

## Liquefaction of sands and its evaluation

(Keynote Lecture)

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**ABSTRACT:** Liquefaction of sands is a major problem in areas of earthquake loading and for the design of large sand structures such as mine tailings and earth dams. The mechanism of soil liquefaction is described and a set of definitions is presented. The evaluation of flow liquefaction and cyclic liquefaction based on both laboratory and field testing is described. Important features of soil behaviour are reviewed and typical results are presented. Loose sands are inherently anisotropic in undrained monotonic loading. The minimum undrained strength can be significantly smaller in extension loading than in compression. Hence, case histories based on back analyses of flow failures provide an average undrained strength which is a function of ground geometry. Laboratory testing on undisturbed samples is becoming increasingly more important, since soil structure can be difficult to replicate using reconstituted samples. Ground freezing is becoming one of the main methods for obtaining undisturbed samples of sandy soils. However, the cost is high and less expensive in-situ testing such as the Cone Penetration Test (CPT) will remain popular for smaller projects and for preliminary stages of larger projects. A new, fully integrated, method to evaluate both flow and cyclic liquefaction potential from the CPT is presented. This method incorporates corrections for grain characteristics such as fines content and mean grain size. This method is global in nature and site specific corrections are encouraged where possible. A worked example is presented to illustrate the methodology and the problems associated with the Standard Penetration Test (SPT).

## 1. INTRODUCTION

Soil liquefaction is a major concern for structures constructed with or on sandy soils. The phenomenon of soil liquefaction has been recognized for many years. Terzaghi and Peck (1948) 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 Yoshimi et al. (1977), Seed (1979), Finn (1981) and Ishihara (1993).

The major earthquakes of Niigata in 1964 and Kobe in 1995 have illustrated the significance and extent of damage 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.

Several phenomena are described as soil liquefaction. In an effort to clarify the different phenomena, the mechanisms will be described and a set of definitions for soil liquefaction will be presented. These definitions will be used throughout the paper in an effort to clearly identify each type of soil liquefaction. A full description of some previous definitions is given in the report by the National Research Council (1985).

The objective of this paper is to summarize the key issues regarding soil liquefaction and to discuss the various methods for evaluating liquefaction potential. Emphasis will be given to in-situ techniques although laboratory techniques will be discussed since much of what we understand about liquefaction has come from laboratory studies. A brief discussion will be given on post liquefaction effects and the consequences of liquefaction.

In this paper, reference will be made to a major research project underway in Canada. This project is the Canadian Liquefaction Experiment

(CANLEX) and is a major collaborative research project to study the characterization of sand for static and dynamic liquefaction. The CANLEX project is a collaboration between Canadian universities, industry and geotechnical consultants (List and Robertson, 1995).

## 2. LIQUEFACTION DEFINITIONS

Figure 1 shows the results from undrained triaxial compression tests on Toyoura sand presented by Ishihara (1993). These results present a clear picture of sand behaviour in undrained shear, since they show results at the same void ratio, but at different effective confining stresses. The results are presented in the form of deviator stress,  $q$ , versus axial strain and stress paths in  $q$  versus mean normal effective stress,  $p'$ . Very loose sand (density index = 16%), shows a marked strain softening response during undrained shear. The shear stress reaches a peak then strain softens, eventually reaching an ultimate condition referred to as critical or steady state. In this paper this ultimate condition will be referred to as 'ultimate state' (US). The stress path during strain softening appears to follow a 'collapse surface', as suggested by Sladen et al. (1985). However, at a lower confining stress, sand at the same void ratio shows a strain hardening response before reaching ultimate state. For the same sand at a higher density, a similar behaviour is seen, except that the ultimate state condition is reached at a higher stress level (Ishihara, 1993). For dense sand, the response is predominately strain hardening since the ultimate state strength is very large. This confirms the basic behaviour suggested by Castro (1969) and that embodied in critical state soil mechanics (Roscoe et al., 1958). Been et al. (1991) showed that steady state and critical state are the same condition and are independent of the stress path followed to reach this ultimate state. The steady state or critical state represents an ultimate state that can be represented by a line in  $e$ - $p'$  space, where  $p'$  is the effective mean normal stress,  $q$  is the deviator stress and  $e$  is the void ratio.

Figure 2 shows a summary of the behaviour of a granular soil loaded in undrained triaxial compression. In  $e$ - $p'$  space, a soil with an initial void ratio higher than the ultimate state line (USL) will strain soften (SS) at large strains, whereas a soil with an initial void ratio lower than the USL will strain harden (SH) at large strains. 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

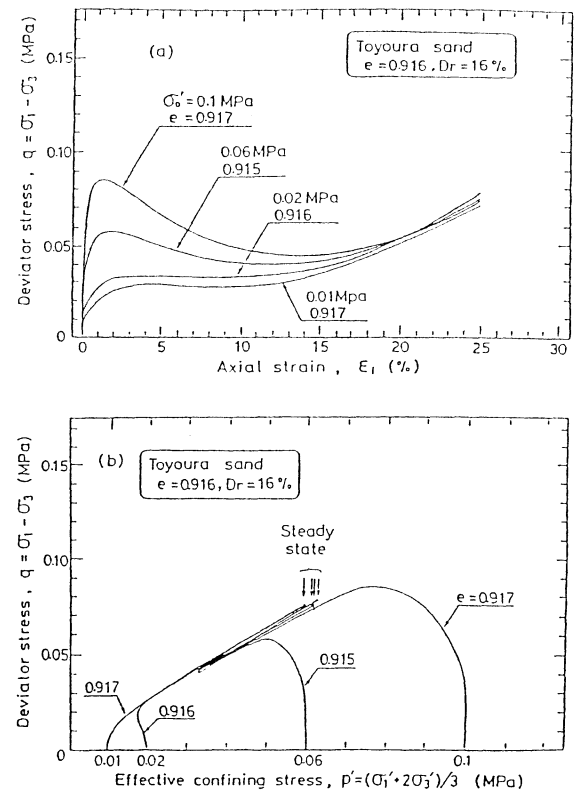


Figure 1 Undrained behaviour of Toyoura sand (after Ishihara, 1993).

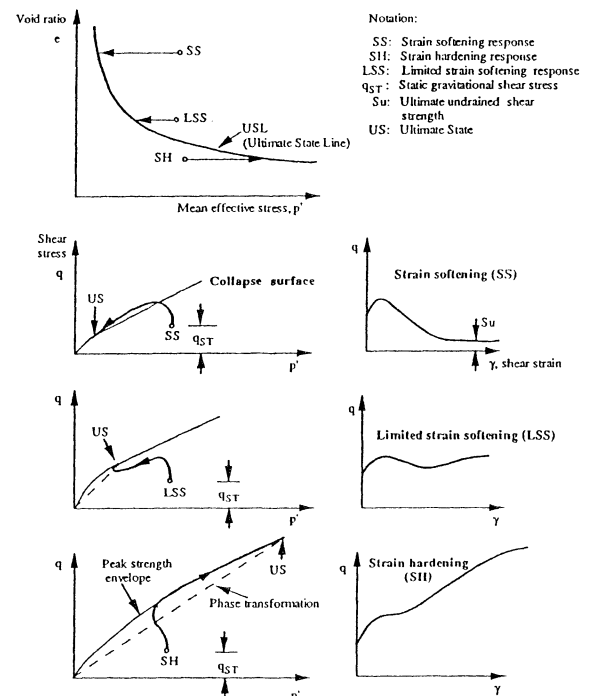


Figure 2 Schematic of undrained monotonic behaviour of sand in triaxial compression.

large strains are required to reach the ultimate state, and in some cases conventional triaxial equipment may not reach these large strains ( $\epsilon_a > 20\%$ ) (see Figure 1).

If a soil slope or structure, such as an earth dam or tailings dam, is composed entirely of a strain softening soil and the in-situ gravitational shear stresses are larger than the ultimate state strength (i.e. a relatively steep slope consisting of very loose sand), a catastrophic collapse and flow slide can occur if the soil is triggered to strain soften. Sasitharan et al. (1994) have shown that the collapse surface is a state boundary and controls the onset of soil structural collapse. The collapse can be triggered by either cyclic or monotonic undrained loading. Sasitharan et al. (1994) have also shown that undrained collapse can be triggered by certain types of drained monotonic loading (e.g. a slow rise in groundwater level).

If a soil structure is composed entirely of strain hardening soil, undrained collapse can generally not occur, unless the soil can become looser due to pore water redistribution. If a soil structure is composed of strain softening (SS) and strain hardening (SH) soil and the SS soil is triggered to strain soften, a collapse and slide will occur only if, after stress redistribution due to softening of the SS soil, the SH soil can not support the gravitational shear stresses. A flow slide will occur only if a kinimatically admissible mechanism can develop. In general, a kinimatically admissible mechanism can not form under level ground conditions. The trigger mechanism for a catastrophic flow slide can be cyclic, such as earthquake loading, or monotonic, such as a rise in groundwater level or a rapid undrained loading. Gu et al. (1993) used the collapse surface approach to explain the failure of the Lower San Fernando Dam which occurred shortly after the 1971 San Fernando earthquake. Gu et al. (1994) also used the collapse surface approach to explain the continued buildup of pore pressures and deformation after the 1987 Superstition Hills earthquake at the Wildlife Site in the Imperial Valley, California.

During cyclic undrained loading, almost all saturated cohesionless soils develop positive pore pressures due to the contractant 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 Figure 3. For shear stress reversal to occur, ground conditions must be generally level, gently sloping, or steeply sloping, but of moderate height (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,

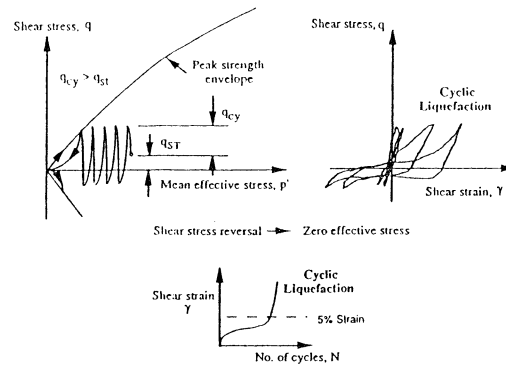


Figure 3 Schematic of undrained cyclic behaviour of sand illustrating cyclic liquefaction.

except for those due to local pore pressure redistribution. Gu et al. (1994) showed that the deformations due to pore pressure redistribution were very small at the Wildlife Site in the Imperial Valley. 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 dilative (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 (Pando and Robertson, 1995).

Based on the above description of soil behaviour in undrained shear and following the work by Robertson (1994), the following definitions of liquefaction are suggested:

## 2.1 Flow liquefaction

Applies to strain softening soils only.

- Requires a strain softening response in undrained loading resulting in constant shear stress and effective stress, as illustrated in Figure 2.
- Requires in-situ shear stresses greater than the ultimate or minimum undrained shear strength.
- Flow liquefaction can be triggered by either monotonic or cyclic loading.
- For failure of a soil structure to occur, such as a slope, a sufficient volume of material must strain soften. The resulting failure can be a slide or a flow depending on the material characteristics and ground geometry. The resulting movements are due to internal causes and can occur after the trigger mechanism occurs.

- Can occur in any metastable saturated soil, such as very loose granular deposits, very sensitive clays and loess (silt) deposits.

## 2.2 Cyclic softening

Two terms can be used to define cyclic softening. Applies to both strain softening and strain hardening soils.

### Cyclic liquefaction

- Requires undrained cyclic loading during which shear stress reversal occurs or zero shear stress can develop (i.e. occurs when in-situ static shear stresses are low compared to cyclic shear stresses), as illustrated in Figure 3.
- Requires sufficient undrained cyclic loading to allow effective stress to reach essentially zero.
- At the point of zero effective stress no shear stress exists. When shear stress is applied, pore pressure drops as the material tends to dilate, but a very soft initial stress strain response can develop resulting in large deformations.
- Deformations during cyclic loading can accumulate to large values, but generally stabilize when cyclic loading stops. The resulting movements are due to external causes and occur only during the cyclic loading.
- Can occur in almost all sands provided that the cyclic loading is sufficiently large in size and duration.
- Clayey soils can experience cyclic liquefaction but deformations are generally small due to the cohesive strength at zero effective stress. Deformations in clay soils are often controlled by rate effects (creep).

### Cyclic mobility

- Requires undrained cyclic loading during which shear stresses are always greater than zero; i.e. no shear stress reversal develops.
- Zero effective stress will not develop.
- Deformations during cyclic loading will stabilize, unless the soil is very loose and flow liquefaction is triggered. The resulting movements are due to external causes and occur only during the cyclic loading.
- Can occur in almost any sand provided that the cyclic loading is sufficiently large in size and duration and no shear stress reversal occurs.
- Clays can experience cyclic mobility but deformations are usually controlled by rate effects (creep).

Note that strain softening soils can also

experience cyclic softening (cyclic liquefaction or cyclic mobility) depending on the ground geometry.

Figure 4 presents a suggested flow chart (after Robertson, 1994) for the evaluation of liquefaction according to the above 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, geometry of the ground, amount and type of trigger mechanism, brittleness of strain softening soil and drainage conditions. Soils that are only temporarily strain-softening (i.e. experience a QSS 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.

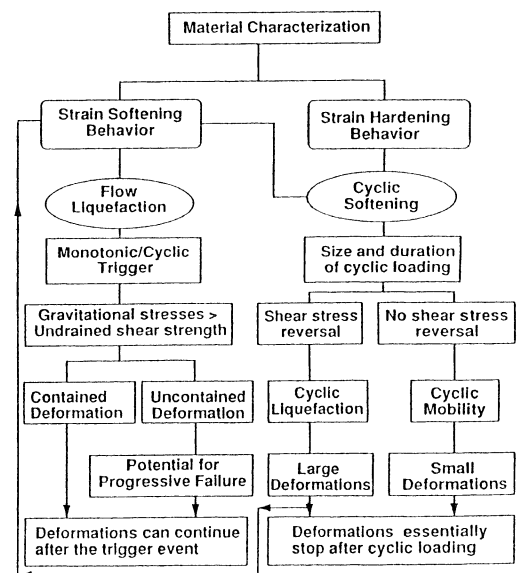


Figure 4 Suggested flow chart for evaluation of soil liquefaction (after Robertson, 1994).

If the soil is strain hardening, flow liquefaction will generally not occur. However, cyclic softening can occur due to cyclic undrained loading. The amount and extent of deformations during cyclic loading will depend on the density of the soil, the size 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 in the form of sand boils, damaged lifelines (pipelines, etc.), lateral spreads, slumping of small embankments and ground surface cracks.

If cyclic liquefaction occurs and drainage paths are restricted due to overlying less permeable layers, the sand near the surface can become looser 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 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.

### 3. FLOW LIQUEFACTION

The flow chart in Figure 4 shows that the first step in any liquefaction analysis should be to estimate the in-situ state of the soil to determine if flow liquefaction is possible (i.e. if the soil is strain-softening in undrained shear). The following sections discuss the influence of in-situ soil state on the undrained soil response and the various methods available to evaluate in-situ state.

#### 3.1 *Flow liquefaction potential based on laboratory testing*

Much has been published on the response of sands to various types of loading. This section will concentrate on the recent advances related to monotonic undrained loading of saturated sands, as it relates to our ability to predict if a soil is strain softening in undrained shear. The phenomenon associated with cyclic loading will be covered in a later section.

Ishihara (1993) presented a comprehensive review of test results on reconstituted Toyoura sand to illustrate the response of a clean sand to monotonic undrained loading. Most of the test results presented by Ishihara (1993) were on isotropically consolidated reconstituted samples tested in triaxial compression (as shown in Figure 1). These results clearly show the following main points:

1. There exists a steady/critical (ultimate) state at large strains in terms of void ratio and mean normal effective stress, which is independent of drainage conditions and initial soil fabric (i.e. sample preparation).

2. The projection of the ultimate state line (USL) is curved even in void ratio - log mean normal effective stress space.

3. At smaller strains some samples show a quasi-steady state (QSS), which is a function of both the initial state (void ratio and consolidation stress) and soil fabric.

The condition of quasi-steady state, during which there is a drop in resistance to some minimum value followed by a strain hardening response is illustrated in Figure 1. The shear stress mobilized at quasi-steady state is significantly smaller than the strength at the ultimate state (i.e. at very large strains). This type of behaviour has been called "flow with limited deformation". The shear strength at quasi-steady state will be referred to in this paper as the minimum undrained shear strength ( $S_{min}$ ). For very loose sand this can be equal to the ultimate undrained shear strength.

More recent laboratory testing (e.g. Georgiannou et al., 1990; Vaid et al., 1995a and 1995b) has

illustrated the importance of the initial consolidation stress state (i.e.  $K_0$ ) and direction of loading (e.g. triaxial compression, triaxial extension or simple shear) on the response of saturated sand to undrained monotonic loading. Figure 5 presents the results of anisotropically consolidated ( $K_0 = 0.5$  or  $K_c = \sigma'_{1c}/\sigma'_{3c} = 2$ ) undrained triaxial compression and extension tests on loose water pluviated reconstituted samples of Syncrude sand as part of the CANLEX Project (Vaid et al., 1995a). It is clear

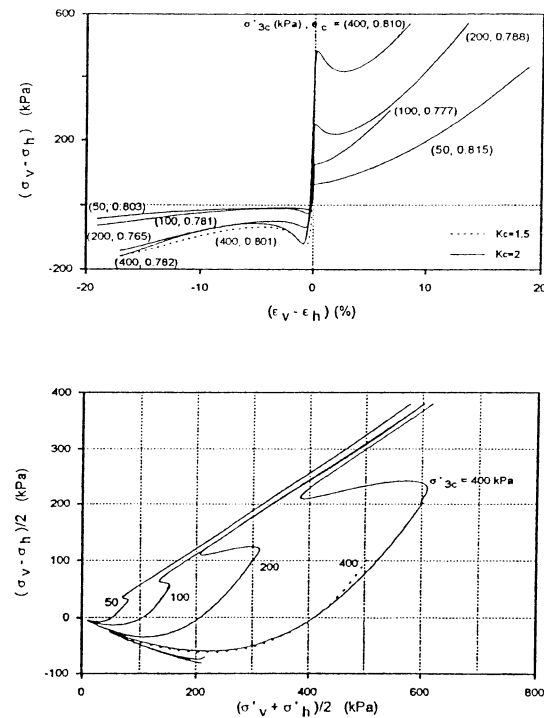


Figure 5 Triaxial undrained compression and extension behaviour of anisotropically consolidated loose water pluviated Syncrude sand (after Vaid et al., 1995a)

from Figure 5 that there are different responses in compression and extension loading and that the extension direction of loading produces a much lower quasi-steady state (minimum) strength than does the compression direction of loading. The amount of additional loading required to induce the strain softening response is, however, much less in compression than in extension due to the initial stress state ( $K_0 = 0.5$ ). The response in compression is, therefore, more brittle than the response in extension. Also, the application of static shear at constant confining stress promotes a more strain softening response despite a slight decrease in void ratio (Vaid et al., 1995a). The tests shown in

Figure 5 could not be carried out to a sufficiently large strain to evaluate if the ultimate state in  $e$ - $p'$  space was the same for both directions of loading. However, the stress paths for all tests are moving toward a common ultimate state. Been et al. (1991) showed results to suggest that the ultimate state is unique for a given sand, but that the strain levels required to attain ultimate state can sometimes exceed the capabilities of the triaxial equipment. The differences between undrained response in triaxial compression and extension have been partially attributed to inherent anisotropy in sands (Arthur and Menzies, 1972). The lower undrained strengths in extension can be at least partially explained by critical state soil mechanics theory, which predicts lower ratios of  $q_{us}/p'_{us}$  (i.e.  $M$ ) in extension than in compression for a constant value of  $\phi'_{us}$ .

