEIGHT AFTER CONSOLIDATION, CM	=
VOLUME OF VOIDS, CU CM	=	VOLUME AFTER CONSOLIDATION, CU CM	=
DEGREE OF SATURATION	=	X-AREA, SQ CM = , SQ IN	=
		VOLUME OF VOIDS, CU CM	=
VOID RATIO	=	VOID RATIO	=
MOISTURE CONTENT, %	=	MOISTURE CONTENT, %	=
UNIT WEIGHT DRY, PCF	=	UNIT WEIGHT DRY, PCF	=
UNIT WEIGHT MOIST, PCF	=	UNIT WEIGHT MOIST, PCF	=

Tested by N.A Date 31 Oct. 73	SOIL MECHANICS LABORATORY
Calculated by N.A Date 1 Feb 73	Department of Civil Engineering
Checked by N.A Date 1 Feb 73	University of Manitoba Fort Garry Manitoba

SM 1

SOIL MECHANICS LABORATORY DEPARTMENT OF CIVIL ENGINEERING UNIVERSITY OF MANITOBA FORT GARRY MANITOBA	DIRECT SHEAR TEST Project..... MSc. THESIS Test for CANADA CEMENT LAFARGE OIL Address WINNIPEG CANADA TANK

Sample no. ________ Test Hole no. 1 DEpth 26-28 FT.

Sample Description UNDISTURBED GREY CLAY

Comments ________

Load on Shear pan (lbs.)	Strain Dial (ins.)	Total Shearing Strain(ins)	Xsectional Area of sample (sq. ins.)	Unit Vertical Stress (p.s.i.)	Net Shearing Load (lbs.)	Unit Shearing Stress (p.s.i.)
390	0.5000	0.000	5.52	69	0	0
	0.4800	0.020			25.8	4.67
	0.4600	0.040			60.2	10.90
	0.4400	0.060			90.3	16.35
	0.4200	0.080			112.0	20.30
	0.4000	0.100			131.2	23.80
	0.3800	0.120			136.0	24.65
	0.3600	0.140			139.2	25.20
	0.3400	0.160			140.0	25.40
	0.3200	0.180			142.0	25.70
	0.3000	0.200			143.5	26.00
	0.2800	0.220			144.5	26.20
	0.2600	0.240			144.5	26.20
	0.2100	0.260			145.2	26.30
	0.2200	0.280			146.0	26.45
	0.5000	0.280			0	0
	0.4800	0.300			21.5	3.90
	0.4600	0.320			47.3	8.57
	0.4400	0.340			116.0	21.00
	0.4200	0.360			124.8	22.60
	0.2000	0.380			127.2	23.00
	0.3800	0.400			130.0	23.60
	0.3600	0.420			131.0	23.70
	0.3400	0.440			132.5	24.00
	0.3200	0.460			134.0	24.30
	0.3000	0.480			135.0	24.45
	0.2800	0.500			136.0	24.65
	0.2635	0.517			137.0	24.80
	0.2400	0.540			137.0	24.80
	0.2200	0.560			137.7	25.00
	0.5000	0.560			0	0
	0.4700	0.591			4.3	0.77
	0.4460	0.612			76.5	13.85
	0.4150	0.645			117.0	21.20
	0.3800	0.680			122.0	22.10
	0.3560	0.704			124.0	22.45

Tested by N.A Date 31 Oct 73 Calculated by N.A Date 1 Feb 73

SM 2

SOIL MECHANICS LABORATORY
DEPARTMENT OF CIVIL ENGINEERING
UNIVERSITY OF MANITOBA
FORT GARRY MANITOBA

DIRECT SHEAR TEST

Project..... MSc. THESIS
Test for CANADA CEMENT LAFARGE OIL
Address WINNIPEG CANADA TANK

Sample no. ________ Test Hole no. 1 DEpth 26 - 28 FT.

