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LIMITED FLOW BEHAVIOUR OF SAND WITH FINES UNDER MONOTONIC AND CYCLIC LOADING

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Experimental studies on a sand with 10% fines were conducted to establish the linkage between monotonic and cyclic instability under undrained loading, in particular for the case of limited flow. Results of special strain path testing were presented to offer a theoretical explanation on the lack of clear field evidence of limited flow.

INTRODUCTION

Significant progress has been made, since the 80s, in linking liquefaction to instability; and as such the strain softening behaviour under monotonic undrained loading provides the key for understanding cyclic liquefaction behaviour. Mohamad and Dorby (1986) reported that the monotonic behaviour of soil must be considered in analyzing the undrained cyclic behaviour of saturated sand. Georgiannou et al. (1991) demonstrated that the monotonic bounding envelope of Ham river sand as determined in undrained loading was also applicable in determining cyclic response. Konrad (1993) found that the undrained peak strength envelope in the effective stress pace could be used to define the triggering of strain softening in both monotonic and cyclic undrained loading. This envelope could be taken to be a unique line for samples at similar void ratio. Yamamuro and Covert (2001) confirmed that cyclic liquefaction in loose Nevada sand with 40% silt was triggered by “crossing” the instability line. Vaid and Sivathayalan (2000) reported that strain softening under undrained cyclic loading occurred at the instant when the mobilized friction angle attained the value that triggered strain softening under static loading. Gennaro et al. (2004) suggested that the response in undrained monotonic shearing contribute to the prediction of undrained response in cyclic loading.

Based on the above studies, the conceptual framework for linking liquefaction under cyclic loading to static liquefaction is presented in Figure 1. Unless stated otherwise to the contrary, shearing is performed under an undrained mode. In this framework liquefaction is a manifestation of instability and this is different from cyclic mobility. We can define an instability stress ratio, η_{IS}, by the effective stress state at peak undrained strength under monotonic loading. When the effective stress state crosses the line defined by η_{IS} as a result of pore water pressure generation due to either monotonic or cyclic loading, instability will be triggered. It is evident from Figure 1 that the stress pulse required to trigger instability under cyclic loading is less than that in monotonic loading. Furthermore, the effective stress path of monotonic loading defines the boundary of admissible stress state during cyclic loading.

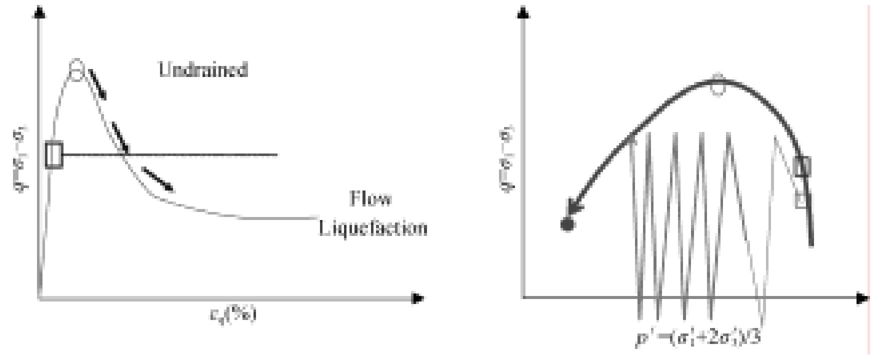


Figure 1. Instability under static and cyclic loading

However, experimental evidence directly verifying the above conceptual framework is relatively limited. There are also issues related to limited flow. First is how the above framework be applied to the case of limited flow. Second is whether limited flow is a real soil behaviour (Zhang and Garga, 1997). Although laboratory evidence indicates that limited flow can be real (Chu, 1999; Vaid et al., 1999a; Yoshimine, 1999), there has been a lack of clear field evidence on the existence of limited flow (Zhang and Garga, 1997). It is also recognized that most loose sandy soil have some fines, whereas experimental studies addressing the issue of limited flow has largely based on clean sand. The objective of the paper is to present experimental evidence that demonstrates the linkage between monotonic and cyclic instability in the case of limited flow, and explain via strain path testing the absence of clear field evidence of limited flow.