Figure 6 shows the results of hollow cylinder torsion tests (HCT) carried out on loose water pluviated reconstituted samples of Syncrude sand under approximately plane strain conditions ( $b = [\sigma_2 - \sigma_3] / [\sigma_1 - \sigma_3] = 0.5$ ) to study the influence of direction of loading relative to the bedding planes ( $\alpha_\sigma$ ) (Vaid et al., 1995a). The

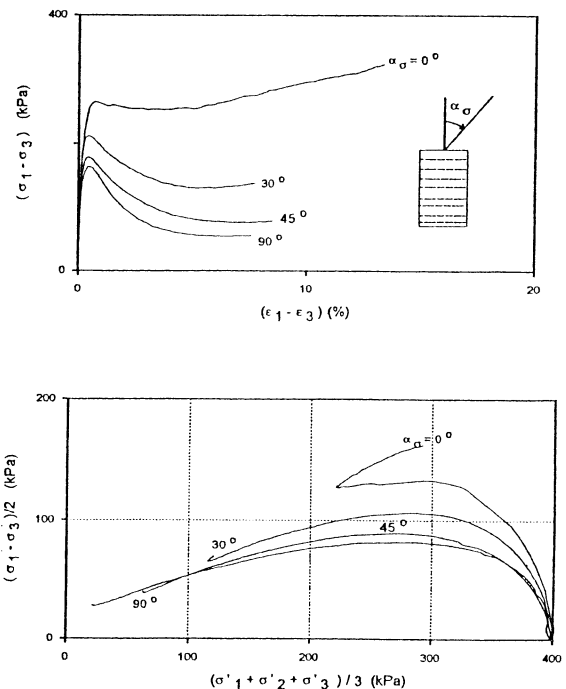


Figure 6 Undrained anisotropy in loose water pluviated Syncrude sand as measured in the hollow cylinder torsion test (after Vaid et al., 1995a).

variation in the direction of loading confirms the basic results shown in Figure 5 in that triaxial extension ( $\alpha_\sigma = 90^\circ$ ) shows a lower quasi-steady state strength than triaxial compression ( $\alpha_\sigma = 0^\circ$ ).

Simple shear deformation represents an initial, one-dimensional,  $K_0$ -consolidation state followed by a gradual rotation of the principal stress angle  $\alpha_\sigma$  from zero to about  $45^\circ$  at shear strains in excess of about 1 to 2% (Roscoe, 1970). This deformation mode evokes an expression of the inherent anisotropic nature of the sand deposit. The values of both  $\alpha_\sigma$  and the intermediate principal stress ratio (b) undergo changes during undrained simple shear. Results from simple shear tests, with an initial static shear ratio ( $\tau_{st} / \sigma'_{vc}$ ) of 0.3, but with no shear stress reversal during shear (i.e. monotonic tests), for loose water pluviated Syncrude sand (Vaid et al., 1995a) are shown in Figure 7. At low confining stress, the samples are strain softening but at higher confining stresses the samples become somewhat more strain hardening. The results also show that in undrained simple shear loading, loose sand requires very little additional shear to induce very large strains and an essentially strain softening response.

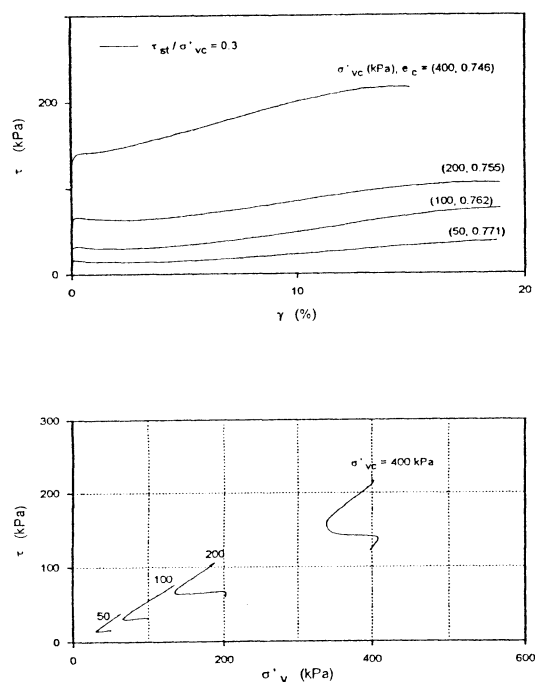


Figure 7 Constant volume simple shear behaviour of loose water pluviated Syncrude sand with static shear and no stress reversal (after Vaid et al., 1995a).

Figure 8 shows the results of undrained simple shear tests on Syncrude sand reconstituted to similar void ratios, but using different sample preparation techniques (Vaid et al., 1995b). These results illustrate that soil fabric has a significant influence on the undrained response at intermediate levels of strain ( $1\% < \epsilon_a < 15\%$ ). At very large strain levels (beyond those shown in Figure 8), the initial soil fabric should have almost no effect (Been and Jefferies, 1990; Ishihara, 1993).

The above results suggest the following general comments:

1. The response of sands is inherently anisotropic and is similar to that observed in many natural structured clay soils (e.g. Leroueil and Vaughan, 1990).

2. Anisotropically consolidated loose sands may strain soften in undrained shear after the application of very little additional shear.

Therefore, to evaluate the potential for possible flow liquefaction requires the following:

1. Evaluation of the in-situ state of the sand in terms of void ratio, effective confining stresses and soil fabric.

2. Laboratory testing under the appropriate direction of loading and in-situ state of the sand to determine the correct soil response.

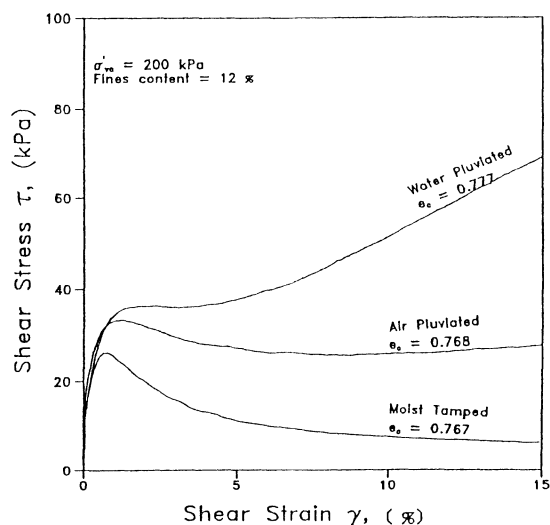


Figure 8 Undrained simple shear results on Syncrude sand using different reconstituting methods (after Vaid et al., 1995b).

For most sandy soils this would require testing of undisturbed samples. This testing should be carried out under anisotropic consolidation stresses. It is possible that testing could be performed on reconstituted samples provided that the in-situ soil

fabric could be reasonably reproduced. Tokimatsu and Hosaka (1986) suggested that either the small strain shear modulus or shear wave velocity measurements could be used to improve the value of laboratory testing of reconstituted samples of sand. Samples can be reconstituted in the laboratory in such a way that their small strain stiffnesses (i.e. shear wave velocities) and densities are equal to the in-situ values. The principle is that a reconstituted sample with the same elastic shear modulus and density as the same soil in-situ would possess the same response as the soil in-situ (Tokimatsu and Hosaka, 1986). The evidence for this was based on the cyclic resistance of sands. Although no evidence is yet available for monotonic response, the same conclusion is likely. This approach would only apply to uncemented sands. High quality geophysical logging is seen as a useful method for obtaining profiles of in-situ density (Plewes et al., 1990).

Recently, in-situ ground freezing has been used to obtain undisturbed samples of sandy soils (Yoshimi et al., 1978; Yoshimi et al., 1989; Yoshimi et al., 1994; Sego et al., 1994 and Hofmann et al. 1995). As part of the CANLEX project, in-situ ground freezing has been carried out at a total of four sites in Western Canada where loose, essentially saturated, sand deposits exist. The first site was at the Syncrude Canada Ltd. tailings storage facility in Fort McMurray, Alberta. The tailings sand is the result of oil extraction from natural oil sand and is used to hydraulically construct the containment dykes and supporting beaches of the storage facility. Syncrude sand is a fine, uniform, subrounded to subangular sand with traces of silt and clay. The predominately quartz sand has a  $D_{50} = 0.15$  mm and contains about 12% fines. The second and third sites (KIDD 2 and Massey South, respectively) are located near Vancouver, B.C. Both sites contain natural alluvial sediments as part of the Fraser River delta. At both sites, the target zone for sampling is within a 20 to 30 m thick complex of distributary channel sands that underlies most of the delta plain (Monahan et al., 1995). Fraser River sand is predominately quartz with some mica and feldspar, has a  $D_{50} = .30$  mm and contains on average about 5% fines. The fourth site is also located at the Syncrude Canada Ltd. site in Fort McMurray, but is the result of hydraulic placement of the sand tailings into water that was filling an old overburden pit. The resulting deposit is loose and very young (i.e. less than 6 months old). Extensive field and laboratory testing has been carried out at all four sites. The in-situ  $K_0$  conditions were estimated from high quality self-boring pressuremeter tests (Hughes et al., 1995).  $K_0$  was found to generally have a value close to 0.5 at all of the sites. The soil stratigraphy and in-situ density variations were measured using extensive Cone Penetration Tests

(CPT), seismic CPT, Standard Penetration Tests (SPT) with rod energy measurements (Campanella et al., 1995) and down-hole geophysical logging (Küpper et al., 1995). High quality conventional samples were also obtained using a thin-walled fixed piston sampler (Plewes and Hofmann, 1995). The results of some of the reconstituted testing on Syncrude sand are shown in Figures 5 to 8 and are described more fully by Vaid et al. (1995a and 1995b).

Figure 9 shows a summary of the data from tests on Fraser River sand in terms of in-situ void ratio against log mean normal effective stress,  $p'$ . Included in Figure 9 are the laboratory consolidation lines for isotropically consolidated reconstituted samples using both water pluviated and moist tamped techniques at their respective loosest states. Also indicated are regions that represent the in-situ state of the two CANLEX sites (KIDD 2 and Massey South) in their target zones, based on the void ratio values from undisturbed samples and geophysical logging. Note that some samples from the Massey South site are looser than can be achieved using water pluviated laboratory techniques. Superimposed on Figure 9 is the projection of the ultimate state line (USL) based on triaxial compression tests on isotropically consolidated reconstituted loose moist tamped Fraser River sand (Chillarige et al., 1995). Although the CANLEX sites were selected based on the estimated loose state of their sands, only some of the undisturbed samples from the Massey South site plot above the projection of the USL; all remaining samples plot below the USL. Based on the  $e_{max}$  values measured using ASTM standards (dry method), some of the Massey South samples have a relative density as loose as 5%.

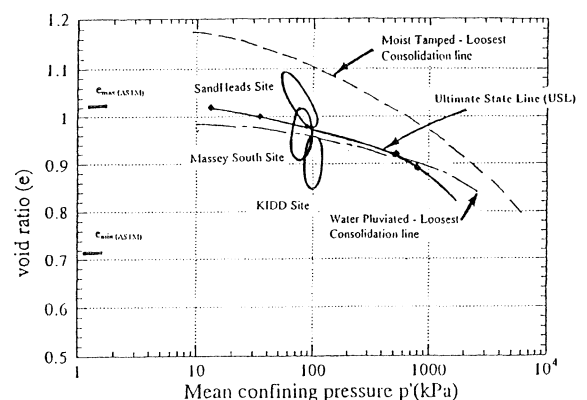


Figure 9 Summary of in-situ void ratio at different sites, laboratory consolidation curves and ultimate state line (USL) for Fraser River sand.

In order to model the in-situ sand behaviour, stress-strain relations are required. Due to the natural variability of most sand deposits, obtaining representative undisturbed samples and testing them in the appropriate direction of loading is not a simple process, since a very large amount of testing would be required. Hence, there is a need to define a framework within which the response of the sand can be described. A limited amount of laboratory testing can then be combined with continuous in-situ test results to provide a more complete picture of the expected ground response. The following sections attempt to describe such a framework.

There is considerable evidence to suggest that the response of sands can be described within a Critical State framework similar to that applied to clay soils (e.g. Atkinson, 1993; Coop et al., 1995). Pestana and Whittle (1995) have shown that there appears to be a limiting consolidation line for a given sand beyond which the sand can not exist, at least in a reconstituted state. Coop et al. (1995) refer to this line as the intrinsic normal compression line (NCL). This NCL is similar to the loosest state consolidation line for a given sand suggested by Ishihara (1993). Ishihara (1993) suggested that a knowledge of the location of the loosest consolidation state relative to the ultimate state line provided a measure of how loose the sand could possibly exist in a reconstituted state. The NCL is analogous to the intrinsic consolidation line (ICL) for clays suggested by Burland (1990). Burland (1990) showed that natural clays could exist outside the ICL due to processes, such as aging and cementation. Likewise, Cuccovillo and Coop (1993) showed that a calcarenite (carbonate sand/silt) could exist outside the reconstituted consolidation line (i.e. NCL) due to natural aging and cementation processes, as illustrated in Figure 10. Coop and Atkinson (1993) showed that the distance that the state moved outside the intrinsic line depended on the degree of cementation and that yielding was associated with fracture of both the cement and the carbonate grains. Atkinson (1993) suggested that the results of laboratory (drained and undrained) tests on soils could be presented in a normalized manner by normalizing the mean normal effective stress ( $p'$ ) and deviator stress ( $q$ ) by the mean normal effective stress on the projection of the critical (ultimate) state line at the same void ratio ( $p'_{us}$ ). Normalized stress paths for triaxial compression tests on intact calcarenite from Cuccovillo and Coop (1993) are shown in Figure 11. Also included on Figure 11 are broken lines which define the intrinsic state boundary surface obtained from tests on reconstituted samples. The intrinsic state boundary surface (SBS) is the surface that samples will follow when loaded in shear from their intrinsic consolidation line (NCL) to their critical (ultimate)

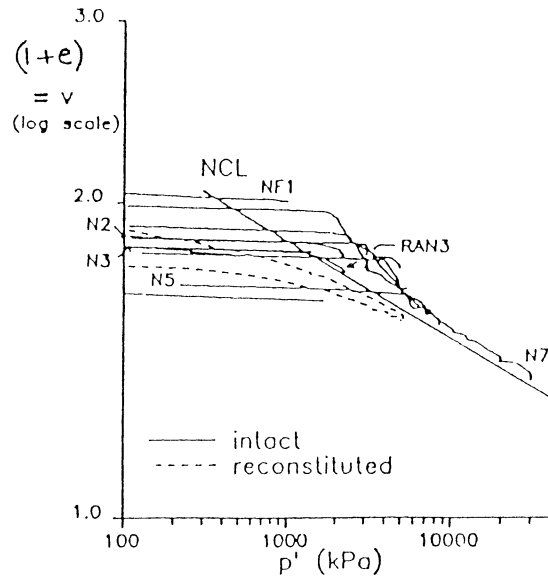
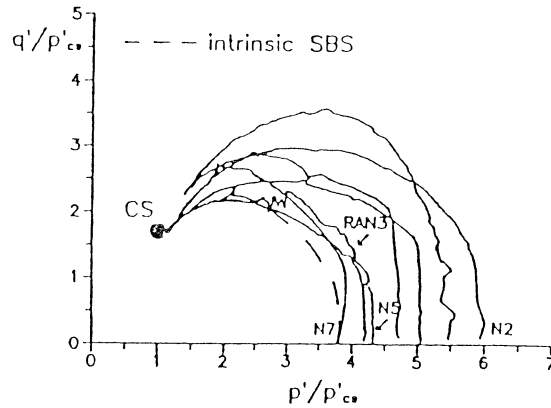


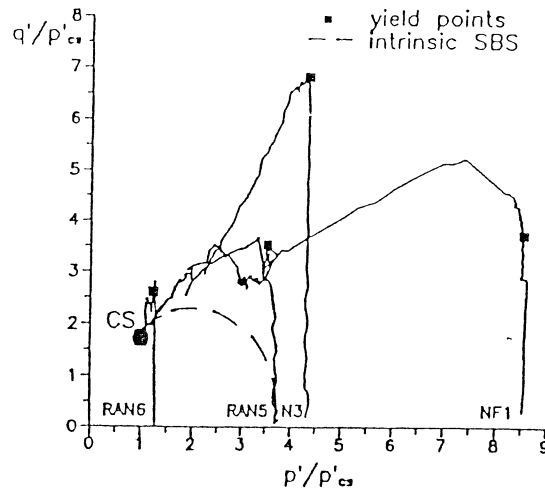
Figure 10 Isotropic compression curves for calcarenite (after Cuccovillo and Coop, 1993).

state. The results in Figure 11(b) show that the undisturbed intact calcarenite samples can have a very brittle response due to brittle fracture of the grains (Coop and Lee, 1993). The stress paths in Figure 11(a) are for samples which were initially compressed to states in excess of their yield stress; the stress paths are similar to those for the reconstituted samples. The ratio  $p'/p'_{us}$  represents a measure of the state of the soil relative to the ultimate state. The initial value of this ratio shall be referred to as the Reference Stress Ratio ( $RSR = p'/p'_{us}$ ) (Fear et al., 1995). If RSR is greater than 1, the soil would be expected to contract to ultimate state, as shown in Figure 11. The greater the value of RSR, the more contractant and potentially more brittle and strain softening is the soil. The suggested application of RSR for sands is similar to the current application of OCR (overconsolidation ratio) for clays.

