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SCREW PLATE TESTING OF SOFT CLAY

BY

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SUBMITTED TO THE SCHOOL OF GRADUATE
STUDIES, CARLETON UNIVERSITY AS PARTIAL
FULFILLMENT FOR THE REQUIREMENTS OF
THE DEGREE OF MASTER OF ENGINEERING

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Thursday, April 19, 1984

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1

ABSTRACT

A review of the literature has indicated that the undrained modulus of deformation and undrained shear strength of sensitive marine clay soils is greatly affected by sampling disturbance caused by the sampling technique employed to retrieve the sample. Block sampling techniques are superior to sampling techniques using either thin walled tube samples or stationary piston samples to obtain undisturbed samples. Block samples can now be obtained from the base of boreholes to depths of approximately 10 metres by using novel sampling techniques such as the Sherbrooke sampler. Nonetheless, when the results of laboratory tests, carried out on samples carved from block samples, are compared to in situ undrained shear strengths and undrained modulus of elasticity values derived from in situ plate load tests or the observed actual settlement of structures the laboratory test results are less than the in situ values.

Presently used in situ testing techniques such as the cone penetration test, the standard penetration test, and the pressuremeter test should only be used in areas where local knowledge of the soil being tested is available. Local empirical correlations are used to analyze the results of such tests. The test results derived from these tests are highly dependant on the sensitivity of the clay. Doubt has also been placed on results derived from the vane shear tests as on occasions the undrained shear strength as derived from the vane shear tests have been shown to be unconservative. These high results are usually observed within clays which exhibit a highly cemented structure. A need for better in situ testing techniques is necessary to better determine the in situ undrained shear strength and deformability characteristics of sensitive clay soils.

This investigation has centered on the performance of the screw plate test to determine the strength and deformability characteristics of soft clay soil in the light of classical theories of isotropic elasticity and rigid plastic behaviour. These relationships are used to derive the in situ undrained shear strength of a cohesive soil from the load-displacement relationship measured during the screw plate test.

A series of screw plate tests were carried out at the Gloucester test fill site and the results were compared to results obtained from laboratory tests carried out on good quality samples. The undrained modulus and undrained shear strength values derived from the screw plate tests at the 2.3 metre depth compare favourably with the results obtained from previous investigations at the Gloucester test fill site. At the 4.1 metre depth however, the results show no appreciable increase in the undrained modulus or undrained shear strength as reported in previous studies.

It is concluded that more testing is necessary at greater depths to determine if the screw plate test can accurately predict the in situ strength and deformability characteristic of sensitive clay soil.

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TABLE OF CONTENTS

	Page
INTRODUCTION	1
CHAPTER 1	4
1.0 The Estimation of Strength and Deformability Characteristics of Soft Clay: Laboratory vs. In situ Testing	4
1.1 Introduction	4
1.1.1 Sensitivity	4
1.2 Sampling Disturbance in Sensitive Clays	5
1.2.1 Effects on the Undrained Shear Strength	5
1.2.2 Effects on the Undrained Modulus of Elasticity	18
1.3 In situ Testing Techniques	28
1.3.1 Introduction	28
1.3.2 Standard Penetration Test (SPT)	28
1.3.3 Cone Penetration Test (CPT)	34
1.3.4 The Vane Shear Test (VST)	39
1.3.5 Pressuremeter Test (PMT)	47
1.3.6 Comparison of Testing Methods as Related to E_u and c_u	55
CHAPTER 2	67
2.0 Theoretical Assessment of the Screw Plate Test	67
2.1 Introduction	67
2.2 Theoretical Development	68
2.2.1 Uniform Flexible Load in an Infinite Elastic Medium	71
2.2.2 Rigid Circular Plate Embedded in full Bonded Contact with an Elastic Medium	73
2.2.3 Rigid Circular Plate Smoothly Embedded and in Partial Contact with the Elastic Medium	73

2.2.4	Rigid Circular Plate Embedded in Bonded Partial Contact with the Elastic Medium . . .	75
2.2.5	Flexible Circular Plate Embedded in Bonded Contact with the Elastic Medium . . .	75
2.2.6	Rigid Spheroidal Inclusion Embedded in Bonded Contact with an Elastic Medium . . .	81
2.2.7	Deep Borehole in an Elastic Medium Loaded by a Uniform Flexible Load	81
2.2.8	Deep Borehole in an Elastic Medium Loaded by a Rigid Circular Plate	85
2.3	Comparison of λ Values	85
2.4	Determination of the Undrained Shear Strength from Screw Plate Test Results	93
CHAPTER 3		97
3.0	Experimental Program	97
3.1	Conventional Screw Plate Test Apparatus	97
3.2	Carleton University Screw Plate Apparatus	97
3.2.1	Screw Plates and Loading Shafts	100
3.2.2	Test Frame	104
3.2.3	Ground Anchors	104
3.2.4	Load and Displacement Measurement	107
3.3	Test Procedure	109
3.4	Testing Program	110
3.4.1	Preliminary Tests (Arnprior)	111
CHAPTER 4		114
4.0	Experimental Results and Discussion	114
4.1	Introduction	114
4.2	Results of Analysis on the Second Load Cycle	118
4.3	Results of Analysis on the Third Load Cycle	125

	page
4.4 The Unload Undrained Modulus (E_R)	125
4.5 Undrained Shear Strength	135
4.6 Ratio E_u/c_u	135
4.7 Comparison of Results	141
CHAPTER 5	148
5.0 Conclusions and Recommendations	148
5.1 Summary and Conclusions	148
5.2 Recommendations for Future Work	150
REFERENCES	152
APPENDICES	161

LIST OF FIGURES

Figure		page
1.1	Effect of sampling and size of test specimen on the preconsolidation pressure measured in soft marine clay (after Bozozuk, 1971)	8
1.2	Sample disturbance caused by various sampling techniques (after Conlon and Isaacs, 1970)	9
1.3	Comparison of test results obtained from various sampling techniques (after Eden, 1970)	11
1.4	Effect of sampler type on stress-strain properties (after Raymond <i>et al.</i> , 1971)	13
1.5	Comparison of the undrained shear strength from penetrometer tests on block and Shelby tube samples (after Domaschuk, 1977)	16
1.6	Stress-strain diagram indicating the undrained modulus of elasticity	19
1.7a	Unconfined compression test results (after Milovic, 1970)	22
1.7b	Consolidated undrained test results (after Milovic, 1970)	22
1.7c	Reload Modulus vs. stress ratio for sample size and test type shown (after Soderman <i>et al.</i> , 1968)	24
1.7d	Reload Modulus vs. stress ratio for sample size and test type shown (after Soderman <i>et al.</i> , 1968)	25
1.8	Split spoon sampler used in Standard Penetration test (after Peck <i>et al.</i> , 1974)	29
1.9	Some correlations between N and q_c in clays (after Schmertmann, 1975)	31
1.10	Variation of N with sensitive clays (after Schmertmann, 1975)	32
1.11	Dutch cone penetration test.	35
1.12	Cone penetration test with friction sleeve	36
1.13	Worm gear assembly for Vane Shear Apparatus	42
1.14	Bjerrum's 1973 Vane strength correction factors	44

Figure		Page
1.15	Undrained shear strength decrease vs. the diameter of the swept area	46
1.16	Typical Pressuremeter Apparatus	48
1.17	Variation of the pressuremeter calibration curves with changes in fluid temperature and pressure increment	49
1.18	Typical pressuremeter curve	50
2.1	Uniform flexible circular load	71
2.2	Fully bonded rigid circular plate	74
2.3	Smoothly embedded partially separated rigid circular plate	76
2.4	Bonded partially separated rigid circular plate	77
2.5	Bonded thin flexible plate	78
2.6	The effects of relative rigidity on the central deflection of an embedded circular foundation	80
2.7	Rigid spheroidal inclusion in bonded contact with an isotropic elastic medium	82
2.8	Variation of λ_s with η	83
2.9	Deep borehole; subjected to a uniform flexible circular load	84
2.10a	Central deflection of a uniform loading at the base of a borehole	86
2.10b	Average deflection of a uniform loading at the base of a borehole	87
2.11	Deep borehole; rigid circular plate	88
2.12	Deflection of a rigid plate in smooth contact at the base of a borehole	89
2.13	Variation of λ values with Poisson's Ratio	90
2.14	Determination of c_u from screw plate test results	96
3.1	Schematic of conventional screw plate testing apparatus	98

Figure		page
3.2	Schematic of Carleton University screw plate testing apparatus	99
3.3	Connection detail of the loading shafts	102
3.4	Connection detail of the screw plate to the loading shafts	103
3.5	Plan view of the base and support triangle layout	106
3.6	Gloucester test fill site	112
3.7	Soil profile and screw plate test depths	113
4.1	Plan view of the test fill and the screw plate test locations	115
4.2	Load displacement curve derived from the screw plate test	116
4.3	Determination of the undrained modulus from typical load cycle curves	117
4.4a	Histograms of the E_u values obtained at the 2.3 m depth at the Gloucester test fill site; 2nd load cycle results	121
4.4b	Histograms of the E_u values obtained at the 4.1 m depth at the Gloucester test fill site; 2nd load cycle results	122
4.5	The effect of the coefficient of kurtosis on the shape of frequency distribution	123
4.6a	Histograms of the E_u values obtained at the 2.3 m depth at the Gloucester test fill site; 3rd load cycle results	128
4.6b	Histograms of the E_u values obtained at the 4.1 m depth at the Gloucester test fill site; 3rd load cycle results	129
4.7a	Histograms of the E_R values obtained at 2.3 m depth at the Gloucester test fill site	133
4.7b	Histograms of the E_R values obtained at the 4.1 m depth at the Gloucester test fill site	134
4.8a	Histograms of the c_u values obtained at the 2.3 m depth at the Gloucester test fill site	139

Figure

page

4.8b	Histograms of the c_u values obtained at the 4.1 m depth at the Gloucester test fill site	140
4.9	E_u/c_u minimum and maximum values 2.3 m depth	143
4.10	E_u/c_u minimum and maximum values 4.1 m depth	144

LIST OF PLATES

<i>Plate</i>		<i>page</i>
2.1	Disturbance caused by penetration action of model screw plate device	70
3.1	Screw plates used in field testing program	101
3.2	Screw plate tripod test frame	105
3.3	Screw plate test detail of the plate displacement measurement setup	108

LIST OF TABLES

Table		Page
1.1	Contribution of side friction and end bearing to the N value obtained from the standard penetration test	33
1.2	Some variables that influence N used in the estimation of c_u from cone penetration tests	38
1.3	Compressibility as indicated by the static cone resistance	40
1.4	Undrained shear strength as indicated by various testing methods	56-60
1.5	Undrained modulus as indicated by various testing methods	61-62
1.6	Undrained modulus results (after Milovic, 1970)	64
1.7	Comparison of the ratio of undrained modulus, E_u , to the undrained shear strength, c_u , of the St. Alban clay	65
2.1	λ values used for the estimation of the undrained modulus from screw plate tests	92
2.2	Estimation of the undrained shear strength from screw plate tests	94
4.1a	Determination of the undrained modulus from screw plate tests at the 2.3 m depth; Gloucester test fill site, 2nd load cycle	119
4.1b	Determination of the undrained modulus from screw plate tests at the 4.1 m depth; Gloucester test fill site, 2nd load cycle	120
4.2	Summary of statistical data, 2nd load cycle	124
4.3a	Determination of the undrained modulus from screw plate tests at the 2.3 m depth; Gloucester test fill site, 3rd load cycle	126
4.3b	Determination of the undrained modulus from screw plate tests at the 4.1 m depth; Gloucester test fill site, 3rd load cycle	127
4.4	Summary of statistical data, 3rd load cycle	130

Chapter		Page
4.5	Table of the unload moduli determined at the 2.3 m and 4.1 m depths; Gloucester test fill site	131
4.6	Summary of statistical data, unload moduli results	132
4.7a	Determination of the undrained shear strength from screw plate tests at the 2.3 m depth; Gloucester test fill site	136
4.7b	Determination of the undrained shear strength from screw plate tests at the 4.1 m depth; Gloucester test fill site	137
4.8	Summary of the statistical data of the undrained shear strength results	138
4.9	Determination of the minimum and maximum E_u/c_u values	142
4.10	Comparison of results obtained at the Gloucester test fill site from screw plate test and other testing techniques	147

LIST OF SYMBOLS

a	half the length of the major axis of oblate spheroid, or the radius of a circular loaded area, or radius of screw plate
A_r	area ratio equal to blade cross sectional area divided by area swept in vane shear apparatus
b	half the length of the minor axis of oblate spheroid
C_c	compression index
cc	abbreviation cubic centimetres
CIU	consolidated isotropically undrained triaxial test
cm	abbreviation centimetres
CPT	abbreviation Cone Penetration Test
c_u	undrained shear strength
$\bar{c}_u$	normalized undrained shear strength
e	void ratio
E_{oed}	undrained modulus of deformation from Oedometric compression test
E_p	undrained modulus of deformation as derived from pressuremeter test or Young's Modulus of plate material
E_R	unload modulus of deformation as derived from screw plate test

E_u	undrained modulus of deformation also called undrained Young's Modulus
$\bar{E}_u$	normalized undrained modulus of deformation
f_s	side friction of standard penetration test device
m	abbreviation metres
mm	abbreviation millimetres
N	penetration resistance derived from standard penetration test
N_c	bearing capacity factor for a deep circular foundation embedded in an ideal cohesive soil
NC	normally consolidated clay
NGI	abbreviation Norwegian Geotechnical Institute
OC	over consolidated clay
P_o, P_f, P_ℓ	pressures corresponding to the in situ stress state, the failure pressure and limit or ultimate pressure as derived by pressuremeter test
p	average pressure over the area of the screw plate
p_{ult}	maximum pressure or failure pressure obtained from pressure displacement curve of screw plate test results
PMT	abbreviation pressuremeter test
Q	ratio between diameter of borehole and diameter of pressuremeter probe

q_c	end bearing of standard penetration test device
R	radius of pressuremeter probe, or relative rigidity of the soil plate system
R_f	f_s/q_c friction ratio side friction to end bearing of standard penetration device
SPT	abbreviation standard penetration test
U	unconfined compression test
UU	unconsolidated undrained triaxial test
V_c	initial volume of central measuring cell of pressuremeter probe with the volumeter at zero
V_m	the volume of water injected under pressure change $\Delta p = (p_o + p_f)/2$ in pressuremeter test
V_o	initial volume of pressuremeter at start of test
VST	abbreviation vane shear test
w	displacement of screw plate; subscripts are used to denote particular test being modelled
α	non dimensional constant to correlate E_{oed} to q_c in cone penetration test
β	non dimensional constant to correlate pressuremeter test results to c_u
γ_z	total overburden pressure at depth z

- η radial deformation of pressuremeter probe $\Delta R/R$ or b/a in oblate spheroid model of screw plate test
- λ a non dimensional constant which depends on Poisson's ratio and geometry of the problem being used to model the screw plate test
- ν Poisson's ratio
- ξ_a radius of the circular area over which the axial load is applied in the problem considering a flexible circular plate embedded in bonded contact with the elastic medium
- σ_o vertical effective overburden stress
- $\sigma_{\phi\phi}$ circumferential stress
- σ_{rr}, σ_{zz} stresses referred to the cylindrical polar coordinate system
- σ_{rz}

LIST OF APPENDICES

	page
A Calibration of Proving Ring to Measure Load	161
B Screw Plate test data and load displacement curves for tests carried out at the Gloucester test fill site	164
C Log ₁₀ Pressure vs. percent displacement curves used to derive undrained shear strength measured by the screw plate test at the Gloucester test fill site	237

CONVERSION OF UNITS

SYMBOLS AND CONVERSION FACTORS FOR SI UNITS

 $m = \text{metre} = 39.37 \text{ inches}$ $mm = \text{millimetre} = 0.03937 \text{ inches}$ $N = \text{Newton} = 0.22481 \text{ pound force}$ $kN = \text{Newton} = 224.81 \text{ pound force}$

CONVERSION FROM IMPERIAL TO SI UNITS.

 $1 \text{ inch} = 25.4 \text{ mm}$ $1 \text{ foot} = 304.8 \text{ mm} = 0.3048 \text{ m}$ $1 \text{ lb force} = 4.4482 \text{ N}$ $1 \text{ kip} = 4448.2 \text{ N} = 4.4482 \text{ kN}$

CONVERSION OF UNITS FOR PRESSURE AND STRESS

 $1 \text{ psi} = 1 \text{ lb force per square inch} = 6.8947 \text{ kPa}$

or

 $1 \text{ kPa} = 1 \text{ Kilo Newton per square metre} = 0.14504 \text{ psi}$

INTRODUCTION

In order to predict the initial or instantaneous deformation of structural foundations resting on soil media, it is necessary to have a prior knowledge of the deformation characteristics of the soil medium. Similar results concerning the undrained shear strength of the soil medium is important to the estimation of the end of construction or short term ultimate bearing capacity of the foundation. The deformation and strength characteristics of cohesive soils are usually determined from laboratory tests conducted on relatively *undisturbed* samples obtained at various depths within the soil profile. These samples are commonly obtained by devices such as the open thin walled tube sampler (50 mm minimum diameter), or the stationary piston sampler (76 mm minimum diameter). On occasions (Bozozuk, 1963) block samples with sides of approximately 30 cm can be obtained, but the depths at which such samples can be retrieved are comparatively shallow (Bozozuk and Leonards, 1972). Block samples taken at greater depths during the excavation of the foundation are certainly important to the further study of the material behaviour, but these results provide only peripheral information which may be of distinct use for a foundation under construction.

In situ plate load tests and large scale footing tests have been used as means for the accurate determination of the deformation and strength characteristics of cohesive soils. Such tests are, however, both expensive and time consuming. Furthermore, these tests can only be contemplated on very large scale geotechnical projects. Other, less expensive, in situ testing devices are now being used to obtain information related to the strength and/or deformation characteristics of the entire soil deposit. Devices such as the field vane apparatus, the dutch cone penetration apparatus and the pressuremeter have found widespread use.

In the early 1950's the Norwegian Geotechnical Institute developed (under the direction of the late Dr. L. Bjerrum) the screw plate test in an attempt to:

- i) eliminate the need for the time consuming and difficult surface plate load test and

- ii) to obtain accurate plate load test data at large depths below the ground surface and below the ground water table in both cohesionless and cohesive soils.

In early applications (Kummenje and Eide, 1961) the screw plate test was used primarily as a sounding device, similar to the cone penetration test, to observe the change in density of loose sand deposits due to blast effects. Further field studies of screw plate testing of a granular soil deposit are reported by Gould (1967). Schmertmann (1970) and Janbu and Senneset (1973) have used the screw plate test to measure in situ deformability characteristics of cohesionless soil deposits. Dahlberg (1975) has used the test to ascertain the influence of precompression on deformability of granular media. More recently, studies which involve time independent and time dependent screw plate testing of silty clays are reported by Schwab (1976) and Schwab and Broms (1977). Further investigations are also reported by Berg and Olssen (1978).

Since the primary application of the screw plate test has been concerned with testing in granular soil deposits, the interpretation of the test result invariably involved certain ad hoc empirical assumptions. The empiricism that is usually associated with the treatment of settlement and deformation in granular media has therefore prompted the use of similar relationships for the estimation of *moduli of deformation* from screw plate tests. When dealing with screw plate tests conducted in cohesive soil media, the assessment of the in situ deformability characteristics of the soil medium has to be approached in a manner that will enable a comparison with results derived from other classes of in situ tests, such as the pressuremeter test.

This thesis examines the performance of the screw plate test in the light of the classical theories of isotropic elasticity and rigid plastic behaviour. The relationship proposed by Selvadurai and Nicholas (1979) and Selvadurai, Bauer and Nicholas (1979) are used to determine or interpret the in situ undrained elastic modulus and the in situ undrained shear strength of a cohesive soil from the load-displacement relationship measured in a screw plate test. This thesis also documents the results of a series of screw plate tests that were conducted at the Gloucester test fill adjacent to

the Canadian Forces Station (CFS) Gloucester, approximately 20 kilometers east of Ottawa. The test fill at this site has been the subject of extensive field and laboratory investigations (see eg, Bozozuk and Leonards, 1972; Lo *et al.*, 1976). These investigations form the basis for the comparison of results derived from the screw plate test.

CHAPTER ONE

1.0 The Estimation of Strength and Deformability Characteristics of Soft Clay: Laboratory vs. In situ Testing.

1.1 Introduction

The main objective of this chapter is to provide a brief outline of the difficulties encountered by geotechnical engineers in attempting to obtain relatively undisturbed samples of sensitive marine clay from which the deformability and strength characteristics can be assessed in the laboratory. The extent to which disturbances caused by sampling and handling affect the undrained shear strength and modulus of elasticity of the soil are presented. Difficulties in the interpretation of these parameters from laboratory results obtained from conventional laboratory testing methods, are presented and compared with settlements observed of full scale structures.

Over the past forty years various in situ testing devices have been developed in attempts to correlate laboratory and in situ strength and/or deformability characteristics. Four such devices are briefly described and the factors which affect the results of these tests are presented. A comparison of results obtained from these in situ test methods and results from relatively *undisturbed* samples (see eg. Eden and Hamilton, 1956; Simons, 1957; La Rochelle and Lefebvre, 1970; and Shields and Bauer, 1975) are presented in tabular form.

1.1.1 Sensitivity

The sensitivity of a clay is defined as the ratio of its undisturbed strength to its remoulded strength at constant water content. Skempton and Northey (1952), conducted tests on six different clays to determine why certain clays exhibit high sensitivities and clays of similar composition are insensitive or exhibit low sensitivity. They observed that thixotropy and

changes in the microstructure due to leaching are two major reasons which account for the extreme difference in sensitivity exhibited by clays of similar physical composition. Sensitive clays, such as Leda clay, have developed a microstructure due to its past geological environment. Once this structure is destroyed by mechanical action such as remoulding, it cannot be reconstituted artificially (see Torrance, 1975). This characteristic limits the confidence of geotechnical engineers in their ability to sample and to interpret laboratory tests for the estimation of the deformation properties of sensitive soil media.

1.2 *Sampling Disturbances in Sensitive Clays*

1.2.1 *Effects on the Undrained Shear Strength*

In the early 1960's considerable attention was given to the sampling and laboratory investigations of soft and sensitive clays (Coates and McRostie, 1963; Crawford, 1963; Hall, 1963; Skempton and Sowa, 1963; Ladd and Lambe, 1963; and Noorany and Seed, 1965). Skempton and Sowa (1963) and Noorany and Seed (1965) investigated the relationship between the strength measured with unconsolidated undrained triaxial tests on *perfectly undisturbed* samples and the strength mobilized in situ. A *perfectly undisturbed sample* is one in which there is no mechanical disturbance due to sample retrieval, transportation or handling. In each study, identical samples were anisotropically consolidated in triaxial cells to represent the in situ stress state of the sample. Upon completion of consolidation, the drainage valves were closed and one sample was then tested to failure in undrained compression by increasing the deviator stress. This test represented the in situ strength of the clay. The axial and radial stresses on the other sample were simultaneously removed to simulate the stress change during a perfect sampling operation. The sample was then tested to failure in unconsolidated undrained compression. Skempton and Sowa (1963) conducted these tests on a clay with a sensitivity of 2. They found that the perfect sample undrained shear strength

was 1 to 2 percent less than the undrained shear strength of the samples which simulated the in situ state. It was conjectured that there would be a greater deviation in the two strengths for clays of higher sensitivities. Noorany and Seed (1965) tested a clay with a sensitivity of 8. The difference in the strength values from their study was in the order of 6 percent. They noted that simply the release of stresses from an in situ element of undisturbed, normally consolidated clay induced sufficient strain and change in pore water pressure to cause the results of unconsolidated undrained tests to give low strengths.

In practice, however, considerably larger discrepancies in the undrained shear strength were being reported. For example, Coates and McRostie (1963) stated that shear strengths obtained from hand cut block samples were up to twice the magnitude of those obtained by conducting undrained triaxial tests on samples obtained from Shelby tube samplers. The use of a piston type sampler increases the average shear strength from unconfined compression tests by approximately 25 percent over open tube samplers. However, the results of unconfined compression tests of samples from different types of piston samplers varied by as much as 40 percent. The consolidated undrained strength obtained from block samples are 50 to 80 percent higher than the results from samples obtained by open Shelby tube samplers. Unconsolidated undrained strengths from block samples were as much as 200 percent greater than strengths obtained from samples retrieved by open Shelby tube samplers.

Hall (1963) used 152 mm diameter Shelby tubes and a 76 mm diameter Swedish foil sampler to obtain samples from the San Francisco Bay area. The undrained shear strength obtained from the Shelby tube samples was consistently 30 percent lower than from the Swedish foil sampler. This indicated that a larger size diameter of sampler does not necessarily imply a higher quality sample.

Investigations have been conducted by Berre *et al.* (1969); Bozozuk (1970); Conlon and Isaacs (1970); Eden (1970); La Rochelle and Lefebvre (1970); and Raymond *et al.* (1971) to

name a few. These studies emphasized the effects of the type and size of samplers used in a site investigation on the sample disturbance. Bozozuk (1970), conducted consolidation tests and anisotropically consolidated undrained triaxial tests on samples obtained from the Gloucester test fill site. The samples were retrieved using a 54 mm diameter NGI piston sampler and a 127 mm diameter Osterberg hydraulic sampler. The consolidation tests on the NGI piston samples produced a more gradual drop off than the Osterberg sampler, therefore indicating some degree of disturbance (see Figure 1.1). Since the disturbances due to handling and trimming was assumed to be the same, it was concluded that the disturbance that caused the gradual decrease in the test results was due to the size of the NGI piston sampler. The results of the triaxial tests were essentially the same. This was probably due to the consolidation of the samples before testing.

Cohnon and Isaacs (1970) obtained block samples as well as 54 mm, 76 mm and 127 mm diameter tube samples from a hand dug test hole 1.8 m in diameter and 23 m deep. A qualitative review of the results on the effects of sample disturbance on the undrained shear strength is shown on Figure 1.2.

Eden (1970) used the site of a sewage treatment plant on the eastern outskirts of Ottawa, Ontario to investigate the degree of disturbance caused by various samplers. The soil profile at this site consists of approximately 23 m of sensitive overconsolidated Leda clay with sensitivities ranging from 28 at a depth of 6 m to 200 at a depth of 15 m. Sensitivities greater than 500 were observed beyond the depth of 15 m. Block samples, used as the *undisturbed* reference state, were obtained at six depths during the excavation of the pumping wells. Tube samples were obtained by using a 54 mm diameter NGI piston sampler with a fixed piston, a 50 mm diameter Swedish piston sampler also with a fixed piston, a 63 mm diameter Swedish foil sampler and a 124 mm diameter Osterberg sampler with a fixed piston. Approximately 50 consolidation tests and 280 undrained shear strength tests were conducted. Samples with failure strains of greater than 5 percent

CODE	SAMPLING METHOD	AREA OF SPECIMEN
▲	51 mm dia. tube	20 cm ²
○	51 mm dia. tube	20 cm ²
●	127 mm dia. tube	20 cm ²
□	127 mm dia. tube	40 cm ²
■	127 mm dia. tube	60 cm ²

FIG. 1.1 EFFECT OF SAMPLING AND SIZE OF TEST SPECIMEN ON THE PRECONSOLIDATION PRESSURE MEASURED IN SOFT MARINE CLAY

(AFTER BOZOCUK, 1971)

FIG. 1.2. SAMPLE DISTURBANCE CAUSED BY VARIOUS SAMPLING TECHNIQUES

(AFTER CONLON AND ISAACS, 1970)

were discarded. The results from the remainder of the tests are shown on Figure 1.3. These results, in effect represent laboratory test results conducted on high quality samples. Even these results indicate that the undrained shear strengths are on average approximately 40 percent less than the results obtained from block samples. Owing to the extremely wide scatter in the results it is difficult to draw any conclusions regarding which sampler provided the best results consistent with undisturbed behaviour. Eden (1970), however, concluded that the Swedish foil sampler had the largest clearance ratio and the most divergent results.

La Rochelle and Lefebvre (1970) studied the effects sampling techniques had on the cementation bonds of sensitive cohesive soils. A complete discussion of these cementation bonds can be found in an article by Crawford (1963) and in the recent text by Mitchell (1977). The undrained shear strength derived from a 54 mm diameter NGI piston sampler was 40 to 50 percent lower than the strength obtained from block samples at two sites, St. Vallier and St. Louis de Bonsecours in Quebec. Consolidated isotropically drained and consolidated isotropically undrained tests were also performed at various confining pressures on samples taken from these sites. It was concluded that high confining pressures considerably effect the undrained shear strength of the clay. This was attributed to the breakdown of the cementation bonds between particles. Tests were also conducted to determine the level of lateral strain at which the cementation bonds begin to disintegrate. At a lateral strain of 0.02 the undrained shear strength was reduced to 70 percent of its original value.

La Rochelle and Lefebvre (1970) proposed that due to the type of sampler used, this lateral strain was most probably exceeded (during the tube penetration) due to the clearance ratio. Lateral expansion could therefore account for most of the strength loss associated with sampling.

The investigation of Raymond *et al.* (1971) follows the same pattern adopted by Eden (1970) and La Rochelle and Lefebvre (1970). Raymond *et al.* (1971) extended their study to include the examination of sampling disturbance caused not only by the best

FIG. 1.3 COMPARISON OF TEST RESULTS OBTAINED FROM
VARIOUS SAMPLING TECHNIQUES

(AFTER EDEN, 1970, SITE: OTTAWA)

sampling methods but also to the examination of sampling methods commonly used in native site investigations. Six different sampling techniques were used to obtain soil samples from two different depths at a site near Ottawa. The exact location of the site is recorded as the junction between Heron Road and Bronson Avenue. The samples used in the investigation included:

- i) block samples
- ii) 127 mm diameter samples obtained by an Osterberg sampler with an inside clearance ratio of 0.42 percent
- iii) 50 mm diameter samples obtained with the Swedish piston sampler with an inside clearance ratio of 0.4 percent
- iv) 50 mm diameter samples obtained with a piston sampler using standard thin walled Shelby tubes which were ground to have a sharp cutting edge and an inside clearance ratio of 0.5 percent
- v) 50 mm diameter samples obtained with a piston sampler using standard thin walled Shelby tubes which were not ground and had an inside clearance ratio of 0.5 percent, and
- vi) 50 mm diameter samples obtained with a standard open Shelby tube sampler.

The results of unconfined compression tests indicated that the maximum shear strength (c_u) was substantially affected by sampling methods as was the strain at the maximum stress (see Figure 1.4). It was conjectured that the Shelby piston sampling with a sharp cutting edge would be superior to the Swedish sampler since the cutting edges were similar and the Shelby tubes thinner. In some instances, a power hack saw was used to extract the sample from the Shelby tube. Tests on these samples all indicated considerably more sample disturbance than samples which were extracted by vertical jacking. The vibrations of the power hack saw induced considerable disturbance within the soil sample located in the tube. Since the jacking of the complete sample also caused considerable disturbances, it was suggested that a shorter length of sample in the sampling tube be retrieved. Alternative suggestions included the use of some form of lacquered liner in the tubes.

In continuing the line of investigation adopted by La Rochelle

FIG. 1.4 EFFECT OF SAMPLER TYPE ON STRESS-STRAIN PROPERTIES

(AFTER RAYMOND *ET AL.*, 1971)

and Lefebvre (1970) (regarding the effects of confining pressure on the undrained shear strength) Raymond *et al.* (1971) noted that the sampling process using 50 mm diameter Shelby tubes caused considerable disturbance to the cementation bonds resulting in lower strengths at low confining pressures. In contrast the strength values from samples obtained by using the 50 mm diameter Shelby tubes were much higher than those obtained from triaxial tests on the best samples at higher confining pressures (but less than the preconsolidation pressure). These results would suggest that on occasion, the use of Shelby tubes can result in the prediction of undrained shear strength values which lie on the unsafe side.

Investigations at the Norwegian Geotechnical Institute (Berre, Schjetne and Sollie, 1969) resulted in the development of a new piston sampler for the retrieval of 95 mm diameter samples using thin walled tubes hereby to replace the commonly used 54 mm diameter sampler. It was felt that the 54 mm diameter sampler introduced considerable disturbance so as to make the definition of *undisturbed sample* virtually inapplicable. A statistical analysis of results from oedometric compression tests indicated that less disturbed samples were obtained with the 95 mm piston sampler than with the 54 mm sampler. In this investigation no attempt was made to estimate the manner in which the preconsolidation stress measured from the 95 mm diameter sampler compares with oedometric tests conducted on samples trimmed from block samples.

Recently Domaschuk (1977) and Lefebvre and Poulin (1979) have reported on the development of sample retrieval equipment designed for the retrieval of large diameter samples from the bottom of unlined boreholes. Domaschuk (1977) describes a sampling device which is capable of retrieving block samples approximately 230 mm square by 460 mm in height. Briefly, the sampler consists of a circular cutter about 300 mm in diameter and 460 mm in height containing a square cutter approximately 230 mm square and 480 mm in length. After the sample has been retrieved from the borehole the circular and square cutters are separated. The sample inside the square cutter is then extruded by vertical jacking and the block sample is prepared for transportation to the laboratory.

Samples of Winnipeg, Manitoba brown and blue clay were retrieved using this sampler and tested at several depths within each sample using a pocket penetrometer and a lab vane apparatus. From his findings Domaschuk (1977) concluded that there was virtually no sample disturbance to within about 25 mm of the sampler walls measured from the center of the vane. For comparison purposes penetrometer readings were taken on a sample of brown clay recovered with a conventional 76 mm diameter Shelby tube. The results of the pocket penetrometer tests on both the block sample and the Shelby tube sample are shown in Figure 1.5. The results indicate that the reductions in strength near the faces of the block samples are minor and comparable to those associated with Shelby tube sampling. As a result a larger volume of relatively undisturbed sample can be retrieved using the block sampling technique. It should be noted however that Shelby tube sampling is not considered to retrieve the same calibre of sample as the Osterberg or the NGI sampler. Domaschuk (1977) does not report on any comparison of results of unconfined compression or confined triaxial tests on samples obtained with the block sampler with results of the same tests on samples obtained by either Osterberg or NGI sampling techniques.

