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Undrained Behaviour of Silty Glacial Sand under K_0-Consolidation

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ABSTRACT

Instability of loose to medium dense granular soil is the primary cause of slope failures, landslides and earthquake-induced/static liquefactions. Instability occurs due to the rapid pore water pressure generation during undrained loading. The triggering of instability for sandy soils has been studied in the past under laboratory condition, mostly for isotropically consolidated soils. However, a natural level ground, most likely, has a history of K_0-consolidation. Recent studies showed that the history of consolidation, namely Isotropic and K_0 have a significant effect on the triggering of instability. Therefore, the effect of consolidation on instability was investigated in this study. South Australian silty glacial sand which contains about 10% fines content was used for triaxial tests. It was observed that instability trigger earlier, in terms of on-set stress ratio, for K_0 than isotropically consolidated specimen.

Introduction

The instability/liquefaction is one of the major causes of sandy soil failure during earthquake/static loading. Instability may trigger due to increase in pore water pressures under static or dynamic loading. Undrained triaxial testing is one of the most commonly applied laboratory test for studying this phenomenon. The typical undrained behaviour of sand at various density conditions are illustrated in Figure 1. While the deviatoric stress, q of loose sand reaches to a peak and then drops to steady state (SS) at large axial strain, medium dense sand shows a temporary drop of q after initial peak and then rises to a higher q at SS. The temporary drop of q to a minimum is called the quasi-steady state, QSS (Alarcon-Guzman, *et al.*, 1988, Ishihara, 1993). The former is known as "flow" type behaviour and the latter is known as "limited flow" behaviour (Chu and Leong, 2002, Yang, 2002, Rahman, *et al.*, 2008). On the other hand, dense soil continues to increase strength over large axial strain until it reaches SS. This behaviour is referred to as "non-flow" type behaviour.

The term instability was used as deviatoric strain softening for loose to medium dense sand is a form of instability and is often characterised by instability line, IL (Lade, 1993, Chu and Leong, 2002) or instability stress ratio, $\eta_{IS} = (q/p')_{IS}$; where the subscript "IS" denotes onset of instability. Chu and Leong (2002) showed that η_{IS} correlated with e_0; where '0' in the subscript

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indicates at the beginning of shearing or the end of consolidation. The pathway to that η_{IS} can be related to state parameter (Yang, 2002, Chu, *et al.*, 2012) which can be link to predict cyclic instability (Baki, *et al.*, 2014, Rahman, *et al.*, 2014a). However, these understandings were developed based on experimental studies on isotropically consolidated (i.e. K $= \sigma_3'/\sigma_1' = 1.0$) specimens, whereas natural ground may have an in-situ consolidated state, most probably K_0-consolidated (i.e. $K_0 = (\sigma_3'/\sigma_1')_{\varepsilon_h=0}$, where ε_h is horizontal strain) below level ground. Therefore, these understandings may not be directly transferable to field condition. However, only few studies have discussed the effect of anisotropic consolidation on undrained monotonic behaviour of sands and a consensus is yet to be reached (Kato, *et al.*, 2001, Finno and Rechenmacher, 2003, Chu and Gan, 2004, Fourie and Tshabalala, 2005, Chu and Wanatowski, 2008). Chu and Wanatowski (2008) found that η_{IS} remained same for K_0 and K = 1.0 at the same initial states, while Kato, *et al.* (2001) and Fourie and Tshabalala (2005) obtained higher η_{IS} for K_0 than for K = 1.0 for loose sandy soil. Subsequently, the question raised in transferring existing knowledge to field condition requires further evaluation. Therefore, this study investigated the undrained behaviour of natural silty sand under isotropically and K_0-consolidated condition.