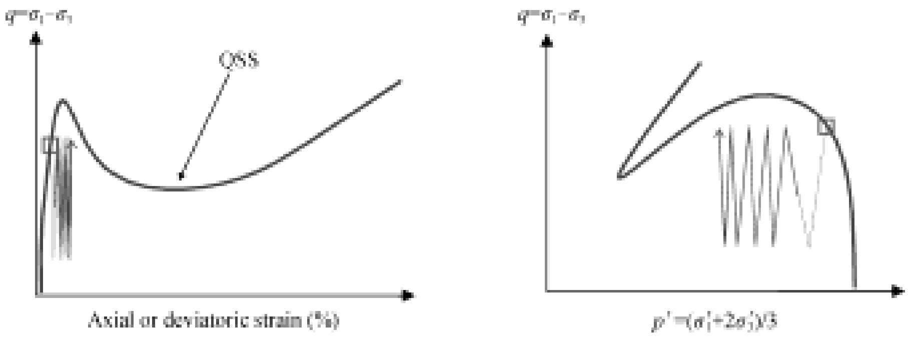


Figure 2. Limited flow

EXPERIMENTAL STUDY

Material tested

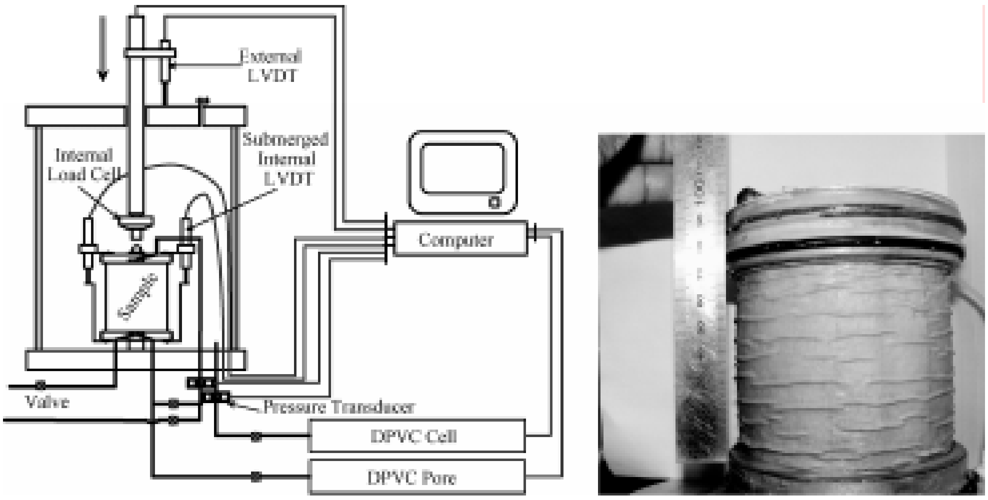
The material tested is a uniform quartz sand with 10% fines. The host sand is Sydney sand, a clean uniform size quartz sand (SP) with a mean size of 0.30mm and it's index properties can be found in Lo et al. (1989). The fines is a low plasticity fines (PI=11, LL=28) with a uniformity coefficient of 12.56. It is composed of 2/3 of well-graded silt from the Majura River and 1/3 commercial kaolin.

Specimen preparation

A specimen was formed by a modified moist tamping method. A pre-determined quantity of moist soil was carefully placed and then tamped lightly into a prescribed thickness using a standardized plastic strip with a tamping area of 8.5mm×20 mm. A total of 10 layers were placed in forming a specimen with a dimension of 100mm in both diameter and height. Free ends with enlarged platens were used to minimize end restraint. This technique has been proven to be successful in achieving essentially uniform deformation for a range of soil type (Chu et al., 1993; Lo and Wardani, 2002; Lo and Chen, 1999). Bedding and membrane penetration errors were reduced to an insignificant value by using the liquid rubber technique developed by Lo et al. (1989). Saturation was achieved by vacuum flushing with a low head followed by back pressure saturation. Details of the specimen preparation method are contained in Bobei (2005).

Experiment procedures

A schematic diagram of the experimental setup is shown in Figure 3(a). A triaxial testing system with PC-controlled data logging and stress/strain control capabilities was used for this study. Axial load was measured with an internal load cell. The axial deformation was measured with a pair of internal LVDTs mounted directly across the top platen plus an external LVDT. The former was used in the early stage of shearing whereas the latter was used at large deformation. Cell pressure was controlled by a large capacity Digital Pressure Volume Controller (DPVC). The pore pressure line was connected to a small capacity DPVC which served three purposes: i) controlling back pressure and measurement of volume change in drained stage, ii) ensuring nil volume change and measuring pore water generation in undrained shearing, and iii) controlling the ratio $d\varepsilon_v/d\varepsilon_q$ in a strain path test. The concept and implementation of a strain path test will be discussed at a later section. Pressure transducers were mounted at both the top and bottom platens to verify pore pressure equilibrium.