Lefebvre and Poulin (1979) describe a sampler developed at the University of Sherbrooke which carves a cylinder of soil about 25 cm in diameter from the base of an unlined borehole. The sample is carved by three cutting tools having an annular motion permitting the opening of a slot about 5 cm wide around the clay cylinder. At each cutting tool, water or bentonite mud is fed from the surface to help evacuate the cuttings. After carving a soil cylinder of about 35 cm in height, components of a horizontal diaphragm, fixed at each cutting tool and contained in the annular slot when in the opened position, gradually close as the apparatus continues to rotate. The penetration of the diaphragm under the sample is helped by small cutting tools fixed at the inner ends of the diaphragm components. Closure of the diaphragm separates the sample from the deposit and forms a support under the sample to lift it out of the borehole. On average it takes a 4 man crew

FIG. 1.5 UNDRAINED SHEAR STRENGTH: (a) Shelby tube sample using a pocket penetrometer, (b) comparison of block and Shelby tube strengths

(AFTER DOMASCHUK, 1977)

about 3 hours to obtain and prepare each block sample for shipment to the laboratory.

In 1976 this sampler was used at Nicolet, Quebec to obtain samples at a depth of about 5 m. These samples were obtained only a few metres from a trench dug in 1974 where conventional block samples were obtained at a depth of 4.5 m. Of 7 unconfined compression tests conducted on specimens cut from a block sample obtained with the University of Sherbrooke sampler, the undrained shear strength compared very closely with the undrained shear strength obtained from samples carved from conventional block samples. The results of this investigation indicate that much of the sample disturbance during the sampling operation is caused by the pushing of a tube into the soil deposit. This device now provides an economic means of obtaining high quality large diameter samples from the base of a borehole as compared to excavation methods.

From the results of careful field studies conducted by the above investigators, it becomes evident that the magnitude of the undrained shear strength of a sensitive soil with a fragile microfabric is highly sensitive to the effects of disturbance introduced by sample retrieval (and perhaps testing). If unconsolidated undrained triaxial tests are conducted on samples confined at low confining pressures, the resulting undrained shear strengths are low compared to similar results obtained from block samples. Unconsolidated undrained triaxial tests on samples confined at higher confining pressures (which are less than the preconsolidation pressure) can give unsafe shear strength results. Undrained tests carried out on samples which have been preconsolidated to the in situ vertical effective stress tend to give higher strengths in clays that exhibit a cemented structure. Eden (1970) concluded that sampling techniques available (at that time) for obtaining undisturbed samples were in general inadequate. Only a block sample would provide a reliable approximation to an undisturbed soil sample. Since sampling techniques have changed very little in the past ten years one can only conclude that samples of sensitive clays obtained by standard sampling techniques are still an inadequate substitute to the undisturbed soil sample. Tests

conducted on samples obtained using the University of Sherbrooke block sampler compare closely with test results obtained from samples carved from conventional block samples. To the author's knowledge, the Sherbrooke sampler, however, has not been used to obtain block samples from depths greater than about 10 metres.

1.2.2 Effects on the Undrained Modulus of Elasticity

The undrained modulus of elasticity (E_u) shall hereby be defined as the ratio of the stress to the strain taken over the initial linear region of the stress-strain curve as shown in Figure 1.6. As indicated the undrained modulus of elasticity may represent an initial tangent modulus or secant modulus at some specified stress level. Results of Figure 1.4 clearly indicate that the value of the undrained modulus of elasticity (E_u) can be adversely affected by the degree of sample disturbance. It is generally assumed that sample disturbance has a greater effect on the in situ stress-strain properties of a soil than on its in situ shear strength (Hvorslev, 1949). It has been observed (Skempton, 1948; Eden, 1970; Bozozuk, 1970; Raymond *et al.*, 1971; and Bjerrum, 1972) that the ratio (E_u/c_u) generally remains constant for a relatively undisturbed soil and tends to decrease with increasing degree of disturbance. Raymond *et al.* (1971) consolidated samples prior to undrained triaxial shear. They observed that by changing the magnitude of the consolidation pressure, the ratio E_u/c_u changed. They also observed that the optimum consolidation pressure depended on the degree of sample disturbance. The changes in E_u/c_u were not considered to be substantial, and as a general guide they recommended that a consolidation pressure from 1/2 to 3/4 of the preconsolidation pressure could be used for consolidated undrained triaxial tests.

As observed by Ladd (1963), the value of the undrained modulus derived from laboratory tests were also found to be highly dependent on the exact experimental procedure used in its evaluation. Test results showed that moduli determined from unconfined compression tests could be 2 to 3 times lower than moduli determined from unconsolidated undrained triaxial tests. The moduli values

FIG. 1.6 TYPICAL STRESS-STRAIN CURVE

determined from unconsolidated undrained tests were one tenth to one fifteenth of those moduli values determined from undrained triaxial tests in which the samples were isotropically consolidated at pressures that equalled or exceeded the in situ vertical effective stress. The undrained moduli values determined by Skempton and Sowa (1963) from undrained triaxial tests were several times higher than those derived from unconfined compression tests. The results of the unconsolidated undrained tests were in turn approximately 0.7 to 0.9 times those derived from the isotropically consolidated undrained triaxial tests. Simon (1957) reports the results of case studies conducted in Norway, which involved the measurement of settlements of two buildings on normally consolidated clays with sensitivities of 2 and 20. Undrained modulus values were determined from unconfined compression tests and from isotropically consolidated undrained tests with consolidation pressures equal to the vertical effective stress. Simons (1957) concluded that the modulus values from the unconfined compression tests were 3 to 5 times lower than moduli values back calculated from the actual settlements of the buildings. The modulus values derived from the isotropically consolidated undrained tests were, however, 2 to 4 times higher than the back calculated moduli. Bozozuk (1963) on the other hand, obtained very close agreement between the initial tangent undrained modulus of elasticity derived from unconfined compression tests on an overconsolidated clay and the undrained modulus back calculated from the elastic heave of an excavation near Ottawa, Ontario at the site of the Ottawa Sewage Treatment Plant. Samples for the unconfined compression tests were trimmed from large block samples. In the estimation of the elastic heave it was assumed that the soil possessed a Poisson's ratio of 0.4.

In studying the effects of sample disturbance on the deformability and strength characteristics of a sensitive clay, Milovic (1970) sampled overconsolidated clays from sites near St. Simon and Nicolet in Quebec. Samples were retrieved using 69 mm diameter NGI piston samplers and 69 mm diameter Shelby tube samplers. Block samples were also obtained at various depths and were used as the reference state for undisturbed soil.

Unconfined compression, consolidated undrained triaxial and consolidation tests were conducted on the samples retrieved. The results (see Figures 1.7a and 1.7b) show the variations in the stress-strain curves derived from unconfined compression tests and consolidated undrained tests. The samples were consolidated at approximately the existing in situ vertical effective stress. The maximum value of the undrained secant modulus (calculated at a stress level of 0.5) and the undrained shear strength were approximately the same for the unconfined compression tests and the isotropically consolidated undrained triaxial tests for the clay at the St. Simon site (sensitivity of 10). The results, however, were more consistent for the latter tests. Isotropically consolidated undrained triaxial tests on samples from the site of Nicolet (sensitivity of 15) gave undrained secant moduli values that were approximately 60 to 170 percent greater than the undrained secant moduli values derived from unconfined compression tests. These results (from St. Simon and Nicolet) indicate that variations in results derived from different test methods may be dependent on the sensitivity of the clay.

Plate load tests were also conducted at each site. The undrained secant moduli values derived from isotropically consolidated undrained triaxial tests on specimens carved from the block samples were approximately 76 percent of the in situ undrained secant modulus of the St. Simon clay and 79 percent of the in situ undrained secant modulus of the Nicolet clay. These results seem to be in contradiction with the findings of Simons (1957) discussed earlier. The reason for this disagreement may be explained by the fact that the investigation of Simons (1957) dealt with normally consolidated clays whereas both of the clayey soil deposits discussed by Milovic (1970) exhibit over consolidation effects.

Soderman, Kim and Milligan (1968) conducted extensive tests on both remoulded and undisturbed samples of a stiff to firm grey clay till with a low sensitivity of 2 to 3. A test trench was excavated to depths of 1.8, 3.0 and 4.3 metres. At each elevation, 50 mm and 100 mm thin walled Shelby tubes (with

FIG. 1.7a UNCONFINED COMPRESSION TEST RESULTS

(AFTER MILOVIC, 1970)

FIG. 1.7b CONSOLIDATED UNDRAINED TEST RESULTS

(AFTER MILOVIC, 1970)

an inside clearance ratio of zero percent) were pushed into the clay to a depth of 38 cm, and then dug out to minimize disturbance. Block samples were also obtained at each elevation. Plate load tests using a 152 mm diameter plate and a cyclic loading procedure were carried out at each elevation. These tests were conducted in both the horizontal and vertical directions. Figure 1.7c shows a complete summary of the variations in E_r (reload modulus), (see Figure 1.6), with respect to:

- i) the stress ratio
- ii) the type of sample and
- iii) the testing procedure.

Figure 1.7d shows the variation in the reload modulus at each of the three depths. The reload moduli have been calculated from a secant drawn from the zero load level to a stress level of one third. The reload moduli are shown with respect to the type of sample tested and the method of testing.

Although the clay till was a relatively insensitive clay, the value of the reload modulus was found to be quite dependent on the degree of sample disturbance. It is pertinent to question the use of sampling tubes with a zero inside clearance ratio. The methods of extracting the samples were not indicated. A zero clearance does not allow the sample to swell, therefore the release of internal stresses is less than would be the case if a sampler with an inside clearance ratio was used. It is conceivable that a greater degree of disturbance could be caused when the sample (obtained from a tube sampler with a zero inside clearance ratio) is extruded. Since the sample cannot expand inside the tube there is an increased amount of friction between the sampler tube walls and the soil sample. As discussed previously for a more sensitive clay, the process of pushing the sampling tube into the ground and then extracting the sample from the tube will cause very severe disturbance.

Shields and Bauer (1975) compared the undrained modulus as determined by various in situ and laboratory tests with the undrained modulus back calculated from a full scale footing test. The in situ tests conducted were the plate load test and the

FIG. 1.7c RELOAD MODULUS vs. STRESS RATIO FOR SAMPLE SIZE AND TEST TYPE SHOWN

(AFTER SODERMAN ET AL., 1968)

FIG. 1.7d RELOAD MODULUS vs. DEPTH FOR SAMPLE SIZE AND TEST TYPE

(AFTER SODERMAN ET AL., 1968)

pressuremeter test. The laboratory tests conducted on relatively undisturbed samples were unconsolidated undrained triaxial, consolidated undrained triaxial and stress path tests. The site of the investigation was Algonquin College Campus, in Ottawa. The campus is underlain by overconsolidated lacustrine and marine clay. Samples for the triaxial testing program were obtained with thin walled tube samplers. The plate used in the load tests was 46 cm in diameter and was considered to be rigid. Only the plate load test gave results that were comparable to the prototype footing. The undrained moduli values derived from the consolidated undrained tests were approximately 90 percent of the moduli value back calculated from the proto-type footing tests. Undrained moduli values derived from the unconsolidated undrained, stress path tests, and the pressuremeter tests were in the order of 33 to 50 percent of the moduli value back calculated from the proto-type footing test.

The previous studies have shown that in sensitive clays, the determination of both the undrained shear strength and the undrained modulus of elasticity are affected by sampling disturbances associated with the sampling method, the testing method and the procedure. The degree of sample disturbance is to a large extent governed by the sensitivity of the clay mass, and the past stress history of the deposit. In normally consolidated clays Ladd (1963) suggested consolidated undrained tests for the determination of the undrained modulus of elasticity. Consolidated undrained tests, with consolidation pressures of approximately $1/2$ to $3/4$ of the preconsolidation pressure yield results that are more reliable than either unconfined compression or unconsolidated undrained triaxial tests (Raymond *et al.*, 1971). As pointed out by Simons (1957) and Ladd (1963), moduli values are usually 2 to 4 times greater than observed in situ moduli values.

In practice it is customary to obtain 50 mm diameter piston samples. Raymond *et al.* (1971) found that 50 mm diameter samplers cause considerable disturbance to the cementation bonds. This results in lower strengths at low confining pressures and higher unsafe strengths at higher confining pressures which are, however,

lower than the preconsolidation pressures. The investigations of Soderman *et al.*, (1968); Bozozuk, (1970); Conlon and Isaacs, (1970); Eden, (1970); Milovic, (1970); and Raymond *et al.*, (1971) indicate that sample disturbance due to the size of the sampler and the sampling technique, are major factors which contribute to the reduction in laboratory measured values of the undrained shear strengths and undrained modulus of elasticity. The sensitivity of the clays investigated by the above authors ranged from 2 (Soderman *et al.*, 1968) to greater than 200 (Eden, 1970). Milovic (1970) found that for overconsolidated clays, the maximum value of the undrained shear strength and the undrained modulus of elasticity obtained from unconfined compression and consolidated undrained tests were comparable for the two clays examined. These clays had sensitivities of 10 to 15. Due to the sensitivity of the clay, laboratory test results are more divergent (even for good quality samples) making the results difficult to interpret.

The findings and conclusions of most research workers engaged in the correlation of performance of sample retrieval techniques seem to suggest that block samples are perhaps the best source for the laboratory estimation of the in situ deformability and strength characteristics of cohesive (sensitive and insensitive) soil media. However, it becomes increasingly difficult to obtain such samples at large depths within the soil mass. This deficiency is, to a certain extent, overcome by novel sampling devices such as the *Sherbrooke Sampler* (Lefebvre and Poulin, 1979) which is capable of obtaining large diameter (250 mm) block samples at depths up to 10 metres. A similar device has been reported by Domaschuk (1977). Even with such sampling devices the extent of sampling disturbances has to be clearly identified. With sensitive cohesive soils the presence of a high degree of sample disturbance in retrieved samples has prompted the increased application of in situ techniques for the estimation of in situ deformability and strength characteristics. In situ techniques usually provide a fast and economical solution for obtaining information pertaining to the deformation and strength characteristics of a cohesive soil media. Furthermore, many of the in situ testing devices are capable

of providing a detailed description of the mechanical properties of the soil medium throughout the depth of the soil deposit with relative ease.

1.3 In situ Testing Techniques

1.3.1 Introduction

The purpose of this section is to present a brief overview of the more widely used in situ testing methods as they relate to the determination of the undrained modulus of elasticity and/or the undrained shear strength. The four methods to be discussed are:

- i) the standard penetration test (SPT)
- ii) the quasi-static cone penetration test (CPT)
- iii) the vane shear test (VST), and
- iv) the pressuremeter test (PMT).

Particular attention is given to the identification of the various factors that affect the results derived for each in situ testing technique.

1.3.2 Standard Penetration Test (SPT)

The standard penetration test had its origin in the United States around 1902 with the driving of a 25 mm diameter open end pipe into the soil during the wash boring process. Toward 1930 the test was standardized by the stipulation of the size of the device and the driving effort (ie. a 51 mm split sampler spoon, driven into the soil with a 63.5 kilogram weight having a free fall of 76 cm, see Figure 1.8). The number of blows required to drive the spoon a distance of 30 cm, after an initial penetration of 15 cm, is referred to as the penetration resistance, N . The SPT has been used mainly to provide a qualitative indication of soil density with depth and to provide samples of the soil for the purpose of soil classification. The test is considered by many to be extremely variable.

FIG. 1.8 SPLIT SPOON SAMPLER USED IN STANDARD PENETRATION TEST

(AFTER PECK, HANSEN AND THORNBURN, 1974)

(1 inch = 2.54 cm)

The majority of the work with the SPT has been conducted in sandy soil. However some research has been conducted on SPT in clayey soils (Houston, 1960; Fletcher, 1965; Schmertmann, 1967; and De Mello, 1971). Figure 1.9 shows correlation data between the SPT penetration resistance, N , and the unconfined compressive strength c_u , for clays of low, medium and high plasticity indices. Such charts, however, should only be used in areas where they have been developed or for similar soil types. The correlations shown of Figure 1.9 do not take into account the effects of overburden pressure. For normally consolidated clays any increase in overburden pressure will produce a proportional increase in the undrained shear strength. Also these charts do not consider the effects of overconsolidation. Schmertmann (1975) and De Mello (1971) agree that in non-fissured, saturated clay N values will increase with the overconsolidation ratio compared to the case when the same clays are considered to be normally consolidated.

In sensitive clays the SPT is known to produce N values that are much too low considering the N vs. undrained shear strength correlations obtained in much less sensitive clays. Figure 1.10 shows the variation in N with sensitive clays. Schmertmann (1975) indicates that the reduction in the blow count is likely due to the high positive pore pressures generated in sensitive clays during the sampler penetration. Schmertmann (1975) derived Table 1.1 by assuming that the end bearing and side friction resistance is proportional to q_c and f_s (as measured by the cone penetration test). He found a dramatic increase in the importance of side friction on the sampler as the friction ratio, $R_f \hat{=} f_s/q_c$ increases. An insensitive clay usually has a high friction ratio and most of its N value is due to side friction. The remoulding action of the sampler greatly reduces the side friction in sensitive clays and hence there would be a great reduction in the N value.

In conclusion, correlations between the N value and the undrained shear strength should be approached with extreme caution. The use of published charts may give extremely erroneous results unless the chart is based on a clear representation of local soil

FIG. 1.9 SOME CORRELATIONS BETWEEN N AND q_c IN CLAYS
(AFTER SCHMERTMANN, 1975)

FIG. 1.10 INFLUENCE OF SENSITIVITY ON PENETRATION
RESISTANCE
(AFTER DE MELLO 1971)

TABLE 1.1

R _f %	TYPICAL SOIL TYPES	% OF N DUE TO	
		END BEARING	SIDE FRICTION
1	SAND, ABOVE AND BELOW GROUND WATER	56	44
2.5	SILT AND CLAYEY SAND	34	66
4	SANDY CLAY, CLAY (S _t = 4)	25	75
8	INSENSITIVE CLAY AND PEATY SOIL	14	86

$$R_f = f_s / q_c \times 100$$

where f_s = side friction as measured by CPT
 q_c = end bearing as measured by CPT

After Schmertmann, 1975

conditions. Sensitivity has been found to have a great effect on the measured N value. The use of penetration resistance, N , and undrained shear strength correlation charts that do not take into account the soil sensitivity can lead to an under-estimation of the undrained shear strength.

1.3.3 Cone Penetration Test (CPT)

In its simplest form the CPT apparatus consists of a 60 degree cone with a base area of 10 square centimetres attached to the bottom of a rod protected by a casing (See Figure 1.11). The cone is pushed into the soil by either hydraulic or mechanical jacking equipment at a rate of 2 cm/sec. This form of the CPT is referred to as the static CPT. The cone resistance, q_c , is the force required to advance the cone divided by the base area. A refined form of the apparatus (See Figure 1.12) consists of a cone with a friction sleeve in which the point resistance and friction on the outside of the casing can each be measured. The penetrometer tips can be either

- i) mechanical, (such as the Delft mantle and Begemann friction cone (Peck, Hansen and Thornburn, 1974) or
- ii) electrical, (such as the Fugro cylindrical shape, or Delft shape) (Schmertmann, 1975).

Another form of the CPT is the dynamic CPT. This procedure is commonly used in the United States and Canada because it is much easier to conduct. Although the cone is basically the same as for the static CPT the method of driving the cone through the soil media is different. In the dynamic CPT the cone is driven by a 65 kilogram hammer free falling 75 cm. The number of blows to drive the cone through 30 cm is counted.

The static CPT method is considered to be superior to the dynamic CPT method to produce data of quantitative usefulness for design, including the determination of the in situ strength (Schmertmann, 1975). This is because variations in static penetration resistance of a sounding rod, which is pushed or jacked slowly into the soil,

FIG. 1.11 DUTCH CONE PENETRATION TEST

FIG. 1.12 CONE PENETROMETER WITH FRICTION SLEEVE

can be determined with greater accuracy than variations in dynamic resistance.

The CPT does not measure the shear strength directly but measures cone bearing capacity, q_c , and soil-steel friction, f_s , both of which depend on the shear strength. At present, geotechnical engineers use an equation of the form

$$c_u = \frac{q_c - \gamma Z}{N_c} \quad (1)$$

where c_u is the undrained shear strength

γZ is the total overburden pressure

and N_c is the bearing capacity factor appropriate for a deep

circular foundation embedded in an ideal cohesive soil.

This equation is essentially derived from bearing capacity theories similar to those proposed by Terzaghi (1946), Skempton (1948) and Meyerhof (1956). Ladanyi and Eden (1969) and Schmertmann (1975) have indicated that the value of N_c is very dependent on the geometry of the penetrometer, the testing technique and the geological history of the clay. The specific value of N_c to be used in equation 1 is generally calculated for a specific geographical area. N_c is compared to a reference undrained shear strength (ie. undrained shear strength as derived from vane shear tests), thereby correlating N_c with undrained shear strength values as derived from other tests. N_c values reported have ranged from 5 to 7.5 (Schmertmann, 1975). Table 1.2 indicates some of the factors that have been found to influence the value of N_c . Ladanyi and Eden (1969) carried out penetration tests in the sensitive clays in the Ottawa, Ontario area by using a Borros Penetrometer. They found that the N_c value varied from about 5.5 to 8.0. The lower limit of N_c occurred in clays of high sensitivity. Many studies have led to empirical correlations between penetration resistance and compressibility. Most of the correlations, however, have been made using the static cone penetration resistance rather than the dynamic cone resistance which is used more frequently in practice. Moreover all correlations have been developed by correlation with one dimensional settlement of normally consolidated clays. The equation employed in the interpretation of compressibility of soils has the form

TABLE 1.2
SOME VARIABLES THAT INFLUENCE N_c USED IN
THE ESTIMATION OF c_u FROM CPT TESTS

VARIABLE	APPROX. N_c FACTOR POTENTIAL	EFFECTS	INVESTIGATION
CHANGING THE TEST METHOD FOR OBTAINING REFERENCE c_u	2 to 3	BETTER SAMPLING, THINNER VANES USE OF PRESSUREMETER ALL DECREASE N_c	SCHMERTMANN (1975)
CLAY STIFFNESS RATIO = G/c_u	3	INCREASES WITH INCREASING STIFFNESS	VESIC (1972)
RATIO INCREASING/ DECREASING MODULUS E/E_R AT PEAK c_u	3	DECREASING WITH DECREASING RATIO	LADANYI (1967)
EFFECTIVE FRICTION $\tan \phi'$	2 to 3	INCREASING WITH INCREASING ϕ'	JANBU (1974)
K_o OR OCR	3	INCREASING WITH INCREASING K_o OR OCR	JANBU (1974)
SHAPE OF PENETRATION TIP	2	CLAY ADHESION ON MANTLE OF MECHANICAL TIPS INCREASES N_c	AMAR (1975)
	1.2	REDUCED DIAMETER ABOVE CONE CAN DECREASE N_c IN VERY SENSITIVE CLAYS	SCHMERTMANN (1972)
RATE OF PENETRATION	1.2	INCREASING RATE INCREASES N_c	SCHMERTMANN (1972)
METHOD OF PENETRATION	1.2	CONTINUOUS ELECTRICAL TIPS) PENETRATION DECREASES N_c COMPARED TO INCREMENTAL (MECHANICAL TIPS) BECAUSE OF HIGHER PORE PRESSURES	SCHMERTMANN (1975)

$$E_{\text{oed}} = \frac{2.3}{C_c + 1} \sigma_o \quad (2)$$

where $i = \frac{2.3 (1+e) \sigma_o}{C_c + 1}$

σ_o = the effective overburden stress

C_c = the compression index for normally consolidated soils

e = in situ void ratio.

Several values of i as determined from various investigations are given in Table 1.3. Recommended values of i range from about 1.5 to 8.0 for normally consolidated soils, with the higher values associated with the softer, more compressible soils. Due to the complex influence of soil fabric, stress history and sensitivity on compressibility and strength, it would be unreasonable to anticipate that all deposits of a given soil type would give the same value of i . As noted by Meigh and Corbett (1969) good agreement was obtained between E_{oed} determined from laboratory tests and the corresponding E_{oed} value determined from cone penetration tests, when i varied from 8 at a depth of 3 metres to 5 at a depth greater than 15 metres. It is therefore recommended that unless a large amount of data is available for the particular area being investigated the cone penetration test should only be used as an index test and designs should not be based on the results from such tests.

1.3.4 The Vane Shear Test (VST)

The VST is widely used for the determination of undrained shear strength characteristics of undisturbed and remoulded saturated cohesive soils tested under laboratory and field conditions. (Cadling and Odenstad, 1950; Eden and Hamilton, 1956; Anderson and Bjerrum, 1957; Skempton, 1948; Aas, 1965 and 1967; Richardson et al., 1975; Schmertmann, 1975; and Menzies and Mailey, 1976). In general, the vane shear device consists of a vane, a series of torque or connecting rods and a torque application and measuring device. The vane consists of four thin blades

TABLE 1.3

COMPRESSIBILITY AS INDICATED BY STATIC CONE RESISTANCE

$$E_{oed} = \alpha q_c$$

REFERENCE	RELATIONSHIP	SOIL TYPE
BACHELIER & PAREZ (1965)	$\alpha = 7.7$	SOFT CLAY
MEIGH & CORBETT (1969)	$\alpha = 8$ AT DEPTH OF 3 METRES AND DECREASED LINEARLY TO 5 AT A DEPTH OF 15 METRES AND GREATER	SOFT SILTY CLAY
GIELLY ET AL. (1969) SANGLERAT ET AL. (1972)	$q_c < 7 \text{ BARS}^*$ $3 < \alpha < 8$ $7 < q_c < 20 \text{ BARS}$ $2 < \alpha < 5$ $q_c > 20 \text{ BARS}$ $1 < \alpha < 2.5$ $q_c > 20 \text{ BARS}$ $2 < \alpha < 6$	CLAYS OF LOW PLASTICITY HIGHLY PLASTIC SILTS & CLAYS
ROBERTSON & CAMPANELLA (1983) AFTER MITCHELL & GARDENER (1975)	$q_c < 7 \text{ BARS}$ $3 < \alpha < 8$ $7 < q_c < 20 \text{ BARS}$ $2 < \alpha < 5$ $q_c > 20 \text{ BARS}$ $1 < \alpha < 2.5$ $q_c > 20 \text{ BARS}$ $3 < \alpha < 6$ $q_c < 20 \text{ BARS}$ $1 < \alpha < 3$ $q_c < 20 \text{ BARS}$ $2 < \alpha < 6$ $q_c < 12 \text{ BARS}$ $2 < \alpha < 8$ $q_c < 7 \text{ BARS}$	CLAYS OF LOW PLASTICITY (CL) SILTS OF LOW PLASTICITY HIGHLY PLASTIC SILTS AND CLAYS ORGANIC SILT
	$50 < w < 100$ $1.5 < \alpha < 4$ $100 < w < 200$ $1 < \alpha < 1.5$ $w > 200$ $0.4 < \alpha < 1$	PEAT AND ORGANIC CLAY

* 1 BAR = 10 dynes/cm²
= 95.9 x 10³ kN/m²

brazed or welded at right angles on a small circular shaft. The vane blades can be triangular, circular or polygonal in shape but the most widely used blade shape is rectangular. A torque is gradually applied to the upper end of the connecting rods until the soil yields along the surface swept by the boundary of the vanes. An expression for the undrained shear strength, which is assumed to be mobilized at this boundary, is computed in terms of the external torque and the vane dimensions by purely statical equilibrium considerations. The VST can be carried out in a borehole (cased or uncased) or as a penetration device in soft clays. In the latter case (See Figure 1.13), the vane and rods are sometimes encased by a protective outer sheath. The device is pushed into the clay to a depth of approximately 0.5 metres above the depth at which the soil is to be tested. The vane is then pushed out of the protective sheath, the test is carried out and the vane is then returned into its protective sheath. The device is then pushed through the already tested zone into the next zone which is to be tested. The torque is usually applied by a worm gear assembly as shown on Figure 1.13. The rate of angular rotation of 0.1 degrees per second is controlled by the assembly of gears and usually corresponds to one revolution of the worm gear handle per second.

The vane shear device was developed in Sweden and Germany during the late 1920's. About 1947 the Swedish Geotechnical Institute improved the apparatus. In the ensuing period little effort was made to standardize the VST. Several aspects of the VST have now been standardized. These included vanes with

- i) a height to diameter ratio of 2
- ii) a rotation rate of 0.1 degrees per second, and
- iii) an area ratio, A_r , of less than 10 percent.

The area ratio A_r is defined as the cross sectional area of the vane blades divided by the area swept by the blades as the vanes are rotated.

In the 1950's the VST became widely accepted as a reliable method for the in situ determination of the undrained shear strength (Osterberg, 1956; Gibbs, 1956; Fenske, 1956; Hill, 1956;

FIG. 1.13 WORM GEAR ASSEMBLY FOR VANE SHEAR APPARATUS
(AFTER EDEN AND HAMILTON, 1956)

Eden and Hamilton, 1956; and Cadling and Lindskog, 1956). In most cases the undrained shear strength determined from the vane shear test was equal to or greater than the maximum value of the undrained shear strength as determined from unconfined compression tests conducted on samples obtained from piston sampling techniques. Bjerrum (1972), Pilot (1972), Janbu (1975) and others noted that there seemed to be many situations in which undrained shear strength values, obtained from the VST, produced non-conservative stability calculation results. Bjerrum (1972) produced correction factors (See Figure 1.14) which were based on the plasticity index of the clay. These correction factors could then be applied to compute undrained shear strength values to be used in slope stability and bearing capacity calculations. Schmertmann (1975) showed that clay fabric affected the undrained shear strength and may provide a possible explanation for Bjerrum's (1972) development of the correction factors. La Rochelle *et al.* (1974) and Ladd (1973) pointed out non-conservative exceptions to Bjerrum's correction factors showing that the plasticity index was not the only factor which attributes to the misleading safety factors. La Rochelle and Lefebvre (1970) found that the undrained shear strength values determined from the VST were consistently lower than undrained shear strengths derived from triaxial compression tests on block samples. This behaviour was attributed, in part, to progressive failure taking place at the tips of the vane blades.

From the test results obtained by Leblanc (1975), Roy (1975) stated that the difficulty in the interpretation of vane strength data lies in the assumption of a cylindrical failure surface at maximum torque. Leblanc (1975) showed that the maximum torque occurs at a very small amount of angular rotation, less than 0.09 radians. X-ray photographs taken during laboratory tests, showed that a cylindrical failure pattern only developed for rotations in excess of 45 degrees. Arman *et al.* (1975) verified the work of Skempton (1948) by showing that the failure surface produced by the vane blades, was slightly greater than the diameter of the vane. They noted that a 5 percent increase in the diameter would produce a 16 percent reduction in the computed undrained

FIG. 1.14 BJERRUM'S 1973 VANE STRENGTH CORRECTION FACTORS

shear strength values. Figure 1.15 shows how the undrained shear strength decreases as the diameter of the swept area increases. Ladd (1975) and Richardson et al. (1975) point out that the anisotropic behaviour of sensitive clays may be a prominent reason why safety factors computed using results from the VST are unconservatively high. The failure plane for the vane shear test is prescribed by the test itself and does not model a preferential failure mode. Ladd (1975) also points out that the effects of strain rate are believed to be more important with highly plastic clays. Undrained shear strength values measured by an in situ test causing failure in a few minutes may be much greater than the undrained strength that can be mobilized in the field over a period of weeks or months.

As a result of these studies a considerable amount of doubt has been placed on the vane shear test. Schmertmann (1975) believes that the test has outlived its usefulness and eventually should be phased out. Ladd (1975) agrees that because of the uncertainties associated with the test, its use should be suspect and results from the test should be used with a great deal of caution, but does not agree in phasing out the test completely. It is generally believed that the test gives reasonable results in normally consolidated clays and tends to give erroneous results in soils which exhibit a high degree of over consolidation and sensitivity. The vane shear test has several draw backs and a variety of factors that could influence the measured value of the undrained shear strength. It is a very simple test, however, which

- i) is useful as perhaps an indexing test
- ii) can be carried out by an engineer or technician with a limited amount of site investigation experience, and
- iii) the amount of soil disturbance associated with the vane penetration is perhaps lower than that introduced by the pressuremeter or any other self boring device.

Furthermore, it is easy to carry out the vane shear test over a large area in a short time. For this reason it still should be regarded as a viable test which could be used in conjunction with charts similar to those developed by Bjerrum.

FIG. 1.15 DECREASE IN THE UNDRAINED SHEAR STRENGTH AS THE DIAMETER OF THE SWEEPED AREA INCREASES.

The pressuremeter test was developed by Menard in 1950. It is used to determine the stress-strain characteristics of soils tested under plain strain conditions. The pressuremeter developed by Menard consists of three basic components: a probe, a pressure and volume unit, and the connecting tubes (See Figure 1.16). The probe consists of a metal cylinder covered with an inflatable rubber membrane within which three independent cells are located. The three cells consist of two outer guard cells and an inner measuring cell. The guard cells ensure the development of a state of near plane strain around the measuring cell. Usually air or gas is used to pressurize the guard cells and water is used in the measuring cell. In this manner the volume changes of only the measuring cell are measured by reading the volume of water expelled from the volumeter of the control unit. The volumeter consists of a cylindrical reservoir with a graduated transparent tube. The reservoir is pressurized using a pressure reducing valve connected to the compressed gas tank (See Figure 1.16). The tubing from the measuring cell runs co-axially through the tubing of the guard cells in order to reduce the errors in volume readings which would result from the dilation of the tubing connected to the measuring cell.

Before any pressuremeter test can be performed in situ, a calibration curve must be obtained for the membrane stiffness. This is done by conducting a standard test with the probe in the air. In the case of soils with a low shear strength, the calibration curve has to be determined with great care. Roy et al. (1975) have shown that the calibration curve can be altered by the pressure increment used during the calibration procedure as well as the viscosity and temperature of the liquid used in the measuring cell (See Figure 1.17).

From the shape of the pressuremeter curve (See Figure 1.18) three characteristic pressures can be defined. The first stage of the curve represents the rapid volume increase at low pressures to bring the membrane in full contact with the borehole wall.

