Estimating liquefaction-induced ground settlements from CPT for level ground

G. Zhang, P.K. Robertson, and R.W.I. Brachman

Abstract: An integrated approach to estimate liquefaction-induced ground settlements using CPT data for sites with level ground is presented. The approach combines an existing 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 on 17 October 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.

Key words: liquefaction, settlements, earthquake, sand, in situ testing.

Résumé : On présente une approche intégrée au moyen de données de CPT pour estimer les tassements du terrain induits par liquéfaction pour les sites ayant une surface à niveau. L'approche combine une méthode existante basée sur le CPT pour estimer la résistance à la liquéfaction avec des résultats d'essais en laboratoire sur le sable propre pour évaluer les déformations volumétriques induites par liquéfaction pour les sols sableux et silteux. La méthode proposée a été utilisée pour évaluer les tassements aux deux sites de Marina District et de Treasure Island endommagés par liquéfaction durant le tremblement de terre de Loma Prieta, Californie, le 17 octobre 1989. On a trouvé une bonne concordance entre les tassements de terrain induits par liquéfaction calculés et mesurés. On discute aussi en détail les principaux facteurs qui influencent l'évaluation des tassements du sol induits par liquéfaction. On présente les recommandations pour prendre en compte les effets de ces facteurs dans l'évaluation des tassements du sol induits par liquéfaction au moyen de l'approche proposée basée sur le CPT. On suggère que la méthode proposée peut être utilisée pour estimer les tassement induits par liquéfaction pour les projets de risques moyens à faibles, et aussi pour fournir des estimations préliminaires pour les projets à risques plus élevés.

Mots clés : liquéfaction, tassements, tremblement de terre, sable, essais in situ.

[Traduit par la Rédaction]

Introduction

Liquefaction of loose, saturated granular soils during earthquakes poses a major hazard in many regions of the world. 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 penetra-

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tion 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 conditions. 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 on 17 October 1989. Good agreement between the calculated and measured liquefaction-induced ground settlements was found. The influence of maximum surface acceleration, fines content, 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) on the calculated liquefaction-induced ground settlements are discussed. Guidance for taking the effects of these factors into account when 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 (Youd and Idriss 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.

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. 1999*a*). In addition, Juang et al. (1999*b*) 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 20 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 quantities is required for the evaluation of liquefaction potential of soils. These quantities are: the seismic demand placed on a soil layer by a given earthquake, expressed in terms of the cyclic stress ratio (CSR); and the capacity of the soil to resist liquefaction, expressed in terms of the cyclic resistance ratio (CRR) (Youd and Idriss 1997). If CSR exceeds CRR, liquefaction of the soil is highly likely during the earthquake.

An updated flow chart for evaluating the CRR of sandy soils using the CPT-based method proposed by Robertson and Wride (1998) is shown in Fig. 1. A new set of values of the stress exponent, n, for normalizing cone penetration resistance is introduced in Fig. 1. These updated values of n were proposed by Robertson (1999) to produce a smoother variation of n for soils from clean sands to clays, because a somewhat discontinuous variation of n was applied in Robertson and Wride (1998). Note that the new values of n for clean sands and clays are the same as those used in Robertson and Wride (1998) and are only different for silty soils. Nevertheless, the choice of n value has little effect on the calculated normalized penetration resistance if the in situ effective overburden stresses are in the order of 50–150 kPa.

Important parameters in Robertson and Wride's method 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, $(q_{c1N})_{cs}$. The soil behavior type index I_c is a function of the normalized CPT penetration resistance (Q) and the normalized friction ratio (F). The CRR 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 CSR 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 elsewhere (Youd and Idriss 1997).

Integrated CPT-based approach to estimate liquefaction-induced ground settlements