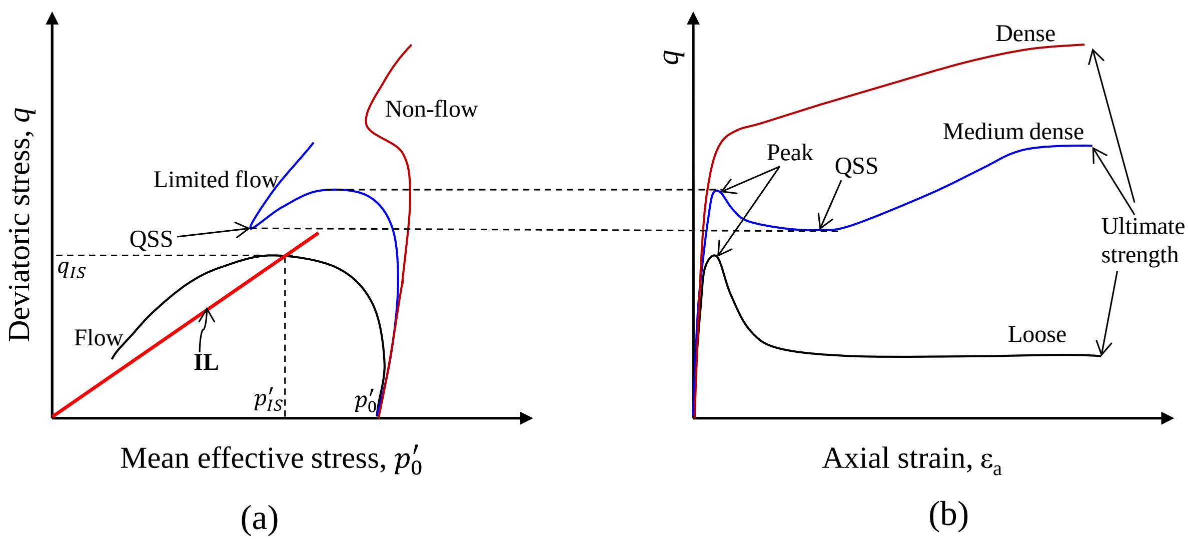

Figure1: Typical undrained behaviour of sand.

Testing Materials

In this study a yellow colour silty sand sample was used which was collected from the slopes of a hilly region from the Mount Compass area of South Australia and was named as Yellow sand (YS). Yellow sand has about 10% fines content and thus, wet sieving method was chosen over dry sieving to break down any aggregation of silt or clay. The sand samples were dispersed in a solution of sodium hexametaphosphate (40 g/litre) overnight and then washed on an ASTM #200 sieve to separate the fine particles from sand. The particle size distribution of sand was obtained by wet sieving and fine particles (< 0.075 mm) by hydrometer analysis. The grain size distribution curve is shown in Figure 2. The sand was poorly graded sand (SP-SM).

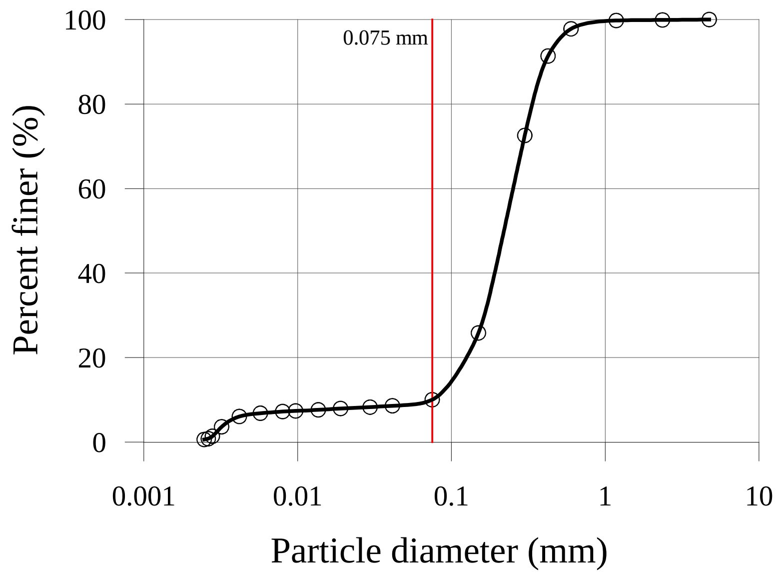

Figure 2: Grain size distribution of Yellow sand (YS)

The coefficient of uniformity (C_u) and coefficient of curvature (C_c) were 3.16 and 1.23, respectively. The maximum and minimum void ratios are 0.423 (1.86g/cm^3) and 0.892 (1.40g/cm^3) according to ASTM D-4253 (2004b) and ASTM D-698 (2004a) methods, respectively. The clean sand is uniform and sub-rounded in shape. The liquid limit (LL) and plasticity index (PI) of the fines were found to be 40.0% and 9.3% respectively and so the fines material was classified as low plasticity silt (ML) according to the Unified Soil Classification System (USCS).

Specimen Preparation

There are several methods for specimen preparation has been found in the literature, namely, air pluviation, water sedimentation and moist tamping. Although none of these methods can simulate exact field conditions and soil fabric, each of these methods has different advantages in resembling certain field conditions and fabrics. The moist tamping method was thought to be most suitable for this study as it reduces segregation of fines from sand (Rahman, 2009, Baki, *et al.*, 2012) and relatively loose specimens can be prepared with very high void ratio, which is an important aspect for static liquefaction studies (Been and Jefferies, 1985, Ishihara, 1993). Moist tamping gives an initial soil fabric more representative of sandy deposits placed on land, such as embankment fill or colluvium (Rahman, *et al.*, 2014b). Therefore, a modified moist tamping method, as presented in Bobei, et al. (2009), was used to prepare soil specimens.