Figure 12 shows the normalized stress paths for anisotropically consolidated ( $K_0 = 0.5$ ) Fraser River sand loaded in undrained triaxial compression and extension. The broken lines represent the estimated state boundary surface (SBS) for the loosest moist tamped reconstituted samples (stress paths indicated by the thinner solid lines). The thicker solid lines represent the stress paths for the loosest undisturbed samples (Massey South site). For the loosest undisturbed samples at this site, the sand is below the intrinsic consolidation line (NCL) (see Figure 9) and the samples are therefore not as strain softening as the loosest reconstituted moist tamped samples.



(a) Samples yielding during compression.



(b) Samples yielding during shearing.

Figure 11 State paths for intact calcarenite (after Cuccovillo and Coop, 1993).

The key components that can represent the stress-strain curves of a soil loaded in undrained shear are: brittleness, minimum shear strength (ultimate or quasi-steady-state strength), strain that occurs at minimum strength, ultimate strength (generally this would be close to the strength at the end of the test), and stress ratio ( $M = q'/p'$ ) at both peak and ultimate state (Fear et al., 1995). Figure 13 illustrates a method of quantifying the various components of response for both triaxial compression and extension loading. Note that Brittleness Index ( $I_B$ ) is defined as follows:

$$I_B = \frac{S_p - S_{\min}}{S_p - S_i}$$

for triaxial compression

$$I_B = \frac{S_p - S_{\min}}{S_p + S_i} \quad (1b)$$

for triaxial extension

where:

$S_p$  = magnitude of the peak shear strength

$S_{\min}$  = magnitude of the minimum shear strength

$S_i$  = magnitude of the initial static shear stress.

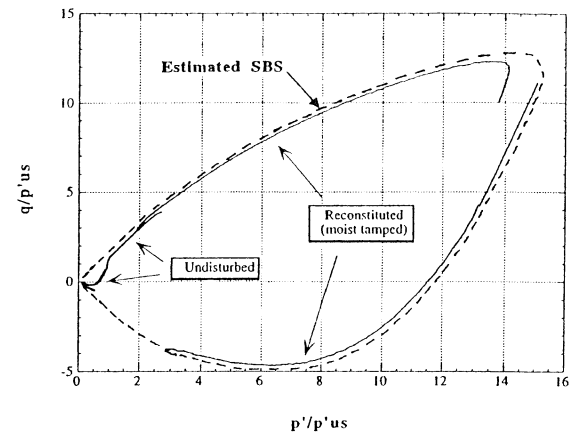
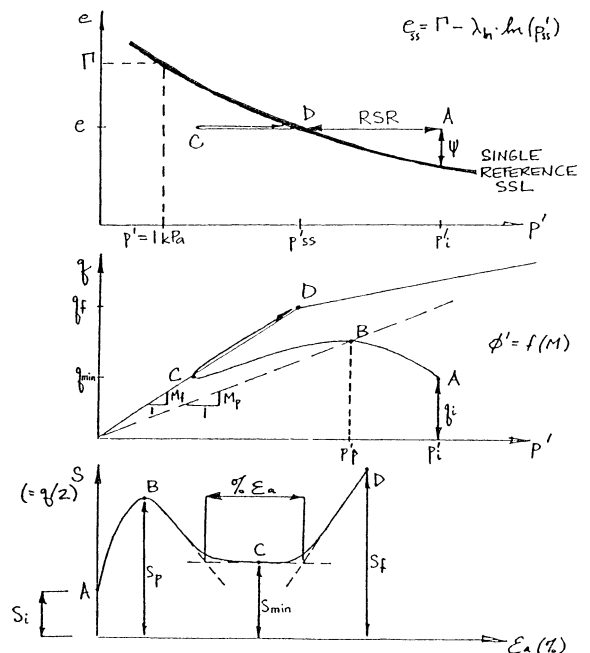


Figure 12 Normalized stress paths for anisotropically consolidated Fraser River sand; loosest moist tamped state and typical undisturbed state.



(1a) Figure 13a Schematic of components of soil response for triaxial compression loading (after Fear et al., 1995).

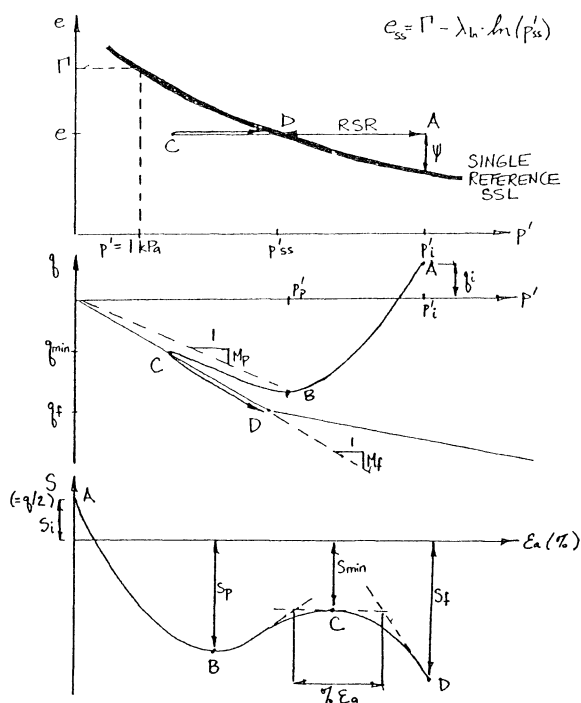


Figure 13b Schematic of components of soil response for triaxial extension loading (after Fear et al., 1995).

Using this definition of Brittleness Index, it is possible to quantify the loss in strength more clearly for anisotropically consolidated samples (i.e.  $S_i > 0$ ).

Figure 14 illustrates the suggested interaction between field characterization and the response of samples in the laboratory. The link between the two can be RSR; however, to calculate RSR, the location of the ultimate state line (USL) is required. The

USL is controlled by the grain characteristics of the soil. Hence, the incorporation of the USL provides a link to grain characteristics. For the CANLEX data, the USL was determined using very loose moist tamped reconstituted samples of the same sand tested in undrained triaxial compression. Although this may not represent a unique USL, it is relatively easy to determine and forms a useful reference line. Figure 15(a) shows the initial states of various samples of Fraser River sand relative to the reference USL based on the work by Chillarige et al. (1995). Figures 15(b) to (d) show the variation of Brittleness Index, minimum shear strength and axial strain at minimum strength with RSR for anisotropically consolidated undisturbed (U) samples of Fraser River sand. Also included in Figure 15 are values from tests on anisotropically consolidated reconstituted samples using both water pluviated (WP) and moist tamped (MT) techniques. The results show that the response of the undisturbed samples at the Massey South site is closer to that of the water pluviated reconstituted samples. The Massey South site is a very recent (i.e. less than 200 years old), predominately quartz, deposit (Monahan et al., 1995). Hence, it appears reasonable that the undisturbed samples would be little affected by aging and have a response similar to reconstituted samples. The undisturbed samples show a much lower minimum shear strength in triaxial extension than in compression. Samples were strain hardening in compression loading but showed limited strain softening in triaxial extension. Two samples were also strain softening in simple shear.

The results of laboratory testing on undisturbed samples can be linked to the field characterization data using the method illustrated in Figure 14. The following section will describe the recent advances in field characterization.

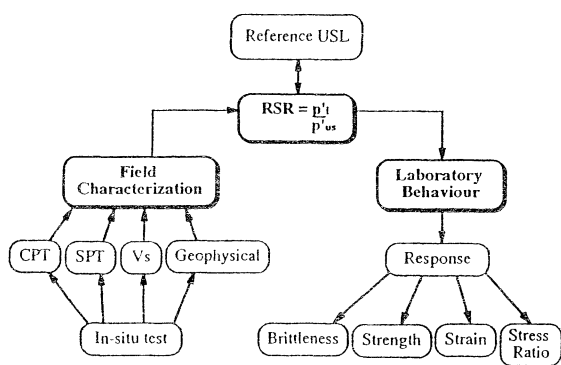


Figure 14 Suggested interaction between field characterization and in-situ response from laboratory testing (after Fear et al., 1995).

### 3.2 Flow liquefaction potential based on field testing

Most of the early work to estimate the in-situ state of a sand was based on estimates of relative density. However, research has clearly shown that relative density alone can not describe the in-situ state of a sand. In-situ sand state requires a knowledge of both the density (void ratio) and effective stresses, as well as the soil structure. Been and Jefferies (1985), based on the early work of Wroth and Bassett (1965), suggested the use of state parameter ( $\psi$ ) to define sand state. State parameter is defined as the difference in void ratio between the in-situ void ratio and the void ratio on the ultimate state line (USL) at the same mean normal effective stress ( $p'$ ). If the USL can be assumed to be straight in  $e$ - $\ln p'$  space

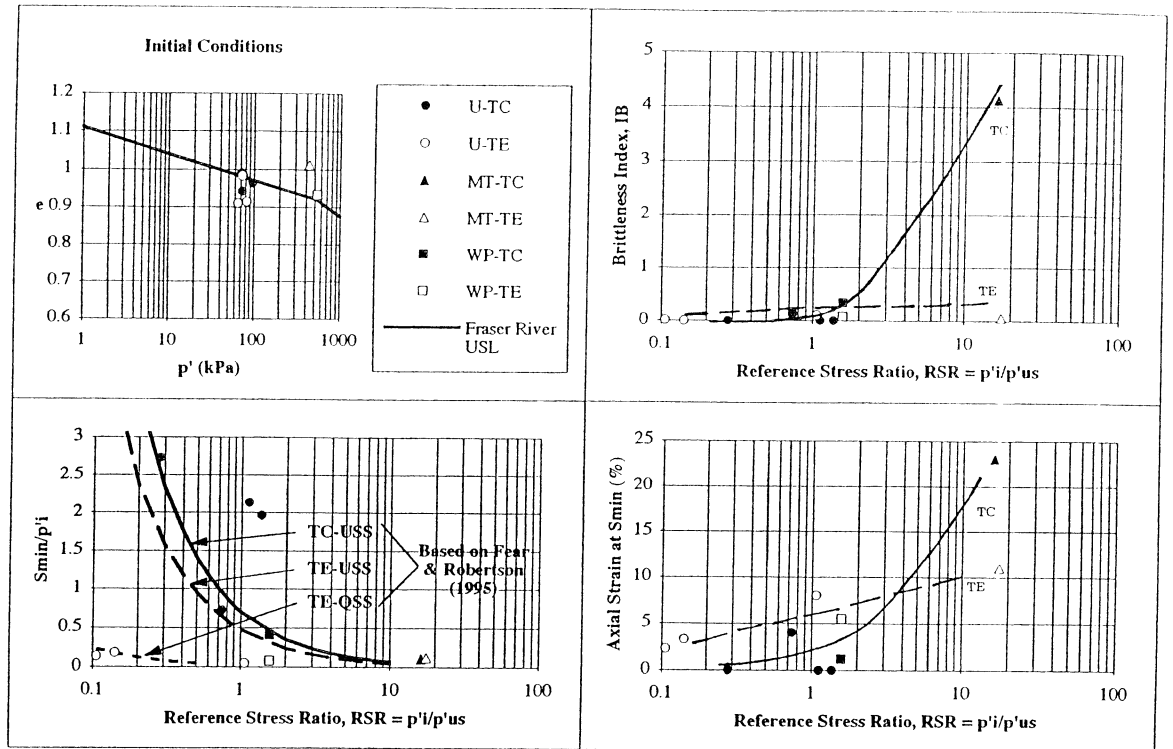


Figure 15 Results of triaxial tests on samples of Fraser River sand. (a) Initial states relative to the Fraser River ultimate state line (USL). Variation with Reference Stress Ratio (RSR) of (b) Brittleness Index, (c) minimum shear strength and (d) axial strain at minimum strength.

over a given range of void ratio, the state parameter ( $\psi$ ) and Reference Stress Ratio (RSR) are related as follows:

$$RSR = \frac{p'_i}{p'_{us}} = \exp\left(\frac{\psi}{\lambda_{ln}}\right) \quad (2)$$

where:  $\lambda_{ln}$  = slope of USL in  $e$ - $\ln p'$  space

Since the slope of the USL ( $\lambda_{ln}$ ) is a function of grain characteristics and stress level, RSR is a more generic measure of state for a wide range of soils. When  $\psi = 0$ ,  $RSR = 1.0$  and the state falls on the USL in  $e$ - $\ln p'$  space.

Been et al. (1986) suggested a method to estimate the in-situ state parameter using CPT results. This method was based on results from large calibration chamber studies. However, there is some uncertainty over the corrections made for chamber size effects. Since the results were based mostly on relatively dense sand samples, for which corrections for chamber size effects were large, and the resulting correlations needed extrapolation into the loose sand region, there is some uncertainty in the resulting

correlations for loose sands. Been et al. (1986) also noted that the resulting correlations are not unique for all sands but depend on sand compressibility, which is reflected indirectly in the slope of the ultimate state line ( $\lambda_{ln}$ ). Sand compressibility also has a major influence on the empirical correlations for relative density (Robertson and Campanella, 1983).

Because of the difficulty in estimating in-situ sand state from penetration resistance, Robertson et al. (1995a) recently suggested a method for estimating the state of a young, uncemented sand using shear wave velocity ( $V_s$ ). This method links measurements of  $V_s$  on small samples in the laboratory at known states with in-situ measurements of  $V_s$ . Shear wave velocity is primarily a function of soil void ratio and effective confining stresses, but is also influenced by soil structure (i.e. age, cementation and fabric). For young, uncemented sands, shear wave velocity is controlled primarily by void ratio and effective stresses. Hence, the potential exists for estimating the in-situ state of a young, uncemented sand from in-situ shear wave velocity measurements. Shear wave velocity has the advantage that it can be easily

measured both in the field and in the laboratory without the need for difficult and uncertain corrections. In the field, shear wave velocity can be measured using the seismic Cone Penetration Test (SCPT). In the laboratory, it can be measured using bender elements (Dyvik and Madhus, 1985).

Figure 16 shows the relationship between normalized shear wave velocity ( $V_{s1}$ ) and void ratio ( $e$ ) for a wide range of freshly deposited uncemented sand samples in the laboratory. Normalized shear wave velocity ( $V_{s1}$ ) is defined as follows:

$$V_{s1} = V_s (P_a / \sigma_{vo}')^{0.25} \quad (3)$$

where:

$V_{s1}$  = normalized shear wave velocity

$V_s$  = measured shear wave velocity

$P_a$  = atmospheric pressure, usually = 100 kPa

$\sigma_{vo}'$  = vertical effective stress in kPa.

The relationship between  $V_{s1}$  and  $e$  can be approximated over a given range of void ratio by:

$$V_{s1} = (A - Be) K_o^{0.125} \quad (4)$$

where:

$A$  and  $B$  = constants for a given sand, both in m/s.

$K_o$  = the coefficient of earth pressure at rest.

$e$  = void ratio

$V_{s1}$  is in m/s.

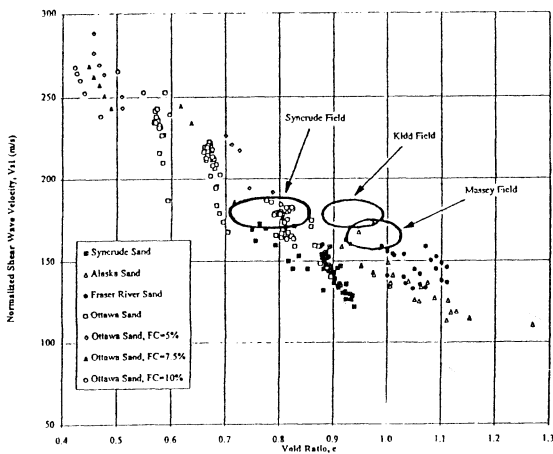


Figure 16 Variation of normalized shear wave velocity with void ratio for a range of sands.

There is evidence that the above linear relationship is limited to a certain range of void ratios and that a more general relationship would be non-linear (Reilly, 1995).

The in-situ  $V_s$  is controlled by the effective stresses in both the direction of wave propagation and the direction of particle motion. For the seismic CPT, this means that  $V_s$  is controlled by the vertical and horizontal effective stresses. Hence, Equation 3 should include the horizontal effective stress as well as the vertical effective stress. However, in practice, the horizontal effective stress is generally unknown; therefore, the recommended normalization includes only the vertical effective stress. The error in excluding the horizontal effective stress is generally less than 10%. However, when using in-situ shear wave velocity measurements to estimate in-situ void ratio, it is recommended to account for  $K_o$  conditions, as shown in Equation 4.

Figure 17 shows the resulting contours of shear wave velocity for  $K_o = 1$ , based on Figure 16 and Equation 4, superimposed over the USL for Syncrude sand in void ratio ( $e$ ) versus the logarithm of mean normal effective stress ( $p'$ ). It is clear from Figure 17 that the state of a sand can be evaluated from the in-situ shear wave velocity, provided that the location of a reference USL is known.

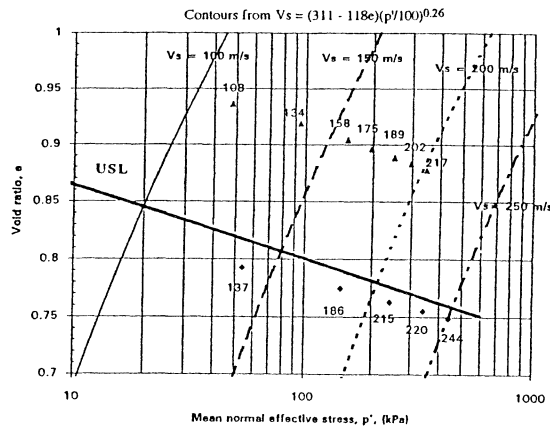


Figure 17 Comparison between the ultimate state line (USL) and contours of constant shear wave velocity for Syncrude sand (after Robertson et al., 1994).