Postliquefaction volumetric strain from laboratory tests

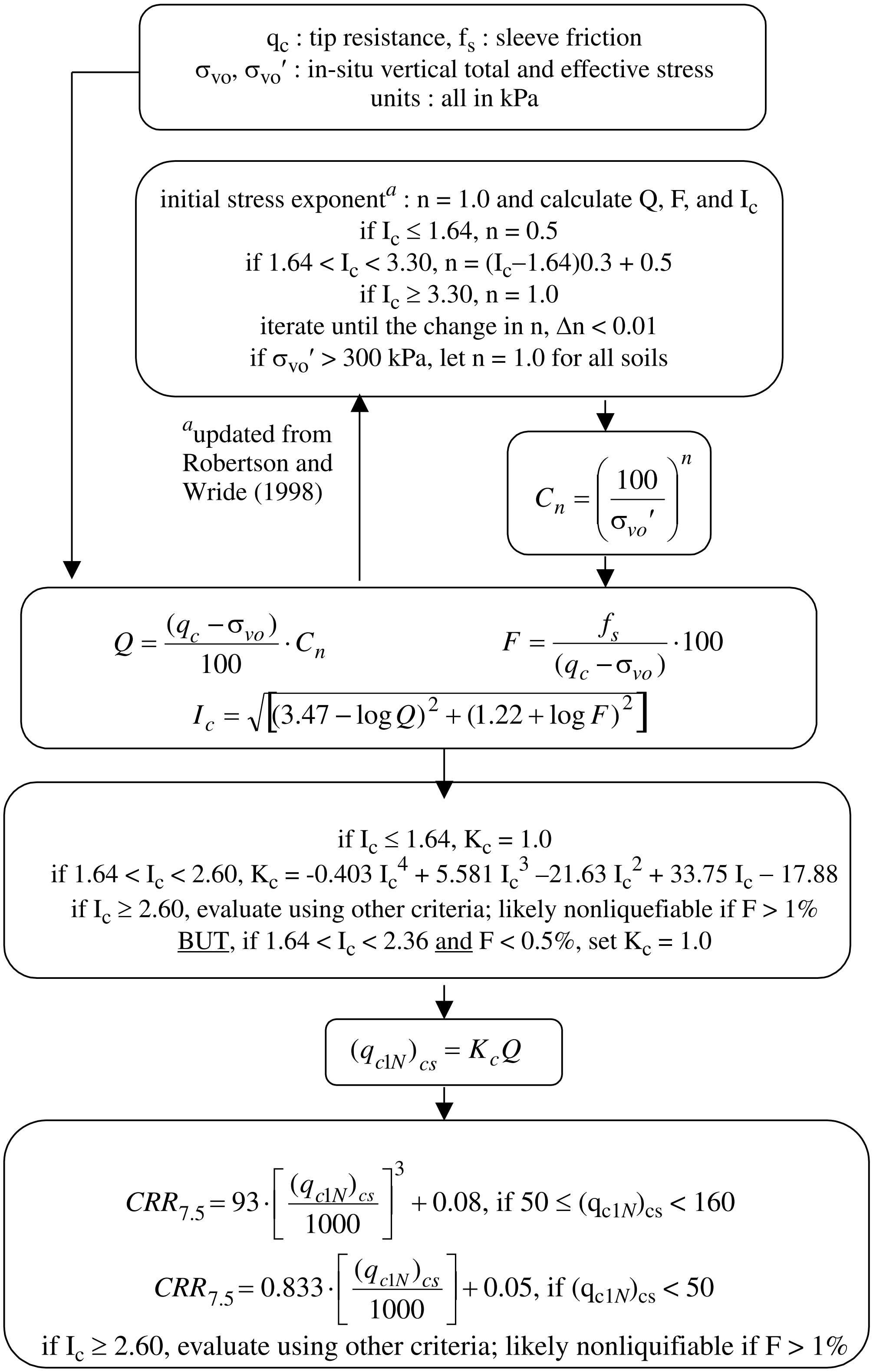
Nagase and Ishihara (1988) conducted cyclic simple shear tests on saturated loose, medium-dense, and dense samples of Fuji River sand. Both unidirectional and multidirectional loading conditions were simulated by employing irregular time histories of motions observed during major earthquakes in Japan 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 reconsolidation 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 Fig. 2, from which the volumetric strain resulting from dissipation of pore-water pressures was correlated with relative density (D_r) and the factor of safety against liquefaction (FS) for clean sands. These curves are used to estimate postliquefaction volumetric strain for clean sands in this paper.

Relative density from the CPT

Relative density D_r cannot 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 lab-

Fig. 1. Flowchart for evaluation of $CRR_{7.5}$ using Robertson and Wride's (1998) CPT-based method.



oratory 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 the grain characteristics of sands may affect the correlation between D_r and q_c. The 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

Fig. 2. Curves for estimating the postliquefaction volumetric strain of clean sands (modified from Ishihara and Yoshimine 1992).

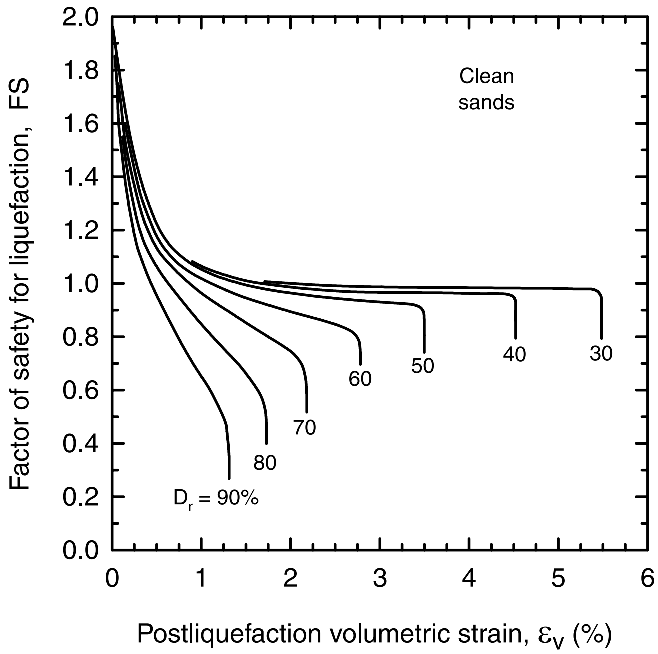
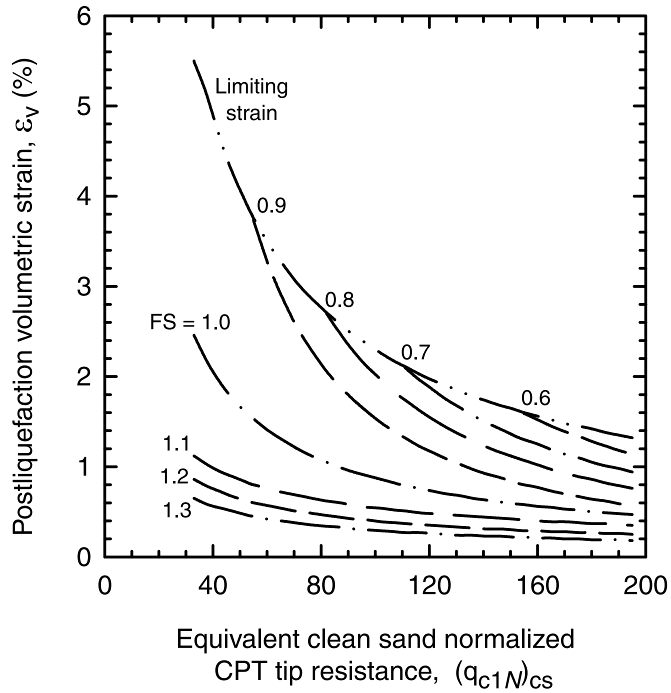


