Chapter 2

Background Information and Literature Review

2.1 Introduction

The purpose of this chapter is to summarize the information presented in the literature related to the behavior of cohesionless soils during monotonic and cyclic loading, cone penetration testing (*CPT*), and the development of excess pore pressures in sands during undrained loading. General discussions of steady-state principles and the definition of liquefaction are also presented in the chapter. The chapter is concluded with a summary of the common methods for evaluating the liquefaction potential of soil deposits based on cone penetrometer testing and energy-based concepts.

2.2 Behavior of Saturated Sands During Undrained Loading

Saturated sandy soils exposed to shear stresses may have a tendency to undergo a shear induced volume decrease. The shear stresses can be monotonic, as in the case of the self-weight of an earth embankment, or applied cyclically, as in the case of an earthquake. If the loading condition and soil properties permit drainage, the water is easily evacuated from the pores, resulting in an increase in the effective stress with increasing strain. The decrease in volume associated with this drainage results in a rearrangement of the soil particles into a denser state. However, if the soil properties or loading conditions do not permit drainage, the water in the system is not expelled from the soil voids during the loading. Instead, the water induces a transfer of stress from the soil skeleton onto the water, elevating the water pressure and subsequently decreasing the effective stress in the soil. If the combination of the in-situ and applied stress in the soil is larger than the soils' resistance to shear, deformation will occur until the applied stress and shear resistance reaches an equilibrium condition (Castro 1969). This equilibrium can occur after the soil has experienced large strains or can occur quickly after small strains.

Presented as Figures 2.1 and 2.2 are examples of the behavior of saturated sandy soils when subjected to undrained monotonic and cyclic triaxial loading. The behaviors of both loose (contractive) and dense (dilative) materials are considered. Anisotropic stress conditions ($\sigma_v \neq \sigma_h$) were assumed for all tests to simulate in-situ field conditions.

The response of contractive soil loaded monotonically and cyclically in undrained triaxial compression is presented as Figure 2.1. Soils associated with this behavior are generally low-density materials that contain voids that are abundant in both size and number or soils loaded at high confining stresses. When shear stresses are applied to these soils, the soil grains have a tendency to move into the void space between the grains, resulting in an overall compression of the soil structure. Since the volume of these samples is constant during the undrained loading, the tendency for compression is manifested by a pre-peak shear induced increase in pore water pressure (Figure 2.1b). The pore water pressure continually increases as the soil is sheared during monotonic loading and accumulates with the load cycles during cyclic loading. Additional loading beyond the peak strength results in large strength loss, as identified by the substantial post peak loss of shear strength (Figure 2.1a). The shear strength decreases until it reaches a large strain residual strength value, which is defined as S_{us} in Figure 2.1

The relationship of the initial stress to the undrained residual strength is further characterized through the stress paths shown in Figure 2.1c. Positive induced pore pressures drive the stress paths to the left, eventually reaching a failure condition and subsequent strength loss. The strength loss stopped at point X_1 for both the monotonic and cyclic loading conditions, suggesting the shear stresses and the shear strength reached a point of equilibrium at this stress condition. As noted through Figure 2.1, the residual strength of the contractive soil is independent of the type of loading (monotonic or cyclic) applied to the soil.

The diagrams included in Figure 2.2 contain the soil response for a material that exhibits dilative behavior during monotonic shear. This dilative behavior is characteristic of high density soils that exhibit a low void ratio or soils loaded at low confining stresses. When these materials are sheared, the soil grains can not move into the void space between the particles and are therefore forced to move up and over the adjacent soil grains. As shown in Figure 2.2 this type of behavior exhibits the tendency for the soil to increase in volume, generating an overall shear-induced increase in effective stress and decrease in pore water pressure in the soil.

The dilative behavior generated during the monotonic loading is initially preceded by contraction of the soil structure, as indicated by the increase in pore pressure at small strains. As shown in Figure 2.2b, this small incremental increase in pore water pressure accumulates during cyclic loading, resulting in an overall increase in pore water pressure and subsequent reduction in effective stress as loading continues. The behavior of the soil after this point depends on a relationship of the initial stress to the magnitude and duration of the induced cyclic load. If the cyclic loading conditions are substantial enough to reverse the shear stress direction and possibly bring the soil to zero effective stress, the soil will undergo large deformations (Figure 2.2c). Once the cyclic loading stops, however, the soil exhibits a strain hardening behavior and deformations cease. If the cyclic loading conditions are not substantial enough to reverse the shear stress direction, the soil exhibits a strain hardening behavior after the limiting strain condition is reached. Periods of zero effective stress do not occur and deformations are generally small (Figure 2.2c).

2.3 Steady-state Principles

The concept of steady-state was introduced by Castro (1975) to explain large strain soil behavior during drained and undrained shear. The concept was based on the earlier works of Casagrande (1936), who showed that all soils tested at the same confining pressure in drained triaxial compression approached the same density when sheared to large strains (Kramer 1996). The void ratio at this density was termed the *critical void ratio,* and later developments revealed that the critical void ratio was uniquely related to the confining pressure, as described by the *critical void ratio line (CVL).*

Castro (1969) advanced the critical void ratio concept through the use of stress-controlled undrained triaxial compression tests to measure soil behavior during undrained shear. His stress-controlled tests allowed for the identification of the large post-peak strength loss in loose soils and the potential for pore pressure accumulation during cyclic loading in dense soils. He confirmed the unique relationship between the critical void ratio and effective confining stress, i.e. the CVL, professed by Casagrande (1936) through undrained tests, and further revealed that the CVL obtained from the undrained tests fell below the CVL obtained from the drained tests. He attributed this difference to the generation of a *flow structure* during undrained shear, and used this undrained CVL to explain the liquefaction failure of the San Fernando Dam (Castro et al. 1982). Castro

suggested the soil in the flow structure was in its *steady-state*, which he defined as a "state of deformation of a soil mass where the deformation is occurring at a constant volume, normal effective stress, shear stress, and velocity" (Castro et al. 1982). He noted that the steady-state of deformation is only achieved after all particle orientation has reached a steady-state 'flow' type of condition and after all particle breakage is complete. A soil can only exist in this type of condition if the velocity and shear stress needed for continuous deformation are constant. If any variation from these 'constant' conditions is encountered, the soil will develop shear strength and the large shear induced deformations will cease.

Castro and Poulos (1977) introduced the concept of the *steady-state line* into the engineering community through a landmark paper presented at the ASCE Annual Convention in Philadelphia, PA. They identified the steady-state line (*SSL*) as the 3-D graphical representation of the locus of states in which a soil will flow at the constant shear stress, volume, effective stress, and velocity conditions mentioned above (Castro and Poulos 1977). They noted that the axes of the plot would include shear stress (τ), void ratio (e), and minor effective principle stress (σ_3') in 3-D space, but could also be plotted in a pair of 2-D plots with one common axes. Examples of these relationships are presented as Figure 2.3. They suggested that the steady-state line concept can be used to evaluate the soil response during undrained monotonic and cyclic shear by considering the position of the in-situ soil relative to the SSL. Soils that plot above the line would tend to contract and develop positive pore pressures when exposed to undrained monotonic and cyclic shear, resulting in a reduction in the effective stress and possible large strains. If the initial soil conditions plot below the SSL, the sample will tend to dilate during monotonic undrained shear, resulting in a decrease in the pore water pressure and a subsequent increase in effective stress in the soil. However, soils in these conditions may develop positive pore water pressures during undrained cyclic shear, which can decrease the effective stress in the soil and possibly result in momentary points of zero effective stress and large strains.

Several examples of the relationship between the in-situ void ratio and the soil behavior undrained shear are presented in Figure 2.3. Two of the samples (X_1 and X_2) are located at the same void ratio but at different confining stress levels. The third sample (X_3) is tested at the same confining stress as X_1 but at a much higher void ratio. Both static and cyclic loading conditions are considered in the analysis.

As shown in Figure 2.3, the soils at points X_{1A} and X_2 move towards the same point on the steady-state diagram (Point A), suggesting that the steady-state shear strength (Su_{s2}) is the same for both samples. The post peak behavior of the soil under load was different for the two samples, as shown through the dilative behavior of X_{1A} and the contractive behavior of X_2. The soil at X_{1A} also exhibits a different behavior for the two types of loading, as shown through the strain hardening during the monotonic loading and the cyclic softening during the cyclic loading. For both cases of loading of the soil at X_{1A}, the dense state of the soil and the absence of shear stress reversal during the cyclic loading prohibited post peak softening. For soils at X_2, however, both the monotonic and cyclic loading resulted in a large post peak strength loss and large deformations.

The soil condition at X_1 was also consolidated to an isotropic stress state (X_{1B}) and exposed to a cyclic shear stress that was large enough in magnitude and duration to cause an extended shear stress reversal, resulting in conditions where the soil reached momentary points of zero effective stress. The soil generally exhibits periods of large deformations when the soil is exposed to periods of zero effective stress, but when the cyclic loading stops the soil reaches an equilibrium condition and the deformations cease.

