Evaluating cyclic liquefaction potential using the cone penetration test

P.K. Robertson and C.E. (Fear) Wride

Abstract: Soil liquefaction is a major concern for structures constructed with or on sandy soils. This paper describes the phenomena of soil liquefaction, reviews suitable definitions, and provides an update on methods to evaluate cyclic liquefaction using the cone penetration test (CPT). A method is described to estimate grain characteristics directly from the CPT and to incorporate this into one of the methods for evaluating resistance to cyclic loading. A worked example is also provided, illustrating how the continuous nature of the CPT can provide a good evaluation of cyclic liquefaction potential, on an overall profile basis. This paper forms part of the final submission by the authors to the proceedings of the 1996 National Center for Earthquake Engineering Research workshop on evaluation of liquefaction resistance of soils.

Key words: cyclic liquefaction, sandy soils, cone penetration test.

Résumé : La liquéfaction des sols est un risque majeur pour les structures faites en sable ou fondées dessus. Cet article décrit les phénomènes de liquéfaction des sols, fait le tour des définitions s'y rapportant et fournit une mise à jour des méthodes permettant d'évaluer la liquéfaction cyclique à partir de l'essai de pénétration au cône (CPT). On décrit une méthode qui permet d'estimer les caractéristiques granulaires à partir du CPT et d'incorporer directement les résultats dans une des méthodes d'évaluation de la résistance au chargement cyclique. Un exemple avec solution est aussi présenté pour illustrer comment la nature continue du CPT peut donner une bonne idée du potentiel de liquéfaction cyclique sur la base d'un profil d'ensemble. Cet article fait partie de la contribution finale des auteurs aux comptes-rendus du séminaire NCEER tenu en 1996 sur l'évaluation de la résistance des sols à la liquéfaction.

Mots clés : liquéfaction cyclique, sols sableux, CPT

[Traduit par la Rédaction]

Introduction

Soil liquefaction is a major concern for structures constructed with or on saturated sandy soils. The phenomenon of soil liquefaction has been recognized for many years. Terzaghi and Peck (1967) referred to "spontaneous liquefaction" to describe the sudden loss of strength of very loose sands that caused flow slides due to a slight disturbance. Mogami and Kubo (1953) also used the term liquefaction to describe a similar phenomenon observed during earthquakes. The Niigata earthquake in 1964 is certainly the event that focused world attention on the phenomenon of soil liquefaction. Since 1964, much work has been carried out to explain and understand soil liquefaction. The progress of work on soil liquefaction has been described in detail in a series of state-of-the-art papers, such as those by Yoshimi et al. (1977), Seed (1979), Finn (1981), Ishihara (1993), and Robertson and Fear (1995). The major earthquakes of Niigata in 1964 and Kobe in 1995 have illustrated the significance and extent of damage that can be caused by soil liquefaction. Liquefaction was the cause of much of the damage to the port facilities in Kobe in 1995. Soil liquefaction is also a major design problem for large sand structures such as mine tailings impoundments and earth dams.

The state-of-the-art paper by Robertson and Fear (1995) provided a detailed description and review of soil liquefaction and its evaluation. In January 1996, the National Center for Earthquake Engineering Research (NCEER) in the United States arranged a workshop in Salt Lake City, Utah, to discuss recent advances in the evaluation of cyclic liquefaction. This paper forms part of the authors' final presentation (Robertson and Wride 1998) to the proceedings of that workshop (Youd and Idriss 1998). The objective of this paper is to provide an update on the evaluation of cyclic liquefaction using the cone penetration test (CPT). Several phenomena are described as soil liquefaction. In an effort to clarify the different phenomena, the mechanisms will be briefly described and definitions for soil liquefaction will be reviewed.

Liquefaction definitions

Before describing methods to evaluate liquefaction potential, it is important to first define the terms used to explain the phenomena of soil liquefaction.

Figure 1 shows a summary of the behaviour of a granular soil loaded in undrained monotonic triaxial compression. In void ratio (e) and mean normal effective stress (p') space, a soil with an initial void ratio higher than the ultimate state line (USL) will strain soften (SS) at large strains, eventually

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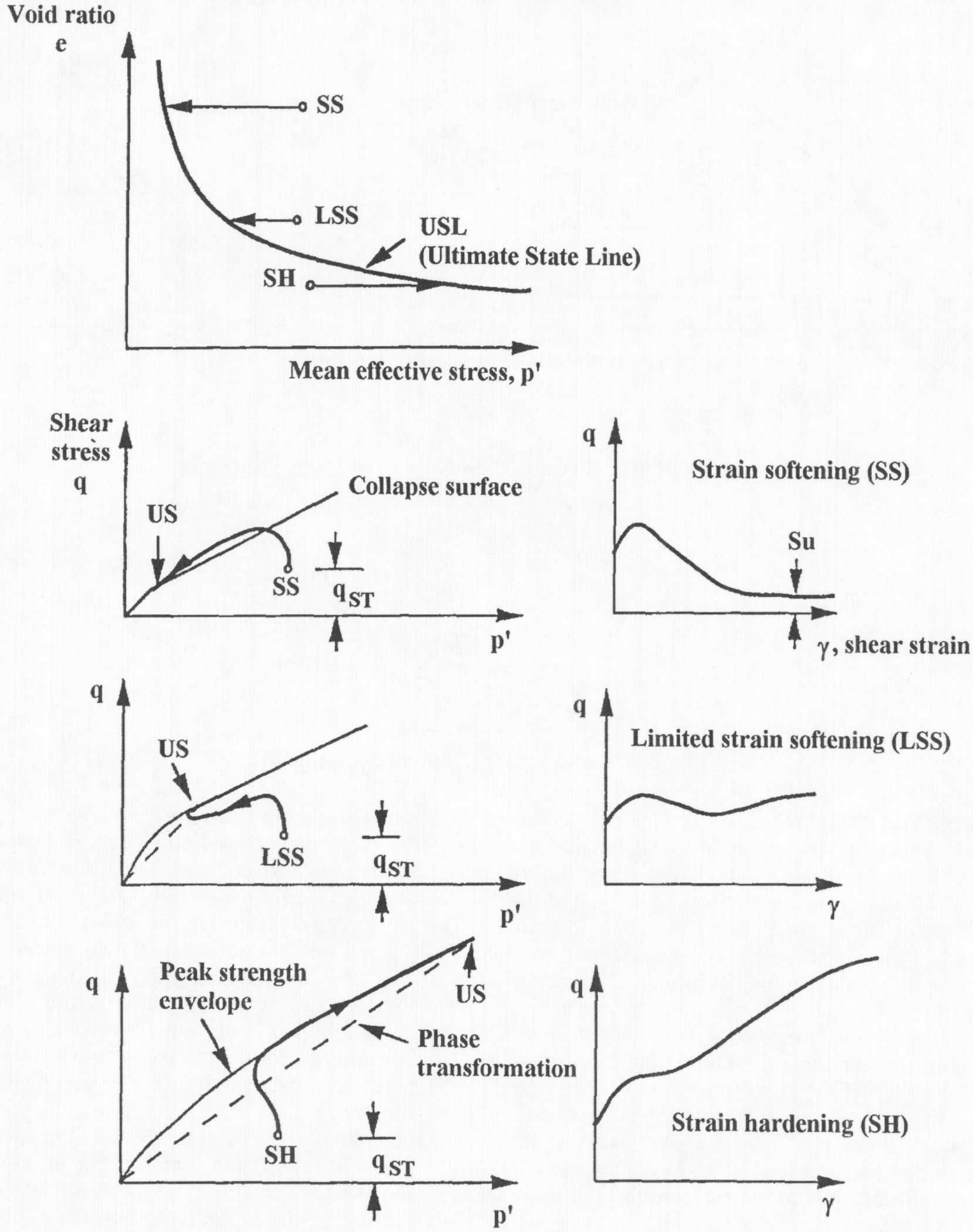
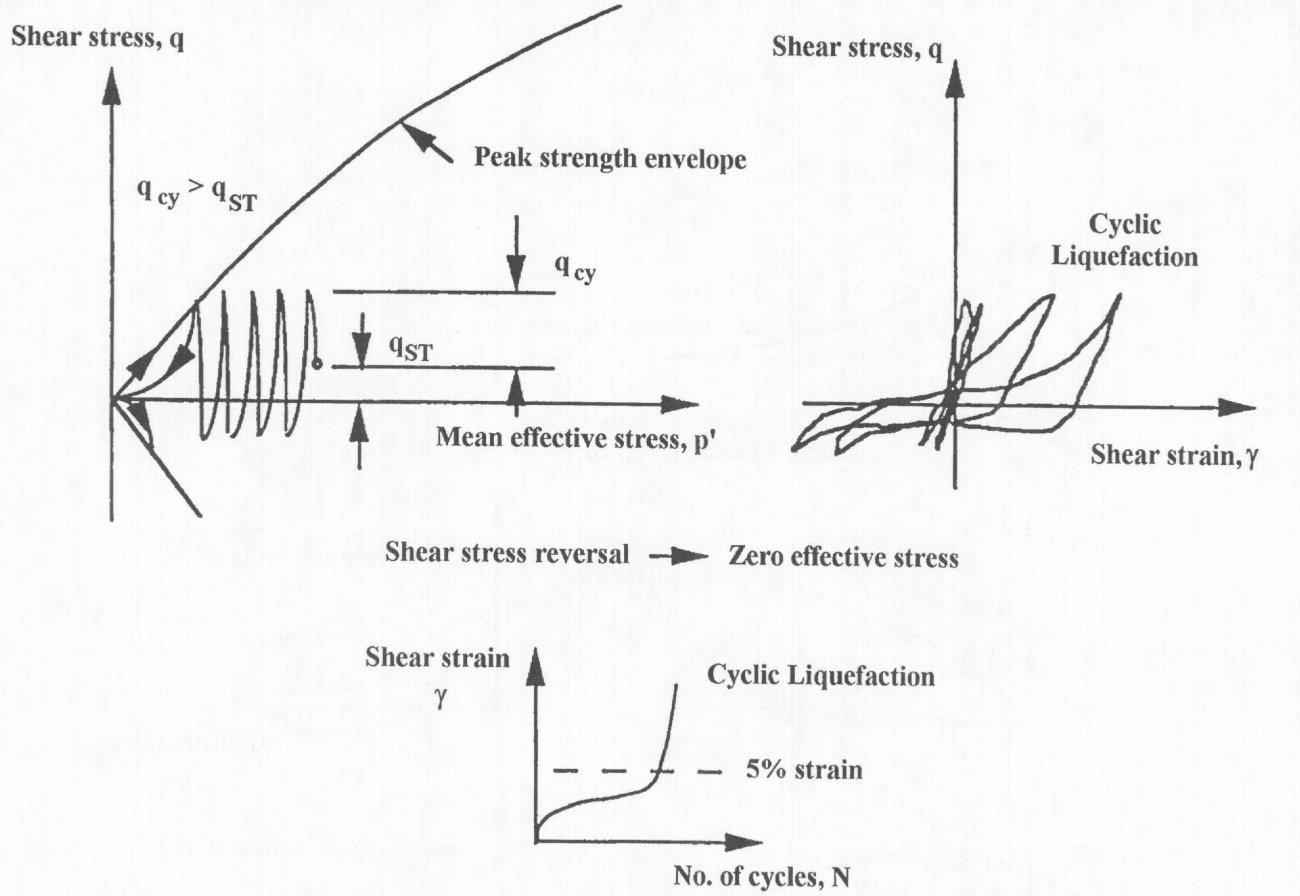
Fig. 1. Schematic of undrained monotonic behaviour of sand in triaxial compression (after Robertson 1994). LSS, limited strain-softening response; q_{ST}, static gravitational shear stress; S_u, ultimate undrained shear strength; SH, strain-hardening response; SS, strain-softening response; US, ultimate state.



reaching an ultimate condition often referred to as critical or steady state. The term ultimate state (US) is used here, after Poorooshasb and Consoli (1991). However, a soil with an initial void ratio lower than the USL will strain harden (SH) at large strains towards its ultimate state. It is possible to have a soil with an initial void ratio higher than but close to the USL. For this soil state, the response can show limited strain softening (LSS) to a quasi steady state (QSS) (Ishihara 1993), but eventually, at large strains, the response strain hardens to the ultimate state. For some sands, very large strains are required to reach the ultimate state, and in some cases conventional triaxial equipment may not reach these large strains (axial strain, $\varepsilon_a > 20\%$).