Fig. 3. Relationship between postliquefaction volumetric strain and equivalent clean sand normalized CPT tip resistance for different factors of safety (FS).



al. (1990) is used in this paper, since this method provides slightly smaller and more conservative estimates of D_r than the correlation by Jamiolkowski et al. (1985) when q_c is less than about 10 MPa.

Correction for grain characteristics

The curves of Fig. 2 were based on laboratory test results on clean sand. If these curves are used to estimate the postliquefaction 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 postliquefaction volumetric strain at certain values of FS.

An alternate approach is to estimate the D_r of silty soils using the CPT and then use D_r and FS to evaluate the postliquefaction 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 postliquefaction 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 postliquefaction volumetric strain. The proposed CPT-based method in this paper uses the first approach (i.e., $(q_{c1N})_{cs}$ accounts for grain characteristics or fines content) to estimate postliquefaction volumetric strains for sandy and silty soils and will be further discussed later in this paper. The correlations between $(q_{c1N})_{cs}$ and postliquefaction volumetric strain (ε_v) for different FS were developed on the basis of the curves of Ishihara and Yoshimine (1992) and are shown in Fig. 3. The equations for these relationships are given in Appendix A.

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 postliquefaction volumetric strain can then be estimated using Fig. 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 eq. [1], the result should be an appropriate index of potential liquefaction-induced ground settlement at the CPT location due to the design earthquake,

$$S = \sum_{i=1}^{j} \varepsilon_{vi} \Delta z_i \qquad [1]$$

where S is the calculated liquefaction-induced ground settlement at the CPT location; ε_{vi} is the postliquefaction volumetric strain for the soil sublayer i; Δz_i is the thickness of the sublayer i; and j is the number of soil sublayers.

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 and cyclic stress ratio CSR, and factor of safety against liquefaction FS, respectively. The data in Figs. 4*a* and 4*b* can be directly obtained from the CPT sounding. Figures 4*c*, 4*d*, and 4*e* show the results calculated based on the procedure shown in Fig. 1. Note that, according to Robertson and Wride's approach, the CRR is not calculated when the soil behavior type index is greater than 2.6. These soils are assumed to be nonliquefiable 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 Fig. 5. Figures 5*a*–5*d* show the profiles of equivalent clean sand normalized tip resistance $(q_{c1N})_{cs}$, factor of safety FS, postliquefaction volumetric strain ε_v, and liquefaction induced ground settlement S, respectively. Data in Figs. 5*a* and 5*b* are from the liquefaction potential analysis. Data in Fig. 5*c* are calculated from the curves of Fig. 3. The settlement shown in Fig. 5*d* is obtained using eq. [1], and the volumetric strains are from Fig. 5*c*.

Evaluation of proposed integrated CPT-based approach by case histories

In the past 20 years, a number of postliquefaction CPTs have been conducted at sites around the world. 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 CPT-based method for estimating liquefaction-induced ground settlements by comparing estimated settlements with those measured in the field. Of 15 case history sites in the U.S.A. with CPT soundings, eight involved lateral spreads and five have postliquefaction 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 — 1989 Loma Prieta earthquake

The Marina District is located on the north side of San Francisco, California and experienced significant damage, 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, the U.S. Geological Survey (Reston, VA) conducted a subsurface investigation that included five CPT soundings and measured the vertical settlements caused by the earthquake (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 2–3 m and thus were not 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. The locations of the nine CPTs (M1, M2, M3, M4, M6, C2, C8, C9, and C12) used to evaluate the proposed CPT-based method are shown in Fig. 6.

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 also shows the general locations of the geologic units in the Marina District. The upper 8 m of the western part of the district contains mainly beach sand deposits. CPT soundings M1, M2, M3, C8, and C9 penetrated in this material. The upper 8 m towards the central part of the district consists of sand to silty-sand fill. Much of this fill was hydraulically placed in 1912 with no compaction (Rollins and McHood 1998). CPT soundings M4, C2, and C12 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 with no bay mud beneath the dune sand. CPT M6 penetrated the dune sand sediment.

