TRIAXIAL TESTING ON SOILS FROM NEWFOUNDLAND AREAS: RESULTS OF MONOTONIC AND CYCLIC TESTS

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TRIAXIAL TESTING ON SOILS FROM NEWFOUNDLAND AREAS: RESULTS OF MONOTONIC AND CYCLIC TESTS

by

© Zille Ahmad Khan, B.Sc. Eng. (Hons)

A thesis submitted in partial fulfillment
of the requirements for the
degree of Master of Engineering

Faculty of Engineering and Applied Science
Memorial University of Newfoundland
April, 1985
St.John's, Newfoundland, Canada

TO MY PARENTS

ACKNOWLEDGEMENT

The author wishes to acknowledge the valued guidance and assistance received from many people. Especially Drs. Pierre Morin and John Christian without whose help and advice this thesis would not have been possible. A great debt is owed to Dr. Morin, my principal supervisor, for his motivated and continuous support throughout the project. Dr. G. R. Peters, Dean, and Dr. T. R. Chari, Associate Dean, graduate studies, Faculty of Engineering and Applied Science provided much help and encouragement.

Dr. F. A. Aldrich, Dean, school of graduate studies supported the author's graduate work. Finally author wishes to acknowledge Memorial University of Newfoundland for its financial support throughout the M. Eng. program and Mr. Jeff Webb for reading the text of this thesis.

ABSTRACT

The study presents the laboratory testing of various types of soils using triaxial equipment recently established at Memorial University of Newfoundland. Sand from Hibernia area, Clayey Silt from Placentia Bay area and Silty Clay from West Coast of Newfoundland were submitted to standard monotonic triaxial tests. Isotropically consolidated undrained (CIU) compression tests, isotropically consolidated drained (CID) compression tests were performed as well as several extension tests or pore pressure controlled tests. Cyclic triaxial tests were also carried out on Sand and Clayey Silt.

Results showed that, despite known limitations, the equipment was adequate in providing reliable geotechnical parameters. A discussion of these results and a comparison with earlier work by other researchers are presented.

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NOMENCLATURE

The symbls listed below and used in this thesis generally conform to those published in Canadian Geotechnical Journal (Barsvary et al. 1980). They are also defined when they first appear in the text.

A, B = Skempton's pore pressure coefficients

A_0 = initial area of the sample

c' = effective cohesion intercept

C_v = coefficient of consolidation

C_u = uniformity coefficient

D_0 = initial diameter of the sample

D_R = relative density

e_0 = initial void ratio

H_0 = initial height of the sample

I_D = density index

K_f = failure envelop in Mohr-Coulomb's representation

K_0 = coefficient of earth pressure at rest

[L] = test performed with enlarged lubricated ends

N = number of cycles for failure

[S] = test performed with standard ends

u = pore pressure

V_0 = initial volume of the sample

γ_d = dry unit weight of soil

ρ_{min} = minimum density of soil

ρ_{max} = maximum density of soil

ρ_s = density of solid particles

ϕ' = effective angle of internal friction

σ_3 = initial effective confining pressure

Chapter 1
INTRODUCTION

1.1. Introduction to the present work

The Triaxial test has been accepted as a routine test for determining the strength of a soil sample. It is commonly used in practice to investigate the behaviour of soil samples before and at failure. Despite the advent of various other testing machines the triaxial test still has many important applications in soil mechanics research.

The relationship between the behaviour of soil under particular drainage conditions and the strength characteristics expressed in terms of effective stresses depends on the magnitude of pore-pressure induced during the test. In order to clarify pore-pressure response to different combinations of applied loads, it is convenient to use a concept incorporating pore-pressure parameters, expressed in terms of empirical coefficients, which are based upon the separation between all round pressure and deviatoric stress increments. This concept can be used to predict the behaviour of soil masses by numerical methods (like the finite element method, for example) but it also provides a basis for estimating the magnitude of pore-pressure to be encountered in the field. Indeed in many problems involving the assessment of stability and consolidation of soil masses, it is necessary to estimate the magnitude of the effective stress changes.

Laboratory testing and particularly triaxial test results constitute a first tool that the designer should use in order to reduce the margin of uncertainty in his engineering analysis. The second step, as it has been proved for embankments design is to use field data before and during construction to check the consistency of the assumptions made earlier.

In the long term behavior of soil masses where pore pressure changes are not that critical, laboratory triaxial testing also constitutes one of the tools that the designer can use together with other tests.

1.2. Objectives

In the present study, laboratory triaxial testing on different types of soils from Newfoundland areas are reported. Standard monotonic triaxial tests, using standard and enlarged lubricated ends were performed. Comparison of tests using standard and enlarged lubricated ends are made. Several special tests were performed as well as cyclic tests. The objectives of the study were to provide reliable triaxial results and compare them to previous data by other researchers. For that purpose, a comprehensive description of the equipment used at Memorial University as compared to the other organizations is made and the influence of various testing parameters evaluated.

Chapter 2
LITERATURE REVIEW

2.1. Geotechnical Properties of Offshore Soils

2.1.1. Introduction

Although construction of offshore drilling platforms began in the Gulf of Mexico as early as 1937, geotechnical data for the soils encountered in these sites are very limited. Shear strength data are mostly unconfined compression or laboratory vane shear test results. In several investigations in which direct shear apparatus was used, the results are unreliable because of uncontrolled and ambiguous drainage conditions. Very few data giving strength parameters in terms of effective stresses are available (Noorany 1970).

In recent years more attention has been given to the engineering properties of submarine sediments other than those found in relatively shallow coastal waters. Published studies on the properties of submarine sediments mainly pertain to physical properties such as sound velocity or density related to consolidation and depositional history.

About thirty years ago the Swedish Deep Sea Expedition investigated the sediments of the East Pacific Ocean, followed a few years later by American offshore investigations near California and in the Gulf of Mexico due to the interest in oil exploration. During the last ten years geotechnical studies of offshore marine sediments have been reported in special conferences on Marine Geotechnique held regularly since 1967.

From the available literature the shear strength properties of the previously investigated soils are summarized here. An attempt will be made to compare some of these results with the findings of the present work, in spite of unavoidable regional differences.

2.1.2. Index Properties of Offshore Soils

Due to the action of water, offshore sands consist usually of relatively rounded particles. Such sands are frequently of fine and fairly uniform grain size with uniformity coefficients of less than about 3 to 4. Moreover, uniform sands and silts in a loose state are susceptible to liquefaction. Figure 2.1 shows the grain size distribution of some offshore sands. The plasticity characteristics of the near and offshore clays, usually from shallow depths, are shown in figure 2.2 (Meyerhof 1979).

2.1.3. Geotechnical Properties for the Canadian East Coast

Keller (1969) summarized the analysis of over 300 samples from the North Atlantic and gave the following range of geotechnical properties for the Canadian East Coast surficial sediments. Measurement of shear strength was made by either laboratory vane shear or unconfined compression test.

Sediment type: Fluvial-marine (sand-silt sizes greater than 0.016 mm)

Shear strength: 3.4 to 6.9 kPa

Water content: 50 to 100%

Saturated unit weight: 14.8 to 17.1 kN/m^3

Fifteen short core samples from the southern Grand Banks were tested by Geocon (1969). Water depth was approximately 95 m, core lengths ranged from 0.38 to 1.37 m, the average length being 0.80m. The results of these tests are summarized below:

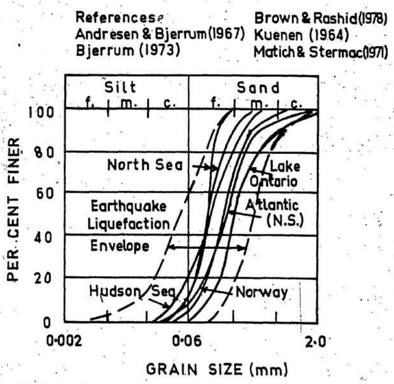


Figure 2-1: Grain size distribution of offshore sands (Meyerhof 1979)

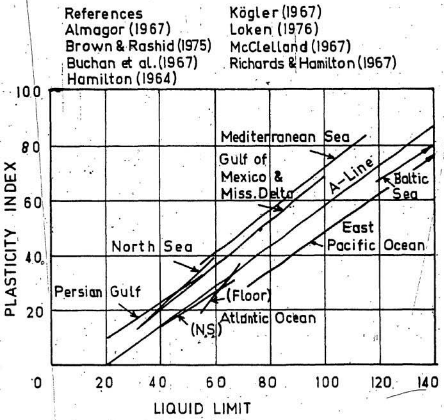


Figure 2-2: Plasticity characteristics for offshore clays (Meyerhof 1979)

Sediments type: sand and combination of sand - silt and gravel

Saturated unit weight: 14 to 20 kN/m^3

Angle of internal friction: 34 to 57^0 (from direct shear tests on disturbed samples)

A series of three bore-holes on the Grand Banks by McClelland Engineers (1971) indicated 4 to 26 m of dense sand as the upper layer underlain by hard silts or stiff clays occasionally with sand seams. A value of angle of internal friction $\phi' = 35^0$ for the sand was suggested which is near the range of values of angle of internal friction $\phi' = 39$ to 46^0 given by Geocon (1969).

Based on the results of three boreholes drilled in 1980 (Geocon 1980) in the Hibernia area of the Grand Banks, the upper 5 to 6m was found to be dense clean sand with traces of gravel, silt, clay and shells. The sand was underlain by various strata from silty clay to clayey silt and silty sand. Direct shear tests on reconstituted samples from upper sand layers gave angle of internal friction ϕ' values of 36 to 40^0.

From the efforts required to obtain vibrocore samples in 70 m depth at Hibernia, Amos and Barrie (1980) concluded that the seabed was hard with lag gravel. Because of the dense nature of the seabed (Lewis 1981) the load cells of the penetrometer failed by buckling. Tests on van veen grab samples from these locations showed the following:

Location: Hibernia area, Grand Banks

Sediment type: medium to fine sand poorly graded

Saturated unit weight: 19 to 21 kN/m^3

Angle of internal friction: 27 to 32^0

2.1.4. Geotechnical Properties for Davis Strait Area

Some geotechnical data are also available for Davis Strait area. MacLaren Atlantic Limited (1976) studied two piston cores in 540 m water depths. Graham and Nixon (1979) discuss the results of tests of 6 cm diameter piston cores from 400 m water depth in the southern Davis Strait. Geotechnical properties for 400 m water depth at Davis Strait area are as follows:

Location: Davis Strait (400 m water depth)

Sediment type: Clay (CL) less than 40% sand and gravel

Saturated unit weight: 22 kN/m^3

Shear strength: 17 to 31 kPa

Cohesion: 7 kPa

Angle of internal friction: 34^0

Additional geotechnical tests on 10 piston core samples from the Davis Strait with 300 m water depth provided the following properties (Chari et al. 1983).

Sediment type: sand and silt mixture

Saturated unit weight: 20 kN/m^3

Angle of internal friction: 25^0 (direct shear tests on reconstituted samples)

2.1.5. Geotechnical Properties of Sediments for the North Pacific

North Pacific site lies between 30^0 and $31^0\ 30'$ N , and longitude 157^0 and 159^0 W some 1000 km north of Hawaiian Islands. The surface sediments are fine grained quartz - illite - rich clays. Consolidation tests of the deeper samples in the North Pacific clays indicate that the sediments column is normally consolidated. The in situ coefficient of permeability within the cored depth of 25 m is relatively constant at 10^{-9} m/s. Isotropically consolidated undrained (CIU) triaxial tests were conducted on undisturbed and reconstituted - reconsolidated specimens (remolded samples) of sediments. The Mohr - Coulomb parameters of effective cohesion c' and effective friction angle ϕ' from these tests are listed in table 2.1 (Silva et al. 1984). Also listed are friction angles for similar sediments reported by other investigators.

Fine grained soils are common on the continental slope, continental rise, and certain areas of the deep sea floor. Table 2.2 (Richards et al. 1967) represents the brief summary of some reported extremes of geotechnical properties of sea floor sediments the majority of which are from water depths in excess of 130 m (430 ft). Examples of fine grained soils are presented by Keller (1969).

2.1.6. Geotechnical Properties for Gulf of Mexico Area

Noorany (1970) has summarized the findings regarding shear characteristics of submarine soils. The most thoroughly investigated region having sediments is in the Gulf of Mexico near the north of Mississippi River. Because of the rapid rate of deposition the modern prodelta clays are highly compressible and have extremely low shear strengths of 0.96 kPa to 9.6 kPa. Results of a laboratory vane shear tests on clays from the samples on Guadalupe Island in the Pacific indicate and undrained shear strength ranging from 2.8 kPa to 16.3 kPa. A series of consolidated undrained triaxial tests with pore pressure measurements indicated an effective cohesion $c' = 57.5$ kPa and an effective angle of internal friction $\phi' = 31^0$ for these samples. The pattern of pore pressure change with strain was

Table 2.1: Strength Properties of Sediments (Silva et al. 1984)

	$\bar{c}$ kPa	$\bar{\phi}$ degrees
Smectite		
LL-44 GPC-3, 1800 cm	1.6	35.5
PS-9 Vertical (Remolded)	1.1	37.3
PS-18 Vertical (Remolded)	0.9	36.3
Illite		
MA-02 GC-04 200 cm	0.0	34.8
MA-02 GC-04 100 cm	3.0	33.0
Pl-18 Vertical (Remolded)	9.9	30.4
Pl-2 Vertical (Remolded)	1.3	24.6
Lee and Hamilton (1974)		
Illice (depth: uper 10 meters)	1.8-3.3	35.0-36.0
Smectite (depth: upper 10 meters)	3.4-4.1	37.0-38.0
Silva and Clukey (1975)		
Illite (depth: upper meter)	0.0	34.0-35.8

Table 2.2: Range of geotechnical properties of sea floor sediments (Richards et al. 1967)

	Min	Max
Liquid limit	25	130
Plastic limit	15	45
Plasticity index	<2	92
Liquidity index	0.8	33
Water content	27%	375%
Void ratio	0.7	9
Saturated unit weight kN/m^3	11.7	22.6
Shear strength, natural kg/m^2	20	3000
Shear strength, remolded kg/m^2	20	500
Sensitivity	1.6	26

typical of normally consolidated soils and Skempton's pore pressure coefficients A measured at failure ranges between 0.1 and 1.0. In many locations such as South Timbalier area in the Gulf of Mexico the clays are overconsolidated having undrained shear strength as high as 210 kPa. The undrained shear strength characteristics of clays samples from Texas and Mexican continental slope varied from 2.6 kPa to 41 kPa.

2.2. Historical Development of Triaxial Test

The aim of laboratory testing is to study the behavior of a soil sample under conditions similar to those encountered in the field and to define representative parameters which describe this behavior. In a laboratory test the specimen is intended and generally assumed to represent a single point in a soil medium. The validity of this assumption depends on the uniformity of stress and strain distribution within the soil sample.

Obtaining parameters for constitutive equations and modeling has resulted in the development of improved equipment and testing techniques: multiaxial (true triaxial) test or hollow cylinder triaxial test are two of them. Although these devices are more versatile, the conventional solid cylinder triaxial test is still the most popular (Saada et al. 1980).

It appears that the first real soil shear test was described by Collins (1956). In this test a long specimen of clay 4 cm^2 in cross section was loaded transversely at its center until it failed in double direct shear. Triaxial testing of soils seems to have evolved simultaneously in Germany (Seiffert 1933), the Netherland (Business 1934), and the United States (Housel 1936; Stanton et al. 1934). In the early thirties a machine was built in Germany at the Prussian Waterways Experimental Station for studying the consolidation of clays under conditions of negligible side friction; the surrounding liquid was entirely confined and temperature and leakage had to be closely controlled. Several investigators recognized that the apparatus could be used to measure the ratio of the axial and lateral pressure prior to and at failure, the first result appearing to be those of Stanton and Hveen (1934). Positive control over the lateral pressure was developed independently by Rendulic (1936) and Housel (1936). A larger variety of stress paths can be applied in the triaxial cell with the limitation that two of the three principal stresses are always equal. The volume of the confining fluid may remain fixed (DeBeer 1950) or may vary. Bishop and Henkel (1962) cover in great detail all aspects of this test.

Kjellman (1936) reported test results on cubical specimens on which the three principal stresses could be varied independently. In the last few years several devices involving automated control of the principal stresses and strains applied to prismatic specimens have appeared. Hollow circular cylinders subjected to different internal and external pressures as well as to axial and torsional loads are also in use. Among the earliest investigations in soil mechanics taking advantage of that last configuration are those of Cooling and Smith (1935) and Geuze and Tan (1953).

Due to historical factors as well as to the simplicity of the test, the solid cylinder triaxial test remains the most commonly used in routine investigation. It provides information on volume change and pore pressure characteristics during the consolidation as well as during the shear. Minor modifications of the system permit a wide variety of stress paths to be investigated.

2.3. Systems for Triaxial Loading

2.3.1. System Developed at Imperial College (Bishop and Henkel)

The main components of a triaxial system are the cell, the external measuring systems (pressure, volume, mechanical or electrical feed back and controls) and the loading system.

Following the description by Bishop and Henkel(1962), the common cell consists of three components: (1) the base, which forms the pedestal on which the sample rests and incorporates various pressure connections, (2) the removable cylinder and top cap, which enclose the sample and enable fluid pressure to be applied, and (3) the loading ram, which applies the deviatoric stress to the sample.

2.3.1.1. The Cell for 1.5 in. and 4 in. Diameter Samples

The 1.5 in.[1] diameter specimen is the generally accepted standard in Great Britain for testing soils free from stones. Indeed where 4 in. diameter undisturbed samples are obtained, three 1.5 in. diameter samples can be cut from any one layer, which greatly facilitates the investigation of the shear characteristics of the strata having a rapid variation in strength in the vertical direction. A sample height of 3 in. is usually adopted for clays. This is sometimes increased to 3.5 in. The cell itself is designed to allow plenty of space around the sample for special fittings.

The cell for 4 in. diameter samples differs from small cell mainly due to the much greater forces involved. The height of the specimen is usually 8 in. Soil with a maximum grain size of 3/4 in. may thus be tested and the cell can be used in general purpose loading press. The cell is also used to test undisturbed samples taken with a 4 in. diameter sampler. Details of the 1.5 in. and 4 in. cells are shown in figures 2.3 and 2.4 respectively.

2.3.1.2. Apparatus for Controlling Cell Pressure

In the most common types of triaxial compression tests, the cell pressure is held constant throughout each stage of the test. The consolidation stage where the sample is allowed to change in volume under a constant effective pressure may take as much as 3 days for a soil of low permeability. The duration of the undrained shear stage may widely vary from 10 minutes, if pore pressure measurements are not required to several hours if pore pressure are measured. Shearing under drained conditions may need upto 3 days. It is therefore important to avoid any change in the confining cell pressure during this period. For some

[1] 1 in. = 25.4 mm.

