LIQUEFACTION BEHAVIOUR AND ASSESSMENT IN ENGINEERING PRACTICE

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ABSTRACT

A procedure for evaluation of liquefaction susceptibility of a granular soil mass is presented. The basis for such proposition lies in the soil behaviour exhibited in conventional laboratory triaxial tests. While the behaviour of clean sand is similar to that reported in many technical publications, the addition of a small amount of fines dramatically changes the soil behaviour. Such unusual behaviour appears compatible to the framework of critical soil mechanics due to changes in shape and the relative position between the critical state and normal consolidation lines. A modified state parameter is defined to predict the undrained behaviour of both clean sand and sand with fines. Cavity expansion simulations indicate that a good correlation exists between the modified state index and spherical cavity pressure ratio. On the basis of spherical cavity expansion analogue to CPT testing, a relationship is proposed to estimate the granular soil behaviour based on in-situ CPT data.

Keywords: static liquefaction, sand with fines, critical state, triaxial, piezocone, dilatometer

1 INTRODUCTION

1.1 General

Liquefaction is an important geotechnical problem which affects natural slopes and man-made structures. The devastating effects manifest when instability occurs at an in-situ stress state located under the failure surface envelope. The reduction in shear resistance is sudden with a potential to initiate flow-like landslides of high velocities that travel significant distances to affect loss of property and more importantly human life.

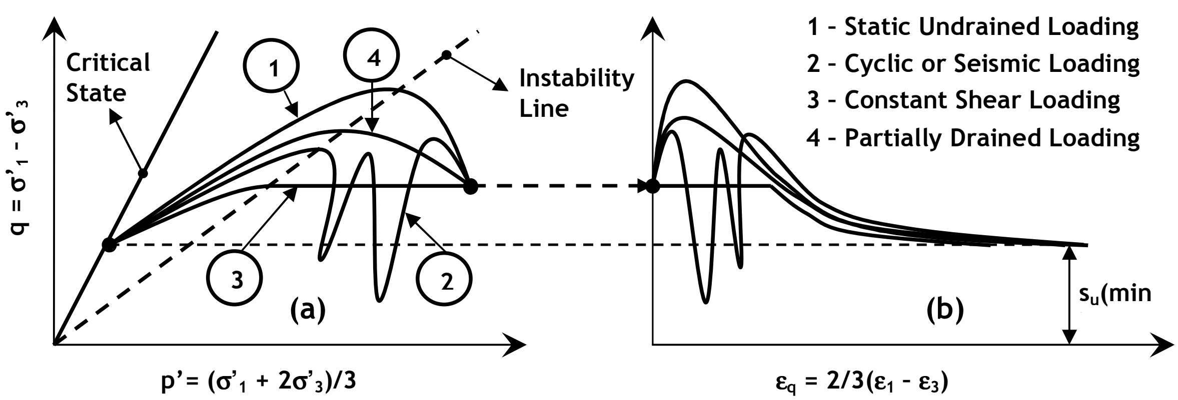

Figure 1. Triggering mechanism of instability: (a) Stress paths (b) Stress-strain curves.

Figure 1 schematically illustrates the triggering mechanism of four modes of instability. A common feature of all modes of instability is the strain softening branch of monotonic loading as it provides a boundary of admissible stress states. When the stress path attempts to cross this boundary, it triggers instability which is accompanied by a sudden and dramatic increase in the excess pore water pressure. At large displacements a minimum shear strength, s_u(min), is reached. The shear strength s_u(min) may correspond to either the critical state (CS) or a transient state referred to by Alarcon-Guzman et al. (1988) as the quasi-steady state (QSS). If the in-situ driving shear stress is less than s_u(min), then the soil mass is not susceptible to liquefaction. Such condition at s_u(min) differentiates between the mechanism of cyclic liquefaction and cyclic mobility. During cyclic mobility the shear stress is usually smaller than s_u(min) and excess pore water pressure gradually accumulates without dramatic increases.

The behavior during static loading appears to hold the key when predicting the mechanism of flow failure deformation. Hence, potential zones susceptible to liquefaction can be identified based on the static liquefaction response.

1.2 Paper Objectives

The objective of this paper is to present an experimental and analytical study which forms the basis for an in-situ CPT assessment of static liquefaction.