The input data for the proposed CPT-based method includes the following: 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 mainshock accelerograms was located on bedrock in Pacific Heights, approximately 1.5 km south of the Marina District (Boatwright et al. 1992). Based on the work by Idriss (1990), Seed et al. (1994), and others, values of a_{max} of 0.12, 0.16, 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 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 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. The calculated settlements are quite similar to the measured/estimated settlements with the calculated settlements 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. Their calculated settlements, using a peak ground acceleration of 0.2g and an earthquake magnitude of 7.1, are presented in Table 2. Results from Rollins and McHood (1998) obtained using the same SPT-based method, but with a peak ground acceleration of 0.15 ± 0.05g and an earthquake magnitude of 6.75, are also given in Table 2. 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 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

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

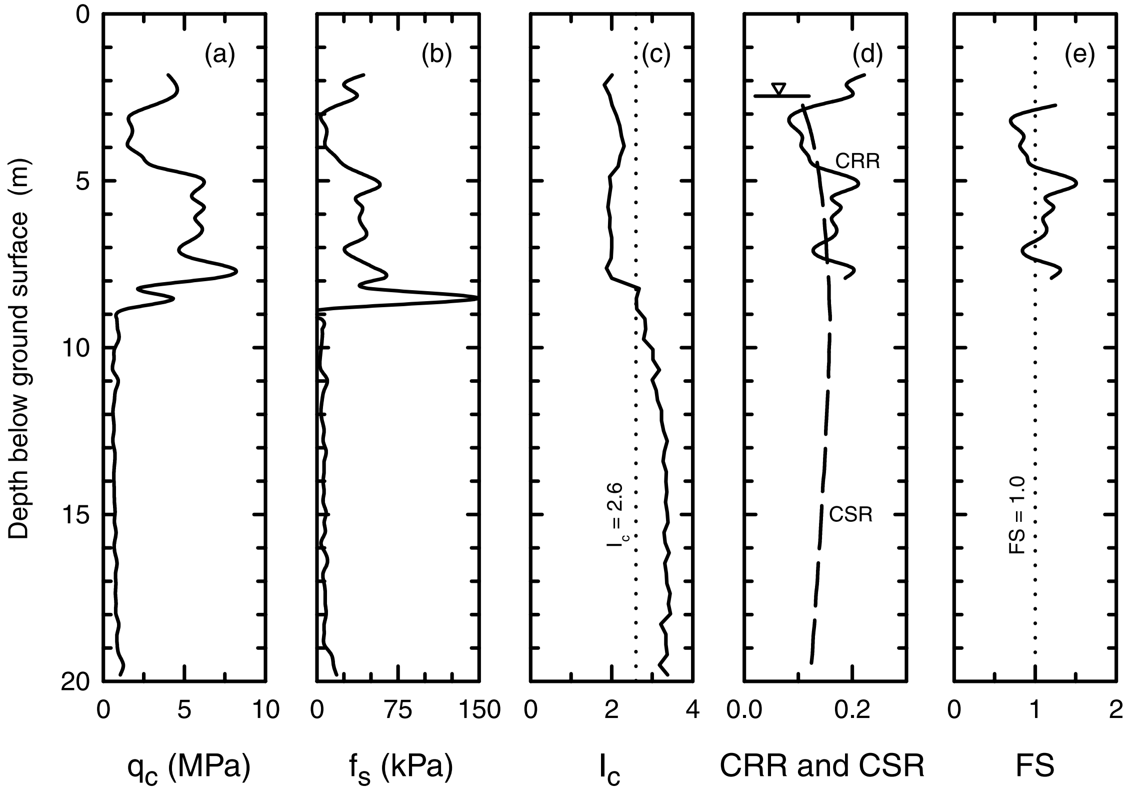


Fig. 5. Example plots illustrating the major procedures in estimating liquefaction-induced ground settlements using the proposed CPT-based method.

