Is conventional laboratory testing a reflection of field behaviour?

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Keywords: liquefaction, sand, fines, stress path, strain path

ABSTRACT

Recent concerns have been raised on the suitability of laboratory measured limited flow response of sands, as no similar field occurrences have been observed to date. An experimental approach has been adopted in this study with the main purpose of exploring a possible rational avenue for explaining such discrepancies. The laboratory testing has been carried out using state of the art equipment and testing techniques, on a subject material consisting of a composite mix of sand with a small amount of fines. The results indicated that in conventional triaxial testing, the undrained (i.e. $d\varepsilon_v = 0$) static response was consistently observed as a fundamental soil behaviour. However, in some tests where the $d\varepsilon_v/d\varepsilon_q$ ratio was controlled during loading, the experimental results showed no occurrence of the limited flow behaviour. On such premises, it is emphasised the need for a practising engineer to employ careful judgement when interpreting the conventional test results for infrastructure designs.

1 INTRODUCTION

It has become nowadays a usual practice to conduct detailed geotechnical investigations for careful planning of any infrastructure development. Although sometimes the importance of the development as well as budget limitations dictate the detail of geotechnical investigations, these will traditionally consist of a combination of field investigations and laboratory testing. Based on these investigations, the geotechnical engineer will make an assessment as to the parameters to adopt in analyses, as well as the constitutive models that are best suited for each particular problem. However this judgement is not always a straightforward exercise, especially when dealing with the liquefaction response of granular materials. In conventional laboratory tests, liquefaction is usually associated with a soil response in undrained conditions by restricting the volumetric changes (i.e. $d\varepsilon_v = 0$) during shearing. A large amount of work has been conducted in this fashion by Vaid et al (1990), Ishihara (1993), Lade and Yamamuro (1997), Bobei and Lo (2001) and variations in void ratio and confining pressure have generally been found to lead to flow, limited flow or non-flow types of behaviour as schematically illustrated in Figure 1. In spite of such laboratory observations, Zhang and Garga (1997) point out that whilst flow and non-flow type responses are likely field occurrences, the behaviour of limited flow behaviour has to their knowledge not been encountered or observed in sand deposits after seismic events. Consequently, the limited flow response where a decrease in the shear strength is noted to be a temporary state, the Quasi-Steady

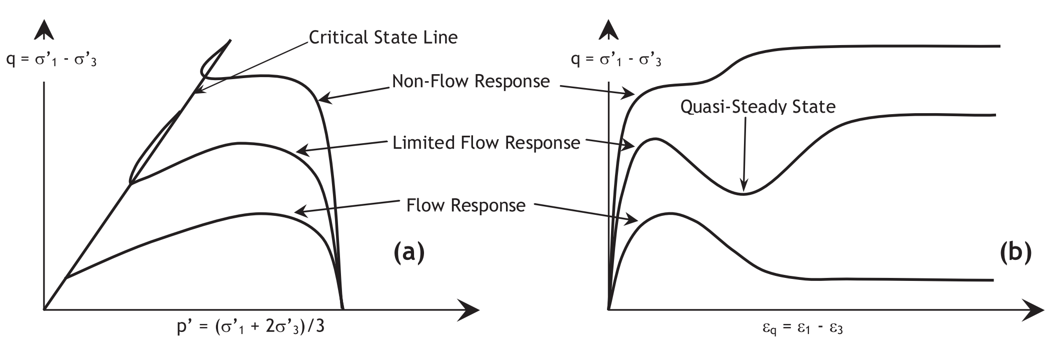

Figure 1 Typical undrained behaviour of granular materials: (a) Stress paths (b) Stress-strain curves.

State (QSS), followed by an increase in strength with further deformation, was suggested to be an artefact of laboratory testing caused primarily by experimental errors, limitations and procedures.

The intent of the paper is to examine the presence/absence of limited flow response in controlled laboratory conditions. An extension of these observations to field situations and implications for engineering practice are also discussed.

2 MATERIALS AND TESTING TECHNIQUES

2.1 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 a commercial medium plasticity kaolin and 2/3 a low plasticity silt. The index properties of these materials are presented in Table 1.

