EFFECT OF SOIL-STRUCTURE INTERACTION ON THE BEHAVIOR OF
OFFSHORE PILES EMBEDDED IN NONLINEAR POROUS MEDIA

by

Mohamad Jawad K. Essa Al-Younis

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As members of the Dissertation Committee, we certify that we have read the dissertation prepared by Mohamad Jawad K. Essa Al-Younis entitled Effect of Soil-Structure Interaction on the Behavior of Offshore Piles Embedded in Nonlinear Porous Media and recommend that it be accepted as fulfilling the dissertation requirement for the Degree of Doctor of Philosophy.

Date: 12/19/2012
Dr. Chandrakant S. Desai

Date: 12/19/2012
Dr. Achintya Haldar

Date: 12/19/2012
Dr. Hamid Saadatmanesh

Date: 12/19/2012
Dr. Lianyang Zhang

Final approval and acceptance of this dissertation is contingent upon the candidate’s submission of the final copies of the dissertation to the Graduate College.

I hereby certify that I have read this dissertation prepared under my direction and recommend that it be accepted as fulfilling the dissertation requirement.

Date: 12/19/2012
Dissertation Director: Dr. Chandrakant S. Desai
STATEMENT BY AUTHOR

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SIGNED: ___________________

Mohamad Alyounis
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DEDICATION

I would like to dedicate this dissertation

to the memory of my father,

my mother and my wife
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ABSTRACT

Pile foundations that support offshore structures are required to resist not only static loading, but also dynamic loading from waves, wind and earthquakes. The purpose of this study is to gain a better understanding of the behavior of offshore piles under cyclic or dynamic loading using the finite element approach. To achieve this goal, an appropriate constitutive model is required to simulate the behavior of soils and interfaces. The DSC constitutive model is developed for saturated interfaces to study the behavior under severe shear deformation at the soil-pile interface.

Monotonic and cyclic simple shear experiments are conducted on Ottawa sand-steel interfaces under drained and undrained conditions using the Cyclic-Multi-Degree-of-Freedom shear device with porewater pressure measurement (CYMDOF-P). The effect of various parameters such as normal stress, surface roughness of steel, type of loading, and the amplitude and frequency of the applied displacement in two-way cyclic loading are investigated. The data from the simple shear tests on saturated interfaces are used to calculate the parameters in the DSC model. The resulting parameters are then used to verify the DSC model by back predicting tests from which parameters are determined and independent tests that are not used in parameters determination. The model predictions, in general, were found to provide a highly satisfactory correlation with the observations.

In the context of DSC, the concept of critical disturbance is developed to identify initiation of liquefaction in saturated Ottawa sand-steel interfaces. This method is based
on using microstructural changes in material as an indication of liquefaction identification.

The finite element method, along with DSC constitutive model, is used to investigate the response of offshore piles to dynamic loading. These include cyclic loading of axially loaded instrumented pile in clay and full-scale laterally loaded pile in sand. The DSC model is used to model the nonlinear behavior of saturated soils and interfaces. A nonlinear dynamic finite element program DSC-DYN2D based on the DSC modeling approach and the theory of nonlinear porous media is used for this purpose. Results from numerical solutions are compared with field measurements. Strong agreement between numerical predictions and field measurements are an indication of the ability to solve challenging soil-structure interaction problems.

Based on the results of this research, it can be stated that the finite element-DSC model simulation allows realistic prediction of complex dynamic offshore pile-soil interaction problems, and is capable of characterizing behavior of saturated soils and interfaces involving liquefaction.
CHAPTER 1: INTRODUCTION

1.1. General

Pile foundations are very common and important structures for offshore engineering, which can be used as a foundation for offshore platforms, offshore ports, wind energy plants, long-span cross-bay bridges, etc. Offshore platforms have been widely used for a variety of applications such as oil and gas production, military and navigational purposes. About 20% of the world’s oil production is extracted from offshore reservoirs. Currently, there are over 3000 offshore platforms in the United States and an equal number of offshore structures around the world. Pile foundations are the most common for Steel Jacket Platform and Tension Leg Platform.

Steel Jacket Platform, Fig.(1.1), is essentially a space truss constructed of tabular steel members and fastened to the sea floor with piles. Tension Leg Platform, Fig.(1.2), is a floating buoyant platform in which very large tensile forces are transmitted to the pile foundation.

Offshore wind energy plants promise to become an important source of energy in the near future. It is expected that within 10 years, wind farms with a total capacity of thousands of megawatts will be installed in the seas around the world. An offshore pile foundation for wind energy plants, for example the tripod system, Fig.(1.3), consists of steel frame transferring the forces from the tower to primarily tension and compression forces in three steel piles driven into the seabed.
Due to incomplete knowledge concerning offshore structures as well as their environmental factors and resulting inappropriate design, some marine structure failure incidents have occurred in recent years (Smith and Gordon 1983, Lundgren et al., 1989, Silvester and Hsu 1989). Therefore, a safe design of offshore piles remains a challenge for ocean engineers. Historically, research on piles has been directed primarily toward the determination of static ultimate capacity. For pile foundations with constant, determinate service loads, and with no environmental loading, such an approach to the design may be adequate. For offshore piles, the environmental loads may equal or exceed the service loads. Experiments have shown that, under the action of repeated loading, the actual pile capacity may be reduced to only a fraction of the static capacity (Matlock and Holmquist, 1967). Clay softens under repeated loading, and saturated sands demonstrate excess pore pressure buildup and sometimes bring the soil skeleton close to failure.

Offshore piles are subjected to several forms of cyclic or repeated loading which is frequently the most critical design situation for the foundation. In any marine location the environmental loading by wind and waves can cause enormous forces on the offshore structures, particularly, in rough weather conditions. The proximity of seismically active plate margin areas adds earthquake loading as a probability within the structure’s design lifetime. All of these situations present the question of how the capacity of piles is influenced by dynamic loading.

Cyclic load applied to submerged soil, such as offshore pile foundations, generates transient pore pressure motion relative to the skeleton for the transient redistribution of pore pressure. The excess porewater pressure may enhance or erode
material strength depending upon whether the pore fluid is stressed in tension or compression. For example, rapid loading such as an earthquake on saturated cohesionless soils can lead to generation of high excess pore pressure in the soil mass which may cause liquefaction. Offshore piles founded on a soil that dynamically liquefy will fail regardless of its structural integrity, as observed during the 1964 Niigata earthquake in Japan and the 2010 Chile earthquake. Figure (1.4) shows the collapse of Shawa Bridge during the 1964 Niigata earthquake. Similar failure was also observed of the Bio-Bio Bridge during the 2010 Chile earthquake, Fig.(1.5). A better understanding to the generation of excess pore pressure, which is the main cause of soil liquefaction, requires consideration of porous media theory.

Porous geologic media consists of soil skeleton and water in voids between soil particles. Deformability of porous media depends on the resistance of the individual phases against deformation and coupling between the two phases. When an external excitation acts upon fully saturated geologic media, resistance is developed by the soil skeleton and by water in the pores. This changes the stress in the pore water resulting in a motion of pore water. As the pore water flow continues, the pressure is gradually transferred to the soil skeleton, which results in its continuous deformation. In some cases, excitation would be so rapid that there would be no time for the dissipation of pore water pressure. This may bring the soil skeleton to a state close to failure.

For modeling the behavior of saturated soil-structure interaction, it is necessary to characterize the nonlinear behavior of the soils and interfaces between soils and structures. In order to obtain improved and realistic solutions for such problems, use of
appropriate constitutive model is important. In the past, design practices often ignore or grossly simplify the influence of behavior of interfaces on the overall response of the structure-foundation system. Under loading, relative movements can occur between the structure and the foundation which can eventually lead to failure of the entire system. Thus it is believed that ignoring or simplifying the role of interfaces can cause significant errors in design of structures and foundations. This area of research has attracted researchers to study the basic material behavior of interfaces and its modeling, under static and dynamic loading conditions, and establish the influence of interfaces on the overall behavior of the structure-foundation system.

Advanced constitutive models have been recently proposed to describe the behavior of geological materials, structures, and interfaces. Besides elastic models, classical plasticity models such as Mohr-Coulomb and Drucker-Prager models, advanced continuous yield models such as the critical state, cap, and Hierarchical Single Surface (HISS) models have been developed. To account for discontinuities, the disturbed state concept (DSC) model has been proposed based on the idea of observed behavior dependent on components of the material. The DSC model has been successfully verified with respect to test data for a wide range of geotechnical materials.

Considerable amount of research has been done to model and test interfaces under monotonic and quasi-static loading. Some research toward cyclic loading under dry conditions has been reported. Since many practical applications such as offshore piles involve factors like cyclic loading, saturated interfaces and porewater pressure, it is
desirable to undertake research towards an enhanced understanding of interface behavior under such conditions.

1.2. Scope of Research

The proposed research aims at;

1. Modify the CYMDOF-P device for saturated interfaces.

2. Perform a series of normal compression tests and interface simple-shear tests with dry and saturated Ottawa sand-steel with different steel roughnesses using CYMDOF-P device to identify important aspects of interface material behavior.

3. Define and determine of surface roughness parameters for the steel samples used in this study.

4. Develop the Disturbed State Concept (DSC) for interface behavior of saturated sand-steel interfaces.

5. Develop techniques for determination of DSC model parameters using laboratory results of saturated Ottawa sand-steel interface.

6. Verify the DSC model with respect to the tests used in parameter determination and back predicting independent tests that are not used in parameter determination.

7. Use the extended Biot’s theory with finite element procedure to predict the nonlinear coupled behavior of soil skeleton and porewater pressure.

8. Utilize and extend the method of determining liquefaction by using critical disturbance ($D_c$) under cyclic loading for different steel roughnesses.
9. Investigate, by using finite element code (DSC-DYN2D), the effects of soil nonlinearity, soil-structure interaction, developing of pore water pressure, and liquefaction for selected applications on offshore piles.

10. Identify the effect of no interface (bond) and with interface (relative motion) conditions on the behavior of pile-soil systems.

11. Provide critical comments and suggestions for analysis and design of offshore piles using the proposed model.

1.3. Organization of the Dissertation

The dissertation contains eight chapters and two appendices. A summary of the content of each chapter is presented in this section. Chapter 1 in an introduction describing the significance of the research in a brief background and also describes the scope of the research. Chapter 2 reviews some literature relevant to the subject which includes: analysis of offshore piles, interface testing devices, behavior of soils and interfaces, interface and surface roughness, constitutive relations for soils and interfaces, and the porous media theory.

Chapter 3 describes in detail the experimental program conducted in this study. The results of Ottawa sand-steel interface tests performed with the CYMDOF-P device are presented in Appendix (A) and discussed in this chapter. Profilometer test results of steel samples used in this study are summarized in this chapter while detailed principles, instruments, and specifications are given in Appendix (B).
Chapter 4 is devoted to the review of Biot’s original theory for porous media and its finite element formulation. The Disturbed State Concept model is presented in chapter 5. It includes details of the constitutive relations of the relative intact state, fully adjusted state and the average stress-strain relation for both soil and interface materials.

Chapter 6 presents the procedure for evaluating DSC model parameters and the parameters for saturated Ottawa sand-steel interface are found. This chapter also includes back predictions of laboratory observations and verification of the resulting model by predicting independent tests that are not used in parameter determination.

In chapter 7, the analysis of two independent pile tests are conducted using non-linear finite element approach with DSC constitutive model to assess the accuracy of finite element-DSC simulation.

Finally, in chapter 8, the summary of this work presents conclusions and recommendations.
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CHAPTER 2: LITERATURE REVIEW

2.1. Analysis of Offshore Piles

2.1.1. General

The response of pile-supported offshore structures is inherently a complex problem dominated by nonlinear behavior of soil-structure interaction and pore pressure generated during dynamic loading. The available models used for the analysis can be classified into two main approaches: empirical and theoretical. Empirical approaches rely on load-deflection relationships derived from field testing or physical laboratory experiments to characterize the pile-soil system. Winkler soil models and $p-y$ curves belong to that approach. Theoretical approaches characterize the pile-soil system using analytical or numerical methods and are based on derived solutions including finite element and boundary element methods. The following sections provide a brief review of the most used models in the analysis of offshore piles.

2.1.2. Empirical Models

Winkler (1867) introduced a theory of modeling the soil as a series of closely spaced discrete springs to simulate the soil resistance to pile loading. The theory ignores the shear transfer between adjacent soil layers. Furthermore, it does not take into account the non-linear behavior of soil, and so it is only useful for approximate or small displacement analysis. Matlock (1970) developed the Beam on Nonlinear Winkler Foundation model for laterally loaded piles in soft clay. The soil-pile interaction is approximated using parallel nonlinear soil-pile springs. The stiffness of these springs is
described by nonlinear load-deflection relations called the \( p-y \) curves. These curves were derived from static and cyclic loading of piles from experimental tests results on several piles installed in soft clay. This method is adopted in the API Recommended Practice (API 1993).

Reese and coworkers (Reese et al., 1974, Reese and Welch 1975, Reese et al., 1975) introduced a number of criteria for developing \( p-y \) curves for piles in sand and clay based on field pile tests. The proposed \( p-y \) curves for sand are adopted in the API Recommended Practice (API 1993). The \( p-y \) curves approach is further extended for the dynamic response analysis of pile foundations by adding a dashpot parallel to the nonlinear spring in order to account for the damping effect (Matlock et al., 1978).

For axially loaded piles, a similar approach called \( t-z \) curves was originally proposed and developed by Seed and Reese (1957), with further refinement by Coyle and Reese (1966). The method involves modeling the pile as a number of segments that are assumed to be connected by discrete springs which represent the resistance of the soil skin friction and end-bearing. A \( t-z \) nonlinear spring relates the load transfer between the soil and the pile shaft with the corresponding vertical displacements of the pile. The complete load-settlement curve is constructed by applying a small displacement at the pile, and by calculating the forces and displacements for each segment of the pile shaft. This process is repeated for a number of base displacements until a sufficient range of pile loads and displacements are obtained.

Chin and Poulos (1991) developed the \( t-z \) curves for the cyclic axial loading analysis of single pile embedded in a layered soil. They found that the pile-soil stiffness
ratio, soil stiffness ratio and pile length to diameter ratio have significant influence on the cyclic pile response. The comparison with the results of field cyclic pile load test showed that the general trends from the numerical and field measurements were generally consistent.

The $p$-$y$ and $t$-$z$ approaches are the popular methods for pile design; however, these models have some limitations. One on the major limitations is that they assume no significant interaction between successive pile segments and the soil surrounding these segments and hence, these models could not reflect the interface action of the pile-soil system. In real situations, soil exhibits continuity, whereas soil resistance is modeled with discontinuous springs. Boundary effects can be considered only indirectly. The field tests didn’t necessarily excite the mechanisms involved in seismic loading such as load transfer from the soil profile or generation of pore pressure. Therefore, they are not necessarily applicable to other types of dynamic loadings such as seismic loading. Furthermore, it has been observed from centrifuge experiments that the inertial forces and soil resistance are not always in phase and hence the maximum stresses in piles are generally over predicted (Brandenberg et al., 2007).

2.1.3. Theoretical Methods

Theoretical methods of pile analysis can be classified into two main groups: elastic continuum methods and numerical methods. The elastic continuum methods are based on Mindlin’s (1936) closed form solution for the application of point loads to a semi-infinite mass. The elastic continuum models are convenient for developing explicit solutions, which can accommodate the dynamic effects such as inertial forces and
damping. The solution of these models is conducted for constant, linear, and parabolic soil modulus. These models are more appropriate for small strain, steady state vibration problems. However, the inclusion of nonlinear behavior in these models is very difficult. Therefore, they are not suitable for the nonlinear dynamic analysis.

The major numerical method used in the pile-soil analysis is the finite element method. It has the advantage of using nonlinear soil models and allowing for permanent deformations. Finite element studies on pile behavior have been conducted by many investigators. Desai and Appel (1976) performed a three-dimensional finite element analysis with both linear and nonlinear soil behavior (hyperbolic stress-strain relationship). Pile-soil interface elements were considered in the finite element study. The results showed that the relative movement at the pile-soil interface can have significant influence on pile behavior. Simple plasticity models, such as Mohr-Coulomb, von Mises and Drucker-Prager models have been often used for soils due to their relative simplicity. Brown and Shie (1990) performed a three-dimensional finite element analysis on laterally loaded piles using von Mises and extended Drucker-Prager constitutive laws. The $p-y$ curves were derived from the finite element analysis. A comprehensive nonlinear finite element analysis of a vertically loaded pile was carried out using ABAQUS (Trochanis et al., 1991). In this study, the soil was modeled using extended Drucker-Prager plasticity while piles were modeled as linearly elastic material. Non-associative Drucker-Prager and von Mises models were also used by Yang and Jeremic (2002 and 2005) to study interaction between distinct soil layers for a laterally loaded pile in multi-layered soil profiles. Non-associative Mohr-Coulomb model for the soil was used in finite element by
Lee et al., (2002) to compute the distribution of dragloads due to negative skin friction in pile groups. They concluded that the dragload is normally overestimated from empirical methods and elastic analyses.

The advantage of the finite element method over other methods on the analysis of piles can be summarized, as reported by Fan and Long (2005) as: (1) it can take into account various boundary conditions and pile geometry, (2) various types of material constitutive models can be included in the system, (3) the continuity of soil mass and pile-soil interface behavior can be taken into account, (4) effects of various pile or soil properties on the pile responses can be studied systematically. In addition, coupled behavior of porous media and development of excess pore pressure can be accounted for.

Although nonlinear discrete systems have been extensively used due to their simplicity, severe limitations remain in the proper assessment of the empirical model parameters. With the advancement of computing technology, the finite element method is increasingly attractive as a versatile and powerful tool for realistic soil-structure interaction analysis.

2.2. Interface Testing Devices

2.2.1. General

Most early work on the experimental investigation of interfaces and joints concentrated mainly on the Mohr – Coulomb failure criteria in which strength parameters of the interface were sought, namely the angle of friction $\phi$ and the adhesion $c_a$. As the aim was to predict when slip or failure would occur, little attention was paid to deformation at the interface. Later, experimenters have studied the development of
relative motions of the interface, the plastic, or irreversible response, the nature of shear resistance both before and after the initiation of failure, and the effects of fluids and other environmental aspects. This section presents a selective review of test devices used for testing interfaces and joints.

2.2.2. Direct Shear Device

The direct shear device was first developed to measure the shear strength of soils; subsequently, the device was modified to allow for rock joint and interface testing. In this test, two specimens, usually having different properties, are placed in contact in the shear box as shown in Fig.(2.1(a)). A normal load is applied by a loading platen to the top of the soil specimen, and then a horizontal load, is applied to shear the interface between the two materials.

Potyondy (1961) used direct shear boxes to determine the friction between different types of soils and construction materials, both by strain controlled and stress controlled methods. The specimens of construction materials were placed in the lower portion of the box, and the soil was placed in the upper half. Coyle and Suleiman (1967), Desai (1974, 1976), Kulhawy and Peterson (1979), Acar et al., (1982), Uesugi (1987), and others have reported direct shear test data to evaluate interface strength parameters.

The direct shear device is a commonly available and the simplest system in sample preparation and test procedure. The stress concentration at the ends, the reduction of interface area during shear, and also the impossibility of separating the sliding displacement and displacement due to soil deformation are the major disadvantages of this type of device.
2.2.3. Annular Shear Device

To eliminate (or reduce) some of the discrepancies of a direct shear test, Brummund and Leonard (1973) proposed a modified test device for determining interface or joint parameters under static and dynamic loading conditions. The test configuration is shown schematically in Fig. (2.1(b)). The interface is introduced as the circumference of a circular rod which is inserted coaxially into a cylinder of soil. The soil sample is encased in a rubber membrane. By evacuating air from the membrane, a normal stress is applied to the sand-rod interface and then the rod is caused to slip relative to the sand by gradually applying static forces to the rod in the axial direction. This device is also used by Miyamoto et al., (1975).

This device maintains a constant interface area and is geometrically identical to shaft resistance of piles. The disadvantages with this system are the unknown normal stress on the interface, stress concentration occurs at the ends, sample preparation is difficult and has a direct effect on the test results, and finally the displacement components cannot be identified separately.

2.2.4. Torsional Ring Shear Device

Yoshimi and Kishida (1981a and 1981b) used a ring torsion apparatus shown in Fig. (2.1(c)). Dry sand was rained into an annular container lined with a rubber membrane. A ring-shaped metal specimen was placed on the sand as the construction material. A static torque was applied to shear the interface under a constant normal load. In addition to measurements of circumferential and vertical displacements of the metal ring, the deformation of the sand and slippage at the soil-metal contact were also measured.
This device has a constant interface area, the stresses and strains within the specimen are nearly uniform and because the specimen is endless, it is free from progressive failure that would initiate at the ends of a direct shear device. However, this device is a complicated system in terms of sample preparation and measurement of shear displacement. In addition, it is not possible to separate the displacement due to soil deformation from the tangential displacement.

2.2.5. Simple Shear Device

Uesugi and Kishida (1986a and 1986b) developed a simple shear type device which was capable of measuring both sliding displacement between steel and soil as well as shear deformation of soil mass. Fig.(2.1(d)) shows the simple shear apparatus. The contact area between the soil and the structural material remained constant during a test even if sliding occurred, since the steel plate was longer than the friction surface. Normal and tangential loads were applied by the vertical and horizontal hydraulic actuators. The container of sand specimen was a stack of rectangular 2 mm thick aluminum plates. The surface of each plate was lubricated to allow the container to follow the shearing deformation of sand with minimum friction resistance. This apparatus was also used for one-way and two-way repeated loading of interfaces between sand-steel and sand-concrete (Uesugi et al., 1989 and 1990).

In addition to the above advantages of this device, the sample preparation and test procedures are easy. It can suffer, however, from stress concentrations at the ends.
2.2.6. *The Cyclic-Multi-Degree-of-Freedom Shear Device (CYMDOF)*

Desai and coworkers [Desai (1980 and 1981), Zaman et al., (1984), Desai et al., (1985), Drumm and Desai (1986), Desai and Nagaraj (1988), Desai and Fishman (1991)] have developed and used the Cyclic-Multi-Degree-of-Freedom Shear device (CYMDOF) for static and cyclic testing of dry interfaces and joints. Tests were conducted using a variety of interfaces: concrete joints, sand-concrete and wood-rock ballast. Desai and Rigby (1997) built a new device called CYMDOF-P device which is similar to the old device mentioned above. Most of the interface devices do not operate under undrained conditions nor do they allow for cyclic loading under simple shear conditions. The new device is capable of performing direct and simple shear tests, drained and undrained interface testing, and static or cyclic loading under load control or displacement control procedures.

The CYMDOF-P shear device is intended for the general testing of interfaces formed between materials such as concrete, steel, rock, sand, clay, geosynthetics and similar materials. Details of the device and method of operation is discussed in chapter 3.

2.3. *Behavior of Soils and Interfaces*

2.3.1. *General*

The triaxial shear test seems to be the most common soil test used so far to evaluate the shear strength of soils. However, shear tests such as torsional shear or simple shear use stress paths that are closer to real field environment during loading. Moreover, these devices are capable of testing similar materials such as soils and dissimilar
materials such as joints and interfaces. Therefore, those testing methods have gained considerable popularity for strength analysis during the past 30 years.

There has been a large volume of material published on the behavior of soils and interfaces under static and cyclic loading conditions. Results of some of these works which are related to the topic of this research are discussed here.

2.3.2. Behavior of Saturated Sand

Undrained conditions occur in practical situations whenever external loads change at a rate much faster than the rate at which the induced pore water pressure can dissipate. For sands, undrained strength is relatively less important for static loading, but may be very important for problems involving dynamic loading.

A quantitative approach to the prediction of liquefaction was presented by Seed and Lee (1966) and was further developed by Lee and Seed (1967a and 1967b). In this approach, saturated sand samples are consolidated in the triaxial apparatus under appropriate stress conditions and alternating shear loads of controlled magnitudes and frequencies are applied with no drainage allowed. Measurements of the pore water pressure changes and the deformation behavior of the sample are made, so that the onset of liquefaction can be detected. The main conclusions derived from these triaxial studies is that the larger the cyclic stresses, the looser the samples, and the lower the confining pressure, a fewer number of cycles of a given stress are required to cause liquefaction.

Finn et al. (1970) presented two cyclic simple shear tests (tests Nos. 34 and 46) and one cyclic triaxial test (test No. 40) to demonstrate the behavior of saturated sand during liquefaction. The simple shear tests were run with an initial normal stress of 2
kg/cm². The sample in test No. 34 was vibrated to a medium dense state (relative density = 63%) but the sample in test No. 46 was loose (relative density = 42%). They found that the cyclic loads cause the pore water pressure to increase steadily and after a number of cycles liquefaction occurs and the shear strains rapidly increased. The denser sample showed greater resistance to liquefaction than the looser sample and the shear strains developed more slowly after initial liquefaction.

The triaxial test was run for medium density sand (relative density = 50%). The sand was first isotropically consolidated to 2 kg/cm² followed by application of cyclic deviator stress with amplitude of 0.58 kg/cm². Liquefaction occurred after 26 cycles and the triaxial strain suddenly increased. The porewater pressure increased gradually and showed fluctuations after liquefaction took place. The relevant data for simple shear tests and triaxial test are shown in Figs.(2.2 , 2.3 and 2.4), respectively.

Undrained cyclic triaxial shear test and simple shear tests on Ottawa sand were reported by Finn et al., (1971). The mean diameter of sand particles was D_{50} = 0.4 mm. All tests were run at a frequency of 2 Hz with a square waveform for the deviator stress. All sand samples were prepared to a medium dense state before testing. Typical results are presented in Fig.(2.5) for a triaxial test and Fig.(2.6) for a simple shear test. The axial strain in triaxial test and the shear strain in simple shear test remain small for the major part of all tests and then suddenly increase as liquefaction occurs. The pore pressure continues to rise during the test. The rate of development of pore pressure increases in the later stages of the test and the final significant increase to the values associated with liquefaction takes place over only a few cycles.
Ishihara and Yamazaki (1980) studied the liquefaction of Toyoura sand by using a multi-directional simple shear apparatus. They showed that under undrained cyclic rotational shear loading, the number of cycles required to cause liquefaction of sand was up to 35% less than that under unidirectional loading conditions. Typical results of their works are shown in Figs.(2.7 and 2.8).

Studies on the behavior of sands in both drained and undrained cyclic simple shear tests were conducted at the University of British Columbia and reported by Finn et al., (1982). Typical test results on Ottawa sand are shown in Figs.(2.9 and 2.10) for an undrained test and Fig.(2.11) for a drained test. In the undrained test, Fig.(2.9), the softening shows up in the decreasing shear stresses required to generate the required strain. The energy loss due to friction is represented by the almost rectangular hysteresis loop. As the number of load cycles increases the development of porewater pressure becomes limited to the unloading part of the strain cycle. The decrease in lateral effective stress accompanying the increase in porewater pressure is shown in Fig.(2.10).

Stress-strain behavior during a drained test is illustrated in Fig.(2.11), for a constant cyclic strain test. The sand strengthens during cyclic loading and each successive cycle requires a greater shear stress to achieve the prescribed strain level. This strain-hardening is accompanied by volumetric strain $\varepsilon_{vd}$. The development of volumetric strain with loading cycles is shown in Fig.(2.11-b).

A cooperative test program of cyclic undrained triaxial tests was performed on Toyoura sand by Toki et al., (1986). The grain size distribution and index properties are shown in Fig.(2.12). Fig.(2.13) shows typical results of the test. They concluded that for
the test procedures followed in their paper, the scattering of the data among five different laboratories is small whereas the effect of the specimen diameter on the results is appreciable.

Gyi (1996) studied the static and dynamic behavior of dry and saturated Ottawa sand by using a cubical multiaxial device. This device is capable of applying the general three-dimensional state of stress. Fig.(2.14) presents the results of a consolidated undrained cyclic shear test subjected to normal stress of 138 kPa, and Fig.(2.15) shows typical stress-strain behavior of the same test.

Mao and Fahey (2003) reported the behavior of three sediments (muddy silt, silt and sand) from offshore areas on the North-West Shelf of Australia. The behavior was studied through undrained simple shear tests. The cyclic phase transformation (PT) state, which separates undrained cyclic stress paths into contractive and dilative paths, was identified. Fig.(2.16) presents the determination of a cyclic phase transformation $PT_{cyc}$ state for Gorgon muddy silt from the stress path of undrained cyclic shear tests. The $PT_{cyc}$ state line was determined by linking elbows of the stress path loops. They concluded that $PT_{cyc}$ is unique for a particular soil.

2.3.3. Behavior of Dry Interfaces

A ring torsion apparatus was used by Yoshimi and Kishida (1981 b) to evaluate friction between dry sand and a steel surface over wide ranges of surface roughness and sand density. Fig.(2.17) shows typical results on friction between steel and medium dense Tonegawa sand. During the initial stage of the test, both the shear stress ratios and the volumetric strains are unaffected by the roughness of the steel surface. After passing the
initial stage, however, the curve begins to diverge, with rougher surfaces reaching higher stress ratios. They concluded that the friction resistance is primarily governed by the roughness of the steel surface, irrespective of the density of the sand.

Desai and his coworkers have been one of the active research groups studying interfaces between dissimilar materials. Using CYMDOF shear device, Desai et al., (1985) reported comprehensive test results on dry sand-concrete interfaces. The results are used to express shear stress as function of normal stress, relative displacement, number of loading cycles and initial density of sand.

Fig.(2.18) shows typical shear stress-relative displacement curves at cycles; N = 1, 10 and 100 for relative densities $D_r = 15\%$ and 30\%. The peak or mobilized shear stress increases with N for both densities. However, the increase for the higher density is not as rapid as that for the lower density. They concluded that, for cohesionless soils, the interface response stiffens or hardens with the number of cycles and the rate of such hardening decreases with increasing N. The influence of normal stress is indicated in Fig. (2.19). They found that the interface exhibits increased stiffness as the normal stress is increased.

Uesugi et al., (1989) and Uesugi et al., (1990) presented some very interesting results by application of a simple shear apparatus. Fig.(2.20) shows the relationship between shear stress ratio and the slip at the interface during cyclic test. The amplitude of the total displacement $\delta$ was 1 mm and the frequency was 1/100 Hz. In the early numbers of loading cycles, the interface sliding $\delta_i$ was much smaller than the total displacement $\delta$. 
After several cycles, the amplitude of sliding $\delta_1$ becomes close to the total displacement $\delta$. The maximum shear stress ratio becomes constant after several cycles of loading.

Fig. (2.21) shows an example of the shear stress ratio $\tau/\sigma$ and volumetric strain $\varepsilon_x$ in relation to the (a) total tangential displacement $\delta$, (b) sliding displacement $\delta_1$ and (c) shear strain of sand $\delta_2$. The difference between $\delta$ and $\delta_1$ is the displacement due to the shear deformation of sand mass $\delta_2$. They found that the maximum coefficient of friction $\mu_y$ is the peak shear stress ratio in the first cycle of loading. The residual coefficient of friction $\mu_r$ is the shear stress ratio when the total displacement is $\delta = 4$ mm in the 15th cycle of loading.

