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DESIGN CONSIDERATIONS FOR LIQUEFACTION

REFLECTIONS SUR LA CONCEPTION POUR LA LIQUEFACTION

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SYNOPSIS: The consequences of liquefaction depend on the type of liquefaction that can occur. A set of definitions has been proposed that help define the different types of liquefaction and hence, the resulting consequences. A flow chart has also been proposed to act as a guide in the process of designing for potential liquefaction. Normalised values of in-situ test results from the SPT, CPT and shear wave velocity measurements have been proposed as initial screening values to estimate if flow liquefaction is possible. If flow liquefaction is not possible, existing empirical design charts based on the SPT, CPT and shear wave velocity can be used to estimate the cyclic resistance ratio to resist seismic (cyclic) liquefaction.

INTRODUCTION

To evaluate either the response of existing structures or to design new structures on or with soil that may liquefy it is important to understand the consequences of the possible liquefaction event. There is often much confusion in the literature about the definition and hence, consequences of soil liquefaction. Castro (1969) defined liquefaction as the strain softening and collapse of a loose sand to an ultimate state of constant effective stress and deformation. Seed et al. (1983), using the results of extensive laboratory testing, defined liquefaction as the condition of zero effective confining stress due to cyclic loading. At zero effective stress a granular soil becomes very soft and develops large deformations during cyclic loading. Ishihara (1993) suggests that liquefaction can be defined in terms of the magnitude of cyclic stress ratio required to produce a given level of strain, typically 5% double amplitude axial strain.

The objective of this paper is to briefly review the behavior of granular soils to undrained loading, present a clear set of definitions for liquefaction and to suggest a flow chart to evaluate liquefaction for design.

BEHAVIOR OF SAND IN UNDRAINED LOADING

Ishihara et al (1991) presented a set of results from undrained triaxial compression tests on a clean sand. These results present a clear picture of sand behavior in undrained shear, since they show three sets of results at different values of void ratio and varying effective confining stress. For the very loose sand (density index = 16%), shown in Figure 1, the sand shows a marked strain softening response during undrained shear. The shear stress reaches a peak then strain softens to an ultimate condition at steady state. The stress path during strain softening appears to follow a 'collapse surface', as suggested by Sladen et al, (1985). However, at lower confining stress the sand at the same void ratio shows a strain hardening response before reaching steady state. For the same sand at a higher density a similar behavior is seen except that the steady state condition is reached at a higher stress level. For the very dense sand, the response is predominately strain hardening since the steady state strength is very large. This confirms the basic behavior suggested by Castro (1969) and that embodied in critical state Figure 2 shows a schematic summary of the behavior of a granular soil loaded in undrained triaxial compression. A soil with an initial state higher than the steady state will be strain softening (SS) at large strains, whereas a soil with an initial state lower than the steady state will be strain hardening (SH) at large strains. It is possible to have a soil with an initial state higher than the steady state but close to steady state. For this soil state the response can show limited strain softening (LSS), but eventually at large strains the response becomes strain hardening to the ultimate steady state. For a soil with limited strain softening (LSS) there can be a quasi-steady state before strain hardening occurs to the ultimate steady state. The response at quasi-steady state appears to be controlled by the sand structure and method of deposition.

If a soil 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 steady state strength (i.e. a relatively steep slope), a catastrophic collapse and flow slide can occur if the soil is triggered to strain soften. Sasitharan et al, (1993) 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 (1993) have also shown that undrained collapse can also be triggered by certain types of slow monotonic loading. If a soil structure is composed entirely of a limited strain softening soil and the in-situ gravitational shear stresses are larger than the quasi-steady state strength, a catastrophic flow slide is unlikely. However, large deformations can occur before the soil stiffens as it strain hardens towards its ultimate state.