Based on the definition of state parameter, Robertson et al. (1994) developed the following expression to estimate sand state from shear wave velocity:

$$\Psi = \left( \frac{A}{B} - \Gamma \right) - \left( \frac{V_{s1}}{B (K_o)^{na}} - \lambda_{ln} \ln \left[ \frac{\sigma_{vo}'}{3} (1 + 2K_o) \right] \right) \quad (5)$$

where:

$\Gamma$  = intercept of USL at  $p' = 1$  kPa

$\lambda_{ln}$  = slope of USL in  $e$ - $\ln p'$  space, over a given range of void ratio

A and B = constants for a given sand, both in m/s  
 $n_a$  = stress exponent; typically,  $n_a = 0.125$   
 $V_{s1}$  is in m/s and  $\sigma_{v0}'$  is in kPa.

This relationship assumes that the USL can be represented by a straight line in  $e-\ln p'$  space (i.e. constant  $\Gamma$  and  $\lambda_{ln}$ ) over a specific range of void ratio. Figure 18 shows the resulting correlation between  $V_s$  and vertical effective stress ( $K_0 = 0.4$ ) for  $\psi = 0$  for Syncrude sand. It is clear from Equation 5 that the correlation between  $V_{s1}$  and in-situ state ( $\psi$ ) is primarily controlled by the location of the USL in  $e-\ln p'$  space (i.e. the values of  $\Gamma$  and  $\lambda_{ln}$ ) for a given sand.

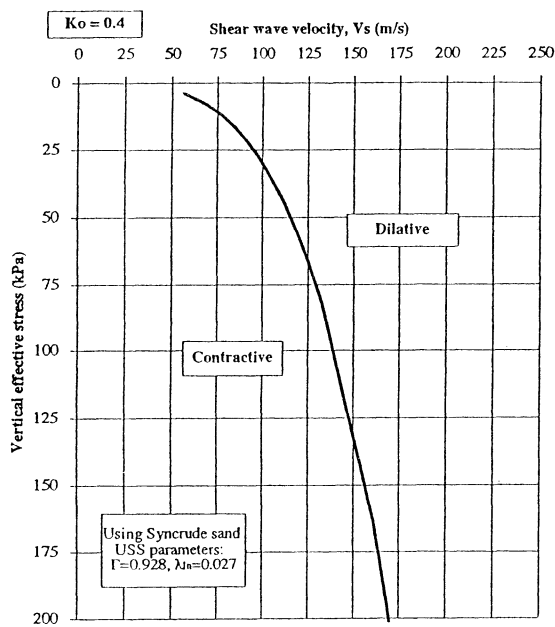


Figure 18 Relationship between shear wave velocity and vertical effective stress for state parameter ( $\psi$ ) = 0 for Syncrude sand when  $K_0 = 0.4$  (after Robertson et al., 1994).

If in-situ shear wave velocity is measured using the seismic CPT, it is possible to develop a site specific correlation between  $V_s$  and cone penetration resistance ( $q_c$ ), and hence develop a site specific correlation between  $q_c$  and in-situ sand state ( $\psi$ ). Since cone penetration resistance and shear wave velocity vary with overburden effective stress in different ways, it is important to compare normalized values. Fear and Robertson (1995) suggested that the relationship between normalized shear wave velocity and normalized cone penetration

resistance can be approximated by the following equation:

$$q_{c1} = (V_{s1} / Y)^4 \quad (6)$$

where:

$q_{c1}$  = normalized cone penetration resistance, in MPa

$Y$  = conversion factor between  $V_{s1}$  and  $q_{c1}$ , in  $(m/s)/(MPa)^{0.25}$

$V_{s1}$  is in m/s

The normalized cone penetration resistance is given by:

$$q_{c1} = q_c (P_a / \sigma_{v0}')^{0.5} \quad (7)$$

where:  $q_c$  = measured cone penetration resistance

The parameter  $Y$  appears to be controlled by grain characteristics, sand compressibility, age and degree of cementation. For more compressible sands, the parameter  $Y$  will increase since the cone penetration resistance,  $q_{c1}$ , will decrease while the shear wave velocity ( $V_{s1}$ ) will remain essentially constant. For aged or cemented sands,  $Y$  will also increase since shear wave velocity will increase faster than penetration resistance (Robertson et al. 1995b).

Since shear wave velocity is little influenced by sand compressibility, while the cone penetration resistance is strongly influenced by sand compressibility, there appears to be the potential for identifying sand compressibility by comparing shear wave velocity and cone penetration resistance in a sand at the same depth. This can be accomplished using the seismic CPT in which both cone resistance and shear wave velocity are measured during the same sounding in the same soil. The slope of the USL ( $\lambda_{ln}$ ) is an indirect measure of the inherent grain compressibility of a sand. Based on a number of sites where seismic CPT has been carried out in addition to laboratory testing to determine  $\lambda_{ln}$ , a preliminary correlation between  $\lambda_{ln}$  and the parameter  $Y$  has been developed (Robertson et al., 1995b) and is shown in Figure 19. Included in Figure 19 are estimated contours of age. Figure 19 applies only to sands that are uncemented.

Over a given range of void ratio (i.e. straight USL with constant  $\lambda_{ln}$ ), the RSR for a sand can be estimated from either the CPT or in-situ shear wave velocity measurements since state parameter ( $\psi$ ) is linked to RSR by Equation 2.

As a part of CANLEX, seismic CPTs have been carried out at the four sites where in-situ undisturbed samples of sand have been obtained using in-situ

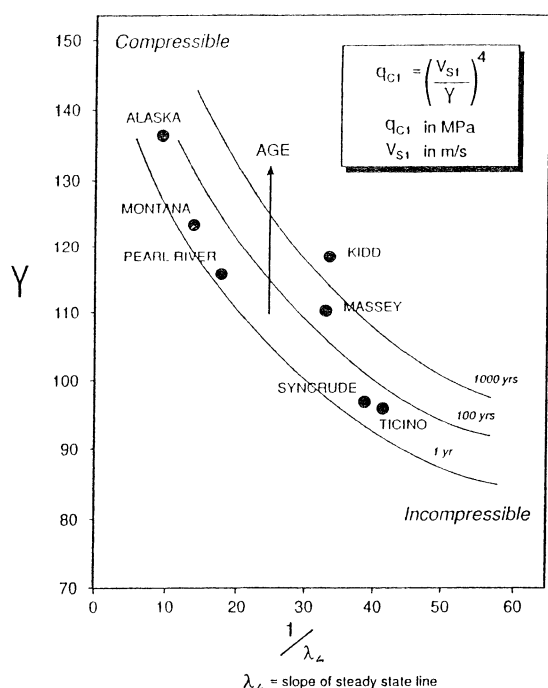


Figure 19 Proposed correlation between shear wave velocity and CPT tip resistance and the slope of the ultimate state line  $1/\lambda_u$  (after Robertson et al., 1995).

ground freezing (Sego et al., 1994). Based on a comparison between predicted and measured void ratios, the influence of aging on normalized shear wave velocity can be quantified. Figure 20 shows a plot of the change in normalized shear wave velocity versus age for the CANLEX sites. Although limited data are available for the influence of aging on CPT penetration resistance, there is evidence of small effects of aging on SPT penetration resistance (Skempton, 1986; Kulhawy and Mayne, 1990). A similar small increase in CPT penetration resistance with age can also be expected.

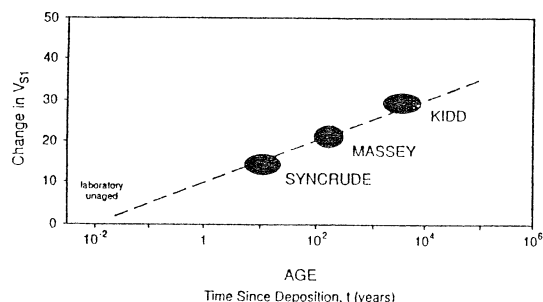


Figure 20 Change in normalized shear wave velocity with age for uncemented sands (after Robertson et al., 1995).

Independent of the CANLEX project, Chillarige et al. (1995) studied the stability of a submarine slope at the mouth of the Fraser River delta near the Sandheads lighthouse following a large flow slide that occurred in 1985 (McKenna et al., 1992). This slide involved about 1,400,000 m<sup>3</sup> of sediments which appeared to have flowed out into Georgia Strait. Chillarige et al. (1995) investigated various potential trigger mechanisms and concluded that the slide was triggered by a combination of factors, as follows:

1. The sandy sediments were very loose (RSR > 1) and strain softening in undrained shear.
2. The sediments were partly saturated due to small amounts of gas formed by the rapid sedimentation and biodegradation of organisms.
3. The sandy sediments were triggered to strain soften by a decrease in effective stresses during a falling tide.

The small amounts of gas were sufficient to produce a time lag in the pore pressure response during tidal changes and the falling tide produced a decrease in effective confining stresses. This decrease in effective confining stresses produced a stress path having essentially constant static shear stresses due to the sloping ground ( $q = \text{constant}$ ), but decreasing  $p'$ . Sasitharan et al. (1994) showed that a  $q = \text{constant}$  and slowly decreasing  $p'$  stress path could trigger undrained collapse of very loose sand samples in the laboratory. This same mechanism was probably the cause of the 1985 flow slide at Sandheads. Based on in-situ CPT and shear wave velocity measurements at the top of the submarine slope adjacent to the Sandheads lighthouse, and using the method described above to estimate in-situ state, the young sandy sediments appear to have an in-situ state looser than that at the Massey South site. The estimated in-situ state of the sand at Sandheads is included in Figure 9. The sand at Sandheads has essentially the same grain characteristics as that found at the Massey South site. The very loose nature of the Sandheads sediments is thought to have been formed by the unique sedimentation environment found at the mouth of the river and the very young age of the surficial deposit (less than 1 or 2 years old). The observation that the sand has a very loose state is supported by the frequent occurrence of rapid submarine flow slides at the mouth of the Fraser River.

The major difficulties associated with the interpretation of penetration tests, such as the CPT, relate to the influence of variations in fines content and layer thickness. Most of the existing interpretation methods for the CPT and SPT apply only to clean sands. Increasing the fines content of a sand generally decreases its penetration resistance due to the changing permeability and compressibility of the soil. Ishihara (1993) presented a correction to

measured values of CPT penetration resistance for estimating the state of sandy soils. The shear wave velocity method described above overcomes much of this problem since shear wave velocity, being a small strain measurement, is almost unaffected by fines content. The seismic CPT offers a combination of penetration resistance and shear wave velocity measurements to allow for a site specific correlation between soil state and penetration resistance. However, most methods which are used to measure shear wave velocity provide a rather coarse measure of the ground profile compared to the detail provided by the CPT.

It is possible to estimate grain characteristics, such as fines content, of sandy soils directly from the CPT and, hence, correct the measured penetration resistance to an equivalent clean sand value. In recent years, charts have been developed to estimate soil type from CPT data (Olsen and Malone, 1988; Robertson and Campanella, 1988; Robertson, 1990). Experience has shown that the friction ratio increases with increasing fines content, as illustrated in Figure 21. Hence, fines content can be estimated from CPT data using soil behaviour charts, such as that shown in Figure 22. The addition of pore pressure data can also provide valuable additional guidance in estimating fines content. Robertson et al. (1992) suggested a method to estimate fines content based on the rate of pore pressure dissipation ( $t_{50}$ ) during a pause in the CPT.

Based on experience and the results shown in Figure 21, it is possible to estimate grain characteristics, such as fines content and grain size

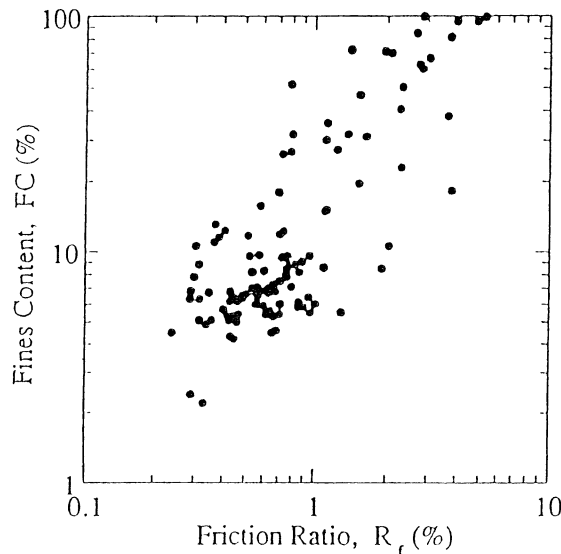


Figure 21 Variation of fines content with CPT friction ratio (after Suzuki et al., 1995).

directly from CPT results using the soil behaviour type chart shown in Figure 22. The boundaries between soil behaviour type zones 2 to 7 can be approximated as concentric circles about a common point. The radius of each circle can then be used as a soil behaviour type index. The soil behaviour type index,  $I_c$ , can be defined as follows:

$$I_c = [(3.47 - \log Q)^2 + (\log F + 1.22)^2]^{0.5} \quad (8)$$

where:

- $Q$  = normalized penetration resistance, dimensionless  
 $= (q_c - \sigma_{vo}) / \sigma_{vo}'$
- $F$  = normalized friction ratio, in percent.  
 $= [f_s / (q_c - \sigma_{vo})] \times 100\%$
- $f_s$  = CPT sleeve friction stress
- $\sigma_{vo}$  = total overburden stress

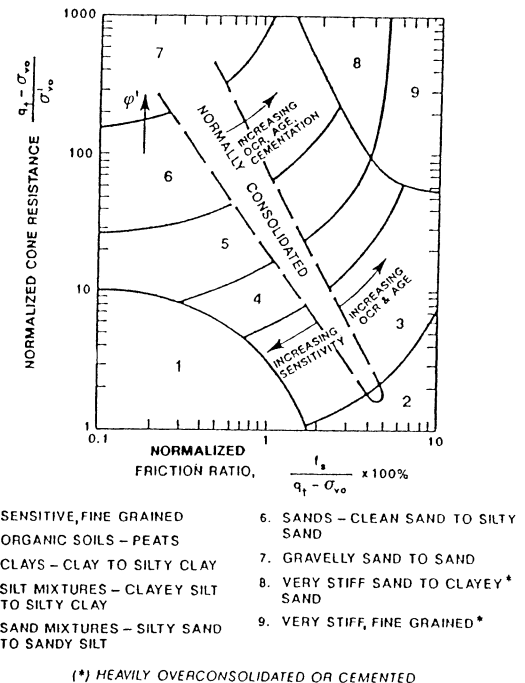


Figure 22 Normalized CPT soil behaviour type chart proposed by Robertson (1990).

The boundaries of soil behaviour type are then 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.

Table 1: Boundaries of soil behaviour type

| Soil Behaviour Type Index, $I_c$ | Zone | Soil Behaviour Type (see Figure 22)      |
|----------------------------------|------|------------------------------------------|
| $I_c < 1.31$                     | 7    | Gravelly 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                                    |
| $I_c > 3.60$                     | 2    | Organic soils: peats                     |

The soil behaviour type index can then be combined with the relationship between fines content and friction ratio suggested in Figure 21 to give the following simplified relation:

$$\text{Fines Content, FC (\%)} = 1.375 I_c^3 - 3.5 \quad (9)$$

The range of potential correlations is illustrated in Figure 23, which shows the variation of soil behaviour type index ( $I_c$ ) with fines content. The suggested relationship given in Equation 9 is shown. A comparison between measured (based on SPT samples) and predicted fines content for three locations at the Phase III CANLEX site is shown in Figure 24. In general, the predicted fines content is

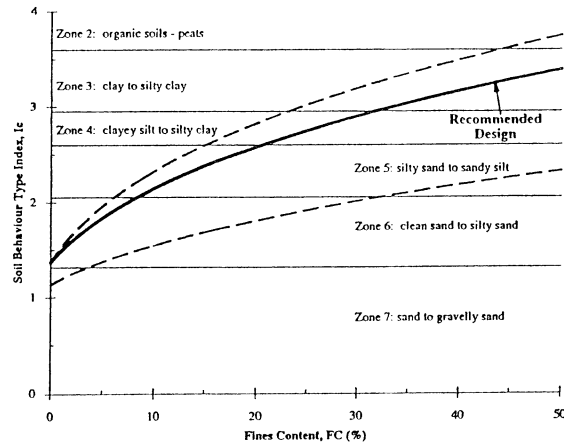


Figure 23 Variation of soil behaviour type index ( $I_c$ ) with fines content.

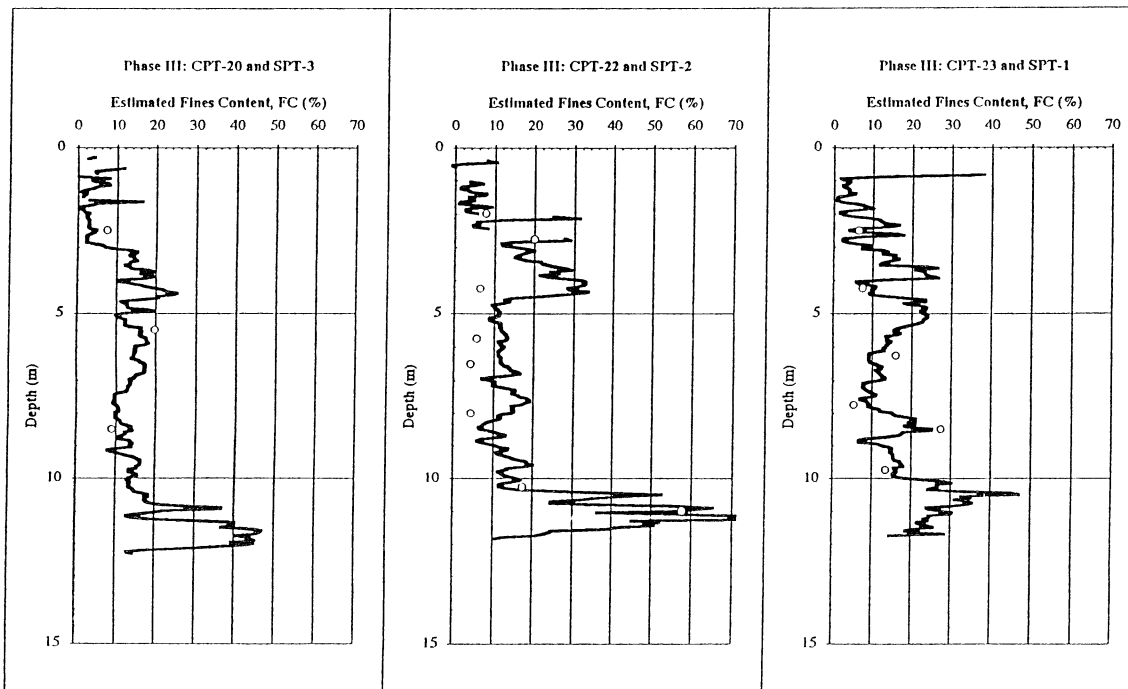


Figure 24 Comparison between measured fines content and that predicted using the CPT for three profiles at CANLEX Phase III site.

close to the measured values, although there is some scatter which may be due, in part, to the averaging effect of the sampling as well as the approximate nature of the correlation. It is interesting to note the rapid variation of fines content with depth at this particular site and how well the CPT appears to track this variation.

