E3S Web of Conferences **92**, 08009 (2019) https://doi.org/10.1051/e3sconf/20199208009
IS-Glasgow 2019

Cyclic resistance of a decomposed granite

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Abstract. An important issue surrounding the identification of liquefaction susceptibility using laboratory testing is how well the soil sample being tested represents the soil in the field. Undisturbed samples are difficult and costly to obtain, while reconstituted soil samples must have a structure and fabric that represents in situ conditions as closely as possible. Recent laboratory tests on sand samples revealed that liquefaction resistance is strongly affected by the sample preparation technique, as different techniques result in different fabrics and structures. The same may be true for silty sand samples, although they have not been given the same research attention. Thus, this paper presents cyclic triaxial test results on non-plastic silty sand samples (a decomposed granite) and determines the number of cycles required to cause liquefaction, considering different failure criterion as well as different confining pressures, cyclic strength ratios and sample preparation techniques. The techniques include dry and slurry deposition. The experimental results show that the preparation technique does not have a significant influence on cyclic resistance.

1 Introduction

Liquefaction is a problem of great significance to geotechnical engineering. Cyclic loading of a soil, for example caused by an earthquake, may cause the pore water pressure to build up to an extent that the soil weakens, becomes unstable and liquefies.

Liquefaction is widely studied with the assistance of cyclic triaxial tests on small soil samples in a laboratory. Within the laboratory studies there are multiple ways that liquefaction failure can be defined and the resistance to liquefaction quantified. For example, liquefaction failure occurs when the ratio between the pore water pressure and the initial total pressure, denoted:

$$Ru = \frac{\Delta u}{\sigma_c} \tag{1}$$

becomes equal to 1. *Ru*=1 corresponds to zero effective stress and the sample becoming fluidised. The number of cycles required to cause *Ru*=1 is a commonly used measure for liquefaction resistance [1]. Alternatively, liquefaction failure occurs when a specified value of strain amplitude [2] is reached, with the value chosen to indicate the commencement of a sudden pore pressure increase. The number of cycles required to reach the strain amplitude is a measure of liquefaction resistance. Further, liquefaction failure occurs when an unusual behaviour during cyclic loading becomes evident [3].

The liquefaction resistance in the cyclic triaxial tests depends on many factors. A major one is the cyclic shear stress applied to the samples, defined using the Cyclic Stress Ratio (CSR):

$$CSR = \frac{\sigma_d}{2\sigma'_0} \tag{2}$$

where σ_d denotes deviatoric stress (kPa) and σ'_0 denotes initial confining stress (kPa).

The fabric of the soil, where the fabric is influenced by the sample preparation method, may also have an influence on liquefaction resistance [4]. Yang and Sze [5] conducted a comprehensive investigation on sand involving many combinations of density, non-symmetrical cyclic loading and initial shear stress. They also investigated the impact of sample preparation on different failure modes using the same soil [3]. Flow-type failure, cyclic mobility and plastic strain accumulation were observed in samples prepared by moist tamping and dry deposition. However, limited flow followed by cyclic mobility, and limited flow followed by strain accumulation, were observed in samples prepared using dry deposition. The samples formed by dry deposition were more sensitive to the degree of stress reversal as the particle packing tended to be more anisotropic than that for samples prepared by moist tamping. Høeg et al [6] compared undrained shear-strength behaviours of undisturbed and reconstituted samples of a natural silt and a tailings prepared using different methods. Their findings revealed that samples prepared by water pluviation and slurry deposition had similar fabrics to the undisturbed samples. Thevanayagam [7] observed the micromechanical effects of fines on the liquefaction potential, stress-strain behaviour and fragility of a silt-sand mixture. The study adopted intergranular, interfine and equivalent interfine void ratios as indicators which control the contact density between particles and thus the tendency for liquefaction. Xenaki and Athanasopoulos [8] observed the effect of non-plastic fines content on the liquefaction resistance of saturated silty sands. They defined a fines content threshold (FC_{th}) 44%. At the same intergranular void ratio, when the fines content is less than FC_{th}, an increase in fines content

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increases liquefaction resistance. When the fines content is greater than FC_{th}, liquefaction resistance decreases with increasing values of fines content. Chang et al. [9] compared behaviours of undisturbed and reconstituted gold tailings and assessed the effectiveness in replicating an undisturbed sample in the laboratory. Their work revealed that slurry deposition provides a similar fabric to an undisturbed sample. It also revealed that moist tamping produced a flocked fabric consisting of platy particles unlike the undisturbed sample.

Despite these studies, it is not clear how the sample preparation technique influences the liquefaction resistance when defined using the alternate definitions mentioned above. This study aims to investigate the effect of sample preparation on the liquefaction resistance for the various definitions.

