Estimating liquefaction induced ground settlements from CPT for level ground

G. Zhang, P.K. Robertson and R.W.I. Brachman*
Department of Civil and Environmental Engineering
University of Alberta
Edmonton, CANADA
T6G 2G7

Phone: (780) 492-5112
Fax: (780) 492-8198
Email: rbrachman@civil.ualberta.ca

Submitted to the Canadian Geotechnical Journal: October 25, 2000

*Corresponding Author

Estimating liquefaction induced ground settlements from CPT for level ground

G. Zhang, P.K. Robertson and R.W.I. Brachman
Department of Civil and Environmental Engineering
University of Alberta
Edmonton, CANADA, T6G 2G7

Abstract: An integrated approach to estimate liquefaction-induced ground settlements using CPT data for sites with level ground is presented. The approach combines a CPT-based method to estimate liquefaction resistance with laboratory test results on clean sand to evaluate the liquefaction-induced volumetric strains for sandy and silty soils. The proposed method was used to estimate the settlements at both the Marina District and Treasure Island sites damaged by liquefaction during the Loma Prieta, California, earthquake of October 17, 1989. Good agreement between the calculated and measured liquefaction-induced ground settlements was found. The major factors that affect the estimation of liquefaction-induced ground settlements are also discussed in detail. The recommendations for taking the effects of these factors into account in estimating liquefaction-induced ground settlements using the proposed CPT-based approach are presented. It is suggested that the proposed method may be used to estimate liquefaction-induced settlements for low to medium risk projects, and also to provide preliminary estimates for higher risk projects.

INTRODUCTION

Liquefaction of loose, saturated granular soils during earthquakes is a major hazard for construction of facilities in many regions. Liquefaction-induced ground deformations have caused significant damage to engineered structures and lifelines during past earthquakes. Both ground settlements and lateral spreads are the pervasive types of liquefaction-induced ground deformations for level to gently sloping sites. This paper focuses on estimating liquefaction-induced ground settlements.

Liquefaction-induced ground settlements are essentially vertical deformations of surficial soil layers caused by the densification and compaction of loose granular soils following earthquake loading. Several methods have been proposed to calculate liquefaction-induced ground deformations, including numerical and analytical methods, laboratory modeling and testing, and field-testing-based methods. The expense and difficulty associated with obtaining and testing high quality samples of loose sandy soils may only be feasible for high-risk projects where the consequences of liquefaction may result in severe damage and large costs. Semi-empirical approaches using data from field tests are likely best suited to provide simple, reliable and direct methods to estimate liquefaction-induced ground deformations for low to medium risk projects, and also to provide preliminary estimates for higher risk projects.

Several field tests are commonly used for the evaluation of liquefaction resistance of sandy soils, including the cone penetration test (CPT), the standard penetration test (SPT), shear-wave velocity measurement, and the Becker penetration test. To date, only an SPT-based method (Tokimatsu and Seed 1987) has been used to calculate liquefaction-induced ground settlements. Recently, the CPT has become very popular for site characterization because of its greater repeatability and the continuous nature of its profile as compared with other field tests. The CPT has also been increasingly used in predicting liquefaction potential in geotechnical practice. However, a method to estimate liquefaction-induced ground settlements based on the CPT has not yet been fully developed.

The purpose of this paper is to present a CPT-based approach to estimate liquefaction-induced ground settlements at sites with essentially level ground during earthquakes. The proposed CPT-based method was used to estimate the settlements at both the Marina District and Treasure Island sites damaged by liquefaction during the Loma Prieta, California, earthquake of October 17, 1989. Good agreement between the calculated and measured liquefaction-induced ground settlements was found. It is suggested that the proposed method may be used to estimate liquefaction-induced settlements for low to medium risk projects, and also to provide preliminary estimates for higher risk projects.

The major factors that affect estimation of liquefaction-induced ground settlements using CPT are also discussed in detail in this paper. These factors include maximum surface acceleration, fines content or mean grain size, the transitional zone or thin sandy soil layers, three-dimensional distribution of liquefied soil layers, the correction factor K_c, and the cutoff line of the soil behavior type index (I_c) equal to 2.6. Guidance for taking the effects of these factors into account in estimating liquefaction-induced ground settlements using the proposed CPT-based approach is provided.

CPT-BASED LIQUEFACTION POTENTIAL ANALYSIS

Several CPT-based methods have been proposed for predicting liquefaction resistance of sandy soils (NCEER 1997). Most require the fines content and/or the mean grain size, D_{50}, and/or plasticity index of the fine fraction to be known for silty sands or sandy silts. Soil samples are therefore needed from a location close to the CPT position. To overcome the disadvantages of the previous CPT-based methods, Robertson and Wride (1998) developed an integrated procedure to evaluate the liquefaction resistance of sandy soils based solely on CPT data. This procedure can be used to estimate the cyclic liquefaction resistance of sandy soils for low- and medium-risk projects and also to provide a preliminary estimate of the liquefaction resistance of sandy soils for high-risk projects using CPT data only. It provides a convenient approach for geotechnical practitioners.

Comparison of Robertson and Wride's CPT-based method with SPT-based methods and other CPT-based methods has demonstrated that Robertson and Wride's method is reliable and convenient (Gilstrap 1998; Juang et al. 1999a). In addition, Juang et al. (1999b) found that the degree of conservatism in the Robertson and Wride method is comparable to that in the Seed and Idriss (1971, 1982) SPT-based method that has been widely used in geotechnical practice around the world for more than twenty years. The method of Robertson and Wride (1998) is used to evaluate the liquefaction resistance as one step in the proposed CPT-based approach to estimate liquefaction-induced ground settlements in this paper.

Calculation or estimation of two variables is required for evaluation of liquefaction potential of soils. These variables are: the seismic demand placed on a soil layer by a given earthquake, expressed in terms of cyclic stress ratio (CSR); and the capacity of the soil to resist liquefaction, expressed in terms of cyclic resistance ratio (CRR) (NCEER 1997). If CSR exceeds CRR, liquefaction of the soil is highly likely during the earthquake.

An updated flow chart for evaluating the cyclic resistance ratio of sandy soils using the CPT-based method proposed by Robertson and Wride (1998) is shown in Figure 1. Important parametres in this approach are the soil behavior type index (I_c), the correction factor for the grain characteristics of the soil (K_c), and the equivalent clean sand normalized CPT penetration resistance, $(q_{c1N})_{cs}$. I_c is a function of the normalized CPT penetration resistance (Q) and the normalized friction ratio (F). The cyclic resistance ratio profile for an earthquake of magnitude (M) equal to 7.5, denoted as $CRR_{7.5}$, can be estimated directly from the CPT sounding.

A simplified method to estimate the cyclic stress ratio profile caused by a given earthquake was developed by Seed and Idriss (1971) based on the maximum ground surface acceleration (a_{max}) at the site. An update of this simplified approach is described in detail in a recent NCEER publication (NCEER 1997).

INTEGRATED CPT-BASED APPROACH TO ESTIMATE LIQUEFACTION-INDUCED GROUND SETTLEMENTS

Post-liquefaction volumetric strain from laboratory tests

Nagase and Ishihara (1988) conducted cyclic simple shear tests on saturated loose, medium-dense and dense samples of Fuji River sand. Both uni-directional and multi-directional loading conditions were simulated by employing irregular time histories of motions observed during major earthquakes in Japan between 1964 and 1983. Following the undrained application of the irregular loading, pore water pressures were allowed to dissipate and the resulting volumetric strains of the samples were measured. The amount of re-consolidation volumetric strain found from the tests provides a measure of the volumetric strain that may occur for in-situ deposits of sands following liquefaction during earthquakes.

Based mainly on the laboratory results of Nagase and Ishihara (1988), Ishihara and Yoshimine (1992) established a family of curves, as shown in Figure 2, from which the volumetric strain resulting from dissipation of pore water pressures was correlated with relative density (or density index) and the factor of safety against liquefaction (FS) for clean sands. These curves are used to estimate post-liquefaction volumetric strain for clean sands in this paper.

Relative density from CPT

Relative density (D_r) was used by Ishihara and Yoshimine (1992) to quantity the state of density of a sand. However, D_r can not be measured directly from the CPT. Several empirical correlations between D_r and cone tip resistance (q_c) have been proposed (e.g., Jamiokowski et al. 1985; Tatsuoka et al. 1990).

The curves proposed by Ishihara and Yoshimine (1992) were based mainly on results from laboratory tests conducted on Fuji River sand. However, no calibration chamber testing has been carried out to establish the relationship between D_r and q_c for Fuji River sand. Ishihara and Yoshimine (1992) recommended use of the correlation proposed by Tatsuoka et al. (1990) for Toyoura sand.

It is known that grain characteristics of sands may affect the correlation between D_r and q_c. Grain characteristics of Fuji River sand are similar to those of the five sands used by Jamiolkowski et al. (1985) and the sand used by Tatsuoka et al. (1990). Hence, the correlation by Tatsuoka et al. (1990) is used in this paper since this method provides slightly smaller and more conservative estimates of relative density than the correlation by Jamiolkowski et al. (1985) when q_c is less than about 10 MPa.

Correction for grain characteristics

The curves of Figure 2 proposed by Ishihara and Yoshimine (1992) were based on laboratory test results on clean sand. If these curves are used to estimate the post-liquefaction volumetric strains of silty sands using the CPT, some modifications for grain characteristics or fines content on the CPT soundings and their interpretations must be made.

