LABORATORY INVESTIGATION OF SOIL FROM NEW PROPOSED LOCATION FOR TEXAS LOW-LEVEL RADIOACTIVE WASTE DISPOSAL FACILITY

A REPORT FOR
TEXAS LOW-LEVEL RADIOACTIVE WASTE DISPOSAL AUTHORITY
AUSTIN, TEXAS

THE UNIVERSITY OF TEXAS
AT EL PASO

VOLUME I: PRESENTATION OF RESULTS

RESEARCH REPORT 90-2

CENTER FOR GEOTECHNICAL & HIGHWAY MATERIALS RESEARCH
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LABORATORY INVESTIGATION OF SOIL FROM
NEW PROPOSED LOCATION FOR TEXAS
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VOLUME I: PRESENTATION OF RESULTS

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AUGUST 1990
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CHAPTER ONE

INTRODUCTION

1.1 SCOPE OF WORK

Cyclic triaxial tests were performed on two soils from the trench at the proposed location of the Texas Low-Level Waste Disposal facility in Hudspeth County, Texas. These two soils were identified as a clay, and a silt. Six clay specimens were tested at in-situ water content. All six specimens were trimmed from block samples, which were hand carved from the wall of the trench. Attempts were made to obtain intact specimens of silt as well. However, the silt was weak, and intact specimens could not be carved from the block samples. Therefore, specimens were reconstituted at in-situ water content and bulk density, and used in the cyclic triaxial tests. Twenty four reconstituted silt specimens were tested for dynamic properties. Of these twenty four specimens, twelve were tested at in-situ water content, and the other twelve were saturated and consolidated before testing.

Cyclic triaxial tests were performed to evaluate the effects of confining pressure and strain level on Young's modulus and damping ratio of the material tested. In addition, for the reconstituted silt specimens, the effects of the degree of saturation on the modulus and the damping ratio were also studied. The liquefaction potential of the reconstituted silt specimens was addressed in terms of a "threshold strain". When a specimen is tested under strain-controlled undrained cyclic triaxial tests, the threshold strain is defined as the minimum strain level, below which there is no build-up of pore pressure.

Static triaxial tests were also performed on reconstituted silt specimens. Tests were performed to evaluate the effects of confining pressure, rate of applied axial strain, and the degree of saturation on the stress-strain behavior of the soil. Out of a total of eighteen specimens tested, four were saturated and consolidated before testing.

This project was funded by the Texas Low-Level Radioactive Waste Disposal Authority (TLLRWDA). Mr. Ruben Alvarado of the TLLRWDA was the project manager. His technical advice and support is highly appreciated. All tests reported herein were performed utilizing the facilities of the University of Texas at El Paso. However, the quality control/quality assurance of the project was managed by the Bureau of Economic Geology (BEG) of the University of Texas at Austin.
1.2 ORGANIZATION

The report consists of two volumes. In Volume I of this report, the results of tests are presented and analyzed. Volume II consists of the raw and reduced data collected for each test.

This section provides a brief overview of the organization of Volume I. The theory of the cyclic triaxial test, and a brief description of the equipment used, are included in Chapter Two. A review of some of the existing literature is presented in Chapter Three. In Chapter Four, the tests performed, and their nomenclature are defined. The results of the static tests are included in Chapter Five, while results of the dynamic tests are presented in Chapters Six and Seven. The conclusions of this study, and recommendations for future studies are presented in Chapter Eight.
CHAPTER TWO

CYCLIC TRIAXIAL TESTS

2.1 INTRODUCTION

Cyclic triaxial tests were performed to evaluate the dynamic properties of two soils from the proposed location of the Texas low-level radioactive waste disposal facility. These tests were performed at relatively high strain levels (greater than 0.01 percent), on specimens of clay and silt (as classified in Chapter Five). For each soil, cyclic triaxial tests were performed on specimens at in-situ water content, as well as on specimens that were saturated and consolidated before testing. These tests were conducted in the strain controlled mode. The theory of the cyclic triaxial tests and the equipment used during this program are described and discussed in this Chapter.

2.2 DAMPING AND MODULUS

The modulus is a measure of the dynamic stiffness of the material. In general terms, the modulus value of a sample is obtained from its stress-strain curve. The definition of modulus is depicted in Figure 2.1. The secant modulus is defined as the ratio of stress and strain at a given strain level. The initial tangent modulus (also known as the maximum modulus or the elastic modulus) is simply the tangent to the curve at very low strain levels. Typically, the variation in modulus with strain is demonstrated in a normalized fashion. On this normalized curve, the abscissa is the logarithm of the strain and the ordinate is the ratio of the secant modulus to initial tangent modulus. Such a curve is shown in Figure 2.2. Three distinct regions are apparent in this curve. In the initial portion, with a constant normalized modulus of about 1, the modulus is independent of strain. In the second region, the modulus decreases drastically as the strain level increases. In the third portion, the modulus more or less tends to a constant value again. Practically speaking, the initial portion, with a constant modulus, can be accurately measured using seismic methods or the resonant column device. The resonant column device is also quite effective in defining most of the second part of the curve. However, for strain levels above 0.01 percent, cyclic triaxial tests are more appropriate for determining the modulus.

Damping is a measure of the energy absorbed by the material. In geotechnical earthquake engineering, two types of damping are usually considered, viz. hysteretic damping and viscous damping. Typically, cyclic triaxial tests yield hysteretic damping; while with the resonant column device, viscous damping is obtained.
Figure 2.1  Variation in Secant Moduli with Strain (after Stokoe et al, 1988)

Figure 2.2  Generalized Variation in Young's Modulus with Axial Strain (after Stokoe et al, 1988)
For a linear viscoelastic material subjected to a sinusoidal strain, the resulting stress is also sinusoidal in nature. Stress and strain cycle at the same frequency. However, there exists a phase difference of $\delta$, between the two parameters. If the imposed strain, $\varepsilon(t)$, is expressed as:

$$
\varepsilon(t) = \varepsilon_{\text{max}} \cos(\omega t),
$$

the resulting stress, $\sigma(t)$, is given by:

$$
\sigma(t) = \sigma_{\text{max}} \cos(\omega t - \delta),
$$

where $\varepsilon_{\text{max}}$ and $\sigma_{\text{max}}$ are the maximum strain and stress amplitudes, respectively, and $\omega$ ($= 2\pi f$) is the angular frequency. A plot of stress versus strain for a given cycle represents a hysteresis loop. A schematic hysteresis loop is shown in Figure 2.3.

The energy dissipation in a single cycle of deformation, $\Delta W$, is given by:

$$
\Delta W = \frac{2\pi}{\omega} \int_0^T \frac{\sigma d\varepsilon}{dt} dt
$$

Upon integration, this yields:

$$
\Delta W = \pi \varepsilon_{\text{max}} \sigma_{\text{max}} \sin \delta
$$

Physically, $\Delta W$ corresponds to the area of the hysteresis loop (see Figure 2.3). The maximum strain energy, $W$, stored in a perfectly elastic material at the same amplitude is given by:

$$
W = 0.5 \varepsilon_{\text{max}} \sigma_{\text{max}}.
$$

The specific loss is defined as $\Delta W/W$ for a linear viscoelastic material and is given by:

$$
\Delta W/W = 2 \pi \sin \delta.
$$

Damping ratio, $D$, is defined as:

$$
D = 1/(4\pi) \times \Delta W/W = 1/2 \sin \delta.
$$

In cyclic triaxial tests, the modulus for a particular cycle is obtained by plotting the stress versus the strain and by joining the points of maximum strain and maximum stress with the points of minimum strain and minimum stress, as shown in Figure 2.3. The damping for a particular cycle could also be determined from the hysteresis loop for that cycle. The damping is directly proportional to the area of the hysteresis loop.
Figure 2.3 Schematic of a Hysteresis Loop from Cyclic Triaxial Tests

\[ A_1 = \text{Area (abcdefa)} \]
\[ A_t = \text{Area (oag)} \]
Mathematically,

\[ D = A_i / (4 \pi A_i) \]  \hspace{1cm} (2.8)

where \( A_i \) and \( A \) are defined in Figure 2.3

Cyclic triaxial tests typically yield Young's modulus at a given axial strain amplitude. Axial and shear strains, and Young's and shear moduli are interrelated through Poisson's ratio. Shear strain, \( \gamma \), and the axial strain, \( \varepsilon \), are related by:

\[ \gamma = \varepsilon / (1 + \nu). \]  \hspace{1cm} (2.9)

Similarly, Shear Modulus, \( G \), and the Young's Modulus, \( E \), can be related by:

\[ G = E / 2(1 + \nu) \]  \hspace{1cm} (2.10)

where \( \nu \) is Poisson's ratio.

2.3 CYCLIC TRIAXIAL TESTS

In cyclic triaxial tests, a specimen of known dimensions is placed between load platens. A confining pressure is applied around the specimen, to simulate the earth pressure that an element of soil experiences at a certain depth below the ground surface. A cyclic deviatoric stress is applied about this confining pressure.

Before the cyclic load is applied, the soil sample is subjected to an isotropic state of stress. The Mohr circle representing this state of stress in the element, is a point (see Figure 2.4). The figure also shows the states of stress and the corresponding Mohr circles during cyclic loading. During the first half cycle of loading, the vertical stress on the element is larger than the lateral stress. During the next half of the load cycle, the vertical stress is smaller than the lateral stress. Therefore, the shear stresses acting on a plane passing through the element, and inclined at 45°, cycle between the values of \( + \Delta \sigma_v / 2 \) and \( - \Delta \sigma_v / 2 \) (where \( \Delta \sigma_v / 2 \) is the vertical deviatoric stress). Presumably, this pattern of cyclic loading reproduces the shear stress reversals developed in a soil element during an earthquake.

A more accurate reproduction of the state of stress would be obtained by cycling the confining pressure at the same frequency as the vertical cyclic load, but 180° out of phase, as shown in Figure 2.5. The variation in the magnitude of the confining pressure would be one half the magnitude of the vertical deviatoric stress. However, it is rather difficult to cycle the confining pressure, and hence this is not usually done. Moreover, when dealing with a saturated sample, cycling the confining pressure results in changes
Figure 2.4  Mohr Circle Total Stress Representation for a Cyclic Triaxial Strength Test for an Isotropically Consolidated Specimen (from Silver, 1976)
Figure 2.5 Stress Conditions for Triaxial Test on a Saturated Sand under Simulated Earthquake Loading Conditions (from Seed and Lee, 1966)
in pore pressure only, and not in the effective stresses. Consequently, there would be no change in the deformation characteristics of the specimen. Instead of actually cycling the confining pressure, a simulation can be achieved by applying a correction factor to the values of the pore pressure.