The third soil (X_3) included in the figure was tested at the same confining stress as X_1 but at a much looser condition. Unlike sample X_1, the undrained loading of this sample exhibited a highly contractive behavior for the monotonic loading case. This behavior is further identified through the path of the confining stress in the soil, which suggests that the soil exhibits a decrease in effective stress and subsequent strength loss at failure.

As noted in the previous section and shown through Figure 2.3, the steady-state line may be used to identify the potential of a soil deposit to exhibit a large strength loss during shear. Soils located above the SSL exhibit the tendency to collapse upon loading, which may result in large deformations and/or potentially unstable ground conditions. Soils at these states behave independent of the type of loading, resulting in the same undrained shear strength whether loaded monotonically or cyclically. Soils located below the SSL, however, will generally exhibit a tendency to dilate when loaded monotonically. They will exhibit periods of significant deformation only if the load is cyclic, is large enough to generate a shear stress reversal, and is substantial in duration.

2.4 The Phenomena of Liquefaction

One of the most significant factors leading to ground failure during earthquakes is the liquefaction of loose and medium dense saturated sands. *Soil liquefaction* is generally described as the rapid drop in the shear strength of a soil during a loading event, most notably due to the buildup of excess pore water pressure in the soil during the loading (Castro 1982). The loading process can be cyclic, as in the case of an earthquake, or monotonic, as in the case of a slope failure. Hazen (1920) was the first to relate this large strength loss to "liquefiable" behavior, while Terzaghi (1925) was the first to coin the term *liquefaction*, where he stated:

> "Liquefaction can occur if saturated soil collapses, resulting in a transfer of the collapsed weight of the solid particles to the surrounding water. As a consequence, the hydrostatic water pressure at any depth increases, which is then close to the submerged unit weight of the soil." (Castro 1969)

Terzaghi (1925) originally introduced the term *liquefaction* into the engineering community in the classical book *Erdbaumechanik* (Castro 1969), and in 1936 Casagrande (1936) used the term to explain the massive soil failures at the Fort Peck Dam. The concept of liquefaction gathered worldwide attention in the 1960's, when in 1964 large magnitude earthquakes located near Anchorage, Alaska and Niigata, Japan caused massive structural damage through soil failure. An abundance of work has been performed in the 35 years since these earthquakes, resulting in several state-of-the-art papers relating to liquefaction evaluation (e.g. Seed and Idriss 1971; Seed 1979; Robertson and Campanella 1985; Mitchell and Tseng 1990; Ishihara 1993; Robertson and Fear 1995a, 1995b; Youd and Idriss 1997). Although much was learned, there seems to be a continual discrepancy in the literature relating to a standard definition of and an evaluation procedure for liquefaction. Although numerous definitions of liquefaction were noted, they all seem to originate from one of two main schools of thought:

- The cyclic liquefaction approach proposed by Seed and his co-workers.
- The critical void ratio and steady-state approach proposed by Casagrande and his co-workers.

The actual behavior of pore water during undrained conditions was not measured in the laboratory until the late 1950's, where monotonic undrained triaxial tests were performed by the Corps of Engineers on very loose soils for liquefaction characterization studies. Seed and Lee (1966) first introduced constant volume cyclic triaxial tests into the laboratory in the mid 1960's, from which they evaluated pore pressure and effective stress relationships in different density sands during dynamic loading. The results of their investigation revealed a cumulative increase in pore water pressure during cyclic loading in samples of a wide range densities. This cumulative increase in pore water pressure continued to a point where the pore water pressure equaled the confining pressure, indicating conditions where the effective stress in the soil reached a value of zero. This point of zero effective stress was termed the point of *initial liquefaction*, which they suggested was dependent on the confining pressure, the initial principal stress ratio, and the magnitude of the deviatoric load applied to the sample (Seed and Lee 1966). As the cyclic loading continued, large strains developed in the loose samples with very few additional loads. In dense samples, however, this large strain behavior was only noted after 1000's of cycles. This large strain behavior was termed *complete liquefaction*, most likely due to the large strains associated with the deformation.

As shown through the two definitions, the term *liquefaction* was associated with the point during the loading/unloading cycle when the soil reached a point of zero effective stress. The concept was based on the test results from isotropically consolidated cyclic triaxial tests, which always involved a shear stress reversal during the load/unload cycles of the test and subsequent points where the effective stress is equal to zero. As noted by Castro et al. (1982), however, soils tested under anisotropic conditions did not always have a shear stress reversal during the loading. Soils tested under these conditions may exhibit a large post peak strength loss but did not exhibit periods where the effective stress in the soil equaled zero. According to the definitions of liquefaction presented by Seed and his co-workers, the soil did not liquefy even though a large post peak strength loss was observed.

Casagrande (1936) observed liquefaction related behavior in the laboratory through a series of simple experiments involving the placement of a weight on the surface of a very loose saturated sand soil and the penetration of a wooden rod into the sand adjacent to the weight. The addition of the rod into the sand resulted in a collapse of the sand adjacent to the rod, which quickly propagated laterally and vertically throughout the soil mass. As the soil collapsed, the pore pressures in the sand increased, resulting in a reduction of the

frictional resistance of the soil supporting the weight and the submergence of the weight into the sand. As Casagrande (1936) noted, the small stress changes imparted onto the initially metastable soil structure through the penetration of the rod was significant enough to ultimately cause collapse of the structure.

Castro (1969), under Casagrande's guidance, further investigated the mechanics of liquefaction through an extensive laboratory study involving constant volume cyclic and monotonic stress-controlled triaxial tests on loose and dense sands. He observed large post peak strength loss and subsequent positive pore pressures during undrained shear of soils above the CVL, while strength increases and decreasing pore pressures were observed for soils below the CVL. He used this behavior to develop the concept of *steady-state*, which he later defined as the *residual strength* of the soil that is present after rapid post peak strength loss during undrained shear (Castro 1975).

Castro (1969) also noted that dense sands exhibited positive pore pressure generation at small strains when loaded cyclically, which directly confirmed the concept of *cyclic mobility* originally proposed by Casagrande (1936). The accumulation of these small increments of positive pore pressure eventually brought the soil to momentary points of zero effective stress. However, these samples quickly mobilized strength through dilation and did not exhibit any signs of flow behavior. It was suggested by Castro (1969) that these momentary lapses of high pore pressures were attributed to redistribution of the void ratio and water contents in the sample and were not representative of field behavior. This principle of void ratio redistribution was further addressed and confirmed by Casagrande (1976) through a series of laboratory gyratory/cyclic simple shear tests. It was at this time that Casagrande introduced the term *cyclic liquefaction*, which was proposed to define the points during a laboratory test when void ratio redistribution and soil softening allow for conditions where cyclic pore pressure is equal to the effective confining pressure.

Castro (1975) continued his laboratory investigations related to the behavior of soils during cyclic loading and in 1975 redefined the definition of *cyclic mobility*. He used the definition of cyclic mobility to describe the process of a cumulative increase in pore pressure during cyclic loading, eventually reaching a point of minimal shear resistance. At this point the soil exhibits strain, which ultimately results in a shear resistance increase and pore pressure decrease. He associated this behavior to that of dense soils and emphasized that the void ratio redistribution and sampling inadequacies associated with

laboratory testing in dense materials can result in conservative estimations of the field liquefaction potential.

Robertson (1994) summarized the liquefaction-related terminology developed by Casagrande and Castro through a classification scheme that incorporates both the type of loading and the stress/strain behavior of the soil. A flow chart for each of the definitions is presented as Figure 2.4. The definitions are related to critical void ratio and steady-state principles and are summarized as follows:

- *Flow Liquefaction* occurs when a saturated and contractive soil exhibits a strain softening response to monotonic or cyclic undrained loading. For flow liquefaction to occur the in-situ shear stress imparted onto the soil must be greater than the undrained shear strength of the soil.

- *Cyclic Softening* refers to the buildup of excess pore pressure during cyclic loading in soils that would normally dilate during monotonic loading. The pore pressure increase implied through this definition occurs prior to the soil reaching its limiting strain condition. The post limiting strain behavior of the soil is dependent on the magnitude of the initial shear stress relative to the applied shear stress and whether or not the applied shear stress is large enough to cause a shear stress reversal in the soil. The post limiting strain behavior is described in two manners:

    - ✓ *Cyclic Liquefaction* occurs when the cyclic shear stress imparted onto the soil is large enough to generate a shear stress reversal. Conditions of zero effective stress may develop, resulting in large deformations. Deformation can be large during the cyclic loading, but generally stabilizes once the cyclic loading is stopped.

    - ✓ *Cyclic Mobility* occurs when the cyclic shear stress imparted onto the soil is not large enough to generate a shear stress reversal. Conditions of zero effective stress do not develop and deformation is generally small.

The definitions of flow liquefaction, cyclic softening, cyclic liquefaction, and cyclic mobility are all related to the pre and post peak strain behavior of the soil. Each of the conditions is located on the soil behavior diagrams presented as Figure 2.1 and 2.2. The definitions are all based on undrained conditions during the loading where the pore pressure incrementally increases prior to failure. If limiting strain conditions are reached,

the soil will exhibit either strain hardening or strain softening with added cycles of loading. If the load increment applied to the soil is large enough, the soil can undergo large increments of strain.