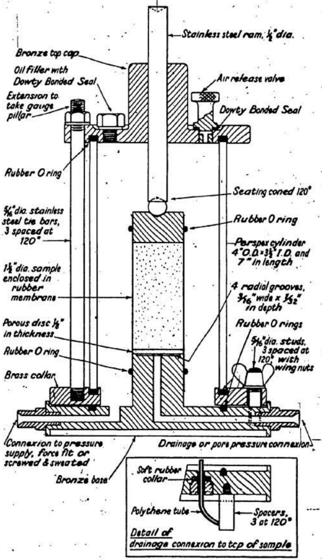


Figure 2-3: The triaxial cell for 1.5 in. diameter samples (Bishop et al. 1962)

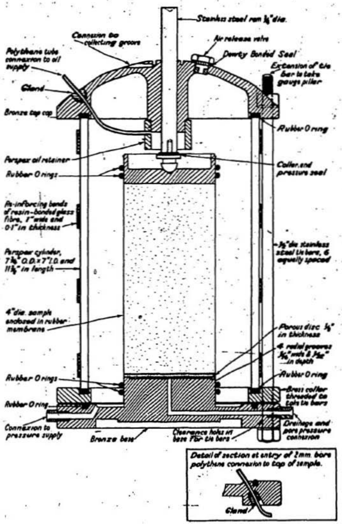


Figure 2-4: The triaxial cell for 4 in. diameter samples (Bishop et al. 1962)

special tests (creep test, for example) a constant cell pressure is required for a period of several weeks or even months.,

The maintenance, with sufficient accuracy, of a constant pressure over long periods presents considerable difficulty. The self compensating mercury control was reported as the most satisfactory method. The principle of the self compensating mercury control is shown in figure 2.5. The pressure of water in triaxial cell results from the difference in level between the mercury surfaces in two small cylinders connected by a thin flexible pressure tube which form, in effect the two limbs of a manometer. The stiffness of the spring is adjusted for compensating eventual leakages in the water system. That system is safe in the sense that there is only liquid pressure involed. On the other hand the use of mercury may be an health hazard.

2.3.1.3. Apparatus for Measuring Pore Pressure

. The usual laboratory methods of measuring pressure (Bourdon tube type) cannot be applied directly to the measurement of the pore pressure in a small sample of soil owing to the volume of pore water which would have to flow from sample to cause the instrument to register. This flow of pore water will modify boundary conditions and cause a time lag in the response. As an example, a soil with low compressibility would not be able to expel enough water to allow a pressure reading with this type of measuring device. Similarly, a soil of low permeability, even compressible, will need a long time for stabilization of the pore pressure readings.

These drawbacks can be greatly avoided by using a diaphram gauge in which the deflections are small and are measured by electric strain gauges. They can also be avoided by the use of a null method of pressure measurement.

The form in which the null method of pressure measurement was originally used at Imperial College is shown in figure 2.6. An increase in pore pressure in the sample will tend to depress the mercury in the limb b of the U-tube. This can be

$$\text{cell pressure} = (h_1 - h_2)\, r_m - (h_3 - h_2)\, r_w$$

$$\text{stiffness of spring} = A\, r_m \frac{1}{2 - r_m/r_w} - W$$

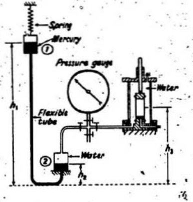


Figure 2-5: The principle of the self compensating mercury control (Bishop et al. 1962)

immediately balanced by adjusting the piston in the cylinder e to increase the pressure in limb c by an equal amount, which is registered on a pressure gauge d. The only flow of pore water which can occur results from the elastic deformation of the tube connecting the cell to the limb b, which is negligible for most practical purposes or from the compression of air bubble trapped in the system between the base of the sample and the mercury surface in the limb.

2.3.1.4. Apparatus for Measuring Volume Change

A change in the cell pressure or in axial load generally results in a change in the volume of the sample. In the particular case of an undrained test on a fully saturated sample this volume change is negligible due to the low compressibility of the water in the pore space and of the material forming the soil particles. The measurement of volume changes during drained tests is of importance both in determining the compressibility of the sample and in calculating the actual cross sectional area of the sample at failure, on which the stress values are based.

The methods commonly used are based on (1) the volume of fluid entering the cell to compensate for the change in volume of the sample, (2) the volume of the fluid expelled from the pore space of the soil, and (3) the direct measurement of the change in length and diameter of the sample.

In partly saturated soils a volume change occurs due to the gas compressibility and solubility. This is measured by observing the quantity of water entering or leaving the cell as the cell pressure and axial load are changed. A method to carry out this measurement is to check the displacement of an interface between the pore pressure fluid and a colored fluid (kerosene, for example).

In practice a small diameter cylinder is preferable, to give greater accuracy in the volume readings, and special measuring unit is used (figure 2.7). This is a self compensating U-tube, the spring being so adjusted that displacement of mercury from one limb to the other can occur without resulting in a significant head across the apparatus.

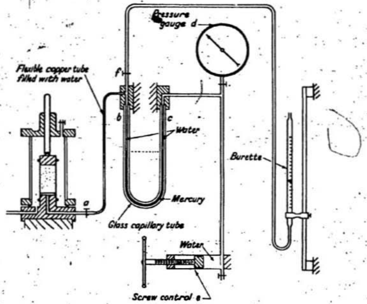


Figure 2-6: Null method of pore pressure measurement; original arrangement (Bishop et al. 1962)

Routine testing of partly saturated soils is generally performed on 4 in. diameter samples, and the volume change apparatus is therefore designed primarily for the large cell. A calibrated transparent tube about 20 in. in length and 1 in. in internal diameter is suitable for this purpose. The level of mercury surface is read on a scale fixed to a strip of mirror. Volume changes can be read to about 0.01% of the initial volume of the sample.

2.3.1.5. Loading System

The methods of applying the axial load to the sample are influenced both by the requirements of the test and the need of mechanical simplicity. Two classes of procedure may be broadly distinguished: (1) controlled rate of strain and (2) controlled rate of loading. For routine tests and for common research tests the use of controlled rate of strain has many advantages and is generally accepted. The loading system may consist either of a screw jack operated by an electric motor and gear box, or of a hydraulic ram operated by an oil pump.

The main features of the Imperial College Triaxial system can be briefly summarized: two different types of triaxial cells have been used for two different diameter size of samples. A sample diameter of 1.5 in (height of 3 in.) was usually adopted for intact clays and sands, a sample diameter of 4 in. (8 in. in height) was adopted for remolded soils or where the material was too coarse. Bishop and Henkel (1962) developed the self-compensating mercury controlled methods to apply the confining cell pressure. For measuring the pore pressure, a diaphram gauge was used in which the deflections are small and are measured by electric strain gauges. A null method of pressure measurement was also used. The 6000 lbs. capacity general purpose testing machine was used for loading both 1.5 in. and 4 in. diameter samples.

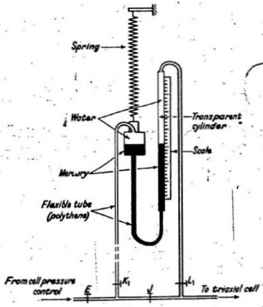


Figure 2-7: Apparatus for measuring volume change under pressure (Bishop et al. 1962)

2.3.2. Triaxial Testing at Norwegian Geotechnical Institute

At the begining, triaxial tests at Norwegian Geotechnical Institute (NGI) were used mainly to determine shear strength parameters in compression tests where the axial stress increases and the lateral stress is kept constant (Berre 1981). In the later years it was gradually realized that laboratory specimens had to be subjected, as closely as possible, to the same stresses and stress changes as in the field. Equipment and procedures were then developed so that any combination of vertical and horizontal stresses can be applied.

Figure 2.8 represents the layout of a triaxial test unit. The sample can be loaded in three ways, (1) by the motor driven press (2) by the dead weights on the hanger and (3) by the air-operated double acting piston on the top of the loading frame.

Two valve selector blocks for cell pressure and pore-pressure are connected to the triaxial cell respectively. Between the two blocks a mercury differential pressure manometer can measure the difference between the cell and pore-pressures very accurately, independently of the magnitude of the back pressure. Since both blocks have an outlet to the atmosphere, the manometer can also measure low absolute cell pressures or low absolute pore-pressures. In addition to the measuring devices, the triaxial test unit is also equipped with electronic transducers for automatic data logging. The air pressure supply, coming into each of the valve blocks, together with electronic transducers, enable automatic regulations of stresses and strains.

For maximum utilization of the loading press and its auxiliary equipment two extra consolidation units can be included in the triaxial test unit. A consolidation unit consists of triaxial cell where the piston can be locked, a cell pressure unit and hangers for application of dead weights on the piston. After completion of consolidation, the piston is locked in position and the cell moved to the loading press.

Two types of triaxial cells are used, one for static loading and one for cyclic loading. The one used for static loading has a few improvements over the one used by Bishop and Henkel(1962). The surface of the piston is hardchromed, and the rotating bushing is nitrid-hardened in order to reduce friction. A wheel on the dial gauge arm prevents rotation of the piston. The connection between piston and top cap can take both compression and tension forces.

The cell and the pore pressures are usually applied by bleeding air pressure valves. The system includes two valves, connected to the low and high pressure supplies respectively. Both valves can be adjusted, either manually or automatically. Except for the consolidation stage, all readings for continuously loaded static tests are taken automatically by the electronic data logger. Cyclic tests are logged by strip-chart recorder.

2.4. Experimental Factors Affecting Triaxial Results

The results obtained in the triaxial testing are mainly affected by the following factors

(1) effects of end plates and porous stone (smoothness, size, permeability)

(2) saturation of the sample and circuitry

(3) effect of the latex membrane surrounding the sample and lateral filter paper drains

(4) sample preparation

(5) intrinsic response lag of the system and other factors (sample disturbance, size of the specimen)

The purpose of the following review is to draw attention on the relative weight of these factors on triaxial test results.

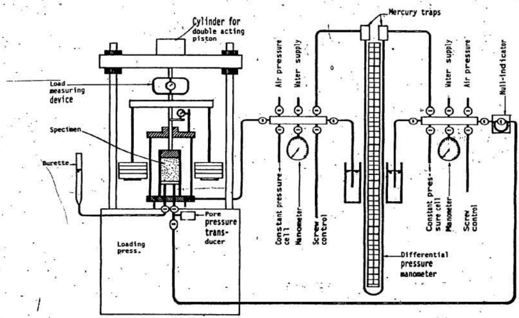


Figure 2-8: General layout of a typical triaxial test unit (Berre 1981)

2.4.1. End Effects

The effect of end restraints during compression tests is an important feature in the triaxial test. The use of "guard" areas, like in permeability which may provide an homogeneous state of strain and stress is not possible in the test. The experimental work by Taylor (1941) in 1940s summarized by Rutledge (1947), led to the conclusion that reliable results could be obtained with soil specimens between usual platens, provided length to diameter ratio was in the range of 1.5 to 3.0. In 1960 Shockley and Ahlvin (1961) conducted an investigation on the nonuniform conditions in the triaxial tests. From tests on dry sand ranging from loose to dense state they found that there was a volume increase in the middle third of the sample and a volume decrease at both ends. Tests on large triaxial specimens of dry sand showed higher than average values of both vertical stress and vertical strain in the portion of the mass near the vertical axis just below midheight where the maximum bulge take place with lesser values toward edges and the ends.

Rowe (1962) was apparently the first one to use a combination of rubber sheeting and silicon grease to develop frictionless end for triaxial compression test specimens. Rowe and Barden (1964) grease rubber system is the popular system of minimizing end restraints (fig. 2.9). Sometimes a short porous dowel is used at the center of the platens to avoid side slipping of the sample. The paper by Rowe and Barden (1964) produced interesting discussions (Lee et al. 1964; Olsen et al. 1964; Turnbull 1964) and started a wave of research to bring out the advantages and disadvantages of oversized lubricated end plates. Barden and McDermott (1965) tested compacted clays as well as remolded normally consolidated and overconsolidated clays with lubricated and nonlubricated platens. They concluded that lubricated ends markedly reduced the vertical and radial pore-pressure gradients together with the moisture migration. Barelling was minimized but the effective strength parameters were not altered when the results were compared with a length to diameter ratio of 2 tested between ordinary platens. Bishop and Green (1965) tested one type of sand and reached the same conclusion regarding maximum angle of shearing. Short samples with lubricated ends show larger axial strain and a larger dilation at failure than long samples without lubrication. Barden and Khayatt (1966) referred to the necking encountered during extension tests, and show that the lubricated ends go a long way in increasing uniformity .

Duncan and Dunlop (1968) tested undisturbed clay, and reached the conclusion that unless it was necessary to measure volumetric strains in drained tests on sand, the advantages gained from the use of lubricated ends were not worth the additional bother. Roy and Lo (1971) ran comparative drained triaxial tests at high confining pressures on strong-grained and weak-grained granular material with ordinary and lubricated ends. They found that the stress-strain relations were significantly influenced by the end conditions. For high pressure tests, lubrication resulted in a much more uniform strain, volume change, and crushing of particle throughout the sample. Raju et al (1972) found that, while the well known failure plane develops in specimen of dense sand tested in compression between ordinary plates, no such plane occured if the plates were lubricated. They deduced that the occurance of this plane was not a property of the sand but was due to the testing procedure. Kirkpatrick et al (1974), using dense and loose sand, measured stress at the platens by means of diaphram gages and found that lubricated platens lead to a reasonably uniform stress distribution, while nonlubricated platen resulted in nonuniformities which became more severe as the strains increased.

Finally, Lee (1978) reviewed most of the aforementioned research and extended it to sand under the undrained conditions. He concluded that for medium to dense sand there was significant increase in static undrained strength with lubricated ends as compared with tests using regular ends. The effect was found to be significantly greater than observed for other studies pertaining to drained tests on sands and undrained tests on clays. He relates the influence of the friction to the tendency of the material to change volume, thus explaining why Duncan and Dunlop (1968) did not find much difference in their results using regular and lubricated platens.

2.4.2. Water Content and Saturation of the Sample

If meaningful data are to be obtained in the laboratory, sample must be tested in the same water content and saturation conditions as in situ (Ballivy et al. 1976). Therefore, if properties of saturated soils are investigated, the laboratory samples must be fully saturated in order to get the actual pore pressure response and to determine the effective stress path.

2.4.3. Membrane Effects

The penetration of the lateral membrane into a triaxial sample affects either the volumetric strains in drained test or the pore pressures in undrained tests. The effect is important for granular materials, and may also be significant for silty clays as shown by Kiekbush and Schuppener (1977). To determine the effective stress-strain relation for soil, it is necessary either to reduce the influence of the membrane penetration by means of experimental devices (Kiekbush et al. 1977; Lade et al. 1977) or to correct the results obtained.

In drained triaxial tests the volume changes are normally determined by the quantity of water expelled from or drawn into a fully saturated sample. The test in which the cell pressure is also changed are alway accompanied by a change of the membrane penetration. Therefore a prediction of the volume changes based on the measurement of water leaving or entering the specimen will be incorrect.

In undrained tests errors will occur in the measurements of pore-pressure. Here the assumption is made that the volume of the specimen remains constant throughout a test. This assumption will be violated for the following reasons. At the begining of the shear, the rubber membrane completely transmits the cell pressure to the grain structure as shown in figure 2.10. In the course of the test part of the cell pressure is transferred to the water as excess pore water pressure and a change in membrane penetration will occur. For volumetric compatibility this volume change due to membrane penetration must be accompanied by a corresponding volume change of the grain structure in the opposite direction. In situ, however, the deformation of the grain structure during rapid loading may occur without volume change. The excess pore-water pressures and effective stresses measured in test where membrane penetration occurs may differ from those in-situ. It has been noticed that the influence of membrane penetration can be far more important than all other effects for specimens of granular soils.

2.4.4. Sample Preparation

Effects of methods of specimen preparation on fabric and compressibility were investigated in medium grain sized sand by Mahmood et al (1976). The effect of vibratory compaction was compared with the effect of pluviation by characterizing the particle arrangements and measuring compressibility in a specially fabricated oedometer.

The loose, pluviated specimens were much more compressible than the dense specimens prepared by the same procedure. The dense specimens when prepared by vibration were less compressible than the pluviated specimens. The lateral stresses were higher in the loose pluviated specimens than in the dense pluviated specimens during loading. On unloading, the same trend was present initially but reversed at lower axial load. This trend was also measured in the specimens prepared by vibration.

Mahmood et al. (1976) interpreted these results by analyzing the structure of the sample. Specimens densified by vibration in layers had random grain orientation up to 100 percent relative density. When vibration was continued beyond the time interval needed for achieving 100 percent relative density, the grain acquired a preferred orientation. Pluviated specimens had randomly oriented grains at both low and high densities.

Sample preparation control is a complicated problem to address. The main difficulty is that a reliable measurement of the sample density is possible but beyond the capabilities of an ordinary laboratory.

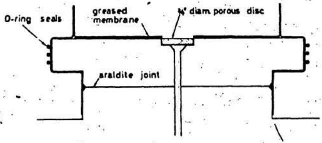


Figure 2-9: Enlarged frictionless end platen with central drainage for triaxial tests (Rowe et al. 1964)

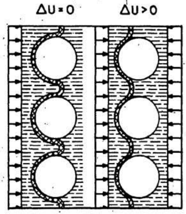


Figure 2-10: Membrane penetration into interstices of sample of sand due to change of excess pore pressure (Kiekbush et al. 1977)

2.4.5. Other Factors

Hvorslev (1941) in his work on subsurface exploration listed the requirements for a suitable undisturbed laboratory specimen: no disturbance of the soil structure, no change in water constituents or chemical composition. Such criteria are difficult to meet.

Various methods and tools for borehole sampling have evolved, and cores obtained with thin-walled tubes have been accepted universally. In spite of warnings, such as one by Terzaghi and Peck (1968) that "If tube samples have been taken, it is always desirable to investigate the extent to which the consistency of clay has been affected", little attention is paid to the extent of disturbance in most daily work.

Bozozuk (1970) showed that the long term storage of marine clays reduced the measured preconsolidation pressure by 4.8 percent. Thus consolidation tests should be done as soon as possible after sampling. Kallstenius (1971) found a general lowering of the strength of samples with storage time.

Many investigators (Lang 1971; Sone 1971; Shackel 1971) have found that tube sampling causes disturbance, resulting from excessive friction along the wall of the tube. Similar disturbances also occur during the extrusion of the samples.

The selection of the size of the specimen is still largely left to personal preference and convinience. Past work indicates that small specimens trimmed from large samples provide better test results. One recommendation is that the trimmed surfaces should be as small as possible in relation to original size of the specimen (Lang 1971). To avoid disturbance caused by remolding of the outer surfaces, large samples should be obtained and trimmed so that any remolded material will be removed (Holtz 1963). The shear strength of the Leda clay was shown also to be affected by the sample diameter (Coats et al. 1963). It was shown that the sample size is the most important factor influencing the shear behaviour of specimen (Lo et at. 1970).

The history of the clay sample prior to testing may have an appreciable effect on the properties of the clay measured in the laboratory. In the case of soft sensitive clays, observation by various authors have shown that the samples should be manipulated with great care during transportation, preparation and trimming to avoid any shocks, vibration, or stress concentration, which are found to disturb the clay structure and affect the measured properties.

