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Effect of strain rate on isotropically consolidated kaolin over a wide range of strain rates in the triaxial apparatus

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ABSTRACT: Soils display strain rate dependant behaviour which may pose a problem in geotechnical engineering applications. In this study a modified electromechanical triaxial apparatus was used to shear normally consolidated reconstituted Speswhite kaolin samples over a range of strain rates from 1%/hr to 180,000%/hr. The purpose of the investigation was to explore the variation of rate dependant soil response to increased in both drained and undrained conditions. Results show that the shear strength of the soil to be rate dependant at all of the strain rates investigated. In the case of a drained response overall strength decreased with increasing strain rate, with the rate effects displaying strain level dependency. Where the response was undrained the soil achieved a greater strength with increasing strain rate which increased at a greater rate than the generally assumed 10% increase in strength per log cycle increase in strain rate.

1. INTRODUCTION

In geotechnical engineering soils may be subjected to vastly different rates of strain. The difference between the soil resistance as a result of slow and rapid strain rate events is often referred to as the strain rate effect (Whitman, 1957). Geotechnical applications where soil resistance may be affected by variation of strain rate include driven pile installation, pile capacity testing (Brown and Hyde, 2008a), shear vane testing (Biscontin and Pestana, 2001), CPT investigations (Dayal and Allen, 1975), as well as laboratory testing including triaxial tests (Briaud *et al*, 1985), fall cone penetration, direct and simple shear testing amongst others. In terms of velocity the variation between tests can be from 0.0001mm/s for a 38 mm diameter triaxial test to 2000 mm/s during a Statnamic pile test (Brown et al, 2006). This is a difference of 7 orders of magnitude.

Soil strength parameters derived from rate dependent tests may be unsuitable for use with other applications involving a different strain rate from which the parameters are derived. To quantify the rate effect for a soil requires costly and specialist testing, and the rate effect is thus often ignored or underestimated in design. Generally the strain rate effect is taken as a 10% increase in strength per log cycle increase in strain rate as recommended by Kulhawy and Mayne (1990).

The variability of soil resistance with increased velocity is highlighted in Figure 1, which presents normalised cone resistance against normalised penetration rate CPT tests. In Figure 1, results from Randolph and Hope (2004) are obtained from CPT tests in kaolin, and results from Bemben and Meyers (1974) are from CPT tests in a lightly over consolidated varved clay. The results from the CPT tests are compared to the 10% increase in strength per log cycle increase in strain rate. To normalise the results from different soils a non-dimensional ‘velocity’ V is used (Finnie *et al*, 1994).

$$V = \frac{vd}{c_v} \tag{1}$$

Where v is the velocity of penetration, d is a characteristic length (often an assumed to be the drainage path length), and c_v is the coefficient of vertical consolidation.

In Figure 1 the response of the soil to increased V is clearly non-linear with increasing log cycles. For the tests in kaolin, a decrease in resistance with increasing V is found. This is attributed to increasing excess pore pressure generation and less time available for the soil to consolidate during the event. The rate of decrease in resistance reduces until $V \approx 100$, where the response changes, beyond which an increase in penetration resistance was measured. In the case of field CPT tests on varved clays, at values of V between 8 to 30 the soil resistance does not vary significantly as the soil response is thought to be fully drained. Beyond V=30 the resistance reduces, again due to increases in pore pressure and reduced consolidation, until a minimum resistance is measured at $V \approx 100$. For both soils when V is increased beyond 100 the soil resistance to penetration increases. The increase in soil resistance is thought to be a viscous response, as the soil reaction is assumed to have become fully undrained beyond V=100. There is clearly a large difference between the suggested

10% rate effect per log cycle increase in *V* and the results presented by Bemben and Meyers (1974)

An idealised version of soil response to increasing strain rate based upon the results shown in Figure 1 is presented in Figure 2. Four zones of differing soil response are identified.

This paper presents data from 7 tests conducted at various strain rates on 100mm diameter samples of normally consolidated reconstituted Speswhite kaolin.

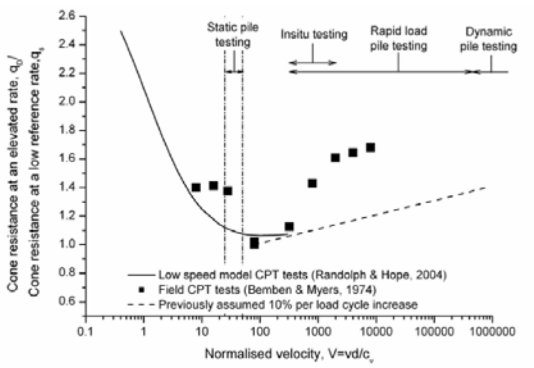

Figure 1. Variation of rate effect from cone penetration testing compared with suggested rate effect prediction proposed by Kulhawy and Mayne (1990).