Fakhrarian and Evgin (1997) presented the results on the cyclic behavior of sand-steel interface under constant normal stiffness conditions. The experiments were performed by using a simple shear testing apparatus called C3DSSI. They aimed to study the reduction in the maximum shear stress with cycles. Fig. (2.22) shows typical behavior of sand-steel interface under a constant normal stiffness ($K=400$ kPa/mm). The specimen was subjected to 50 cycles of displacement at a frequency of 1/200 Hz with total tangential displacement amplitude of 0.75 mm. They found that the maximum shear stress was reduced with an increase in the number of cycles in such a way that the rate of reduction was higher within the first few cycles.

2.3.4. Behavior of Saturated Interfaces

The behavior of interface between Ottawa sand-steel were studied by Alanazy and Desai (1996) using CYMDOF-P device. A series of static and cyclic displacement-controlled interface tests were performed considering type of loading, normal stress,
drainage conditions, and rate of loading as variables. Figs. (2.23 and 2.24) present some results from undrained two-way cyclic shear test. In their tests, the soil sample didn’t reach the liquefaction state because of the leakage of a membrane system during the tests. The porewater pressure stabilized or approached a constant value lower than the applied normal stress.

Desai and Rigby (1997) performed a number of tests on clay-steel interfaces using the CYMDOF-P shear testing device. They used remolded clay and smooth steel surfaces, and tested the interface under two different displacement amplitudes (0.5 mm and 1.5 mm). For each displacement amplitude, three different normal stresses were applied. Typical results are shown in Figs. (2.25 and 2.26). They concluded that most of the shear stress degradation and porewater pressure build up occurred at the beginning of the test.

Desai et al., (2005) reported the results of a series of interface tests conducted on Ottawa sand-concrete interface. Typical results of undrained two-way cyclic loading are presented in Fig. (2.27). The cyclic response shows degradation of shear stress with number of cycles, in which the shear stress approaches the asymptotic (residual) stress after about four cycles. The excess porewater pressure during the cyclic test shows a gradual increase and then approaches the initial normal stress indicating liquefaction after about four cycles. The plot of degradation of the effective normal stress versus time shows a gradual decrease and liquefaction after about four cycles.

2.4. Interface and Surface Roughness

The behavior at the interface between soils and structural materials is the limiting factor in the performance of many systems including deep foundations, tunnels,
underground conduits and retaining structures. The ability to predict and control the
interface behavior can result in improved constructability and/or performance of such
structures. Therefore, the behavior of interfaces has received significant attention in the
literature over the past 40 years.

Yoshimi and Kishida (1981a) proposed the quantification of the role of	
continuum surface roughness on interface strength. Using a simple ring shear device, the
interface friction was shown to be primarily dependent on the continuum surface
roughness with the maximum interface friction angle always less than that of the test sand
and in particular the interface friction angle for smooth surfaces less than one-half the
internal friction angle of the sand. Uesugi and Kishida (1986a and 1986b), used a simple
shear test device to study the interface behavior between several kinds of soils and steel
plates of different roughnesses. Their comprehensive work concluded that surface
roughness had an important influence on the friction coefficient at yielding. Kishida and
Uesugi (1987), made a modification to the evaluation of steel surface roughness. The
gauge length \(L\) was modified to 50% diameter of sand i.e. \(R_{\text{max}} (L = D_{50})\). A normalized
roughness \(R_n\) was defined as \(R_n = R_{\text{max}} (L = D_{50})/ D_{50}\). A good correlation was obtained
between \(R_n\) and the coefficient of interface friction over a wide range of sand diameter.
Uesugi et al., (1988) tracked the particle behavior near the interface using close-up
photographs and observed the formation of a shear zone with a thickness of \(5D_{50}\) within
the sand mass along the rough interface. Frost et al., (1999) provided a further
development by quantifying the structural evaluation of sand particles adjacent to
geomembranes during shearing. For smooth geomembranes, the particles slide along the
interface with minor variations in density limited to a zone with two particle diameters ($2D_{50}$) or less adjacent to the geomembranes. As the surface roughness increased, the size of the affected zone increased up to a maximum of about six particle diameters ($6D_{50}$) from the interface.

Frost et al., (2002) performed a series of laboratory tests on a selection of sand-continuum material interfaces to study the effect of surface roughness and hardness on interface shear behavior and strength. They concluded a strong interaction between the effects of surface roughness and hardness and provided an understanding and framework from which continuum material surfaces can be optimized for performance in geotechnical applications.

Hu and Pu (2004) performed a series of direct shear tests with a charge-coupled-device camera used to observe the sand particle movements near the interface. They recognized two different failure modes during interface shearing; elastic perfectly-plastic failure mode occurs along the smooth interface, while strain localization occurs in a rough interface accompanied by strong strain-softening and bulk dilatancy.

2.5. Constitutive Relations for Soils and Interfaces

2.5.1. General

Different materials with the same geometrical characteristics may exhibit different responses for the same external excitations. This is due to the differences in the internal constitution of materials. Mathematical models that describe the behavior of materials are called constitutive laws or models. In this study, only the constitutive models that describe the mechanical stress-strain response of materials are discussed.
Under the application of external mechanical excitations the material undergoes a number of microstructural changes and transformations. These transformations are manifested by various effects such as elastic strains, plastic strains, creep, degradation or softening and fracture or failure. In this article various constitutive models are discussed that include one or more of these effects.

2.5.2. Elasticity Models

A material is ideally elastic if it recovers its original form completely upon removal of forces causing the deformation. The most general form of linear stress-strain relations for an elastic material can be represented by the generalized Hooke’s Law as:

$$\sigma_{ij} = C_{ijkl} \epsilon_{kl} \tag{2.1}$$

where $C_{ijkl}$ is a fourth-order symmetric tensor of elastic material constants, $\sigma_{ij}$ is stress tensor, and $\epsilon_{kl}$ is strain tensor. In the most general case, there are 81 different constants, but for a linear elastic isotropic material the number is reduced to two constants. For isotropic materials, $C_{ijkl}$ in Eq. (2.1) can be expressed as:

$$C_{ijkl} = \lambda \delta_{ij} \delta_{kl} + \mu (\delta_{ik} \delta_{jl} + \delta_{il} \delta_{jk}) \tag{2.2}$$

where $\lambda$ and $\mu$ are Lame’s constants and $\delta_{ij}$ is the Kronecker delta.

This type of linear model has a quite limited range and applicability to geological materials. To overcome such limitations, nonlinear elastic models have been developed and applied for a variety of problems under monotonic loading (Desai and Siriwardane, 1984). However, these models fail to identify plastic deformation when unloading occurs. Therefore, they may not be suitable for geological materials having the characteristic of plasticity under general loading conditions.
2.5.3. Perfect Plasticity Models

In these models, it is assumed that the material is elastic until the shear stress reaches a certain level, called the yield stress; thereafter, the material deforms plastically at constant stress (Fig. 2.28). In the first part, stress-strain relation is modeled as linear elastic. In the second part, the shear stress holds constant and plastic strain increases indefinitely. These models are called perfectly plastic models. Several of these models commonly used are Tresca, von Mises, Drucker-Prager and Mohr-Coulomb. The Tresca criterion assumes that the material yields when the maximum shear stress reaches a certain value. The von Mises criterion assumes that material yields when the second invariant of the deviatoric stress tensor \( J_2 \) reaches a certain value. Both criteria assume the yield stress is independent of the mean stress \( J_1 \), where \( J_1 \) is the first invariant of stress tensor \( \sigma_{ij} \). Moreover, material strength in tension and compression are assumed to be equal which is not true for many geologic materials. The yield surfaces for Tresca and von Mises are shown in Figs. (2.29 and 2.30).

For frictional materials such as soil, rock and concrete, yield strength increases with confining pressure. The Mohr-Coulomb model is the first attempt to capture this behavior. It assumes that the shear strength \( (\tau) \) increases with increasing normal stress \( (\sigma) \) on the failure plane (Desai and Siriwardane, 1984):

\[
\tau = c + \sigma \tan \phi
\]

(2.3)

where \( c \) is the cohesion and \( \phi \) is the angle of friction. Even though this criterion predicts different strength in extension and compression, it ignores the effect of intermediate principal stress. A generalization of Mohr-Coulomb criterion is proposed by Drucker and
Prager (1952) where all the principal stresses are considered. The shapes of the yield surfaces for the two criteria are same on the $J_1 - \sqrt{J_2D}$ plane and different on the octahedral plane, Figs.(2.31 and 2.32).

Although Drucker-Prager criterion includes the effect of mean pressure, it has severe limitations. For example, the model does not permit any plastic volume change under pure isotropic compression state of stress. Also, under a combined stress path, it always predicts plastic volumetric expansive strains if the normality rule is adopted. Consequently, this model may overpredict the observed dilatant behavior of some granular materials.

2.5.4. Hardening Plasticity Models

Experimental observations indicate an increase in yield strength with deformation for most materials. This is called the work or strain hardening. Such behavior may be attributed to the micro-structural rearrangements as well as locking of crystals during plastic deformations (Hill 1950). Hardening models are intended to define the process of strength gain in a material due to permanent straining in comparison to perfectly plastic response. Common types of hardening are isotropic hardening, kinematic hardening and anisotropic hardening.

Isotropic hardening assumes that with increasing plastic work or strains the yield surface expands without changing its shape, orientation and location in the stress space. Thus, as plastic deformation continues, the elastic limit increases and the yield surface expands isotropically without changing its origin. Kinematic hardening was introduced by Ishilinsky (1954) and Prager (1956). It is assumed that the yield surface does not
change its size but translates in the stress space as a rigid body during the inelastic straining process, whereas anisotropic hardening combines both isotropic and kinematic hardening. It is assumed that the yield surface expands as well as translates in the stress space. However, the shape of the yield surface remains the same. The three types of hardening models are shown in Figs.(2.33, 2.34 and 2.35), respectively.

2.5.5. Continuous Yielding Models

It has been observed that many geologic materials experience plastic deformations almost from the initiation of loading. In other words, if the material is unloaded from a given state during loading, it does not return to its original configuration, and experiences inelastic or plastic deformations. Thus the material undergoes a process of continuous yielding until it finally reaches the conventional failure or ultimate state. Furthermore, even during hydrostatic loading, many materials experience plastic deformations (Desai and Siriwardane, 1984).

It is possible to define the above behavior by defining a series of yield surfaces prior to reaching the failure or the final yield surface. Since the behavior is plastic under hydrostatic loading, the successive yield surfaces should intersect the \( J_1 \) axis, Fig.(2.36). This idea was first proposed by Drucker et al., (1957) with respect to the behavior of a soil specimen under conventional triaxial test. The major two continuous yielding models are the Cam-Clay model and Cap model.

Roscoe and coworkers (1957) developed the concept of critical state based on the findings of Rendulic (1936) and Hvorslev (1936, 1938). The underlying foundation of this model is the concept of critical void ratio or critical density. When a soil sample is
subjected to shear stress, it goes through phases of progressive yielding. This process of yielding continues until the material reaches a critical void ratio, after which the void ratio remains constant and the sample experiences large shearing strain and finally collapses. The yield surface in the Cam-Clay model is shown in Fig. (2.37). It is observed that the linear failure surface and the circular shape of the yielding cap, proposed by Drucker et al., (1957), may not compare with the experimental data with reasonable accuracy. Subsequently, several extensions and modifications were proposed. Dimaggio and Sandler (1971) found that the linear failure surface in the plane over predicts the dilatancy for granular materials. An exponential decay failure surface is assumed rather than linear. To represent the plastic volumetric changes under hydraulic compressions, they proposed an elliptical yielding cap as shown in Fig. (2.38).

One of the major drawbacks of the continuous yielding models is that the yielding is controlled by two separate yield functions which intersect each other with a slope discontinuity, Fig. (2.36). This results in nonuniqueness of the normal at the point of intersection. In the associated theory of plasticity, the incremental plastic strain is assumed to be normal to the yield surface at the loading point. Thus, the direction of incremental plastic strain is not defined at the point of intersection of the two yield surfaces. This problem can be eliminated if a single yield surface is used instead.

2.5.6. HISS Models

Desai (1980) expressed the yield and failure surfaces with a single mathematical function in terms of stress invariants. Based on this idea, Desai and coworkers (Desai and Faruque 1983, Baker and Desai 1984, Desai et al., 1986, Desai et al., 1990, Desai et al.,
1991) have developed the hierarchical single surface (HISS) models. This allows for progressive development of models of higher grades corresponding to different levels of complexities such as elastic-perfectly plastic, hardening and continuous yielding. Details of various specializations of the HISS model are given in Desai (2001).

This approach has been successfully used to model solids, interfaces and joints (Navayogarajah et al., 1992, Wathugala and Desai 1993, Desai 1994, Huang et al., 2004, Zaman 2008). The basic form in the HISS family model, called the $\delta_0$ model, allows for the associative and isotropic hardening plasticity.

2.5.7. Disturbed State Concept (DSC)

Desai (1974) characterizes the softening response of an overconsolidated soil by expressing the observed response in terms of its response in a normally consolidated state as the reference state. This idea was later formalized as the Disturbed State Concept (DSC).

DSC model has been successfully used to characterize behavior of soil (Desai et al., 1988, Liu et al., 2003), cohesive soil (Wathugala and Desai 1993, Katti and Desai 1995, Xiao and Jian 2009) cohesionless soil (Armaleh and Desai 1994, Park and Desai 2000, Jiajum and Shuanghuang 2010), and saturated soil (Desai and Wang 2003, Kim et al., 2004). It is also used to simulate the behavior of interfaces (Desai and Nagaraj 1988, Desai and Ma 1992, El – Hoseiny and Desai 2005), steel-sand interface (Alanazy and Desai 1996), saturated clay-steel interface (Desai and Rigby 1997), sand geosynthetic interface (Pal and Wathugala 1999), and sand-concrete interface (Pradhan and Desai 2006). Details and derivation of the DSC model is given in chapter five.
2.6. The Porous Media Theory

Many practical engineering problems involve quasi-static or dynamic loading of a saturated porous medium. Due to the presence of a fluid in the pores of the skeleton, responses of such media to external excitation are time dependent. Researchers have been trying to formulate this complex phenomenon involving deformation of porous skeleton and motion of pore fluid.

Terzaghi’s one-dimensional consolidation theory was the first such attempt reported in the literature, in which he uncoupled the deformation process from the pressure in the pore fluid. This simple one-dimensional consolidation theory is still being used by practicing engineers. Biot (1941), extending the early work of Terzaghi, proposed a two-phase continuum formulation for consolidation behavior of saturated porous media based on the assumption that a porous solid is linear elastic and isotropic. Moreover, the fluid flow through the solid phase obeys Darcy’s law. He showed that Terzaghi’s one-dimensional consolidation theory is a special case of his general three-dimensional formulation. The three-dimensional theory was later extended to the general case of anisotropic solid skeleton (Biot 1955).

Biot (1956a, 1956b, 1956c, and 1956d) was the first to develop a dynamic theory of the propagation of stress waves in porous media containing a compressible viscous fluid. The latest literature concerning the dynamic equations published by Biot (1962 a) presented a complete summary of the dynamic formulation of porous media.

Numerical approaches for flow in porous media have been successfully applied to the case of steady as well as the non-steady Darcy flow through rigid non-homogeneous


Owing to the increasing interest in nonlinear applications, a generalized incremental form was derived by Zienkiewicz (1982), Zienkiewicz et al., (1977c and 1980a), Prevost (1980, 1981, and 1982), Prevost and Hughes (1980) in which both large strain and nonlinear material behavior were included.

Zienkiewicz et al., (1980) reexamined Biot’s theory of wave propagation in the saturated porous elastic medium and suggested a possible simplification by ignoring the relative acceleration of pore fluid with respect to the soil skeleton \(u\) and porewater pressure \(p\). Validity of this new formulation, known as the \(u–p\) formulation, was examined for a broad range of frequencies encountered in soil dynamics problems. It was shown that for the range of frequencies usually encountered in earthquake engineering, a simplified form is acceptable and practically equivalent to the complete form.

Zienkiewicz and Shiomi (1984) give a detailed discussion of various possible numerical solutions to the generalized Biot formulation. Shiomi solved the full governing
equation using the finite element program (DIANA) with the displacement of the solid and the velocity of the fluid as the primary unknowns.

Another generalized form for the nonlinear analysis of porous media was derived by Prevost (1980a, 1980b, 1981b, 1982, 1983a, 1983b). In Prevost’s approaches, the porous media consisted of an inelastic porous skeleton and a viscous fluid and provided for transient analysis of saturated porous media. He also showed that the formulation of mixture theory is equivalent to Biot’s equations for the linear cases.

Based on the numerical solutions of the generalized incremental forms, many computer codes resulted from these works such as DYNAFLOW (Prevost 1981), DIANA (following Zienkiewicz and Shiomi 1984) and DSC-DYN2D (Desai, 2000b).
Fig.(2.1): Interface Testing Devices (Kishida and Uesugi 1987)
Fig.(2.2): Liquefaction Simple Shear Test on Medium Dense Ottawa Sand

(Finn et al., 1970)

Fig.(2.3): Liquefaction Simple Shear Test on Loose Ottawa Sand (Finn et al., 1970)
Fig.(2.4): Cyclic Triaxial Test on Medium Dense Ottawa Sand (Finn et al., 1970)
Fig.(2.5): Cyclic Triaxial Test on Medium Dense Ottawa Sand (Finn et al., 1971)

Fig.(2.6): Cyclic Simple Shear Test on Medium Dense Ottawa Sand

(Finn et al., 1971)
Fig.(2.7): Cyclic Unidirectional Simple Shear Test on Toyoura Sand

(Ishihara and Yamazaki, 1980)

Fig.(2.8): Cyclic Rotational Simple Shear Test on Toyoura Sand

(Ishihara and Yamazaki, 1980)
Fig.(2.9): Stress-Strain Loops in Constant Strain Cyclic Simple Shear Test

(Finn et al., 1982)

Fig.(2.10): PWP and Effective Lateral Stresses During Cyclic Loading (Finn et al., 1982)
Fig.(2.11): Strain-Stress Loops and Volumetric Strain in Drained Cyclic Loading Test (Finn et al., 1982)
Fig.(2.12): Grain Size Distribution and Index Properties of Toyoura Sand

(Toke et al., 1986)

Fig.(2.13): Typical Recorded Time Histories of Axial Load, Axial Deformation, and Porewater Pressure (Toke et al., 1986)
Fig.(2.14): Results of Consolidated Undrained Cyclic Shear Test with Confining Pressure of 138 kPa (Gyi, 1996)
Fig.(2.15): Stress-Strain Plot of Cyclic Test with Confining Pressure of 138 kPa (Gyi, 1996)
Fig. (2.16): $PT_{\text{cyc}}$ State Lines for Two-Way Undrained Cyclic Simple Shear Tests on Gorgon Muddy Silt (Mao and Fahey, 2003)

Fig. (2.17): Typical Test Results of Steel-Tonegawa Sand Interface

(Yoshimi and Kishida, 1981b)
Fig.(2.18): Shear Stress versus Relative Displacement for Different $N$.

(a) $D_r = 15\%$  (b) $D_r = 80\%$ (Desai et al., 1985)

Fig.(2.19): Shear Stress versus Relative Displacement for Different $\sigma_n$.

(a) $D_r = 15\%$  (b) $D_r = 80\%$ (Desai et al., 1985)
Fig.(2.20): Two-Way Cyclic Test of Steel-Toyoura Sand Interface (Uesugi et al., 1989)

Fig.(2.21): Typical Test Results Under Two-Way Repeated Loading (Uesugi et al., 1990)

(a) Total Displacement, (b) Interface Behavior, (c) Shear Deformation of Sand
Fig.(2.22): Two-Way Cyclic Shear Test under Constant Normal Stiffness Condition (Fakharian and Evgin, 1997)
Fig.(2.23): Shear Stress vs. Total Displacement for Two-Way Cyclic Loading (Alanazy and Desai, 1996)

Fig.(2.24): Effective Normal Stress and PWP vs. Number of Cycles for Two-Way Cyclic Loading (Alanazy and Desai, 1996)
Fig. (2.25-a): Shear Stress - Relative Shear Displacement for Sabine Clay-Steel Interface (Desai and Rigby, 1997)

Fig. (2.25-b): Peak Shear Stress with Time at Different Cycles (Desai and Rigby, 1997)
Fig.(2.26): Excess Pore Pressure and Effective Normal Stress with Time at Different Cycles (Desai and Rigby, 1997)
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(a) Yield Surfaces,   (b) Hardening Caps
Fig.(2.37): Cam-Clay Model in $J_1 - \sqrt{J_{2D}}$ Plane

Fig.(2.38): Schematic of the Cap Model (Dimaggio and Sandler 1971)
3.1. Introduction

In order to solve soil-structure interaction problems using numerical procedures, the behavior of interface between soil and structure must be represented by a suitable mathematical model. The parameters involved in this model should be determined from realistic interface behavior, evaluated from appropriate laboratory and/or field tests. In the past, a number of laboratory devices have been developed and techniques have been proposed for testing of interfaces and joints under various loading conditions.

In this work, simple shear tests and normal compression tests on Ottawa sand-steel interface were performed using Cyclic Multi Degree of Freedom device with porewater pressure measurements (CYMDOF-P). The test program also includes the measurements of surface roughness parameters for steel samples used in this study. In this chapter a brief review of testing devices will be presented.

3.2. Surface Roughness Parameter

Surface roughness is a quantitative measure of surface topography which describes the profile of the surface. It is an important factor influencing interface shear behavior in geologic materials. A review of previous research indicates that surface roughness parameters have generally been developed for interfaces for geotechnical applications.

Ward (1982) summarized 23 different international standard measurements of roughness. However, each of these parameters was developed for a particular application.
Only some of these parameters are considered appropriate for the study of interface behavior with geological materials. Table (3.1) summarizes some of these parameters which are reviewed below within the context of interface behavior.

**Average roughness ($R_a$):** Average roughness represents an average of the asperity heights (absolute value) of the roughness profile ordinates.

**Root mean square roughness ($R_q$):** It is essentially the standard deviation of the asperity height above and below the datum.

**Surface roughness ($R_s$):** It is the ratio of actual surface area $A_s$ to the projected area $A_o$.

Based on measurements made using optical profile microscopy (OPM) method, Dove and Frost (1996) proposed a geomembrane roughness classification scheme based on ranges of $R_s$ values as shown in Table (3.2).

**Maximum peak to valley roughness ($R_{max}$):** Yoshimi and Kishida (1981) used $R_{max}$ as a roughness parameter to quantify the roughness of machined steel surfaces. It was defined as the relative vertical distance between the highest peak and the lowest valley along a surface profile over a gauge length of 2.5mm. The main advantage of using $R_{max}$ is that it is easy to measure. However, the use of $R_{max}$ may not provide a true representation of the surface profile. This is due to the fact that, within a gauge length, a significant amount of small amplitude roughness would be ignored which affects the interface shear strength when shearing occurs adjacent to a fine-grained soil. This means that $R_{max}$ is an absolute roughness parameter rather than a relative one.

To overcome these problems, Uesugi and Kishida (1986a) developed a modified $R_{max}$ by changing the gauge length to equal the $D_{50}$ of the soil at the interface. Modified
$R_{\text{max}}$, however, has the same problems as $R_{\text{max}}$. The intermediate amplitude surface roughnesses within the gauge length $D_{50}$ can be ignored and this can be an important characteristic when the soil being studied has a well graded particle size distribution. The soil particles with diameter less than $D_{50}$ can interact with the intermediate amplitude surface roughness features.

**Normalized Roughness ($R_n$):** Uesugi and Kishida (1986b) found that the interface shear strength between soils and machined steel surfaces was influenced by the surface roughness of the material, the $D_{50}$ of the sand, and the interaction between these factors. Based on this, they concluded that the surface roughness could be better correlated with interface shear strength when normalized by the sand particle size. They suggested that the normalized roughness parameter $R_n$ be defined as follows:

$$R_n = \frac{R_{\text{max}}(L=D_{50})}{D_{50}}$$  \hspace{1cm} (3.1)

where:

$R_{\text{max}} (L = D_{50}) = R_{\text{max}}$ when $L = D_{50}$,

$D_{50} =$ mean grain size, and,

$L =$ gauge length of $R_{\text{max}}$.

In their study, $R_n$ ranged from 0.001 to 0.1 for machined steel surfaces.

The main advantage of using $R_n$ over $R_{\text{max}}$ is that $R_n$ accounts for the size of soil and produces a relative roughness parameter. In other words, $R_n$ can be correlated with the coefficient of interface friction over a wide range of particle sizes.
The surface roughness parameters for the steel samples used in this study have been measured by NANOVEA Inc. (NANOVEA, 2011) using a ST 400 3D optical profilometer. Details of test principles, instruments, and specifications are shown in Appendix B. Table (3.3) summarizes different roughness parameters for the three steel samples.

Based on the review of various surface roughness parameters, it is clear that the average roughness $R_a$ and the root mean roughness $R_q$ are absolute roughness parameters and cannot give enough information to explain the nature of surface characteristics. The maximum peak to valley roughness $R_{\text{max}}$ and the normalized roughness $R_n$ depend on the profile location and orientation. Moreover, the roughness contributed by small amplitude profile can be ignored. Conversely, the surface roughness $R_s$ is a three-dimensional surface roughness parameter and does not depend on the profile location and orientation. Also, the roughnesses contributed by small amplitude profile are considered. Therefore, the surface roughness parameter $R_s$ will be used in this study to define the surface roughness characteristics of the steel samples.

3.3. Interface Thickness

In the finite element analysis, the interface is treated as a solid element with a small finite thickness (Desai et al. 1984). An important question arises as to what value of thickness is most appropriate.

From a detailed parametric study and experimental verification on shear box test, Desai et al. (1984) found that the distribution of stress is not affected significantly by the
thickness of the element, while a choice of thickness \( t \) in the range of \( 0.01 \leq t/B \leq 0.1 \) can give satisfactory computations, where \( B \) is the width of the sample. Yoshimi and Kishida (1981a and 1981b) and Uesugi et al. (1988) estimate the shear zone thickness by studying the particle displacement during shear test. They used x-ray photography to measure the deformation of the sand-steel interface. The tangential displacement at the interface was found to consist mostly of slip for smooth surfaces. For rough surfaces, the displacement consisted mostly of shear zone distortion along the interface. Yoshimi and Kishida (1981a and 1981b) used sand material with average diameter \( D_{50} = 0.3 \) mm and steel roughness \( R_{\text{max}} = 510 \) μm and they found that the shear zone thickness is about 5 - 8 times the mean diameter of sand particles. Uesugi et al. (1988) used sand material with average diameter ranging between 0.5 - 0.6 mm and steel roughness \( R_n = 0.068 \). They found that the shear zone thickness ranged between 3 – 4 times the average diameters of sand particles.

In this study, the interface thickness for the three roughnesses used are assumed as 5, 7, and 8 times the average diameter of Ottawa sand for smooth, slightly textured, and moderately textured roughnesses, respectively. Knowing that \( D_{50} = 0.37 \) mm for Ottawa sand, the interface thickness becomes 1.85 mm, 2.6 mm and 3.0 mm for the three roughnesses, respectively.

**3.4. Description of Cyclic Shear Apparatus (CYMDOF-P)**

**3.4.1. General**

The Cyclic Multi Degree of Freedom (CYMDOF) shear device was originally designed by Desai (Desai 1980, Desai 1981) and then developed by Desai and Rigby
(1997). The modified shear device, CYMDOF-P (P is for pore pressure) allows static and
cyclic testing under direct and simple shear deformation, drained and undrained
conditions, and includes measurements of the interface fluid pressure. The details of the
device are shown in Fig.(3.1).

In this study, simple shear tests and normal compression tests were performed
under drained and undrained conditions. In the simple shear tests, the two materials are
placed one above the other in the two halves of the box with a provision for allowing
simple shear deformation by confining the geologic specimen by annular smooth
confining rings, which are coated with Teflon as shown in Fig.(3.2). The rings are
16.5cm diameter and 2.03cm height. The displacement due to the soil deformation is
measured by attaching a pair of small submersible LVDT’s to the top thin aluminum ring
as shown in Fig.(3.3). The benefit of using the simple shear rings is illustrated in Fig.(3.4).
The rings are essential in defining the relative displacement of the interface. This
advantage is not available in the direct shear device where the relative displacement
cannot be defined accurately.

The circular test box, which includes the interface system, is illustrated in
Figs.(3.5). The upper part is the structural material (steel in this study) and has a diameter
of 20.3 cm (8 in). The lower part which holds the soil specimen (Ottawa sand in this
study) has a diameter of 16.5cm (6.5 in). The lower sample is fixed and the upper sample
is allowed to move vertically and horizontally in the direction of shear.

In the case of pore water pressure measurements, a rubber membrane is installed
on the outside of both upper and lower box and is confined by confining rings as shown
in Figs.(3.6 and 3.7). Porewater pressure in interface is measured through a 2.54 cm (1 in.) porous stone mounted in the center of steel sample as shown in Fig.(3.8). A schematic representation of the device is show in Fig.(3.9).

The hydraulic control system is capable of applying one-way monotonic, one-way cyclic and two-way cyclic loads. Loads or displacements are applied by the vertical and horizontal actuators. The quantities that can be measured during a test include the normal stress, shear stress, normal displacement, shear displacement, relative displacement, and the porewater pressure.

The porewater pressure at the interface and the chamber pressure are measured by two 690 kPa (100 psi) Sensotec Pressure Transducers as illustrated in Figs.(3.10 and 3.11).

The fluid pressure saturation system is composed of the external water supply reservoir, the pressure saturation board, the air pressure system, the vacuum pump, the membrane confining system, and the device itself. This system is used for undrained interface testing. Fig.(3.12) shows a schematic drawing for the system. In this study, the membrane confining system has been developed to prevent leakage in the saturation system during the test. A new type of flexible annular membrane with high-strength steel confining rings is used, Fig.(3.6). The purpose of the membrane confining system is to create a water tight chamber out of the upper and lower boxes of the device.

3.4.2. Load Control and Data Acquisition

A single hydraulic pump supplies pressure to operate hydraulic actuators which generate the load or displacement needed for performing tests. Each actuator is connected
to a servovalve which regulates the oil flow that controls the actuator, the servo valve, in turn, is controlled by MTS-Test Star II control system. The control system consists of two parts: the digital controller and the workstation computer as shown in Fig.(3.11).

A software program TestWare-SX (MTS, 1999) is installed in the computer which helps in running the test procedures and acquiring the data during the test.

3.4.3. Preparation of Soil Sample

A known weight of sand (722 gm) is placed in the confining rings (volume = 435 cm³) to get a unit weight of 1.66 g/cm³ which corresponds to relative density $D_r = 60\%$. The soil is placed into three layers with each layer compacted by 25 blows of a (0.6 lb) rammer (Figs. 3.13 and 3.14). Compaction effort has been distributed uniformly on the sand specimen to assure uniform density and uniform distribution of void ratio. All specimens received the same specimen preparation to assure consistency.