During tests in this program, even for the saturated samples, only a total stress analysis was performed. For tests on samples at natural water content the drainage lines were kept open to avoid any pore pressure build-up. For the saturated samples, the drainage lines were kept closed, and the build-up of pore pressure was carefully monitored.

2.4 TYPES OF CYCLIC TRIAXIAL TESTS

Cyclic triaxial tests can be performed either under strain-controlled, or under stress-controlled conditions. In the strain-controlled tests, a pre-determined strain is applied to the specimen, and the load response is monitored. In the stress controlled cyclic triaxial tests, a pre-determined load is imposed on the specimen, and the deformation of the specimen is observed.

The modulus and damping ratio of the material are determined utilizing strain-controlled tests. About four cycles are applied to the specimen, and the load and displacement records are collected.

Liquefaction tests are performed under load controlled conditions. The specimen is saturated and consolidated at a given effective confining pressure. A cyclic deviatoric stress is applied under undrained conditions. The number of cycles of load required to cause liquefaction is recorded. The test is terminated when the specimen undergoes liquefaction, or at the completion of a pre-determined number of load cycles. In these tests, liquefaction is defined as the condition when the effective confining pressure reduces to zero.

One main drawback of the load controlled liquefaction tests is the extreme sensitivity of the test results to sample disturbance, and the unacceptable scatter of test data when testing reconstituted specimens (Dobry et al, 1988). Simultaneously, there is a significant body of evidence that shows the pore pressure buildup in a specimen in strain-controlled cyclic triaxial tests, where the amplitude of strain remains constant throughout the test, is relatively insensitive to the soil fabric and the void ratio of the specimen. This may suggest that the cyclic strain (Dobry et al, 1988), rather the cyclic stress, is a more fundamental parameter governing the seismic behavior of the soil. Based on previous investigations (Dobry et al, 1988), it can be concluded that there is a "threshold strain", below which there is no buildup of pore pressure irrespective of the
number of cycles applied to the specimen.

2.5 EQUIPMENT

An MTS closed-loop servo-valve system was used to perform the cyclic triaxial tests. The system consists of several interacting units as shown in Figure 2.6, and consists of three main components: (1) a load unit, (2) a controller, and (3) a hydraulic power supply. Each component is briefly described below.

2.5.1. Load Unit. The load unit, as shown in Figure 2.7, consists of two smooth vertical columns that join two stiff structural members; i.e. a movable crosshead and a fixed platen. The crosshead is vertically adjustable to accommodate specimens of varying lengths. A vertical load can be applied to the specimen using a hydraulic actuator. The actuator is mounted on the crosshead.

The load unit is provided with a triaxial cell, so that soil samples can be tested with an all round confining pressure. The triaxial cell basically consists of stainless steel upper and lower load platens housed between upper and lower support plates. A push rod extends through the upper support plate and is attached to the upper load platen to apply an axial load to the specimen. A shut-off valve manifold at the base of the triaxial cell provides control of specimen saturation, pore pressure measurement, and removal of entrapped air in the platen. The upper platen is connected to the shut-off valve manifold with a flexible tubing.

A close-up of the triaxial cell is shown in Figure 2.8. The load platens and the support plates are enclosed by a translucent cylindrical shell which acts as a sealed confining chamber and allows observation of the specimen. Access clamps on the upper support plate restrain the shell during testing. Specimen mounting and removal are accomplished by unlocking the access clamps and raising the shell.

The triaxial cell push-rod is rigidly mounted to the actuator via a load cell. The position of the push-rod is monitored by a linear variable differential transformer (LVDT). The triaxial cell is also equipped with two transducers; one to measure cell pressure, and the other to monitor pore pressure within the sample.

The confining pressure can be applied in two different ways. Either a hydraulic actuator mounted at the base of the load frame can be utilized; or confining pressure can be applied pneumatically to the specimen through the pressure ports in the upper support plate. Generally, the first option is utilized for short term testing at low confining pressures. The second option is used for saturation of the sample and long-term tests at high confining pressures.
Figure 2.7 Components of Load Unit
Figure 2.8  Triaxial Testing Cell
The upper load platen consists of two steel parts as shown in Figure 2.9. The upper part remains attached to the push-rod at all times, while the second part rests on the sample. The two parts are held together by means of a vacuum applied between them. The vacuum is applied through a port in the shut-off valve manifold at the base of the triaxial cell (see Figure 2.8).

An additional service manifold is attached to the load frame to accommodate reservoirs for the confining fluid and the pore fluid. Compressed air (obtained from an external air compressor), applied on the water in the pore fluid reservoir, causes water to flow into the specimen under pressure. A valve and a pressure gage are provided to control the pore pressure.

2.5.2. Controller. The MicroConsole controls and monitors the operation of the load unit. It also provides chassis connections for functional plug-in modules. Connectors are provided on the rear panel for transducers, servovalves, hydraulic service manifolds, etc.

A picture of the controller is included in Figure 2.10. Three plug-in modules are provided: an AC Controller, a DC Controller, and an Auxiliary span-control. The Auxiliary span-control was not used during these tests, and will not be discussed herein. Either the AC Controller or the DC Controller can be used to operate the actuator mounted at the top of the load frame. The AC Controller and the DC Controller control the movement of and the load applied by the actuator rod, respectively. Depending on the selected active controller, the test can be run in strain- or stress- controlled mode.

An expansion MicroConsole panel houses another set of three plug-in modules. Of these, two control the hydraulic actuator at the bottom of the frame, which in turn controls the confining pressure. The third module is used to monitor the pore pressure.

The MicroConsole is also equipped with two arbitrary waveform generators manufactured by Wavetek Inc. These two function-generators which are used to control the motion of the two hydraulic actuators, have several built-in standard functions, such as sine, cosine, square, and halver sine. The amplitude, frequency, and the DC-offset of these waves can be set to any arbitrary values. In addition, these standard waveforms can be readily edited to produce more complex functions. These input functions, as well as the output from any of the transducers mounted on the load frame or the load cell, can be monitored by an oscilloscope mounted above the waveform generators.

2.5.3. Hydraulic Power Supply. The hydraulic power supply provides the high pressure fluid required for the operation of the system. The high pressure fluid is applied to one side of the actuator piston, causing it to move. A servovalve controls the movement of the actuator, by opening or closing in response to the Controller. The valve can be opened in either of two directions, allowing the high pressure fluid to flow into the
Figure 2.9  Components of Upper Load Platen of Triaxial Cell
Figure 2.10 Components of Controller

A - Oscilloscope
B - Waveform Generators
C - Microconsole
D - Expansion Microconsole
cylinder on either side of the piston. This causes movement of the piston in either of two directions.
CHAPTER THREE

BEHAVIOR OF SOILS UNDER DYNAMIC LOADS

3.1 INTRODUCTION

The stress-strain behavior of sands under dynamic loads has been the subject of several investigations in the past. A vast majority of these investigations have involved laboratory tests to explain the phenomenon of liquefaction, and study the effects of various parameters affecting liquefaction.

Seed (1976) defines liquefaction as the condition when the soil undergoes continuous deformation under a low constant residual stress, due to buildup and maintenance of pore water pressure. The rise in pore pressure may be due to static or cyclic loads. Initial liquefaction is the condition when the pore pressure first becomes equal to the applied confining pressure. This definition does not involve any implications regarding the deformation of the soil. Initial liquefaction with limited strain potential/cyclic mobility/cyclic liquefaction is the condition when initial liquefaction develops, but the deformation of the soil is limited due to remaining resistance of the soil to deformation. The deformations may also be limited due to dilation of the specimen, and consequent reduction in pore pressure, and subsequent stabilization of the soil under the applied loads.

Seed (1976) also explains why most liquefaction theories are based on the assumption that the sand is under undrained conditions. In layers close to the ground surface, the pore pressure that may buildup as a direct result of ground vibrations, will dissipate so rapidly that a liquefied condition could not possibly develop. In fact, it is more likely that liquefaction of the near surface layers, is caused by the upward flow of water from one or more of the underlying layers that have liquefied. Observations of actual pore pressure variation during liquefaction caused by an earthquake also support this hypothesis. It has been observed, that liquefaction is initiated in one or more of the underlying layers during the earthquake. The near surface layers liquify after a time lag, the duration of which depends on the depth from the ground surface.

It is therefore apparent that liquefaction cannot occur during an earthquake unless one or more layers of soil in the profile liquifies. Furthermore, if the soil permits quick drainage, there is a continuous dissipation of pore water pressure, and liquefaction cannot develop. In practice therefore, every soil in the profile could be tested for susceptibility to liquefaction. If each soil in the profile exhibits resistance to liquefaction, then it would be reasonable to assume that vibrations will not induce pore pressure build-up and liquefaction in the soil.
Though conservative, this approach is reasonable considering the state-of-knowledge of the rate of pore pressure buildup and dissipation, both during and after an earthquake.

3.2 DAMAGE DUE TO LIQUEFACTION

Failure due to liquefaction could have several manifestations, depending on the geometry of the ground, the permeability characteristics of the soil deposit, and the homogeneity of the soil (National Academy Press, 1985). Some of the common modes of failure due to liquefaction are discussed below.

1. Sand boils: Sand boils alone do not cause failure of structures, as they do not produce ground deformations. However, sand boils sometimes cause economic loss due to sediment deposition and flooding. If liquefaction occurs in a zone at some depth below the ground surface, the excess pore pressure dissipates by the upward flow of water. When the zone of liquefaction is close to the ground surface, the sand-laden pressurized water can break through the surface, and form a water spout. If the overlying soil is cohesionless and relatively permeable, there is a general settlement of the ground surface, with the uniformity of settlement depending on the homogeneity of the soil. If the soil is cohesive, continued ground shaking may cause crack formation in the soil, and the venting of the water occurs through these cracks.

2. Flow Failures: Flow failures are the most catastrophic ground failures caused by liquefaction. The flow may consist of completely liquefied soil, or partially intact blocks riding on a layer of liquefied soil. Many such flows have been observed in coastal areas. The earthquake of 1964, produced submarine flows, that destroyed large sections of the port facilities in Valdez, Alaska (National Academy Press, 1985). Flow failures on land are usually called debris flows, and are often more devastating than submarine flows.