(a) Schematic diagram of experimental setup (b) Photo of specimen after loaded to an axial strain > 35%

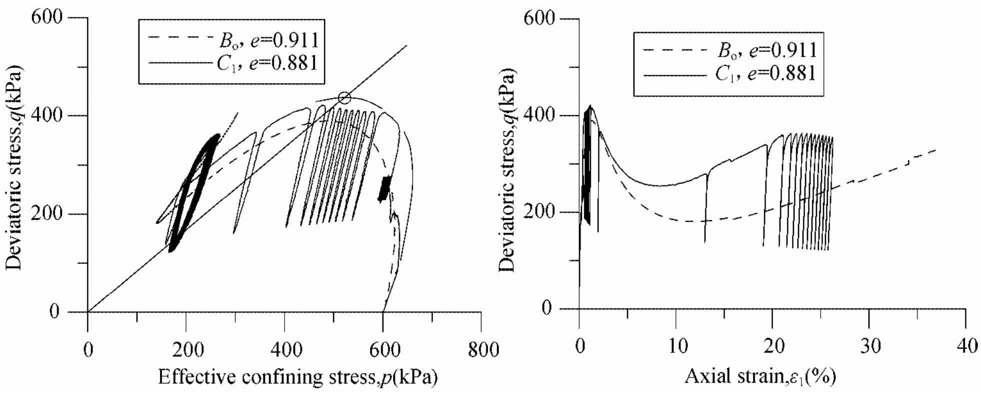
Figure 3. Experimental methodology

COMPARISON BETWEEN MONOTONIC AND CYCLIC INSTABILITY

Monotonic loading and limited flow

A monotonic loading test that clearly manifested limited flow was used as the benchmark for comparison. This test, denoted as B_o, was isotropically consolidated at an effective confining stress pressure of 600kPa from which monotonic shearing began. Its behaviour in undrained shearing was shown as dotted line in Figures 4a-b. As evident from Figure 4a, the resultant effective stress path (ESP) clearly showed limited flow behaviour. However, the ESP did not showed a sharp peak and η_{IS} was in the range of 0.75 to 0.81.

A distinct peak was, however, manifested in the stress strain curve of Figure 4b. The peak deviator stress, q_{peak}, of 390 kPa was mobilized at ~3% axial strain and the specimen strain softened to a q_{QSS} of 190 kPa, where subscript "QSS" denotes quasi-steady state. Although the minimum deviator stress point occurred at an axial strain of ~10%, clear strain hardening resumed after axial strain exceeded 15%. This means unless the deformation of the specimen remains essentially uniform deformation when sheared well beyond 15%, QSS cannot be clearly differentiated from SS. This specimen, as shown in Figure 3b, did not showed any sign of platen restraint even after the specimen was loaded to an axial strain of ~35% and therefore limited flow and QSS could be reliably confirmed.

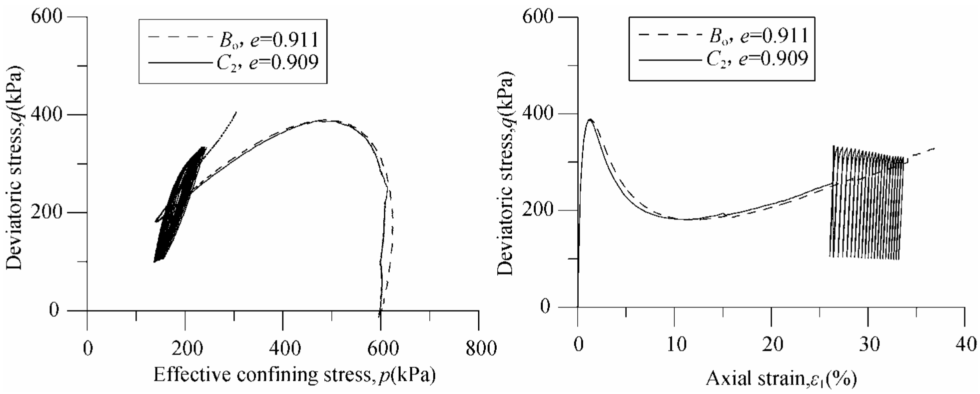


(a) p'-q response (b) q-ε_1 response

Figure 4. Comparison of tests B_o & C_1

Cyclic loading and limited flow

Two one-way cyclic loading tests were conducted in a load-controlled mode. For both tests, the specimen was first brought to a non-zero deviator stress via undrained monotonic loading prior to the application of cyclic loading. For the first test C_1, cyclic loading commenced prior to attaining q_{peak}. For the second test C_2, the specimen was sheared beyond QSS prior to application of cyclic loading. The consolidated states of both C_1 and C_2 are closed to that of B_0 (in monotonic loading) so thus a meaningful comparison can be made.