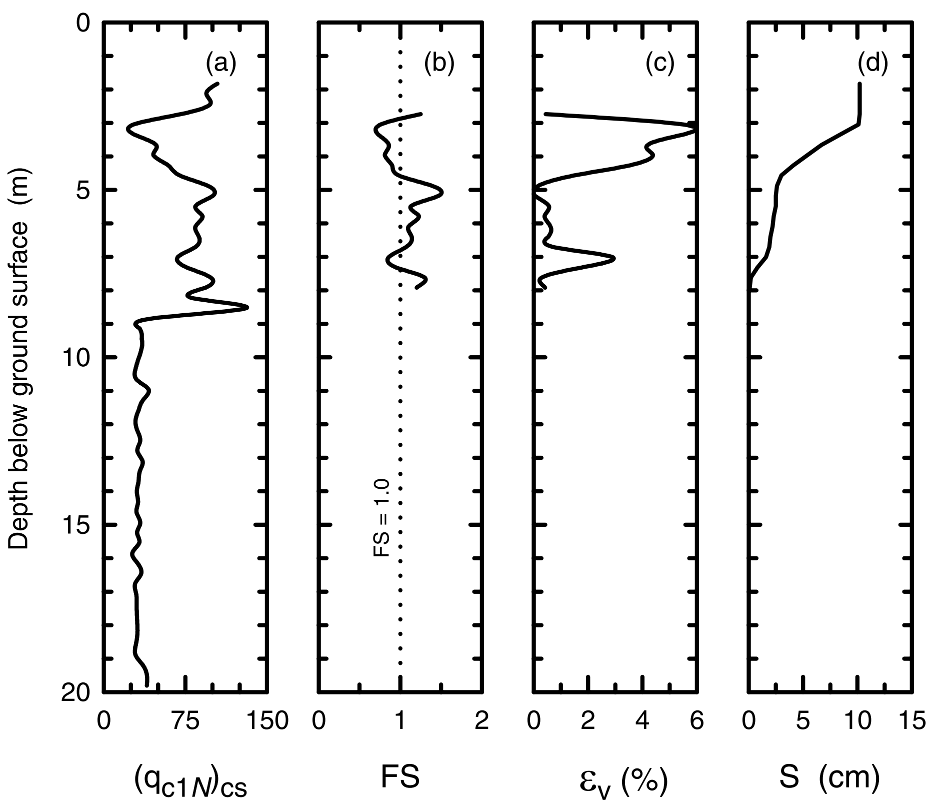
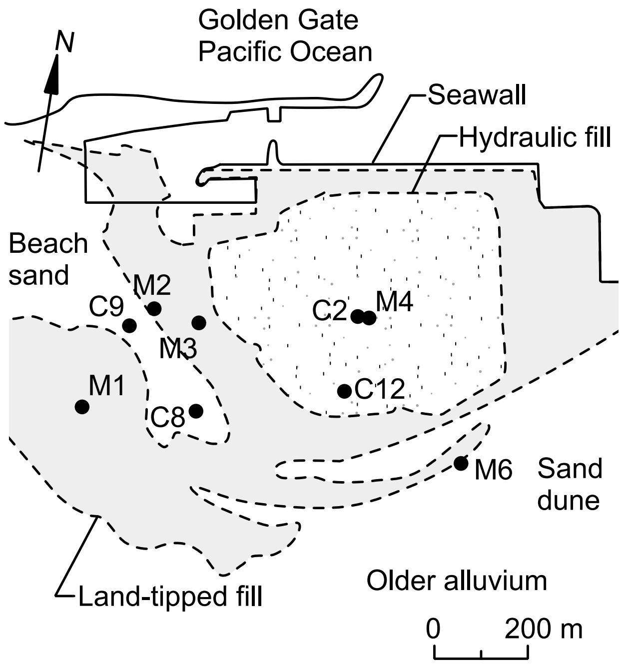


Fig. 6. Plan view showing the major geologic units and CPT locations at Marina District (modified from Bennett 1990).



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 — 1989 Loma Prieta earthquake

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 and (or) in the island were damaged by the ground settlements and lateral displacements (Egan and Wang 1991).

Following the 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 42 locations around the island (Power et al. 1998; Hryciw 1991). However, only 12 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 generally consist of hydraulically placed sand fill, native shoal sand and clay, recent bay sediments, and older bay sediments (Power et al. 1998), as shown in Fig. 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 perimeter of the island. 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 to 15 m. The recent bay mud consists primarily of soft to stiff silty clay and ranges in thickness from about 4.5 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.

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
C8	0.16	2.7	3.0	1.9
C9	0.16	2.6	0.1	0.0–3.4
M4	0.24	2.4	11.2	9.6
C2	0.24	2.3	12.1	9.6–10.7
C12	0.24	2.3	9.4	7.0–10.7

Treasure Island is relatively flat. The ground water levels in the island are typically at depths of 1.5–2.4 m below the ground surface and a depth of 2.0 m was assumed for this study. Ground motion was recorded at the fire station on Treasure Island during the earthquake. The recorded peak acceleration was 0.16g (Hryciw et al. 1991). Ground response analyses (Hryciw et al. 1991; Power et al. 1998) have shown that the intensity of ground shaking did not vary greatly in different places on the island. Consequently, a peak acceleration of 0.16g was used in this study. Again, a moment magnitude of 7.0 was used to model the 1989 Loma Prieta earthquake. Average total unit weights of 15.0 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 were available at the locations where the CPTs were conducted. However, the pre-earthquake and postearthquake data from nine survey benchmarks in the island indicated that the total settlements generally ranged from 5 to 15 cm (2–6 in) (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 in) (Power et al. 1998).

The calculated settlements and the ranges of observed liquefaction-induced ground settlements for Treasure Island are provided in Table 3. The proposed CPT-based method has correctly predicted large settlements within zone two. In

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

Fig. 7. Typical ground conditions at Treasure Island (modified from Power et al. 1998).

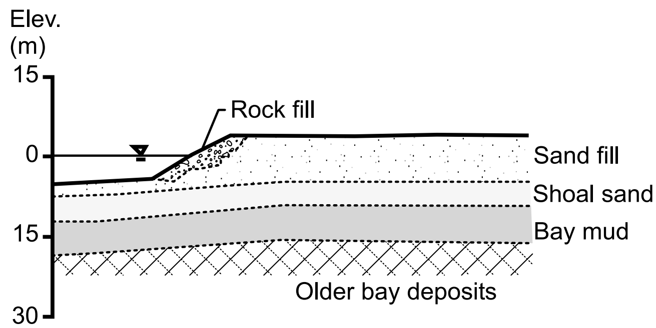


Table 3. Comparison of the liquefaction-induced ground settlements measured and calculated using the proposed CPT-based method for Treasure Island.

CPT	Calculated settlement using proposed CPT-based approach (cm)	Observed settlement (Power et al. 1998) (cm)
Zone one		
C28	15.9	5–10 (~2–4 in)
C32	14.4	
C33	15.3	
C34	12.2	
C35	10.7	
C37	11.8	
C42	17.2	
Zone two		
C29A	18.8	10–15 (~4–6 in)
C30	27.0	
C31	33.3	
C39	23.6	
UM10	25.8	

general, the calculated settlements are larger than the observed values. This implies that the proposed method appears to be conservative for this particular case history. The possible reasons for the difference between measured and calculated settlements are discussed later in this paper. However, it is useful to reflect on the accepted accuracy of current calculations 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% 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).