3. Lateral Flows: Lateral flows occur on gentle slopes (between 0.3 to 3.0 degrees). Lateral flows occur due to displacement of large superficial blocks of soil, as a result of liquefaction of a subsurface layer. Movement occurs as a response to gravitational and inertial forces produced by the earthquake. Lateral flows typically do not cause catastrophic damage. But, they can be severely disruptive to foundations, pipelines and other structures located on or across the failure.
(4) Loss of Bearing Capacity: When a soil deposit liquifies, it loses its bearing capacity. Structures supported on the soil may settle and tilt without undergoing structural damage. During the 1964 earthquake, several spectacular bearing failures were observed in Niigata, Japan. Many of the buildings were repositioned, underpinned and reused (National Academy Press, 1985).

3.3 LABORATORY TESTING

Ladd (1976) studied the effect of specimen preparation on the liquefaction of sand. These investigations revealed that the moisture content of the soil at the time of specimen preparation had a significant effect on the cyclic strength of soil. Specimens prepared using moist soil were between 50 and 100 percent stronger than those specimens that were prepared using oven dry soil. The cyclic strength of the specimens was relatively insensitive to the method of compaction used.

Ladd (1976) attributes this change in cyclic strength to the variation in the fabric of the sand. The sand fabric parameters that could affect the cyclic strength of the sand are: (1) differences in grain and inter-particle contact orientations, (2) variations of void ratio within the individual specimen, and (3) segregation of particles.

Mullilis et al (1977) studied some of the variations in testing techniques that could affect the liquefaction characteristics of sand. Monterey sand 0 was used in the test program. The effects of specimen preparation, B-values, and the waveform used during testing, on the liquefaction characteristics of the sand were evaluated.

Specimens were prepared using dry rodding, moist rodding, and moist tamping. The investigators found that specimens prepared by moist rodding were significantly stronger than the prepared by dry rodding. The cyclic stress ratio required to cause initial liquefaction showed an increase of 58% at 50 cycles, and 38% at 30 cycles.

Specimens were tested using sinusoidal, triangular, and square waveforms. Specimens tested with a triangular waveform were about 13% stronger than those tested with a square waveform. Specimens tested with a sinusoidal waveform were found to be 15% stronger than those tested with triangular waveforms, and about 30% stronger than those tested with a square waveform.

The results of this program also indicated that variation in the B values between 0.91 to 0.98, did not significantly affect the liquefaction characteristics. Specimens prepared using the method of undercompaction (Ladd, 1978), were about 10% stronger than those specimens prepared not using this method. With reference to these two mentioned, the investigators note that the results may be applicable only when the type
of sand, the relative density, and the initial confining pressure are the same as those used in this test program.

Ladd (1978) suggests one method for preparing specimens of uniform density. The procedure incorporates a tamping method of compacting the soil in layers. This method of specimen recognizes the fact that the compacting effort for any layer, will densify the layers of soil below. To compensate for this densification, each layer is compacted to give a density that is lower than the final density by a certain pre-determined amount, called "undercompaction". The layer at the bottom has the maximum value for undercompaction. The amount of undercompaction is reduced for each successive layer. Thus, when the final layer is compacted, the entire specimen has a uniform density.

Ladd (1978) describes several methods to determine an appropriate value of undercompaction for the first layer. Excessive necking or bulging of one or more layers of the specimen during cyclic loading, non-uniform vertical strains during unconsolidated-undrained loading, a honeycomb structure at either the top or bottom of the specimen, are all indications of an inappropriate value of undercompaction for the first layer.

Stage testing is a very popular technique, and is widely used in laboratory investigations of the dynamic properties of soil. In this method, the specimen is cycled at the lowest possible vertical strain under undrained conditions. Any excess pore water pressure generated is released. The drainage is closed, and the specimen is cycled at the next higher strain level. Silver et al (1975) investigated the method of stage testing, to determine if the values of modulus and damping ratio obtained from stage tests were different from tests where a fresh specimen was used for each strain increment. A uniform angular quartz with a specific gravity of 2.65, and an air dry water content of less than 1% was used during this program.

Silver et al (1975) conclude that for dry specimens subjected to less than 25 cycles of loading, results obtained from stage tests could be used to evaluate the dynamic behavior of fresh specimens. In terms of effective confining pressure, the modulus values for fresh saturated and stage tested saturated specimens, were equivalent as long as the volume changes were small. The damping ratios for saturated stage tested specimens, and saturated fresh specimens, are more or less equal. However, the values for damping ratios for saturated stage tested specimens were significantly higher than the values for dry stage tested specimens.
CHAPTER FOUR

OVERVIEW OF TESTS PERFORMED

4.1 INTRODUCTION

This Chapter provides a brief overview of the various types of tests performed during the course of this program. Also included are tables that identify the specimen used in each test, the type of test, and the conditions under which the test was performed.

The procedure to determine the cementation of the soil is presented in Section 4.7.

4.2 TYPES OF SOILS TESTED

Two soils were tested in this test program. They were a clay, and a silt with very low cementation. The geotechnical classification of these soils, and a discussion on some of their static properties, are presented in Chapter Five. Both soils were obtained from an exploratory trench excavated at the site.

4.3 DESCRIPTION OF THE SITE

The site is located in the Trans-Pecos region of Texas, in the Basin and Range physiographic region, about 60 miles Southeast of El Paso, and about 12 miles northeast of Fort Hancock in Hudspeth County. The region has a subtropical arid climate, with hot summers and mild winters. Rainfall events are locally intense but short lived, and the mean annual precipitation over the years 1951-1980 was 9 inches (Kreitler, 1986). The site is situated in an area of low relief that is gently sloping downward to the southwest. The vegetation at the site is predominantly low scrub brush and grasses. The site is approximately 4200 feet above sea level.

The trench was approximately 120 feet long, and 100 feet wide. The trench was excavated to a depth of about 20 feet. The first two feet of soil was brown silty sand, with low cementation. The cementation was weak, and the soil could be penetrated with the foot. The next three feet was also a silty sand, but with a lower cementation than the first layer. Below this was a 5 feet thick layer of silty clay. Clay specimens tested herein were taken from this layer. This layer of clay overlies a 5 feet thick layer of lightly cemented silt. The cementation of this sand was variable, with very strong cementation in localized areas. Generally, the cementation decreased with depth in this layer. All silt specimens
were reconstituted from this material. A hard clay layer comprised the two feet between approximately 18 and 20 feet.

4.4 SPECIMEN IDENTIFICATIONS

Cyclic triaxial and static triaxial tests were performed on a clay, and a silt. Intact specimens of clay were tested. These specimens were hand carved from block samples obtained from the walls of the trench. Specimens of silt were reconstituted at in-situ water content and density, before being tested.

The Test Number, the type of test, and the confining pressure for each of the static and dynamic triaxial tests, are presented in Tables 4.1 through 4.3. Tables 4.1 through 4.2 also include whether an intact specimen or a reconstituted specimen was used during the test, and if the test was performed on a specimen at in-situ water content, or if the specimen was saturated and consolidated before testing. Table 4.3, which contains the test numbers for reconstituted silt specimens used in static triaxial tests, the strain rate at which the specimen was sheared is also included.

4.5 TYPES OF SPECIMENS

4.5.1. Intact Specimens. The specimens of clay used in the various tests, were hand carved out of block samples obtained from the walls of the exploratory trench. A 2 cubic foot block of soil was carved out from the wall of the trench, using a geological hammer, and hand saw. This block was waxed, and transported to the Geotechnical laboratory at the University of Texas at El Paso. Sand paper was used to trim out cylindrical specimens from these blocks. The specimens are referred to as "intact specimens" in this report.

4.5.2. Reconstituted Specimens. Intact specimens of silt could not be carved out, as the cementation was extremely weak, and the soil crumbled easily. Therefore, specimens of silt were reconstituted on the base pedestal of the triaxial cell. Care was taken to ensure that the specimen was at in-situ water content and density.

4.6 TYPES OF TESTS

4.6.1. Repeatability Tests. These tests were performed on three clay specimens, to verify that the results of the cyclic triaxial tests were repeatable. Each specimen was tested at confining pressure of 5, 10, 20, and 40 psi, at low strain levels. The modulus at a particular confining pressure were compared. If the variation in the values was within acceptable limits, the results were considered to be repeatable.
<table>
<thead>
<tr>
<th>TYPE OF TEST</th>
<th>SAMPLE CONDITION (INTACT OR RECONSTITUTED)</th>
<th>WATER CONTENT (IN-SITU OR SATURATED)</th>
<th>CONFINING PRESSURE (Psi)</th>
<th>TEST NO.</th>
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<td>REPEATABILITY</td>
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<td>IN-SITU</td>
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<td></td>
<td>-do-</td>
<td>-do-</td>
<td>5,10,20,40</td>
<td>RDC02</td>
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<td>-do-</td>
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<td>RDC03</td>
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<td>IN-SITU</td>
<td>10</td>
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<td>40</td>
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Table 4.2  Silt Specimens for Cyclic Triaxial Tests

<table>
<thead>
<tr>
<th>TYPE OF TEST</th>
<th>CONDITION (INACT OR RECONSTITUTED)</th>
<th>WATER CONTENT (IN-SITU OR SATURATED)</th>
<th>CONFINING PRESSURE (Psi)</th>
<th>TEST NO.</th>
</tr>
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</table>

*RECONSTITUTED
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<th>WATER CONTENT (IN-SITU OR SATURATED)</th>
<th>CONFINING PRESSURE (Psi)</th>
<th>STRAIN RATE (PERCENT/SEC)</th>
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<td></td>
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<td>RTX10</td>
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4.6.2. High Strain Tests. For the two types of soils tested, some specimens were tested at high strain levels (of the order of 1 percent). The aim of these tests was to study the modulus and damping ratio of the soils at high strain levels. The specimen was first tested at the lowest strain level. The strain was then increased in increments. After each increment, the specimen was tested at the lowest strain level, to check for degradation. The term "degradation" is defined later in Chapter Six. The high strain tests were performed on specimens at in-situ water content, as well on specimens that were saturated and consolidated before testing.

The clay specimens were tested at in-situ water content only since a sufficient number of specimens were not available. Moreover, pore pressure build-up would not be a problem for a clay.

4.6.3. Threshold Strain Tests. Tests were performed to determine the threshold strain for the reconstituted silt specimens. The aim of this series of tests was to observe the build-up of pore pressure in the specimen during strain-controlled cyclic triaxial tests. The specimen was first saturated and consolidated at the effective confining pressure at which the test was to be performed. Fifty cycles of the lowest strain level were imposed on the specimen under undrained conditions. At the end of fifty cycles, the outlet valves to the specimen were opened, and the excess pore pressure built up was released. The outlet valves were closed, and the specimen was then tested at the next higher strain level.