(a) p'-q response (b) q-ε_1 response

Figure 5. Comparison of tests B_o & C_1

Test C1: This test had two stages of cyclic loading and the test results are shown as solid lines in Figures 4a-b. The first stage of cyclic loading was conducted with the deviator stress, q, cycling between 220 to 270 kPa. It is pertinent to mentioned that ESP manifested in bring the specimen from the isotropic stress state to q = 270 kPa essentially followed that of B_0, thus confirming that the responses of these two tests could be compared meaningfully. During the first stage of cycling, the ESP moved slightly to the left (ie increase in p') with load repetition. There was also minimal accumulation of permanent strain. Therefore, the specimen showed no sign of approaching instability. This specimen was then brought in undrained mode to a high deviator stress and cyclied between a cycle-trough of 190 kPa to a cycle-peak of 410 kPa. Despite the impose cycle- peak of 410 kPa was higher than q_{peak} of 390 kPa (for B_0 in monotonic loading), the corresponding stress ratio η @ cycle-peak was less than η_{IS} because p' at cycle-peak was higher. The higher p' (due to lower pore water generation) was most likely to be due to the fact that C_2 had a lower void ratio. Thus a "projected" ESP for monotonic loading at an "identical" void ratio was shown as dashed line in Figure 4a. This projected ESP has η_{IS} of 0.81, which is the upper bound estimation of η_{IS} for B_0. During the first 7 load cycles, the ESP moved gradually to the left and the shift per cycle is about the same. However, the leftward shift of the ESP increased for the 8$^{\text{th}}$ cycle. It is noted that η_{IS} was exceeded in the 8$^{\text{th}}$ cycle. For the 8$^{\text{th}}$ to 10$^{\text{th}}$ load cycles, the ESP shifted left considerably. Furthermore, the maximum deviator stress cannot reach the prescribed cycle-peak of 410 kPa. Instead, it traced a downward path that followed closely, but located above, the downward (instability) segment of the resultant ESP in monotonic loading. From the 11$^{\text{th}}$ cycle onwards, the ESP traced a closed and stable loop and with the cyclic-peak located approximately on the post-QSS segment of B_0. A similar behaviour was also manifested in the stress-strain plot of Figure 4b, the cycle-peak traced a stress-strain curve that showed strain softening to a minimum deviator stress followed by strain hardening. However, the stress strain curve formed by the cycle-peak of cyclic loading was located considerably above the corresponding monotonic response of B_0.

Test C_2: This test had a void ratio very close to that of B_0. As cyclic loading was applied only after the specimen had been sheared to beyond QSS, the equivalence of these two specimen was assured. The test results of C_2 are shown as solid lines in Figures 5a-b. The imposed cyclic loading was from a cycle-trough of 105 kPa to a cycle-peak of 335 ka. The ESP during cyclic loading manifested a close loop with minimal shifting of location with load cycles. Furthermore, the stress state of cycle-peak was close to the last (upward) segment of the ESP of B_0. However, the stress strain curve of C_2 during cyclic loading was different from the monotonic curve of B_0. Initially, the cycle-peak was located well above the monotonic stress strain curve of B_0. With load repetition, it eventually merged back to the monotonic stress strain curve of B_0. Whether this difference is due to a load rate effect or due to some other mechanisms is unclear.

IN-SITU LIQUEFACTION MECHANISM

The occurrence of limited flow behaviour has been well established in element testing, but why this has not been clearly observed under field scale needs investigation. The so-called "Mechanism C" proposed by NCR (1985), in conjunction with behaviour under strain path testing, offers an explanation. The principle of "Mechanism C" is illustrated in Figure 6. Liquefaction in the field is not element behaviour as the in-situ stress states, say within a slope, are not uniform. Liquefaction initiates within the slope. This will leads to void ratio and pore water pressure re-distribution, with the soil above the initially liquefied zone having increase in void ratio. As this re-distribution is a continuous process, it can be represented by shearing under the condition of $d\varepsilon_v/d\varepsilon_q < 0$.

Independent of the NRC report, Lo and co-worker initiates an experimental program of studying the behaviour of sandy soil in strain path testing (Chu and Lo 1991). In essence, strain path testing is shearing along a constant strain increment ratio path, with the strain increment ratio, $d\varepsilon_v/d\varepsilon_q$, being controlled to a prescribed value. This is achieved by having the DPVC connected to the pore water line controlled by special software developed in-house. Strain path control can be activated at any stage, and the strain increment ratio can be change during shearing. Undrained shearing is a special form of strain path testing with $d\varepsilon_v/d\varepsilon_q=0$. The re-distribution of void ratio can be simulated as a process by strain path testing.