The experimental study is conducted to understand the behaviour of sand, as well as on how the behaviour changes in the presence of a small amount of fines. The original state parameter (Been and Jefferies 1985) is reformulated to include a dependency on the stress level and curvature of critical state line. The new state index is customarily termed "modified state parameter" (ψ_m) as additional attributes are added to the original state parameter (ψ). Values of ψ_m are separated in well-defined ranges corresponding to behavioural trends noted in undrained shearing.

In addition to behaviour, the stress–strain characteristics of soil specimens with the same ψ_m are investigated with a relationship presented between spherical cavity pressure ratio (Q_{sph}) and ψ_m. Such correlation forms the basis for estimates of state index ψ_m from in-situ values of CPT tip resistance.

2 EXPERIMENTAL STUDY

2.1 Soil Materials

A mix of sand with fines fraction consisting of both plastic and non-plastic constituents was tested in this study. The sand was a typical Sydney beach material; whereas the fines (i.e. particles less than 75µm) were added to the mix as 10% by weight in the following proportions: 1/3 commercial medium plasticity kaolin and 2/3 low plasticity silt.

2.2 Undrained Behaviour

The triaxial stress-strain response and effective stress paths of sand with fines are shown in Figure 2. The shearing starts from an isotropically consolidated state in the range of p'_0 = 100kPa to 1,115kPa. All stress-strain curves display initially a sharp increase in the deviatoric stress to a peak strength corresponding to $\varepsilon_q \leq 0.5\%$, which is then followed by strain softening.

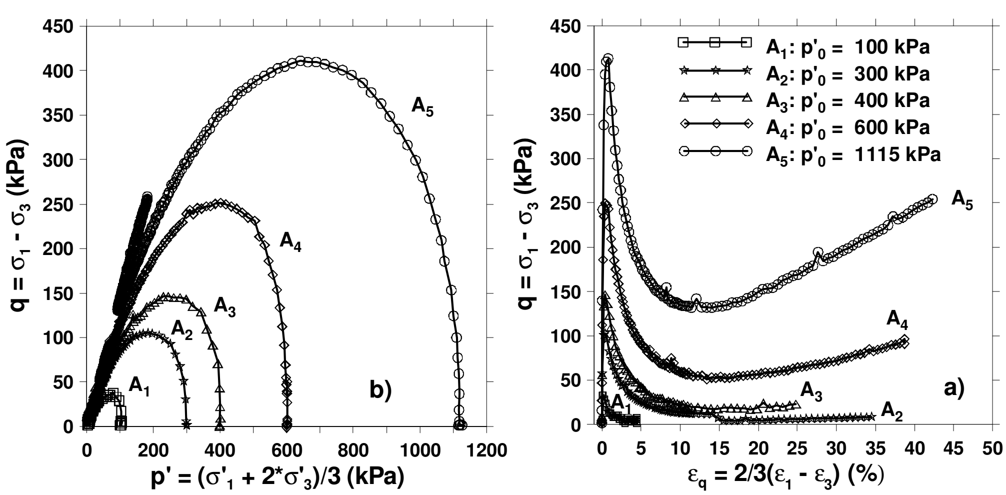

Figure 2. Response of sand with fines in undrained shearing for a range of confining pressures p'_0 = 100 to 1,115 kPa. a) Stress-strain relationships b) Effective stress paths.

Whilst the deviatoric stress in tests A_1 to A_3 reaches a steady value after the peak strength, in tests A_4 and A_5 a typical QSS is noted. In general, the effective stress paths show a response which changes from flow liquefaction at low p'_0 values to limited flow at higher p'_0 values. This behaviour conforms to the pattern of 'reverse behaviour' previously reported by Yamamuro and Lade (1998).

2.3 Critical and Steady State

To interpret the 'reverse behaviour' in the critical state framework, the uniqueness of critical state (CS) and steady state (SS) of sand with fines was studied by conducting a number of drained and undrained tests. The data was found to trace along a curve in Figure 3 and consequently no distinction is made between CS and SS states throughout this paper.