Despite all due precautions with regard to manipulation, the properties of the clay samples may be altered by other phenomena, such as water migration within the sample. That problem was recognized by Hvorslev (1949) who suggested that "Seriously disturbed parts of the sample should, as far as possible, be separated from the undisturbed parts in order to avoid migration of pore water from the disturbed to the undisturbed parts". Bjerrum (1973) has reported on some observations made at NGI on Norwegian clays. Clay samples were tested immediately after sampling and after a storage time of three days, during this relatively short storage time water migration resulted in water content which were 3 to 4 percent higher in the core than in the outer zone of the sample, moreover a reduction of 15 percent of the undrained shear strength was observed even if the samples had been reconsolidated to the field stresses. The reduction was attributed to the internal swelling which had occured within the sampling tube during that fairly short period of time.

The restraints imposed by the filter paper is of importance and is more difficult to estimate accurately than that of the rubber membrane alone. Test on 1.5 in. (38 mm) diameter specimen, with and without drains, has shown that at failure which occurred around 15 percent axial strain, a total correction of approximately 14 kPa (2 psi) has to be applied to the compression strength to allow for both rubber membrane and drains. No direct experimental evidence on the correction necessary for the drains on 4 in. diameter specimen is available but assuming the same mechanism and using the data from the tests on 1.5 in. (38 mm) diameter samples, the magnitude of the correction can be calculated (Bishop et al. 1962).

In engineering practice the description of the geological origin of the material and certain physical properties such as structure and minerology is often sufficient. Rarely is any description of the chemical composition of the pore fluid. In some instances the lack of chemical information may lead to incomplete understanding of results or to misinterpretation. The influence of chemical factors on marine clay behavior has been examined in Norway and in Canada by various investigators. The results show that even small amount of chemicals in the pore water, such as salts, may affect the strength of soil considerably (Torrance 1976; Sangrey et al. 1971).

2.5. Cyclic Triaxial Testing

2.5.1. Systems for cyclic loading and most commonly performed tests

Cyclic loading problems in offshore environment arise from two primary sources: wave action and earthquake shaking. The physical behaviour of seafloor soils under such loading conditions has a major influence on design of piled and gravity platforms and seafloor anchor systems. The stability of natural slopes and for any excavation is also affected by cyclic loading.

Various systems for cyclic loading in soil mechanics have been developed (Klementev 1983). The simple ones are mechanical, like that of Grainger and Lister (1962), sophisticated systems are electro-hydraulic, which enable programable closed-loop controlled loading, such as the system of Cullingford, Lashine and Parr (1972). The basic control and monitoring circuit layout is shown in figure 2.11. Hydraulically operated triaxial apparatus can be easily adapted for cyclic loading by adding a hydrodynamic pulsator and by employing limiters to keep the maximum and minimum values of load unchanged. Such a cyclic triaxial apparatus is shown diagramatically in figure 2.12.

Figure 2.10 presents the layout of a triaxial test system used at NGI which is suitable for both static and cyclic loading tests. The details of this apparatus has been given in section 2.2.2. Figure 2.13 shows the triaxial cell used for cyclic loading tests. Cell of this type have been used for several years at MIT. The reason for making the cell in this way is to facilitate the connection process between piston and the top cap. This connection is more complicated for cyclic than for static loading tests, because no dead movement can be tolerated in cyclic loading tests. The connection are shown in figure 2.14. During mounting of the specimen, the piston with a connection piece at its bottom end is lowered over a top sticking up from the top cap. Then the two connection screws are fastened. The top cap is fixed to the piston. The universal joint, which is modified to give a minimum of false deformation, allows some tilting of the top cap. The gap between the top cap and the connection piece limits the tilting to ± 4 degrees.

During laboratory cyclic loading tests on saturated sands, the pore pressure response is normally recorded under undrained conditions. In field loading conditions, induced pore pressure may redistribute or dissipate during loading to an extent depending on: (1) the induced pore pressure gradients (2) the permeability and compressibility of the soil and (3) the frequency of loading. The application of drained cyclic loading to loose sands results in a progressive decrease in volume.

The most commonly performed cyclic tests are: (1) undrained cyclic loading tests and (2) drained cyclic loading tests. Undrained cyclic tests are performed to record the deformation and pore pressure response of the sample and drained cyclic tests are performed to record volume change of the sample and to calculate the cyclic modulus.

2.5.2. Factors Affecting Cyclic Triaxial Results

Field observations of loose saturated sand deposits that were liquefied during strong earthquakes have led to extensive laboratory studies on liquefaction resistance of sands. Most of the studies on this subject have been limited to the granular materials such as clean sands or gravel containing little or no fines. There are a few case where soils known to have liquefied were identified as silty sands or sandy silts containing 10 to 50 percent fines (Marshal 1961; Ikehara 1970; Yamanouchi et al. 1976).

Investigations by Pyke (1973), Ladd (1974), Mulilis et al (1977), Marcuson and Townsend (1974), and Mitchell et al (1976) have showed evidence that the liquefaction characteristics of saturated sands under cyclic loading are very much influenced by the method of specimen preparation or soil deposition.

The influence of many testing variables on sand samples are described in the literature and systematic testing procedures have been proposed (Lee et al. 1962; Seed et al. 1971; Seed 1976). The significance of cyclic loading rates has been

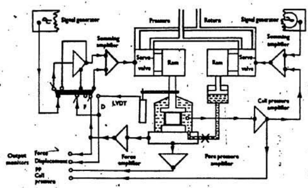


Figure 2-11: Schematic servo loop for dynamic triaxial testing of soils (Marshal 1962)

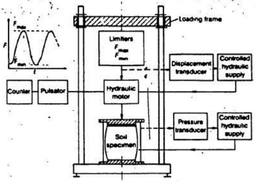


Figure 2-12: Diagrammatic layout of the triaxial apparatus (Klementev 1983)

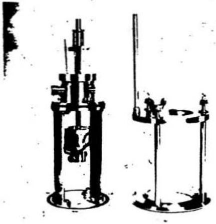


Figure 2-13: Triaxial cell for cyclic loading tests (Berre 1981)

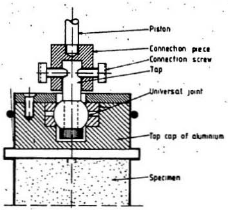


Figure 2-14: Details of connection between piston and top cap for cyclic loading cells (Berre 1981)

noted for selected studies on both sands and clays (Seed et al. 1976; Thiers et al 1969; Krizek et al. 1969; Arango et al. 1974). Cyclic triaxial test has steadily increased in usage as the predominant laboratory test for evaluating earthquake response of soils. The test has even broadened its utility to foundation problems of offshore structures under storm action (Lee et al. 1975; Bjerrum 1973; Peacock 1968). Peacock et al (1968) and Seed and Peacock (1971) realized that strengths measured in cyclic simple shear tests were about 35 to 50 percent less than comparable to strengths measured in cyclic triaxial tests and provided "correction factors" to more closely align cyclic triaxial strengths with estimated field response. Ladd (1974) focussed attention on the effects that specimen preparation procedures have on the cyclic triaxial strengths of sands. The most comprehensive study regarding specimen preparation effects on cyclic triaxial strength of sands was conducted by Mulilis et al (1977) who evaluated 11 different specimen preparation procedures. Mulilis et al (1978) also presented results concerning effects using a procedure of undercompaction during specimen preparation.

Marcuson and Townsend (1976) reported that isotropically consolidated undisturbed specimens from Fort Peck Dam were approximately 70 to 80 percent more stable than specimens of the same material reconstituted to the same density using a "dry rodding" specimen preparation procedures. They also reported that five other laboratories observed that reconstituted specimens generally gave lower strengths than undisturbed specimens.

Mulilis et al (1977) tabulated for various soils the effects of reconstitution, which shows that the ratio of strengths of undisturbed to reconstituted samples varies from 1.0 to 2.0, i.e. from the absence of effects due to reconstitution to a 100 percent strength loss depending upon the method of reconstitution and site.

Seed and Lee (1966) reported that the number of cycles required for liquefaction increased with confining pressure for specimens prepared such that they had the same void ratio after consolidation. Finn et al (1971) examined the data from previous triaxial tests by Lee and Seed (1967) and observed that the relationship between cyclic shear stress required to cause initial liquefaction in a specific number of cycles and effective confining pressure was linear for any given void ratio. Castro and Poulos (1976) also show that stress ratio, $R=\sigma_{dc}/2\sigma_c$ decreases with increasing confining pressure for various relative densities and soil types. Lee and Focht (1975) pointed out that for practical purposes within small ranges of pressure, cyclic strength is directly proportional to effective confining pressure.

Lee and Fitton (1969) found that triangular loading wave forms gave somewhat higher strengths than rectangular loading wave forms. Seed and Chan (1964) and Thiers (1965), in investigation on loading wave effects on undisturbed sensitive clays, found that triangular loading wave shapes gave 5 to 20 percent higher strengths than would be obtained under rectangular loading. Silver et al (1976) showed that the rectangular loadings with fast rise times caused stress waves in the specimen and corresponding "ringing" in the pore-pressure traces, resulting in cyclic strengths 15 percent lower than those tested using sine wave or degraded rectangular loadings. Mulilis et al (1978) compared the effects of rectangular, triangular, and sine wave loadings and found that the order of increasing strength was rectangular, triangular, and sine. Ishihara and Yasuda (1972) and Annaki and Lee (1976) have confirmed the validity of the equivalent uniform cycle concept by performing cyclic triaxial tests with irregular loading wave forms. Annaki and Lee (1976) observed that when converting from irregular to uniform cycles, extension peak produced about 90 percent of the total damage, because the undrained strength of sand is less in extension than compression.

Lee and Fitton (1969) and Lee and Focht (1975) found that the slower loading frequencies produced slightly less than 10 percent lower strength. Wong et al (1975) Mulilis (1975), and Wang (1972) found that slower frequencies gave slightly higher strengths. Although this is conflicting, it can be safely concluded that frequency effects have only minor effect on cyclic strength of cohesionless soil.

Lee and Fitton (1969) and Wang (1972) compared the effects of size on 35.6 mm and 71.1 mm diameter specimens and found that it has very little effect. Wang

(1972) compared the effects of height to diameter ratios of 1.0 to 2.3 and found that the specimens with a height to diameter ratio of 1.0 were approximately 20 to 50 percent stronger than the standard specimens.

Wang (1972) conducted tests at a frequency of 2 Hz comparing very rough and smooth end platens. His data showed that cyclic strength was insensitive to cap and base roughness, with smooth end platens being approximately 5 percent stronger.

The effect of relative density on cyclic strength was recognized early in the history of cyclic tests. Lee and Seed (1967) reported that cyclic stress required to cause initial liquefaction increased linearly to approximately 60 percent relative density.

Lee and Fitton (1969) compared the effects of particle size based upon mean grain diameter D_{50} and found that as grain size increases, the cyclic strength also increases, with very fine sands having about half the cyclic strength of gravels. Ishihara et al (1978), show a 28 percent cyclic strength increase as D_{50} decreases from 0.1 to 0.001 mm. Wong et al (1975) found that well graded material was somewhat weaker than uniformly graded material.

Finn et al (1970) evaluating the effects of reliquefaction in cyclic triaxial tests, found that once a specimen has liquefied and reconsolidated to a denser structure, despite this densification, the specimen is much weaker to reliquefaction. Mori et al (1977) investigated the effects of sampling on strain history.

Lee and Focht (1975) observed an increase in cyclic stress ratio, $\sigma_{dc}/2\sigma_c$ of about 30 percent for an overconsolidation ratio (OCR) of 3 for very dense sand. Likewise, Ladd (1976) observed an increase in cyclic strength of about 20 percent for an OCR of 2. Ishihara et al (1978) has shown that the cyclic strength increases as OCR and fines content increases.

Two series of cyclic triaxial tests were performed by Ishihara et al (1978) on soil containing fines (passing # 200 US sieve) from 0 to 100 percent by weight. The first series included the testing of laboratory prepared reconstituted specimens overconsolidated to over consolidation ratio (OCR) values of 1.0 to 2.0. It was shown that the overconsolidation had a definite effect in increasing the cyclic resistance of the specimens, and its effect became more pronounced as the contents of the fines increased. The second series dealt with the tests on undisturbed specimens of alluvial deposits in Tokyo as well as tests on remolded specimens of the same soil. It was shown that the cyclic strength of the undisturbed specimens was about 15 percent greater than that of the reconstituted ones. Comparison of the results show that the greater cyclic strength of the undisturbed specimens over that of the remolded specimens might have been caused by a slight overconsolidation which existed in the alluvial deposit of the sands containing the fines.

There is no generally accepted method for conducting cyclic loading tests on clays, although several methods have been described (Holtz 1963; La Rochelle et al. 1974; Sangrey 1969). For sands systematic testing procedures have been proposed (Lang 1971; Sone 1971; Shackel 1971). However, most of this work was done using relatively rapid rates of loading since the practical application of the results was to earthquakes. The significance of cyclic loading rate has been noted for selected studies on both sands and clays (Holtz 1963; Coats et al. 1963; Lo et al. 1970; Bjerrum 1973). These studies have shown that under a rapid cyclic loading rate it usually takes more cycles to achieve a particular result, either failure or some level of strain, than for an identical test run under a slower loading rate. On the basis of these observations it is not clear whether failure will occur under some loading rates but not others for a particular stress level. One reason for this uncertainty is that testing errors have not been separated from testing effects.

Conducting an accurate cyclic loading test requires as a minimum, all of the care and control necessary in normal soil testing (Bishop et al. 1962). In addition, several factors can introduce significant limitations in cyclic triaxial tests. The most important of these are pore pressure measurement and equalization problems and the effects of air diffusion. Even if the initial state of stress in-situ can be applied to test specimen, the actual cyclic loading will involve fluctuations both in normal stress and shear stress and also involve rotation of principal stresses. These conditions can not be duplicated with conventional testing equipment. Consequently, any triaxial test involves a great simplification even without considering the problems of duplicating the time history of cyclic loading. Several studies have been dealing with these problems (Sone 1971; La Rochelle et al. 1974).

It was found that specimen preparation methods, differences between intact and reconstituted specimens, density, and prestraining have major effects on the cyclic strength. Confining stress, loading wave form, material grain size and gradation, overconsolidation ratio, and consolidation stress ratio (K_0) may also be of influence. Finally some factors have minor effects: freezing intact specimens, loading frequency, specimen size, and frictionless caps and bases.

The effects of these various testing and material factors on the cyclic triaxial strength of cohesionless soils and their relative magnitudes are summarized in table 2.3 (Townsend 1978).

Table 2-1: Summary of factors affecting triaxial strength of cohesionless soil Ref. [94]

Variable Effect of	Testing Conditions and Materials	Effect(Result)
Testing laboratory and equipment	"standard sand" tested by eight different laboratories following specified testing procedures and conditions. Monterey No. 0 sand.	excellent agreement between laboratories provided strict adherence to testing procedures followed
Specimen preparation	specimens formed by pluviation through air of water, vibration or tamping in dry or moist condition. Monterey No. 0 sand, 50% D_r and other soils.	weakest specimens formed by pluviation through air, while strongest formed by vibrating in moist condition. Difference in stress ratio for failure can be 110%
Reconstituted versus intact	"intact (relatively undisturbed) specimens tested, then remolded and reconstituted to same density and retested under same conditions as intact. A variety of sands and specimen preparation techniques for reconstitution used.	intact specimens stronger than reconstituted. Strength decreases range from 0 to 100% depending upon material and reconstitution method
Freezing intact specimens	intact specimens transported to laboratory in frozen and unfrozen condition, reconstituted specimens frozen in laboratory and compared with unfrozen specimens	no effect, with testing variation, due to freezing
Confining stress σ_3	tests conducted at a variety of confining stresses on variety of sands	within small range of pressures, cyclic strength is directly proportional to confining stress
		cyclic stress ratio decreases with increasing confining pressure 0.0007 to 0.004 per psi increase in σ_3
Loading wave form	tests conducted using rectangular, rounded rectangular, triangular, and sine wave loading shapes	order of increasing strengths: rectangular, degraded rectangular or triangular, sine. Sine wave approximately 30% stronger than rectangular
	irregular wave forms simulating earthquake stress histories evaluating equivalent cycle concept	equivalent cycle approach valid. Cyclic triaxial affected by extension more than compression

Variable Effect of	Testing Conditions and Materials	Effect(Result)
Frequency	frequencies with combined range of 1 to 1000 cpm for various sands. Typical ranges	slower loading frequencies have slightly higher strengths. For range of 1 to 60 cpm, effect is 10%
	water and air used as confining media while evaluating frequency effects	water may affect results at 5Hz
Specimen size	strengths of 35.6- and 70-mm -dia specimens compared with strenghts of 70- and 300-mm-dia specimens	no effect for this range 300 mm dia approximately 10% weaker
Frictionless caps and bases	specimens with friction and frictionless caps and bases at frequencies slow enough to allow grease to function	no effect between full friction and frictionless caps and bases
Relative density	tests conducted on variety of cohensionless soils over wide range of stress and testing conditions	cyclic strength increases dramatically with increasing density. Linear relationship between cyclic stress ratio and relative density to approximately 60% D_r, but slope is dependent on soil type, fabric, confining pressure, and failure strain
Particle size and gradation	cyclic strengths of different soils at comparable testing conditions compared on basis of mean grain diameter, D_{50}	sands have D_{50} = 0.1 mm have least resistance to cyclic loading. As D_{50} increases from 0.1 to 30 mm, a 60% strength increase observed; as D_{50} decreases from 0.1 mm to silt and clay sizes, rapid increase in strength observed.
Prestraining	well-graded material compared with uniformly graded material both having same D_{50} size	well graded somewhat weaker than uniformly graded material
	specimens liquefied by cyclic loading, reconsolidated and reliquefied by applying same original cyclic load	despite increase in density due to consolidation, liquefaction causes considerably weaker specimen

Variable Effect of	Testing Conditions and Materials	Effect(Result)
Overconsolidation ratio (OCR)	specimens prestrained to 50 to 80% pore pressure response, reconsolidated, and reloaded cyclicly	precycling greatly strengthens cyclic strength
	specimens overconsolidated by consolidating to higher stresses and rebounded to lower testing stresses	OCR's of 1 to 4 and 1 to 8 increased cyclic simple shear stress ratios 75 and 150%, respectively
		OCR's of 1 to 2 increased cyclic strengths 30 to 80% depending upon amount of fines. Amount of fines affect OCR
Consolidation K_c	specimens consolidated anisotropically for variety of confining stresses and subjected to reversing and nonreversing cyclic stresses	maximum deviator stress required for failure increases with K_c ratio for given $\bar{\sigma}_{3c}$
		method of presenting data influences conclusions: τ_{cf} versus σ_{fc} recommended
		isotropic consolidation may not always provide conservative results

Chapter 3
IDENTIFICATION OF INVESTIGATED SOILS

3.1. Type of Soils Tested

The following three types of soils were investigated under static loading (1) sand, (2) clayey silt, and (3) silty clay. The sand and the clayey silt were also tested under cyclic loading.