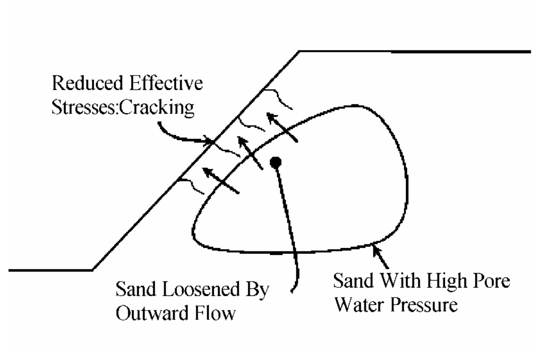


Figure 6. "Mechanism C" proposed by NCR (1985)

An undrained test B_1 was compared to a strain path tests SP in Figure 7. The strain path test was conducted in 4 stages, therefore denoted as SP_1 to SP_4, and each stage was prescribed a different $d\varepsilon_v/d\varepsilon_q$ value. SP_1 was conducted with $d\varepsilon_v/d\varepsilon_q = 0$ and should showed a response identical to B_1. As shown in Figure 7, B_1 manifested a limited flow response, with QSS attained at ~10% axial strain and strain-hardening re-commenced clearly at an axial strain of 15%.

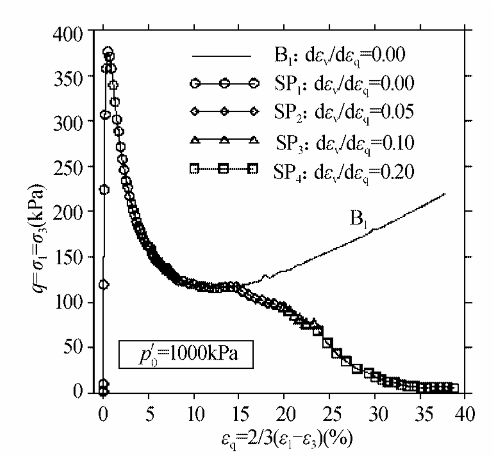


Figure 7. Simulation of void ratio re-distribution by strain path testing

Since the responses of B_1 and SP_1 are essentially identical over a strain range of ~15%, the two specimens can be considered as replicate. Strain path control was activated on test SP at an axial strain of ~15% where the behaviour of clearly returned to strain hardening. A slight dilative strain path control defined by $d\varepsilon_v/d\varepsilon_q = -0.05$ was prescribed. This simulates a slight void ratio redistribution. As evident from Figure 7, the behaviour of SP_2 for $d\varepsilon_v/d\varepsilon_q = -0.05$ was strain softening! By any means, $d\varepsilon_v/d\varepsilon_q = -0.05$ is very small and more significant void ratio re-distribution can occur. Therefore, the prescribed value of $d\varepsilon_v/d\varepsilon_q$ was changed to -0.10 (for SP_3) and then -0.20 (for SP_4). As he test result showed that the more dilative the prescribed strain increment ratio, the more significant is the strain softening. This means, for the soil above the initially liquefied zone, the occurrence of a slight void ratio re-distribution will change the behaviour from limited flow for undrained shearing to that of strain softening. This change will be more significant for higher void ratio re-redistribution. This offers a plausible and theoretically sound explanation on why limited flow has not been clearly observed in the field.

CONCLUSION

This paper synthesizes some of the test results on the instability behaviour of sand with a small amount of low plasticity fines. The findings are:

- The effective stress conditions for triggering strain softening in cyclic and monotonic undrained loading is essentially identical: an effective stress ratio exceeding η_{IS}. This criterion also applies to the strain softening segment of limited flow.
- Once instability commenced, the cycle-peak appears to trace along the effective stress path of monotonic undrained loading. This applies to both the pre-QSS strain softening segment and the post-QSS strain hardening segment of the response, and irrespective of whether cyclic loading commenced from a pres-QSS state or a post-QSS state.
- "Mechanism C" as proposed by NCR (1985) can be simulated by strain path testing. Strain path tests that simulate a slight void ratio re-distribution in the soil above the

initially liquefied zone will turn a limited flow behaviour (for undrained mode) into continued strain softening. This offers a plausible and theoretically sound explanation on why limited flow has not been observed in the field.

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