The proposed correlation between CPT soil behaviour index ( $I_c$ ) and fines content is approximate, since the CPT responds to many factors affecting soil behaviour. Hence, when possible, it is recommended that the correlation shown in Figure 23 and given by Equation 9 be evaluated and modified to suit a specific site and project. However, for small projects, the above correlation provides a useful guide. Caution must be taken in applying the relationship in Equation 9 to sands that fall close to or in zone 1 in Figure 22 so as not to confuse very loose clean sands with denser sands containing fines.

As already outlined, if seismic CPT are performed, it is possible to develop a site specific correlation between shear wave velocity and cone resistance, so that the soil state can be estimated directly from cone resistance, increasing the level of detail and continuity of the data. Based on limited data, Figure 25 shows an approximate correlation between the Y parameter and soil behaviour index ( $I_c$ ), with the resulting relationship:

$$Y = 72.5 (I_c - 1.5) + 50 \quad (10)$$

where:

$$1.8 < I_c < 2.5$$

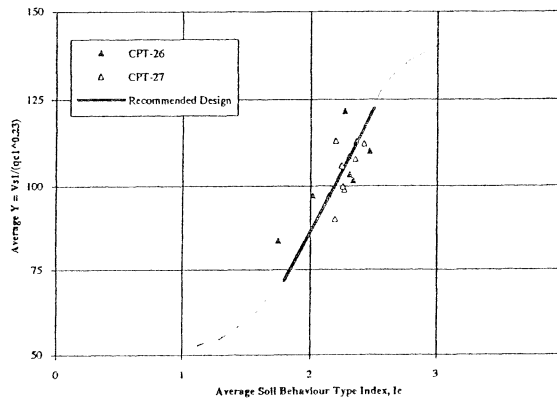


Figure 25 Link between  $V_{s1}$  and  $q_{c1}$  based on soil behaviour type index ( $I_c$ ) at CANLEX Phase III site.

The other problem associated with the interpretation of penetration tests 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 2 to 3 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. Care should therefore 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) has provided some insight as to how to correct cone data in thin layers. Vreugdenhil et al. (1994) has shown that the error in the measured cone resistance within thin stiff layers 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. 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. Based on this work, a correction factor for cone resistance ( $K_c$ ) as a function of layer thickness is shown in Figure 26. These corrections appear to have a reasonable trend, but are rather large. Therefore, a more conservative correction is recommended (corresponding to  $q_{c2}/q_{c1} = 2$ ) as is shown in Figure 26 and given by the following expression:

$$K_c = 0.5 \left( \frac{H}{1000} - 1.45 \right)^2 + 1.0 \quad (11)$$

where:

- $q_{c2}/q_{c1} = 2$
- $H$  = layer thickness, in mm.
- $q_{c2}$  = tip resistance in the layer
- $q_{c1}$  = tip resistance in the soil surrounding the layer.

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.

The evaluation of the in-situ potential for flow liquefaction can be carried out in a variety of ways depending on project requirements. For large projects for which the consequences of flow liquefaction can result in major financial decisions, the recommended method is to obtain high quality undisturbed samples and perform appropriate

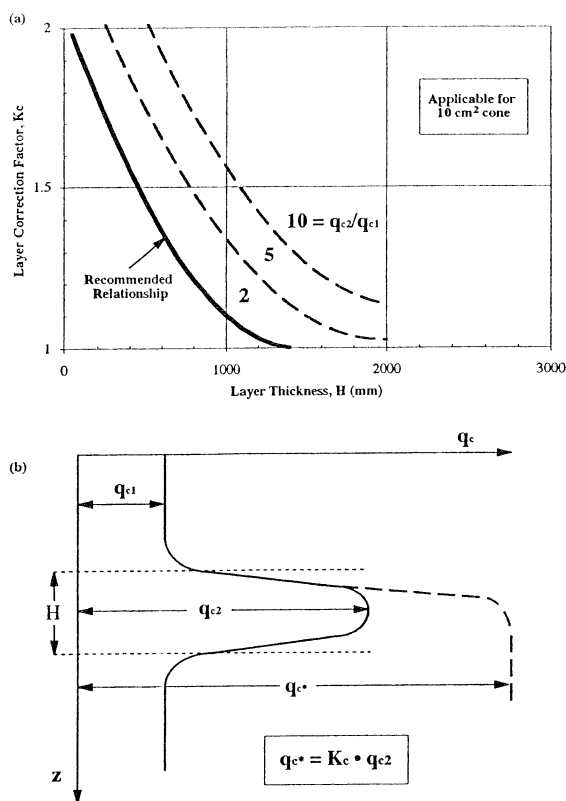


Figure 26 Suggested correction ( $K_c$ ) to CPT penetration resistance in thin sand layers (based on results by Vreugdenhil et al., 1994).

laboratory testing to evaluate the response of the soil. In-situ ground freezing is seen as the most promising technique for obtaining undisturbed samples. Large diameter sampling can also provide useful results in fine sands (Konrad et al, 1995). Undisturbed sampling and laboratory testing is expensive and will only find an application for large projects.

For somewhat smaller projects, high quality geophysical logging (Plewes et al, 1990; Küpper et al, 1995) can provide excellent continuous profiles of in-situ density and, hence, void ratio. Geophysical logging must be carried out in high quality mud rotary drill holes with smooth, uniform borehole wall geometry. Corrections are also required for site specific soil mineralogy (Plewes et al, 1990). The profiles of void ratio can then be compared to the USL obtained from testing of reconstituted samples to see if the in-situ state is loose or dense of the USL. The quality of the laboratory testing on reconstituted samples can be further improved by consolidating samples under the in-situ anisotropic effective stresses to the in-situ

void ratio plus matching the in-situ shear wave velocity values, as suggested by Tokimatsu and Hosaka, (1986). The direction of loading (e.g. compression, extension or simple shear) should also be appropriate to the project and ground geometry.

The above approaches based on laboratory testing (on either undisturbed or reconstituted samples) still require a link with a more general site characterization to allow for adequate extrapolation of the often limited laboratory results to the complete ground profile. A suggested link was illustrated in Figure 14 using Reference Stress Ratio (RSR) obtained from CPT, seismic CPT, or geophysical logging.

For small projects, or in the preliminary stages of large projects, the response of sandy soils can be estimated from RSR estimated directly from the CPT or seismic CPT. If the soil profile is interbedded, containing several thin sand layers embedded in softer deposits, the cone resistance within the sand layer should be corrected using Figure 26 (Equation 11). Then, using the soil behaviour type CPT classification chart (Figure 22) and the soil behaviour index ( $I_c$ ), the fines content should then be estimated (Figure 23) and a correction made to produce an in-situ equivalent clean sand CPT profile. Based on either reconstituted samples or average parameters, the equivalent clean sand CPT profile for  $\psi = 0$  (RSR = 1) should be compared with the in-situ equivalent clean sand CPT profile. This methodology is approximate and global in nature and can be improved by using seismic CPT results to develop a more site specific relationship. The response data shown in Figure 15 can also be used to guide in the link between estimated RSR and soil response.

As part of the CANLEX project (Phase III), a liquefaction event was planned and constructed. Figure 27 shows both a plan and a cross-section of the event configuration. An old borrow pit (J-pit) at Syncrude in Ft. McMurray, Alberta, was filled with tailings to create a relatively loose sand deposit with a water table at a depth of approximately 0.5 m below the ground surface. Characterization of the site indicated that the sand below the water table (BBW) had an average SPT ( $N_1$ )<sub>60</sub> of approximately 3 and an average CPT  $q_{c1}$  of approximately 2 MPa. The beach above the water table (BAW) was denser. A 8 m high clay embankment with side slopes ranging from 2:1 to 2.5:1 was constructed over the loose sand, with the resulting geometry illustrated in Figure 27. A compacted sand berm was then constructed behind, but connected to, the clay embankment, as shown in Figure 27. Together, the sand berm and the clay embankment formed a dike within which water and tailings could be impounded.

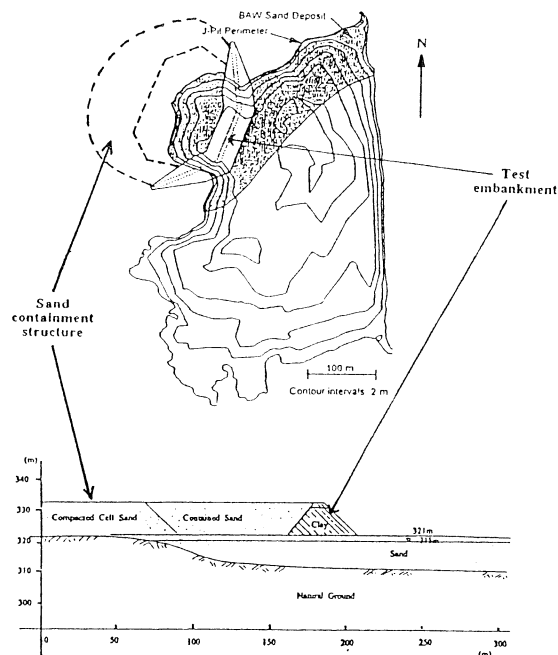


Figure 27 Plan and cross-section of CANLEX Phase III liquefaction event.

Limited CPT characterization of the sand beneath the embankment indicated that increases in  $q_{c1}$  in the sand due to construction of the embankment were small.

The event was intended to generate flow liquefaction of the loose sand due to monotonic loading by rapidly filling the impounded area with sand tailings. Tailings were pumped into the impounded area on a continuous basis, with an outlet controlling the maximum water level. Within 12 hours, the pool was full of water and within 24 additional hours, the pool was full of sand. Instrumentation, consisting of inclinometers and survey targets, indicated that movements were very small during the entire filling. Pore pressures increased at various locations beneath the embankment, as a result of the steady state seepage. The pore pressures then dissipated with time, once the filling was complete. A flow liquefaction failure was not generated, even though the initial average SPT  $(N_1)_{60}$  in the sand was approximately 3. Detailed analyses of the in-situ site characterization, the laboratory testing of undisturbed frozen samples of the sand, and the results of the event will be reported by CANLEX in the near future. At the present time, the possibility of dynamically triggering a liquefaction event using blasting is being considered.

The above comments have shown that for soils to experience flow liquefaction, a complex set of conditions is required, as follows:

1. The sediments must be very loose ( $RSR > 1$ ).
2. The loading must be such that a sufficient quantity of the sediments are strain softening in undrained shear.
3. A trigger mechanism must exist to promote an undrained response.
4. The static shear stresses must be sufficiently high (i.e. greater than the ultimate or minimum undrained strength) for instability to result.
5. The geometry of the ground must be such that a kinimatically admissible failure mechanism can form.

These unique set of conditions are probably not very common and, hence, flow liquefaction failures are also not common. However, flow liquefaction failures that have been documented appear to occur very suddenly with little or no warning and the resulting slide is often catastrophic in nature. The above conditions apply equally to flow slides in very sensitive (quick) clays (Tavenas and Leroueil, 1990) and metastable loess deposits (Ishihara, 1993).

### 3.3 Minimum undrained shear strength

If flow liquefaction is possible, it is important to estimate the minimum undrained shear strength ( $S_{min}$ ) of the sand. Seed and Harder (1990) suggested an empirical correlation (later modified by Stark and Mesri, 1992) between SPT  $(N_1)_{60}$  and  $S_{min}$  based on limited field performance data. However, the field performance data have considerable uncertainty associated with them due to the complex and difficult back-analyses for estimating  $S_{min}$  especially when the inherent anisotropy of sands is considered.

As an alternative method, Fear and Robertson (1995) suggested a framework to estimate  $S_{min}$  based on shear wave velocity. Figure 28 shows the variation of minimum or ultimate undrained strengths for Fraser River sand based on normalized shear wave velocity ( $V_{s1}$ ) and normalized cone resistance ( $q_{c1}$ ). The correlations shown in Figure 28 illustrate the highly non-linear nature of the relationship between a measure of in-situ state (i.e.  $V_{s1}$  or  $q_{c1}$ ) and the shear strength at ultimate or quasi-steady state in triaxial compression or extension.

Evaluation of the stability of a soil structure with respect to flow liquefaction is dependent on the geometry of the potential failure surface since different contributions will be made by the inherent anisotropy of the soil. A similar problem exists for stability calculations in clay soils.

## 4. CYCLIC SOFTENING

The most common form of soil liquefaction

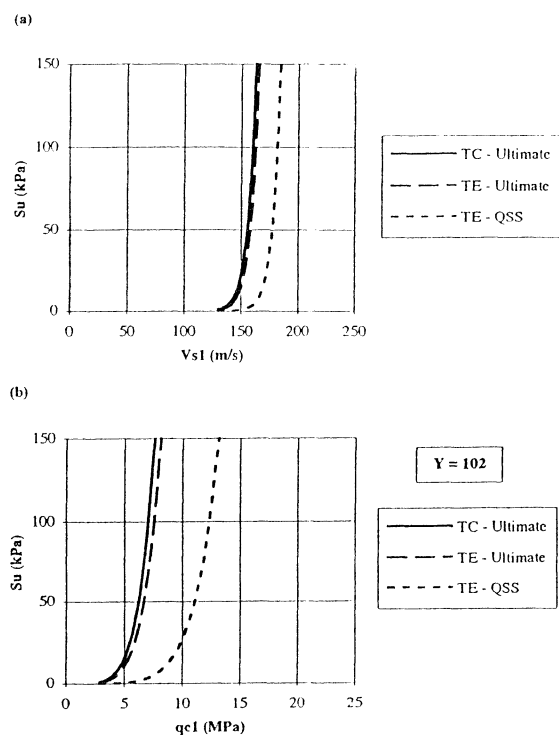


Figure 28 Variation of minimum or ultimate undrained shear strengths in triaxial compression and extension for Fraser River sand based on (a) normalized shear wave velocity values and (b) normalized cone resistance (after Fear and Robertson, 1995).

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.

#### 4.1 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 or cyclic resistance ratio (CRR). For cyclic simple shear tests, CRR is taken as the ratio of the cyclic shear stress to the initial vertical effective stress; i.e.  $(CRR)_{ss} = \tau_{cyc} / \sigma_{vo}'$ . For cyclic triaxial tests, CRR is taken as the ratio of the maximum cyclic shear stress 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 7.5.

It is common practice to define the point of 'liquefaction' in a cyclic laboratory test 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 to 4% double-amplitude shear strain in a cyclic simple shear test. For loose sand samples subjected to shear stress reversal, this often occurs close to the point at which the effective confining stress is essentially zero and deformations develop rapidly; hence, the definition is the same as that for cyclic liquefaction (see Figure 3). However, for denser sand samples, the 5% double-amplitude strain criteria can occur well before sufficient pore pressure has developed to take the sample to the state of essentially zero effective stress. Hence, the criteria for liquefaction typically applied to laboratory results may well be unduly conservative, since deformations may actually be progressing rather slowly.

While 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 fabric similar to that in-situ. Unfortunately, it is very difficult to determine the in-situ fabric of natural sands. As a result, there is often some uncertainty in the evaluation of CRR based on laboratory testing of reconstituted samples. As mentioned above, Tokimatsu and Hosaka (1986) suggested that 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.

Based on the above observations, 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 a clean sand from Niigata, as shown in Figure 29. Yoshimi et al. (1994) also showed that sand samples obtained using conventional high quality fixed piston samplers produced different CRR values than undisturbed in-situ frozen samples, as shown in Figure 30. 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 to the in-situ frozen samples. The difference in CRR became more pronounced as the density of the sand increased.

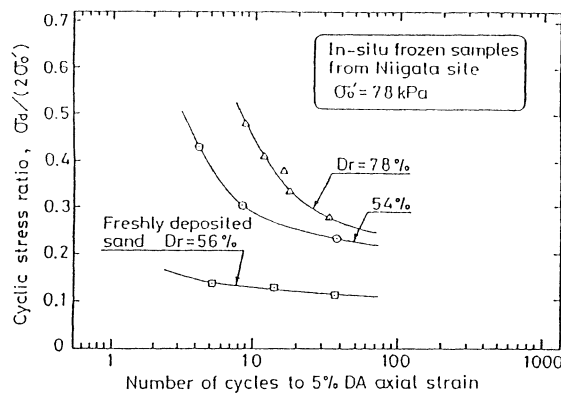


Figure 29 Comparison between triaxial cyclic resistance ratio for a clean Niigata sand based on recently deposited and aged samples (after Yoshimi et al., 1989).

The relationship, shown in Figure 30, (Yoshimi et al., 1994) between  $(CRR)_{tx}$  to cause 5% double-amplitude axial strain after 15 cycles and normalized SPT N value at 100 kPa effective overburden stress and 60% energy  $(N_1)_{60}$  obtained from adjacent soundings was based on undisturbed samples of sand obtained using ground freezing. It would appear that dense sand with a normalized SPT  $(N_1)_{60}$  between 30 and 40 has a  $(CRR)_{tx}$  less than 1.0. This is in conflict with field observation (Seed et al., 1985) and is almost certainly associated with the definition of 'liquefaction' based on a limiting double-amplitude axial strain of 5%. As explained earlier, dense sand samples can progressively develop 5% double-amplitude axial strain but may not have achieved the condition of rapid deformation associated with essentially zero effective confining stress. Hence, it is important to clearly define the onset of 'liquefaction'. In general, for design purposes, cyclic liquefaction is the point at which the soil experiences large uncontrolled deformations.