Oven dried natural silty sand mixed with a small amount of water (about 5% by dry mass) and cured for 24 hours in a controlled room temperature. Enlarged end platens with free ends, as described by Lo, *et al.* (1989), were used to minimize end restraint and ensure uniform deformation at high axial strains to almost 30% (Bobei, *et al.*, 2009, Lo, *et al.*, 2010). A predetermined quantity of moist soil was tamped in ten layers in a split mould of 100 mm in diameter and 100 mm in height; the thickness of each layer was controlled using a height controlled tamping rod. CO_2 was flushed through the specimen to replace the air inside the specimen. Saturation of the specimen was done by two stages; vacuum water flushing and back pressure saturation. A Skempton's pore pressure parameter B ≈ 0.98 was achieved for almost all the specimens.

Testing Apparatus

A newly commissioned computer controlled triaxial testing system was used in this study. Two automated pressure regulators were used to control the cell pressure and pore water pressure. Cell pressure and pore water pressure were also measured using pressure transducers mounted just outside the cell to cross-check the controlled pressure and to measure pressure in the specimen, independent of the control system. An Advanced Digital Pressure Volume Controller (DPVC) with 2 MPa pressure and 1000 cubic cm volume capacity was used to apply back pressure and to monitor or control volume change of the specimen. A Linear Variable Displacement Transducer (LVDT) was used to measure axial deformation. The ram movement was controlled by a force actuator driven by a stepper motor located at the bottom of the machine. Axial load was measured by a load cell located inside the cell, just above the top platen. The computer program for triaxial testing can control the ram movement by either strain control or stress control. The strain control mode was used for K_0-consolidation.

Control module for K_0-consolidation

A strain controlled triaxial system was used to achieve the K_0-consolidated soil specimen by applying constant axial strain to target an effective vertical stress, σ_1'. Firstly, a constant axial strain rate is applied producing an increase in the effective vertical stress, σ_1', up to the target value. During this vertical straining, zero radial strain is enforced by the DPVC in the manner described by Menzies (1988). At a user selected time step (default 6 seconds) the computer program calculates a volume, Δv equal to the axial deformation increment times the current cross sectional area of the specimen. The program then commands the DPVC to extract the volume Δv to maintain zero radial strain according to the following equation:

$$\varepsilon_v = \varepsilon_a + 2\varepsilon_r \quad \rightarrow \quad \varepsilon_r = 0 = \frac{(\varepsilon_v - \varepsilon_a)}{2} \quad \rightarrow \quad \varepsilon_a = \varepsilon_v \tag{1}$$

where, $\varepsilon_a, \varepsilon_r, \varepsilon_v$ are the axial strain, radial strain and volumetric strains respectively. This extraction of volume causes a subsequent reduction of Δu, to the pore pressure (back pressure). To avoid the potential problem that a lower pore pressure may cause partial desaturation, the program commands the cell pressure regulator to increase the cell pressure by the drop in pore pressure. Allowing that $\Delta u = B\Delta\sigma_3$, where B is Skempton's pore pressure parameter, and assuming that the specimen is nearly saturated ($B \approx 1$) , this will restore the back pressure to the desired level. If B is somewhat less than one, there may be a lag in restoration of the back pressure.

The simpler process of lowering of the pore pressure was used by Uchida, *et al.* (2003). At the same time, they used higher stress levels initially, starting with a back pressure of nearly

1500kPa. However, the equipment used in this study is not capable of achieving such a high back pressure and the accompanying cell pressure. The reduction of relatively small back pressures (400 kPa) as used in this study during the K_0-consolidation may risk partial de-saturation of the specimen. To avoid this, cell pressure was increased in this module to keep pore pressure constant.

Testing program

A series of isotropically consolidated undrained (CIU) tests with void ratios, e_0 ranges from 0.480 to 0.655 and p'_0 from 50 to 350 kPa were performed to investigate the undrained behaviour of silty sand. A series of K_0-consolidated undrained (K_0U) triaxial tests was carried out to see the effect of consolidation path on the subsequent undrained behaviour of silty sand. A total of 21 CIU tests and 14 K_0U tests were performed in this study. The following sections discuss the test results.