4.6.4. Static Triaxial Tests. Reconstituted silt specimens were tested in axial compression, at confining pressures of 5, 10, 20, and 40 psi. Three specimens were tested at a confining pressure of 40 psi, and in-situ water content to verify the repeatability of the results. The specimens were sheared at a rate of approximately 0.0075 percent per second. In addition to these tests, one specimen each was tested at confining pressures of 5, 10, 20, and 40 psi, and strain rates of 0.0075, 0.075, and 7.5 percent per second.

Four specimens were saturated and consolidated before shearing at a rate of 0.0075 percent per second. The tests were performed under undrained conditions, with pore pressure measurements.

4.7 DETERMINATION OF CEMENTATION.

Tests were also performed following procedures recommended by Jackson (1979) to estimate the calcium carbonate content of the specimens. In this method, the soil was oven dried and weighed. Dilute hydrochloric acid was added to the soil, until the calcium carbonate was dissolved. Once the reaction was complete, and no more carbon dioxide was evolved, the mixture was allowed to stand for about four hours to ensure completion
of the chemical reaction. The supernatant liquid was poured. Distilled water was added to the soil, and the mixture centrifuged. This process of washing was repeated several times. The residue was then oven dried. The loss of weight, expressed as a percentage of the original oven dry weight, is reported as the carbonate or the cementation.
CHAPTER FIVE

STATIC PROPERTIES

5.1. INTRODUCTION

Two soils were investigated. These soils can be broadly classified as a clay, and a silt. The grain size distribution curves of both soils were determined, using the sieve and hydrometer analysis. The Atterberg Limits were measured, in order to classify the soils. In addition to these tests, triaxial tests were performed on the reconstituted specimens of the silt, to determine the stress-strain characteristics of the soil.

5.2. STATIC PROPERTIES FOR CLAY

5.2.1. Physical Appearance. The soil was brown in color, and appeared to have a very low moisture content. The soil particles were extremely fine, but were strongly cemented together. The cementation was not uniform however, and the block samples appeared to have several cracks, and zones of low cementation. The soil was odorless, and showed little evidence of containing organic matter.

5.2.2. Index Properties. The cylindrical specimens for the cyclic triaxial tests were carved out of two block samples (hereafter referred to as Block 1 and Block 2). The index properties of both blocks were determined.

The sieve analysis was performed as per ASTM standard D422-63. Both dry and wet sieve analysis were performed. For both the block samples, the amount of soil passing the Number 200 (0.075 mm) sieve during wet sieving was substantially greater than the amount passing during dry sieving. During dry sieving, about 48 percent of the soil was finer than the Number 200 (0.075 mm) sieve, while after wet sieving, a total of about 73 percent of the soil passed the Number 200 (0.075 mm) sieve. For Block 2, the corresponding values were approximately 41 percent and 64 percent respectively. The grain size distribution for Block 1 is presented in Figure 5.1. The grain size distribution for Block 2 is presented in Figure 5.2. These graphs contain the data from both, the wet sieve and dry sieve, analyses. The Atterberg Limits were determined in accordance with ASTM procedure D4318-84. For Block 1, the liquid limit was 34 percent, and the plastic limit was about 21 percent. Therefore, the plasticity index was 14 percent. For Block 2, the liquid limit was 25.3 percent, and the plastic limit was 18 percent, yielding a plasticity index of 7.3. As per the Unified Soil Classification System and using the Atterberg Limits mentioned above, the soil from both Blocks classifies as an inorganic clay of low to medium plasticity (CL).
Figure 5.1  Grain Size Distribution for Block 1 of Clay
Figure 5.2  Grain Size Distribution for Block 2 of Clay
Some of the properties of clay specimens tested in this program are tabulated in Table 5.1.

5.3 STATIC PROPERTIES FOR SILT

5.3.1. Physical Appearance. The soil was brown in color, and apparently was very dry. The cementation in the soil seemed non-uniform. Most of the soil was weakly cemented, although there were a few zones that were strongly cemented. Even in these cemented zones there were several cracks, making it impossible to carve out cylindrical specimens required for the various tests. There were a few decayed roots of plants in the soil. Despite this, the soil contained little organic matter.

5.3.2. Index Properties. The index properties were determined for two samples. These samples will hereafter be referred to as Sample 1 and Sample 2. The dry and wet sieve analyses were performed as per ASTM D422-63. In addition, the wet and dry sieve analysis were also performed after the calcium carbonate in the soil was chemically removed. The calcium carbonate was dissolved using dilute hydrochloric acid, followed by successive washing with distilled water, and centrifuging.

When the hydrometer analysis was performed on soil from Sample 1 without removal of the calcium carbonate, the soil settled to the bottom of the jar immediately after dispersion, and no reading could be taken. When the test was performed on soil after treatment with dilute hydrochloric acid however, this problem was no longer encountered. One possible explanation for this behavior, could be that the presence of salts in the water significantly affects the results of the hydrometer analysis (El Jurf, 1989).

The grain size distribution for Sample 1, before and after treatment with hydrochloric acid are shown in Figures 5.3 and 5.4, respectively. For Sample 1, during dry sieving before treatment with hydrochloric acid, 14.6 percent of the soil was finer than the Number 200 (0.075 mm) sieve. Wet sieving of the same soil yielded a fraction of 52.9 percent of soil passing through the Number 200 (0.075mm) sieve. After removal of calcium carbonate from the soil, the corresponding value were 87.4 percent, and 98.0 percent.

The amount of soil passing the Number 200 (0.075 mm) sieve for Sample 2, during dry and wet sieving, were 18.1 percent and 52.4 percent respectively. The grain size distribution for the dry and wet sieve analysis are presented Figure 5.5. These values are quite similar to the corresponding values for Sample 1. After treatment with dilute hydrochloric acid however, 73.5 percent of the soil was finer than the Number 200 sieve, while after wet sieving 75.6 percent of the soil passed through the Number 200 sieve. The results of the grain size analysis test after treatment with hydrochloric acid are shown in Figure 5.6.
Table 5.1  PROPERTIES OF CLAY SPECIMENS

<table>
<thead>
<tr>
<th>TEST NO.</th>
<th>DIAMETER (in.)</th>
<th>LENGTH (in.)</th>
<th>VOID RATIO</th>
<th>MOISTURE CONTENT AFTER TEST (PERCENT)</th>
<th>CEMENTATION (PERCENT)</th>
</tr>
</thead>
<tbody>
<tr>
<td>RDC01</td>
<td>1.443</td>
<td>2.809</td>
<td>0.81</td>
<td>9.77</td>
<td>16.3</td>
</tr>
<tr>
<td>RDC02</td>
<td>1.418</td>
<td>3.051</td>
<td>0.88</td>
<td>4.62</td>
<td>17.4</td>
</tr>
<tr>
<td>RDC03</td>
<td>1.411</td>
<td>2.815</td>
<td>0.83</td>
<td>3.11</td>
<td>15.8</td>
</tr>
<tr>
<td>SDC01</td>
<td>1.418</td>
<td>3.051</td>
<td>0.88</td>
<td>4.62</td>
<td>17.4</td>
</tr>
<tr>
<td>SDC02</td>
<td>1.411</td>
<td>2.815</td>
<td>0.83</td>
<td>3.11</td>
<td>15.8</td>
</tr>
<tr>
<td>SDC03</td>
<td>1.407</td>
<td>3.221</td>
<td>0.84</td>
<td>6.63</td>
<td>13.9</td>
</tr>
</tbody>
</table>

SPECIFIC GRAVITY = 2.65 (ASSUMED)
Figure 5.3 Grain Size Distribution for Sample 1 of Silt, before Treatment with HCl
Figure 5.4  Grain Size Distribution for Sample 1 of Silt, after Treatment with HCl
Figure 5.5  Grain Size Distribution for Sample 2 of Silt, before Treatment with HCl
Figure 5.6  Grain Size Distribution for Sample 2 of Silt, after Treatment with HCl
The liquid limit and plastic limit of the soil from Sample 1 were 34 percent and 25 percent, respectively. The plasticity index is thus 9 percent. For the soil from Sample 2, the liquid limit, plastic limit, and plasticity index are 38 percent, 28 percent and 10 percent, respectively. As per the Unified Soil Classification System, Samples 1 and Sample 2 classify as inorganic silts (ML).

The dimensions of reconstituted silt specimens, along with their void ratios before testing, and the moisture contents after testing, are all presented in Table 5.2.

5.3.3 Triaxial Tests on Reconstituted Silt Specimens. Triaxial tests were performed on specimens of reconstituted silts to study the effects of confining pressure, strain rate, and degree of saturation on the strength parameters of the soil. First, three specimens were tested at in-situ water content, and an effective confining pressure of 40 psi, at a strain rate of approximately 0.0075 percent per second. This was done to check the repeatability of the results. Next, specimens were sheared at strain rates of 0.0075, 0.075 and 7.5 percent per second. For each strain rate, one specimen each was tested at confining pressures of 5, 10, 20, and 40 psi. These specimens were at in-situ water content at the time of testing. In the final series of tests, four specimens were back-pressure saturated and consolidated. One specimen each was then sheared at effective confining pressures of 5, 10, 20, and 40 psi, under undrained conditions. All tests on the saturated specimens were performed at a strain rate of about 0.0075 percent per second.

5.3.3.1. Specimens at In-situ Water Content. The results from the three specimens tested at a confining pressure of 40 psi, and a strain rate of 0.0075 percent per second, are presented in Figure 5.7. As seen in the graph, stress-strain behavior of the three specimens are very similar to one another, and show little variation. This shows that the results are very repeatable, and reliable. Up to a strain level of about 1 percent, the stress increases almost linearly with strain. Beyond this strain level, the soil behaves non-linearly. The maximum stress was reached at a strain of approximately 10 percent, and the maximum stress attained was about 200 psi. The maximum strain imposed on the specimen was of the order of 15 to 20 percent. Higher strains were not applied, as the specimen failed, and a well defined failure plane could be observed during testing.