3.1.1. Sand

The sand was obtained from the Hibernia field of the Newfoundland Grand Banks at a depth of 91.0 m. of water using a IKU Grab Sampler. The location of the site was latitude 47^0 15' 14" N and longitude 49^0 16' 80" W and is shown in figure 3.1. The sand samples were taken by the Center for Cold Ocean Resources Engineering (C-CORE), Memorial University of Newfoundland and were brought to the soils laboratory. The sand was air dried before performing preliminary tests. Particles with diameter of grains greater than No. 4 U. S. sieve were removed. Table 3.1 shows the physical properties of sand. Particles were of subangular shape.

Sieve analysis was done according to the ASTM D 422-63 (Reapproved 1972) as shown in figure 3.2. Uniformity Coefficient C_u and Coefficient of Curvature C_c were found to be 1.66 and 1.0 respectively. According to the United Soil Classification (USC), this material is a poorly graded medium to fine clean sand (SP) with little fines. The density ranges from 1580 to 1640 kg/m^3 for loose sand, 1650 to 1710 kg/m^3 for medium sand and 1720 to 1780 kg/m^3 for dense sand.

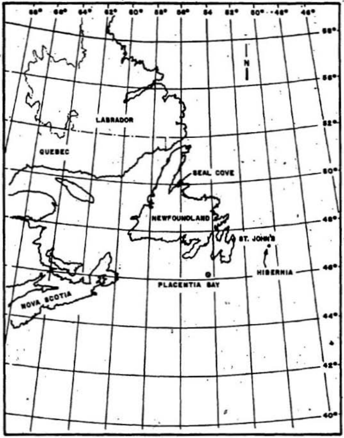


Figure 3-1: Location of Project Sites on Newfoundland Map

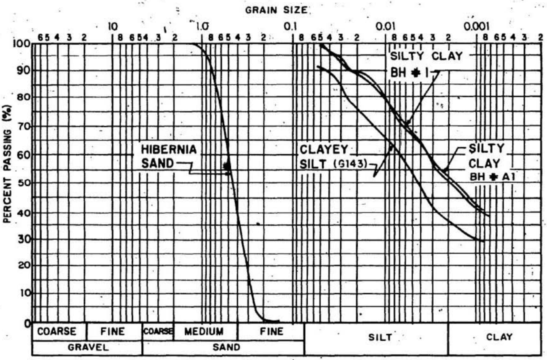


Figure 3-2: Grain Size Distribution

In monotonic triaxial tests, the specimens were given the following code: HIB stands for Hibernia, the second symbol 00 stands for monotonic tests and the last number is the number of the specimen tested. In cyclic tests, the middle number 00 has been replaced by CY, which stands for cyclic.

3.1.2. Clayey Silt

In May 1978, the Ocean Engineering Group at Memorial University in cooperation with the personnel from the Bedford Institute of Oceanography (BIO), Huntex ('70) Ltd., and the Geological Survey of Canada undertook a multi-device high density sea floor survey in the outer Placentia Bay area on the Newfoundland Grand Banks (figure 3.1), using BIO ship CSS Hudson. Data was collected in a site bounded by latitude 46^0 to 47^0 N and longitude 54^0 10' to 55^0 10' W. A few stations were selected within each area to collect grab samples, piston cores, and penetrometer data as well as selected data. The physical properties of soil are shown in table 3.1 .

In monotonic triaxial tests, the first symbol in the name of the specimens is "HUD", which stands for Hudson, the second symbol "G143" stands for borehole number and the last number is the specimen number tested. In cyclic tests, the third number "CY" stands for cyclic.

Hydrometer analysis was done to plot the particle size distribution as shown in figure 3.2. According to USC classification, the soil is classified as MH, inorganic clayey silt (35% passing υm sieve with little sand (7%)). (Figure 3.3).

3.1.3. Silty Clay

Samples were obtained from West Coast of Newfoundland from a location called Seal Cove at latitude 49^0 31' and longitude 57^0 51' using a 4 in (101 mm.) diameter thin-walled Sampler. They were part of a project conducted by Golder Associate, Consulting Geotechnical and Mining Engineers, St. John's and were brought for additional testing to the soils laboratory at Memorial University.

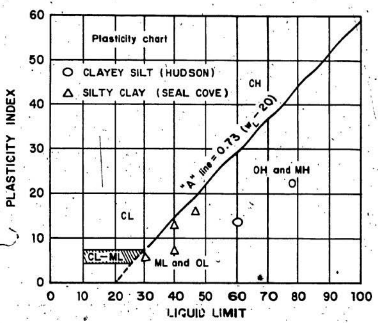


Figure 3-3: Plastic Properties of Investigated Soils (USC)

Samples were taken at various depths. After sampling, both ends of the sample tubes were sealed with heavy wax coating. The material was then stored in a humidity controlled room untill required. At this time the samples were extracted from the sampling tube and sliced into lengths of approximately 0.12 m. (5.0 in.). These specimens were carefully wrapped in a thin plastic sheet and sealed with wax. Before performing the actual triaxial test identification tests were carried out. The results are shown in table 3.1.

Hydrometer analysis was performed in order to calculate the particle size as shown in figure 3.2. For USC classification see figure 3.3. According to the USC classification the soil is MH.

In monotonic triaxial tests, the first symbol in the name of the specimen is "GA", which stands for Golder Associates, the second symbol stands for borehole number, the third number stands for the position of the sample, i.e. PS1 stands for shallow water and PS2 stands for deep water, and the last symbol is the number of the specimen tested.

An oedometer test was also performed on the silty clay from borehole # A1 in order to have some information about its preconsolidation pressure. The sample was taken at a depth of 18.3 m. which means an estimated 140 kPa overburden effective pressure. That value is based on a water table at ground surface. Under artesian conditions likely to occur on this site, the overburden effective pressure could drop to 130 kPa.

The test was carried out in a Clockhouse Oedometer frame with a 0.002 mm accuracy of dial gauge, and the results are shown in figure 3.4. The preconsolidation pressure, as estimated by Casagrande's method is 60 kPa. The discripancy between that value and the estimated overburden pressure is probably due to the sample disturbance. This problem is aggravated by the need to use small diameter samplers at great depth and the long storage duration.

Table 3-1 Physical Properties of Soils

Sand

Relative Density of the grains (D_R)	= 2.65
Minimum Density (ρ_{min})	= 1570 kg/m^3
Maximum Density (ρ_{max})	= 1780 kg/m^3

Clayey Silt

Station	Grid Identifier	Relative Density of grains (D_R)	Liquid Limit w_l%	Plastic Limit w_p%	Plasticity Index I_p%
1	G121	2.65	78.0	55.3	22.7
1	G143	2.78	60.0	45.6	14.4

Silty Clay

Bore Hole #	Depth (m.)	Relative Density of grains (D_R)	Liquid Limit w_l%	Plastic Limit w_p%	Plasticity Index I_p%
1	11.5–12.1	2.83	40.0	32.5	7.5
1	16.5–16.9	2.78	46.0	29.1	16.9
1A	18.0–18.7	2.83	40.0	16.9	13.1
4	10.7–11.3	2.80	30.0	22.4	7.6

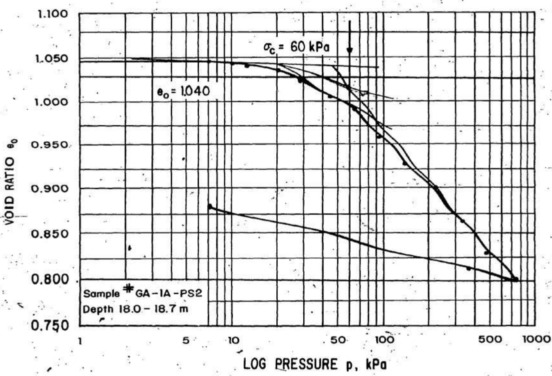


Figure 3-4: Oedometer Curve for Silty Clay

Chapter 4

APPARATUS AND TECHNIQUES USED FOR MONOTONIC LOADING OF VARIOUS SOILS

4.1. Triaxial System Used

The triaxial system implemented in the soil laboratory at Memorial University is suitable for monotonic and cyclic triaxial testing of various soils. Specimen preparation methods were established for laboratory triaxial testing of samples. The system is composed of four main parts: (1) the triaxial cell (2) the pressure control device (3) the volume change device and (4) the loading press. Assembly of the experimental set up is shown in figure 4.1.

4.1.1. Triaxial Cell

The triaxial cell has the following features:

1. 38.0 mm. - 75.0 mm. diameter specimen may be accomodated
2. Capability of compression as well as extension loading.
3. Low friction seal resistance between loading piston and top cap.
4. Built in load cell and pressure transducer.
5. Cell may be used for both monotonic and cyclic loading.

The cell was bought commercially and a few modifications were made to suit the present testing of soils. A picture of the triaxial cell is shown in figure 4.2. The cylindrical acrylic chamber is 125.6 mm. (4.9 in.) in outside diameter, 6.5 mm. (0.25 in.) thick, and 295.0 mm. (11.6 in.) long. O-rings are used for sealing the top cap and the base plate to the acrylic chamber. The cell may be used for a maximum allowable internal pressure of 690 kPa (100 psi). Four 1.27 mm. (0.05 in.) diameter stainless steel rods and tightening screws are used to flatten out the O-rings between the acrylic chamber and the plates.

4.1.2. Volume Change Device

The volume change device was designed and developed in the laboratory, and was ready for use at the time when candidate started his research. The following paragraphs provide some of its features. The use of a burette was chosen for measuring volume change. The fluid used in the burette was a red coloured kerosene which creates a flat maniscus that was clearly visible through the outer transparent cylinders. Kerosene also does not wet the glass or plexiglass and this was convenient for keeping the burette clean. The glass burette used was graduated in cm^3 and any change in volume can be directly measured. A two way valve was used to reverse the direction of movement of fluid when the limit of the volume change scale is reached, hence eliminating any disconnection of lines. Another two way valve is provided on the panel which allows volume change to be measured through the burette or not. The burette was located in a 10.2 mm. (4.0 in.) diameter cylinder as shown in figure 4.3a. For example when the back pressure is applied to saturate the soil sample, water is generally pushed into the sample. The amount of water pushed into the sample is read on the burette. This volume change device has a sensitivity of 0.1 cm^3. The outer cylinder acts as an compensation cylinder to avoid expansion of both inside cylinder and burette. This is an improvement on Imperial College System (commercialized under Wykeham Farrance patent) for accurate measurement of volume during the saturation.

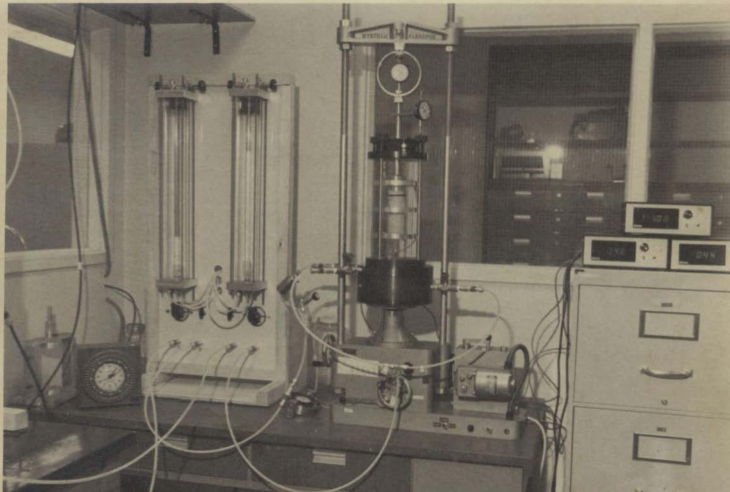


Figure 4.1; Experimental set up for monotonic loading tests

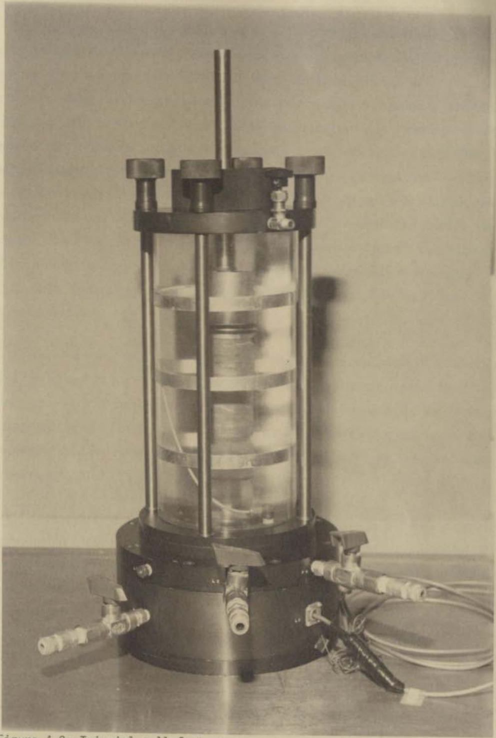


Figure 4.2: Triaxial cell features

4.1.3. Pressure Control Device

The following paragraphs provide some of its functions and limitations. The cell and the back water pressure applied to the specimen during testing are provided by an air compressor through air-oil-water interface located in a pressure reservoir. The reservoir has been designed for a maximum pressure of 828 kPa (120 psi). The pressure can be accurately maintained for any desired period of time using air pressure regulators that maintain the desired pressures regardless of minor fluctuations in the air line pressure. The air pressure regulators used have a capacity ranging from 690 to 3450 kPa (100 to 500 psi) and will maintain the constant pressure to within 0.69 kPa (0.1 psi). As an additional feature the system allows for refilling of the reservoir under pressure during a test without disconnecting the line. This is particularly useful for permeability measurement or long tests testing where unavoidable leakages through the cell must be compensated.

All the connecting lines on the pressure control device are made of 6.35 mm. (1/4 in.) diameter high pressure plastic tubing. The pressure control device is made up of four identical separate systems that can be connected to the volume change device and / or directly to the cell. An air pressure dial gauge is provided to roughly set the cell and back pressures. That gauge can read a cell and back pressure upto 2070 kPa (300 psi).

A "quick connect" system for line connections (Swagelock) provides a great versatility in conducting tests and avoids air bubbles. Accurate measurement of the water pressure both in the cell and in the sample are made by mean of Kulite relative pressure transducers.

A schematic picture of the apparatus is shown in figure 4.3b.

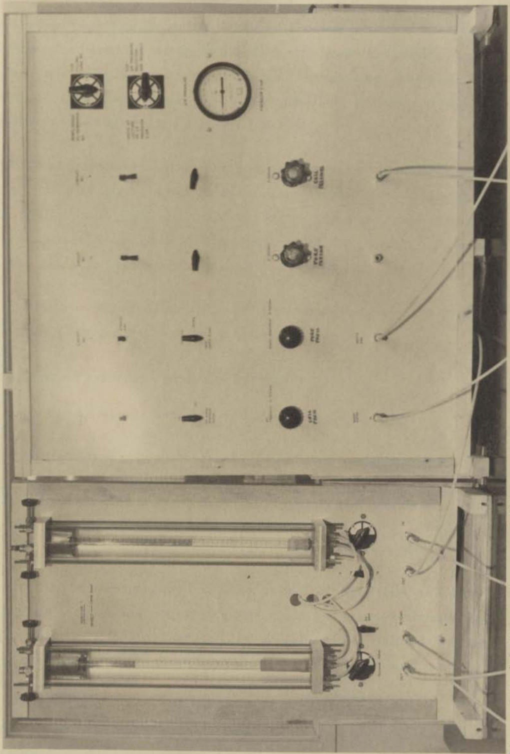


Figure 4.3: (a) Volume change and (b) pressure control devices

4.1.4. Load Press

The load press used for monotonic triaxial compression tests was a Wykeham Farrance load press. It may be used for both compression and extension tests. A wide range of displacement rates are available through a five gear system. The rates of displacement ranges from 0.0006 mm./min. to 1.52 mm./min. The capacity of the loading frame is 9810 N. (1000 kgf.).

The load is applied by a mechanical loading jack operated by a motorized gear box. The motor forces the jack up or down at a continuous rate depending upon the gear setting. Rigid steel rods carrying an adjustment rigid cross beam, mounted in a strengthened alloy housing, ensures that virtually no distortion occurs within the frame work of the machine during the test.

The machine has sufficient clearance to accomodate the triaxial cell previously described. The load press is shown in figure 4.4.

4.2. Tests Performed with the System

Following tests were performed on sand, clayey silt, and silty clay.

1. Isotropically consolidated undrained (CIU) triaxial compression tests.

2. Isotropically consolidated drained (CID) triaxial compression tests.

3. Isotropically consolidated undrained (CIU) triaxial extension tests.

Sand samples were mounted in a loose, medium or dense state, as defined in Chapter 3. Samples were tested using both standard or enlarged lubricated ends. All samples tested for clayey silt were of remolded soil using standard or enlarged lubricated ends. Samples of silty clay used in the tests were either remolded or intact and mounted with standard ends only.

All samples were back pressure saturated and isotropically consolidated before shearing.

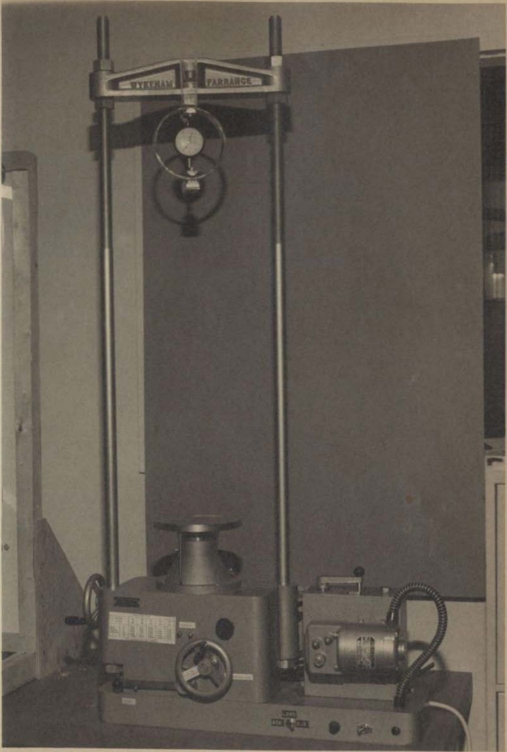


Figure 4.4: Load press: W.F: (model 10021)

4.3. Sample Preparation

Soil preparation methods used in the laboratory for mounting various types of soils are now described.

4.3.1. Sand

Several methods of pouring sand into the mold were tested to determine which method provided a sample close to the minimum density for a loose sample. The method finally adopted was to pluviate the sand through a funnel. No degradation effect was noticed and the density obtained was fairly constant. For dense samples, sand was saturated with a small amount of water, poured into the mold and then compacted. For further details on mounting the sample see appendix A.

4.3.2. Clayey Silt

All samples tested for clayey silt are of remolded soil because of the poor conditions of the samples when extracted from the sampling tubes. Soil was thus extracted from the plastic tubes and remolded. Remolded soil poured into the triaxial mold in layers and compacted at the natural water content. For further details on mounting of the sample see appendix A.