Several studies related to the cyclic loading of soils in the laboratory were reviewed as part of this study. Although most of the researchers identified similar soil responses for a given testing condition, the terminology used to describe the soil behavior during the loading was different. The multi-tiered approach presented by Robertson (1994) incorporates the pre and post peak soil behavior into specific definitions for liquefaction, and has been adopted by the 1997 NCEER committee as an adequate means for identifying the soil response to loading. As such, the definitions of liquefaction presented by Robertson (1994) were adopted for this study.

2.5 Dynamic Loading of Soils in the Laboratory

A considerable amount of work has been performed over the last 35 years related to the phenomena of liquefaction. Much of this work involved detailed testing on reconstituted samples in the laboratory, mainly using cyclic triaxial, direct simple shear, and torsional shear testing devices. Included in this section is an outline of several of the developments related to the dynamic loading of soils in the laboratory. A discussion of the advantages and disadvantages of using the lab based responses to estimate the in-situ soil behavior during dynamic loading is also included.

Seed and Lee (1966) were the first to expand on the critical void ratio concept defined by Casagrande (1936) through investigation on the behavior of saturated sands during undrained cyclic triaxial loading. They concluded that the behavior of the soil during dynamic loading was dependent on the initial void ratio, confining pressure, magnitude of cyclic stress, and the number of stress cycles exposed to the sample. They revealed that the pore water pressure accumulated during cyclic loading in liquefiable soils until a zero effective stress condition was reached, at which time large strains developed. They also observed this accumulation of pore water pressure in dense samples, but did not identify large strains after zero effective stress conditions were reached.

Castro (1969; 1975) and Castro and Christian (1976) furthered the understanding of the behavior of saturated clean sands when loaded cyclically in the laboratory. The

investigations involved constant volume, cyclic and monotonic triaxial tests on loose and dense sands, resulting in the confirmation of the concepts of *flow structure*, *flow liquefaction*, and *cyclic mobility* developed by Casagrande. They also introduced the concept of the *steady-state*, which was outlined in detail in Section 2.3. Later investigations by Castro and Christian (1976) and Castro and Poulos (1977) resulted in the rebuttal of the concept *cyclic mobility*, suggesting the phenomena was primarily due to the presence of void ratio distribution in the lab samples and was not indicative of actual field behavior.

Finn et al. (1971) performed a detailed study on the behavior of cohesionless soils during dynamic loading using simple shear and cyclic triaxial testing devices. The investigation involved direct comparison of the test results from both types of testing at different confining stresses. They indicated that the shear stress needed to cause post-peak strength loss increased as the confining stress increased for a given number of cycles, and further concluded that the ratio of the peak cyclic shear stress to the initial mean effective normal stress (τ_{cyc}/σ_m') was uniquely related to the number of cycles to failure. They noted that the sample was exposed to different stress conditions for the two testing apparatuses, but the τ_{cyc}/σ_m' ratio was the same for both apparatuses at a given number of cycles.

Seed and Peacock (1971) also evaluated the ability of simple shear and triaxial compression tests to evaluate the behavior of saturated cohesionless soils during cyclic loading. They investigated the limitations of the testing equipment and procedures, the different stress conditions present during the test, and the deviation of the soil fabric and loading stress path from the field conditions. This resulted in the development of a correction factor that related laboratory to field data that was applied to the τ_{cyc}/σ_m' ratios noted by Finn et al. (1971). The correction factors ranged in value from 0.55 to 0.7 for relative densities between 40% and 85%.

Martin et al. (1975) performed simple shear tests in saturated sands to evaluate the progressive pore water pressure increase during undrained cyclic loading. The approach resulted in the development of a theoretical model for estimating the pore pressure buildup during loading based on the effective stress parameters of the sand. The investigation revealed that volume changes during cyclic loading are dependent on cyclic shear strain rather than shear stress amplitudes, and that changes in volume were

independent of the vertical stress and frequency used in the testing (0.2 Hz to 2 Hz) (Martin et al. 1975). Similar test results were noted by Youd (1972).

Mitchell et al. (1976) conducted an investigation related to the effects of sample fabric on the undrained strength and volume change capabilities of saturated soils exposed to cyclic triaxial loading. Different sample preparation methods were considered in the study, each of which generated different soil responses during the cyclic loading. They indicated that samples prepared through dry pluviation resulted in an orientation of the particles that generated the lowest strength of all of the sample preparation methods tested.

Cyclic triaxial tests were performed by Singh et al. (1979) on 'undisturbed samples' to evaluate the effects of different sampling methods on the initial void ratio of clean sands. The techniques of block, push tube, and freezing were all evaluated for sand at a relative density of approximately 60%. They revealed that both the block and tube sampling generated a large enough density change in the soil to modify the cyclic strength by 15%. They also noted that soils tested in the laboratory were not allowed to age, did not exhibit the same fabric as in-situ soils, and exhibited a lower strength and sensitivity to disturbance than in-situ soils. They suggested these effects may be considerable, and noted fluctuations of soil strength of 75% to 100% for some sand types. Similar qualitative results were reported by Seed and De Alba (1986).

Castro et al. (1982) investigated the application of steady-state principles to the concept of liquefaction through an investigation involving cyclic triaxial tests in the laboratory. The influence of the stress history, loading stress paths, and grain size on the steady-state value were all considered in the investigation. They indicated that the steady-state line is independent of the stress history and loading stress paths and is unique for a given soil type. They noted that the slope and position of the steady-state line determined in the laboratory is effected by the angularity and grain size distribution of the soil, respectively. An empirical procedure was recommended to correct the lab based steady-state line to that representative of field conditions.

An evaluation of the steady-state procedure for evaluating the liquefaction potential of soil deposits was performed by Kramer (1989). The analysis confirmed the work of Castro et al. (1982) by providing statistical evidence that the slope and position of the steady-state line are dependent on soil properties. Kramer (1989) further suggested that

these soil properties can be variable within a soil deposit, and revealed through a statistical and probabilistic analysis that this variability may add a significant uncertainty to the shear strength estimated through the steady-state procedure. He noted that this soil variability can add uncertainty to the overall liquefaction potential evaluation and proposed correction factors to modify the position of the steady-state line.

Sladen and Handford (1987) identified a source of systematic error present during the determination of the steady-state parameters for sands. They suggested that a significant amount of specimen densification occurs during the saturation and backpressure application stages of the test, which can result in void ratio changes as much as 0.15 in very loose materials. This volumetric change in the soil sample can significantly influence the slope and position of the steady state line, resulting in errors in the undrained strength estimations and unconservatism in liquefaction evaluations. A correction to the steady-state line determined from conventional measures was therefore recommended.

Vaid and Thomas (1995) performed an experimental study of the behavior of saturated sand during monotonic and dynamic loading. The investigation involved compression and extension load tests using a cyclic triaxial testing apparatus. Test results revealed a dilative response of the soil to compressional loading for all densities and stresses considered, but identified a contractive behavior of the soil during triaxial extension at relative densities less than 60%. The observed response to undrained loading is contradictory to the results of the works of several of the investigations previously mentioned. The test results suggest serious implications for the use of steady-state concepts for design, which usually considers a steady-state line uniquely determined from compression tests (Vaid and Thomas 1995). Similar conclusions were presented by Negussey and Islam (1994).

Porter (1998) performed an experimental and analytical study to examine the validity of steady-state shear strength estimations obtained from conventional ICU triaxial tests. The results of the investigation revealed that the measured response at large strains during the test may be inaccurate due to testing errors and therefore may not accurately represent in-situ material behavior at large strains.

Based on the investigations presented in the preceding paragraphs, the following general conclusions were made regarding the use of laboratory testing to determine the in-situ behavior of soils under dynamic loading:

- The sample preparation method can effect the soil structure of the specimen, which can significantly alter the response to the loading.

- The SSL is generally determined from triaxial compression tests. It was noted that the procedures associated with this test may result in estimations of the undrained steady state strength that are not representative of field conditions.

- Discrepancies were noted in the literature regarding the magnitude of the correction factors needed to correlate test results from different testing apparatuses. These correction factors are applied to the test results because the sample follows different stress path to failure for each of the different testing units, suggesting that the strength estimation in the laboratory is dependent on the stress path to failure.

As noted through these considerations, the response of the soil to dynamic loading may fluctuate with the sample preparation and test procedure, soil fabric, load type, load stress path, load history, and sample compositional variability. A testing program that measures the soil response to dynamic loading in-situ may remove some of the uncertainties associated with the use of laboratory testing to estimate in-situ soil behavior.

2.6 Cone Penetration Testing in Sands

2.6.1 General

In-situ testing has played a major role in evaluating the engineering characteristics of cohesionless soils for decades. The major surge of these in-situ evaluations is primarily due to the non-uniformity of natural sand deposits and the difficult and expensive tasks of obtaining high quality undisturbed samples for laboratory testing. The cone penetration tests (CPT) is one such in-situ testing technique, and is often used in static load problems and settlement analyses, allowable bearing pressure determinations, and density and friction angle estimations. More recently, however, the cone penetration test (*CPT*) has gained worldwide attention for determining the liquefaction potential of cohesionless

soils for earthquake engineering purposes (Seed and De Alba 1986). In particular, the CPT is used extensively to estimate the response of soil during dynamic loading through empirical penetration resistance versus liquefaction potential correlations.

