Small and large strain monitoring of unsaturated soil behavior by means of multiaxial testing and shear wave propagation

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A Dissertation

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by

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Abstract

The deformation and strength behavior of dry and saturated soils is controlled by the effective stresses as defined by Terzaghi. However, Terzaghi’s definition of the effective stresses fails for unsaturated soils, as capillarity force influence is also important. The effects of capillarity forces in soil are evaluated by the concept of matrix suction. Several techniques are used to evaluate soil suction however their applications involve difficult calibrations and tedious methodology. Furthermore, suction is a microscopic property and it is influenced by interparticle soil attraction, which can change by sampling disturbance.

This research program evaluates the effect of suction on stiffness and strength of soils at small strain (at constant fabric) and large strain (with fabric changes) levels. The phenomena are studied using a modified oedometer cell and a multi-axial device with matric suction control that have been equipped with bender elements for shear-wave velocity measurements. The test program consists on testing dry and unsaturated specimens under different boundary conditions: \( K_0 \)-loading and multi-axial loading. To test the \( K_0 \)-loading condition, the soil is loaded in the oedometer cell while the bender-elements monitor the changes in state of stresses by evaluating the changes in the velocity of wave propagation. Similarly, triaxial compression and conventional triaxial compression tests, along with monitoring of shear wave velocities, are conducted on 10-cm side cubical specimens of reconstituted soil specimens to study the stress-strain behavior of an unsaturated soil over a range of degrees of suctions and stress paths and the effect they have on the propagation velocity of shear waves.

Results show the adequacy of methods and equipment used in this investigation to monitor the behavior of unsaturated soils under the application of a range of suctions and several stress paths. Experimental results are analyzed using simple, yet robust wave propagation models and geo-material behavior. Their interpretation bring a better understanding to low and large strain-stress behavior of near sub-surface soils. Results provide a stronger base for the development of models for the imaging of near-surface geo-materials using elastic wave-based imaging techniques and for better interpretation of geotechnical models of design.
Chapter 1
Introduction

The research program is motivated by the need to have a better understanding of the small strain behavior of unsaturated soils under both small and large strain conditions. Results can be applied to the interpretation of the elastic wave-based geophysical testing (i.e., travel time tomography and electrical resistivity tomography) and design of near surface geotechnical systems where the presence of unsaturated soil is the norm and not the exception. To evaluate the effect of suction in soils, two sets of experiments are developed: tests under $K_0$-condition (laterally constrained) and multi-axial state of stress. These tests are run using soils with different grain size distribution and moisture content.

1.1 Evaluation of Small Strain Stiffness under $K_0$-Conditions in Dry and Unsaturated Soils

A set of tests in a modified oedometer cell are designed to study the effect of matric suction and stress anisotropy under $K_0$ conditions on the state of effective stresses and hence on low strain stiffness of soils. The tests consist in monitoring the variation of shear wave velocities as the vertical stresses are varied in an oedometer cell. The bender elements are placed in such a way to collect data from shear waves polarized in the vertical and horizontal directions. This feature permits estimating the anisotropic state of stresses.

The experimental study involves running a series of tests on sand and silt under dry and partially saturated conditions. In all these tests, suction is independently monitored with a small tip tensiometer. The evaluation of the $K_0$ coefficients and stiffness measurement results using soils with different grain size distribution and moisture content complement this part of the study. The analysis of the data includes signal processing.

1.2 Evaluation of Small and Large Stiffness under Multi-axial State of Stress Conditions in Unsaturated Soil with Suction Control

To study the effect of stress and suction on the stiffness of soils several sets of tests on a multi-axial device (triaxial compression TC and conventional triaxial compression CTC tests) are conducted on remolded silty soil and sand specimens. The specimens are compacted inside the true triaxial device at constant initial water content and unit weight. During the different loading procedures, the stress versus deformation data and the shear wave velocity in two different polarization directions are collected.
These data permit the evaluation of both low strain and large strain behavior of unsaturated soils. The complete set of tests conducted are summarized in Table 1.1

| Table 1.1: Summary of tests conducted for sand and silt soil types |
|-------------------|-------------------|
| Material          | Clayey silt       | Silica sand      |
|                   | Specific surface  | Specific surface |
| Index properties  | Atterberg’s limits| Particle size distribution |
| Uniaxial testing  | Oedometer         | Oedometer        |
| Multiaxial testing| Triaxial compression | Triaxial compression |
|                   | Conventional triaxial compression | Conventional triaxial compression |
|                   | Hydrostatic compression | Hydrostatic compression |

1.3 Organization

A brief description of the chapters included in this dissertation follows. Chapter 2 presents fundamental concepts related to the mechanical behavior of unsaturated soils. It introduces the concept of effective stress for unsaturated soil and highlights the importance of the stress state variables in interpreting the mechanical behavior of unsaturated soils. It reviews the state of the art on the small strain behavior of soils and presents the techniques used in the determination of low strain parameters and elastic wave velocity.

Chapter 3 is dedicated to describe the uniaxial testing program under no lateral strain conditions and the modifications made to the equipment to measure velocities of shear waves. Detailed descriptions of the equipment used are included. The complete set of test results and the analysis of data are presented.

Chapter 4 describes the multiaxial testing program including the materials and methods employed. It portrays the difficulties encountered in this part of the investigation and the modification made to the true triaxial device to control and monitor the application of stress, suction, and the generation/reception of elastic waves. This chapter also presents the data and the analysis of the testing results from silty soil specimens.

Chapter 5 continues with the description of tests performed on true triaxial device. The results of tests performed on sand specimens are presented and discussed. It also includes discussion about the differences between results obtained from test conducted on sand and silt soils.
Chapter 6 is dedicated to the interpretation and discussion of results from both oedometer and muti-axial tests conducted on the silt and sand soil specimens. Simple physical/mechanical models are used in the evaluation of the data.

Chapter 7 presents conclusions of this investigation and recommendations for future work.

Suction is a fundamental parameter in the determination of the mechanical behavior of unsaturated soils. For example, the stability of natural slopes and the speed of wave propagation in shallow deposits are highly dependent on the capillarity effects (Brand 1981; Krahn et al. 1988; Anderson and Sitar 1995; Cho and Santamarina 2000). Suction may also be used to evaluate the effective stresses in unsaturated soils and to calculate heave deformations in expansive soils (Fredlund and Rahardjo 1993). The use of elastic wave propagation and more specifically, the velocity of shear waves, is used in this study to monitor the effect of suction and effective stress on the stiffness of soils at very small strains. The use of a true triaxial apparatus to monitor both the application of suctions and the velocity of S-waves makes this study unique in this area.
Chapter 2

Mechanical Behavior of Unsaturated Soils at Small and Large Strain

2.1 Introduction

Traditional geotechnical engineering studies the behavior of water or air saturated soils, that is, soils consisting of two phases: solid particles and water or solid particles and air. At a macroscopic level, the interaction between two phases is not complex. However, natural and man-made soil deposits, in most cases, are not completely saturated or completely dry. Shallow deposits are subjected to processes of evaporation from ground surface and evapo-transpiration from vegetation that creates an upward flux of moisture leaving the soil, situation particularly persistent in relatively dry surroundings. There is also a downward flux of water through the soil caused by infiltration of surface water. In unsaturated soils, both air and water fill the voids between solid particles. Unsaturated soils are thus three-phase materials, comprising soil solids, water, and air. A phase is identifiable in a medium when it has matter, distinctive properties, and a clear limit. In reality, none of the phases correspond exactly with this denomination since there may be some water adsorbed by soil particles, air dissolved into water, and water dissolved into gas state as water vapor. Besides, some researchers (Fredlund and Rahardjo 1993) recognize the air-water interface, the so-called contractile skin, as a fourth phase that acts as a membrane between the air and water phases. Most soils used as construction material are in unsaturated condition during construction and may remain in that condition during the working life of the soil structure.

The behavior of unsaturated soils is of importance in a diverse range of geotechnical and environmental civil engineering projects. Examples can be found in earth dams, transportation projects, such as road and railway embankments, and also in environmental projects, such as waste containment in landfill sites. Phenomena like capillarity, suction, and swelling/shrinking are of utmost importance in understanding the behavior of unsaturated soils. Furthermore, geotechnical engineers increasingly use elastic wave-based geophysical and non-destructive evaluation techniques to image and gather more information about the near subsurface. However, elastic wave speeds are directly dependant on the effective stresses and capillary forces that control the stiffness of soils. For all these reasons, understanding the low and high strain
behavior of unsaturated soils is essential in the advancement of research and practice of geotechnical engineering today.

Furthermore, the study of the behavior of particulate media at very small strain is important to evaluate deformation of soils and thus of structures at working loads. The value of the shear modulus at very small strains ($G_{\text{max}}$ or $G_0$) is considered a fundamental soil property to determine its stiffness. This Chapter reviews concepts related to the small strain parameters in soils and the current methods used to measure them in the laboratory. Consequently, notions of elastic wave propagation are introduced and the use of piezoelectric bender elements to generate and monitor elastic waves is covered in detail. Results from previous research on the use of bender elements to measure the velocity of propagation of shear waves are included.

### 2.2 Effective Stress Concept in Unsaturated Soils

The principle of effective stress proposed by Terzaghi (Terzaghi and Peck 1958; Scott 1963; Ladd and de Boer 1997) is intended to describe the mechanical behavior of saturated soil. The derivation of the effective stress concept is based on force equilibrium in the system composed of particles, liquid and gas phases (Figure 2.1). The equilibrium of normal forces along the cross sectional plane is:

\[
P = p_s A_s + u_w A_w + u_a (A - A_s - A_w)
\]  

\[P = p_s A_s + u_w A_w + u_a (A - A_s - A_w)
\]

\[
(2.1)
\]

Figure 2.1: The concept of effective stress: force equilibrium between phases.

where $P$ is the normal force to the cross sectional, $p_s$ is the solid-solid contact pressure, $u_w$ is the pore water pressure, $u_a$ is the pore air pressure, $A$ is the gross area, $A_s$ is the solid-solid contact area, and $A_w$ is the water contact area. Dividing both sides of Equation 2.1 by the gross area $A$:  

$$
P = p_s A_s + u_w A_w + u_a (A - A_s - A_w)
$$  

Figure 2.1: The concept of effective stress: force equilibrium between phases.
the equation for total stress $\sigma$ is obtained, where $a$ and $\chi$ are the solid-solid contact and water contact area ratios. It is important to note that strictly speaking, the parameter $\chi$ should indicate not only the ratio of fluid over gross area ratio (and dependant in the degree of saturation) but also the magnitude of the surface tension $T_s$. Now, if the soil is fully saturated, $1-a-\chi=0$ and Equation 2.3 simplifies to:

$$\sigma = p_a a + u_w \cdot (1-a-\chi) = p_a a + u_w \cdot (1-a) + (u_a - u_w)(1 - a - \chi)$$

(2.4)

furthermore, the area ratio “$a$” diminishes as the solid-to-solid contact area becomes very small with respect to the whole cross sectional area at low stress levels. However, as the ratio “$a$” approaches null, $p_s$ increases and the first term in Equation 2.4 does not disappear. Then, defining $\sigma’ = p_a a$ as the effective, Equation 2.4 may be rewritten as:

$$\sigma’ = \sigma - u_w$$

(2.5)

effective stress equation for saturated soils

The effective stress $\sigma’$ controls the stiffness and strength of saturated particulate media and its application is very important in geotechnical engineering design. However as indicated in Equations 2.1 to 2.3, the presence of three phases (solid particles, water and air) changes the equilibrium equation, as the relative pressure of the air and water phases also contribute to the behavior. Therefore, to determine the controlling deformation and strength parameter in soils, a modified effective stresses equation for unsaturated is sought. This equivalent effective stresses equation for unsaturated soils requires the use of two independent stress variables, which may be obtained combining total stress, pore water pressure, and pore air pressure. The pair of stress variables most commonly used is formed by the net normal stress ($\sigma-u_a$) and matric suction ($u_a-u_w$ -Fredlund and Rahardjo 1993). It is clear that matric suction must be one of the variables
because it increases the existing forces at interparticle contact points due to the presence of capillarity forces. The other variable, the net normal stress, uses the air pressure as a reference which is almost constant if taken as the atmospheric pressure. Therefore, rearranging Equation 2.3 and assuming that ratio \( a \) is close to zero, yields:

\[
\sigma' = \left(\sigma - u_a\right) + \chi \left(u_a - u_w\right) \quad \text{(unsaturated particulate media)} \tag{2.6}
\]

where \( \chi \) is zero for dry condition and one for full saturation. It is important to state that the effects of matric suction and net stresses induced by external loads at particle contact points are uncoupled, thus, the two stress variables must be independent (Vinale et al. 2001; Cho and Santamarina 2001). Although the Bishop’s single tensor equation shown in Equation 2.6 is commonly used and it combines the effects of menisci water pressure and total stress, it has several limitations because it mixes local and global conditions within the medium (Fredlund and Rahardjo 1993). To avoid this problem, it is better to present stress-strain results in terms of the two state variables: net pressure \( \sigma - u_a \) and suction \( u_a - u_w \). These two stress variables separate the contribution of external and internal stresses. This scheme facilitates the study and description of stress paths. Besides, Equation 2.6 applies only to pure water as the presence of soluble adds another term to the suction, the osmotic suction (Fredlund and Rahardjo 1993; Mitchell 1993). Osmotic suction \( \pi \) is also important in soil systems (Mitchell 1993; Tindall and Kunkel 1999):

\[
\pi = kT\Delta c \tag{2.7}
\]

where \( k \) is the Boltzmann’s constant, \( T \) is the absolute temperature and \( \Delta c \) is the chemical concentration difference across a semipermeable membrane. This phenomenon will not be considered in this research study.

Matric suction \( \Delta u = u_a - u_w \), may also be expressed using the Laplace’s equation. This formulation includes the effect of surface tension \( T_s \):

\[
\Delta u = u_a - u_w = T_s \left(\frac{1}{r_1} + \frac{1}{r_2}\right) \quad \text{(matric suction)} \tag{2.8}
\]
Figure 2.2: Definition of radii in the Laplace’s equation.

where and \( r_1 \) and \( r_2 \) are the radii of the water menisci (see Figure 2.2). Cho and Santamarina (2001) proposed micromechanics-based equivalent effective stress equations. These equations show the contribution of the capillarity forces to the contact forces. For simple particle arrangements of single size spherical particles, the equivalent effective stresses are:

\[
\sigma'_{eq} = \frac{\pi T_s}{4R} \left[ 2 - \left( \frac{8}{9} G_s w \right)^{\frac{1}{2}} \right] \quad \text{simple cubic packing} \tag{2.9}
\]

\[
\sigma'_{eq} = 2\sqrt{2} \frac{\pi T_s}{4R} \left[ 2 - \left( \frac{8}{9} G_s w \right)^{\frac{1}{2}} \right] \quad \text{tetrahedral packing} \tag{2.10}
\]

where \( R \) is the radius of the particles, \( G_s \) is the specific gravity, and \( w \) is the water content. Equations 2.9 and 2.10 may be combined with Hertz’ theory (Richart et al. 1970) to find the equivalent combined effective stress that considers both contact and capillarity forces. There is no exact solution for the equation for the equivalent combined effective stress, as the menisci radii change with the contact area between particles. A solution for the combined effect of capillary and Hertz’ contact forces can be found using an iterative solution. This combined equivalent stress shows the effect of the capillary forces is important at the low applied stresses and its effect becomes negligible as the applied stresses increase (Figure 2.3 - Cho and Santamarina 2001).
Another important implication is that given a deformation, the menisci may be disrupted and its re-formation depends on the degree of saturation and the re-formation is not immediate. Several researchers have shown this problem during testing. For example, Cho and Santamarina (2001) show the disruption and generation of capillary forces by measuring shear wave velocity (Figure 2.4) and Macari and Hoyos (2001) present the same phenomena as function of loading rate (Figure 2.5). These studies indicate that both loading and deformation during testing must be carefully controlled to avoid the disruption of capillary forces between particles.

**Figure 2.3:** Effect of equivalent effective stress versus boundary stresses (after Cho and Santamarina 2001).

**Figure 2.4:** Effect of loading in the disruption of menisci in unsaturated particulate media. The shear wave velocity recovers during the re-formation of the menisci. The higher the saturation the faster is the stiffness recovery (after Cho and Santamarina 2001).
2.3 Stress-Strain Behavior of Unsaturated Soils

Extensive research has been performed in field and laboratory measurements of soil suction, analysis of shrink-swell-collapse behavior, and assessment of soil-water characteristic curve (SWCC) for soil drainage aspects. Very few studies have been focused on stress-strain-strength behavior of unsaturated soils via suction-controlled testing.

Previous research have suggested that two independent sets of stress state variables, suction and net stress (Equation 2.6), may be used to describe the shear strength behavior and the volume change properties of an unsaturated soil, eliminating the need to find a single-valued effective stress equation that is applicable to both shear strength and volume change problems (see Equations 2.1 to 2.6).

The effective stress determines the mechanical behavior of unsaturated soils although it requires the following assumptions (Bishop and Eldin 1950; Skempton 1960):

- Soil particles are incompressible,
- Confining pressure is independent of the stress controlling the area of contact and shear strength between soil particles.

For the typical near-subsurface stresses, these assumptions can be considered valid. Although the effective stress principle is not a physical law, it has been a tool of incalculable value in the study of saturated soils. Since the 1950’s several scientists have tried to extend the

![Figure 2.5: Effect of loading rate in the disruption of menisci in unsaturated soil. The soil was loaded (after Macari and Hoyos 2000).](image)
effective stress principle of unsaturated soils. The major efforts are aimed at finding an adequate expression for the effective stress equation. Jennings and Burland (1962) present results of oedometer tests and isotropic compression conducted on unsaturated soils of different particle size ranging from sand to clays. The results show that for values under a certain degree of saturation, the Terzaghi’s principle of effective stress fail to predict the behavior of the soils. Aitchison and Bishop (1960) propose Equation 1.6 for the effective stress:

\[ \sigma' = \sigma - u_a + \chi(u_a - u_w) = \sigma - \left(\chi u_w + (1 - \chi)u_a\right) \]  

(2.11)

As previously indicated, if the soil is saturated, \( \chi \) equals unity; if the soil is dry then \( \chi \) becomes null. Bishop and Blight (1963) analyzed four series of tests (Figure 2.6) trying to determine whether a unique relation exists between \( \chi \) and the degree of saturation \( S_r \). They conclude that the behavior of unsaturated soils can be studied only if the stress path and stress state are referred to two stress components (\( \sigma - u_a \)) and (\( \sigma - u_w \)). The relation between shear strength and effective stress seem to depend little on the stress path but the values of \( \chi \) are not unique. Jennings and Burland (1962) compared a theoretical model made by Bishop and Donald (1961) and test results from five different soils (Figure 2.7). They found that, for the same soil, during a collapse process the value of \( \chi \) is positive, whereas during an expansion process it is negative.