2 Experimental Program

2.1 Materials and equipment used

The material used in the experimental program was Lyell silty sand, a decomposed granite taken from the catchment area of Lyell Dam, NSW, Australia. It is non-plastic, well graded and classified as SM according to Unified Soil Classification System (USCS). Void ratio maximum (e_{max}) and void ratio minimum (e_{min}) are 0.69 and 0.26 respectively. Index properties are listed in Table 1. The particle size distribution is shown in Figure 1.

Table 1. Index properties of Lyell silty sand

Property	Plasticity Index	Specific Gravity	Sand (%)	Silt (%)	Clay (%)
Value	N/A	2.6	72	19	9

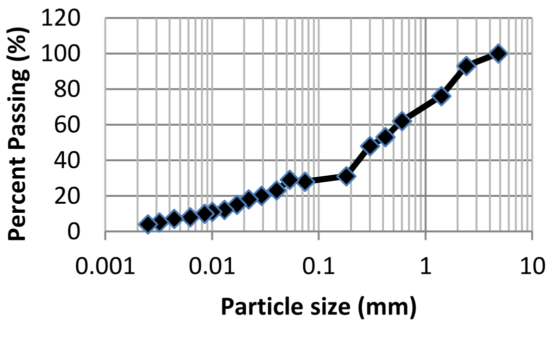

Fig. 1. Particle size distribution of Lyell silty sand

A number of cyclic triaxial tests were performed (Table 2). Soil samples were initially isotropically consolidated to 50 kPa or 95 kPa, resulting in the void ratios listed in Table 2. The cyclic loading was performed at 0.1 Hz using a range of CSRs.

Table 2. Undrained cyclic triaxial tests performed at 0.1 Hz

Deposition Method	Consolidation Pressure (σ'_c) - kPa	CSR	Void ratio
Dry Deposition	95	0.075	0.410
		0.1	0.411
		0.125	0.409
		0.15	0.409
	50	0.075	0.413
		0.1	0.412
		0.125	0.414
		0.15	0.419
Slurry Deposition	95	0.075	0.411
		0.1	0.413
		0.125	0.413
		0.175	0.415
	50	0.075	0.413
		0.125	0.41
		0.15	0.416

2.2 Methods of sample preparation

To prepare samples the silty sand was deposited in five layers in a 50 mm diameter of 100 mm high mould using two methods: slurry deposition (SD) and dry deposition (DP).

Slurry deposition involved mixing dry soil with de-aired water (38% by mass). The slurry was then deposited into the mould using a spoon, dropping it in to place from a minimum height. Trials were conducted to determine the weight of the slurry required to produce a 20 mm high layer of settled soil. After a layer had settled the excess bleed water on the top was removed using a syringe. A collar was attached to the top of the mould for deposition of the top layer. This procedure resulted in a void ratio of 0.42 (D_r=0.62) and water content of 13% being achieved. The process took around 4 hours to complete. Upon settlement of the top layer the soil was levelled with a spatula, then covered by plastic cling film, then placed in a freezer for 10 hours. The frozen sample was then removed from the mould and placed on the base pedestal of the triaxial apparatus, covered with a membrane, connected to the top cap and sealed using o-rings. A 10 kPa suction was then applied to avoid any cell installation error during subsequent test set-up. Thawing of the frozen sample was conducted under 20 kPa of cell pressure for four hours. A non-uniform stresses distribution may develop during thawing although is relatively small when the sample is subjected to a small hydrostatic stresses [10]. The volume change measured during the thawing process is around 0.1% and any alterations to this non-plastic soil's fabric are assumed negligible. The first B-value reading was always around 0.5. Back pressure was increased gradually until the B-value reached 0.95. Figure 2 presents duplicate test results demonstrating repeatability of the slurry deposition method.