The cone penetration test has been used to evaluate the in-situ properties of soils for over 50 years. Schmertmann (1978) and Robertson and Campanella (1983) performed detailed evaluations of the cone penetrometer that resulted in the early guidelines and recommendations for use. The recommended procedure involves pushing a cone with a standard geometry into the ground at a rate of penetration of 2 cm/sec. Cones with a diameter of 35.8mm and a conical tip area of 10-cm^2 are generally accepted as industry standard, although cones 45.8mm in diameter with a tip area of 15-cm^2 are also used (Schmertmann 1978). Standard cone penetrometers have load cells or strain gauges at the tip and sleeve locations, although additional sensors such as pore pressure transducers, inclinometers, and accelerometers were added in recent years to provide additional soil information.

The penetration resistance of a soil deposit can be measured directly or estimated using theoretical or empirical relationships. Presented in the following subsection is a general review of the different approaches for *estimating* the penetration resistance value. Also included in this section is a general discussion of the factors that control the penetration resistance, as determined through calibration chamber testing.

2.6.2 Penetration Resistance Estimations

Four general approaches are commonly used to estimate the cone penetration resistance of sands: (1) bearing capacity, (2) cavity expansion theory, (3) finite element analysis, and (4) calibration chamber testing. A complete discussion of each of these methods has been presented in Yu and Mitchell (1996) and Mitchell and Brandon (1998). An outline of these reviews is presented in the following paragraphs.

The initial approach for estimating the penetration resistance of a soil deposit was based on bearing capacity theory. The approach assumes that the cone penetration resistance is equal to the failure load of a deep foundation. The magnitude of this load is defined through either limit equilibrium or slip line methods, which are based on either conditions of global equilibrium or conditions of equilibrium and assumed failure criterion. For sands, numerical expressions and chart solutions were developed for the bearing capacity factor, which is multiplied by the overburden stress to determine the

penetration resistance value (e.g. Durgunoglu and Mitchell 1974; 1975). These bearing capacity factors are dependent on the magnitude of the friction angle for the limit equilibrium approach and the friction angle and the geometry of the slip surface for the slip line method.

The cavity expansion approach for estimating the penetration resistance assumes that the pressure required to advance a cone into a soil mass is equal to the pressure needed to expand a cavity from a finite radius to the radius of the cone penetrometer. The approach requires the computation of the *limit pressure* during the 'expansion' of the cavity, which is in turn related to the penetration resistance. A recent approach developed by Salgado et al. (1997; 1998) involves the computation of the limit pressure in the soil through an iterative technique, considering both the constitutive behavior of the soil within the zone of influence of the penetrometer and the rotation of the stresses and strain fields generated during penetration. The rational of this technique is incorporated into the computer program CONPOINT, which can be used to estimate the penetration resistance of a soil mass of any lateral dimension and boundary condition.

A limited number of investigations were identified that involve the estimation of the cone penetration resistance in sands using finite element methods. The few investigations that are available assume large strain analyses, and often result in numerical difficulties that result in an inaccurate cone factor, which in turn can generate errors in the estimation of the penetration resistance (Mitchell and Brandon 1998).

Calibration chambers have been used for over thirty years to determine empirical correlations between testing parameters and the properties of the soil. The most common correlative relationships relate penetration resistance to the peak friction angle (e.g. Houlsby and Hitchmann 1988) and relative density (e.g. Schmertman 1976), although relationships with other parameters also exist. Much was learned from this testing about the factors that control penetration resistance, mainly because the testing conditions are known prior to the penetration of the cone. Several general relationships between the testing conditions and the penetration resistance values were identified through the literature review:

- The penetration resistance is highly dependent on the initial lateral effective stress in the soil and virtually independent of the magnitude of the initial vertical effective stress.

- The penetration resistance is highly dependent on the relative density of the soil, which is an indirect measure of the drained friction angle.

- The penetration resistance is dependent on the fabric, compressibility, Over Consolidation Ratio (*OCR*), stiffness, and age of the soil deposit.

- The penetration resistance measured in calibration chambers may be influenced by the type of boundary condition as well as the size of the chamber relative to the cone penetrometer.

The first calibration chamber used for geotechnical engineering purposes was built in 1969 by the Country Roads Board (*CRB*) in Melbourne, Australia to evaluate the performance of a full-size cone penetrometer under conditions of controlled stress, strain, and soil properties (Holden 1991). The only large-scale CPT testing noted prior to this point was in rigid wall test pits in Germany and France where the lateral strains were restricted and the boundary stresses uncontrolled (Chapman 1974).

A number of large-scale calibration chambers were built around the world by various research institutions for the purpose of generating correlative relationships between CPT measurements and engineering parameters. A complete list of the existing large-scale calibration chambers, along with their sample size and boundary conditions, is included in Lunne et al. (1997). The design and setup for each of the chambers are based on the initial design of the CRB chamber, with modifications mainly directed at the sample size and the boundary condition present. Presented as Figure 2.5a. is a comparison of the penetration resistance values measured at different calibration chamber diameter to cone diameter ratios (D_{cc}/d_{cp}) during tests in loose ($D_r = 30\%$) and dense ($D_r = 90\%$) sands. Included as Figure 2.5b are the different boundary conditions present in the tests. As shown by the figures, both the type of boundary condition and the magnitude of the D_{cc}/d_{cp} ratio can significantly affect the measured penetration resistance value, especially for penetration tests in dense sands at low D_{cc}/d_{cp} ratios.

A number of investigations were identified in the literature that relate penetration resistance to an engineering design parameter (e.g. friction angle, relative density, OCR, in-situ horizontal effective stress, Young's modulus, constrained modulus, small strain shear modulus, etc.). The bulk of these correlations came from calibration chamber testing, and most of these relationships were established empirically. The reader is

referred to Lunne et al (1997) for a listing of such correlations. However, one such correlation relating penetration resistance to relative density was developed by Kulhawy and Mayne (1990) from an extensive set of calibration test data. The chamber used in this investigation simulated flexible wall (*BC1*) conditions. Issues related to the compressibility, OCR, and age of the soil were all considered through empirical factors. A similar relationship was developed from the same data set by Kulhawy and Mayne (1990) that allows for the estimation of the horizontal effective stress if both the relative density and the penetration resistance value are known. Both of these relationships are used in Section 5.12 of Chapter 5 to evaluate the penetration resistance values measured as part of this study.

The bulk of the calibration chamber studies presented in the literature recognize that the boundary conditions present during the testing may influence the magnitude of the measured penetration resistance value. The problem of the influence of the calibration chamber size on the penetration resistance measurement has been extensively addressed in the literature (e.g. Parkin and Lunne 1981; Jamiolkowski et al. 1985; Baldi et al. 1986; Ghionna and Jamiolkowski 1991; Schnaid and Houlsby 1991; Lunne et al. 1997; Salgado et al. 1998). Although discrepancies existed between the studies in the actual quantified relationships, the following general behaviors were noted:

- Penetration tests performed at low D_{cc}/d_{cp} ratios have a lower penetration resistance than at higher D_{cc}/d_{cp} ratios in constant stress boundary conditions (*BC1*). Opposite results were noted at boundary conditions corresponding to zero lateral strain (*BC3*).

- The effect of the boundary condition on the penetration resistance measurement is more pronounced as the relative density of the soil increases.

- The effect of the boundary condition on the penetration resistance measurement increases as the effective stress in the soil increases.

- Boundary effects were less pronounced as the compressibility of the soil increased.

Four approaches were presented for estimating the penetration resistance of a soil mass. The three theoretical approaches have shown favorable results, provided the slip surface (bearing capacity), the limit pressure and constitutive properties of the soil (cavity expansion theory), and the strain estimations (finite element methods) are accurately

estimated. The empirical correlations determined from calibration chamber testing seem to provide an accurate way of estimating field conditions, provided the boundary conditions, soil gradation and compressibility, stress and strain levels, and relative density of the soil are accurately identified during the test.

2.7 Measurement of Pore Pressure in Sands Through Large Scale Testing

2.7.1 During Cone Penetration Testing

The magnitude and fluctuation of the water pressure generated during cone penetration is measured through pore pressure transducers on the cone face, at the cone shoulder, and/or at the sleeve location. Schmertmann (1974a, 1974b) first noted the need for the measurement of water pressure during penetration to help explain why higher penetration resistance values were recorded in tests in dry samples versus saturated samples. He also observed increases in penetration resistance as the penetration rate was decreased. He hypothesized that the penetrating cone generated zones of elevated pore water pressures below and away from the penetrometer in the saturated samples, which affected the stress conditions in the soil and corresponding penetration resistance value.