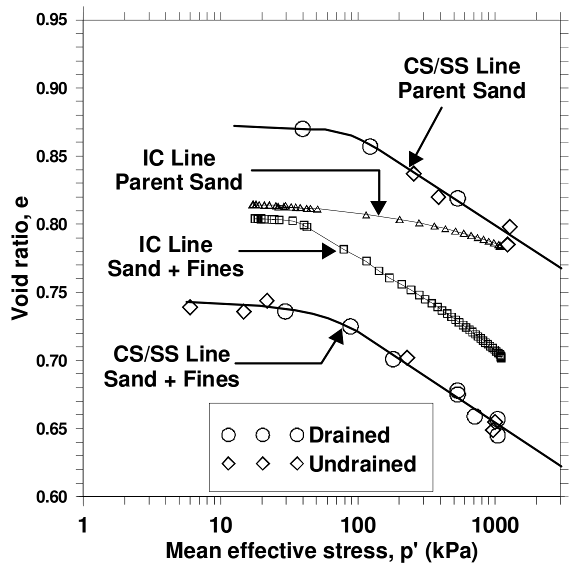

Figure 3. Critical state and normal consolidation lines of parent sand and sand with fines.

Bobei (2004) also found a similar CS and SS line for parent sand as illustrated in Figure 3. A typical NC line of sand initially plots under the CS line. With the increase in confining pressure, the NC shows a trend of intersecting the CS at p' = 1,100 to 1,200 kPa. The behaviour of sand also changes from non-flow in undrained shearing below $p'_0 < 1{,}000$ kPa and $\psi < 0$ to limited flow when $p'_0 = 1{,}200$ kPa and $\psi \sim 0$. As expected, the response of sand conforms to the critical state framework.

3 INTERPRETATION OF EXPERIMENTAL RESULTS

3.1 Modified State Parameter

A state parameter (ψ) was formulated by Been and Jefferies (1985) as the vertical distance between the initial state and CS line as illustrated in Figure 4. This formulation is proposed to quantify the potential for contraction and dilation based on the proximity of the soil state to the critical state, such that: the larger the distance is from the CS, the higher is the potential for contraction or dilation.

While there are experimental results in support of ψ to predict the behavior of sand, it is not known how ψ will predict the behavior of sand with fines. Figure 3 indicates that NC line of sand with fines remains parallel to CS for $p'_0 > 100$kPa. In this range, the undrained shearing response changes from flow to limited flow. Provided the state parameter remains unchanged, it appears that ψ in its original definition may not adequately capture the changing behaviour of sand with fines.

An important outcome of Figure 3 is the visibly curved shape of CS in the region of lower mean effective stresses. Accordingly, a new state index is proposed to factor the changing slope of CS. This index builds additional attributes to the definition of the original state parameter, and is appropriately named "modified state parameter" (ψ_{m0}).

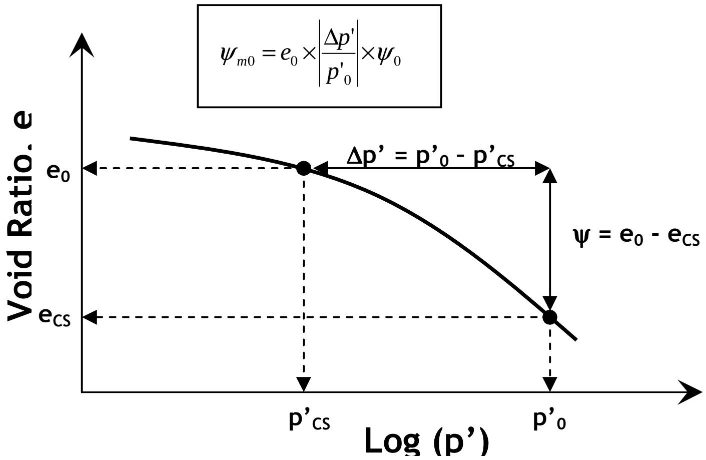

Figure 4. Definition of modified state parameter.

3.2 Range of Modified State Parameters

Bobei (2004, 2005 & 2009) studied the variation of undrained soil response for a wide range of ψ_{m0} values.

4 ANALYTICAL APPROACH

4.1 Cavity Expansion

The numerical analysis of an expanding cavity was found useful in the past to interpret field results of pressuremeter tests, bearing capacity of a driven pile, or CPT results. The description of CPT penetration was analytically treated based on an analogue displacement controlled spherical cavity. On this assumption the displacements are radial around the expanding cavity, and essentially a complex 3D analysis is reduced to a 1D problem. The merits of the analogy drawn between spherical cavity expansion and CPT have been investigated by Ladanyi and Roy (1987).