Test Results

Typical undrained behaviour of CIU specimens

Figure 3 (a) and (b) showed the stress-strain relationship and effective stress path (ESP) of CIU specimens prepared at an 'as-placed' void ratio, e_{ap} of 0.65 and they reached e_0 = 0.610±0.004 after consolidation. The initial mean effective stress, p'_0 varies from 100 to 350 kPa. All three specimens showed clear peak deviator stress, q at smaller strain and then follow rapid strain softening to a steady state deviator stress, q_{SS} at large strain. The peak q of these tests increased with increasing p'_0, however, they reached to a close SS strength, q_{SS}. These specimens showed 'flow' behaviour for the tested range of p'_0. The ESP of all three tests travel leftward and plummeting towards the origin until reached failure stress ratio, M =1.355. The η_{IS} for individual specimens shown in Figure 3 (b) is slightly different (varies from 0.728 to 0.820) and thus, may not be appropriate to present them by a single line, i.e. IL line (Chu and Leong, 2002). To enhance visibility near the origin, η_{IS} which is also the slope of IL, are represented by a line, which is often not shown connected to the origin. The IL of YS-CIU09 was extended with dotted line to show the variation with other two η_{IS}. Therefore, η_{IS} for individual tests may depend on their state (e, p'). This is in line with the findings of Rahman and Lo (2012).

Typical undrained behaviour of K_0U specimens

Figure 4 showed the undrained shearing behaviour of K_0-consolidated specimens for e_0 = 0.557 ± 0.004. After K_0-consolidation, strain-controlled undrained shearing was performed at an axial strain rate, the same as that of CIU tests. Figure 4 (a) showed that peak q reached quickly at a very small axial strain after the commencement of undrained shearing. After reaching peak q, the specimens exhibited strong strain softening behaviour and q decreased quickly with further strain. Steady state deviator stress, i.e. q_{SS} at large axial strain increased slightly with increasing p'_0. Similar to the CIU specimens, instability stress ratio, η_{IS} varies slightly and it may not be appropriate to draw as single IL. However, in general slight increasing tendency of η_{IS} was found with decreasing e_0 even though the variation of e_0 was small as shown in Figure 4 (b). It can also be observed that the η_{IS} ranges from 0.726 to 0.820 which is similar to that observed in the CIU specimens shown in Figure 3 (b) even though the void ratio for K_0U specimens were relatively smaller than that of the CIU specimens ~~ranges are different~~. Since the η_{IS} value decreases with increasing void ratio, the onset of instability would be triggerred early (i.e. at smaller values of η_{IS}) for K0U specimens with relatively higher e_0, i.e. similar void ratio ranges of CIU specimens. Therefore, η_{IS} for both CIU and K_0U specimens were plotted against their void ratios after consolidation, e_0 in Figure 5. It was observed that K_0U specimens showed similar instability stress ratio at significantly smaller void ratios than that of the CIU specimens and they followed a different trend in $\eta_{IS} - e_0$ plot, which is somewhat different than the data published by Chu and Wanatowski (2008). This also suggests that the triggering of instability/liquefaction is consolidation history dependent.

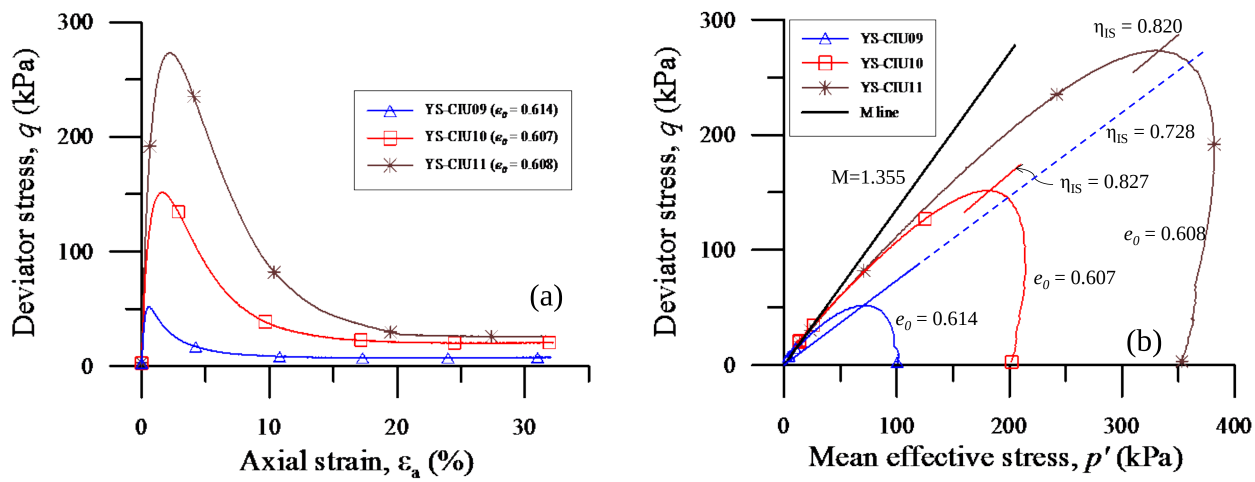

Figure 3: Undrained behaviour of CIU tests under varying p_0'; (a) stress-strain relationship, (b) mean effective stress.