After the sand specimen was prepared to the right density in the rings and the top surface was leveled, the testing device components were assembled and the steel specimen was placed at the top of the sand specimen.

3.4.4. Saturation Process

For undrained tests, it is necessary to bring the sand specimen to a fully saturated condition to fill all voids in the specimen with water. Saturation is usually accomplished by applying back pressure to the specimen after applying vacuum to the specimen and the saturation system and allowing deaired water to saturate the system while maintaining the vacuum. Back pressure causes an increase in pore water pressure which will increase the degree of saturation of the specimen by two ways. First, the increased pressure in the
pore fluid causes the air to compress and second, the compressed air is dissolved in the water.

Back pressure is applied to the specimen porewater simultaneously with an equal increase in the normal stress. This was done to avoid disturbing the sand specimen or changing the effective stress. Xia and Hu (1991) pointed out that the back pressure has a direct effect on the shear resistance of the sand if subjected to cyclic loading. For this reason, a low back pressure (5 psi) was used in all of the undrained tests, and it was applied for about one hour before starting the tests.

3.5. Properties of Interface Materials

3.5.1. Lower Sample – Ottawa Sand

The Ottawa sand used in this study is silica sand with uniform rounded or subrounded fine grains. The particle size distribution according to ASTM-D422 (2009) is given in Fig.(3.15). Some other properties of Ottawa sand are listed in Table (3.4) (Gyi, 1996). The initial density of the tested samples is 1.66 g/cm³ (103.6 lb/ft³) which corresponds to about 60% relative density.

3.5.2. Upper Sample – Steel

The top sample is a steel disc 20.3 cm (8.0 in) diameter and 7.76 cm (3.05 in) thickness fabricated from a hot-rolled structural carbon steel ASTM A36. Three samples of steel surfaces were investigated in this study: smooth (RI), slightly textured (RII) and moderately textured (RIII). Roughness parameters for the three steel samples used in this study are summarized in Table (3.3).
3.6. Test Program Description

In order to model the behavior of saturated Ottawa sand-steel interface, a series of drained and undrained shear and normal compression tests were performed with three different steel roughnesses. All tests were conducted using CYMDOF-P shear device. Ottawa sand of 60% relative density was used in the tests.

3.6.1. Normal Compression Test

Two groups of a total of six (6) drained normal compression tests were conducted to evaluate the normal stiffness behavior. In the first group, the sand samples were dry and in the second were saturated. The main variable in each group is the surface roughness of steel. All tests were run under load controlled mode and covered all ranges of normal stresses in simple shear tests (100 – 300 kPa). Table (3.5) summarizes all the normal tests.

Rigby and Desai (1995) indicated that vertical friction is negligible in the upper box under dry testing. However, with undrained testing, an o-ring is used around the upper sample which creates resistance that is significant enough to require a friction correction to the vertical load. The stick slip movement of the upper sample is observed which produces an average friction correction of 1000 N. This amount is added to the total applied normal load.

3.6.2. Simple Shear Test

A total of seventy two (72) simple shear tests were performed for sand-steel interface. They were divided into two groups: dry interfaces and saturated interfaces. Each group involves eleven (11) one-way monotonic shear tests, eleven (11) one-way
cyclic shear tests, and fourteen (14) two-way cyclic shear tests. The controlling variable of all tests was the shear displacement. The other parameters adopted as variables in the program are:

- Normal Stress.
- Surface roughness of steel
- Frequency of the cyclic loading
- Amplitude of the cyclic loading.

Two components of horizontal resistance are considered in the correction of shear strength of the interfaces. First, the constant friction component due to friction between the guide arms of the upper box and the bronze plates. The second component resulted from stretching of the rubber membrane during shear motion. Series of tests were performed on Teflon-Teflon interface to estimate these components which are subtracted from the total horizontal friction.

Tables (3.6 and 3.7) summarize dry and saturated tests, respectively. Test results are given in Appendix A.

3.7. Test Results

Results of the laboratory tests conducted by CYMDOF-P device on dry and saturated Ottawa sand-steel interfaces are presented in this section. This section is divided into two parts: normal compression test results and simple shear test results.

The average normal and shear stresses were obtained by dividing the relevant load by nominal area of the interface. The effective normal stress $\sigma_n'$ is determined by subtracting the porewater pressure from the total normal stress $\sigma_n$. The relative shear
displacement $u_r$, as defined before, is the difference between the total displacement $u_t$ and the displacement due to sand deformation $u_s$, Fig.(3.4). The relative displacement $u_r$ should be considered in interface modeling. The total displacement is imposed on the interface and applied by moving the top specimen (steel). When $u_t$ is applied, sand deformation is observed and $u_s$ is measured. Hence, monitoring the displacement due to sand deformation $u_s$ is essential to obtain the relative displacement $u_r$. The test result graphs for both normal compression tests and simple shear tests are given in Appendix A.

3.7.1. Normal Compression Test Results

This is a load controlled test in which the normal load is applied in a nature of one-way cyclic loading to enable the evaluation of normal stiffness from the unloading part of the resulting stress-displacement relation. The normal load is applied in increments with a rate of 25 N/sec and the corresponding displacement is recorded. The total normal stress applied (300 kPa) covered the range of normal stresses involved in this study.

Figures (A.1 - A.3) show the variation of normal stress with normal displacement for dry interfaces. The results for saturated interfaces are presented in Figs.(A.4 - A.6). In the two sets of graphs, the normal stiffness which is defined as the slope of the unloading-reloading part remains essentially constant as the number of cycles increases.

3.7.2. Simple Shear Test Results

In this test, the normal stress is load controlled and the shear stress is displacement controlled. As mentioned before, the simple shear tests are divided into two groups: dry interfaces and saturated interfaces. In each group, three different types of
loading were used. The test results are presented and discussed below in three separate categories based on the type of the applied load.

**3.7.2.1. One-Way Monotonic Shear Test Results**

The most basic interface behavior is the behavior under one-way monotonic loading. Under this type of loading, eleven (11) simple shear tests were performed on dry interfaces and a similar number on saturated interfaces. The main variables in this test are the normal stress and the surface roughness as shown in Tables (3.6 and 3.7). The normal stress is ranges between 50-300 kPa. Three different roughnesses were used: smooth (RI), slightly textured (RII), and moderately textured (RIII). The maximum total shear displacement is 6 mm with a rate of loading of 0.05 mm/sec.

Figures (A.7 - A.17) present the variation of shear stress with relative shear displacement resulting from one-way monotonic simple shear tests on dry interfaces. Similar relations for saturated interfaces are shown in Figs.(A.43 - A.53).

It is clear from the above relations that the maximum shear stress increases with the increase of the normal stress and occurs at a larger relative displacement. Moreover, the increasing of surface roughness for the upper steel sample also causes an increase in the maximum shear stress. All graphs show no stress softening after reaching the peak shear stress. The shear stress is gradually increased as the shear displacement increases until an approximately constant shear stress is attained. This behavior is characteristic of loose to medium dense sand. Graphs also indicate a small difference between the total relative shear displacement and the total applied shear displacement (6 mm). This indicated that sand deformation was small and formed only a small percentage of the total
displacement and the total displacement consisted mostly of interface sliding or relative displacement.

In saturated interfaces, an access porewater pressure is built up as shown in Figs. (A.54 – A.64). The access porewater pressure reduces the effective stress acting normally at the interface. When the effective normal stress decreases it causes a decrease in the shear strength of the interface. Such behavior is clearly recognized when the relations of shear stress versus relative shear displacement of saturated interfaces are compared with the corresponding relations from dry interfaces.

3.7.2.2. One-Way Cyclic Shear Test Results

This test was performed by loading the interface to a certain increment of the total displacement, unloading to zero shear stress, and then reloading to the next increment of total displacement until the final total displacement value is reached. Eleven (11) tests were performed following this type of loading for dry interfaces and similar number for saturated interfaces. The maximum displacement applied is 6 mm with a rate of loading equal to 0.05 mm/sec. The main variables used in this test were the normal stress and surface roughness. The normal stress ranged between 50-300 kPa and three surface roughnesses were used for the steel sample: smooth (RI), slightly textured (RII), and moderately textured (RIII). The summary of variables used in this test is found in Tables (3.6 and 3.7).

Figures (A.18 – A.28) present shear stress variation with relative shear displacement for dry one-way cyclic shear tests. Similar relations yield for saturated
samples are presented in Figs.(A.65 – A.75). These relations are used in estimating the shear stiffness for Ottawa sand-steel interfaces.

3.7.2.3. Two-Way Cyclic Shear Test Results

Two-way cyclic shear loading is presented in the field during earthquakes, under wind and/or wave loads. This type of loading is important because it involves loading in the reverse direction which may involve severe changes in the behavior of the soil-structure system compared to any other type of loading. Under this type of loading, fourteen (14) dry and fourteen (14) saturated Ottawa sand-steel interfaces were tested. The variables used in this test were: surface roughness, normal stress, and amplitude and frequency of the applied shear displacement. Three surface roughnesses were considered: smooth (RI), slightly textured (RII), and moderately textured (RIII). The shear stress in this test is displacement controlled via the control unit which receives a sinusoidal displacement function. The amplitude and frequency of the applied displacement were set constants for the tests with slightly and moderately surface roughnesses and variables for the tests with smooth surface roughness as shown in Tables (3.6 and 3.7). Two amplitude values (2.5 and 5.0 mm) were used for smooth interface tests and one amplitude value (5.0 mm) was used for the tests with other roughnesses. The frequency of applied displacement was (0.04, 0.1, 0.25, and 0.625 Hz) for smooth interface tests and one value (0.1 Hz) was used for other interfaces. Cyclic tests were performed with three different applied normal stresses: 50, 100, and 200 kPa for slightly and moderately textured surface roughnesses and four different applied normal stresses: 50, 100, 150, and 200 kPa for smooth surface roughness.
The results of this test for dry interfaces are presented in Figs.(A.29 – A.42) and for saturated interfaces they are presented in Figs.(A.76 – A.89). By comparing the relative shear displacement with the amplitude values of the applied displacements, it can be concluded that the total displacement consists mostly of the sliding displacement or the relative displacement. The amplitude of the controlled displacement was large enough to pass the peak shear stress in all tests. It is also clear that as the normal stress increases, the corresponding shear resistance also increases for a given amplitude of shear displacement.

The effect of surface roughness on shear behavior can also be noticed in the mentioned figures. As surface roughness increases, the peak shear stress increases for the same amplitude, frequency and applied normal stress.

The effect of displacement amplitude on the behavior of shear stress for smooth surface roughness can be observed by comparing Figs.(A.29 and A.30) for dry interfaces and Figs.(A.76 and A.77) for saturated interfaces. For dry tests, it is clear that the behavior of 2.5 mm amplitude has the same basic shape of the 5.0 mm amplitude. While for saturated interfaces, the degradation of shear stress is more severe in the 5.0 mm amplitude test compared to 2.5 mm amplitude test.

The degradation of shear stress for saturated interfaces is also affected by the surface roughness of steel sample. Figs.(A.81, A.77, and A.83) show the variation of shear stress with relative shear displacement for smooth interface with applied normal stress of 50, 100, and 200 kPa, respectively. The corresponding relations for slightly textured interfaces are presented in Figs.(A.84 – A.86) and for moderately textured ones.
are presented in Figs.(A.87 – A.89). It is found that the increasing of surface roughness of steel sample causes rapid decrease in the shear stress as the number of cycles increases. The degradation of peak shear stress with time are shown in Figs.(A.90 – A.103).

The magnitude of porewater pressure and the rate at which it developed has a direct effect on the shear stress behavior. Porewater pressure build up reduces the effective normal stress at the interface and as a result, the shear resistance reduces. Hence, the change in the shear stress behavior is affected by the magnitude and rate of porewater pressure build up. Figs.(A.104 – A.117) present the development of porewater pressure with time for saturated interface tests. The important aspect which should be noticed during the development of porewater pressure is the number of cycles or time at which the porewater pressure becomes equal to the applied normal stress; this condition is often called liquefaction.

Table (3.8) shows the time $t_{liq}$ and number of cycles $N_{liq}$ at liquefaction for the saturated two-way cyclic shear test. It is shown that $t_{liq}$ and $N_{liq}$ increase as the applied normal stress increases. More cycles are required to build up higher porewater pressure. The surface roughness as well as the amplitude of shear displacement has the same impact on the porewater pressure. As surface roughness or shear displacement amplitude increases, the number of cycles required to create liquefaction is reduced.

It is clear from Table (3.8) that the frequency of shear displacement has a direct effect on the development of porewater pressure and the rate of its development. As the frequency increases, the time and number of cycles at liquefaction decrease. Low frequencies permit the sand particles to seek the path of least resistance around each other
and results in a specimen with smaller voids and lower permeability. High frequencies decrease the effect of interlocking behavior of the sand particles since sufficient time is not allowed for the particles to move around each other in the most efficient path. It may be concluded that the higher frequencies, the greater the potential of failure and liquefaction.

It should be mentioned here that the device didn’t respond properly for the first 5-7 cycles in this type of loading. The applied shear displacement was found to be less than the target displacement (2.5 mm or 5.0 mm). The behavior within these cycles is excluded in calculations of parameters in Chapter 6.
Table (3.1): Definition of Roughness Parameters (After DeJong et al. 2002)

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Definition</th>
<th>Reference</th>
</tr>
</thead>
<tbody>
<tr>
<td>Average Roughness, $R_a$</td>
<td>$R_a = \frac{1}{A} \int_A</td>
<td>z(x,y)</td>
</tr>
<tr>
<td>Root Mean Square Roughness, $R_q$</td>
<td>$R_q = \left[ \frac{1}{A} \int_A z^2(x,y) dx dy \right]^{1/2}$</td>
<td>Stachowiak and Batchelor (1993)</td>
</tr>
<tr>
<td>Surface Roughness Parameter, $R_s$</td>
<td>$R_s = \frac{Actual\ surface\ area\ A_s}{projected\ area\ A}$</td>
<td>Underwood and Banerji (1987)</td>
</tr>
<tr>
<td>Maximum Peak to Valley Roughness, $R_{max}$</td>
<td>$R_{max} = \text{Height between the highest peak and deepest valley}$</td>
<td>Yoshimi and Kishida (1982)</td>
</tr>
<tr>
<td>Normalized roughness, $R_n$</td>
<td>$R_n = \frac{R_{max}(L=D_{50})}{D_{50}}$</td>
<td>Kishida and Uesugi (1986b)</td>
</tr>
</tbody>
</table>

Where:
- $z(x,y)$: profile height above datum
- $A$: Sample projected area
- $A_s$: Actual surface area
- $D_{50}$: mean grain size
- $L$: Gauge length of $R_{max}$.
Table (3.2): Textural Classification (After Dove and Frost, 1996)

<table>
<thead>
<tr>
<th>Textural Description</th>
<th>Rₜ Range</th>
</tr>
</thead>
<tbody>
<tr>
<td>Smooth</td>
<td>1.00 to 1.10</td>
</tr>
<tr>
<td>Slightly Textured</td>
<td>1.10 to 1.35</td>
</tr>
<tr>
<td>Moderately Textured</td>
<td>1.35 to 1.60</td>
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<tr>
<td>Heavily Textured</td>
<td>Greater than 1.60</td>
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Table (3.3): Surface Roughness Parameters for Steel Samples

<table>
<thead>
<tr>
<th>Parameter</th>
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<tbody>
<tr>
<td></td>
<td>RI</td>
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<tr>
<td>Rₐ (µm)</td>
<td>2.331</td>
</tr>
<tr>
<td>Rₖ (µm)</td>
<td>3.040</td>
</tr>
<tr>
<td>A (mm²)</td>
<td>25</td>
</tr>
<tr>
<td>Aₚ (mm²)</td>
<td>26.551</td>
</tr>
<tr>
<td>Rₛ</td>
<td>1.062</td>
</tr>
<tr>
<td>Rₘₚ (µm)</td>
<td>32.709</td>
</tr>
<tr>
<td>Rₙ</td>
<td>0.09</td>
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*L=D₅₀
Table (3.4): Index Properties of Ottawa Sand

<table>
<thead>
<tr>
<th>Property</th>
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<tbody>
<tr>
<td>Unfiled Soil Classification System</td>
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<tr>
<td>Specific Gravity</td>
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<tr>
<td>$D_{10}$, mm</td>
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</tr>
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<td>$D_{50}$, mm</td>
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<td>$D_{30}$, mm</td>
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</tr>
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<td>$C_c$</td>
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$C_c$: coefficient of curvature = $(D_{30})^2 / (D_{60})(D_{10})$

$C_u$: coefficient of uniformity = $D_{60}/D_{10}$

Table (3.5): Normal Compression Tests on Dry and Saturated Interfaces

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Fig.(3.1): Cyclic Multi Degree of Freedom Device

Fig.(3.2): Simple Shear Confining Rings
Fig.(3.3): Close-up View of Submersible LVDT

Fig.(3.4): Measurements of the Interface Displacement (Kishida and Uesugi, 1987)
Fig. (3.5): View of the Upper Part and the Upper Guide Box

Fig. (3.6): Membrane Confining System
Fig. (3.7): Membrane Confining Rings

Fig. (3.8): Upper Steel Sample with Porous Stone
Fig.(3.9): Schematic Drawing of CYMDOF-P Shear Device
Fig.(3.10): Sensotec Pressure Transducers

Fig.(3.11): LVDT and Sensotec Controllers
Fig.(3.12): Schematic Representation of the Saturation System.

(Desai and Rigby, 1997)
Fig.(3.13): Filling the Lower Part with Ottawa Sand

Fig.(3.14): Compaction of Ottawa Sand
Fig.(3.15): Grain Size Distribution of Ottawa Sand
CHAPTER 4: DYNAMICS OF POROUS MEDIA

4.1. Introduction

Porous media, such as soil and rock, are a multiphase system consisting of a skeleton formed by individual particles and pores in between them. When the pores are completely filled with water, such as a saturated soil, the system responds as a two-phase system. Under loading, the skeleton deforms partly due to deformation of individual particles and partly due to relative motions between particles. Water pressure (or pore pressure), on the other hand, changes and the fluid flows from high to low pressure regions, and excess pore pressure dissipates with time. For geotechnical engineering problems, the behavior of saturated porous media is usually modeled by considering the interaction of the two phases.

In order to understand the mechanics of deformation of solid-fluid system, it is necessary to consider the following:

i- Deformation of individual solid particles,

ii- Interparticle sliding and rearrangement of particles – deformation of skeleton,

iii- Deformation of fluid, and

iv- Ability of fluid to flow through pores of the skeleton.

The mathematical theory describing the time dependent dissipation of pore pressure through the flow of porewater and associated deformation of the soil skeleton is called consolidation theory. It was first developed in one – dimension by Terzaghi in 1923 by adopting the effective stress principle.
Biot (1941) established the equations governing the behavior of three-dimensional deformation of saturated porous media including the interaction of the solid and fluid media for quasi-static problems. Later, Biot (1956, 1962a, 1962b) extended these equations to dynamic problems. The porous media was treated as elastic in all the above analyses. Following this, nonlinear material behavior was taken into account by Zienkiewicz (1982) and Zienkiewicz and Shiomi (1984). A finite element program based on this theory was developed by Galogoda and Desai (1986) and was modified by Wathugala and Desai (1990) to apply models in the HiSS family. DSC model is implemented in the program by Shao and Desai (2000) which is used in this study.

4.2. Definition of Basic Variables

Fig. (4.1) shows a typical element of saturated soil which consists of two parts: solid skeleton and fluid in the pores. The amount of pores in a solid skeleton is defined by the term porosity which denotes the relative volume of the pores to total volume:

\[ \bar{n} = \frac{V_v}{V} = \frac{V_f}{V_f + V_s} \]  \hspace{1cm} (4.1)

where \( V_v \) is the volume of pores which is equivalent to the volume of fluid \( V_f \) and \( V_s \) is the volume of solids.

From the definition of density and Eq. (4.1), the density \( \rho \) of the solid-fluid mixture can be derived in terms of porosity \( \bar{n} \) and the densities of solid \( \rho_s \) and fluid \( \rho_f \) as:

\[ \rho = (1-\bar{n}) \rho_s + \bar{n} \rho_f \]  \hspace{1cm} (4.2)

The displacements in the two phases are shown schematically in Fig. (4.2). Since the fluid can move relative to the solid skeleton, the displacement of solid \( u_i \) and the
displacement of fluid \( v_i \) are independent. The volume of fluid \( Q_i \) moving in \( i \)-direction is given by:

\[
Q_i = A_i \bar{n} (v_i - u_i)
\]  

(4.3)

where \( A_i \) is the area of solid skeleton normal to the \( i^{th} \) direction.

Thus, the displacement of fluid relative to the solid skeleton, averaged over the face of the solid skeleton is given by:

\[
w_i = \frac{Q_i}{A} = \bar{n} (v_i - u_i)
\]  

(4.4)

This situation is depicted in Fig.(4.2b).

The change of fluid volume \( \zeta \) in a unit volume of the skeleton is given by:

\[
\zeta = w_{i,i}
\]  

(4.5)

where double \( i \) denote summation of \( i \) from 1 to 3 and the comma denotes differentiation with respect to a coordinate axis.

From classical continuum kinematics, the strain of the soil skeleton can be related to the displacement by:

\[
\varepsilon_{ij} = \frac{1}{2} (u_{i,j} + u_{j,i})
\]  

(4.6)

where \( \varepsilon_{ij} \) is the strain tensor.

### 4.3. Generalized Formulation for a Two-Phase Media

The set of equations governing the response of saturated porous media can be derived from three basic laws: constitutive law, law of conservation of momentum, and law of conservation of mass. These laws respectively yield: constitutive relation, equilibrium equations of both solid and liquid phases, and pore pressure relation.
Biot (1941, 1960) presented a general set of equations governing the behavior of a saturated linear elastic porous solid under dynamic conditions. Zienkiewicz et al. (1980) generalized these equations to non-linear material behavior by writing the constitutive relation in incremental form. This formulation is defined as \( u-w \) form since the primary variables are \( u \) and \( w \). The summary of \( u-w \) formulation is given below (Zienkiewicz et al. (1980)).

The total incremental stress tensor \( d\sigma_{ij} \) can be decomposed into two parts as:

\[
d\sigma_{ij} = d\sigma'_{ij} + dp \delta_{ij}
\]

where \( d\sigma'_{ij} \) is the incremental effective stress tensor and \( dp \) is the incremental pore pressure. Similarly, incremental strains \( d\epsilon_{ij} \) are also decomposed into two parts:

\[
d\epsilon_{ij} = (d\epsilon_{ij})_p + (d\epsilon_{ij})_{\sigma'}
\]

where \( (d\epsilon_{ij})_p \) are the strains caused by the deformation of solid grains due to pore pressure and \( (d\epsilon_{ij})_{\sigma'} \) are the strains caused by the deformation of solid grains and the skeleton due to the effective stress. Strains in the solid grains caused by pore pressure can be estimated by assuming elastic behavior of solid grains which gives:

\[
(d\epsilon_{ij})_p = \left(\frac{dp}{3K_s}\right) \delta_{ij}
\]

where \( K_s \) is the bulk modulus of the solid grains. On the other hand, deformation of solid skeleton caused by the effective stress \( d\sigma'_{ij} \) is related to \( d\sigma'_{ij} \) through the constitutive tensor \( C_{ijkl} \):

\[
d\sigma'_{ij} = C_{ijkl}^{ep} (d\epsilon_{kl})_{\sigma'}
\]

Combining Eqs.(4.7, 4.8, 4.9, and 4.10), the following relation is obtained:
The last two terms without \( dp \) of Eq.(4.11) can be decomposed into a diagonal tensor \( a\delta_{ij} \) and a deviatoric tensor \( b_{ij} \): 

\[
\delta_{ij} - \frac{c_{ijkl}^{ep}}{3K_s} \delta_{kl} = a \delta_{ij} + b_{ij} \tag{4.12}
\]

where \( a \) is a scalar. By multiplying Eq.(4.12) by \( \delta_{ij} \), the term \( a \) is found as:

\[
a = 1 - \frac{\delta_{ij}c_{ijkl}^{ep}\delta_{kl}}{9K_s} \quad (\delta_{ij} b_{ij} = 0) \tag{4.13}
\]

Zienkiewicz and Shiomi (1984) have recommended neglecting the deviatoric part \( b_{ij} \) of Eq.(4.12) to simplify Eq.(4.11) to:

\[
d\sigma_{ij} = C_{ijkl}^{ep}d\varepsilon_{kl} + a dp\delta_{ij} \tag{4.14}
\]

For elastic porous materials:

\[
C_{ijkl}^{ep} = C_{ijkl}^{e} = 2\mu \delta_{ik}\delta_{jl} + \lambda \delta_{ij}\delta_{kl} \tag{4.15}
\]

then \( a \) can be found from Eqs.(4.13 and 4.15) as:

\[
a = 1 - \frac{\lambda + \frac{2\mu}{K_s}}{K_s} = 1 - \frac{K}{K_s} \tag{4.16}
\]

where \( K \) is the bulk modulus of the soil skeleton. For elastoplastic materials, 

\[
K = \frac{\delta_{ij}c_{ijkl}^{ep}\delta_{kl}}{9}.
\]

To obtain a relation for pore pressure, the continuity equation of fluid flow has to be considered. The total volume change of the mixture consists of the sum of the following parts:

a) The volume change due to fluid flowing out, \( d\zeta \). This causes a decrease of the volume of the mixture.
b) The volume change in solid grains due to the pore pressure change $dp$, $\frac{dp}{K_s} (1 - \bar{n})$. This causes an increase of the volume of the mixture.

c) The volume change in fluid due to pore pressure change $dp$, $\frac{dp}{K_f} \bar{n}$, where $K_f$ is the bulk modulus of the fluid. This causes an increase in the volume of the mixture.

d) The volume change in the solid grains due to effective stress change $d\sigma'_{ij}, \frac{d\sigma'_{ij}}{3K_s}$. This causes the increase of the volume of the mixture.

Thus, the total volume change of the soil mixture $d\varepsilon_{ii}$ is given by:

$$d\varepsilon_{ii} = -d\xi + \frac{dp}{K_s} (1 - \bar{n}) + \frac{dp}{K_f} \bar{n} + \frac{d\sigma'_{ij}}{3K_s}$$  \hspace{1cm} (4.17)

Zienkiewicz and Shiomi (1984) simplified the above equation by using similar assumptions as for the derivation of Eq.(4.14) which yields:

$$dp = M(a \ d\varepsilon_{kk} + d\xi)$$ \hspace{1cm} (4.18)

where $a$ is given by Eq.(4.13) and $M$ is given by:

$$\frac{1}{M} = \frac{\bar{n}}{K_f} + \frac{a-\bar{n}}{K_s}$$ \hspace{1cm} (4.19)

where $K_s$ and $K_f$ are the bulk moduli for the solid and the fluid, respectively.

It should be noted that Eqs.(4.14 and 4.18) are similar to the incremental form of those derived by Biot (1941, 1956, and 1962) for elastic porous materials. Thus, Eqs.(4.14 and 4.18) may be used for both linear elastic systems and elastoplastic systems with constant bulk moduli of solid grains and fluid.
4.4. Dynamic Equilibrium Equations

Since the process of formulation involves two different phases, two different dynamic equilibrium equations can be derived: one for the mixture and the other for the pore fluid. For the mixture, the dynamic equilibrium equation can be written as (Biot 1962, Zienkiewicz 1980):

\[
\sigma_{ij,j} + (1 - \bar{n})\rho_s b_i + \bar{n}\rho_f b_i + (1 - \bar{n})\rho_s \ddot{u}_i - \bar{n}\rho_f \ddot{v}_i = 0
\]  

(4.20)

where \( b_i \) is the component of body force per unit mass, and overdots denote time derivatives. In the above equation, \((1 - \bar{n})\rho_s\) and \(\bar{n}\rho_f\) represent the amount of solid mass and fluid mass in a unit volume of bulk material, respectively. Substitution of Eqs.(4.2 and 4.4) into Eq.(4.20) yield;

\[
\sigma_{ij,j} + \rho b_i - \rho \ddot{u}_i - \rho_f \ddot{v}_i = 0
\]  

(4.21)

The equilibrium of fluid phase is described using the generalized form of Darcy’s equation as;

\[
\dot{\psi}_i = K_{ij} h_j
\]  

(4.22)

where \(\dot{\psi}_i\) is the average velocity of fluid relative to the solid skeleton, \(K_{ij}\) is the permeability tensor, and \(h_j\) is the gradient of the fluid head in \(j^{th}\) direction.

In general, the gradient of the fluid head is composed of three terms: pressure gradient, body force gradient and fluid inertia;

\[
h_{ii} = p_{ii} + \rho_f b_i - \rho_f \ddot{v}_i
\]  

(4.23)

where \(p_{ii}\) is the pore pressure gradient.

Substitution of Eqs.(4.4 and 4.22) into Eq.(4.23) gives;

\[
p_{ii} + \rho_f b_i = \rho_f \ddot{u}_i + \frac{\rho_f}{\bar{n}} \ddot{v}_i + K_{ij}^{-1} \dot{\psi}_j
\]  

(4.24)
where $K_{ij}^{-1}$ are components of the inverse permeability tensor.

Equilibrium equations (Eqs.(4.21 and 4.24)), strain displacement relations (Eqs.(4.5 and 4.6)), and constitutive equations (Eqs.(4.7, 4.13, 4.14, and 4.18)) completely define the behavior of a nonlinear porous media. This formulation is generally identified as $u$-$w$ formulation since the primary variables are $u_i$ and $w_i$.

A simplified form of above equations was presented by Zienkiewicz et al. (1980) for the case where the rate of change of relative velocity $\ddot{w}_i$ is negligibly small, then $u_i$ and $p$ are chosen as primary variables and $w_i$ is eliminated. This simplified form is identified as $u$-$p$ formulation in the literature. For consolidation and quasi-static problems, the same above formulation can be used with $\ddot{u} = \dot{w} = 0$. In this dissertation, the $u$-$w$ formulation is used in solving the pile problems.

4.5. Finite Element Formulation

The finite element method is an effective numerical technique for finding approximate solutions to differential equations governing physical problems. It is widely used to solve the boundary value problems in engineering whose closed-form solutions are usually difficult to obtain.

In the finite element analysis, the structure body is approximated as an assemblage of discrete finite elements with elements being interconnected at nodal points on the element boundaries. The displacements and other primary variables within each element are approximately related to the nodal values in an element through the interpolation (or shape) functions. Thus the total degrees-of-freedom of the system are reduced to those at the nodes. The virtual work principle or minimum potential energy
principle is then applied to the whole system to get a set of algebraic equations with nodal quantities as variables.