There are two potential approaches to account for the effect of grain characteristics. One approach is to use the equivalent clean sand normalized CPT penetration resistance, $(q_{c1N})_{cs}$, defined by Robertson and Wride (1998) to account for the effect of grain characteristics or fines content on CPT soundings. The parameter, $(q_{c1N})_{cs}$, can then be treated as the cone tip resistance for a clean sand and used directly to estimate the post-liquefaction volumetric strain at certain values of FS.

An alternate approach is to estimate the relative density (D_r) of silty soils using the CPT and then use D_r and FS to evaluate the post-liquefaction volumetric strain based on the curves by Ishihara and Yoshimine (1992). This implicitly assumes that silty soils with the same D_r and FS may result in the same post-liquefaction volumetric strains under cyclic loading. Unfortunately, no generally accepted correlation between D_r and CPT soundings is available for silty soils.

One major advantage of the first approach is that it is convenient to get both FS and $(q_{c1N})_{cs}$ from the liquefaction potential analysis and then estimate the post-liquefaction volumetric strain. The proposed CPT-based method in this paper uses the first approach to estimate post-liquefaction volumetric strains for sandy and silty soils. The correlations between $(q_{c1N})_{cs}$ and post-liquefaction volumetric strain (ε_v) for different FS were developed on the basis of the curves of Ishihara and Yoshimine (1992), as shown in Figure 3.

With the CPT sounding, the design earthquake (M and a_{max}) and other input parameters (ground water table, unit weight, etc.), the equivalent clean sand normalized CPT penetration resistance, $(q_{c1N})_{cs}$, and FS for sandy and silty soils can be obtained from the CPT-based liquefaction potential analysis proposed by Robertson and Wride (1998). The post-liquefaction volumetric strain can then be estimated using Figure 3 for every reading in the CPT sounding.

Calculating ground settlement

For sites with level ground, far from any free face (e.g., river banks, seawalls), it is reasonable to assume that little or no lateral displacement occurs after the earthquake, such that the volumetric strain will be equal or close to the vertical strain. If the vertical strain in each soil layer is integrated with depth using Equation [1], the result should be an appropriate index of potential liquefaction-induced ground settlement at the CPT location due to the design earthquake.

[1] $$S = \sum_{i=1}^{n} \varepsilon_{v_i} \Delta z_i$$

where S is the calculated liquefaction-induced ground settlement at the CPT location; ε_{vi} is the post-liquefaction volumetric strain for the soil sub-layer i; Δz_i is the thickness of the sub-layer i; n is the number of soil sub-layers.

The procedure to estimate liquefaction-induced ground settlements with the proposed CPT-based method can be illustrated using a CPT profile from the Marina District site in California. This site is discussed in detail in the next section. Figure 4 illustrates the major steps in the CPT-based liquefaction potential analysis and shows the profiles of measured CPT tip resistance q_c, sleeve friction f_s, soil behavior type index I_c, cyclic resistance ratio CRR & cyclic stress ratio CSR, and factor of safety against liquefaction FS, respectively. The data in Figures 4a and 4b can be directly obtained from the CPT sounding. Figures 4c, 4d and 4e show the results calculated based on the procedure shown in Figure 1. Note that, according to Robertson and Wride's approach, CRR is not calculated when the soil behavior type index is greater than 2.6. These soils are assumed to be non-liquefiable in Robertson and Wride's approach.

The four key plots for estimating liquefaction induced ground settlements by the proposed CPT-based method are presented in Figure 5. Figures 5a to 5d show the profiles of equivalent clean sand normalized tip resistance $(q_{c1N})_{cs}$, factor of safety FS, post-liquefaction volumetric strain ε_v, and liquefaction induced ground settlement S, respectively. Data in Figures 5a and 5b are from the liquefaction potential analysis. Data in Figure 5c are calculated from the curves of Figure 3. The settlement shown in Figure 5d is obtained using Equation [1] and the volumetric strains from Figure 5c.

EVALUATION OF PROPOSED INTEGRATED CPT-BASED APPROACH BY CASE HISTORIES

In the past 20 years, a number of post-liquefaction CPTs have been conducted at sites around the world, especially in the USA. In addition, earthquake-induced ground deformations have also been measured at some of these sites. These case histories provide an opportunity to evaluate the proposed integrated CPT-based method for estimating liquefaction-induced ground settlements by comparing estimated settlements with those measured in the field.

One hundred and thirty three CPT soundings have been collected from fifteen case history sites in the USA. Eight case history sites are associated with lateral spreads and five case history sites have post-liquefaction phenomena of sand boils and cracks but no reported values of liquefaction induced ground settlements. Hence, only two case history sites (Marina District and Treasure Island) are available to evaluate the proposed CPT-based liquefaction-induced ground settlement method.

Marina District

The Marina District is located on the north side of San Francisco, California. During the 1989 Loma Prieta earthquake the area was significantly damaged, even though it was more than 100 km from the epicenter. Liquefaction induced sand boils, ground fissures, and ground settlements were observed and recorded.

Following the 1989 earthquake, several groups conducted post-earthquake investigations in the area. A subsurface investigation that included five CPT soundings were performed and vertical settlements caused by the earthquake were measured by the US Geological Survey (Bennett 1990). Bardet and Kapuskar (1991) also conducted a subsurface investigation including nine CPTs in the Marina District. Two of these CPTs only penetrated into soils down to 2 to 3 m and thus can not be used in this study. Three of the CPTs were conducted at locations near the seawall where both lateral spreads and ground settlements occurred during the earthquake due to the adjacent free face. A total of nine CPTs (M1, M2, M3, M4, M6, C-2, C-8, C-9, C-12) were used to evaluate the proposed CPT-based method.

The stratigraphy in the Marina District generally consists of three distinct sand deposits overlying the San Francisco Bay Mud or bedrock (Holzer and O'Rourke 1990). Figure 6 is a plan showing the general locations of the geologic units in the Marina District. The western part of the district in the upper 8 m contains mainly beach sand deposits. CPT soundings M1, M2, M3, C-8, and C-9 penetrated in this material. The central part of the district is underlain by sand to silty sand fill in the upper 8 m. Much of this fill was placed back hydraulically in 1912 as a slurry with no compaction (Rollins and McHood 1998). CPT soundings M4, C-2, and C-12 penetrated this sediment. Both the beach sand and the hydraulic fill overlie bay mud. The eastern part of the district is underlain by dune sand in the upper 11 m and no bay mud is beneath the dune sand. CPT M6 penetrated the dune sand sediment.

The input data for the proposed CPT-based method include: CPT soundings (cone tip resistance and sleeve friction) with depth, moment magnitude of the earthquake, maximum surface acceleration during the earthquake, depth to ground water table, and the unit weights of the soils.

No accelerograph was located in the Marina District before the earthquake. The closest site that recorded main-shock accelerograms was located on bedrock of the Pacific Heights, approximately 1.5 km south of the Marina District (Boatwright et al. 1992). As a result, many researchers (Bardet et al. 1992; Bennett 1990; Boatwright et al. 1992; Holzer and O'Rourke 1990; O'Rourke et al. 1991; Rollins and McHood 1998; Taylor et al. 1992) have conducted studies to estimate the maximum surface acceleration (a_{max}) at the Marina District during the Loma Prieta earthquake. The variation of the maximum surface accelerations either calculated or assumed ranged from 0.12g to 0.32g for the Marina District during the Loma Pritea earthquake.

There are generally three distinct geological zones in the Marina District area, which are different in soil type, soil compressibility, and soil thickness of each layer. It is reasonable to assume that they had a different response to the earthquake and thus the different values of a_{max}. Based on the work by Idriss (1990), Seed et al. (1994), and others, values of a_{max} of 0.12g, 0.16g, and 0.24g were used in this work for the eastern (dune sand, no bay mud), western (beach sand over thinner bay mud), and central zones (hydraulic fill over thicker bay mud) of the Marina District.

The depth to the ground water table varied between 2.3 m and 5.5 m within the Marina District during the earthquake (Bonilla 1992). A moment magnitude of 7.0 was used to model the 1989 Loma Prieta earthquake (Boulanger et al. 1995; Gilstrap 1998). Average total unit weights of 15.0 kN/m^3 and 19.4 kN/m^3 were assumed for soil above and below the ground water table, respectively.

Table 1 presents the liquefaction-induced ground settlements measured and calculated using the proposed CPT-based method for the Marina District. The calculated settlements are quite similar to the measured/estimated settlements. In general, the calculated settlements are slightly larger than the actual values.

O'Rourke et al. (1991) used the SPT based method of Tokimatsu and Seed (1987) to estimate the liquefaction-induced settlement at the Marina District. They assumed a peak ground acceleration of 0.2 g and an earthquake magnitude of 7.1 in their analyses. The calculated settlements by O'Rourke et al.

(1991) are presented in Table 2. Rollins and McHood (1998) also computed the settlements at six SPT locations in the Marina District using the SPT based method of Tokimatsu and Seed (1987). However, they adopted a peak ground acceleration of 0.15 ± 0.05g and an earthquake magnitude of 6.75 as the input. Although Tokimatsu and Seed (1987) did not specify a procedure for correcting for fines content in the settlement computation, Rollins and McHood (1998) corrected for fines content by adjusting the volumetric-strain curves in a manner consistent with the correction of Seed et al. (1985) for liquefaction triggering. The ranges of settlements calculated by Rollins and McHood (1998) are also presented in Table 2.

