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Partially drained response of cohesive soils subjected to cyclic loading

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ABSTRACT: One of the main concerns in the design of infrastructure earthworks on soft cohesive soils is the deformation of subsoil under traffic loading. A series of cyclic triaxial tests were conducted under undrained and partially-drained conditions on kaolinite specimens. A model based on the test results is presented to examine the partially-drained behaviour of an elastic-viscoplastic material considering different influence factors such as cyclic stress ratio and frequency. For a given cyclic stress ratio it is shown that a greater number of load cycles are required to dissipate the initial accumulation of pore pressures under partially-drained conditions for high frequency tests. The accumulated cyclic strain is reduced by partially-drained cyclic stress-controlled loading resulting in greater post-cyclic strength when compared to undrained test specimens. Numerical analyses for the prediction of the developent of partially drained pore pressures have been undertaken. Considering the long term response of a subsoil to cyclic loading, the residual pore pressures under cyclic loading can be predicted.

Keywords: cyclic loading; triaxial test; partial drainage; frequency; cyclic stress ratio

1 INTRODUCTION

The development of new rail infrastructure and the expansion of cities means routes are being governed more by the availability of existing land. Typically this land comprises soft ground previously avoided by contractors due to its high compressibility and low permeability. As a result, it is susceptible to significant settlements that have an impact on the overlying rail infrastructure, which has strict tolerances on vertical alignment, resulting in reduced speeds and line services and ultimately costly repairs. This settlement behaviour is increased by the cyclic loading conditions imposed on the soil as a result of the train passage.

The response of soils under cyclic loading conditions has been investigated by a number of authors. Sangrey *et al*. (1969) undertook low frequency undrained cyclic triaxial tests. More recently, Guo *et al*. (2013) examined the long term response of soft clays to cyclic loading. The analytical work of Hu (2010) suggested that the long term settlement of a clay subsoil subject to cyclic loading could consider the drained condition such that the soil settlement was governed by the deformation behaviour of the soil neglecting the pore pressure response. However, acknowledging the low permeability of clay soils, a number of authors have examined the long term response of the soil considering partially drained conditions, whereby pore pressures are simultaneously generated and dissipated. Sakai *et al*. (2003) stated this response was more representative of in situ soil behaviour, previously acknowledged by Hyodo *et al.* (1998) stating drainage conditions had a significant impact on the settlement response. Ni (2012) acknowledged the contribution of geosynthetic vertical drains in the dissipation of excess pore pressures and subsequently reducing the settlement of soft soils. Cyclic triaxial tests undertaken with a single PVD inserted in a 300mm diameter kaolinite sample showed a significant reduction in the generation of excess pore pressure. In the case of a failed sample, a greater number of load cycles were applied to the sample with a PVD inserted. Subsequently, Ni (2012) and Indraratna *et al.* (2015) presented a partially drained cyclic model for soft soils.

This paper presents the undrained and partially drained cyclic triaxial tests undertaken on reconstituted kaolinite samples and based on the partially drained model presented by Ni (2012) and Indraratna *et al*. (2015), numerical analysis of the response of a kaolinite soil subject to partially drained cyclic loading.

2 PARTIALLY DRAINED TRIAXIAL TESTS

Undrained and partially drained cyclic triaxial tests were undertaken using the GDS Enterprise Level Dynamic Triaxial Testing (ELDYN) System. The kaolinite samples were anisotropically consolidated to an initial vertical effective stress of 120 kPa with a radial effective stress of 72 kPa, representing the confining conditions of a soil element at approximately 6.0 m depth with a K_0 consolidation ratio of 0.6. In this study, tests were conducted on kaolinite soil with a specific gravity of $G_s = 2.68$; a plastic limit of w_p = 25%; a liquid limit of w_L = 55% and a plasticity index of I_p = 30%. Reconstituted samples were mixed to a slurry at 150% water content prior to consolidation and cut to cylindrical samples of 38mm diameter and 76mm height.

Undrained tests were completed with drainage lines of the triaxial machine closed. Partially drained conditions were achieved by allowing the drainage lines to remain open. Due to the low permeability of the kaolinite samples and therefore the inability of the sample to fully dissipate the generated excess pore pressures the samples can be defined as partially drained. A summary of the undrained and partially drained tests completed is presented in Table 1 for undrained tests and Table 2 for partially drained tests.