During cyclic undrained loading (e.g., earthquake loading), almost all saturated cohesionless soils develop positive pore pressures due to the contractive response of the soil at small strains. If there is shear stress reversal, the effective stress state can progress to the point of essentially zero effective stress, as illustrated in Fig. 2. For shear stress reversal to occur, ground conditions must be generally level or gently sloping; however, shear stress reversal can occur in steeply sloping ground if the slope is of limited height

Fig. 2. Schematic of undrained cyclic behaviour of sand illustrating cyclic liquefaction (after Robertson 1994). q_{cy}, cycling shear stress.

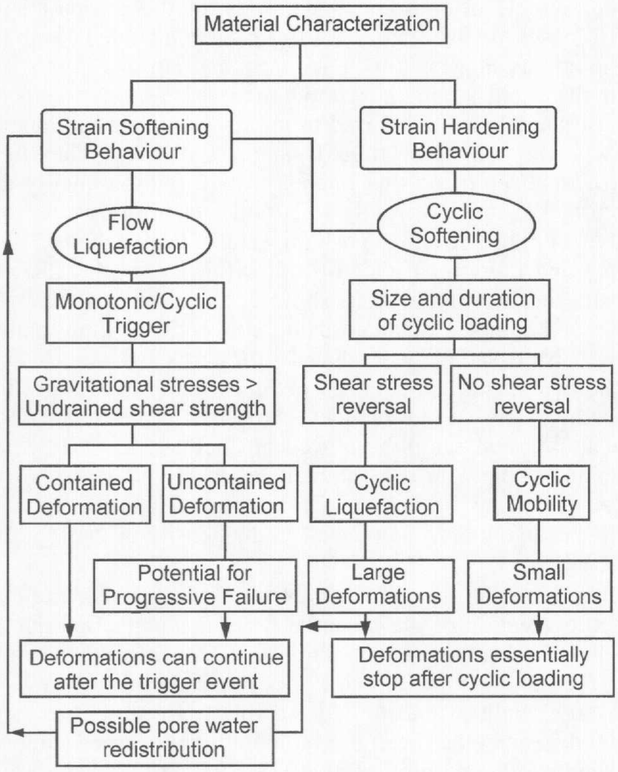


(Pando and Robertson 1995). When a soil element reaches the condition of essentially zero effective stress, the soil has very little stiffness and large deformations can occur during cyclic loading. However, when cyclic loading stops, the deformations essentially stop, except for those due to local pore-pressure redistribution. If there is no shear stress reversal, such as in steeply sloping ground subjected to moderate cyclic loading, the stress state may not reach zero effective stress. As a result, only cyclic mobility with limited deformations will occur, provided that the initial void ratio of the sand is below the USL and the large strain response is strain hardening (i.e., the material is not susceptible to a catastrophic flow slide). However, shear stress reversal in the level ground area beyond the toe of a slope may lead to overall failure of the slope due to softening of the soil in the toe region.

Based on the above description of soil behaviour in undrained shear, Robertson and Fear (1995), building on earlier work by Robertson (1994), proposed specific definitions of soil liquefaction which distinguished between flow liquefaction (strain-softening behaviour; see Fig. 1) from cyclic softening. Cyclic softening was further divided into cyclic liquefaction (see Fig. 2) and cyclic mobility. A full description of these definitions is given by Robertson and Wride (1998) in the NCEER report (Youd and Idriss 1998). Figure 3 presents a flow chart (after Robertson 1994) for the evaluation of liquefaction according to these definitions. The first step is to evaluate the material characteristics in terms of a strain-softening or strain-hardening response. If the soil is strain softening, flow liquefaction is possible if the soil can be triggered to collapse and if the gravitational shear stresses are larger than the ultimate or minimum strength. The trigger mechanism can be either monotonic or cyclic. Whether a slope or soil structure will fail and slide will depend on the amount of strain-softening soil relative to strain-hardening soil within the structure, the brittleness of the strain-softening soil, and the geometry of the ground. The resulting deformations of a soil structure with both strain-softening and strain-hardening soils will depend on many factors, such as distribution of soils, ground geometry, amount and type of trigger mechanism, brittleness of the strain-softening soil, and drainage conditions. Soils that are only temporarily strain softening (i.e., experience a minimum strength before dilating to US) are not as dangerous as very loose soils that can strain soften directly to ultimate state. Examples of flow liquefaction failures are Fort Peck Dam (Casagrande 1965), Aberfan flowslide (Bishop 1973), Zealand flowslide (Koppejan et al. 1948), and the Stava tailings dam. In general, flow liquefaction failures are not common; however, when they occur, they take place rapidly with little warning and are usually catastrophic. Hence, the design against flow liquefaction should be carried out cautiously.

If the soil is strain hardening, flow liquefaction will generally not occur. However, cyclic softening can occur due to cyclic undrained loading, such as earthquake loading. The

Fig. 3. Suggested flow chart for evaluation of soil liquefaction (after Robertson 1994).



amount and extent of deformations during cyclic loading will depend on the density of the soil, the magnitude and duration of the cyclic loading, and the extent to which shear stress reversal occurs. If extensive shear stress reversal occurs, it is possible for the effective stresses to reach zero and, hence, cyclic liquefaction can take place. When the condition of essentially zero effective stress is achieved, large deformations can result. If cyclic loading continues, deformations can progressively increase. If shear stress reversal does not take place, it is generally not possible to reach the condition of zero effective stress and deformations will be smaller, i.e., cyclic mobility will occur. Examples of cyclic softening were common in the major earthquakes in Niigata in 1964 and Kobe in 1995 and manifested in the form of sand boils, damaged lifelines (pipelines, etc.), lateral spreads, slumping of small embankments, settlements, and ground-surface cracks. If cyclic liquefaction occurs and drainage paths are restricted due to overlying less permeable layers, the sand immediately beneath the less permeable soil can loosen due to pore-water redistribution, resulting in possible subsequent flow liquefaction, given the right geometry.

Both flow liquefaction and cyclic liquefaction can cause very large deformations. Hence, it can be very difficult to clearly identify the correct phenomenon based on observed deformations following earthquake loading. Earthquake-induced flow liquefaction movements tend to occur after the cyclic loading ceases due to the progressive nature of the load redistribution. However, if the soil is sufficiently loose and the static shear stresses are sufficiently large, the earthquake loading may trigger essentially spontaneous liquefaction within the first few cycles of loading. Also, if the soil is sufficiently loose, the ultimate undrained strength may be close to zero with an associated effective confining stress very close to zero (Ishihara 1993). Cyclic liquefaction movements, on the other hand, tend to occur during the cyclic loading, since it is the inertial forces that drive the phenomenon. The post-earthquake diagnosis can be further complicated by the possibility of pore-water redistribution after the cyclic loading, resulting in a change in soil density and possibly the subsequent triggering of flow liquefaction. Identifying the type of phenomenon after earthquake loading is difficult and, ideally, requires instrumentation during and after cyclic loading together with comprehensive site characterization.

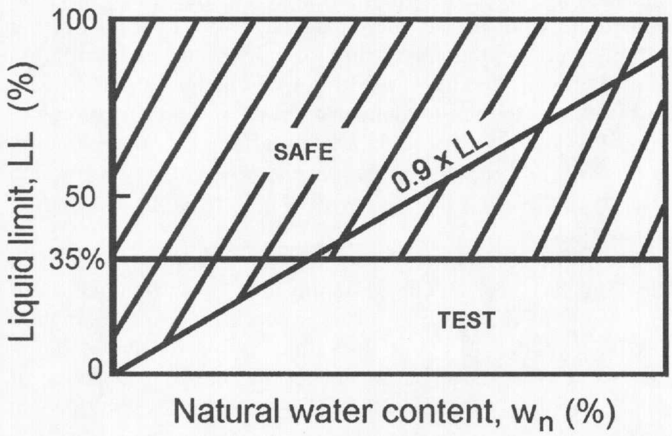
The most common form of soil liquefaction observed in the field has been cyclic softening due to earthquake loading. Much of the existing research work on soil liquefaction has been related to cyclic softening, primarily cyclic liquefaction. Cyclic liquefaction generally applies to level or gently sloping ground where shear stress reversal occurs during earthquake loading. This paper is concerned primarily with cyclic liquefaction due to earthquake loading and its evaluation using results of the CPT.

Cyclic resistance based on laboratory testing

Much of the early work related to earthquake-induced soil liquefaction resulted from laboratory testing of reconstituted samples subjected to cyclic loading by means of cyclic triaxial, cyclic simple shear, or cyclic torsional tests. The outcome of these studies generally confirmed that the resistance to cyclic loading is influenced primarily by the state of the soil (i.e., void ratio, effective confining stresses, and soil structure) and the intensity and duration of the cyclic loading (i.e., cyclic shear stress and number of cycles), as well as the grain characteristics of the soil. Soil structure incorporates features such as fabric, age, and cementation. Grain characteristics incorporate features such as grain-size distribution, grain shape, and mineralogy.

Resistance to cyclic loading is usually represented in terms of a cyclic stress ratio that causes cyclic liquefaction, termed cyclic resistance ratio (CRR). The point of "liquefaction" in a cyclic laboratory test is typically defined as the time at which the sample achieves a strain level of either 5% double-amplitude axial strain in a cyclic triaxial test or 3–4% double-amplitude shear strain in a cyclic simple shear test. For cyclic simple shear tests, CRR is the ratio of the cyclic shear stress to cause cyclic liquefaction to the initial vertical effective stress, i.e., $(CRR)_{ss} = \tau_{cyc}/\sigma'_{vo}$. For cyclic triaxial tests, CRR is the ratio of the maximum cyclic shear stress to cause cyclic liquefaction to the initial effective confining stress, i.e., $(CRR)_{tx} = \sigma_{dc}/2\sigma'_{3c}$. The two tests impose different loading conditions and the CRR values are not equivalent. Cyclic simple shear tests are generally considered to be better than cyclic triaxial tests at closely representing earthquake loading for level ground conditions. However, experience has shown that the $(CRR)_{ss}$ can be estimated quite well from $(CRR)_{tx}$, and correction factors have been developed (Ishihara 1993). The CRR is typically taken at about 15 cycles of uniform loading to represent an equivalent earthquake loading of magnitude (M) 7.5, i.e., $CRR_{7.5}$.