Although the results shown in Figure 30 apply to

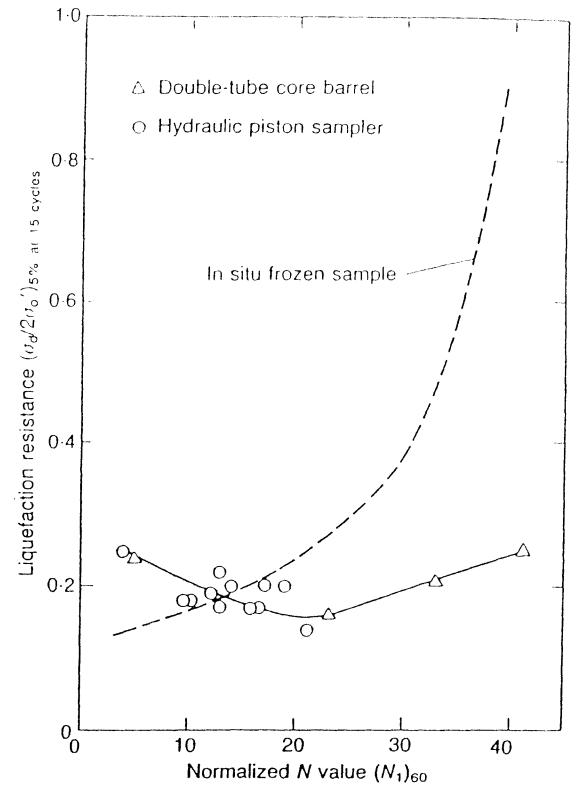


Figure 30 Comparison between triaxial cyclic resistance ratio and SPT  $(N_1)_{60}$  values for clean sands based on tube samples and undisturbed in-situ frozen samples (after Yoshimi et al., 1994).

a range of sands from Japan, it is likely that changes in grain characteristics will influence the correlation between CRR and SPT  $(N_1)_{60}$ . Based on the same laboratory test results on undisturbed in-situ frozen sand samples as those shown in Figure 30 plus one additional site, Suzuki et al. (1995) suggested a correlation between CRR and normalized Cone Penetration Test (CPT) penetration resistance. To account for the variation due to differences in grain characteristics, Suzuki et al. (1995) suggested a modification to the CPT normalization to incorporate the minimum void ratio ( $e_{min}$ ), as a measure of the grain characteristics, as follows:

$$q_{tN} = \frac{q_{c1}}{f(e_{min})} = \frac{q_c \left( \frac{P_a}{\sigma'_{vo}} \right)^{0.5}}{f(e_{min})} \quad (12)$$

where:

$q_{tN}$  = modified normalized cone penetration resistance

$q_{c1}$  = normalized cone penetration resistance

$P_a$  = atmospheric pressure, usually = 100 kPa

$\sigma'_{vo}$  = vertical effective stress.

$f(e_{min}) = (2.17 - e_{min})^2 / (1 + e_{min})$ .

The resulting correlation is shown in Figure 31. Suzuki et al. (1995) suggested that the correlation in Figure 31 would be applicable to a wide range of sandy soils (i.e. sandy soils with various grain distributions, grain shapes and/or mineralogy). It is interesting to note that the modification using  $f(e_{min})$  accounts for a correction to the traditional normalized cone penetration resistance by a factor of 0.65 to 0.96 when  $e_{min}$  varies from typical values of 0.6 to 0.8. However, the incorporation of  $e_{min}$  into the correlation is cumbersome and difficult to apply and appears to have a small influence for most sands.

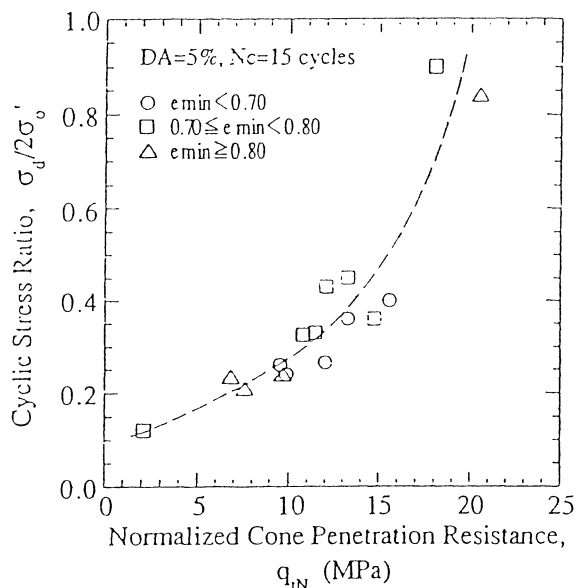


Figure 31 Correlation between triaxial cyclic resistance ratio and modified normalized cone penetration resistance for a wide range of sands (after Suzuki et al., 1995).

When a soil is fine-grained or contains some amount of 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 the most important index property influencing CRR is the plasticity index of the fines contained in the sand (Ishihara and Koseki, 1989). Figure 32 shows the results of cyclic triaxial tests versus plasticity index ( $I_p$ ) for a variety of sandy soils (Ishihara, 1993) and illustrates that the  $(CRR)_{tx}$  changes little below  $I_p = 10$ , but begins to increase thereafter with increasing plasticity index.

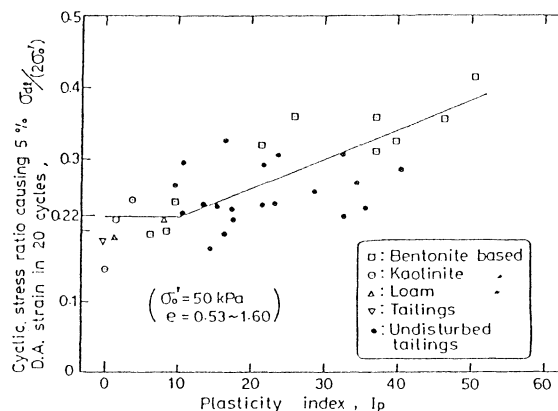


Figure 32 Effect of soil plasticity on cyclic resistance ratio of fines-containing sands (after Ishihara, 1993).

#### 4.2 Cyclic resistance based on field testing

The above comments have shown that the most reliable method for estimating the CRR of sandy soils is testing high quality undisturbed samples under the appropriate cyclic loading. However, sampling saturated sandy soils is expensive and can only be carried out for large projects for which the consequences of liquefaction may result in large financial decisions. Therefore, there will always be a need for simple, economic procedures for estimating the CRR of sandy soils. Currently, the most popular simple method for estimating CRR makes use of the penetration resistance from the Standard Penetration Test (SPT) although, more recently, the Cone Penetration Test (CPT) has become very popular due to its greater reliability and repeatability and the continuous nature of its profile.

Seed (1979) developed a method to estimate the CRR for a sand under level ground conditions based on the SPT. This method was based on extensive field performance data from essentially level ground sites which either had or had not experienced cyclic softening (liquefaction) due to earthquake loading. Liquefaction was assumed to have occurred based on the presence of observable surface features such as sand boils and ground cracks. A summary of the SPT based method to estimate CRR for clean sand is shown in Figure 33 (Seed et al., 1985). Other SPT based methods have been developed (Ishihara, 1993; Fear and McRoberts, 1995), but the correlation by Seed et al. (1985) appears to maintain the most popularity, especially in North America. The correlation shown in Figure 33 includes the estimated limiting shear strain values associated with 'liquefaction'. Seed recognized that dense sands ( $(N_1)_{60} > 20$ ) generally experienced less

deformation for a given cyclic loading (i.e. they experienced cyclic mobility) than loose sands (which experienced cyclic liquefaction). Hence, the definition of 'liquefaction' became flexible in that dense sand would not develop very large strains (i.e. would not reach the condition of essentially zero effective confining stress). This is supported by laboratory test results (Yoshimi et al., 1994).

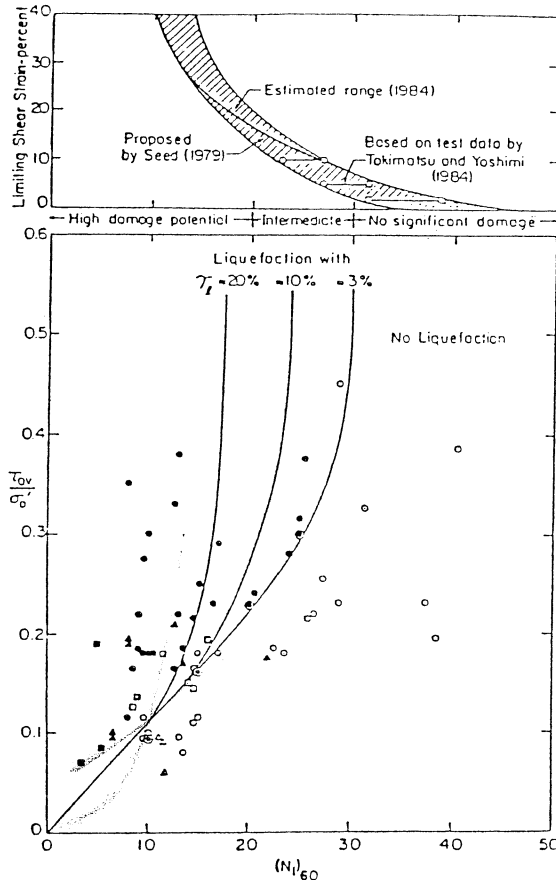


Figure 33 Cyclic resistance ratio for clean sands under level ground conditions based on SPT and field performance data (after Seed et al., 1985).

Seed et al. (1985) developed the correlation further to include the influence of fines content, as shown on Figure 34. The correlation showed that, for the same CRR, the penetration resistance in silty sands was smaller. This is most likely due to the greater compressibility and decreased permeability of silty sands which reduces penetration resistance and moves the penetration process toward an undrained penetration, respectively.

The late Professor H.B. Seed and his co-workers developed a comprehensive methodology to estimate the potential for cyclic softening due to earthquake

loading. The methodology 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 et al. (1985) based on the maximum ground surface acceleration ( $a_{max}$ ) at the site. This simplified approach can be summarized as follows (after Tokimatsu and Yoshimi, 1983):

$$CSR = \frac{\tau_{av}}{\sigma_{v0}'} = 0.1 (M - 1) \left( \frac{a_{max}}{g} \right) \left( \frac{\sigma_{v0}}{\sigma_{v0}'} \right) (1 - 0.015z) \quad (13)$$

where:

- $\tau_{av}$  = average cyclic shear stress
- $M$  = earthquake magnitude, commonly  $M = 7.5$
- $a_{max}$  = maximum horizontal acceleration at ground surface
- $g$  = acceleration due to gravity =  $9.81 \text{ m/s}^2$
- $\sigma_{v0}$  = total vertical overburden stress
- $\sigma_{v0}'$  = effective vertical overburden stress
- $z$  = depth in meters (for  $z < 25\text{m}$ ).

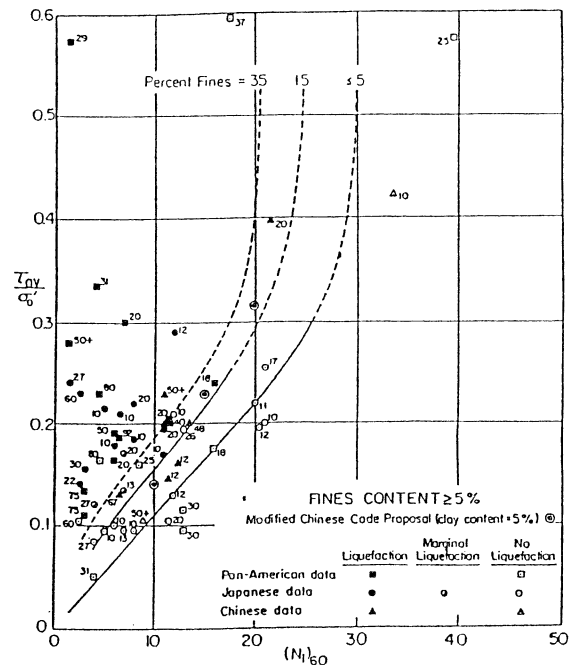


Figure 34 Relationship between cyclic resistance ratio and SPT for sands and silty sands based on field performance data (after Seed et al., 1985).

The CSR profile from the earthquake can then be compared to the estimated CRR profile for the soil deposit. At any depth, if CSR is greater than CRR, cyclic softening (liquefaction) is possible. This methodology is the most commonly used technique in most parts of the world to estimate soil liquefaction due to earthquake loading.

The methodology based on the SPT has many problems, primarily due to the unreliable nature of the SPT. The main factors affecting the SPT test have been reviewed (e.g. Skempton, 1986; Robertson et al., 1983) and correction factors to be applied to measured N values have been identified. The single most important factor affecting SPT results is the energy delivered to the SPT sampler. This is normally expressed in terms of the rod energy ratio (ER). An energy ratio of 60% has generally been accepted as the reference value which represents the approximate historical average SPT energy. The value of ER (%) delivered by a particular SPT set-up depends primarily on the type of hammer/anvil system and the method of hammer release. Values of the correction factor to modify the SPT results to 60% energy (ER/60) can vary from 0.3 to 1.6 corresponding to field values of ER of 20% to 100%. Additional correction factors are also required for rod lengths less than 10 m, borehole diameters outside the recommended interval (65 - 115 mm) and samplers without internal liners. Since the SPT N value also varies with the effective overburden stress level, an overburden stress correction factor is often also applied to provide a consistent reference (i.e.  $(N_1)_{60}$ ). A value of ER is often assumed based on published global average values (Seed et al., 1985). However, the actual value of ER can vary significantly from the global averages. It is recommended that ER be measured during the actual site investigation to improve the level of reliability of the SPT.

Due to the inherent difficulties and poor repeatability associated with the SPT, Robertson and Campanella (1985) proposed a modified chart to estimate the CRR for clean sands and silty sands using normalized CPT penetration resistance ( $q_{c1}$ ). This chart was developed from the Seed SPT chart using SPT-CPT conversions. 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 the main different CPT charts is shown in Figure 35.

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). The recent field performance data have shown that the existing CPT-based correlations to estimate CRR are generally good for both clean sands and silty sands.

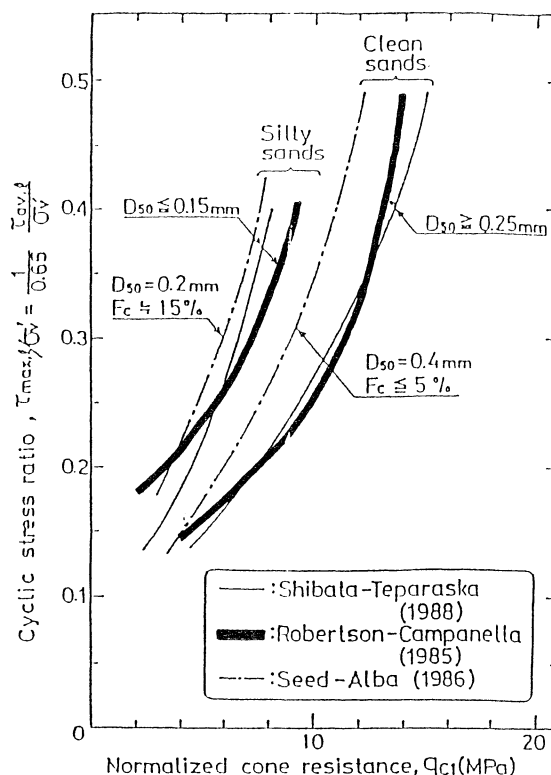


Figure 35 Comparison between various CPT based charts for estimating cyclic resistance ratio for clean sands.

The recent comprehensive databases put together by Stark and Olson (1995) and Suzuki et al. (1995) show that the correlation between CRR and  $q_{c1}$  by Robertson and Campanella (1985) for clean sands provides a good estimate of CRR, as shown in Figure 36. Based on data from 180 sites, Stark and Olson (1995) also developed a set of correlations between CRR and  $q_{c1}$  for various sandy soils based on fines content and mean grain size, as shown in Figure 37. The combined database is now larger than the original SPT-based database proposed by Seed et al. (1985). It is important to note that the Seed methodology based on either the SPT or extrapolated to 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 (Fear and McRoberts, 1995). A detailed review of the CPT data, similar to that carried out by Fear and McRoberts (1995) on SPT data, would be required to investigate the degree of conservatism contained in Figures 35 to 37. The correlations are also sensitive to the amount and plasticity of the fines within the sand.

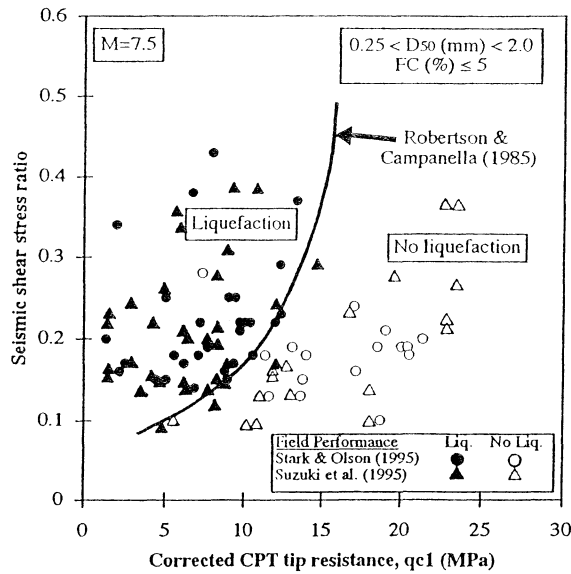


Figure 36 Comparison between CPT based method for clean sands proposed by Robertson and Campanella (1985) and recent field performance data from Stark and Olson (1995) and Suzuki et al. (1995).

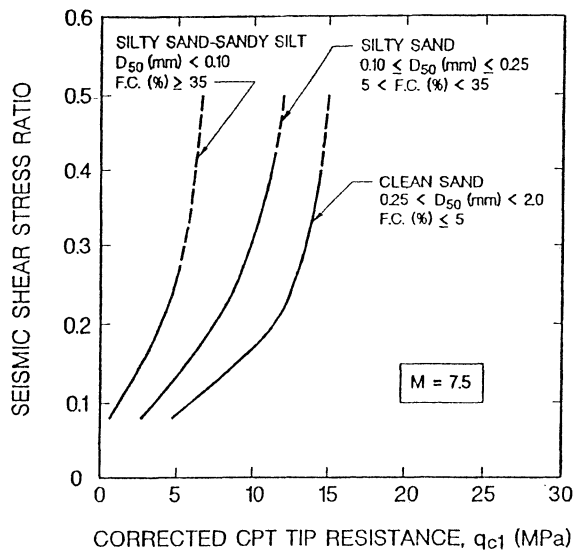


Figure 37 Summary of variation of cyclic resistance ratio with fines content based on CPT field performance data (after Stark and Olson, 1995).