Table 1. Summary of undrained cyclic triaxial tests

Sample	CSR	Frequency (Hz)	Number of cycles	Fail?
U01	0.6	0.1	6000	No
U02	0.6	1	15000	No
U03	0.6	2	30000	No
U04	0.6	5	34200	No
U05	0.8	0.1	1726	Yes
U06	0.8	1	10523	Yes
U07	0.8	2	18256	Yes
U08	0.8	5	29565	Yes

Table 2. Summary of partially drained cyclic triaxial tests

Sample	CSR	Frequency (Hz)	Number of cycles	Fail?
PD01	0.6	0.1	6000	No
PD02	0.6	1	15000	No
PD03	0.6	2	30000	No
PD04	0.6	5	34200	No
PD05	0.8	0.1	1965	Yes
PD06	0.8	1	14535	Yes
PD07	0.8	2	21463	Yes
PD08	0.8	5	33210	Yes

$$CSR = \frac{q_{cyc}}{q_f} \quad (1)$$

The cyclic stress ratio, CSR, is defined as the ratio between the cyclic deviatoric stress q_{cyc} (kPa) applied to the sample in testing and the static deviatoric stress at failure q_s (kPa) for the sample, defined in Equation 1. As such, there exists a critical *CSR* value, whereby above this value (sample with a value of q_{cyc} close to q_s) the sample will fail and below this value that sample will remain stable upon repeated cycles of loading. Samples were tested for between 6000 (0.1Hz) and 34200 cycles of load or until failure.

Results of tests from the undrained triaxial tests and partially drained triaxial tests are presented in Figure 1 for a frequency of 0.1Hz and Figure 2 for a frequency of 5Hz for undrained and partially drained samples with a cyclic stress ratio of 0.6 and 0.8. The results demonstrate a reduction in the pore pressure as a result of partially drained conditions. Considering the response of normalised excess pore pressure, the partially drained samples show an initial peak in the generation of pore pressure followed by subsequent dissipation with an increased number of cycles to a stable residual pore pressure increment. Comparing the pore pressure response of the partially drained triaxial tests with the undrained triaxial tests, the partially drained pore pressures reduce from their peak value at a similar point to where there is no longer a rapid accumulation in pore pressures in the undrained tests. Considering the undrained pore pressure response, it can be seen that when there is a negligible increment of pore pressure development, the pore pressures in partially drained tests begin to decrease as the dissipation of pore pressures is able to occur. Eventually, as for the case of the undrained tests, the pore pressures converge to a stable pore pressure value.

Examining Figures 1 and 2 for frequencies of 0.1Hz and 5Hz respectively, it can be seen that for high frequency loading, a greater number of cycles is required in order to dissipate the peak excess pore pressures.

From observation of the pore pressure response alone, it is unclear if the sample with a cyclic stress ratio of 0.8 has failed, therefore the axial strains must also be considered. For partially drained samples, the samples can withstand a greater number of load cycles prior to failure. The results show a similar trend to the results of tests on Ariake clay previously undertaken by Sakai *et al*. (2003) and the findings of Ni (2012).

$$\Delta\varepsilon_v = \chi\, N^{\xi}\, m_{vr}^{*}\, \Delta u_d \quad (2)$$

Sakai *et al*. (2003) presented the relationship between the incremental volumetric strain ($\Delta\varepsilon_v$) and dissipation of excess pore pressure generation (Δu_d) using Equation 2, where m_{vr}^{*} is the post-cyclic coefficient of volume compressibility, N is the number of cycles and χ and ξ are experimental parameters.

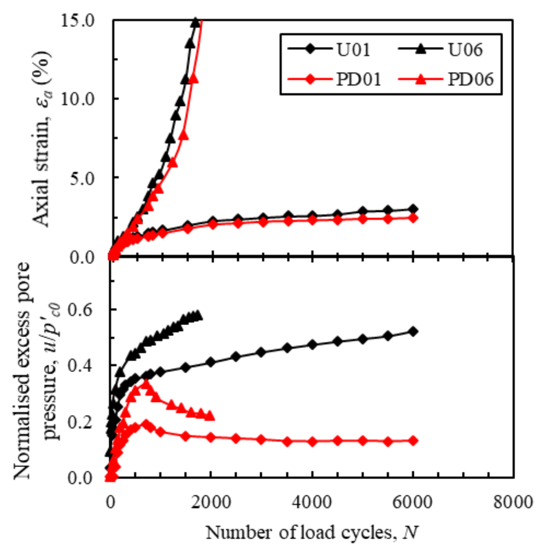

Figure 1. Undrained and Partially drained cyclic triaxial response considering number of load cycles; f=0.1Hz

$$m_{vr}^{*} = \frac{0.434\, \alpha_c\, C_s}{(1+e_c)\left(1-{}^{u_p}/_{p'_{c0}}\right)p'_{c0}} \quad (3)$$

m^*_{vr} is described in Equation 3 where e_c defines the critical void ratio, α_c and C_s are determined from oedometer tests, u_d is the summation of generated and dissipated excess pore pressures in partially drained tests and u_p and p'_{c0} define the peak excess pore pressure and mean effective confining pressure respectively.