Table 1 Basic Soil Properties

Soil Material	Sand	Clay	Silt
Specific Gravity	2.63	2.42	2.49
Mean Diameter, D_{50} (mm)	0.25	-	-
Coefficient of Uniformity, U_c	1.5	-	-
Maximum Void Ratio, e_{max}	0.565	-	-
Minimum Void Ratio, e_{min}	0.855	-	-
Liquid Limit, LL (%)	-	63	28
Plastic Limit, PL (%)	-	32	17
Index of Plasticity, IP (%)	-	31	11

2.2 Testing Arrangement

The experimental work was conducted using a state of the art triaxial system in which the stress and strain parameters as well as the data logging were controlled by an in-house designed computer software. The soil specimens were axially loaded using a digital force actuator programmed to operate in deformation controlled mode. For accurate measurements, the axial load and displacements were measured inside the triaxial chamber by submersible load cell and a pair of LVDTs (Linear Variable Differential Transducers) mounted directly on the top soil specimen cap. The control of the cell pressure was achieved by a DPVC (Digital Pressure Volume Controller), and the monitoring/control of the pore water pressure/volumetric change was achieved by a second DPVC.

2.3 Preparation of Soil Specimens

The method of moist tamping was used to prepare homogeneous and uniform loose soil specimens. A total of ten (10) layers of pre-determined quantities of moist soil were compacted in prescribed thicknesses using a standardised plastic strip with a tamping area of 8.5 X 20 mm. The dimensions of as placed soil specimens were 100mm in both diameter and height. The preparation technique also included:

(a) The use of free ends (i.e. lubricated latex discs) with enlarged platens to minimise the end restraints and development of non-homogeneous deformations; and
(b) Liquid rubber technique as developed by Lo et al. (1989) to reduce the bedding errors and membrane penetration to insignificant levels.

The soil specimen water saturation was accomplished in two stages as:

(a) Air replacement by carbon dioxide followed by water displacement under a double vacuum flushing system with a small (i.e. 300mm) and constant water head; and

(b) Application of minimum 300 kPa back-pressure for a high degree of saturation corresponding to a B-value of 98%.

In all saturation stages, the specimen was maintained at a constant effective confining pressure of 20 kPa, so that isotropic compression always started from a consistent state with a void ratio between 0.830 and 0.850.

3 NOTATIONS

The notations adopted in this study are defined as follows:

(a) Deviatoric stress $q = \sigma'_1 - \sigma'_3$;
(b) Mean effective stress $p' = (\sigma'_1 + 2\,\sigma'_3)/3$;
(c) Deviatoric strain $\varepsilon_q = \varepsilon_1 - \varepsilon_3$;
(d) Volumetric strain $\varepsilon_v = \varepsilon_1 + 2\,\varepsilon_3$;
(e) Axial and radial principal effective stresses σ'_1 and σ'_3 respectively;
(f) Axial and radial principal strains ε_1 and ε_3 respectively; and
(g) Increment in effective stress and strain $d\sigma'$ and $d\varepsilon$ respectively.

4 CONVENTIONAL UNDRAINED TESTING

Undrained effective stress paths and corresponding stress-strain curves from three (3) tests labelled A_1 to A_3 are illustrated on Figure 2. The undrained tests commenced from isotropic confining pressures in the range of 400 to 1115 kPa. With the increase in confining pressure, it was noted a well defined transition from flow to limited flow. This behavioural trend observed in sand with fines was termed 'reverse behaviour' by Lade and Yamamuro (1997). Further analyses and discussions of these results in the framework of critical state soil mechanics are detailed in Bobei and Lo (2005).

5 STRAIN PATH TESTING

In spite of minimising the experimental errors, the undrained results suggest that limited flow behaviour is a fundamental behaviour of granular materials. Following on from these findings, the research program pursued an alternative approach where the liquefaction response was studied using the technique of strain path testing as detailed in Chu and Lo. (1993) and Vaid and Eliadorani (1998). Limited attention has been given to such laboratory testing, mainly due to experimental difficulties when controlling the rate of $d\varepsilon_v/d\varepsilon_q$ during shearing.