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 unreliability of SPT data. With the increasing interest in the CPT due to its greater

reliability, it is now possible to estimate fines content and grain size from CPT data and incorporate this directly into the evaluation of liquefaction potential. The following is a summary of the suggested approach based on the CPT.

Based on the comprehensive database presented by Stark and Olson (1995), Figure 38 shows the suggested correction ( $\Delta q_{c1}$ ) to the measured CPT penetration resistance required to obtain an equivalent clean sand normalized penetration resistance,  $(q_{c1})_{cs}$ , based on fines content and grain size. The correction is not constant for all values of penetration resistance, but a reasonable average can be defined by the following:

$$\begin{aligned} \Delta q_{c1} &= 5 \text{ MPa} & \text{if } FC \geq 35\% \\ \Delta q_{c1} &= 0 & \text{if } FC \leq 5\% \\ \Delta q_{c1} &= \frac{FC - 5}{6} \text{ MPa} & \text{if } 5\% < FC < 35\% \end{aligned} \quad (14)$$

where: FC is in percent.

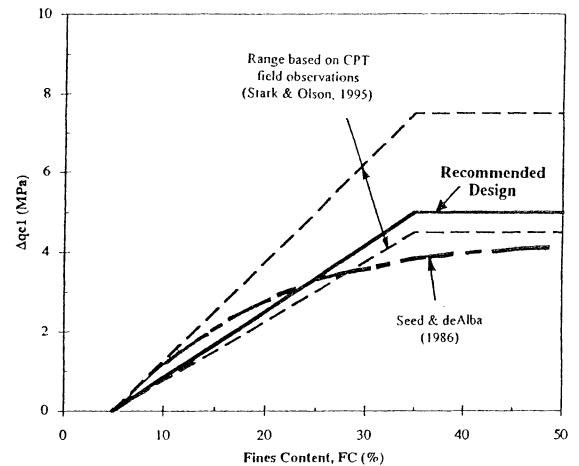


Figure 38 Suggested correction to normalized cone resistance for fines content based on field performance data.

The range of correction values are compared to those suggested by Seed and de Alba (1986). The more recent CPT field observation data suggest a slightly higher correction at high fines content.

In an earlier section, a method was suggested to estimate fines content directly from CPT results using Equation 9. Equations 9 and 14 can be combined so that the correction to obtain the equivalent clean sand normalized penetration resistance  $(q_{c1})_{cs}$  can be estimated directly from the measured CPT data. Then, using the equivalent clean sand normalized penetration resistance  $(q_{c1})_{cs}$ , CRR can be estimated from Figure 36 using the following simplified equation for  $M = 7.5$ :

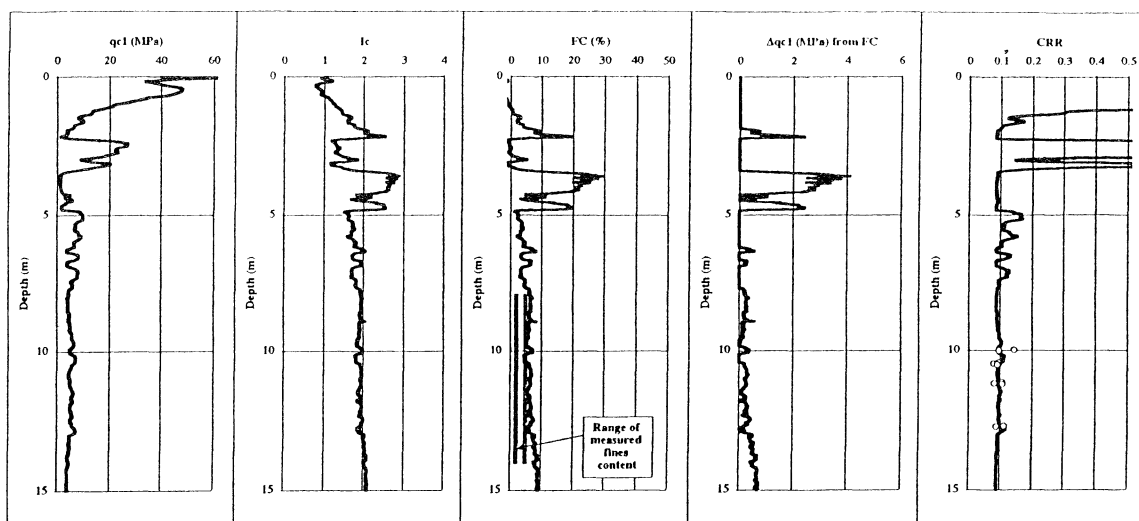


Figure 39 Example of applying the integrated CPT method to estimate cyclic resistance ratio (CRR) and comparison of predicted CRR with results of cyclic simple shear tests on in-situ frozen samples from CANLEX Phase II site (Massey South Site).

$$CRR = 93 \left( \frac{(q_{c1})_{cs}}{100} \right)^3 + 0.08 \quad (15)$$

where:

$(q_{c1})_{cs}$  is in MPa and  $3 \text{ MPa} < (q_{c1})_{cs} < 16 \text{ MPa}$ .

An example of this proposed method is shown in Figure 39 for one of the Phase II sites for the CANLEX project (Massey South). The measured cone resistance is corrected for overburden stress to  $q_{c1}$  and the soil behaviour type index ( $I_c$ ) is calculated. Based on  $I_c$ , the fines content, FC (%), is estimated and the correction  $\Delta q_{c1}$  is calculated. The final continuous profile of CRR at 15 cycles ( $M = 7.5$ ) is then calculated based on the equivalent clean sand values of  $q_{c1}$  and Equation 15. Included in Figure 39 are results from cyclic simple shear tests on undisturbed samples using ground freezing converted to 15 cycle equivalent values of CRR using the conversions proposed by Seed et al. (1985). A good comparison is seen, although, at this site, the fines content is generally less than 10 % and the corrections to CPT tip resistance for fines content are very small.

With the development of the seismic CPT, shear wave velocity ( $V_s$ ) has been proposed as an independent measure of in-situ sand state (Robertson et al., 1995a). Robertson et al. (1992) and Tokimatsu et al. (1991) both suggested similar charts to estimate the CRR of sands from normalized  $V_s$ . The chart by Robertson et al. (1992) is shown in Figure 40. The relationship by Robertson et al. (1992) was based on limited field performance data, whereas the relationship by Tokimatsu et al. (1991)

was based on laboratory results. Recent results from the Loma Prieta earthquake (Kayen et al., 1992; Boulanger et al., 1995) have shown that the shear wave velocity relationship by Robertson et al. (1992) provided good agreement with more recent field performance results. As for the SPT and CPT charts, the shear wave velocity chart may contain conservatism in the way in which representative values of shear wave velocity were selected at various sites.

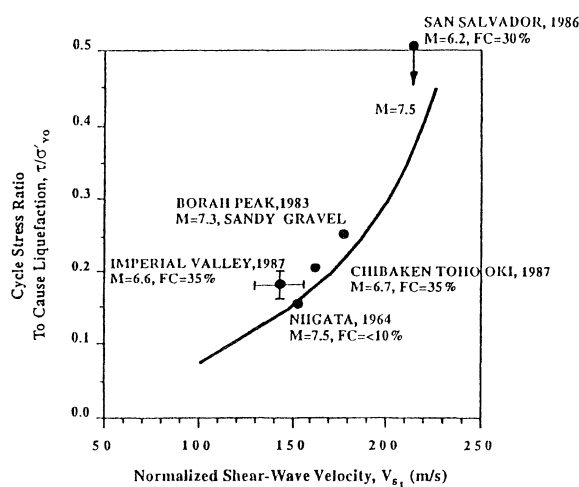


Figure 40 Correlation between normalized shear wave velocity and cyclic resistance ratio based on field performance data (after Robertson et al., 1992)

Correction factors have been developed to account for high overburden stresses and sloping ground (Seed and Harder, 1990). The early correction factors for high overburden stresses ( $K_\sigma$ ) suggested by Seed and Harder (1990) may not be applicable to all sands, as reflected in the results of cyclic simple shear tests carried out on undisturbed samples from Duncan Dam (Byrne et al., 1994) and shown in Figure 41. The correction factors for static shear ( $K_\sigma$ ) are a function of sand state. Very loose sand can have an in-situ state very close to collapse for which any small disturbance could trigger collapse and flow liquefaction. Dense sands, however, will have a very high ultimate undrained strength and the application of static shear will generally improve the resistance to cyclic loading. Pillai (1991) and Pando and Robertson (1995) suggested that the correction factor  $K_\sigma$  should be related to state parameter (or RSR) instead of relative density.

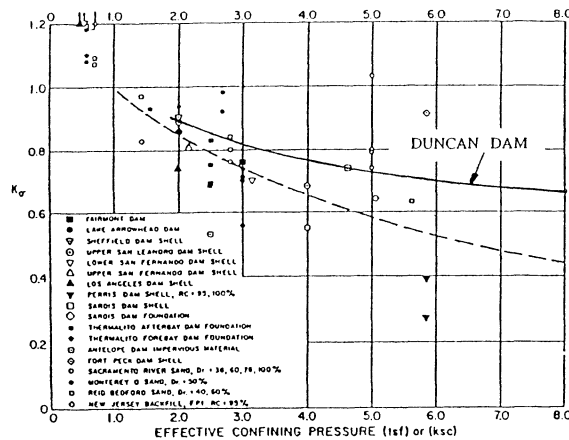


Figure 41 Comparison between overburden stress correction ( $K_\sigma$ ) and effective confining pressure (after Byrne et al., 1994).

For cyclic liquefaction to occur, there must be some level of shear stress reversal so that the effective stresses can progress toward zero. For sloping ground, it is not always clear when shear stress reversal will occur. Pando and Robertson (1995) performed two-dimensional dynamic analyses to estimate for a given  $K_0$  what level of ground acceleration was required to induce shear stress reversal along horizontal planes for different slope geometries. The results of this study showed that the zone of shear stress reversal was controlled by the size of the cyclic loading, the height of the slope and the slope angle. In general, shear stress reversal is likely to occur in most slopes when the slope height is less than about 20 m and the slope angle is less than about 20 degrees. The actual distribution of cyclic shear stresses is a complex

function of site location, ground geometry, earthquake size and duration and soil conditions. However, it appears that shear stress reversal is common for most ground conditions during large earthquake loading.

### 4.3 Cyclically induced deformations

One of the difficulties encountered when evaluating the potential for cyclic liquefaction is to estimate the consequences of liquefaction. Most cyclic liquefaction design problems are controlled by the level of expected deformation. Empirical methods exist for estimating the vertical settlement subsequent to earthquake loading or the lateral displacement during and after an earthquake.

Tokimatsu and Seed (1987) and Ishihara (1993) developed empirical charts to estimate the vertical settlement subsequent to an earthquake. The chart by Ishihara (1993) is shown in Figure 42. Bartlett and Youd (1995) have developed an empirical method for estimating lateral spread deformations in gently sloping ground. The limiting shear strain curves proposed by Seed et al. (1985) and shown in Figure 33 can also be used to estimate the extent of

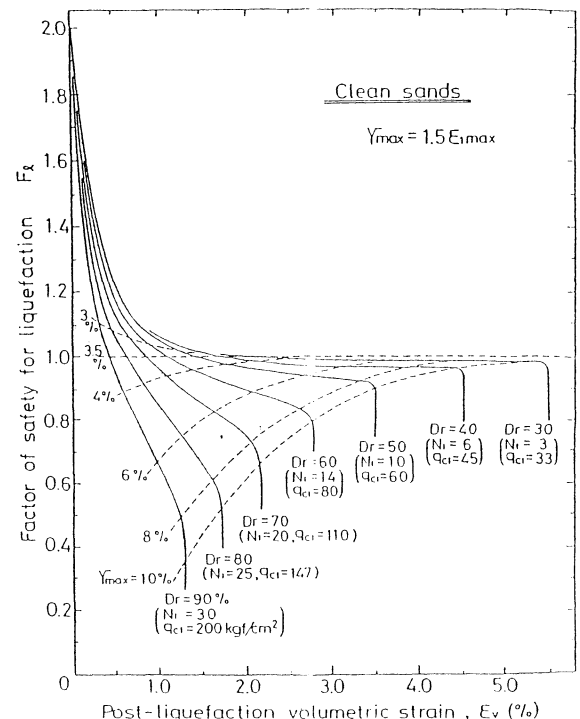


Figure 42 Post cyclic liquefaction volumetric strain curves using CPT or SPT results (after Ishihara, 1993).

ground oscillations during earthquake loading. These are similar to the shear strain values suggested by Ishihara (1993) and shown in Figure 42. One advantage of the CPT methodology for estimating cyclic liquefaction is the continuous nature of the interpretation. Hence, the contribution of each soil layer can be included in estimating the extent of deformation.

It is clear from reports of damage following some of the recent major earthquakes that the extent of damage to structures is a complex function of the interaction of the soil profile, the earthquake loading and the foundation design. Hence, there are valuable lessons for future foundation designs to be learned from observations after major earthquakes. The potential for cyclic liquefaction in a thin layer below an existing or proposed structure may not lead to failure of the structure. Ishihara (1993) provided some useful guidelines for the amount of non-liquefied soil that should exist above a layer of liquefied soil such that no damage would occur at the ground surface.

## 5. EXAMPLE

Recently, the ASCE Group in Seattle, Washington, USA, held a one-day seminar on soil liquefaction. As part of the seminar, a detailed study was carried

out at a typical alluvial soil site near Seattle. Ten local drilling companies donated the services of eleven different drill rigs to drill ten 15 m and two 10 m holes on a small site owned by the Washington State Department of Transportation (WSDOT). The drilling techniques and SPT reflect typical local practice. The site is located on a wide floodplain of the Green River. The site was selected because liquefiable soils were expected in the area and because site access was good and subsurface conditions were expected to be somewhat horizontal. Two CPT profiles were also carried out to a depth of 16 m. Table 2 shows a summary of the testing carried out and the equipment used for the SPT. Figure 43 shows a summary of one of the CPT profiles. The ground conditions consist of about 1.5 m of sand fill overlying alluvial sands and silty sands. The sand layer from 3.6 m to 7.4 m is very loose. Figure 44 shows a summary of the measured field data in terms of the SPT N values, the two CPT  $q_c$  profiles (C-2 and C-4) and the  $V_s$  profile from the seismic CPT (C-4). The SPT shows considerable scatter compared to the CPT. Energy measurements were carried out on most of the SPT equipment and energies varied from 20% to 90% of the theoretical energy. Many of the SPTs were carried out in hollow stem auger (HSA) holes in which significant disturbance at the base of the drill hole was experienced. The conclusion from the study was that the raw SPT results were unreliable. The two

Table 2: Summary of drilling equipment used at Kent Site, Washington, U.S.A.

| Exploration Designation and Equipment Used                                                                                                                    |                                                                                                                                                                    |                                                                                                                                                    |                                                                                                                                                                     |                                                                                                   |                                                                                                                                        |
|---------------------------------------------------------------------------------------------------------------------------------------------------------------|--------------------------------------------------------------------------------------------------------------------------------------------------------------------|----------------------------------------------------------------------------------------------------------------------------------------------------|---------------------------------------------------------------------------------------------------------------------------------------------------------------------|---------------------------------------------------------------------------------------------------|----------------------------------------------------------------------------------------------------------------------------------------|
| <b>A-1</b><br>HSA<br>NW Rods<br>SPT Sampler<br>Downhole Hammer (140 lb) on wireline with manual release<br>CME 75-high Torque<br><br>Logged by RZA-AGRA       | <b>A-2</b><br>HSA<br>BW Rods<br>SPT sampler<br>Safety Hammer (140 lb) on 1.5 dia. polypropylene based rope, cathead<br>B61 equivalent<br><br>Logged by Converse NW | <b>A-3</b><br>HSA<br>AW-J Rods<br>SPT Sampler<br>Automatic Hammer (140 lb / free fall)<br>CME 75-high Torque<br><br>Logged by Golder Assoc.        | <b>A-4</b><br>HSA<br>HWJ Rods<br>D&M Sampler<br>Safety Hammer (300 lb) on 1.0 in Dia. polypropylene-based rope, cathead<br>Mobile B61<br><br>Logged by GeoEngineers | <b>A-5</b><br>Becker / Hammer Drill, driven by Link-belt diesel hammer<br><br>Logged by CH2M Hill |                                                                                                                                        |
| <b>B-1</b><br>HSA<br>BJ Rods<br>SPT Sampler<br>Safety Hammer (140 lb) on rope to steel cable over pulley, cathead<br>Mobile B61<br><br>Logged by GeoEngineers | <b>B-2</b><br>HSA<br>AW-J Rods<br>SPT Sampler<br>Automatic Hammer (140 lb / free fall)<br>Longyear / Brinard-Killman BK-81<br><br>Logged by Golder Assoc.          | <b>B-3</b><br>HSA<br>NW-J Rods<br>SPT Sampler<br>Safety Hammer (140 lb) on wireline with manual release<br>Mobile B59<br><br>Logged by Converse NW | <b>B-4</b><br>HSA<br>SPT Sampler<br>Downhole Hammer (140 lb) on wireline with manual release<br>Mobile B61<br><br>Logged by RZA-AGRA                                | <b>B-5</b><br>Becker / Hammer Drill, driven by Link-Belt diesel hammer<br><br>Logged by CH2M Hill | <b>B-6</b><br>Mud Rotary<br>AW-J Rods<br>HQ Inner Casing<br>Automatic Hammer (140 lb / free fall)<br>CME 45<br><br>Logged by CH2M Hill |
| <b>C-1</b><br>Mud Rotary<br>NW-J Rod<br>SPT Sampler<br>Safety Hammer (140 lb) on wireline with manual release<br>Mobile B-61<br>Logged by Hart Crowder        | <b>C-2</b><br>CPT (electric cone)                                                                                                                                  | <b>C-3</b><br>Pressure meter                                                                                                                       | <b>C-4</b><br>CPT (seismic cone)                                                                                                                                    |                                                                                                   |                                                                                                                                        |

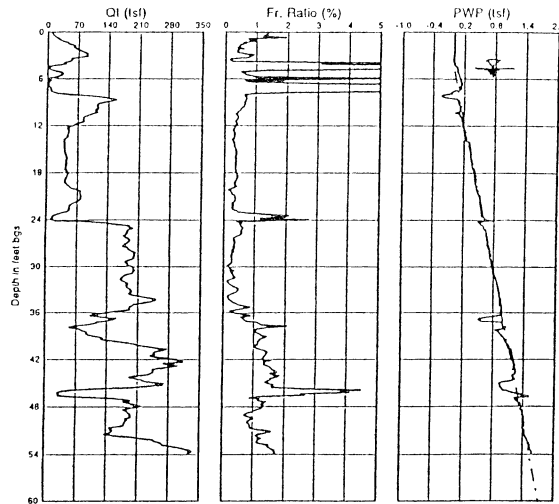


Figure 43 Summary of CPT profile (CPT C-2) at Kent Site, Washington, USA.