Fig. 4. Graphical representation of liquefaction criteria for silts and clays from studies by Seed et al. (1973) and Wang (1979) in China (after Marcuson et al. 1990): < 15% finer than 0.005 mm, liquid limit (LL) < 35%, and water content > 0.9 × liquid limit.



The CRR for any other size earthquake can be estimated using the following equation:

$$[1] \quad CRR = (CRR_{7.5})(MSF)$$

where MSF is the magnitude scaling factor (recommended values are provided in the report by NCEER (Youd and Idriss 1998), which summarizes the results of the 1996 NCEER workshop).

When a soil is fine grained or contains some fines, some cohesion or adhesion can develop between the fine particles making the soil more resistant at essentially zero effective confining stress. Consequently, a greater resistance to cyclic liquefaction is generally exhibited by sandy soils containing some fines. However, this tendency depends on the nature of the fines contained in the sand (Ishihara 1993). Laboratory testing has shown that one of the most important index properties influencing CRR is the plasticity index of the fines contained in the sand (Ishihara and Koseki 1989). Ishihara (1993) showed that the $(CRR)_{tx}$ appears to increase with increasing plasticity index. Studies in China (Wang 1979) suggest that the potential for cyclic liquefaction in silts and clays is controlled by grain size, liquid limit, and water content. The interpretation of this criterion as given by Marcuson et al. (1990) and shown in Fig. 4 can be useful; however, it is important to note that it is based on limited data and should be used with caution. Figure 4 suggests that when a soil has a liquid limit less than 35% combined with a water content greater than 90% of the liquid limit, it is unclear if the soil can experience cyclic liquefaction, and the soil should be tested to clarify the expected response to undrained cyclic loading.

Although void ratio (relative density) has been recognized as a dominant factor influencing the CRR of sands, studies by Ladd (1974), Mulilis et al. (1977), and Tatsuoka et al. (1986) have clearly shown that sample preparation (i.e., soil fabric) also plays an important role. This is consistent with the results of monotonic tests at small to intermediate strain levels. Hence, if results are to be directly applied with any confidence, it is important to conduct cyclic laboratory tests on reconstituted samples with a structure similar to that in situ. Unfortunately, it is very difficult to determine the in situ fabric of natural sands below the water table. As a result, there is often some uncertainty in the evaluation of CRR based on laboratory testing of reconstituted samples, although, as suggested by Tokimatsu and Hosaka (1986), either the small strain shear modulus or shear wave velocity measurements could be used to improve the value of laboratory testing on reconstituted samples of sand. Therefore, there has been increasing interest in testing high-quality undisturbed samples of sandy soils under conditions representative of those in situ. Yoshimi et al. (1989) showed that aging and fabric had a significant influence on the CRR of clean sand from Niigata. Yoshimi et al. (1994) also showed that sand samples obtained using conventional high-quality fixed piston samplers produced different CRR values than those of undisturbed samples obtained using in situ ground freezing. Dense sand samples showed a decrease in CRR and loose sand samples showed an increase in CRR when obtained using a piston sampler, as compared with the results of testing in situ frozen samples. The difference in CRR became more pronounced as the density of the sand increased.

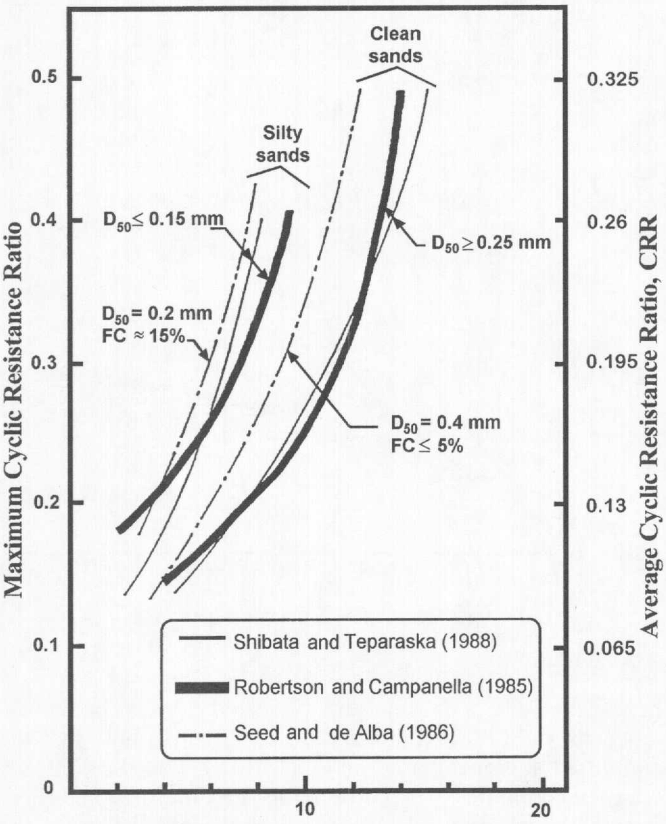
Based on the above observations, for high-risk projects when evaluation of the potential for soil liquefaction due to earthquake loading is very important, consideration should be given to a limited amount of appropriate laboratory tests on high-quality undisturbed samples. Recently, in situ ground freezing has been used to obtain undisturbed samples of sandy soils (Yoshimi et al. 1978, 1989, 1994; Sego et al. 1994; Hofmann et al. 1995; Hofmann 1997). Cyclic simple shear tests are generally the most appropriate tests, although cyclic triaxial tests can also give reasonable results.

Cyclic resistance based on field testing

The above comments have shown that testing high-quality undisturbed samples will give better results than testing poor quality samples. However, obtaining high-quality undisturbed samples of saturated sandy soils is very difficult and expensive and can only be carried out for large projects for which the consequences of liquefaction may result in large costs. Therefore, there will always be a need for simple, economic procedures for estimating the CRR of sandy soils, particularly for low-risk projects and the initial screening stages of high-risk projects. Currently, the most popular simple method for estimating CRR makes use of penetration resistance from the standard penetration test (SPT), although, more recently, the CPT has become very popular because of its greater repeatability and the continuous nature of its profile.

The late Professor H.B. Seed and his coworkers developed a comprehensive SPT-based approach to estimate the potential for cyclic softening due to earthquake loading. The approach requires an estimate of the cyclic stress ratio (CSR) profile caused by a design earthquake. This is usually done based on a probability of occurrence for a given earthquake. A site-specific seismicity analysis can be carried out to determine the design CSR profile with depth. A simplified method to estimate CSR was also developed by Seed and Idriss (1971) based on the maximum ground surface acceler-

Fig. 5. Comparison between three CPT-based charts for estimating cyclic resistance ratio (CRR) for clean sands (after Ishihara 1993). D_{50}, average grain size; FC, fines content.



ation (a_{max}) at the site. This simplified approach can be summarized as follows:

$$\text{[2]} \quad \mathrm{CSR} = \frac{\tau_{av}}{\sigma'_{vo}} = 0.65\left(\frac{a_{max}}{g}\right)\left(\frac{\sigma_{vo}}{\sigma'_{vo}}\right)r_d$$

where τ_{av} is the average cyclic shear stress; a_{max} is the maximum horizontal acceleration at the ground surface; g = 9.81 m/s^2 is the acceleration due to gravity; σ_{vo} and σ'_{vo} are the total and effective vertical overburden stresses, respectively; and r_d is a stress-reduction factor which is dependent on depth. Details about the r_d factor are summarized by Robertson and Wride (1998), based on the recommendations by Seed and Idriss (1971) and Liao and Whitman (1986). The CSR profile from the earthquake can be compared to the estimated CRR profile for the soil deposit, adjusted to the same magnitude using eq. [1]. At any depth, if CSR is greater than CRR, cyclic softening (liquefaction) is possible. This approach is the most commonly used technique in most parts of the world for estimating soil liquefaction due to earthquake loading.

The approach based on the SPT has many problems, primarily due to the inconsistent nature of the SPT. The main factors affecting the SPT have been reviewed (e.g., Seed et al. 1985; Skempton 1986; Robertson et al. 1983) and are summarized by Robertson and Wride (1998). It is highly recommended that the engineer become familiar with the details of the SPT to avoid or at least minimize the effects of some of the major factors, the most important of which is the energy delivered to the SPT sampler. A full description of the recommended modifications to the simplified SPT method to estimate cyclic liquefaction is given by Robertson and Wride (1998) in the NCEER report (Youd and Idriss 1998).

Cone penetration test (CPT)

Because of the inherent difficulties and poor repeatability associated with the SPT, several correlations have been proposed to estimate CRR for clean sands and silty sands using corrected CPT penetration resistance (e.g., Robertson and Campanella 1985; Seed and de Alba 1986; Olsen 1988; Olsen and Malone 1988; Shibata and Teparaska 1988; Mitchell and Tseng 1990; Olsen and Koester 1995; Suzuki et al. 1995*a*, 1995*b*; Stark and Olson 1995; Robertson and Fear 1995).

Although cone penetration resistance is often just corrected for overburden stress (resulting in the term q_{c1}), truly normalized (i.e., dimensionless) cone penetration resistance corrected for overburden stress (q_{c1N}) can be given by

$$\text{[3]} \quad q_{c1N} = \left(\frac{q_c}{P_{a2}}\right)C_Q = \frac{q_{c1}}{P_{a2}}$$

where q_c is the measured cone tip penetration resistance; $C_Q = (P_a/\sigma'_{vo})^n$ is a correction for overburden stress; the exponent n is typically equal to 0.5; P_a is a reference pressure in the same units as σ'_{vo} (i.e., P_a = 100 kPa if σ'_{vo} is in kPa); and P_{a2} is a reference pressure in the same units as q_c (i.e., P_{a2} = 0.1 MPa if q_c is in MPa). A maximum value of C_Q = 2 is generally applied to CPT data at shallow depths. The normalized cone penetration resistance, q_{c1N}, is dimensionless.