The effects of confining pressure are illustrated in Figures 5.8 through 5.10. These graphs correspond to strain rates of 0.0075, 0.075, and 7.5 percent per second, respectively. As expected, the maximum deviatoric stress attained increases as the confining pressure increases. The same data is presented in Figures 5.11 through 5.14, with each graph showing the effect of strain rate on the stress-strain behavior, at one confining pressure. Failure of the specimens occur at strain levels of about 1, 5, 7, and 10 percent, for confining pressures of 5, 10, 20, and 40 psi. The strain rate apparently has little effect on the stress-strain behavior of the specimen. However, the specimen tested at a strain rate of 7.5 percent per second appear to be more ductile than the specimens tested at strain rates of 0.0075 and 0.075 percent per second.
### Table 5.2  PROPERTIES OF RECONSTITUTED SILT SPECIMENS

<table>
<thead>
<tr>
<th>TEST NO.</th>
<th>DIAMETER (in.)</th>
<th>LENGTH (in.)</th>
<th>VOID RATIO</th>
<th>MOISTURE CONTENT AFTER TEST (PERCENT)</th>
</tr>
</thead>
<tbody>
<tr>
<td>RSD1</td>
<td>1.392</td>
<td>3.500</td>
<td>0.72</td>
<td>2.33</td>
</tr>
<tr>
<td>RSD2</td>
<td>1.392</td>
<td>3.460</td>
<td>0.68</td>
<td>2.33</td>
</tr>
<tr>
<td>RSD3</td>
<td>1.392</td>
<td>3.500</td>
<td>0.72</td>
<td>2.28</td>
</tr>
<tr>
<td>RSD4</td>
<td>1.392</td>
<td>3.415</td>
<td>0.72</td>
<td>2.66</td>
</tr>
<tr>
<td>RSD5</td>
<td>1.392</td>
<td>3.440</td>
<td>0.71</td>
<td>2.33</td>
</tr>
<tr>
<td>RSD6</td>
<td>1.392</td>
<td>3.410</td>
<td>0.71</td>
<td>2.33</td>
</tr>
<tr>
<td>RSD7</td>
<td>1.392</td>
<td>3.451</td>
<td>0.70</td>
<td>2.66</td>
</tr>
<tr>
<td>RSD8</td>
<td>1.392</td>
<td>3.422</td>
<td>0.71</td>
<td>2.38</td>
</tr>
<tr>
<td>RDS9</td>
<td>1.392</td>
<td>3.461</td>
<td>0.70</td>
<td>2.30</td>
</tr>
<tr>
<td>RSD10</td>
<td>1.392</td>
<td>3.400</td>
<td>0.71</td>
<td>2.29</td>
</tr>
<tr>
<td>RSD11</td>
<td>1.392</td>
<td>3.420</td>
<td>0.71</td>
<td>2.23</td>
</tr>
<tr>
<td>RSD12</td>
<td>1.392</td>
<td>3.430</td>
<td>0.71</td>
<td>2.36</td>
</tr>
<tr>
<td>RSW1</td>
<td>1.392</td>
<td>3.411</td>
<td>0.71</td>
<td>27.07</td>
</tr>
<tr>
<td>RSW2</td>
<td>1.392</td>
<td>3.410</td>
<td>0.70</td>
<td>27.63</td>
</tr>
<tr>
<td>RSW3</td>
<td>1.392</td>
<td>3.440</td>
<td>0.70</td>
<td>24.83</td>
</tr>
<tr>
<td>RSW4</td>
<td>1.392</td>
<td>3.400</td>
<td>0.71</td>
<td>26.32</td>
</tr>
<tr>
<td>RSW5</td>
<td>1.392</td>
<td>3.420</td>
<td>0.71</td>
<td>26.21</td>
</tr>
<tr>
<td>RSW6</td>
<td>1.392</td>
<td>3.420</td>
<td>0.71</td>
<td>27.33</td>
</tr>
</tbody>
</table>

Specific Gravity = 2.66
Moisture content before test = 2.33 %
In situ void ratio = 0.70
Cementation = 4.3 %
Table 5.2 (contd) PROPERTIES OF RECONSTITUTED SILT SPECIMENS

<table>
<thead>
<tr>
<th>TEST NO.</th>
<th>DIAMETER (in.)</th>
<th>LENGTH (in.)</th>
<th>VOID RATIO</th>
<th>MOISTURE CONTENT AFTER TEST (PERCENT)</th>
</tr>
</thead>
<tbody>
<tr>
<td>RTS01</td>
<td>1.392</td>
<td>3.410</td>
<td>0.71</td>
<td>27.63</td>
</tr>
<tr>
<td>RTS02</td>
<td>1.392</td>
<td>3.410</td>
<td>0.70</td>
<td>26.33</td>
</tr>
<tr>
<td>RTS03</td>
<td>1.392</td>
<td>3.441</td>
<td>0.68</td>
<td>29.74</td>
</tr>
<tr>
<td>RTS04</td>
<td>1.392</td>
<td>3.401</td>
<td>0.71</td>
<td>28.39</td>
</tr>
<tr>
<td>RTS05</td>
<td>1.392</td>
<td>3.420</td>
<td>0.71</td>
<td>24.46</td>
</tr>
<tr>
<td>RTS06</td>
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<td>3.422</td>
<td>0.70</td>
<td>25.55</td>
</tr>
<tr>
<td>RTX01</td>
<td>1.392</td>
<td>3.433</td>
<td>0.72</td>
<td>2.36</td>
</tr>
<tr>
<td>RTX02</td>
<td>1.392</td>
<td>3.411</td>
<td>0.71</td>
<td>2.35</td>
</tr>
<tr>
<td>RTX03</td>
<td>1.392</td>
<td>3.400</td>
<td>0.70</td>
<td>2.33</td>
</tr>
<tr>
<td>RTX04</td>
<td>1.392</td>
<td>3.416</td>
<td>0.69</td>
<td>2.33</td>
</tr>
<tr>
<td>RTX05</td>
<td>1.392</td>
<td>3.429</td>
<td>0.69</td>
<td>2.36</td>
</tr>
<tr>
<td>RTX06</td>
<td>1.392</td>
<td>3.433</td>
<td>0.72</td>
<td>2.37</td>
</tr>
<tr>
<td>RTX07</td>
<td>1.399</td>
<td>3.400</td>
<td>0.72</td>
<td>26.71</td>
</tr>
<tr>
<td>RTX08</td>
<td>1.391</td>
<td>3.440</td>
<td>0.72</td>
<td>28.21</td>
</tr>
<tr>
<td>RTX09</td>
<td>1.390</td>
<td>3.454</td>
<td>0.71</td>
<td>25.23</td>
</tr>
<tr>
<td>RTX10</td>
<td>1.400</td>
<td>3.555</td>
<td>0.70</td>
<td>25.00</td>
</tr>
<tr>
<td>RTX11</td>
<td>1.392</td>
<td>3.400</td>
<td>0.70</td>
<td>2.40</td>
</tr>
<tr>
<td>RTX12</td>
<td>1.392</td>
<td>3.411</td>
<td>0.70</td>
<td>2.40</td>
</tr>
<tr>
<td>RTX13</td>
<td>1.392</td>
<td>3.413</td>
<td>0.71</td>
<td>2.33</td>
</tr>
<tr>
<td>RTX14</td>
<td>1.392</td>
<td>3.432</td>
<td>0.71</td>
<td>2.29</td>
</tr>
<tr>
<td>RTX15</td>
<td>1.392</td>
<td>3.422</td>
<td>0.69</td>
<td>2.26</td>
</tr>
<tr>
<td>RTX16</td>
<td>1.392</td>
<td>3.415</td>
<td>0.68</td>
<td>2.33</td>
</tr>
<tr>
<td>RTX17</td>
<td>1.392</td>
<td>3.444</td>
<td>0.70</td>
<td>2.44</td>
</tr>
<tr>
<td>RTX18</td>
<td>1.392</td>
<td>3.455</td>
<td>0.70</td>
<td>2.42</td>
</tr>
</tbody>
</table>

SPECIFIC GRAVITY = 2.66
MOISTURE CONTENT BEFORE TEST = 2.33 %
INSITU VOID RATIO = 0.70
CEMENTATION = 4.3 %
Figure 5.7 Stress-Strain Variation for Three Reconstituted Specimens at a Confining Pressure of 40 Psi, at In-situ Water Content
Figure 5.8  Effect of Confining Pressure on Stress-Strain Behavior of Reconstituted Specimens at In-situ Water Content, at a Strain Rate of about 0.0075 Percent/Second
Figure 5.9  Effect of Confining Pressure on Stress–Strain Behavior of Reconstituted Specimens at In-situ Water Content, at a Strain Rate of about 0.075 Percent/Second
Figure 5.10  Effect of Confining Pressure on Stress-Strain Behavior of Reconstituted Specimens at In-situ Water Content, at a Strain Rate of about 7.5 Percent/Second
Figure 5.11 Effect of Strain Rate on Stress-Strain Behavior of Reconstituted Specimens at In-situ Water Content, at a Confining Pressure of 5 Psi
Figure 5.12  Effect of Strain Rate on Stress Behavior of Reconstituted Specimens at In-situ Water Content, at a Confining Pressure of 10 Psi
Figure 5.13 Effect of Stain Rate on Stress-Strain Behavior of Reconstituted Specimens at In-situ Water Content, at a Confining Pressure of 20 Psi
Figure 5.14 Effect of Strain Rate on Stress-Strain Behavior of Reconstituted Specimens at In-situ Water Content, at a Confining Pressure of 40 Psi
The Mohr Circles representing the state of stress at failure, for specimens tested at strain rates of 0.0075, 0.075 and 7.5 percent per second, are presented in Figures 5.15 through 5.17. From the Mohr circles at failure, the angles of internal friction at strain rates of 0.0075, 0.075 and 7.5 percent are 43, 45 and 45 degrees, respectively. The failure envelope passes through the origin, indicating that the soil tested is cohesionless. The strain rate and the confining pressure appear to have little effect on the strength parameters of the soil.

The effective angle of internal friction was also calculated for each of the tests, using the relationship

\[ (\sigma_1 / \sigma_3)_i = \tan^2 (45^\circ + \phi/2) \]

(5.1)

where \((\sigma_1 / \sigma_3)_i\) is the ratio of the major and minor principal stresses at failure, and \(\phi\) is the effective angle of internal friction.

For the three specimens tested at a confining pressure of 40 psi, and at a strain rate of 0.0075 percent per second, the values of internal friction varied between 42.3 and 40.7 degrees, with an average of 41.4 degrees. For the twelve combinations, involving four confining pressures (5, 10, 20, and 40 psi), and three strain rates (0.0075, 0.075, and 7.5 percent per second), the angle of internal friction varied between 43.8 and 39.5 degrees, with an average value of 41.4 degrees. This calculated value for the angle of internal friction differs from the angle of internal friction obtained using Mohr circles. This small difference is perhaps due to the unavoidable inaccuracies in drawing of the Mohr circles.

It must be noted here that the above computations were performed on the premise that the soil was non-cohesive. This assumption is based on the fact that the failure envelope to the Mohr circles at failure passes through the origin.