If a soil structure is composed entirely of strain hardening soil, undrained collapse and flow slide can not occur and deformation will, in general, be small. If a soil structure is composed partly of strain softening (SS) and strain hardening (SH) soil and the SS soil is triggered to strain soften, a collapse and flow slide will only occur if, after stress redistribution due to the softening of the SS soil, the SH soil can not support the gravitational shear stresses. The trigger mechanism can be cyclic, such as earthquake loading, or monotonic, such as a rise in ground water level or a rapid undrained loading. Gu et al (1993a) used the collapse surface approach to explain the failure of the Lower San Fernando Dam shortly after the 1971 San Fernando earthquake. Gu et al (1993b) also used the collapse surface approach to explain the continued buildup of pore pressure and deformation after the

1987 Superstition Hills earthquake at the Wildlife Site in the Imperial Valley, California.

During cyclic undrained loading, almost all granular 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 zero effective stress, as illustrated in Figure 3. For shear stress reversal to occur, ground conditions are generally level or gently sloping. When a soil element reaches the condition of zero effective stress, the soil has very little stiffness and large deformations can occur during cyclic loading. For very dense soils, the cyclic loading may not be sufficient to reduce the state to zero effective stress and hence, deformations will be smaller. When cyclic loading stops the deformations essentially stop, except those due to local pore pressure redistribution. Gu et al (1993b) 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 the stress state can not reach zero effective stress and cyclic mobility with limited deformations will occur.

PROPOSED DEFINITIONS OF LIQUEFACTION

Based on the above description of soil behavior in undrained shear, the following definitions of liquefaction are suggested.

Gravitational (Flow) Liquefaction

- Requires strain softening response in undrained loading resulting in constant shear stress and effective stress, (i.e. ultimate steady or critical state).
- Requires that in-situ shear stress is greater than undrained residual or steady state shear strength.
- Gravitational (flow) liquefaction can be triggered by either monotonic or cyclic loading.
- For failure of a soil structure to occur, such as a dam or a slope, a sufficient volume of material must show strain softening response. The resulting failure can be a slide or a flow depending on the material characteristics and slope geometry. The resulting movements are due to internal causes and can occur after the trigger mechanism.
- Can occur in saturated, very loose granular deposits, very sensitive clays and loose loess deposits.

Seismic Liquefaction

Cyclic Liquefaction

- Requires undrained cyclic loading where shear stress reversal or zero shear stress can develop (i.e. where in-situ static gravitational shear stress is low compared to cyclic shear stress).
- Requires sufficient undrained cyclic loading to allow effective confining stress to reach zero.
- At point of zero effective confining stress no shear stress can exist. When shear stress is applied, pore pressure drops and a very soft initial stress strain response can develop resulting in large deformations. Soil will strain harden with increasing shear strain.
- Deformations during cyclic loading when effective stress is approximately zero can be large, but deformations stabilize when cyclic loading stops. The resulting movements are due to external causes and only occur during the cyclic loading.
- Can occur in almost all sands provided size and duration of cyclic loading is sufficiently large. For very dense sands the size and duration of cyclic loading will be large and hence, the condition of zero effective confining stress may not always be achieved.
- Clays can experience cyclic liquefaction but deformations at zero effective stress are generally small due to the cohesive strength at zero effective stress and deformations are often controlled by rate effects (creep).

Cyclic Mobility

- Requires undrained cyclic loading where shear stress is always greater than zero, i.e. no shear stress reversal develops.
- Zero effective stress does not develop.
- Deformation during cyclic loading will stabilize. The resulting movements are due to external causes and only occur during the cyclic loading.
- Can occur in almost any sand provided size and duration of cyclic loading is sufficiently large and no stress reversal occurs. Can also occur in very dense sand with shear stress reversal provided cyclic loading is not sufficient to cause zero effective stress to develop.
- Clays can experience cyclic mobility but deformations are often controlled by rate effects (creep).