The results in Table 2 show that the calculated settlements using the proposed CPT-based method are much closer to the measured values than those calculated using the SPT based method for the Marina District site. This is especially the case for the hydraulic fill zone where large settlements occurred due to the earthquake and where the SPT based approaches overestimate the settlements by up to a factor of two. The discontinuous nature in a SPT and low resolution with its readings in a very loose or loose sandy soil layer may partially contribute to the poor performance of the SPT-based method.

Treasure Island

Treasure Island is a 400 acre man-made island situated in San Francisco Bay approximately midway between the cities of San Francisco and Oakland, California. During the 1989 Loma Prieta earthquake, liquefaction-related phenomena, including sand boils, ground surface settlements, and lateral spreading movements, were evident at many locations across the island. The buildings and underground utilities on/in the island were damaged by the ground settlements and lateral displacements (Egan and Wang 1991).

Following the 1989 earthquake, several groups conducted post-earthquake investigations in the area (Power et al. 1998; Hryciw 1991). Liquefaction-induced ground settlements and lateral movements were recorded at nine existing benchmarks on the island. The liquefaction-induced differential settlements between the ground and piled buildings on the island were also observed (Bennett 1998). CPT data were collected at forty-two locations around the island (Power et al. 1998; Hryciw 1991). However, only twelve of the CPT soundings can be used to evaluate the proposed CPT-based method for this case history site since the majority of the CPT locations were close to the perimeter of the island where both lateral spreads and ground settlements occurred.

Subsurface materials at Treasure Island can generally be divided into four strata: hydraulically placed sand fill, native shoal sand and clay, recent bay sediments, and older bay sediments (Power et al. 1998) as shown in Figure 7. The hydraulically placed sand fill was dredged from various borrow sources located within San Francisco Bay during filling operations and consisted of mostly fine-to-medium-grained sand material containing different amounts of gravel, silt and clay depending on the location. The sand fill is supported by a rock mound placed on either the native soil or fill materials to act as a retaining dike along the island's perimeter. The shoal sand is similar to the fill deposit but with higher density and shell content. The sand fill and shoal sand range in combined thickness from approximately 7.5 m to 15 m. The recent bay mud consists primarily of soft to stiff silty clay and range in thickness from about 4.5 m to 40.5 m. The older bay sediments consist of very stiff sandy, silty, and/or peaty clay and dense sand and overlie the bedrock that is about 85 m below the ground surface.

Treasure Island is relatively flat. The ground water levels in the island are typically at depths of 1.5 to 2.4 m below the ground surface. In this study, the ground water table was assumed to be 2.0 m below the ground surface. Ground motion was recorded at the fire station on Treasure Island during the 1989 Loma Prieta earthquake. The recorded peak acceleration was 0.16 g (Hryciw et al. 1991). The ground response analyses (Hryciw et al. 1991; Power et al. 1998) had shown that the intensity of ground shaking did not vary greatly in different places on the island. Consequently, a peak acceleration of 0.16 g was used in this study. A moment magnitude of 7.0 was used to model the 1989 Loma Prieta earthquake. Average total unit weights of 15.0 kN/m^3 and 19.4 kN/m^3 were assumed for soil above and below the ground water table, respectively.

No measured values of liquefaction-induced ground settlements are available at the locations where the CPTs used in this study were penetrated. However, the pre-earthquake and post-earthquake data from nine survey benchmarks in the island indicated that the total settlements generally ranged from 5 to 15 cm (2 to 6 inches) (Power et al. 1998). Furthermore, observations of the ground-surface settlements adjacent to piled structures also indicated that the settlements were generally as much as approximately 15 cm (6 inches) (Power et al. 1998).

The calculated settlements and the ranges of observed liquefaction-induced ground settlements for Treasure Island are shown in Table 3. The proposed CPT-based method has correctly predicted large settlements in the region of zone two. In general, the calculated settlements are larger than the observed values. This implies that the proposed method appears to be conservative for this case history site. The possible reasons for this conservativeness will be discussed in detail in next few sections.

However, it is useful to reflect on the accepted accuracy of current calculation of ground settlements in sand for the simple case of static vertical loading. For example, Tan and Duncan (1991) proposed that the most accurate static-loading settlement predictions should be multiplied by a factor of about 1.7 to ensure that 85 percent of the measured settlements would be less than the computed settlements (Rollins and McHood 1998). Thus, considering the complexity involved in the estimation of liquefaction-induced ground settlements under earthquake loading, the agreement here between observed and calculated settlements is encouraging.

EFFECTS OF OTHER MAJOR FACTORS ON CALCULATED SETTLEMENTS

Maximum surface acceleration

The amplification of earthquake motions is a complex process and is dependent on soil properties, thickness, frequency content of motions and local geological settings. For a given earthquake and geological setting, the amplification increases with the increase of soil compressibility and with soil thickness (Law 1990).

Maximum surface acceleration at a site is one important parameter used in evaluating liquefaction potential of sandy soils. However, its determination is difficult without recorded accelerograghs for a given earthquake because it may vary with soil stratigraphy, soil properties, earthquake properties, the relative location of the site to the epicenter and even ground geometry. Ground response analysis may help to solve the problem but still leave some uncertainty in the results.

As an example, Table 4 shows the different values of maximum surface acceleration estimated or assumed by the different researchers for the Marina District site under the 1989 Loma Prieta earthquake. The values vary from 0.12 g to 0.32 g. Obviously, this wide range of the values will produce uncertainty in evaluating liquefaction potential and estimating liquefaction-induced ground settlements. The effects of these variations on the calculated settlements may be much different for the different CPT soundings as illustrated in Figure 8. The calculated settlement for the CPT M6 is very sensitive to change of the peak surface acceleration for values less than 0.2 g. The change of the peak surface acceleration from 0.12 g to 0.20 g would cause the calculated ground settlement increasing from 2 cm to 20 cm for the CPT M6. On the contrary, the variation in the peak surface acceleration from 0.12 g to 0.20 g only causes slightly change of the calculated settlement for the CPT M2. And for all the three CPT soundings studied the calculated ground settlements almost does not change if the peak surface acceleration is greater than 0.20 g at which the calculated volumetric strains have reached their maximums.

Fines content or mean grain size

Lee and Albaisa (1974) conducted laboratory cyclic triaxial tests to study earthquake induced settlements in saturated sands with different grain sizes. They found that grain size has a significant effect on the re-consolidation volumetric strains when "initial liquefaction" occurs or the peak pore pressure ratio reaches to 100% in soils. Their test results indicated that volumetric strains would increase with increasing mean grain size at a given relative density. Generally, increase of fines content in sands will result in decreasing mean grain size of the sands. Therefore, it can be concluded that post-liquefaction volumetric strains would decrease with increasing fines content in sands at a given relative density.

Both Tokimatsu and Seed (1987) and Ishihara and Yoshimine (1992) methods are applicable to saturated clean sands only. However, it is a practical necessity to include sands having fines content greater than five percent, ranging from sands with little silt to silty sands.

Seed and Harder (1990) stated that "triggering" (i.e. initial liquefaction) and "post-triggering" (i.e. liquefaction induced deformations) analyses for liquefaction are inherently different as they relate to different phenomena. Unfortunately, at present there are no systematic laboratory "post-triggering" testing data available on silty sands as reported for clean sands. Equally, there are no systematic field "post-triggering" data accumulated on liquefaction induced volumetric strains relating to silty sands. As a result, a compromised alternative procedure is to adopt similar approaches with "post-triggering" (i.e. liquefaction induced volumetric strains) analysis for sands with fines as with "triggering" (i.e. liquefaction resistance) analysis.

The effect of fines content on liquefaction resistance of sand had been investigated by several researchers (Iwasaki et al. 1978; Tatsuoka et al. 1980; Tokimatsu and Yoshimi 1981; Zhou 1981; et al.). They found that silty sands are considerably less vulnerable to liquefaction than clean sands with similar SPT blow-counts. Based on this observation, "correction factors" of SPT blow-counts or "corrected" cyclic resistance ratio for sands with different fines contents or mean grain sizes had been widely used in liquefaction potential analyses using SPT or CPT based methods (Seed and Idriss 1982; Robertson and Campanella 1985; Seed et al. 1985; Robertson and Wride 1998; et al.). Robertson and

Wride (1998) used $(q_{c1N})_{cs}$ to incorporate the effect of sand grain characteristics on cyclic resistance ratio in liquefaction potential analysis.

$(q_{c1N})_{cs}$ is the value with consideration of the appropriate "correction" values for the apparent fines content in liquefaction potential analysis. In this study, $(q_{c1N})_{cs}$ is also used to estimate post-liquefaction volumetric strains for sands with fines. This approach is based on an assumption that both liquefaction resistance and post-liquefaction deformation properties of sandy soils including silty sands can be quantified using the same method and formula as clean sands if equivalent clean sand normalized cone penetration resistance, $(q_{c1N})_{cs}$, is used. This implies that no further correction procedure is needed for the effect of fines content or mean grain size if $(q_{c1N})_{cs}$ is used to estimate the liquefaction induced settlements of sandy soils including silty sands. Because $(q_{c1N})_{cs}$ will increase with increase of fines content with sands for a given cone tip resistance, the resultant calculated post-liquefaction volumetric strains will decrease with increase of fines content for a given factor of safety. Therefore, in general, this approach appears to indirectly account partially or wholly for the effect of grain characteristics on post-liquefaction volumetric strains and cast the same trend as observed by Lee and Albaisa (1974).