![Figure 2.6: Measured values of parameter \( \chi \) in four compacted soils (Bishop and Blight 1963).](image-url)
Aitchison (1965) identified three possible sources of error when using a single-valued effective stress (for example Equation 2.6):

- Air pressure should be present as an independent variable,
- Changes on interstitial pressure are caused not just by increasing applied stresses, and
- The effective stress principle can not be applied to certain soils with complex behavior such as cemented soils

The author also presented curves of volumetric changes against the independent stress variables ($\sigma - u_a$) and ($u_a - u_w$), as shown in Figure 2.8.

**Figure 2.7.** Relation between parameter $\chi$ and degree of saturation for several soils (Jennings and Burland 1962).

**Figure 2.8:** Response of axial strain with effective stress for difference values of suction (Aitchison 1965).
Therefore, the use of a single variable of stress state should be avoided. The suction should be included as a second variable since it increases the existent stresses between solid particles due to capillarity effect. However, the effect of suction on the deformation and strength of soils cannot be reproduced just by adding stress between particles because, for instance, collapse or expansion phenomena cannot be explained using that simple approach. In summary, it is necessary to use two independent stress state variables to describe the behavior of unsaturated soils.

Several pairs of stress state variables have been proposed: Coleman (1962), Bishop and Blight (1963), Matyas and Radhakrishna (1968), Barden et al. (1969), Aitchinson and Woodborn (1969), Fredlund and Mogerstern (1977), being (σ-ua) and (ua-uw) the pair of stress variables most frequently referred. Fredlund and Mogerstern (1977) proposed three possible pairs of stress state variables (Table 2.1). Pairs A and B separate external from internal stresses, which facilitates the description and study of stress paths. Pair A uses air pressure as the reference, which is usually the atmospheric pressure and thus is almost constant. Throughout this study, the pair formed by (σ-ua) and (ua-uw) is used to describe the state of stress of unsaturated soil.

<table>
<thead>
<tr>
<th>Pair</th>
<th>First variable</th>
<th>Second variable</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>σ-ua</td>
<td>ua-uw</td>
</tr>
<tr>
<td>B</td>
<td>σ-uw</td>
<td>ua-uw</td>
</tr>
<tr>
<td>C</td>
<td>σ-ua</td>
<td>σ-uw</td>
</tr>
</tbody>
</table>

The pair of stress state variables formed by (σ-ua) and (ua-uw) has been used to formulate constitutive models describing the strength and deformation of unsaturated soils. For instance, Muraleetharan and Wei (2000) develop a set of governing equations for unsaturated porous media based on the Theory of Mixtures with Interfaces (TMI) that explicitly considers the interfacial effects and provides a theoretical basis for the use of two independent stress variables.

In general, stress tensors are needed to describe the three dimensional state of stress in soils:
\[
\sigma - u_a = \begin{bmatrix}
\sigma_x - u_a & \tau_{yx} & \tau_{zx} \\
\tau_{xy} & \sigma_y - u_a & \tau_{zy} \\
\tau_{xz} & \tau_{yz} & \sigma_z - u_a \\
\end{bmatrix}
\] (2.12)

\[
u_a - u_w = \begin{bmatrix}
u_a - u_w & 0 & 0 \\
0 & u_a - u_w & 0 \\
0 & 0 & u_a - u_w \\
\end{bmatrix}
\] (2.13)

Certain boundary conditions exist, concretely; the value of the components of the state of stress is limited by the following inequality:

\[
\sigma \geq u_a \geq u_w
\] (2.14)

That is, if the air pressure exceeded the total pressure, the solid particles would not touch each other and the soil, at the extreme, would explode. In fact, this restriction is always used in the pressure plate apparatus and the specimen can be considered surrounded by a flexible membrane, so the air pressure acts as the total pressure. The water pressure can rise until it equates the air pressure. In this case, the matric suction is zero and the soil is fully saturated. Additionally, the water pressure can not exceed the external pressure. This case is studied in conventional Soil Mechanics and is called piping.

### 2.4 Experimental Determination of the Stress State Variables

To determine experimentally the validity of the stress variables \((\sigma - u_a)\) and \((u_a - u_w)\), it is necessary to monitor the behavior of soil specimens over a range of these variables. This requires the application of high values of matric suction. In order to apply high suction values, and considering the air (atmospheric) pressure constant, it is necessary to decrease the pore water pressure. If during a test the water pressure becomes lower than -101.3 kPa (-1 atm), cavitation would occur with consequent formation of air bubbles inside the measuring system. The axis-translation technique permits avoiding this undesired phenomenon. This laboratory technique involves increasing the air pressure and the water pressure in the same amount, keeping in this way the value of matric suction. Using this technique, it is possible to obtain any suction value, as long as the laboratory equipment used supports it.
The axis-translation technique permits the evaluation of the stress state variables. For instance, Hilf (1956) presents results from several specimens tested in a pressure plate apparatus. Each specimen is submitted to an increase in air pressure and it is observed that the interstitial pressure augments automatically in the same amount and the specimen behavior experience no change (see Figure 2.9). They conclude that as long as the suction is maintained constant, the specimen do not suffer any change, even though the individual suction components are changed. Therefore, suction is an adequate state of stress variable.

**Figure 2.9.** Determination of matric suction using the axis-translation technique (Hilf 1956).

Bishop and Blight (1963) made several undrained compression tests with Selset clay. These results are presented in Figure 2.10. The air pressure is increased in several steps separated by certain time intervals to allow the air pressure to be homogeneous in the specimen. The water pressure is affected on the same amount by the changes in air pressure. Furthermore, they observe that the air pressure applied does not affect the strength.
Fredlund and Morgenstern (1977) propose the following criterion to verify experimentally the validity of the selected stress state variables: a set of independent stress variables will not produce shear strain or volumetric strain when the individual components of each stress variable, $\sigma$, $u_a$, $u_w$, are varied without altering the global valance on the stress variables. Using the null testing procedure and the axis-translation technique, they conducted nineteen tests on compacted kaolinite specimens. In the null test, the individual stress components are varied equally in order to maintain the stress state variables constant.

$$\Delta\sigma_x = \Delta\sigma_y = \Delta\sigma_z = \Delta u_a = \Delta u_w$$  \hspace{1cm} (2.15)

If the proposed stress state variables ($\sigma-u_a$) and ($u_a-u_w$) are valid, then the bulk volume, the water volume and, consequently, the degree of saturation of the specimen should remain constant throughout the null test. The results show no appreciable change in the overall volume and water volume of the specimen, verifying the suitability of the stress state variables ($\sigma-u_a$) and ($u_a-u_w$) to describe the mechanical behavior of unsaturated soils.
2.5 Volume Changes in Unsaturated Soils

Biot (1941) publishes a general theory of consolidation for soils containing air bubbles. This theory assumes that the soil behaves isotropically and linear- elastically. Two equations are proposed to relate stresses and strains for the soil skeleton and the water phase using four volumetric strain coefficients. Bishop (1959) and Jennings and Burland (1962) try to model the behavior of an unsaturated soil with a single-valued effective stress equation. Figure 2.11 shows typical results and it indicates that no unique relationship exists between volume change and effective stress for the majority of unsaturated soils, especially under a critical value of degree of saturation. This critical saturation value depends mostly on particle size distribution, varying from 20% for sands, 40-50% for silts silt, up to 85% for clays. For example, silt specimens with saturation degrees above the proposed value of 40-50% do not collapse, whereas those specimens with degrees of saturation below the proposed value do. In other words, specimens below a certain degree of saturation do not meet the effective stress principle since they suffer deformations even though the effective stresses are maintained constant.

Figure 2.11: Isotropic compression test results for silt specimens under different applied stresses (Jennings and Burland 1962).
Coleman (1962) proposed three stress-strain relations for an element of unsaturated soil under axisymmetric condition. The relations include nine coefficients of compressibility which depend exclusively on the current values of net mean stress ($\sigma_m-u_a$), shear stress ($\sigma_1-\sigma_3$), and suction ($u_a-u_w$), as well as on the stress history of the soil. Coleman (1962) suggested the constitutive Equation 2.16 for the volume change associated with the water phase, Equation 2.17 for the volume change associated with the soil skeleton, and Equation 2.18 describes the change in shear strain.

$$-\frac{dV_w}{V} = C_{11}(du_a - du_w) + C_{12}(d\sigma_m - du_a) + C_{13}(d\sigma_1 - d\sigma_3)$$  \hspace{1cm} (2.16)

$$-\frac{dV_w}{V} = C_{21}(du_a - du_w) + C_{22}(d\sigma_m - du_a) + C_{23}(d\sigma_1 - d\sigma_3)$$  \hspace{1cm} (2.17)

$$-(d\varepsilon_1 - d\varepsilon_3) = C_{31}(du_a - du_w) + C_{32}(d\sigma_m - u_a) + C_{33}(d\sigma_1 - d\sigma_3)$$  \hspace{1cm} (2.18)

Furthermore, Blight (1965) indicates that the stress-strain paths should be represented in three dimensions, using the axis for ($\sigma$-$u_a$) and ($u_a$-$u_w$) independently with respect to the volumetric strain (Figure 2.12).

Figure 2.12: Volume changes plotted on void ratio, ($\sigma$-$u_a$), and ($u_a$-$u_w$) space (Blight 1965).
Burland (1965) supports the same idea and represented the void ratio as function of logarithm of the hydrostatic pressure \( \log(p) \) and water pressure \(-u_w\) as in Figure 2.13. The stress variables \((\sigma-u_a)\) and \((u_a-u_w)\) are acknowledged as the best fitted to describe volume changes.

Matyas and Radhakrishna (1968) introduced the concept of state parameters for unsaturated soils: \(\sigma-u_a\), \(u_a-u_w\), \(e\), and \(S_r\). Results of tests conducted on specimens made with the same soil type, compaction energy, and initial water content are presented in three-dimensional \((e:\sigma-u_a:u_a-u_w)\) and \((S_r:\sigma-u_a:u_a-u_w)\) plots. The void ratio \(e\) is used to represent the deformation of the soil structure, and the degree of saturation for the deformation of the water phase.

The state surface presented in Figure 2.14a possesses the property of uniqueness, giving decreasing values of void ratio with increments in effective stress or declines of suction. The surface is therefore constitutive, as long as the applied stress path results in an increase on the degree of saturation. However, hysteresis observed on unsaturated soils on wetting and drying cycles, avoids the uniqueness property in other cases. Uniqueness is not seen in the water phase of Figure 2.14b, the reason being that they do not obtain full saturation at zero suction.
Barden et al. (1969) conduct tests with no lateral strain on specimens of unsaturated soil trying to explain the volumetric behavior at different stress paths shown in Figure 2.15. They conclude that the volumetric behavior depends on the stress path followed, either at wetting or drying stage. Therefore, the independent stress state variables should be included when studying the volume strain behavior of unsaturated soil.

Figure 2.14: State parameter surfaces as presented by Matyas and Radhakrishna (1968).

Figure 2.15: Volumetric deformation for different stress path in an unsaturated soils (Barden et al. 1969).
Lloret and Alonso (1985) published linear and non-linear equations that describe constitutive surfaces of unsaturated soils loaded isotropically and under $K_o$ conditions.

![Figure 2.16: Linear and non-linear constitutive surfaces as presented by Lloret and Alonso (1985).](image)

Using optimization techniques the authors obtained the best fitting functions of previously published data seen in Figure 2.16, which shows the constitutive surfaces in terms of void ratio $e$ and degree of saturation $S_r$.

### 2.6 Swell/Shrink Behavior

The swell and shrink of fine-grained soils depend on the soil water content which is influenced by weather conditions. Swelling occurs when water infiltrates between and within the clay particles, causing them to separate. Shrinkage occurs when clays dry due to evaporation and diffusion of the pore water. The particular chemical composition and crystalline structure of the clay minerals are responsible for the swelling mechanisms (Fredlund and Rahardjo 1993; Mitchell 1993; Santamarina et al. 2001). The volume changes that occur in these types of clays result in the problems of cracking and moving of building foundations, highway pavements, and other light structures. Figure 2.17 shows the effect of drying on an initially saturated Kaolinite clay specimen.

There are a number of procedures used to quantify swelling potential of clays. These methods help in determining the degree of expansiveness, and ultimately estimating the in-situ heave (O’Neill and Poormoayed 1980).
The measurement systems for expansive potential of fine-grained soils can be direct or indirect. Direct methods consist of laboratory swelling tests, indirect methods are based on physical-chemical properties and mineralogical composition of soils, and there are also combined methods that use results of both direct and indirect examination (Fredlund and Rahardjo 1993). Direct laboratory methods are labor and time consuming and results from various methods differ considerably (Sridharan et al. 1986; Abduljauwad and Sulaimani 1993; O’Neill and Poormoayed 1980). Alternatively, several indirect methods to estimate the potential swell make use of simple soil parameters such as water content, Atterberg’s limits, vertical pressures, percent of fines, etc. These methods relate empirically the soil properties to its expansiveness. Seed et al. (1962) evaluated the usefulness of the plasticity index as a single factor for predicting expansion potential. The authors show that this parameter alone can provide an assessment of swelling potential that is probably correct to within ±35 % and is a useful indicator of swelling characteristics. Neither the clay content nor the shrinkage limit is found suitable for this purpose in their study. For this reason, plasticity index and liquid limit are commonly considered more reliable parameters. However other authors have found them useful. In general, water content and Atterberg’ limits have been used consistently as indicators of swell potential (Snethen 1984; Sowers 1970; Vijayvergiya and Gazzaly 1973).

Although one may agree that the change in moisture content is the single most important factor affecting the swelling and shrinking of clay, it does not give the complete picture. O’Neill and Poormoayed (1980) and McKeen (1992) stated that soil suction is a more fundamental

![Figure 2.17: Shrinkage on a Kaolinite specimen. (a) Initially saturated specimen. (b) Shrinkage after one week of air-drying. (c) Shrinkage after three weeks of air-drying.](image)
measure and a more sensitive indicator than water content and index properties to quantify swelling or shrinkage potential. Furthermore, Mou and Chu (1981) concluded that the use of different compaction methods (static or kneading compaction) for specimen preparation results in a different soil structure or fabric of the compacted specimens. Although both specimens might have the same water content, the difference in soil fabric is reflected in the measured soil suctions and the percentage of swell. Therefore, they concluded that any difference in the soil fabric of expansive clay formations may be a significant factor that affects the swelling characteristics of the clay formations, and thus, the measurement of soil suction would provide helpful information in the investigation of the volume-change behavior of expansive clays. Furthermore, suction is influenced by interparticle soil attraction, which can change by sampling disturbance (O’Neill and Poormoayed 1980). Finally, Mitchell (1993) and Garbulewsky et al. (1994) maintained that swelling and suction phenomena can be explained through mechanisms influenced by the same factors such as the diffuse double layer, the concentration of electrolytes, the type of electrolyte ions, and the contact type of clay particles.

Although using soil water content as a major variable in the evaluation of swelling is a convenient and practical approach, it is very useful to include soil suction as an additional variable. Moreover, soil suction affects water flow, water storage, volume change, and shear strength (Shuai and Fredlund 2000). Soil suction is a microscopical property that indicates the intensity with which the soil will attract water (see for example Cokca and Birand 2000). Soil suction is defined quantitatively as the difference between the pore air pressure, u_a, and the soil pore water pressure, u_w.

More fundamentally, the swelling/shrinkage deformation is related to the state of effective stresses and suction by:

\[
\Delta h = \frac{C_s}{1 + e_0} h \log \left( \frac{\sigma_v + \Delta \sigma_v - u_{wf}}{(\sigma_v - u_a) + (u_a - u_w)_e} \right)
\]

where \(\Delta h\) is the swelling/shrinkage deformation, \(C_s\) is the swelling index, \(\sigma_v\) is the vertical total stress, \(\Delta \sigma_v\) is the change in the total vertical stresses, \(u_{wf}\) is the final pore water pressure, and \((u_a - u_w)_e\) is the matric suction equivalent (Fredlund and Rahardjo 1993).
2.7 Measurement of Suction

Soil suction is a key parameter for determining the deformation and shear strength behavior of partially saturated soils, thus reliable techniques are needed for the measurement of suction. Several techniques are available including direct (pressure plates, pressure membranes, and tensiometers) and indirect methods (filter paper, porous blocks, and heat dissipation sensors) as classified by Zapata et al. (2000). Determination of soil suction, however, is not exempt from drawbacks. All the instruments used in these methods need to be calibrated before making any suction measurement. The accuracy of the soil suction measurements therefore depends on these calibrations.

2.8 Small Strain Parameter of Soils

The measurement of soil stiffness at small strains assumes greater importance in the study of soil mechanics and its applications to geotechnical design (Matthews et al. 2000; Stokoe and Santamarina 2000). Furthermore, small-strain behavior of soils is paramount in predicting performance of earth structures during construction and subsequent working stages (Brand 1981; Johnson and Sitar 1990; Macari and Hoyos 2000; Vinale et al. 2001).

As the small strain parameters are dependant on the contact forces, they have been successfully used as indicators of the behavior of unsaturated and saturated soils. In unsaturated soils, capillarity forces may control the stiffness and strength of soil at low confinement level (Santamarina 2001). The parameter that controls the propagation of elastic waves also controls the small strain behavior of soils, i.e., stiffness (Richart et al. 1970). For this reason elastic wave propagation is an important tool in evaluating the behavior of soils and they are used in this investigation to monitor deformation and anisotropic loading process in unsaturated soils.

The typical variation of shear or bulk stiffness with strain for most soils is given in Figure 2.18. The curve depicts non-linear soil stiffness from very small strains to pre-failure conditions. It is known that the strain-dependent curve depends mainly on soil plasticity in fine soils (Vucetic and Dobry 1991) and is affected by the mean effective stress in coarse soils (Ishibashi and Zhang 1993). Furthermore, it is believed that most soils behave elastically at very small strains (i.e., strain smaller than 0.001%) giving rise to a constant stiffness. The strain induced by the propagation of seismic waves is within this range and hence provides a measure of the upper bound for stiffness \( (G_{\text{max}}) \). The upper bound stiffness is clearly a fundamental parameter in
defining this curve and hence the use of seismic measurements of stiffness is becoming more relevant.

2.9 Elastic Wave Propagation

In an infinite medium, two fundamental modes of seismic wave propagation exist: compression waves and shear waves (Kolsky 1963; White 1981; Achenbach 1991). The particle motion generated by these two types of seismic waves differs (Figure 2.19). In compression waves, also called primary waves (P-waves), the particle motion is held on a plane (polarized) parallel to the direction of propagation, whereas in shear waves, known as secondary waves (S-waves), the particle motion is perpendicular to the direction of propagation.

![Monotonic Loading Curve](image)

**Figure 2.18:** Typical shear modulus degradation for most soils.

**Figure 2.19:** Particle motion on (a) P-waves and (b) S-waves (Kramer 1995).
The use of elastic waves permits monitoring the elastic (stiffness) and inertial (density) properties of soils. More specifically, elastic waves allow monitoring the state of effective stresses by checking shear waves velocity polarized in different directions (Roessler 1979). This is inferred because the stiffness of soils is controlled by the effective stresses.