The governing equations of the problem of porous media, as given in Eqs.(4.21 and 4.22), are:

\[
\begin{align*}
\sigma_{ij,j} + \rho b_i - \rho \dot{u}_i - \rho_f \dot{w}_i &= 0 \\
p_i + \rho_f b_i - \rho_f \dot{u}_i - \frac{\rho_f}{n} \dot{w}_i - k_{ij}^{-1} \dot{w}_j &= 0
\end{align*}
\]  
(4.21)  
(4.24)

The principle of virtual work requires that for an arbitrary virtual displacement, the work done through Eqs.(4.21, and 4.24) over the domain of interest must be equal to zero. This requirement on Eq.(4.21) is given by:

\[
\int_V (\sigma_{ij,j} + \rho b_i - \rho \dot{u}_i - \rho_f \dot{w}_i) \delta u_i dV = 0
\]  
(4.25)

where \( V \) is the domain of interest, \( \delta u_i \) are the components of virtual displacement compatible with \( u_i \). Similarly, application of the virtual work principle to Eq.(4.24) results in the following:

\[
\int_V \left( p_i + \rho_f b_i - \rho_f \dot{u}_i - \frac{\rho_f}{n} \dot{w}_i - k_{ij} \dot{w}_j \right) \delta w_i dV = 0
\]  
(4.26)

where \( \delta w_i \) are components of virtual displacement compatible with \( w_i \).

Using Gauss’s theorem, Eqs.(4.25 and 4.26) can be reduced to:

\[
\begin{align*}
\int_V \rho \ddot{u}_i \delta u_i dV + \int_V \rho_f \ddot{w}_i \delta u_i dV + \int_V \sigma_{ij} \delta u_{i,j} dV &= \int_S T_i \delta u_i ds + \int_V \rho b_i \delta u_i dV \\
\int_V (\rho \ddot{u}_i \delta w_i) dV + \int_V \left( \frac{\rho_f}{n} \ddot{w}_i \delta w_i \right) dV + \int_V (k_{ij} \dot{w}_j \delta w_i) dV + \int_V (p \delta w_{i,i}) dV &= \int_S (p_i \delta w_i) ds + \int_V (\rho_f b_i \delta w_i) dV
\end{align*}
\]  
(4.27)  
(4.28)

where \( T_i = \sigma_{ij} n_j \) is the traction on the boundary, \( n_j \) is the unit vector normal to the boundary surface \( S \).
The displacements are related to the corresponding nodal values as:

\[ u_i = N_u^a U_i^a \quad (i = 1...3, a = 1...N_{ue}) \]  
\[ \delta u_i = N_u^a \delta U_i^a \quad (i = 1...3, a = 1...N_{ue}) \]  
\[ \ddot{u}_i = N_u^a \ddot{U}_i^a \quad (i = 1...3, a = 1...N_{ue}) \]  
\[ w_i = N_w^b W_i^b \quad (i = 1...3, b = 1...N_{we}) \]  
\[ \delta w_i = N_w^b \delta W_i^b \quad (i = 1...3, b = 1...N_{we}) \]  
\[ \ddot{w}_i = N_w^b \ddot{W}_i^b \quad (i = 1...3, b = 1...N_{we}) \]

where \( N_u \) and \( N_w \) are shape functions for \( u \) and \( w \), respectively, and \( \delta \) denotes variation. \( N_{ue} \) and \( N_{we} \) are node numbers per element for \( u \) and \( w \). \( U_i \) is the nodal values of \( u \) and \( W_i \) is the nodal values of \( w \). The definition of shape functions \( N \) for different elements can be found in the finite element books such as Desai (1979).

Substituting Eq.(4.29) into Eq.(4.27) and into Eq.(4.28) and eliminating the arbitrary nodal displacements \( \delta u_i \) and \( \delta w_i \), respectively yield:

\[ \sum_{V_e} \left\{ \int_{V_e} \sigma_{ij} N_{u,j}^a dV + \ddot{u}_i^a \int_{V_e} \rho N_{u}^c N_{u}^a dV + \dddot{w}_i^b \int_{V_e} \rho_j N_{w}^b N_{w}^a dV \right\} \]

\[ = \sum_{V_e} \left\{ \int_{S_e} N_{u}^a T_i dS + \int_{V_e} \rho b_i N_{u}^a dV \right\} \]

\[ \sum_{V_e} \left\{ \int_{V_e} p N_{w,i}^b dV + \dddot{w}_i^b \int_{V_e} \rho_j N_{w}^b N_{w}^a dV + \dddot{w}_i^d \int_{V_e} \rho_j N_{w}^b N_{w}^d dV + \dddot{w}_i^e \int_{V_e} k^{-1} N_{w}^b N_{w}^d dV \right\} \]

\[ = \sum_{V_e} \left\{ \int_{S_e} p N_{w}^a dS + \int_{V_e} \rho_j b_i N_{w}^a dV \right\} \]

where \( V_e \) is the volume of an element.

Equations (4.30 and 4.31) are a system of equations with the variables as nodal displacements \( U_i \) and \( W_i \) and may be rewritten in the compact form as;
where \( a, c = (1 \text{ to } N_u); \ b, d = (1 \text{ to } N_w) \) with \( N_u = N_w \) = number of nodes in the whole domain, and \( i, j = (1,2,3) \) and;

\[
\begin{align*}
M^{ac}_{uuij} &= \delta_{ij} \int_V \rho N_u^a N_u^a \, dV \\
M^{ad}_{uuij} &= \delta_{ij} \int_V \rho_f N_u^a N_w^d \, dV \\
M^{bc}_{wuij} &= \delta_{ij} \int_V \rho_f N_w^b N_u^c \, dV \\
M^{bd}_{wwij} &= \delta_{ij} \int_V \frac{\rho_f}{\alpha} N_w^b N_w^d \, dV \\
C^{bd}_{wwij} &= \int_V k^{-1} N_w^b N_w^d \, dV \\
f^{a}_{ui} &= \int_S N_u^a T_i \, dS + \int_V \rho b_i N_u^a \, dV \\
f^{b}_{wi} &= \int_S p N_w^b n_i \, dS + \int_V \rho_f b_i N_w^b \, dV
\end{align*}
\]

The primary unknowns developed in the finite element equations are the nodal quantities \( u_i \) and \( w_i \) and the secondary quantities are \( \sigma_{ij} \) and \( p \). This set of equations has to be solved together with strain displacement relations and constitutive equations given in Eqs.(4.5, 4.6, 4.14, and 4.18).

4.6. Time Integration for Dynamic Problems

The dynamic equilibrium equations (Eq. 4.32) which govern the dynamic response of a system of finite element may be written as:

\[
M_{ij} \ddot{x}_j + C_{ij} \dot{x}_j + K_{ij} x_j = f_i \quad i, j = 1 \text{ to } N_{DOF}
\]

where \( M_{ij}, C_{ij}, K_{ij} \) are the mass, damping, and stiffness matrices, \( f_i \) is the external load vector, and \( x_i, \dot{x}_i \) and \( \ddot{x}_i \) are the displacement, velocity and acceleration vectors of the
finite element assemblage. To solve the initial value problem is to find $x_j = x_j(t)$ satisfying Eq.(4.40) and the given initial conditions $x_j(0)$ and $\dot{x}_j(0)$ for a given force function $f(t)$.

The most general approach for the solution of the dynamic response of structural system is the direct numerical integration of the dynamic equilibrium equations. This involves, after the solution is defined at time zero, the attempt to satisfy dynamic equilibrium at discrete points in time. There exist a large number of accurate, higher-order, multi-step methods that have been developed for the numerical solution of differential equations.

Newmark (1959), presented a family of single-step integration for the solution of structural dynamic problems for both blast and seismic loading. During the past 50 years Newmark’s method has been applied to the dynamic analysis of many practical engineering problems. The well-known Newmark’s method is used in this study.

The time domain is divided into time steps as shown in Fig.(4.3). Time integration schemes are to find the values of $x_j$, $\dot{x}_j$ and $\ddot{x}_j$ at $t_{n+1}$ when their values at $t_n$ are known. Here $t_n$ is the time at $n^{th}$ step and $t_{n+1}$ is given by:

$$ t_{n+1} = t_n + \Delta t $$  \hspace{1cm} (4.41)

where $\Delta t$ is the time increment from $t_n$ to $t_{n+1}$. By using Taylor’s series expansion, $x_i(t_{n+1})$ is expressed as:

$$ x_i(t_{n+1}) = x_i(t_n) + \Delta t \dot{x}_i(t_n) + \frac{\Delta t^2}{2} \ddot{x}_i(t_{n+\alpha_1}); \quad t_n \leq t_{n+\alpha_1} \leq t_{n+1} $$  \hspace{1cm} (4.42)

In Newmark’s method, $\ddot{x}_i(t_{n+\alpha_1})$ is approximated as:

$$ \ddot{x}_i(t_{n+\alpha_1}) = (1 - 2\beta) \ddot{x}_i(t_n) + 2\beta \ddot{x}_i(t_{n+1}) $$  \hspace{1cm} (4.43)
where \( \beta \) is a parameter in Newmark’s scheme. From Eqs.(4.42 and 4.43), \( \ddot{x}_i(t_{n+1}) \) may be expressed as:

\[
\ddot{x}_i(t_{n+1}) = \frac{1}{\beta \Delta t^2} \{ x(t_{n+1}) - x(t_n) - \Delta t \ddot{x}_i(t_n) \} - \frac{(1-2\beta)}{2\beta} \dddot{x}_i(t_n) 
\] (4.44)

Similarly, \( \ddot{x}_i(t_{n+1}) \) can be expressed as:

\[
\ddot{x}_i(t_{n+1}) = \ddot{x}_i(t_n) + \Delta t \dddot{x}_i(t_{n+\alpha_2}); \quad t_n \leq t_{n+\alpha_2} \leq t_{n+1} 
\] (4.45)

and \( \dddot{x}_i(t_{n+\alpha_2}) \) in Eq.(4.45) is approximated as:

\[
\dddot{x}_i(t_{n+\alpha_2}) = (1 - \gamma) \dddot{x}_i(t_n) + \gamma \dddot{x}_i(t_{n+1}) 
\] (4.46)

where \( \gamma \) is another parameter in Newmark’s scheme. From Eqs.(4.45 and 4.46), \( \ddot{x}_i(t_{n+1}) \) may be expressed as:

\[
\ddot{x}_i(t_{n+1}) = \ddot{x}_i(t_n) + \Delta t \{(1 - \gamma) \dddot{x}_i(t_n) + \gamma \dddot{x}_i(t_{n+1})\} 
\] (4.47)

Substitution of Eqs.(4.44 and 4.47) into Eq.(4.40) yields:

\[
K_{ij}^* x_j(t_{n+1}) = f_i^* 
\] (4.48)

where:

\[
K_{ij}^* = \frac{1}{\beta \Delta t^2} M_{ij} + \frac{\gamma}{\beta \Delta t} C_{ij} + K_{ij} 
\] (4.49)

\[
f_i^* = f_i(t_{n+1}) + M_{ij} \left\{ \frac{x_j(t_{n})}{\beta \Delta t^2} + \frac{\dot{x}_j(t_{n})}{\beta \Delta t} + \left( \frac{1}{2\beta} - 1 \right) \ddot{x}_j(t_n) \right\} 
\]

\[
+ C_{ij} \left\{ \frac{\gamma}{\beta \Delta t} x_j(t_{n}) + \left( \frac{\gamma}{\beta} - 1 \right) \dot{x}_j(t_{n}) + \left( \frac{\gamma}{2\beta} - 1 \right) \Delta t \ddot{x}_j(t_{n}) \right\} 
\] (4.50)

In the above equations, mass matrix \( M_{ij} \) and damping matrix \( C_{ij} \) are constant. For linear problems, stiffness \( K_{ij} \) is constant and \( x_i(t_{n+1}) \) can be obtained by solving Eq.(4.48). For nonlinear problems, the stiffness matrix \( K_{ij} \) depends on \( x_i(t_{n+1}) \), and iterative techniques such as Newton-Raphson method have to be used to solve for
After \( x_i(t_{n+1}) \) are found from Eq.(4.48), \( \ddot{x}_i(t_{n+1}) \) and \( \dot{x}_i(t_{n+1}) \) can be found from Eqs.(4.44 and 4.47).

The stability of Newmark’s scheme for linear systems has been investigated by many researchers (Bathe and Wilson 1973, and Hughes 1983) and found to be:

\( 2\beta \geq \gamma \geq 0.5 \) unconditionally stable

\( \gamma \geq 0.5 \) conditionally stable with the restriction on the time step given by:

\[
\omega \Delta t \leq \Omega_{\text{crit}} = \xi \left( \gamma - \frac{1}{2} \right) + \left[ \frac{\gamma - \beta + \xi^2 (\gamma - \frac{1}{2})^2}{\frac{\gamma}{2} - \beta} \right]^\frac{1}{2} \tag{4.51}
\]

where \( \omega \) is the maximum natural frequency and \( \xi \) is the damping ratio of the system. This condition has to be satisfied for the maximum natural frequency of the system. The recommended values of \( \alpha \) and \( \beta \) are 0.5 and 0.25, which give stable results for the dynamic problems.
Fig. (4.1): Element of Porous Material and Porosity

(a). Element of Solid Skeleton

\[ V_f \quad \bar{n} \]

\[ V_s \quad 1 - \bar{n} \]

(b). Porosity Diagram

Fig. (4.1): Element of Porous Material and Porosity
Fig. (4.2): Deformation of the Two-Phase Element

a). Displacements in Two Phases

b). Relative Displacements of Fluid

Fig. (4.2): Deformation of the Two-Phase Element
Fig.(4.3): Time Steps in the Dynamic FEM
CHAPTER 5: DISTURBED STATE CONCEPT MODEL

5.1. Introduction

The disturbed state concept (DSC) is based on the well-recognized idea that a mixture's response can be expressed in terms of the responses of its interacting components (Desai, 2001). The components are considered to be material parts in the relatively intact (RI) or “continuum” state and the fully adjusted (FA) state. Under external excitation (mechanical or thermal effects), the material is assumed to transform progressively from the RI state to the FA state, Fig (5.1). The transformation involves microstructural changes that cause microcracking and damage.

The fully adjusted state is an asymptotic state that cannot be further disturbed. Hence, the observed response of the material is expressed in terms of the response of relatively intact state, which excludes the effects of disturbance, and that of the fully adjusted state (critical state). The transformation of material from RI state to FA state is defined by the disturbance function $D$, Fig (5.2). The original conceptual model was proposed by Desai (1974) and subsequently formalized for various applications. The following description is adapted from various publications, e.g., Desai (2001).

5.2. Relative Intact (RI) State

The RI behavior can be characterized by using an elastoplastic model which includes only hardening behavior and excludes the disturbance effects. Desai (2001), proposed the $\delta_0$ – version in the hierarchical single surface (HiSS) plasticity model to represent the RI state. The $\delta_0$ model is based on associative plasticity and isotropic hardening. The
behavior deviated from that predicated by the $\delta_0$ model can be considered to be caused by disturbance.

The yield surface $F$ in the $\delta_0$ model is defined as (Wathugala and Desai, 1993):

$$ F = \frac{J_{2D}}{p_a^2} - F_b F_s = 0 \quad (5.1) $$

where:

$$ F_b = -\alpha \left( \frac{J_1 + 3R}{p_a} \right)^n + \gamma \left( \frac{J_1 + 3R}{p_a} \right)^2 \quad (5.2) $$

$$ F_s = (1 - \beta S_r)^{-0.5} \quad (5.3) $$

$$ S_r = \frac{\sqrt{27} J_{3D}}{2 J_{2D}} \quad (5.4) $$

and:

$J_1, J_{2D}$ and $J_{3D}$ : invariants of stress tensor $\sigma_{ij}$.

$p_a$ : atmospheric pressure.

$F_b$ : function related to the shape of the yield surface in $J_1 - \sqrt{J_{2D}}$ plane as shown in Fig.(5.3) (with $S_r = \text{constant}$).

$F_s$ : function related to the shape of the yield surface in the octahedral plane as shown in Fig.(5.4) (with $J_1 = \text{constant}$).

$R$ : bonding stress.

$n$ : parameter related to the phase change point (a point where material changes from compaction to dilation). Phase change points are connected by a line called a phase – change line.

$\gamma$ : parameter related to the ultimate yield surface.

$\beta$ : parameter related to the shaped of $F$ in the $\sigma_1 - \sigma_2 - \sigma_3$ space as shown in Fig.(5.4).
$S_r$: stress ratio.

$\alpha$: growth or hardening parameter, which can be expressed in terms of internal variables.

Based on the development of the hardening function for a wide range of solid materials, interfaces and joints, it was found that use of the plastic strain trajectory provides a more consistent formulation (Desai, 2001). The basic form of $\alpha$ is given as:

$$\alpha = \frac{h_1}{\xi h_2} \tag{5.5}$$

where $h_1$ and $h_2$ are material constants and $\xi$ is trajectory of plastic strain expressed as

$$\xi = \int \sqrt{d\varepsilon_{ij}^\text{p} d\varepsilon_{ij}^\text{p}} \tag{5.6-a}$$

or

$$d\xi = \sqrt{d\varepsilon_{ij}^\text{p} d\varepsilon_{ij}^\text{p}} \tag{5.6-b}$$

The trajectory of total plastic strains $\xi$ composed of deviatoric $\xi_D$ and volumetric $\xi_V$ plastic strain trajectories:

$$\xi = \xi_D + \xi_V = \int \sqrt{dE_{ij}^\text{p} dE_{ij}^\text{p}} + \int \frac{1}{\sqrt{5}} |d\varepsilon_{ij}^\text{p}| \tag{5.7}$$

where $E_{ij}^\text{p}$ is the tensor of deviatoric plastic strain:

$$E_{ij}^\text{p} = \varepsilon_{ij}^\text{p} - \frac{1}{3} \delta_{ij} \varepsilon_{kk}^\text{p} \tag{5.8}$$

From plasticity theory with small strains, the total strain $\varepsilon_{ij}$ can be decomposed into elastic strain $\varepsilon_{ij}^e$ and plastic strain $\varepsilon_{ij}^p$ components as:

$$\varepsilon_{ij} = \varepsilon_{ij}^e + \varepsilon_{ij}^p \tag{5.9}$$

or in incremental form:

$$d\varepsilon_{ij} = d\varepsilon_{ij}^e + d\varepsilon_{ij}^p \tag{5.10}$$
The stress is assumed to be related to the elastic strain by the elastic constitutive relation as:

$$d\sigma_{ij} = C^e_{ijkl} d\varepsilon^e_{kl}$$  \hspace{1cm} (5.11)

According to plasticity theory (associative flow rule), the plastic strain increment is orthogonal to the potential surface \((Q = F)\). Thus, the plastic strain increment can be expressed as:

$$d\varepsilon^p_{ij} = \tilde{\lambda} n^Q_{ij}$$  \hspace{1cm} (5.12)

where \(\tilde{\lambda}\) is a scalar factor of proportionality and \(n^Q_{ij}\) is a unit tensor normal to the potential surface \(Q\) given as:

$$n^Q_{ij} = \frac{\partial Q}{\partial \sigma_{ij}} = \frac{\partial Q}{\partial \sigma_{ij}} \left( \frac{\partial Q}{\partial \sigma_{ij}} \right)^{1/2}$$  \hspace{1cm} (5.13)

where \(\|\|\) denotes norm. Substitution of Eq.(5.12) into Eq. (5.6b) leads to:

$$d\xi = \tilde{\lambda}$$  \hspace{1cm} (5.14)

Using Eqs. (5.10, 5.11 and 5.12), we can write:

$$d\sigma_{ij} = C^e_{ijkl}(d\varepsilon_{kl} - d\varepsilon^p_{kl}) = C^e_{ijkl}(d\varepsilon_{kl} - d\varepsilon_{kl} n^Q_{kl})$$  \hspace{1cm} (5.15)

Differentiating yield function, \(F = 0\), yields consistency condition as:

$$dF = \frac{\partial F}{\partial \sigma_{ij}} d\sigma_{ij} + \frac{\partial F}{\partial \xi} d\xi = 0$$  \hspace{1cm} (5.16)

Substitution of Eq. (5.15) into Eq. (5.16) gives:

$$d\xi \left( = \tilde{\lambda} \right) = \frac{\partial F}{\partial \sigma_{ij}} C^e_{ijkl} d\varepsilon_{kl}$$

Now substitution of \(d\xi\) in Eq. (5.15) leads to:
where

\[ (5.18) \]

\[
\begin{aligned}
\frac{d\sigma_{ij}}{d\varepsilon_{kl}} &= C_{ijkl}^e d\varepsilon_{kl} - \frac{c_{ijkl}^e Q}{\sigma_{pq} Q} \frac{\partial F}{\partial \sigma_{pq}} C_{ijkl}^e d\varepsilon_{kl} \\
&= C_{ijkl}^{e_p} d\varepsilon_{kl}
\end{aligned}
\]

Equation (5.18) is the incremental form of the stress – strain relation for the RI state.

5.3. Fully Adjusted (FA) State

The material parts that have reached the FA state during deformation can be considered to be at the critical state. When the material is at the critical state, there is no further change in its volume or void ratio, and it deforms in shear under constant value of the shear stress. Such critical state often lies on a straight line called "Critical State Line (CSL)" with slope \(m\) as shown in Fig.(5.5):

\[ (5.20) \]

\[ \sqrt{J_{2D}} = m J_1^c \]

The final void ratio \(e^c\) of the material attained in a shear test is related to the hydrostatic stress at the critical state \(J_1^c\) as:

\[ (5.21) \]

\[ e^c = e_0^c - \lambda \ln \left( \frac{J_1^c}{3p_a} \right) \]

or

\[ (5.22) \]

\[ J_1^c = 3p_a \exp \left( \frac{e_0^c - e^c}{\lambda} \right) \]

where \(\lambda\) is the slope of the critical state line, Fig.(5.6), \(e_0^c\) is the value of \(e^c\) corresponding to \(J_1^c = 3p_a\), where \(p_a\) is the atmospheric pressure.
5.4. Disturbance Function $D$

As mentioned above, the material deformation process is characterized as a mixture of a two-phase material. The observed response of a material is defined by the contribution of the behavior of each reference state of the material connected through the disturbance function $D$. In general, the material damage is not isotropic and therefore $D$ is a tensor rather than a scalar. It is usually difficult to measure the directional properties of deforming materials. However, for practical purposes, it may often be sufficient to treat $D$ as scalar, in an average and weighted sense (Desai, 2001).

There are a number of ways to define $D$. The most direct way is to measure mass, volume, or area of the material in the fully adjusted state and express $D$ as a ratio of these quantities to the corresponding total mass, volume, or area of the material. However, it is difficult to measure these quantities due to the lack of proper testing devices and hence, indirect approaches are needed to express disturbance $D$. One way is to define $D$ from measurements of stress – strain response. Another way to express $D$ is in terms of some internal material variables such as plastic strain trajectory.

From the stress – strain curve, the disturbance $D$ can be defined as:

$$D = \frac{\sigma^d - \sigma^a}{\sigma^i - \sigma^c}$$  \hspace{1cm} (5.23)

where $i$, $a$ and $c$ denote the relatively intact, observed and fully adjusted stress states and $\sigma$ denotes the measure of stress such as $\tau$ or $\sqrt{J_{2D}}$.

As a material progresses from its initial relative intact state to its final fully adjusted state, the disturbance increases from its initial value of approximately zero to an asymptotic value of 1.0, as shown in Fig.(5.7).
The disturbance curve shown in Fig.(5.7) can be expressed by a Weibull Distribution as (Weibull, 1949):

\[ D = D_u(1 - e^{-A_\xi D}) \]  
(5.24)

where \( D_u \) is the ultimate disturbance, \( A \) and \( Z \) are material parameters, and \( \xi D \) is deviatoric plastic strain trajectory as defined earlier. Figure (5.8) shows the disturbance function with two sets of different constants: \((A_1, Z_1)\) and \((A_2, Z_2)\).

5.5. Observed or Actual Behavior

The Observed or actual response of the material is expressed in terms of the responses of material parts in the reference states (RI and FA states). The disturbance \( D \) denotes the deviation of the observed response from that of the reference states. Figure (5.9) shows a symbolic and schematic representation of disturbance in the DSC. The observed or average response (denoted by \( a \)) is then expressed in terms of the RI response (denoted by \( i \)) and the FA response (denoted by \( c \)) by using the disturbance function \( D \) as an interpolation and coupling mechanism.

Consider the equilibrium of a material element with total area \( A \) and thickness \( t \) subjected to a total force \( F \) as shown in Fig.(5.10). It is assumed that the force \( F \) is carried by the material in both the RI and FA states. Hence:

\[ F = F^i + F^c \]  
(5.25)

Division of both sides by the total area \( A \) leads to:

\[ \frac{F}{A} = \frac{F^i}{A^i} \cdot \frac{A^i}{A} + \frac{F^c}{A^c} \cdot \frac{A^c}{A} \]  
(5.26)

By definition, the disturbance \( D \) could be expressed as:

\[ D = \frac{\nu^c}{\nu} = \frac{A^c t}{A t} = \frac{A^c}{A} \]  
(5.27)
Substitution of Eq.(5.27) into Eq.(5.26) and using tensor notation for stresses leads to:

\[
\sigma_{ij}^a = (1 - D)\sigma_{ij}^1 + D\sigma_{ij}^c \tag{5.28}
\]

where \(\sigma_{ij}^a\), \(\sigma_{ij}^1\) and \(\sigma_{ij}^c\) are the stresses in the observed, RI, and FA states, respectively.

Contracting the index of the above equation \((J_1 = \sigma_{ii})\) gives:

\[
J_1^a = (1 - D)J_1^1 + DJ_1^c \tag{5.29}
\]

Subtracting one third of Eq.(5.29) from Eq.(5.28) leads to a similar relation for the deviatoric stress, \(S_{ij}\):

\[
S_{ij}^a = (1 - D)S_{ij}^1 + DS_{ij}^c \tag{5.30}
\]

It is assumed that the deviatoric stress \(S_{ij}\) in the FA state is proportional to that in the RI state. Then the following relation can be derived as:

\[
S_{ij}^c = \frac{\sqrt{J_{2D}^c}S_{ij}^1}{\sqrt{J_{2D}^1}} = \frac{\sqrt{J_{2D}^c}S_{ij}^a}{\sqrt{J_{2D}^c}} \tag{5.31}
\]

Equations (5.30) and (5.31) lead to:

\[
S_{ij}^a S_{ij}^a = [(1 - D)S_{ij}^1 + DS_{ij}^c][(1 - D)S_{ij}^1 + DS_{ij}^c]
\]

\[
= [1 - D + D\frac{\sqrt{J_{2D}^c}}{\sqrt{J_{2D}^1}}] S_{ij}^i S_{ij}^i
\]

\[
= 2 [1 - D]\sqrt{J_{2D}^i} + D\sqrt{J_{2D}^c}]^2 \tag{5.32}
\]

That is

\[
\sqrt{J_{2D}^a} = (1 - D)\sqrt{J_{2D}^i} + D\sqrt{J_{2D}^c} \tag{5.33}
\]
5.6. Incremental Formulation for DSC (Solid)

The incremental form of the stress relation can be found by differentiating Eq. (5.28) as:

\[
d\sigma_{ij}^c = (1 - D)d\sigma_{ij}^e + Dd\sigma_{ij}^c + dD(\sigma_{ij}^c - \sigma_{ij}^l)
\]  
(5.34)

Using Eqs. (5.31 and 5.20), \(\sigma_{ij}^c\) can be expressed as:

\[
\sigma_{ij}^c = S_{ij}^c + \frac{1}{3}J_2^c\delta_{ij} = \frac{\sqrt{J_2^c}}{\sqrt{J_{2D}^c}}S_{ij}^l + \frac{1}{3}J_2^c\delta_{ij}
\]

\[
= \frac{mJ_2^c}{\sqrt{J_{2D}^c}}S_{ij}^l + \frac{J_2^c}{3}\delta_{ij} = J_2^c \left( \frac{mS_{ij}^l}{\sqrt{J_{2D}^c}} + \frac{1}{3}\delta_{ij} \right)
\]  
(5.35)

Differentiating Eq. (5.22) yields:

\[
dJ_2^c = 3p_a \exp \left( \frac{e_0^c - e^c}{\bar{\lambda}} \right) \left( -\frac{d\sigma^c}{\bar{\lambda}} \right) = \frac{J_2^c}{\bar{\lambda}} (1 + e_o) d\varepsilon^c_{li}
\]  
(5.36)

and

\[
dS_{ij}^l = d\sigma_{ij}^l - \frac{1}{3}\delta_{ij}d\sigma_{mm}^l
\]

\[
= C_{ijkl}^{ep}d\varepsilon_{kl}^l - \frac{1}{3}\delta_{ij}C_{mmkl}^{ep}d\varepsilon_{kl}^l
\]

\[
= \left( C_{ijkl}^{ep} - \frac{1}{3}\delta_{ij}C_{mmkl}^{ep} \right) d\varepsilon_{kl}^l
\]  
(5.37)

\[
dJ_2^l = d \left( \frac{1}{2}S_{ij}^lS_{ij}^l \right) = S_{ij}^ldS_{ij}^l = S_{ij}^l d \left( \sigma_{ij}^l - \frac{J_2^l}{3}\delta_{ij} \right)
\]

\[
= S_{ij}^l d\sigma_{ij}^l = S_{ij}^l C_{ijkl}^{ep} d\varepsilon_{kl}^l \quad \left[ S_{ij}^l \delta_{ij} = S_{li}^l = 0 \right]
\]  
(5.38)

Differentiation of Eq. (5.35) leads to:

\[
d\sigma_{ij}^c = dJ_2^c \left( \frac{m}{\sqrt{J_{2D}^l}}S_{ij}^l + \frac{1}{3}\delta_{ij} \right) + J_2^c \frac{m}{\sqrt{J_{2D}^l}} \left( dS_{ij}^l - \frac{S_{ij}^l}{2J_{2D}^l} dJ_2^l \right)
\]
The $dD$ term is evaluated by differentiating Eq. (5.24) as:

$$dD = \frac{dD}{d\xi_D} d\xi_D = D_u A e^{-A \xi_D^2} Z \xi_D^{Z-1} d\xi_D$$

(5.40)

where $d\xi_D$ can be derived from Eqs. (5.7), (5.8), (5.12) and (5.17) as:

$$d\xi_D = (n_{Dpq}^Q n_{Dpq}^Q) \frac{1}{2} \frac{\partial \xi}{\partial \sigma_{ij}^e} \frac{\partial e_{ijkl}^D}{\partial \xi}$$

(5.41)

Hence:

$$dD = D_u A e^{-A \xi_D^2} Z \xi_D^{Z-1} (n_{Dpq}^Q n_{Dpq}^Q) \frac{1}{2} \frac{\partial \xi}{\partial \sigma_{ij}^e} \frac{\partial e_{ijkl}^D}{\partial \xi}$$

(5.42)

where:

$$n_{Dij}^Q = n_{ij}^Q - \frac{1}{3} \delta_{ij} n_{kk}^Q$$

(5.43)

The average stress in incremental form is finally obtained as:

$$d\sigma_{ij}^e = (1 - D) C_{ijkl}^{ep} d\varepsilon_{kl}^i + D \Psi_{ij} \delta_{kl} d\varepsilon_{kl}^i + D \Omega_{ijkl} d\varepsilon_{kl}^i + (\sigma_{ij}^e - \sigma_{ij}^l) R_{kl} d\varepsilon_{kl}^i$$

(5.44)

where:

$$\Psi_{ij} = \frac{\mu}{\lambda} (1 + e_o) \left( \frac{\mu \delta_{ij}^l}{\lambda^l} + \frac{1}{3} \delta_{ij} \right)$$

(5.45)
The strains in RI part and FA part are usually different. Assume:

\[ d \varepsilon_{ij}^c = \beta_1 d \varepsilon_{ij}^l \]  
\[ d \varepsilon_{ij}^l = \beta_2 d \varepsilon_{ij}^a \]  

Where \( \beta_1 \) and \( \beta_2 \) are two scalar numbers. Then Eq. (5.44) can be rewritten as:

\[ d \sigma_{ij}^a = C^{DSC}_{ijkl} d \varepsilon_{kl}^a \]  

where:

\[ C^{DSC}_{ijkl} = (1 - D)C^{ep}_{ijkl} + D \Psi_{ij} \delta_{kl} \beta_2 + D \Omega_{ijkl} \beta_2 + (\sigma_{ij}^c - \sigma_{ij}^l)R_{kl} \beta_2 \]  

In case of undrained loading, there is no volume change, hence \( d \varepsilon_{ii} = 0 \). Thus:

\[ d \varepsilon_{ij}^c = d \varepsilon_{ij}^l = d \varepsilon_{ij}^a \]  

which leads to:

\[ \beta_1 = \beta_2 = 1 \]

Then \( C^{DSC}_{ijkl} \) in Eq. (5.51) reduced to:

\[ C^{DSC}_{ijkl} = (1 - D)C^{ep}_{ijkl} + D \Psi_{ij} \delta_{kl} + D \Omega_{ijkl} + (\sigma_{ij}^c - \sigma_{ij}^l)R_{kl} \]  

5.7. DSC Formulation for Interfaces

An interface is an actual or idealized thin region between two solid bodies in which stresses and displacements may vary substantially from those of the neighboring solid bodies. To mathematically model a joint or interface, an idealization is used which assumes the interface zone to be a planar surface with an averaged thickness \( t \), (Fig. 5.11).
A coordinate system can be established in which the planar surface is considered the tangential direction with shear stress $\tau$ and relative shear displacement $u_r$ and the direction orthogonal to the planar surface is the normal direction with normal stress $\sigma_n$ and relative normal displacement $v_r$. Interface stresses are defined as the averaged applied force over the nominal planar area of the interface. If $T$ and $N$ are the tangential and normal forces applied, and $A_o$ is the interface area, then the normal and shear stresses are:

$$\tau \equiv \frac{T}{A_o} \quad (5.54)$$

$$\sigma_n \equiv \frac{N}{A_o} \quad (5.55)$$

The displacements of an interface are different from the deformation of a continuum in that slip displacements are included. For example, consider the shear displacements measured by the CYMDOF-P device, the displacement of the steel solid body $u_t$ is measured by the device and the simple shear displacement of the soil $u_s$ is measured by the simple shear rings. The difference between $u_t$ and $u_s$ is the relative displacement $u_r$. The relative shear displacement in the interface zone is composed of elastic shear displacement $u^e$, plastic shear displacement $\bar{u}^p$, and slip displacement between the top of the soil and the steel solid body $u^s$:

$$u_r = u^e + \bar{u}^p + u^s \quad (5.56)$$

In an analogous manner the normal displacements can be defined as:

$$v_r = v^e + \bar{v}^p + v^s \quad (5.57)$$

Since the last two displacements in Eqs.(5.56 and 5.57) are difficult to separate and are irrecoverable, they are combined as $u^p$ and $v^p$ as:
In terms of a two-dimensional idealization of interfaces, the strain–displacement relations are:

\[ u_r = u^e + u^p \]  \hspace{1cm} (5.58)
\[ v_r = u^e + u^p \]  \hspace{1cm} (5.59)

and the related stress components are:

\[
\begin{bmatrix}
\varepsilon_x \\
\varepsilon_n \\
\gamma
\end{bmatrix} \approx \begin{bmatrix}
0 \\
v_r/t \\
u_r/t
\end{bmatrix}
\]  \hspace{1cm} (5.60)

5.7.1. Relative Intact (RI) State

The RI behavior can be simulated using an elastoplastic model. The \( \delta_o \)-version of the HiSS plasticity model is usually used (Desai, 2001). The equations of HiSS model have been specialized for interfaces by Desai and Fishman (1991). This specialization for interfaces will be briefly discussed.