4.2 Correlation Between CPT and Soil State

Simulations using Carter et al. (1986) large strain spherical cavity solution are conducted for a range of ψ_{m0} values. These values are representative of flow and limited flow range of soil behaviours. For each ψ_{m0}, a value of elastic shear modulus is assigned while the confining stress is varied between 20kPa and 600kPa.

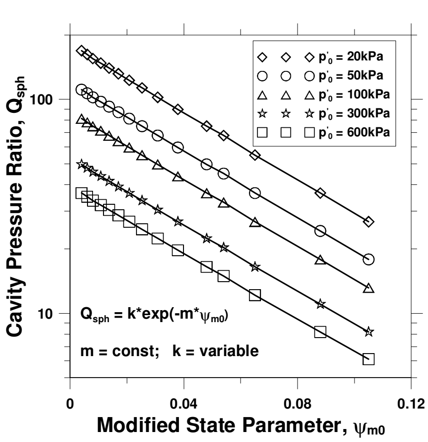

Figure 5. Variation of relationship between spherical cavity pressure ratio and modified state parameter with the confining stress.

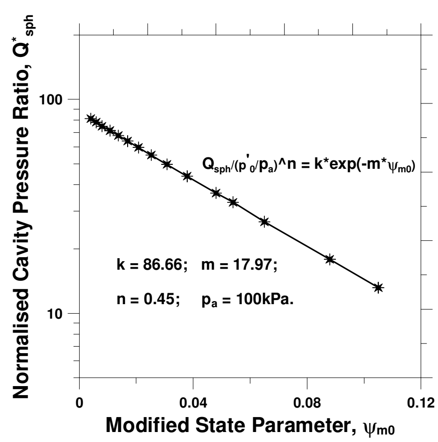

Figure 6. Relationship between normalised spherical cavity pressure ratio and modified state parameter.

The logarithm of spherical cavity pressure ratio (Q_{sph}) is found to decrease linearly with ψ_{m0} as plotted in Figure 5 and 6. The confining pressure has a strong influence on Q_{sph}-ψ_{m0} relationship, such that the increase in confining stress is found to shift downwards the position of the linear prediction. The slope of linear relationship does however remain unaffected.

Based on Ladanyi and Roy (1987) experimental findings that Q_{sph} and Q_p offset by a factor of two, the value of modified state parameter can finally be retrieved as follows:

$$\psi_{m0} = -\frac{\ln\left(\dfrac{Q_p}{2\times k\times\left(\dfrac{p_0'}{p_a}\right)^n}\right)}{m} \tag{1}$$

5 EXAMPLE OF STATIC LIQUEFACTION ASSESSMENT

An example on the applicability of the proposed static liquefaction evaluation procedure is presented in this section. The evaluation is conducted based on a typical CPT and DMT sounding reported by GHD at the site of Sydney Desalination Plant located within the Kurnell Peninsula (see Figure 7).