From the two types of body waves generated from an energy source, the one of most interest in this study is the shear wave. Since the pore fluid does not have shear stiffness, only the soil skeleton propagates S-waves. As a result, S-wave velocities can be related more easily to shear moduli and other properties used in engineering calculations (U.S. Army Corps of Engineers, 1995; Santamarina et al. 2003). On contrast, P-waves can be propagated also by the pore fluid. Therefore, in saturated, unconsolidated materials, P-wave velocities are often controlled by the bulk stiffness of water. Furthermore, the stress anisotropy may also be monitored by changing the polarization of the S-waves in two different directions.

The approach used in the interpretation of wave propagation is based on the fact that the state of effective stress imposed on a soil impacts the velocity of shear waves propagating through it. The generalized relationship between P-wave velocity and effective isotropic stress $\sigma'$ in a simple cubic packing is (White 1983):

$$ V_p = \left[ \frac{3 \cdot E_s^2}{8(1 - v_s^2)} \right]^{\frac{\nu}{2}} \left[ \frac{6}{\pi \cdot \rho_s} \right]^{\frac{\nu}{2}} \sigma'^{\frac{\nu}{2}} \quad (2.20) $$

where $E_s$, $v_s$, $\rho_s$ are the Young’s modulus, Poisson’s ratio, and mass density of the spherical particles. For S-wave velocity, the relationship is (White 1983):

$$ V_s = \left[ 3 \cdot (1 - v_s^2) \right] E_s \left[ \frac{3}{(2 - v_s)(1 + v_s) \pi \cdot \rho_s} \right]^{\frac{\nu}{2}} \sigma'^{\frac{\nu}{2}} \quad (2.21) $$

Calling $\theta = \left[ 3 \cdot (1 - v_s^2) \right] E_s \left[ \frac{3}{(2 - v_s)(1 + v_s) \pi \cdot \rho_s} \right]^{\frac{\nu}{2}}$ and the exponent of the effective stress is $\beta$, Equation 2.21 reduces to:

$$ V_s = \theta \sigma'^{\beta} \quad (2.22) $$

26
where $\theta$ varies with the type of packing (porosity and coordination number), fabric changes, and contact behavior. The exponent $b$ takes into account the contact effects (ideal soil, cemented soil, Hertzian contacts, cone to plane contact, spherical particles with yield, Coulombian forces).

Roessler (1979) and Stokoe et al. (1991) present a similar expression that considers the effective stress only in the plane of S-wave polarization. The predictive semi-empirical equations for shear-wave velocities are expressed as:

$$V_s = \theta \left( \frac{\sigma'_{h} + \sigma'_{v}}{2\sigma_{ref}} \right)^b$$

valid for saturated or dry soils

The relationship presented in Equation 2.23 may be further extended to partially saturated soils as (Cho and Santamarina 2000; Santamarina et al. 2001):

$$V_s \approx V_{S(\text{for } S_r=1.0)} \left[ 1 + \frac{(u_a - u_w)}{0.75 \sigma'_{v}} S_r \right]^b$$

(2.24)

where $S_r$ is the the degree of saturation and $V_{S(\text{for } S_r=1.0)}$ is the shear wave velocity at full saturation. Equation 2.24 shows the interplay between competing effects, as the degree of saturation decreases, the suction in the soil increases.

**Figure 2.20:** Relationship $\beta$-exponent and $\theta$-factor between different types of soils (Santamarina et al. 2001).
2.10 Elastic Wave Measurement Techniques

Estimations of stiffness are traditionally made in a triaxial apparatus using small deformation and displacement transducers. However these techniques are not precise enough and cannot be used at very low strain levels. Dynamic methods for the measurement of soil stiffness at very small strains using resonant columns and, more recent, piezo-ceramic plates (bender elements) are used to provide better quality measurements of very low strain levels (Shibuya et al. 1997; Fiorovante and Capoferri 2001).

The evaluation of the shear wave velocity at small strains in the laboratory is typically performed under isotropic confinement using a resonant column device. But as Thomann and Hryciw (1990) claim, “in situ soils are generally under a condition of no lateral strain during vertical loading…therefore the vertical and horizontal stresses may be quite different“.

Santamarina and Cascante (1996) present a modified resonant column device that allows the application of deviatoric stresses into the soil specimen and then measurement of the shear wave velocity under anistropic conditions (Figure 2.21). Fratta and Santamarina (1996) and Santamarina and Fratta (2002) presented techniques to measure first mode and wide band wave propagation parameters in both rocks and jointed rock masses (Figure 2.22).

**Figure 2.21:** Modified resonant column device: (a) Testing setup. (b) Typical test result on clean sand: both isotropic and anisotropic loading (after Santamarina and Cascante 1996).
Another commonly used technique uses bender elements to send and receive S-waves in soils. Piezo-ceramic elements distort or bend when subjected to a change in voltage (and generate a voltage when bent). Two such elements placed opposite one another provide a particularly convenient design forming a bimorph. These bimorphs consist of two transverse expander plates bonded together so that a voltage applied to the electrodes causes the plates to deform in different directions (one contracts in longitude while the other expands). This opposition causes the element to bend. Conversely, mechanical bending of the element causes it to develop a voltage between the electrodes.

Bender elements may be assembled to operate in either series or parallel (Figure 2.23). The series bimorphs develop twice the voltage as the parallel, but provide only half the displacement for the same applied voltage. Accordingly, a suitable setting should use a parallel bender element as the source and a series bimorph as the receiver.

Mounted as cantilever beams, these bimorphs or bender elements are inserted a small distance into a soil sample for the generation of elastic disturbance into the soil and for the reception of the elastic disturbance coming from the soil. The voltage in one element is varied
which makes the bender element to vibrate. This vibration generates mechanical waves that propagate through the soil sample and are received by the opposite element (Figure 2.24), which converts the motion to an electrical signal. The input voltage (created using a function generator) and the received signal are monitored using a digital oscilloscope, allowing the travel time to be determined using Equation 2.25.

\[ \nu = \frac{L}{t} \]  

where \( \nu \) is the wave velocity, \( L \) is the distance between the tips of source and receiver bender elements, and \( t \) is the travel time. The dynamic elastic shear modulus \( G \) can then be determined as
\[ G = \frac{V_s}{\rho} \]  

(2.26)

where \( \rho \) is the soil mass density, and \( V_s \) is the shear wave velocity in the continuum.

Although measurements of small-strain shear wave velocities on soils using piezoelectric bender elements for determination of soil stiffness is feasible, the convenience of bender element tests is limited by subjectivity associated with identifying wave travel time and uncertainties surrounding the validity of some interpretation methods. Several studies have been performed aimed to improve understanding of the results from dynamic testing of soils using bender elements. See for example Arulnathan et al. 1998 and Lo Presti et al. 2001. Additional doubts exist regarding the influence of transducer support conditions on the characteristics of transmitted waves and the importance of reflected components on received waveforms (Dyvik and Madshus 1985; Brignoli et al. 1996; Viggiani and Atkinson 1995a, 1995b; Nakagawa et al. 1996).

The received signals can be distorted by near field effects, cross-talking, multiple reflections, etcetera. Different methods to determine the travel times of elastic waves from piezoceramic bender elements for measuring the shear wave velocity of laboratory soil specimens have been proposed which are classified into two categories: time domain techniques and frequency domain techniques (Dyvik and Madshus 1985; Brignoli et al. 1996; Ferreira 2003; Kawaguchi 2003).

Currently there is no agreement on which method most closely estimates the true small strain stiffness of a soil. Ferreira (2003) indicates that time domain techniques seem to overestimate shear wave velocity and Go. In contrast, Jovicic (2003) claims that the measurement should be taken in the time domain preferably directly from the screen because automatic interpretation of arrival times using numerical processing (frequency domain) usually does not consider changes in boundary conditions during the course of a test.

In summary, there is no method to determine travel time that is appropriate for every case essentially because their use depends on particular testing conditions.
2.11 Previous Work on Elastic Wave Measurement Using Bender Elements

2.11.1 Use of Bender Elements on Saturated Soils.

Thomann and Hryciw (1990) work on the construction of a bender element-equipped oedometer to determine the shear wave velocity in cohesionless soils under a $K_o$ condition. They measure the lateral stress with a piston embedded in the sidewall of the oedometer and place piezoceramic bender elements in the top and bottom of the device to determine the shear wave velocity along the vertical direction. They presented a comparison of shear wave velocities between this apparatus and a resonant column with bender elements placed on it. Figure 2.25 shows that there is a good agreement between the measurements in the two apparatus. These authors claim that their results indicate that the device is capable of reliably measuring the lateral stresses and the shear wave velocity under $K_o$ conditions.

In another study using another modified oedometer, Fam and Santamarina (1995) discussed the results of the implementation of complementary elastic and electromagnetic wave measurements intended to study and monitor different processes in geomaterials. The authors stated that particulate geomaterials can be distinctively studied with wave-based techniques. They present typical measurements conducted during consolidation, chemical diffusion, and cementation. Figure 2.26 presents the change in shear wave velocity in a saturated kaolinite specimen during the consolidation process: as void ratio decreases, the shear of the material increases.

![Figure 2.25: Comparison of shear wave velocity from resonant column and bender element (Thomann and Hryciw 1990).](image-url)
Zeng and Ni (1999) evaluated the stress-induced anisotropy of elastic shear moduli of sand under a $K_o$ condition. They used bender elements to measure shear moduli of two types of sand in multiple stress paths under $K_o$ conditions. Based on their results, Zeng and Ni (1999) reported that the stress-induced anisotropy is controlled by the value of $K_o$ and that it is different during loading and during unloading. During loading, it is mainly affected by the internal friction angle of particulate material, as given by the stress-induced anisotropic shear modulus coefficient $A_{VH}$:

$$A_{VH} = \frac{|G-V - G_{II}|}{G_V} = \left| 1 - K_o^{5/2} \right|$$

(2.27)

where $K_o$ during loading can be estimated by $K_{o_l} = a(1 - \sin \phi)$, therefore the stress-induced anisotropic shear modulus coefficient $A_{VH}$ during loading is:

$$A_{VH}^l = \left| 1 - [a - a \sin(\phi)]^{n/2} \right|$$

(2.28)
where \( a \) is a soil constant that varies between 1.0 for pool filter sand and 0.75 for Ottawa sand, \( n \) is a constant that is equal to 0.5 for sand, and \( \phi \) is the internal friction angle. During unloading, the stress-induced anisotropic shear modulus coefficient \( A_{VH} \) depends on both the internal friction angle, and the value of the overconsolidation ratio (OCR), as given by the following equations:

\[
K_{0ul} = a \left( 1 - \sin \phi \right) \frac{\sin (\phi)}{OCR}^{2}
\]

\[
A_{VH}^{ui} = \left[ 1 - \left[ a - a \sin (\phi) \right]^{n} \right]^{OCR} \left( \frac{\sin (\phi)}{4} \right)
\]

Zeng and Ni (1999) also indicated that shear modulus in a non-principal stress plane is dependent on the shear moduli in the principal stress planes and the direction of wave propagation (see Figure 2.27). Regarding problems with identification of travel time arrivals, the authors claim that a bender element technique can produce reliable results if the device and electric pulse are properly designed.

Baig et al. (1997) measured low strain moduli of cemented sands using bender elements and compared the results with those obtained utilizing a resonant column device. Values obtained with piezoceramic bender elements shown in Figure 2.28a range from 5 to 40 % higher to those obtained with a torsional resonant column. The authors attribute the difference to the lower strain levels induced by the bender elements. They also show that the effect of confining pressure on maximum shear modulus \( G_{\text{max}} \) of artificially cemented sand specimens is negligible. Instead, the main variable found by the authors influencing the determination of the cemented sands’ moduli is the percentage of cementation, as seen in Figure 2.28b (see also Fernandez and Santamarina 2001).

Jardine et al. (2001) and Firovante and Capoferri (2001) document the results of a comprehensive experimental study of anisotropic state of stresses in sand and glass beads using piezoelectric bender elements mounted in a triaxial cell (Figure 2.29). These authors develop techniques intended to use the evaluation of S-wave propagation in different directions to determine anisotropic elastic parameters.
Figure 2.27: Stress state, direction of wave propagation and particle vibration (Zeng and Ni 1999).

Figure 2.28: (a) Comparison of shear modulus measured with resonant column and piezoelectric bender elements on the same specimen. (b) Effect of confining pressure on shear modulus in cemented sand specimens as measured with piezoelectric bender elements (Baig et al. 1997).
Figure 2.29: Sketch of the setup for measuring shear wave velocity in triaxial cells using bender elements oriented in different directions: (a) Jardine et al. (2001). (b) Fioravante and Capoferri (2001).

Jardine et al. (2001) evaluated the variation of shear wave velocity in specimens of river sand and glass beads. These specimens are K₀-consolidated and loaded in the modified triaxial cell shown in Figure 2.29(a). Figure 2.30 shows the variation of wave velocity of differently polarized waves. In their discussion of the velocity trends in the two specimens they indicate that even when the specimens are expected to show stiffness anisotropy due to specimen preparation, at isotropic confinement during the start of the tests, the specimens show minor differences in S-wave velocities regardless of the direction or polarization. They also observed that the S-wave velocity anisotropy becomes evident when the specimen is loaded under K₀-conditions. Furthermore, they found that the S-wave velocities decrease in the following order (1) velocity in the vertical direction polarized in the horizontal direction, (2) velocity in the horizontal direction polarized in the vertical direction, and (3) velocity in the horizontal direction polarized in the horizontal direction.

Jardine et al. (2001) argue that the large difference between the velocities in the waves in the vertical direction polarized in the horizontal direction and the waves in the horizontal direction polarized in the horizontal direction is surprising. They indicate that the wave velocity
is highly dependent of the stress in the direction of wave propagation and it is very lightly dependant on the stresses in the direction of particle motion.

Fioravante and Capoferri (2001) found similar relations for Ticino sand specimens (Figure 2.31) tested in a triaxial cell.

![Figure 2.30: Response of shear velocities under increasing hydrostatic pressures p’ in anisotropic (a) Ham River Sand at e₀=0.659 and (b) Glass Ballotini at e₀=0.699 (after Jardine et al. 2001).](image)

Pennington et al. (1997) present results of shear wave velocity measurement on remolded Gault specimen. Their results (Figure 2.32) indicate that the shear stiffness of the clay greatly depends on the fabric anisotropy. The degree of inherent anisotropy reportedly does not depend on the isotropic state of stress and remain constant during loading and unloading up to about an over consolidation ratio of ten. Pennington et al. report that even when the shear wave velocity in the clay is stress dependent, the inherent anisotropy plays a controlling role.
Figure 2.31: Variation of the wave velocity versus effective stresses in Ticino sand specimens (void ratio e: 0.86 to 0.90): (a) P-wave velocities and (b) S-wave velocity (after Fioravante and Capoferri 2001).

Figure 2.32: Effect of stress anisotropy on shear wave velocity wave propagation (after Pennington et al. 1997)
2.11.2 Use of Bender Elements on Unsaturated Soils.

Mancuso et al. (2002) carried out an experiment using a resonant column – torsional shear cell fitted for controlled-suction to investigate the small strain behavior of unsaturated compacted silty sand. Specifically, they analyzed the effects of suction and fabric on soil behavior. Shear stiffness measurements are taken during constant-suction tests (Figure 2.33).

Their data indicated an S-shaped initial shear stiffness versus suction variation, which can be explained considering the progressive change from a bulk-water regulated soil response to a menisci-water regulated soil response. Most of the effects are detected for suctions ranging from 0 to about 200 kPa. For values higher than 200 kPa, $G_o$ tends toward a threshold that depends on the net stress level.

Qian et al. (1991) used a resonant column device to measure the shear of stiffness of unsaturated soils. They tested soil specimens with different mean grain size $D_{50}$, degree of saturation $S_r$, and external confining pressure $\sigma$. They observed that shear stiffness has a peak at a certain degree of saturation and that this peak depends on the confining pressure and on the fine fraction, however it does not depend on the mean grain size (see Figures 2.34a to 2.34c). Qian et al. (1991) also showed that the peak of the shear modulus versus inversely depends on the void ratio and the coordination number of the structure decreases.

**Figure 2.33:** (a) Initial shear stiffness in controlled-suction resonant column tests. (b) Response of shear stiffness to suction at a mean net stress of 400 kPa (Mancuso et al. 2002).
Figure 2.34: Shear stiffness of unsaturated soils versus (a) degree of saturation and confining pressure, (b) degree of saturation and fine content, and (c) degree of saturation and confining pressure as tested in a resonant column device (after Qian et al. 1991).

2.12 Summary

This chapter reviews several concepts related to the mechanical behavior of unsaturated soils and the state of the art on the small strain behavior of unsaturated soils found in published literature about the topics. It starts with a discussion of the effective stress concept in unsaturated soils, contrasting the effective stress equation for saturated and for unsaturated soils that have been used in engineering practice. It is concluded that to describe the shear strength behavior and

Summary of Results

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</tbody>
</table>

Note: $C_s = \frac{W_{\text{dry}} - W_{\text{finer S-400}}}{W_{\text{dry}}}$ where $W$ is the weight of the soil.
the volume change behavior of an unsaturated soil it is useful to present stress-strain results in
terms of two independent stress state variables: net normal pressure \((\sigma-u_a)\) and matric suction
\((u_a-u_w)\), because equations of this kind show the contribution of both contact and capillarity
forces. This approach is used in this investigation. The experimental determination of the stress
state variables is treated as well as the axis-translation technique to explain the procedure used
here to induce matric suction in the specimens. The concepts of volume changes in unsaturated
soils and swell/shrink behavior are discussed, highlighting the importance of including soil
suction as a major variable in the evaluation of swelling since it is a key parameter for
determining the deformation and shear strength behavior of partially saturated soils.

This chapter also includes the concept of small strain behavior on unsaturated soils. The
use of elastic wave propagation to monitor elastic and inertial properties of soils is presented.
Dynamic methods for measurement of soil stiffness at very small strains in the laboratory are
discussed emphasizing the use of piezo-ceramic plates (bender elements). Results on elastic
wave measurement from published literature are presented.
Chapter 3
Testing Under K₀-Conditions

3.1 Introduction
To study the effect of matric suction and stress anisotropy on the low strain stiffness of soils, a set of tests are conducted in the modified oedometer cell. Bender elements are used here to investigate elastic properties of soils under anisotropic condition. They are mounted on an odometer cell that restricts lateral deformations. This chapter presents test results from two particulate materials at four moisture contents that render different initial matric suctions. The variation of shear wave velocities polarized in the vertical and horizontal directions as the vertical stresses are varied in an oedometer cell is recorded. This permits monitoring the anisotropic state of stresses. Vertical deformations produced by the application of stresses, are recorded as well at each variation of stress. Suction is independently monitored with a small tip tensiometer.

3.2 Soil Characterization
To evaluate the effect of suction on stress anisotropy of soils, two soils with different grain size distribution (silt and sand) were tested in the laboratory under K₀ conditions. The two soils are uniform fine sand and clayey silt. They have different specific surfaces and particle sizes. The mean particle size and the specific surface (the ratio of its surface area to its mass) of particles yield qualitative information about the relative contribution of the electric forces (related to the surface of the particles and assuming the surface charge is known), capillary forces (related to the diameter of the particles), and the gravimetric forces (related to the volume of the particles) in the global behavior of the arrangement of particle system.