In Eq. (5.1), \( F_s \) controls the shape of the yield function in principal stress space (in the octahedral planes). When specialized to interfaces, \( F \) is plotted only in the \( \sigma_n \) versus \( \tau \) plane. Therefore, \( F_s \) will be taken as unity (\( \beta = 0 \)). The parameter \( n \) will control the shape of \( F \) plotted in \( \sigma_n \) versus \( \tau \) space and act as the phase change parameter denoting transition from a contractive to dilative response. For interfaces the following analogies are made:

\[ J_1 \rightarrow \sigma_n \]  \hspace{1cm} (5.62)
\[ J_{2D} \rightarrow \frac{1}{3} \sigma_n^2 + \tau^2 \]  \hspace{1cm} (5.63)
Substituting the above equations into Eqs. (5.1 and 5.2) with \( R = 0 \) (for cohesionless material) gives:

\[
F = \left( \frac{r}{p_a} \right)^2 + \alpha \left( \frac{\sigma_n}{p_a} \right)^n - \bar{\gamma} \left( \frac{\sigma_n}{p_a} \right)^2 = 0
\]  
(5.64)

where \( \sigma_n \) is the effective stress. The hardening growth function (as defined before) expressed as:

\[
\alpha = \frac{h_1}{\xi h_2}
\]  
(5.65)

where \( h_1 \) and \( h_2 \) are the hardening parameters and \( \xi \) is the trajectory of plastic shear and normal displacements, expressed as:

\[
\xi = \int (d u^p_r d u^p_r + d v^p_r d v^p_r)^{\frac{1}{2}} = \xi_D + \xi_v
\]  
(5.66)

Here, \( u^p_r \) and \( v^p_r \) are the relative plastic shear and normal displacements respectively. The total values of relative shear and normal displacements \( u_r \) and \( v_r \) are defined in Eqs. (5.58 and 5.59). For interfaces, the plastic strains related to \( v_r \) are often small. Since the clearly dominant plastic strains are shear strains, a hardening function of the following form is proposed:

\[
\alpha = \frac{h_1}{\xi_D^{h_2}}
\]  
(5.67)

where \( h_1 \) and \( h_2 \) are model parameters and:

\[
\xi_D = \int |d \gamma^p|
\]  
(5.68)

and

\[
d \gamma^p = d \gamma - d \gamma^e = d \gamma - \frac{1}{G} d \tau
\]  
(5.69)
Derivation of the intact incremental stress – strain relationship follows the traditional elastoplasticity formulation procedure (Wathugala and Desai 1990, Ma and Desai 1990). The stress and total displacement vectors can be defined as:

\[ \{\sigma^I\} = \begin{bmatrix} \tau^I \\ \sigma_n \end{bmatrix} \]  \hspace{1cm} (5.70)

\[ \{\mathbf{u}^I\} = \begin{bmatrix} \mathbf{u}^{ie} \\ \mathbf{v}^{ie} \end{bmatrix} \]  \hspace{1cm} (5.71)

The elastic displacement and the relative displacement are expressed as:

\[ \{\mathbf{u}^I\}^e = \begin{bmatrix} \mathbf{u}^{ie} \\ \mathbf{v}^{ie} \end{bmatrix} \]  \hspace{1cm} (5.72)

and

\[ \{\mathbf{u}^I\}^r = \begin{bmatrix} \mathbf{u}^{ir} \\ \mathbf{v}^{ir} \end{bmatrix} \]  \hspace{1cm} (5.73)

The elastic stiffness matrix is expressed as:

\[ [K] = \begin{bmatrix} K_s & 0 \\ 0 & K_n \end{bmatrix} \]  \hspace{1cm} (5.74)

where \( K_s \) and \( K_n \) are elastic shear and normal stiffnesses and the coupling terms are ignored. The incremental stress – elastic displacement relation can be written as:

\[ \{d\sigma^I\} = [K]\{d\mathbf{u}^I\}^e \] \hspace{1cm} (5.75)

Following the same procedure given in section (5.2), we can derive the final stress – displacement relation as:

\[ \{d\sigma^I\} = \left[K^I\right]^{ep}\{d\mathbf{u}^I\} \]  \hspace{1cm} (5.76)

where:
5.7.2. Fully Adjusted (FA) State

As expressed in the case of solids, fully adjusted state for interfaces can be characterized by critical state concept as discussed in section (5.3). The void ratio – pressure relationship for interfaces can be expressed as:

\[ e^c = e_o^c - \lambda \ln \left( \frac{\sigma_n^c}{p_a} \right) \]  \hspace{1cm} (5.78)

where \( e_o^c \) is the void ratio at \( \sigma_n^c = p_a \) and \( \lambda \) is the slope of \( e^c - \ln \left( \frac{\sigma_n^c}{p_a} \right) \) line as shown in Fig.(5.12). At critical state, the maximum shear stress is expressed as:

\[ \tau^c = \bar{m} \sigma_n^c \]  \hspace{1cm} (5.79)

where \( \bar{m} \) is the slope of the critical state line as shown in Fig.(5.13).

5.7.3. Disturbance Function

The disturbance function \( D \) can be defined based on the shear stress \( \tau \) versus relative shear displacement, \( u_r \) (Fig.5.14) as:

\[ D = \frac{\tau^i - \tau^a}{\tau^i - \tau^c} \]  \hspace{1cm} (5.80)

where \( i, a, \) and \( c \) denote the relatively intact, observed and fully adjusted stress states.

For cyclic behavior, disturbance as a function of cycles \( N \) is calculated based on the RI and peak values of the quantities:

\[ D (N) = \frac{\tau^i - \tau^a}{\tau^i - \tau^c} \]  \hspace{1cm} (5.81)
In the above equation, \( \tau^l \) is the relatively intact term which is the peak of the first cycle, 
\( \tau^p \) is the peak of each cycle, and \( \tau^c \) is the critical shear stress (peak) at the last cycle in the test.

The disturbance Eqs. (5.80 and 5.81) can be expressed in terms of deviatoric plastic strain trajectory \( \xi_D \) as:

\[
D = D_u (1 - e^{-A \xi_D})
\]  
(5.82)

**5.7.4. Incremental Relations**

Eq. (5.28) can be specialized for interfaces as:

\[
\begin{align*}
\left\{ \frac{\tau^a}{\sigma_n^a} \right\} &= (1 - D) \left\{ \frac{\tau^l}{\sigma_n^l} \right\} + D \left\{ \frac{\tau^c}{\sigma_n^c} \right\}
\end{align*}
\]  
(5.83)

Differentiating Eq. (5.83):

\[
\begin{align*}
\left\{ \frac{d\tau^a}{d\sigma_n^a} \right\} &= (1 - D) \left\{ \frac{d\tau^l}{d\sigma_n^l} \right\} + D \left\{ \frac{d\tau^c}{d\sigma_n^c} \right\} + D \left\{ \frac{\tau^c - \tau^l}{\sigma_n^c - \sigma_n^l} \right\}
\end{align*}
\]  
(5.84)

Taking the derivative of Eq. (5.78), the incremental normal strain can be written as:

\[
d\varepsilon_n^c = -\frac{de}{1+e_0} = \frac{\lambda}{1+e_0} \frac{d\sigma_n^c}{\sigma_n^c}
\]  
(5.85)

or

\[
d\sigma_n^c = \sigma_n^c \frac{1+e_0}{\lambda} d\varepsilon_n^c = \frac{1+e_0}{\lambda t} \sigma_n^c d\nu^c
\]  
(5.86)

Differentiating Eq. (5.79):

\[
d\tau^c = \overline{m} d\sigma_n^c = \frac{\overline{m}}{\lambda t} (1 + e_0) \sigma_n^c d\nu^c
\]  
(5.87)

Substituting Eqs.(5.76, 5.79, 5.86 and 5.87) into Eq.(5.84) gives the DSC incremental stress-strain equations:

\[
\begin{align*}
\left\{ \frac{d\tau^a}{d\sigma_n^a} \right\} &= (1 - D) [K_i]^{ep} \left\{ \frac{du^l}{dv^l} \right\} + D \left\{ \frac{1+e_0}{\lambda t} \frac{\tau^c du^c}{\sigma_n^c dv^c} \right\} + D \left\{ \frac{\tau^c - \tau^l}{\sigma_n^c - \sigma_n^l} \right\}
\end{align*}
\]  
(5.88)
or in terms of matrix notation,

\[
\{d\sigma^a\} = (1 - D) [K^i]^{ep} \{du^i\} + [M] \{du^c\} + dD \{\sigma^c - \sigma^i\}
\] (5.89)

and combining common terms since \(\{du\} = \{du^i\} = \{du^c\}:

\[
\{d\sigma^a\} = [K]^{DSC} \{du\} + dD \{\sigma^c - \sigma^i\}
\] (5.90)

where:

\[
[K]^{DSC} = (I - D) [K^i]^{ep} + [M]
\] (5.91)

\[
[M] = \{D\} \frac{1 + e_0}{\lambda t} \{\sigma^c\}
\] (5.92)

and \([K]^{DSC}\) is the DSC stiffness matrix.
Fig. (5.1): Schematic of Growth of Fully Adjusted (FA) State

Intact Zones

Fully Adjusted Zones

Initial  Intermediate  “Failure”

Fig. (5.2): Schematic of Stress-Strain Curve with Disturbance

$\sigma^c$

$\sigma^i$

$\sigma^a$

Ultimate

Relative intact behavior

Observed behavior

Critical behavior (FA behavior)

Peak

1-D

$\sigma$

Strain

Stress
Fig. (5.3): Yield Surface in $J_1$-$J_{2D}$ Plane

Fig. (5.4): Yield Surface in Octahedral Plane Defined by $F_S$
Fig.(5.5): Plot of Critical State line in $J_1^c - \sqrt{J_2^c}$ Plane

Fig.(5.6): Plot of Critical State Line in $\ln \left( \frac{J_1^c}{3p_a} \right)$ - $e^c$ Plane
Fig.(5.7): Disturbance Function

Fig.(5.8): Schematics of Disturbance Function $D$ vs. $\xi_D$
Fig. (5.9): Representation of DSC

(a) Symbolic Representation of DSC

(b) Schematic of Stress-Strain behavior

Fig. (5.9): Representation of DSC
Fig.(5.10): Schematic of a Disturbed Element Subjected to Loading

Fig.(5.11): Interface Definitions

\[ u_r = u_t - u_s \]
Fig. (5.12): Plot of Critical State Line in $\ln \left( \frac{\sigma_n^c}{p_a} \right) - e^c$ Plane

Fig. (5.13): Plot of Critical State Line in $\sigma_n^c - \tau^c$ Plane
Fig.(5.14): Schematic of Shear Stress – Relative Shear Displacement
6.1. Introduction

The proposed interface model involves a number of material parameters which can be determined from a series of tests on interfaces. The basic parameters in the DSC model for interfaces can be grouped into four categories as shown in Table (6.1). This chapter presents procedures and typical plots for determining material parameters from laboratory test data. Once these parameters are defined, verification of the proposed model is discussed by back prediction of both monotonic and cyclic shear tests.

Three normal compression tests and thirty one simple shear tests: nine one-way monotonic, nine one-way cyclic, and thirteen two-way cyclic tests were used to find the parameters of saturated Ottawa sand-steel interfaces. These tests were divided into three groups each with a specific roughness of a steel surface. Three tests SS1, SS3 and SS29 were excluded from parameters calculation and are used to verify the proposed model.

6.2. Elastic Parameters

For elastic, isotropic material, two independent constants are required in the constitutive tensor: Young's modulus $E$ and Poisson's ratio $\nu$ (or bulk modulus $K$ and shear modulus $G$). However, in the case of interfaces, normal stiffness $K_n$ and shear stiffness $K_s$ are usually used instead.

For geomaterials, virgin loading behavior is nonlinear even from the beginning of loading. Therefore, the elastic normal and shear stiffnesses, $K_n$ and $K_s$, are calculated
from the unloading slopes of normal stress – normal displacement relation and shear stress - relative shear displacement relation, respectively. These two relations are obtained from a one-way cyclic normal compression test and a one-way cyclic shear test.

Fig. (6.1) shows a typical determination of normal stiffness from the unloading part of normal stress – normal displacement plot. The average values of $K_n$ for different roughnesses for saturated interfaces are listed in Table (6.2). Fig. (6.2) shows typical determination of shear stiffness from the unloading part of shear stress – relative shear displacement plot. It is found that $K_s$ varies with the normal stress and it is expressed as a function of $\sigma_n$ as shown in Figs. (6.3 - 6.5). Table (6.3) summarizes the values of $K_s$ for saturated one-way cyclic shear tests with different roughnesses.

Referring to Tables (6.2 and 6.3), it is clear that the values of normal stiffness and average shear stiffness vary with the surface roughness of steel. Power regression between surface roughness $R_s$ and normal stiffness $K_n$ as shown in Fig. (6.6) gives the following relation with coefficient of regression $R^2 = 0.958$:

$$K_n = 280.9 \ R_s^{1.442} \quad (6.1)$$

and the plot between surface roughness $R_s$ and shear stiffness $K_s$ as shown in Fig. (6.7) gives (with $R^2 = 0.97$):

$$K_s = 94.2 \ R_s^{1.596} \quad (6.2)$$

### 6.3. Plasticity Parameters

The HISS plasticity model, which is used for the RI behavior in the DSC, involves six parameters: $\beta$, $\gamma$, $n$, $h_1$, $h_2$ and $R$. The parameter $R$ is related to the adhesive strength of a material. For sand and normally consolidated clay, $R$ is usually set to zero.
The parameter $\beta$ is related to the shape of yield function $F$ in the $\sigma_1 - \sigma_2 - \sigma_3$ space. In the case of interfaces, the state of stress reduced to two components only $\sigma$ and $\tau$, and $F$ can be represented in plane rather than in three dimensional stress space. Therefore, $F_s$ in Eq.(5.1) should be constant and this is possible if the value of $\beta$ is not relevant. The other plasticity parameters are discussed in details below.

**6.3.1. Ultimate Parameter $\gamma$**

This parameter represents the asymptotic ultimate stress. The yield surface $F$ becomes an approximate straight line in the $\tau - \sigma_n$ space when the hardening parameter $\alpha = 0$, with a slope of $\sqrt{\gamma}$. Substituting $\alpha = 0$ in Eq.(5.64) gives:

$$\sqrt{\gamma} = \frac{\tau}{\sigma_n}$$  \hspace{1cm} (6.3)

Figs.(6.8 - 6.10) show the plots of ultimate shear stress with normal stress for saturated interfaces. It is found from the plots that the values of $\gamma$ for smooth, slightly textured, and moderately textured roughnesses are, respectively, 0.196, 0.377 and 0.418. These values are plotted against corresponding values of the surface roughness $R_s$ as shown in Fig.(6.11). The graph gives the following relation (with $R^2 = 0.993$):

$$\gamma = 0.173 \ R_s^{2.318}$$  \hspace{1cm} (6.4)

**6.3.2. Phase Change Parameter $n$**

This parameter is related to the state of stress at which the material passes through the state of zero volume change. Fig (6.12) shows the schematic of yield surface, phase change line and ultimate line. At point $A$, the soil experiences transition from contraction to dilation. This can be expressed mathematically as:
\[
\frac{\partial F}{\partial \sigma_{nA}} = 0
\]  
(6.5)

Substitution of \( F = 0 \) (Eq. 5.64) into Eq. (6.5) yields:

\[
\frac{\partial F}{\partial \sigma_{nA}} = \alpha n \frac{\sigma_{nA}^{n-1}}{p_a^2} - 2 \gamma \frac{\sigma_{nA}}{p_a} = 0
\]  
(6.6)

or

\[
\alpha n \left( \frac{\sigma_{nA}}{p_a} \right)^{n-2} - 2 \gamma = 0
\]

which leads to:

\[
\frac{\sigma_{nA}}{p_a} = \left( \frac{2 \gamma}{\alpha n} \right)^{\frac{1}{n-2}}
\]  
(6.7)

At point A, Eq. (5.64) can be written as:

\[
\frac{\tau_A^2}{p_a^2} = -\alpha \frac{\sigma_{nA}^n}{p_a^2} + \gamma \frac{\sigma_{nA}^2}{p_a^2}
\]  
(6.8)

Thus, the slope of the phase change line becomes:

\[
S_{pc} = \frac{\tau_A}{\sigma_{nA}} = \sqrt{-\alpha \left( \frac{\sigma_{nA}}{p_a} \right)^{n-2} + \gamma} = \sqrt{(1 - \frac{2}{n})\gamma}
\]  
(6.9)

The slope of the ultimate line is defined as:

\[
S_{ul} = \sqrt{\gamma}
\]  
(6.10)

Therefore, the ratio of slopes of the phase change line and the ultimate line can be obtained as:

\[
\frac{S_{pc}}{S_{ul}} = \sqrt{\frac{n-2}{n}}
\]  
(6.11)

or

\[
n = \frac{2}{1 - \left( \frac{S_{pc}}{S_{ul}} \right)^2}
\]  
(6.12)
Equation (6.12) is used in this research to find the value of $n$ for saturated interfaces. The slope of the ultimate line, or $\sqrt{\gamma}$, is calculated in the previous section and the slope of phase change line can be calculated from stress path plots for the tests with different normal stresses. Figs.(6.13 - 6.15) show a typical stress path for saturated interfaces with different surface roughnesses. The phase change line is drawn passing through the origin and approximately the phase change points, where the stress path reverses at $d\sigma = 0$. Following these procedures, the stress path plots result from two-way cyclic loading give the average slope values of phase change line as; 0.254, 0.380 and 0.425 for saturated interfaces with smooth, slightly textured, and moderately textured roughnesses, respectively. The corresponding $n$ values calculated from Eq.(6.12) are; 2.98, 3.24 and 3.52. Fig.(6.16) shows the variation of $n$ with $R_s$. The phase change parameter $n$ can be expressed in terms of surface roughness $R_s$ (with $R^2 = 0.976$) as:

$$n = 2.88 R_s^{0.491}$$  \hspace{1cm} (6.13)

6.3.3. Hardening Parameters $h1$ and $h2$

The hardening function $\alpha$ as stated in Eq. (5.67), is expressed in terms of deviatoric plastic strain trajectory $\xi$. Taking the natural logarithm of both sides of Eq. (5.67), one can obtain:

$$\ln \alpha = \ln h_1 - h_2 \ln \xi,$$ \hspace{1cm} (6.14)

where $\ln h_1$ is the intercept and $h_2$ is the slope of $\ln \alpha - \ln \xi$ relation.

The procedure to find $h_1$ and $h_2$ is to find $\alpha$ and $\xi$ for each stress point ($\sigma_n, \tau$) and then determine the best fit line to the set of points ($\ln \alpha, \ln \xi$). The $\xi$ values are
obtained from Eqs. (5.68 and 5.69) and $\alpha$ is found from the following expression obtained by solving Eq. (5.64) for $\alpha$:

$$\alpha = \frac{\gamma (\frac{p_{sa}}{p_{a}})^{2} - (\frac{x}{p_{a}})^{2}}{(p_{sa} \lambda n)}$$  \hspace{1cm} (6.15)

Figs. (6.17 - 6.19) show typical plots of $ln \alpha - ln \xi_D$ for one–way monotonic shear tests for the three different roughnesses. It is found that the value of $h_1$ is related to the applied normal stress as shown in Figs. (6.20 - 6.22) for the three roughnesses. Table (6.4) summarizes the values of $h_1$ and $h_2$ for saturated interfaces based on the results of one-way monotonic shear tests and one–way cyclic shear tests.

Average values of $h_1$ and $h_2$ are plotted against the surface roughness as shown in Figs. (6.23 and 6.24), respectively. The first plot gives the following relation:

$$h_1 = 0.025 R_s^{2.707}$$  \hspace{1cm} (6.16)

and the second plot gives:

$$h_2 = 0.288 R_s^{0.105}$$  \hspace{1cm} (6.17)

It is shown that $h_1$ is well related to $R_s$ with coefficient of regression ($R^2 = 0.894$), while $h_2$ is poorly related to $R_s$ with ($R^2 = 0.04$).

6.4. Parameters for the Fully Adjusted State

As mentioned in chapter 5, the critical state parameters are used in this study to simulate the behavior of interfaces for the fully adjusted state. The two basic equations in the critical state model are:

$$e^c = e_0^c - \lambda \ln \left(\frac{\sigma_{n}^c}{p_{a}}\right)$$  \hspace{1cm} (5.78)

and
\[ \tau^c = \bar{m} \sigma_n^c \]  
(5.79)

The parameters in these equations, \( \bar{m} \), \( \lambda \) and \( e_0^c \), are found by substituting in these equations the stress values and void ratios corresponding to the critical state found from the laboratory experiments.

**6.4.1. Parameter \( \bar{m} \)**

The parameter \( \bar{m} \) is found by plotting the values of shear stress \( \tau^c \) versus normal stress \( \sigma_n^c \) at critical state. The slope of the best fitting line passing through the origin gives the parameter \( \bar{m} \). Plots of \( \tau - \sigma_n^c \) are presented in Figs. (6.25 - 6.27) for steel roughnesses RI, RII, and RIII, respectively. The slopes of these curves which represent the values of \( \bar{m} \) were found as: 0.422, 0.564, and 0.597, respectively. Fig. (6.28) shows the variation of \( \bar{m} \) with respect to the surface roughness \( R_s \). The power regression between \( \bar{m} \) and \( R_s \) gives the following relation (with \( R^2 = 0.956 \)):

\[ \bar{m} = 0.4 R_s^{1.085} \]  
(6.18)

**6.4.2. Parameters \( \lambda \) and \( e_0^c \)**

The relationship between critical void ratio \( e^c \) and the corresponding \( \ln \left( \frac{\sigma_n^c}{p_a} \right) \) is found from the critical state of the material. Each test plots as a point on the \( e^c - \ln \left( \frac{\sigma_n^c}{p_a} \right) \) plane. The best fit line of such relation gives the value of \( \lambda \) as the slope and the value of \( e_0^c \) as the intercept at \( \sigma_n^c = p_a \). Since the void ratio remains constant during an undrained interface test, the void ratio at the critical state \( e^c \) is equal to the initial void ratio \( e_0 \).

Referring to Figs. (6.29 to 6.31), the values of \( \lambda \) were determined to be 0.071, 0.05 and 0.049 and the values of \( e_0^c \) were found to be 0.518, 0.536, and 0.527 for RI, RII, and RIII,
respectively. The variations of $e^c_0$ and $\lambda$ with surface roughness $R_s$ are shown in Figs.(6.32 and 6.33), respectively. It is found that $\lambda$ is well related to $R_s$ with correlation factor ($R^2 = 0.903$) while $e^c_0$ is poorly related to $R_s$ with ($R^2 = 0.402$). These two figures, respectively, yield the following expressions:

$$e^c_0 = 0.52 R_s^{0.065}$$ (6.19)

$$\lambda = 0.074 R_s^{-1.182}$$ (6.20)

6.5. Disturbance Parameters

As mentioned in chapter 5, the disturbance function $D$ is expressed in terms of plastic strain trajectory $\xi_D$ as:

$$D = D_u [1 - \exp (-A \xi^Z_D)]$$ (6.21)

where $D_u$ is the ultimate disturbance and $A$ and $Z$ are material parameters. Many researchers assume the parameter $D_u$ as unity (Desai and Ma 1992, Armaleh and Desai 1994, Katti and Desai 1995, Alanazy and Desai 1996, Desai and Rigby 1997). The parameters $A$ and $Z$ control the rate and shape of the disturbance function. Rearranging Eq.(6.21) and taking logarithms twice on both sides yield:

$$\ln [-\ln (1 - D)] = \ln A + Z \ln \xi_D$$ (6.22)

The slope of the best fit line of the above relation represents $Z$ while its interception represents $\ln A$.

To plot Eq.(6.22) for each test, it is required to find the values of deviatoric plastic strain trajectory $\xi_D$ and the corresponding disturbance $D$. The values of $\xi_D$ are obtained from Eqs.(5.68 and 5.69) as mentioned before. For two-way cyclic loading, the
disturbance $D$ can be defined as a function of the number of cycles ($N$) with respect to observed, intact, and critical shear stresses, as:

$$D(N) = \frac{\tau^l - \tau^p}{\tau^l - \tau^c}$$

(6.23)

where $\tau^l$ is the relative intact term which is the peak of the first cycle, $\tau^p$ is the peak of each cycle and $\tau^c$ is the critical shear stress or the peak shear stress at the last cycle in the test.

Figs.(6.34 through 6.36) show typical plots of $\ln \xi_D$ versus $\ln [-\ln (1 - D)]$ for the three roughnesses: RI , RII , and RIII. The average values of $A$ and $Z$ are summarized in Table (6.5). The variations of $A$ and $Z$ with surface roughness $R_s$ are shown in Figs.(6.37 and 6.38), respectively. The resulting $A$–$R_s$ relation gives the following expression (with $R^2 = 0.985$):

$$A = 0.002 R_s^{3.891}$$

(6.24)

and the $Z$–$R_s$ relation gives (with $R^2 = 0.695$):

$$Z = 1.2 R_s^{-0.175}$$

(6.25)

The disturbance function $D$ can be predicted by back substitution of the calculated parameters $A$ and $Z$ into Eq.(6.21). Figs.(6.39 - 6.41) show typical plots of variation of the predicted disturbance function $D$ in comparison to that from test data.

6.6. Summary of Parameters

In the previous sections, DSC model parameters were calculated for Ottawa sand-steel interfaces. Calculation of these parameters was based on the results of saturated normal compression tests and simple shear tests. The procedures used in calculating the parameters for smooth interface was repeated for slightly textured and moderately
textured interfaces. Table (6.6) presents the summary of DSC parameters for saturated interfaces for the three roughnesses used in this research.

6.7. Verification of the Model

6.7.1. General

In this section, verifications of the DSC model were obtained by back predicting simple shear tests using the parameters obtained in the previous sections. Verification of the model includes back predictions of one-way monotonic shear and two-way cyclic shear tests. Back prediction includes both parameter's dependent tests (tests used in parameters calculation) and parameter's independent tests (tests not used in parameters calculation). Plots resulting from back prediction were compared with those which resulted from experimental shear tests.