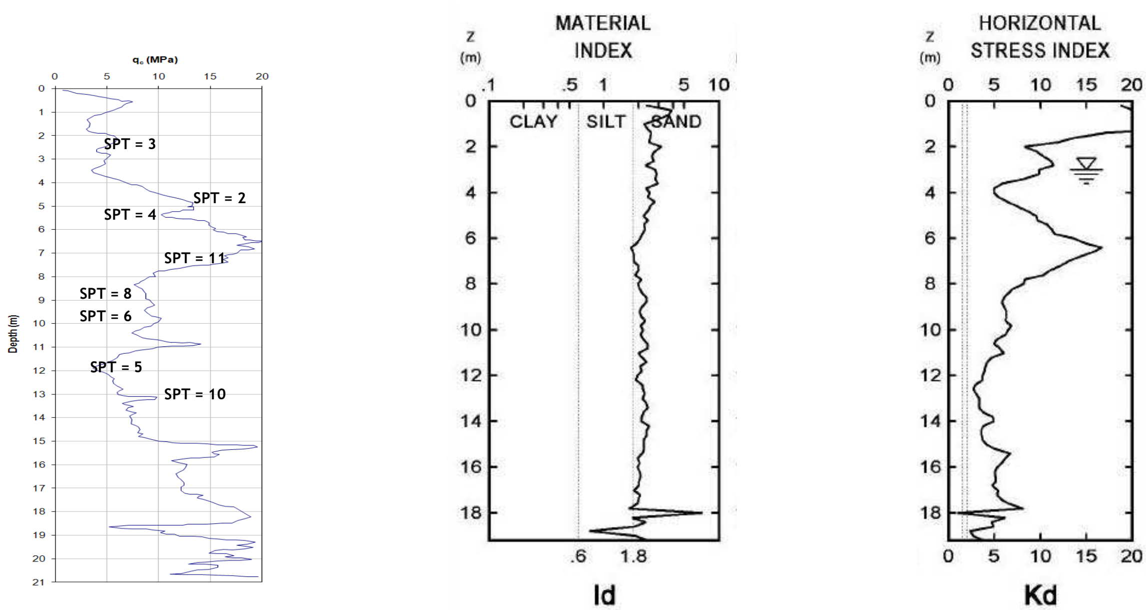

Figure 7. Typical CPT and dilatometer logs reported by GHD at the site of Sydney Desalination plant, Kurnell Peninsula.

The calculations can be easily implemented in an excel spreadsheet to obtain the variation of ψ_{m0} with depth as plotted in Figure 8. Some comments on the liquefaction potential evaluation are as follows:

- The values of ψ_{m0} indicate a potential for flow and limited flow to manifest in the top 6.0m. Below this depth, the susceptibility for liquefaction reduces as the sand is evaluated to behave in a non-flow manner.
- The SPT values from the nearby borehole are plotted on top of CPT cone tip resistance trace in Figure 7. Low SPT N values between 2 and 4 are reported in the top 6.0m. These values indicate the presence of a very loose material which explains the liquefaction susceptibility in the upper 6.0m layers.
- Below 6.0m depth, the SPT N values generally range between 8 and 11 blow counts with occasional 5 to 6 values recorded when clay lenses are present. The SPT values indicate that penetration progressed through an essentially loose to medium dense material. A granular

material of such density is expected to behave in a non-flow manner which is in agreement with the response estimated based on ψ_{m0} values.

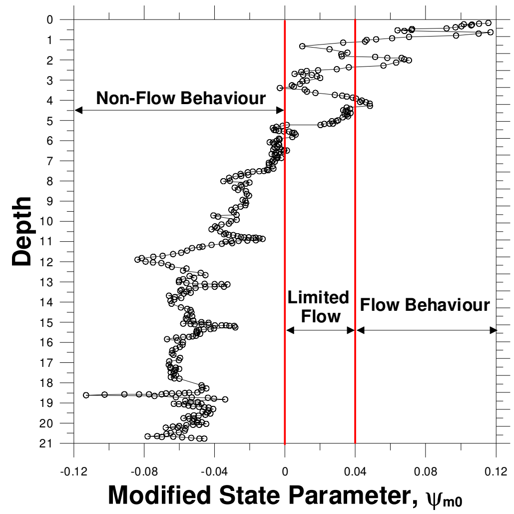

Figure 8. Assessment of soil behaviour based on CPT data.

6 CONCLUSIONS

The paper has set out an approach for evaluation of static liquefaction in granular materials. The evaluation framework is built around the behaviour of clean sands and sands with a small amount of fines. The main findings of the paper are summarised below:

- The undrained response of clean sand conforms to the normal behaviour of soils, whereas the addition of fines has a dramatic effect of changing the soil behaviour. This unusual behaviour is called "reverse behaviour" and is found consistent with previous reports by Yamamuro and Lade (1998).
- The reverse behaviour is explained in the critical state framework because of significant changes in the shape of NC line and its relative position with respect to the CS line.
- The modified state parameter has the additional advantage over the original state parameter as it considers the curvature of critical state.
- Numerical simulations based on spherical cavity expansion theory indicate that a good correlation exists between the modified state parameter and spherical cavity pressure ratio. On the basis that spherical cavity expansion is analogue to CPT testing, a single relationship estimates the granular soil behaviour based on in-situ CPT and DMT data.
- An example is provided to evaluate the static liquefaction of granular materials based on the proposed assessment framework. The liquefaction evaluation based on ψ_{m0} is consistent to traditional procedures employing the SPT penetration techniques.

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