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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

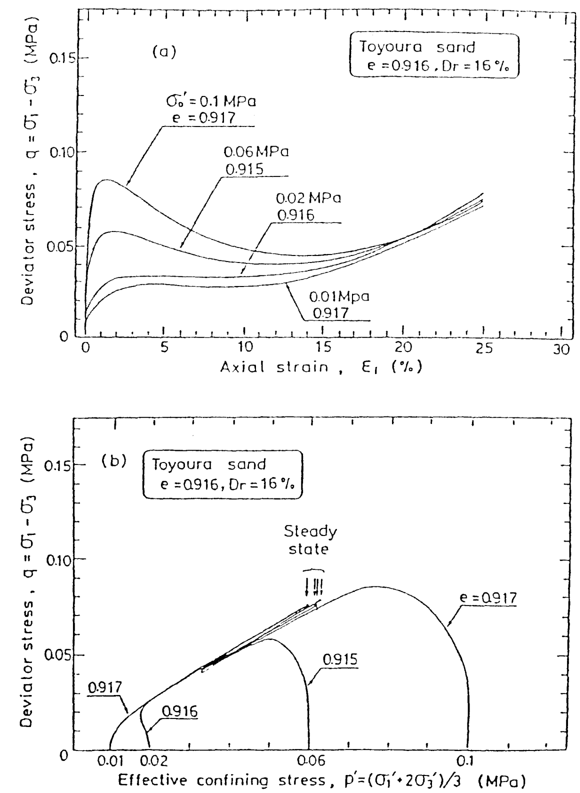

Figure 1 Undrained behaviour of Toyoura sand (after Ishihara, 1993).

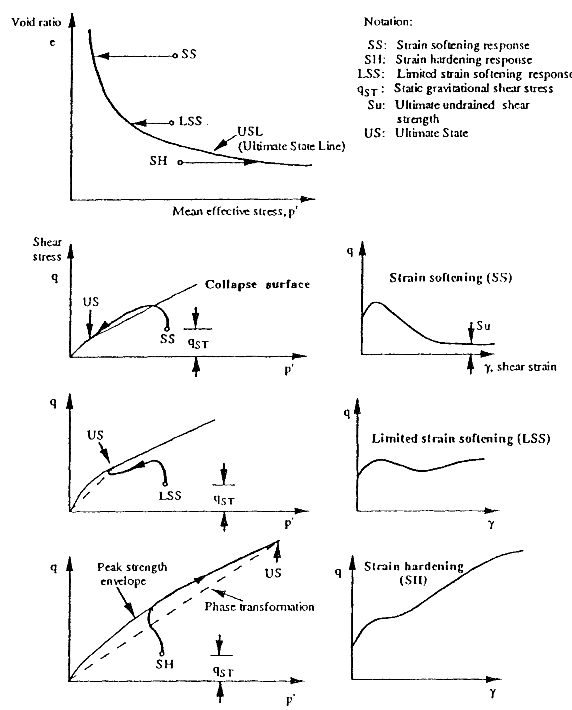

Figure 2 Schematic of undrained monotonic behaviour of sand in triaxial compression.

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

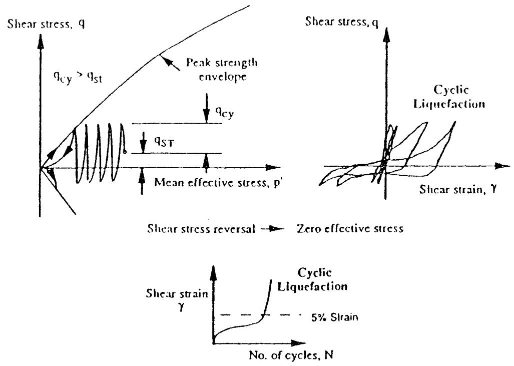

Figure 3 Schematic of undrained cyclic behaviour of sand illustrating cyclic liquefaction.

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.

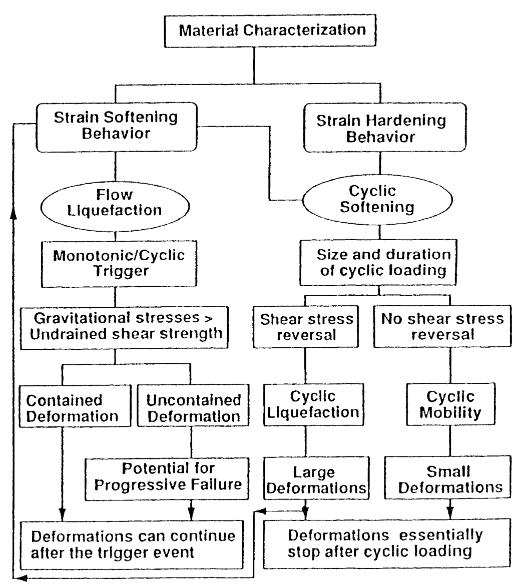

Figure 4 Suggested flow chart for evaluation of soil liquefaction (after Robertson, 1994).