3.2.1 Determination of Grain Size Distribution
The procedure described in ASTM D422-63(2002) is used to determine the grain size distribution of the proposed tested soils. This test method covers the quantitative determination of the distribution of particle sizes in soils. The distribution of particle sizes larger than 0.075 mm (retained on the No. 200 sieve) is determined by sieving, while the distribution of particle sizes smaller than 0.075 mm is determined by a sedimentation process, using a hydrometer to acquire the necessary data.
3.2.2 Determination of Specific Gravity

The procedure described in ASTM D854-02 is used to determine the specific gravity of the proposed tested soils. These test methods cover the determination of the specific gravity of soil solids that pass the 4.75-mm (No. 4) sieve, by means of a water pycnometer.

3.2.3 Determination of Soil Specific Surface

The specific surface of a soil gives indication of the sensitivity of soils to changes in degree of saturation, and then can be used in approximating several engineering parameters including swelling/shrinkage potential. As the specific surface increases, the importance of electrical and capillary contact-level forces increases. Surface related forces must be taken into consideration when the specific surface of the soil approaches and exceeds ~ 1 m²/g (Qian et al. 1991; Santamarina et al. 2001). The aim is to determine the specific surfaces of the clayey silt using the Methylene Blue (MB) Adsorption method. When MB dye (chemical formula C₁₆H₁₈N₃SCl, molecular weight 319.87 g/mole) is dissolved in water, the MB cations and chlorite anions formed can replace the existing cations in a given clay mineral surface (Cokca and Birand 1993).

Finer soils have larger specific surface, then, the amount of cationic MB dye adsorbed by the negatively charged clay surfaces is influenced predominantly by the clay size fraction of the soil. Therefore, the specific surfaces can be determined by the amount of adsorbed MB. The MB adsorption method is used in a wide range of disciplines because the materials necessary to run the test are inexpensive and readily available, the test can be performed easily by a laboratory technician, it is done relatively fast ranging from 10 to 40 min depending on the soil type (Cocka and Birand 1993). Furthermore, the MBA has the advantage that is done in water; therefore, expansive minerals can expose all surface area (Santamarina et al. 2002). Nevertheless, the MB absorption method suffers from disadvantages too. For instance, Santamarina et al. (2002) indicated that the area covered by one methylene blue molecule is in best case assumed and that the uncertainty in this assumption can affect the estimation of specific surface in more than 100%. Furthermore, excess salts in the solution compete with MB to be adsorbed. The methylene blue adsorption technique has been used to successfully measure the surface areas of kaolinite, illite, and montmorillonite (Pham and Brindley 1970). In this study, this method is used on a clayey silt containing 15 % of material smaller than 0.075 mm.
Methylene Blue (MB) Adsorption Procedure. The MB solution consists of 1.0 g of MB powder on 200 mL of deionized water. The soil suspension is prepared with 10 g of oven dry soil on 30 mL of deionized water. MB solution is added to soil suspension in 0.5 mL increments and agitated for approximately one minute. One drop is placed on filter paper (Fisherbrand P5) after each increment. When a light blue halo forms around the soil spot, stop the increments (there is unadsorbed MB which is indication that the cation exchange capacity of the clay has been reached). Specific surface is then determined using the following expression (Kandhal and Parker 1998):

\[
S_s = \frac{1g}{319.87 \text{ g/mole}} \cdot \frac{1}{200 \text{ ml}} (N \cdot 0.5 \text{ ml} \cdot A_v \cdot A_{MB}) \cdot \frac{1}{10 \text{ g}} \tag{3.1a}
\]

\[
S_s = 4.707 \cdot 10^{17} \cdot \frac{N \cdot A_{MB}}{g} \tag{3.1b}
\]

where \( N \) is the number of MB increments added to the soil suspension solution, \( A_v \) is Avogadro’s number equal to \( 6.02 \cdot 10^{23} / \text{mole} \), and \( A_{MB} \) is the area covered by one MB molecule which is assumed to be \( 130 \text{ Å}^2 \) (Chen et al. 1999).

### 3.3 Description of the Testing Setup

To study the effect of suction and stress anisotropy under \( K_o \) conditions (i.e., zero horizontal strain \( \varepsilon_h = 0 \)) using S-wave velocity, a set of oedometer tests are performed. The oedometer tests consist on testing soils by applying vertical effective stresses while a rigid ring prevents horizontal strains. The experimental study involves the use of bender elements for the generation and reception of shear waves and a small tip tensiometer-pressure transducer system for the measurement of soil matric suction.

#### 3.3.1 Shear Wave Velocity Monitoring

A diagram of the machined oedometer cell is shown in Figure 3.1. Four openings 90° apart from each other are cut on the wall of the PVC oedometer cell to let the bender elements being introduced into a remolded soil specimen once it has been placed and compacted in the oedometer cell. Each opening consists of two sections: a groove in the interior part of the wall with dimensions slightly bigger than those of a coated bender element through which the bender element is sliced into the cell, and a threaded cut on the wall’s exterior where a receptacle containing the glued bender element and its base are screwed. This pushes the bender elements into place inside the cell. This design intends to reduce the transmission of signals through the
cell walls. Finally, two openings are cut at the bottom of the cell to permit the installation of small tip tensiometer porous cups for the monitoring of the matric suction.

Piezoceramic bender elements or bimorphs of the series mode (Morgan Matroc Inc.) with nominal dimensions of 12.7mm x 6.35mm x 0.53 mm (0.50” x 0.25” x 0.021”) are used in this
investigation. Only series bender elements were used in this project, which showed to be inconvenient due to cross-talk between them (Santamarina and Lee, 2003). Bender elements are electro-mechanical devices, so besides producing mechanical waves, they may generate unwanted electromagnetic signals that are picked by the receiver bender element, a phenomenon called cross-talk. The amplitude of this noise, also known as electromagnetic interference (EMI), can be several times the amplitude of the mechanical waves which in most cases impedes to observe the body wave’s first arrivals clearly. To avoid or reduce the EMI, bender elements need to be shielded with a metallic cover and grounded. Figure 3.2 shows a comparison of traces with and without grounding. These problems may be also avoided with the use of parallel connected bender elements.

![Figure 3.2](image.png)

**Figure 3.2:** Traces from bender elements: grounded and not grounded.

Grounding the bimorphs greatly reduces the EMI. However, achieving a stable response from the series bender elements in this study was not easy principally because they were exposed to moist conditions for several days. This project required conducting tests lasting up to a week. Bender elements were prepared before testing for the geo-environment in which they were placed, which usually required insulating them from humidity. This insulation, which can consist of two or more layers of a waterproof sealant, serves also to strengthen these fragile pieces but at the expense of sensitivity. The first intent was to insulate the bender elements from moisture using two layers of polyurethane, and then to shield them from EMI with a silver paint. In several occasions, the bender elements short-circuited after applying the metallic paint due to the action of its solvent on the polyurethane. Therefore, several layers of polyurethane were needed.
which required several days for curing and ended up in a thick cover of this sealant. Moreover, there were cases where the response of the bender elements started to decrease (detected by decreasing resistivity) during a test. To avoid such nuisance, Teflon tape and aluminum fold were used to insulate from moisture and shield from EMI respectively (Figure 3.3). The use of these materials eliminated the curing time needed with the previous ones and, more importantly, the occurrence of short circuits.

![Figure 3.3: Pre-test preparation of bender elements](image)

**Figure 3.3:** Pre-test preparation of bender elements

An Exact Electronics Inc. function generator Model 506 was used to generate a pulse to the source bender element. Signals detected on the receiver bender element were enhanced with a Krohn-Hite Corporation multi-channel filter model 3944, and digitized with an Agilent Technologies oscilloscope model 54624A. Figure 3.4 shows a schematic representation of the setting used to acquire bender element readings.

![Figure 3.4: Representation of the equipment setup used to acquire data from bender elements](image)

**Figure 3.4:** Representation of the equipment setup used to acquire data from bender elements
3.3.2 Monitoring Soil Matric Suction.

For the determination of the soil matric suction, a dual system consisting of a small tip tensiometer and an electric pressure transducer (Tensimeter by Soil Measurement Systems) was used (Figure 3.5). A description of the procedure used to prepare a tensiometer prior to operation follows. The ceramic cup (6.3 mm diameter, 28.8 mm length) was connected (through nylon tubing) to a glass tube that provides the column of water to keep the cup saturated. The bond of the elements is done using waterproof epoxy to provide good seal. The tubing was filled up to approximately ¾ of its height with deionized deaired water and any entrapped air was expelled using a vacuum pump (650 mm Hg). Next, the ceramic cup was allowed to soak in deionized water overnight to be sure that the ceramic was thoroughly saturated. Use of deionized water is recommended in order to keep the instruments clean longer and avoid any unwanted reaction of the ions encountered in tap water with those in the soil. The tube was filled to capacity and the upper stopper was placed onto place. Immediately after, the ceramic cup was introduced into the soil specimen and the recording of the readings began.

![Figure 3.5: Sketch of a tensiometer-pressure transducer for measuring matric suction.](image)

The other part of the system used in the measuring of soil suction is an electrical pressure transducer Tensimeter. The Tensimeter is a handheld, battery-operated meter consisting of a high quality pressure transducer and a digital readout. The pressure transducer probe contains a needle.
which is inserted through the stopper of the tensiometer when taking a reading, and the soil matric potential is displayed on the digital readout in milibars. It has a range of operation from –1 bar to +2 bar, with a sensitivity of 1/1000 of a bar.

The manufacturer’s calibration of the tensiometer is checked following one of the procedures recommended by the company. The method uses a vertical tube filled with water, having a septum stopper at the upper end and with the lower end in a container with water. The reading of suction displayed on the digital readout should match the vertical distance between the water level in the tube below the stopper and the water level in the container times 0.98 (100 cm of H₂O is equal to 98 mbar). This is repeated at several levels of water on the tube, corresponding to different suctions.

During the first readings, it is observed that if the needle is introduced again shortly after a first time, the second reading would be invariably higher (less negative) than the first one. This occurs because each time the needle is injected there is going to be some change in air volume. In order to reduce this difficulty, it was decided to leave the needle injected inside the tensiometer for the duration of each test.

A factor limiting the use of tensiometers is that their measuring range does not cover the complete range of possible water tension. Since this system is a direct method to measure the soil matric suction, it uses the atmospheric pressure as the reference. Because of cavitation (formation of water vapor bubbles) of water at low pressure, the range of operation of a tensiometer is limited to a minimum of -90 kPa.

When using this system and with the transducer probe needle at the top of the tube (see Figure 3.5), the readings displayed on the digital readout correspond to the water tension at the top of the water column. Therefore, these readings need to be corrected for the length of the vertical water column above the center of the tensiometer cup. To make this correction, this distance, multiplied by 0.98, is added to the tensiometer reading, which gives the real tension in the soil around the cup.

3.3.3 Specimen Preparation and Testing Procedure.
Remolded specimens are prepared in the PVC oedometer cell shown in Figure 3.1, which has dimensions of 63.3 mm in diameter and 114.3 mm in height. Taking account of the well-know relationship that suction decreases with increasing water content (see Section 2.2), the suction in these set of test specimens is modified by varying the water content of the soil-water blend at the
mixing stage. Altering the molding water content produced different initial suctions in specimens in the 'as compacted' state. Four specimens are mixed at varying water contents of 0, 5, 10, and 15 percent. The specimens are compacted in five layers of soil using a uniform tapping process.

The uniaxial pressure was applied to the specimen using a Geonor swelling/consolidation oedometer load frame. The tests are performed at predetermined level of vertical stresses, ranging from 0 to 624 kPa at uniform loading steps. At each loading steps, the bender elements are excited with 5 Hz square waves and the responses from the receiver bender elements were saved. The signals are stacked sixty-four times to improve the signal-to-noise ratio. Figure 3.6 shows the beneficial effect of stacking or averaging the signals. This procedure is repeated for both vertically and horizontally aligned pairs during the loading and unloading of the specimens.

![Figure 3.6: Effect of averaging on traces from bender elements](image)

To further improve the identification of first arrivals, signals are also collected using phase reversals. Figure 3.7 shows the procedure. The bottom trace is result of subtracting the two at the top, which eliminates the electromagnetic interference caused by cross-talking between the bender elements.

The use of elastic waves permits monitoring the elastic (i.e., stiffness) and inertial (i.e., mass density) properties of soils. However as stated in section 2.10, the identification of wave arrivals and the associated travel times may be difficult because of distortions produced by near field effects, cross-talking, among others.
The use of elastic waves permits monitoring the elastic (i.e., stiffness) and inertial (i.e., mass density) properties of soils. However as stated in section 2.10, the identification of wave arrivals and the associated travel times may be difficult because of distortions produced by near field effects, cross-talking, etc. There are several methods aimed at reducing the subjectivity associated with identifying wave travel times. The approach used in this study to identify the arrival of the shear waves is based on the time domain first arrival method with consideration of near field effect (Dyvik and Madshus 1985; Brignoli et al. 1996; Ferreira 2003; Kawaguchi 2003), portrayed in Figure 3.8.

**Figure 3.7:** Enhancement of wave arrivals by phase reversal

![Figure 3.7: Enhancement of wave arrivals by phase reversal](image)

**Figure 3.8:** Time domain technique to determine S-wave arrival considering near field effect (Kawaguchi 2003).
Figure 3.8 presents received traces affected by near-field effect. The first deflection on the curve at point A can be erroneously taken as the first wave arrival, however it is caused by near field effect. In this case, the correct arrival is suggested to occur at point C or at zero voltage after first inflection. In situations where the near field effect is significant as in curve (b), the arrival time is considered in between points B and D again. Note how the arrival can be disguised by this phenomenon. The approach selected in this study to identify time arrival of the S-wave is supported by a travel time calculation of compression and shear-waves to verify that the first arrival does not correspond to reflected p-waves but to s-waves. The basis of this calculation follows. Compression and shear wave velocities are related through Poisson’s ratio \( \nu \) (Stokoe and Santamarina, 2000) as:

\[
V_p = V_s \sqrt{\frac{1 - \nu}{1 - 2\nu}} 
\]  

(3.2)

Furthermore, the small-strain Poisson’s ratio for unsaturated soils is lower than 0.15 (Santamarina et al. 2002). Therefore, knowing the travel length that P-waves and S-waves will travel in a particular geometrical configuration, it is possible to determine the travel times corresponding to each kind of waves. Table 3.1 presents the ratio of P-wave travel time / S-wave travel time for the dimensions of the oedometer cell (Figure 3.8). For the bender elements polarized vertically, the p-wave arrives latter than the s-wave to the receiver bender element \((t_P/t_S > 1.4)\). For the case of bender elements polarized horizontally, the arrival of both type of waves is almost simultaneous, with the P-waves arriving slightly later.

**Figure 3.8:** Schematic of the oedometer cell showing p-wave and s-wave trajectories and tip-to-tip distances between bender elements.
Table 3.1: Travel time calculation for compression and shear waves in the oedometer cell

<table>
<thead>
<tr>
<th>Poisson’s ratio ( \nu )</th>
<th>Horizontally Polarized Waves</th>
<th>Vertically Polarized Waves</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Ratio of P-wave travel time / S-wave travel time</td>
<td></td>
</tr>
<tr>
<td>0.05</td>
<td>1.501</td>
<td>1.073</td>
</tr>
<tr>
<td>0.10</td>
<td>1.454</td>
<td>1.039</td>
</tr>
<tr>
<td>0.15</td>
<td>1.400</td>
<td>1.000</td>
</tr>
</tbody>
</table>

The presence of two pairs of bender elements is also illustrated in Figure 3.8. One pair of bender elements is placed horizontally, so that the shear wave polarizes the soil particles vertically. That is, when the source bender element vibrates the particle motion produced by the shear wave excitation is vertical. The other pair of bender elements is placed vertically to polarize the soil particles horizontally. Although both P-waves and S-waves are identified and recorded from these plots, only the S-waves are used to evaluate the response of the soil. The reason, is that only the latter are related to the stiffness of the soil skeleton and it permits the identification of the effective stresses in independent directions.

3.4 Test Results

3.4.1 Tested Soil Properties

Table 3.2 shows results from the specific surface of the soil as tested with the Methylene Blue (MB) adsorption method and the Atterberg’s tests results. The selection of the soils shows different specific surfaces and main grain size distributions.

Table 3.2: Preliminary results of specific surface and Atterberg’s limits tests conducted on silica sand, and silt.

<table>
<thead>
<tr>
<th>Spec. Surface ( (m^2/g) )</th>
<th>Liquid Limit ( (%) )</th>
<th>Plastic Limit ( (%) )</th>
<th>Plasticity Index ( (%) )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Silica sand</td>
<td>0.023*</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>Silt</td>
<td>0.98</td>
<td>48</td>
<td>22</td>
</tr>
</tbody>
</table>

*Specific surface of silica sand determined by assuming the following relation: \( S_s = \frac{6}{(d \rho_s G_s)} \); where \( d \) (diameter) is 100 \( \mu m \).
3.4.2 Tested Results

Figures 3.9 to 3.12 present traces from the receiver bender element corresponding to travel times of the arriving elastic waves. Traces are recorded during loading and unloading silica sand specimens constructed at four different water contents. Initial matric suction for each moisture content is indicated.

The general trend in every plot is that as higher vertical stresses are imposed on the specimen, the travel times of the elastic waves to get from the source bender element to the receiver bender element decrease. Thus, the shear wave propagation velocities and shear modulus $G_o$ (which is an indication of stiffness) rise during loading and decrease when the specimen is being unloaded. This behavior is expected, as equation 2.23 indicates. Shear wave velocity is function of the effective state of stress on.

Figures 3.13 to 3.16 show the relationship between the effective vertical stress applied to the specimen, and the S-wave velocities during loading and unloading. There are two sets of curves on these figures: one corresponds to the vertically polarized shear waves while the other one corresponds to the horizontally polarized shear waves. It is observed that the velocity of the vertically polarized shear wave is higher than the velocity of the horizontally polarized shear wave, especially at high stress levels. This behavior can be explained due to the existence of higher stresses in the vertical direction than in the horizontal direction.

Both sets of shear waves show higher velocities during the unloading stage. This is because during loading, the soil specimens suffer variations in fabric that do not return to the original condition once it is unloaded. Curve fitting was performed to get the needed parameters needed to evaluate the coefficient of lateral stress at rest $K_o$ for virgin and overconsolidated soils.

Parameters from the curve fittings are summarized in Table 3.3. Although the data meets the linear regression assumption of independence between observations, it is necessary to consider that the fitting parameters would change if this experiment were repeated, since each value of the response variable (S-wave velocity) has its own distribution which is assumed to be normal. For this reason, the power model used in this study to approximate the relationship between effective stress and shear wave velocity, is suitable to evaluate the results within this project but it might be inappropriate to use for comparison among different studies conducted under other conditions.
Oedometer Test
Soil: Sand
$K_0$ condition
Moisture content: 0 %
Initial Suction: Dry specimen.