Sample Description UNDISTURBED GREY CLAY

Comments

Load on Shear pan (lbs.)	Strain Dial (ins.)	Total Shearing Strain(ins)	Xsectional Area of sample (sq. ins.)	Unit Vertical Stress (p.s.i.)	Net Shearing Load (lbs.)	Unit Shearing Stress (p.s.i.)
390	0.3300	0.730	5.52	69	124.5	22.55
	0.2925	0.768			127.0	23.00
	0.2600	0.800			128.0	23.20
	0.2250	0.835			131.0	23.70
	0.5000	0.835			0	0
	0.4750	0.860			2.5	0.45
	0.4460	0.889			61.0	11.05
	0.4200	0.915			102.0	18.50
	0.4000	0.935			112.0	20.30
	0.3750	0.960			117.0	21.20
	0.3340	1.001			118.0	21.40
	0.2860	1.049			121.0	21.90
	0.2650	1.070			123.0	22.30
	0.2400	1.095			125.0	22.65
	0.5000	1.095			0	0
	0.4740	1.121			79.0	14.30
	0.4460	1.149			104.0	18.85
	0.4170	1.178			110.0	19.90
	0.3720	1.223			113.5	20.60
	0.3320	1.263			115.0	20.80
	0.3000	1.295			119.5	21.65
	0.2790	1.316			122.0	22.10
	0.2420	1.353			125.0	22.65
	0.2170	1.378			125.5	22.75
	0.5000	1.378			0	0
	0.4650	1.413			31.0	5.76
	0.4350	1.443			96.2	17.43
	0.4060	1.472			117.0	21.20
	0.3830	1.495			118.7	21.50
	0.3630	1.515			120.0	21.75
	0.3360	1.542			122.0	22.10
	0.3160	1.562			125.0	22.65
	0.5000	1.562			0	0
	0.4730	1.589			6.9	1.25
	0.4400	1.614			92.0	16.70
	0.4212	1.641			111.0	20.25

Tested by H. A Date 31 Oct 73 Calculated by H. A Date 1 Feb 73

SM 3

SOIL MECHANICS LABORATORY DEPARTMENT OF CIVIL ENGINEERING UNIVERSITY OF MANITOBA FORT GARRY MANITOBA	DIRECT SHEAR TEST Project MSc. THESIS Test for CANADA CEMENT LAFARGE OIL TANK Address WINNIPEG CANADA

Sample no. ________ Test Hole no. 1 DEpth 26 - 28 FT.

Sample Description UNDISTURBED GREY CLAY

Comments

Load on Shear pan (lbs.)	Strain Dial (ins.)	Total Shearing Strain (ins)	Xsectional Area of sample (sq. ins.)	Unit Vertical Stress (p.s.i.)	Net Shearing Load (lbs.)	Unit Shearing Stress (p.s.i.)
390	0.4020	1.660	5.52	69	116.0	21.00
	0.3510	1.660			118.0	21.40
	0.3140	1.718			119.5	21.65
	0.3160	1.746			120.2	21.80
	0.2880	1.774			122.0	22.10
	0.2660	1.794			124.0	22.45
	0.2333	1.823			127.0	23.00
	0.2280	1.834			128.0	23.20
	0.5000	1.834			0	0
	0.4800	1.854			5.1	0.93
	0.4354	1.893			69.5	12.60
	0.4150	1.919			112.7	20.40
	0.3680	1.966			117.0	21.20
	0.3280	2.006			119.5	21.65
	0.3092	2.024			121.0	21.90
	0.2620	2.073			124.8	22.60
	0.5000	2.072			0	0
	0.4770	2.095			1.7	0.31
	0.4330	2.133			14.6	2.64
	0.4150	2.157			107.5	19.50
	0.3900	2.182			111.0	20.11
	0.3700	2.202			112.0	20.30
	0.3400	2.232			114.2	20.70
	0.3160	2.254			115.0	20.80
	0.2830	2.284			118.0	21.40
	0.2630	2.319			120.0	21.75
	0.2280	2.327			122.0	22.10
	0.2170	2.355			125.0	22.65
	0.5000	2.355			0	0
	0.4000	2.375			5.1	0.92
	0.4632	2.392			37.3	6.05
	0.4300	2.425			106.0	19.20
	0.4000	2.455			124.0	63.50

Tested by N.A Date 31 Oct 73 Calculated by N.A Date 1 Feb 73

BIBLIOGRAPHY

1. Bishop, W. A., and Henkel, D. J.: "The Triaxial Test", London, Arnold, 1957.

2. Bjerrum, L., and Simons, N. E.: "Comparison of Shear Strength Characteristics of Normally Consolidated Clay", Research Conference on Shear Strength of Cohesive Soils, June 1960, p.p. 711-724.