PROPOSED FLOW CHART TO EVALUATE LIQUEFACTION

Figure 4 presents a suggested flow chart for the evaluation of liquefaction according to the above definitions. The first step is to evaluated the material characteristics in terms of strain softening or strain hardening response. If the soil is strain softening, gravitational (flow) liquefaction is possible if the soil can be triggered to collapse and if the gravitational shear stresses are larger than the ultimate residual or steady state strength. The trigger to cause collapse 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 or limited strain softening soil relative to the strain hardening soil within the structure and on the brittleness of the strain softening soil. Dawson et al (1993) have shown that at high effective stresses some strain softening granular soils appear to become less brittle with increasing confining stress. 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 structure, amount and type of trigger mechanism, brittleness of strain softening soil and drainage conditions.

If the soil is strain hardening, gravitational (flow) liquefaction will not occur. However, cyclic liquefaction can occur due to cyclic (seismic) undrained loading. The amount and extent of deformations during cyclic loading will depend on the size and duration of the cyclic loading and on whether shear stress reversal occurs. If shear stress reversal occurs it is possible for the effective stress to reach zero and hence, cyclic liquefaction can take place. At the condition of zero effective stress large deformations can occur. If shear stress reversal does not take place or if the sand is very dense and it is not possible to reach the condition of zero effective stress, deformations will be smaller, hence, cyclic mobility will occur.

METHODS TO EVALUATE LIQUEFACTION

The preferred method to evaluate the response of a soil to a given loading is to obtain high quality undisturbed samples and perform relevant laboratory testing. However, this process is difficult in cohesionless soils such as sand. In-situ freezing has been recently used to successfully obtain high quality undisturbed samples of cohesionless soils (Yoshimi et al., 1984; 1989; Hatanaka et al., 1985; Sego et al, 1993). However, the cost of these techniques is very high and is currently restricted to larger projects. For smaller projects and in the initial stages of large projects, in-situ testing has been the most commonly used approach to evaluate liquefaction potential. The most commonly used in-situ methods are the Standard Penetration Test (SPT) and the Cone Penetration Test (CPT). Recently the self-boring pressuremeter test (SBPT), the flat dilatometer test (DMT) and shear wave velocity measurements using either cross-hole, seismic CPT or SASW techniques have been proposed to evaluate liquefaction potential. Most of the existing methods to evaluate liquefaction potential from in-situ test results are applicable to seismic liquefaction and are based on the cyclic stress ratio to trigger a given level of deformation (either cyclic liquefaction or cyclic mobility). Most methods are for level ground conditions however, corrections are provided for sloping ground (Seed et al, 1983). In general, these methods have proven to be very good at

predicting cyclic liquefaction for essentially level ground conditions (Kayen et al, 1992). However, they are generally too conservative to predict the strain softening response of very loose sands in steeply sloping ground that may result in flow liquefaction. Robertson et al, 1992, and Ishihara (1993) have suggested normalised values of penetration resistance from the SPT and CPT to evaluate if strain softening response and hence, the potential for flow liquefaction, is possible. Robertson et al (1992) also suggested values of normalised values of shear wave velocity to evaluate the in-situ state of cohesionless soils.

The following normalized values can be used as an initial screening to identify clean sands that may show a strain softening or limited strain softening response in undrained loading and hence, may result in flow liquefaction or large deformations;

SPT	$(N_1)_{60}$	=	13 blows/foot
CPT	q_{c1}	=	6.5 MPa
Shear wave velocity	V_{s1}	=	160 m/s

These normalized values are global in nature and will not apply to all sands. However, they can be useful as an initial screening of sand deposits.

The shear wave velocity has the advantage that measurements can be made both in-situ and in the laboratory. A relationship between shear wave velocity, void ratio and effective confining stress can be developed based on a small number of undrained triaxial compression tests on very loose samples of the given sand. The in-situ shear wave velocity and an estimate of the in-situ effective stress can then be used to evaluate if the sand has the potential for flow liquefaction (Sasitharan et al, 1993c).