5.3.3.2. Specimens Saturated and Consolidated Before Testing. The aim of these tests was to study the strength parameters of the soil after saturation. The specimens were saturated by the method of back pressured. The back pressure was increased in steps to a maximum pressure of 200 psi. It was ensured that the effective confining pressure at each increment of back pressure did not exceed the final effective confining pressure at which the test was to be performed. The specimen was considered to be saturated when the B-coefficient (Skempton, 1954), was 0.95 or greater. After saturation, the specimen was consolidated at the final effective confining pressure. The specimen was sheared under undrained conditions, at a strain rate of about 0.0075 percent per second. Four specimens were tested, one each at confining pressures of 5, 10, 20, and 40 psi.

The results from these tests are presented in Figures 5.18 through 5.21, for
Figure 5.15  Mohr Circles for all Reconstituted Silt Specimens Tested at In-situ Water Content, and at a Strain Rate of About 0.0075 Percent/Second
Figure 5.16  Mohr Circles for all Reconstituted Silt Specimens Tested at In-situ Water Content, and at a Strain Rate of About 0.075 Percent/Second
Figure 5.17 Mohr Circles for all Reconstituted Silt Specimens Tested at In-situ Water Content, and at a Strain Rate of About 7.5 Percent/Second
effective confining pressures of 5, 10, 20, and 40 psi, respectively. It can be seen from the plots of principal stress difference versus axial strain, that the load increases monotonically with the increase in axial strain. In other words, the curve does not show a peak and drop. Therefore, the test was discontinued after an axial strain of approximately 20 percent.

Specimen RTX07, tested at an effective confining pressure of 5 psi, showed very little change in the pore pressure during shearing (see Figure 5.18). The specimen exhibited a very slight dilative behavior, indicated by negative pore pressures generated during the test. It must be emphasized, that the negative pore pressures generated were extremely small. The other three specimens in the set, RTX08, RTX09, and RTX10, tested at effective confining pressures of 10, 20, and 40 psi respectively, showed a tendency to contract during shearing. A positive build-up of pore pressure during shearing is an indication of this type of behavior (see Figures 5.19 to 5.21).

The angle of internal friction for each specimen tested was calculated using Equation 5.1. The angles of internal friction corresponding to confining pressures of 5, 10, 20 and 40 psi, were 36.9, 41.8, 37.3, and 40.1 degrees respectively. From principal stress difference conditions however, the angle of internal friction was zero. The reason for this apparent discrepancy is not known at this time.
Figure 5.18  R-Test Results for Specimen RTX07
Figure 5.19  R-Test Results for Specimen RTX08
Figure 5.20  R-Test Results for Specimen RTX09
Figure 5.21  R-Test Results for Specimen RTX10
CHAPTER SIX

DYNAMIC PROPERTIES OF CLAY

6.1 INTRODUCTION

This series of tests were performed on intact specimens that were carved out by hand from block samples obtained from the walls of the exploratory trench. The calcium carbonate content in the block samples was not uniform. Therefore, it was difficult to carve cylindrical specimens from the blocks, and only few specimens could be obtained for testing. As such, this series of tests is not as comprehensive as other series in this program.

All tests in this series were performed on specimens at in-situ water content. Of the seven tests, four were used to test the repeatability of the results. Each specimen was tested at low strain levels (below the onset of degradation) at confining pressures of 5, 10, 20, and 40 psi. The other three tests in the series were also conducted at confining pressures of 10, 20 and 40 psi, but the specimens were tested up to strain levels of almost 1 percent, to study the behavior of the soil at high strain levels. It must be noted here that a high-strain test was not performed at a confining pressure of 5 psi, since sufficient specimens were not available. Also, threshold strain or liquefaction susceptibility was not studied because of the nature of the soil (i.e. clay).

6.2. REPEATABILITY TESTS

6.2.1. Young’s Moduli. The values for the Young’s Modulus obtained from tests at confining pressures of 5, 10, 20, and 40 psi on each of the three clay specimens tested for repeatability, are graphically illustrated in Figures 6.1 through 6.3, respectively. As expected, the Young’s Modulus decreases with increase in strain, and increases with increase in confining pressure. The data is very well behaved, and shows little scatter. The moduli from the specimens tested for repeatability are presented in Figures 6.4 through 6.7. The data corresponding to a confining pressure of 5 psi is shown in Figure 6.4. The results form specimens RDC01 and RDC02 are almost identical, with the variation in the modulus being less than 10 percent at any given strain level. The values from specimen RDC03 however, were lower than the moduli from RDC01 and RDC02, by almost 25 percent. For confining pressures of 10, 20, and 40 psi (see Figures 6.5, 6.6, 6.7, respectively), the moduli for all specimens are within ten percent each other. The results from the high strain tests compare well with the results from the specimens tested for repeatability. At this time, the above mentioned discrepancy in the data at a confining pressure of 5 psi cannot be explained.
Figure 6.1 Variation in Modulus with Strain and Confining Pressure for Specimen RDC01
Figure 6.2 Variation in Modulus with Strain and Confining Pressure for Specimen RDC02
Figure 6.3 Variation in Modulus with Strain and Confining Pressure for Specimen RDC03
Figure 6.4 Variation in Modulus with Strain for Three Specimens Tested at In-situ Water Content, at a Confining Pressure of 5 Psi
Figure 6.5 Variation in Modulus with Strain for Four Specimens Tested at In-situ Water Content, at a Confining Pressure of 10 Psi
Figure 6.6 Variation in Modulus with Strain for Four Specimens Tested at In-situ Water Content, at a Confining Pressure of 20 Psi
Figure 6.7 Variation in Modulus with Strain for Four Specimens Tested at In-situ Water Content, at a Confining Pressure of 40 Psi
6.2.2. Damping Ratios. The variation in damping ratio with strain and confining pressure for the three specimens tested for repeatability, are shown in Figures 6.8 through 6.10. The damping ratio increases almost linearly with the logarithm of strain, and reaches a peak at about 0.1 percent strain. For the repeatability tests, the maximum damping ratio is approximately 25 percent. The variability in damping ratios as a function of strain for the specimens tested at in-situ water content, are shown in Figures 6.11 through 6.14. These graphs represent the data collected for each specimen at confining pressures of 5, 10, 20, and 40 psi. The damping ratio increases almost linearly with the logarithm of strain, reaches a peak, and then decreases slightly. This trend is apparent in the above mentioned sets of graphs. The data shows some scatter; but the amount of scatter decreases with increase in confining pressure. The damping ratio for these specimens varies between 3 and 25 percent.

6.3. SPECIMENS TESTED TO HIGH STRAIN LEVELS.

6.3.1. Young's Moduli. The data from the three specimens tested to high strain levels are shown in Figure 6.15. The data exhibits the expected trends on increase in modulus with confining pressure, and a reduction in the modulus with increasing strain levels. The rate of decrease in modulus with strain is much higher at low strain levels, as compared to the rate of reduction at higher strains. In the graphical representation of the data, this change in the rate of reduction with strain is seen as a steeper slope at low strains, and a gradual flattening of the curve at the higher strains. Also, at lower strains, the effect of confining pressures is more prominent. On the other hand, at strain levels beyond about 0.1 percent, the three curves approach each other.

A clearer picture of the effect of confining pressure on the Young's Modulus is presented in Figure 6.16. In this Figure, the modulus at a particular strain level is plotted as a function of confining pressure on a log-log scale. For a given strain level, the relationship between the logarithm of the modulus and the logarithm of the confining pressure is almost linear, with the modulus increasing with increase in confining pressure. The slopes of the best fit lines corresponding to strains of 0.05, 0.1, and 0.2 percent, are 0.58, 0.59 and 0.58 respectively. These values are slightly higher than 0.5 proposed in literature for low strain moduli.

6.3.2. Damping Ratios. The damping ratios for the three specimens tested at high strain levels, is presented in Figure 6.17. The damping ratio increases with strain, up to strain levels of about 0.1 percent. Beyond this strain, the damping ratio decreases once again. For these tests, the damping ratio varies between 10 and 25 percent. There is appreciable scatter in the data, especially for strains of about 0.1 percent.

This variation of the damping ratio is not typical. One possible reason for this could be the friction in the bearing at the top of the triaxial cell.
Figure 6.8 Variation in Damping Ratio with strain and Confining Pressure for Specimen RDC01 Tested at In-situ Water Content
Figure 6.9  Variation in Damping Ratio with Strain and Confining Pressure for Specimen RDC02 Tested at In-situ Water Content
Figure 6.10 Variation in Damping Ratio with Strain and Confining Pressure for Specimen RDC03 Tested at In-situ Water Content
Figure 6.11: Variation in Damping Ratio with Strain for Specimens Tested at In-situ Water Content at a Confining Pressure of 5 Psi
Figure 6.12  Variation in Damping Ratio with Strain for Specimens Tested at In-situ Water Content at a Confining Pressure of 10 Psi
Figure 6.13 Variation in Damping Ratio with Strain for Four Specimens Tested at In-situ Water Content at a Confining Pressure of 20 Psi
Figure 6.14 Variation in Damping Ratio with Strain for Four Specimens Tested at In-situ Water Content at a Confining Pressure of 40 Psi
Figure 6.15. Variation in Modulus with Strain for Three Specimens Tested to Failure at In-situ Water Content
Figure 6.16  Variation in Modulus with Confining Pressure and Strain for Clay Specimen
Figure 6.17  Variation in Damping Ratio with Strain for Three Specimens Tested to Failure at In-situ Water Content
CHAPTER SEVEN

DYNAMIC PROPERTIES OF RECONSTITUTED SILT

7.1. SPECIMENS TESTED AT IN-SITU WATER CONTENT

7.1.1. Introduction. Attempts were made to carve out intact specimens from block samples obtained from the wall of the exploratory trench. However, these blocks had a very low cementation, and crumbled easily. This made it impossible to carve out intact specimens from the blocks. Therefore, a decision was made to perform tests on specimens reconstituted at their in-situ density and water content.

In order to determine the in-situ density, chunks of the soil were carefully weighed. The surface was then coated with molten paraffin, and allowed to cool. The weight of the sample coated with the paraffin was noted. The coated sample was then immersed in water, and the volume dispersed by the sample was accurately measured. Using this measured data, along with the specific gravity and moisture content of the soil, the in-situ bulk density was determined. A total of six specimens were tested. The specific gravity varied between 2.69 and 2.61, with an average of 2.66. The moisture content of the samples was in the range of 1.98 and 2.52 percent, with an average of 2.33 percent. Eight specimens were tested to determine the in-situ bulk density. The bulk density of the soil was bound between 94.7 and 103.4 pounds per cubic foot. The average bulk density was 99.7 pounds per cubic foot.