3. Simons, N. E.: "The Effect of Over Consolidation on the Shear Strength Characteristics of an Undisturbed Oslo Clay", Research Conference on Shear Strength of Cohesive Soils, June 1960, p.p. 747-762.

4. Samarasingha, Nalin P.: "Triaxial Test for Typical Winnipeg Clay", M. Sc. Thesis, Department of Civil Engineering, University of Manitoba, 1966.

5. Whitman, R. V.: "Some Considerations and Data Regarding the Shear Strength of Clays", Research Conference on Shear Strength of Cohesive Soils, June 1960, p.p. 581-610.

6. Scott, R. F.: "Principles of Soil Mechanics", Addison-Wesley, 1963, p.p. 356-359.

7. Lowe, J.: "Use of Back Pressure to Increase Degree of Saturation of Triaxial Test Specimen", Research Conference on Shear Strength of Cohesive Soils, June 1960, p.p. 819-836.

8. Bishop, A. W., and Bjerrum, L.: "The Relevance of the Triaxial Test to the Solution of Stability Problem", Research Conference on Shear Strength of Cohesive Soils, June 1960, p.p. 437-493.

9. Kenny, C.: "Recent Research on Soft Sensitive Clay", Canadian Geotechnical Journal, Volume 5, No. 2, p.p. 97-117.

10. Kvorslev, M. J.: "Physical Component of the Shear Strength of Saturated Clays", Research Conference on Shear Strength of Cohesive Soils, June 1960, p.p. 169-261.

11. Terzaghi, K.: "Theoretical Soil Mechanics", New York, Wiley, 1943, Chapter 8.

12. Skempton, A. W., and Bishop, A. W.: "The Measurement of the Shear Strength of Soils", Geotechnique, Volume 2, p.p. 90-107.

13. Loh, A. K.,and Holt, R. T.: "Variation in Strength and Fabric of Winnipeg Clay", Unpublished records.

14. Bjerrum, L., and Kjaernsli, B.: "Analysis of the Stability of Some Norwegian Natural Clay Slopes", Geotechnique, Volume 7, No. 1, p.p. 1-15.

15. Henkel, D. J., and Skempton, A. W.: "A Landslide at Jackfield Shropshire in a Heavily Overconsolidated Clay", Geotechnique, Volume 5, No. 2, p.p. 139-149.

16. Skempton, A. W., and Brown, J. D.: "A Landslide on Boulder Clay at Selset, Yorkshire", Geotechnique, Volume 11, No. 4, p.p. 280-292.

17. Skempton, A. W.: "Long Term Stability of Clay Slopes", Geotechnique, Volume 14, No. 2, p.p. 77-100.

18. Bjerrum, L.: "Progressive Failure in Slopes of Over-Consolidated Plastic Clay and Clay Shales", ASCE, Journal of SMBFE, Volume 93, No. SM5, 1967, p.p. 1-46.

19. Thomson, S.: "Analysis of a Failed Slope", Canadian Geotechnical Journal, Volume 8, p.p. 596-599.

20. Thomson, S.: "Riverbank Stability Study at the University of Alberta, Edmonton", Canadian Geotechnical Journal, Volume 7, p.p. 157-168.

21. Lo, K. Y., and Stermac, A. G.: "Failure of an Embankment Founded on Varved Clay", Canadian Geotechnical Journal, Volume 2, p.p. 234-252.

22. Insley, A. E.: "A Study of a Large Compacted Clay Embankment-Fill Failure", Canadian Geotechnical Journal, Volume 2, p.p. 274-286.

23. Taylor, D. W.: "Fundamentals of Soil Mechanics", New York, Wiley and Sons, 1948, Chapter 16.