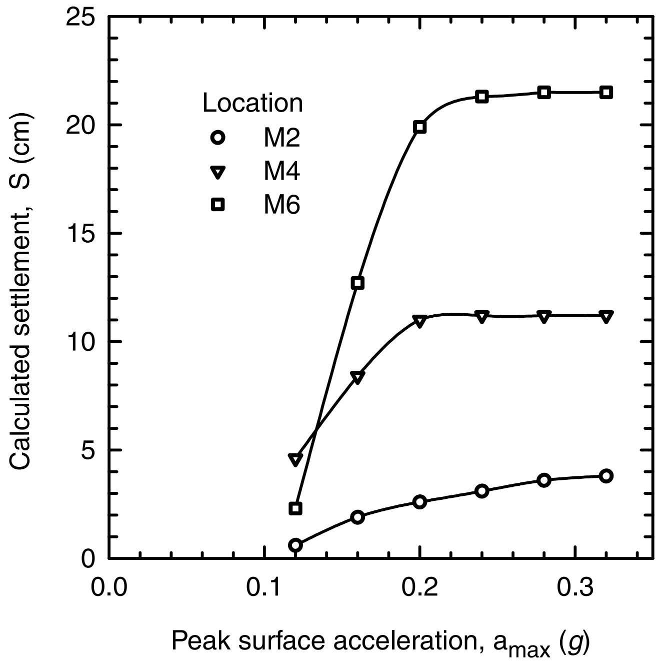
Maximum surface acceleration at a site is one important parameter used in evaluating the liquefaction potential of sandy soils. However, its determination is difficult without recorded accelerographs for a given earthquake because it likely varies with soil stratigraphy, soil properties, earthquake properties, 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 for the Marina District site and the 1989 Loma Prieta earthquake (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) ranging from 0.12 to 0.32*g*. This large variation creates uncertainty in evaluating liquefaction potential and estimating liquefaction-induced ground settlements. The effect of uncertainty in a_{max} on the calculated settlements is illustrated in Fig. 8. The calculated settlement for CPT M6 is very sensitive to the peak surface acceleration, as changing the peak surface acceleration from 0.12 to 0.20*g* would cause the calculated ground settlement to increases from 2 to 20 cm. The sensitivity of the calculated settlement to the selection of a_{max} is much less for CPT M2. For the remaining three CPT soundings studied, the calculated ground settlement almost does not change if the peak surface acceleration is greater than 0.20*g*, since the calculated volumetric strains have reached their maximums (Fig. 3).

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

Fig. 8. Influence of peak surface acceleration on the calculated settlement for three locations at the Marina District.



Fines content or mean grain size

Although there are many practical situations where liquefaction settlements need to be estimated for sands with little silt to silty-sands, the volumetric strains plotted in Fig. 2 are applicable to saturated clean sands only. It is therefore necessary to consider the effects of fines content on postliquefaction volumetric strains and liquefaction potential.

Volumetric strains tend to increase with increasing mean grain size at a given D_r (Lee and Albaisa 1974). Since increasing the fines content of a sand will result in a decrease of the mean grain size, it is postulated that postliquefaction volumetric strains would decrease with increasing fines content in sands at a given D_r.

Silty sands have been found to be considerably less vulnerable to liquefaction than clean sands with similar SPT blow-counts (Iwasaki et al. 1978; Tatsuoka et al. 1980; Tokimatsu and Yoshimi 1981; Zhou 1981; et al.). Consequently, modification factors to SPT blow-counts or to the CRR for sands with different fines contents or mean grain sizes had been widely used in liquefaction potential analyses (Seed and Idriss 1982; Robertson and Campanella 1985; Seed et al. 1985; Robertson and Wride 1998).

In this study, the equivalent clean sand normalized CPT tip resistance $(q_{c1N})_{cs}$ was used to account for the effects of sand grain characteristics and apparent fines content on the CRR (Robertson and Wride 1998). It is also used to estimate postliquefaction volumetric strains for sands with fines. This approach assumes that both the liquefaction resistance and postliquefaction deformations of silty sandy soils can be quantified using the same method and formulas as for clean sands provided that the equivalent clean sand normalized cone penetration resistance, $(q_{c1N})_{cs}$, is used. This implies that no further modification is required for the effects 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 an increase of fines content with sands for a given cone tip resistance, the calculated postliquefaction volumetric strains will decrease with an increase of fines content for a given factor of safety. This approach appears to indirectly account partially or wholly for the effect of grain characteristics on postliquefaction volumetric strains and provides the same trend as observed by Lee and Albaisa (1974).

Transitional zone or thin sandy soil layers

Soil layering influences the cone resistance measured by the CPT (Sanglerat 1972; Campanella and Robertson 1988; Berg 1994; Vreugdenhil 1995; Robertson and Fear 1995; Robertson and Wride 1998). For example, 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 cone for a stiff frictional deposit (e.g., sand) in between softer soil layers. Vreugdenhil (1995) 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 (Youd and Idriss 1997) suggested a simplified correction to the measured CPT tip resistance in a sand layer sandwiched by softer clay layers.