4.3.3. Silty Clay

After removing the wax around the sample, the specimen was trimmed to a diameter of approximately 38.0 mm. The length of the sample was around 90.0 mm. For remolded samples, enough soil was remolded, poured into the mold and compacted. The size of the mold chosen was greater than the final size of the sample. The mold was then unclamped and the sample mounted in the trimming apparatus. Trimming is being done as for intact samples. Care has been taken to keep the same density for all the remolded samples.

Two thin membranes (thickness = 0.1 mm.) have been used to enclose the sample in order to make sure that there is no leak through the membranes. For detailed procedures see appendix A.

4.4. Backpressure Saturation

Since Lowe and Johnson's (1960) investigation, specimen saturation by applying a backpressure has become a widely used technique. The methods and magnitudes of backpressure required to saturate specimens are provided by various authors (Bishop et al. 1962; Laboratory Soil Testing 1976). Lee and Black (1972) provide theoretical and experimental data for time and magnitude of backpressure to dissolve air bubbles.

The procedure which has generally been adopted for backpressure saturation is to incrementally increase the cell pressure and pore pressure simultaneously, allowing equalization at each increment. After equalization, the value of $B = \Delta u/\sigma_3$ (Skempton 1954) is measured by applying the next increment. Several variables are involved in this procedure: (1) the magnitude and duration of backpressure increment, (2) the magnitude of the effective consolidation pressure during the saturation which may or may not permit the specimen to swell, (3) the magnitude of the cell pressure increase when checking B-parameter, and (4) the magnitude of backpressure applied which should not pre-stress the specimen, i.e. the effective confining pressure should not be greater than that under which the specimen is to be sheared.

Two other methods were applied to saturate the sand samples and to remove the air bubbles from the samples: (1) circulation of water through the sample under a slight hydraulic gradient (1.0 m. of water head), (2) partial vacuum applied after the circulation of water to a few sand samples in order to make sure that there was no air bubble left in the circuitry. Finally backpressure has been applied to dissolve any gas remaining in the sample.

4.5. Consolidation

Although in-situ stress conditions are usually anisotropic, isotropic stress conditions are generally used in routine triaxial tests. The reason usually given is that anisotropic consolidation requires more time and complicated procedures. In addition it was thought formerly that the angle of shearing resistance in terms of effective stress was not significantly affected by method of consolidation. However, this is not the case for all the soils especially for intact clays.

In this study all the soil samples were isotropically consolidated.

4.6. Rates of Loading

The rate of loading may significantly affect the magnitude of the shear strength. Increased rates of loading produced increased strengths and in case of extremely slow loading rates creep effects will cause lower measured strengths. If saturated specimens are tested, the selected loading rate must be slow enough so that excess pore pressures do not build up (drained test) or have enough time to equalize throughout the specimen (undrained test). The selection of the loading rate is of considerable importance as the time required to perform a test is directly related to cost.

Bishop and Henkel (1962) proposed that a 95 percent pore pressure dissipation throughout the sample will be acceptable. That criteria leads to the following estimate of the time to failure T_f.

$$T_f = 20H^2/nC_v$$

where

T_f = time to failure,

H = 1/2 the specimen height,

C_v = coefficient of consolidation, and

n = a coefficient related to drainage boundary condition.

In the case of undrained tests, it has been found (Casagrande et al. 1964) that for contractive soils, in the extreme, creep leads to failure without an increase in strength and thus modulus values as well as strength are low. In the case of dilative soils, it has been found (Donaghe et al. 1978; Casagrande et al. 1964) that the strength is not greatly affected by loading rate, however modulus values may be. For normally consolidated soils, Donaghe (1971) reports a 5 percent decrease in strength per tenfold increase in time to failure.

Methods of applying the axial load to the sample are influenced both by the requirements of the test and the need for mechanical simplicity. Two types of loading are generally used, stress controlled and strain controlled.

For routine tests and for some common research tests the use of strain controlled has many advantages and is generally accepted. The rate of strain at failure is accurately known and the influence of rheological factors on the observed strength can thus be taken into account. The shape of the stress-strain curve beyond the peak can also be observed. The duration of the test can be predicted with reasonable accuracy.

The values adopted for loading rate for monotonic tests in this research are as follows: Sand: 10 percent of deformation in two hours for both drained and undrained tests i.e. $8.3 * 10^{-2}\%$/ min. For clayey silt and silty clay: 10 percent of deformation in ten hours for undrained tests i.e. $1.7 * 10^{-2}\%$ / min. and 10 percent of deformation in 24 hours for drained tests i.e. $6.9 * 10^{-3}\%$ / min.

4.7. Special Tests

The following special tests had been performed on sand and clayey silt.

1. Isotropically consolidated undrained (CIU) triaxial compression test with control of the pore pressure on sand samples.

2. Isotropically consolidated drained (CID) triaxial compression test with constant axial stress σ_1 and decreasing lateral stress σ_3 on clayey silt samples.

Chapter 5
MONOTONIC TRIAXIAL TESTS: RESULTS

5.1. Sand

Sixteen monotonic triaxial compression tests with constant confining pressures, 241.5 to 345.0 kPa (35 psi to 50 psi) were performed on the Hibernia sand. All the samples were saturated under back pressure, isotropically consolidated and sheared under drained or undrained conditions. There were two main reasons for the saturation of the samples: (1) to simulate the hydrostatic neutral pressure that prevails in offshore conditions (2) to permit reliable measurement of volume changes in drained tests or pore pressure changes in the undrained tests. Samples of loose, medium and dense sand were tested.

Parameters of Mohr-Coulomb failure criteria were determined for the loose medium and dense samples.

Six loose, six medium and three dense samples were tested. The average minimum density for loose, medium and dense samples were 1610 kg/m^3, 1670 kg/m^3 and 1750 kg/m^3 respectively. The accuracy of the densities determination was estimated as 2 percent i.e. $\pm$300 kg/m^3. The good repeatability of the density determination means that the sample preparation gave in average satisfactory results. The densities were calculated using dry sample weights, initial sample heights and diameters measurements. No corrections for the change in volume of the samples were made for isotropically consolidated drained tests (CID). The relative density of the grains was 2.66, the initial void ratio e_0 ranged between 0.62 - 0.67 for the loose samples and 0.52 - 0.54 for the dense samples. Samples with standard ends (S) as well as samples with enlarged lubricated ends (L) were used. The rate of shearing for both undrained and drained tests was 0.15 mm/min.

The effective stress paths for undrained and drained tests for loose, medium and dense samples are shown in figures 5.1 & 5.2 and in figures 5.3 & 5.4 respectively. On these figures, the strains at the end of the test are indicated in percentage. The effective angle of internal friction ϕ' was evaluated assuming a cohesionless behavior.

The stress versus strain and pore pressure versus strain relationships for undrained tests, for loose, medium and dense samples are shown in figures 5.5 & 5.6 and in figures 5.7 & 5.8 respectively.

The pore pressure plots in both loose and dense samples using standard ends show that the pore pressure increases slightly initially and then decreases. Also, the same trend has been noticed using enlarged lubricated ends. It should also be emphasized that the excess of pore pressure generated during the undrained shear of the sample is less for the sample with enlarged lubricated ends than for the standard ones. The stress versus strain and volume change versus strain plots for drained tests, for loose, medium and dense samples are shown in figures 5.9 & 5.10 and figures 5.11 & 5.12 respectively.

The volumetric strain plots for loose, medium and dense samples using standard ends show a decrease in volume during the shear. The same trend has been noticed when using enlarged lubricated ends. Volumetric strain was defined as positive when total specimen volume increased and negative when the volume decreased. The dilation effect has not been noticed in loose or dense samples using standard or enlarged lubricated ends. Table 5.1 summarizes the tests conditions.

Special tests were performed in which the deviator stress was kept constant and the pore pressure was increased. It has been noticed that pore pressure parameters and failure criteria on effective stress path remain unchanged as shown in figure 5.13.

5.2. Clayey Silt

Thirteen tests were performed on samples of remolded clayey silt from Placentia Bay area. The average diameter and height of the samples were 72.1 mm. and 157.9 mm. respectively. Total confining pressure was 345 kPa (50 psi) except in a few tests in which it was 276 kPa (40 psi). All specimens were isotropically consolidated under 138 kPa (20 psi) except for one test in which the consolidation pressure was 69 kPa (10 psi) effective. All samples were back pressure saturated under a 198 kPa (30 psi). After applying an increment of cell pressure the corresponding build up in pore pressure was measured, allowing therefore the determination of the coefficient $B = \Delta u / \Delta \sigma_3$. Sufficient time was thus given after applying the back pressure to allow the water to flow into the sample. The B parameter was employed to check the degree of saturation. A value of 98 percent was thought sufficient to ensure a reliable degree of saturation.

After saturation has been achieved, generally within 15 hours, the back pressure was kept constant and cell pressure increased to get desired isotropic effective consolidation pressure 69 or 138 kPa (10 or 20 psi). The consolidation procedure was monitored by recording the amount of water flowing from the sample until the end of the primary consolidation as defined by the Bishop and Henkel (1962).

In isotropically consolidated undrained (CIU) test following consolidation the sample was axially compressed at a constant rate of strain under undrained conditions. Excess of pore pressure were measured by a pressure transducer located at the triaxial base. Axial compression or extension was continued generally upto an axial strain of 10 percent. For isotropically consolidated drained (CID) tests, drainage valves were kept open during the axial compression and the change in volume was recorded through the volume measurement device.

It was necessary to select a strain rate which allowed pore pressure equalization throughout the sample volume. The strain selected for undrained test was 10 percent of sample deformation in 8 hours (0.000003 s^{-1}) and for drained test, 10 percent of sample deformation in 24 hours (0.000001 s^{-1}).

The dry unit weight of the thirteen remolded samples ranges from 1182 to 1378 kg/m^3. The relative density of the grains was 2.78, the average initial void ratio e_0 was 1.173. The effective angle of internal friction, ϕ' was calculated for each test assuming that the value of cohesion, c' was equal to zero. The effective stress paths for undrained triaxial compression and extension tests are shown in figure 5.14. The effective stress paths for drained triaxial compression tests are shown in figures 5.15. The stress, pore pressure and volume change versus strain curves are shown in figures 5.16, 5.17, 5.18 and 5.19.

The plot of the pore pressure versus the strain shows that the pore pressure increased continuously and becomes almost constant at around 10 percent strain. The volumetric plots using standard and enlarged lubricated ends shows that the volume decreases under deviatoric stress. Volumetric strain is defined as positive when total specimen volume increased.

A special test was performed in which the axial stress σ_1 was kept constant under increasing deviator stress by decreasing the cell pressure. The stress-strain, volume change versus strain and effective stress path are shown in figure 5.20, 5.21 and 5.22 respectively.

Table 5.2 summarizes the tests conditions on clayey silt.

5.3. Silty Clay

Twelve triaxial compression tests were performed on silty clay samples from the West Coast of Newfoundland. The confining pressure was kept constant during the test and its value is given in table 5.3. All the samples tested were back pressure saturated and isotropically consolidated in the same way as the calyey silt, the values of pressures are shown in table 5.3. Again, a 98% value of B parameter was taken as criteria for sufficient saturation. No lateral drains were used during the consolidation step and the samples were allowed sufficient time for consolidation. The samples were sheared under drained or undrained condition. The samples trimmed directly from the sampling tube are called "intact" as opposed to "remolded" sample, which are constituted of remolded material from the same tube.

The rate of strain chosen for drained test was 10% of sample deformation in 24 hours i.e. 0.006 mm/min. For undrained tests, it was 10% of sample deformation in 12 hours i.e. 0.012 mm/min.

The stress paths corresponding to CIU and CID tests are shown in figures 5.23 and 5.24 together with the Mohr - Coulomb failure parameters for both "intact" and "remolded" samples. The failure envelop can be estimated for "intact" soil as $c' = 13$ kPa, $\phi' = 22.7^0$ for the upper envelop of all tests, and as $c' = 0$, $\phi' = 22.7^0$ for the "large deformation" envelop. For "remolded soil, the results are very close to the previous one, ϕ' being changed to 22.3^0.

The stress-strain curves are shown in figures 5.25, 5.27, 5.29 and 5.31. The pore pressure changes are plotted in figures 5.26, 5.28 and 5.30 for CIU tests. The volumetchange for CID test is shown in figure 5.32. During the CIU tests, the pore pressure is generally increasing upto the failure and then remains more or less constant or slightly decreasing.

5.4. Pore pressure parameters during undrained tests

The use of pore pressure parameters deduced from triaxial laboratory testing for interpreting field conditions had been widely developed by Skempton (1954), Bishop (1954) and Henkel (1960). In number of problems involving the undrained shear strength of soils the change in pore pressure occuring under changes in total stresses must be known. The basic idea is to relate the increase in pore pressure in a sample tested under undrained conditions with the changes in external loading and is given by

$$\Delta u = B\,[\sigma_3 + A\,(\sigma_1 - \sigma_3)]$$

where,

Δu= excess of pore pressure due to change in external loading

$\Delta\sigma_1$, $\Delta\sigma_3$= changes in principal axial and lateral total stresses respectively

A, B = pore pressure parameters

Lo (1969) proposed a more general approach of these parameters, relating the excess pore pressure with principal stress increments invarients and also with strains in the sample.

The pore pressure coefficients have proved increasingly useful during the past few years. Firstly, in providing a qualitative picture of the type of pore pressure changes which can be expected in any particular problem; and, secondly, in cases where the stress changes are known with reasonable accuracy, in providing the basis for quantitative design methods.

Figure 5.33 (a,b) shows a typical set of curves describing a CIU test on silty clay. From these data Δu versus $\Delta(\sigma_1-\sigma_3)$ plot is shown in figure 5.33 (c). The parameter A is the slope of the secant drawn from the origin and the plot of A versus strain is shown in figure 5.34 (a).

From a practical point of view, it would be interesting to work on increments of pore pressure at a given state of stress and strain rather than on cumulative values as in Skempton's interpretation. Indeed the level of strains for which there is no more build up in excess pore pressure is likely to be related to the critical void ratio of the material as defined by Schofield et al (1968). According to that theory, the stress path associated with an undrained axial compression test should end on the critical state line where the sample is sheared under constant effective stresses (no build up in pore pressure, no change in void ratio). However, in many instances, particularly for sands, the actual stress paths do not show that theoretical shape and the excess pore pressure has a tendency to decrease after passing its peak value together with an increase of the deviatoric stress. In terms of critical void ratio, it means that the undrained conditions of the test is not a constant volume condition: the stress path that is measured during the test is not to be associated with the average volume of the sample (which is constant), but with that part of the soil which is actually involved in the yielding process.

Therefore, the range of strain for which there is no pore pressure build up corresponds to an equilibrium between the part of the sample, which tends to dilate and those parts which tend to contract. Figure 5.35 shows the influence of the level of strain on the incremental build up in pore pressure for sand.

The interpretation of "tangent" pore pressure parameters has been made for sand, clayey silt and silty clay and the results are shown in figures 5.36, 5.37 and 5.38 respectively.

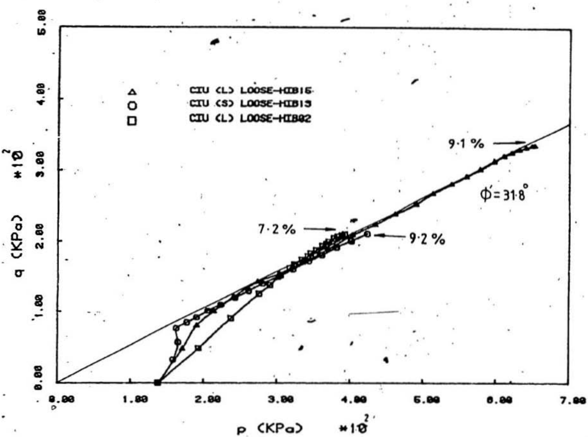


Fig.5.1: Effective stress paths for undrained triaxial compression tests on Hibernia sand.

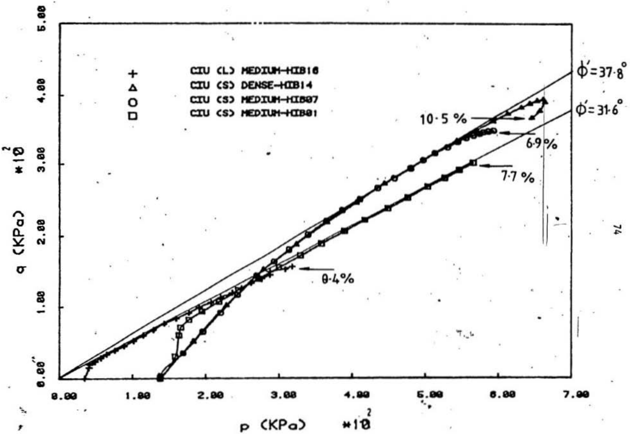


Fig.5·2: Effective stress paths for undrained triaxial compression tests on Hibernia sand.

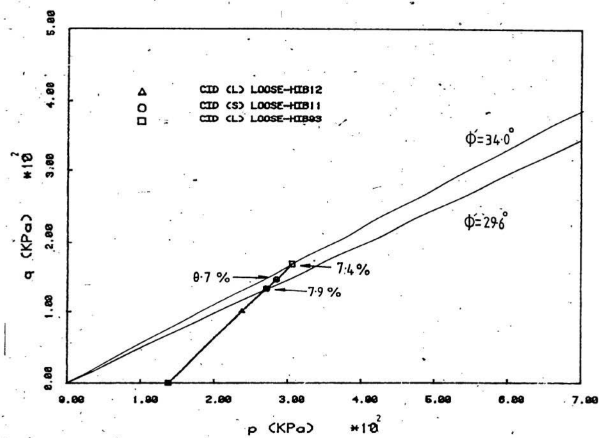


Fig.5·3: Effective stress paths for drained triaxial compression tests on Hibernia sand.

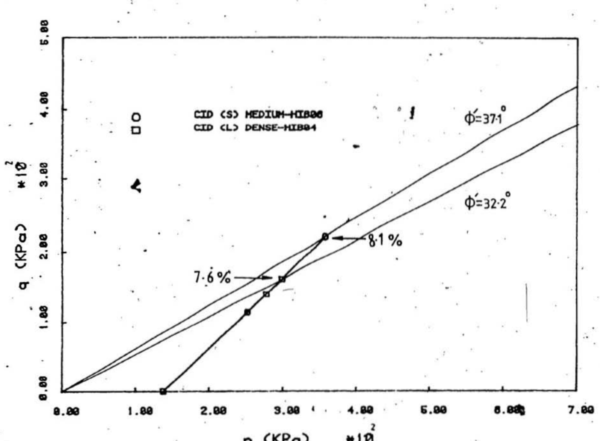


Fig.5·4: Effective stress paths for drained triaxial compression tests on Hibernia sand.