Robertson and Campanella (1985) developed a chart for estimating CRR from corrected CPT penetration resistance (q_{c1}) based on the Seed et al. (1985) SPT chart and SPT–CPT conversions. Other similar CPT-based charts were also developed by Seed and de Alba (1986), Shibata and Teparaska (1988), and Mitchell and Tseng (1990). A comparison between three of these CPT charts is shown in Fig. 5. In recent years, there has been an increase in available field performance data, especially for the CPT (Ishihara 1993; Kayen et al. 1992; Stark and Olson 1995; Suzuki et al. 1995*b*). The recent field performance data have shown that the existing CPT-based correlations to estimate CRR are generally good for clean sands. The recent field performance data show that the correlation between CRR and q_{c1N} by Robertson and Campanella (1985) for clean sands provides a reasonable estimate of CRR. Based on discussions at the 1996 NCEER workshop, the curve by Robertson and Campanella (1985) has been adjusted slightly at the lower end to be more consistent with the SPT curve. The resulting recommended CPT correlation for clean sand is shown in Fig. 6. Included in Fig. 6 are suggested curves of limiting shear strain, similar to those suggested by Seed et al. (1985) for the SPT. Occurrence of liquefaction is based on level ground observations of surface manifestations of cyclic liquefaction. For loose sand (i.e., $q_{c1N} < 75$) this could involve large deformations resulting from a condition of essentially zero effective stress being reached. For denser sand (i.e.,

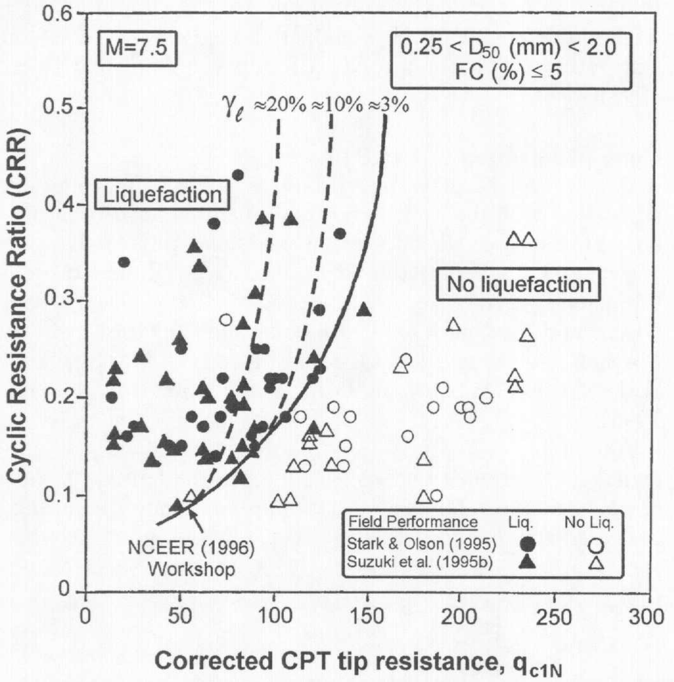


Fig. 6. Recommended cyclic resistance ratio (CRR) for clean sands under level ground conditions based on CPT. γ_l, limiting shear strain.

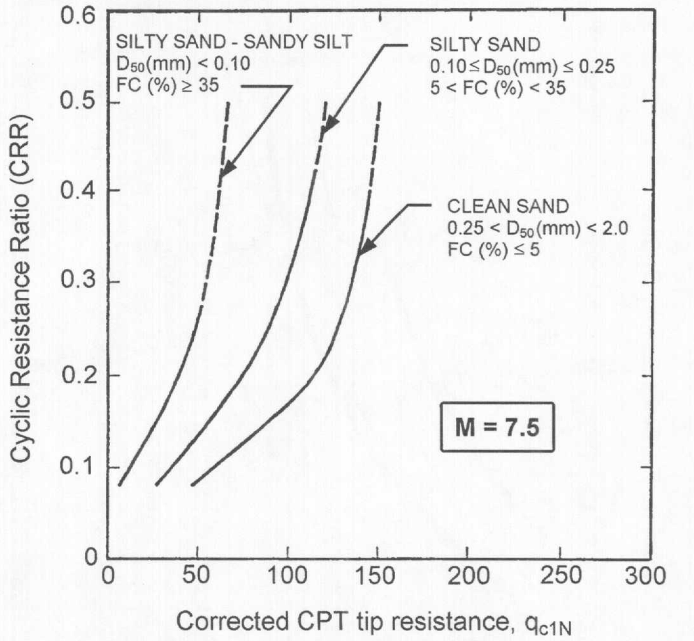


Fig. 7. Summary of variation of cyclic resistance ratio (CRR) with fines content based on CPT field performance data (after Stark and Olson 1995).

$q_{c1N} > 75$) this could involve the development of large pore pressures, but the effective stress may not fully reduce to zero and deformations may not be as large as those in loose sands. Hence, the consequences of liquefaction will vary depending on the soil density as well as the size and duration of loading. An approximate equation for the clean sand CPT curve shown in Fig. 6 is given later in this paper (eq. [8]).

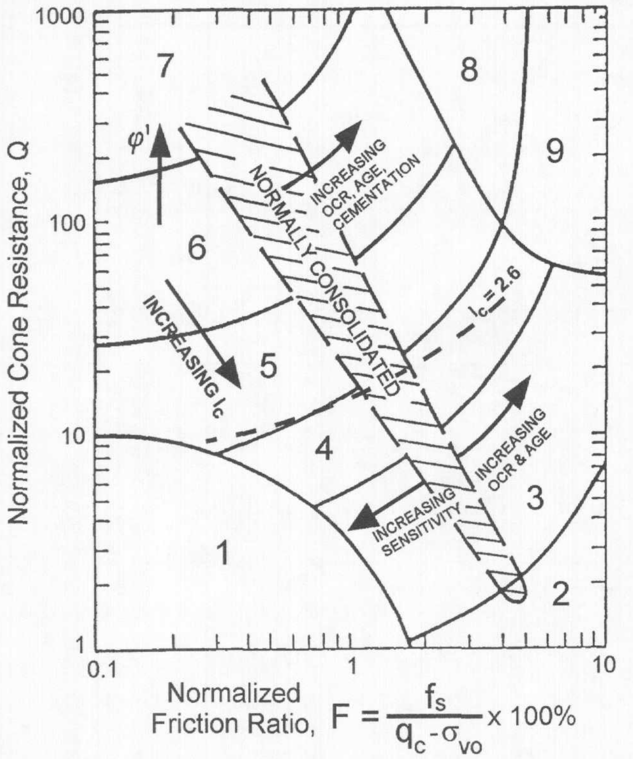
Based on data from 180 sites, Stark and Olson (1995) also developed a set of correlations between CRR and cone tip resistance for various sandy soils based on fines content and mean grain size, as shown in Fig. 7, in terms of q_{c1N}. The CPT combined database is now larger than the original SPT-based database proposed by Seed et al. (1985).

The field observation data used to compile the CPT database are apparently based on the following conditions, similar in nature to those for the SPT-based data: Holocene age, clean sand deposits; level or gently sloping ground; magnitude M = 7.5 earthquakes; depth range from 1 to 15 m (3–45 ft) (84% is for depths < 10 m (30 ft)); and representative average CPT q_c values for the layer that was considered to have experienced cyclic liquefaction.

Caution should be exercised when extrapolating the CPT correlation to conditions outside of the above range. An important feature to recognize is that the correlation appears to be based on average values for the inferred liquefied layers. However, the correlation is often applied to all measured CPT values, which include low values below the average, as well as low values as the cone moves through soil layer interfaces. Therefore, the correlation can be conservative in variable deposits where a small part of the CPT data could indicate possible liquefaction. Although some of the recorded case histories show liquefaction below the suggested curve in Fig. 6, the data are based on average values and, hence, the authors consider the suggested curve to be consistent with field observations. It is important to note that the simplified approach based on either the SPT or the CPT has many uncertainties. The correlations are empirical and there is some uncertainty over the degree of conservatism in the correlations as a result of the methods used to select representative values of penetration resistance within the layers assumed to have liquefied. A detailed review of the CPT data, similar to those carried out by Liao and Whitman (1986) and Fear and McRoberts (1995) on SPT data, would be required to investigate the degree of conservatism contained in Figs. 6 and 7. The correlations are also sensitive to the amount and plasticity of the fines within the sand.

For the same CRR, SPT or CPT penetration resistance in silty sands is smaller because of the greater compressibility and decreased permeability of silty sands. Therefore, one reason for the continued use of the SPT has been the need to obtain a soil sample to determine the fines content of the soil. However, this has been offset by the poor repeatability of SPT data. With the increasing interest in the CPT due to its greater repeatability, several researchers (e.g., Robertson and Campanella 1985; Olsen 1988; Olsen and Malone 1988; Olsen and Koester 1995; Suzuki et al. 1995*a*, 1995*b*; Stark and Olson 1995; Robertson and Fear 1995) have developed a variety of approaches for evaluating cyclic liquefaction potential using CPT results. It is now possible to estimate grain characteristics such as apparent fines content and grain size from CPT data and incorporate this directly into the evaluation of liquefaction potential. Robertson and Fear (1995) recommended an average correction, which was dependent on apparent fines content, but not on penetration resistance. This paper provides modifications to and an update of the CPT approach suggested by Robertson and Fear (1995). The proposed equation to obtain the equivalent clean sand nor-

Fig. 8. Normalized CPT soil behaviour type chart, as proposed by Robertson (1990). Soil types: 1, sensitive, fine grained; 2, peats; 3, silty clay to clay; 4, clayey silt to silty clay; 5, silty sand to sandy silt; 6, clean sand to silty sand; 7, gravelly sand to dense sand; 8, very stiff sand to clayey sand (heavily overconsolidated or cemented); 9, very stiff, fine grained (heavily overconsolidated or cemented). OCR, overconsolidation ratio; φ′, friction angle.



malized CPT penetration resistance, $(q_{c1N})_{cs}$, is a function of both the measured penetration resistance, q_{c1N}, and the grain characteristics of the soil, as follows:

[4] $$(q_{c1N})_{cs} = K_c q_{c1N}$$

where K_c is a correction factor that is a function of the grain characteristics of the soil, as described later in this paper.

Grain characteristics from the CPT

In recent years, charts have been developed to estimate soil type from CPT data (Olsen and Malone 1988; Olsen and Koester 1995; Robertson and Campanella 1988; Robertson 1990). Experience has shown that the CPT friction ratio (ratio of the CPT sleeve friction to the cone tip resistance) increases with increasing fines content and soil plasticity. Hence, grain characteristics such as apparent fines content of sandy soils can be estimated directly from CPT data using any of these soil behaviour charts, such as that by Robertson (1990) shown in Fig. 8. As a result, the measured penetration resistance can be corrected to an equivalent clean sand value. The addition of pore-pressure data can also provide valuable additional guidance in estimating fines content. Robertson et al. (1992) suggested a method for estimating fines content based on the rate of pore-pressure dissipation (t_{50}) during a pause in the CPT.

Based on extensive field data and experience, it is possible to estimate grain characteristics directly from CPT results using the soil behaviour type chart shown in Fig. 8. The boundaries between soil behaviour type zones 2–7 can be approximated as concentric circles (Jefferies and Davies 1993). The radius of each circle can then be used as a soil behaviour type index. Using the CPT chart by Robertson (1990), the soil behaviour type index, I_c, can be defined as follows:

[5] $$I_c = [(3.47 - Q)^2 + (\log F + 1.22)^2]^{0.5}$$

where $Q = \left(\frac{q_c - \sigma_{vo}}{P_{a2}}\right)\left(\frac{P_a}{\sigma'_{vo}}\right)^n$

is the normalized CPT penetration resistance (dimensionless); the exponent n is typically equal to 1.0; $F = [f_s/(q_c - \sigma_{vo})]100$ is the normalized friction ratio, in percent; f_s is the CPT sleeve friction stress; σ_{vo} and σ'_{vo} are the total and effective overburden stresses, respectively; P_a is a reference pressure in the same units as σ'_{vo} (i.e., P_a = 100 kPa if σ'_{vo} is in kPa); and P_{a2} is a reference pressure in the same units as q_c and σ_{vo} (i.e., P_{a2} = 0.1 MPa if q_c and σ_{vo} are in MPa).

The soil behaviour type chart by Robertson (1990) uses a normalized cone penetration resistance (Q) based on a simple linear stress exponent of n = 1.0 (see above), whereas the chart recommended here for estimating CRR (see Fig. 6) is essentially based on a normalized cone penetration resistance (q_{c1N}) based on a stress exponent n = 0.5 (see eq. [3]). Olsen and Malone (1988) correctly suggested a normalization where the stress exponent (n) varies from around 0.5 in sands to 1.0 in clays. However, this normalization for soil type is somewhat complex and iterative.