FIG. 1.16. TYPICAL PRESSUREMETER APPARATUS

	FLUID	TEMPERATURE (°C)	PRESSURE INCREMENT (Pa)
1	WATER	21	0.1×10^5
2	WATER	21	0.25×10^5
3	ANTIFREEZE	21	0.25×10^5
4	ANTIFREEZE	7	0.25×10^5

FIG. 4.17 VARIATION OF THE CALIBRATION CURVES WITH CHANGES IN FLUID, TEMPERATURE AND PRESSURE INCREMENT

(AFTER ROY ET AL., 1975)

FIG. 1.18 TYPICAL PRESSUREMETER CURVE

At this point a pressure, p_f , is reached which should theoretically correspond to the in situ total horizontal stress in the ground. The volume injected into the probe increases linearly with pressure up to a pressure, p_f , which should correspond to the yield stress. The final stage of the curve is the plastic stage. Above the yield pressure the volume increases indicating the development of soil failure around the probe. With increasing pressures, the volume vs. pressure curve tends to an asymptotic limit. The practical limit pressure, p_p , is reached when the initial volume, V_0 , is doubled to $2V_0$.

Prior to the calculation of any properties of the soil the pressuremeter curve must be corrected. First, the hydro-static pressure must be added. This is the fluid pressure exerted by the difference in head between the middle of the probe and the manometer. Second, the pressure corresponding to the stiffness of the membrane covering the probe at the volume at which the pressure is to be determined must be subtracted.

The undrained shear strength, c_u , for the Menard type pressuremeter, is given by the semi empirical formula

$$c_u = \frac{p_f - p_0}{5} \quad (4)$$

where 5 (Amar et al. 1975)

p_f limit pressure

p_0 pressure which corresponds to the in situ total horizontal stress in the soil.

Recently, Ladanyi (1972) derived a method based on an arbitrary non-linear soil behaviour by combining the original theory of Gibson and Anderson (1961) with a strain-displacement relationship. The strain-displacement relationship developed by Ladanyi (1961) related the volumetric strain ($\Delta V/V$) with the radial distance a particle displaces as the cavity of the borehole expands during the pressuremeter test. When this method is applied to the pressuremeter results, it is possible to determine the undrained stress strain behaviour of the cohesive soil up to axial strains of about 20 percent. (See also Selvadurai, 1984). Amar et al. (1975)

determined the undrained shear strength by taking the maximum value of the shear curve which is related to the pressuremeter curve by the equation

$$f(\eta) = \frac{\eta}{2} \frac{dF}{d\eta} (1-\eta) + (2+\eta) \quad (5)$$

where η = radial deformation of the probe $\Delta R/R$

$F(\eta)$ = the expansion curve

$f(\eta)$ = the shear curve

$$dF = d(p_f - p_o)$$

For small radial deformations equation (5) reduces to

$$f(\eta) = \eta \frac{dF}{d\eta} \quad (6)$$

From equation (6) it is seen that the shear curve is indicative of the slope of the pressuremeter expansion curve.

Schmertmann (1975) has shown that a -20 percent error in p_f would produce a +40 percent error in the undrained shear strength, (c_u). Roy et al. (1975) indicate that the major difficulty in the methods proposed by Ladanyi (1972) and Amar et al. (1975) resides in the numerical differentiation of an experimental curve which is very sensitive to the scatter of experimental points. They reported that the peak and ultimate strengths calculated on the basis of the pressuremeter test were respectively 3 and 2 times the vane shear strength. The peak strength was at least 50 percent more than the undrained shear strength measured by any other method. When Menard's theory was used to calculate undrained strengths, the values obtained were mid way between the vane strengths and the ultimate strengths.

The pressuremeter modulus, E_p , is determined from the pseudo-elastic phase of the corrected curve between the pressures p_o and p_f . It is usually expressed as

$$E_p = \frac{2(1-\nu)(V_c + V_m)}{\Delta V} \frac{\Delta p}{\Delta V} \quad (7)$$

where V_c = the initial volume of the central measuring cell of the probe with the voluneter at zero

V_m = the volume of water injected under the pressure

$$\Delta p = \frac{p_o + p_f}{2}$$

ν = Poisson's ratio

$\frac{dV}{dV}$ the slope of the pressuremeter curve
between p_0 and p_f .

Tavenas *et al.* (1974), Shields and Bauer (1975) and Roy *et al.* (1975), reported that borehole disturbance has a great effect on the pressuremeter modulus. In order to study the extent to which borehole disturbances and testing techniques alter the pressuremeter modulus, Roy *et al.* (1975) carried out pressuremeter tests on four sites in Quebec. Three of these sites were mainly composed of soft sensitive clay. The fourth site was a relatively stiff and insensitive soil deposit. Three different probe sizes were used and three different testing techniques were employed. The three techniques were: the sampling hole technique, the auger technique and the driving technique.

In the sample hole technique a NGI fixed piston sampler was first used to obtain a sample and then the pressuremeter test is performed in the cavity left by the sampler. Although this technique is suitable in some stiff clays, it produces considerable disturbances in soft clays. Neither the value of p_0 nor the values of the pressuremeter modulus or the undrained shear strength could be determined in this test. Only limit pressure data was obtained by using this technique. Even then the results were suspect because the test series gave the lowest value when compared to all other values.

In the auger hole technique a specially designed auger is used to make the borehole. This auger incorporates a system of mud injection and is fed into the ground at a rate of 25 cm/min with a rate of rotation of 100 rpm. The density of the mud varied from 1.04 gm/cc to 1.09 gm/cc. Two factors influenced the results. They were:

- i) the density of the mud and
- ii) the parameter, Q , which represents the ratio between the diameter of the borehole and the diameter of the probe.

As the density of the mud increased the value of p_0 increased. It would appear that as the probe expands it must displace the mud from the side of the borehole. When $Q = 1$, p_0 was not affected whereas when $Q < 1$, p_0 could not be determined. When $Q > 1$, the pressuremeter

modulus, E_p , was quite consistent but less than E_1 values derived from the tests where $Q = 1$. The overall difference in the pressuremeter modulus, E_p , when $Q < 1$, $Q = 1$, and $Q > 1$ ranged from 45 to 66 percent.

In the driving technique the probe is simply pushed into the clay mass. When this method was used, p_0 could not be measured whether there was a relaxation time or not. No indication, however, was given as to the amount of relaxation time which was allowed in these tests. The pressuremeter modulus, E_p , could not be measured if no relaxation time was allowed after the driving of the probe. When time was allowed for relaxation between the driving of the probe and the beginning of the test, the soil around the probe consolidated as was indicated by much higher values of E_p . These values were considered unrepresentative of the true behaviour of the soil in its natural state. The limit pressure in this case was lower than that obtained by the auger hole technique.

The results of this study indicate that soil disturbance affects all the characteristics of the pressure-volume curve. The sample hold technique and the driving technique both produce unreliable results when used in soft sensitive clays. The pressuremeter modulus, E_p , is very dependent of the ratio Q . The value of E_p can vary between 45 and 66 percent depending on the value of the ratio Q . Recent results by Law and Eden (1980) also indicate the nature of soil disturbance that is associated with the use of self boring pressuremeter such as the Camkometer or the autotest in the testing of soft sensitive clays.

As noted previously the pressuremeter undrained shear strength and pressuremeter modulus are quite often considerably greater than values obtained by any other test. Ladd *et al.* (1977) noted that empirical correlation factors have to be applied (See eg. Baguelin *et al.* 1978) for design of both shallow and pile foundations. There appears to be a great deal of uncertainty involved when pressuremeter test results and empirical design formulas are used in design problems in soils which differ in structure from the soil in which the correlations were first determined.

In the preceding section, several testing methods and procedures have been presented. Each method prescribes in its own procedure for the determination of the undrained modulus of deformation, E_u , and/or the undrained shear strength, c_u , of the soil. Tables 1.4 through 1.7 have been prepared as a means for comparing the results from various testing procedures. Table 1.4 shows normalized results of the undrained shear strength as determined by the various testing methods. In each case, the value of c_u has been normalized to c_u as determined from the vane shear tests. The investigations of Eden (1970), Raymond et al. (1971) and La Rochelle and Lefebvre (1974) clearly indicate that sample disturbance has a very significant effect on the value of c_u . Because of the large variation in N_c values, Table 1.4 also indicates the importance of a clear knowledge of the local soil conditions and its behaviour.

The variation of the undrained modulus, E_u , indicated by the various test methods are shown in Table 1.5. The values in Table 1.5 have been normalized by E_u as derived by either the unconfined compression or the unconsolidated undrained triaxial test. The low values of E_u determined from the pressuremeter test, reported by Tavenas et al. (1974) and Shields and Bauer (1975) should also be noted. In both investigations it was reported that borehole disturbance was a major cause for the low values. It is particularly interesting to note the very large range in the values obtained from consolidated isotropically undrained, CIU, tests. It has been suggested by Ladd (1964) and Raymond et al. (1974) that the results of CIU tests are less sensitive to disturbance than the results from unconfined compression and unconsolidated undrained tests. This, however, may only account for very small deviations and not for the large deviations as noted by Simons (1957) and Ladd (1964). Ladd (1964) concluded that the increase in effective stress is primarily responsible for the large increase in the undrained modulus when tests conducted using unconsolidated undrained triaxial tests and consolidated undrained triaxial tests of normally consolidated clays are compared.

TABLE 1.4

LEGEND

SPT	STANDARD PENETRATION TEST
CPT	CONE PENETRATION TEST
PMT	PRESSUREMETER TEST
U	UNCONFINED COMPRESSION
UU	UNCONSOLIDATED UNDRAINED TRIAXIAL
CIU	CONSOLIDATED ISOTROPICALLY UNDRAINED TRIAXIAL

TABLE 1.4

NORMALIZED UNDRAINED SHEAR STRENGTH ($\bar{c}_u$) AS INDICATED BY VARIOUS TESTING METHODS

$$\bar{c}_u = (c_u)_{\text{test}} / (c_u)_{\text{vane test}}$$

AUTHOR	SOIL & SITE	SPT	CPT	PMT	UNDIST. SAMPLES		REMARKS
					U	UU CIU	
ANDERSON & BJERRUM (1957)	OSLO, NORWAY, soft and sensitive clay				1.12		Thin walled fixed piston sampler
EDEN & HAMILTON (1956)	OTTAWA, CANADA sensitive clay				0.5- 1.1		71 mm piston sampler
HALL (1963)	SAN FRANCISCO BAY AREA, soft clay				0.7- 0.9		Swedish foil sampler 68 mm dia.
CRAWFORD & EDEN (1965)	OTTAWA, CANADA sensitive clay				0.5 - 1.0		Thin walled piston samples and block samples.
LADANYI & EDEN (1969)	GLOUCESTER TEST FILL OTTAWA, CANADA soft sensitive clay			1.0- 1.5			$N_c = 6.73$
	NATIONAL RESEARCH COUNCIL, OTTAWA			0.9- 1.0			$N_c = 6.73$

TABLE 1.4

NORMALIZED UNDRAINED SHEAR STRENGTH ($\bar{c}_u$) AS INDICATED BY VARIOUS TESTING METHODS

$$\bar{c}_u = (c_u)_{\text{test}} / (c_u)_{\text{vane test}} \quad (\text{cont'd})$$

AUTHOR	SOIL & SITE	UNDIST. SAMPLES				REMARKS
		SPT	CPT	PMT	U UU CIU	
BOZOUK (1970)	GLOUCESTER TEST FILL. OTTAWA, CANADA soft sensitive clay				1.4	124 mm dia. piston sampler
					1.2	54 mm dia. piston sampler
EDEN (1970)	OTTAWA, CANADA OC: sensitive clay				1.5	Block samples
					0.83	124 mm dia. Osterberg
					0.82	54 mm dia. NGI
					0.79	50 mm dia. Swedish piston
					0.63	63 mm dia. Swedish foil sampler
LAROCHELLE & LEFEBVRE (1970)	ST.-LOUIS, QUEBEC sensitive clay				1.6	Block samples
					1.0	54 mm dia. NGI piston sampler
RAYMOND ET AL. (1971)	OTTAWA, CANADA soft sensitive clay				2.5	Block samples
					1.5	127 mm dia. Osterberg
					1.1	50 mm dia. Swedish piston
					1.0	50 mm dia. piston sampler sharpened cutting edge
					0.98	50 mm dia. piston sampler
					0.80	50 mm dia. open Shelby tube

TABLE 1.4

NORMALIZED UNDRAINED SHEAR STRENGTH ($\bar{c}_u$) AS INDICATED BY VARIOUS TESTING METHODS

$$\bar{c}_u = (c_u)_{\text{test}} / (c_u)_{\text{vane test}} \quad (\text{cont'd})$$

AUTHOR	SOIL & SITE	SPT	CPT	PMT	UNDIST. SAMPLES			REMARKS
					U	UU	CIU	
RAYMOND (1973)	TIMISKAMING, ONT., CANADA soft clay				0.8			70 mm dia. Shelby with fixed piston
TÁVERNAS ET AL. (1974)	SAINT ALBAN QUEBEC	1			1.3	1.8		$N_c = 6.0$
AMAR ET AL. (1975)	CRAN PROVINS BOSSE GALIN FRANCE soft clays			1.2 1.25 1.85				Autoforeur probe
ARMAN ET AL. (1975)	LAPLACE soft clay				0.54			71 mm dia. thin walled sampler coefficient of correlation=0.92
LUNNE (1975)	Norwegian NC. clays	1						N_c varies from 13 to 20

PLATE 2.1 DISTURBANCE CAUSED BY PENETRATION ACTION OF
MODEL SCREW PLATE DEVICE

TABLE 1.5

UNDRAINED MODULUS $\bar{E}_u$ AS INDICATED BY VARIOUS TESTING METHODS

$$\bar{E}_u = (E_u)_{\text{test}} / (E_u)_{\text{unconfined compression test}}$$

AUTHOR	SITE & SOIL	PMT	CIU	PLATE	FOOTING	REMARKS
SIMONS (1957)	OSLO, NORWAY NC CLAYS		9.0 to 11.0		2.0 to 5.0	SETTLEMENT OF TWO STRUCTURES
LADD (1964)	NC CLAYS		5.0 to 6.0			
SODERMAN ET AL. (1968)	FIRM GREY TILL OC		1.5 2.2			110 mm TUBE SAMPLES BLOCK SAMPLES PLATE TESTS
BOZOUK (1970)	OTTAWA SENSITIVE CLAY			2.7	0.96	E_u MAX. VALUE OBTAINED FROM BLOCK SAMPLES BACKCALCULATED FROM EXCAVATION HEAVE; $v = 0.4$

TABLE 1.5

UNDRAINED MODULUS $\bar{E}_u$ AS INDICATED BY VARIOUS TESTING METHODS

$$\bar{E}_u = (E_u)_{\text{test}} / (E_u)_{\text{unconfined compression test}} \quad (\text{cont'd})$$

AUTHOR	SITE & SOIL	PMT	CIU	PLATE	FOOTING	REMARKS
MILOVIC (1970)	ST. SIMON NICOLET QUEBEC		1.0 1.6	1.3 1.3		LAB RESULTS BASED ON BLOCK SAMPLES
TAVENAS ET AL. (1974)	SAINT ALBAN QUEBEC	0.2	2.0			
SHEILDS AND BAUER (1975)	OTTAWA LEDA CLAY	1.0	2.5	3.0	3.0	
MARSLAND (1975)	LONDON OC STIFF FISSURED CLAY			5.0		
MARSLAND & RANDOLPH (1977)	LONDON OC STIFF FISSURED CLAY	0.66				COMPARED TO E_u FROM PLATE TESTS 865 mm DIA. PLATES

PMT PRESSUREMETER TEST
CIU CONSOLIDATED ISOTROPICALLY UNDRAINED TRIAXIAL TESTS

Table 1.6 shows the results obtained by Milovan (1973). In this table each value of E_u has been normalized by the E_u value obtained from unconfined compression tests. These results indicate that sample disturbance and sampling technique have a very large effect on E_u . Although the results derived from consolidated undrained triaxial tests are more consistent, sample disturbance still causes a substantial decrease in the modulus value.

Table 1.7 shows the results obtained by Roy (1975). Comparisons are made between the ratio of the undrained modulus to the undrained shear strength of a sensitive clay obtained by:

- i) various in situ tests
- ii) laboratory tests on high quality samples, and
- iii) back analysis of the observed immediate settlements of three test embankments in Saint - Alban, Quebec.

In this study a self boring pressuremeter (See Wroth 1975, Wroth and Windle 1975) was used as well as the Menard type pressuremeter. This pressuremeter has a grinding bit inside a cutting shoe at the front of the unit. The cutting shoe and grinding bit cut a hole for the measuring unit. The results indicate that the self boring pressuremeter is far superior to the standard Menard type pressuremeter. The standard Menard type pressuremeter results were greatly influenced by the technique of installing the probe. The results from the self boring pressuremeter were closest to the observed values back calculated from the embankment performance. The self boring pressuremeter, however, over estimated the embankment performance. Results obtained from laboratory tests on undisturbed samples retrieved by a 200 mm diameter thin walled Sherbrooke sampler were far in excess of those from the observed settlement of the test embankment.

Results from various laboratory and in situ testing methods have been presented in this chapter. The large range of results obtained by these methods are shown in Tables 1.4 through 1.7. It is evident that no one method can yield acceptable results for all types of cohesive soils. Shear vane results provide reliable information for the mechanical behaviour of normally consolidated clays. Due to the occurrence of possible progressive failure, vane shear tests tend to give erroneous results in over

TABLE 1.6

NORMALIZED UNDRAINED MODULUS RESULTS

(AFTER MILOVIC, 1970)

$$\bar{E}_u = E_{u \text{ test}} / E_{u \text{ unconfined compression block sample}}$$

SITE	SAMPLE METHOD	UNCONFINED COMPRESSION	CONSOLIDATED UNDRAINED
ST. SIMON	BLOCK	1.00	0.92
QUEBEC $S_t = 10$	PISTON	0.72	0.77
	SHELBY	0.40	0.66
NICOLET	BLOCK	1.00	1.60
QUEBEC $S_t = 15$	PISTON	0.76	1.30
	SHELBY	0.26	0.71

TABLE 1.7
COMPARISON OF THE RATIO OF UNDRAINED MODULUS E_u
TO THE UNDRAINED SHEAR STRENGTH (c_u) OF THE ST. ALBAN CLAY

TESTING METHOD	E_u/c_u vane	
	0 to 6 m	6 m to 9 m
MENARD PRESSUREMETER		
PROBE < HOLE	55	70
PROBE = HOLE	95	170
PROBE > HOLE	175	215
PROBE PUSHED & RELAXED	250	450
SELF - BORING PRESSUREMETER		
INITIAL TANGENT MODULUS	840	1050
SECANT MODULUS	500	720
STATIC CONE PENETRATION		
COMBINED WITH VANE	450	1500
LABORATORY TESTS		
CK _{0U} ON 3" SAMPLES	380	600
CK _{0U} ON 8" SAMPLES	900	1900
PLANE STRAIN ON 8"	900	1500
TEST EMBANKMENT		
MODULUS BACKFIGURED FROM OBSERVATIONS	600	1000

(AFTER ROY ET AL., 1970)

Consolidated clays. Standard penetration tests in sensitive clays (exhibiting a cemented structure) produces undrained shear strength values that are too low unless correction factors are applied. The cone penetration tests is usually calibrated using the vane shear test. Consolidated undrained triaxial tests on normally consolidated clays yield modulus values that are much higher than values derived from the observation of settlements of structures, whereas unconfined compression and unconsolidated undrained triaxial tests give modulus values that are too low. Borehole disturbance significantly effects results obtained from the standard Menard type pressuremeter. Finally, results from the self boring pressuremeter are usually considerably higher than results from other testing methods and observed settlements of embankments (Tavenas, 1975).

CHAPTER TWO

Extensive deposits of glacially derived marine and lacustrine silts and clays are encountered in regions of Eastern Canada and in the Scandinavian countries. The accurate estimation of the mechanical properties of these sensitive soil deposits is in itself an important aspect of geotechnical engineering. It is generally recognized that routine sampling techniques introduce considerable disturbance errors in the laboratory estimates for the strength and deformability characteristics of such sensitive cohesive soils. Detailed accounts of the factors which contribute to such disturbance effects are documented in the researches of Crawford and Burn (1962), Bjerrum (1964), Soderman *et al.* (1968), Hanna and Adams (1968), Eden (1970), LaRoche and Lefebvre (1970) and Raymond *et al.* (1971). Eden (1970) suggests the use of block samples as a means for minimizing the degree of sample disturbance. With this factor in mind efforts have been made (Domaschuk, 1977; Lefebvre and Poulin, 1979) to develop techniques of obtaining large diameter block samples directly from boreholes. The development of techniques is still at its early stages and the procedures involved are by no means routine. In any event the degree of disturbance associated with these "undisturbed" sample retrieval procedures constitutes an unknown factor. Therefore, where possible, recourse must be made to calibrate the laboratory parameters with equivalent results obtained from in situ tests. The self boring pressuremeter and the screw plate test are two such in situ tests which have potential application in the estimation of deformability and strength characteristics of soft sensitive clays. Accounts of pressuremeter testing of Champlain Sea clays are reported by Roy *et al.* (1975) and Eden and Law (1980). Similarly, the screw plate testing of Scandinavian soft clays are reported by Janbu and Senneset (1973), Schwab (1976), Schwab and Broms (1977) and Berg and Olssen (1978).

The performance of the screw plate test can be reviewed in the light of two theories of material behaviour:

- i) linear elasticity
- ii) ideal plasticity

The linear elastic modelling is used to evaluate the undrained load displacement response of the screw plate test. The ideal plasticity modelling is used to estimate the undrained failure response of the screw plate test. The theoretical developments of both these models will be discussed in detail in the ensuing sections.

The theoretical problem concerning the mechanics of the screw plate test can be formulated as an axisymmetric problem in the classical theory of elasticity. The model chosen is that of a circular plate like region embedded in a homogeneous isotropic elastic medium. The plate like region is subjected to an axial load. In the theoretical assessment of the deformability characteristics consideration has been given to the following factors:

- i) soil medium
- ii) the test plate
- iii) the soil-plate interface, and
- iv) soil disturbance

i) Soil Medium

Selvadurai (1978) indicated that when the test plate is located at a depth greater than about 6 to 8 times the plate diameter, the effects of the free boundary on the deflection of the embedded plate is almost negligible. Since the screw plate test is usually performed at a depth substantially greater than the plate diameter, it may be assumed that the screw plate is located in a homogeneous isotropic elastic medium of infinite extent.

ii) The Test Plate

The circular area over which the applied loads are transmitted to the cohesive soil may be assumed to be completely flexible, infinitely rigid, or possessing finite flexural rigidity characteristics commensurate with the plate dimensions and its elastic characteristics. The screw plate in the above cases has been

idealized as a flat circular plate. The effect of the helical nature of the screw plate, however, may also have an influence on the effective plate dimensions. In this instance the behaviour of the screw plate may be approximated by that of a rigid spheroidal region embedded in an elastic medium.

iii) The Soil-plate Interface

The interface between the soil medium and the screw plate can exhibit a variety of interface phenomena. It can range from complete adhesion (or bonded) to the completely frictionless (or smooth) conditions. Some form of finite adhesion may occupy an intermediate position. Since the analysis of the latter case is mathematically complex and additional information is required regarding the cohesive properties of the soil, the results corresponding to perfectly bonded and smooth interfaces provide probable limits for the assessment of the screw plate test results. It is also possible to visualize a situation wherein bonding exists only on the underside of the plate and the soil detaches from the top surface of the plate creating a void above the plate region.

iv) Soil Disturbance

The extent of soil disturbance associated with the introduction of any type of testing device into a soil medium can be quantified only in rare cases. The screw plate test is no exception. It is, however, conceivable that the soil disturbance associated with the penetration action of the screw plate and the loading shaft would perhaps be no more severe than the penetration action of either the Camkometer or similar device in an otherwise unlined borehole. Laboratory studies (Selvadurai, Nicholas and Moore, 1979) conducted with a model screw plate inserted into a stratum of compacted kaolin indicates that the structure of the soil medium is not significantly altered by the penetration action of the testing device (See Plate 2.1). Once the screw plate is inserted to the required depth, the loading will examine a region of soil which is relatively unaffected by the driving action. In the extreme case, it is assumed that the extent of soil disturbance is such that the cylindrical column of soil, along the driven path

PLATE 2.1 DISTURBANCE CAUSED BY PENETRATION ACTION OF
MODEL SCREW PLATE DEVICE

of the screw plate, for $\lambda = 0$ (zero stiffness). In this instance, the screw plate test degenerates to a plate load test carried out at the base of the unlined borehole (See eq. Burland (1960), Marsland (1977), Ellis and Turner (1978) and Carrier and Christian (1978)).

In the following sections some of the solutions that may be applicable to the theoretical assessment of the insitu deformability characteristics of a soil medium by the screw plate test will be examined. The solutions have been presented in the non-dimensional form

$$\frac{w}{pa/E} = \text{CONSTANT}$$

- where w the settlement of the plate at the loaded pressure
 p the average pressure over the area of the plate
 a the radius of the plate
 E the undrained Young's Modulus
 λ is a constant which depends on Poisson's ratio and the geometry of the problem.

2.2.1 Uniform Flexible Load by a Definite Elastic Medium

The model proposed here, represents a plate of infinitesimal thickness and zero rigidity which is in bonded contact with the surrounding soil medium (See Figure 2.1). The solution to this problem can be obtained by a simple integration of Kelvin's (1880) solution for the internal loading of an infinite elastic medium by a concentrated force directed along the axis of symmetry. Solutions are available for the central, edge and average displacement of the loaded area.

a) Central displacement

$$\frac{w_o}{pa/E} = \frac{(3 - 4\nu)(1 + \nu)}{4(1 - \nu)} = \lambda_o \quad (9)$$

b) Edge displacement

$$\frac{w_a}{pa/E} = \frac{2(3 - 4\nu)(1 + \nu)}{\pi 4(1 - \nu)} = \lambda_a \quad (10)$$

FIG. 2.1 UNIFORM FLEXIBLE CIRCULAR LOAD

$$\frac{w_m}{pa/E} = \frac{0.64 (3 - 4\nu) (1 + \nu)}{4(1 - \nu)} = \dots \quad (11)$$

where: w_o , w_a and w_m represent the central, edge and average displacement of the plate respectively.

E Young's modulus

ν Poisson's ratio

p the average stress over the loaded area

a the radius of the loaded area.

As in the previous case, the plate is considered to be of infinitesimal thickness and in bonded contact with the soil medium. The plate in this case, however, is considered to be completely rigid (See Figure 2.2). The exact analytical solution for this particular problem was obtained by Collins (1962). He employed integral equation methods in connection with the problem of the penny-shaped inclusion embedded in an infinite elastic solid. Kanwal and Sharma (1976) employed singularity methods to obtain the same solution. Selvadurai (1976) used direct spheroidal harmonic function techniques to obtain this solution as a limiting case for the oblate spheroidal anchor problem. The rigid displacement, w_{rc} , of a fully bonded circular plate is given by

$$\frac{w_{rc}}{pa/E} = \frac{1(3 - 4\nu)(1 + \nu)}{16(1 - \nu)} \quad (12)$$

where $p = (P/\pi a^2)$ is the average stress applied over the plate area.

2.2.3 Rigid Circular Plate Smoothly Embedded and in Partial Contact with the Elastic Medium

In this formulation, the contact between the rigid plate and elastic medium is assumed to be frictionless and, in addition, the elastic medium is loaded by the contact which is present only on the bottom of the circular plate. A void, or separation, occurs on the top of the test plate as the plate settles within

FIG. 2.2 FULLY BONDED RIGID CIRCULAR PLATE

2

The following expression for w_{rf} is obtained from the solution presented by Hunter and Jamieson (1974). This solution was obtained in accordance with the theory of a rigid circular disc ground under tension in an elastic soil either with adhesion or without adhesion.

$$w_{rf} = \frac{P}{4\pi E} \frac{(3-4\nu) + (1-\nu/4)(1-2\nu)^2}{(1-\nu)} \quad (13)$$

As in the previous model, separation is assumed to take place at the upper surface of the test plate. In this case, however, the interface between the rigid plate and elastic medium is assumed to be in bonded contact on the underside of the test plate (See Figure 2.4). Keer (1975) has considered the boundary value problem related to a rigid circular plate which is in bonded contact on only one of its faces. The load-displacement (w_{rf}) relationship for the plate inclusion has been evaluated in exact closed form.

$$w_{rf} = \frac{P}{4\pi E} \frac{(1-\nu)(3-4\nu)}{1 + \log_e(3-4\nu)} \quad (14)$$

2.1.1 Flexible Circular Plate Embedded in Bonded Contact with the Elastic Medium

In this particular idealization of the screw plate test the circular plate is assumed to possess a finite flexural rigidity ranging from the limits of completely flexible to completely rigid. The plate is assumed to be of thickness h , which is much less than the diameter of the plate. Also, the loaded area of the test plate can vary from that of a point load directed along the vertical axis to a uniformly distributed load over the entire area of the test plate (See Figure 2.5). Selvadurai (1979) made use of a variational formulation wherein the deflection of the plate was represented by a second degree curve. The central deflection (w_{of}) of the circular plate is given by

FIG. 2.3 SMOOTHLY EMBEDDED PARTIALLY SEPARATED
RIGID CIRCULAR PLATE

FIG. 2.4 BONDED PARTIALLY SEPARATED RIGID CIRCULAR PLATE

FIG. 2.5. BONDED THIN FLEXIBLE PLATE

$$\frac{w_{of}}{pa/E} = \frac{\pi(3-4\nu)(1+\nu)(144-60\xi^2+90R)}{16(1-\nu)(64+90R)} = \lambda_{of} \quad (15)$$

where R is the relative rigidity of the soil-plate system defined by

$$R = \frac{\pi(3-4\nu)(1+\nu)}{12(1-\nu_p)(1-\nu)} \frac{E_p h^3}{E a^3} \quad (16)$$

a is the radius of the circular area over which the axial load is applied

p is the average stress on the entire plate area

h is the thickness of the plate

E is Young's modulus of the plate material, and

ν is Poisson's ratio of the plate material.

As R approaches infinity, solution (15) becomes identical to that of a rigid circular plate embedded in complete bonded contact with an isotropic elastic medium (equation 12). In the particular case when ξ approaches 1 and R approaches zero, the interaction problem is reduced to that of the internal loading of an infinite elastic medium by a circular area of radius " a " and stress intensity p . For this case, the central deflection, w_o , as determined from equation 15 is given by

$$\frac{w_o}{pa/E} = \frac{\pi(3-4\nu)(1+\nu)}{16(1-\nu)} \frac{21}{16} \quad (17)$$

The exact solution, as obtained in section 2.2.1, may be rewritten as

$$\frac{w_o}{pa/E} = \frac{\pi(3-4\nu)(1+\nu)}{16(1-\nu)} \frac{4}{\pi} \quad (18)$$

A comparison of equations 17 and 18 indicates that the energy estimate for the central deflection of the embedded flexible circular foundation overpredicts the exact solution only by approximately 3 percent. Equation 17 may therefore be used to obtain very close approximations of load-displacement relationships for embedded plates of any relative rigidity. Figure 2,6 shows the variation of the central deflection of an embedded circular foundation with relative rigidity values ranging from 1 to 1000. The influence of the value of ξ is also shown. When the relative rigidity is approximately 200 or greater, the foundation or plate may be considered rigid.

$$w_0 = \bar{w}_0 P(3 - 4\nu)/32 G_s(1 + \nu) a$$

$$R = \frac{\pi(3 - 4\nu)(1 + \nu)}{12(1 - \nu p)} \frac{E_p}{E} \left(\frac{h}{a}\right)^3$$

FIG. 2.6 THE EFFECTS OF RELATIVE RIGIDITY ON THE CENTRAL DEFLECTION OF AN EMBEDDED CIRCULAR FOUNDATION

2.2.5 Rigid Spheroidal Inclusion in Bonded Contact with an Elastic Medium

In the previous idealizations of the screw plate test it was assumed that the screw plate occupied a flat circular region within the soil medium. The helical screw plate, however, has finite dimensions and may be assumed to occupy a space approximating an oblate spheroidal inclusion. In this particular idealization, it is assumed that the helical plate and soil contained within the single cycle of the helix acts as a rigid oblate spheroidal inclusion which is in bonded contact with the surrounding soil medium. In this instance the half pitch of the screw plate may be set equal to the minor axis dimension, b , and the plate diameter may be set equal to the major axis dimension, $2a$, of the oblate spheroid (See Figure 2.7). Selvadurai (1976) developed a closed form analytical solution for this problem by using direct spheroidal harmonic function techniques. The rigid deflection is given by

$$\frac{w_s}{pa/E} = \frac{(1+\nu) \{ \sinh \alpha - \left[\frac{\sinh^2 \alpha - (3-4\nu)}{\sqrt{1-\eta^2}} \right] \chi \}}{8(1-\nu) \sqrt{1-\eta^2}} = \lambda_s \quad (19)$$

$$\text{where } \sinh \alpha = \frac{\eta}{\sqrt{1-\eta^2}}$$

$$\chi = \cot^{-1} (\sinh \alpha)$$

$$\eta = b/a$$

$$b = \text{half the length of the minor axis}$$

$$a = \text{half the length of the major axis}$$

Figure 2.8 shows the range of λ_s values obtained from (19) for η values ranging from 0 to 1 and for different values of Poisson's ratio.

2.2.7 Deep Borehole in an Elastic Medium Loaded by a Uniform Flexible Load

The settlement at the base of a deep borehole subjected to a uniform flexible load (See Figure 2.9) has been evaluated by Burland (1969), Christian and Carrier (1978), and Pells and Turner (1978) using the finite element method. These investigations give the following estimates for the central deflection (w_{obl}) and the average deflection (w_{abr}).

— isotropic elastic medium
of infinite extent

FIG. 2.7 RIGID SPHEROIDAL INCLUSION IN BONDED CONTACT WITH
AN ISOTROPIC ELASTIC MEDIUM

FIG. 2.8 VARIATION OF λ_s WITH η

FIG. 2.9 DEEP BOREHOLE SUBJECTED TO A UNIFORM FLEXIBLE CIRCULAR LOAD

$$\frac{w_{obf}}{p_a/E} = \lambda_{obf} \quad (\text{Figure 2.10a}) \quad (20)$$

$$\frac{w_{mbf}}{p_a/E} = \lambda_{mbf} \quad (\text{Figure 2.10b}) \quad (21)$$

As shown in Figure 2.10a there are small discrepancies between the results of Burland (1969) and Pells and Turner (1978) for the solutions associated with the central deflection of the plate. These discrepancies may be due to one of several factors which include:

- i) differences in the computing techniques of the finite element programs used in the different studies
- ii) differences in the distance from the plate to the rigid boundary
- iii) the boundary conditions at the edge of the finite element meshes, and
- iv) the configuration and refinement of the finite element mesh especially around the base of the deep borehole.