The specimens were reconstituted on the base pedestal of the triaxial cell, using a split forming mold. Initially, the method of undercompaction (Ladd, 1978) was to be used to prepare the specimens. However, the density of the specimens prepared by this method, was less than the in-situ density. Therefore, the specimens were prepared in six layers of equal thickness, by the method of dry tamping. The total amount of soil required to prepare the specimen was determined by multiplying the volume of the mold with the required bulk density of the soil.

Twelve specimens were tested at in situ water content. Tests at confining pressures of 5, 10, 20 and 40 psi were carried out. At each water content, three specimens were tested. The objective of these tests was to study the variation in the modulus and damping ratio of the material with confining pressure, and strain levels. Three specimens were tested at each confining pressure, to verify the repeatability of the results obtained. The lowest strain level at which the tests were performed was about 0.01 percent (peak-to-peak), while the highest strain level was about 0.60 percent (peak-to-peak). Each specimen was tested at increasing strain levels. After testing at each strain level was completed, the specimen was once again tested at the lowest strain level
to determine if stiffness degradation had occurred on the specimen.

7.1.2. Young’s Moduli. Typical variation in Young’s modulus with strain, at confining pressures of 5, 10, 20, and 40 psi are illustrated in Figures 7.1 through 7.4 respectively. In each figure, the data obtained from the three specimens tested at the corresponding confining pressure are shown. At each confining pressure, the variation in the modulus obtained from the three specimens tested, is less than 10 percent. This low variation in the moduli for the three specimens, is an indication of a high degree of repeatability in specimen preparation and testing procedure.

The variation in Young’s modulus with strain, for all twelve specimens in this test series is shown in Figure 7.5. A best-fit curve is drawn through the data obtained from the three specimens tested at each confining pressure. The data generally exhibit the expected trends. The modulus increases with increase in confining pressure, and decreases with increasing strain levels.

The effect of confining pressure on the modulus is depicted in the form of a log-log plot, in Figure 7.6. It is noted from this graph that the logarithm of the modulus increases almost linearly with increase in the logarithm of the confining pressure. The slopes of the best fit lines corresponding to strain levels of 0.05, 0.1, and 0.2 percent are 0.50, 0.50, and 0.51, respectively. These values are quite comparable with 0.50 typically suggested.

7.1.3. Damping Ratios. Damping ratios for the specimens tested at confining pressures of 5, 10, 20, and 40 psi are shown in Figures 7.7 through 7.10, respectively. Each figure contains the data obtained from the three specimens tested at that particular confining pressure. The damping ratios for the three specimens tested at a confining pressure of 5 psi show a large scatter. The specimens tested at a confining pressure of 10 psi exhibit less scatter. The damping ratio increases almost linearly with the logarithm of the strain. At confining pressures of 20, and 10 psi, the damping ratio also increases with increase in strain level, reaches a peak, and then reduces slightly with increase in strain.

The damping ratios show some scatter. However, as the confining pressure increases, the amount of scatter decreases. For the three specimens tested at a confining pressure of 40 psi, the variation is less than 20 percent.

7.2. TESTS ON SATURATED SPECIMENS

7.2.1. Introduction. Six reconstituted specimens were saturated and consolidated, before being tested. Of these six specimens, three were tested at an effective confining pressure of 20 psi to check the repeatability of the results, while one each was tested at confining pressures of 5, 10, and 40 psi. Each specimen was backpressured in increments, until the total confining pressure was about 180 psi. During the
Figure 7.1: Variation in Modulus with Strain at a Confining Pressure of 5 Psi, for 3 Reconstituted Specimens, at In-situ Water Content
Figure 7.2 Variation in Modulus with Strain at a Confining Pressure of 10 Psi, for 3 Reconstituted Specimens, at In-situ Water Content
Figure 7.3 Variation in Modulus with Strain at a Confining Pressure of 20 Psi, for 3 Reconstituted Specimens, at In-situ Water Content
Figure 7.4 Variation in Modulus with Strain at a Confining Pressure of 40 Psi, for 3 Reconstituted Specimens, at In-situ Water Content
Figure 7.5  Variation in Modulus with Strain and Confining Pressure, for 3 Reconstituted Specimens, at In-situ Water Content
Figure 7.6 Variation in Modulus with Confining Pressure and Strain for Reconstituted Specimens at In-situ Water Content
Figure 7.7 Variation in Damping Ratio with Strain for Three Reconstituted Specimens, at In-situ Water Content at a Confining Pressure of 5 Psi
Figure 7.8  Variation in Damping Ratio with Strain for Three Reconstituted Specimens, at In-situ Water Content at a Confining Pressure of 10 Psi
Figure 7.9 Variation in Damping Ratio with Strain for Three Reconstituted Specimens, at In-situ Water Content at a Confining Pressure of 20 Psi
Figure 7.10 Variation in Damping Ratio with Strain for Three Reconstituted Specimens, at In-situ Water Content at a Confining Pressure of 40 Psi
backpressuring process, the effective confining pressure was maintained at 5 psi. After saturation was complete, the specimen was consolidated at the effective confining pressure at which the test was to be performed.

7.2.2. Young's Moduli. The variation of modulus with strain for three reconstituted specimens, tested at an effective confining pressure of 20 psi, is shown in Figure 7.11. The expected trend is seen in the data (i.e., the modulus decreases with increase in strain). The variation in the modulus for the three specimens is less than 10 percent, indicating that the test results are repeatable. The results of tests on four specimens, one each tested at confining pressures of 5, 10, 20 and 40 psi, are presented in Figure 7.12. Little scatter is seen in the data. As expected, the modulus increases with increasing confining pressure, and decreases with increase in strain. There is a linear relationship between the logarithm of the modulus and the logarithm of the confining pressure. This can be seen in Figure 7.13, which is a log-log plot of modulus versus confining pressure. The slopes of the best fit lines are 0.33, 0.37, and 0.37 for strain levels of 0.05, 0.1 and 0.2 percent, respectively. These values for the slopes are lower than 0.50, which is the typical value reported.

The moduli obtained from the tests on the saturated and consolidated specimens, at effective confining pressures of 5, 10, 20 and 40 psi, are compared with the results of tests on specimens tested at in situ water content, at the corresponding confining pressures in Figures 7.14 through 7.17 respectively. At all four confining pressures, the saturated specimens appear to be stiffer at low strain levels than the specimens tested at in situ water content. For different confining pressures, the two curves intersect at a strain level between approximately 0.10 and 0.30 percent. At higher strain levels and a confining pressure of 40 psi, the specimen at in situ water content exhibits higher modulus than the corresponding saturated specimen. Further investigations demonstrate that this matter is partially due to the friction of the piston-cell assembly. The total confining pressure in the triaxial cell when testing saturated specimens is of the order of 200 psi, while testing specimens at in-situ water content, the confining pressure varies between 5 and 40 psi. The friction in the bearing at the top of the triaxial cell where the actuator rod enters the cell, is logically higher at higher confining pressures, resulting in higher modulus values.

7.2.3. Damping Ratios. The damping ratios for the three specimens tested at an effective confining pressure of 20 psi are presented in Figure 7.18. The damping ratios for the four specimens tested at confining pressures of 5, 10, 20 and 40 psi, are presented in Figure 7.19. The general trend observed is an increase in the damping ratio with increase in strain. Some scatter is evident in the data, at higher strain levels.

7.3. THRESHOLD STRAIN TESTS ON RECONSTITUTED SPECIMENS

7.3.1. Introduction. Tests were performed to determine the threshold strain for the
Figure 7.11 Variation in Modulus with Strain for Three Reconstituted Specimens Saturated and Consolidated before Testing at an Effective Confining Pressure of 20 Psi
Figure 7.12 Variation in Modulus with Strain and Confining Pressure, for Specimens Saturated and Consolidated before Testing
Figure 7.13 Variation in Modulus with Confining Pressure and Strain for Reconstituted Specimens Saturated and Consolidated before testing
Figure 7.14 Comparison of Moduli from Reconstituted Specimens at In-situ Water Content, and Specimens Saturated and Consolidated before Testing, at an Effective Confining Pressure of 5 Psi.
Figure 7.15 Comparison of Moduli from Reconstituted Specimens at In-situ Water Content, and Specimens Saturated and Consolidated before Testing, at an Effective Confining Pressure of 10 Psi
Figure 7.16 Comparison of Moduli from Reconstituted Specimens at In-situ Water Content, and Specimens Saturated and Consolidated before Testing, at an Effective Confining Pressure of 20 Psi
Figure 7.17 Comparison of Moduli from Reconstituted Specimens at In-situ Water Content, and Specimens Saturated and Consolidated before Testing, at an Effective Confining Pressure of 40 Psi
Figure 7.18 Variation in Damping Ratio with Strain for Three Reconstituted Specimens, Saturated and Consolidated before Testing at an Effective Confining Pressure of 20 Psi
Figure 7.19  Variation in Damping Ratio with Strain and Confining Pressure for Specimens Saturated and Consolidated before Testing
reconstituted specimens. As discussed earlier, threshold strain is defined as the minimum strain level below which no build-up in pore pressure is observed, when a saturated specimen is subjected to cyclic strain under undrained conditions. The pore pressure is normalized against the effective confining pressure at which the test was performed. This ratio of the pore pressure to the effective confining pressure is called the "residual pore pressure".

Six reconstituted specimens were tested to determine their threshold strains. These include three specimens tested at an effective confining pressure of 40 psi to test repeatability of the results. One specimen each was tested at effective confining pressures of 5, 10, and 20 psi.

Variations in residual pore pressure with axial strain for three reconstituted specimens tested at an effective confining pressure of 40 psi are shown in Figures 7.20 through 7.22. The general trend is quite apparent. Up to a certain strain level called the "threshold strain" the residual pore pressure is almost constant, and is relatively insensitive to the number of cycles of strain imposed on the specimen. For the specimens tested in this program, the threshold strain is of the order of 0.04 percent. Beyond the threshold strain however, there is a rapid increase in the residual pore pressure as the strain increases. At this stage, the residual pore pressure is also affected by the number of strain cycles applied to the specimen. Generally, the residual pore pressure increases as the number of cycles applied increases.

The variation in residual pore pressure with strain amplitude for the three specimens tested at an effective confining pressure of 40 psi, at the end of 5, 10, 25 and 50 cycles are presented in Figures 7.23 through 7.26, respectively. The variation in the pore pressure for the three specimens is of the order of 20 percent. This relatively low variation suggest fairly high degrees of repeatability and reliability of the test results.

The effect of confining pressure on the residual pore pressure is illustrated in Figures 7.27 through 7.30. Although there is some scatter in the data, there is a general trend of an increase in the residual pore pressure with increase in confining pressures, indicating a more rapid build-up of pore pressure at higher confining pressures.