Transitional zone or thin sandy soil layers

Many researchers (Sanglerat 1972; Campanella and Robertson 1988; Berg 1994; Vreugdenhil 1995; Robertson and Fear 1995; Robertson and Wride 1998) have recognized the influence of soil layering on CPT cone resistance. Based on the results of the experiments and numerical analyses for a two layered system, Berg (1994) concluded that a thickness of at least 40 – 50 cm is required to ensure reaching full tip resistance in a CPT with a 10 cm^2 base area cone for a stiff frictional deposit (e.g., sand) sandwiched by softer soil layers. Vreugdenhil (1995) also concluded that the error in the measured cone resistance within a thin stiff layer is a function of the thickness of the layer as well as the stiffness of the layer relative to that of the surrounding softer soil. Robertson and Wride (NCEER 1997) suggested a simplified correction to the measured CPT tip resistance in a sand layer sandwiched by softer clay layers.

It is also recognized that transitional zones between soft clay layers and stiff sandy soil layers have an influence on the results of a liquefaction potential analysis and calculated liquefaction-induced settlements in this study. However, it should be noted that the influence of the transitional zones on calculated $(q_{c1N})_{cs}$, and FS has been partially counteracted implicitly in Robertson and Wride's method. Generally, the measured tip resistance in a sandy soil layer close to a soft soil layer (usually a clayey soil layer) is smaller than the "actual" tip resistance (if no layer interface existed) and the resultant friction ratio is greater than the "actual" friction ratio due to the influence of the soft soil layer. As a result, the calculated I_c will increase, therefore, the correction factor K_c, $(q_{c1N})_{cs}$, and FS will increase as well. So, finally, the $(q_{c1N})_{cs}$ and FS may be close to the "true" values in a same sandy soil layer that is not influenced by the soft soil layer. Therefore, the calculated ground settlements would be close the "actual" values because of this implicit correction incorporated with Robertson and Wride's method.

In this study, no further correction is taken to quantify the influences of both the transitional zones and thin sandy layers on the tip resistance of a sandy soil layer because of its complexity. This is on the conservative side in estimating liquefaction potential and liquefaction related deformations. Further research is needed to quantify the influence of transitional zones or thin sandy soil layers on calculated FS and liquefaction-induced ground settlements.

Three dimensional distribution of liquefied soil layers

Both Tokimatsu & Seed (1987) and Ishihara & Yoshimine (1992) suggested that the surface ground settlements could be calculated by multiplying the volumetric strain by the thickness of the liquefied layer and adding them together through the depth. However, the three-dimensional distribution of liquefied soil layers may affect ground surface settlements.

The vertical distribution of liquefied layers may play a role on ground surface settlements. Liquefaction of a relatively thick but deep sandy soil (Figure 9a) may have minimal effect on the performance of an overlying structure founded on shallow foundations. However, liquefaction of a near surface thin layer of soil (Figure 9b) may have major implications on the performance of the same structure. Ishihara (1985) investigated the effect of thicknesses of liquefiable soil and non-liquefied surface layer on liquefaction-induced damage. He used observations from case history sites affected by the 1983 Nihonkai-Chubu earthquake and 1976 Tangshan earthquake to develop boundary curves for site identification of liquefaction-induced damage with different peak ground acceleration levels. A study to evaluate and verify Ishihara's (1985) criteria was performed by Youd and Garris (1995) using the data calculated from a wide range of earthquakes and site conditions. Youd and Garris found that the thickness bounds proposed by Ishihara appear to be valid for sites not susceptible to ground oscillation or lateral spread, however, the bounds suggested by Ishihara are not valid for the prediction of ground-surface disruption for sites susceptible to ground oscillation (includes ground settlements) or lateral spread. O'Rourke and Pease (1997) also evaluated Ishihara's (1985) criteria by using the data from the Marina, South of Market and Mission Creek case sites. They generally agreed with the conclusions of Youd and Garris (1995). Gilstrap (1998) concluded that Ishihara's (1985) relationships for predicting liquefaction-induced surface effects may be over-simplified on the basis of his case history studies. Furthermore, the application of Ishirara's criteria in practice for cases with multiple liquefied layers (Figure 9c) is not clear.

Besides the effect of vertical distribution of liquefied layers, the horizontal extent of liquefied layers may also have effect on ground surface settlements. A small locally liquefied soil zone with limited horizontal extent (Figure 9d) would have limited extent of surface manifestation than that for a horizontally extensive liquefied soil zone with the same soil properties and vertical distribution of the liquefied layer. On the other hand, the locally liquefied soil zone may be more damaging to the engineered structures and facilities due to the potential large differential settlements. However, no quantitative study has been reported for the effect of horizontal extent of liquefied layers on ground surface settlements.

Ignoring the effect of three-dimensional distribution of liquefied layers on ground surface settlements may result in over-estimating liquefaction-induced ground settlements for some sites. Engineering judgement is needed to consider the effect to avoid an overly conservative design. Case histories from previous earthquakes have indicated that little or no surface manifestation was observed for cases where the depth from ground surface to the top of the liquefied layer was greater than 20 m. Based on this observation, it may be reasonable to expect that a liquefied layer beneath a thick non-liquefied layer of 20 m would not contribute to the surface ground settlement. On the other hand, caution should be paid to locally liquefied soil zones since potential differential settlements around the zones may be more significant even though the total ground settlements are same as those for horizontally extensive liquefied soil zones.

Correction factor K_c

Robertson and Wride (1998) recommended that the correction factor K_c is set to be equal to one instead of using K_c of 1.0 to 2.14 when the CPT data plot in the zone defined by $1.64 < I_c < 2.36$ and $F < 0.5\%$. The purpose of this recommendation was to avoid confusing very loose clean sands with denser sands containing fines because both very loose clean sands and denser sands containing fines may fit in the same zone. As a result, if a soil with its CPT data fit in the zone is a denser sand containing fines, the calculated $(q_{c1N})_{cs}$ for the soil with this recommendation may be reduced to only 50% of the "real" value calculated without the recommendation. This recommendation is on the conservative side in evaluating liquefaction potential of sandy soils. However, on the other hand, this recommendation may result in over-estimating of liquefaction-induced ground settlements for sites with denser sands containing fines that fit in that zone.

This seems to be true for some of the CPT soundings in the two case histories studied in this paper. For example, based on soil profiles, CPT profiles, and engineering judgement, a soil should be assessed as a denser sand containing fines, but a portion of the soil was evaluated as a very loose clean sand with K_c set to be one due to the recommendation. To investigate the effect of this recommendation on the calculated settlements for these two case history sites, the settlements were re-calculated without this recommendation and are shown in Table 5. The effect can be seen from the differences between the values calculated with and without this recommendation in Table 5. The differences are up to 14% for several CPT soundings tested in Treasure Island and negligible for the soundings tested in the Marina District (excluding the Eastern Part). It is understandable that the effect of this recommendation on calculated ground settlements may vary with sites and will depend on the amount of the soils that fit in the zone defined by $1.64 < I_c < 2.36$ and $F < 0.5\%$ within a soil profile for a site studied. If a large amount of the soils fit in the zone for the site, the effect would be much more significant than that for the two case history sites studied above. Therefore, soil sampling is recommended to further clarify soil properties for the specific sites where a large amount of the soils fit in the zone defined by $1.64 < I_c < 2.36$ and $F < 0.5\%$.

Cutoff line of I_c equal to 2.6

A cutoff line of I_c equal to 2.6 is set in the Robertson and Wride (1998) to distinguish the sandy and silty soils with clayey soils which are believed non-liquefiable in general. Gilstrap (1998) studied the case histories by using Robertson and Wride's method and compared the I_c calculated using the CPT soundings with the index test results of the samples that were taken from the boreholes close to the CPT locations at the case history sites. He found that more than 95% of the samples that had the associated CPT soundings with calculated I_c greater than 2.6 were classified as clayey soils based on the index test results. He then concluded that the I_c cutoff line of 2.6 is generally reliable for identifying clayey soils. However, he also noticed that 20% to 50% of the samples that had the associated CPT soundings with calculated I_c ranging from 2.4 to 2.6 were classified as clayey soils as well based on the index test results. This implies that the cutoff line of I_c equal to 2.6 appears slightly conservative.

To investigate the sensitivity of the calculated settlements to this cutoff line for the two case histories studied in this paper, a cutoff line of I_c equal to 2.5 was also tested. The calculated settlements using the new cutoff line are shown in Table 5. The differences between the calculated settlements for the cutoff lines of I_c equal to 2.6 and 2.5 are up to about 17% for several CPT soundings tested in Treasure Island and minor for the soundings tested the Marina District. It is understandable that the effect of the cutoff line with I_c equal to 2.6 on calculated ground settlements may vary with sites and will depend on the amount of the soils having calculated I_c ranging from 2.5 to 2.6 within a soil profile for a site studied. If a large amount of the soils has the calculated I_c ranging from 2.5 to 2.6 for the site, the effect would be much greater than that for the two case history sites studied above.

The combination influence of the recommendation for K_c and the cutoff line of I_c equal to 2.6 on the calculated settlements for the two case history sites was also investigated. The differences in the calculated settlements for the cases with and without the combination influence are up to 25% as shown in Table 5 for some of the CPT soundings in Treasure Island and minor for all the CPT soundings in the Marina District (except for the Eastern Part). As mentioned above, the effect may vary with sites and will depend on the amount of the soils having calculated I_c ranging from 2.5 to 2.6 or/and that fit in the zone defined by $1.64<I_c<2.36$ and $F< 0.5\%$ within the sites studied.