2.2.2 Rigid Plate in an Elastic Medium Indented by a Rigid Circular Plate

In this case the finite element technique is used to examine the deflection of a rigid circular plate placed in smooth contact with the base of the borehole (See Figure 2.11). The assumption of smooth contact at the interface would evidently be consistent if substantial softening of the soil occurs along the driven path of the screw plate. The deflection of the rigid plate (w_{rb}) is given by

$$\frac{w_{rb}}{p_a/E} = \lambda_{rb} \quad (22)$$

where λ_{rb} may be obtained from Figure 2.12 which shows the results obtained by Pells and Turner (1978) for the given problem.

2.3 Comparison of λ Values

Figure 2.13 shows a comparison of the λ values, as calculated from the previous sections, for various values of Poisson's ratio. Particular values for the oblate spheroidal inclusion, λ_s , and the flexible circular plate, λ_{of} , have been calculated based on the

FIG. 2.10a CENTRAL DEFLECTION OF A UNIFORM
LOADING AT THE BASE OF A BOREHOLE

FIG. 2.10b AVERAGE DEFLECTION OF A UNIFORM
LOADING AT THE BASE OF A BOREHOLE

FIG. 2.11 'DEEP BOREHOLE RIGID CIRCULAR PLATE'

FIG. 2.12 DEFLECTION OF A RIGID PLATE IN SMOOTH CONTACT AT THE BASE OF A BOREHOLE

FIG. 2.13 VARIATION OF λ VALUES
WITH POISSON'S RATIO

following assumptions. The screw plate was assumed to have an effective η value of 0.25 in the oblate spheroid problem. The half pitch of the screw plate used in the in situ studies was set equal to the minor axis dimension of the spheroid, b . The diameter of the screw plate was set equal to the major axis dimension $2a$.

For the flexible circular plate problem the relative rigidity of the soil-plate system was calculated to be approximately 10. The ξ value used conforms with that of the screw plate used in the field studies. The rigidity of the soil-plate interface was calculated assuming a plate diameter of 29.5 cm (11.625 inches), a plate thickness of 1 cm (0.375 inches), a Young's modulus of the soil of 8.5×10^3 kN/m², a Poisson's ratio of the plate material of 0.3 and a Young's modulus of the plate of 216×10^6 kN/m².

Table 2.1 is a catalogue of λ values based on undrained conditions (i.e. $\nu = 0.5$). The λ values of Table 2.1 show a range of values from 0.477 to 0.82. Some of the values, however, are of limited interest and are quoted here only for purposes of comparison and reference. The λ values associated with the internal loading of an elastic medium and that of a uniform flexible load at the base of a borehole do not represent any physical testing condition. Testing plates commonly used in deep plate load tests are of sufficient thickness to ensure at least moderate values of relative rigidity.

For undrained elastic behaviour of the soil medium, the range of λ values applicable to the screw plate test vary from $\lambda_{rs} = 0.75$ to $\lambda_{rb} = 0.54$. Each result relates to physically admissible interpretations of the screw plate test. The lower bound, however, represents a situation in which there is a complete loss of stiffness along the driven path of the screw plate. Based on the experience gained by using the screw plate test in soft clays, this is perhaps too strict a restriction on the screw plate performance. The screw plate does not completely remould the entire volume of soil encountered along the driven path. It is foreseeable that if the screw plate is power driven into a soft sensitive clay, the behaviour of the screw plate test load-displacement relationship might then approximate that of a plate

TABLE 2.1

VALUES USED FOR THE ESTIMATION OF THE UNDRAINED MODULUS
FROM SCREW PLATE TESTS

SOLUTION	$\frac{w}{pa/E_u}$	REFERENCE
(1)	0.750	Kelvin (1890)
(2)	0.477	
(3)	0.630	
(4)	0.589	Collins (1962), Kanwal and Sharma (1976), Selvadurai (1976)
(5)	0.750	Hunter and Gamblen (1974)
(6)	0.750	Keer (1975)
(7)	0.635	Selvadurai (1979)
(8)	0.576	Selvadurai (1976)
(9)	0.820	Christian and Carrier III (1978)
(10)	0.740	Pells and Turner (1978)
(11)	0.540	

SOLUTION	REMARKS
(1)	central displacement of uniform load
(2)	edge displacement of uniform load
(3)	average displacement of uniform load
(4)	displacement of fully bonded rigid disc
(5)	displacement of a smoothly embedded rigid disc
(6)	displacement of a partially bonded rigid disc
(7)	central displacement of flexible disc *
(8)	displacement of rigid spheroidal region +
(9)	central displacement of uniform load at base of deep bore- hole
(10)	average displacement of uniform load at base of deep bore- hole
(11)	displacement of rigid plate at base of deep borehole

* $R = 10$

+ $\eta = 0.25$

load test at the base of an unlined borehole which is left open by a borehole composed of the remoulded soil medium.

The remaining λ values yield the following approximate results for the assessment of undrained deformability characteristics from screw plate tests.

$$\frac{w}{pa/E_u} = \{0.60 \text{ to } 0.75\} \quad (23)$$

where w is the settlement of the plate at the point of application of the load P

p is the average stress over the area of the screw plate $= P/\pi a^2$

a is the radius of the plate

E_u is the undrained Young's modulus.

2.4 Determination of the Undrained Shear Strength from Screw Plate Test Results.

In this section some solutions which may be used to estimate the undrained shear strength values from results derived from screw plate tests are examined. Table 2.2 summarizes a set of parameters for the ratio p_{ult}/c_u which can be utilized to estimate the undrained shear strength.

The results of Shield (1955) and Eason and Shield (1961) are based on classical plasticity solutions which invoke the Harr-von Karman postulate (ie. the circumferential stress $\sigma_{\phi\phi}$ in the axisymmetric problem is set equal to the algebraic value of one of the principal stresses derived from the stress system σ_{rr} , σ_{zz} and σ_{rz}). Both studies, however, deal with the problem of a rigid circular punch on a half space and are of limited use for the deeply embedded problem.

The result of Meyerhof (1951) is based on Hencky's approximate method involving a plane strain slip-line field. Skempton's (1950) result is developed for the analysis of deep circular foundations. These results essentially correspond to the loading of a circular plate at the base of an unlined borehole. The results of Selvadurai and Szymanski (1979) are based on classical plasticity solutions developed for smooth and rough circular rigid anchor plates

TABLE 2.2
ESTIMATION OF UNDRAINED SHEAR STRENGTH FROM SCREW PLATE TESTS

SOLUTION	P_{ult}/c_u	REFERENCE
(a)	5.69	Shield (1955)
(b)	6.05	Eason and Shield (1960)
(c)	9.00	Skempton (1951)
(d)	9.34	Meyerhof (1951)
(e)	10.97	Selvadurai and Szymanski (1979)
(f)	11.35	Selvadurai and Szymanski (1979)

P_{ult} = failure stress

c_u = undrained shear strength

SOLUTION	REMARKS
(a)	rigid circular punch on halfspace - smooth interface
(b)	rigid circular punch on halfspace - rough interface
(c)	circular foundation at large depth - empirical
(d)	circular foundation at large depth - approximate solution - rough interface
(e)	deeply embedded rigid circular plate - smooth interface
(f)	deeply embedded rigid circular plate - rough interface

embedded in a cohesive soil medium. Here again, the Harg-von Karman postulate ($\sigma_1 = \sigma_2 = \sigma_3$) is invoked to render the problem statically determinate. Considering the theoretical results given in Table 2.2 for a circular foundation at large depths and the deeply embedded rigid circular plate, the following bounds may be assigned for the assessment of the undrained shear strength of a cohesive soil medium from the ultimate load measured in a screw plate test.

$$\frac{p_{ult}}{c_u} = \{9.0 \text{ to } 11.35\} \quad (24)$$

where p_{ult} is the failure stress

c_u is the undrained shear strength.

It has been observed (Schwab, 1976; and Schwab and Broms, 1977) that the failure stress from screw plate tests is less discernible due to the sinkage of the screw plate. To eliminate this ambiguity in the definition of the limit load, it is convenient to adopt a modified plot of the load-displacement curve for the screw plate test. The average stress is plotted in a logarithmic scale and the plate settlement is expressed as a percent of the plate diameter (See Figure 2.14). The failure stress is defined as either the peak value of the modified stress-settlement curve or the stress at which the curve reaches a relatively steady low rate of increase.

FIG. 2.14 DETERMINATION OF c_u FROM SCREW PLATE TEST RESULTS

CHAPTER THREE

3.0 *Experimental Program*

3.1 *Conventional Screw Plate Test Apparatus*

The conventional screw plate apparatus, also called the field compressometer (See Figure 3.1), consists of a helical screw plate mounted on a hydraulic jack unit. The hydraulic jack is connected to an outer pipe system which will transmit both the external forces during installation and the reaction forces during load testing. Oil pressure is transmitted to the jack through a hydraulic hose placed in an inner pipe system. The inner pipe is directly fixed to the piston of the jack and thereby to the screw plate. All displacements of the screw plate can therefore be measured at the ground surface by measuring the movement of the inner pipe system. During installation of the screw plate, the outer and inner pipes are firmly connected, locking the jack in its inner position. When the screw plate is at the test depth desired, the connection between the outer and inner pipes is released. The reaction forces during loading is taken up by an anchor frame with four ground anchors and by skin friction along the outer pipe. Settlement gauges are fixed to an independent anchored beam. Compressed nitrogen gas and a pressure accumulator is used to maintain a constant pressure during each loading step.

3.2 *Carleton University Screw Plate Apparatus*

The Carleton University screw plate apparatus consists of the load shafts and screw plate, the frame, the anchoring system and the load and displacement measuring systems. A schematic of the Carleton University screw plate apparatus is shown on Figure 3.2.

FIG. 3.1 SCHEMATIC OF CONVENTIONAL SCREW PLATE TESTING APPARATUS

FIG. 3.2 SCHEMATIC VIEW OF THE CARLETON UNIVERSITY SCREW PLATE DEVICE

3.2.1 *Screw Plates and Loading Shafts*

Two sizes of screw plates were used in this study. A 16 cm diameter screw plate with a 4.5 cm pitch and a 30 cm diameter screw plate with a pitch of 7.0 cm. The 16 cm diameter screw plate is the same as that used in the conventional screw plate test. The larger screw plate was machined from a 30 cm diameter Chance wrench type screw anchor (Catalogue No. E102-0821). This anchor is commonly used by Ontario Hydro for guying purposes. The anchor consists of a 1.0 cm thick plate welded in a helix around a 3.8 cm square hub. The pitch of the helix is 7.0 cm. The square hub of the anchor was removed and the plate was then welded to a 5 cm diameter mild steel shaft which was machined to have a 60 degree cone on the end. The edge of the screw plate was then machined on the top side in an attempt to improve the cutting action of the plate and decrease the amount of soil disturbances associated with the cutting action. Plate 3.1 shows the large and small screw plates used in this study.

The loading shafts were machined from type 304 stainless steel tubing with an outside diameter of 4.83 cm and a wall thickness of 0.508 cm. Five loading shafts were machined to a length of 0.91 metres. A type 306, 2.8 cm Snap on Tool socket drive was inserted and welded inside each end of the loading shafts. This serves as the primary means of transmitting the torque along the shafts when the screw plate is being installed. Slots were also machined 180 degrees apart to a depth of 2.5 cm by 90 degrees wide at each end of the loading shafts. These slots served as a secondary means of torque transfer. Details of the shaft connection is shown on Figure 3.3.

The screw plate is connected to the loading shaft by a nose bolt which passes through the screw plate hub and threads into a hub at the end of the first loading shaft. Torque is transferred from the loading shafts to the screw plate by four 0.64 cm diameter high strength pins which are installed before the bolt is threaded into the loading shaft. Details of the lower assembly of the screw plate apparatus is shown on Figure 3.4.

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Images en couleur

PLATE 3.1 SCREW PLATES USED IN FIELD TESTING PROGRAM

FIG. 3.3 CONNECTION DETAIL OF THE LOADING SHAFTS

FIG. 3.4 CONNECTION OF SCREW PLATE AND LOADING SHAFT

3.2.2 Test Frame

A tripod test frame (See Plate 3.2) was designed for this particular study. This tripod frame is somewhat different than the more conventional reaction beam type of reaction system supplied by the screw plate test apparatus manufacturer. The main advantage of the tripod frame is that it can be dismantled and easily carried by three persons. A reaction beam is generally heavy and difficult to transport to a testing site unless the site is accessible by a vehicle. The support tripod consists of 3.2 x 3.2 x 0.32 cm hollow structural sections which telescope in 3.8 x 3.8 x 0.32 cm hollow structural steel sections. This telescope action enables the raising and lowering of the apex of the frame to accommodate the length of the loading shafts above the ground surface. Holes were drilled at 10 cm spacing along the length of both sections to accommodate steel pins. The base triangle is constructed of 5.1 x 5.1 x 0.48 cm hollow structural steel sections which are pinned to the bottom of the support tripod legs. The base triangle restrains lateral movement of the ground anchors during testing. It also serves as a support for two other triangles which provide vertical support for the loading shafts extending above the ground surface (See Figure 3.5). When connected to the tripod frame the base triangle is an equilateral triangle with sides of 2.6 metres. The distance from the centroid of this triangle to any of the ground anchors is 1.52 metres.

3.2.3 Ground Anchors

The frame is anchored by three 15 cm diameter Thimbleye Chance ground anchors (Catalogue No. 6346-1). These anchors have a helical plate, 0.64 cm in thickness, welded around a 1.9 cm diameter steel shaft. The cutting edges of the helical plate were sharpened to aid in the installation of the ground anchors. A 0.32 cm thick steel plate was welded to each side of the thimbleye. A 2.5 cm diameter hole was drilled through the two plates to accept a 2.5 cm diameter bolt which connects the test frame to the anchors.

PLATE 3.2 SCREW PLATE TRIPOD TEST FRAME

FIG. 3.5. PLAN VIEW OF BASE AND SUPPORT TRIANGLE LAYOUT

These plates provided a slack free connection between the ground anchors and the test frame.

3.2.4 Load and Displacement Measurement

A 35.8 kN hydraulic car jack was used in the preliminary studies at Arnprior, Ontario. This jack was replaced by an Enerpac double acting 35.8 kN jack having a 15 cm stroke. The pumping unit of this jack is separate from the jack itself and therefore, no moment is unduced by the pumping action during load application. The piston of the jack is screwed into the apex plate of the tripod frame. The base of the jack is bolted to a 2.54 cm thick mild steel plate which serves as a seat for a 13.3 kN capacity proving ring. Load from the jack is transmitted to the proving ring through a 1.6 cm diameter ball bearing which is recessed into the steel plate. The original base of the proving ring was replaced by a base which could be attached to the top of the loading shafts. The deformation of the proving ring is measured by a Starrett dial gauge (Catalogue No. 25-611) which is accurate to 0.0025 cm. The proving ring was calibrated on a 267 kN-Tinius Olsen Universal Testing machine. The calibration was performed with the steel plate and new base attached to the proving ring. The calibration curve and data are shown in Appendix A.

The displacement of the screw plate is measured by four Starrett dial gauges (Catalogue No. 25-441). These gauges can measure a total displacement of 2.54 cm in 0.025 cm divisions. The gauges are supported by 0.64 x 1.27 cm mild steel bars which are bolted to the base of the proving ring. The gauges rest on a 0.635 cm thick plexiglass plate which was clamped to an independent reference beam. The reference beam is a 5.08 x 5.08 x 0.635 cm hollow structural section 2.4 m long. It is supported at each end by adjustable stands. Plate 3.3 shows the configuration of the dial gauges as well as the load and displacement measuring system.

PLATE 3.3 SCREW PLATE TEST DETAIL OF THE PLATE
DISPLACEMENT MEASUREMENT SETUP

3.3 Test Procedure

The base triangle is first fastened to the support tripod and the anchor connectors are inserted into the bottom of the support tripod legs. The frame is then moved to the area where the test is to be performed. After removal of the topsoil (in the areas around the anchors and screw plate) the first anchor is installed and the frame is pinned to it using a 2.54 cm diameter steel bolt. The second anchor is installed to a depth which is level with the first anchor. The third anchor is then installed and leveled across to the first and second anchors. The center line of the screw plate is then established by plumbing from the apex of the tripod frame to the ground.

The the Gloucester test fill site the upper 1.2 m of the crust was removed by hand augering. A 25 cm diameter post hole auger was used for this purpose. The hole was then enlarged (with a shovel) to a diameter sufficient for the screw plate to be lowered into the hole. This procedure was followed because it was difficult for two persons to manually insert the screw plate through the silty sand deposit and maintain the loading shafts in a vertical position. There was also occasional rounded gravel from 2.5 to 5.0 cm in diameter throughout the silty sand deposit.

The center of the support triangle was placed in alignment by plumbing from the apex of the support tripod and clamped in position. The screw plate loading shafts were oiled with a fine machine oil, plumbed and slowly turned into the soil. The oil was used to prevent adhesion between the loading shafts and the surrounding disturbed clay. At the desired depth the frame was adjusted in height and the jack and proving ring were positioned. The reference beam was then placed beside the loading shaft to position the reference beam supports. The topsoil was removed to a depth of approximately 0.3 m to provide adequate seating for the reference beam supports. The reference beam was then positioned beside the loading shafts and leveled at an appropriate height. The plexiglass sheet was clamped to the reference beam and the displacement gauges were set to zero. The gauges were observed

for several minutes before proceeding with the load test to ensure the reference beam supports were stable. The load was then applied in 0.16 kN increments to a level of approximately $c_u/3$. The value of c_u was estimated from the results given by Bozozuk and Leonards (1972). The load was cycled at this level for two load cycles and then the soil was loaded to failure. Settlement readings were taken within 5 seconds after the load was stabilized. At load levels close to failure, readings were taken as soon as the jack operator stabilized the load. At these stress levels the readings exhibited substantial drift within a short time.

3.4 Testing Program

3.4.1 Preliminary Tests (Arnprior)

Preliminary tests were conducted at a pit in Arnprior, Ontario belonging to the Dochart Brick & Tile Company Ltd. The clay at this site falls into the category of an inorganic clay of medium plasticity under the Unified Soil Classification System (Kempthorne, 1978). Guillet (1967) explained that the clay sediments were deposited on the bed of the Champlain Sea, making it a stratified marine clay interbedded with silt and sand lenses. The silt content of this clay was between 25 and 35 percent. Further information pertaining to the Arnprior clay is reported by Guillet (1967) and Kempthorne (1978).

The tests at the Arnprior site served as trial tests on the testing apparatus and testing technique. Several aspects of the original apparatus and testing technique were altered as a result of the experience gained from the tests at Arnprior. The tests at Arnprior were carried out using the small or 16 cm diameter screw plate. Due to the softer, more sensitive silty clay at the Gloucester test fill site, only the large 30 cm diameter screw plate was used at the Gloucester test fill site. In the original apparatus the load was supplied by a simple 36 kN hydraulic car jack reacting against the test frame. The deficiency in this method of load application was that a moment was induced at the jack by the pumping action. Two additional dial gauges were added to measure the screw plate displacement. The steel plates, mentioned previously, were added to the thimble ground anchors to correct the loose fitting connection between the

frame and the ground anchors. Before this adjustment was made a preload of approximately 1.0 kN had to be applied before the test was conducted. The preload ensured the load was being resisted by the anchors at the start of the test. The largest deficiency caused by this loose fitting connection was that load cycling was extremely difficult. There was no control over the release of the load due to the hydraulic release valve.

3.4.2 Gloucester Test Fill Site

The main test series was conducted at a test site adjacent to the Canadian Forces Station (CFS) in Gloucester approximately 20 kilometers east of Ottawa (See Figure 3.6). The test fill constructed at this site has been the subject of extensive field and laboratory investigations (See Bozozuk and Leonards, 1972; and Lo *et al.*, 1976). These investigations enable the comparison of screw plate test results with reliable results on high quality samples from other sources.

The soil profile at the site (See Figure 3.7) consists of a surface crust approximately 1.8 m in depth with a lower 0.9 m composed of grey-brown silty sand. This crust rests on approximately 0.75 m of soft grey-brown silty clay containing some organic material. Underlying this layer is a soft grey silty marine clay to a depth of approximately 18 m. A more complete description of the soil profile may be found in Bozozuk and Leonards (1972).

FIG. 3.6 GLOUCESTER TEST SITE

FIGURE 3.7 SOIL PROFILE AND SCREW PLATE TEST DEPTHS

GLOUCESTER TEST SITE

114
CHAPTER FOUR

In this chapter the results obtained from tests conducted at the Gloucester test fill site are analyzed. The statistical analysis uses small sample theory. Also in this analysis the Student T distribution has been assumed for the calculation of the confidence intervals.

Tests were conducted at 13 locations at the Gloucester test fill site at depths of 2.3 m and 4.1 m. The test locations are shown on Figure 4.1 with respect to the major and minor axis of the test fill. Test number 9 and number 13 have been omitted from the analysis. During test number 9, one of the ground anchors pulled out during the loading cycle causing a misalignment of the jack and loading shafts. In test number 13 similar problems were encountered with the alignment of the jack and the loading shafts.

A typical load-displacement curve obtained from screw plate tests at the Gloucester test fill site is shown on Figure 4.2. Typical undrained modulus, E_u , and unload undrained modulus, E_R , values are also shown. Figure 4.3 illustrates the method used for deriving the E_u values and the E_R values from the load-displacement curves. Only typical load and unload cycles have been shown in this figure. As shown in Figure 4.3 the moduli values are assessed by computing the slope of the line of best fit to the experimental points. Screw plate test data as well as load-displacement curves are presented in Appendix B. Equation 23 has been used to calculate the possible range of the undrained moduli.

The analysis of the undrained modulus, E_u , has only been analyzed on the second and third load cycles. The first load cycle indicates that the screw plate causes some disturbance, however, the extent of this disturbance is unknown.

FIG. 4.1 PLAN VIEW OF TEST FILL AND SCREW PLATE TEST LOCATIONS

FIG. 4.2 A TYPICAL LOAD-DISPLACEMENT CURVE FROM A SCREW PLATE TEST

FIG. 4.3 DETERMINATION OF THE UNDRAINED MODULUS FROM TYPICAL LOAD CYCLE CURVES

4.2 Results of Analysis on the Second Load Cycle

Tables 4.1a and 4.1b indicate the range of the undrained modulus as calculated on the second load cycle for the tests carried out at the 2.3 m and 4.1 m depths respectively. At the 2.3 m depth the histograms on Figure 4.4a as well as the summary of statistical data in Table 4.2 indicate that the frequency distribution (for undrained modulus values based on the limits of $\lambda = 0.60$ to $\lambda = 0.75$) is skewed slightly to the right and platykurtic (See Figure 4.5) in geometry. The mean undrained modulus value, E_u , is $7.78 \times 10^3 \text{ kN/m}^2 \pm 0.82 \times 10^3 \text{ kN/m}^2$ assuming a Student's T distribution and 95 percent confidence intervals.

The results obtained at the 4.1 m testing depth are shown on Figure 4.4b and in Table 4.2. The results indicate that the frequency distribution is also skewed to the right, but leptokurtic (See Figure 4.5) in geometry. The degree of skewness at the 4.1 m depth is greater than the skewness at the 2.3 m depth. This could possibly be due to the occurrence of some friction along the loading shafts causing higher estimates of E_u in some of the results. This explanation seems to be consistent with the results discussed previously. By comparing the E_u values obtained from all tests at the 2.3 m depth with those obtained at the 4.1 m depth (See Table 4.1a and 4.1b) (except test number 1, 3 and 12) the E_u values derived from the tests at the 2.3 m depth appear to be greater than the values derived from the tests at the 4.1 m depth. It is the tests 1, 3 and 12 that form the right handed tail on the frequency distributions at the 4.1 m depth. The range of undrained moduli, E_u , values, $7.15 \times 10^3 \text{ kN/m}^2 \pm 1.23 \times 10^3 \text{ kN/m}^2$ represents the sample mean with a 95 percent confidence interval. The deviation of results at the 4.1 m depth is greater than at the 2.3 m depth because of the greater skewness of the frequency distribution for E_u values derived from tests at the 4.1 m depth. The skewness to the right could be an indication that the results which form the right hand tail may be influenced to a greater extent by friction along the loading shafts.

TABLE 4.1a

DETERMINATION OF UNDRAINED MODULUS E_u ($\times 10^3$ KN/m²)
SECOND LOAD CYCLE

2.3 m DEPTH

TEST NO.	LOAD = P (KN)	DISPLACEMENT = w (mm)	$E_u = \frac{0.6 P}{\pi a w}$	$E_u = \frac{0.75 P}{\pi a w}$
1	3.90	0.59	8.55	10.7
2	4.75	1.40	4.39	5.49
3	5.75	0.88	8.45	10.6
4	4.75	0.69	8.90	11.1
5	5.50	1.10	6.47	8.09
6	3.55	0.68	6.75	8.44
7	6.55	1.15	7.37	9.21
8	6.75	1.10	7.94	9.92
10	5.00	1.25	5.17	6.47
11	5.50	1.16	6.13	7.67
12	6.90	1.50	5.95	7.44

TABLE 4.1b

DETERMINATION OF UNDRAINED MODULUS E_u ($\times 10^{-3}$ KN/m²)
SECOND LOAD CYCLE

4.1 m DEPTH

TEST NO.	LOAD = P (KN)	DISPLACEMENT = w (mm)	E_u	$\frac{0.6 P}{\pi a w}$	E_u	$\frac{0.75 P}{\pi a w}$
1	6.00	0.67		11.6		14.5
3	9.00	1.31		8.89		11.1
4	8.00	1.96		5.28		6.60
5	7.00	2.08		4.35		5.44
6	6.00	1.68		4.62		5.78
7	5.00	1.04		6.22		7.77
8	7.50	1.60		6.06		7.58
10	8.00	2.35		4.40		5.50
11	7.50	1.73		5.61		7.01
12	7.50	1.50		6.47		8.09

i) $E_u \times 10^3 \text{ kN/m}^2$ ii) $E_u \times 10^3 \text{ kN/m}^2$ iii) $E_u \times 10^3 \text{ kN/m}^2$ FIG. 4.4a. HISTOGRAMS FOR E_u VALUES OBTAINED AT THE 2.3 m DEPTH AT GLOUCESTER TEST FILL SITE

- i) E_u based on $\lambda = 0.6$
 ii) E_u based on $\lambda = 0.75$
 iii) E_u based on $\lambda = 0.6$ to 0.75

FIG. 4.4b HISTOGRAMS OF E_u FROM THE 2nd LOAD CYCLE AT THE 4.1 m DEPTH AT GLOUCESTER TEST FILL SITE

- i) E_u based on $\lambda = 0.60$
- ii) E_u based on $\lambda = 0.75$
- iii) E_u based on $\lambda = 0.60$ to 0.75

FIGURE 4.5 THE EFFECT OF THE COEFFICIENT OF KURTOSIS
ON THE FREQUENCY DISTRIBUTION

TABLE 4.2
SUMMARY OF STATISTICAL DATA
SECOND LOAD CYCLE

	DATA	$E_u \times 10^3 \text{ kN/m}^2$	$S \times 10^3 \text{ kN/m}^2$	$V \%$	a_3	a_4
2.3 m DEPTH	$E_u (\lambda = 0.60)$	6.92	1.46	21.1%	-0.18	1.16
	$E_u (\lambda = 0.75)$	8.65	1.83	21.1%	-0.18	1.59
	COMBINED	7.78	1.84	23.7%	0.10	2.05
4.1 m DEPTH	$E_u (\lambda = 0.60)$	6.35	2.28	35.8%	1.17	3.11
	$E_u (\lambda = 0.75)$	7.94	2.84	35.8%	1.18	3.15
	COMBINED	7.15	2.64	36.9%	1.28	3.87

E_u mean
 S standard deviation
 V coefficient of variation
 a_3 coefficient of skewness
 a_4 coefficient of kurtosis

4.3 Results of Analysis on the Third Load Cycle

The E_u values derived from the third load cycle of each test is shown in Tables 4.3a and 4.3b. Histograms and the summary of the statistical analysis are shown on Figures 4.6a and 4.6b and in Table 4.4 respectively. At the 2.3 m depth the frequency distribution derived from the results, is skewed slightly to the right and platykurtic in geometry. The mean undrained modulus is $7.61 \times 10^3 \text{ kN/m}^2 \pm 0.80 \times 10^3 \text{ kN/m}^2$ based on a 95 percent confidence interval. At the 4.1 m depth, the E_u values derived from the results on the third load cycle are generally lower than those derived from tests at the 2.3 m depth. The frequency distribution exhibits the same characteristics as those present in the second load cycle test results at the 4.1 m depth. The frequency distribution of E_u exhibits more positive skewness at the 4.1 m depth than at the 2.3 m depth. The shape of the frequency distribution is again leptokurtic. The mean undrained modulus value, E_u , derived from the third load cycle of the tests is $7.35 \times 10^3 \text{ kN/m}^2 \pm 1.09 \times 10^3 \text{ kN/m}^2$.

4.4 The Unload Undrained Modulus (E_R)

An unload undrained modulus (E_R) has been calculated for each load cycle of each test (See Table 4.5). The results have been grouped to include the first, second and third unload segments of each test. A summary of the statistical analysis is shown in Table 4.6. The E_R frequency distributions are shown on Figures 4.7a and 4.7b for the tests at the 2.3 m depth and the 4.1 m depth respectively. At the 2.3 m depth the E_R frequency distribution is skewed slightly to the right and is very close to a normal distribution in geometry. The mean unload undrained modulus, E_R , is $8.77 \times 10^3 \text{ kN/m}^2 \pm 0.81 \times 10^3 \text{ kN/m}^2$ for a 95 percent confidence interval. At the 4.1 m depth the E_R frequency distributions is skewed further to the right than at the 2.3 m depth. The frequency distribution of the E_R values at the 4.1 m depth is very leptokurtic in geometry indicating that the results are grouped very close about

TABLE 4.3a

DETERMINATION OF UNDRAINED MODULUS E_u ($\times 10^3$ kN/m²)
THIRD LOAD CYCLE

30 cm DIAMETER SCREW PLATE

2.3 m DEPTH

TEST NO.	LOAD = P (kN)	DISPLACEMENT = w (mm)	$E_u = \frac{0.6}{\pi a w} P$	$E_u = \frac{0.75}{\pi a w} P$
1	5.00	0.28	23.1*	28.9*
2	4.00	0.86	6.02	7.52
3	6.00	0.92	8.44	10.5
4	5.00	0.70	9.24	11.6
5	6.00	1.50	5.17	6.47
6	7.50	1.38	7.03	8.79
7	7.00	1.33	6.81	8.51
8	7.00	1.19	7.61	9.51
10	8.00	2.00	5.17	6.47
11	6.00	1.24	6.26	7.82
12	7.50	1.65	5.88	7.35

* TEST NOT USED IN STATISTICAL ANALYSIS

TABLE 4.3b

DETERMINATION OF UNDRAINED MODULUS E_u ($\times 10^3 \text{ kN/m}^2$)
THIRD LOAD CYCLE

30 cm DIAMETER SCREW PLATE

4.1 m DEPTH

TEST NO.	LOAD = P (kN)	DISPLACEMENT = w (mm)	$E_u = \frac{0.6}{\pi a w} P$	$E_u = \frac{0.75}{\pi a w} P$
1	6.00	0.69	11.2	14.1
3	9.00	1.32	8.82	11.0
4	8.00	1.68	6.16	7.70
5	7.00	1.82	4.98	6.22
6	6.00	1.46	5.32	6.65
7	5.00	1.15	5.62	7.03
8	7.50	1.55	6.26	7.82
10	8.00	2.00	5.17	6.47
11	6.00	1.24	6.26	7.82
12	7.50	1.75	5.54	6.93

FIG. 4.6a HISTOGRAMS OF E_u FROM THE 3rd LOAD CYCLE AT THE 2.3 m DEPTH; GLOUCESTER TEST FILL SITE

- i) E_u based on $\lambda = 0.6$
- ii) E_u based on $\lambda = 0.75$
- iii) E_u based on $\lambda = 0.6$ to 0.75

FIG. 4.6b HISTOGRAMS OF E_u FROM THE 3rd LOAD CYCLE AT THE 4.1 m DEPTH; GLOUCESTER TEST FILL SITE

- i) E_u based on $\lambda = 0.6$
- ii) E_u based on $\lambda = 0.75$
- iii) E_u based on $\lambda = 0.6$ to 0.75

TABLE 4.4
SUMMARY OF STATISTICAL DATA
THIRD LOAD CYCLE

	DATA	E_u $\times 10^3 \text{ kN/m}^2$	S $\times 10^3 \text{ kN/m}^2$	V %	a_3	a_4
2.3 m DEPTH	$E_u(\lambda = 0.6)$	6.76	1.35	20.0%	0.46	1.81
	$E_u(\lambda = 0.75)$	8.45	1.69	20.0%	0.48	1.84
	COMBINED	7.61	1.72	22.6%	0.56	2.51
4.1 m DEPTH	$E_u(\lambda = 0.6)$	6.53	1.97	30.1%	1.36	3.43
	$E_u(\lambda = 0.75)$	8.17	2.48	30.4%	1.39	3.52
	COMBINED	7.35	2.34	31.8%	1.43	4.30

E_u mean
 S standard deviation
 V coefficient of variation
 a_3 coefficient of skewness
 a_4 coefficient of kurtosis

TABLE 4.5

TABLE OF UNLOAD MODULI
 $E_R (\times 10^3 \text{ KN/m}^2)$

		UNLOAD CYCLE 1		UNLOAD CYCLE 2		UNLOAD CYCLE 3	
TEST NO.							
		$\frac{0.6 P}{a w}$	$\frac{0.75 P}{a w}$	$\frac{0.6 P}{a w}$	$\frac{0.75 P}{a w}$	$\frac{0.6 P}{a w}$	$\frac{0.75 P}{a w}$
2.3 m DEPTH	1						
	2	9.03	11.3	9.14	11.4	24.0*	30.0*
	3	13.8	17.3	13.3	16.6	9.74	12.2
	4	10.0	12.5	8.44	10.6		
	5	5.33	6.66	5.89	7.37	4.83	6.04
	6	10.0	12.5	5.74	7.17	9.36	11.7
	7	7.33	9.16	7.54	9.43	5.77	7.22
	8	7.78	9.72	7.09	8.87	12.9	16.1
	10	8.86	11.1	6.30	7.88	5.02	6.28
	11	7.07	8.84	7.28	9.10	6.06	7.57
	12	5.52	6.90	5.52	6.90	3.51	4.38
4.1 m DEPTH	1	28.8*	36.0*	42.6*	53.2*	20.6*	25.8*
	3	13.4	16.7	9.07	11.3		
	4	8.23	10.3	7.97	9.96	7.19	8.99
	5	4.85	6.06	5.69	7.12	5.59	6.99
	6	7.40	9.25	6.97	8.71	6.61	8.26
	7	5.84	7.30	6.30	7.87	4.93	6.16
	8	7.78	9.72	7.09	8.87	9.94	12.4
	10	5.94	7.43	5.36	6.71	5.88	7.34
	11	4.73	5.92	4.53	5.66	5.74	7.17
	12	7.68	9.60	6.61	8.26	4.68	5.85

* TEST NOT USED IN STATISTICAL ANALYSIS

TABLE 4.0

SUMMARY OF STATISTICAL DATA
E_p MODULUS

DATA		E_F	S	V	a_3	a_4
		$\times 10^3 \text{ KN/M}^2$	$\times 10^3 \text{ KN/M}^2$	%		
DEPTH 2.31 m	$E_F (\lambda = 0.6)$	6.79	1.61	33.0%	1.72	2.78
	$E_F (\lambda = 0.75)$	8.74	3.26	33.0%	1.72	2.79
	COMBINED	8.77	3.38	35.0%	1.83	3.27
DEPTH 4.1 m	$E_F (\lambda = 0.6)$	6.77	1.94	28.7%	1.66	5.88
	$E_F (\lambda = 0.75)$	8.46	2.42	28.7%	1.66	5.88
	COMBINED	7.62	2.33	30.6%	1.33	6.05

E_p mean
 S standard deviation
 V coefficient of variation
 a₃ coefficient of skewness
 a₄ coefficient of kurtosis

FIG. 4.7a HISTOGRAM OF UNLOAD MODULUS (E_R): 2.3 m DEPTH

- i) E_R based on $\lambda = 0.60$
- ii) E_R based on $\lambda = 0.75$
- iii) E_R based on $\lambda = 0.60$ to 0.75

FIG. 4.7b HISTOGRAMS OF UNLOAD MODULUS (E_R); 4.1 m DEPTH

- i) E_R based on $\lambda = 0.6$
- ii) E_R based on $\lambda = 0.75$
- iii) E_R based on $\lambda = 0.6$ to 0.75

the mean. The mean unload undrained modulus, E_u , is $7.62 \times 10^3 \text{ kN/m}^2 \pm 0.65 \times 10^3 \text{ kN/m}^2$. The confidence limits at the 4.1 m depth are less than at the 2.3 m depth because of the leptokurtic geometry of the former distribution.