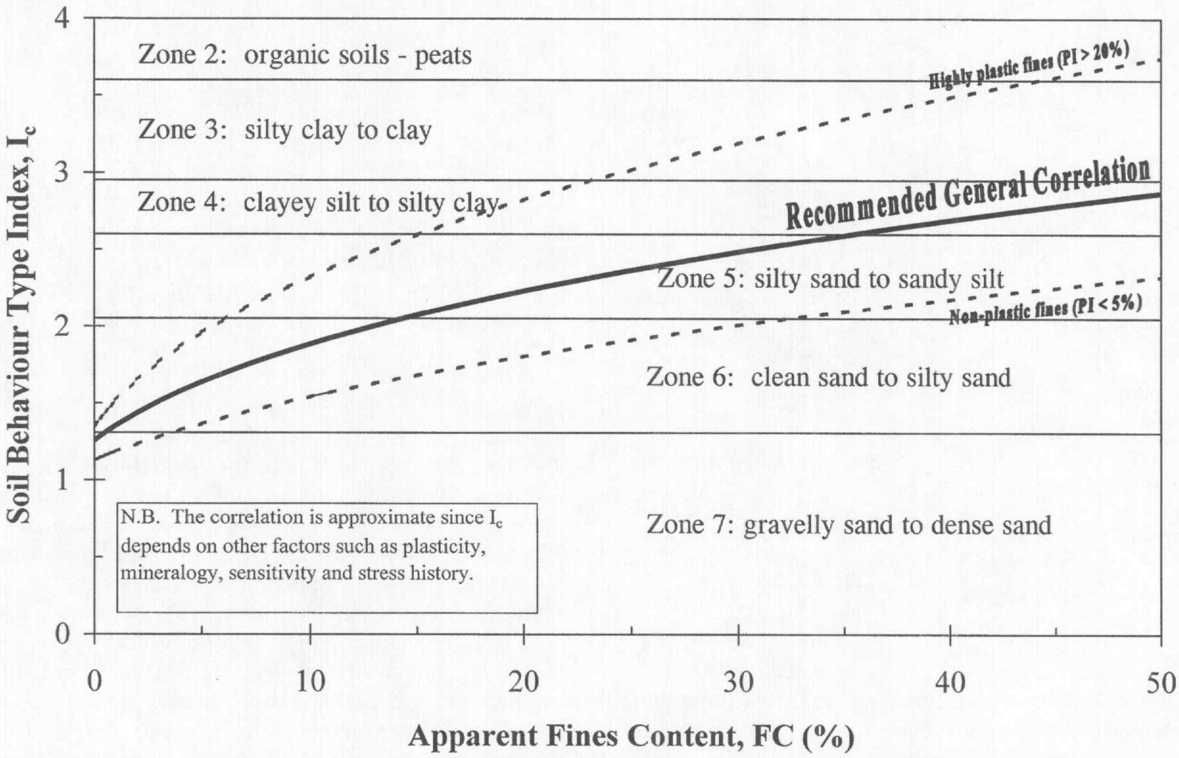
The Robertson (1990) procedure using n = 1.0 is recommended for soil classification in clay type soils when I_c > 2.6. However, in sandy soils when $I_c \le 2.6$, it is recommended that data being plotted on the Robertson chart be modified by using n = 0.5. Hence, the recommended procedure is to first use n = 1.0 to calculate Q and, therefore, an initial value of I_c for CPT data. If I_c > 2.6, the data should be plotted directly on the Robertson chart (and assume q_{c1N} = Q). However, if $I_c \le 2.6$, the exponent to calculate Q should be changed to n = 0.5 (i.e., essentially calculate q_{c1N} using eq. [3], since $\sigma_{vo} << q_c$) and I_c should be recalculated based on q_{c1N} and F. If the recalculated I_c remains less than 2.6, the data should be plotted on the Robertson chart using q_{c1N} based on n = 0.5. If, however, I_c iterates above and below a value of 2.6, depending which value of n is used, a value of n = 0.75 should be selected to calculate q_{c1N} (using eq. [3]) and plot data on the Robertson chart. Note that if the in situ effective overburden stresses are in the order of 50–150 kPa, the choice of normalization has little effect on the calculated normalized penetration resistance.

The boundaries of soil behaviour type are given in terms of the index, I_c, as shown in Table 1. The soil behaviour type index does not apply to zones 1, 8, or 9. Along the normally consolidated region in Fig. 8, soil behaviour type index increases with increasing apparent fines content and soil plasticity, and the following simplified relationship is suggested:

Table 1. Boundaries of soil behaviour type (after Robertson 1990).

Soil behaviour type index, I_c	Zone	Soil behaviour type (see Fig. 8)
$I_c < 1.31$	7	Gravelly sand to dense sand
$1.31 < I_c < 2.05$	6	Sands: clean sand to silty sand
$2.05 < I_c < 2.60$	5	Sand mixtures: silty sand to sandy silt
$2.60 < I_c < 2.95$	4	Silt mixtures: clayey silt to silty clay
$2.95 < I_c < 3.60$	3	Clays: silty clay to clay
$I_c > 3.60$	2	Organic soils: peats

Fig. 9. Variation of CPT soil behaviour type index (I_c) with apparent fines content in or close to the normally consolidated zone of the soil behaviour chart by Robertson (1990). PI, plasticity index.



[6*a*] if $I_c < 1.26$ apparent fines content FC (%) = 0

[6*b*] if $1.26 \le I_c \le 3.5$

$$\text{apparent fines content FC (\%)} = 1.75 I_c^{3.25} - 3.7$$

[6*c*] if $I_c > 3.5$ apparent fines content FC(%) = 100

The range of potential correlations is illustrated in Fig. 9, which shows the variation of soil behaviour type index (I_c) with apparent fines content and the effect of the degree of plasticity of the fines. The recommended general relationship given in eq. [6] is also shown in Fig. 9. Note that this equation is slightly modified from the original work by Robertson and Fear (1995) to increase the prediction of apparent FC for a given value of I_c.

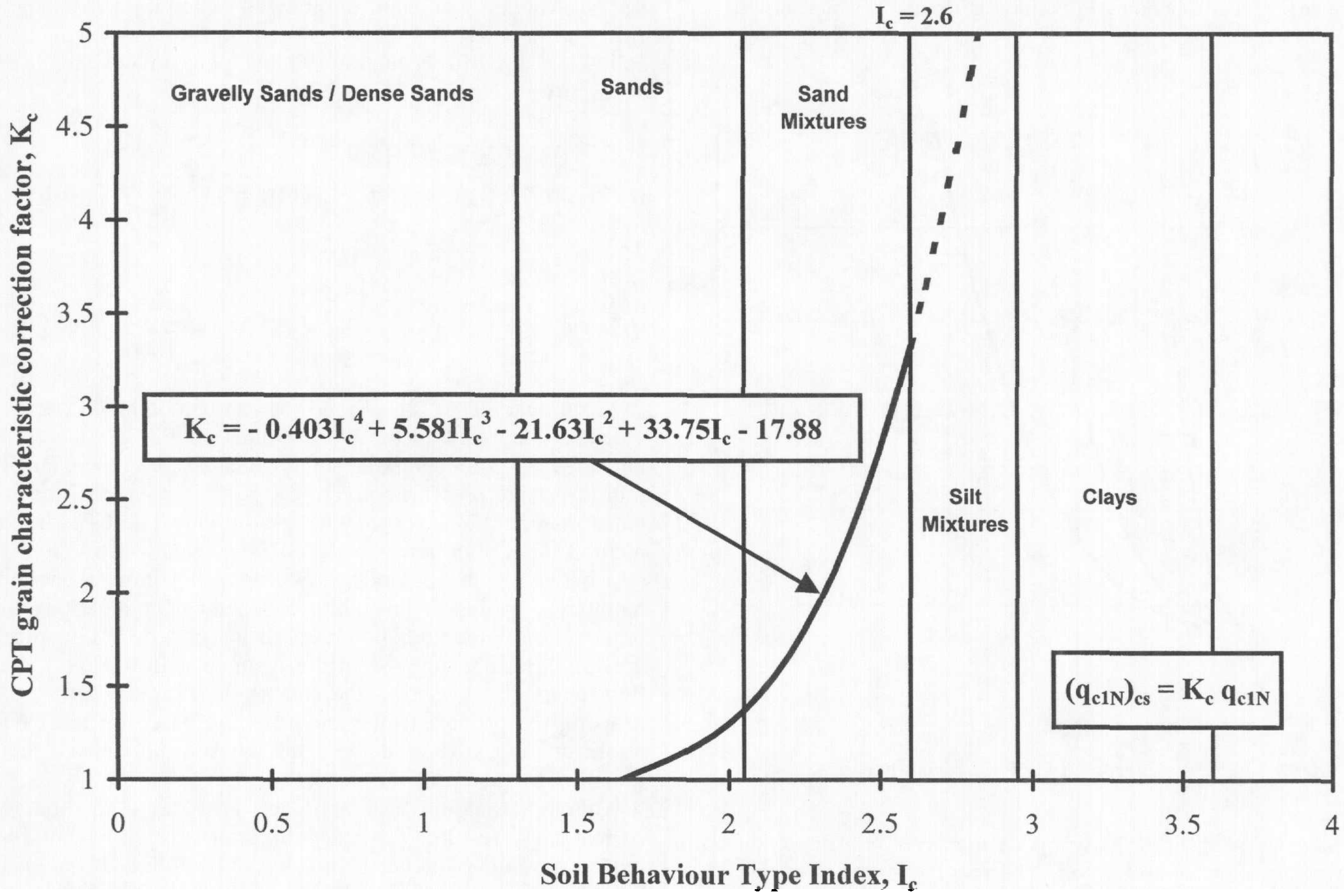
The proposed correlation between CPT soil behaviour index (I_c) and apparent fines content is approximate, since the CPT responds to many other factors affecting soil behaviour, such as soil plasticity, mineralogy, sensitivity, and stress history. However, for small projects, the above correlation provides a useful guide. Caution must be taken in applying eq. [6] to sands that plot in the region defined by $1.64 < I_c < 2.36$ and $F < 0.5\%$ in Fig. 8 so as not to confuse very loose clean sands with denser sands containing fines. In this zone, it is suggested that the apparent fines content is set equal to 5%, such that no correction will be applied to the measured CPT tip resistance when the CPT data plot in this zone. To evaluate the correlation shown in Fig. 9 it is important to show the complete soil profile (CPT and samples, e.g., see Fig. 13), since comparing soil samples with an adjacent CPT at the same elevation can be misleading due to soil stratigraphic changes and soil heterogeneity.

Based on the above method for estimating grain characteristics directly from the CPT using the soil behaviour index (I_c), the recommended relationship between I_c and the correction factor K_c is shown in Fig. 10 and given by the following equations:

[7*a*] if $I_c \le 1.64$, $K_c = 1.0$

[7*b*] if $I_c \le 1.64$, $K_c = -0.403 I_c^4 + 5.581 I_c^3 - 21.63 I_c^2 + 33.75 I_c - 17.88$

Fig. 10. Recommended grain characteristic correction to obtain clean sand equivalent CPT penetration resistance in sandy soils.