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_o) 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 ($K_o = 0.5$ 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

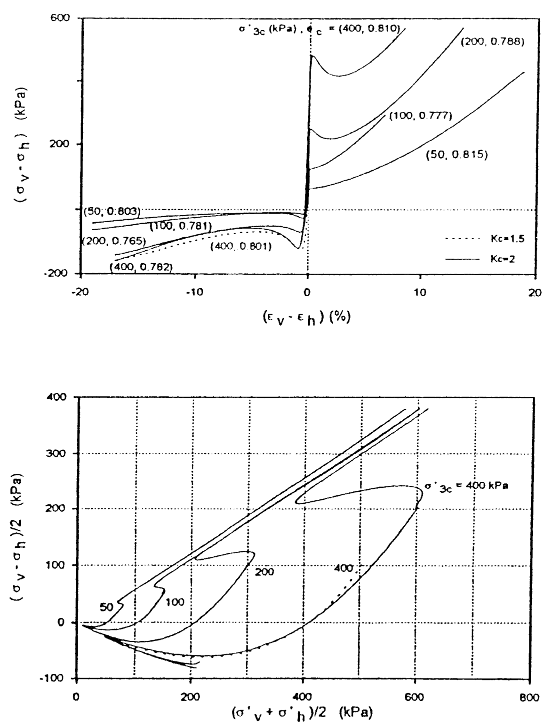

Figure 5 Triaxial undrained compression and extension behaviour of anisotropically consolidated loose water pluviated Syncrude sand (after Vaid et al., 1995a)

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 ($K_o = 0.5$). 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 ϕ'_{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 (α_σ) (Vaid et al., 1995a). The

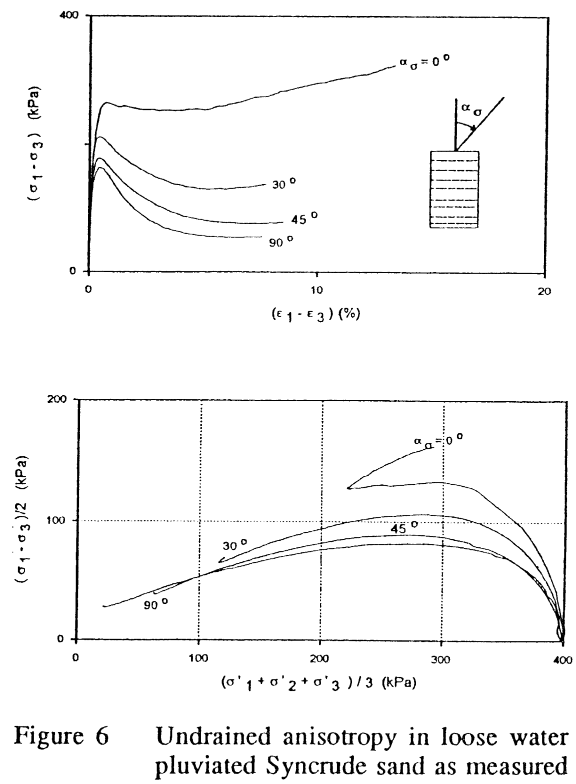

Figure 6 Undrained anisotropy in loose water pluviated Syncrude sand as measured in the hollow cylinder torsion test (after Vaid et al., 1995a).

variation in the direction of loading confirms the basic results shown in Figure 5 in that triaxial extension (α_σ = 90°) shows a lower quasi-steady state strength than triaxial compression (α_σ = 0°).

Simple shear deformation represents an initial, one-dimensional, K_0-consolidation state followed by a gradual rotation of the principal stress angle α_σ from zero to about 45° 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 α_σ and the intermediate principal stress ratio (b) undergo changes during undrained simple shear. Results from simple shear tests, with an initial static shear ratio (τ_{st} / σ'_{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.

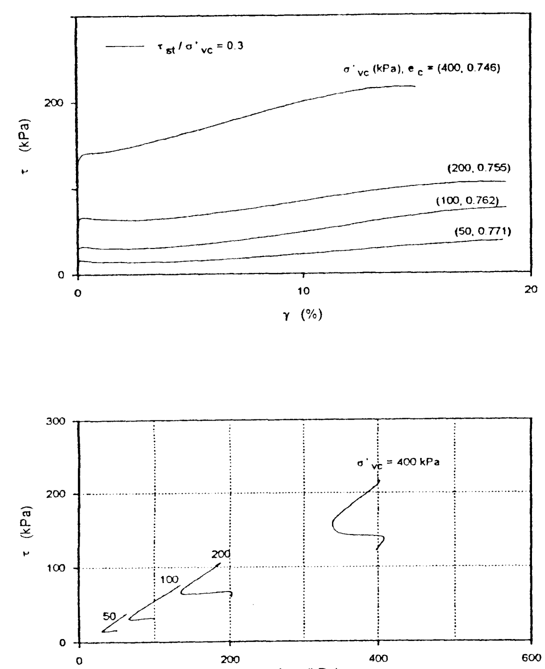

Figure 7 Constant volume simple shear behaviour of loose water pluviated Syncrude sand with static shear and no stress reversal (after Vaid et al., 1995a).

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% < ε_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.

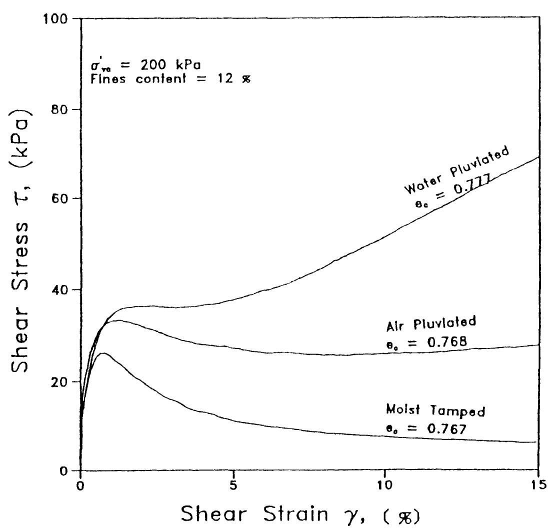

Figure 8 Undrained simple shear results on Syncrude sand using different reconstituting methods (after Vaid et al., 1995b).

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