Ignoring influence of the recommendation for K_c and the cutoff line of I_c equal to 2.6 on the calculated ground settlements is on the conservative side. However, it may cause over-estimation of liquefaction-induced ground settlements for some sites where a large amount of the soils have a calculated I_c close to 2.6 or/and fit in the zone defined by $1.64<I_c<2.36$ and $F< 0.5\%$. Therefore, soil sampling is recommended to further clarify grain characteristics for the specific zones mentioned above.

RECOMMENDATIONS

Reasonable agreement between calculated settlements by the proposed CPT-based method and measured settlements at the two case history sites provides encouragement that the proposed methodology captures the dominant factors influencing liquefaction-induced ground settlements. Although further evaluations are required with future case history data, the proposed method appears to provide a satisfactory estimate of liquefaction-induced ground settlements, and should be useful for low to medium risk projects and also provide preliminary estimation for higher risk projects.

A number of factors may affect the accuracy of calculated settlements in estimating liquefaction-induced ground settlements. Maximum surface acceleration is one of the major factors. Its determination without measured values for the studied site is difficult and often leaves much uncertainty on the estimated liquefaction-induced ground settlements. For important projects, a site specific response analysis is required to determine maximum surface accelerations.

Fines content or mean grain size of sandy soils may affect liquefaction-induced ground settlements. However, their effect on the calculated settlements may be partially included in the proposed CPT-based approach. More studies are required to investigate this effect in the future research and no further correction is recommended at this stage.

The effects of transitional zone between a sandy soil and a soft clayey soil on cone tip resistance are obvious. They may also affect the estimation of liquefaction-induced ground settlements by using

CPT. However, these effects on the calculated settlements may also be partially incorporated with the proposed CPT-based approach. More studies are also required for the effects and no further correction is recommended at this stage, since the proposed CPT-based method is generally conservative in these cases.

Both vertical and horizontal distribution of liquefied layers in a site may play a role on ground surface settlements. Ignoring the effect of the three-dimensional distribution of liquefied layers on ground surface settlements may result in over-estimate liquefaction-induced ground settlements for some sites. Unfortunately, no reliable measure is available to quantify this effect at this stage. Therefore, engineering judgement is needed to consider the effect in order to avoid an overly conservative design.

Robertson and Wride's method may be conservative in evaluating liquefaction potential and estimating liquefaction-induced ground settlements for some sites where a large amount of the soils have a calculated I_c close to 2.6 or/and fit in the zone defined by $1.64<I_c<2.36$ and $F< 0.5$. Therefore, soil sampling with some index tests is strongly recommended to further clarify grain characteristics for the specific zones mentioned above.

CONCLUSIONS

An integrated CPT-based approach has been presented to estimate liquefaction-induced ground settlements for sites with level ground using CPT data. The approach combines an established CPT-based method for liquefaction potential analysis with laboratory test results to estimate the liquefaction-induced volumetric strains for sandy and silty soils.

The proposed methodology was used to estimate the liquefaction-induced ground settlements at the Marina District and Treasure Island case history sites devastated by liquefaction during the 1989 Loma Prieta earthquake. Good agreement between the calculated and measured liquefaction-induced ground settlements was found. The proposed CPT-based method provided better results when compared with the existing SPT-based method. It is suggested that the proposed CPT-based method may be used to estimate liquefaction-induced settlements for low to medium risk projects and also provide preliminary estimates for higher risk projects.

The major factors that affect estimation of liquefaction-induced ground settlements using CPT are also discussed in detail in this paper. These factors include maximum surface acceleration, fines content or mean grain size, the transitional zone at layer boundaries, three-dimensional distribution of liquefied soil layers, the correction factor K_c, and the cutoff line of I_c equal to 2.6. The recommendations for taking the effects of these factors into account in estimating liquefaction-induced ground settlements using the proposed CPT-based approach are also presented.

ACKNOWLEDGEMENTS

The authors would like to thank Professor J. P. Bardet from the University of Southern California and Dr. Ronaldo Luna from the Tulane University for providing the CPT soundings tested in the Marina District and Treasure Island, respectively and used in this research. Research funding was provided by the FS Chia PhD Scholarship awarded to the first author from the University of Alberta, and by the Natural Science and Engineering Research Council of Canada.

REFERENCES

Bardet, J. P., and Kapuskar, M. 1991. Site investigation of the Marina District of San Francisco in September 1990. Report to the National Science Foundation, University of Southern California, February, p. 94.

Bardet, J. P., Kapuskar, M., Martin, G. R., and Proubet, J. 1992. Site-response analyses. The Loma Prieta, California, Earthquake of October 17, 1989 - Marina District, U. S. Geological Survey Professional Paper 1551-F, pp. F85-F114.

Bennett, M. J. 1990. Ground deformation and liquefaction of soil in the Marina District. Chapter D of Effects of the Loma Prieta Earthquake on the Marina District, San Francisco, California, U.S. Geological Survey Open-File Report 90-253, pp. D1-D36.

Bennett, M. J. 1998. Sand boils and settlement on Treasure Island after the earthquake. The Loma Prieta, California, Earthquake of October 17, 1989 - Liquefaction, U. S. Geological Survey Professional Paper 1551-B, pp. B121-B128.

Berg, P. van den. 1994. Analysis of soil penetration. Delft University Press, the Netherlands, p. 175.

Boatwright, J., Seekins, L. C., Fumal, T. E., Liu, H. P., and Mueller, C. S. 1992. Ground-motion amplification. The Loma Prieta, California, Earthquake of October 17, 1989 - Marina District, U. S. Geological Survey Professional Paper 1551-F, pp. F35- F49.

Bonilla, M. G. 1992. Geologic and historical factors affecting earthquake damage. The Loma Prieta, California, Earthquake of October 17, 1989 – Marina District, U. S. Geological Survey Professional Paper 1551-F.

Boulanger, R. W., Idriss, I. M., and Mejia, L. H. 1995. Investigation and evaluation of liquefaction related ground displacements at Moss Landing during the 1989 Loma Prieta earthquake. Department of Civil & environmental Engineering, University of California at Davis, report No. UCD/CGM-95/02.

Campanella, R. G., and Robertson, P. K. 1988. Current status of the piezocone test. Proceedings of the International Symposium on Penetration Testing, ISOPT-1, Orlando, Balkema Pub., Rotterdam, Vol. 1, pp. 93 – 116.

Egan, J. A., and Wang, Z. L. 1991. Liquefaction-related ground deformation and effects on facilities at Treasure Island, San Francisco, during the 17 October 1989 Loma Prieta Earthquake. Proceedings of the Third Japan-U.S. Workshop on Earthquake Resistant Design of Lifeline Facilities and Countermeasures for Soil Liquefaction, Tech. Report NCEER-91-0001, pp.57-76.

Gilstrap, S. D. 1998. CPT based liquefaction resistance analyses evaluated using case histories, Master of Science Thesis, Department of Civil Environmental Engineering, Brigham Young University, Technical Report CEG-98-01, July.

Holzer, T. L., and O'Rourke, T. D., 1990. Effects of the Loma Prieta earthquake on the Marina District, San Francisco, California. U.S. Geological Survey, Open-file report 90-253.

Hryciw, R. D. 1991. Post Loma Prieta earthquake CPT, DMT and shear wave velocity investigations of liquefaction sites in Santa Cruz and on Treasure Island. Final Report to the U. S. Geological Survey, Award No. 14-08-0001-G1865, December.

Hryciw, R. D., Rollins, K. M., and Homolka, M. 1991. Soil amplification at Treasure Island during the Loma Prieta earthquake. Proceedings : Second International Conference on Recent Advances in Geotechnical Earthquake Engineering and Soil dynamics, March 11-15, St. Louis, Missouri, pp. 1679-1685.

Idriss, I. M. 1990. Response of soft soil sites during earthquakes. Proceedings H. B. Seed Memorial Symp., Vol. 2, BiTech Publishers, Vancouver, BC, May, pp.273-289.

Ishihara, K. 1985. Stability of natural deposits during earthquakes. Proceedings of the Eleventh International Conference on Soil Mechanics and Foundation Engineering, San Francisco, 12-16, August, pp. 321-376.

Ishihara, K., and Yoshimine, M. 1992. Evaluation of settlements in sand deposits following liquefaction during earthquakes. Soils and Foundations, **32**(1): 173-188.

Iwasaki, T., Tatsuoka, F., Tokida, K., and Yasuda, S. 1978. A practical method for assessing soil liquefaction potential based on case studies at various sites in Japan. Proceeding of the Second International Conference of Microzonation, San Francisco, CA, Vol. 2, pp.885-896.

Jamiolkowski, M., Ladd, C. C., Germaine, J. T., and Lancellotta, R. 1985. New developments in field and laboratory testing of soils. Proceedings of the Eleventh International Conference on Soil Mechanics and Foundation Engineering, San Francisco, 12-16 August, Vol. 1, pp. 57-153.

Juang, C. H., Chen, C. J., and Tien, Y. M. 1999a. Appraising CPT-based liquefaction resistance evaluation methods -- artificial neural network approach. Canadian Geotechnical Journal, **36**(3): 443-454.

Juang, C. H., Rosowsky, D. V., and Tang, W. H. 1999b. Reliability-based method for assessing liquefaction potential of soils. Journal of Geotechnical and Geoenvironmental Engineering, ASCE, **125**(8): 684-689.