It is recognized that transitional zones between soft clay layers and stiff sandy soil layers influence the calculated liquefaction-induced settlements in this study. However, 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 value of I_c will increase, and therefore the correction factor K_c, $(q_{c1N})_{cs}$, and FS will increase as well. Both $(q_{c1N})_{cs}$ 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 to 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 investigation is required 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

The thickness, depth, and lateral distribution of liquefied layers may play an important role on ground surface settlements. Liquefaction of a relatively thick but deep sandy soil (Fig. 9*a*) 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 (Fig. 9*b*) may have major implications on the performance of the same structure.

Ishihara (1985) provided some guidance on the effect of thickness and depth to the liquefied layer on potential liquefaction-induced damage that may be reasonable provided that the site is not susceptible to ground oscillation or lateral spread (Youd and Garris 1995, O'Rourke and Pease 1997). Gilstrap (1998) concluded that Ishihara's relationship for predicting surface effects may be oversimplified. Additionally, the application of Ishirara's criteria in practice for cases with multiple liquefied layers (Fig. 9*c*) is not clear.

The lateral extent of liquefied layers may also have an effect on ground surface settlements. A small locally liquefied soil zone with limited lateral extent (Fig. 9*d*) would have a limited extent of surface manifestation compared to 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 on the effect of lateral extent of liquefied layers on ground surface settlements.

Neglecting the effect of three-dimensional distribution of liquefied layers on ground surface settlements may result in overestimating liquefaction-induced ground settlements for some sites. Engineering judgement is needed 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. Care is required to detect local zones of soil that may liquefy and to estimate the potential differential settlements that may occur.

The K_c factor

Robertson and Wride (1998) recommended the modification factor K_c be set equal to one rather than using K_c between 1.0 and 2.14 when the CPT data plot in the zone defined by $1.64 < I_c < 2.36$ and $F < 0.5\%$ to avoid confusion of very loose clean sands with denser sands containing fines. However, if the CPT data of a dense sand with fines lies within the zone ($1.64 < I_c < 2.36$ and $F < 0.5\%$), the calculated $(q_{c1N})_{cs}$ value for the dense sand would be reduced by about one-half. Although this recommendation is conservative for evaluating the liquefaction potential of sandy soils, it may result in overestimating 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 portion of the soil that should have been assessed as dense sand containing fines, was classified as very loose clean sand with K_c equal to one.

Settlements for the two case histories were recalculated without following the recommendation of K_c equal to one for $1.64 < I_c < 2.36$ and $F < 0.5\%$, and the results are given in Table 5. There is almost no effect for the western and central parts of the Marina District and only small (up to 14%) effects for Treasure Island. However the calculated settlements for the eastern part of the Marina District are reduced by a factor of 2.6 without following the recommendation for K_c. The effect of this recommendation on calculated ground settlements depends 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 this zone, the effect would be much more significant than that for the two case history sites studied above. Soil sampling is therefore recommended to clarify soil properties for sites where a large amount of soil plots in the zone $1.64 < I_c < 2.36$ and $F < 0.5\%$.

Cutoff of $I_c = 2.6$

A cutoff of $I_c = 2.6$ is used to distinguish sandy and silty soils from clayey soils, which are believed to be nonliquefiable (Robertson and Wride 1998). Gilstrap (1998) concluded that the I_c cutoff of 2.6 recommended by Robertson and Wride (1998) is generally reliable for identifying clayey soils, but noticed that 20–50% of the samples with I_c between 2.4 and 2.6 were classified as clayey soils based on index tests. This implies that the cutoff of $I_c = 2.6$ appears slightly conservative.

The sensitivity of the calculated settlements to this cutoff was investigated for the two case histories using a cutoff of $I_c = 2.5$. These results are also reported in Table 5. The calculated settlements with the cutoff of $I_c = 2.5$ are slightly smaller than with the cutoff of 2.6. For these two cases, only a small portion of the soil in the profiles had I_c ranging from 2.5 to 2.6, thus the use of a cutoff of $I_c = 2.6$ does not greatly overestimate the settlements.

Table 5 also contains calculations of settlement when the influence of K_c and the cutoff for I_c are combined. Again, the influences on calculated settlements are small for the

Fig. 9. Four hypothetical cases showing the importance of the three-dimensional distribution of liquefied layers.

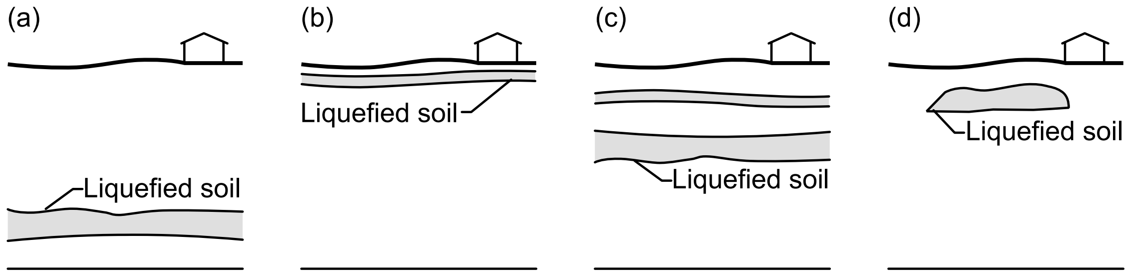


Table 5. Liquefaction-induced ground settlements measured and calculated using the CPT-based approach with and without some modifications for the Marina District and Treasure Island.