$\sigma_v' = \sigma_v + \Delta \sigma$

$\sigma_h' = K_0 \sigma_v'$

**Figure 3.9**: Travel time data in the oedometer test (loading and unloading): (a) description of stress paths during testing, (b) travel time data for vertically polarized bender elements, and (c) travel time data for horizontally polarized bender elements.
Figure 3.10: Travel time data in the oedometer test (loading and unloading): (a) description of stress paths during testing, (b) travel time data for vertically polarized bender elements, and (c) travel time data for horizontally polarized bender elements.
Figure 3.11: Travel time data in the oedometer test (loading and unloading): (a) description of stress paths during testing, (b) travel time data for vertically polarized bender elements, and (c) travel time data for horizontally polarized bender elements.

Oedometer Test
Soil: Sand
Ko condition
Moisture content: 10 %
Initial Suction: 4.6 kPa

σv = σo + Δσ
σh' = Ko σv'

(a)

(b)

(c)
**Figure 3.12:** Travel time data in the oedometer test (loading and unloading): (a) description of stress paths during testing, (b) travel time data for vertically polarized bender elements, and (c) travel time data for horizontally polarized bender elements.
Once the arrivals of the S-waves are identified and the travel times determined, the propagation velocities can be computed.

**Figure 3.13:** Relationship between effective vertical stress and S-wave velocity at the dry silica sand specimen. The arrows indicate the direction of loading and unloading. For comparison the fitted velocity versus the applied effective vertical stresses are presented.

Vertically polarized S-wave velocities:
- Loading: \[ V_S = 91 \frac{m}{s} \left( \frac{\sigma'_v}{p_r} \right)^{0.211} \]
- Unloading: \[ V_S = 147 \frac{m}{s} \left( \frac{\sigma'_v}{p_r} \right)^{0.144} \]

Horizontally polarized S-wave velocities:
- Loading: \[ V_S = 89 \frac{m}{s} \left( \frac{\sigma'_v}{p_r} \right)^{0.178} \]
- Unloading: \[ V_S = 165 \frac{m}{s} \left( \frac{\sigma'_v}{p_r} \right)^{0.087} \]

\[ p_r = 1 \text{ kPa} \]

**Figure 3.14:** Relationship between effective vertical stress and S-wave velocity at the sand specimen with suction of 5.2 kPa. The arrows indicate the direction of loading and unloading. For comparison the fitted velocity versus the applied effective vertical stresses are presented.

Vertically polarized S-wave velocities:
- Loading: \[ V_S = 135 \frac{m}{s} \left( \frac{\sigma'_v}{p_r} \right)^{0.121} \]
- Unloading: \[ V_S = 140 \frac{m}{s} \left( \frac{\sigma'_v}{p_r} \right)^{0.122} \]

Horizontally polarized S-wave velocities:
- Loading: \[ V_S = 134 \frac{m}{s} \left( \frac{\sigma'_v}{p_r} \right)^{0.09} \]
- Unloading: \[ V_S = 179 \frac{m}{s} \left( \frac{\sigma'_v}{p_r} \right)^{0.05} \]

\[ p_r = 1 \text{ kPa} \]
**Figure 3.15:** Relationship between effective vertical stress and S-wave velocity at the sand specimen with suction of 4.6 kPa. The arrows indicate the direction of loading and unloading. For comparison the fitted velocity versus the applied effective vertical stresses are presented.

Vertically polarized S-wave velocities:

Loading: \( V_S = 83 \frac{m}{s} \left( \frac{\sigma'_v}{p_r} \right)^{0.194} \)

Unloading: \( V_S = 176 \frac{m}{s} \left( \frac{\sigma'_v}{p_r} \right)^{0.079} \)

Horizontally polarized S-wave velocities:

Loading: \( V_S = 102 \frac{m}{s} \left( \frac{\sigma'_v}{p_r} \right)^{0.127} \)

Unloading: \( V_S = 198 \frac{m}{s} \left( \frac{\sigma'_v}{p_r} \right)^{0.029} \)

\( p_r = 1 \text{ kPa} \)

**Figure 3.16:** Relationship between effective vertical stress and S-wave velocity at the sand specimen with suction of 3.6 kPa. The arrows indicate the direction of loading and unloading. For comparison the fitted velocity versus the applied effective vertical stresses are presented.

Vertically polarized S-wave velocities:

Loading: \( V_S = 74 \frac{m}{s} \left( \frac{\sigma'_v}{p_r} \right)^{0.205} \)

Unloading: \( V_S = 159 \frac{m}{s} \left( \frac{\sigma'_v}{p_r} \right)^{0.086} \)

Horizontally polarized S-wave velocities:

Loading: \( V_S = 97 \frac{m}{s} \left( \frac{\sigma'_v}{p_r} \right)^{0.141} \)

Unloading: \( V_S = 170 \frac{m}{s} \left( \frac{\sigma'_v}{p_r} \right)^{0.061} \)

\( p_r = 1 \text{ kPa} \)
<table>
<thead>
<tr>
<th>Suction [kPa]</th>
<th>Velocity parameters</th>
<th>Vertically Polarized</th>
<th>Horizontally Polarized</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Loading</td>
<td>Unloading</td>
</tr>
<tr>
<td>Dry sand</td>
<td>θ [m/s]</td>
<td>91</td>
<td>147</td>
</tr>
<tr>
<td></td>
<td>β [ ]</td>
<td>0.211</td>
<td>0.144</td>
</tr>
<tr>
<td>5.2</td>
<td>θ [m/s]</td>
<td>135</td>
<td>140</td>
</tr>
<tr>
<td></td>
<td>β [ ]</td>
<td>0.121</td>
<td>0.122</td>
</tr>
<tr>
<td>4.6</td>
<td>θ [m/s]</td>
<td>83</td>
<td>176</td>
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<td>β [ ]</td>
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<td>0.079</td>
</tr>
<tr>
<td>3.6</td>
<td>θ [m/s]</td>
<td>74</td>
<td>159</td>
</tr>
<tr>
<td></td>
<td>β [ ]</td>
<td>0.205</td>
<td>0.086</td>
</tr>
</tbody>
</table>

To better observe the effect that matric suction has on the stiffness of soils and thus on the values of shear wave velocity, relationships between effective vertical stress and shear wave velocities of the four sand specimens are presented together in Figures 3.17 and 3.18. Only the loading stages are plotted here to help perceiving the pattern. The effect of matric suction is evident on the values of shear wave velocities. Specimens with higher initial matric suctions had higher s-wave velocities, due to increased bonding of the soil particles by menisci effect. Although the dry specimen does not have the beneficial effect of decreasing suction on soil stiffness since the wetted area of contact between particles is zero, it has the highest value on S-wave velocity. This is explained by the fact that this specimen has also the lowest initial void ratio, a parameter that controls soil stiffness.

The state of effective stresses can be inferred by measuring shear waves velocity polarized in different directions because effective stresses control the stiffness of soils (Pennington et al. 1997). Furthermore, the stress anisotropy may also be monitored by changing the polarization of the S-waves in two different directions (Zeng and Ni, 1999, Jardine et al. 2001, and Firovante and Capoferri 2001). For example, the interlocking between soil particles prevents the development of isotropic state of stresses. The ratio between the effective horizontal and vertical stresses is the coefficient of lateral stress at rest $K_0$ and it is determined here to evaluate the anisotropy of the specimens during the test (see details of the analysis in section 6.1). Results from this analysis are presented in Figure 3.19.
**Figure 3.17:** Relationship between effective vertical stress and S-wave velocity at the sand specimen at various suctions (Vertically polarized bender elements).

**Figure 3.18:** Relationship between effective vertical stress and S-wave velocity at the sand specimen at various suctions (Horizontally polarized bender elements).
Figure 3.19: Relationship between effective vertical stress and coefficient of lateral stress at rest $k_0$ on sand specimens with different initial matric suction.

It is observed that the initial soil matric suction has an influence on the anisotropy of the tested material. Higher values of matric suction give lower variations of anisotropy during the application of vertical stress.

Results from silt specimens follows. Figure 3.20 to 3.23 show the travel time data as captured on the digital oscilloscope. Travel times of S-waves on unsaturated specimens show small variation with applied vertical stress even though the specimens experience changes in void ratio during the test. This may be a result of the increased stiffness caused by matric suctions.

Figures 3.24 to 3.27 present the relationship between effective vertical stress and the velocity of shear waves observed in the clayey silt specimens. It is evident the low variation on shear wave velocity on the unsaturated specimens motivated by the effect of matric suction. Figures 3.28 and 3.29 summarize the loading parts of all tests for observation of matric suction effect on stiffness. With the exception of the specimen with the lowest initial matric suction, the other two unsaturated specimens show that higher S-wave velocities are obtained at higher matric suctions. The remolded dry specimen shows a higher variation of S-wave velocity with applied vertical effective stress because it is not affected by the suction effect seen on the unsaturated specimens.
Figure 3.20: Travel time data in the oedometer test (loading and unloading): (a) description of stress paths during testing, (b) travel time data for vertically polarized bender elements, and (c) travel time data for horizontally polarized bender elements.
Figure 3.21: Travel time data in the oedometer test (loading and unloading): (a) description of stress paths during testing, (b) travel time data for vertically polarized bender elements, and (c) travel time data for horizontally polarized bender elements.
Figure 3.22: Travel time data in the oedometer test (loading and unloading): (a) description of stress paths during testing, (b) travel time data for vertically polarized bender elements, and (c) travel time data for horizontally polarized bender elements.
Figure 3.23: Travel time data in the oedometer test (loading and unloading): (a) description of stress paths during testing, (b) travel time data for vertically polarized bender elements, and (c) travel time data for horizontally polarized bender elements.
Figure 3.24: Relationship between effective vertical stress and S-wave velocity at the dry silt specimen. The arrows indicate the direction of loading and unloading. For comparison the fitted velocity versus the applied effective vertical stresses are presented.

Figure 3.25: Relationship between effective vertical stress and S-wave velocity at the silt specimen with suction of 35 kPa. The arrows indicate the direction of loading and unloading. For comparison the fitted velocity versus the applied effective vertical stresses are presented.
Figure 3.26: Relationship between effective vertical stress and S-wave velocity at the silt specimen with suction of 23 kPa. The arrows indicate the direction of loading and unloading. For comparison the fitted velocity versus the applied effective vertical stresses are presented.

Figure 3.27: Relationship between effective vertical stress and S-wave velocity at the silt specimen with suction of 8 kPa. The arrows indicate the direction of loading and unloading. For comparison the fitted velocity versus the applied effective vertical stresses are presented.
Figure 3.28: Relationship between effective vertical stress and S-wave velocity at the silt specimen at various suctions (Vertically polarized bender elements).

Figure 3.29: Relationship between effective vertical stress and S-wave velocity at the silt specimen at various suctions (Horizontally polarized bender elements).
3.5 Summary

Tests in the modified oedometer cell are conducted to study the effect of matric suction and stress anisotropy on the low strain stiffness of soils. Using shear waves generated by bender elements, elastic properties of soils under anisotropic condition is investigated. Two particulate materials, silica sand and clayey silt, are tested at four moisture contents that render different initial matric suctions. In general, higher S-wave velocities are observed on specimens with higher initial matric suctions. The variation of S-wave velocity with applied vertical effective is higher on the sand specimens than in the ones constructed with clayey silt. This behavior may seem to be caused by the higher matric suctions observed on the silt specimens. A power law model is used for curve fitting. The model seems appropriate to compare the results obtained in this study, but its applicability to compare results from different studies under different conditions might be inappropriate. This is because the response variable on the model has its own distribution (normal assumed) and the curve fitting parameters $\theta$ and $\beta$ would be different even within a particular study if it were repeated. Nevertheless, the power model can be applied for curve fitting the shear wave velocity – stress relation for soils of different plasticity without altering its form as long the soils are dry. The model’s form varies for unsaturated soils (see chapter 6).
Chapter 4
Multiaxial Testing on Silt Specimens

4.1 Introduction

To study the effect of stress and suction on the stiffness of soils, a modified true triaxial device is used in this thesis. True triaxial devices control the three principal stresses or strains independently (Figure 4.1) and allow for any type of stress or strain paths. The Stress/Suction – Controlled True Triaxial Device used in this investigation is capable of controlling suction and any type of stress paths that soils maybe subjected to under geotechnical engineering applications (Hoyos and Macari 2001). Moreover the Stress/Suction – Controlled True Triaxial Device is modified to include two pairs of bender elements for monitoring elastic wave velocities and for the evaluation of low-strain soil stiffness (see for example Silva et al. 2002).

This chapter describes the modified Stress/Suction – Controlled True Triaxial Device and the results testing results for a silty soil under two different stress paths. Chapter 5 presents the similar stress paths in a sandy soil. The interpretations of the experimental results are presented in Chapter 6.

![Figure 4.1: Stresses (strains) applied to a cubical specimen within a true triaxial cell.](image)

4.2 True Triaxial Testing

To evaluate deformation properties and shearing resistance of soils in the lab, the most common equipment is the conventional triaxial apparatus. This equipment is capable of applying only axisymmetric state of stress though. In contrast, soils in the field are usually subjected to three principal stresses presenting cross-anisotropic behavior. The true triaxial apparatus
provides independent variations of the three principal stresses thus permitting to study the effects of the intermediate principal stress (which has influence on the strength and stress-strain properties of a material – Lade and Musante 1978; Budhu 1984; Prashant and Penumadu 2004) and cross-anisotropic behavior.

True triaxial devices are usually classified according to the applied boundary conditions into three categories: rigid-boundary type (Pearce 1972), flexible-boundary type (Ko and Scott 1967), and a mixture of rigid and flexible boundary type (Lade and Duncan 1973). The advantages and disadvantages of the three types have been discussed by Sture (1979), Saada and Townsend (1981), and Arthur (1988) and they are summarized next. Since a rigid boundary true triaxial device is strain controlled, the strains can be measure accurately and it is possible to get uniformity of strains. Loading plates allow easy installation of pressure cells and pore-water pressure instrumentation. However, the uniformity of stresses is difficult to verify and the apparatus does not allow shear distortions. Besides, interference of the loading plates may occur at large strains. On the other hand, a flexible boundary true triaxial device is stress controlled, hence, it permits to get normal principal stress on the loading faces and it is possible to get uniform stress distributions on all faces. Besides, no significant boundary interferences occur even at large strains and shear distortions are possible to obtain and measure. The main drawbacks in the flexible boundary true triaxial device are that the uniformity of large strains can be difficult to maintain and it is not easy to accommodate pore water-pressure instrumentation. A mixed boundary true triaxial apparatus tries to avoid some of the disadvantages of both rigid and flexible boundary types. Thus, boundary interference is usually avoided if the rigid boundary is placed on the compressive deviator direction while having flexible boundaries on the extension deviator direction. Furthermore, facilities for measuring pore-water pressure are easy to install on the rigid boundary. Nevertheless, heterogeneity of stressed and strains occur near the boundaries.

Modified True Triaxial Device. The device used in this research is a mixed-boundary type, having a high-air-entry ceramic disk at the bottom face of the apparatus (rigid boundary) and five flexible latex membranes at the other sides. The device is originally developed by Atkinson (1972) for multiaxial testing of rock materials, and is updated and modernized for further testing on silty and clayey soils (Ne-Smith 1997; Hoyos Jr. 1998). Hoyos and Macari (2001) present a detailed description of the apparatus emphasizing the modifications made to induce and control
suction (Figure 4.2). A new modification has been done to the apparatus to incorporate piezoelectric bender elements onto the lateral flexible membranes (Silva et al. 2002).

Here is a brief description of its main components with information from Hoyos and Macari (2001) updated for the present study. The modified true triaxial device consists of 10 main modules.

1. A steel frame that supports the top and lateral wall assemblies, the cubical specimen, and the bottom wall assembly containing the ceramic disk. The inner square cavities (pressure cavities) with dimension of 10.35 cm accommodate the membranes.

2. A top and four lateral wall assemblies (designated Z+, X+, X-, Y+, and Y-, respectively) machined from solid aluminum seal the interior pressure cavity. Each wall assembly
contains two fluid pressure inlet/outlet connections, three holes threaded to receive the stainless steel housing of three linear variable differential transformers (LVDT), and a centrally threaded pore-air pressure control port.

3. A deformation measuring system consisting of 15 high-pressure sealed LVDTs (from Lucas Control Systems Products, Inc.). Three LVDTs are used per each of the top and lateral faces (Figure 4.3). Calibration of the LVDTs was done using steel plates 0.1 inch in thickness and the LABTECH-NOTEBOOK computer software (from Laboratory Technologies Corp.)

4. A stress application/control system, which is a computer-driven electro-hydraulic pressure system for application and control of hydraulic pressure to the top and lateral flexible membranes. The fluid is pressurized by a 20-gal 6HP air compressor (from Porter-Cable Corp., Jackson, TN), which can deliver a maximum output pressure of 930 kPa (135 psi). The pressure is adjusted and controlled by electronic pressure regulators (from Proportion-Air, McCordsville, IN) with pressure range of 760 mmHg to 3445 kPa (500 psi) and accuracy ±0.2 % F.S. The electronic regulators receive analog input signals (Volt) from the Data Acquisition and Process Control System DA/PCS to regulate the pressure output.

5. Five flexible latex membranes (manufactured by Atlanta Plastics & Chemical Corp.) with medium-to-high tear strength and low stiffness form the barrier between the hydraulic fluid acting against the specimen and the specimen (Figure 4.3).

Figure 4.3: Photograph of the LVDTs extension rods, facilities for the application of pore air pressure, and a flexible latex membrane.

6. A pore-air pressure application set, consisting of a 1.3-cm side, pentagon-shaped copper block coupled to a 1.8-cm diameter, copper stem with the flexible membrane in between.
The pore air is introduced to the specimen via 3.2-mm diameter flexible nylon tubing and through an opening in the center of the stem (Figure 4.3). A 47 mm-diameter glass microfibre filter (from Whatman International LTD., Maidstone, England) is placed between the flexible membrane and the specimen to distribute the air pressure, supplied through the stem, to the pores of the soil. The pore air is applied only through the top face of the specimen since the lateral faces receive the bender elements.

7. A pore-water pressure application set consisting of a 1.05-cm thick, 7.95-cm diameter, 5-bar porous disk (Soilmoisture Equipment Corp.) seated onto the bottom wall assembly, which has a grooved water compartment underneath the disk. The water pressure is applied to the water compartment beneath the disk via nylon tubing. A flushing mechanism releases the entrapped air formed as a result of diffusion that accumulates beneath the disk to the atmosphere.

8. An air/water-supply pressure system that includes manual regulators (Bellofram Corp., Newell, WV) with maximum supply pressure of 1722 kPa (250 psi) used to regulate/control the air/water pressures applied to the specimen. An electrical pressure transducer (from Validyne Engineering Corp. Northridge, CA) monitored both the air and water pressures. The air/water lines are pressurized using the same air compressor utilized in the stress application/control system.