A series of calibration chamber studies involving cone penetration testing and pore pressure development were performed at the University of Florida in the 1970's under the tutelage of Professor John Schmertmann (Caillemer 1975; Reese 1975; Lhuer 1976; Bunnell 1978). Caillemer (1975) evaluated the pore pressure distribution around a penetrating cone through a series of calibration chamber tests in loose and dense sands. Transducers were placed in the soil during pluviation and prior to saturation to measure the magnitude of the pore water pressure in the soil during penetration of the cone penetrometer. A 10-cm^2 Fugro cone was used sparingly in the testing that had a pore pressure transducer at the shoulder location. An examination of the test data revealed very small increases in water pressure at the imbedded transducer location when the cone penetrated into the loose soils. Negative pore pressure values were noted at the imbedded transducer location during penetration tests in dense samples when the cone was considerable distances away from the transducer. As the cone approached the transducer, however, these negative values turned positive, revealing a zone of compression directly beneath and adjacent to the cone penetrometer. Tests performed in loose and medium dense sands with the Fugro cone revealed pore pressure values that were significantly less than those values recorded at the imbedded transducer.

Lhuer (1976) performed a series of cone penetration tests in dry and saturated fine sands to evaluate pore pressure development around an advancing cone. The effects of saturation on the measured penetration resistance values were also examined. The pore water pressure was measured on the penetrating cone at the shoulder location and in the soil mass through several pore pressure transducers. He indicated that the saturation of the sand had no effect on the penetration resistance values. The test results also revealed a small positive excess pore pressure at the cone and in the soil mass during penetration in loose sands, while negative values were noted at the higher densities. The zone of compression noted by Caillemer (1975) was not observed for tests in dense samples. Further inspection of the test data revealed that the pore pressure measured by the cone penetrometer was always less than the value recorded at any of the transducers imbedded in the soil mass.

Bunnell (1978) attempted to gain a further understanding of the pore pressure generated by an advancing cone by performing penetration tests in loose and dense saturated samples with a Wissa cone. The pore pressure transducer was located on the Wissa cone at the tip location. Numerous pore pressure transducers were also added to the soil sample at different elevations and lateral distances away from the cone penetration path to supplement the information provided by the cone penetrometer. Test results reveal positive pore pressures at both the penetrometer tip and away from the penetrometer for penetration tests in loose sands. Negative pore pressures were recorded away from the penetrometer in penetration tests in dense sands, although positive values were noted at the cone tip location. The vertical and lateral zone of compression noted by Caillemer (1975) was not observed for tests in dense samples. Contrary to the previously noted investigation, the pore pressure values recorded on the penetrometer were always greater than those recorded below and away from the penetrometer.

Filho (1979; 1982) performed an experimental laboratory study to evaluate the influence of excess pore pressure on the penetration resistance measurement and the implication of this pore pressure value on the existing penetration resistance correlations. Penetration tests were performed in loose and medium dense sands of variable fines content with a cone penetrometer that had a pore pressure transducer on the cone face. He noted that the penetration resistance measured in loose dry samples was considerably greater than that in saturated samples, while the reverse was true for dense samples. He attributed this to the development of pore pressures around the penetrating cone, and suggested that the existing correlations developed from data bases of clean, dry sands are

not valid for soils that deviate in gradation, compressibility, and degree of saturation from those considered in the correlative database (Filho 1982).

Last (1982), in a detailed theoretical and experimental analysis of cone penetration tests in granular soils, indicated that tests performed in dry samples exhibited a 25% higher penetration resistance than those performed in equivalent saturated samples. He also indicated that the penetration resistance values estimated using conventional bearing capacity techniques were approximately equal to that measured during the dry tests. He concluded that this difference between the dry and saturated test results was attributed to the development of excess pore pressures around the penetrometer tip, but was not able to verify this conclusion due to the absence of pore pressure measurements during the testing.

Tumay et al. (1981) performed a series of field penetration tests in a fine-grained sandy soil to evaluate the pore pressure values recorded from cone penetrometers of different size and shape. One of the penetrometers used in the testing was identical to the Fugro 15-cm^2 cone used in this study. They concluded that the pore pressure generated during penetration was dependent on the cone shape, with the lowest induced pore pressure and subsequent highest penetration resistance generated by the lowest angled tips (Tumay et al. 1981).

Robertson and Campanella (1983) provided a general guide for interpreting CPT data. They noted that the geometrical design of the standard cone penetrometer results in an ambient water pressure that acts normal to the shoulder to the cone and opposite the force applied to the tip of the cone. They referred to the influence of this water pressure as the *unequal end area effect*, and suggested that it can influence the penetration resistance value when the water pressure is high or the penetration resistance is low. The magnitude of the unequal end area effect is determined through a simple calibration in the laboratory, which is in turn used to correct the penetration resistance measurement. A further discussions of this correction is included in Section 3.5.1 of Chapter 3.

A number of investigations using the piezocone penetrometer were reported at the Second European Symposium on Penetration Testing in 1982 (e.g. Ruiter 1982; Jones and Rust; Torstensson 1982). All of these investigations were simultaneously able to measure tip resistance, sleeve friction, and pore water pressure during penetration of the

penetrometer. The correlations generated from these investigations mainly related one of the three cone measurements to the grain size of the soil.

Bruzzi and Battaglio (1987) performed field penetration tests in different soils with different cone penetrometers to evaluate the effects of model type, transducer location, and filter material on the penetration resistance and pore pressure measurements. They noted that the degree of saturation of the filter and transducer housing significantly influences the magnitude and response time of the pore pressure measurement, and suggested that measurements obtained from unsaturated cones would not accurately portray the underlying stratigraphy or the proper magnitude of the induced pore pressure. They indicated that axial loads oriented directly onto the filter element could influence the pore pressure measurement. They further identified that wear, abrasion, and clogging of the filter material may result in a reduction of the filter permeability, which could affect the magnitude and response of the pore water pressure. Penetration test results also revealed that the magnitude of the pore pressure measurement is greatest at the penetrometer tip and reduces as the measurement location moves toward the friction sleeve.

2.7.2 During Field Investigations

Baez (1995) evaluated different design configurations of stone columns for liquefaction reduction purposes. Issues related to densification, drainage, and stress redistribution were all considered as part of the study. A Keller "S" type vibrator operating at a centrifugal force of 176 kN and a frequency of 30.5 Hz was used for the project. The investigation involved placing pore pressure transducers and geophones into the ground adjacent to the proposed stone column location and then recording the output from the instruments during the vibration portion of the construction. The instruments were placed 4.4m into the ground and 1m, 1.5m, and 2.2m away from the centerline of the penetrating probe. The results of the investigation revealed elevated pore water pressures at defined distances laterally and vertically away from the probe. Periods where the pore pressure ratio ($\Delta u/\sigma_v'$) equaled a magnitude of one were noted at several points during the test at the 1m and 1.5m measurement locations. This behavior was particularly noted when the vibrator continued after the penetration was stopped. Periods where the pore pressure equaled the vertical effective stress were also observed at these locations when the vibratory unit approached the depth of the imbedded transducer and when the vibrator was allowed to run at the end of penetration. An acceleration of 1.7g

was noted at the 1m measurement location, which decreased rapidly to a value of 0.6g at the 2.2m location.

Moller and Bergdahl (1981) evaluated the dynamic pore pressure generated in dense sands during the driving of mini piles in a testing box. Pore pressure transducers were fitted on the pile tip and placed in the sand 10mm from the pile. They reported zones of slightly positive pore water pressure at the pile tip during the driving of the pile, which turned negative and then approached hydrostatic in a very short period of time after the vibration stopped (within milliseconds). Elevated water pressures were not observed during static penetration of the piles.

Nogami et al. (1997) performed a numerical analysis of the pore pressure generated in a cohesionless soil during the vertical vibration of a pile. They noted that the soil around the pile shaft fits the conditions of the *Winkler model* when the frequency of vibration is greater than the natural frequency of the soil, and modified this model to account for the influences of excess pore water pressure on the shear modulus and the interface shear strength. Numerical outputs from the model indicate that shear strain at the soil/pile interface increases as the vibration frequency is increased. The shear strain laterally away from the pile dissipates faster as the frequency of vibration increases, suggesting that slippage at the soil/pile interface hampers the magnitude of shear force transmitted into the soil and the corresponding induced pore water pressure.

A literature review was performed to provide information regarding the pore pressures generated around a statically penetrating cone penetometer. Information related to the pore pressure generated in a soil mass during vibratory pile driving was also reviewed. It appears that the existing calibration chamber investigations report mixed results regarding the magnitude of the pore pressure generated at the soil/cone interface during penetration. Discrepancies also exist in the magnitude of the pore pressure in the soil mass away from the penetrating cone. An elevated pore water pressure was measured on a pile tip during vibratory penetration of the pile. These elevated pore water pressures dissipated rapidly after the vibration ceased. Elevated water pressures were also observed in the soil laterally and vertically away from a vibrating pile that approached levels where the mean effective stress in the soil was zero. The magnitude of these elevated pore water pressures was dependent on the proximity of the measurement location to the vibrating unit and the duration of the vibratory loading. Estimations of the magnitude of the pore pressure using a numerical approach revealed similar results, and

further suggested that the induced pore pressure is a function of the frequency of vibration and the strain generated in the soil through the dynamic loading.