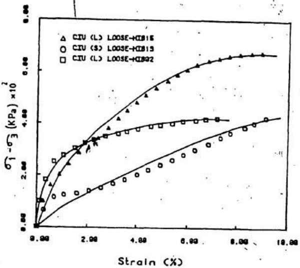


Figure 5-5: Stress-strain curves for undrained triaxial compression tests on Hibernia sand

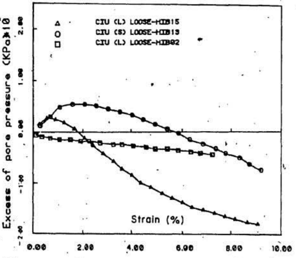


Figure 5-6: Excess pore pressure versus strain curves for undrained triaxial compression tests

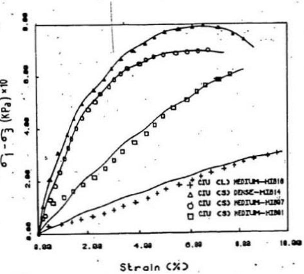


Figure 5-7: Stress-strain curves for undrained triaxial compression tests on Hibernia sand

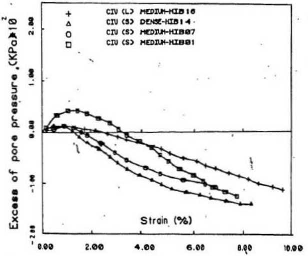


Figure 5-8: Excess pore pressure versus strain curves for undrained triaxial compression tests

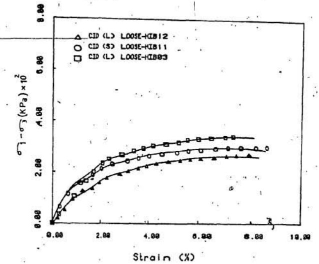


Figure 5-9: Stress-strain curves for drained triaxial compression tests on Hibernia sand

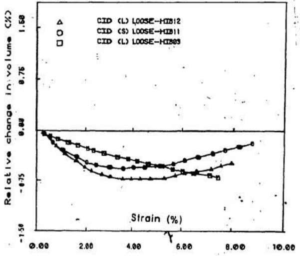


Figure 5-10: Relative change in volume versus strain curves for drained triaxial compression tests

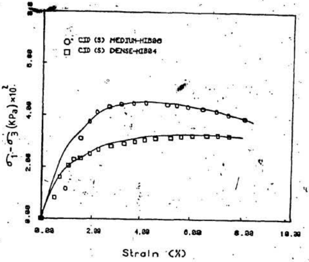


Figure 5-11: Stress-strain curves for drained triaxial compression tests on Hibernia sand

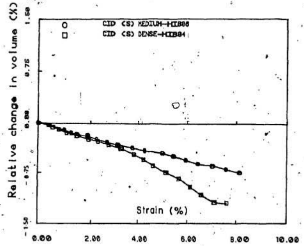


Figure 5-12: Relative change in volume versus strain curves for drained triaxial compression tests

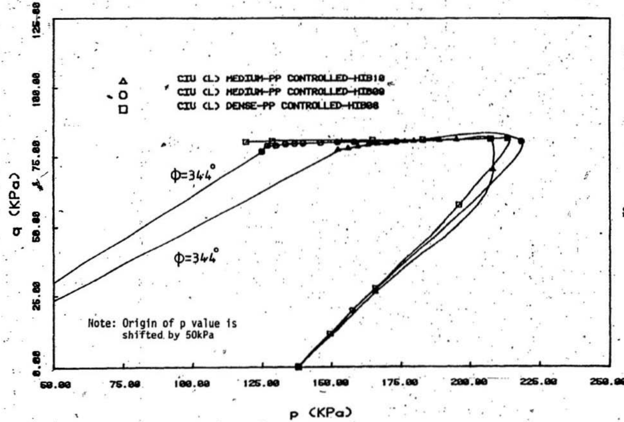


Fig.5.13: Effective stress paths for undrained triaxial compression tests on Hibernia sand.

Table 5-1: Summary of tests conditions on sand

Test No.	Test Type	Soil Condition	Height H_0 mm	Diameter D_0 mm	Dry Density kg/m^3	Void Ratio e_0	Cell Pressure kPa(psi)	Back Pressure kPa(psi)	Effective Press.$(\sigma_3')_0$ kPa (psi)
H1B-00-01	CIU(S)	Medium	156.0	72.0	1,670	0.59	345(50)	207(30)	38(20)
H1B-00-02	CIU(L)	Loose	166.0	71.4	1,630	0.63	345(50)	207(30)	38(20)
H1B-00-03	CIU(L)	Loose	166.0	71.4	1,630	0.63	345(50)	207(30)	38(20)
H1B-00-04	CIU(S)	Dense	160.0	73.2	1,720	0.54	345(50)	207(30)	38(20)
H1B-00-06	CIU(S)	Medium	150.0	72.6	1,680	0.57	345(50)	207(30)	38(20)
H1B-00-07	CIU(S)	Medium	150.0	72.9	1,690	0.56	345(50)	207(30)	38(20)
H1B-00-08	Special	Dense	165.0	72.0	1,720	0.54	345(50)	207(30)	38(20)
H1B-00-09	Special	Medium	165.0	72.0	1,700	0.56	345(50)	207(30)	38(20)
H1B-00-10	Special	Medium	165.0	73.2	1,650	0.70	345(50)	207(30)	38(20)
H1B-00-11	CIU(S)	Loose	147.0	72.3	1,590	0.67	345(50)	207(30)	38(20)
H1B-00-12	CIU(L)	Loose	165.0	70.4	1,590	0.66	345(50)	207(30)	38(20)
H1B-00-13	CIU(S)	Loose	148.0	72.4	1,600	0.65	345(50)	207(30)	38(20)
H1B-00-14	CIU(S)	Dense	148.0	74.2	1,740	0.52	345(50)	207(30)	38(20)
H1B-00-15	CIU(L)	Loose	166.0	71.4	1,640	0.62	345(50)	207(30)	38(20)
H1B-00-16	CIU(L)	Medium	166.0	70.8	1,660	0.59	241(35)	207(30)	35(05)

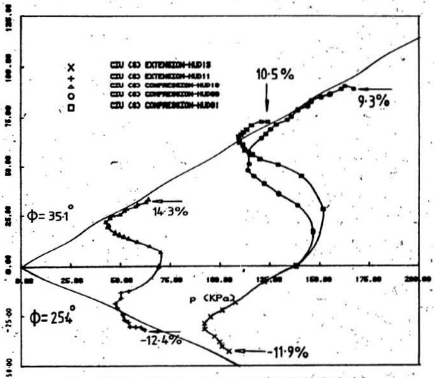


Figure 5-14r .Effective stress paths for undrained triaxial compression and extension tests on clayey silt

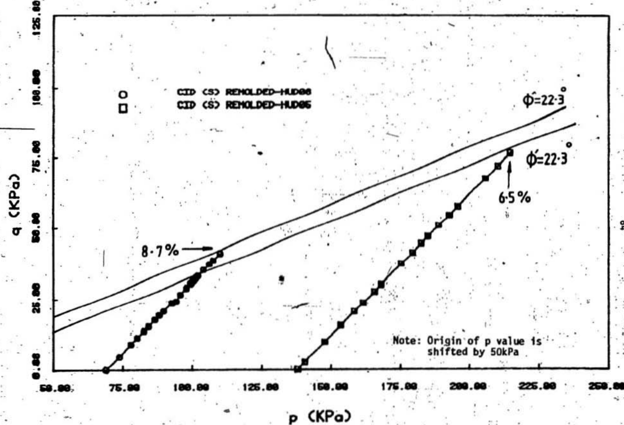


Fig.5.15: Effective stress paths for drained triaxial compression tests on clayey silt.

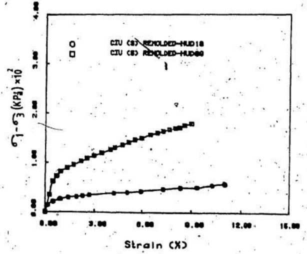


Figure 5-16: Stress-strain curves for undrained triaxial compression tests on clayey silt

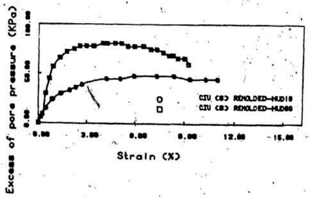


Figure 5-17: Pore pressure versus strain curves for undrained triaxial compression tests on clayey silt

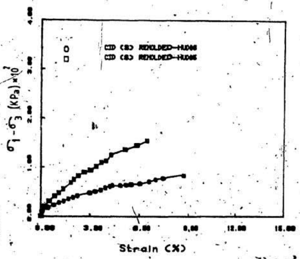


Figure 5-18: Stress-strain curves for drained triaxial compression tests on clayey silt.

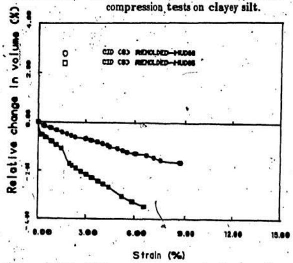


Figure 5-19: Volume change versus strain curves for drained triaxial compression tests on clayey silt

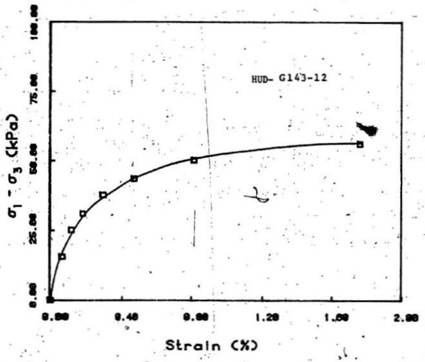


Figure 5-20: Stress-strain curve for drained triaxial extension test on clayey silt

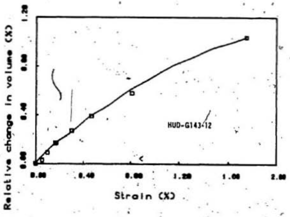


Figure 5-21: Volume change versus strain curve for drained triaxial extension test on clayey silt

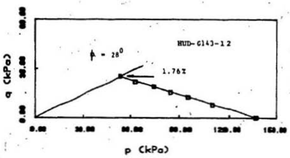


Figure 5-22: Effective stress paths for drained triaxial extension test on clayey silt

Table 5-2: Summary of tests conditions on clayey silt

Test No.	Test Type	Height H_0 mm	Diameter D_0 mm	Dry Density kg/m^3	Void Ratio e_0	Cell Pressure kPa(psi)	Back Pressure kPa(psi)	Effective Press. $(\sigma_3')_0$ kPa(psi)
HUD-G143-01	CIU(S)	145.0	72.0	1,210	1.29	345(50)	207(30)	138(20)
HUD-G143-02*	CIU(L)	166.0	70.9	1,180	1.35	345(50)	207(30)	138(20)
HUD-G143-03*	CIU(L)	166.0	71.1	1,190	1.33	345(50)	207(30)	138(20)
HUD-G143-04*	CIU(S)	160.0	72.1	1,230	1.25	345(50)	207(30)	138(20)
HUD-G143-05	CID(S)	155.0	72.2	1,200	1.35	345(50)	207(30)	138(20)
HUD-G143-06*	CID(L)	166.0	71.2	1,250	1.22	345(50)	207(30)	138(20)
HUD-G143-07*	CID(L)	166.0	71.2	1,320	1.11	345(50)	207(30)	138(20)
HUD-G143-08	CID(S)	165.0	72.6	1,280	1.17	276(40)	207(30)	69(10)
HUD-G143-09	CIU(S)	155.0	73.1	1,380	1.02	345(50)	207(30)	138(20)
HUD-G143-10	CIU(S)	150.0	72.4	1,300	1.21	276(40)	207(30)	69(10)
HUD-G143-11	CIU(S)	150.0	72.5	1,380	1.02	276(40)	207(30)	69(10)
HUD-G143-12	CID(S)	152.0	73.0	1,390	0.99	345(50)	207(30)	138(20)
HUD-G143-13	CIU(S)	157.0	72.4	1,420	0.96	345(50)	207(30)	138(20)

*The results of these tests have not been plotted due to mechanical problems during the test

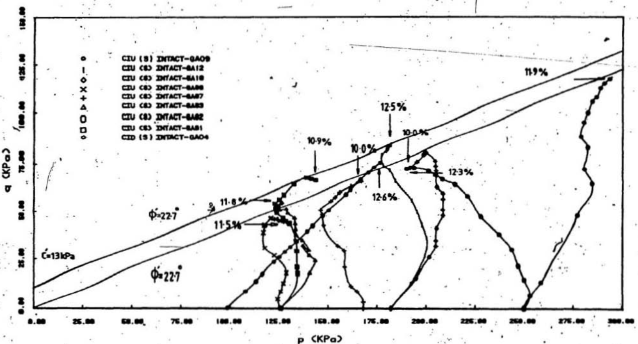


FIG.5-23: EFECTIVE STRESS PATHS FOR UNDRAINED TRIAXIAL COMPRESSION TESTS ON SILTY CLAY.

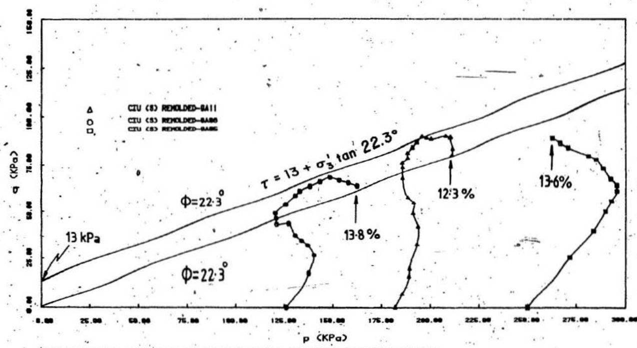


FIG 5.24: EFFECTIVE STRESS PATHS FOR UNDRAINED TRIAXIAL COMPRESSION TESTS ON SILTY CLAY.

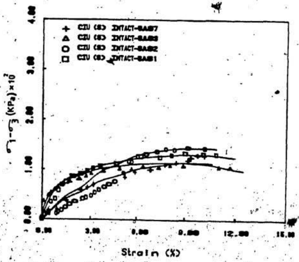


Figure 5-25: Stress-strain curves for **undrained** triaxial compression tests on **intact silty** clay

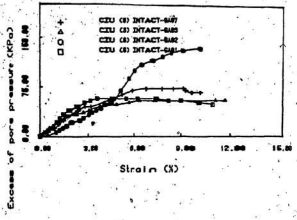


Figure 5-26: Pore **pressure** versus **strain** curves for **undrained triaxial** compression **tests** on intact silty **clay**

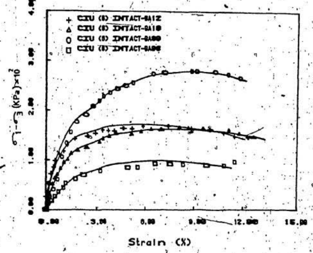


Figure 5-27: Stress-strain curves for undrained triaxial compression tests on intact silty clay

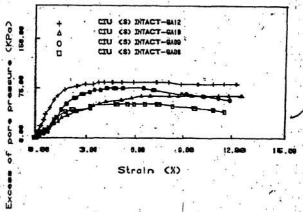


Figure 5-28: Pore pressure versus strain curves for undrained triaxial curves for intact silty clay

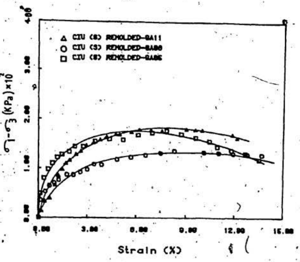


Figure 5-29: Stress-strain curves for undrained triaxial compression tests on remolded silty clay

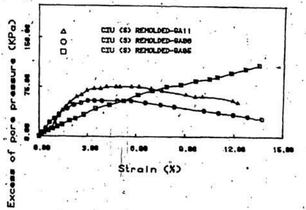


Figure 5-30: Pore pressure versus strain curves for undrained triaxial compression tests on remolded silty clay

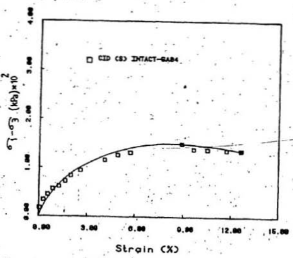


Figure 5-31: Stress-strain curve for drained triaxial compression test on intact silty clay

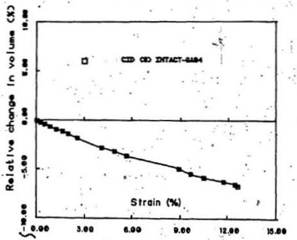


Figure 5-32: Volume change versus strain curve for drained triaxial compression test on intact silty clay

Table 5-3: Summary of tests conditions on silty clay

Test No.	Test Type	Soil Condition	Height H_0 mm	Diameter D_0 mm	Dry Density kg/m^3	Void Ratio e_0	Cell Pressure kP_a(psi)	Back Pressure kP_a(psi)	Effective Press.$(\sigma_3')_0$ kP_a(psi)
GA-BH1-PS2-01	CIU	Intact	81.3	38.0	1,499	0.923	221(32)	97(14)	124(18)
GA-BH1-PS1-02	CIU	Intact	89.2	37.2	1,356	1.086	345(50)	97(14)	124,248(18,36)
GA-BH1-PS1-03	CIU	Intact	88.6	37.9	1,363	1.076	221(32)	97(14)	124(18)
GA-BH1-PS1-04	CID	Intact	93.8	39.1	1,437	0.968	193(28)	97(14)	97(14)
GA-BH1-PS1-05	CIU	Remolded	86.1	37.2	1,534	0.844	345(50)	97(14)	124,248(18,36)
GA-BH1-PS1-06	CIU	Remolded	83.2	38.1	1,563	0.816	221(32)	97(14)	124(18)
GA-BH1-PS1-07	CIU	Intact	88.5	37.7	1,205	1.348	331(48)	179(26)	83,166(12,24)
GA-BHA1-PS2-08	CIU	Intact	84.2	37.5	1,385	1.043	221(32)	97(14)	124(18)
GA-BHA1-PS2-09	CIU	Intact	88.5	38.8	1,517	0.865	345(50)	97(14)	124,248(18,36)
GA-BHA1-PS2-10	CIU	Intact	84.8	38.0	1,451	0.951	345(50)	166(24)	179(26)
GA-BHA1-PS2-11	CIU	Remolded	88.8	37.6	1,535	0.843	345(50)	166(24)	179(26)
GA-BH4-PS2-12	CIU	Intact	87.6	37.3	1,709	0.638	345(50)	166(24)	179(26)

Note: In a few cases effective pressure applied was in two steps.