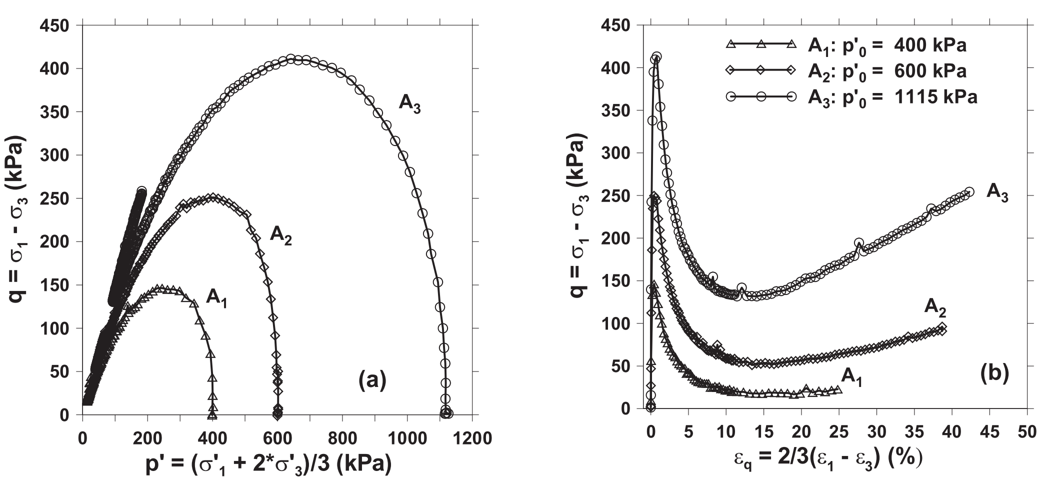

Figure 2 Undrained response for a range of confining pressures between 400 to 1115 kPa: (a) Effective stress paths (b) Stress-strain curves.

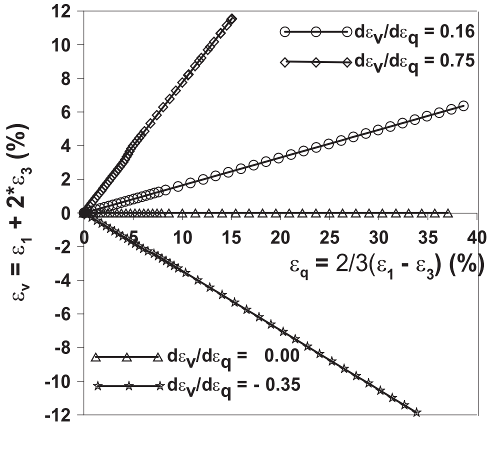

Figure 3 Comparison of measured and targeted strain paths.

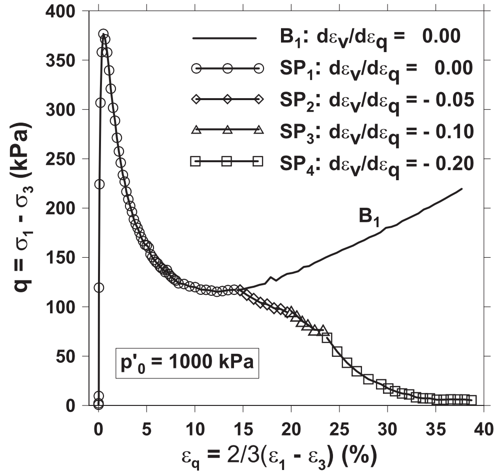

Figure 4 Stress-strain curves in multi-linear strain path segments.

Bobei and Lo (2003) proposed to classify the strain path tests as follows:

(a) Positive strain paths: $d\varepsilon_v/d\varepsilon_q > 0$ where during shearing the pore water is allowed to flow out of the soil element; and
(b) Negative strain paths: $d\varepsilon_v/d\varepsilon_q < 0$ where during shearing the pore water is allowed to flow into the soil element due to soil dilatancy. Such tests were conducted on soil specimens that were saturated under elevated back-pressures (i.e. of minimum 600 kPa) to prevent occurrence of water cavitation during shearing due to reduction in pore water pressure.

It also follows from above, that undrained behaviour may be simply regarded as a special case of strain path testing when $d\varepsilon_v/d\varepsilon_q = 0$.

An adequate control of the strain path was achieved during shearing, and a range of theoretical vs. recorded responses along positive and negative constant strain paths are illustrated on Figure 3.

Within the above framework, the limited flow response was studied along multi-linear strain paths where the strain increment ratio $d\varepsilon_v/d\varepsilon_q$ was gradually decreased as illustrated on Figure 4. The shearing of the soil specimen commenced initially from a confining pressure of 1000kPa in an undrained condition (i.e. a state where limited flow would be observed during shearing as shown on Figure 4 for a replicate tests B_1). After the onset of Quasi-Steady State, the loading condition was changed to proceed along a strain increment of $d\varepsilon_v/d\varepsilon_q = -0.05$. For such a small departure from an