2.8 The Use of the CPT to Evaluate the Behavior of Soils During Dynamic Loading

2.8.1 Static Penetration Approaches

The initial approach for evaluating behavior of soils in the field during dynamic loading was developed by Seed and Idriss (1971). The procedure is referred to as the *simplified procedure*, and involves the comparison of the seismic stresses imparted onto a soil mass during an earthquake (*CSR*) to the resistance of the soil to large magnitude strain and strength loss (*CRR*). The CSR estimation is based on the estimated ground accelerations generated by an earthquake, the stress conditions present in the soil, and correction factors accounting for the flexibility of the soil mass (Youd and Idriss 1997). An expression for the CSR is given by the following:

$$CSR = \tau_{av} / \sigma_v' = 0.65 \cdot \frac{a_{\max}}{g} \cdot \frac{\sigma_v}{\sigma_v'} \cdot r_d \tag{2.1}$$

where τ_{av} is the equivalent cyclic shear stress, σ_v is the total overburden pressure, a_{max} is the peak horizontal ground acceleration, g is the acceleration of gravity, σ_v' is the effective overburden pressure, and r_d is a stress reduction factor that accounts for the non-rigidity of the soil mass.

The value of the CSR computed through Equation 2.1 is compared to the resistance of the soil to large magnitude shear induced strength loss (*CRR*). This CRR value was initially approximated through standard penetration test values (N_{spt}), but work performed by Seed at al. (1983) allowed for CRR estimations using CPT data through a standard q_c/N_{spt} conversion factor developed by Schmertman (1978). This conversion factor resulted in liquefaction evaluation curves for a clean and silty sands exposed to a M_w = 7.5 earthquake. Empirical correction factors were also presented to correct the CSR for earthquakes of different magnitude and duration.

Robertson and Campanella (1985) expanded the concept of the q_c/N_{spt} conversion factor by suggesting that the conversion factor was a function of the mean particle size (D_{50}) of the soil. They also suggested that the N_{spt} value used in the conversion should be

corrected to account for energy loss issues associated with the testing technique. The influence of the mean particle size on the q_c/N_{spt} conversion factor was further investigated and confirmed by Seed and De Alba (1986).

Sweeney (1987) evaluated the liquefaction potential of a dry sand deposit through an investigation involving miniature cone penetrometer tests in the Virginia Tech calibration chamber. A complete discussion of the Virginia Tech calibration chamber, pluviation apparatus, and sample preparation and testing procedure is included in Chapter 4. The penetration resistance values measured during the tests were correlated to CSR values determined through cyclic triaxial testing to generate penetration resistance curves that bound liquefiable and non-liquefiable conditions. The curves generated through this procedure fell below the liquefaction curves generated for the conventional 10-cm^2 cone (Figure 2.6).

Liquefaction potential curves were also developed by Shibata and Teparaska (1988) through a comprehensive field investigation of over 35 sites of where liquefaction was and was not observed in Japan (Figure 2.6). The investigation involved comparing the measured penetration resistance values from each of the sites to the CSR value calculated from Equation 2.1. The a_{max} values used in the investigation were obtained from existing earthquake records. Ishihara et al. (1985) performed a similar investigation, but the CSR values used in this study were determined from cyclic triaxial testing in the laboratory. As noted by Mitchell and Brandon (1998), the liquefaction resistance curves developed through these studies agree surprisingly well with proposed curves developed for clean sands using the corrected q_c/N_{spt} ratios noted by Seed and De Alba (1986) (Figure 2.6).

Mitchell and Tseng (1990) developed liquefaction resistance curves using the test results from cyclic triaxial testing in the laboratory and cone penetration resistance values predicted through cavity expansion theory. The procedure is based on the premise that the factors that affect the dynamic soil behavior in the laboratory also influence the magnitude of the penetration resistance value (Mitchell and Tseng 1990). The curves developed through this study were not dependent on the correlations of q_c/N_{spt} noted above or the uncertainties of deformation observations in the field. As shown in Figure 2.6, the curves generated through this technique agree well with the previously mentioned curves (Mitchell and Brandon 1998).

Stark and Olson (1995) compiled information from over 180 sites where earthquake records were available and CPT testing was performed to develop empirical liquefaction potential relationships based on CPT measurements (Figure 2.6). The presence of soil large strain behavior was identified at each of the sites by the surficial evidence of ground settlement, sand boils, or lateral spreading. Non-liquefaction conditions were suggested when the aforementioned features were absent. The results of the investigation involved the development of liquefaction potential curves for sandy soils ranging from clean sand to sandy silts, while additional curves were presented for gravelly soils. Suzuki et al. (1995) presented similar field based curves, which seem to correlate well with the curves presented by Stark and Olsen (1995) (Figure 2.6).

Olsen (1994) and Olsen and Koester (1995) proposed a technique for estimating the soil behavior during dynamic loading that considers both the tip resistance and sleeve friction measurements obtained during a penetration test. Both parameters were normalized by the vertical effective stress, which was modified by a *stress exponent*. A complete discussion of the stress exponent and the penetration resistance normalization procedure is presented in Section 5.4.2 of Chapter 5. The evaluation of the dynamic soil behavior of a given soil deposit using this technique involves comparing the normalized penetration resistance and sleeve friction values on a plot to determine the soil type. Superimposed on this plot are the results of normalized cyclic resistance ratio values determined from cyclic triaxial tests. Thus, for a given soil type, the CRR needed to prevent shear induced strength loss is determined for a given soil type. As noted by Olsen and Koester (1995) most of the previously mentioned liquefaction evaluation procedures require knowledge of the grain size distribution of the soil, which suggests that a field investigation outside of the CPT testing is needed to obtain soil samples. The proposed technique, however, indirectly gives estimations of the soil type so that separate investigations to retrieve soil samples are not necessary.

Robertson and Fear (1995b) proposed a method for evaluating the potential of a soil deposit to exhibit large seismically induced strains using estimations of the grain size and apparent fines content of the soil. The procedure involves an iterative estimation of the *soil behavior index* (I_c) from the normalized penetration resistance and sleeve friction values measured during the penetration test. The I_c value provides an estimation of the soil type encountered during the investigation. It is used to determine an empirical correction factor (K_c) that converts the normalized penetration resistance ($q_{c(N)}$) into a clean sand equivalent penetration resistance (q_{ce}). This q_{ce} value is used to determine the

CRR, which is then compared to the CSR estimated from a characteristic earthquake to provide an estimation of the potential for the soil deposit to exhibit significant strength loss and large strains during dynamic loading.

Robertson and Wride (1998) compared the relationship presented by Robertson and Fear (1995b) to the field measurements compiled by Suzuki et al. (1995) from 68 sites in Japan where earthquake induced soil deformation was and was not observed. The analysis revealed that the soil behavior curves presented by Robertson and Fear (1995b) simulated the field behavior for I_c values less than 1.65 but were unconservative for I_c values greater than 1.65. Correction factors were therefore presented to modify the I_c values greater than 1.65 so that the liquefaction curves approximated the field data.

A 1996 workshop sponsored by the National Center for Earthquake Engineering (NCEER) was conducted to review developments and establish a professional and academic consensus for the procedure of evaluating the behavior of soil deposits during dynamic loading using the CPT. The resulting CPT correlation recommended by the workshop committee is presented in Figure 2.6. The criteria allows for the direct calculation of the CRR based on corrected CPT data, and is represented through the following equations:

$$If\ \left(q_{c1N}\right)_{cs} < 50, \quad CRR_{7.5} = 0.833\left[\left(q_{c1N}\right)_{cs} / 1000\right] + 0.05 \tag{2.2}$$

$$If\ 50 < \left(q_{c1N}\right)_{cs} < 160, \quad CRR_{7.5} = 93\left[\left(q_{c1N}\right)_{cs} / 1000\right]^3 + 0.08 \tag{2.3}$$

where $(q_{c1N})_{cs}$ is the normalized clean sand cone penetration resistance. The clean sand penetration resistance value is normalized to a 100 kPa pressure and includes corrections to the measured penetration resistance value to account for grain characteristics and fines content of the soil. As stated by Youd and Idriss (1997) the NCEER committee did not conclusively agree on the correction factor that modified the measured penetration resistance into an equivalent clean sand value. Although a portion of the workshop committee endorsed the proposed criteria, some members of the committee suggested that the behavior index parameter (I_c) and the corresponding correction factor were inadequately developed and verified, and suggested that further investigative work be performed prior to implementation of the technique (Youd and Idriss 1997).

Included as Figure 2.6 are liquefaction characterization curves generated from each of the investigations noted in the preceding paragraphs. Clean sands and a magnitude M_w = 7.5 earthquake are assumed in the comparison. Excluding the curve generated by Sweeney (1987), the liquefaction curves generated from the each of the other investigations are in general agreement for the defined testing condition. However, as noted by Mitchell and Brandon (1998), there is a less consensus between the procedures as the fines content in the soil increases and as the magnitude of the earthquake deviates from the M_w = 7.5 reference value.