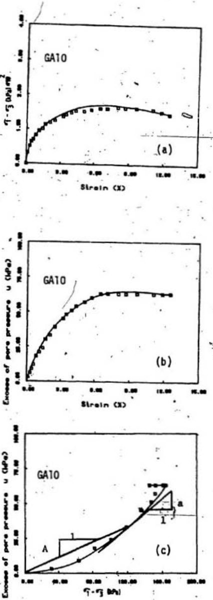
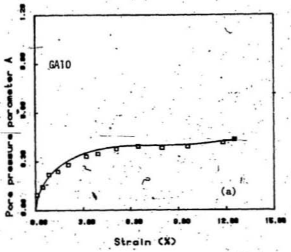


Figure 5-33: (a) stress versus strain, (b) change in pore pressure versus strain, and (c) change in pore pressure versus stress



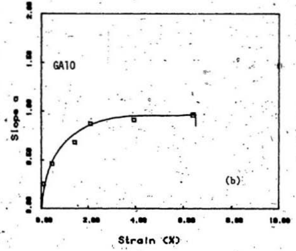


Figure 5-34: (a) Pore pressure parameter A versus strain and (b) Tangent pore pressure parameter a versus strain (refer to fig. 5.33(c))

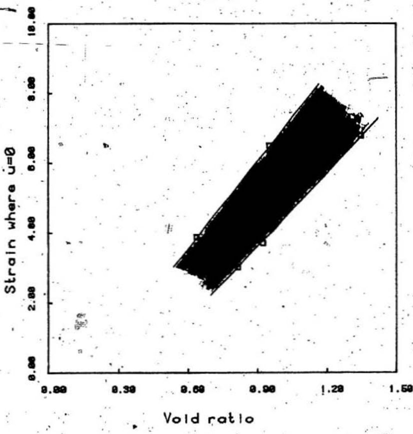


Figure 5-35: Influence of initial void ratio on the axial strain at which no more pore pressure builds up ($\dot{u}=0$) for silty clay

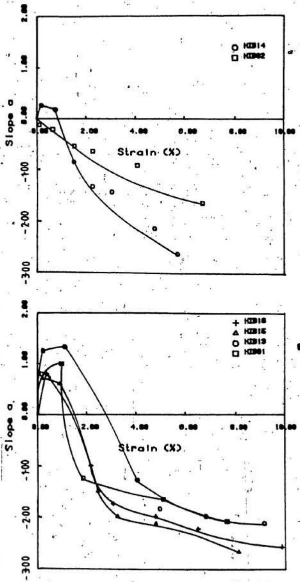


Figure 5-36: Plot of the tangent pore pressure parameter a versus strain for sand

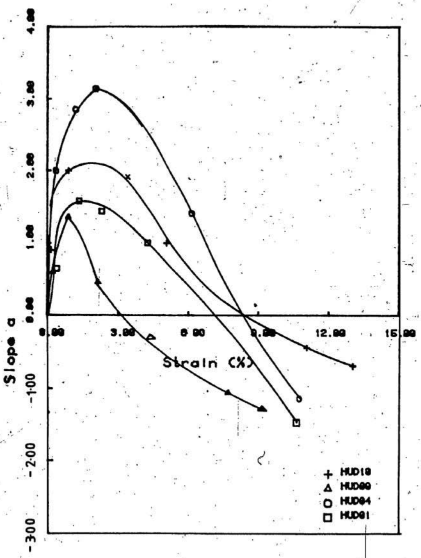
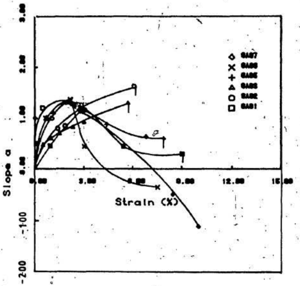


Figure 5-37: Plot of the tangent pore pressure parameter a versus strain for clayey silt



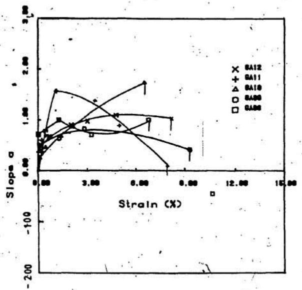


Figure 5-38: Plot of the tangent pose pressure parameter a versus strain for silty clay

Chapter 6
MONOTONIC TRIAXIAL TESTS: DISCUSSIONS

6.1. Effect of the Saturation and Pore fluid Properties

For a few tests previously presented, the value of the pore pressure parameter B during the saturation process was not close to unity due to the presence of the air bubbles in the circuitry or in the sample. It has been suggested that for these tests, the pore pressure response was not as good as under fully saturated samples. That underestimation of the excess of the pore pressure can lead to an error in estimation of the angle of internal friction ϕ'.

Before applying back pressure, all the sand samples were saturated by allowing the water to flow through the samples. Back pressure was applied in steps, in an increments of 35 kPa (5 psi). Sufficient time was given between the two increments to allow the water to flow through the sample. In almost all sand samples at 207 kPa (30 psi) B $= \Delta u/\Delta\sigma_3$ value was close to 1. Figure 6.1 shows the degree of saturation with back pressure for two sand samples. A typical example of the improvement of the measured B value during the back pressure saturation is shown in figure 6.2 for sand, clayey silt and silty clay samples.

In this research, it was not tried to keep the back pressure as low as possible, using additional techniques, like CO_2 ciculation (for sand) or vacuum. It was thought that high back pressures were more realistic for simulating neutral in situ pressures.

Another problem that may be of importance in laboratory testing as compared with in situ testing is the pore fluid properties: the use of the fresh water at 18^{0}C is certainly far from the actual in situ conditions (for example at Hibernia, temperature of 0^{0}C to 2^{0}C must be expected with salt water). These parameters have not been studied here. From a qualitative point of view they should not affect the results for monotonic loading.

6.2. Influence of End Effects

In case of isotropically consolidated undrained (CIU) tests on loose sand the effect of the end conditions on the stress-strain curve are not prominent, as they are for CIU tests on medium and dense sand or CID tests on loose, medium and dense sand. The stress-strain curves for enlarged lubricated ends are below those for standard ends.

In case of CIU tests on loose, medium and dense sand, it has been noticed that maximum excess pore pressure is higher for standard ends then for lubricated ends. For CID tests on loose, medium and dense sand, the change in volume is greater for enlarged lubricated ends than for standard ends. The influence of ends effects has not been studied for the other materials.

From the tests on sand it appears that the use of enlarged lubricated ends tends to smooth the pore pressure or volume changes. This response was expected since the distribution of the strain in the sample is more uniform in this case.

Neverthless, end effects appear to have much less importance for effective failure parameters than for the parameters that depend on the distribution of the strain in the sample.

6.3. Effect of the Sample Disturbance

The effect of the sample disturbance is always difficult to highlight. The "intact" state of the soil is something that can only be defined by extrapolation from more or less disturbed specimen. It also becomes more difficult to assess the degree of disturbance as the soil is more silty.

In this study the specimen had encountered a long disturbance history: the sampling was carried out under difficult conditions, the storage was long and the laboratory manipulations were far from perfection.

The consolidation test performed on a sample taken at a 18.3 m depth where the vertical in situ stress can be estimated at 140 kPa, show a preconsolidation pressure estimated at 60 kPa, which is an indication of disturbance of the sample.

On the other hand the stress path associated with triaxial testing on the "intact" specimens are very close to those on the "remolded" specimens. The shape of the stress paths also look like a typical shape for silty soils or remolded silty clays.

The cohesion intercept that may be taken into account for both "intact" and "remolded" samples can therefore be interpreted either as a curvature of the failure envelop, or as a remainder of overconsolidation or a slight dilatancy effect.

6.4. Other Factors

Some researchers mentioned that the size of the sample has an effect on the sample strength. The smaller the height to diameter ratio, the higher the strength. In this research all the samples were of the same size for one particular type of the soil, therefore nothing can be said regarding the effect of sample size on soil strength. Nevertheless, the diameter of the tested samples for sand or clayey silt was large enough to avoid the scattering that might happen from the non-homogeneous distribution of the unit weight in the sample.

Sample preparation control is a complicated problem to address, since a reliable measurement of sample density is difficult to obtain. It is possible but beyond the capabilities of an ordinary laboratory.

The influence of the rate of shearing was beyond the scope of this study, which placed emphasis on actual laboratory problems rather than on properties of investigated soils.

Only loose and dense samples were tried for Sand. Density of samples slightly higher than loose density were classified as medium. Density difference between loose and medium samples were therefore small, and the difference in angle of internal friction ϕ' is not significant. Further, density depends on the precise measurement of the weight of the soil, height of the sample and diameter of the sample. Any inaccuracies will therefore have a influence on the value of the density.

The failure envelops from undrained extension (fig. 5.14) and drained extension (fig. 5.22) tests were found to be consistent with compression tests.

In figure 5-24, for the range of consolidation pressures used in the tests, the failure envelop may be approximated by the equation:

$$\tau = 13 + \sigma_3' \tan 22.3^0$$

where σ_3' and τ are in kPa.

6.5. Comparison with Other Results

Comparing the sand from Hibernia area, Grand Banks, used in this research with that reported by Meyerhof (1979), it is concluded that offshore sands are frequently of fine and uniform grain size with uniformity coefficients of less than 3. Hibernia sand also fits into the category of offshore sands repoted by Meyerhof (1979).

Comparing the tests results mentioned earlier in Chapter 2, with the present tests results on Hibernia Sand. From the isotropically consolidated undrained (CIU) triaxial tests, angle of internal friction $\phi' = 31.6$ and 37.8^0 for loose and dense sand samples respectively. Similarly, from isotropically consolidated drained (CID) triaxial tests, angle of internal friction $\phi' = 29.6$ and 37.1^0 for loose and dense sand samples respectively. Some of the values reported by the other researchers are higher than found in the present study. The reason for that could be that shear box tests give higher values as compared to triaxial tests. But the values found in the present research fit in well into the average range of values given by the other researchers.

A comparison of the results of the isotropically consolidated undrained (CIU) triaxial tests on undisturbed and remolded samples of silty clay from West Coast of Newfoundland with Silva et al. (1984), given in table 2.1, showed that the average angle of internal friction ϕ' found for undisturbed and remolded samples were 22.7^0 and 22.3^0 respectively. The angle of internal friction ϕ' from undisturbed and remolded samples was found to be close. It was concluded that the samples had undergone a lot of disturbance in sampling. The ϕ' values found in this research are lower than the values given by Silva et al. (1984) in table 2.1. The difference in tests results could be due to shape of the particles and minerology.

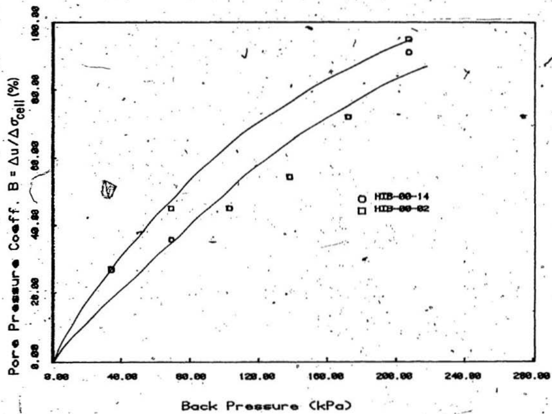


Figure 6.1 Improvement of Saturation with Back Pressure.

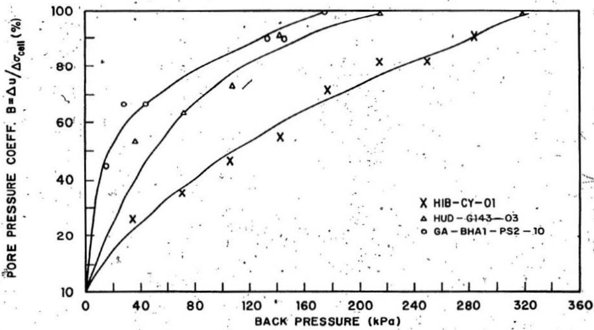


Figure 6-2: Improvement of Saturation with Back Pressure

Chapter 7

CYCLIC TRIAXIAL TESTS ON SAND AND CLAYEY SILT

7.1. Material

The Sand and Clayey Silt tested under cyclic loading were the same as for under monotonic loading and their description is given in Chapter 3.

7.2. Testing Equipment

The load press used for cyclic loading tests is an Instron machine, model 1123. The range of loading goes up to 2,500 kilograms. The machine performs the primary functions of load weighing, crosshead drive and control, and recorder drive and control.

The load is applied by means of the crosshead drive system and controlled by load weighing transducer located under the bottom plate.

The crosshead is of high stiffness and rigidly guided to prevent side loading during the test. The motion of the crosshead is obtained by rotating leadscrews. Beside the manual control of the unit, an electronic console provides the following features:

1. Recording of the axial load applied to the testing device and various conviniences (lower and upper set points for example).

2. Display of the displacement of the crosshead and possibility to limit it to given values.

3. An extra strain gauge output for a displacement transducer or any other strain gauge transducer.

All recordings are made on a strip chart recorder with an adjustible speed.

The same triaxial cell has been used for the cyclic loading as for the monotonic loading tests. The load applied externally and the load which has been actually transfered into the sample as measured by the internal load cell are on the chart. Because of the friction between the ram and the top cap, the load applied externally is not the same as actual load transfered to the sample. A linear displacement transducer was fixed to the crosshead of the machine and recorded on a separate two channel plotter. The internal pore pressure transducer was also connected to the same plotter. The plotter used was a J J instruments CR 600 recorder. The linear displacement transducer used was Schaevitz SCM series. Internal pressure transducer is a Viatran relative pressure gauge (range 0-100 psi). The experimental set up is shown in figure 7-1.

All cyclic triaxial tests were performed at a frequency of around 1 Hz. Four variables were monitored continously during each test: i.e. external and internal axial load, axial deformation, and pore pressure.

Besides the plotting of the results, electronic readouts designed at Memorial University allow a direct control of each variable by the operator. Calibration of the transducers were provided before each test by means of a "shunt" resistor associated to each readout. This ensures a high reliability of the results and no drift in time.

In order to assess the results, all measuring systems are duplicated so that cross calibration is always possible. For example all transducers are recorded and can be read on a digital readout. This allows a great flexibility in changing the sensitivity of the recording devices during tests without intermediate calibration. All equipments were carefully calibrated prior to testing.

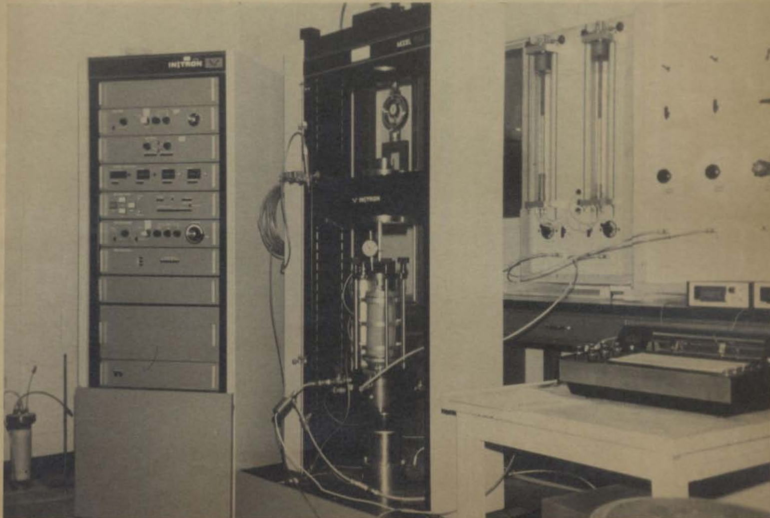


Figure 7.1: Experimental set up for cyclic loading tests

7.3. Types of Tests

Isotropically consolidated drained and undrained tests were performed on sand and undrained tests were performed on clayey silt under cyclic loading.

7.3.1. Drained tests on sand

Two tests, one on loose sand and the other one on dense sand were performed under drained conditions. The samples were deformed up to a given strain and then sheared under cyclic loading. It led to the determination of deformation moduli for these densities and level of strain. This may be of interest for numerical program analysis.

A typical stress-strain curve is shown in figure 7-2 for dense sand.

7.3.2. Undrained tests on sand

Cyclic loading of sands under undrained conditions leads to progressive build up in pore pressure. These excess positive pore pressures reduce the effective stress to sufficiently low levels so that an effective stress failure condition, termed liquifaction, will develop with the large strains and low strength.

Twenty three samples were tested under undrained conditions. All the samples were saturated under a backpressure of 210 kPa (30 psi) and isotropically consolidated under an effective pressure of 69 or 138 kPa (10 or 20 psi). Then the drainage valves were closed and the axial load increased up to a pre-determined value, the load is cycled between zero and this value at a frequency of approximately 1 Hertz.

A typical stress path associated with this type of test is shown in figure 7-3. The test can be described in terms of load ratio. The load ratio is defined as the ratio of the deviatoric cyclic stress $(\sigma_1 - \sigma_3)$ to the initial effective consolidation stress $(\sigma_3')_0$. In some publications, half of this value is taken into account. Depending on the results of the test, the number of cycles necessary to reach the failure may be associated with the load ratio. The results of such an interpretation are shown in figure 7.4.

It must be emphasized that there is no reversal of the deviatoric stress during the cycle, resulting in particular deformations and failure mode. If the load ratio is not sufficient to generate significant excess of pore pressure, a continous deformation take place corresponding to the relative rearrangement of the grains, like in the monotonic tests. If the load ratio is sufficient, pore pressure gradually builds up in the sample, leading to the failure by reaching the Mohr-Coulomb envelop.

After the cyclic loading, an undrained compression test was performed in order to study the effect of cyclic loading on the monotonic undrained shear strength.

7.3.3. Undrained tests on clayey silt

Seventeen samples were tested under undrained conditions. All the samples were backpressure saturated under a pressure of 210 kPa (30 psi) and isotropically consolidated under effective pressure of 69 or 138 kPa (10 or 20 psi). Backpressure saturation takes six to eight hours and consolidation process takes between sixteen to twenty hours. Once the consolidation was over, the sample is sheared in undrained conditions under cyclic loading. The load was cycled between zero and the pre-determined load at a frequency of approximately 1 Hertz.

After failure by cycling, or after a sufficient number of cycles, the samples were sheared like in monotonic compression tests (CIU) to check the consistency with standard tests.

Figure 7.5 shows a typical stress-strain relationship obtained in CU-cyclic-monotonic triaxial compression tests. Figure 7.6 the shows effective stress path for cyclic and monotonic loading. Figures 7.7 & 7.8 show a relationship between load ratio and variation of pore pressure during cyclic loading tests versus number of cycles respectively.

7.4. Sample Preparation

The procedure followed for mounting a sand sample was exactly the same as for monotonic loading and complete details are given in Chapter 4. The procedure for mounting the clayey silt sample is almost the same as for the remolded silty clay sample except the mould chosen for the clayey silt sample had a diameter of approximately 38.0 mm. and no trimming had been done. Other details are given in Chapter 4 under silty clay.

7.5. Results and Interpretations

7.5.1. Sand

Tables 7.1 provides the main results for sand. Figure 7.1 shows the number of cycles required for failure as a function of load ratio. As the load ratio increases, the number of cycles to failure decreases. For a ratio greater than 0.38, the failure occures at the first cycle. For a load ratio less than 0.18, the build up in pore pressure is not sufficient to cause failure. Table 7.2 shows the angle of internal friction obtained at failure by cycling, as well as the final angle after shearing under undrained monotonic conditions. The values of the angle of internal friction for monotonic tests range from 30.9^0 to 34.7^0, which is consistent with the standard results (30.4^0 to 33.0^0). The angle of internal friction at failure by cycling is about the same as the standard one for loose samples. Medium and dense samples show a higher value of the angle for cyclic loading than for the monotonic loading. It may be interpreted as the influence of the initial compaction.