9. A data acquisition and process control system DA/PCS to automatically control the external pressures applied to the specimen and to monitor and record its resulting deformation. The electronic regulators of the pressure system receive an analog input signal from an analog output signal-conditioning interface kit (IOB120-01 from Analog Devices, Inc.), which is connected to the analog-to-digital converter (RTI-815 board from Analog Devices, Inc.) plugged into the CPU of the PC-based computer. The analog input signals delivered by the LVDT are also converted into digital output signals by the analog-to-digital converter RTI-815 board. A computer software (LABTECH-NOTEBOOK from Laboratory Technologies Corp.) makes it possible to monitor and record the application of pre-selected stress paths to the specimen and the corresponding deformations at rates determined by the operator. An additional analog input signal port was assigned to the pressure transducer monitoring the value of induced matric suction.
10. To generate and receive the elastic S-waves, two pairs of bender elements are mounted on the copper stems placed onto the lateral flexible membranes (stems were previously used for the application of pore air pressure through the lateral sides of the specimen). Prior to being installed on the stems, series type bender elements (from Morgan Matroc Inc., Bedford, Ohio) are treated with an electric shield and a moisture barrier (details on section 3.3).

The experimental setup involves the use of bender elements for the generation and reception of shear waves within the true triaxial test system (Figure 4.4). In more detail, changes on shear wave velocities are continuously monitored during the application of a range of net normal stresses to soil specimens with different induced matric suction values. During testing, a source-receiver pair of bender elements is placed vertically on the Y direction wall assemblies and another pair is placed horizontally on the X direction wall assemblies.

A tip-to-tip distance of 84 mm separates each pair of bender elements. The bender elements placed vertically polarize the shear waves in the horizontal direction. This configuration permits monitoring the effects of the effective horizontal stresses independently of the changes in the applied vertical stress. The bender elements placed horizontally polarize the shear waves in the vertical direction enabling them to monitor the effect produced by variations of effective vertical stress. The same equipment set-up employed on the oedometer testing to get shear wave travel time readings from the bender elements, is used on the multiaxial testing (see Chapter 3). Shear-wave arrivals were identified using a time domain first arrival method with consideration of near field effect, portrayed in Chapter 3 (Dyvik and Madshus 1985; Brignoli et al. 1996; Ferreira 2003; Kawaguchi 2003). A ratio of first arrival travel times for compression and shear waves was determined to validate the S-wave arrival identification technique used here. P-waves arrive slightly later than S-waves in the bender element set-up configuration used in the triaxial apparatus, as shown in Table 4.1.

<table>
<thead>
<tr>
<th>Poisson’s ratio ν</th>
<th>Ratio of P-wave travel time/S-wave travel time</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Horizontally Polarized Waves</td>
</tr>
<tr>
<td>0.05</td>
<td>1.086</td>
</tr>
<tr>
<td>0.10</td>
<td>1.052</td>
</tr>
<tr>
<td>0.15</td>
<td>1.013</td>
</tr>
</tbody>
</table>
Figure 4.4: Cross sectional view of cubical setup with bender elements.

4.3. Testing Program

Since the wetted area of contact between the soil particles decreases with an increase in the soil suction, it is expected that the effective stress will increase and the velocity of wave propagation will rise (see for example Equation 2.22). Yet the application of stresses needs to be done slowly enough to avoid menisci break and loss of suction as Cho and Santamarina (2001) documented.

4.3.1 Loading Rate Selection

To select the loading rate that would avoid menisci disruption, five silt specimens are loaded following a CTC stress path, each at different load rate. The tests are run at conditions favorable for menisci disturbance, that is, a low confinement and a relatively high suction. Thus, an effective confinement of 25 kPa and a suction of 50 kPa (which is the highest suction value used in this study) were selected. Each specimen was loaded incrementally up to 75 kPa of deviatoric stress. Stress-strain results of these tests are presented in Figure 4.5, where it is observed that slower loading rates cause smaller strains. Loading rates of 3 and 6 kPa/hour produce similar strains. Therefore a loading rate of 6 kPa/hour is selected to complete the testing.
4.3.2 Matric Suction Regulation

Matric suction states are applied using the axis-translation technique. The axis-translation technique for controlling soil suction allows turning the pore-water pressure positive by elevating the pore air pressure being applied to a soil sample. Unsaturated soils on the field present negative pore water pressure relative to the atmospheric pressure (gauge pressure = 0 kPa). In the laboratory, air pressures higher than the atmospheric pressure can be applied to a soil specimen. If both pore air pressure and pore water pressure receive an increment of the same magnitude, the soil matric suction is maintained without variation. This technique enables measuring pore water pressure using conventional pressure transducers as well as avoiding cavitation of water in the measuring system (Hilf 1956; Hoyos and Macari 2001). Matric suctions in this study are induced by increasing the air pressure in the triaxial cell while maintaining constant the water pressure below the high air entry porous disk.

4.3.3 Material and Testing Methodology

Two sets of tests (triaxial compression - TC - and conventional triaxial compression - CTC) were conducted on remolded silty soil specimens compacted in five layers to constant
height inside the true triaxial device at 10 % moisture content and 17.3 kN/m³ unit weight. Figure 4.6 summarizes the physical properties of the tested particulate media.

![Figure 4.6: Physical properties of tested soil. (a) Grain size distribution (D₅₀ = 0.056 mm). (b) Standard Proctor test compaction curve (wₑₒₚ=13.6%; γmax=17.9 kN/m³). Tests are run on specimens with 10 % moisture content and 17.3 kN/m³ unit weight.](image)

Each Triaxial Compression (TC) test is run at a constant level of matric suction (0, 25, or 50 kPa) and included three shearing processes started at hydrostatic confinements of 25, 50 and 100 kPa. The testing procedure for the multi-stage triaxial compression tests is explained next in more detail.

- **Hydrostatic confinement.** The test starts with a gradual increase in the hydrostatic confinement of the specimen up to a specified stress (25, 50, or 100 kPa). The increase in hydrostatic confinement is made gradually at a rate of 6 kPa/hour to avoid breaking the menisci with a corresponding loss of suction. The specimen is also subjected to a predetermined suction that is kept constant throughout the test. The suction is induced by subjecting the specimen to 50 kPa of water pressure and to an appropriate air pressure that will bring the specimen to the desired value of suction for a particular test. (i.e., suction equal to 0, 25, or 50 kPa).

- **Equalization.** The specimen is then allowed to equilibrate (suction gradually builds up to the desired value) which is a slow process. Duration of this process depends not only on the permeability of the soil under study but also on the permeability of the high air entry disk being used for the application of pore water pressure. In this case, the permeability of the porous ceramic disk governed the homogenization rate since it has a lower coefficient of
permeability (1.21 x 10^{-7} \text{ cm/s} - \text{Soilmoisture Corp.}) than the common values for sand and silty soils (10^{-3} to 10^{-7} \text{ cm/s} - \text{Bardet 1997; Budhu 2000})

- **Shearing.** The specimen is subjected to an increase in pressure in the vertical direction (Z) with a decrease in stresses on the horizontal directions (X-Y plane) of one half the increment in Z. This keeps constant the net mean stress $p'$. A loading rate of 6 kPa/hour was selected for the shearing stage to avoid the breaking the menisci.

- **Unloading.** The specimen is then brought to the state of stresses it had at the end of the equalizing stage, and the three-stage cycle is repeated for different confining pressures (i.e., 50 and 100 kPa).

The testing procedure on the conventional triaxial compression (CTC) tests varies from the previous one only in the shearing stage. Here, the pressure in the vertical direction is increased gradually while maintaining constant the pressure in the horizontal directions. The hydrostatic confinements of the specimens (25, 50, and 100 kPa) and the induced suctions (0, 25, or 50, kPa) were the same to those applied in the TC test.

Calculation of velocities are corrected using the tip-to-tip distance between bender elements. Although this distance varies during the test, its variation (in the horizontal axis) does not affect the velocity patterns significantly. Variation of the tip-to-tip distance between bender elements on a CTC test with a zero suction specimen are plotted in Figure 4.7.

A summary of tests that conform the true triaxial suction-controlled testing program are presented in Table 4.2 and the stress paths followed on each are shown in Figure 4.8. During the different loading procedure, the stress versus deformation data and the shear wave velocity is collected. These data permits the evaluation of both low strain and large strain behavior of unsaturated soils.

<table>
<thead>
<tr>
<th>Suction $u_a - u_w$ (kPa)</th>
<th>Net Stress $\sigma-u_a$ (kPa)</th>
<th>25</th>
<th>50</th>
<th>100</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>TC, CTC</td>
<td>TC, CTC</td>
<td>TC, CTC</td>
<td></td>
</tr>
<tr>
<td>25</td>
<td>TC, CTC</td>
<td>TC, CTC</td>
<td>TC, CTC</td>
<td></td>
</tr>
<tr>
<td>50</td>
<td>TC, CTC</td>
<td>TC, CTC</td>
<td>TC, CTC</td>
<td></td>
</tr>
</tbody>
</table>

Notation: TC: Triaxial compression test and CTC: Conventional triaxial compression test
Figure 4.7: Tip-to-tip distance between horizontally polarized bender elements at shear on a CTC test on a specimen subjected to zero suction and 25, 50, and 100 kPa of initial confinement.

Figure 4.8: Stress paths followed in TC and CTC tests
4.4 Results

4.4.1 Results from Triaxial Compression (TC) Tests - Small Strain Data

Figures 4.9 and 4.10 portray characteristic traces of arriving waves at the receiver bender element during a triaxial compression test conducted on a silt specimen with induced matric suction of 25 kPa (The complete set of traces from these tests are presented in appendix ). Traces of successive waves corresponding to different state of stress during hydrostatic compression and shearing processes can be seen. Figure 4.9 presents the traces from vertically polarized bender elements, whereas traces from horizontally polarized bender elements are shown in Figure 4.10.

In general, S-wave travel times between horizontally polarized bender elements decreases (and subsequently velocity increases) with increasing hydrostatic confinement. Regarding the shearing stages, the S-wave travel times of horizontally polarized bender elements increase during loading (decline in velocity) and decrease during unloading (rise in velocity). The rate of variation in velocities decreases at higher confinement stress. These results may be easily explained. At the loading part of TC test, stresses in the vertical direction (Z) are increased gradually while the horizontal stresses (in the X-Y plane) are decreased in half of the magnitude of the vertical stress increment. During unloading the stress path is reversed until the original hydrostatic stress is reached. Bender elements polarized horizontally effectively sensed changes on effective stress on that plane.

Results from S-waves polarized in the vertical direction seem right as well as the shear wave velocities increase with hydrostatic confinement (Figure 4.9b). On the shearing stages though, it is expected to observe increment in velocities (decrease on travel times) at the loading part and decreasing velocities during unloading, as the stress path on this test suggests. Instead, the shear-wave velocity remains in general almost invariant (Figure 4.9c). This seems to indicate that the exponents that control the relationship between wave velocity and the effective stresses are different: one for the direction of wave propagation and another for the direction of particle motion. In this case, it is clear that the exponent that corresponds to the stress in the direction of wave propagation is greater than the exponent corresponding to the stress perpendicular to the direction of wave propagation.

Shear-wave velocities calculated with Equation 2.23 are plotted against deviator stress in Figure 4.11.
Figure 4.9. Travel time data for vertically polarized bender elements: (a) description of stress paths during testing (suction $u_a-u_w = 25$ kPa), (b) travel time data during isotropic consolidation, and (c) travel time data during shearing (loading and unloading).
Figure 4.10. Travel time data for horizontally polarized bender elements: (a) description of stress paths during testing (suction $u_a-u_w = 25$ kPa), (b) travel time data during isotropic consolidation, and (c) travel time data during shearing (loading and unloading).
Figure 4.11: S-wave velocity data during a triaxial compression TC test on silt specimens at matric suctions of (a) 0 kPa, (b) 25 kPa, and (c) 50 kPa. References: ♦, ■, and ▲ indicate vertically polarized S-waves; ◊, □, and Δ indicate horizontally polarized shear waves; $\sigma_o$ indicates initial net hydrostatic stress.

Figure 4.11 summarizes the trends obtained during shearing stages on each test. The trends discussed above are more easily seen here. It is evident the effect of hydrostatic confinement on
shear wave velocity. Velocity of propagation of shear waves increases with increasing hydrostatic confinement.

With respect to the effect of matric suction on S-wave velocity (and thus in stiffness), a direct relationship between the two parameters is expected especially at low confinements. More precisely, due to the increased meniscus force caused by surface tension of the negative pore water pressure at higher matric suctions, the soil’s stiffness is expected to increase. Figure 4.12 shows the relationship between net mean stress $p'$ and shear wave velocity at matric suctions of 0 and 50 kPa. As expected, values of s-wave velocity at different suctions present higher variation at low confinements. At higher confinements the s-wave velocity data come nearer regardless the level of suction since the behavior is controlled by stress.

![Figure 4.12: Effect of matric suction on s-wave velocity. Data from horizontally polarized bender elements during a TC test on silt](image)

4.4.2 Results from Triaxial Compression (TC) Tests - Large Strain data

Figure 4.13 presents the typical octahedral shear stress versus the strain in the x, y, and z directions during shearing (loading and unloading) for a silt specimen tested at suction of 25 kPa and with different initial effective confinements. The octahedral shear stress is defined as:

$$
\tau_{\text{oct}} = \frac{1}{3} \sqrt{(\sigma_1 - \sigma_2)^2 + (\sigma_2 - \sigma_1)^2 + (\sigma_3 - \sigma_1)^2}
$$

(4.1)

where $\sigma_1$, $\sigma_2$, and $\sigma_3$ are the applied principal stresses and coincide with the stresses in the x, y, and z directions. As expected the strength of the particulate media increases with increasing...
confinement. A summary of results from the shearing process (loading only for comparison) at three different soil suctions is in Figure 4.14. Unsurprisingly, a direct relationship between, hydrostatic confinement and soil strength was observed throughout. The effect of suction however is more easily detected at low confinement. Similar trends are presented by Silva et al (2002).

**Figure 4.13:** Octahedral shear stress versus strain for silt specimen tested at suction = 25 kPa and at initial effective confinement stress of (a) p’=25 kPa, (b) p’=50 kPa, and (c) p’=100 kPa.
4.4.3 Results from Conventional Triaxial Compression (CTC) Tests - Small Strain Data

Conventional triaxial tests were also run in the modified true triaxial device and traces of arriving waves at the receiver bender element are presented in Figures 4.15 and 4.16. These results correspond to tests conducted on a silt specimen with induced matric suction of 25 kPa (Complete data from all these tests are documented in Appendix A). The travel time data contained in these figures are from hydrostatic compression and shearing processes. Figure 4.15 presents the traces from vertically polarized bender elements, whereas traces from horizontally polarized bender elements are shown in Figure 4.16.

**Figure 4.14**: True triaxial testing results. Octahedral shear stress versus vertical strain for specimen tested at (a) suction = 50 kPa, (b) suction = 25 kPa, and (c) suction = 0 kPa.
Figure 4.15: Travel time data for vertically polarized bender elements: (a) description of stress paths during testing (suction $u_a-u_w = 25$ kPa), (b) travel time data during isotropic consolidation, and (c) travel time data during shearing (loading and unloading).
Figure 4.16: Travel time data for horizontally polarized bender elements: (a) description of stress paths during testing (suction \( u_a - u_w = 25 \text{ kPa} \)), (b) travel time data during isotropic consolidation, and (c) travel time data during shearing (loading and unloading).
**Figure 4.17:** S-wave velocity data during a conventional triaxial compression CTC test on silt specimens at matric suctions of (a) 0 kPa, (b) 25 kPa, and (c) 50 kPa. References: ♦, ■, and ▲ indicate vertically polarized S-waves; ◊, □, and ∆ indicate horizontally polarized shear waves; $\sigma_o$ indicates initial net hydrostatic stress.
Similar to the behavior on the TC tests, S-wave travel time between both horizontally and vertically polarized bender elements decreases (velocity increases) with increasing hydrostatic confinement. On the shearing stages, the s-wave travel times of vertically polarized bender elements decrease during loading (increment in velocity) and increase during unloading (velocity decreasing). The rate of variation in velocities decreases at higher confinement stress. At the loading part of a CTC test, stresses in the vertical direction (Z) are increased gradually while the horizontal stresses (in the X-Y plane) are constant. Bender elements polarized vertically effectively sense the changes on effective stress on that direction. Results from bender elements polarized horizontally seem right as well as shear wave velocities remain practically unaffected at shearing. Figure 4.17 summarizes these trends obtained during shearing stages on each test. Only loading parts are included here for easy comparison. Again, the effect of hydrostatic confinement on shear wave velocity is clearly seen. Velocity of propagation of shear waves increases with increasing hydrostatic confinement.

As expected, soil stiffness indicated by the s-wave velocity, increases with suction, which is especially evident at low confinement (Figure 4.18). Values of s-wave velocity at different suction levels present higher variation at low confinements. At higher confinements the s-wave velocity data from the two tests run at different suctions come closer as the behavior is controlled by stress.

Figure 4.18. Effect of matric suction on s-wave velocity. Data from horizontally polarized bender elements during a CTC test on silt
4.4.4 Results from Conventional Triaxial Compression (CTC) Tests—Large Strain Data

Octahedral shear stress versus strain in the x, y, and z directions during shearing (loading and unloading) for a silt specimen tested at suction of 25 kPa and with different initial effective confinements are depicted in Figure 4.19.

\[ \text{Diagram showing octahedral shear stress versus strain for silt specimen tested at suction = 25 kPa and initial effective confinement stress of (a) } p' = 25 \text{ kPa, (b) } p' = 50 \text{ kPa, and (c) } p' = 100 \text{ kPa.} \]

**Figure 4.19:** Octahedral shear stress versus strain for silt specimen tested at suction = 25 kPa and at initial effective confinement stress of (a) $p' = 25$ kPa, (b) $p' = 50$ kPa, and (c) $p' = 100$ kPa.
Trends are alike to the ones seen on the TC test, i.e. the strength of the particulate media increases with increasing confinement. Figure 4.20 summarizes results from the shearing process (loading only for comparison) at three different soil suctions. Specimens were loaded only at a stress level where 2 to 3% of vertical strain was obtained because of high incidence of membrane failures at higher strains.

**Figure 4.20**: True triaxial testing results. Octahedral shear stress versus vertical strain for specimen tested at (a) suction = 50 kPa, (b) suction = 25 kPa, and (c) suction = 0 kPa.
4.5 Repeatability

First attempts in the experimental program resulted on large number of unsuccessful tests. Several were the problems causing the failed tests. Inappropriate loading rate and uncompleted homogenization of stresses before the application of shear caused obtaining erratic results. After these problems were resolved, short-circuited bender elements and membrane failures were still a source of troubles during testing. Adding the delays originated by these difficulties, not enough time was left in the program to replicate the test factorial. Nevertheless, data from tests that were stopped because of a membrane or bender element problem are compared in this section with results presented in this chapter. For instance, Figure 4.21 shows together traces of travel time data from two silt specimens during a CTC test at 25 kPa of matric suction. The data correspond to bender elements The arrivals of S-waves for both specimens are signalized with arrows. It is clear the proximity of the arrivals for the two different specimens.

![Figure 4.21](image)

**Figure 4.21:** S-wave arrivals on two different clayey silt specimens at 25 kPa of matric suction during a CTC test. Data correspond to vertically polarized s-waves
Shear wave velocities for the two specimens are presented in Figure 4.22. The maximum difference in velocities between the specimens is 4.7 percent, evidencing the capability of the system to produce consistent, repeatable results.