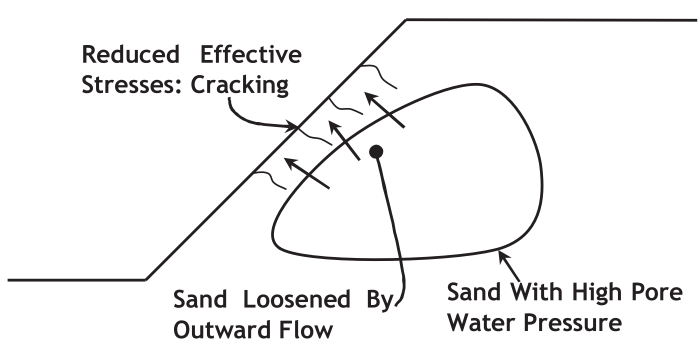

Figure 5 Mechanism C by NCR (1985): Example of a potential situation for failure by outward flow of excess pore pressure with global volume changes.

undrained condition, $d\varepsilon_v/d\varepsilon_q = 0$, the deviatoric stress ceased to increase and decreased to a steady value. While further strain increment reductions (i.e. $d\varepsilon_v/d\varepsilon_q = -0.10$ and $d\varepsilon_v/d\varepsilon_q = -0.20$) had a similar effect of reducing the deviatoric stress, the larger the $d\varepsilon_v/d\varepsilon_q$ rate the larger the reduction in deviatoric stress. A complete reduction in deviatoric stress was observed along $d\varepsilon_v/d\varepsilon_q = -0.20$ at large deformations.

6 DISCUSSION OF THE RESULTS

The experimental results presented in this paper were aimed at reflecting the conflicting and difficult task of evaluating the liquefaction behaviour of silty sand materials. However, whilst the laboratory test results would explain the occurrence/non-occurrence of Quasi-Steady State as a stress or strain path soil dependent behaviour, a basic question still remains on why on a larger in-situ scale the Quasi-Steady State may not observed after seismic events. In this regard, it is interesting to note a concept of ‘void ratio re-distribution’ following an earthquake loading as proposed by NRC 1985, and for the purpose of this discussion Mechanism C was schematically reproduced in Figure 5. Although the concept is not specifically accounted for in the engineering practice, it indicates the possibility of a secondary effect. As such, assuming that during the seismic motion the sand liquefies in a zone inside the slope embankment, when shaking stops the excess pore water pressures is considered to dissipate under an upward hydraulic gradient in the upper portion of the slope. When the upward water seepage occurs, this is expected to have different effects inside the slope such as:

(a) A reduction in void ratio (i.e. $d\varepsilon_v/d\varepsilon_q > 0$) of sand in the lower parts of the liquefied zone due to a reduction in excess pore water pressure; and
(b) An opposite effect of increase in void ratio (i.e. $d\varepsilon_v/d\varepsilon_q < 0$) in upper parts of the liquefied zone due to an additional water inflow.

The multi-linear strain path test results illustrate the importance of considering the physical mechanism of void ratio re-distribution. As such and for the sake of this argument, it is considered that during the earthquake the sand manifests a limited flow type and the undrained shear resistance at Quasi-Steady State is sufficiently large to maintain the slope in a stable condition. When the shaking stops, a small amount of water inflow (i.e. small $d\varepsilon_v/d\varepsilon_q > 0$ change) was noted experimentally to reduce the sand shear resistance below that reached at the Quasi-Steady State. This reduction in the shear strength could possibly drive instability in the upper parts of the slope embankment and a complete loss of shear resistance should be considered as an extreme case.

7 COMMENTS

From a field observer point of view, the potential for void ratio redistribution is not easily verified but rather inferred to occur. This together with the limited amount of research on this topic, can lead to doubts on whether the laboratory determined Quasi-Steady State behaviour is a likely field occurrence. Finally, it is hoped the paper will foster further thought on the subject which could lead to sound engineering design guidelines that account for the void ratio redistribution effects.

8 CONCLUSIONS

An evaluation of liquefaction behaviour of sand with a small percentage of fines by employing conventional and non-conventional experimental techniques was presented. Despite disagreement in the profession, the limited flow in conventional testing was not found to be an artefact of laboratory testing due to experimental errors. As an alternative approach, limited flow was studied along non-conventional strain path tests. The strain increment (i.e. $d\varepsilon_v/d\varepsilon_q$) was decreased in stages and the deviatoric stress was found to reduce with loading as opposed to the continuous increase usually recorded in conventional testing. The experimental findings were interpreted to explain some salient features behind the void ratio re-distribution of Mechanism C by NRC.

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