CPT profiles were very consistent and clearly identified the various sand and silty sand layers of different density. The shear wave velocity profile was consistent with the CPT profiles.

The energy values for each of the SPT profiles were estimated from CPT C-2 prior to learning the actual values. This was done by converting the CPT profile to an equivalent SPT profile (Robertson et al., 1983) and comparing the results with the actual measured SPT values. The equivalent SPT profile from the CPT was based on 60 % energy and, hence, the energies corresponding to each SPT set-up could be estimated from the comparison. The

estimated SPT energies from the CPT-SPT comparison were generally very close to the actual measured values.

Figure 45 presents a summary of the evaluation of flow liquefaction for all of the field data. This evaluation was based primarily on the shear wave velocity data and estimated values for the USL for the sand deposit. Site specific correlations between  $V_s - q_c - N$  were used to extrapolate to the CPT and SPT profiles. The analysis indicates that flow liquefaction is possible from 3.5 to 7.5 m based on all data. Flow liquefaction is also possible from 9.5 to 11.5 m in the area of CPT C-4. Figure 45 also shows the estimated values of minimum (ultimate) undrained shear strength in triaxial compression based on the method by Fear and Robertson (1995). The estimated undrained shear strength is very low in the layers in which flow liquefaction is possible. Flow liquefaction will occur only if the soil can be triggered to deform in an undrained manner and if a kinematically possible mechanism can form to produce a flow slide. Since the site consists of essentially level ground, a flow slide is highly unlikely. However, if a large structure were constructed on the site and the soil was not densified by the construction, there would be the possibility of a bearing capacity failure if undrained loading were to develop, such as during an earthquake.

Figure 46 presents a summary of the evaluation of cyclic softening for all of the field data using conventional approaches described previously. The analysis was based on a design earthquake that would produce a maximum surface acceleration of 0.2 g from a Magnitude 7.0 earthquake. Cyclic softening is predicted from 3.5 to 7.5 m. This cyclic

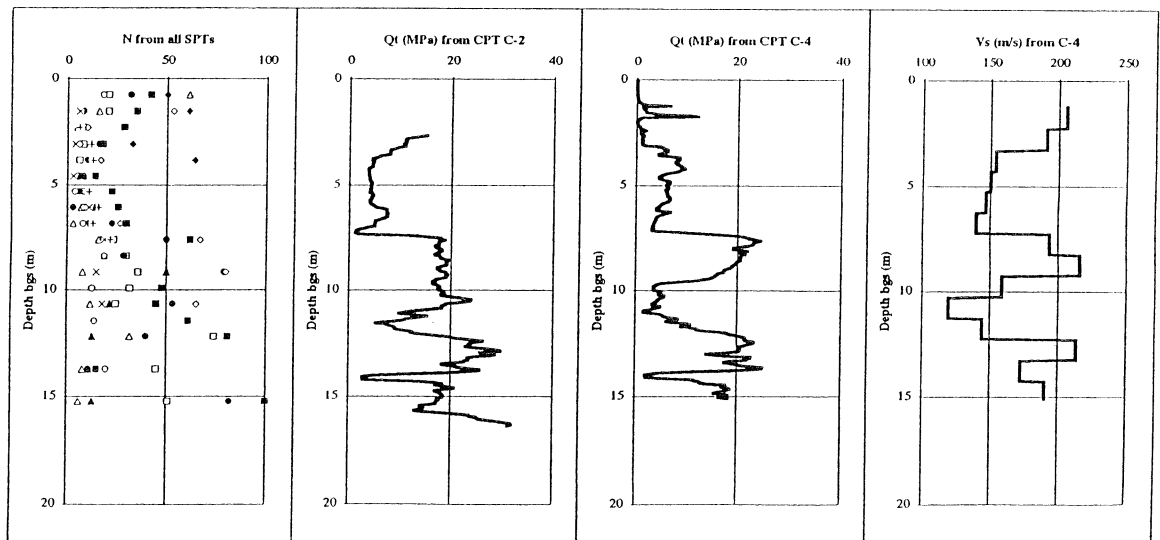


Figure 44 Summary of measured field data in terms of SPT, CPT (C-2 and C-4) and shear wave velocity profiles at Kent Site, Washington, USA..

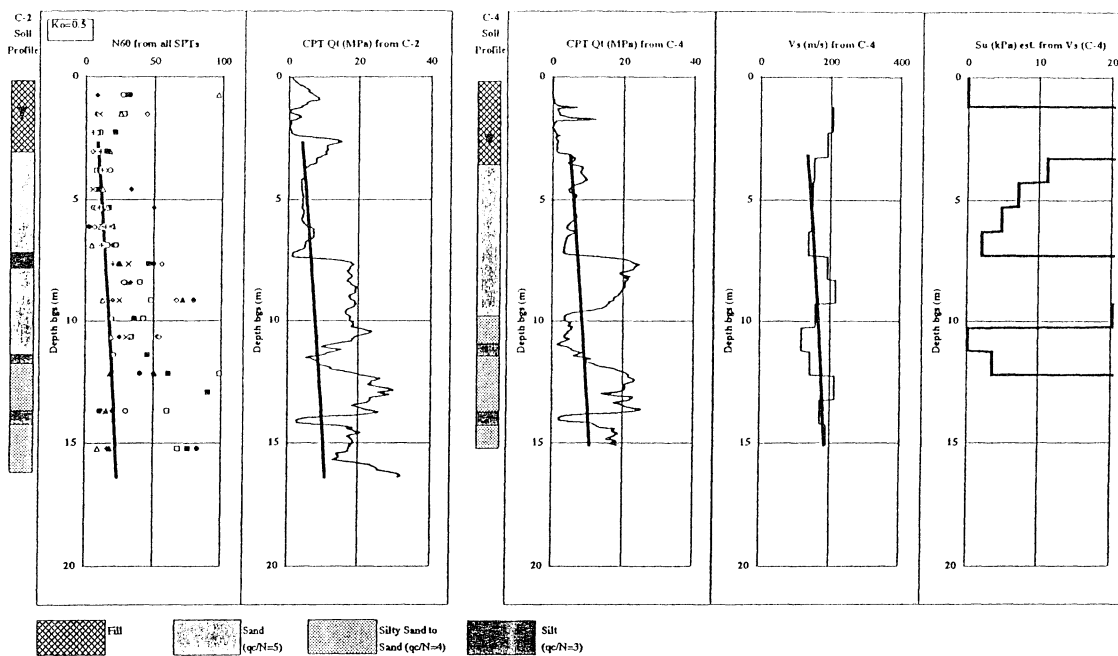
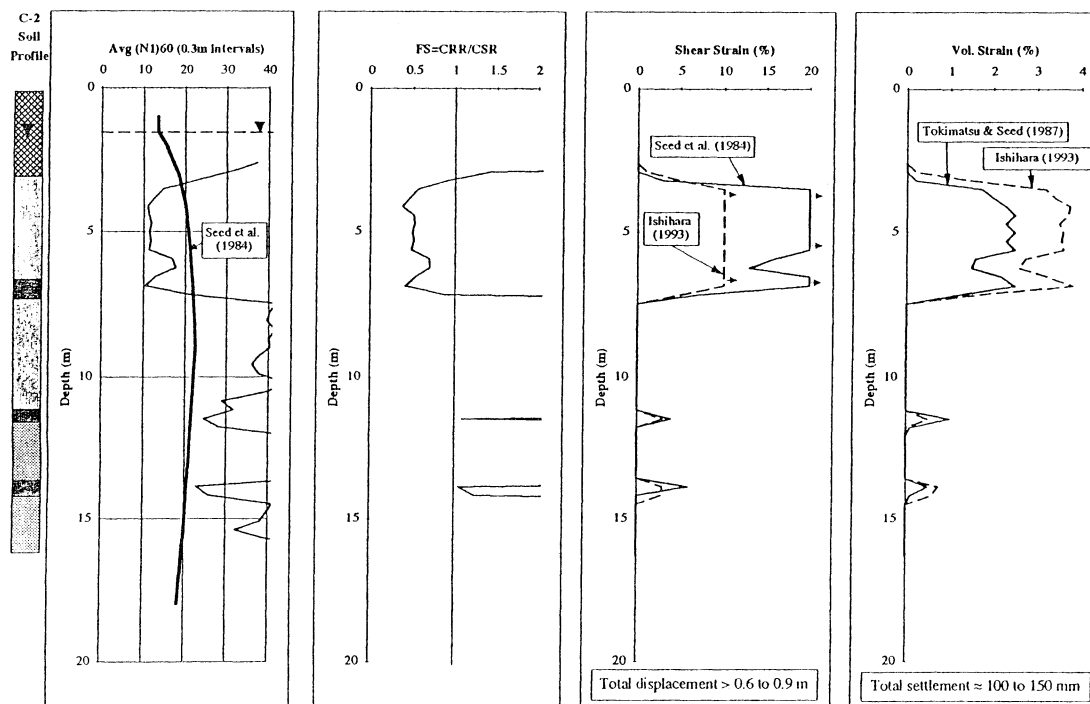


Figure 45 Summary of evaluation of potential for flow liquefaction at Kent Site, Washington, USA.



Note: Deformations in soil profile C-4 would be larger due to thicker second loose layer

Figure 46 Summary of conventional evaluation of potential for cyclic liquefaction and resulting deformations at Kent Site, Washington, USA.

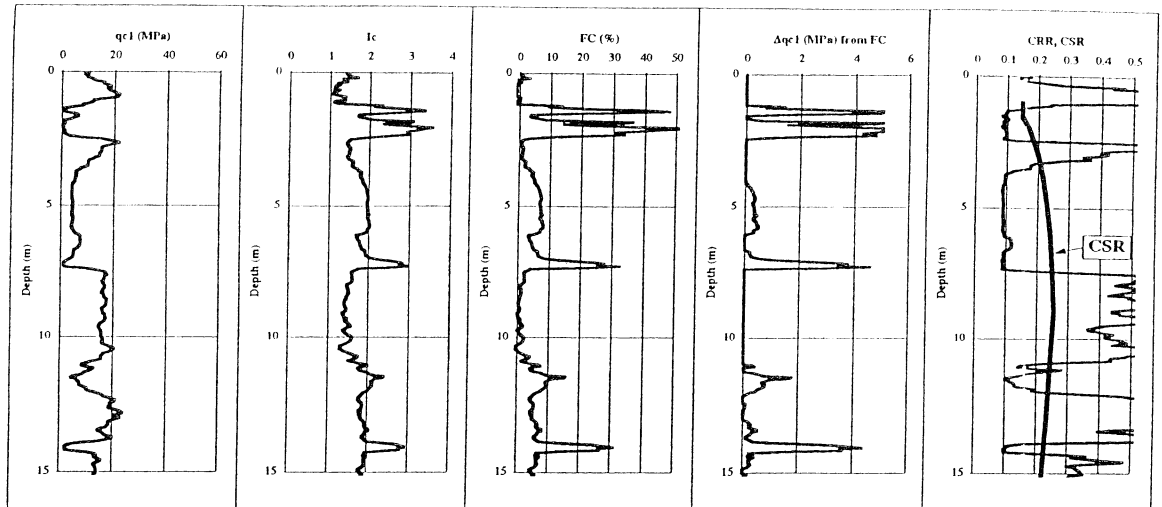


Figure 47 Application of new integrated CPT approach to Kent Site, Washington, USA.

softening could be expected to produce horizontal displacements greater than 0.6 to 0.9 m during the earthquake and at least 100 to 150 mm of vertical settlement after the earthquake. Figure 47 applies the integrated CPT approach, proposed here, to the same field data. The predicted zones of cyclic liquefaction (i.e.  $CRR < CSR$ ) at this site agree well with the results of the conventional approach.

they are applied to unaged, uncemented, predominately silica sands.

An increased amount of field performance data is now available to add confidence to the empirical charts for estimating the cyclic resistance ratio (CRR) of sandy soils for cyclic softening based on CPT penetration resistance. Existing CPT soil

## 6. SUMMARY

A detailed description of the mechanisms involved in soil liquefaction has been presented and definitions of soil liquefaction have been suggested. A flow chart to guide in the assessment of soil liquefaction has also been provided. The primary step in any liquefaction evaluation is to estimate the in-situ state of the sandy soil. If the saturated sand is very loose and contractant ( $RSR > 1$ ) the soil could be strain softening in undrained shear and flow liquefaction may be possible. If the saturated sand is very loose to medium dense, cyclic softening is possible and will depend on the size and duration of cyclic loading. The concept of potential CPT penetration resistance profiles to evaluate these different phenomena are illustrated in Figure 48. The profile to estimate the potential for flow liquefaction based on penetration resistance is influenced by the location of the ultimate state line (USL) and sand characteristics. The profile to estimate the potential for cyclic softening (cyclic liquefaction) is controlled by the size and duration of the cyclic loading as well as sand characteristics. The available field performance data for cyclic softening suggests that sand characteristics have only a small influence on the correlations, provided

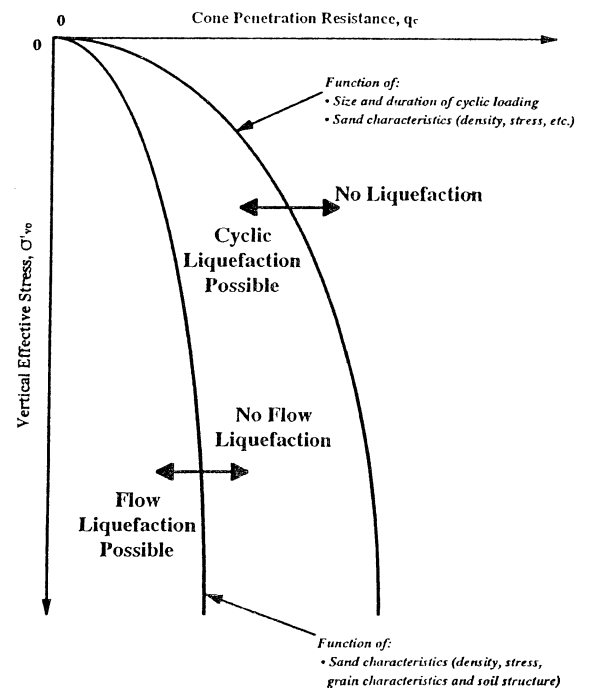


Figure 48 Schematic example of variation in CPT penetration resistance to evaluate the potential for flow liquefaction and cyclic liquefaction.

behaviour charts can be used to estimate the fines content and grain size of sandy soils which can then be incorporated directly into the evaluation process. Shear wave velocity measurements from the seismic CPT can also be used to provide an independent assessment of the potential for cyclic softening.

The addition of shear wave velocity measurements to the CPT can also provide valuable additional independent data to estimate the in-situ state of a sand for flow liquefaction analyses. A procedure to estimate in-situ state of a sand based on shear wave velocity measurements has been described. This approach has the advantage that no corrections are required for calibration chamber size effects since the correlation for any particular sand can be developed using simple laboratory equipment modified to include bender elements. The disadvantage with the shear wave velocity approach is the difficulty in measuring  $V_s$  with the same degree of accuracy and detail as is possible when measuring the CPT penetration resistance,  $q_c$ . However, the seismic CPT provides the opportunity to measure both parameters in the same soil profile and, hence, develop a site specific correlation between  $V_s$  and  $q_c$ .

The shear wave velocity values can also be used to improve the value of laboratory testing of reconstituted samples of sand (Tokimatsu and Hosaka, 1986). Samples can be reconstituted in the laboratory in such a way that their small strain stiffnesses (i.e. shear wave velocities) and densities are equal to the in-situ values estimated from the SCPT. The principle is that a reconstituted sample with the same elastic shear modulus and density as the same soil in-situ would possess the same liquefaction resistance as the soil in-situ (Tokimatsu and Hosaka, 1986).

The CPT and SCPT provide valuable data for the evaluation of liquefaction potential. The CPT provides the required level of detail to evaluate the soil profile and to identify even relatively thin layers of sand that could liquefy. The pore pressure data provides the required information regarding groundwater conditions as well as additional information to estimate grain size and fines content in sandy deposits. Shear wave velocity and cone penetration resistance both provide valuable information to estimate the in-situ state of sandy soils. From these, it is possible to provide continuous profiles of liquefaction potential in a repeatable and cost effective manner.

Evaluation of liquefaction potential is difficult due to the complex phenomena involved and the variations in grain characteristics and in-situ soil state. However, recent developments in undisturbed sampling is providing valuable insight into sand response to both monotonic and cyclic loading. Recent results suggest that current methods for estimating the cyclic resistance ratio (CRR) for

cyclic liquefaction analyses are quite good. A new method has been described that provides a fully integrated CPT approach for estimating CRR.

Evaluation of the in-situ state of a sand and its subsequent response to monotonic undrained loading (i.e. its flow liquefaction potential) is more difficult. Sands appear to be inherently anisotropic with different responses in different directions of loading. Hence, the evaluation of flow liquefaction potential requires careful selection of the minimum undrained shear strength appropriate to the in-situ soil state and ground geometry and level of tolerable deformations. Approximate methods have been described but further developments are still required.

## 7. ACKNOWLEDGMENTS

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