6.7.2. Back Prediction Program

An interface back prediction program was developed using MATLAB software. This program integrates numerically the DSC constitutive equations described in chapter 5 (Eqs. 5.77 and 5.90) using an incremental technique. Small increments of relative tangential displacement are input and the corresponding increments of shear stress are computed. As in all numerical techniques, the solution obtained in this manner is an approximate one; the accuracy increases with a decrease to the increment size. In this study, relative tangential displacement increments of 0.1 mm were found to yield acceptable results.
6.7.3. **Back Prediction Results**

One-way monotonic shear tests (SS1 - SS11) and two-way cyclic shear tests (SS23 - SS36) were back predicted using parameters determined in the previous sections. Figs.(6.42 - 6.52) show the observed and predicted shear stress versus relative shear displacement for one-way monotonic shear tests. Figs.(6.42 and 6.44) are independent tests and the rest are dependent tests. These figures illustrate the model's ability to represent the actual behavior of the Ottawa sand-steel interfaces with different roughnesses.

The results of back prediction for two-way cyclic loading are shown in Figs.(6.53 - 6.66) compared with the observed behavior of the same test. It is clear that the model back predicts the observed behavior reasonably well. The degradation during cycles is in good agreement with the actual laboratory test data. An independent test (SS29) with normal stress of 150 kPa is plotted using the predicted shear stiffness and hardening parameter values, while for the remaining of the parameters the average values were used. The actual and the predicted behaviors, as shown in Fig (6.59), are close and knowing that the back prediction graph is extrapolated from the results of other tests.

The verification indicates that DSC model proposed in this study can predict the laboratory test results quite reliably.

Figs.(6.67 - 6.80) present the observed and predicted behavior of the variation of peak shear stress versus time. From these figures it can be seen that the model back predicts the observed behavior well. The degradation of peak shear stress during cycles compares very well with the laboratory data.
6.8. Liquefaction Identification

During an earthquake or cycling loading, the application of cyclic shear stresses induced causes the soil to contract, resulting in an increase in porewater pressure. Because dynamic loading often occurs so quickly, the saturated sands even with their good drainage characteristics cannot dissipate the induced excess porewater pressure. Thus soils are cyclically loaded under undrained conditions. If the excess porewater pressure builds to the point at which it is equal to the overburden pressure, the effective stress becomes zero, the sand loses its strength completely, and it develops a liquefaction state (Seed and Lee, 1966).

The problem of liquefaction of saturated sands during dynamic loads has led to the widespread use of laboratory cyclic loading tests to assess the liquefaction characteristic of sands. In such tests, samples of saturated sand under undrained conditions are subjected to cyclic loading to simulate dynamic loading in the field (Seed and Lee 1966, Lee and Seed 1967a and 1967b, Martin et al., 1978, etc.).

The study of soil liquefaction has gained fresh impetus as a result of the great damages which occurred in Niigata, El Centro, Chile, Mexico, Kobe, Loma Prieta, and Northridge earthquakes. Much of these damages were caused by the liquefaction and consequent subsidence of sandy foundation soils or by liquefaction induced landslides (Finn et al. 1971). These damages to property and life have spurred significant research activities over the last 40 to 50 years. Some of the procedures and criteria for the identification of the liquefaction potential have been based on empirical considerations.
They often employ index properties such as blow count, and critical stress or strain criteria based on laboratory observations (Desai, 2000).

Desai et al., (1998) proposed a new method based on the Disturbed State Concept (DSC) to identify liquefaction. During cyclic loading, the soil experiences particle motions such as sliding and rotation; this can lead to instability in the microstructure of the material. The disturbance grows during loading and it can be used as the measure of micro-structural changes. Park and Desai (2000), found that when the disturbance reaches the critical value $D_c$, Fig.(6.81), its rate of change experienced a transition, that is, it increases but at a decreasing rate, toward the ultimate condition $D_u$. This condition ($D_c$) represents the state at which the microstructure develops instability leading to its collapse.

The critical disturbance $D_c$ can be found as the intersection of tangents to the early and latter parts of the disturbance curve. Figure (6.82) presents a typical determination of $D_c$. Knowing the value of $D_c$, the time $t_c$ and the number of cycles $N_c$ at critical disturbance state can be determined. Table (6.7) summarizes the values of $D_c$, $t_c$, and $N_c$ and also the time $t_{liq}$ and number of cycles $N_{liq}$ at liquefaction for the saturated two-way cyclic shear tests. The values of $t_c$ and $N_c$ based on critical disturbance are comparable to the corresponding values of $t_{liq}$ and $N_{liq}$ found from the laboratory test results. So, it can be concluded that the new procedure based on critical disturbance can allow identification of liquefaction in a simplified manner. As the difference between the values of $D_c$ for different tests is not large, the average value of $D_c = 0.91$ is adopted to define the initiation of liquefaction for saturated Ottawa sand–steel interface.
Table (6.1): DSC Model Parameters for Interfaces

<table>
<thead>
<tr>
<th>Group</th>
<th>Parameters</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>Elastic Parameters</td>
<td>$K_n$</td>
<td>Normal Stiffness</td>
</tr>
<tr>
<td></td>
<td>$K_s$</td>
<td>Shear Stiffness</td>
</tr>
<tr>
<td>Plasticity Parameters</td>
<td>$\gamma$</td>
<td>Ultimate Parameter</td>
</tr>
<tr>
<td></td>
<td>$n$</td>
<td>Phase Change Parameter</td>
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<td>$h_1$</td>
<td>Hardening Parameters</td>
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<tr>
<td></td>
<td>$h_2$</td>
<td></td>
</tr>
<tr>
<td>Critical State Parameters</td>
<td>$\bar{m}$</td>
<td>$\tau^c = \bar{m} \sigma_n^c$</td>
</tr>
<tr>
<td></td>
<td>$\lambda$</td>
<td>$e^c = e_0^c - \lambda \left(\frac{\sigma_n^c}{\rho_u}\right)$</td>
</tr>
<tr>
<td></td>
<td>$e_0^c$</td>
<td></td>
</tr>
<tr>
<td>Disturbance Parameters</td>
<td>$D_u$</td>
<td>$D = D_u \left[1 - \exp\left(-A \frac{Z}{D_u}\right)\right]$</td>
</tr>
<tr>
<td></td>
<td>$Z$</td>
<td></td>
</tr>
<tr>
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<td>$A$</td>
<td></td>
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Table (6.2): Normal Stiffness for Saturated Interfaces

<table>
<thead>
<tr>
<th>Test</th>
<th>Roughness (R_s)</th>
<th>Normal Stiffness (kPa/mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>NS1</td>
<td>RI</td>
<td>300</td>
</tr>
<tr>
<td>NS2</td>
<td>RII</td>
<td>440</td>
</tr>
<tr>
<td>NS3</td>
<td>RIII</td>
<td>476</td>
</tr>
</tbody>
</table>

Table (6.3): Shear Stiffness for Saturated Interfaces

<table>
<thead>
<tr>
<th>Test</th>
<th>Normal Stress $\sigma_n$ (kPa)</th>
<th>Roughness</th>
<th>Shear Stiffness $K_s$ (kPa /mm)</th>
<th>$\sigma_n - K_s$ Relation</th>
<th>Average Shear Stiffness (kPa/mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>SS13</td>
<td>100</td>
<td>RI</td>
<td>75</td>
<td>$K_s = 8.75 \sigma_n^{0.467}$ ($R^2 = 0.999$)</td>
<td>101.7</td>
</tr>
<tr>
<td>SS15</td>
<td>200</td>
<td>RI</td>
<td>105</td>
<td></td>
<td></td>
</tr>
<tr>
<td>SS16</td>
<td>300</td>
<td>RI</td>
<td>125</td>
<td></td>
<td></td>
</tr>
<tr>
<td>SS17</td>
<td>100</td>
<td>RII</td>
<td>110</td>
<td>$K_s = 9.27 \sigma_n^{0.534}$ ($R^2 = 0.984$)</td>
<td>153.3</td>
</tr>
<tr>
<td>SS18</td>
<td>200</td>
<td>RII</td>
<td>150</td>
<td></td>
<td></td>
</tr>
<tr>
<td>SS19</td>
<td>300</td>
<td>RII</td>
<td>200</td>
<td></td>
<td></td>
</tr>
<tr>
<td>SS20</td>
<td>100</td>
<td>RII</td>
<td>135</td>
<td></td>
<td></td>
</tr>
<tr>
<td>SS21</td>
<td>200</td>
<td>RII</td>
<td>170</td>
<td>$K_s = 23.81 \sigma_n^{0.375}$ ($R^2 = 0.992$)</td>
<td>170.0</td>
</tr>
<tr>
<td>SS22</td>
<td>300</td>
<td>RII</td>
<td>205</td>
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</table>
Table (6.4): Hardening Parameters $h_1$ and $h_2$ for Saturated Interfaces

<table>
<thead>
<tr>
<th>Test</th>
<th>Normal Stress $\sigma_n$ (kPa)</th>
<th>$h_1$</th>
<th>$h_1 - \sigma_n$ Relation</th>
<th>Average $h_1$</th>
<th>$h_2$</th>
<th>Average $h_2$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. Smooth Interface</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>SS2</td>
<td>100</td>
<td>0.038</td>
<td>$h_1 = 0.108e^{-0.008\sigma_n}$</td>
<td>0.027</td>
<td>0.177</td>
<td></td>
</tr>
<tr>
<td>SS4</td>
<td>200</td>
<td>0.026</td>
<td></td>
<td></td>
<td>0.325</td>
<td></td>
</tr>
<tr>
<td>SS5</td>
<td>300</td>
<td>0.009</td>
<td></td>
<td></td>
<td>0.258</td>
<td>0.279</td>
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<tr>
<td>SS13</td>
<td>100</td>
<td>0.056</td>
<td></td>
<td></td>
<td>0.288</td>
<td></td>
</tr>
<tr>
<td>SS15</td>
<td>200</td>
<td>0.023</td>
<td></td>
<td></td>
<td>0.240</td>
<td></td>
</tr>
<tr>
<td>SS16</td>
<td>300</td>
<td>0.01</td>
<td></td>
<td></td>
<td>0.384</td>
<td></td>
</tr>
<tr>
<td>2. Slightly Textured Interface</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>SS6</td>
<td>100</td>
<td>0.095</td>
<td>$h_1 = 0.194e^{-0.007\sigma_n}$</td>
<td>0.061</td>
<td>0.350</td>
<td></td>
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<tr>
<td>SS7</td>
<td>200</td>
<td>0.033</td>
<td></td>
<td></td>
<td>0.205</td>
<td></td>
</tr>
<tr>
<td>SS8</td>
<td>300</td>
<td>0.019</td>
<td></td>
<td></td>
<td>0.626</td>
<td></td>
</tr>
<tr>
<td>SS17</td>
<td>100</td>
<td>0.127</td>
<td></td>
<td></td>
<td>0.285</td>
<td>0.327</td>
</tr>
<tr>
<td>SS18</td>
<td>200</td>
<td>0.049</td>
<td></td>
<td></td>
<td>0.274</td>
<td></td>
</tr>
<tr>
<td>SS19</td>
<td>300</td>
<td>0.041</td>
<td></td>
<td></td>
<td>0.219</td>
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<td>3. Moderately Textured Interface</td>
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<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>SS9</td>
<td>100</td>
<td>0.104</td>
<td>$h_1 = 0.27e^{-0.008\sigma_n}$</td>
<td>0.063</td>
<td>0.183</td>
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<td>SS10</td>
<td>200</td>
<td>0.049</td>
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<td></td>
<td>0.256</td>
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<td>SS11</td>
<td>300</td>
<td>0.016</td>
<td></td>
<td></td>
<td>0.479</td>
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<tr>
<td>SS20</td>
<td>100</td>
<td>0.118</td>
<td></td>
<td></td>
<td>0.246</td>
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</tr>
<tr>
<td>SS21</td>
<td>200</td>
<td>0.064</td>
<td></td>
<td></td>
<td>0.231</td>
<td></td>
</tr>
<tr>
<td>SS22</td>
<td>300</td>
<td>0.027</td>
<td></td>
<td></td>
<td>0.293</td>
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Table (6.5): Average Values of Disturbance Parameters for Saturated Interfaces

<table>
<thead>
<tr>
<th>Roughness</th>
<th>Disturbance Parameters</th>
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</thead>
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<tr>
<td></td>
<td>A</td>
</tr>
<tr>
<td>RI</td>
<td>0.0026</td>
</tr>
<tr>
<td>RII</td>
<td>0.0067</td>
</tr>
<tr>
<td>RIII</td>
<td>0.0092</td>
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Table (6.6): Summary of DSC Parameters for Saturated Interfaces

<table>
<thead>
<tr>
<th>Group</th>
<th>Parameter</th>
<th>Surface Roughness</th>
</tr>
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<tbody>
<tr>
<td></td>
<td></td>
<td>RI</td>
</tr>
<tr>
<td>Elastic Parameters</td>
<td>$K_n$ (kPa/mm)</td>
<td>300</td>
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<tr>
<td></td>
<td>$K_s$ (kPa/mm)</td>
<td>101.7</td>
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<tr>
<td>Plasticity Parameters</td>
<td>$\gamma$</td>
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<tr>
<td></td>
<td>$n$</td>
<td>2.98</td>
</tr>
<tr>
<td></td>
<td>$h_1$</td>
<td>0.027</td>
</tr>
<tr>
<td></td>
<td>$h_2$</td>
<td>0.279</td>
</tr>
<tr>
<td>Critical State Parameters</td>
<td>$\bar{m}$</td>
<td>0.422</td>
</tr>
<tr>
<td></td>
<td>$\lambda$</td>
<td>0.071</td>
</tr>
<tr>
<td></td>
<td>$e_0^c$</td>
<td>0.518</td>
</tr>
<tr>
<td>Disturbance Parameters</td>
<td>$D_u$</td>
<td>1.0</td>
</tr>
<tr>
<td></td>
<td>$Z$</td>
<td>1.178</td>
</tr>
<tr>
<td></td>
<td>$A$</td>
<td>0.0026</td>
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</table>
Table (6.7): Liquefaction and Critical Disturbance Parameters for Saturated Two-Way Cyclic Shear Tests

<table>
<thead>
<tr>
<th>Test</th>
<th>Based on Lab. Test Results</th>
<th>Based on Critical Disturbance Concept</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>( t_{liq} ) (sec)</td>
<td>( D_c )</td>
</tr>
<tr>
<td>SS25</td>
<td>2600</td>
<td>0.92</td>
</tr>
<tr>
<td>SS23</td>
<td>1000</td>
<td>0.93</td>
</tr>
<tr>
<td>SS26</td>
<td>360</td>
<td>0.92</td>
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<tr>
<td>SS27</td>
<td>120</td>
<td>0.91</td>
</tr>
<tr>
<td>SS28</td>
<td>400</td>
<td>0.90</td>
</tr>
<tr>
<td>SS24</td>
<td>440</td>
<td>0.91</td>
</tr>
<tr>
<td>SS29</td>
<td>560</td>
<td>0.90</td>
</tr>
<tr>
<td>SS30</td>
<td>680</td>
<td>0.93</td>
</tr>
<tr>
<td>SS31</td>
<td>200</td>
<td>0.94</td>
</tr>
<tr>
<td>SS32</td>
<td>320</td>
<td>0.88</td>
</tr>
<tr>
<td>SS33</td>
<td>440</td>
<td>0.91</td>
</tr>
<tr>
<td>SS34</td>
<td>200</td>
<td>0.93</td>
</tr>
<tr>
<td>SS35</td>
<td>320</td>
<td>0.88</td>
</tr>
<tr>
<td>SS36</td>
<td>400</td>
<td>0.91</td>
</tr>
</tbody>
</table>

\((D_c)_{\text{average.}} = 0.91\)
Fig. (6.1): Determination of Normal Stiffness for Saturated Interface (Roughness RI)

Fig. (6.2): Determination of Shear Stiffness for Saturated Interface (Roughness RI)
Fig. (6.3) Variation of Shear Stiffness with Normal Stress for Saturated Interface (Roughness RI)

\[ K_s = 8.75 \sigma_n^{0.467} \]

Fig. (6.4) Variation of Shear Stiffness with Normal Stress for Saturated Interface (Roughness RII)

\[ K_s = 9.27 \sigma_n^{0.534} \]
Fig. (6.5) Variation of Shear Stiffness with Normal Stress for Saturated Interface (Roughness RIII)

\[ K_s = 23.81 \sigma_n^{0.375} \]

Fig. (6.6) Variation of Normal Stiffness with Surface Roughness for Saturated Interfaces

\[ K_n = 280.9 R_s^{1.442} \]
Fig.(6.7) Variation of Shear Stiffness with Surface Roughness for Saturated Interfaces

$$K_s = 94.2 R_s^{1.596}$$

Fig.(6.8): Variation of Ultimate Shear Stress with Normal Stress (Roughness RI)

Roughness: RI

Roughness: RI

slope = 0.442
Fig.(6.9): Variation of Ultimate Shear Stress with Normal Stress (Roughness RII)

slope = 0.614

Fig.(6.10): Variation of Ultimate Shear Stress with Normal Stress (Roughness RIII)

slope = 0.646
Fig.(6.11): Variation of $\gamma$ with Surface Roughness $R_s$  

$$\gamma = 0.173 R_s^{2.318}$$

Fig.(6.12): Yield Surfaces and Phase Change Line in $\sigma_n-\tau$ Plane
Fig. (6.13): Stress Path for Saturated Simple Shear Test ($\sigma_n=100$ kPa, Roughness: RI)

Fig. (6.14): Stress Path for Saturated Simple Shear Test ($\sigma_n=100$ kPa, Roughness: RII)
Fig.(6.15): Stress Path for Saturated Simple Shear Test ($\sigma_n$ =100 kPa, Roughness: RIII)

\[ n = 2.88 R_s^{0.491} \]

Fig.(6.16): Variation of Phase Change Parameter with Surface Roughness
Fig. (6.17): Determination of the Hardening Parameters $h_1$ and $h_2$

($\sigma_n=200$ kPa, Roughness: RI)

Fig. (6.18): Determination of the Hardening Parameters $h_1$ and $h_2$

($\sigma_n=200$ kPa, Roughness: RII)
Fig. (6.19): Determination of the Hardening Parameters $h_1$ and $h_2$

($\sigma_n=200$ kPa, Roughness: RIII)

\[ \ln \alpha = -0.256 \xi_D - 3.013 \]

Fig. (6.20): Variation of Hardening Parameter $h_1$ with Applied Normal Stress (Roughness RI)

\[ h_1 = 0.108 e^{-0.008\sigma_n} \]
Fig. (6.21): Variation of Hardening Parameter $h_1$ with Applied Normal Stress (Roughness RII)

$$h_1 = 0.194 e^{-0.007\sigma_n}$$

Fig. (6.22): Variation of Hardening Parameter $h_1$ with Applied Normal Stress (Roughness RIII)

$$h_1 = 0.27 e^{-0.008\sigma_n}$$
Fig. (6.23): Variation of Hardening Parameter $h_1$ with Surface Roughness

$$h_1 = 0.025 R_s^{2.707}$$

Fig. (6.24): Variation of Hardening Parameter $h_2$ with Surface Roughness

$$h_2 = 0.288 R_s^{0.105}$$
Fig. (6.25): Determination of Critical State Parameter $\bar{m}$ (Roughness RI)

\[ \tau^c = 0.422 \sigma_n^c \]

Fig. (6.26): Determination of Critical State Parameter $\bar{m}$ (Roughness RII)

\[ \tau^c = 0.564 \sigma_n^c \]
Fig.(6.27): Determination of Critical State Parameter $\bar{m}$ (Roughness RIII)

\[ \tau^c = 0.597 \sigma_n^c \]

\[ \bar{m} = 0.4 R_s^{1.085} \]

Fig.(6.28): Variation of $\bar{m}$ with Surface Roughness
Fig.(6.29): Determination of Critical State Parameters $e_o^c$ and $\lambda$ (Roughness RI)

$$e^c = -0.071 \ln(\sigma'_n / p_o) + 0.518$$

Fig.(6.30): Determination of Critical State Parameters $e_o^c$ and $\lambda$ (Roughness RII)

$$e^c = -0.05 \ln(\sigma'_n / p_o) + 0.536$$
Fig. (6.31): Determination of Critical State Parameters $e^c_o$ and $\lambda$ (Roughness RIII)

\[ e^c_o = -0.049 \ln\left(\frac{\sigma_n}{p_o}\right) + 0.527 \]

Fig. (6.32): Variation of $e^c_o$ with Surface Roughness $R_s$

\[ e^c_o = 0.52 R_s^{0.065} \]
Fig. (6.33): Variation of $\lambda$ with Surface Roughness $R_s$

$$\lambda = 0.074 R_s^{-1.182}$$

Fig. (6.34): Determination of Disturbance Parameters $A$ and $Z$

$$\ln(-\ln(1-D)) = 1.322 \ln \xi_D - 7.418$$

$(\sigma_n=200 \text{ kPa, Roughness: RI})$
Fig. (6.35): Determination of Disturbance Parameters $A$ and $Z$

($\sigma_n = 100$ kPa, Roughness: RII)

\[\ln(-\ln(1-D)) = 1.015 \ln \xi_D - 4.646\]

Fig. (6.36): Determination of Disturbance Parameters $A$ and $Z$

($\sigma_n = 200$ kPa, Roughness: RIII)

\[\ln(-\ln(1-D)) = 1.064 \ln \xi_D - 4.812\]
Fig.(6.37): Variation of Disturbance Parameter $A$ with Surface Roughness $R_s$

\[ A = 0.002 R_s^{3.891} \]

Fig.(6.38): Variation of Disturbance Parameter $Z$ with Surface Roughness $R_s$

\[ Z = 1.2 R_s^{-0.175} \]
Fig. (6.39): Plot of Disturbance $D$ vs. $\xi_D$ ($\sigma_n = 200$ kPa, Roughness: RI)

Fig. (6.40): Plot of Disturbance $D$ vs. $\xi_D$ ($\sigma_n = 100$ kPa, Roughness: RII)
Fig.(6.41): Plot of Disturbance $D$ vs. $\tilde{\xi}_D$ ($\sigma_n = 200$ kPa, Roughness: RIII)

Fig.(6.42): Observed and Back Predicted Shear Stress vs. Relative Shear Displacement for One-way Monotonic Loading ($\sigma_n = 50$, RI)
Fig. (6.43): Observed and Back Predicted Shear Stress vs. Relative Shear Displacement for One-way Monotonic Loading \((\sigma_n = 100, \text{RI})\)

Fig. (6.44): Observed and Back Predicted Shear Stress vs. Relative Shear Displacement for One-way Monotonic Loading \((\sigma_n = 150, \text{RI})\)
Fig.(6.45): Observed and Back Predicted Shear Stress vs. Relative Shear Displacement for One-way Monotonic Loading ($\sigma_n =200$, RI)

Fig.(6.46): Observed and Back Predicted Shear Stress vs. Relative Shear Displacement for One-way Monotonic Loading ($\sigma_n =300$, RI)
Fig.(6.47): Observed and Back Predicted Shear Stress vs. Relative Shear Displacement for One-way Monotonic Loading ($\sigma_n = 100$, RII)

Fig.(6.48): Observed and Back Predicted Shear Stress vs. Relative Shear Displacement for One-way Monotonic Loading ($\sigma_n = 200$, RII)
Fig.(6.49): Observed and Back Predicted Shear Stress vs. Relative Shear Displacement for One-way Monotonic Loading ($\sigma_n = 300$, RII)

Fig.(6.50): Observed and Back Predicted Shear Stress vs. Relative Shear Displacement for One-way Monotonic Loading ($\sigma_n = 100$, RIII)
Fig.(6.51): Observed and Back Predicted Shear Stress vs. Relative Shear Displacement for One-way Monotonic Loading ($\sigma_n = 200$, RIII)

Fig.(6.52): Observed and Back Predicted Shear Stress vs. Relative Shear Displacement for One-way Monotonic Loading ($\sigma_n = 300$, RIII)
Fig.(6.53): Observed and Back Predicted Shear Stress vs. Relative Shear Displacement for Two-way Cyclic Loading ($\sigma_n = 100$ kPa, Ampl.=2.5 mm, Freq.=0.1 Hz, RI)
Fig.(6.54): Observed and Back Predicted Shear Stress vs. Relative Shear Displacement for Two-way Cyclic Loading ($\sigma_n = 100$ kPa, Ampl.=5.0 mm, Freq.=0.1 Hz, RI)
Fig.(6.55): Observed and Back Predicted Shear Stress vs. Relative Shear Displacement for Two-way Cyclic Loading ($\sigma_n$ =100 kPa, Ampl.=2.5 mm, Freq.=0.04 Hz, RI)
Fig.(6.56): Observed and Back Predicted Shear Stress vs. Relative Shear Displacement for Two-way Cyclic Loading ($\sigma_n = 100$ kPa, Ampl.=2.5 mm, Freq.=0.25 Hz, RI)
Fig.(6.57): Observed and Back Predicted Shear Stress vs. Relative Shear Displacement for Two-way Cyclic Loading ($\sigma_n=100$ kPa, Ampl.=2.5 mm, Freq.=0.625 Hz, RI)
Fig.(6.58): Observed and Back Predicted Shear Stress vs. Relative Shear Displacement for Two-way Cyclic Loading ($\sigma_n=50$ kPa, Ampl.=5.0 mm, Freq.=0.1 Hz, RI)
Fig.(6.59): Observed and Back Predicted Shear Stress vs. Relative Shear Displacement for Two-way Cyclic Loading ($\sigma_n = 150$ kPa, Ampl. = 5.0 mm, Freq. = 0.1 Hz, RI)
Fig.(6.60): Observed and Back Predicted Shear Stress vs. Relative Shear Displacement for Two-way Cyclic Loading ($\sigma_n=200$ kPa, Ampl.=5.0 mm, Freq.=0.1 Hz, RI)
Fig.(6.61): Observed and Back Predicted Shear Stress vs. Relative Shear Displacement for Two-way Cyclic Loading ($\sigma_n = 50$ kPa, Ampl. = 5.0 mm, Freq. = 0.1 Hz, RII)
Fig.(6.62): Observed and Back Predicted Shear Stress vs. Relative Shear Displacement for Two-way Cyclic Loading ($\sigma_n = 100$ kPa, Ampl.=5.0 mm, Freq.=0.1 Hz, RII)
Fig.(6.63): Observed and Back Predicted Shear Stress vs. Relative Shear Displacement for Two-way Cyclic Loading ($\sigma_n=200$ kPa, Ampl.=5.0 mm, Freq.=0.1 Hz, RII)
Fig.(6.64): Observed and Back Predicted Shear Stress vs. Relative Shear Displacement for Two-way Cyclic Loading ($\sigma_n = 50 \text{ kPa}$, Ampl.=5.0 mm, Freq.=0.1 Hz, RIII)
Fig.(6.65): Observed and Back Predicted Shear Stress vs. Relative Shear Displacement for Two-way Cyclic Loading ($\sigma_n = 100$ kPa, Ampl.=5.0 mm, Freq.=0.1 Hz, RIII)
Fig.(6.66): Observed and Back Predicted Shear Stress vs. Relative Shear Displacement for Two-way Cyclic Loading ($\sigma_n=200$ kPa, Ampl.=5.0 mm, Freq.=0.1 Hz, RIII)
Fig.(6.67): Observed and Back Predicted Peak Shear Stress vs. Time for Two-way Cyclic Loading ($\sigma_n=100$ kPa, Ampl.=2.5 mm, Freq.=0.1 Hz, RI)
Fig.(6.68): Observed and Back Predicted Peak Shear Stress vs. Time for Two-way Cyclic Loading ($\sigma_n=100$ kPa, Ampl.=5.0 mm, Freq.=0.1 Hz, RI)
Fig.(6.69): Observed and Back Predicted Peak Shear Stress vs. Time for Two-way Cyclic Loading ($\sigma_n = 100$ kPa, Ampl.=2.5 mm, Freq.=0.04 Hz, RI)
Fig. (6.70): Observed and Back Predicted Peak Shear Stress vs. Time for Two-way Cyclic Loading ($\sigma_n = 100$ kPa, Ampl.=2.5 mm, Freq.=0.25 Hz, RI)
Fig.(6.71): Observed and Back Predicted Peak Shear Stress vs. Time for Two-way Cyclic Loading ($\sigma_n = 100$ kPa, Ampl.=2.5 mm, Freq.=0.625 Hz, RI)
Fig.(6.72): Observed and Back Predicted Peak Shear Stress vs. Time for Two-way Cyclic Loading ($\sigma_n = 50$ kPa, Ampl.= 5.0 mm, Freq.= 0.1 Hz, RI)
Fig.(6.73): Observed and Back Predicted Peak Shear Stress vs. Time for Two-way Cyclic Loading ($\sigma_n = 150$ kPa, Ampl.=5.0 mm, Freq.=0.1 Hz, RI)
Fig.(6.74): Observed and Back Predicted Peak Shear Stress vs. Time for Two-way Cyclic Loading ($\sigma_n = 200$ kPa, Ampl.=5.0 mm, Freq.=0.1 Hz, RI)
Fig.(6.75): Observed and Back Predicted Peak Shear Stress vs. Time for Two-way Cyclic Loading ($\sigma_n = 50$ kPa, Ampl.=5.0 mm, Freq.=0.1 Hz, RII)
Fig.(6.76): Observed and Back Predicted Peak Shear Stress vs. Time for Two-way Cyclic Loading ($\sigma_n=100$ kPa, Ampl.=5.0 mm, Freq.=0.1 Hz, RII)
Fig. (6.77): Observed and Back Predicted Peak Shear Stress vs. Time for Two-way Cyclic Loading ($\sigma_n = 200$ kPa, Ampl.=5.0 mm, Freq.=0.1 Hz, RII)
Fig.(6.78): Observed and Back Predicted Peak Shear Stress vs. Time for Two-way Cyclic Loading ($\sigma_n = 50$ kPa, Ampl.=5.0 mm, Freq.=0.1 Hz, RIII)
Fig.(6.79): Observed and Back Predicted Peak Shear Stress vs. Time for Two-way Cyclic Loading ($\sigma_n = 100$ kPa, Ampl.=5.0 mm, Freq.=0.1 Hz, RIII)
Fig. (6.80): Observed and Back Predicted Peak Shear Stress vs. Time for Two-way Cyclic Loading ($\sigma_n = 200$ kPa, Ampl. = 5.0 mm, Freq. = 0.1 Hz, RIII)
Fig.(6.81): Determination of Critical Disturbance (Park and Desai, 2000)

Fig.(6.82): Determination of $D_c$ for Test SS24
CHAPTER 7: APPLICATION OF DSC MODEL FOR OFFSHORE PILES

7.1. Introduction

Dynamic soil-structure interaction analysis involving pile foundations is one of the more complex problems in geotechnical engineering. The analysis involves modeling soil-pile interaction, nonlinear soil response, and in case of saturated soil such as in offshore piles, it incorporates dynamically induced porewater pressure. In the last few decades, several numerical and analytical methods have been developed to predict the dynamic response of pile foundations accounting for soil-pile interaction. Some of these methods assumed linear response for the surrounding soil. Nevertheless, under moderate and strong dynamic loading, pile foundations undergo large displacements and the behavior of the system can be strongly nonlinear. Other models are based on semi-empirical approaches such as p-y and t-z curves (Matlock 1970, Reese et al. 1974 and 1975). Empirical methods are only as good as the available data base on the particular problem. They may not be applicable for new types of soils or loading situations. Moreover, those methods cannot provide information about porewater pressure development during dynamic loading which is the most important factor that controls the behavior of saturated soil-pile system under dynamic loading.