4.6 Undrained Shear Strength

As mentioned previously (Chapter 2) the undrained shear strength, c_u , can be derived from a plot of log pressure ($\log p$) vs. percentage displacement (ie w/B) (Schwabs and Broms, 1976). These plots for the tests at the Gloucester test fill site are presented in Appendix C. Equation 24 (Chapter 2) has been used to estimate the undrained shear strength, c_u , for each tests location. Table 4.7a and 4.7b summarize the limits of the undrained shear strength results obtained for the soft clay at the Gloucester test fill site. Table 4.8 represents a statistical summary pertaining to the undrained shear strength results. Histograms of the undrained shear strength for the tests conducted at the 2.3 m and the 4.1 m depth are shown on Figures 4.8a and 4.8b.

At the 2.3 m depth the undrained shear strength was calculated to be $9.13 \text{ kN/m}^2 \pm 0.92 \text{ kN/m}^2$ assuming a Student's T distribution and a 95 percent confidence interval. The undrained shear strength distributions shown on Figure 4.8-iii exhibits a slightly positive skewness of 0.092, and a platykurtic geometry. At the 4.1 m depth the undrained shear strength was calculated to be $10.53 \text{ kN/m}^2 \pm 0.90 \text{ kN/m}^2$. The frequency distribution for the tests at the 4.1 m depth (See Figure 4.8b-iii) has a positive skewness of 0.40 and is also platykurtic in geometry.

4.7 Ratio E_u/c_u

It has been observed (Ladd, 1964; Bozozuk, 1971; and Raymond et al., 1971) that the ratio E_u/c_u , generally remains constant for relatively undisturbed soil and tends to decrease with increasing sample disturbance. A summary of the range of the ratio E_u/c_u for the screw plate tests carried out at the Gloucester

TABLE 4.7a

DETERMINATION OF THE UNDRAINED SHEAR STRENGTH

 c_u (kN/m²)

2.3 m DEPTH

TEST NO.	LOG PRESSURE (kN/m ²)	LIMIT PRESSURE _u (kN/m ²) (p_u)	$c_u = \frac{p_u}{9.0}$		$c_u = \frac{p_u}{11.35}$	
			c_u		c_u	
1	1.80	63.1	7.01		5.55	
2	1.86	72.4	8.05		6.38	
3	2.00	100.0	11.1		8.80	
4						
5	1.90	79.4	8.80		7.00	
6	2.05	112.2	12.5		9.90	
7	1.96	91.2	10.1		8.04	
8	1.96	91.2	10.1		8.04	
10	2.04	109.6	12.2		9.70	
11	1.94	89.1	9.70		7.70	
12	2.04	109.6	12.2		9.70

TABLE 4.7b
DETERMINATION OF THE UNDRAINED SHEAR STRENGTH
 c_u (kN/m²)
4.1 m DEPTH

TEST NO.	LOG PRESSURE (kN/m ²)	LIMIT PRESSURE (kN/m ²) (p_L)	$c_u = \frac{p_L}{9.0}$	$c_u = \frac{p_L}{11.35}$
1	1.96	91.2	10.1	8.00
3	2.10	126.0	14.0	11.1
4	2.08	100.0	11.1	8.80
5	2.00	100.0	11.1	8.80
6	1.95	89.1	9.90	7.90
7	-	-	-	-
8	2.00	100.0	11.1	8.80
10	2.10	126.0	14.0	11.1
11	2.04	110.0	12.2	9.70
12	2.04	110.0	12.2	9.70

TABLE 4.8
SUMMARY OF STATISTICAL DATA FOR c_u

DATA		c_u kN/m ²	S kN/m ²	V %	a_3	a_4
2.3 m DEPTH	c_u (9.0)	10.18	1.86	18.3%	-0.23	1.57
	c_u (11.35)	8.08	1.48	18.3%	-0.23	1.58
	c_u (9.0 to 11.35)	9.13	1.96	21.4%	0.092	2.02
4.1 m DEPTH	c_u (9.0)	11.74	1.50	12.8%	-0.39	1.57
	c_u (11.35)	9.32	1.18	12.7%	0.39	1.57
	c_u (9.0 to 11.35)	10.53	1.81	17.2%	0.40	2.19

c_u mean
 S standard deviation
 V coefficient of variation
 a_3 coefficient of skewness
 a_4 coefficient of kurtosis

FIG. 4.8a HISTOGRAMS OF c_u FOR GLOUCESTER TEST FILL SITE,
2.3 m DEPTH

- i) based on $c_u = \text{pult}/9.0$
- ii) based on $c_u = \text{pult}/11.35$
- iii) based on $c_u = \text{pult}/9.0$ to $\text{pult}/11.35$

FIG. 4.8b HISTOGRAMS OF c_u FOR GLOUCESTER TEST FILL SITE
4.1 m DEPTH

- i) $c_u = p_{ult}/9.0$
 ii) $c_u = p_{ult}/11.35$
 iii) $c_u = p_{ult}/9.0$ to $p_{ult}/11.35$

test fill site is shown in Table 4.9. Since the calculations of the undrained modulus, E_u , and the undrained shear strength, c_u , are based on equations 23 and 24 (Chapter 2) the possible minimum and maximum values of E_u/c_u are tabulated. Figure 4.9 and Figure 4.10 are a graphical representation of the results shown in Table 4.9. At the 2.3 m depth the E_u/c_u ratio ranges from a minimum of 680 to a maximum of 1050. At the 4.1 m depth the E_u/c_u ratio ranges from a minimum of 520 to a maximum of 880. The shaded portions shown on Figures 4.9 and 4.10 correspond to the possible minimum and maximum E_u/c_u ratios as calculated from E_u and c_u discussed previously. If, however, it is assumed that the minimum E_u value will not occur with the maximum c_u value or the maximum E_u value will not occur with the minimum c_u value the band shown in Figures 4.9 and 4.10 would be much thinner. If the maximum E_u and c_u values are used in calculating the ratio E_u/c_u at the 2.3 m depth the resultant is 857. Likewise, if only the minimum E_u and c_u are used in the calculation the resultant is 845. At the 4.1 m depth the use of the maximum E_u and c_u value would result in a E_u/c_u ratio of 730. The use of the E_u minimum value and the c_u minimum value would result in a E_u/c_u ratio of 618.

4.2 Comparison of Results

This section compares the results derived from screw plate tests conducted at the Gloucester test fill site with the results reported by Bozozuk (1971), Bozozuk and Leonards (1972), and Lo et al., (1976).

The results presented by Bozozuk (1971) were obtained from triaxial tests conducted on samples which were obtained with the Norwegian (NGI) piston sampler and the Osterberg hydraulic sampler. The NGI sampler used was 54 mm in diameter and 0.9 m long. The inside clearance ratio was 1 percent. The NGI samples were taken using a single rapid thrust with the hydraulic feed of a drill rig. Precautions were taken to ensure that the piston remained stationary and that the samples were not overdriven. The Osterberg

TABLE 4.9
DETERMINATION OF MINIMUM AND MAXIMUM E_u c_u VALUES

	$E_u (\times 10^3 \text{ kN/m}^2)$		$c_u (\text{kN/m}^2)$		E_u / c_u		
TEST NO.	1	2	3	4	MINIMUM 1/3	MAXIMUM 2/4	
	$\frac{0.6 P}{a w}$	$\frac{0.75 P}{a w}$	$\frac{P_{ult}}{9.0}$	$\frac{P_{ult}}{11.35}$			
2.3 m DEPTH	1	8.55	10.70	7.01	5.55	1220	1928
	2	4.39	5.49	8.05	6.38	545	861
	3	8.45	10.60	11.1	8.80	761	1205
	4	8.90	11.10				
	5	6.47	8.09	8.80	7.00	735	1156
	6	6.75	8.44	12.5	9.90	540	853
	7	7.37	9.21	10.1	8.04	730	1145
	8	7.94	9.92	10.1	8.04	786	1234
	10	5.17	6.47	12.2	9.70	424	667
	11	6.13	7.67	9.70	7.70	632	996
	12	5.95	7.44	12.2	9.70	488	767
	4.1 m DEPTH	1	11.6	14.5	10.1	8.00	1149
3		8.89	11.1	14.1	11.1	630	1000
4		5.28	6.60	11.1	8.80	476	750
5		4.35	5.44	11.1	8.80	392	618
6		4.62	5.78	9.90	7.90	467	732
7		6.22	7.77				
8		6.06	7.58	11.1	8.80	546	861
10		4.40	5.50	14.0	11.1	314	495
11		5.61	7.01	12.2	9.70	460	723
12		6.47	8.09	12.2	9.70	530	834

FIG. 4.9 E_u/c_u MINIMUM AND MAXIMUM VALUES
2.3 METRE DEPTH

+ maximum value

○ minimum value

FIG. 4.10 E_u/c_u MINIMUM AND MAXIMUM VALUES
4.1 METRE DEPTH

+ maximum value

○ minimum value

sampler used was 124 mm in diameter and 170 mm long. The inside clearance ratio was .41 percent. This sampler has a fixed piston and a safeguard to prevent overdriving. The sampling thrust is provided by water pressure. The samples were carefully transported to the laboratories at the National Research Council of Canada, Ottawa, where they were extracted by backing vertically in the same direction that the sample had entered the tube. Triaxial tests were carried out on test specimens 110 mm in diameter and 70 mm in height. The samples were trimmed from the 124 mm diameter and 124 mm diameter cores taken from borings isolated within a few feet of each other. The specimens were consolidated anisotropically to the in situ stress condition and then loaded in an undrained condition to failure at a rate of strain of .1 percent per second.

The results of plane tests indicated that the shear strengths measured on specimens trimmed from the two sample sizes compared well with each other and with in situ shear strengths estimated from in situ vane tests. The in situ vane shear strengths measured varied from 11.6 kN/m² at the 2.3 m depth to 14.3 kN/m² at a depth of 5.5 m.

Bozozuk and Leonards (1971) report results of undrained moduli measurements carried out on high quality triaxial samples obtained with the NGI and Osterberg samplers. These results indicate that for the deep deposit of grey clay, the undrained elastic modulus approximately corresponds to the empirical relationship $E_u = 630 c_u$ in the upper region (from 0 to 7.0 m depth) and $E_u = 1070 c_u$ in the lower region (depth greater than 7.0 m). Since the screw plate tests were conducted in the upper region of the grey clay stratum it would appear that the undrained elastic modulus as inferred from the shear strength data of Bozozuk and Leonards (1972) and the above empirical relationship would approximately correspond to 8.0×10^3 to 10.0×10^3 kN/m² at a depth of 2.3 m and 11.0×10^3 to 13.0×10^3 kN/m² at a depth of 4.1 m.

Lo *et al.* (1976) report that in situ vane shear strengths varied from 10.0 kN/m² at a 2.3 m depth to 32.6 kN/m² at a depth of 7.0 m. In this investigation Lo *et al.* (1976) used an Osterberg sampler to obtain 127 mm diameter samples. These soil samples were

taker to a depth of 1.8 m and were taken from a location of 33 m west of the embankment. Block samples were recovered from a test trench excavated adjacent to the test fill, to a depth of 2.4 m. The undrained shear strength derived from isotropic and anisotropic consolidated undrained triaxial tests were in good agreement with the undrained shear strengths estimated from in situ vane tests.

A comparison of the results obtained from the screw plate tests of the present study and the results obtained from the above mentioned studies is presented in Table 4.16.

TABLE 4.10

GLOUCESTER TEST FILL RESULTS

	DEPTH (m)	*SCREW PLATE TEST	+ PREVIOUS STUDIES
E_u (kN/m ³)	2.3	$(7.8 \pm 0.8) \times 10^3$	$(8.0 \text{ to } 10.0) \times 10^3$
	4.1	$(7.2 \pm 1.2) \times 10^3$	$(11 \text{ to } 13) \times 10^3$
c_u (kN/m ²)	2.3	9.13 ± 0.9	10.0
	4.1	10.53 ± 0.9	19.6
E_u/c_u	2.3	680 - 1050	800 - 1000
	4.1	520 - 880	560 - 660

* Second load cycle values

+ Bozozuk (1971), Bozozuk and Leonards (1972), Lo *et al.* (1976)

A review of the literature related to the estimation of strength and deformability characteristics of soft clays; laboratory vs. in situ testing has indicated that present day methods of sample retrieval and the analysis of results derived from laboratory tests carried out on such samples are inadequate to accurately predict the deformability and strength characteristics of soft clay. The use of block sampling techniques has been shown to give much higher undrained shear strengths and undrained modulus values than any other sampling technique. Block samples at large depths can now be obtained with novel sampling techniques such as described by Domaschuk (1977), Lefebvre and Poulin (1978), and La Rochelle (1981). Such samples are, however, expensive to retrieve, and extreme care must be exercised after retrieval to minimize disturbance of the clay during transportation and handling. La Rochelle and Lefebvre (1979) and Raymond et al. (1971) showed that the release of the in situ state of stress of a clay soil can cause sufficient disturbance of the clay so as to make the results of any testing unreliable.

A review of the in situ testing techniques most commonly used in practice today has indicated that no one method gives reliable results for all situations. The vane shear test has however, been more widely accepted than any other testing technique to determine the undrained shear strength of soft clays. Deficiencies of this testing technique have been pointed out, however, by several investigators. (Bjerrum (1972), Pilot (1972), Ladd (1973), Janbu (1975), La Rochelle et al. (1974), and Schmertmann (1975) to name a few.) The standard penetration test and the cone penetration test have found very little acceptance as reliable in situ testing techniques where soft clays are involved. Much of the work done has been in correlating the results of these tests to the vane

shear test and considerable local experience in the area being examined is necessary to evaluate the results with any degree of certainty.

The pressuremeter test has been shown to give variable results which depend on the ratio of the hole diameter to the probe diameter.

Recent results by Law and Eden (1980) indicate the nature of soil disturbance that is associated with the use of self boring pressuremeters such as the Camkometer or the autoforeur in the testing of soft sensitive clays. The result derived from pressuremeter tests have to be adjusted before the results can be used for geotechnical design purposes. These adjustments are made based on experience and empirical correlations which have been obtained from the use of the pressuremeter in France and other European countries.

(Baquelin, F., Jezequel, J. F. and Shields, D. H. (1977)).

In the theoretical analysis of the screw plate test, several realistic models of the screw plate test have been analyzed and the results bounded deriving simple and easy to use formulae to determine both the undrained modulus of deformation and the undrained shear strength of sensitive silty clay soil.

A testing apparatus was designed and what the author terms as preliminary testing has been carried out at the Gloucester test fill site in Gloucester, Ontario. The results of this testing has shown the following:

1. The screw plate device is a rather simple apparatus to use and installation of the plate into soft clays is quite easy.
2. The first load cycle of the screw plate test indicates that some degree of disturbance is associated with the placement of the helical plate within the soil.
3. The second and third load cycles test results give consistent undrained modulus of deformation. At the 2.3 metre depth the average undrained modulus of deformation as derived from the second and third load cycles was 7.78×10^3 and 7.61×10^3 kN/m² respectively. At the 4.1 metre depth

the average undrained modulus of deformation as derived from the second and third load cycles was $7.15 \times 10^3 \text{ kN/m}^2$ and 7.35 kN/m^2 respectively.

4. The unload modulus gave undrained modulus values which were consistent although slightly higher than the second and third cycle results. The average undrained unload modulus at the 4.1 meter depth was $7.62 \times 10^3 \text{ kN/m}^2$.
5. Shaft friction could have influenced some of the test results. This is evidenced by the right handed tail in some of the frequency distributions causing a positive skewness.
6. There was no appreciable increase in either the undrained modulus of deformation or the undrained shear strength at the relatively shallow depths tested.
7. Screw plate test results derived from the theories of elasticity and ideal plasticity yield moduli of deformation and undrained shear strength results that are comparable in magnitude to those derived by testing and from undisturbed samples.

5.2 Recommendations for Future Work

This study has attempted to investigate the performance of the screw plate test in the light of the classical theories of isotropic elasticity and rigid plastic behaviour. To date only a limited number of experimental field tests have been performed and further research is necessary to clearly establish the performance and applicability of the screw plate testing apparatus to determine the deformability characteristics of soft clay soils. It is recommended that this initial program of research be continued, with the following suggestions for future work:

1. It is suggested that more tests be carried out to verify the observations of this investigation. In particular, it is suggested that tests be carried out at greater depths.
2. It is recommended that the screw plate testing device be modified to eliminate friction along the shaft-disturbed clay interface.
3. As all screw plate tests in this study were carried out by manually inserting the screw plate in the soil it is recommended that other installing techniques such as power augering be investigated.
4. The screw plate test in the future could be modified to measure the build up of pore water pressure. This could enable carrying out the test under drained conditions. This, however, would probably be of academic interest as such tests are expected to be very long term and expensive to carry out. A better understanding of the consolidation characteristics of soft clay soils could be determined.
5. It is strongly recommended that the screw plate test apparatus be modified, so it can be used on a standard drilling rig such as a CME 55 or CME 75. It is considered that the screw plate device could be used in conjunction with hollow stem augers and standard B size drilling rods thus providing a casing and therefore eliminating friction between the soil and loading shafts. It is the author's opinion that unless such modifications are made, the screw plate test in routine field testing will be unduly postponed.

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APPENDIX A

ALLEGATION DATA - 1970 VIN. RIN. FBI T. MEASURED DATA

PROVIN PING CALIBRATION DATA

LOAD	GAUGE DISPLACEMENT
(lb)	(inches x 10 ⁻³)
100	1.45
200	1.65
300	1.85
400	1.78
500	2.22
600	2.67
700	3.11
800	3.56
900	4.00
1000	4.45
1100	4.89
1200	5.34
1300	5.78
1400	6.23
1500	6.67
1600	7.11
1700	7.56
1800	8.01
1900	8.45
2000	8.90
2100	9.34
2200	9.79
2300	10.23
2400	10.68
2500	11.11
2600	11.56
2700	12.01
2800	12.45
2900	12.90
3000	13.34

PROVING RING CALIBRATION CURVE

APPENDIX B

SCREW PLATE TEST DATA AND LOAD-DISPLACEMENT CURVES FOR
TESTS CARRIED OUT AT THE GLOUCESTER TEST FILL SITE

RAW DATA PAGE 165

REDUCED DATA PAGE 192

LOAD-DISPLACEMENT CURVES PAGE 216

165
TEST NO. 1

2.3 m DEPTH

INCREMENT	DISPLACEMENT GAUGE OF PROVING RING (INCHES $\times 10^3$)	DISPLACEMENT w (INCHES $\times 10^3$)			
		GAUGE NO. 1	GAUGE NO. 2	GAUGE NO. 3	GAUGE NO. 4
1	20	0.500	0.000	1.000	0.000
2	40	10.00	0.000	8.000	0.000
3	60	21.00	-1.00	16.00	1.000
4	80	33.50	-5.00	24.00	1.500
5	100	0.000	0.000	0.000	0.000
6	120	3.000	-3.00	0.000	3.000
7	140	5.000	-2.50	1.000	5.000
8	160	6.000	-2.50	2.000	6.000
9	180	9.000	-2.00	4.000	8.000
10	200	12.00	0.000	6.000	10.00
11	220	15.00	0.000	7.000	10.00
12	240	15.50	2.000	10.00	13.00
13	260	17.00	3.000	11.00	14.00
14	280	20.00	4.000	12.00	16.00
15	300	21.00	5.000	13.00	19.00
16	340	21.00	5.000	14.00	19.50
17	360	22.00	6.000	15.00	21.00
18	380	23.00	7.000	16.00	22.50
19	400	26.00	8.000	17.00	24.00
20	420	27.00	9.000	19.00	26.00
21	440	28.00	10.00	20.00	27.00
22	460	30.00	11.00	22.00	29.00
23	480	32.00	12.00	24.00	31.00
24	500	33.00	13.00	25.00	33.50
25	300	32.00	13.00	23.00	31.50
26	320	32.50	13.00	24.00	32.50
27	340	33.00	13.00	24.50	33.00
28	360	34.00	14.00	26.00	34.00
29	380	35.00	15.00	27.00	35.00
30	400	36.50	16.50	27.00	36.00
31	420	38.00	17.00	29.00	38.00
32	440	42.00	18.00	31.00	40.00
33	460	46.00	19.00	33.00	42.50
34	480	57.00	18.00	34.00	45.00
35	500	59.00	19.00	35.00	46.00
36	300	59.00	15.50	32.00	43.00
37	320	60.00	16.00	33.00	44.00
38	340	60.00	16.50	33.50	44.50
39	360	60.00	18.00	34.00	45.00
40	380	60.00	18.00	35.00	45.00
41	400	60.00	18.00	36.00	45.00
42	420	60.00	18.00	36.50	46.00
43	440	60.00	19.00	36.50	47.00
44	480	61.00	20.00	39.00	49.50

TEST NO. 1

4.1 m DEPTH

INCREMENT	DISPLACEMENT GAUGE OF PROVING RING (INCHES $\times 10^3$)	DISPLACEMENT w (INCHES $\times 10^3$)			
		GAUGE NO. 1	GAUGE NO. 2	GAUGE NO. 3	GAUGE NO. 3
1	320	2.500	0.000	2.000	2.000
2	340	6.000	0.000	4.000	4.000
3	360	15.00	-2.00	8.000	7.000
4	380	17.00	-2.00	10.00	9.000
5	400	21.00	-1.00	12.00	11.50
6	420	26.00	-1.00	14.00	14.00
7	440	31.00	-1.00	17.00	16.00
8	460	35.00	-1.00	19.00	18.50
9	480	40.00	-1.00	21.00	21.00
10	500	44.00	0.000	24.00	24.00
11	520	47.00	0.000	26.00	26.50
12	540	50.00	1.000	28.00	29.00
13	560	54.00	2.000	30.00	32.00
14	580	58.00	3.000	33.00	36.00
15	600	62.00	4.000	36.00	40.00
16	300	55.00	-1.00	31.00	27.50
17	350	57.00	-1.00	32.00	29.00
18	400	59.00	0.000	34.00	32.00
19	420	60.00	0.500	34.50	33.00
20	440	61.00	1.000	35.00	34.00
21	460	62.50	1.500	36.00	35.50
22	480	64.00	2.000	37.00	36.50
23	500	65.50	2.500	38.00	38.00
24	520	66.00	3.000	39.00	39.00
25	540	68.00	3.500	40.00	40.00
26	560	70.00	4.000	41.00	42.00
27	580	72.00	5.000	42.50	45.00
28	600	75.00	5.000	43.50	46.00
29	400	70.00	3.500	40.00	40.00
30	420	70.00	3.500	40.00	40.00
31	440	70.00	4.000	41.00	40.50
32	460	71.00	4.000	41.00	41.00
33	480	72.00	4.000	42.00	42.00
34	500	73.00	4.000	42.00	42.00
35	520	74.00	5.000	43.50	45.00
36	540	75.00	5.500	44.00	45.50
37	560	76.50	6.000	45.00	47.50
38	580	78.00	7.000	46.00	48.00
39	600	80.50	7.000	47.00	50.00
40	620	85.00	9.000	50.00	52.00
41	640	88.00	9.000	51.00	54.00
42	680	95.00	10.00	54.00	57.00
43	700	100.0	11.00	56.00	60.00
44	300	79.00	7.000	48.00	45.00

TEST NO. 2

2.3 m DEPTH

INCREMENT	DISPLACEMENT GAUGE OF PROVING RING (INCHES $\times 10^3$)	DISPLACEMENT w (INCHES $\times 10^3$)			
		GAUGE NO. 1	GAUGE NO. 2	GAUGE NO. 3	GAUGE NO. 4
1	220	1.000	2.000	4.000	1.000
2	240	6.500	7.500	13.00	5.000
3	260	8.000	10.00	15.00	8.000
4	280	10.00	12.00	17.00	9.000
5	300	13.00	18.00	22.00	13.00
6	320	14.00	20.50	26.00	16.00
7	340	16.00	24.00	29.00	18.50
8	360	18.00	27.00	32.00	20.00
9	380	20.00	30.00	35.00	22.00
10	400	22.00	33.00	39.00	25.50
11	200	28.00	10.00	26.00	15.00
12	220	28.50	13.50	30.00	17.00
13	240	30.00	16.00	32.00	18.00
14	260	31.00	18.50	35.00	20.00
15	280	32.00	21.00	38.00	22.50
16	300	33.50	23.50	40.00	24.00
17	340	35.50	28.00	45.00	27.50
18	380	38.00	32.00	50.00	31.00
19	420	41.00	37.50	55.00	36.00
20	300	38.00	30.50	47.00	30.00
21	340	39.00	34.00	51.00	33.50
22	400	42.00	39.00	56.00	36.50
23	300	36.00	25.00	38.00	25.00
24	360	39.00	28.00	45.00	29.00
25	400	41.00	32.00	50.00	32.00
26	440	44.00	36.50	54.00	36.00
27	460	45.00	39.00	57.00	37.50
28	480	46.00	41.50	59.00	39.00
29	500	47.50	44.00	61.00	41.00
30	200	42.00	36.00	53.00	35.00

TEST NO. 3

2.3 m DEPTH

INCREMENT	DISPLACEMENT GAUGE OF PROVING RING (INCHES $\times 10^3$)	DISPLACEMENT w (INCHES $\times 10^3$)			
		GAUGE NO. 1	GAUGE NO. 2	GAUGE NO. 3	GAUGE NO. 4
1	20	-0.500	2.000	0.000	2.500
2	40	-2.000	5.000	0.000	4.500
3	60	-4.000	8.000	0.000	7.500
4	80	-5.000	10.00	0.000	10.00
5	100	-6.000	13.00	0.500	11.00
6	120	-6.000	16.00	1.000	12.50
7	140	-6.000	18.00	2.000	14.00
8	160	-6.000	21.00	4.00	15.00
9	180	-6.000	22.00	4.000	16.00
10	200	-6.000	26.00	6.000	16.50
11	17	-13.00	20.00	0.000	8.000
12	20	-13.00	20.00	0.000	7.500
13	40	-13.00	20.00	0.000	8.500
14	70	-11.00	20.00	0.500	10.00
15	80	-11.00	20.00	1.000	10.50
16	100	-11.00	20.00	1.000	11.00
17	120	-11.00	20.00	2.000	12.00
18	140	-11.00	22.00	2.000	13.00
19	160	-10.00	23.00	3.000	14.00
20	180	-9.500	25.00	4.000	14.50
21	200	-8.000	27.00	5.000	15.50
22	220	-8.000	28.00	6.000	16.50
23	240	-7.000	30.00	8.000	18.00
24	260	-6.000	30.00	9.000	19.00
25	280	-5.000	32.00	10.00	20.50
26	300	-3.000	35.00	12.00	22.00
27	18	-14.00	25.00	2.000	9.500
28	40	-13.00	24.00	2.000	10.00
29	80	-12.00	24.50	2.000	11.00
30	100	-12.00	24.50	3.000	12.00
31	120	-12.00	26.00	3.000	12.00
32	140	-11.00	28.00	4.000	14.00
33	160	-11.00	28.00	5.000	14.50
34	200	-9.000	31.00	7.000	16.00
35	220	-7.500	31.50	8.500	17.50
36	240	-6.000	33.00	10.00	18.50
37	260	-5.000	35.00	11.00	19.50
38	280	-4.500	35.00	11.50	20.50
39	300	-3.500	37.00	13.00	22.00
40	320	-2.000	37.00	14.00	23.00
41	340	-0.500	39.00	16.00	24.50
42	360	1.000	40.00	17.50	25.50
43	380	4.000	41.00	19.00	27.50
44	400	6.000	43.00	21.00	29.50

TEST NO. 3

2.3 m DEPTH
(cont'd)

INCREMENT	DISPLACEMENT GAUGE OF PROVING RING (INCHES $\times 10^3$)	DISPLACEMENT w (INCHES $\times 10^3$)			
		GAUGE NO. 1	GAUGE NO. 2	GAUGE NO. 3	GAUGE NO. 4
45	440	8.000	43.00	23.00	32.50
46	480	12.00	47.00	25.50	36.00
47	520	16.00	50.00	29.00	40.00
48	560	21.00	53.00	33.50	43.50
49	600	24.00	58.00	38.00	48.00
50	640	28.00	60.00	41.00	52.00
51	700	35.00	66.50	47.00	60.00
52	720	38.00	71.00	51.00	64.00
53	740	42.00	75.00	55.00	67.50
54	760	46.00	79.00	58.00	70.50
55	19	3.500	50.00	22.00	31.00

TEST NO. 1

4.1 m DEPTH

INCREMENT	DISPLACEMENT GAUGE OF PROVING RING (INCHES $\times 10^3$)	DISPLACEMENT w (INCHES $\times 10^3$)			
		GAUGE NO. 1	GAUGE NO. 2	GAUGE NO. 3	GAUGE NO. 4
1	40	-2.000	9.000	5.000	1.500
2	60	-4.000	17.00	7.00	2.500
3	100	-7.000	27.00	11.00	6.000
4	140	-7.000	30.00	12.00	10.00
5	160	-6.000	30.00	12.00	11.50
6	180	-4.500	31.00	13.00	13.00
7	200	-1.500	31.50	15.00	13.50
8	220	0.000	32.00	16.50	14.50
9	240	2.500	34.00	19.00	15.50
10	260	3.000	34.00	19.00	16.50
11	300	7.000	37.00	22.50	18.50
12	19	-11.00	35.00	14.00	2.500
13	40	-11.00	36.50	14.50	4.000
14	85	-10.00	35.50	14.50	7.000
15	120	-9.000	36.00	14.00	10.50
16	160	-5.000	34.00	14.00	14.00
17	200	-1.500	34.00	16.00	16.00
18	240	2.000	35.00	18.00	19.50
19	280	6.000	37.00	20.50	24.00
20	320	9.000	39.00	23.00	27.00
21	360	18.00	42.00	30.00	29.00
22	400	22.00	46.00	34.50	32.00
23	16	-8.000	37.00	15.00	8.000
24	50	-9.000	38.00	15.00	10.00
25	100	-8.000	39.00	15.00	13.00
26	150	-3.500	38.00	15.50	20.00
27	200	4.000	38.00	19.50	25.00
28	250	6.500	39.00	21.00	28.00
29	300	10.00	41.00	23.00	31.00
30	350	16.00	45.50	29.00	35.00
31	400	18.00	47.00	30.00	38.00
32	450	26.00	50.00	36.00	43.00
33	500	30.00	54.00	41.00	46.00
34	550	34.00	58.00	44.00	51.50
35	600	39.50	66.00	51.00	56.00
36	650	43.50	70.00	55.00	63.00
37	700	50.00	76.00	61.00	68.50
38	750	54.00	81.00	66.00	76.00
39	800	63.00	91.00	76.00	82.00
40	850	71.00	99.00	83.00	95.00
41	900	80.00	110.0	93.00	101.0
42	950	88.00	118.0	102.0	116.0
43	1000	102.0	136.0	120.0	126.0
44	1050	110.0	146.0	128.0	146.0