Law, K. T. 1990 Geotechnical aspects. The 1989 Loma Prieta (San Francisco Area) Earthquake: Site Visit Report, Internal Report No. 594, National Research Council Canada, May, pp. 9-20.

Lee, K. L., and Albaisa, A. 1974. Earthquake induced settlements in saturated sands. Journal of Geotechnical Engineering, ASCE, **100**(GT4): 387-406.

Nagase, H. and Ishihara, K. 1988. Liquefaction-induced compaction and settlement of sand during earthquakes. Soils and Foundations, **28**(1): 65-76.

NCEER 1997. Proceeding of the NCEER Workshop on Evaluation of Liquefaction Resistance of Soils, Edited by Youd, T. L., and Idriss, I. M., Technical Report NCEER-97-0022, Salt Lake City, Utah, Decemeber 31, 1997.

O'Rourke, T. D., Gowdy, T. E., Stewart, H. E., and Pease, J. W. 1991. Lifeline performance and ground deformation in the Marina During 1989 Loma Prieta Earthquake. Proceedings of the Third Japan-U.S. Workshop on Earthquake Resistant Design of Lifeline Facilities and Countermeasures for Soil Liquefaction, Tech. Report NCEER-91-0001, pp.129-146.

O'Rourke, T. D., and Pease, J. W. 1997. Mapping liquefaction layer thickness for seismic hazard assessment. Journal of Geotechnical and Geoenvironmental Engineering, ASCE, **123**(1): 46-56.

Power, M. S., Egan, J. A., Shewbridge, S. E., deBacker, J., and Faris, J. R. 1998. Analysis of liquefaction-induced damage on Treasure Island. The Loma Prieta, California, Earthquake of October 17, 1989 - Liquefaction, U. S. Geological Survey Professional Paper 1551-B, pp. B87-B120.

Robertson, P. K., and Campanella, R. G. 1985. Liquefaction potential of sands using the cone penetration test. Journal of Geotechnical Engineering, ASCE, **22**(3): 298-307.

Robertson, P. K., and Fear, C. E. 1995. Liquefaction of sands and its evaluation. Proceedings, IS Tokyo '95, 1st International Conference on Earthquake Geotechnical Engineering, Keynote Lecture.

Robertson, P. K., and Wride C. E. 1998. Evaluating cyclic liquefaction potential using the CPT. Canadian Geotechnical Journal, **35**(3): 442-459.

Rollins, K. M., and McHood, M. D. 1998. Comparison of computed and measured liquefaction-induced settlements in the Marina District, San Francisco. The Loma Prieta, California, Earthquake of October 17, 1989 - Liquefaction, U. S. Geological Survey Professional Paper 1551-B, pp. B223-B240.

Sanglerat, G. 1972. The penetrometre and soil exploration. Elsevier Scientific Publishing Company, Amsterdam-Oxford-New York.

Seed, H. B., and Idriss, I. M. 1971. Simplified procedure for evaluation soil liquefaction potential. Journal of the Soil Mechanics and Foundations Division, ASCE, **97**(SM9): 1249-1273.

Seed, H. B., and Idriss, I. M. 1982. Ground motions and soil liquefaction during earthquakes. Earthquake Engineering Research Institute, p.134.

Seed, H. B., Tokimatsu, K., Harder, L. F., and Chung, R. M. 1985. Influence of SPT procedures in soil liquefaction resistance evaluations. Journal of Geotechnical Engineering, ASCE, **111**(12): 1425-1440.

Seed, R. B., and Harder, L. F. 1990. SPT-based analysis of cyclic pore pressure generation and undrained residual strength. Proceedings H. B. Seed Memorial Symp., Vol. 2, BiTech Publishers, Vancouver, BC, May, pp.351-376.

Seed, R. B., Dickenson, S., and Mok, C. M. 1994. Site effects on strong shaking and seismic risk: recent developments and their impact on seismic design codes and practice. Strctural Congress XII, Vol. 1, ASCE, New York, pp.573-578.

Tan, C. K., and Duncan, J. M. 1991. Settlement of footings on sands-accuracy and reliability, in Geotechnical Engineering Congress, Boulder, Colo., Proceedings: American Society of Civil engineerings Special Publication 27, V. 1, pp. 446 –455.

Tatsuoka, F., Iwasaki, T., Tokida, K., Yasuda, S., Hirose, M., Imai, T., and Kon-no, M. 1980. Standard penetration tests and soil liquefaction potential evaluation. Soils and Foundations, JSSMFE, **20**(4): 95-111.

Tatsuoka, F., Zhou, S., Sato, T., and Shibuya, S. 1990. Method of evaluating liquefaction potential and its application. Seismic Hazards in the Soil Deposits in Urban Areas, pp. 75-109 (in Japanese).

Taylor, H. T., Cameron, J. T., Vahdani, S., and Yap, H. 1992. Behavior of the seawalls and shoreline during the earthquake. The Loma Prieta, California, Earthquake of October 17, 1989 - Marina District, U. S. Geological Survey Professional Paper 1551-F, pp. F141-F153.

Tokimatsu, K. and Yoshimi, Y. 1981. Field correlation of soil liquefaction with SPT and grain size. Proceedings of Eight World Conference on Earthquake Engineering, San Francisco, CA, 95-102.

Tokimatsu, K., and Seed, H. B. 1987. Evaluation of settlements in sands due to earthquake shaking. Journal of Geotechnical Engineering, ASCE, **113**(8): 861-879.

Vreugdenhil, R. A. 1995. Interperation of poezocone data and its use in estimating seismic soil liquefaction potential. Ph.D. Thesis, University of Canterbury, New Zealand, May, p. 183.

Youd, T. L., Garris, C. T. 1995. Liquefaction-induced ground-surface disruption. Journal of Geotechnical and Geoenvironmental Engineering, ASCE, **121**(11): 805-808.

Zhou, S. G. 1981. Influence of fines on evaluating liquefaction of sand by CPT. Proceedings of International Conference on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics, St. Louis, MO, 1, 167-172.

Table 1. Comparison of the liquefaction-induced ground settlements measured and calculated using the proposed CPT-based method for the Marina District

CPT	a_{max}	Ground water level (Bonilla 1992) (m)	Calculated settlement using proposed CPT-based method (cm)	Measured settlement (Bennett 1990) (cm)
M6	0.12	5.5	2.3	0.0 – 1.6
M1	0.16	2.3	5.9	0.0 – 3.4
M2	0.16	2.7	1.9	0.0 – 3.4
M3	0.16	2.7	1.0	1.1
C-8	0.16	2.7	3.0	1.9
C-9	0.16	2.6	0.1	0.0 – 3.4
M4	0.24	2.4	11.2	9.6
C-2	0.24	2.3	12.1	9.6 – 10.7
C-12	0.24	2.3	9.4	7.0 – 10.7

Table 2. Comparison of the liquefaction-induced ground settlements measured and calculated using the SPT based method and the proposed CPT-based method for the Marina District

Soil type at the Marina District	Liquefaction-induced ground settlement (cm)			
	Measured (Bennett 1990)	SPT-based method (O'Rourke et al. 1991)	SPT-based method (Rollins and McHood 1998)	Proposed CPT-Based Method
Dune sand at the lower eastern part	0 – 2.0	3.0 – 4.0	0.5 – 1.5	2.3
Beach sand or old fill at the western part	0.1 – 4.0	5.0 – 6.0	0.5 – 8.0	0.1 – 5.9
Hydraulic fill at the central part	7.0 – 12.0	17.0 – 24.0	12.5 – 24.5	9.4 – 12.1

Table 3. Comparison of the liquefaction-induced ground settlements measured and calculated using the proposed CPT-based method for Treasure Island

Zone	CPT	Calculated settlement using proposed CPT-based approach (cm)	Observed settlement (Power et al. 1998) (cm)
Zone one	C-28	15.9	5 to 10 cm (about 2 to 4 inches)
	C-32	14.4	
	C-33	15.3	
	C-34	12.2	
	C-35	10.7	
	C-37	11.8	
	C-42	17.2	
Zone two	C-29A	18.8	10 to 15 cm (about 4 to 6 inches)
	C-30	27.0	
	C-31	33.3	
	C-39	23.6	
	UM10	25.8	

Table 4. Recorded, calculated, and assumed a_{max} associated with the Marina District during the Loma Prieta earthquake

a_{max} (g)	Method	Reference	Comments
0.05 – 0.11,	Recorded	Bardet et al., 1992	at five bedrock sites within 7 km from the Marina District
0.13 – 0.17	Recorded	Bardet et al., 1992	at three artificial fill sites within 7 km from the Marina District
0.12 – 0.15	Calculated	Bardet et al., 1992	one-dimensional site response analysis
0.20 – 0.23	Calculated	Bardet et al., 1992	two-dimensional site response analysis
0.15 ± 0.05	Calculated	Rollins and McHood, 1998	one-dimensional site response analysis
0.16 – 0.32	Estimated	Holzer and O'Rourke, 1990	possible acceleration range
0.12 – 0.17	Estimated	Taylor et al., 1992	possible acceleration range
≥ 0.25	Estimated	Boatwright et al., 1992	possible acceleration in the central part (hydraulic fill)
0.16 and 0.32	Assumed	Bennett, 1990	liquefaction potential analysis using SPT data
0.2	Assumed	O'Rourke et al., 1991	liquefaction potential analysis using SPT data
0.24	Assumed	Gilstrap, 1998	liquefaction potential analysis using CPT data