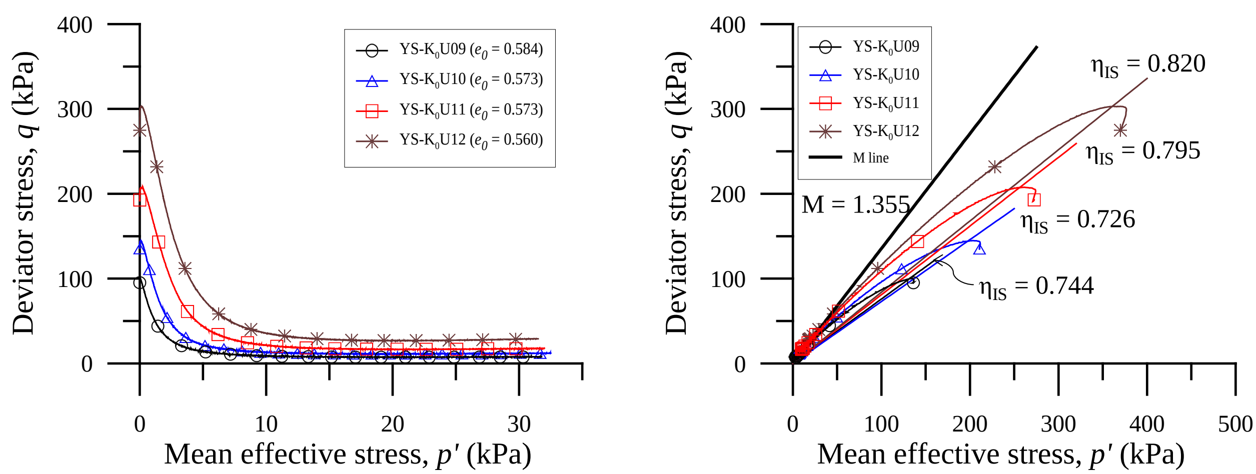

Figure 4: Undrained behaviour of K_0U tests under varying p_0'; (a) stress-strain relationship, (b) mean effective stress.

Conclusions

This paper examined the laboratory undrained instability behaviour of silty sand under different consolidation conditions. The major findings are:

- For the range of void ratio experienced in the experimental program, both isotropic and K_0 consolidated specimen showed flow type of behaviour. In both cases, instability triggered at the stress ratio, $\eta_{IS} = (q/p')_{IS}$ at peak deviatoric stress, q.
- Instability stress ratio, η_{IS} increased with decreasing void ratio, e_0 for isotropic and K_0 consolidated specimens. However, instability data points in $\eta_{IS} - e_0$ possessed different trends depending on consolidation history.
- For the same void ratio, e_0 would occur at a lower η_{IS} for K_0 consolidated specimen than that for an isotropically consolidated specimen.

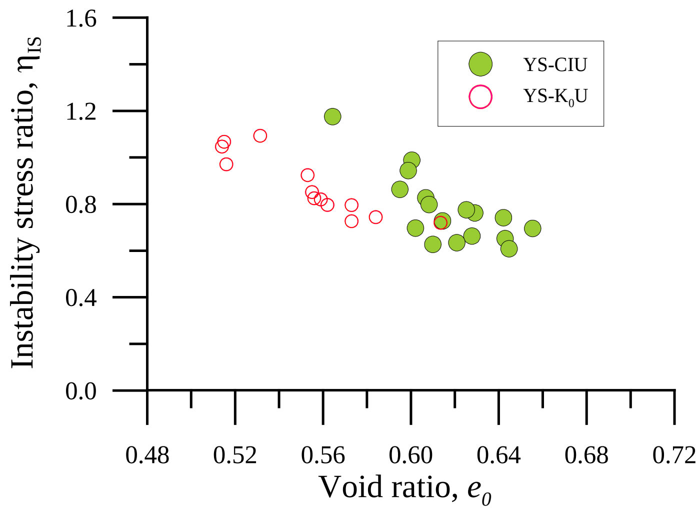

Figure 5: Relation between void ratio and instability stress ratio

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