TEST NO. 4

2.3 m DEPTH

INCREMENT	DISPLACEMENT GAUGE OF PROVING RING (INCHES $\times 10^3$)	DISPLACEMENT w (INCHES $\times 10^3$)			
		GAUGE NO. 1	GAUGE NO. 2	GAUGE NO. 3	GAUGE NO. 4
1	25	14.00	-10.00	-3.500	25.00
2	50	13.00	-6.000	-1.500	25.00
3	75	13.00	-4.000	2.000	25.00
4	100	15.00	0.000	5.000	25.00
5	125	15.50	6.000	7.000	28.00
6	150	16.00	8.000	9.000	29.00
7	175	17.00	10.00	11.00	30.50
8	200	18.00	15.50	14.50	31.00
9	225	18.00	18.00	16.50	30.00
10	250	18.00	24.00	20.00	29.00
11	275	19.50	27.00	23.00	31.00
12	300	20.00	31.50	25.00	32.00
13	325	21.00	35.00	27.50	34.00
14	350	22.50	39.00	30.50	36.00
15	18	12.00	10.00	3.500	41.00
16	57	11.00	12.00	5.000	40.00
17	75	11.00	15.00	6.500	38.50
18	100	11.50	18.00	9.000	38.50
19	125	11.50	20.00	10.00	38.00
20	150	12.00	25.00	14.00	38.00
21	175	12.00	28.00	16.00	37.00
22	200	12.50	31.00	17.50	37.00
23	225	12.00	34.00	19.00	38.00
24	250	13.00	37.00	21.50	38.50
25	275	15.00	39.00	24.00	40.00
26	300	16.50	44.00	27.50	41.00
27	325	16.50	46.00	28.00	43.00
28	350	17.50	55.00	34.00	43.50
29	17	11.50	18.00	8.000	41.00
30	55	10.00	20.00	8.000	41.00
31	100	10.00	27.00	13.50	38.00
32	150	10.50	32.00	17.00	39.50
33	200	10.00	38.00	20.00	41.00
34	250	11.00	44.00	23.50	44.00
35	300	13.00	50.00	27.00	46.00
36	350	14.00	58.00	32.00	50.00
37	400	16.00	69.00	37.00	54.00
38	450	12.50	74.00	32.50	59.00

3 3

OF / DE

1.0

1.1

1.25

1.4

1.6

1.8

2.0

2.2

2.5

2.8

3.2

3.6

4.0

TEST NO. 4

4.1 m DEPTH

INCREMENT	DISPLACEMENT GAUGE OF PROVING RING (INCHES $\times 10^3$)	DISPLACEMENT w (INCHES $\times 10^3$)			
		GAUGE NO. 1	GAUGE NO. 2	GAUGE NO. 3	GAUGE NO. 4
1	25	1.500	2.500	0.000	19.00
2	50	38.00	-32.00	-7.500	46.00
3	75	42.50	-30.00	-3.000	45.00
4	100	45.5	-26.50	2.000	45.50
5	125	48.00	-22.00	6.000	47.00
6	150	51.00	-16.50	10.50	50.00
7	175	55.00	-12.00	15.00	54.00
8	200	56.00	-6.000	18.00	57.00
9	225	57.00	-3.000	20.50	60.00
10	250	60.00	3.000	26.00	62.00
11	275	64.00	6.500	30.00	64.00
12	300	65.00	10.00	31.50	68.00
13	325	66.00	18.00	36.00	72.00
14	350	68.00	22.00	39.00	74.00
15	19	59.00	-13.00	12.00	68.00
16	55	58.00	-10.00	14.50	66.00
17	100	58.00	-2.000	22.00	61.00
18	125	58.00	2.000	23.50	62.00
19	150	61.00	6.000	27.00	64.00
20	175	61.00	8.000	28.00	65.00
21	200	63.00	12.50	32.00	66.50
22	225	64.00	16.00	34.00	69.00
23	250	64.00	18.00	35.00	71.00
24	275	66.00	21.00	37.00	72.00
25	300	66.00	23.50	39.00	74.50
26	325	70.50	26.50	43.00	76.00
27	350	71.00	29.00	44.00	78.50
28	17	61.50	-7.500	16.00	73.00
29	60	61.00	2.000	20.50	68.50
30	100	61.00	5.000	26.00	66.00
31	150	63.00	12.00	31.00	68.00
32	200	64.00	17.00	34.50	70.00
33	250	66.00	22.00	39.00	74.00
34	300	68.00	26.50	42.00	77.00
35	350	72.50	33.00	47.00	81.00
36	400	75.00	38.00	50.00	85.00
37	450	79.00	45.00	55.50	89.00
38	500	81.00	51.00	60.00	94.50
39	550	86.00	59.00	66.00	98.00
40	600	90.00	66.00	72.00	106.0
41	650	96.00	76.50	80.00	110.0
42	700	102.0	85.00	87.00	119.0
43	750	109.0	97.00	99.00	125.0
44	800	117.0	110.0	107.0	140.0
45	12	91.50	27.00	51.00	99.00

TEST NO. 5

2.3 m DEPTH

INCREMENT	DISPLACEMENT GAUGE OF PROVING RING (INCHES $\times 10^3$)	DISPLACEMENT w (INCHES $\times 10^3$)			
		GAUGE NO. 1	GAUGE NO. 2	GAUGE NO. 3	GAUGE NO. 4
1	25	11.50	0.000	10.00	-7.000
2	50	19.00	1.000	17.00	-9.500
3	75	21.00	4.000	21.00	-11.00
4	100	28.00	8.000	29.00	-15.00
5	125	31.00	11.00	33.50	-17.00
6	150	43.50	17.00	47.00	-22.50
7	175	48.00	20.00	54.00	-23.50
8	200	53.00	22.00	58.00	-24.00
9	225	58.00	23.00	62.00	-25.00
10	250	62.00	26.00	67.00	-25.00
11	285	65.00	28.00	70.00	-25.00
12	300	70.00	32.00	76.00	-23.50
13	15	28.00	15.00	32.00	-13.50
14	55	30.00	15.00	34.50	-15.50
15	75	34.00	15.00	39.00	-17.00
16	100	42.00	20.00	47.00	-21.00
17	125	45.00	20.00	50.00	-23.00
18	150	49.00	21.00	54.00	-24.00
19	175	55.00	25.00	62.00	-27.00
20	200	59.00	26.00	65.00	-27.00
21	225	61.00	29.00	68.00	-26.50
22	250	62.00	30.00	70.00	-26.00
23	275	65.00	33.00	73.00	-25.50
24	300	67.00	35.00	76.00	-25.00
25	18	29.00	18.00	35.00	-12.50
26	50	32.00	17.00	37.00	-14.00
27	100	43.00	20.00	48.00	-19.00
28	150	54.00	25.00	60.00	-25.00
29	200	60.00	29.00	67.50	-26.00
30	250	63.00	32.00	72.00	-26.00
31	300	68.00	36.00	77.00	-24.50
32	350	75.00	44.00	86.00	-22.50
33	400	79.00	48.00	90.00	-20.00
34	450	84.00	54.00	96.00	-16.50
35	500	92.00	61.00	105.0	-12.00
36	550	100.0	66.00	112.0	-8.000
37	600	109.0	75.00	122.0	-1.000
38	650	126.0	88.00	140.0	7.500
39	9	42.00	30.00	43.00	18.00

TEST NO. 5

4.1 m DEPTH

INCREMENT	DISPLACEMENT GAUGE OF PROVING RING (INCHES $\times 10^3$)	DISPLACEMENT w (INCHES $\times 10^3$)			
		GAUGE NO. 1	GAUGE NO. 2	GAUGE NO. 3	GAUGE NO. 4
1	25	-15.00	13.00	-2.000	3.500
2	50	-15.50	14.00	-3.500	7.000
3	75	-13.00	17.50	0.000	10.00
4	100	-10.00	1.500	18.00	12.00
5	125	-5.000	21.00	6.000	16.50
6	150	0.000	23.00	10.00	18.50
7	175	8.000	27.00	16.00	23.00
8	200	15.00	31.00	23.00	27.00
9	225	17.00	32.00	24.00	29.00
10	250	19.50	34.00	27.00	31.50
11	275	24.00	37.00	31.00	34.00
12	300	31.00	44.00	40.00	38.00
13	15	2.000	17.00	7.000	22.00
14	50	3.000	20.00	8.500	23.50
15	75	4.000	20.00	9.000	24.00
16	100	6.000	23.00	12.00	25.50
17	125	9.000	26.00	15.50	27.00
18	150	14.00	30.00	21.00	28.50
19	175	17.00	34.00	25.00	29.00
20	200	18.00	34.00	26.00	30.50
21	230	22.00	37.00	30.00	33.50
22	250	25.00	40.00	33.00	35.00
23	275	28.00	42.00	36.00	37.00
24	300	30.00	45.00	45.00	39.50
25	20	7.000	24.00	14.00	25.00
26	50	6.000	23.00	13.00	26.00
27	100	9.000	26.00	15.00	28.00
28	150	16.00	34.00	24.00	30.00
29	200	21.00	38.00	30.00	33.50
30	250	25.00	40.00	34.00	36.50
31	300	30.00	43.00	37.00	40.00
32	350	36.00	51.00	46.00	44.50
33	400	40.00	54.00	49.00	48.00
34	450	41.00	58.00	52.00	52.00
35	500	49.00	63.00	58.00	55.50
36	550	59.00	71.00	67.00	64.50
37	600	61.00	78.00	71.00	70.00
38	650	66.00	82.00	76.00	76.00
39	700	73.00	88.00	82.00	81.00
40	750	80.00	93.00	89.00	87.50
41	800	86.00	99.00	95.00	93.00
42	25	14.00	42.00	22.00	43.50

TEST NO. 6

2.3 m DEPTH

INCREMENT	DISPLACEMENT GAUGE OF PROVING RING (INCHES $\times 10^3$)	DISPLACEMENT w (INCHES $\times 10^3$)			
		GAUGE NO. 1	GAUGE NO. 2	GAUGE NO. 3	GAUGE NO. 4
1	25	-2.000	-1.000	-1.000	1.000
2	50	2.000	3.000	2.000	-1.000
3	90	2.000	7.000	5.000	-2.000
4	125	1.000	14.00	11.00	-3.000
5	150	1.000	18.00	13.00	-1.000
6	175	1.000	24.00	16.00	0.000
7	200	1.000	28.00	18.00	1.000
8	225	2.000	33.00	21.00	4.000
9	250	2.000	36.00	22.00	6.000
10	277	3.000	41.00	26.00	8.000
11	305	3.000	44.00	27.00	10.00
12	10	5.000	10.00	7.000	8.000
13	25	5.000	10.00	7.000	8.500
14	60	5.000	12.50	9.000	7.000
15	106	5.000	18.00	13.50	5.000
16	151	5.000	23.00	16.00	6.000
17	192	5.000	30.00	20.00	7.500
18	222	5.000	35.00	22.50	8.000
19	270	5.000	43.00	27.00	9.500
20	300	5.000	48.00	29.00	11.00
21	85	5.000	6.000	3.500	13.00
22	52	5.000	10.00	7.000	10.50
23	106	5.000	19.00	14.00	6.000
24	150	8.000	24.00	19.00	7.000
25	210	6.000	35.00	23.00	8.000
26	250	6.000	40.00	26.00	9.000
27	300	6.000	48.00	30.00	11.00
28	348	7.000	52.00	32.00	15.00
29	400	9.000	63.00	38.00	21.00
30	460	6.000	39.00	66.00	16.00
31	500	8.000	71.00	42.00	18.00
32	552	8.000	78.00	45.00	21.00
33	600	8.000	86.00	50.00	25.00
34	650	10.00	96.00	56.00	31.00
35	700	12.00	110.0	64.00	38.00
36	750	13.00	128.0	72.00	48.00
37	800	15.00	143.0	80.00	58.00
38	850	18.00	163.0	90.00	65.00
39	900	00.00	213.0	100.0	78.00
40	9	45.00	68.00	59.00	44.00

TEST NO. 6

4.1 m DEPTH

INCREMENT	DISPLACEMENT GAUGE OF PROVING RING (INCHES $\times 10^3$)	DISPLACEMENT w (INCHES $\times 10^3$)			
		GAUGE NO. 1	GAUGE NO. 2	GAUGE NO. 3	GAUGE NO. 4
1	25	32.00	-21.00	9.000	6.000
2	50	88.00	-62.00	18.00	25.00
3	80	102.0	-68.00	22.00	31.00
4	100	106.0	-69.00	22.00	35.00
5	125	112.0	-70.00	25.00	36.00
6	150	119.0	-71.00	25.00	39.00
7	175	130.0	-68.00	37.00	43.00
8	200	136.0	-65.00	42.00	46.00
9	225	142.0	-65.00	46.00	47.00
10	250	148.0	-63.00	51.00	51.00
11	22	122.0	-64.00	36.00	35.00
12	50	124.0	-65.00	37.00	35.00
13	80	127.0	-64.50	39.00	36.00
14	100	129.0	-65.00	40.00	38.00
15	125	135.0	-64.00	43.00	43.00
16	150	138.0	-63.00	45.00	45.00
17	175	140.0	-63.00	46.00	46.00
18	200	147.0	-62.00	50.00	50.00
19	225	151.0	-62.00	53.00	51.00
20	250	155.0	-61.00	56.00	52.00
21	275	159.0	-60.00	58.00	54.00
22	300	163.0	-59.00	61.00	56.00
23	16	129.0	-62.00	41.00	37.00
24	50	129.0	-62.00	42.00	37.50
25	100	134.0	-62.00	44.00	40.00
26	150	142.0	-61.00	49.00	46.00
27	200	149.0	-61.00	54.00	48.00
28	250	156.0	-60.00	57.00	50.00
29	300	162.0	-58.00	62.00	55.00
30	350	176.0	-55.00	72.00	61.00
31	400	182.0	-50.00	78.00	66.00
32	450	190.0	-46.00	83.00	71.00
33	500	198.0	-41.00	89.00	80.00
34	550	205.0	-35.00	95.00	85.00
35	600	210.0	-30.00	102.0	93.00
36	650	220.0	-23.00	110.0	100.0
37	700	229.0	-15.00	120.0	110.0
38	15	156.0	-36.00	67.00	65.00

TEST NO. 7

2.3 m DEPTH

INCREMENT	DISPLACEMENT GAUGE OF PROVING RING (INCHES $\times 10^3$)	DISPLACEMENT w (INCHES $\times 10^3$)			
		GAUGE NO. 1	GAUGE NO. 2	GAUGE NO. 3	GAUGE NO. 4
1	25	6.000	3.500	5.000	-5.000
2	57	6.500	10.00	8.000	-5.000
3	75	6.500	13.50	10.00	-5.000
4	100	8.000	16.00	12.00	-5.000
5	125	10.00	18.00	14.50	-5.000
6	150	12.00	20.00	17.00	-4.000
7	175	17.00	22.00	20.00	-3.000
8	200	20.00	24.50	23.00	-0.500
9	225	28.00	26.00	28.00	-0.500
10	250	31.00	28.00	30.00	1.000
11	275	35.00	30.00	34.00	4.000
12	300	40.00	32.00	37.00	6.000
13	325	44.00	38.00	43.00	7.000
14	22	23.00	17.00	20.00	-1.500
15	50	23.00	18.00	21.00	-2.000
16	75	24.00	20.00	22.00	-1.000
17	100	26.00	21.50	24.00	0.000
18	125	28.00	23.00	26.00	1.000
19	150	30.00	24.00	27.50	3.000
20	175	33.00	26.00	30.00	3.500
21	210	36.00	28.00	33.00	5.000
22	225	38.00	29.00	34.00	5.000
23	250	40.00	30.00	36.00	6.500
24	275	42.00	33.00	39.00	8.000
25	300	46.00	35.00	42.00	9.000
26	325	49.00	37.00	44.00	11.50
27	20	27.00	18.00	22.00	1.500
28	50	27.00	20.00	23.00	1.500
29	100	30.00	23.00	27.00	3.000
30	150	34.00	26.00	30.00	6.000
31	200	39.00	29.00	34.00	8.500
32	250	43.00	32.00	39.00	10.00
33	300	49.00	35.00	43.00	12.00
34	350	52.00	40.00	47.00	14.00
35	400	59.00	45.00	54.00	16.50
36	450	64.00	51.00	61.00	19.00
37	500	71.00	58.00	68.00	23.00
38	550	80.00	66.00	78.00	29.00
39	600	88.00	76.00	86.00	38.50
40	650	97.00	87.00	97.00	46.50
41	700	110.0	98.00	110.0	56.00
42	750	123.0	113.0	124.0	67.00
43	800	135.0	128.0	139.0	79.0
44	850	150.0	150.0	160.0	98.00
45	26	89.00	89.00	88.00	63.50

TEST NO. 7

4.1 m DEPTH

INCREMENT	DISPLACEMENT GAUGE OF PROVING RING (INCHES $\times 10^3$)	DISPLACEMENT w (INCHES $\times 10^3$)			
		GAUGE NO. 1	GAUGE NO. 2	GAUGE NO. 3	GAUGE NO. 4
1	25	1.000	5.000	4.500	0.000
2	50	2.000	10.50	8.000	0.500
3	75	4.000	14.00	11.00	1.500
4	100	7.000	17.00	15.00	2.000
5	125	10.00	20.00	18.00	4.000
6	150	14.50	27.00	23.00	6.000
7	175	15.00	30.00	26.00	7.000
8	200	18.00	35.00	30.00	8.500
9	250	23.00	42.00	37.00	13.00
10	275	26.00	49.00	42.00	10.50
11	300	25.00	58.00	45.00	16.00
12	15	10.00	25.00	20.00	0.000
13	11	19.00	21.00	20.00	2.000
14	50	20.00	25.00	22.00	5.000
15	75	22.00	28.00	25.00	7.000
16	100	22.00	32.00	27.00	9.000
17	125	23.00	35.00	29.00	10.00
18	150	24.00	37.00	31.50	12.00
19	175	25.00	41.00	33.00	13.00
20	200	26.00	44.00	36.00	14.50
21	225	28.00	48.00	38.00	16.50
22	250	29.00	51.00	40.00	18.00
23	275	31.00	54.00	43.00	19.50
24	300	33.00	58.00	46.00	21.00
25	22	20.00	28.00	23.50	5.000
26	50	20.00	30.00	25.00	5.000
27	100	22.00	37.00	30.00	8.000
28	150	24.00	43.00	34.00	12.00
29	200	27.00	48.00	38.00	17.00
30	250	30.00	55.00	43.00	19.00
31	300	32.00	51.00	48.00	22.50
32	350	35.00	68.00	53.00	25.00
33	400	38.00	77.00	60.00	28.00
34	450	27.00	114.0	72.00	34.00
35	500	24.00	133.0	85.00	40.00
36	550	28.00	143.0	92.00	55.00
37	600	33.00	155.0	101.0	62.00
38	12	28.00	51.00	36.00	22.50

TEST NO. 8

2.3 m DEPTH

INCREMENT	DISPLACEMENT GAUGE OF PROVING RING (INCHES $\times 10^3$)	DISPLACEMENT w (INCHES $\times 10^3$)			
		GAUGE NO. 1	GAUGE NO. 2	GAUGE NO. 3	GAUGE NO. 4
1	25	0.000	2.000	2.000	1.000
2	50	0.000	7.000	4.000	3.000
3	75	0.000	9.500	4.000	7.000
4	100	-2.000	15.00	6.000	10.50
5	125	-3.000	22.00	8.000	10.50
6	150	-7.000	27.00	9.000	12.50
7	175	-10.00	32.00	11.00	15.50
8	200	-10.00	34.50	13.00	17.00
9	225	-9.000	38.00	15.00	20.00
10	250	-8.000	41.00	17.00	22.50
11	275	-7.000	45.00	20.00	24.00
12	300	-5.000	48.00	23.00	27.00
13	13	-1.000	13.00	5.500	9.000
14	50	-2.000	17.00	7.000	8.000
15	75	-2.500	21.00	9.000	10.50
16	100	-4.500	27.00	11.50	12.00
17	125	-4.000	30.00	12.50	15.00
18	150	-6.000	32.00	11.00	20.00
19	175	-9.000	37.00	13.00	20.50
20	200	-9.000	40.00	15.00	21.50
21	225	-8.000	42.00	18.00	22.00
22	250	-8.000	44.00	19.00	23.50
23	275	-7.000	47.00	22.00	25.00
24	300	-5.000	50.00	24.00	26.00
25	11	3.000	7.000	5.000	7.500
26	50	0.000	16.00	8.000	9.000
27	100	-2.000	27.00	13.00	11.00
28	150	-8.000	35.00	13.00	19.00
29	200	-8.000	40.00	16.00	20.50
30	250	-7.000	45.00	20.00	23.50
31	300	-5.000	50.00	24.00	26.00
32	350	1.000	53.00	30.00	29.00
33	400	5.000	58.00	34.00	33.50
34	450	10.00	65.00	40.00	39.50
35	500	17.00	74.00	48.00	48.00
36	550	25.00	83.00	57.00	57.00
37	600	35.00	95.00	70.00	70.00
38	650	50.00	110.0	85.00	85.00
39	700	70.00	130.0	104.0	106.0
40	750	100.0	160.0	130.0	135.0
41	800	140.0	200.0	170.0	175.0
42	15	144.0	143.0	138.0	143.0

TEST NO. 8

4.1 m DEPTH

INCREMENT	DISPLACEMENT GAUGE OF PROVING RING (INCHES $\times 10^3$)	DISPLACEMENT w (INCHES $\times 10^3$)			
		GAUGE NO. 1	GAUGE NO. 2	GAUGE NO. 3	GAUGE NO. 4
1	25	-4.000	8.000	1.000	4.000
2	50	-6.000	13.00	1.000	10.00
3	75	-6.000	19.00	2.000	15.00
4	100	-6.000	25.00	5.000	20.00
5	125	-6.000	33.00	8.000	25.50
6	150	-7.000	42.00	10.00	32.00
7	175	-7.000	46.00	12.00	35.50
8	200	-4.000	51.00	15.00	39.50
9	225	-6.000	57.00	16.00	45.50
10	250	-5.500	62.00	19.00	49.00
11	275	-5.000	67.00	22.00	53.50
12	300	-4.000	72.00	25.00	55.50
13	15	-7.000	42.00	10.00	32.00
14	50	-8.000	45.00	11.00	33.00
15	75	-8.000	47.00	12.00	35.00
16	100	-8.000	50.00	13.50	37.00
17	125	-7.000	54.00	15.50	41.00
18	150	-6.000	61.00	18.00	45.50
19	175	-6.000	64.00	19.00	48.00
20	200	-5.000	67.00	21.00	50.00
21	225	-4.000	70.00	22.00	54.50
22	250	-4.000	73.00	24.00	57.00
23	275	-3.000	77.00	26.00	59.50
24	300	-3.000	80.00	28.00	60.50
25	15	-7.000	50.00	13.00	38.00
26	50	-7.000	52.00	15.00	39.00
27	100	-6.000	57.00	17.00	43.50
28	150	-6.000	67.00	21.00	51.00
29	200	-4.000	72.00	23.00	57.00
30	250	-3.000	78.00	27.00	60.00
31	300	-2.000	84.00	30.00	63.50
32	350	4.000	90.00	36.00	68.50
33	400	6.000	96.00	40.00	73.00
34	450	12.00	105.0	48.00	79.50
35	500	17.00	112.0	54.00	87.00
36	550	24.00	120.0	62.00	94.50
37	600	27.00	130.0	68.00	100.0
38	650	37.00	140.0	78.00	109.0
39	700	45.00	150.0	87.00	120.0
40	750	55.00	165.0	100.0	133.0
41	800	65.00	180.0	110.0	145.0
42	850	80.00	195.0	125.0	159.5
43	15	48.00	150.0	173.0	95.00

TEST NO. 9

2.3 m DEPTH

INCREMENT	DISPLACEMENT GAUGE OF PROVING RING (INCHES $\times 10^3$)	DISPLACEMENT w (INCHES $\times 10^3$)			
		GAUGE NO. 1	GAUGE NO. 2	GAUGE NO. 3	GAUGE NO. 4
1	25	2.000	1.000	-3.000	0.000
2	50	6.000	3.500	7.500	-1.000
3	75	6.500	6.000	9.500	-5.000
4	100	8.000	7.500	11.50	1.000
5	125	9.000	12.00	14.00	3.000
6	150	9.000	14.00	16.00	5.500
7	175	7.000	17.00	13.00	16.00
8	200	8.000	20.00	13.00	20.00
9	225	10.00	32.00	18.00	15.00
10	250	15.00	32.00	21.50	17.00
11	275	18.00	36.00	25.50	18.00
12	300	25.00	35.00	30.00	18.50
13	15	21.00	18.00	19.00	-4.000
14	50	20.00	19.00	19.00	-1.000
15	75	20.00	20.00	19.00	0.000
16	100	19.00	22.00	20.00	2.000
17	125	13.00	28.00	19.50	5.000
18	150	10.00	32.00	19.50	8.000
19	175	10.00	33.00	19.50	9.000
20	200	16.00	32.00	23.00	12.00
21	225	26.00	30.00	29.00	13.00
22	250	30.00	30.00	31.00	13.00
23	275	38.00	29.00	34.00	13.00
24	300	48.00	27.00	40.00	11.00
25	15	28.00	17.00	25.00	-6.000
26	50	25.00	20.00	24.00	-3.000
27	100	24.50	23.00	25.00	-1.000
28	150	26.00	26.00	27.00	0.000
29	200	33.00	27.00	32.00	5.500
30	250	53.00	22.00	41.00	5.000
31	300	78.00	14.00	58.00	-6.000
32	350	128.0	-5.000	88.00	-25.00
33	400	158.0	-10.00	103.0	-29.50
34	450	168.0	-6.000	115.0	-29.00
35	18	78.00	-20.00	146.0	-65.00

TEST NO. 9

4.1 m DEPTH

INCREMENT	DISPLACEMENT GAUGE OF PROVING RING (INCHES $\times 10^3$)	DISPLACEMENT w (INCHES $\times 10^3$)			
		GAUGE NO. 1	GAUGE NO. 2	GAUGE NO. 3	GAUGE NO. 4
1	25	1.000	2.000	2.000	1.000
2	50	2.500	5.000	3.000	5.000
3	75	3.000	9.000	4.000	9.000
4	100	1.000	16.00	4.000	17.00
5	125	-2.000	26.00	4.000	28.00
6	150	14.00	5.000	-13.00	81.00
7	175	27.00	-9.000	-19.00	106.0
8	200	32.00	-10.00	-20.00	117.5
9	225	35.00	-8.000	-19.00	122.5
10	250	43.00	-11.00	-17.00	128.5
11	275	54.00	-20.00	-12.00	137.0
12	23	82.00	-45.00	-1.000	65.00
13	50	-0.500	2.000	0.000	12.00
14	75	-1.000	3.000	0.000	4.500
15	100	-4.000	10.00	-2.000	11.00
16	125	-16.00	24.00	-6.000	25.00
17	150	-26.00	34.00	-13.00	41.00
18	175	-20.00	34.00	-10.00	47.50
19	225	-3.000	22.00	-4.000	14.50
20	250	85.00	-45.00	33.00	17.00
21	275	94.00	45.00	46.00	16.50
22	300	96.00	-46.00	40.00	22.00
23	22	5.000	-27.00	3.000	36.50
24	50	-1.000	-22.00	1.000	42.50
25	100	-17.00	-4.000	-5.000	61.00
26	150	-33.00	8.000	-13.00	85.00
27	200	-20.00	6.000	-10.00	93.50
28	250	89.00	-62.00	33.00	64.00
29	300	93.00	-67.00	40.00	64.00
30	350	90.00	-50.00	40.00	67.00
31	400	99.00	-37.00	59.00	50.50
32	32	126.0	-70.00	78.00	-86.00

TEST NO. 10

2.3 m DEPTH

INCREMENT	DISPLACEMENT GAUGE OF PROVING RING (INCHES $\times 10^3$)	DISPLACEMENT w (INCHES $\times 10^3$)			
		GAUGE NO. 1	GAUGE NO. 2	GAUGE NO. 3	GAUGE NO. 4
1	25	5.000	-3.000	1.000	2.000
2	50	42.00	-68.00	-23.00	37.00
3	75	47.00	-68.00	-20.00	36.00
4	100	50.00	-67.00	-18.00	38.00
5	125	52.00	-66.00	-16.00	40.50
6	150	55.00	-65.00	-14.00	41.00
7	175	66.00	-75.00	-15.00	48.00
8	200	71.00	-74.00	-11.00	51.00
9	225	74.00	-74.00	-10.00	53.00
10	250	77.00	-71.00	-8.000	54.00
11	275	81.00	-69.00	-2.000	55.00
12	300	85.00	-69.00	0.000	58.00
13	325	88.00	-68.00	2.000	61.00
14	350	94.00	-64.00	5.000	70.00
15	10	47.00	-38.00	2.000	24.00
16	52	71.00	-69.00	-6.000	45.00
17	75	73.00	-70.00	-6.000	47.00
18	100	85.00	-79.00	-5.000	52.00
19	125	90.00	-80.00	-4.000	54.00
20	150	94.00	-80.00	-1.000	57.00
21	175	99.00	-80.00	1.000	60.00
22	200	104.0	-80.00	4.000	63.00
23	230	108.0	-79.00	6.000	66.00
24	250	110.0	-79.00	7.000	68.00

TEST NO. 10

2.3 m DEPTH
(cont'd)

INCREMENT	DISPLACEMENT GAUGE OF PROVING RING (INCHES $\times 10^3$)	DISPLACEMENT w (INCHES $\times 10^3$)			
		GAUGE NO. 1	GAUGE NO. 2	GAUGE NO. 3	GAUGE NO. 4
25	275	113.0	-79.00	8.000	71.00
26	300	117.0	-79.00	10.00	73.00
27	325	120.0	-78.00	12.00	76.00
28	350	123.0	-77.00	14.00	79.00
29	9	79.00	-75.00	-4.000	43.00
30	50	81.00	-74.00	-4.000	46.00
31	100	92.00	-80.00	-2.000	53.00
32	150	100.0	-80.00	2.000	58.00
33	200	108.0	-80.00	6.000	65.00
34	250	114.0	-80.00	10.00	69.00
35	300	121.0	-79.00	13.00	75.00
36	350	127.0	-78.00	16.00	80.00
37	400	134.0	-75.00	20.00	88.00
38	450	142.0	-73.00	26.00	96.00
39	500	154.0	-70.00	32.00	106.0
40	550	165.0	-68.00	38.00	115.0
41	600	179.0	-64.00	47.00	129.0
42	650	192.0	-59.0	56.00	141.0
43	700	204.0	-54.00	65.00	152.0
44	750	220.0	-45.00	77.00	168.0
45	850	231.0	-38.00	85.00	180.0
46	900	243.0	-32.00	90.00	191.0
47	950	254.0	-25.00	100.0	208.0
48	20	90.00	10.00	46.00	71.00

TEST NO. 10

4.1 m DEPTH

INCREMENT	DISPLACEMENT GAUGE OF PROVING RING (INCHES $\times 10^3$)	DISPLACEMENT w (INCHES $\times 10^3$)			
		GAUGE NO. 1	GAUGE NO. 2	GAUGE NO. 3	GAUGE NO. 4
1	35	11.00	1.000	13.00	-2.000
2	50	22.00	-1.000	16.00	-2.000
3	75	31.00	-5.000	20.00	0.000
4	100	37.00	-4.000	24.00	2.000
5	125	42.00	-3.000	27.00	5.000
6	150	48.00	-3.000	30.00	10.00
7	175	52.00	-2.000	32.00	13.00
8	200	58.00	4.000	39.00	16.00
9	225	66.00	11.00	48.00	20.00
10	250	68.00	15.00	50.00	22.00
11	275	72.00	19.00	54.00	27.00
12	300	75.00	22.00	57.00	30.00
13	5	62.00	-8.000	37.00	1.000
14	50	63.00	-7.000	40.00	4.000
15	75	64.00	-5.000	41.00	7.000
16	100	66.00	-5.000	42.00	9.000
17	125	68.00	-3.000	43.00	12.00
18	150	68.00	1.000	45.00	15.00
19	175	71.00	4.000	47.00	18.00
20	200	72.00	8.000	50.00	20.00
21	225	73.00	12.00	52.00	24.00
22	250	75.00	15.00	54.00	27.00
23	275	77.00	19.00	56.00	30.00
24	300	79.00	23.00	59.00	32.00
25	3	65.00	-11.00	37.00	1.000
26	60	65.00	-8.000	41.00	4.000
27	100	68.00	-7.000	42.00	8.000
28	150	72.00	-2.000	46.00	14.00
29	200	75.00	7.000	51.00	19.00
30	250	78.00	14.00	56.00	25.00
31	300	80.00	21.00	60.00	30.00
32	350	87.00	28.00	68.00	37.00
33	400	92.00	36.00	73.00	44.00
34	450	98.00	43.00	80.00	50.00
35	500	106.0	49.00	88.00	56.00
36	550	110.0	56.00	95.00	64.00
37	600	120.0	65.00	102.0	71.00
38	650	128.0	75.00	112.0	80.00
39	700	141.0	87.00	124.0	91.00
40	750	150.0	95.00	130.0	99.00
41	800	160.0	105.0	140.0	109.0
42	850	176.0	116.0	155.0	125.0
43	900	198.0	133.0	175.0	145.0
44	950	210.0	145.0	185.0	157.0
45	1000	250.0	165.0	220.0	172.0
46	5	166.0	95.00	150.0	82.00

TEST NO. 11

2.3 m DEPTH

INCREMENT	DISPLACEMENT GAUGE OF PROVING RING (INCHES $\times 10^3$)	DISPLACEMENT w (INCHES $\times 10^3$)			
		GAUGE NO. 1	GAUGE NO. 2	GAUGE NO. 3	GAUGE NO. 4
1	25	0.000	3.000	2.000	1.000
2	60	1.000	9.000	5.000	5.000
3	100	5.000	16.00	11.00	7.000
4	125	5.000	23.00	14.00	10.00
5	150	5.000	27.00	16.00	12.00
6	175	7.000	31.00	19.00	15.00
7	200	15.00	25.00	20.00	24.00
8	225	17.00	27.00	21.00	30.00
9	250	19.00	30.00	22.00	34.00
10	275	19.00	34.00	24.00	38.00
11	300	20.00	37.00	26.00	40.00
12	9	13.00	8.000	9.000	20.00
13	50	13.00	14.00	11.00	20.00
14	75	13.00	16.00	13.00	20.00
15	100	15.00	18.00	16.00	17.00
16	125	15.00	24.00	19.00	19.00
17	150	19.00	21.00	18.00	27.00
18	175	19.00	23.00	19.00	30.00
19	200	20.00	26.00	21.00	32.00
20	225	20.00	29.00	22.00	33.00
21	250	21.00	23.00	24.00	36.00
22	275	21.00	36.00	26.00	38.00
23	300	23.00	40.00	28.00	40.00
24	325	23.00	44.00	29.00	42.00
25	350	23.00	48.00	31.00	45.00
26	8	17.00	11.00	12.00	23.00
27	50	18.00	15.00	15.00	25.00
28	100	19.00	20.00	19.00	21.00
29	150	23.00	22.00	20.00	31.00
30	200	25.00	27.00	24.00	35.00
31	250	26.00	34.00	27.00	40.00
32	300	26.00	40.00	30.00	43.00
33	350	26.00	48.00	32.00	47.00
34	400	26.00	55.00	35.00	35.00
35	450	27.00	63.00	37.00	64.00
36	500	27.00	71.00	40.00	75.00
37	550	29.00	84.00	44.00	90.00
38	600	31.00	95.00	48.00	104.0
39	650	30.00	110.0	50.00	124.0
40	700	38.00	130.0	63.00	141.0
41	8	60.00	5.000	23.00	84.00