The proposed correction factor, K_c, is approximate, since the CPT responds to many factors, such as soil plasticity, fines content, mineralogy, soil sensitivity, and stress history. However, for small projects or for initial screening on larger projects, the above correlation provides a useful guide. Caution must be taken in applying the relationship to sands that plot in the region defined by $1.64 < I_c < 2.36$ and $F \leq 0.5\%$ so as not to confuse very loose clean sands with sands containing fines. In this zone, it is suggested that the correction factor K_c be set to a value of 1.0 (i.e., assume that the sand is a clean sand).

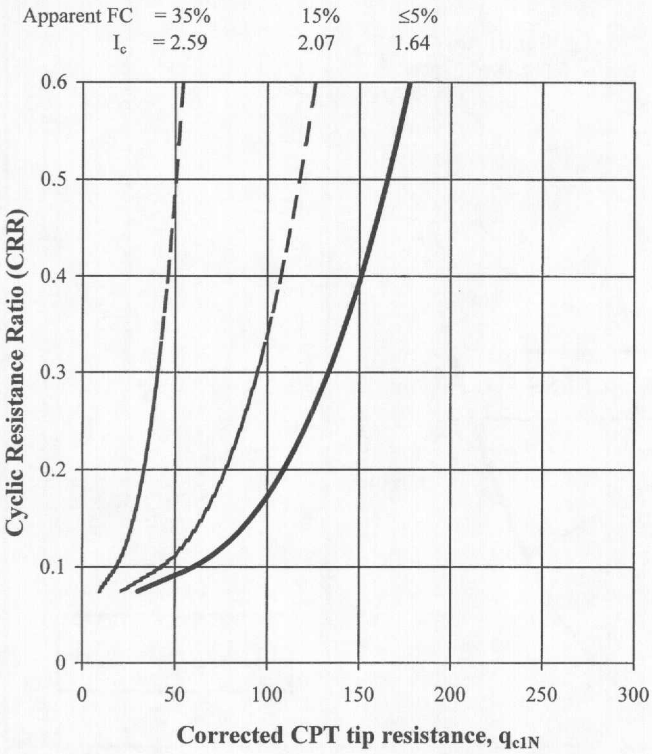
Note that the relationship between the recommended correction factor, K_c, and soil behaviour type index, I_c, is shown in Fig. 10 as a broken line beyond an I_c of 2.6, which corresponds to an approximate apparent fines content of 35%. Soils with $I_c > 2.6$ fall into the clayey silt, silty clay, and clay regions of the CPT soil behaviour chart (i.e., zones 3 and 4). When the CPT indicates soils in these regions ($I_c > 2.6$), samples should be obtained and evaluated using criteria such as those shown in Fig. 4. It is reasonable to assume, in general, that soils with $I_c > 2.6$ are nonliquefiable and that the correction K_c could be large. Soils that fall in the lower left region of the CPT soil behaviour chart (Fig. 8), defined by $I_c > 2.6$ and $F \leq 1.0\%$, can be very sensitive and, hence, possibly susceptible to both cyclic and (or) flow liquefaction. Soils in this region should be evaluated using criteria such as those shown in Fig. 4 combined with additional testing.

Figure 11 shows the resulting equivalent CRR curves for I_c values of 1.64, 2.07, and 2.59 which represent approximate apparent fines contents of 5, 15, and 35%, respectively.

Influence of thin layers

A problem associated with the interpretation of penetration tests in interbedded soils occurs when thin sand layers are embedded in softer deposits. Theoretical as well as laboratory studies show that the cone resistance is influenced by the soil ahead of and behind the penetrating cone. The cone will start to sense a change in soil type before it reaches the new soil and will continue to sense the original soil even when it has entered a new soil. As a result, the CPT will not always measure the correct mechanical properties in thinly interbedded soils. The distance over which the cone tip senses an interface increases with increasing soil stiffness. In soft soils, the diameter of the sphere of influence can be as small as two to three cone diameters, whereas in stiff soils the sphere of influence can be up to 20 cone diameters. Hence, the cone resistance can fully respond (i.e., reach full value within the layer) in thin soft layers better than in thin stiff layers. Therefore care should be taken when interpreting cone resistance in thin sand layers located within soft clay or silt deposits. Based on a simplified elastic solution, Vreugdenhil et al. (1994) have provided some insight as to how to correct cone data in thin layers. Vreugdenhil et al. have shown that the error in the measured cone resistance within thin stiff layers is a function of the thickness of the

Fig. 11. CPT base curves for various values of soil behaviour index, I_c (corresponding to various apparent fines contents, as indicated).



layer as well as the stiffness of the layer relative to that of the surrounding softer soil. The relative stiffness of the layers is reflected by the change in cone resistance from the soft surrounding soil to the stiff soil in the layer. Vreugdenhil et al. validated the model with laboratory and field data.

Based on the work by Vreugdenhil et al. (1994), Robertson and Fear (1995) suggested a conservative correction factor for cone resistance. The corrections apply only to thin sand layers embedded in thick, fine-grained layers. A full description of the proposed correction factor is given by Robertson and Wride (1998) in the NCEER report (Youd and Idriss 1998). Thin sand layers embedded in soft clay deposits are often incorrectly classified as silty sands based on the CPT soil behaviour type charts. Hence, a slightly improved classification can be achieved if the cone resistance is first corrected for layer thickness before applying the classification charts.

Cyclic resistance from the CPT

In an earlier section, a method was suggested for estimating apparent fines content directly from CPT results, using eq. [6]. Following the traditional SPT approach, the estimated apparent fines content could be used to estimate the correction necessary to obtain the clean sand equivalent penetration resistance. However, since other grain characteristics also influence the measured CPT penetration resistance, it is recommended that the necessary correction be estimated from the soil behaviour type index, as described above. Hence, eqs. [4], [5], and [7] can be combined to estimate the equivalent clean sand normalized penetration resistance, $(q_{c1N})_{cs}$, directly from the measured CPT data. Then, using the equivalent clean sand normalized penetration resistance $(q_{c1N})_{cs}$, the CRR (for M = 7.5) can be estimated using the following simplified equation (which approximates the clean sand curve recommended in Fig. 6):

[8*a*]

$$\text{if} \quad 50 \le (q_{c1N})_{cs} < 160, \qquad \text{CRR} = 93\left[\frac{(q_{c1N})_{cs}}{1000}\right]^3 + 0.08$$

[8*b*]

$$\text{if} \quad (q_{c1N})_{cs} < 50, \qquad \text{CRR} = 0.833\left[\frac{(q_{c1N})_{cs}}{1000}\right]^3 + 0.05$$

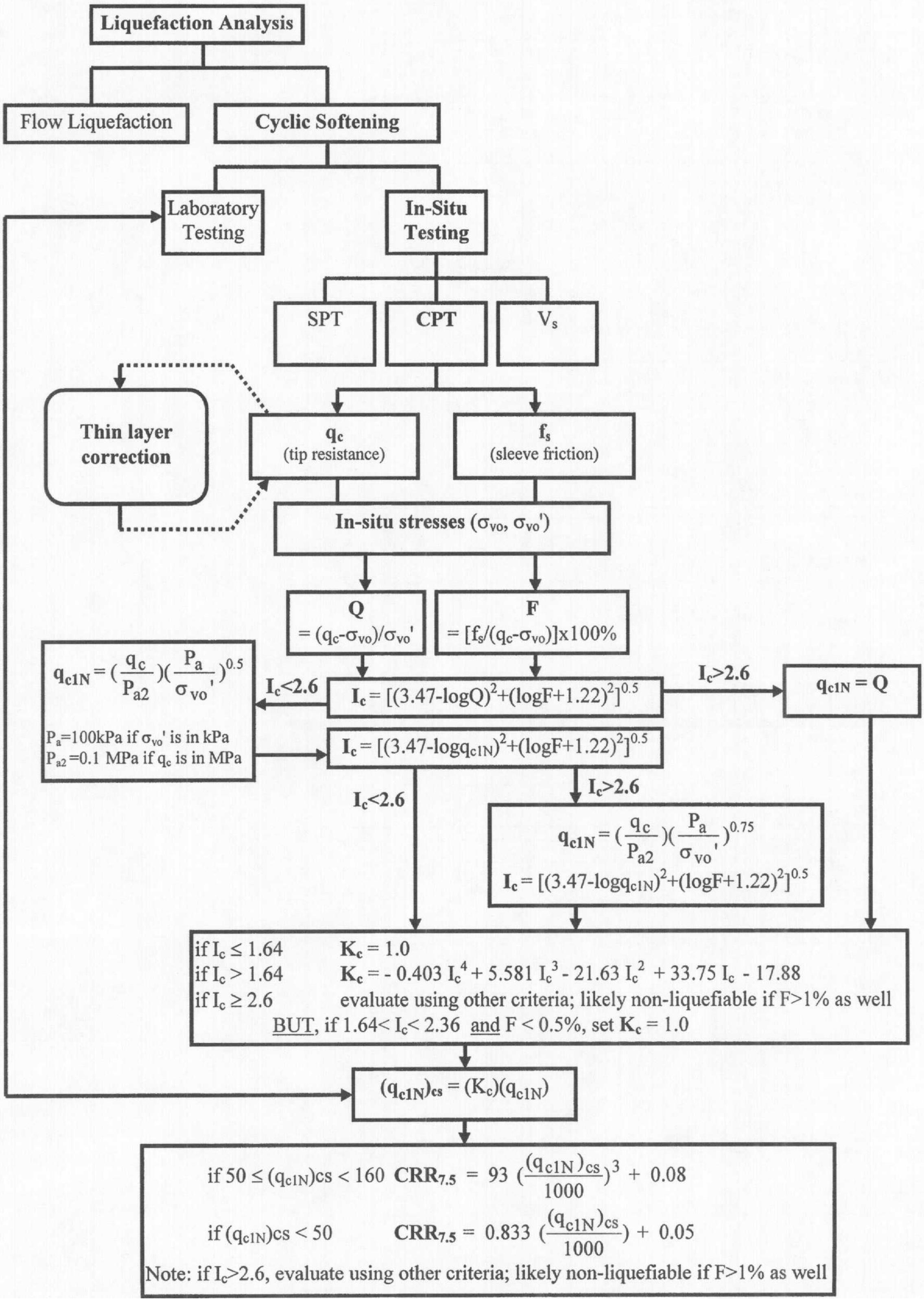
In summary, eqs. [4]–[7] and [8] can be combined to provide an integrated method for evaluating the cyclic resistance (M = 7.5) of saturated sandy soils based on the CPT. If thin layers are present, corrections to the measured tip resistance in each thin layer may be appropriate. The CPT-based method is an alternative to the SPT or shear wave velocity (V_s) based in situ methods; however, using more than one method is useful in providing independent evaluations of liquefaction potential. The proposed integrated CPT method is summarized in Fig. 12 in the form of a flow chart. The flow chart clearly shows the step-by-step process involved in using the proposed integrated method based on the CPT for evaluating CRR and indicates the recommended equations for each step of the process.

Although the proposed approach provides a method to correct CPT results for grain characteristics, it is not intended to remove the need for selected sampling. If the user has no previous CPT experience in the particular geologic region, it is important to take carefully selected samples to evaluate the CPT soil behaviour type classification. Site-specific modifications are recommended, where possible. However, if extensive CPT experience exists within the geologic region, the modified CPT method with no samples can be expected to provide an excellent guide to evaluate liquefaction potential.