SUMMARY

The consequences of liquefaction depend on the type of liquefaction that can occur. A set of definitions have been proposed that help to define the different types of liquefaction and hence, the resulting consequences. A flow chart has also been proposed to act as a guide in the process of evaluating liquefaction potential. The first step of the proposed flow chart is to characterise the soil to evaluate if the soil will show a strain softening or strain hardening response during undrained loading. If the soil is strain softening at large strains either gravitational (flow) liquefaction or seismic liquefaction is possible. However, if the soil is strain hardening at large strains only seismic liquefaction is possible.

Normalised values of SPT and CPT penetration resistance and shear wave velocity (Vs) have been proposed that can be used as screening methods to evaluate the material characteristics in terms of strain softening or strain hardening response. These normalised values are applicable to clean sands up to a vertical effective overburden stress of about 200 kPa. At higher stress levels these values will tend to increase slightly. For silty sands the normalised penetration values will tend to decrease. If a sand has a penetration resistance lower than these proposed values a strain softening response can be expected during undrained loading, hence there is a possibility of flow liquefaction. However, for flow liquefaction to occur requires that the gravitational shear stresses be higher than the undrained ultimate shear strength and that strain softening behaviour must be triggered. The normalised SPT, CPT and Vs values are conservative and should be used only as a screening device to evaluate in-situ state of clean sands. To evaluate the actual soil behavior requires very high quality undisturbed samples and laboratory testing. In-situ freezing has been shown to provide excellent undisturbed samples of cohesionless soils. However, the cost of in-situ freezing can be very high and maybe uneconomical for smaller engineering projects. Hence, in-situ test techniques and the proposed normalised values maybe sufficient for a conservative design evaluation for smaller projects.

If the soil is considered to show a strain hardening response during undrained loading (i.e. in-situ test SPT, CPT and Vs values larger than proposed normalised values) only seismic liquefaction can occur. For seismic liquefaction, either cyclic liquefaction or cyclic mobility can occur, depending on whether shear stress reversal can take place during the seismic loading and on the density of the sand. The empirical charts developed by Seed et al (1983), Tokimatsu and Yoshimi (1983) and Robertson et al (1992) using in-situ test results (SPT, CPT, Vs) can be used to evaluate if seismic liquefaction will be triggered for a given level of cyclic loading represented by the cyclic stress ratio (τ/σ'). If shear stress reversal can occur, deformations during cyclic loading can be large when the cyclic stress ratio (CSR) from the seismic event exceeds the cyclic resistance ratio (CRR) estimated from the empirical charts. If shear stress reversal can not occur or if the sands are very dense, deformations will in general be smaller. The empirical charts to estimate the triggering of seismic liquefaction should not be used to evaluate the triggering of flow liquefaction.

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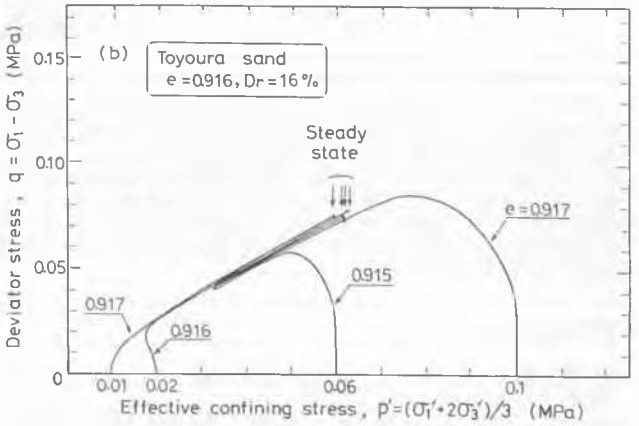

Figure 1. Test results for very loose Toyoura sand on an undrained plane. (After Ishihara et al, 1991).