TEST NO. 11

4.1 m DEPTH

INCREMENT	DISPLACEMENT GAUGE OF PROVING RING (INCHES $\times 10^3$)	DISPLACEMENT w (INCHES $\times 10^3$)			
		GAUGE NO. 1	GAUGE NO. 2	GAUGE NO. 3	GAUGE NO. 4
1	30	12.00	-9.000	0.000	11.00
2	50	43.00	-39.00	-4.000	39.00
3	75	44.00	-35.00	-3.000	43.00
4	100	45.00	-30.00	-1.000	47.00
5	125	45.00	-22.00	2.000	50.00
6	150	45.00	-18.00	4.000	54.00
7	175	47.00	-14.00	7.000	58.00
8	200	47.00	-10.00	8.000	62.00
9	225	52.00	-6.000	12.00	66.00
10	250	54.00	-4.000	14.00	70.00
11	275	56.00	0.000	16.00	74.00
12	300	54.00	15.00	17.00	77.00
13	13	13.00	-4.000	-9.000	54.00
14	50	42.00	-22.00	-4.000	64.00
15	75	45.00	-20.00	0.000	64.00
16	100	48.00	-18.00	3.000	65.00
17	125	48.00	-13.00	5.000	66.00
18	150	48.00	-10.00	8.000	67.00
19	175	50.00	-6.000	9.000	69.00
20	200	52.00	-4.000	12.00	71.00
21	225	53.00	-2.000	13.00	75.00
22	250	54.00	0.000	14.00	76.00
23	275	55.00	4.000	16.00	79.00
24	300	58.00	5.000	15.00	86.00
25	15	14.00	-15.00	-22.00	74.00
26	50	32.00	-20.00	-11.00	74.00
27	100	44.00	-17.00	0.000	71.00
28	150	47.00	-9.000	6.000	73.00
29	200	53.00	-4.000	10.00	77.00
30	250	55.00	0.000	13.00	83.00
31	300	58.00	5.000	17.00	86.00
32	350	64.00	10.00	22.00	92.00
33	400	72.00	15.00	28.00	99.00
34	450	75.00	25.00	35.00	102.0
35	500	78.00	32.00	40.00	110.0
36	550	86.00	43.00	49.00	120.0
37	600	93.00	54.00	57.00	126.0
38	650	98.00	67.00	69.00	130.0
39	700	105.0	80.00	79.00	137.0
40	750	114.0	95.00	90.00	146.0
41	800	124.0	110.0	104.0	155.0
42	850	132.0	128.0	118.0	165.0
43	900	150.0	150.0	138.0	182.0
44	14	115.0	25.00	54.00	140.0

TEST NO. 12

2.3 m DEPTH

INCREMENT	DISPLACEMENT GAUGE OF PROVING RING (INCHES. $\times 10^3$)	DISPLACEMENT w (INCHES $\times 10^3$)			
		GAUGE NO. 1	GAUGE NO. 2	GAUGE NO. 3	GAUGE NO. 4
1	25	7.000	5.000	10.00	-4.000
2	50	15.00	8.000	19.00	-7.000
3	75	18.00	16.00	27.00	-8.000
4	100	20.00	26.00	37.00	-9.000
5	125	21.00	31.00	42.00	-10.00
6	150	23.00	34.00	45.00	-10.00
7	175	24.00	38.00	49.00	-9.000
8	200	24.00	46.00	55.00	-9.000
9	225	26.00	48.00	57.00	-8.000
10	250	30.00	50.00	59.00	-5.000
11	275	32.00	61.00	61.00	-4.000
12	300	35.00	54.00	64.00	-2.000
13	325	38.00	57.00	67.00	0.000
14	350	40.00	59.00	70.00	2.000
15	17	18.00	25.00	30.00	-7.000
16	50	19.00	28.00	33.00	-7.000
17	75	22.00	35.00	41.00	-7.000
18	100	22.00	43.00	47.00	-7.000
19	125	25.00	46.00	53.00	-7.000
20	150	26.00	49.00	55.00	-7.000
21	175	28.00	52.00	59.00	-7.000
22	200	30.00	56.00	62.00	-6.000
23	225	31.00	57.00	64.00	-5.000
24	250	32.00	59.00	65.00	-5.000

TEST NO. 12

2.3 m DEPTH

INCREMENT	DISPLACEMENT GAUGE OF PROVING RING (INCHES $\times 10^3$)	DISPLACEMENT w (INCHES $\times 10^3$)			
		GAUGE NO. 1	GAUGE NO. 2	GAUGE NO. 3	GAUGE NO. 4
25	275	35.00	60.00	67.00	-3.000
26	300	37.00	62.00	70.00	-2.000
27	325	39.00	64.00	72.00	0.000
28	350	41.00	68.00	74.00	2.000
29	20	22.00	30.00	35.00	-6.000
30	50	22.00	31.00	37.00	-5.000
31	100	24.00	46.00	50.00	-6.000
32	150	27.00	53.00	58.00	-6.000
33	200	31.00	58.00	63.00	-5.000
34	250	34.00	60.00	67.00	-3.000
35	300	38.00	63.00	70.00	-1.000
36	350	41.00	68.00	75.00	2.000
37	400	47.00	74.00	82.00	4.000
38	450	50.00	78.00	85.00	8.000
39	500	55.00	84.00	90.00	12.00
40	550	60.00	92.00	96.00	17.00
41	600	68.00	101.0	105.0	24.00
42	650	78.00	115.0	120.0	31.00
43	700	86.00	125.0	130.0	40.00
44	750	95.00	140.0	142.0	49.00
45	800	110.0	164.0	164.0	65.00
46	850	125.0	190.0	185.0	80.0
47	900	145.0	230.0	220.0	103.0
48	20	77.00	75.00	52.00	70.00

TEST NO. 12

4.1 m DEPTH

INCREMENT	DISPLACEMENT GAUGE OF PROVING RING (INCHES $\times 10^3$)	DISPLACEMENT w (INCHES $\times 10^3$)			
		GAUGE NO. 1	GAUGE NO. 2	GAUGE NO. 3	GAUGE NO. 4
1	25	1.000	4.000	2.000	2.000
2	50	6.000	4.000	5.000	6.000
3	75	17.00	0.000	7.000	14.00
4	100	26.00	-3.000	10.00	21.00
5	125	41.00	-11.00	11.00	31.00
6	150	46.00	-12.00	13.00	35.00
7	175	51.00	-12.00	16.00	38.00
8	200	57.00	-11.00	19.00	41.00
9	225	62.00	-11.00	22.00	45.00
10	250	66.00	-9.000	24.00	49.00
11	300	70.00	-6.000	27.00	56.00
12	325	73.00	-4.000	29.00	59.00
13	350	76.00	-5.000	31.00	62.00
14	15	42.00	-6.000	7.000	37.00
15	50	51.00	-24.00	8.000	41.00
16	75	53.00	-22.00	9.000	44.00
17	100	56.00	-20.00	12.00	45.00
18	125	58.00	-20.00	14.00	46.00
19	150	62.00	-19.00	16.00	49.00
20	175	64.00	-17.00	18.00	50.00
21	200	66.00	-12.00	21.00	52.00
22	225	68.00	-10.00	23.00	55.00
23	250	70.00	-10.00	25.00	57.00
24	275	72.00	-9.000	26.00	59.00

TEST NO. 12

4.1 m DEPTH
(cont'd)

INCREMENT	DISPLACEMENT GAUGE OF PROVING RING (INCHES $\times 10^3$)	DISPLACEMENT w (INCHES $\times 10^3$)			
		GAUGE NO. 1	GAUGE NO. 2	GAUGE NO. 3	GAUGE NO. 4
25	300	75.00	-9.000	27.00	61.00
26	325	77.00	-8.000	29.00	64.00
27	350	81.00	-7.000	31.00	67.00
28	.16	50.00	-22.00	7.000	42.00
29	50	54.00	-23.00	9.000	45.00
30	100	60.00	-19.00	14.00	49.00
31	150	65.00	-19.00	18.00	51.00
32	200	68.00	-9.000	23.00	56.00
33	250	72.00	-7.000	26.00	60.00
34	300	75.00	-7.000	29.00	63.00
35	350	82.00	-6.000	31.00	69.00
36	400	88.00	-5.000	35.00	74.00
37	450	95.00	-4.000	40.00	80.00
38	500	104.0	-1.000	45.00	86.00
39	550	112.0	3.000	51.00	93.00
40	600	125.0	10.00	62.00	104.0
41	650	134.0	18.00	70.00	111.0
42	700	145.0	25.00	79.00	120.0
43	750	158.0	32.00	90.00	130.0
44	800	187.0	46.00	110.0	143.0
45	850	196.0	70.00	129.0	160.0
46	900	230.0	110.0	165.0	187.0
47	15	30.00	163.0	99.00	60.00

TEST NO. 1

2.3 m DEPTH

INCREMENT	LOAD (KN)	TOTAL DISPLACEMENT (mm)	SHAFT DEFLECTION (mm)	NET DISPLACEMENT (mm)
1	0.1570	0.0095	0.0032	0.0063
2	0.3140	0.1143	0.0064	0.1785
3	0.4711	0.2349	0.0096	0.2252
4	0.6281	0.3429	0.0128	0.3300
5	0.7851	0.0000	0.0161	-0.016
6	0.9421	0.0190	0.0193	-0.0002
7	1.0992	0.0539	0.0225	0.0314
8	1.2563	0.0730	0.0257	0.0472
9	1.4133	0.1206	0.0290	0.0916
10	1.5703	0.1778	0.0322	0.1455
11	1.7274	0.2032	0.0354	0.1677
12	1.8844	0.2571	0.0386	0.2185
13	2.0414	0.2857	0.0418	0.2438
14	2.1985	0.3302	0.0451	0.2850
15	2.3555	0.3683	0.0483	0.3199
16	2.6695	0.3778	0.0547	0.3230
17	2.8266	0.4064	0.0580	0.3483
18	2.9836	0.4349	0.0612	0.3737
19	3.1406	0.4762	0.0644	0.4118
20	3.2977	0.5143	0.0676	0.4466
21	3.4547	0.5397	0.0708	0.4688
22	3.6117	0.5842	0.0741	0.5100
23	3.7688	0.6286	0.0773	0.5513
24	3.9258	0.6635	0.0805	0.5830
25	2.3555	0.6318	0.0483	0.5834
26	2.5125	0.6477	0.0515	0.5961
27	2.6695	0.6572	0.0547	0.6024
28	2.8266	0.6858	0.0580	0.6277
29	2.9836	0.7112	0.0612	0.6499
30	3.1406	0.7366	0.0644	0.6721
31	3.2977	0.7747	0.0676	0.7070
32	3.4547	0.8318	0.0708	0.7609
33	3.6117	0.8921	0.0741	0.8180
34	3.7688	0.9779	0.0773	0.9005
35	3.9258	1.0096	0.0805	0.9290
36	2.3555	0.9493	0.0483	0.9009
37	2.5125	0.9715	0.0515	0.9199
38	2.6695	0.9810	0.0547	0.9262
39	2.8266	0.9969	0.0580	0.9389
40	2.9836	1.0033	0.0612	0.9420
41	3.1406	1.0096	0.0644	0.9452
42	3.2977	1.0192	0.0676	0.9515
43	3.4547	1.0319	0.0708	0.9609
44	3.7688	1.0763	0.0773	0.9989

TEST NO. 1

4.1 m DEPTH

INCREMENT	LOAD (KN)	TOTAL DISPLACEMENT (mm)	SHAFT DEFLECTION (mm)	NET DISPLACEMENT (mm)
1	2.5125	0.0412	0.0863	-0.0450
2	2.6695	0.0889	0.0917	-0.0028
3	2.8266	0.1778	0.0971	0.0806
4	2.9386	0.2159	0.1025	0.1134
5	3.1406	0.2762	0.1078	0.1683
6	3.2977	0.3365	0.1132	0.2232
7	3.4547	0.4000	0.1186	0.2813
8	3.6117	0.4540	0.1240	0.3299
9	3.7688	0.5143	0.1294	0.3848
10	3.9258	0.5842	0.1348	0.4493
11	4.0828	0.6318	0.1402	0.4915
12	4.2399	0.6858	0.1456	0.5401
13	4.3969	0.7493	0.1510	0.5982
14	4.5539	0.8255	0.1564	0.6690
15	4.7110	0.9017	0.1618	0.7398
16	2.3553	0.7143	0.0809	0.6334
17	2.7481	0.7429	0.0944	0.6485
18	3.1406	0.7937	0.1078	0.6858
19	3.2977	0.8128	0.1132	0.6995
20	3.4547	0.8318	0.1186	0.7131
21	3.6117	0.8604	0.1240	0.7363
22	3.7688	0.8858	0.1294	0.7563
23	3.9258	0.9144	0.1348	0.7795
24	4.0828	0.9334	0.1402	0.7931
25	4.2399	0.9620	0.1456	0.8163
26	4.3969	0.9969	0.1510	0.8459
27	4.5539	1.0446	0.1564	0.8881
28	4.7110	1.0763	0.1618	0.9144
29	3.1406	0.9747	0.1078	0.8668
30	3.2977	0.9747	0.1132	0.8614
31	3.4547	0.9874	0.1186	0.8687
32	3.6117	0.9969	0.1240	0.8728
33	3.7688	1.0160	0.1294	0.8865
34	3.9258	1.0319	0.1348	0.8970
35	4.0828	1.0636	0.1402	0.9233
36	4.2399	1.0795	0.1456	0.9338
37	4.3969	1.1112	0.1510	0.9602
38	4.5539	1.1366	0.1564	0.9802
39	4.7110	1.1716	0.1618	1.0097
40	4.8680	1.2446	0.1672	1.0774
41	5.0250	1.2827	0.1726	1.1101
42	5.3391	1.3716	0.1834	1.1882
43	5.4961	1.4414	0.1888	1.2526
44	2.3555	1.1366	0.0809	1.0557

TEST NO. 2

2.3 m DEPTH

INCREMENT	LOAD (KN)	TOTAL DISPLACEMENT (mm)	SHAFT DEFLECTION (mm)	NET DISPLACEMENT (mm)
1	1.7274	0.0508	0.0354	0.0153
2	1.8844	0.2032	0.0386	0.1645
3	2.0414	0.2603	0.0418	0.2184
4	2.1985	0.3048	0.0451	0.2596
5	2.3555	0.4191	0.0483	0.3707
6	2.5125	0.4857	0.0515	0.4342
7	2.6695	0.5556	0.0547	0.5008
8	2.8266	0.6159	0.0580	0.5579
9	2.9836	0.6794	0.0612	0.6182
10	3.1406	0.7588	0.0644	0.6943
11	1.5703	0.5016	0.0322	0.4694
12	1.7274	0.5651	0.0354	0.5297
13	1.8844	0.6096	0.0386	0.5709
14	2.0414	0.6635	0.0418	0.6216
15	2.1985	0.7207	0.0451	0.6756
16	2.3555	0.7683	0.0483	0.7200
17	2.6695	0.8636	0.0547	0.8088
18	2.9836	0.9588	0.0612	0.8976
19	3.2977	1.0763	0.0676	1.0087
20	2.3555	0.9239	0.0483	0.8755
21	2.6695	1.0001	0.0547	0.9453
22	3.1406	1.1017	0.0644	1.0373
23	2.3555	0.7874	0.0483	0.8760
24	2.8266	0.8953	0.0580	0.9740
25	3.1406	0.9842	0.0644	1.0560
26	3.4547	1.0827	0.0708	1.1480
27	3.6117	1.1335	0.0741	1.1960
28	3.7688	1.1779	0.0773	1.2370
29	3.9258	1.2287	0.0805	1.2850
30	1.5703	1.0541	0.0322	1.1580

TEST NO. 3

2.3 m DEPTH

INCREMENT	LOAD (KN)	TOTAL DISPLACEMENT (mm)	SHAFT DEFLECTION (mm)	NET DISPLACEMENT (mm)
1	0.1570	0.0254	0.0043	0.0210
2	0.3140	0.0476	0.0086	0.0389
3	0.4711	0.0730	0.0129	0.0600
4	0.6281	0.0952	0.0172	0.0779
5	0.7851	0.1174	0.0216	0.0958
6	0.9421	0.1492	0.0259	0.1233
7	1.0992	0.1778	0.0302	0.1475
8	1.2563	0.2159	0.0345	0.1813
9	1.4133	0.2286	0.0388	0.1897
10	1.5703	0.2698	0.0432	0.2266
11	0.1334	0.0952	0.0036	0.0915
12	0.1570	0.0920	0.0043	0.0877
13	0.3140	0.0984	0.0086	0.0897
14	0.5496	0.1238	0.0151	0.1087
15	0.6281	0.1301	0.0172	0.1128
16	0.7851	0.1333	0.0216	0.1117
17	0.9421	0.1460	0.0259	0.1201
18	1.0992	0.1651	0.0302	0.1348
19	1.2563	0.1905	0.0345	0.1559
20	1.4133	0.2159	0.0388	0.1770
21	1.5703	0.2508	0.0432	0.2076
22	1.7274	0.2698	0.0475	0.2223
23	1.8844	0.3111	0.0518	0.2593
24	2.0414	0.3302	0.0561	0.2740
25	2.1985	0.3651	0.0604	0.3046
26	2.3555	0.4191	0.0648	0.3543
27	0.1413	0.1428	0.0038	0.1389
28	0.3140	0.1460	0.0086	0.1374
29	0.6281	0.1619	0.0172	0.1446
30	0.7851	0.1746	0.0216	0.1530
31	0.9421	0.1841	0.0259	0.1582
32	1.0992	0.2222	0.0302	0.1920
33	1.2563	0.2317	0.0345	0.1972
34	1.5703	0.2857	0.0432	0.2425
35	1.7274	0.3175	0.0475	0.2699
36	1.8844	0.3524	0.0184	0.3005
37	2.0414	0.3841	0.0561	0.3280
38	2.1985	0.3968	0.0604	0.3363
39	2.3555	0.4349	0.0648	0.3701
40	2.5125	0.4572	0.0691	0.3880
41	2.6695	0.5016	0.0734	0.4282
42	2.8266	0.5334	0.0777	0.4556
43	2.9836	0.5810	0.0820	0.4989
44	3.1406	0.6318	0.0864	0.5454

TEST NO. 3

2.3 m DEPTH
(cont'd)

INCREMENT	LOAD (KN)	TOTAL DISPLACEMENT (mm)	SHAFT DEFLECTION (mm)	NET DISPLACEMENT (mm)
45	3.4547	0.6762	0.0950	0.5812
46	3.7688	0.7651	0.1036	0.6614
47	4.0828	0.8572	0.1123	0.7449
48	4.3969	0.9588	0.1209	0.8378
49	4.7110	1.0668	0.1296	0.9371
50	5.0250	1.1493	0.1382	1.0111
51	5.4961	1.3240	0.1512	1.1728
52	5.6532	1.4224	0.1555	1.2669
53	5.8102	1.5208	0.1598	1.3610
54	5.9672	1.6097	0.1641	1.4456
55	0.1491	0.6762	0.0041	0.6721

TEST NO. 3

4.1 m DEPTH

INCREMENT	LOAD (KN)	TOTAL DISPLACEMENT (mm)	SHAFT DEFLECTION (mm)	NET DISPLACEMENT (mm)
1	0.3140	0.0857	0.0097	0.0760
2	0.4711	0.1428	0.0145	0.1283
3	0.7851	0.2349	0.0245	0.2106
4	1.0992	0.2857	0.0339	0.2517
5	1.2563	0.3016	0.0388	0.2628
6	1.4133	0.3333	0.0436	0.2897
7	1.5703	0.3714	0.0485	0.3229
8	1.7274	0.4000	0.0533	0.3466
9	1.8844	0.4508	0.0582	0.3926
10	2.0414	0.4603	0.0630	0.3973
11	2.3555	0.5397	0.0727	0.4669
12	0.1491	0.2571	0.0046	0.2525
13	0.3140	0.2794	0.0097	0.2697
14	0.6673	0.2984	0.0206	0.2778
15	0.9421	0.3270	0.0291	0.2979
16	1.2563	0.3619	0.0388	0.3231
17	1.5703	0.4095	0.0485	0.3610
18	1.8844	0.4730	0.0582	0.4148
19	2.1985	0.5556	0.0679	0.4877
20	2.5125	0.6223	0.0776	0.5446
21	2.8266	0.7556	0.0873	0.6683
22	3.1406	0.8540	0.0970	0.7570
23	0.1256	0.3302	0.0038	0.3263
24	0.3925	0.3429	0.0121	0.3307
25	0.7851	0.3746	0.0242	0.3503
26	1.1777	0.4445	0.0363	0.4081
27	1.5703	0.5492	0.0485	0.5007
28	1.9629	0.6000	0.0606	0.5394
29	2.3555	0.6667	0.0727	0.5939
30	2.7481	0.7969	0.0849	0.7120
31	3.1406	0.8445	0.0970	0.7475
32	3.5332	0.9842	0.1091	0.8750
33	3.9258	1.0858	0.1213	0.9645
34	4.3184	1.1906	0.1334	1.0572
35	4.7110	1.3494	0.1455	1.2038
36	5.1035	1.4700	0.1576	1.3124
37	5.4961	1.6224	0.1698	1.4526
38	5.8887	1.7589	0.1819	1.5770
39	6.2813	1.9812	0.1941	1.7871
40	6.6739	2.2098	0.2062	2.0036
41	7.0664	2.4384	0.2183	2.2201
42	7.4590	2.6924	0.2304	2.4620
43	7.8516	3.0734	0.2425	2.8308
44	8.2442	3.3655	0.2547	3.1108

TEST NO. 4

2.3 m DEPTH

INCREMENT	LOAD (KN)	TOTAL DISPLACEMENT (mm)	SHAFT DEFLECTION (mm)	NET DISPLACEMENT (mm)
1	0.1962	0.1619	0.0040	0.1579
2	0.3925	0.1936	0.0080	0.1856
3	0.5888	0.2286	0.0122	0.2164
4	0.7851	0.2857	0.0161	0.2695
5	0.9814	0.3587	0.0202	0.3385
6	1.1777	0.3937	0.0242	0.3694
7	1.3740	0.4349	0.0283	0.4066
8	1.5703	0.5016	0.0323	0.4693
9	1.7666	0.5238	0.0363	0.4874
10	1.9629	0.5778	0.0404	0.5374
11	2.1592	0.6301	0.0444	0.5937
12	2.3555	0.6889	0.0485	0.6404
13	2.5518	0.7461	0.0525	0.6935
14	2.7481	0.8128	0.0566	0.7562
15	0.1413	0.4222	0.0029	0.4193
16	0.4475	0.4318	0.0092	0.4225
17	0.5888	0.4508	0.0121	0.4387
18	0.7851	0.4889	0.0161	0.4727
19	0.9814	0.5048	0.0202	0.4846
20	1.1777	0.5651	0.0242	0.5408
21	1.3740	0.5905	0.0283	0.5622
22	1.5703	0.6223	0.0323	0.5899
23	1.7666	0.6540	0.0363	0.6176
24	1.9629	0.6985	0.0404	0.6580
25	2.1592	0.7493	0.0444	0.7048
26	2.3555	0.8191	0.0485	0.7706
27	2.5518	0.8477	0.0525	0.7951
28	2.7481	0.9525	0.0566	0.8959
29	0.1334	0.4984	0.0027	0.4957
30	0.4318	0.5016	0.0088	0.4927
31	0.7851	0.5619	0.0161	0.5458
32	1.1777	0.6286	0.0242	0.6043
33	1.5703	0.6921	0.0323	0.6598
34	1.9629	0.7778	0.0404	0.7374
35	2.3555	0.8636	0.0485	0.8150
36	2.7481	0.9779	0.0566	0.9213
37	3.1406	1.1176	0.0646	1.0529
38	3.5332	1.1303	0.0727	1.0575

TEST NO. 4

4.1 m DEPTH

INCREMENT	LOAD (KN)	TOTAL DISPLACEMENT (mm)	SHAFT DEFLECTION (mm)	NET DISPLACEMENT (mm)
1	0.1962	0.1460	0.0060	0.1400
2	0.3925	0.2825	0.0121	0.2704
3	0.5888	0.3460	0.0181	0.3278
4	0.7851	0.4222	0.0242	0.3980
5	0.9814	0.5016	0.0303	0.4713
6	1.1777	0.6032	0.0363	0.5668
7	1.3740	0.7112	0.0424	0.6687
8	1.5703	0.7937	0.0485	0.7452
9	1.7666	0.8540	0.0545	0.7995
10	1.9629	0.9588	0.0606	0.8982
11	2.1592	1.0446	0.0667	0.9778
12	2.3555	1.1081	0.0727	1.0353
13	2.5518	1.2192	0.0788	1.1404
14	2.7481	1.2890	0.0849	1.2041
15	0.1491	0.8001	0.0046	0.7954
16	0.4318	0.8159	0.0133	0.8026
17	0.7851	0.8826	0.0242	0.8583
18	0.9814	0.9239	0.0303	0.8936
19	1.1777	1.0033	0.0363	0.9669
20	1.3740	1.0287	0.0424	0.9862
21	1.5703	1.1049	0.0485	1.0564
22	1.7666	1.1620	0.0545	1.1075
23	1.9629	1.1938	0.0606	1.1332
24	2.1592	1.2446	0.0667	1.1779
25	2.3555	1.2890	0.0727	1.2163
26	2.5518	1.3716	0.0788	1.2928
27	2.7481	1.4129	0.0849	1.3280
28	0.1334	0.9080	0.0041	0.9039
29	0.4711	0.9398	0.0145	0.9252
30	0.7851	1.0033	0.0242	0.9790
31	1.1777	1.1049	0.0363	1.0685
32	1.5703	1.1779	0.0485	1.1294
33	1.9629	1.2763	0.0606	1.2157
34	2.3555	1.3557	0.0727	1.2830
35	2.7481	1.4827	0.0849	1.3978
36	3.1406	1.5748	0.0970	1.4774
37	3.5332	1.7050	0.1091	1.5958
38	3.9257	1.8193	0.1212	1.6980
39	4.3184	1.9621	0.1334	1.8287
40	4.7110	2.1209	0.1455	1.9784
41	5.1035	2.3019	0.1576	2.1442
42	5.4961	2.4955	0.1698	2.3257
43	5.8887	2.7305	0.1819	2.5486
44	6.2813	3.0099	0.1940	2.8158
45	0.0942	1.7050	0.0029	1.7021

TEST NO. 5

2.3 m DEPTH

INCREMENT	LOAD (KN)	TOTAL DISPLACEMENT (mm)	SHAFT DEFLECTION (mm)	NET DISPLACEMENT (mm)
1	0.1962	0.0920	0.0040	0.0880
2	0.3925	0.1746	0.0080	0.1665
3	0.5888	0.2222	0.0121	0.2101
4	0.7851	0.3175	0.0161	0.3013
5	0.9814	0.3714	0.0202	0.3512
6	1.1777	0.5397	0.0242	0.5154
7	1.3740	0.6254	0.0283	0.5971
8	1.5703	0.6921	0.0323	0.6598
9	1.7666	0.7493	0.0363	0.7129
10	1.9629	0.8255	0.0404	0.7850
11	2.2377	0.8763	0.0460	0.8302
12	2.3555	0.9810	0.0485	0.9325
13	0.1177	0.3905	0.0024	0.3881
14	0.4318	0.4064	0.0088	0.3975
15	0.5888	0.4508	0.0121	0.4387
16	0.7851	0.5588	0.0161	0.5426
17	0.9814	0.5842	0.0202	0.5639
18	1.1777	0.6350	0.0242	0.6107
19	1.3740	0.7302	0.0283	0.7019
20	1.5703	0.7810	0.0323	0.7487
21	1.7666	0.8350	0.0363	0.7986
22	1.9627	0.8636	0.0404	0.8231
23	2.1592	0.9239	0.0444	0.8794
24	2.3555	0.9715	0.0485	0.9230
25	0.1413	0.4413	0.0029	0.4384
26	0.3925	0.4572	0.0080	0.4491
27	0.7851	0.5842	0.0161	0.5680
28	1.1777	0.7239	0.0242	0.6996
29	1.5703	0.8286	0.0323	0.7963
30	1.9629	0.8953	0.0404	0.8549
31	2.3555	0.9937	0.0485	0.9452
32	2.7481	1.1589	0.0566	1.1023
33	3.1406	1.2509	0.0646	1.1863
34	3.5332	1.3811	0.0727	1.3084
35	3.9258	1.5621	0.0808	1.4812
36	4.3184	1.7145	0.0889	1.6256
37	4.7110	1.9367	0.0970	1.8397
38	5.1035	2.2955	0.1051	2.1904
39	0.7066	0.8445	0.0014	0.8430

TEST NO. 5

4.1 m DEPTH

INCREMENT	LOAD (KN)	TOTAL DISPLACEMENT (mm)	SHAFT DEFLECTION (mm)	NET DISPLACEMENT (mm)
1	0.1962	-0.0031	0.0060	-0.0092
2	0.3925	0.0127	0.0121	0.0005
3	0.5888	0.0920	0.0181	0.0738
4	0.7851	0.1365	0.0242	0.1122
5	0.9814	0.2444	0.0303	0.2141
6	1.1777	0.3270	0.0363	0.2906
7	1.3740	0.4699	0.0424	0.4274
8	1.5703	0.6096	0.0485	0.5610
9	1.7666	0.6477	0.0545	0.5931
10	1.9629	0.7112	0.0606	0.6505
11	2.1592	0.8001	0.0667	0.7333
12	2.3555	0.9715	0.0727	0.8987
13	0.1177	0.3048	0.0036	0.3011
14	0.3925	0.3492	0.0121	0.3371
15	0.5888	0.3619	0.0181	0.3437
16	0.7851	0.4222	0.0242	0.3980
17	0.9814	0.4921	0.0303	0.4618
18	1.1777	0.5937	0.0363	0.5573
19	1.3740	0.6667	0.0424	0.6243
20	1.5703	0.6889	0.0485	0.6404
21	1.8059	0.7778	0.0557	0.7220
22	1.9629	0.8445	0.0606	0.7839
23	2.1592	0.9080	0.0667	0.8413
24	2.3555	1.0128	0.0727	0.9400
25	0.1570	0.4445	0.0048	0.4396
26	0.3925	0.4318	0.0121	0.4196
27	0.7851	0.4953	0.0242	0.4710
28	1.1777	0.6604	0.0363	0.6240
29	1.5703	0.7778	0.0485	0.7293
30	1.9629	0.8604	0.0606	0.7997
31	2.3555	0.9525	0.0727	0.8797
32	2.7481	1.1271	0.0849	1.0422
33	3.1406	1.2128	0.0903	1.1158
34	3.5332	1.2890	0.1091	1.1799
35	3.9258	1.4319	0.1212	1.3106
36	4.3184	1.6605	0.1334	1.5271
37	4.7110	1.7780	0.1455	1.6325
38	5.1035	1.9050	0.1576	1.7473
39	5.4961	2.0574	0.1698	1.8876
40	5.8887	2.2193	0.1819	2.0374
41	6.2813	2.3685	0.1940	2.1745
42	0.1962	0.7715	0.0060	0.7654

TEST NO. 6

2.3 m DEPTH

INCREMENT	LOAD (KN)	TOTAL DISPLACEMENT (mm)	SHAFT DEFLECTION (mm)	NET DISPLACEMENT (mm)
1	0.1962	-0.0190	0.0040	-0.0230
2	0.3925	0.0412	0.0080	0.0331
3	0.7066	0.0762	0.0145	0.0616
4	0.9814	0.1460	0.0202	0.1258
5	1.1777	0.1968	0.0242	0.1725
6	1.3740	0.2603	0.0283	0.2320
7	1.5703	0.3048	0.0323	0.2724
8	1.7666	0.3810	0.0363	0.3446
9	1.9629	0.4191	0.0404	0.3786
10	2.1749	0.4953	0.0447	0.4505
11	2.3947	0.5334	0.0493	0.4840
12	0.0785	0.1905	0.0016	0.1888
13	0.1962	0.1936	0.0040	0.1896
14	0.4711	0.2127	0.0097	0.2030
15	0.8322	0.2635	0.0171	0.2463
16	1.1856	0.3175	0.0244	0.2930
17	1.5075	0.3968	0.0310	0.3658
18	1.7431	0.4476	0.0359	0.4117
19	2.1199	0.5365	0.0436	0.4929
20	2.3555	0.5905	0.0485	0.5420
21	0.6673	0.1746	0.0137	0.1608
22	0.4082	0.2063	0.0084	0.1979
23	0.8322	0.2794	0.0171	0.2622
24	1.1777	0.3683	0.0242	0.3440
25	1.6488	0.4572	0.0339	0.4232
26	1.9629	0.5143	0.0404	0.4739
27	2.3555	0.6032	0.0485	0.5547
28	2.7324	0.6731	0.0562	0.6168
29	3.1406	0.8318	0.0646	0.7671
30	3.6117	0.8064	0.0743	0.7320
31	3.9258	0.8826	0.0808	0.8017
32	4.3341	0.9652	0.0892	0.8759
33	4.7110	1.0731	0.0970	0.9761
34	5.1035	1.2255	0.1051	1.1204
35	5.4961	1.4224	0.1132	1.3092
36	5.8887	1.6573	0.1212	1.5361
37	6.2813	1.8796	0.1293	1.7502
38	6.6739	2.1336	0.1374	1.9961
39	7.0664	2.4828	0.1455	2.3373
40	0.0706	1.3716	0.0014	1.3701