Application of the proposed CPT method

An example of this proposed modified CPT-based method is shown in Fig. 13 for the Moss Landing site that suffered cyclic liquefaction during the 1989 Loma Prieta earthquake in California (Boulanger et al. 1995, 1997). The measured cone resistance is normalized and corrected for overburden stress to q_{c1N} and F and the soil behaviour type index (I_c) is calculated. The final continuous profile of CRR at N = 15 cycles (M = 7.5) is calculated from the equivalent clean sand values of q_{c1N} (i.e., $(q_{c1N})_{cs} = K_c q_{c1N}$) and eq. [8]. Included in Fig. 13 are measured fines content values obtained from adjacent SPT samples. A reasonable comparison is seen between the estimated apparent fines contents and the measured fines contents. Note that for $I_c > 2.6$ (i.e., FC > 35%; see eq. [6]), the soil is considered to be nonliquefiable; however, this should be checked using other criteria (e.g., Marcuson et al. 1990) (see Fig. 4). The estimated zones of soil that are predicted to experience cyclic liquefaction are very similar to those observed and reported by Boulanger et al. (1995, 1997).

Fig. 12. Flow chart illustrating the application of the integrated CPT method of evaluating cyclic resistance ratio (CRR) in sandy soils. V_s, shear wave velocity.



The predicted zones of liquefaction are slightly conservative compared to the observed ground response. For example, when the CPT passes from a sand into a clay (e.g., at a depth of 10.5 m), the cone resistance decreases in the transition zone and the method predicts a very small zone of possible liquefaction. However, given the continuous nature of the CPT and the high frequency of data (typically every 20 mm), these erroneous predictions are easy to identify. In

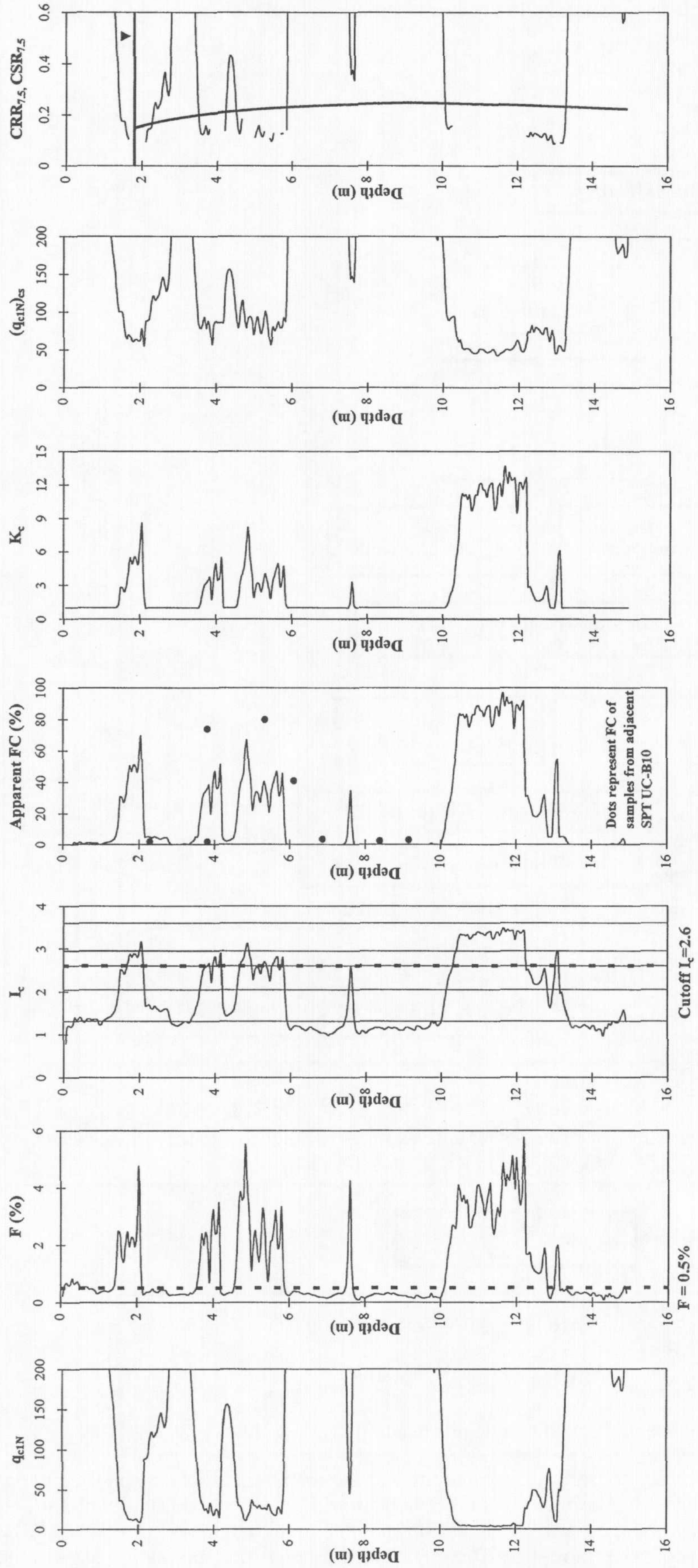
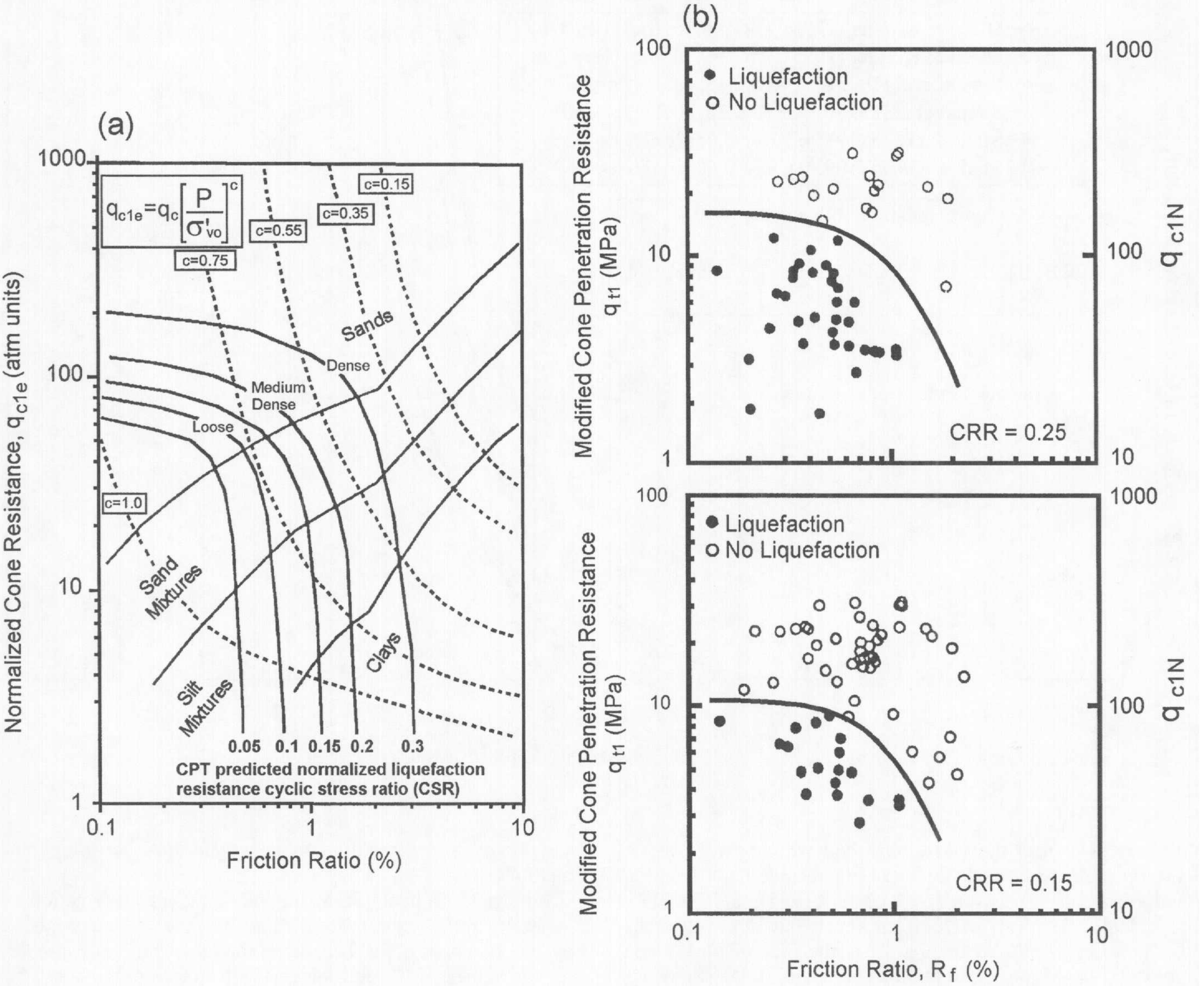


Fig. 13. Application of the integrated CPT method for estimating cyclic resistance ratio (CRR) to the State Beach site at Moss Landing which suffered cyclic liquefaction during the 1989 Loma Prieta earthquake in California.

Fig. 14. Comparison of estimating CRR from the CPT. (*a*) CRR = 0.05 to 0.3, after Olsen and Koester (1995). *c*, stress exponent. (*b*) CRR = 0.15 and 0.25, after Suzuki et al. (1995*b*). σ'_v, vertical effective stress.