Mathematically, the actual pore pressure is defined as the product of the residual pore pressure and the effective confining pressure at which the test is conducted. Therefore, if two specimens tested at different confining pressures exhibit the same residual pore pressure at a given strain level, the actual pore pressure developed in the specimen tested at the higher confining pressure would be greater.

7.4 **YOUNG'S MODULI FROM BENDER ELEMENTS**

7.4.1. Introduction. It has been mentioned on several occasions in this report, that
Figure 7.20  Variation in Residual Pore Pressure with Strain and Number of Cycles, for Reconstituted Specimen RTS01, at an Effective Confining Pressure of 40 Psi
Figure 7.21  Variation in Residual Pore Pressure with Strain and Number of Cycles, for Reconstituted Specimen RTS02, at an Effective Confining Pressure of 40 Psi
Figure 7.22 Variation in Residual Pore Pressure with Strain and Number of Cycles, for Reconstituted Specimen RTS03, at an Effective Confining Pressure of 40 Psi
Figure 7.23 Variation in Residual Pore Pressure with Strain, for 3 Reconstituted Specimens Tested at an Effective Confining Pressure of 40 Psi, at the End of 5 Cycles
Figure 7.24 Variation in Residual Pore Pressure with Strain, for 3 Reconstituted Specimens Tested at an Effective Confining Pressure of 40 Psi, at the End of 10 Cycles.
Figure 7.25  Variation in Residual Pore Pressure with Strain, for 3 Reconstituted Specimens Tested at an Effective Confining Pressure of 40 Psi, at the End of 25 Cycles
Figure 7.26  Variation in Residual Pore Pressure with Strain, for 3 Reconstituted Specimens Tested at an Effective Confining Pressure of 40 Psi, at the End of 50 Cycles
Figure 7.27 Variation in Residual Pore Pressure with Strain and Confining Pressure, for Reconstituted Specimens Tested, at the End of 5 Cycles
Figure 7.28 Variation in Residual Pore Pressure with Strain and Confining Pressure, for Reconstituted Specimens Tested, at the End of 10 Cycles
Figure 7.29 Variation in Residual Pore Pressure with Strain and Confining Pressure, for Reconstituted Specimens Tested, at the End of 25 Cycles
Figure 7.30  Variation in Residual Pore Pressure with Strain and Confining Pressure, for Reconstituted Specimens Tested, at the End of 50 Cycles
friction caused some discrepancies in the modulus of the soil as determined by cyclic triaxial tests. In order to obtain an approximate quantitative estimate of the influence of friction of the values of moduli determined, the initial shear modulus \( G_{max} \) and the Young's modulus \( E_{max} \) for the reconstituted silt specimens were determined using bender elements.

This section describes a method of measuring \( G_{max} \) using bender elements (Dyvik et al., 1985), developed at the Norwegian Geotechnical Institute (NGI). The technique involves the use of piezoceramic bender elements at each end of the soil specimen. The bender element at one end of the specimen is used to generate a shear wave pulse which propagates along the length of the specimen and the other element is used to determine the arrival time of the shear wave at the other end of the specimen. The travel time along the known specimen length produces a direct measurement of the shear wave velocity, \( V_s \), and in turn, \( G_{max} \) for the soil. Traditionally, the resonant column test is used to determine \( G_{max} \). The advantage of using bender elements is that the computations of \( G_{max} \) are much simpler and more direct than using the resonant column test.

### 7.4.2. Description of Bender Elements.

The piezoceramic bender element is an electro-mechanical transducer which is capable of converting mechanical energy (movement) either to or from electrical energy. The element itself consists of two thin piezoceramic plates which are rigidly bonded together with conduction surfaces between them and on the outsides (Dyvik, 1985). The polarization of the ceramic material in each plate and the electrical connections are such that when a driving voltage is applied to the element, one plate elongates and the other shortens. The net result is a bending displacement which is greater in magnitude than the length change in either of the two layers (plates). On the other hand, when the element is forced to bend, one layer will go into tension and the other into compression. This will result in an electrical signal which can be measured through the wire leads to the element. The shape of an element before and after the driving voltage is applied, is shown in Figure 7.31.

There are two different types of bender elements: series connected and parallel connected. The electrical connections to each element are shown in Figures 7.32 and 7.33 respectively. A series connected element is twice as effective as a parallel connected element when used as a generator or a sensor (mechanical to electrical energy). On the other hand, a parallel connected element is twice as effective as a series connected element when used as a motor or source (electrical to mechanical energy). For these reasons, a parallel connected element is used as the transmitter and a series connected element as the receiver of the shear waves in the soil specimen.

\[
V_s = \frac{L_s}{T_s} \quad (7.1)
\]

\[
V_p = \frac{L_p}{T_p} \quad (7.2)
\]
Figure 7.31 Shape of Piezoceramic Bender Elements With and Without Applied Excitation Voltage (from Dyvik et al, 1985)
Figure 7.32 Series Connected Piezoceramic Bender Element (from Dyvik et al, 1985)

Figure 7.33 Parallel Connected Piezoceramic Bender Element (from Dyvik et al, 1985)
where, \( V_s, V_p \) are the shear wave and compression wave velocities, respectively

\[
L
\]

is the length of the specimen between bender elements

\[
T_s, T_p
\]

are the times of travel for the shear and compression waves, respectively

The shear modulus is determined as

\[
G = \rho (V_s)^2
\] (7.3)

where, \( G \) is the shear modulus

\( \rho \) is the mass density of the soil

Knowing the shear wave and compression wave velocities, the Poisson's ratio \( \nu \) is calculated using the relationship

\[
\nu = \left\{ \frac{0.50(V_s / V_p)^2 - 1}{(V_s / V_p)^2 - 1} \right\}
\] (7.4)

The Young's modulus is obtained using the relation

\[
E = 2(1 + \nu)G
\] (7.5)

7.4.3. Test Procedure and Data Reduction. The silt specimen was reconstituted in a conventional triaxial cell. The transmitter element was placed at the top of the specimen, while the receiver was at the bottom. The transmitter element was connected to a waveform generator, that produced the pulse. The signals received by the bender element at the bottom were recorded using a Hewlett Packard digital signal analyzer. A typical trace of the receiver bender signal is shown in Figure 7.34. The arrival of the S- and P- waves is seen clearly. The shear wave velocity and the compression wave velocity are then calculated as

7.4.4. Soil Model. A model used to describe the relationship between the modulus and strain for geotechnical materials (Nazarian et al, 1987) is shown in Figure 7.35. The model is based on numerous resonant column tests on granular samples. It can be seen that both the effect of normal strain and normal stress are considered in the model. Several important points can be deduced from Figure 7.35. First, as the octahedral normal stress increases in the soil, the material becomes stiffer (at constant strain). Secondly, there is a threshold strain level below which the soil behaves elastically. This threshold level is slightly above 0.001 percent. Third, an increase in strain above the threshold level results in a reduction in the value of the modulus of the material; hence,
Figure 7.34 Typical Trace of Receiver Bender Element
Figure 7.35  Nonlinear Model Used for Determining Equivalent Linear Moduli of Geotechnical Materials (from Nazarian et al, 1987)
nonlinear behavior.

7.4.5. **Results and Discussion.** The results of the tests on reconstituted silt specimens using bender elements are compared with the values of modulus obtained from the cyclic triaxial tests in Figure 7.36. There is a linear relationship between the logarithm of the confining pressure and the logarithm of the modulus. The slope of the best fit line is approximately 0.63.

Tests using the bender elements impose strains of less than 0.001 percent (Dyvik et al, 1985). In Figure 7.36, as the strain increases from 0.001 percent to 0.05 percent, there is approximately a drop of 20 percent in the value of the modulus. This is in good agreement with the soil model described in Section 7.4.4. There is one discrepancy in the results, however. At a confining pressure of 10 psi, the modulus obtained from triaxial tests, and tests using bender elements are very nearly equal. One explanation for this behavior is that friction in the system is more prominent at low confining pressures and moduli, where the loads are correspondingly smaller, than when dealing with higher loads encountered at higher confining pressures and higher moduli. The other possible explanation is that the Poisson's ratio could not be found very accurately at low confining pressures.
Figure 7.36  Comparison of Moduli from Cyclic Triaxial Tests and Tests Using Bender Elements
CHAPTER EIGHT

CLOSURE

8.1 SUMMARY

Tests were performed on soils from the proposed location of the low-level radioactive disposal site in Hudspeth County, Texas. Two soils, a clay and slightly cemented silt were tested. Intact clay specimens were tested, while specimens of uncemented silt were reconstituted at in-situ water content and density.

Cyclic triaxial tests were performed on specimens at in-situ water content, as well as on specimens that were saturated and consolidated before testing. These tests were utilized to study the effect of strain level, confining pressure, degree of cementation, and degree of saturation, on the modulus and damping ratio of the soils. Tests were also conducted to determine the threshold strains for the reconstituted silt specimens.

Static triaxial tests were performed on the reconstituted silt specimens at in-situ water content, and saturated and consolidated specimens. The tests were aimed at studying the effect of confining pressure, and strain rate on the stress-strain behavior of the soil.

8.2. CONCLUSIONS

Based on the several tests performed in the course of this laboratory investigation, the following are some of the conclusions that can be deduced:

(1) The results of all sets of repeatability tests conducted are consistent, indicating that the methods employed to prepare the specimens, and the test procedures followed are satisfactory.

(2) In most cases, the modulus is very well behaved, and shows little scatter, when plotted against the logarithm of the axial strain amplitude. The damping ratio however, shows a relatively high scatter. The scatter in the data reduces as the confining pressure increases.

(3) The threshold strains for the reconstituted silt specimens was of the order of 0.04 percent. Below the threshold strain, the
number of cycles of strain imposed on the specimen, did not affect the pore pressure build-up in the specimen.

(4) The rate of axial strain in static triaxial tests has little effect on the stress-strain behavior of the reconstituted silt specimens.

(5) The tests to determine the threshold strain were immune to friction, and hence could be considered to be more reliable than the other tests in the program.

(6) The moduli for reconstituted silt specimens at in-situ water content, determined using bender elements compare well with the values from cyclic triaxial tests. Therefore, the effect of friction is negligible at low total confining pressures and high loads.
REFERENCES

(1) Brettmann, T.T., 1989, "Geotechnical Engineering Properties of Soils at a Proposed Low-Level Radioactive Waste Disposal Site in Hudspeth County, Texas", submitted in partial fulfillment of the requirements for an M.S. degree, the Department of Civil Engineering, The University of Texas at Austin, 134p.