		Liquefaction-induced ground settlement (cm)				
Site	Zone	Calculated[a] (basic procedure)	Calculated[b] (without the recommendation for K_c)	Calculated[c] (with the cutoff line of I_c = 2.5)	Calculated[d] (without the recommendation for K_c and with the cutoff line of I_c = 2.5)	Observed[e]
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
TreasureIsland	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

[a]Calculated settlements using the proposed CPT-based approach with the recommendation of K_c =1 for 1.64 < I_c < 2.36 and F < 0.5% and the cutoff line of I_c = 2.6.
[b]Same as footnote *a* but without the recommendation of setting K_c = 1 for the soils that fit in the zone defined by 1.64 < I_c < 2.36 and F < 0.5%.
[c]Same as footnote *a* but set the cutoff line at I_c = 2.5 instead of I_c = 2.6.
[d]Same as footnote *a* but without the recommendation of setting K_c = 1 and with the cutoff line of I_c = 2.5.
[e]Measured or observed ground settlements in the Marina District (Bennett 1990) and Treasure Island (Power et al. 1998).

western and central parts of the Marina District and up to 25% for Treasure Island. For the eastern part of the Marina District, the selection of K_c had a greater influence on calculated settlements than the cutoff for I_c.

Neglecting the influence of the recommendation for K_c and the cutoff line of I_c = 2.6 on the calculated ground settlements is conservative. However, soil sampling is recommended to avoid unnecessary overestimation 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%.

Summary and conclusions

A CPT-based approach for estimating liquefaction-induced ground settlements for sites with level ground conditions was presented. 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 CPT cone tip resistance and sleeve friction, moment magnitude of the earthquake, maximum surface acceleration during the earthquake, depth to the ground water table, and the unit weights of the soils are required to estimate liquefaction-induced ground settlements. The effect of maximum surface acceleration, fines content or mean grain size, transitional zones at layer boundaries, three-dimensional distribution of liquefied soil layers, correction factor K_c, and cutoff of I_c = 2.6 on calculated ground settlements were discussed.

The proposed methodology was used to estimate the liquefaction-induced ground settlements for the Marina District and Treasure Island case histories devastated by liquefaction during the 1989 Loma Prieta earthquake. Good agreement was found between the calculated and measured liquefaction-induced ground settlements. Results from the proposed CPT-based method provided closer agreement with the measured results than those obtained using the existing SPT-based method, especially for loose, hydraulically placed fill where large settlements occurred. Although further evaluations of the proposed method are required with future case history data from different earthquakes and ground conditions as they become available, 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.

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Appendix A

Equations for the relationships plotted in Fig. 3 are given by

[A1] if FS ≤ 0.5, $\varepsilon_v = 102(q_{c1N})_{cs}^{-0.82}$ for $33 \le (q_{c1N})_{cs} \le 200$

[A2] if FS = 0.6, $\varepsilon_v = 102(q_{c1N})_{cs}^{-0.82}$ for $33 \le (q_{c1N})_{cs} \le 147$

[A3] if FS = 0.6, $\varepsilon_v = 2411(q_{c1N})_{cs}^{-1.45}$ for $147 \le (q_{c1N})_{cs} \le 200$

[A4] if FS = 0.7, $\varepsilon_v = 102(q_{c1N})_{cs}^{-0.82}$ for $33 \le (q_{c1N})_{cs} \le 110$

[A5] if FS = 0.7, $\varepsilon_v = 1701(q_{c1N})_{cs}^{-1.42}$ for $110 \le (q_{c1N})_{cs} \le 200$

[A6] if FS = 0.8, $\varepsilon_v = 102(q_{c1N})_{cs}^{-0.82}$ for $33 \le (q_{c1N})_{cs} \le 80$

[A7] if FS = 0.8, $\varepsilon_v = 1690(q_{c1N})_{cs}^{-1.46}$ for $80 \le (q_{c1N})_{cs} \le 200$

[A8] if FS = 0.9, $\varepsilon_v = 102(q_{c1N})_{cs}^{-0.82}$ for $33 \le (q_{c1N})_{cs} \le 60$

[A9] if FS = 0.9, $\varepsilon_v = 1430(q_{c1N})_{cs}^{-1.48}$ for $60 \le (q_{c1N})_{cs} \le 200$

[A10] if FS = 1.0, $\varepsilon_v = 64(q_{c1N})_{cs}^{-0.93}$ for $33 \le (q_{c1N})_{cs} \le 200$

[A11] if FS = 1.1, $\varepsilon_v = 11(q_{c1N})_{cs}^{-0.65}$ for $33 \le (q_{c1N})_{cs} \le 200$

[A12] if FS = 1.2, $\varepsilon_v = 9.7(q_{c1N})_{cs}^{-0.69}$ for $33 \le (q_{c1N})_{cs} \le 200$

[A13] if FS = 1.3, $\varepsilon_v = 7.6(q_{c1N})_{cs}^{-0.71}$ for $33 \le (q_{c1N})_{cs} \le 200$

[A14] if FS = 2.0, $\varepsilon_v = 0.0$ for $33 \le (q_{c1N})_{cs} \le 20$