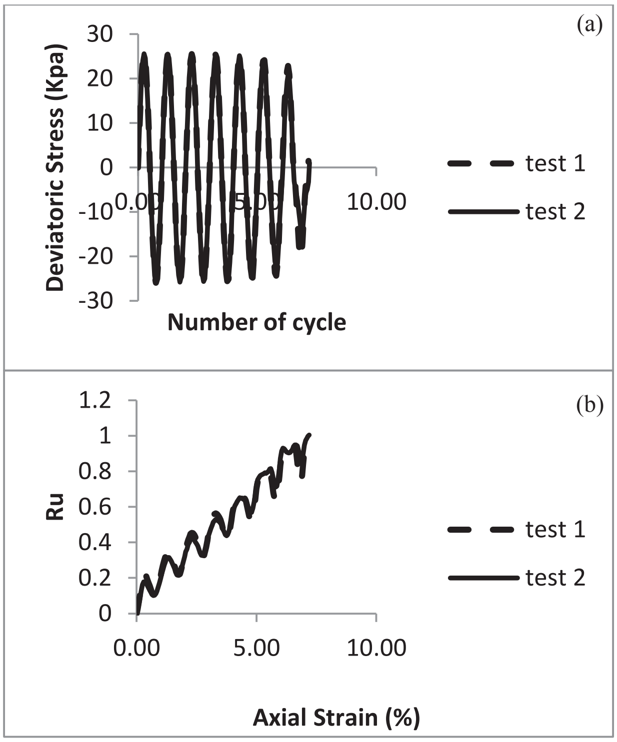

Fig. 2. (a) and (b) Repeatability of SD placement

Dry deposition was used to prepare samples in the mould directly on the triaxial base pedestal. The drop height was kept at zero using a funnel that could be gradually lifted as soil passed through it. To achieve a target density, the same as samples prepared by slurry deposition, a predetermined weight of soil was poured into the mould to fill a 20 mm height. Carbon dioxide was used to accelerate saturation process. Thus, the B-value at the end of saturation process is always more than 0.95.

The sample was then consolidated under a predetermined effective consolidation pressure while monitoring the changing of sample volume. Consolidation was assumed to end when the changing of the volume was less than 0.1%. At the last stage, cyclic loading was performed at 0.1 Hz until the pore water pressure became equal to total pressure or, for some samples, until excess pore water pressure became stable.

3 Results and Discussion

Evaluation of soil resistance to liquefaction needs careful assessment on the failure criterion used. Figure 3 shows a typical test result for the material used in this study.

3.1 Failure Criterion

During cyclic loading, some soil samples undergo pore water pressure increase while the effective stress always remains above zero. In others the effective stress approaches zero and a limited flow is observed accompanied by plastic strain accumulation, with an example shown in Figure 3c. In Figure 3c the excessive strain accumulation begins at the same time that the pore water pressure begins to increase rapidly. The arrow indicates this stage.

In this study when the pore water pressure ratio (Ru) becomes equal to 0.9, and when strain accumulation (PS) exceeds 0.5%, are both considered as possible definitions of failure of the samples under undrained cyclic triaxial compression tests. The limit 0.5% strain accumulation in compression is chosen because the excessive strain accumulation starts at this point followed by rapid increment of pore water pressure.

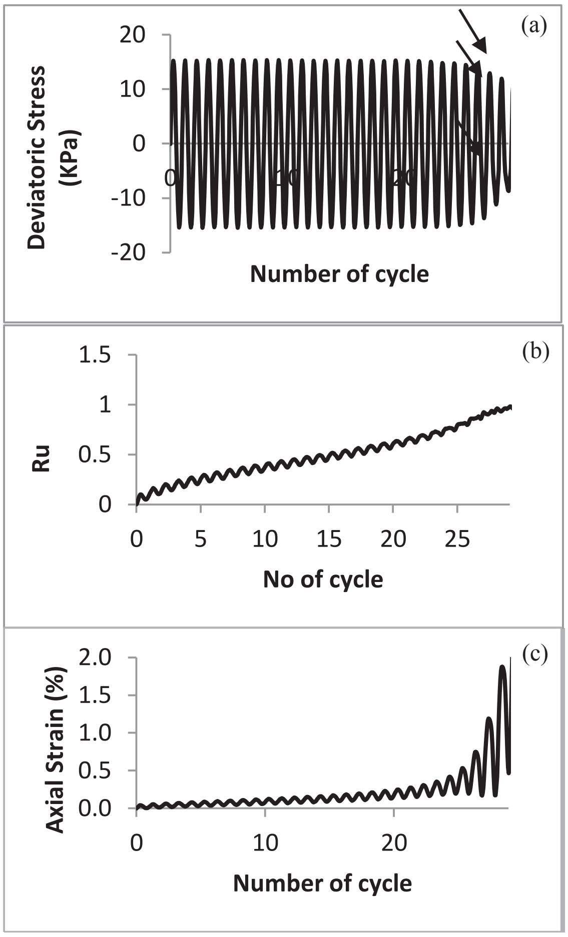

Fig. 3. Dry deposition, CSR 0.075, confining pressure 95 kPa

Figure 4 and Figure 5 present plots of CSR versus the numbers of cycles required to cause failure for the two deposition methods used. The number of cycles to failure is slightly different at 50 and 95 kPa confining pressures. The difference is greater when the lower confining pressure is applied. Another feature of the results is that the two different deposition methods produce very similar cyclic resistances. The possible explanation is that dry deposition and slurry deposition methods produce similar soil fabrics. Both are deposited from zero drop height and allow soil settlement under self-weight.

Using excessive strain accumulation as a failure criterion is beneficial for stability analyses as it occurs sooner than initial liquefaction defined by Ru=0.9 (Figure 5).

Figure 6 illustrates the influences of confining pressure and deposition method for when the strain accumulation criterion is used. The effect of confining pressure is most pronounced at high cyclic stress ratios.