Table 5. Liquefaction-induced ground settlements measured and calculated using the CPT-based approach with and without some modifications for the Marina District and Treasure Island

Site	Zone	Liquefaction-induced ground settlement (cm)				
		Calculated(1) (basic procedure)	Calculated(2) (without the recommendation for K_c)	Calculated(3) (with the cutoff line of $I_c = 2.5$)	Calculated(4) (without the recommendation for K_c and with the cutoff line of $I_c = 2.5$)	Observed(5)
Marina District	Eastern Part	2.3	0.9	2.3	0.9	0.0 – 1.6
	Western Part	0.1 – 5.9	0.1 – 5.3	0.1 – 5.4	0.1 – 4.8	0.0 – 3.4
	Central Part	9.4 – 12.1	9.4 – 11.8	8.8 – 10.0	8.6 – 9.4	7.0 – 10.7
Treasure Island	Zone one	10.7 – 17.2	10.7 – 16.4	9.8 – 15.5	9.7 – 14.7	5.0 – 10.0
	Zone two	18.8 – 33.3	18.1 – 30.6	17.3 – 27.4	16.5 – 24.8	10.0 – 15.0

Note:

(1) Calculated settlement by using the basic CPT-based approach (with the caution recommendation of K_c and the cutoff line of $I_c = 2.6$).
(2) Same as (1) but without the caution recommendation of setting K_c to be one for the soils that fit in the zone defined by $1.64 < I_c < 2.36$ and $F < 0.5\%$.
(3) Same as (1) but set the cutoff line of $I_c = 2.5$ instead of using the cutoff line of $I_c = 2.6$
(4) Same as (1) but without the caution recommendation of setting K_c to be one and with the cutoff line of $I_c = 2.5$
(5) Measured or observed ground settlements in the Marina District (Bennett 1990) and Treasure Island (Power et al. 1998).

q_c : tip resistance, f_s : sleeve friction
σ_{vo} , σ_{vo}' : in-situ vertical total and effective stress
units: all in kPa

initial stress exponent : n = 1.0 and calcualte Q, F, and I_c
if $I_c <= 1.64$, $n = 0.5$
if $1.64 < I_c < 3.30$, $n = (I_c - 1.64)*0.3 + 0.5$
if $I_c >= 3.30$, $n = 1.0$
iterate until the change in n, $\Delta n < 0.01$
if $\sigma_{vo}' > 300$ kPa, let n = 1.0 for all soils

$$C_n = \left(\frac{100}{\sigma_{vo}'}\right)^n$$

$$Q = \frac{(q_c - \sigma_{vo})}{100} \cdot C_n \quad , \quad F = \frac{f_s}{(q_c - \sigma_{vo})} \cdot 100$$

$$I_c = \sqrt{\left[(3.47 - \log Q)^2 + (1.22 + \log F)^2\right]}$$

if $I_c <= 1.64$, $K_c = 1.0$
if $1.64 < I_c < 2.60$, $K_c = -0.4031 I_c^4 + 5.581 I_c^3 - 21.63 I_c^2 + 33.75 I_c - 17.88$
if $I_c >= 2.60$, evaluate using other criteria; likely non- liquefiable if $F > 1\%$
<u>BUT</u>, if $1.64 < I_c < 2.36$ <u>and</u> $F < 0.5\%$, set $K_c = 1.0$

$$(q_{c1N})_{cs} = K_c \cdot Q$$

$$CRR_{7.5} = 93 \cdot \left(\frac{(q_{c1N})_{cs}}{1000}\right)^3 + 0.08 \text{ , if } 50 <= (q_{c1N})_{cs} < 160$$

$$CRR_{7.5} = 0.833 \cdot \left(\frac{(q_{c1N})_{cs}}{1000}\right) + 0.05 \text{ , if } (q_{c1N})_{cs} < 50$$

if $I_c >= 2.60$, evaluate using other criteria; likely non- liquefiable if $F > 1\%$

Figure 1. An updated flow chart for estimating cyclic resistance ratio at earthquake moment magnitude of 7.5, $CRR_{7.5}$, using Robertson and Wride's CPT-based method

Figure 2. Curves for estimating post-liquefaction volumetric strain of clean sands (after Ishihara and Yoshimine, 1992)

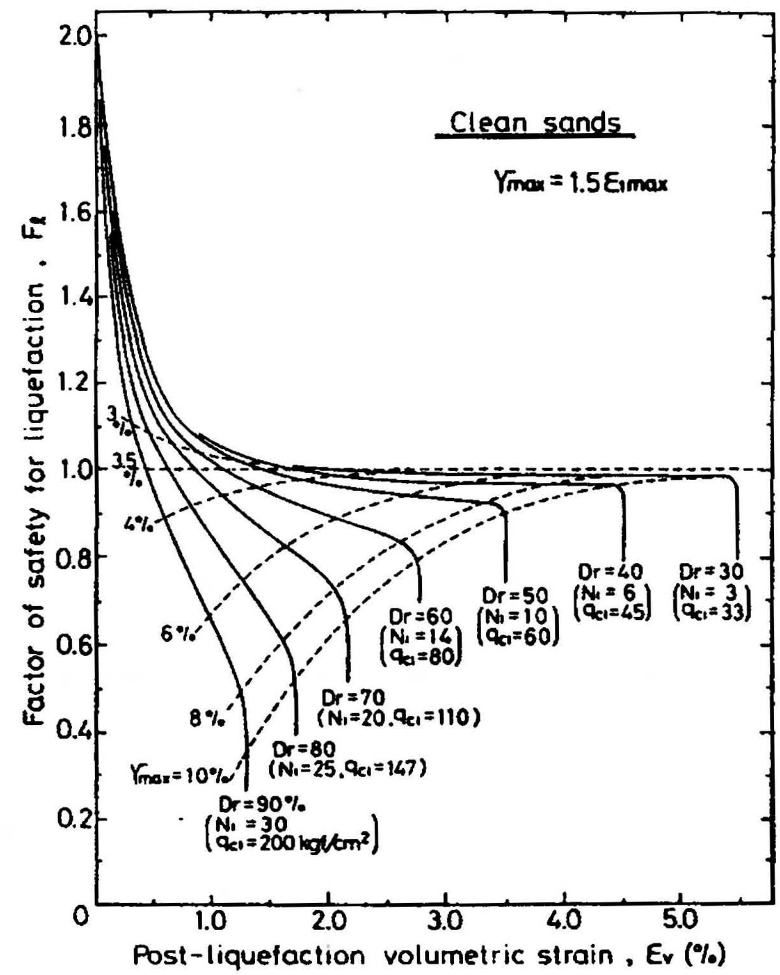

Figure 3. Relationship between ε_v and $(q_{c1N})_{cs}$ for different factor of safety (FS)

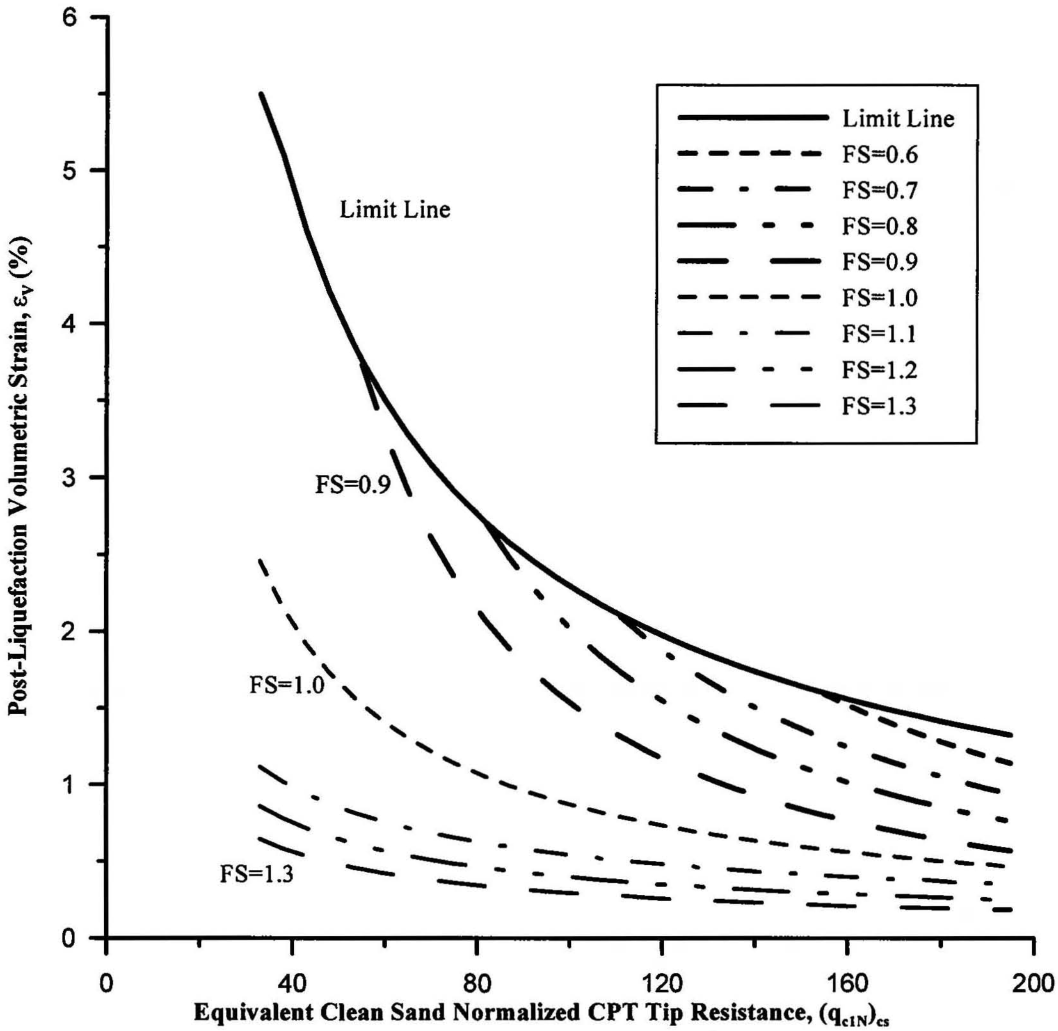

Figure 4. Example plots illustrating major procedures in performing liquefaction potential analysis using the CPT based Robertson and Wride's (1998) method