Several investigations were presented in the previous paragraphs that correlate the information obtained from a CPT test to an estimation of the soils resistance to shear induced strength loss during dynamic loading. All of the correlations indirectly correlate the soil density to the dynamic strength of the soil and do not directly measure the in-situ soil behavior during cyclic loading. As an attempt to remove some of the empirical nature of the correlations, a vibrating piezocone penetrometer (vibrocone) was developed and tested in the Virginia Tech calibration chamber to evaluate the in-situ behavior of saturated soils during vibratory loading. Several investigations using vibrocones for liquefaction evaluations were identified in the literature, a summary of which is included in Section 2.8.2.

2.8.2 Vibratory Penetration Approaches

Versions of the vibrocone were developed and evaluated outside of the U.S. in Japan (e.g. Sasaki and Koga 1982; Sasaki et al. 1984; Sasaki et al. 1985; Koga et al. 1986; Teparaksa 1987; Tokimatsu 1988; and in Canada (e.g. Moore 1987). A prototype vibrator built as part of this project was evaluated by Wise (1998), Wise et al. (1999), and Schneider et al. (1998) at Georgia Tech.

Six versions of the vibrocone were developed in Japan since the mid 1980's to provide an in-situ estimation of the behavior of soils during dynamic loading. Each of the vibrators used in the testing was mounted directly on top of the cone above the friction sleeve. The units applied a horizontal centrifugal force ranging from 313N (75lbs) to 1.6kN (360lbs) at a frequency of 200 Hz and were based on a rotating electric-bar design.

Sasaki and Koga (1982) evaluated the 313N vibrocone through a series of penetration tests in saturated sands in a steel testing box. The soil was placed in the box through foot compaction to reach the desired density. The behavior of the soil during the test was evaluated by comparing the penetration resistance from static and vibratory penetration tests and then computing the penetration resistance decrease due to the vibration. The results of their testing revealed that the reduction in penetration resistance was a fairly constant value of about 0.4 and was independent of the relative density for relative densities less than 50%. The magnitude of this reduction rapidly decreased as the relative density increased above 50%, suggesting that the effect of the vibration on the penetration resistance decreased as the relative density increased. An inspection of the test data also revealed that the magnitude of the vibratory penetration resistance increased as the overburden pressure increased for a given relative density. This implies that the reduction is penetration resistance through the vibration was dependent on both the density and the effective stress in the soil.

Sasaki et al. (1984; 1985) evaluated the in-situ behavior of the soil during dynamic loading by comparing the field penetration resistance values from side by side static and vibratory soundings. The investigations revealed zones where the vibratory penetration resistance was drastically lower than that obtained from adjacent static soundings, which were identified by the authors as zones of high susceptibility to strength loss during cyclic loading.

Koga et al. (1986) proposed a method for evaluating the behavior of a soil deposit during dynamic loading by comparing the shear stress needed to cause liquefaction to the penetration resistance reduction outlined by Sasaki and Koga (1982). A method was also proposed to estimate the large strain shear strength of the soil. The proposed methods were formulated by comparing CSR values from laboratory cyclic triaxial tests on undisturbed samples to the penetration resistance reduction measured in the field. These proposed relationships were considered unique for a given vertical effective stress but were developed independently of the grain size, fines content, and density of the soil. The authors indicated that the extremely large scatter in the data was most likely related to the influence of the aforementioned properties on the behavior of the soil during cyclic loading (Koga et al. 1986).

Tokimatsu (1988) evaluated the test results noted by Sasaki et al. (1984; 1985) and proposed an alternative method for evaluating the behavior of soil during dynamic

loading. The approach involved computing the ratio of the dynamic to static penetration resistance for different density soils in the laboratory and then comparing this ratio to similar values obtained from field testing. He noted from this comparison that ratios less than 0.2, which corresponded to soils with relative densities less than 50%, exhibited a strong potential for strength loss during dynamic loading.

Teparaksa (1987) also used the vibrating cone penetrometer to evaluate the liquefaction potential of soils. The investigation involved the measurement of pore pressures during penetration. He noted that the pore pressures values recorded at the midface location on the on the cone penetrometer were larger than the values recorded during the static penetration. This behavior was particularly evident in tests performed at low confining stresses. The magnitude of this pore pressure ratio seemed to gradually decrease as the effective stress in the sample increased, and was less than 0.01 for both the static and vibratory tests for all stress levels and densities.

A vibrocone was developed at the University of British Columbia in Canada by Moore (1987). The vibrator used in the investigation consisted of an unbalanced mass that was rotated by an electric motor. The vibrator was placed on top of the cone rods and oriented so that the oscillating force was vertical, as opposed to the Japanese vibrators were mounted directly above the cone and induced a centrifugal motion. The electric power for the vibrator was coupled with the power supply of the drill rig, which resulted in fluctuations in the frequency of the vibrator during the penetration test. An average frequency of 75 Hz was noted, although no estimations of the force applied to the soil was mentioned. The cone penetrometer used in the testing was a Fugro penetrometer with a pore pressure element at the shoulder location. Field test results of adjacent static and vibratory tests revealed a small reduction in the penetration resistance through the vibration at certain points in the soil profile. Excess pore pressures were not observed at locations where the vibration reduced the penetration resistance value.

Applied Research Associates (ARA) recently developed a sonic cone penetrometer for the purpose of advancing cone penetrometers in dense sands and gravels. The vibration associated with system is generated through a pair of counter rotating masses. The vibratory unit is mounted onto the top of the drill rod, operates at frequencies greater than 75 Hz, and is capable of adding a dynamic force of 295kN (14,000 lbf) to the static force in sandy soils (Shinn 1999). Tests results indicate that the vibration induced through the sonic unit reduces both the tip resistance and sleeve friction to approxiamtely

zero, even in very dense soils. No pore pressure measurements were reported with the sonic testing to date.

A prototype vibrator was developed and field tested by the Georgia Tech group as part of this study e.g. (Schneider et al. 1998; Wise 1998; Wise et al. 1999). The prototype unit mounts on the top of the cone penetrometer, operates at frequencies between 1-10 Hz, and is based on a pneumatic impulse design. Impulse load tests conducted in an MTS machine revealed that the force imparted onto the penetrometer tip through the impact motion was less than 4N (1lb). Field tests results reported by Wise (1998), Wise et al. (1999), and Schneider (1998) do not reveal a reduction in vibratory tip resistance when compared to static tests. An excess pore water pressure generated through the vibration was noted at a few locations, but field variablity in the soil stratum does not indicate whether the induced pore pressure is attributed to the vibratory loading or to partially undrained behavior due to the presence of silt in the sand.

Several investigations were identified in the literature related to vibrocone testing in sandy soils. Each investigation was performed for the purpose of evaluating the in-situ soil behavior during dynamic loading. The Japanese vibrators used in the testing were based on a rotating mass design that generated a constant force and frequency of vibration. The vibration generated by the Canadian and American units was vertical and fluctuated in both force and frequency. All penetration tests performed with the vibratory units were either performed in the field or under conditions where the testing parameters were not precisely controlled.

2.9 The Use of the Energy Based Methods to Evaluate the Behavior of Soils During Dynamic Loading

A review of the energy-based approaches for evaluating the behavior of soils during dynamic loading was also performed as part of this study. The initial work related to energy evaluations was performed by Gutenberg and Richter (1956), where the seismic energy released during an earthquake was related to estimations of the surface wave magnitude. Numerous approaches were developed within the last few decades that relate earthquake energy to the dynamic behavior of soils, several of which were included herein.

2.9.1 Laboratory Techniques

Simcock et al. (1983) performed cyclic triaxial tests on clean sands to evaluate the energy dissipated in a soil specimen during cyclic loading. The energy dissipated during the loading was computed from the cumulative area of the axial force vs. axial deformation hysteresis loops generated during the cyclic loading. This energy computation was normalized by the confining stress to generate what the authors referred to as the *energy density per unit volume of soil*. The energy density per volume of soil value was compared to the pore pressure ratio present during the loading to determine empirical correlations that were dependent on the CSR applied to the soil.

Towhata and Ishihara (1985) performed cyclic triaxial and torsional shear tests on cohesionless soils to evaluate the effects of the rotation of the principle stresses and principle strain on the magnitude of the pore pressure development. A portion of the investigation involved the computation of the energy dissipated in the soil sample through the use of the stress vs. strain hysteresis loops. Correlations were made with this dissipated energy to the pore pressure generated during the cyclic loading. The dissipated energy value computed in this investigation was not normalized by the confining stress in the soil. A further discussion of the use of this approach to compute the energy dissipated in a soil mass during dynamic loading is presented in Section 3.3.2 of Chapter 3.

Law et al. (1990) developed an energy approach for evaluating the liquefaction potential of cohesionless soils based on laboratory tests and seismic records from past earthquakes. The investigation included cyclic triaxial and cyclic simple shear testing on samples at different densities and stress levels. The sum of the hysteresis loops generated from the stress-strain behavior during the loading was calculated for each test, which was empirically related to the normalized excess pore pressure generated during the loading. The authors related this relationship to the equation for attenuation of seismic energy noted by Nuttli (1979), thereby correlating pore pressure development and energy dissipation in the laboratory to field estimations. A correlation of seismic energy factors to N_{spt} values was generated from the study (Law et al. 1990).