Figure 4.22: S-wave velocities during shearing of two different clayey silt specimens at 25 kPa of matric suction during a CTC test.

4.6 Summary

This chapter presents results from the modified true triaxial device, i.e. triaxial compression TC and conventional triaxial compression CTC test results on remolded silty soil specimens at low strain and large strain behavior soils are presented. These tests are the building blocks to study the small and large strain behavior of unsaturated soils under controlled state of stress conditions. Detailed descriptions of the soil characterization program and the testing equipment used in the investigation are included here. Emphasis is given to describe the modifications made to the true triaxial apparatus to accommodate the devices used to create the elastic waves (bender elements).
5.1 Introduction

Sand specimens are also tested on the modified true triaxial device to study the effect of stress and suction on the stiffness of this particulate material. Differences in the observed mechanical behavior between clayey silt (Chapter 4) and silica sand specimens are described throughout this chapter. The intention in this Chapter is to compare the behavior of these two different particulate materials. The specimens’ preparation procedure and testing program used on both types of soils are the same. Therefore, this chapter focuses mainly on presenting the results from the set of tests conducted on the sand specimens without repeating procedures already explained in the previous chapter. Analysis of results includes determination of fundamental parameters of mechanical behavior at both small and large strain. Results show more susceptibility of the clayey silt to variation of induced matric suction than the silica sand.

5.2 Testing Program

The same procedures are used on both silt and sand testing programs. These procedures include specimen preparation, testing factorial, stress paths, and presentation of results. A brief description of the most significant procedures follows. For a more detailed explanation of methods see Chapter 4.

All specimens were constructed at 10 percent moisture content and at the same void ratio. Multistage triaxial compression (TC) and conventional triaxial compression (CTC) tests are run at induced matric suctions of 0, 25, and 50 kPa. Each test comprises three shearing stages with initial confinements of 25, 50, and 100 kPa. The loading rate at both hydrostatic compression and shearing stages is 6 kPa/hour (i.e., 0.5 kPa-stress increments every five minutes). Continuous recording of stress, deformation, and shear wave velocity data is performed during the different loading procedures for posterior evaluation of the low strain and large strain behavior.

The silica sand used in this study is labeled E P K Sand (from The Feldespar Corporation, Edgar, Florida, C.A.S. No. 14808-60-7). Figure 5.1 gives the grain size distribution of this material. It is remarkable the particle size uniformity of this particulate media. As with the silt, specimen preparation is conducted on sand at 10 % water content.
5.3 Results

5.3.1 Results from Triaxial Compression (TC) Tests - Small Strain Data

Typical traces of arriving waves at the receiver bender element during a triaxial compression test are depicted on Figures 5.2 and 5.3. These figures correspond to the specimens run at induced matric suction of 25 kPa (the complete sets of traces from these tests are presented in appendix A). Traces from both hydrostatic compression and shearing stages are included. Figure 5.2 presents the traces from vertically polarized bender elements, whereas traces from horizontally polarized bender elements are shown in Figure 5.3.

The general trend during hydrostatic compression stages is that the shear wave velocity increases with increasing hydrostatic confinement. The trend is applicable to both horizontally and vertically polarized bender elements. Regarding the shearing stages of TC tests, the S-wave velocity of horizontally polarized bender elements decreases during loading and increases during unloading. These variations on S-wave velocity match the changes on horizontal stresses during a TC test. It is observed that the rate of variation in velocities decreases at higher confinement stress. The velocity of vertically polarized s-waves on the other hand, remains practically unaltered at the shearing stage of TC tests. Same general behavior is obtained in both types of soils: silt and sand. Traces from tests on sand are however usually cleaner from noise than those from the silty soil specimens (see Chapter 4). The reduced noise facilitates the identification of first arrivals.

![Figure 5.1: Grain size distribution of silica sand. (D<sub>50</sub> = 0.20 mm).](image-url)
Figure 5.2: Travel time data for vertically polarized bender elements: (a) description of stress paths during testing (suction \( u_a-u_w = 25 \) kPa), (b) travel time data during isotropic consolidation, and (c) travel time data during shearing (loading and unloading).
Figure 5.3: Travel time data for horizontally polarized bender elements: (a) description of stress paths during testing (suction $u_a - u_w = 25$ kPa), (b) travel time data during isotropic consolidation, and (c) travel time data during shearing (loading and unloading).
Figure 5.4: S-wave velocity data during a triaxial compression TC test on sand specimens at matric suctions of (a) 0 kPa, (b) 25 kPa, and (c) 50 kPa. References: ♦, ■, and ▲ indicate vertically polarized S-waves; ◊, □, and ∆ indicate horizontally polarized shear waves; $(\sigma_o - u_a)$ indicates initial net stress (hydrostatic).

Figure 5.4 presents a summary of the S-wave velocity versus deviatoric stress trends observed on the TC tests during the loading part of shearing stages started at different values of net hydrostatic stresses. The effect of net hydrostatic confinement on shear wave velocity is
easily seen here. Velocity of S-waves propagation increases with increasing net hydrostatic confinement, however when deviatoric stresses are applied the velocity of the vertically polarized S-waves remains constant while the velocity of the horizontally polarized S-waves decreases following the changes in the applied net stresses. The difference in velocity between vertically polarized S-waves ($V_{SV}$) and horizontally polarized S-waves ($V_{SH}$), however, does not stay constant among specimens with different matric suctions. Smaller difference in velocities is seen on specimens with higher suction values, which indicates the effect of matric suction on the stiffness of these sand specimens. The computed variations between $V_{SV}$ and $V_{SH}$ from Figure 5.4 are summarized in Figure 5.5.

![Figure 5.5: Effect of matric suction on small strain stiffness of sand specimens during shearing on TC tests.](image)

5.3.2 Results from Triaxial Compression (TC) Tests - Large Strain Data

Octahedral shear stress versus the strain in the X, Y, and Z directions during shearing (loading and unloading) are presented on Figure 5.6. The results correspond to a sand specimen tested at suction of 0 kPa and with different initial effective confinements. The octahedral shear stress is defined as in equation 4.1. As anticipated, that the strength of the particulate media increases with increasing confinement.
5.3.3 Results from Conventional Triaxial (CTC) Compression Tests - Small Strain Data

Figures 5.7 and 5.8 present traces of arriving waves corresponding to tests conducted on a sand specimen with induced matric suction of 25 kPa (Complete data from all these tests are documented in Appendix A). Traces from vertically polarized bender elements are in Figure 5.7 whereas Figure 5.8 presents traces from horizontally polarized bender elements. The trend obtained on hydrostatic compression stages is the same on every test, i.e. S-wave velocity increases with increasing hydrostatic confinement. This trend is valid for both horizontally and vertically polarized bender elements on TC and CTC tests, and with both type of particulate materials.
Figure 5.7: Travel time data for vertically polarized bender elements: (a) description of stress paths during testing (suction $u_a-u_w = 25$ kPa), (b) travel time data during isotropic consolidation, and (c) travel time data during shearing (loading and unloading).
Figure 5.8: Travel time data for horizontally polarized bender elements: (a) description of stress paths during testing (suction $u_a-u_w = 25$ kPa), (b) travel time data during isotropic consolidation, and (c) travel time data during shearing (loading and unloading).
Figure 5.9: S-wave velocity data during a conventional triaxial compression CTC test on sand specimens at matric suctions of (a) 0 kPa, (b) 25 kPa, and (c) 50 kPa. References: ♦, ■, and ▲ indicate vertically polarized S-waves; ◊, □, and ∆ indicate horizontally polarized shear waves; $\sigma_o$ indicates initial net hydrostatic stress.

On the shearing stages, the S-wave velocity of vertically polarized bender elements increases during loading and decrease during unloading. These changes on S-wave velocity are accord with the variation of vertical stresses on a CTC test, portraying the sensitivity of bender
elements to sense such variations. The rate of variation in velocities decreases at higher confinement stress. Results from horizontally polarized S-waves during shearing show that velocities remain practically unaltered as the horizontal stress on this stage are constant. Figure 5.9 summarizes these trends obtained during shearing stages on each test. Only loading parts are included here for easy comparison. The same trend is observed repeatedly: velocity of propagation of shear waves increases with increasing hydrostatic confinement.

The effect of the induced matric suction on S-wave velocity is again, as in the TC test results, manifested in the difference in velocities of S-waves vertically and horizontally polarized (see Figure 5.10). This difference becomes smaller on specimens with higher values of matric suction, that is, on stiffer specimens. This may be caused because matric suction is isotropic which reduces the anisotropy of effective stresses on different directions.

![Figure 5.10. Effect of matric suction on small strain stiffness of sand specimens during shearing on CTC tests.](image)

5.3.4 Results from Conventional Triaxial Compression Tests (CTC) – Large Strain Data

Octahedral shear stress versus strain in the X, Y, and Z directions during shearing (loading and unloading) for a sand of 25 kPa and with different initial effective confinements are depicted specimen tested at suction in Figure 5.11. Trends are alike to the ones seen on the TC test with both particulate materials, i.e. the strength of the particulate media increases with increasing net stresses.
Figure 5.11: Octahedral shear stress versus strain for sand specimen tested at suction = 25 kPa and at initial effective confinement stress of (a) $p'=25$ kPa, (b) $p'=50$ kPa, and (c) $p'=100$ kPa.
5.4 Repeatability

Reasons for not replicating the entire experimental program are exposed in section 4.5. However, the ability of the system to generate repeatable and trustful results is illustrated in Figure 5.12, which shows traces of travel time data from two sand specimens during hydrostatic compression and CTC test at 0 kPa of induced matric suction. The arrivals of S-waves for the two specimens are extremely close.

![Figure 5.12: S-wave arrivals on two different sand specimens at 0 kPa of matric suction during hydrostatic compression and CTC tests. Data correspond to horizontally polarized s-waves.](image)

5.5 Summary

Test results from triaxial compression TC and conventional triaxial compression CTC test conducted on silica sand specimens are presented in this chapter. The modified true triaxial device is used to assess the low strain and large strain behavior of the sand specimens at a range...
of stress and suction states. Relationship between shear wave velocity and effective state of stress from tests on silica sand presented here and tests on silty soil presented in chapter 4, show similar trends. These trends are the increase of S-wave velocity with increasing hydrostatic confinement, and the variation of shear wave velocities is result of variations on the effective stresses on the plane of polarization of S-waves. The effect of the induced matric suction on clayey silt specimens is more evident than in specimens constructed with silica sand. Sand specimens show the effect of matric suction in the homogenization of net stresses within the samples, explicitly, the difference between S-waves polarized in different directions decreases with increasing matric suction.
6.1 Modified Oedometer Device

The semi-empirical Equation 2.23 is used to compare the results obtained in this study to those predicted by it. This equation relates the shear wave velocity and the state of effective stresses in the plane of S-wave polarization. It is semi-empirical in the sense that it has a theoretical base on the Hertz theory of contacts between particles and coordination number or number of contacts. It uses two parameters $\beta$ and $\theta$, which depend on type of contacts between particles and type of packing, to correlate the effective stresses and the velocity. Exponent $\beta$ gives an indication of the susceptibility of the particulate medium to changes in the state of stresses. The factor $\theta$ is the shear wave velocity of the medium at 1 kPa of effective stress. The empirical parameters $\beta$ and $\theta$ obtained from vertically polarized bender elements on the oedometer testing for sand specimens are compared with the values proposed by Santamarina et al. (2001 - fit line for different particulate media) in Figure 6.1.

![Figure 6.1: Parameters $\beta$ and $\theta$ from sand specimens tested under $K_o$ conditions compared with values proposed by Santamarina et al. 2001. Data from vertically polarized bender elements.](image-url)
There are two appreciable differences in the results from the two sets of data on Figure 6.1. The β values during loading are consistently higher than those at unloading because the specimens suffer irrecoverable changes in fabric. The sand specimens are looser during loading and thus, the S-wave velocity on the soil is more susceptible to stress variations. This seems to be the effect of pre-consolidation on S-wave velocity. Furthermore, there are some important differences between the dry specimen and the unsaturated soil. All the points that correspond to the unsaturated specimens form a line that falls lower than the line proposed by Santamarina et al. (2001) and the data presented for the dry soil. This observation seems to be a clear indication of the effect of suction on the specimen: the exponent β decreases its value with suction yielding a parallel, yet θ–β relation. The power function used in this study for fitting the test data can be applied on soils of any gradation without altering its form since it takes into account effects from changes on porosity, coordination number, fabric, and contact behavior (see Equation 2.21). The power low model changes in form for unsaturated soils to take into account the effect of the capillary forces (see Equation 2.24 and 6.6)

This effect occurs mainly because meniscus water causes a compression force that helps to hold particles together due to surface tension and negative water pressure phenomena. A representation of these phenomena is seen in Figure 6.2.

![Figure 6.2: Representation of the effect of meniscus water in matric suction (modified from Mancuso et al. 2002).](image-url)
Figure 6.2 makes clear the beneficial effect of menisci water on suction does not increase indefinitely since the wetted area of contact between the particles reduces with increasing suction until it becomes zero at a completely dry medium.

6.1.1 Proposed Model for the Evaluation of S-wave Velocity

The equation that represent the S-wave velocity presented in Chapter 2 shows that the wave velocity is a physical property that depends on the elastic and inertial properties of the medium of propagation:

$$V_s = \sqrt{\frac{G}{\rho}}$$

(6.1)

where $G$ is the elastic shear modulus and $\rho$ is the mass density. In the case of soils, the value of the shear modulus and mass density depends on many parameters, including applied stresses, degree of cementation, porosity, water content, specific gravity, and grain size distribution (e.g., Roesler 1977; White 1981, Cho and Santamarina 2001; Fernandez and Santamarina 2001; Fratta et al. 2004). Furthermore, the shear stiffness would also depend on the shear stiffness of the soil’s different phases: air, water, and solid minerals grain stiffness, and capillarity force-controlled skeleton shear modulus.

The mass density of the soil $\rho_{soil}$ considering all soil components is:

$$\rho_{soil} = \rho_w G_s (1-n) + \rho_w n S_r$$

(6.2)

where $\rho_w$ is the water mass density, $G_s$ is the specific gravity of the solid minerals, $n$ is the porosity and $S_r$ is the degree of saturation. The shear stiffness of the soil $G_{soil}$ may be expressed as:

$$G_{soil} = \frac{1}{n \left( \frac{S_r}{G_w} + \frac{1-S_r}{G_s} \right) + \frac{1-n}{G_s} + f(\sigma, S_r, c)}$$

(6.3)
where \( G_w, G_a \) and \( G_g \) are the shear stiffness of water, air and solid minerals; and \( f(\sigma, S_r, c) \) is a function of the applied stresses \( \sigma \), the degree of saturation \( S_r \) and the degree of cementation and it represents the shear stiffness of the soil skeleton. In most remolded soils, the effect of cementation is not important, but most near subsurface deposits are unsaturated and the effect of degree of saturation is utmost important. Furthermore, in most cases the skeleton shear stiffness is dominant:

\[
\frac{1}{\left(\frac{S_r}{G_w} + \frac{1-S_r}{G_a}\right) + \frac{1-n}{G_g}} \ll f(\sigma, S_r)
\]  

(6.4)

The challenge is to find a function that would properly represent the effect of both the net stresses \((\sigma-u_a)\) and suction \((u_a-u_w)\) on the stiffness and in the S-wave velocity of soils. It is proposed the following function:

\[
G_{\text{soil}} = f = \theta \cdot \left(\frac{\sigma-u_a}{p_t}\right)^\chi + \xi \cdot (u_a-u_w) \cdot (1-S_r)^\eta
\]

(6.5)

where \( \theta, \chi, \xi, \eta \) are parameters that depend on the type of soils and are documented by Mindlin and Duffy (1954), Richard et al. (1970); Roessler (1977); White (1981), Cho and Santamarina (2002), and Fratta et al. (2004) among other researchers. Replacing Equations 6.2 and 6.5 into Equation 6.1:

\[
V_s = \frac{1}{\rho_w G_s (1-n) + \rho_n n S_r} \theta \cdot \left(\frac{\sigma-u_a}{p_t}\right)^\chi + \xi \cdot (u_a-u_w) \cdot (1-S_r)^\eta
\]

(6.6)

yields an equation that incorporates net pressure and suction as controlling parameters of the S-wave velocity. In equation 6.6, the external stress \( \sigma \) is the average stresses in the plane of wave polarization. For soils tested at very low confining stresses, the first term in Equation 6.5 vanishes and Equation 6.5 may be used to evaluate the effect of degree of saturation in the S-
wave velocity as shown by Fratta et al. (see Figure 6.3). In this study, both the stresses and degree of saturation (suction) are changed and Equation 6.6 is used in the evaluation and interpretation of the wave propagation data.

**Figure 6.3:** Shear wave velocity versus degree of saturation: Experimental data and theoretical model. (a) Granite Powder: the model fits the data between 30 and 100% saturation (0.12 and 0.4 volumetric water content). (b) Sand Boil Sand: the model fits the data between 10 and 90% saturation (0.034 and 0.30 volumetric water content – after Fratta et al. 2004 – experimental data: Cho and Santamarina 2001)
Figure 6.4 presents estimated and measured results of shear wave velocity against effective stress in the polarization plane for the silica sand specimens at different matric suctions. Equation 6.6 is used to estimate the relationship between these parameters. Results from clayey silt specimens are presented in Figure 6.5.

**Figure 6.4:** Estimated and measured results of shear wave velocity against effective stress in the polarization plane for the silica sand specimens (continuous lines indicate estimated results, symbols are measured results)
Figure 6.5: Estimated and measured results of shear wave velocity against effective stress in the polarization plane for the clayey silt specimens (continuous lines indicate estimated results, symbols are measured results).