TEST NO. 6

4.1 m DEPTH

INCREMENT	LOAD (KN)	TOTAL DISPLACEMENT (mm)	SHAFT DEFLECTION (mm)	NET DISPLACEMENT (mm)
1	0.1962	0.1651	0.0060	0.1590
2	0.3925	0.4381	0.0121	0.4260
3	0.6281	0.5524	0.0194	0.5330
4	0.7851	0.5969	0.0242	0.5726
5	0.9814	0.6540	0.0303	0.6237
6	1.1777	0.7112	0.0363	0.6748
7	1.3740	0.9017	0.0424	0.8592
8	1.5703	1.0096	0.0485	0.9611
9	1.7666	1.0795	0.0545	1.0249
10	1.9629	1.1874	0.0606	1.1268
11	0.1727	0.8191	0.0053	0.8138
12	0.3925	0.8318	0.0121	0.8197
13	0.6281	0.8731	0.0194	0.8537
14	0.7851	0.9017	0.0242	0.8774
15	0.9814	0.9969	0.0303	0.9666
16	1.1777	1.0477	0.0363	1.0114
17	1.3740	1.0731	0.0424	1.0307
18	1.5703	1.1747	0.0485	1.1262
19	1.7666	1.2255	0.0545	1.1710
20	1.9629	1.2827	0.0606	1.2221
21	2.1592	1.3398	0.0667	1.2731
22	2.3555	1.4033	0.0727	1.3306
23	1.2563	0.9207	0.0038	0.9168
24	0.3925	0.9302	0.0121	0.9181
25	0.7851	0.9906	0.0242	0.9663
26	1.1777	1.1176	0.0363	1.0812
27	1.5703	1.2065	0.0485	1.1580
28	1.9629	1.2890	0.0606	1.2284
29	2.3555	1.4033	0.0727	1.3306
30	2.7481	1.6129	0.0849	1.5280
31	3.1406	1.7526	0.0970	1.6556
32	3.5332	1.8923	0.1091	1.7831
33	3.9258	2.0701	0.1212	1.9488
34	4.3184	2.2225	0.1334	2.0891
35	4.7110	2.3812	0.1455	2.2357
36	5.1035	2.5844	0.1576	2.4268
37	5.4961	2.8194	0.1698	2.6496
38	0.1177	1.6002	0.0036	1.5966

TEST NO. 7

2.3 m DEPTH

INCREMENT	LOAD (KN)	TOTAL DISPLACEMENT (mm)	SHAFT DEFLECTION (mm)	NET DISPLACEMENT (mm)
1	0.1962	0.0603	0.0040	0.0562
2	0.4475	0.1238	0.0092	0.1146
3	0.5888	0.1587	0.0121	0.1466
4	0.7851	0.1968	0.0161	0.1806
5	0.9814	0.2381	0.0202	0.2179
6	1.1777	0.2857	0.0242	0.2614
7	1.3740	0.3556	0.0283	0.3273
8	1.5703	0.4254	0.0323	0.3931
9	1.7666	0.5175	0.0363	0.4811
10	1.9629	0.5715	0.0404	0.5310
11	2.1592	0.6540	0.0444	0.6095
12	2.3555	0.7302	0.0485	0.6817
13	2.5518	0.8382	0.0525	0.7856
14	0.1727	0.3714	0.0035	0.3679
15	0.3925	0.3810	0.0080	0.3729
16	0.5888	0.4127	0.0121	0.4006
17	0.7851	0.4540	0.0161	0.4378
18	0.9814	0.4953	0.0202	0.4750
19	1.1777	0.5365	0.0242	0.5123
20	1.3740	0.5873	0.0283	0.5590
21	1.6488	0.6477	0.0339	0.6137
22	1.7666	0.6731	0.0363	0.6367
23	1.9629	0.7143	0.0404	0.6739
24	2.1592	0.7747	0.0444	0.7302
25	2.3555	0.8382	0.0485	0.7896
26	2.5518	0.8985	0.0525	0.8459
27	0.1570	0.4349	0.0032	0.4317
28	0.3925	0.4540	0.0080	0.4459
29	0.7851	0.5270	0.0161	0.5108
30	1.1777	0.6096	0.0242	0.5853
31	1.5703	0.7016	0.0323	0.6693
32	1.9629	0.7874	0.0404	0.7469
33	2.3555	0.8826	0.0485	0.8341
34	2.7481	0.9715	0.0566	0.9149
35	3.1406	1.1081	0.0646	1.0434
36	3.5332	1.2382	0.0727	1.1655
37	3.9258	1.3970	0.0808	1.3161
38	4.3184	1.6065	0.0889	1.5176
39	4.7110	1.8320	0.0970	1.7349
40	5.1035	2.0796	0.1051	1.9745
41	5.4961	2.3749	0.1132	2.2617
42	5.8887	2.7114	0.1212	2.5902
43	6.2813	3.0543	0.1293	2.9350
44	6.6739	3.5433	0.1374	3.4058
45	0.2041	2.0923	0.0042	2.0881

TEST NO. 7

4.1 m DEPTH

INCREMENT	LOAD (KN)	TOTAL DISPLACEMENT (mm)	SHAFT DEFLECTION (mm)	NET DISPLACEMENT (mm)
1	0.1962	0.0666	0.0060	0.0601
2	0.3925	0.1333	0.0121	0.1212
3	0.5888	0.1936	0.0181	0.1754
4	0.7851	0.2603	0.0242	0.2360
5	0.9814	0.3302	0.0303	0.2998
6	1.1777	0.4476	0.0363	0.4112
7	1.3740	0.4953	0.0424	0.4528
8	1.5703	0.5810	0.0485	0.5325
9	1.9629	0.7302	0.0606	0.6696
10	2.1592	0.8096	0.0667	0.7429
11	2.3555	0.9144	0.0727	0.8427
12	0.1177	0.3492	0.0036	0.3456
13	0.0863	0.3937	0.0026	0.3910
14	0.3925	0.4572	0.0121	0.4450
15	0.5888	0.5207	0.0181	0.5025
16	0.7851	0.5715	0.0242	0.5472
17	0.9814	0.6159	0.0303	0.5856
18	1.1777	0.6635	0.0363	0.6271
19	1.3740	0.7112	0.0424	0.6687
20	1.5703	0.7651	0.0485	0.7166
21	1.7666	0.8286	0.0545	0.7741
22	1.9629	0.8763	0.0606	0.8156
23	2.1592	0.9366	0.0667	0.8699
24	2.3555	1.0033	0.0727	0.9305
25	0.1727	0.4857	0.0053	0.4804
26	0.3925	0.5080	0.0121	0.4958
27	0.7851	0.6159	0.0242	0.5916
28	1.1777	0.7175	0.0363	0.6811
29	1.5703	0.8255	0.0485	0.7769
30	1.9629	0.9334	0.0606	0.8728
31	2.3555	0.9747	0.0727	0.9019
32	2.7481	1.1493	0.0849	1.0644
33	3.1406	1.2890	0.0970	1.1920
34	3.5332	1.5684	0.1091	1.4593
35	3.9258	1.7907	0.1212	1.6694
36	4.3184	2.0193	0.1334	1.8859
37	4.8110	2.2288	0.1455	2.0833
38	0.0942	0.8731	0.0029	0.8702

TEST NO. 8

2.3 m DEPTH

INCREMENT	LOAD (KN)	TOTAT DISPLACEMENT (mm)	SHAFT DEFLECTION (mm)	NET DISPLACEMENT (mm)
1	0.1962	0.0317	0.0040	0.0277
2	0.3925	0.0889	0.0080	0.0808
3	0.5888	0.1301	0.0121	0.1180
4	0.7851	0.1873	0.0161	0.1711
5	0.9814	0.2381	0.0202	0.2179
6	1.1777	0.2635	0.0242	0.2392
7	1.3740	0.3079	0.0283	0.2796
8	1.5703	0.3460	0.0323	0.3137
9	1.7666	0.4064	0.0363	0.3700
10	1.9629	0.4603	0.0404	0.4199
11	2.1592	0.5207	0.0444	0.4762
12	2.3555	0.5905	0.0485	0.5420
13	0.1020	0.1682	0.0021	0.1661
14	0.3925	0.1905	0.0080	0.1824
15	0.5888	0.2413	0.0121	0.2291
16	0.7851	0.2921	0.0161	0.2759
17	0.9814	0.3397	0.0202	0.3195
18	1.1777	0.3619	0.0242	0.3376
19	1.3740	0.3905	0.0283	0.3622
20	1.5703	0.4286	0.0323	0.3962
21	1.7666	0.4699	0.0363	0.4335
22	1.9629	0.4984	0.0404	0.4580
23	2.1592	0.5524	0.0444	0.5079
24	2.3555	0.6032	0.0485	0.5547
25	0.0863	0.1428	0.0017	0.1411
26	0.3925	0.2095	0.0080	0.2014
27	0.7851	0.3111	0.0161	0.2949
28	1.1777	0.3746	0.0242	0.3593
29	1.5703	0.4349	0.0323	0.4026
30	1.9629	0.5175	0.0404	0.4771
31	2.3555	0.6032	0.0485	0.5547
32	2.7481	0.7175	0.0566	0.6609
33	3.1406	0.8286	0.0646	0.7639
34	3.5332	0.9810	0.0727	0.9083
35	3.9258	1.1874	0.0808	1.1066
36	4.3184	1.4097	0.0889	1.3208
37	4.7110	1.7145	0.0970	1.6175
38	5.1035	2.0955	0.1051	1.9904
39	5.4961	2.6035	0.1132	2.4903
40	5.8887	3.3337	0.1212	3.2125
41	6.2813	4.3497	0.1293	4.2204
42	0.1177	3.6068	0.0024	3.6044

TEST NO. 8

4.1 m DEPTH

INCREMENT	LOAD (KN)	TOTAL DISPLACEMENT (mm)	SHAFT DEFLECTION (mm)	NET DISPLACEMENT (mm)
1	0.1962	0.0571	0.0060	0.0510
2	0.3925	0.1143	0.0121	0.1021
3	0.5888	0.1905	0.0181	0.1723
4	0.7851	0.2794	0.0242	0.2551
5	0.9814	0.3841	0.0303	0.3538
6	1.1777	0.4889	0.0363	0.4525
7	1.3740	0.5492	0.0424	0.5068
8	1.5703	0.6445	0.0485	0.5960
9	1.7666	0.7143	0.0545	0.6598
10	1.9629	0.7905	0.0606	0.7299
11	2.1592	0.8731	0.0667	0.8064
12	2.3555	0.9429	0.0727	0.8702
13	0.1177	0.4889	0.0036	0.4853
14	0.3925	0.5143	0.0121	0.5022
15	0.5888	0.5461	0.0181	0.5279
16	0.7851	0.5873	0.0242	0.5631
17	0.9814	0.6572	0.0303	0.6269
18	1.1777	0.7524	0.0363	0.7160
19	1.3740	0.7937	0.0424	0.7513
20	1.5703	0.8445	0.0485	0.7960
21	1.7666	0.9048	0.0545	0.8503
22	1.9629	0.9525	0.0606	0.8918
23	2.1592	1.0128	0.0667	0.9461
24	2.3555	1.0509	0.0727	0.9781
25	0.1177	0.5969	0.0036	0.5932
26	0.3925	0.6286	0.0121	0.6165
27	0.7851	0.7080	0.0242	0.6837
28	1.1777	0.8445	0.0363	0.8081
29	1.5703	0.9398	0.0485	0.8912
30	1.9629	1.0287	0.0606	0.9680
31	2.3555	1.1144	0.0727	1.0417
32	2.7481	1.2605	0.0849	1.1756
33	3.1406	1.3652	0.0970	1.2682
34	3.5332	1.5526	0.1091	1.4434
35	3.9258	1.7145	0.1212	1.5932
36	4.3184	1.9082	0.1334	1.7748
37	4.7110	2.0637	0.1455	1.9182
38	5.1035	2.3114	0.1576	2.1537
39	5.4961	2.5527	0.1698	2.3829
40	5.8887	2.8765	0.1819	2.6946
41	6.2813	3.1750	0.1940	2.9809
42	6.6739	3.5528	0.2061	3.3466
43	0.1177	2.9591	0.0036	2.9555

TEST NO. 9

2.3 m DEPTH

INCREMENT	LOAD (KN)	TOTAL DISPLACEMENT (mm)	SHAFT DEFLECTION (mm)	NET DISPLACEMENT (mm)
1	0.1962	0.0381	0.0040	0.0340
2	0.3925	0.1016	0.0080	0.0935
3	0.5888	0.1365	0.0121	0.1244
4	0.7851	0.1778	0.0161	0.1616
5	0.9814	0.2413	0.0202	0.2210
6	1.1777	0.2825	0.0242	0.2583
7	1.3740	0.3365	0.0283	0.3082
8	1.5703	0.3873	0.0323	0.3550
9	1.7666	0.4438	0.0363	0.4075
10	1.9629	0.5105	0.0404	0.4701
11	2.1592	0.5867	0.0444	0.5423
12	2.3555	0.6566	0.0485	0.6081
13	0.1177	0.3105	0.0024	0.3081
14	0.3925	0.3295	0.0080	0.3215
15	0.5888	0.3422	0.0121	0.3301
16	0.7851	0.3676	0.0161	0.3515
17	0.9814	0.3835	0.0202	0.3633
18	1.1777	0.4089	0.0242	0.3847
19	1.3740	0.4206	0.0283	0.3923
20	1.5703	0.4946	0.0323	0.4623
21	1.7666	0.5898	0.0363	0.5535
22	1.9629	0.6280	0.0404	0.5876
23	2.1592	0.6851	0.0444	0.6407
24	2.3555	0.7677	0.0485	0.7192
25	0.1177	0.3740	0.0024	0.3716
26	0.3925	0.3866	0.0080	0.3786
27	0.7851	0.4216	0.0161	0.4055
28	1.1777	0.4692	0.0242	0.4450
29	1.5703	0.5867	0.0323	0.5544
30	1.9629	0.7359	0.0404	0.6955
31	2.3555	0.8820	0.0485	0.8335
32	2.7481	1.1487	0.0566	1.0921
33	3.1406	1.3741	0.0646	1.3095
34	3.5332	1.5424	0.0727	1.4697
35	0.1413	0.8503	0.0029	0.8474

TEST NO. 9

4.1 m DEPTH

INCREMENT	LOAD (KN)	TOTAL DISPLACEMENT (mm)	SHAFT DEFLECTION (mm)	NET DISPLACEMENT (mm)
1	0.1962	0.0381	0.0060	0.0320
2	0.3925	0.0984	0.0121	0.0862
3	0.5888	0.1587	0.0181	0.1405
4	0.7851	0.2413	0.0242	0.2170
5	0.9814	0.3556	0.0303	0.3252
6	1.1777	0.5524	0.0363	0.5160
7	1.3740	0.6667	0.0424	0.6243
8	1.5703	0.7588	0.0485	0.7103
9	1.7666	0.8286	0.0545	0.7741
10	1.9629	0.9112	0.0606	0.8505
11	2.1592	1.0096	0.0667	0.9429
12	0.1805	0.6413	0.0055	0.6357
13	0.3925	0.0857	0.0121	0.0735
14	0.5888	0.0412	0.0181	0.0230
15	0.7851	0.0952	0.0242	0.0709
16	0.9814	0.1714	0.0303	0.1411
17	1.1777	0.2286	0.0363	0.1922
18	1.3740	0.3270	0.0424	0.2845
19	1.7666	0.1873	0.0545	0.1327
20	1.9629	0.5715	0.0606	0.5108
21	2.1592	0.7080	0.0667	0.6413
22	2.3555	0.7112	0.0727	0.6384
23	0.1727	0.1111	0.0053	0.1057
24	0.3925	0.1301	0.0121	0.1180
25	0.7851	0.2222	0.0242	0.1979
26	1.1777	0.2984	0.0363	0.2620
27	1.5703	0.4413	0.0485	0.3928
28	1.9629	0.7874	0.0606	0.7267
29	2.3555	0.8255	0.0727	0.7527
30	2.7481	0.9334	0.0849	0.8485
31	3.1406	1.0890	0.0970	0.9919
32	0.2512	0.3048	0.0077	0.2970

TEST NO. 10

2.3 m DEPTH

INCREMENT	LOAD (KN)	TOTAL DISPLACEMENT (mm)	SHAFT DEFLECTION (mm)	NET DISPLACEMENT (mm)
1	0.1962	0.0317	0.0040	0.0277
2	0.3925	-0.0762	0.0080	-0.0842
3	0.5888	-0.0317	0.0120	-0.0438
4	0.7851	0.0190	0.0161	0.0029
5	0.9814	0.0666	0.0201	0.0465
6	1.1777	0.1079	0.0241	0.0837
7	1.3740	0.1524	0.0281	0.1242
8	1.5703	0.2349	0.0322	0.2027
9	1.7666	0.2730	0.0362	0.2368
10	1.9629	0.3302	0.0402	0.2899
11	2.1592	0.4127	0.0443	0.3684
12	2.3555	0.4699	0.0483	0.4215
13	2.5518	0.5770	0.0523	0.4746
14	2.7481	0.6667	0.0563	0.6103
15	0.0785	0.2222	0.0016	0.2206
16	0.4082	0.2603	0.0083	0.2519
17	0.5888	0.2794	0.0120	0.2673
18	0.7851	0.3365	0.0161	0.3204
19	0.9814	0.3810	0.0201	0.3608
20	1.1777	0.4445	0.0241	0.4203
21	1.3740	0.5080	0.0281	0.4798
22	1.5703	0.5778	0.0322	0.5456
23	1.8059	0.6413	0.0370	0.6042
24	1.9629	0.6731	0.0402	0.6328
25	2.1592	0.7175	0.0443	0.6732
26	2.3555	0.7683	0.0483	0.7200
27	2.5518	0.8255	0.0523	0.7731
28	2.7481	0.8826	0.0563	0.8262
29	0.0706	0.2730	0.0014	0.2716
30	0.3925	0.3111	0.0080	0.3030
31	0.7851	0.4000	0.0161	0.3839
32	1.1777	0.5080	0.0241	0.4838
33	1.5703	0.6286	0.0322	0.5964
34	1.9629	0.7175	0.0402	0.6772
35	2.3555	0.8255	0.0483	0.7771
36	2.7481	0.9207	0.0563	0.8643
37	3.1406	1.0604	0.0644	0.9960
38	3.5332	1.2128	0.0725	1.1403
39	3.9258	1.4097	0.0805	1.3291
40	4.3184	1.5875	0.0886	1.4989
41	4.7110	1.8478	0.0966	1.7512
42	5.1035	2.0955	0.1047	1.9908
43	5.4961	2.3304	0.1127	2.2177
44	5.8887	2.6670	0.1208	2.5462
45	6.6739	2.9083	0.1369	2.7713
46	7.0664	3.1242	0.1450	2.9792
47	7.4590	3.4099	0.1530	3.2569
48	0.1570	1.3779	0.0032	1.3747

TEST NO. 10

4.1 m DEPTH

INCREMENT	LOAD (KN)	TOTAL DISPLACEMENT (mm)	SHAFT DEFLECTION (mm)	NET DISPLACEMENT (mm)
1	0.2748	0.1460	0.0094	0.1366
2	0.3925	0.2222	0.0134	0.2087
3	0.5888	0.2912	0.0202	0.2718
4	0.7851	0.3746	0.0269	0.3476
5	0.9814	0.4508	0.0337	0.4171
6	1.1777	0.5397	0.0404	0.4992
7	1.3740	0.6032	0.0472	0.5560
8	1.5703	0.7429	0.0539	0.6890
9	1.7666	0.9207	0.0606	0.8600
10	1.9629	0.9842	0.0674	0.9168
11	2.1592	1.0922	0.0741	1.0180
12	2.3555	1.1684	0.0809	1.0875
13	0.0392	0.5842	0.0013	0.5828
14	0.3925	0.6350	0.0134	0.6215
15	0.5888	0.6794	0.0202	0.6592
16	0.7851	0.7112	0.0269	0.6842
17	0.9814	0.7620	0.0337	0.7282
18	1.1777	0.8191	0.0404	0.7786
19	1.3740	0.8890	0.0472	0.8418
20	1.5703	0.9525	0.0539	0.8985
21	1.7666	1.0223	0.0606	0.9616
22	1.9629	1.0858	0.0674	1.0184
23	2.1592	1.1557	0.0741	1.0815
24	2.3555	1.2255	0.0809	1.1446
25	0.0235	0.5842	0.0008	0.5833
26	0.4711	0.6477	0.0161	0.6315
27	0.7851	0.7048	0.0269	0.6778
28	1.1777	0.8255	0.0404	0.7850
29	1.5703	0.9652	0.0539	0.9112
30	1.9629	1.0985	0.0674	1.0311
31	2.3555	1.2128	0.0809	1.1319
32	2.7481	1.3970	0.0944	1.3026
33	3.1406	1.5557	0.1078	1.4479
34	3.5332	1.7208	0.1213	1.5995
35	3.9258	1.8986	0.1348	1.7638
36	4.3184	2.0637	0.1483	1.9154
37	4.7110	2.2733	0.1618	2.1115
38	5.1035	2.5082	0.1753	2.3329
39	5.4961	2.8130	0.1888	2.6242
40	5.8887	3.0099	0.2022	2.8076
41	6.2813	3.2639	0.2157	3.0481
42	6.6739	3.6322	0.2292	3.4029
43	7.0664	4.1338	0.2427	3.8911
44	7.4590	4.4259	0.2562	4.1697
45	7.8516	5.1244	0.2697	4.8547
46	0.0392	3.1305	0.0013	3.1292

TEST NO. 11

2.3 m DEPTH

INCREMENT	LOAD (KN)	TOTAL DISPLACEMENT (mm)	SHAFT DEFLECTION (mm)	NET DISPLACEMENT (mm)
1	0.1962	0.0381	0.0040	0.0340
2	0.4711	0.1270	0.0096	0.1173
3	0.7851	0.2476	0.0161	0.2315
4	0.9814	0.3302	0.0201	0.3100
5	1.1777	0.3810	0.0241	0.3568
6	1.3740	0.4572	0.0281	0.4290
7	1.5703	0.5334	0.0322	0.5011
8	1.7666	0.6032	0.0362	0.5670
9	1.9629	0.6667	0.0402	0.6264
10	2.1592	0.7302	0.0443	0.6859
11	2.3555	0.7810	0.0483	0.7327
12	0.0706	0.3175	0.0014	0.3160
13	0.3925	0.3683	0.0080	0.3602
14	0.5888	0.3937	0.0120	0.3816
15	0.7851	0.4191	0.0161	0.4029
16	0.9814	0.4889	0.0201	0.4688
17	1.1777	0.5397	0.0241	0.5155
18	1.3740	0.5778	0.0281	0.5496
19	1.5703	0.6286	0.0322	0.5964
20	1.7666	0.6604	0.0362	0.6241
21	1.9629	0.6604	0.0402	0.6201
22	2.1592	0.7683	0.0443	0.7240
23	2.3555	0.8318	0.0483	0.7835
24	2.5518	0.8763	0.0523	0.8239
25	2.7481	0.9334	0.0563	0.8770
26	0.0628	0.4000	0.0012	0.3987
27	0.3925	0.4635	0.0080	0.4554
28	0.7851	0.5016	0.0161	0.4855
29	1.1777	0.6096	0.0241	0.5854
30	1.5703	0.7048	0.0322	0.6726
31	1.9629	0.8064	0.0402	0.7661
32	2.3555	0.8826	0.0483	0.8343
33	2.7481	0.9715	0.0563	0.9151
34	3.1406	0.9588	0.0644	0.8944
35	3.5332	1.2128	0.0725	1.1403
36	3.9258	1.3525	0.0805	1.2720
37	4.3184	1.5684	0.0886	1.4798
38	4.7110	1.7653	0.0966	1.6686
39	5.1035	1.9939	0.1047	1.8892
40	5.4961	2.3622	0.1127	2.2494
41	0.0628	1.0922	0.0012	1.0909

TEST NO. 11

4.1 m DEPTH

INCREMENT	LOAD (KN)	TOTAL DISPLACEMENT (mm)	SHAFT DEFLECTION (mm)	NET DISPLACEMENT (mm)
1	0.2355	0.0889	0.0080	0.0808
2	0.3925	0.2476	0.0134	0.2341
3	0.5888	0.3111	0.0202	0.2909
4	0.7851	0.3873	0.0269	0.3603
5	0.9814	0.4762	0.0337	0.4425
6	1.1777	0.5397	0.0404	0.4992
7	1.3740	0.6223	0.0472	0.5751
8	1.5703	0.6794	0.0539	0.6255
9	1.7666	0.7874	0.0606	0.7267
10	1.9629	0.8509	0.0674	0.7834
11	2.1592	0.9271	0.0741	0.8529
12	2.3555	1.0350	0.0809	0.9541
13	0.1020	0.3429	0.0035	0.3393
14	0.3925	0.5080	0.0134	0.4945
15	0.5888	0.5651	0.0202	0.5449
16	0.7851	0.6223	0.0269	0.5953
17	0.9814	0.6731	0.0337	0.6393
18	1.1777	0.7175	0.0404	0.6770
19	1.3740	0.7747	0.0472	0.7275
20	1.5703	0.8318	0.0539	0.7779
21	1.7666	0.8826	0.0606	0.8219
22	1.9629	0.9144	0.0674	0.8469
23	2.1592	0.9779	0.0741	0.9037
24	2.3555	1.0414	0.0809	0.9604
25	0.1177	0.3238	0.0040	0.3198
26	0.3925	0.4762	0.0134	0.4627
27	0.7851	0.6223	0.0269	0.5953
28	1.1777	0.7429	0.0404	0.7024
29	1.5703	0.8636	0.0539	0.8096
30	1.9629	0.9588	0.0674	0.8914
31	2.3555	1.0541	0.0809	0.9731
32	2.7481	1.1938	0.0944	1.0994
33	3.1406	1.3589	0.1078	1.2510
34	3.5332	1.5049	0.1213	1.3836
35	3.9258	1.6510	0.1348	1.5161
36	4.3184	1.8923	0.1483	1.7440
37	4.7110	2.0955	0.1618	1.9337
38	5.1035	2.3114	0.1753	2.1361
39	5.4961	2.5463	0.1888	2.3575
40	5.8887	2.8257	0.2022	2.6235
41	6.2813	3.1305	0.2157	2.9148
42	6.6739	3.4480	0.2292	3.2188
43	7.0664	3.9370	0.2427	3.6942
44	0.1099	2.1209	0.0037	2.1171

TEST NO. 12

2.3 m DEPTH

INCREMENT	LOAD (KN)	TOTAL DISPLACEMENT (mm)	SHAFT DEFLECTION (mm)	NET DISPLACEMENT (mm)
1	0.1962	0.1143	0.0040	0.1102
2	0.3925	0.2222	0.0080	0.2141
3	0.5888	0.3365	0.0120	0.3244
4	0.7851	0.4699	0.0161	0.4537
5	0.9814	0.5334	0.0201	0.5132
6	1.1777	0.5842	0.0241	0.5600
7	1.3740	0.6477	0.0281	0.6195
8	1.5703	0.7366	0.0322	0.7043
9	1.7666	0.7810	0.0362	0.7448
10	1.9629	0.8509	0.0402	0.8106
11	2.1592	0.9525	0.0443	0.9081
12	2.3555	0.9588	0.0483	0.9105
13	2.5518	1.0287	0.0523	0.9763
14	2.7481	1.0858	0.0563	1.0295
15	0.1334	0.4191	0.0027	0.4163
16	0.3925	0.4635	0.0080	0.4554
17	0.5888	0.5778	0.0120	0.5657
18	0.7851	0.6667	0.0161	0.6506
19	0.9814	0.7429	0.0201	0.7228
20	1.1777	0.7810	0.0241	0.7568
21	1.3740	0.8382	0.0281	0.8100
22	1.5703	0.9017	0.0322	0.8694
23	1.7666	0.9334	0.0362	0.8972
24	1.9629	0.9588	0.0402	0.9185
25	2.1592	1.0096	0.0443	0.9653
26	2.3555	1.0604	0.0483	1.0121
27	2.5518	1.1112	0.0523	1.0589
28	2.7481	1.1747	0.0563	1.1184
29	0.1570	0.5143	0.0032	0.5111
30	0.3925	0.5397	0.0080	0.5316
31	0.7851	0.7239	0.0161	0.7077
32	1.1777	0.8382	0.0241	0.8140
33	1.5703	0.9334	0.0322	0.9012
34	1.9629	1.0033	0.0402	0.9630
35	2.3555	1.0795	0.0483	1.0312
36	2.7481	1.1811	0.0563	1.1247
37	3.1406	1.3144	0.0644	1.2500
38	3.5332	1.4033	0.0725	1.3308
39	3.9258	1.5303	0.0805	1.4498
40	4.3184	1.6827	0.0886	1.5941
41	4.7110	1.8923	0.0966	1.7956
42	5.1035	2.1844	0.1047	2.0797
43	5.4961	2.4193	0.1127	2.3066
44	5.8887	2.7051	0.1208	2.5843
45	6.2813	3.1940	0.1289	3.0651
46	6.6739	3.6830	0.1369	3.5460
47	7.0664	4.4323	0.1450	4.2873
48	0.1570	1.7399	0.0032	1.7367

TEST NO. 12

4.1 m DEPTH

INCREMENT	LOAD (KN)	TOTAL DISPLACEMENT (mm)	SHAFT DEFLECTION (mm)	NET DISPLACEMENT (mm)
1	0.1962	0.0571	0.0067	0.0504
2	0.3925	0.1333	0.0134	0.1198
3	0.5888	0.2413	0.0202	0.2210
4	0.7851	0.3429	0.0269	0.3159
5	0.9814	0.4572	0.0337	0.4234
6	1.1777	0.5207	0.0404	0.4802
7	1.3740	0.5905	0.0472	0.5433
8	1.5703	0.6731	0.0539	0.6191
9	1.7666	0.7493	0.0606	0.6886
10	1.9629	0.8255	0.0674	0.7580
11	2.3555	0.9334	0.0809	0.8525
12	2.5518	0.9969	0.0876	0.9092
13	2.7481	1.0414	0.0944	0.9470
14	0.1177	0.5080	0.0040	0.5039
15	0.3925	0.4826	0.0134	0.4691
16	0.5888	0.5334	0.0202	0.5131
17	0.7851	0.5905	0.0269	0.5635
18	0.9814	0.6223	0.0337	0.5885
19	1.1777	0.6858	0.0404	0.6453
20	1.3740	0.7302	0.0472	0.6830
21	1.5703	0.8064	0.0539	0.7525
22	1.7666	0.8636	0.0606	0.8029
23	1.9629	0.9017	0.0674	0.8342
24	2.1592	0.9398	0.0741	0.8656
25	2.3555	0.9779	0.0809	0.8969
26	2.5518	1.0287	0.0876	0.9410
27	2.7481	1.0922	0.0944	0.9978
28	0.1256	0.4889	0.0043	0.4846
29	0.3925	0.5397	0.0134	0.5262
30	0.7851	0.6604	0.0269	0.6334
31	1.1777	0.7302	0.0404	0.6897
32	1.5703	0.8763	0.0539	0.8223
33	1.9629	0.9588	0.0674	0.8914
34	2.3555	1.0160	0.0809	0.9350
35	2.7481	1.1176	0.0944	1.0232
36	3.1406	1.2192	0.1078	1.1113
37	3.5332	1.3398	0.1213	1.2185
38	3.9258	1.4859	0.1348	1.3510
39	4.3184	1.6446	0.1483	1.4963
40	4.7110	1.9113	0.1618	1.7495
41	5.1035	2.1145	0.1753	1.9392
42	5.4969	2.3431	0.1888	2.1543
43	5.8887	2.6035	0.2022	2.4012
44	6.2813	3.0861	0.2157	2.8703
45	6.6739	3.5242	0.2292	3.2950
46	7.0664	4.3942	0.2427	4.1514
47	0.1177	2.2352	0.0040	2.2312

TEST NO. 1
2.3 m DEPTH

TEST NO. 1
4.1 m DEPTH

TEST NO. 2
2.3 m DEPTH

TEST NO. 3
2.3 m DEPTH

TEST NO. 3
4.1 m DEPTH

TEST NO. 4
2.3 m DEPTH

TEST NO. 4
4.1 m DEPTH

TEST NO. 5
2.3 m DEPTH

TEST NO. 5
4.1 m DEPTH

TEST NO: 6
2.3 m DEPTH

TEST NO. 6
4.1 m DEPTH

TEST NO. 7
2.3 m DEPTH

TEST NO. 7
4.1 m DEPTH

TEST NO. 8
2.3 m DEPTH

TEST NO. 8
4.1 m DEPTH

TEST NO. 10
2.3 m DEPTH

TEST NO. 10
4.1 m DEPTH

TEST NO. 11
2.3 m DEPTH

TEST NO. 11
4.1 m DEPTH

TEST NO. 12
4.1 m DEPTH

APPENDIX C

LOG₁₀ PRESSURE VS PERCENT DISPLACEMENT CURVES USED TO
DERIVE UNDRAINED SHEAR STRENGTHS MEASURED BY THE SCREW
PLATE TEST AT THE GLOUCESTER TEST FILL SITE

TEST NO. 1
2.3 m DEPTH

239

TEST NO. 1
4.1 m DEPTH

TEST NO. 2
2.3 m DEPTH

2

TEST NO. 3
2.3 m DEPTH

TEST NO. 3
4.1 m DEPTH

243

TEST NO. 4
2.3 m DEPTH

TEST NO. 4
4.1 m DEPTH

TEST NO. 5
2.3 m DEPTH

TEST NO. 5
4.1 m DEPTH

TEST NO. 6
2.3 m DEPTH

TEST NO. 6
4.1 m DEPTH

TEST NO. 7
2.3 m DEPTH

TEST NO. 7
4.1 m DEPTH

TEST NO. 8
2.3 m DEPTH

TEST NO. 8
4.1 m DEPTH

253

TEST NO. 9
2.3 m DEPTH

TEST NO. 9
4.1 m DEPTH

TEST NO. 10
2.3 m DEPTH

TEST NO. 10
4.1 m DEPTH

TEST NO. 11
2.3 m DEPTH

TEST NO. 11
4.1 m DEPTH .

254

TEST NO. 12
2.3 m DEPTH

TEST NO. 12
4.1 m DEPTH

END

1 2 0 6 8 6

FIN