Figueroa et al. (1994) performed torsional shear tests on laboratory samples to attempt to evaluate the liquefaction potential using energy-based methods. Sand samples were tested at a variety of different densities, confining stresses, and strain amplitudes, and the results were presented based on the energy per unit volume dissipated in the

sample during the loading. The final expression for dissipated energy noted by the authors was dependent on the confining stress and density of the soil. The expression was dependent on the initial shear modulus and its degradation with loading and was virtually independent of the amplitude of cyclic shear strain exposed to the samples.

2.9.2 Field Techniques

Arias (1970) developed an approach for estimating the intensity of an earthquake that considered seismic destructiveness, the dynamic response of structures, and energy dissipation during shaking. The final expression generated through this study directly relates earthquake intensity to the acceleration measured from field instrumentation. The relationship allows for the computation of the earthquake intensity along mutually perpendicular axes at a given point, which when combined into an intensity tensor, is used to calculate the intensity on a horizontal plane. He suggests that the estimation of intensity using this approach is directly related to the Fourier spectrum of the ground motion, which seems to correlate well with the response spectra (Arias 1970).

Davis and Berrill (1982) analyzed earthquake case histories using the Gutenberg and Richter (1956) energy relationship to develop energy based liquefaction charts based on N_{spt} values. The relationship includes attenuation factors to account for both energy loss with respect to distance from the seismic source and the dissipation of energy per unit volume of soil at the site. This relationship is considered by the authors to relate the field N_{spt} value to dissipated energy, which indirectly gives a measure of the shear strain developed in the soil mass (Davis and Berrill 1982).

Moroto (1995) evaluated the behavior of cohesionless soils during dynamic loading by considering the relationship of the energy arriving at a site to the total energy radiated by an earthquake. The author normalized this energy arriving at the site by the effective overburden pressure, and correlated these normalized energy values to N_{spt} values obtained from field data.

Kayen and Mitchell (1997) used the Arias Intensity developed by Arias (1970) to assess the liquefaction potential of a soil deposit during seismic loading. The approach involves the use of seismograph time histories to determine the energy imparted into the soil during an earthquake. The approach considers any epicentral point of the earthquake and considers both the amplitude and the duration of the shaking in the energy estimation. The approach also considers a 2-D response to the motion and assumes that

the damping characteristics of the soil do not significantly effect the calculated Arias intensity (I_h). The values calculated from this approach were corrected to account for depth within a soil column and then correlated with CPT data obtained from field investigations. The approach seems to completely segregate the penetration test data into zones of liquefaction and non liquefaction, suggesting that considerations of both the shaking intensity and the cone penetration test results may be a viable method for estimating the liquefaction potential of a soil deposit (Kayen and Mitchell 1997).

Included in the previous paragraphs were several methods for calculating the relationship of the energy imparted into the soil during an earthquake to the behavior of the soil during the earthquake motion. The most commonly used lab based technique involves the calculation of the dissipated energy per unit volume of soil, which is determined in cyclic triaxial tests through the cumulative area of the stress-strain loops generated during the loading. Several field based approaches were also presented. The Arias Intensity approach presented by Kayen and Mitchell (1997) expands on the early works of Arias (1970), and uses information from the actual ground motion to determine the energy imparted into the soil from an earthquake. The approach does not, however, consider the effects of confining stress on the energy dissipation of the soil, which may lead to unconservative liquefaction potential estimations at shallow depths in the soil stratum.

2.10 Summary and Conclusions

The information presented in the literature is related to the behavior of cohesionless soils during dynamic loading, CPT testing, pore pressure development in sands, and energy-based principles used to evaluate the energy capacity of the soil. The following general conclusions were derived from the investigations:

a) Numerous of definitions for *liquefaction* were identified in the literature. The multi-tiered definition presented by Robertson and Fear (1995) takes into account the mechanism of loading, the pore pressure response, and pre and post peak strain behavior of the soil.

b) The review of the literature indicates that a wide degree of work was performed over the years to attempt to establish a laboratory testing procedure that provides test results that accurately simulates in-situ soil behavior. It appears, however, that issues related to sample

preparation and testing procedures, soil fabric, loading type, loading stress paths, stress history, soil property variations, and drainage path variations significantly influence the soil behavior during the test. Correction factors are therefore needed to correlate lab data to field conditions. All of these correction factors are empirically based and vary for the different investigations noted, type of laboratory tests performed, and different soil types.

c) Sample preparation procedures, such as saturation and application of backpressure, may influence the void ratio of the soil specimen, which may result in unconservative estimations of the undrained shear strength and the position of the steady state line.

d) Several approaches exist for estimating the cone penetration resistance of sand deposits. The empirical correlations based on calibration chamber testing seem to adequately simulate field behaviors, provided the correlation considers the effects of the chamber boundary, soil compressibility, grain size and gradation, horizontal effective stress, stress history, and relative density on the magnitude of the penetration resistance value. Recent approaches for estimating the penetration resistance based on cavity expansion theory have also shown favorable results.

e) The penetration resistance measurement is highly dependent on the initial lateral effective stress, relative density of the soil, soil fabric, compressibility, OCR, stiffness, and age of the soil deposit. The type of boundary condition as well as the size of the chamber relative to the cone penetrometer may influence the penetration resistance measured in calibration chambers.

f) Tests performed at low D_{cc}/d_{cp} ratios have lower penetration resistance values than at higher D_{cc}/d_{cp} ratios in constant stress boundary conditions (*BC1*). The opposite is true for rigid boundary conditions (*BC3*). The effect of the boundary on the penetration resistance measurement is more pronounced as the relative density and/or the effective stress in the soil increases. Boundary effects can also influence the penetration resistance measurement in low compressibility soils.

g) The bulk of the investigations noted in the literature related to calibration chamber testing were either performed in dry samples or in samples with variable fines content. As such, data related to pore pressure development in clean sands is limited. The few investigations that do provide information reveal that small positive excess pore pressures exist at the penetrometer in loose specimens, while negative values were identified in dense specimens. The magnitude of this pore pressure development seems to be independent of the stress levels in the soil and is dependent on the measurement location on the cone. The largest pore pressure values were recorded at the cone tip, and seemed to decrease towards the shoulder location.

h) Limited data exists in relation to the magnitude of the pore water pressure away from a penetrating cone penetrometer, with the existing data appearing contradictory and not well explained. The effect of the state of saturation on the penetration resistance measurement is also contradictory.

i) Pore water pressures measured at points away from a vibrating probe during stone column construction revealed excess water pressures that approached the vertical effective stress in the soil. An analysis of the soil response during vibratory pile driving suggests that the shear strain at the soil/pile interface increases and the shear strain away from the pile decreases as the frequency of vibration is increased. This indicates that the pore water pressure induced in the soil stratum away from the pile is dependent on both the frequency of vibration and the magnitude of strain at the soil/pile interface.

j) The most commonly used cone penetration technique used for liquefaction evaluation in practice is based on a comparison of the static penetration resistance of the soil to the estimated resistance of the soil to shear induced strength loss during dynamic loading (*CRR*). Several approaches were presented to make this comparion. All of the comparitive methods are highly dependent on the indirect correlation of the soil density, as inferred through static penetration resistance, to the estimated cyclic stress ratio imparted into the soil. None of the methods were developed using the direct in-situ measurement of the deformation behavior or the induced pore water pressure of the soil during the cyclic loading. The methods also use a cyclic stress estimation for the soil that is based on a peak

acceleration estimation that does not account for the frequency or duration of the shaking. Disagreement also exists within the NCEER committee members on how to correct the penetration resistance measured in the field into a clean sand value. A general dissagreement also exists regarding the correction factors used to correct the reference M_w = 7.5 earthquake to other magnitude earthquakes.

k) The vibrocone investigations noted in the literature show preliminary results that are promising, although major design and theoretical complications exist. The Japanese vibrocones were evaluated in either the field or in laboratory settings where the sample uniformity, preparation procedure, and testing conditions were not constant. The vibrocone developed at the UBC operates with a vertical vibration, although the frequency was variable and the force imparted to the soil mass unknown. The vibratory unit developed at ARA shows promising results related to tip reduction through vibration, although the force imparted onto the soil by the system is too large for liquefaction based analysis. Finally, the prototype vibrator developed at Georgia Tech did not impart a large enough force onto the soil mass to affect the tip resistance, nor was the range of frequency of vibration large enough to adequately characterize the harmonics of the soil.

l) Laboratory based methods were presented that allow for the calculation of the energy dissipated in a soil sample during cyclic loading by considering the stress/strain behavior during the loading cycles. Each of these correlations appears to be dependent on the type of testing and the corresponding loading stress path, indicating that field verification is needed prior to implementation of the model into design considerations.

m) Several field methods for evaluating the liquefaction potential of soils using energy based concepts were also reviewed. The approach generated by Kayen and Mitchell (1997) seems to identify a distinct boundary between zones of liquefaction and non liquefaction. However, the technique does not completely account for the effects of confining stress on the dissipated energy of the soil, which may lead to unconservative liquefaction potential evaluations at certain stress conditions.