Evaluation of the effect of matric suction presented in Figures 6.4 for the sand specimens makes evident that specimens with higher values of initial matric suction have higher stiffness as indicated by the S-wave velocity results. As for the case of silts, the results in Figure 6.5 are less clear. Travel times of S-waves present less variation with stress than on sand specimens. This behavior may be motivated by the higher suctions present in the clayey silt specimens, which makes them stiffer. Nevertheless, with the exception of the wettest specimen (not shown), higher initial matric suction produced higher velocity of S-waves. Erratic behavior encountered on the specimen with the highest water content may be motivated because the soil might be approaching saturation.
6.1.2 Determination of Stress Anisotropy Using S-wave Velocity

Measuring S-wave velocities polarized in different and perpendicular directions also makes possible to assess the stress anisotropy of the medium. For this purpose, the coefficient of lateral stress at rest $K_o$ is determined. $K_o$ is the ratio of horizontal to vertical effective stresses and is approximately constant for one-dimensional loading. This parameter is difficult to measure in the field, however the evaluation of the $K_o$ coefficient may be simplified by using shear waves polarized in different directions and using the following analysis (see also Equation 2.21)

$$V_s = \theta \left( \frac{\sigma_{||} + \sigma_{\perp}}{2 \sigma_{\text{ref}}} \right)^\beta$$  \hspace{1cm} (6.7)

where $\sigma_{||}$ and $\sigma_{\perp}$ are the effective stresses in a direction parallel and perpendicular to the direction of wave propagation and $\beta$ is either calculated from a particular set of data or assumed from the range of values in Figure 2.20 (here is calculated from test results on sands to be 0.20):

$$V_{s\parallel} = \theta \left( \frac{\sigma_v (1 + K_o)}{2 \sigma_{\text{ref}}} \right)^\beta$$ \hspace{1cm} (6.8) \hspace{1cm} \text{vertically polarized S-wave velocity}

$$V_{s\perp} = \theta \left( \frac{K_o \sigma_v}{\sigma_{\text{ref}}} \right)^\beta$$ \hspace{1cm} (6.9) \hspace{1cm} \text{horizontally polarized S-wave velocity}

the ratio of Equations 6.8 and 6.9 may be used to evaluate the value of the coefficient of lateral stress at rest $K_o$:

$$\frac{V_{sv}}{V_{sh}} = \left( \frac{1 + K_o}{2K_o} \right)^\beta$$  \hspace{1cm} (6.10)

and finally, the $K_o$ coefficient could be found as:

$$K_o = \frac{1}{2^{\frac{\beta}{\beta}} \left( \frac{V_{sv}}{V_{sh}} \right)^{\frac{\beta}{\beta}} - 1}$$  \hspace{1cm} (6.11)
The results of its determination for sand specimens are presented in Figure 3.19. All of the specimens present a variation of $K_o$ with effective vertical stress, being this variation smaller at higher state of stresses. It is also observed that the variation differs according to the initial matric suction of the specimens. As suction increase, the shear strength of the soil also increases and the ratio of the horizontal over the vertical “effective” stresses as felt by the propagating waves increases and $K_o$ becomes lower. This difference becomes smaller as the applied external stresses increase. Specimens with higher suctions have lower variation of $K_o$.

6.2 Modified True Triaxial Device

6.2.1 Small Strain Analysis

Expected behavior. Behavior from TC and CTC can be anticipated using equation 2.21 and the stress path at which the specimen will be subjected. For instance, the stress path followed on a TC test during shearing is depicted in Figure 6.6. Furthermore, the velocity of propagation of shear waves traveling in the horizontal direction and polarized horizontally ($V_{S-HH}$) can be calculated as:

$$V_{S-HH} = \alpha \left[ \frac{\sigma_x + \sigma_y}{2} \right]^\beta$$  \hspace{1cm} (6.12)

which, using notation from Figure 6.6 gives:

![Figure 6.6: Application of stresses during shearing on a TC test.](image)
\[ V_{S-HH} = \alpha \left[ \frac{\sigma_0 - \Delta \sigma}{2} \right]^{\beta} \]  

(6.13)

that reduces to:

\[ V_{S-HH} = \alpha \left[ \frac{\sigma_0 - \Delta \sigma}{2} \right]^{\beta} \]  

(6.14)

This indicates that an increment in stress will render a decrease in \( V_{S-HH} \). For shear waves traveling in the horizontal direction but polarized vertically (\( V_{S-HV} \)) the velocity of propagation can be obtained by:

\[ V_{S-HV} = \alpha \left[ \frac{\sigma_x + \sigma_z}{2} \right]^{\beta} \]  

(6.15)

using Figure 6.6 notation gives:

\[ V_{S-HV} = \alpha \left[ \frac{(\sigma_0 + \Delta \sigma) + (\sigma_0 - \Delta \sigma)}{2} \right]^{\beta} \]  

(6.16)

that reduces to:

\[ V_{S-HV} = \alpha \left[ \frac{\sigma_0 + \Delta \sigma}{4} \right]^{\beta} \]  

(6.17)

This indicates that an increment in stress will make \( V_{S-HV} \) to increase. Plotting these velocities against deviator stress determined following the same scheme and choosing \( \alpha \) and \( \beta \) parameters typical for sand (i.e., \( \alpha = 0.75 \) and \( \beta = 0.22 \)) renders the relations depicted by lines in Figure 6.7. Symbols in Figure 6.7 indicate measured results.
Figure 6.7: Expected and obtained behavior of sand specimens during shearing on TC test at induced matric suction of (a) 0 kPa, (b) 25 kPa, and (c) 50 kPa. Lines indicate estimated results.

A similar procedure can be followed for the CTC test. In this case the stresses applied to the specimen during shearing are illustrated in Figure 6.8. Here, the velocity of propagation of shear waves traveling in the horizontal direction and polarized horizontally ($V_{S\text{-HH}}$) are obtained using equation 6.12 that transforms to:

Deviator Stress [kPa] = 25 kPa

Deviator Stress [kPa] = 50 kPa

Deviator Stress [kPa] = 100 kPa

S-wave velocity [m/s]

(a)

(b)

(c)
\[ V_{S-HH} = \alpha \left[ \frac{\sigma_0 + \sigma_0}{2} \right]^\beta \]  

(6.18)

**Figure 6.8:** Application of stresses during shearing on a CTC test.

Therefore, \( V_{S-HH} \) is basically constant in this test. The velocity of propagation of shear waves traveling horizontally but polarized vertically (\( V_{S-HV} \)) is determined with:

\[ V_{S-HV} = \alpha \left[ \frac{\sigma_0 + \Delta \sigma + \sigma_0}{2} \right]^\beta \]  

(6.19)

that reduces to:

\[ V_{S-HV} = \alpha \left[ \frac{\sigma_0 + \Delta \sigma}{2} \right]^\beta \]  

(6.20)

Thus, \( V_{S-HV} \) is expected to increase during shearing on the CTC test. Graphically, the expected behavior during a CTC test on sand specimens is illustrated by the continuous lines in Figure 6.9. Measured results on this test are included for comparison.
Figure 6.9: Expected and obtained behavior of sand specimens during shearing on CTC test at induced matric suction of (a) 0 kPa, (b) 25 kPa, and (c) 50 kPa. Lines indicate estimated results.

It is observed in Figures 6.7 and 6.9 that the closest match to estimated values is from specimens with 50 kPa of initial net stress. Measured results of specimens with initial confinement of 25 kPa fall below the expected values, whereas results from specimens with 100 kPa are above the predicted results. This indicates that the parameters $\alpha$ and $\beta$ in the power function used to get estimations of the S-wave velocity should not be constant, but function of
the state of stresses. The variation of S-wave velocities measured with the bender elements
during the tests however, followed in general the changes of stress on the plane of polarization.
The only exception is on the shear wave velocities obtained from vertically polarized bender
elements on a TC test. In that case it is expected that the velocity of S-waves increase during
loading and decrease during unloading. In contrast, S-wave velocity from test results remain in
general constant at these stages and in some cases (at low confinement) they even follow the
trend of horizontally polarized shear waves, i.e. decreasing during loading and increasing at the
loading part of the test.

With respect to the effect that the induced matric suction has on soil stiffness (higher
stiffness on specimens with higher suction), the effect is more evident in the silty soil (see
Figures 4.12 and 4.18). This result is explicable because finer soils are able to sustain higher
values of suction that provide them with additional stiffness. Nevertheless, the matric suction
effect on sands specimens is noticed on the homogenization of net stresses as shown in Figures
5.5 and 5.10.

6.2.2 Large Strain Analysis

In several occasions, loading of specimens in the modified multiaxial device was done until
a strain between 2 and 3 % was reached. A high incidence of membrane’s failures occurred at
larger strains, especially at high confinement. Therefore, the maximum shear strength in many
tests was not achieved. The use of the hyperbolic model (Equation 6.21) permits to estimate the
maximum shear strengths and stiffness for those tests. They are plotted against the stress state
variables net normal stress ($\sigma - u_a$) and matric suction ($u_a - u_w$) in Figure 6.10a. Figure 6.10b
presents the deviatoric stress at failure against the same stress variables and Figure 6.10c depicts
the failure envelope at each test with different matric suction. Each data point in Figure 6.10
corresponds to a different specimen. The failure surfaces obtained show in general that the shear
strength and stiffness of the soil specimens increase with both increments in net stress and
increments in matric suction. Figure 6.11 present the results from CTC tests on silt specimens,
whereas Figures 6.12 and 6.13 show the data from sand specimens on TC and CTC test
respectively.

$$\tau = \frac{\varepsilon_z}{1 + \frac{\varepsilon_z}{\tau_{\text{max}}}}$$  \hspace{1cm} (6.21)
Figure 6.10: Variations of stiffness and strength against net stress and matric suction on clayey silt specimens during a TC test.
Figure 6.11: Variations of stiffness and strength against net stress and matric suction on clayey silt specimens during a CTC test.
Figure 6.12: Variations of stiffness and strength against net stress and matric suction on sand specimens during a TC test.
Figure 6.13: Variations of stiffness and strength against net stress and matric suction on sand specimens during a CTC test.
Chapter 7
Conclusions and Recommendations for Future Work

7.1 $K_o$-condition Testing Results

$K_o$-condition testing results on sand specimens show that the S-wave velocity increases with increasing effective vertical stress. This behavior is in agreement with expected results because the stiffness of particulate media is controlled by the state of effective stress. Higher velocities of S-waves are seen on specimens with higher initial matric suctions. This result shows the effect of matric suction on small strain stiffness, i.e. small strain stiffness increases with increasing soil matric suction especially at low confinement. Plots of the experimentally determined $\theta$ and $\beta$ parameters also show the effect of suction: the exponent $\beta$ decreases as the interpretation mask the effect of suction on the s-wave velocity (this observation is true at low confinement stresses)

The assessment of stress anisotropy on sand specimens tested under no lateral strain, and at different matric suctions, show that the estimated coefficient of lateral stress $K_o$ decreases with increasing effective vertical stress. This variation is observed especially at low confinements, and in specimens with low values of initial matric suctions. $K_o$ is expected to be approximately constant for one-dimensional loading. The variations observed range from 12 to 30 percent, corresponding the highest change to the sand specimen with the lowest matric suction; and conversely, the lowest variation in $K_o$ corresponds to the sand specimen with the highest initial matric suction. Lower $K_o$ coefficients are seen on specimens with higher values of matric suction. This results agrees with expected behavior, explicitly, since soil matric suction is an isotropic property, a reduction in the stress anisotropy on the specimens with higher matric suctions is expected.

Results from unsaturated clayey silt specimens show a small variation of the S-wave travel time with applied vertical stress. This behavior may indicate the effect of suction on stiffness for the clayey silt specimens, where a stronger effect is expected than in sand specimens. In general the unsaturated clayey silt specimens with higher suctions show higher S-wave velocities.

The effect of suction is also observed in the reduction of hysteresis on S-wave velocity during application and removal of vertical stresses as the matric suctions are increased. This
phenomenon is caused by decreased susceptibility of the material to changes in stiffness with variation of applied effective stress.

The use of the power law model to approximate the relationship between S-wave velocity and effective stresses on the plane of shear wave polarization seems appropriate in this study. Results from the modified true triaxial device, however suggest that the effective stresses parallel and perpendicular to the wave propagation should use a different exponent.

7.2. Multiaxial Testing Results

With respect to the evaluation of small strain behavior of these particulate materials, the bender elements used to monitor the small strain stiffness during the stress paths applied, efficiently sense variations of stress on the direction of shear wave polarization on the test conducted on the modified true triaxial device. This is repeatedly observed in hydrostatic compression stages: the velocity of S-waves increases with increasing confinement for both vertically and horizontally polarized S-waves. In most cases, the S-wave velocity increases with the applied stress on the direction of S-wave polarization during shearing stages.

The effect of suction on the small strain stiffness of the particulate media is seen only on the silty soil. The S-wave velocity of sand specimens in this study is not influenced by the induced matric suction. This may be explained because the relatively big and uniform pore sizes of the sand (compared to the silty soil) are not capable to sustain the matric suction values induced. Because of its uniformity, the majority of the pores drain at a given level of matric suction leaving the soil without the beneficial effect of the menisci water on suction, and thus in stiffness.

Large strain analysis of the test results unsurprisingly shows that the shear strength of the particulate materials increases with confinement. The effect of suction on shear strength invariably shows that at low confinements the specimens with higher induced matric suctions present the higher values on shear strength. An intriguing result is observed on the silt specimen with no induced suction at high confinement. At high confinement this specimen with the lowest suction presents the biggest shear strength although at low confinement it has the lowest strength.
7.3 Recommendations for Future Work

Whenever possible, the use of parallel bender elements should be preferred over the series type to reduce the distortion produced by cross-taking and near field effects on the received traces. Their higher cost is compensated with the save in time that it takes to prepare the series bender elements to diminish such effects.

Another alternative for avoiding the problems caused by the exposure of bender elements to humid environments might be the use of another device capable of generating shear waves such as flat shear plates. In view of the difficulties encountered on this investigation for the fragile nature of bender elements, it would be worthy to investigate the feasibility of using other options.

A better assessment of the effect of suction on the stiffness of soils in the modified triaxial device might be to perform multistage testing varying the matric suction on the same specimen. That way, the changes of suction belong to the same soil-water characteristic curve, and they are not individual points from different curves as in the case of using different specimens.

The issue of the non-changing travel time arrivals in the unsaturated silt specimens tested on the oedometer cell needs to be investigated. This phenomenon occurred in repeated specimens in which measurements of matric suction were also being recorded with a tensiometer porous cup.

Although using a tensiometer to make direct measurements of matric suction is relatively simple, it is limited to measurement of matric suctions of up to about 90 kPa (-90 kPa of water tension) because of cavitation of water at tensions approaching -101 kpa. Therefore another method to measure suction on laboratory specimens might be better.

Monitoring of suction on the modified true triaxial device is desirable to verify that the suction induced by the difference of pore air and pore water pressures applied to the soil specimen are achieved and maintain constant during the test.
References


Appendix A
Travel Time Data: Clayey Silt

TC Test
Soil: Silt
Net Pressure: \( \sigma_{o-ua} = 25, 50, \text{ and } 100 \text{ kPa} \)
Suction: \( u_a-u_w = 0 \text{ kPa} \)

**Figure A.1:** Travel time data for vertically polarized bender elements: (a) description of stress paths during testing (suction \( u_a-u_w = 0 \text{ kPa} \)), (b) travel time data during isotropic consolidation, and (c) travel time data during shearing (loading and unloading).
Figure A.2: Travel time data for horizontally polarized bender elements: (a) description of stress paths during testing (suction $u_a-u_w = 0$ kPa), (b) travel time data during isotropic consolidation, and (c) travel time data during shearing (loading and unloading).
Figure A.3: Travel time data for vertically polarized bender elements: (a) description of stress paths during testing (suction $u_a-u_w = 25$ kPa), (b) travel time data during isotropic consolidation, and (c) travel time data during shearing (loading and unloading).
Figure A.4. Travel time data for horizontally polarized bender elements: (a) description of stress paths during testing (suction $u_a-u_w = 25$ kPa), (b) travel time data during isotropic consolidation, and (c) travel time data during shearing (loading and unloading).
Figure A.5. Travel time data for vertically polarized bender elements: (a) description of stress paths during testing (suction $u_a-u_w=50$ kPa), (b) travel time data during isotropic consolidation, and (c) travel time data during shearing (loading and unloading).
Figure A.6. Travel time data for horizontally polarized bender elements: (a) description of stress paths during testing (suction $u_a-u_w = 50$ kPa), (b) travel time data during isotropic consolidation, and (c) travel time data during shearing (loading and unloading).
Figure A.7. Travel time data for vertically polarized bender elements: (a) description of stress paths during testing (suction $u_a-u_w = 0$ kPa), (b) travel time data during isotropic consolidation, and (c) travel time data during shearing (loading and unloading).
Figure A.8. Travel time data for horizontally polarized bender elements: (a) description of stress paths during testing (suction $u_a-u_w = 0$ kPa), (b) travel time data during isotropic consolidation, and (c) travel time data during shearing (loading and unloading).
Figure A.9. Travel time data for vertically polarized bender elements: (a) description of stress paths during testing (suction $u_a-u_w = 25$ kPa), (b) travel time data during isotropic consolidation, and (c) travel time data during shearing (loading and unloading).
Figure A.10. Travel time data for horizontally polarized bender elements: (a) description of stress paths during testing (suction $u_a-u_w = 25$ kPa), (b) travel time data during isotropic consolidation, and (c) travel time data during shearing (loading and unloading).
Figure A.11. Travel time data for vertically polarized bender elements: (a) description of stress paths during testing (suction \( u_a-u_w = 50 \text{ kPa} \)), (b) travel time data during isotropic consolidation, and (c) travel time data during shearing (loading and unloading).
Figure A.12. Travel time data for horizontally polarized bender elements: (a) description of stress paths during testing (suction $u_a-u_w = 50$ kPa), (b) travel time data during isotropic consolidation, and (c) travel time data during shearing (loading and unloading).
Appendix B
Travel Time Data: Silica Sand

Figure B.1: Travel time data for vertically polarized bender elements: (a) description of stress paths during testing (suction $u_a-u_w=0$ kPa), (b) travel time data during isotropic consolidation, and (c) travel time data during shearing (loading and unloading).
Figure B.2: Travel time data for horizontally polarized bender elements: (a) description of stress paths during testing (suction $u_a-u_w = 0$ kPa), (b) travel time data during isotropic consolidation, and (c) travel time data during shearing (loading and unloading).
Figure B.3: Travel time data for vertically polarized bender elements: (a) description of stress paths during testing (suction $u_a-u_w = 25$ kPa), (b) travel time data during isotropic consolidation, and (c) travel time data during shearing (loading and unloading).
Figure B.4: Travel time data for horizontally polarized bender elements: (a) description of stress paths during testing (suction $u_a - u_w = 25$ kPa), (b) travel time data during isotropic consolidation, and (c) travel time data during shearing (loading and unloading).
Figure B.5: Travel time data for vertically polarized bender elements: (a) description of stress paths during testing (suction $u_a-u_w = 50$ kPa), (b) travel time data during isotropic consolidation, and (c) travel time data during shearing (loading and unloading).
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CTC Test  
Soil: Sand  
Net Pressure:  
\( \sigma_{o-ua} = 25, 50, \text{ and } 100 \text{ kPa} \)  
Suction:  
\( u_a - u_w = 50 \text{ kPa} \)

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\begin{align*}
\sigma_1 &= \sigma_o + \Delta \sigma \\
\sigma_2 &= \sigma_o \\
\sigma_3 &= \sigma_o
\end{align*}
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Figure B.12: Travel time data for horizontally polarized bender elements: (a) description of stress paths during testing (suction \( u_a - u_w = 50 \text{ kPa} \)), (b) travel time data during isotropic consolidation, and (c) travel time data during shearing (loading and unloading).
Vita
Oscar Fernando Porras Ortiz was born in Durango, Durango, Mexico, on May 25, 1963. He received his Bachelor of Science in Civil Engineering from Instituto Tecnológico de Durango, Mexico. After working on the construction industry and for Secretaría de Educación Pública for several years, he went to the United States to continue his education. He obtained a Master of Science in Civil Engineering degree in the area of asphalt technology from Louisiana State University in December 2000. Subsequently, he stayed at the same institution pursuing a doctoral degree where he expects to receive the degree of Doctor of Philosophy in civil engineering in December 2004.