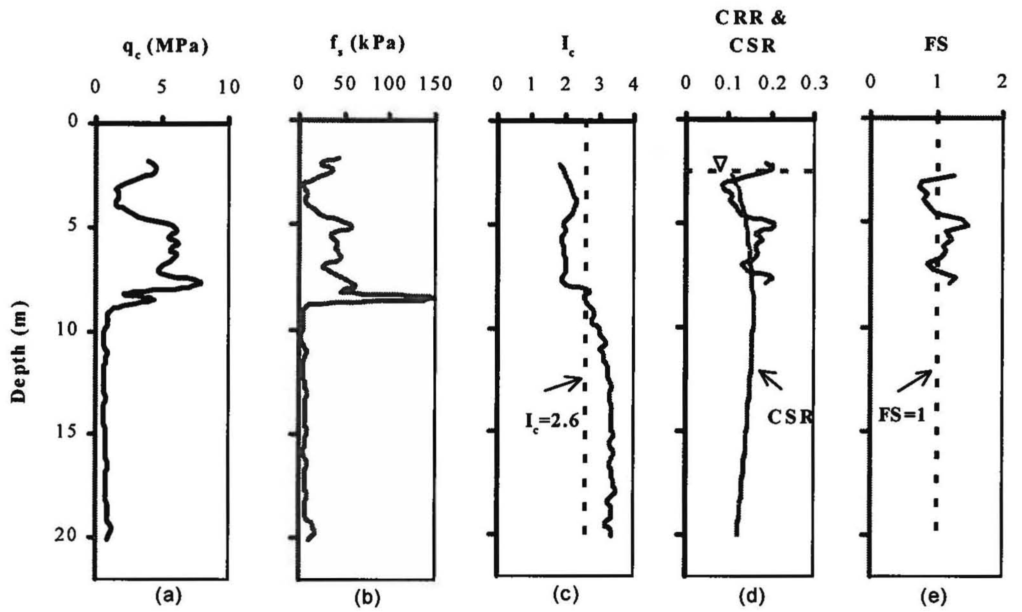

Figure 5. Example plots illustrating major procedures in estimating liquefaction-induced ground settlements using the proposed CPT based method

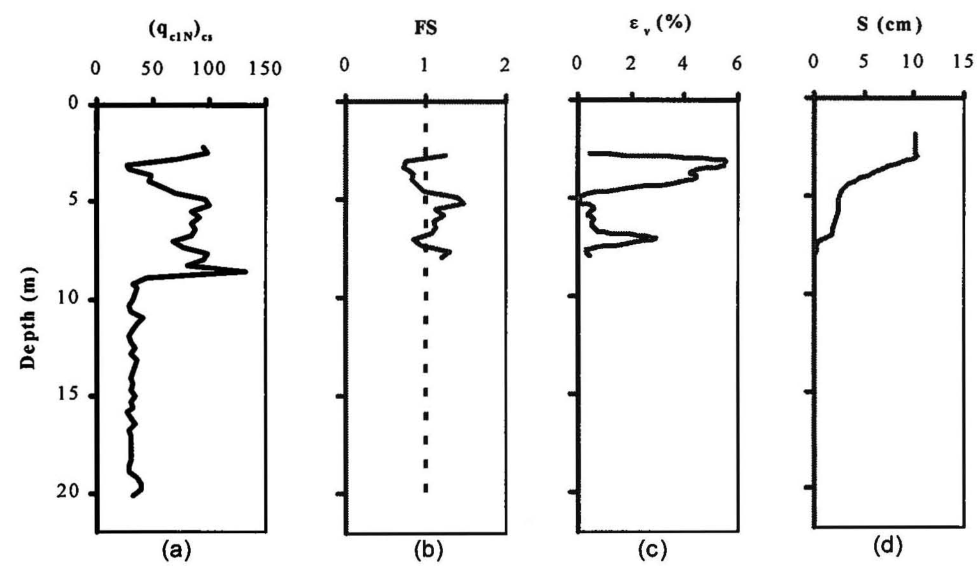

Figure 6. Plan view of the geologic units and CPT locations at the Marina District (after Bennett, 1990)

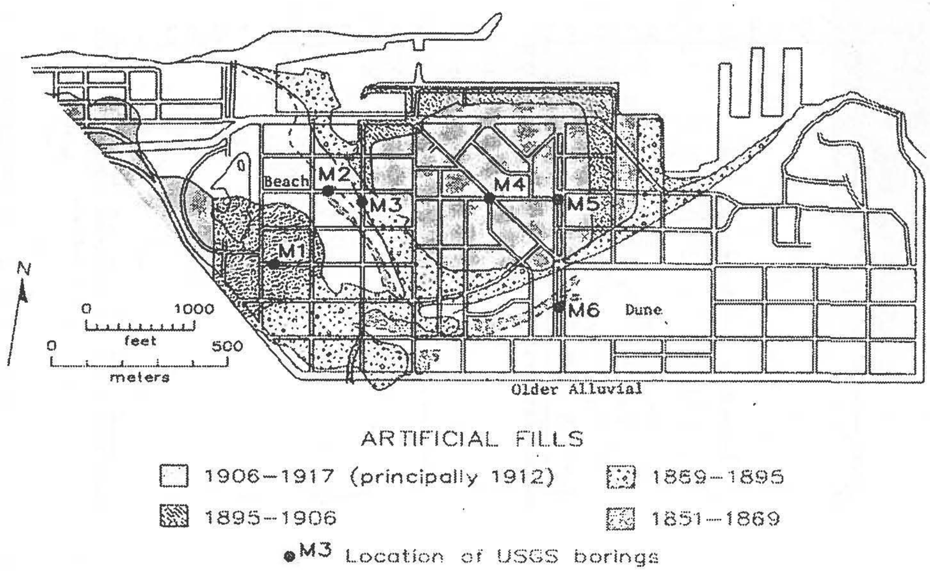

Figure 7. Typical geological profile at Treasure Island (modified from Power et al., 1998)

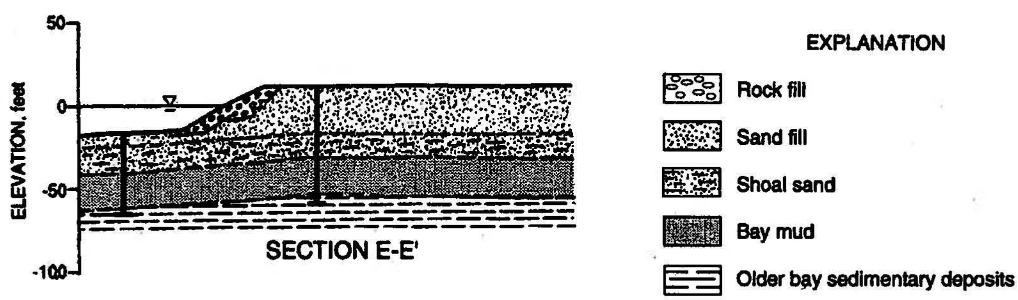

Figure 8. Effect of inputted peak surface acceleration on calculated settlements for some CPTs at the Marina District

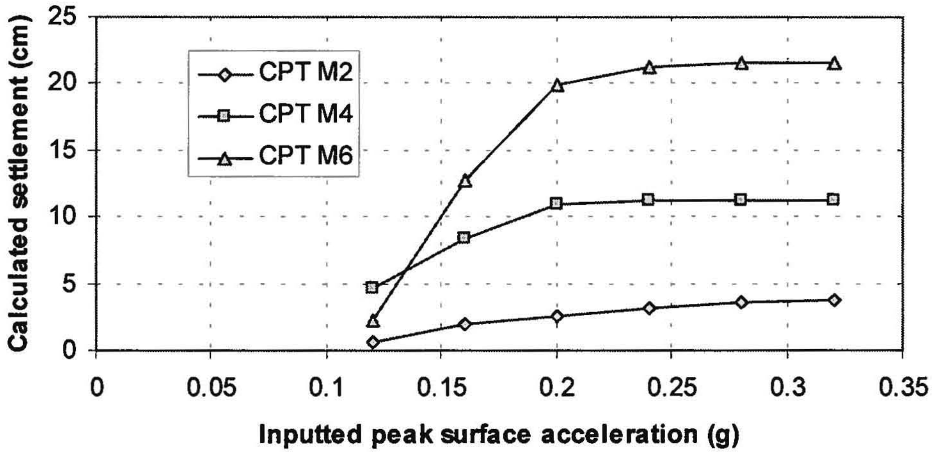

Figure 9. Sketches illustrating three-dimensional distribution of liquefied layers