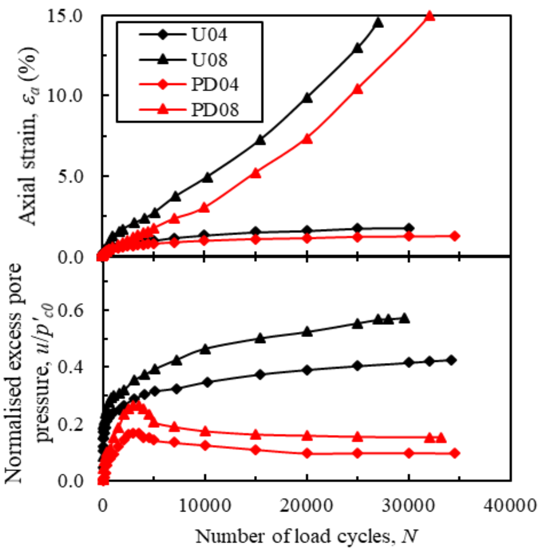

Figure 2. Undrained and Partially drained cyclic triaxial response considering number of load cycles; f=5Hz

The results of the analytical solution of Sakai *et al*. (2003) compared with the partially drained triaxial test pore pressure response is presented in Figure 3. Whilst a good agreement is achieved with the test data, the empirical solution is typically suitable for low amplitude (and therefore low *CSR* values) below the critical *CSR* value. As a result, it is unable to determine if failure has occurred. The solution is based on empirical assumptions with respect to the generation of excess pore pressures. Indraratna *et al*. (2015) presented a numerical model that combined the unit cell method of vertical drains with the cyclic model created by Ni (2012).

Considering the response of the partially drained triaxial tests and their attainment of a residual pore pressure, the solution can be modified to consider partially drained soils without prefabricated vertical drains.

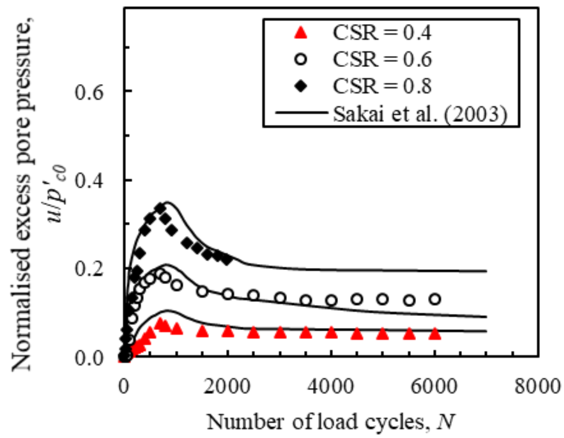

Figure 3. Comparison between partially drained triaxial tests (f=0.1Hz) and Sakai et al. (2003) relationship

3 NUMERICAL ANALYSIS

Ni (2012) extended the response of a soil under cyclic loading, defined previously by Carter et al. (1980) in Equation 4 to include a degradation parameter, Equation 5, that degrades with an increasing number of cycles.

$$\frac{dp'_c}{p'_c} = \theta \frac{dp'_y}{p'_y} \tag{4}$$

The hardening parameter, p'_c is defined as the intersection of the initial yield surface and the p' axis whilst p'_y is defined by Roscoe and Burland (1968).

$$\theta^* = \frac{1}{\xi_1 N + \xi_2} \tag{5}$$

The degradation parameter, θ^* is defined in terms of the number of cycles, N, and two experimentally determined constants, ξ_1 and ξ_2.

Ni (2012) and Indraratna *et al.* (2015) include the dissipation of excess pore pressures using Equation 6.

$$c_h\left(\frac{\delta^2 u}{\delta r^2}+\frac{1}{r}\frac{\delta u}{\delta r}\right)=\frac{\delta(u+u_p)}{\delta t} \tag{6}$$

The coefficient of consolidation, c_h and radial drainage (subscript r) can be written in terms of pore pressure induced by cyclic loading, u_p and the consolidation time, t.

The cyclic loading parameters have been determined based on the laboratory test data, presented in Table 3.

Table 3. Summary of soil parameters

λ	**κ**	**M**	p'_o **(kPa)**	q_o **(kPa)**	e_o
0.157	0.052	1.71	88	48	1.53

Values for cyclic loading parameter ξ_1 were 2.9, 2.7, 2.8 and 2.9 and for ξ_1 were 30, 160, 340 and 490 for frequency of 0.1Hz, 1.0Hz, 2.0Hz and 5.0Hz respectively. The radius was taken to be 18mm in agreement with the triaxial samples completed. Based on the reduction in pore pressure generated in the cyclic triaxial tests, the ratio between undrained and partially drained residual pore pressures was calculated, shown in Figure 4. c_h was modified to account for the partially drained behaviour in the cyclic tests undertaken, acknowledging that both lateral and vertical drainage were permitted under the partially drained test conditions.