7.5.2. Clayey Silt

Table 7.3 gives the main results for clayey silt. Figure 7.7 shows the number of cycles required for failure as a function of load ratio. Again as the number of cycles decreases the load ratio increases. For load ratio 0.4, and effective consolidation pressure of 138 kPa, the failure occurs within fifty cycles. For load ratio 0.6, and effective pressure 69 kPa, the failure occurs within fifty cycles. For load ratios of less than 0.3, and 0.45 for samples consolidated under effective pressure of 138 and 69 kPa respectively, the pore pressure developed is not sufficient to cause failure. Excess pore pressures induced by cyclic loading versus strain are shown in figure 7.8.

In Figure 7.7 even after normalizing, there is apparently a difference in the number of cycles to failure for two different confining pressures. This difference could be due to insufficient saturation or intrinsic properties of soil. It should be further investigated in the future.

7.6. Discussion and Recommendation

An instrumented triaxial cell has been used to saturate and isotropically consolidate the sand and clayey silt sample and then perform cyclic load tests on them. The various facilities include a number of features that allow accurate data to be obtained. Particular emphasis was given to measurement of applied stresses, pore pressures and deformations.

The equipment has been successfully used to obtain data on the effects of drained and undrained cyclic loading tests. The triaxial cell used has a built-in pressure transducer to measure the pore pressure developed and an internal load cell to measure the actual deviatoric stress.

In this investigation, the shape of the stress waves was not controlled but was close to a sine shape. A more sophisticated system would be needed for various types of waves.

A PDP/11 computer was used for computation and drawing standard diagrams. In future, a computer should also be used for automatic data logging and analysis, not only for monotonic, but also for cyclic loads. It should also be used for adjusting piston force, cell pressure and pore pressure during consolidation and shearing.

Table 7-1: Results of cyclic tests on sand

Sample No.	Date of Test ('84)	Initial Dry Unit Weight (kN/m^3)	Saturation Back Pressure (kPa)	Effective Initial$(\sigma_3')_0$ (kPa)	Relative Index I_D(%)	Number of Cycles	Load Ratio $\Delta(\sigma_1' - \sigma_3')/(2\sigma_3')_0$	Comments
HIB-CY-00A	Apr. 18	16.43	310	138	80	---	---	drained test
HIB-CY-00B	Aug. 25	15.93	0	138	38	---	---	drained test
HIB-CY-01	Apr. 25	15.81	345	69	27	130	0.28	undrained test
HIB-CY-02	Apr. 28	16.17	345	69	58	190	0.30	undrained test
HIB-CY-03	May 1	16.07	345	69	50	no failure	---	continuous
HIB-CY-04	May 2	16.12	345	69	54	no failure	---	deformation of the
HIB-CY-05	May 3	16.02	345	69	45	no failure	---	sample.
HIB-CY-06	May 11	15.82	345	138	27	140	0.32	undrained test
HIB-CY-07	May 29	15.88	345	138	33	720	0.33	lack of saturation
HIB-CY-08	May 30	15.70	275	138	17	750	0.27	lack of saturation
HIB-CY-09	May 31	15.63	275	138	11	250	0.27	undrained test
HIB-CY-10	Jun. 1	15.91	275	138	41	40	0.36	undrained test
HIB-CY-11	Jun. 1	15.68	275	138	15	20	0.34	undrained test
HIB-CY-12	Jun. 2	15.51	275	138	0	50	0.36	undrained test
HIB-CY-13	Jun. 2	15.72	275	138	19	failed under the first load		(0.36)
HIB-CY-14	Jun. 3	16.04	275	138	47	15	0.3	undrained test
HIB-CY-15	Jun. 3	15.56	275	138	5	90	0.25	undrained test
HIB-CY-16	Jun. 4	15.71	275	138	18	150	0.23	undrained test
HIB-CY-17	Jun. 4	15.75	275	138	21	280	0.23	undrained test
HIB-CY-18	Jun. 5	15.95	275	138	40	260	0.23	undrained test
HIB-CY-19	Jun. 5	16.04	275	138	47	450	0.23	undrained test
HIB-CY-20	Jun. 6	16.02	275	138	45	320	0.23	undrained test
HIB-CY-21	Jun. 7	16.07	275	138	50	390	0.23	undrained test
HIB-CY-22	Jun. 7	16.17	275	138	58	500	0.23	undrained test
HIB-CY-24	Jun. 8	15.91	275	138	35	420	0.32	undrained test

Table 7.2: Angle of Internal Friction for Sample Submitted to Cyclic Tests

Sample No.	Density Index I_D (%)	Angle of Internal Friction (end of cycling) (degree)	Angle of Internal Friction (large deformations) (degree)
HIB-CY-04	54	29.7	33.1
HIB-CY-05	45	32.8	30.0
HIB-CY-06	27	27.5	27.5
HIB-CY-07	44	29.0	27.5
HIB-CY-08	17	31.8	33.0
HIB-CY-09	11	29.1	30.9
HIB-CY-10	41	44.5	34.0
HIB-CY-14	47	33.7	31.3
HIB-CY-15	5	33.3	34.7
HIB-CY-16	18	34.0	33.9
HIB-CY-17	21	31.7	32.4
HIB-CY-18	40	31.5	32.5
HIB-CY-19	47	32.9	31.6
HIB-CY-20	45	37.0	33.9
HIB-CY-21	50	37.0	33.6
HIB-CY-22	58	39.8	33.6
HIB-CY-23	35	35.0	32.3

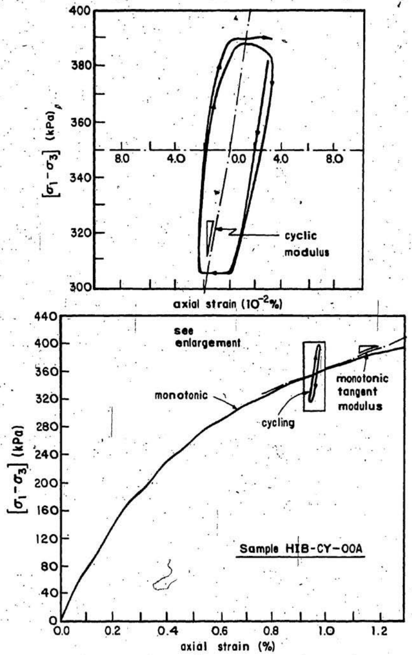


Fig. 7.2 Axial stress-strain curve for monotonic and cyclic loading under drained conditions

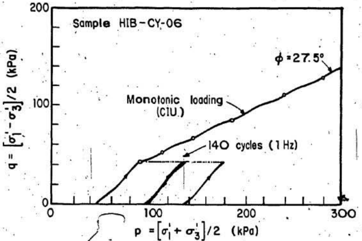


Fig. 7.3: Stress path associated with cyclic loading on Hibernia sands.

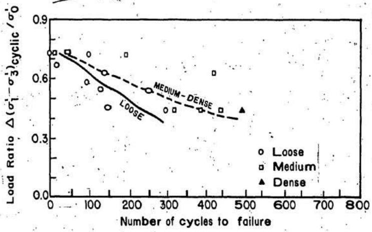


Fig. 7.4: Relationship between Load Ratio and number of cycles to failure.

Table 7-3: Results of cyclic loading on clayey silt

Sample No.	Test Date ('84)	Dry Unit Weight (kN/m^3)	Water Content (%)	Back Pressure Saturation (kPa)	Effective Pressure (kPa) $(\sigma_3')_0$	Load Ratio $\Delta(\sigma_1' - \sigma_3')/(2\sigma_3')_0$	Number of Cycles N	Comments
HUD-G143-CY-01	Oct. 5	13.19	36.1	207	138	0.25	no failure	almost no effect
HUD-G143-CY-02	Oct. 7	14.17	31.8	207	138	0.25	no failure	almost no effect
HUD-G143-CY-03	Oct. 9	14.62	28.3	207	138	0.30	330	continuous deformation
HUD-G143-CY-04	Oct. 12	14.21	29.4	207	138	0.40	60	failed
HUD-G143-CY-07	Oct. 21	13.88	31.6	207	138	0.40	58	failed
HUD-G143-CY-08	Oct. 22	14.46	29.5	207	138	0.35	160	continuous deformation
HUD-G143-CY-09	Oct. 23	14.06	31.7	207	138	0.40	50	failed
HUD-G143-CY-10	Oct. 24	15.00	27.6	207	138	0.35	180	continuous deformation
HUD-G143-CY-11	Oct. 25	14.33	32.1	207	69	0.30	no failure	no effect
HUD-G143-CY-12	Oct. 26	14.39	30.5	207	69	0.35	no failure	no effect
HUD-G143-CY-13	Oct. 27	14.71	28.5	207	69	0.40	no failure	almost no effect
HUD-G143-CY-14	Oct. 28	14.26	29.6	207	69	0.45	175	continuous deformation
HUD-G143-CY-15	Oct. 29	14.53	30.4	207	69	0.50	145	continuous deformation
HUD-G143-CY-16	Oct. 30	14.23	30.0	207	69	0.60	40	failed
HUD-G143-CY-17	Oct. 31	14.57	29.7	207	69	0.65	100	failed

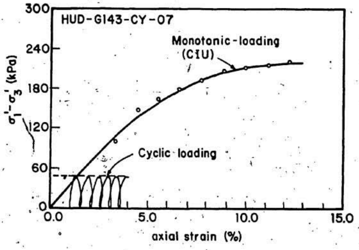


Fig. 7.5 Axial stress-strain curve for cyclic and monotonic loading under undrained conditions.

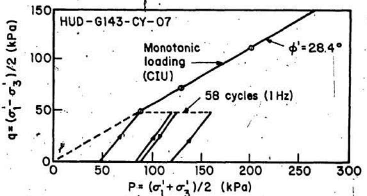


Fig. 7.6 Effective stress path for cyclic loading test on clayey silt

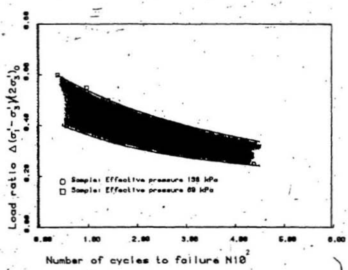


Figure 7-7: Load ratio versus number of cycles

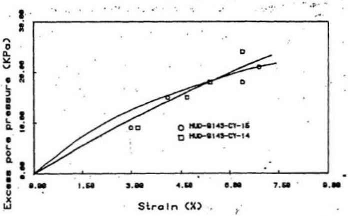


Figure 7-8: Excess pore pressure versus strain

Chapter 8
CONCLUSIONS

The main objective of this research was to test various types of soils under monotonic and cyclic triaxial tests. Tests were performed on three types of soils from the Newfoundland areas. From the results of monotonic and cyclic triaxial tests the following conclusions may be drawn.

The three soils tested in this study have shown to behave very similarly to the other soils of the same type, despite their offshore origin (Hibernia sand and Placentia Bay clayey silt). The disturbance of the silty clay samples appears very clearly in the results.

Lubricated enlarged ends ensure a more homogenous distribution of the strain inside the sample and should be used if a high volume change tendency is to be expected. The effect of lubricated ends is to smooth the pore pressure response of the soil during the undrained tests.

The failure criteria was found to be well defined for the three soils tested during this study, especially for large deformations. The disturbance of the samples in some cases, or the effect of the artificial compaction for remolded samples, lead to a certain scatering in the results at small strain levels.

Some special tests were performed on sand and clayey silt. It was found that the shear strength properties were close to the results obtained from the standard tests.

The pore pressure parameters have been reinterpreted in terms of tangent values, showing that the strain where no more pore pressure is generated in undrained tests may be related to the initial void ratio.

The cyclic tests have shown that the liquefaction, defined as state of large permanent deformation, is associated with the same failure criteria as for monotonic tests. The build up in pore pressure requires also a minimum value of the deviatoric stress ratio. Below that threshold no failure occurs. When the cycled stress level is high, pore pressure will accumulate to the point that a yield failure occurs. At lower level of cycled stress, the pore pressure and strain increase and stabilize to a steady state, where a large number of loading cycles do not produce any further significant change in strain or effective stress.

In view of the unavoidable sample disturbance of offshore soils, especially from great depths, laboratory test results must be considered as a mean of getting an approximation of in situ properties of the seabed. With the development of such reduced scale models as wave tanks and iceberg towing tanks, the need for reliable parameters of soils will increase.

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Appendix A

Sample Preparation

The following preparation methods have been used in the soil laboratory at M.U.N. to prepare satisfactory samples:

Sand

1. Apply the silicon grease around the rubber ring of the base of the cell and fasten the base plate to the base of the cell.

2. Prepare specimen in a split mold that is clamped together and fitted around the base plate. Apply grease lightly over the two halves of the mold to give a good seal for vacuum.

3. Put a cylindrical sheet of plastic inside the mold. The diameter of the sheet is the same as the internal diameter of the mold. Use a water soluble tape to hold the sheet in position.

4. Put two O-rings on each side of the mold. The rubber membrane is then stretched inside the mold and fold the top and bottom portion of the membrane over the mold. The mold has two holes in the wall, which allows vacuum to be applied to hold the membrane against the wall. Vacuum is then applied.

5. If the sand is to be sheared in a saturated state, place a water soaked porous stone with a filter paper on top of the base plate. Apply grease lightly around the base plate, which gives a good seal between base plate and the membrane.

6. Fix the mold around the base plate with the vacuum.

7. Weigh the container of dry sand. Pour the sand into the mold such that the density obtained is close to minimum dry density for loose samples. The use of a funnel is a good way to get an uniform distribution of the density.

8. For compacted sand samples, weigh the container of the dry sand, moist the sand with a small amount of water and weigh the container again. Pour the sand into the mold and compact it in order to get a density close to the maximum density.

9. Leave a portion of the mold empty at the top for filter paper, porous stone and top cap if standard ends are used, otherwise fill the mold upto the top if enlarged lubricated ends are used.

10. Place a porous stone with filter paper and then the top cap above the sand. Coat the outer rim of the top cap with silicone grease to provide a leak proof seal at the top.

11. Roll the membrane off the mold and onto the top and bottom cap and seal it with two O-rings on each side.

12. Attach the flexible tubing from the top cap to the bottom of the cell for top drainage.

13. Stop the vacuum and remove the specimen mold. The specimen will remain straight because of the plastic sheet around the sample. Observe the membrane for obvious holes and leakes. If any found, the sample must be remade using a new membrane.

14. Obtain height measurements at two positions and use the average value for the initial specimen height H_0. Take three diameter readings at the top, midheight and at the base using a vernier caliper. Take these measurements to the nearest 0.1 cm. Compute the average diameter of the specimen and subtract the thickness of the membrane and the plastic sheet for the calculation of the area A_0. Weigh the remaining sand (after drying it if moist) in order to compute the dry unit weight.

15. Apply silicone grease on both sides of the plexiglass cell cylinder and place it on the cell base after removing any soil grains from the seal. Place all the four tension rods around the cylinder and fix up the top cap and screw it tightly around cylinder. Push the ram until it comes in slight contact with the top cap of the sample. This will keep the sample straight.

16. Fill the triaxial cell with water. Once it is filled close the water outlet at the top of the cell. Apply a pressure of approximately 5 kPa from the water tap, the top and bottom drainage being open to atmosphere.

17. Place the cell in the compression machine and just barely make the load contact of the ram and the loading ring which is attached to the compression machine.

18. Attach a deformation dial (reading to 0.01 mm.) to the loading piston so that the deformation of the sample can be obtained.

19. Apply a little cell pressure to the cell from the pressure system to keep the sample in cylindrical shape.

Clayey Silt

All the samples tested for clayey silt were remolded as explained earlier. The procedure followed for mounting a sample is as follows:

6. Steps 1 to 6 are the same as for sand.

7. Remold enough soil for one sample and weigh the container plus soil.

8. Pour some soil into the mold and compact it. Repeat the procedure till the mold is filled. Try to obtain a unit weight close to the estimated in situ unit weight.

9. Take a sample of the remolded soil for water content determination. While compacting sample, care must be taken not to destroy the membrane.

10. Steps 9 to 20 are same as for sand.

Silty Clay

The procedure followed to mount silty clay "intact" and "remolded" samples are as follows 38 mm. (1.5 in.) sample.

1. Apply the silicone grease around the rubber ring of the base of the cell and fasten the 38.0 mm. base plate to the base of the cell.

2. For an intact sample remove the sample from its wax coating and mount it on a trimming apparatus. The sample is brought to a diameter of approximately 38 mm. It is then cut to a length close to 90 mm.

3. For a remolded sample, remold enough soil at the required water content. Take a bigger mold and clamped together its two halves. Compact the soil into it. Unclamped the mold, take out sample and put it in the trimming apparatus. Care should be taken to keep the same unit weight for all the remolded samples.

4. Obtain two measurements at two positions and use the average value for the initial specimen height H_0. Take three diameter readings; at the top, midheight, and at the base using a vernier caliper. Take these measurements to the nearest 0.1 cm. Take the average of three diameters and calculate the initial area A_0, volume V_0 and the unit weight.

5. Weigh the trimmed sample. Place a water soaked porous stone with filter paper on top of the base plate. Place the sample on top of it. Place another water soaked porous stone and filter paper at the top.

6. Take a hollow cylinder of 100 mm. in length and 50 mm. internal diameter,

with a hole in the middle and a small plastic tubing connected so that a vacuum can be applied to stretch the membrane.

7. Put two O-rings on each side of the cylinder. Two membranes are used to surrond the sample and avoid any leakage through the membranes. The membranes are stretched inside the cylinder. The top and bottom portion of the membrane are folded over the cylinder.

8. Put one membrane at a time and apply the vacuum through mouth to hold the membrane against the wall. Place the membrane over the sample and release the vacuum. Membrane will take the shape of the sample.

9. Apply the grease over the bottom half of the top cap. Top cap is in two halves. Put one O-ring at each end to seal the sample. Stretch another membrane and put it on top of the first one and seal it with another pair of O-rings at the top and bottom of the sample.

10. Place top part of the top cap and screw it with the bottom half. Attach the flexible tubing from top to the bottom of the cell for top drainage.

11. All the steps from 15 to 20 for sand are then the same except for step 20 where the cell and back pressures are applied in a multiples of 13.8 kPa (2 psi).

Appendix B

Sample Saturation, Consolidation and Shear

1. Saturate the soil sample by circulating water from the tank located at a height of approximately 1.0 m. from the cell. Wait for some time and let the water flow through the sample. Reverse the the process twice or thrice till no visible air bubbles are left in the circuitry.

2. Make all the necessary connections. Apply the the back pressure to saturate the sample further. Check pore pressure parametr B in order to make sure that the sample is fully saturated. Back pressure applied should always be less than the cell pressure.

3. After applying the back pressure upto the desired level, the cell pressure is raised so that the effective over all pressure required for consolidation is obtained. This can be done in one or more steps.

4. As soon as no more change in sample volume is observed, for the final effective consolidation pressure, set the compression machine to the desired strain rate.

5. Turn on the the compression machine and take simultaneously load, deformation, pore pressure, and /or volume change readings. The value of the cell pressure must not vary $\pm$0.5 psi during the test for standard CIU or CID.

6. Turn off the compression machine when load becomes constant or approximately 15 percent deformation is reached. Take off the load, release the cell and back pressures.

7. Dismount the triaxial cell and clean the whole apparatus for the next test.