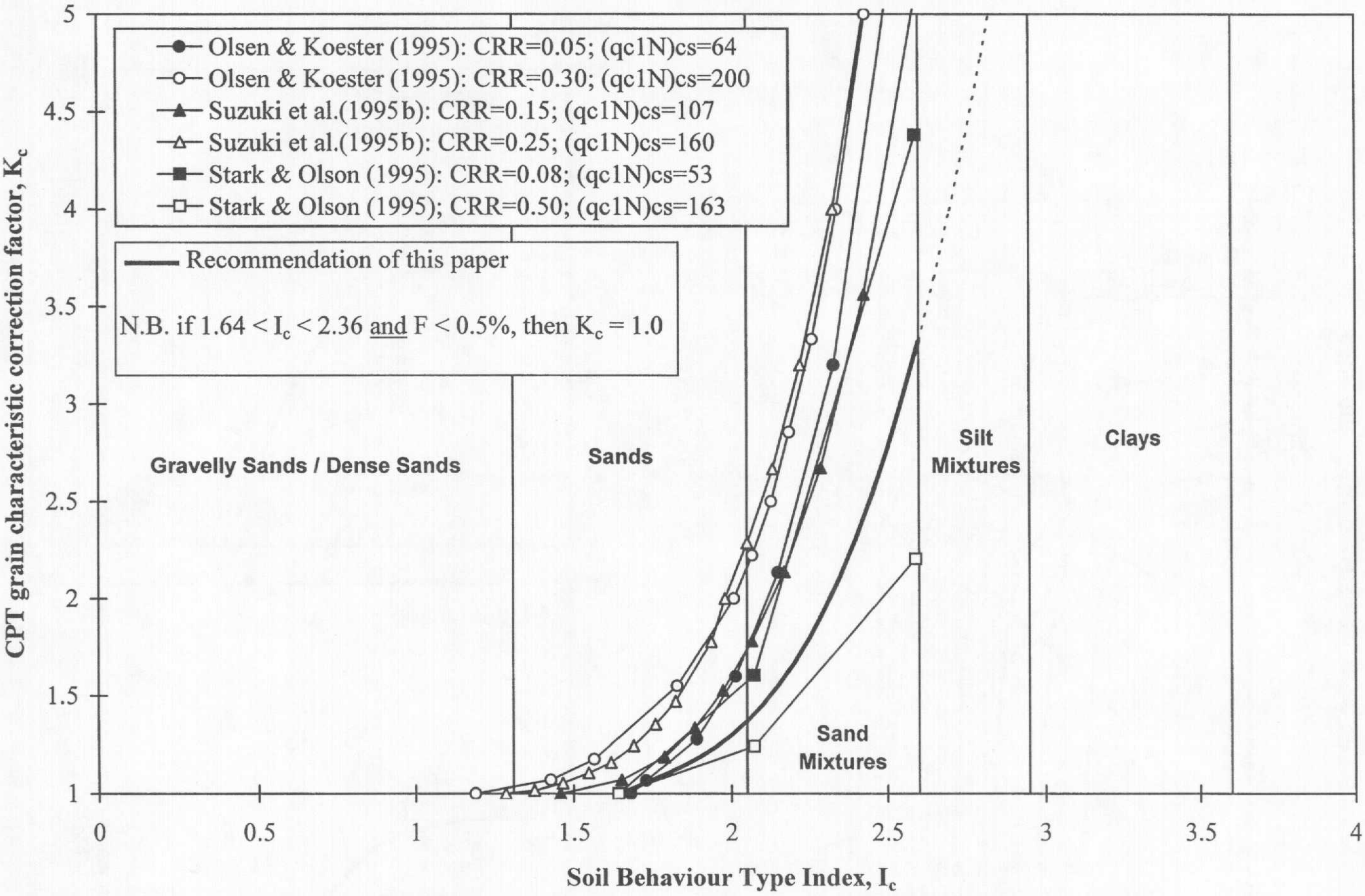
general, given the complexity of the problem, the proposed methodology provides a good overall prediction of the potential for liquefaction. Since the methodology should be applied to low-risk projects and in the initial screening stages of high-risk projects, it is preferred that the method produces, in general, slightly conservative results.

Comparison with other CPT methods

Olsen (1988), Olsen and Koester (1995), and Suzuki et al. (1995*b*) have also suggested integrated methods to estimate the CRR of sandy soils directly from CPT results with the correlations presented in the form of soil behaviour charts. The Olsen and Koester method is based on SPT–CPT conversions plus some laboratory-based CRR data. The method by Suzuki et al. is based on limited field observations. The methods by Olsen and Koester and Suzuki et al. are shown in Fig. 14. The Olsen and Koester method uses a variable normalization technique, which requires an iterative process to determine the normalization. The method by Suzuki et al. uses the q_{c1N} normalization suggested in this paper (eq. [3] with n = 0.5). The Olsen and Koester method is very sensitive to small variations in measured friction ratio and the user is not able to adjust the correlations based on site-specific experience. The friction sleeve measurement for the CPT can vary somewhat depending on specific CPT equipment and tolerance details between the cone and the sleeve and, hence, can be subject to some uncertainty. The method proposed in this paper is based on field observations and is essentially similar to those of Olsen and Koester and Suzuki et al.; however, the method described here is slightly more conservative and the process has been broken down into its individual components.

Built into each of the CPT methods for estimating CRR is the step of correcting the measured cone tip resistance to a clean sand equivalent value. It is the size of this correction that results in the largest differences between predicted val-

Fig. 15. Approximate comparison of various methods for correcting CPT tip resistance to clean sand equivalent values, based on soil behaviour type.



ues of CRR from the various methods. Figure 15 provides an approximate comparison between the methods by Stark and Olson (1995), Olsen and Koester (1995), and Suzuki et al. (1995*b*), in terms of K_c from soil behaviour type index, I_c. Also superimposed in Fig. 15 is the recommended relationship, based on the equations given in this paper. The comparisons are approximate because the different authors use different normalizations when correcting CPT tip resistance for plotting data on a soil classification chart. However, for effective overburden stresses in the order of 100 kPa (~1 tsf (tsf= ton-force per square foot)), all of the normalization methods should give similar values of normalized penetration resistance for the same value of measured penetration resistance. The boundaries between different soil behaviour types are also indicated in Fig. 15, as a guide to the type of soil in which corrections of certain magnitudes are suggested by the various methods.

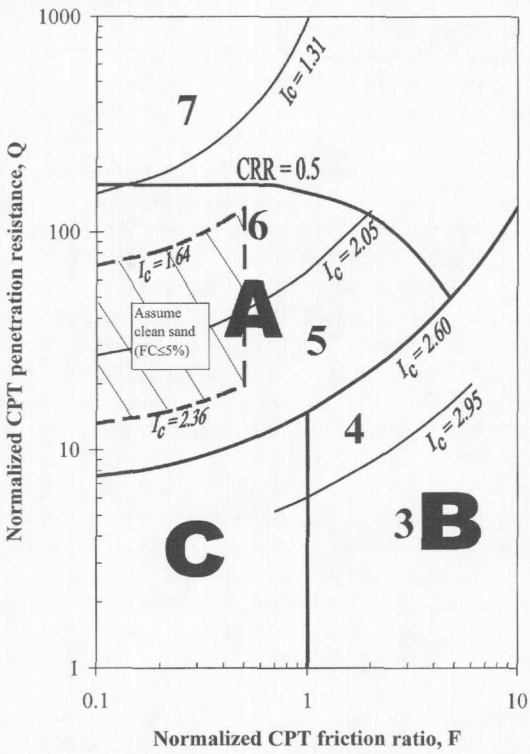
The methods by Suzuki et al. (1995*b*) and Olsen and Koester (1995) appear to be consistent with each other, indicating that K_c increases with increasing CRR. Very large corrections result especially in soils of high I_c. The method by Stark and Olson (1995) indicates that K_c decreases with increasing CRR for a given I_c and does not fit in with the trend of the combined Suzuki et al. and Olsen and Koester lines. The magnitudes of the corrections suggested by Stark and Olson are generally smaller than those of the other methods.

Figure 15 indicates that the recommended correction is generally more conservative than the corrections proposed by the other authors. The other methods generally predict higher values of K_c and suggest that corrections should be applied beginning at lower values of I_c, particularly for higher values of CRR. Note that the recommended relationship between K_c and I_c is shown in Fig. 15 as a broken line beyond I_c = 2.6, which corresponds to an apparent FC of 35% (eq. [6]). This shows that the integrated CPT method for evaluating CRR, as outlined here, does not apply to soils that would be classified as clayey silt, silty clay, or clay. As explained earlier, when interpretation of the CPT indicates that these types of soils are present, samples should be obtained and evaluated using other criteria, such as those given in Fig. 4 (Marcuson et al. 1990). It is logical that in nonliquefiable clay soils, the equivalent correction factor, K_c, could be very large for I_c > 2.6.

Recommendations

For low-risk, small-scale projects, the potential for cyclic liquefaction can be estimated using penetration tests such as the CPT. The CPT is generally more repeatable than the SPT

Fig. 16. Summary of liquefaction potential on soil classification chart by Robertson (1990). Zone A, cyclic liquefaction possible, depending on size and duration of cyclic loading; zone B, liquefaction unlikely, check other criteria; zone C, flow liquefaction and (or) cyclic liquefaction possible, depending on soil plasticity and sensitivity as well as size and duration of cyclic loading.



and is the preferred test, where possible. The CPT provides continuous profiles of penetration resistance, which are useful for identifying soil stratigraphy and for providing continuous profiles of estimated cyclic resistance ratio (CRR). Corrections are required for both the SPT and CPT for grain characteristics, such as fines content and plasticity. For the CPT, these corrections are best expressed as a function of soil behaviour type index, I_c, which is affected by a variety of grain characteristics.

For medium- to high-risk projects, the CPT can be useful for providing a preliminary estimate of liquefaction potential in sandy soils. For higher risk projects, and in cases where there is no previous CPT experience in the geologic region, it is also preferred practice to drill sufficient boreholes adjacent to CPT soundings to verify various soil types encountered and to perform index testing on disturbed samples. A procedure has been described to correct the measured cone resistance for grain characteristics based on the CPT soil behaviour type index, I_c. The corrections are approximate, since the CPT responds to many factors affecting soil behaviour. Expressing the corrections in terms of soil behaviour index is the preferred method of incorporating the effects of various grain characteristics, in addition to fines content. When possible, it is recommended that the corrections be evaluated and modified to suit a specific site and project. However, for small-scale, low-risk projects and in the initial screening process for higher risk projects, the suggested general corrections provide a useful guide. Correcting CPT results in thin sand layers embedded in softer fine-grained deposits may also be appropriate. The CPT is generally limited to sandy soils with limited gravel contents. In soils with high gravel contents, penetration may be limited.

A summary of the CPT method is shown in Fig. 16, which identifies the zones in which soils are susceptible to cyclic liquefaction (primarily zone A). In general, soils with $I_c > 2.6$ and $F > 1.0\%$ (zone B) are likely nonliquefiable. Soils that plot in the lower left portion of the chart (zone C; $I_c > 2.6$ and $F < 1.0\%$) may be susceptible to cyclic and (or) flow liquefaction due to the sensitive nature of these soils. Soils in this region should be evaluated using other criteria. Caution should also be exercised when extrapolating the suggested CPT correlations to conditions outside of the range from which the field performance data were obtained. An important feature to recognize is that the correlations appear to be based on average values for the inferred liquefied layers. However, the correlations are often applied to all measured CPT values, which include low values below the average for a given sand deposit and at the interface between different soil layers. Hence, the correlations could be conservative in variable stratified deposits where a small part of the penetration data could indicate possible liquefaction.

As mentioned earlier, it is clearly useful to evaluate CRR using more than one method. For example, the seismic CPT can provide a useful technique for independently evaluating liquefaction potential, since it measures both the usual CPT parameters and shear wave velocities within the same borehole. The CPT provides detailed profiles of cone tip resistance, but the penetration resistance is sensitive to grain characteristics, such as fines content and soil mineralogy, and hence corrections are required. The seismic part of the CPT provides a shear wave velocity profile typically averaged over 1 m intervals and, therefore, contains less detail than the cone tip resistance profile. However, shear wave velocity is less influenced by grain characteristics and few or no corrections are required (Robertson et al. 1992; Andrus and Stokoe 1998). Shear wave velocity should be measured with care to provide the most accurate results possible, since the estimated CRR is sensitive to small changes in shear wave velocity. There should be consistency in the liquefaction evaluation using either method. If the two methods provide different predictions of CRR profiles, samples should be obtained to evaluate the grain characteristics of the soil.

A final comment to be made is that a key advantage of the integrated CPT method described here is that the algorithms can easily be incorporated into a spreadsheet. As illustrated by the Moss Landing example presented here, the result is a straightforward method for analyzing entire CPT profiles in a continuous manner. This provides an useful tool for the engineer to review the potential for cyclic liquefaction across a site using engineering judgement.

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