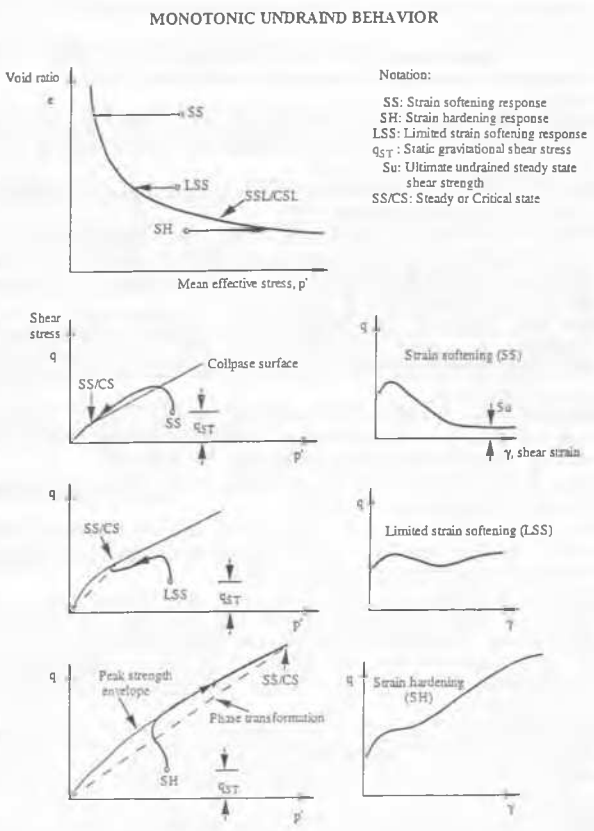

Figure 2. Schematic behaviour of a cohesionless soil in monotonic undrained triaxial compression.

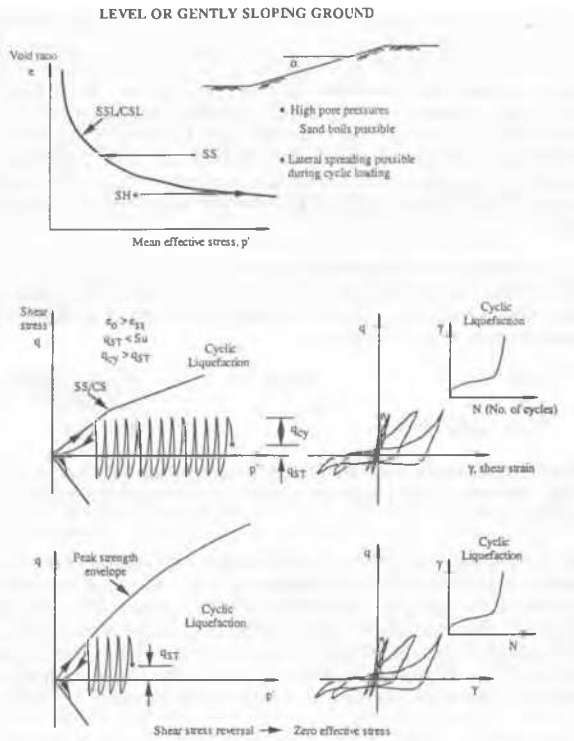

Figure 3. Schematic behaviour of elements of cohesionless soil under cyclic undrained loading in level or gently sloping ground.

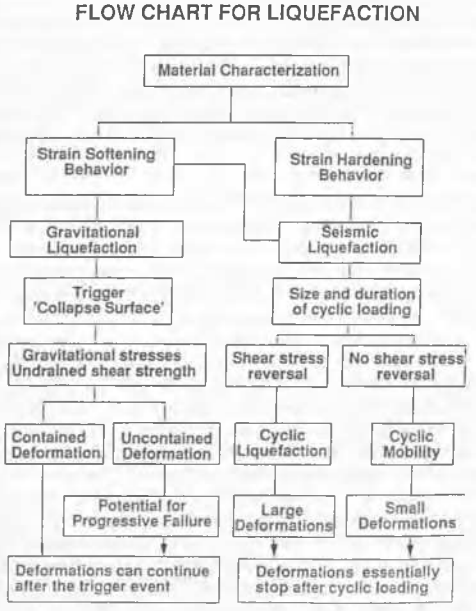

Figure 4. Proposed flow chart for evaluation of liquefaction.