The results of the model developed by Ni (2012) are shown in Figure 5 for frequencies of 0.1Hz, 1.0Hz, 2.0Hz and 5Hz at cyclic stress ratios of 0.6 and 0.8. Unlike in the case of the analytical solution by Sakai *et al.* (2003), the predictions of the modified Indraratna *et al.* (2015) are able to predict the final residual pore pressure under partially drained conditions and in the case of samples that failed (*CSR* = 0.8) were able to accurately predict the number of cycles before failure occurred .

Whilst the results were unable to determine the peak generated as a result of the initial build up of generated pore pressures, in reality the long term response of the embankment will be governed by the soil response over a significant number of cycles which is accurately predicted by the model.

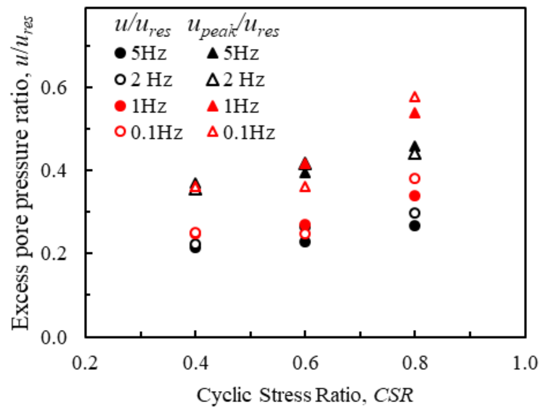

Figure 4. Ratio between partially drained and undrained excess pore pressures

4 CONCLUSIONS

Cyclic undrained and partially drained triaxial tests have been undertaken on reconstituted kaolinite samples. The results demonstrate that less pore pressure and less axial strain develop under partially drained conditions than for undrained conditions. In the case of failure, a critical cyclic stress ratio exists, above which a sample will reach failure. For partially drained soils exposed to cyclic stresses above their critical cyclic stress ratio, the soil can withstand a greater number of load cycles compared to the same soil under undrained conditions. The partially drained model presented by Ni (2012) for PVD improved soils under cyclic loading can adequately predict the long term response of partially drained soils under cyclic loading when the coefficient of consolidation is modified to reflect to ratio between undrained and partially drained pore pressures.

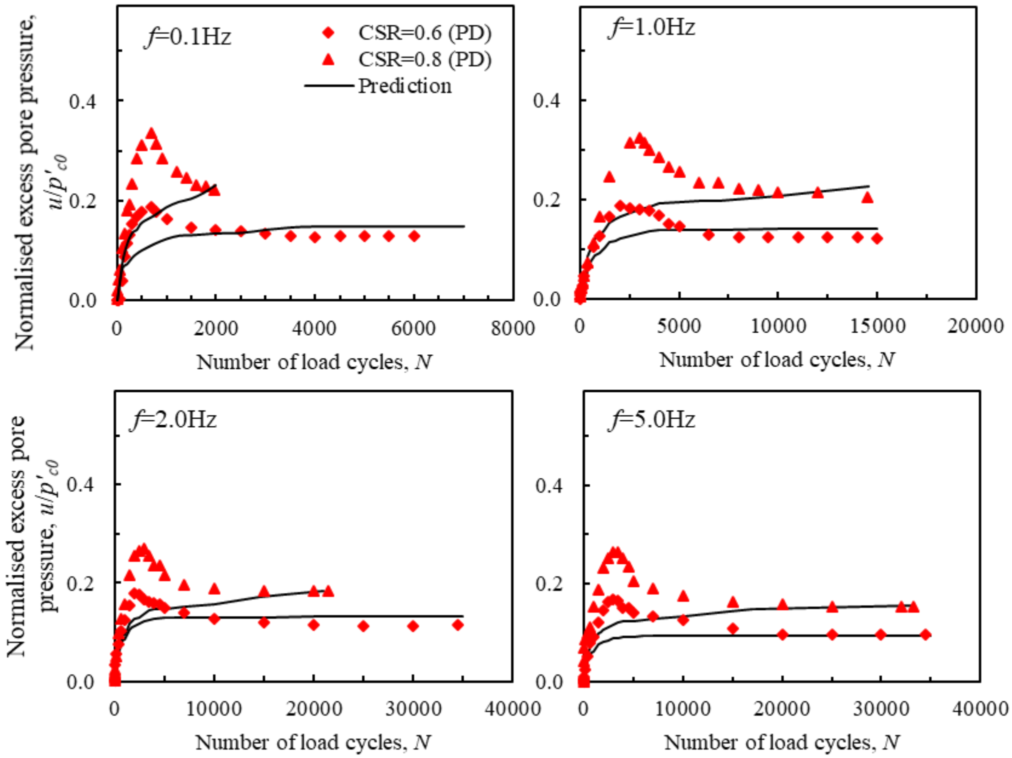

Figure 5. Partially drained prediction results

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