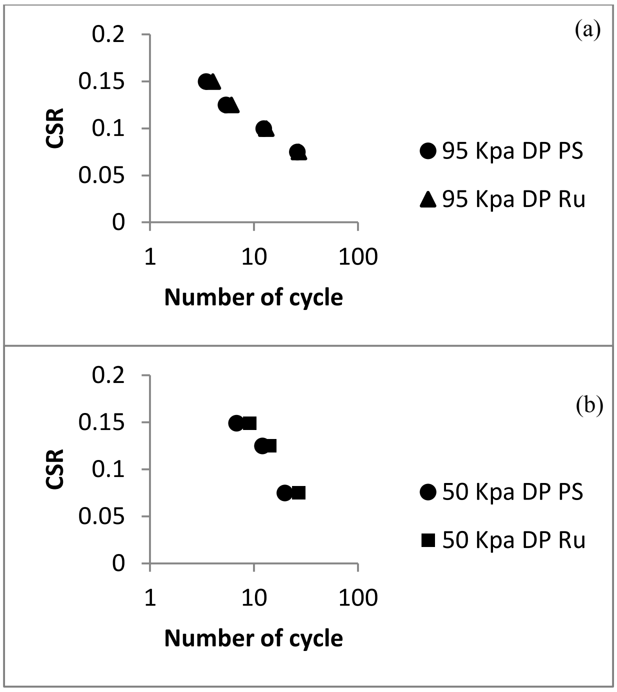

Fig. 4. Failure criterion of dry deposition based on (a) 95 kPa confining pressure (b) 50 kPa confining pressure

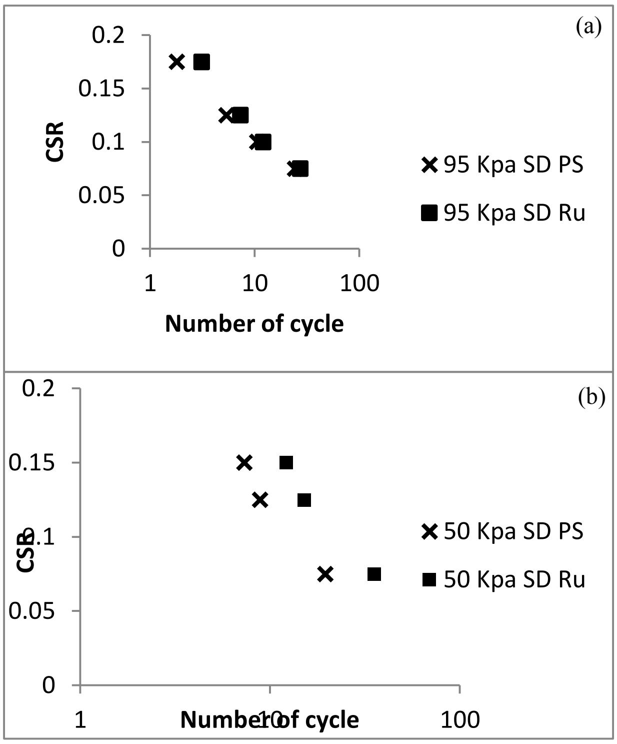

Fig. 5. Failure criterion of slurry deposition based on (a) 95 kPa confining pressure (b) 50 kPa confining pressure

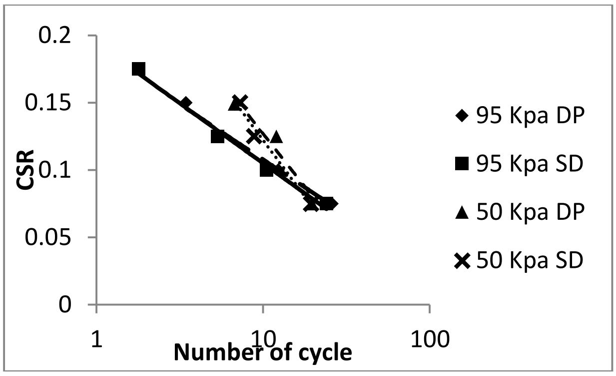

Figure 6. Failure criterion of dry and slurry deposition based on excessive plastic strain accumulation (PS)

4 Conclusion

A series of undrained cyclic triaxial tests were conducted on samples prepared using slurry deposition and dry deposition. Careful interpretation of the cyclic failure considered two failure criteria. One was based on pore water pressure ratio Ru=0.9. The other was based on the beginning of excessive plastic strain accumulation (PS).

Cyclic resistances for the two sample preparation methods were very similar. This is attributed to similar fabrics being present in the samples due to both methods subjecting the soil to similar vertical deposition and densification processes.

Cyclic failure defined using the strain accumulation criterion occurs sooner than failure by Ru=0.9.

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