Realistic modeling of the nonlinear dynamic response of the soil-pile system is very important especially for the dynamic analysis of offshore structures. For such complex analysis, numerical methods like finite element analysis are usually applicable.
to soil-structure interaction problems including realistic material behavior, complex loading and coupled soil-fluid interaction. Appropriate constitutive models for interfaces and soils are important in the finite element method to enable realistic simulation of the behavior of the system. In this study, the DSC constitutive model is adopted for both soil and soil-pile interface.

The constitutive model, problem geometry, and an appropriate loading condition can be combined in the finite element analysis to get a realistic prediction for the behavior of soil-structure system. However, confirmation of such calculations can be achieved by full-scale pile test, instrumented pile test and/or centrifuge test. In this chapter, the finite element method with the DSC model is verified by back predicting the observed behavior of an instrumented axially loaded pile and a full-scale laterally loaded pile.

The analyses of the two pile load tests are conducted using non-linear finite element approach with DSC constitutive model to assess the accuracy of both finite element simulation and material modeling. An instrumented axially loaded pile test at Sabine, Texas (ETC, 1986) and a full-scale laterally loaded pile test at Seal Beach, California (Ertec, 1981) are analyzed using the finite element program DSC-DYN2D (Desai, 2000b). Field tests results are compared with those from finite element analyses for both without and with interface cases. Material parameters for soils and interfaces are obtained from laboratory tests.

Description of the pile load tests, finite element simulation, and comparisons of measured with predicted pile responses are presented in this chapter.
7.2. Simulation of Axially Loaded Pile

7.2.1. General

The Earth Technology Corporation (ETC, 1986) under a joint research project with the University of Arizona (Wathugala and Desai, 1990) performed field experiments on instrumented pile segment models to study the behavior of axially loaded piles in saturated clay soils. The tests were performed in July, 1986 at a test site located 4 miles south of Sabine Pass, Texas. Figure (7.1) shows the location of the site. The soil profile, as shown in Fig.(7.2), consists of a normally consolidated grey marine clay having a liquid limit of 100% and plasticity index of 72. This plots well above the A-line in the region of high plasticity. Miniature field vane shear tests conducted at the site gave average undrained peak strength of 0.75 ksf (35.9 kPa) and residual shear strength of 0.34 ksf (16.3 kPa). An additional advanced laboratory testing program was performed at the University of Arizona (Katti, 1991) on undisturbed clay samples. The tests included conventional cylindrical triaxial tests and also cyclic triaxial tests in the multiaxial device with pore pressure measurements. Some of these tests were used to determine of parameters and verification of the DSC model.

The interface behavior between Sabine clay-steel has been investigated by Rigby (1996) using the CYMDOF-P device. His laboratory test results were used by Shao (1998) for the determination of interface parameters and for verification of the DSC model.

7.2.2. Pile Installation and Test Procedures

Two pile segment models were used, a 1.72-inch (4.37 cm) diameter model, also known as the X-Probe, and a 3-inch (7.62 cm) diameter model. The field experiments
were performed at a depth of 50 ft (15.24 m) for both models. In this study, only the 3-inch (7.62 cm) pile-soil behavior is considered.

The testing system used in the field experiment study consisted of loading system, computer controlled data acquisition system, cutting shoes, and instrumented pile segments. The loading system had a hydraulic ram, with a 12 inch (30.48 cm) stroke, that was able to apply tension or compression loading to the pile model. Screw anchors were used as a reaction mechanism for compression loadings. The displacement of the hydraulic ram was transferred to the pile segment by the N-rods connecting them. A schematic of the loading system is shown in Fig.(7.3). The 3-inch diameter pile segment model was driven into the soil below the bottom of the pre-bored borehole. The general testing sequences for the pile segment models were as follows:

1. Drilling and casing the borehole up to the planned test depth.
2. Setting up the data acquisition system.
3. Installing of the pile segment model and performing continuous measurements of the shear transfer, total radial pressure, and relative pile-soil displacement.
4. Conducting static tension and compression tests at the specified degree of consolidation.
5. Conducting two-way cyclic load tests at the end of consolidation.

There was a slip joint located between the body of the pile segment and the cutting shoe. The body of a DC-LVDT (Direct Current-Linear Voltage Displacement Transducer) is mounted in the body of the pile segment and the core of the LVDT is attached to the cutting shoe, Fig.(7.4). When the pile segment is driven, both the cutting shoe and the
pile segment move together, but during subsequent load tests only the pile segment moves due to presence of the slip joint. The DC-LVDT is used to measure the relative movement between the pile segment and the cutting shoe. In this case, the cutting shoe acts as the reference anchor which does not move relative to the surrounding soil. Thus, accurate displacement measurements of the pile segment can be obtained.

The shear transfer from the pile segment to the soil is obtained by measuring the difference between axial loads at two points in the pile segment. This is achieved by two load cells separated by a distance of 31.6 in. (80.26 cm) as shown in Fig. (7.4).

Lateral earth pressure and pore pressure are measured using two pressure transducers located midway between two load cells. The total pressure transducer is sensitive only to forces normal to the outer face of the unit, thus measuring the total radial pressure acting against the surface of the pile segment.

In this study, only the two-way cyclic load test is considered to verify the DSC model in the finite element simulation.

7.2.3. Finite Element Simulation

The displacement controlled two-way cyclic axial test of the 3-inch (7.62 cm) pile probe is numerically simulated using the finite element program DSC-DYN2D. This problem has been solved by Shao and Desai (2000) when the behavior of Sabine clay-steel interface is considered. Sabine clay and the steel-clay interface are modeled using the Disturbed State Concept model. Material parameters for clay were adopted from Katti and Desai (1995), and those for the interface were adopted from Shao (1998). Those material parameters are summarized in Table (7.1). The pile is assumed rigid, so the pile
movements are simulated as prescribed displacements of the soil at the nodes in contact with the pile segment. To demonstrate the effect of interface on the behavior of pile-soil system, the problem is solved in this study for the case of no interface.

The configuration details of pile installation used in the finite element simulation is shown in Fig.(7.5). Geometric conditions around the pile segment and the applied cyclic displacement can allow the use of axisymmetric finite element analysis. The soil domain considered in this analysis has dimensions of 3.2m radius and 24m height as shown in Fig.(7.6). It consists of 192 elements and 225 nodes.

Five field cycles in compression and tension to failure were performed on the 3-inch probe, Fig.(7.7). The vertical displacement as measured in the field is applied to the nodes in contact with pile segment in 801 time steps, as shown in Fig.(7.8). For the boundary conditions, as indicated in Fig.(7.6), left and right vertical sides of the mesh boundaries are restrained in the x-direction and free in the y-direction. The bottom boundary is restrained in the y-direction, but free in the x-direction, and the top boundary is free in both directions, x and y.

The finite element mesh, loading, boundary conditions, and constitutive parameters as explained above are utilized for the dynamic finite element analysis of the pile segment. The damping ratio was not reported in the field test report. Vucetic and Dobry (1991), reported damping ratio range between 3.5% and 6% for fine grained soils with plasticity index PI = 100. In this study, a value of 5% is used for damping ratio.
7.2.4. Comparison of Results and Discussion

The results of finite element solution of two-way cyclic test for 3-inch instrumented pile are compared with field measurements in this section. The finite element results for the case of a pile with interface are adopted from Shao and Desai (2000) and those for the case of a pile without interface are obtained in this study.

Before using the finite element mesh suggested by Shao and Desai (2000) it is verified for the effects of boundary conditions. In their study, the mesh radius around the pile was \( w = 42D \) and the depth of mesh beneath the pile tip was \( d = 92D \), where \( D \) is the pile diameter. The results from the above mesh are compared with the results of a reduced mesh: \( w = 21D \) and \( d = 39D \). The same results were found from the original and the reduced meshes and therefore the same size mesh used by Shao and Desai (2000) is used in this study.

Results from the finite element solution for both with and without interface cases at element 121 are compared with field measurements and presented in Figs.(7.9-7.13). The predicted values of shear transfer versus pile displacement are compared in Fig.(7.9). The shear transfer for the case of with interface shows good agreement with the field measurements. In contrast, the shear transfer for the case of no interface over predicts the field results. This is because the pile and soil are considered to be glued together. Since no interface was used in this analysis, no relative displacements are allowed and hence larger deformations are transformed to the soil elements resulted in high shear resistance. However, the numerical solution accurately captured the response patterns during the tension-compression cycles as shown in Fig.(7.10). The shear transfer from both field test
and numerical solution show a slight degradation with time. This can be attributed to the slight increase in porewater pressure with time as shown in Fig.(7.11). The predicted pore pressure values are higher than measured values, but the trends are same with slight increase with time in both field test results and finite element results. The variation of total horizontal stress with time is shown in Fig.(7.12). Total horizontal stress did not change significantly during the cyclic test and the finite element results for the case with interface agreed very well with the field measurements. Fig.(7.13) shows the variation of the effective horizontal stress with time. The finite element analysis overpredicts the field measurements for the case of no interface and underpredicts the field measurements for the case of with interface.

7.2.5. Liquefaction Potential and Disturbance

Based on conventional method proposed by Seed and Lee (1966) and Lee and Seed (1967a and 1967b), initial liquefaction can be defined as the excess porewater pressure becoming equal to the initial effective normal stress. Using this criterion, the liquefaction potential (LP) can be defined as the ratio of excess porewater pressure to the initial effective normal stress. Based on this definition, the soil is said to be liquefied when the value of LP approaches 1.

Distribution of excess porewater pressure (PWP), liquefaction potential (LP) and disturbance (D) are drawn as contour images at two points: step 627 and step 738, Fig.(7.7). Figs.(7.14-7.16) present the results at step 627 and Figs.(7.17-7.19) present the results at step 738. For both peaks, the excess pore pressure buildup in the region close to the pile segment is much larger than that in other regions of soil domain. The values of
LP near the pile segment ranged between 0.25 and 0.5 while those in the far regions are below 0.25. The maximum value of LP is less than unity which indicates that the soil didn’t liquefy. The disturbance in the region close to the pile segment approaches 0.4 and this value decreases gradually by moving away from the face of the pile segment.

Both LP and D show similar distribution through the soil domain. The regions of high LP values are the same as those defined by higher disturbance D. So, as mentioned by Desai (2000), the consideration of microstructural behavior can be adopted as an alternative approach to enhance understanding and general procedures for liquefaction analysis.

7.3. Simulation of Laterally Loaded Pile

7.3.1. General

Two steel pipe piles were driven into medium-dense sand during summer of 1981. The soil was saturated fine sand at mean tide level on a beach in Seal Beach, California. One of the piles was then instrumented and used in a series of lateral vibration tests. These vibration tests, sponsored by National Science Foundation (NSF), were performed jointly by the Ertec Western, Inc. (Ertec, 1981) and the California Institute of Technology (Scott et al., 1982). The results from this test program are used in this study to verify the behavior of a laterally loaded offshore pile in the finite element analysis, with DSC constitutive model.

The field test was aimed to study the interaction behavior between the pile and the saturated cohesionless soil under vibratory lateral loading, and to examine the effects of potential liquefaction on the soil-pile system. Such needs require a test site with a soil
profile of loose to medium dense sand or silty sand and with a water table very close to the ground surface. The best location for such soil conditions was found at the southern city limits of Seal Beach, Orange County, California as shown in Fig.(7.20) (Ertec, 1981).

The soil at the Seal Beach test site consists of 18 to 20 ft (5.5 to 6.1 m) of medium dense uniform silty sand overlying strata of silt, clayey silt, sand and siltstone. The upper layer of sand is of the most interest since effects of the pile on the soil below about 10-pile diameters [20 ft (6.1 m)] are negligible (Ertec, 1981). The twice-daily flooding of the area ensure the saturation of the soil. The site soil conditions were evaluated by performing two types of in situ soil tests on the site soil: cone penetration tests and a continuous standard penetration test. Locations of boreholes and soundings are shown in Fig.(7.21). The soil profile is presented in Fig.(7.22).

7.3.2. Test Configuration and Instrumentation

The two steel pipe piles driven at the test site were 40 ft (12.2 m) long and 24 in. (0.61 m) in outside diameter with a uniform wall thickness of 0.5 inch (12.7 mm). The penetration was 32 feet (9.76 m). The diameter of the test piles was chosen to be representative of the size of piles used in conventional construction and small enough to exhibit measurable response during the expected range of lateral loading. A platform was built and attached to the pile to hold both the vibration generators and the lead ingots, Fig.(7.23). Extensive field instrumentations were used to monitor pile bending moments, pile head displacement, and pore pressure in the surrounding soil.
The bending moment along the pile length was measured by attaching electrical strain gages at the surface points to measure mechanical strain at those points. The moment was then determined from the following well-known relation:

\[ \varepsilon = \frac{M y}{E I} \]  

(7.1)

where \( \varepsilon \) is mechanical strain, \( M \) is bending moment, \( y \) is the distance from the neutral axis to point of strain measurement, and \( EI \) is the flexural rigidity of the member. The strain gage configuration is shown in Fig.(7.24).

To measure the fluctuation of porewater pressure around the pile during vibration, piezometers were placed at various depths and distances from the pile. The piezometer units were built into three sections of pipe which were pushed into the ground 11, 23.5, and 59 in. (0.28, 0.6, 1.5 m) away from the pile wall parallel to the axis of loading. The piezometer arrangement is shown in Fig. (7.25).

Lateral deflection of the pile head during loading was determined in two ways: double integration of the moment-depth curve obtained from the strain gage readings, and direct measurement at two points on the pile itself. The latter was accomplished through the use of two Direct-Linear Variable Differential Transducers (DC-LVDT) located at 12 and 36 inches (0.305 and 0.915 m) above the ground surface. Comparison of the computed deflections from double integration of the moment curves and the measured deflections from the DC-LVDT’s shows a maximum difference of less than 0.1 in. (2.54 mm).

The program of field tests consisted of 12-frequency sweep pile shaking tests. Each test began with a frequency sweep at a low level of force and then increased up to
and past the first-mode resonance. Results of test No. 9 are used in this study to verify the DSC-finite element simulation of the pile-soil system. This test was run with a frequency range between 0-3.58 Hz and force range between 0-4709 lbf (0-20.9 kN). Results indicate a natural frequency of 2.01 Hz and peak amplitude of displacement at resonance at 2.0 ft (0.61 m) from the pile head of 0.259 in. (6.58 mm). The peak moment in the pile at resonance was 5.8x10^5 lbf-in. (67 kN-m) while the damping ratio, based on the half bandwidth method, was 5.5% of critical damping. Fig.(7.26) shows the response for test No. 9. The mode shape for a higher force level shaking at resonance is plotted in Fig.(7.27). The maximum moment induced in the pile at resonance for this test is located about 14 ft (4.27 m) from the top of the pile, i.e., 6 ft (1.83 m) below ground surface.

Fig.(7.28) shows oscillations of pore pressure during vibration but no strong tendency for an increase in pressure was observed. However, a rapid increase of 10 in. (245 mm) of water head inside the standpipe piezometers was observed during 30 cycles of vibration. Other phenomena such as small sand boils, bleeding of water at the ground surface, and consolidation of the sand around the pile were observed during the vibration. These observations indicate that a progressive buildup in pore pressure and possibly partial liquefaction of the near surface sand had occurred. A schematic illustration of the above described phenomena during and immediately after vibration test is shown in Fig.(7.29) (Ertec, 1981).

7.3.3. Finite Element Simulation

A laterally loaded pile (test No. 9) is analyzed in two-dimensional finite element condition using DSC-DYN2D. Plane strain idealization is used to approximate the test.
Two independent finite element analyses, without interface and with interface, are carried out to demonstrate the effect of interface on the behavior of pile-soil system. Material behavior of sand and sand-steel interface is modeled using Disturbed State Concept model. The foundation soil, which consists of medium dense fine silty sand, is simulated using the parameters of saturated Ottawa sand with relative density $D_r = 60\%$. These parameters were calculated by Park and Desai (2000). The parameters from the interface tests on Ottawa sand-steel interface calculated in chapter 6 of this study are used for those elements in contact with the pile. Those elements are assigned as interface elements with a thickness of $t = 1.85$ mm. The steel pile is simulated using quadrilateral elements with same outer diameter and flexural rigidity $EI$ as the test pile and modeled using elastic model ($\nu = 0.3$ and $E = 1.335 \times 10^5$). Table (7.2) summarizes the material properties used for soil and interface.

Harmonic sinusoidal wave displacement, Fig.(7.30), is applied in the lateral direction (x-direction) 2 ft (0.61 m) below the top of the pile. The amplitude of the applied displacement function is 0.259 in. (6.58 mm) and the frequency is 2.01 Hz. The analysis is carried out for 12.5 cycles with time step $\Delta t = 0.025$ sec and for a total of 250 time steps. The finite element domain used for this problem is illustrated in Fig.(7.31). It is discretized into 439 elements and 488 nodes. Four-node isoparametric elements are used for the soil, the interface, and the steel pile. For the boundary conditions, boundaries AB, BC, and CD are fixed in both x and y directions and boundary AD is free in both x and y directions. The elements corresponding to piezometers locations are shown in Fig.(7.32).
7.3.4. Effect of Boundary Conditions

The influence of boundary on the pile-soil behavior is investigated by changing the mesh dimensions. Different values of mesh width \( w \) (distance from the side boundary to the pile face) and \( d \) (depth of the mesh below the pile tip) were considered. It is found that the values of \( w \geq 50D \) and \( d \geq 5D \) eliminate the effect of boundary on the pile deflection at step 5 (peak of the first cycle), where \( D \) is the pile diameter. However, the parametric study indicates values of \( w \geq 60D \) and \( d \geq 10D \) reduce the effect of boundary on the prediction of porewater pressure at elements 339, 398, and 399. Figures (7.33-7.36) demonstrate some results of mesh size effects on pile deflection and porewater pressure. Based on these results, the final dimensions of the mesh adopted in this study are: \( w = 70D \) and \( d = 15D \).

7.3.5. Comparison of Results and Discussion

The laterally loaded pile response is presented in terms of deflection, slope, and bending moment along the pile length, at step 5, and the porewater pressure variation with time at three locations in the soil mass, Fig.(7.32). The predicted pore pressure values are smoothed by using 2-point moving average technique. The results from the finite element simulation for both without and with interface cases are compared with measured values.

Figure (7.37) shows the variation of pile deflection at step 5 with the depth of pile. The finite element results for both with and without interface cases slightly underestimate, but in general, are in good agreement with the measured values. This difference can be attributed for in the two-dimensional approximation in the finite element analysis.
However, the consideration of interface behavior slightly improves the predicted deflection. Pile slope and bending moment are calculated from the results of pile deflection using finite difference equations and the results are presented in Figs.(7.38 and 7.39). For both slope and bending moment, the agreement with the measured values is better for the case when interface behavior is considered.

Pore pressure development with time at elements 339,398, and 399, are shown in Figs.(7.40A-7.40C). The locations of the above elements correspond to the locations of piezometers installed in the field. The predicted pore pressure values for both with and without interface cases in the three locations can be considered to be comparable with the field measurements. However, porewater pressures for the case of no interface show irregular variation compared to those from with interface, and the amplitudes differ for the case of without interface.

7.3.6. Liquefaction Potential and Disturbance

Contour images for excess porewater pressure (PWP), liquefaction potential (LP) and disturbance (D) at the two peaks of the eleventh cycle are chosen for discussion. The results of the positive peak (step 204) are presented in Figs.(7.41-7.43) and at the negative peak (step 214) are presented in Figs.(7.44-7.46). Excess pore pressure generated as the pile head is displaced as shown in Figs.(7.41 and 7.44). Positive peak displacement (step 204) resulted in the generation of a positive excess pore pressure in the soil side where the pile head is moving toward that side. The other side of the soil experiences negative excess pore pressure. At the negative peak displacement (step 214) the signs of excess pore pressure are reversed for the two sides of the soil domain. The
higher generation in the excess pore pressure occurs mostly at shallow depths around the pile head and decreases with the increasing of soil depth.

Figures (7.42 and 7.45) show the variation of LP for the positive peak and negative peak displacements, respectively. The values of LP in the shallow depths become equal or greater 1.0 which indicates soil liquefaction due to generation of positive excess pore pressure. However, due to increased overburden pressure with depth, the value of LP reduces and becomes less than 1.0 and hence, the soil does not show the tendency to liquefy.

Fig.(7.43 and 7.46) present the disturbance contours at the two peaks. These figures clearly represent the accumulative microstructural changes occurring in the soil during the cyclic loading. The disturbance values varied between 0.7 and 1.0 immediately around the pile and then reduced and became less than 0.7 in the far regions. Park and Desai (2000) identify the initiation of liquefaction for Ottawa sand at $D_c = 0.84$. For Ottawa sand-steel interface, the critical disturbance corresponding to the state of liquefaction, as calculated in chapter 6, is $D_c = 0.91$. Hence, from Fig.(7.43 and 7.46), liquefaction occurs around the pile and in the shallow soil, as reported by Ertec (1981).

7.4. Conclusion

Behavior of offshore piles based on nonlinear finite element analysis with DSC model are studied using DSC-DYN2D program. This simulation is verified by back predicting the observed behavior of instrumented axially loaded pile and full-scale laterally loaded pile. Results from numerical solutions are compared with field
measurements. Very good agreement between numerical predictions and field measurements were found.

It is concluded that the finite element analysis, along with DSC constitutive model, accounted for nonlinearities arising from soil-pile interaction and the behavior of two-phase porous media under cyclic or dynamic loading. The model is well implemented in the DSC-DYN2D code and capable of predicting the behavior of offshore piles subjected to axial or lateral loading. Moreover, the consideration of interface behavior improves the predicted results.
Table (7.1): Material Parameters for Sabine Clay and Sabine Clay-Steel Interface

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Table (7.2): Material Parameters for Ottawa Sand and Ottawa Sand-Steel Interface

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Fig. (7.1): Site of Instrumented Pile Test, Sabine, Texas (ETC, 1986)
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Fig.(7.19): Disturbance at Step 738
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Fig. (7.22): Seal Beach Site Soil Profile (Ertec, 1981)

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<td>DARK GRAY SILT WITH SOME SAND AND CLAY, MANY SMALL SHELLS, MEDIUM DENSE</td>
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<td>28</td>
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<td>31</td>
<td>S</td>
<td>31</td>
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<td>23</td>
<td>NR</td>
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</table>

ELEVATION: MEAN SEA LEVEL  DATE DRILLED: 11-18-80 (SPT)  
EQUIPMENT USED: FAILING 750 & CPT TRUCK  
WATER LEVEL: 0'  

S STANDARD SPILT SPOON SAMPLE (ASTM D1586)  
NR NO RECOVERY  
"AVERAGE VALUE OF BORINGS 1 & 2 (WHEN APPLICABLE)"  
"STANDARD 140 LB HAMMER DROPPED 30"
Fig. (7.23): Test Configuration (Ertec, 1981)
Fig.(7.24): Strain Gage Configuration (Ertec, 1981)

**PROCEDURE FOR INSTRUMENTING NSF PILE**

1. WELD 1 1/2" SQUARE STEEL TUBES (e) TO INSIDE OF 24" DIAMETER STEEL PILE (d).
2. PREFABRICATE 1" DIAMETER STEEL TUBE (c) BY EPOXYING 8 PAIRS STRAIN GAGES (d) AND (f).
3. POT 2 STRAIN-GAGED TUBES INTO 2 OPPOSITE SQUARE TUBES WITH EPOXY COMPOUND (7).
4. CONNECT EACH OF 8 STRAIN GAGE CABLES (8) [3 SHOWN] IN BRIDGE CONFIGURATION SHOWN BELOW.

**CUTAWAY VIEW OF ONE STRAIN GAGE TUBE**

**CROSS SECTION OF INSTRUMENTED PILE**

**CONFIGURATION OF STRAIN GAGE BRIDGE**

(1 OF 8 IN PILE)
Fig. (7.25): Piezometer Locations (Ertec, 1981)
Fig. (7.26): Displacement Response Curve for Test No. 9 Normalized
to 367 lbf (1.63 kN) Peak Lateral Force (Ertec, 1981)
Fig.(7.27): Peak Strain, Slope, and Lateral Displacement for Test No. 9 (Ertec, 1981)
Fig.(7.28): Records of Pore Pressure with Time (Ertec, 1981)
Fig. (7.29): Schematic Illustration of Vibration-Induced Pore Pressure Buildup Related Response (Ertec, 1981)

1. MINIATURE SAND BOIL
2. PORE PRESSURE DISSIPATION RELATED CONSOLIDATION
3. EXCESS PORE PRESSURE INCREASE (AFTER ABOUT 30 CYCLES OF MEDIUM LEVEL SHAKING)
Fig.(7.30): Applied Displacement on Laterally Loaded Pile
Fig.(7.31): Finite Element Mesh (Not to Scale)
Fig. (7.32): Elements Corresponding to Piezometers Locations (Not to Scale)
Fig.(7.33): Effect of Boundary Depth $d$ on Pile Deflection at Step 5
Fig. (7.34A): Effect of Boundary Depth \( d \) on Excess Pore Pressure at Element 339
Fig.(7.34B): Effect of Boundary Depth \( d \) on Excess Pore Pressure at Element 398
Fig.(7.34C): Effect of Boundary Depth $d$ on Excess Pore Pressure at Element 399
Fig(7.35): Effect of Boundary Width $w$ on Pile Deflection at Step 5
Fig. (7.36A): Effect of Boundary Width $w$ on Excess Pore Pressure at Element 339
Fig.(7.36B): Effect of Boundary Width $w$ on Excess Pore Pressure at Element 398.
Fig. (7.36C): Effect of Boundary Width $w$ on Pore Pressure at Element 399
Fig.(7.37): Effect of Interface on Pile Deflection at Step 5
Fig.(7.38): Effect of Interface on Pile Slope at Step 5
Fig.(7.39): Effect of Interface on Bending Moment of the Pile at Step 5
Fig.(7.40A): Effect of Interface on Excess Pore Pressure at Element 339
Fig.(7.40B): Effect of Interface on Excess Pore Pressure at Element 398
Fig.(7.40C): Effect of Interface on Excess Pore Pressure at Element 399
Fig.(7.41): Excess Porewater Pressure at Step 204
Fig.(7.42): Liquefaction Potential at Step 204
Fig.(7.43): Disturbance at Step 204
Fig.(7.44): Excess Porewater Pressure at Step 214
Fig.(7.45): Liquefaction Potential at Step 214
Fig.(7.46): Disturbance at Step 214
CHAPTER 8: SUMMARY AND CONCLUSIONS

8.1. Summary

The response of offshore structures mainly depends on the supporting foundation, the geologic media on which the structure is placed, and the interaction between them. The behavior of such system is strongly nonlinear, inelastic and affected by factors such as the interface behavior and porewater pressure development. In order to study the real behavior, a proper constitutive model has to be incorporated in finite element formulation. A constitutive model for the stress-strain behavior of soils and interfaces is developed using Disturbed State Concept (DSC). The work accomplished in this study is summarized as follows:

1. Modification of the CYMDOF-P device for saturated interfaces.
2. Performance of the laboratory test program which includes six (6) normal compression tests and seventy two (72) interface simple-shear tests on dry and saturated Ottawa sand-steel and different steel roughnesses using CYMDOF-P device.
3. Determination of physical parameters related to interface behavior for the materials used in the laboratory testing program. This includes the surface roughness parameters for the steel samples and the particle size distribution with index properties of Ottawa sand.
4. Development of new constitutive model based on Disturbed State Concept (DSC) to describe the behavior of saturated interfaces subjected to static and cyclic
loading conditions. This model is capable of capturing essential sand interface behavior including degradation of shear stress, relative (slip) motions, and development of porewater pressure.

5. Determination of DSC model parameters using the laboratory test results of saturated Ottawa sand-steel interfaces. Procedures for finding parameters are discussed in detail.

6. Verification of the DSC model with respect to the results used in parameters determination and back predicting independent tests that are not used in parameters determination.

7. The concept of critical disturbance $D_c$, proposed by Desai (2000), for liquefaction identification is developed for saturated Ottawa sand-steel interfaces.

8. Application of DSC model for offshore piles. The finite element method with the DSC model is verified by back predicting the observed behavior of instrumented an axially loaded pile and a full-scale laterally loaded pile. The nonlinear finite element code DSC-DYN2D is used for this purpose. Identification of liquefaction potential based on critical disturbance $D_c$ is compared with the conventional method given by Seed and Lee (1966) for both problems.

### 8.2. Conclusions

Conclusions achieved from this study can be summarized as follows:

1. From the results of Ottawa sand-steel interface tests, it is observed that:
   
a. The normal stiffness remains constant as the number of cycles increases for both dry and saturated interfaces.
b. The maximum shear strength increases with the increase of the normal stress and occurs at a larger relative displacement.

c. Higher surface roughness for the upper steel sample mobilizes higher peak strength.

d. No stress softening is observed after reaching the peak shear strength.

e. The deformation of sand during shear formed only a small percentage of the total displacement and the total displacement consisted mostly of interface sliding.

f. The development of pore pressure in saturated shear tests reduces the effective normal stress which leads to a reduction of interface shear strength.

g. An increase in amplitude of shear displacement and/or increasing of surface roughness accelerate the rate of shear strength degradation for the two-way cyclic shear tests of saturated interfaces. This degradation is attributed to the development of porewater pressure.

h. The time and number of cycles at liquefaction increase as:

- The applied normal stress increases.
- The surface roughness decreases.
- The amplitude of shear displacement decreases.
- The frequency of shear displacement decreases.

2. The proposed interface constitutive model based on DSC provides reasonable agreement with the measurements of saturated simple shear tests of Ottawa sand-steel interfaces. The applicability of this model has been successfully
demonstrated by back predicting independent laboratory tests that are not used in parameters calculation. It is found that the model can simulate interface behavior under various roughnesses, normal stresses, cyclic displacement amplitudes and frequencies and applicable for both static and cyclic loading.

3. All parameters derived from test results are found to be well related to the surface roughness parameter $R_s$ except for the hardening parameter $h_2$ and the critical void ratio $e_0^c$.

4. The average critical disturbance values are, respectively, found as 0.93, 0.921, and 0.852 for smooth, slightly textured and moderately textured roughnesses. A good correlation is found between the critical disturbance $D_c$ and the surface roughness parameter $R_s$.

5. The finite element method with DSC constitutive model in simulating offshore piles, which include axially loaded pile segment in clay and full-scale laterally loaded pile in sand, provides successful predictions for pile-soil behavior. The simulation accounted for nonlinearities which arise from soil behavior, the soil-pile interaction, and the behavior of two-phase porous media under cyclic or dynamic loading.

6. A finite element mesh with dimensions: $w = 21 \ D$ and $d = 39 \ D$ is found adequate for predictions of shear transfer and porewater pressure for axially loaded pile. However, for laterally loaded pile, reasonable prediction for pile deflection and porewater pressure is found with mesh dimensions: $w = 60 \ D$ and $d = 10 \ D$. 
7. The inclusion of interface behavior in the pile analysis provides a better prediction of pile behavior.

8. The new procedure for identifying liquefaction potential based on critical disturbance concept agrees with the prediction from the conventional method for an axially loaded pile. For a laterally loaded pile, however, the two approaches give different results.

### 8.3 Recommendations for Future Research

Based upon literature review and this research, some future studies are suggested as follows:

1. Development of CYMDOF-P device to enable measuring the interface thickness.

2. Studies of the behaviors of other types of geologic materials such as loose sand, calcareous sands (highly compressible sands), and soft clays. Shear behavior and liquefaction of such materials, and also interface behavior between these types of soils and structural materials are important in practice.

3. Implementation of the DSC model in 3D-finite element program to get higher accuracy for 3D-boundary value problems such as laterally loaded piles.

4. Verification of DSC model and critical disturbance concept for fully liquefied pile-soil system.

5. Investigations of other types of structural materials with different surface roughnesses. Determination of an optimum roughness parameter is necessary to characterize the interface behavior.
6. Parametric studies for offshore pile problems should be conducted and the results of finite element-DSC simulation with other methods of analysis such as \( p-y \) and \( t-z \) curves compared. Such studies may be used to introduce procedures for analysis and design of offshore piles based on the numerical analysis.

7. The interface behavior under earthquakes or cyclic loadings should be evaluated with tests under cyclic loading conditions. However, other dynamic loading conditions such as an impact load may change the interface behavior significantly from that in static and cyclic loading.

8. Although a constant normal stiffness condition is more likely to exist across the interface in situ rather than constant normal stress or displacement, the interface response under constant normal stiffness has not received much attention in the literature.
APPENDIX A: TEST RESULTS

Table (A.1): Normal Compression Tests on Dry and Saturated Interfaces

<table>
<thead>
<tr>
<th>Test</th>
<th>Roughness</th>
<th>Applied Normal Stress (kPa)</th>
<th>Rate of Loading (N/sec)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. Dry Interface</td>
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</tr>
<tr>
<td>ND1*</td>
<td>RI&quot;</td>
<td>300</td>
<td>25</td>
</tr>
<tr>
<td>ND2</td>
<td>RII</td>
<td></td>
<td></td>
</tr>
<tr>
<td>ND3</td>
<td>RIII</td>
<td></td>
<td></td>
</tr>
<tr>
<td>2. Saturated Interface</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>NS1**</td>
<td>RI</td>
<td>300</td>
<td>25</td>
</tr>
<tr>
<td>NS2</td>
<td>RII</td>
<td></td>
<td></td>
</tr>
<tr>
<td>NS3</td>
<td>RIII</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

*ND1 = Normal Dry 1 (and so on)
** NS1 = Normal Saturated 1 (and so on)
# RI = Roughness I (and so on)
Fig. (A.1): Normal Compression Test for Dry Interface (Roughness: RI)

Fig. (A.2): Normal Compression Test for Dry Interface (Roughness: RII)
Fig.(A.3): Normal Compression Test for Dry Interface (Roughness: RIII)

Fig.(A.4): Normal Compression Test for Saturated Interface (Roughness: RI)
Fig. (A.5): Normal Compression Test for Saturated Interface (Roughness: RII)

Fig. (A.6): Normal Compression Test for Saturated Interface (Roughness: RIII)
Table (A.2): Simple Shear Tests on Dry Interfaces

<table>
<thead>
<tr>
<th>Test</th>
<th>Roughness</th>
<th>Normal Stress (kPa)</th>
<th>Maximum Displacement (mm)</th>
<th>Rate of Loading (mm/sec)</th>
<th>Amplitude (mm)</th>
<th>Frequency (Hz)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. One-Way Monotonic Shear Tests</td>
<td>SD1* RI</td>
<td>50</td>
<td></td>
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<td></td>
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<tr>
<td></td>
<td>SD2 RI</td>
<td>100</td>
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<tr>
<td></td>
<td>SD3 RI</td>
<td>150</td>
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<tr>
<td></td>
<td>SD4 RI</td>
<td>200</td>
<td></td>
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<td></td>
<td>SD5 RI</td>
<td>300</td>
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<tr>
<td></td>
<td>SD6 RI</td>
<td>100</td>
<td>6</td>
<td>0.05</td>
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<tr>
<td></td>
<td>SD7 RI</td>
<td>200</td>
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<td></td>
<td>SD8 RI</td>
<td>300</td>
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<td></td>
<td>SD9 RI</td>
<td>100</td>
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<td></td>
<td>SD10 RI</td>
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<td></td>
<td>SD11 RI</td>
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<td>2. One-Way Cyclic Shear Tests</td>
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<td>50</td>
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<td>SD13 RI</td>
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<td></td>
<td>SD14 RI</td>
<td>150</td>
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<td>SD15 RI</td>
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<tr>
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<td>SD16 RI</td>
<td>300</td>
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<td>SD17 RI</td>
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<td></td>
<td>SD18 RI</td>
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<td>SD19 RI</td>
<td>300</td>
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<td></td>
<td>SD20 RI</td>
<td>100</td>
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<td></td>
<td>SD21 RI</td>
<td>200</td>
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<td></td>
<td>SD22 RI</td>
<td>300</td>
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<tr>
<td>3. Two-Way Cyclic Shear Tests</td>
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<td>100</td>
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<td>SD24 RII</td>
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<td></td>
<td>SD25 RII</td>
<td>150</td>
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<td></td>
<td>SD26 RII</td>
<td>200</td>
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<td></td>
<td>SD27 RII</td>
<td>300</td>
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<td>SD28 RII</td>
<td>50</td>
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<td></td>
<td>SD29 RII</td>
<td>200</td>
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<td></td>
<td>SD30 RII</td>
<td>300</td>
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<td></td>
<td>SD31 RII</td>
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<td>SD34 RII</td>
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<td>SD35 RII</td>
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<td></td>
<td>SD36 RII</td>
<td>300</td>
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</table>

*SD1 = Shear Dry 1 (and so on).
Fig. (A.7): Shear Stress vs. Relative Shear Displacement for Dry One-Way
Monotonic Shear Test ($\sigma_n=50$ kPa, Roughness: RI)

Fig. (A.8): Shear Stress vs. Relative Shear Displacement for Dry One-Way
Monotonic Shear Test ($\sigma_n=100$ kPa, Roughness: RI)
Fig.(A.9): Shear Stress vs. Relative Shear Displacement for Dry One-Way Monotonic Shear Test ($\sigma_n=150$ kPa, Roughness: RI)

Fig.(A.10): Shear Stress vs. Relative Shear Displacement for Dry One-Way Monotonic Shear Test ($\sigma_n=200$ kPa, Roughness: RI)
Fig. (A.11): Shear Stress vs. Relative Shear Displacement for Dry One-Way Monotonic Shear Test ($\sigma_n=300$ kPa, Roughness: RI)

Fig. (A.12): Shear Stress vs. Relative Shear Displacement for Dry One-Way Monotonic Shear Test ($\sigma_n=100$ kPa, Roughness: RII)
Fig.(A.13): Shear Stress vs. Relative Shear Displacement for Dry One-Way Monotonic Shear Test ($\sigma_n = 200$ kPa, Roughness: RII)

Fig.(A.14): Shear Stress vs. Relative Shear Displacement for Dry One-Way Monotonic Shear Test ($\sigma_n = 300$ kPa, Roughness: RII)
Fig.(A.15): Shear Stress vs. Relative Shear Displacement for Dry One-Way Monotonic Shear Test ($\sigma_n=100$ kPa, Roughness: RIII)

Fig.(A.16): Shear Stress vs. Relative Shear Displacement for Dry One-Way Monotonic Shear Test ($\sigma_n=200$ kPa, Roughness: RIII)
Fig.(A.17): Shear Stress vs. Relative Shear Displacement for Dry One-Way Monotonic Shear Test ($\sigma_n=300$ kPa, Roughness: RIII)

Fig.(A.18): Shear Stress vs. Relative Shear Displacement for Dry One-Way Cyclic Shear Test ($\sigma_n=50$ kPa, Roughness: RI)
Fig. (A.19): Shear Stress vs. Relative Shear Displacement for Dry One-Way Cyclic Shear Test ($\sigma_n=100$ kPa, Roughness: RI)

Fig. (A.20): Shear Stress vs. Relative Shear Displacement for Dry One-Way Cyclic Shear Test ($\sigma_n=150$ kPa, Roughness: RI)
Fig.(A.21): Shear Stress vs. Relative Shear Displacement for Dry One-Way Cyclic Shear Test ($\sigma_n=200$ kPa, Roughness: RI)

Fig.(A.22): Shear Stress vs. Relative Shear Displacement for Dry One-Way Cyclic Shear Test ($\sigma_n=300$ kPa, Roughness: RI)
Fig. (A.23): Shear Stress vs. Relative Shear Displacement for Dry One-Way Cyclic Shear Test ($\sigma_n=100$ kPa, Roughness: RII)

Fig. (A.24): Shear Stress vs. Relative Shear Displacement for Dry One-Way Cyclic Shear Test ($\sigma_n=200$ kPa, Roughness: RII)
Fig. (A.25): Shear Stress vs. Relative Shear Displacement for Dry One-Way Cyclic Shear Test ($\sigma_n=300$ kPa, Roughness: RII)

Fig. (A.26): Shear Stress vs. Relative Shear Displacement for Dry One-Way Cyclic Shear Test ($\sigma_n=100$ kPa, Roughness: RIII)
Fig. (A.27): Shear Stress vs. Relative Shear Displacement for Dry One-Way Cyclic Shear Test ($\sigma_n=200$ kPa, Roughness: RIII)

Fig. (A.28): Shear Stress vs. Relative Shear Displacement for Dry One-Way Cyclic Shear Test ($\sigma_n=300$ kPa, Roughness: RIII)
Fig.(A.29): Shear Stress vs. Relative Shear Displacement for Dry Two-Way Cyclic Shear Test ($\sigma_n=100$ kPa, Roughness: RI, Amplitude=2.5 mm, Frequency=0.1 Hz)

Fig.(A.30): Shear Stress vs. Relative Shear Displacement for Dry Two-Way Cyclic Shear Test ($\sigma_n=100$ kPa, Roughness: RI, Amplitude=5.0 mm, Frequency=0.1 Hz)
Fig.(A.31): Shear Stress vs. Relative Shear Displacement for Dry Two-Way Cyclic Shear Test ($\sigma_n=100$ kPa, Roughness: RI, Amplitude=2.5 mm, Frequency=0.04 Hz)

Fig.(A.32): Shear Stress vs. Relative Shear Displacement for Dry Two-Way Cyclic Shear Test ($\sigma_n=100$ kPa, Roughness: RI, Amplitude=2.5 mm, Frequency=0.25 Hz)
Fig.(A.33): Shear Stress vs. Relative Shear Displacement for Dry Two-Way Cyclic Shear Test ($\sigma_n=100$ kPa, Roughness: RI, Amplitude=2.5 mm, Frequency=0.625 Hz)

Fig.(A.34): Shear Stress vs. Relative Shear Displacement for Dry Two-Way Cyclic Shear Test ($\sigma_n=50$ kPa, Roughness: RI, Amplitude=5.0 mm, Frequency=0.1 Hz)
Fig. (A.35): Shear Stress vs. Relative Shear Displacement for Dry Two-Way Cyclic Shear Test ($\sigma_n=150$ kPa, Roughness: RI, Amplitude=5.0 mm, Frequency=0.1 Hz)

Fig. (A.36): Shear Stress vs. Relative Shear Displacement for Dry Two-Way Cyclic Shear Test ($\sigma_n=200$ kPa, Roughness: RI, Amplitude=5.0 mm, Frequency=0.1 Hz)
Fig.(A.37): Shear Stress vs. Relative Shear Displacement for Dry Two-Way Cyclic Shear Test ($\sigma_n=50$ kPa, Roughness: RII, Amplitude=5.0 mm, Frequency=0.1 Hz)

Fig.(A.38): Shear Stress vs. Relative Shear Displacement for Dry Two-Way Cyclic Shear Test ($\sigma_n=100$ kPa, Roughness: RII, Amplitude=5.0 mm, Frequency=0.1 Hz)
Fig.(A.39): Shear Stress vs. Relative Shear Displacement for Dry Two-Way Cyclic Shear Test ($\sigma_n=200$ kPa, Roughness: RII, Amplitude=5.0 mm, Frequency=0.1 Hz)

Fig.(A.40): Shear Stress vs. Relative Shear Displacement for Dry Two-Way Cyclic Shear Test ($\sigma_n=50$ kPa, Roughness: RIII, Amplitude=5.0 mm, Frequency=0.1 Hz)
Fig. (A.41): Shear Stress vs. Relative Shear Displacement for Dry Two-Way Cyclic Shear Test ($\sigma_n=100$ kPa, Roughness: RIII, Amplitude=5.0 mm, Frequency=0.1 Hz)

Fig. (A.42): Shear Stress vs. Relative Shear Displacement for Dry Two-Way Cyclic Shear Test ($\sigma_n=200$ kPa, Roughness: RIII, Amplitude=5.0 mm, Frequency=0.1 Hz)
### Table (A.3): Simple Shear Tests on Saturated Interfaces

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<tr>
<th>Test</th>
<th>Roughness</th>
<th>Normal Stress (kPa)</th>
<th>Maximum Displacement (mm)</th>
<th>Rate of Loading (mm/sec)</th>
<th>Amplitude (mm)</th>
<th>Frequency (Hz)</th>
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*SS1 = Shear Saturated 1 (and so on)
Fig.(A.43): Shear Stress vs. Relative Shear Displacement for Saturated One-Way Monotonic Shear Test ($\sigma_n=50$ kPa, Roughness: RI)

Fig.(A.44): Shear Stress vs. Relative Shear Displacement for Saturated One-Way Monotonic Shear Test ($\sigma_n=100$ kPa, Roughness: RI)
Fig.(A.45): Shear Stress vs. Relative Shear Displacement for Saturated One-Way Monotonic Shear Test ($\sigma_n=150$ kPa, Roughness: RI)

Fig.(A.46): Shear Stress vs. Relative Shear Displacement for Saturated One-Way Monotonic Shear Test ($\sigma_n=200$ kPa, Roughness: RI)
Fig. (A.47): Shear Stress vs. Relative Shear Displacement for Saturated One-Way Monotonic Shear Test ($\sigma_n=300$ kPa, Roughness: RI)

Fig. (A.48): Shear Stress vs. Relative Shear Displacement for Saturated One-Way Monotonic Shear Test ($\sigma_n=100$ kPa, Roughness: RII)
Fig.(A.49): Shear Stress vs. Relative Shear Displacement for Saturated One-Way Monotonic Shear Test ($\sigma_n=200$ kPa, Roughness: RII)

Fig.(A.50): Shear Stress vs. Relative Shear Displacement for Saturated One-Way Monotonic Shear Test ($\sigma_n=300$ kPa, Roughness: RII)
Fig.(A.51): Shear Stress vs. Relative Shear Displacement for Saturated One-Way Monotonic Shear Test ($\sigma_n=100$ kPa, Roughness: RIII)

Fig.(A.52): Shear Stress vs. Relative Shear Displacement for Saturated One-Way Monotonic Shear Test ($\sigma_n=200$ kPa, Roughness: RIII)
Fig. (A.53): Shear Stress vs. Relative Shear Displacement for Saturated One-Way Monotonic Shear Test ($\sigma_n=300$ kPa, Roughness: RIII)

Fig. (A.54): Variation of Porewater Pressure with Time for Saturated One-Way Monotonic Shear Test ($\sigma_n=50$ kPa, Roughness: RI)
Fig. (A.55): Variation of Porewater Pressure with Time for Saturated One-Way Monotonic Shear Test ($\sigma_n=100$ kPa, Roughness: RI)

Fig. (A.56): Variation of Porewater Pressure with Time for Saturated One-Way Monotonic Shear Test ($\sigma_n=150$ kPa, Roughness: RI)
Fig. (A.57): Variation of Porewater Pressure with Time for Saturated One-Way Monotonic Shear Test ($\sigma_n=200 \text{ kPa}$, Roughness: RI)

Fig. (A.58): Variation of Porewater Pressure with Time for Saturated One-Way Monotonic Shear Test ($\sigma_n=300 \text{ kPa}$, Roughness: RI)
Fig. (A.59): Variation of Porewater Pressure with Time for Saturated One-Way Monotonic Shear Test ($\sigma_n=100$ kPa, Roughness: RII)

Fig. (A.60): Variation of Porewater Pressure with Time for Saturated One-Way Monotonic Shear Test ($\sigma_n=200$ kPa, Roughness: RII)
Fig. (A.61): Variation of Porewater Pressure with Time for Saturated One-Way Monotonic Shear Test ($\sigma_n = 300$ kPa, Roughness: RII)

Fig. (A.62): Variation of Porewater Pressure with Time for Saturated One-Way Monotonic Shear Test ($\sigma_n = 100$ kPa, Roughness: RIII)
Fig.(A.63): Variation of Porewater Pressure with Time for Saturated One-Way Monotonic Shear Test ($\sigma_n=200$ kPa, Roughness: RIII)

Fig.(A.64): Variation of Porewater Pressure with Time for Saturated One-Way Monotonic Shear Test ($\sigma_n=300$ kPa, Roughness: RIII)
Fig.(A.65): Shear Stress vs. Relative Shear Displacement for Saturated One-Way Cyclic Shear Test ($\sigma_n=50$ kPa, Roughness: RI)

Fig.(A.66): Shear Stress vs. Relative Shear Displacement for Saturated One-Way Cyclic Shear Test ($\sigma_n=100$ kPa, Roughness: RI)
Fig.(A.67): Shear Stress vs. Relative Shear Displacement for Saturated One-Way Cyclic Shear Test ($\sigma_n=150$ kPa, Roughness: RI)

Fig.(A.68): Shear Stress vs. Relative Shear Displacement for Saturated One-Way Cyclic Shear Test ($\sigma_n=200$ kPa, Roughness: RI)
Fig.(A.69): Shear Stress vs. Relative Shear Displacement for Saturated One-Way Cyclic Shear Test ($\sigma_n=300$ kPa, Roughness: RI)

Fig.(A.70): Shear Stress vs. Relative Shear Displacement for Saturated One-Way Cyclic Shear Test ($\sigma_n=100$ kPa, Roughness: RII)
Fig. (A.71): Shear Stress vs. Relative Shear Displacement for Saturated One-Way Cyclic Shear Test ($\sigma_n=200$ kPa, Roughness: RII)

Fig. (A.72): Shear Stress vs. Relative Shear Displacement for Saturated One-Way Cyclic Shear Test ($\sigma_n=300$ kPa, Roughness: RII)
Fig.(A.73): Shear Stress vs. Relative Shear Displacement for Saturated One-Way Cyclic Shear Test ($\sigma_n=100$ kPa, Roughness: RIII)

Fig.(A.74): Shear Stress vs. Relative Shear Displacement for Saturated One-Way Cyclic Shear Test ($\sigma_n=200$ kPa, Roughness: RIII)
Fig. (A.75): Shear Stress vs. Relative Shear Displacement for Saturated One-Way Cyclic Shear Test ($\sigma_n=300$ kPa, Roughness: RIII)

Fig. (A.76): Shear Stress vs. Relative Shear Displacement for Saturated Two-Way Cyclic Shear Test ($\sigma_n=100$ kPa, Roughness: RI, Amplitude=2.5 mm, Frequency=0.1 Hz)
Fig.(A.77): Shear Stress vs. Relative Shear Displacement for Saturated Two-Way Cyclic Shear Test ($\sigma_n=100$ kPa, Roughness: RI, Amplitude=5.0 mm, Frequency=0.1 Hz)

Fig.(A.78): Shear Stress vs. Relative Shear Displacement for Saturated Two-Way Cyclic Shear Test ($\sigma_n=100$ kPa, Roughness: RI, Amplitude=2.5 mm, Frequency=0.04 Hz)
Fig. (A.79): Shear Stress vs. Relative Shear Displacement for Saturated Two-Way Cyclic Shear Test ($\sigma_n=100$ kPa, Roughness: RI, Amplitude=2.5 mm, Frequency=0.25 Hz)

Fig. (A.80): Shear Stress vs. Relative Shear Displacement for Saturated Two-Way Cyclic Shear Test ($\sigma_n=100$ kPa, Roughness: RI, Amplitude=2.5 mm, Frequency=0.625 Hz)
Fig. (A.81): Shear Stress vs. Relative Shear Displacement for Saturated Two-Way Cyclic Shear Test ($\sigma_n=50$ kPa, Roughness: RI, Amplitude=5.0 mm, Frequency=0.1 Hz)

Fig. (A.82): Shear Stress vs. Relative Shear Displacement for Saturated Two-Way Cyclic Shear Test ($\sigma_n=150$ kPa, Roughness: RI, Amplitude=5.0 mm, Frequency=0.1 Hz)
Fig. (A.83): Shear Stress vs. Relative Shear Displacement for Saturated Two-Way Cyclic Shear Test ($\sigma_n=200$ kPa, Roughness: RI, Amplitude=5.0 mm, Frequency=0.1 Hz)

Fig. (A.84): Shear Stress vs. Relative Shear Displacement for Saturated Two-Way Cyclic Shear Test ($\sigma_n=50$ kPa, Roughness: RII, Amplitude=5.0 mm, Frequency=0.1 Hz)
Fig. (A.85): Shear Stress vs. Relative Shear Displacement for Saturated Two-Way Cyclic Shear Test ($\sigma_n=100$ kPa, Roughness: RII, Amplitude=5.0 mm, Frequency=0.1 Hz)

Fig. (A.86): Shear Stress vs. Relative Shear Displacement for Saturated Two-Way Cyclic Shear Test ($\sigma_n=200$ kPa, Roughness: RII, Amplitude=5.0 mm, Frequency=0.1 Hz)
Fig.(A.87): Shear Stress vs. Relative Shear Displacement for Saturated Two-Way Cyclic Shear Test ($\sigma_n=50$ kPa, Roughness: RIII, Amplitude=5.0 mm, Frequency=0.1 Hz)

Fig.(A.88): Shear Stress vs. Relative Shear Displacement for Saturated Two-Way Cyclic Shear Test ($\sigma_n=100$ kPa, Roughness: RIII, Amplitude=5.0 mm, Frequency=0.1 Hz)
Fig. (A.89): Shear Stress vs. Relative Shear Displacement for Saturated Two-Way Cyclic Shear Test ($\sigma_n=200$ kPa, Roughness: RIII, Amplitude=5.0 mm, Frequency=0.1 Hz)

Fig. (A.90): Variation of Peak Shear Stress with Time for Saturated Two-Way Cyclic Shear Test ($\sigma_n=100$ kPa, Roughness: RI, Amplitude=2.5 mm, Frequency=0.1 Hz)
Fig.(A.91): Variation of Peak Shear Stress with Time for Saturated Two-Way Cyclic Shear Test ($\sigma_n=100$ kPa, Roughness: RI, Amplitude=5.0 mm, Frequency=0.1 Hz)

Fig.(A.92): Variation of Peak Shear Stress with Time for Saturated Two-Way Cyclic Shear Test ($\sigma_n=100$ kPa, Roughness: RI, Amplitude=2.5 mm, Frequency=0.04 Hz)
Fig.(A.93): Variation of Peak Shear Stress with Time for Saturated Two-Way Cyclic Shear Test ($\sigma_n=100$ kPa, Roughness: RI, Amplitude=2.5 mm, Frequency=0.25 Hz)

Fig.(A.94): Variation of Peak Shear Stress with Time for Saturated Two-Way Cyclic Shear Test ($\sigma_n=100$ kPa, Roughness: RI, Amplitude=2.5 mm, Frequency=0.625 Hz)
Fig.(A.95): Variation of Peak Shear Stress with Time for Saturated Two-Way Cyclic Shear Test ($\sigma_n=50$ kPa, Roughness: RI, Amplitude=5.0 mm, Frequency=0.1 Hz)

Fig.(A.96): Variation of Peak Shear Stress with Time for Saturated Two-Way Cyclic Shear Test ($\sigma_n=150$ kPa, Roughness: RI, Amplitude=5.0 mm, Frequency=0.1 Hz)
Fig.(A.97): Variation of Peak Shear Stress with Time for Saturated Two-Way Cyclic Shear Test ($\sigma_n=200$ kPa, Roughness: RI, Amplitude=5.0 mm, Frequency=0.1 Hz)

Fig.(A.98): Variation of Peak Shear Stress with Time for Saturated Two-Way Cyclic Shear Test ($\sigma_n=50$ kPa, Roughness: RII, Amplitude=5.0 mm, Frequency=0.1 Hz)
Fig. (A.99): Variation of Peak Shear Stress with Time for Saturated Two-Way Cyclic Shear Test ($\sigma_n=100$ kPa, Roughness: RII, Amplitude=5.0 mm, Frequency=0.1 Hz)

Fig. (A.100): Variation of Peak Shear Stress with Time for Saturated Two-Way Cyclic Shear Test ($\sigma_n=200$ kPa, Roughness: RII, Amplitude=5.0 mm, Frequency=0.1 Hz)
Fig. (A.101): Variation of Peak Shear Stress with Time for Saturated Two-Way Cyclic Shear Test ($\sigma_n=50$ kPa, Roughness: RIII, Amplitude=5.0 mm, Frequency=0.1 Hz)

Fig. (A.102): Variation of Peak Shear Stress with Time for Saturated Two-Way Cyclic Shear Test ($\sigma_n=100$ kPa, Roughness: RIII, Amplitude=5.0 mm, Frequency=0.1 Hz)
Fig. (A.103): Variation of Peak Shear Stress with Time for Saturated Two-Way Cyclic Shear Test ($\sigma_n = 200$ kPa, Roughness: RIII, Amplitude=5.0 mm, Frequency=0.1 Hz)

Fig. (A.104): Variation of Porewater Pressure with Time for Saturated Two-Way Cyclic Shear Test ($\sigma_n = 100$ kPa, Roughness: RI, Amplitude=2.5 mm, Frequency=0.1 Hz)
Fig. (A.105): Variation of Porewater Pressure with Time for Saturated Two-Way Cyclic Shear Test ($\sigma_n = 100$ kPa, Roughness: RI, Amplitude=5.0 mm, Frequency=0.1 Hz)

Fig. (A.106): Variation of Porewater Pressure with Time for Saturated Two-Way Cyclic Shear Test ($\sigma_n = 100$ kPa, Roughness: RI, Amplitude=2.5 mm, Frequency=0.04 Hz)
Fig. (A.107): Variation of Porewater Pressure with Time for Saturated Two-Way Cyclic Shear Test ($\sigma_n=100$ kPa, Roughness: RI, Amplitude=2.5 mm, Frequency=0.25 Hz)

Fig. (A.108): Variation of Porewater Pressure with Time for Saturated Two-Way Cyclic Shear Test ($\sigma_n=100$ kPa, Roughness: RI, Amplitude=2.5 mm, Frequency=0.625 Hz)
Fig. (A.109): Variation of Porewater Pressure with Time for Saturated Two-Way Cyclic Shear Test \((\sigma_n=50 \text{ kPa}, \text{Roughness: RI, Amplitude}=5.0 \text{ mm, Frequency}=0.1 \text{ Hz})\)

Fig. (A.110): Variation of Porewater Pressure with Time for Saturated Two-Way Cyclic Shear Test \((\sigma_n=150 \text{ kPa}, \text{Roughness: RI, Amplitude}=5.0 \text{ mm, Frequency}=0.1 \text{ Hz})\)
Fig. (A.111): Variation of Porewater Pressure with Time for Saturated Two-Way Cyclic Shear Test ($\sigma_n=200$ kPa, Roughness: RI, Amplitude=5.0 mm, Frequency=0.1 Hz)

Fig. (A.112): Variation of Porewater Pressure with Time for Saturated Two-Way Cyclic Shear Test ($\sigma_n=50$ kPa, Roughness: RII, Amplitude=5.0 mm, Frequency=0.1 Hz)
Fig. (A.113): Variation of Porewater Pressure with Time for Saturated Two-Way Cyclic Shear Test \( (\sigma_n=100 \text{ kPa}, \text{Roughness: RII, Amplitude}=5.0 \text{ mm, Frequency}=0.1 \text{ Hz}) \)

Fig. (A.114): Variation of Porewater Pressure with Time for Saturated Two-Way Cyclic Shear Test \( (\sigma_n=200 \text{ kPa}, \text{Roughness: RII, Amplitude}=5.0 \text{ mm, Frequency}=0.1 \text{ Hz}) \)
Fig. (A.115): Variation of Porewater Pressure with Time for Saturated Two-Way Cyclic Shear Test ($\sigma_v=50$ kPa, Roughness: RIII, Amplitude=5.0 mm, Frequency=0.1 Hz)

Fig. (A.116): Variation of Porewater Pressure with Time for Saturated Two-Way Cyclic Shear Test ($\sigma_v=100$ kPa, Roughness: RIII, Amplitude=5.0 mm, Frequency=0.1 Hz)
Fig. (A.117): Variation of Porewater Pressure with Time for Saturated Two-Way Cyclic Shear Test ($\sigma_n=200$ kPa, Roughness: RIII, Amplitude=5.0 mm, Frequency=0.1 Hz)
APPENDIX B: PROFILOMETRY MEASUREMENTS

PROFILOMETRY MEASUREMENTS

Principle

The axial chromatism technique uses a white light source, where light passes through an objective lens with a high degree of chromatic aberration. The refractive index of the objective lens will vary in relation to the wavelength of the light. In effect, each separate wavelength of the incident white light will re-focus at a different distance from the lens (different height). When the measured sample is within the range of possible heights, a single monochromatic point will be focalized to form the image. Due to the confocal configuration of the system, only the focused wavelength will pass through the spatial filter with high efficiency, thus causing all other wavelengths to be out of focus.

The spectral analysis is done using a diffraction grating. This technique deviates each wavelength at a different position, intercepting a line of CCD, which in turn indicates the position of the maximum intensity and allows direct correspondence to the Z height position.

If the sample is composed of several transparent or semitransparent thin layers, each interface between adjacent layers will reflect the light of a different wavelength, and the spectrum of detected lights will be composed of a series of spectral peaks. The chromatic aberration technique allows all interfaces to be detected and their positions to be measured simultaneously.
Instrument Used

ST400 Optical Profiler
The flexible, modern design of the ST400 Optical Profiler includes: 150mm X-Y stages and a large coarse height adjustment to easily accommodate larger sample sizes. The ST400 also has an optional offset camera, with either manual or motorized zooms, to easily identify small features prior to measuring them. The Custom Profiler, a more open configuration, allows for the addition of larger X-Y stages to measure even larger areas, a 3600 rotational stage for measuring spherical or cylindrical parts, and many other custom configurations.

Probe Specifications

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Measurement Parameters

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REFERENCES


Winkler, E., Die Lehre Von Der Elastiziant and Festigkeit, Verlag, 182, Prague, 1867.