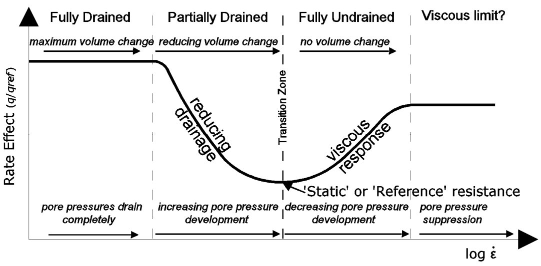

Figure 2. Idealised strain rate effect 'backbone' curve, identifying regions of different soil behaviour.

2. LABORATORY TESTING, MATERIALS, EQUIPMENT AND PROCEEDURES

2.1. Testing programme

This paper presents strain rate effect data from tests in normally consolidated reconstituted kaolin samples conducted in a specially modified triaxial apparatus. The testing programme was designed to define the soil response over a wide range of strain rates in a similar way to that shown in Figure 1. The triaxial tests were conducted while permitting drainage to occur at all strain rates (ie drainage valves open). This was undertaken to the identify the strain rates where the soil response moved between drained, partially drained and undrained (Figure 2).

To better understand the soil behaviour at different strain rates, the tests were conducted with base and mid height pore pressure measurement in an attempt to establish the changes in effective stress, and with axial and radial local Hall effect transducers to investigate strain dependant rate effects (not reported here).

2.2. Equipment

A total of 7 strain controlled tests each of a different constant axial strain rate were conducted for this study. (1%/hr, 5%/hr, 10%/hr, 300%/hr, 1,800%/hr, 18,000%/hr and 180,000%/hr). A bespoke GDS Electromechanical Dynamic Triaxial Testing System was used to conduct the tests. It was modified from a 5Hz cyclic triaxial testing system to allow rapid application of monotonic displacement. The system was capable of axial strain rates of 180,000%/hr (100mm/s). The GDS triaxial system supported 8 channels reading the entire series of channels over 10kHz. Cell pressure and back pressure were controlled by two GDS standard digital controllers. The triaxial cell was provided with a balanced ram system which compensated for the potential increase in cell pressure caused by the axial displacement of the ram. Load was measured from an internal submersible 5kN load cell. The contact between the load cell and the top cap (Figure 3) was by a half ball which seated in the top cap, as specified by Gasparre (2005). The half ball was allowed to rotate freely without transmitting load until full contact was made between the load cell and half ball, thus ensuring no disturbance to the sample during docking with the load cell. Pore water pressure was measured at mid height using a GDS mid height pore water pressure probe, and at the base using a P102 Sherborne pore pressure transducer. Axial bender elements were incorporated into the base pedestal and top cap. Local displacements were measured by both axial and radial Hall effect transducers (Clayton and Khatush 1986). The displacement range of the local transducers was 6mm, with a resolution using a 16bit data acquisition system of 0.1μm, and an accuracy of 0.8%.

2.3. Material

The soil used in this study was reconstituted Speswhite kaolin (I_P 45%, w_L 75%) mixed at 1.6 times the liquid limit and consolidated to 140kPa in a 1D consolidation apparatus which allowed two way axial drainage. When consolidated the kaolin sample was removed and trimmed to 100mm diameter and 200mm height. The sample was saturated to 330kPa for a minimum time period of 24 hours and a saturation test was conducted. All samples achieved a minimum saturation value of B=0.98. The sample was then consolidated to 720kPa under isotropic conditions using radial filter paper drains aligned vertically. The samples were then sheared monotonically whilst control of the back pressure unit was maintained to allow sample drainage if the strain rate allowed.

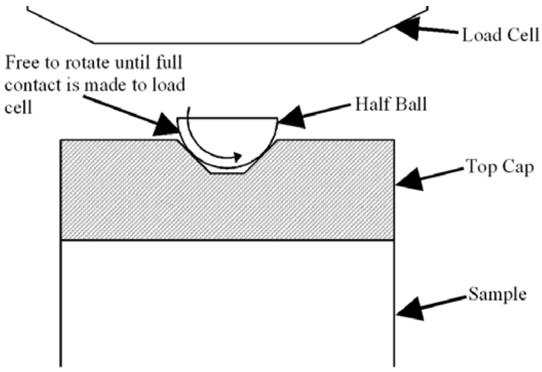

Figure 3. Half Ball connection between load cell and top cap, as per Gasparre (2005).

Property	Value
Plasticity Index, I_p	45
Liquid Limit, w_l	75
Plastic Limit, w_p	30
Γ	2.7245
λ_{NCL}	0.1248
M_{CSL}	0.87
p' (kPa)	720
G_0 (MPa) p'=720kPa	155
Permeability, k (m/s)	$3.4\text{x}10^{-11}$

Table 1. Properties of Speswhite kaolin.

3. RESULTS OF TRIAXIAL SHEAR TESTS

3.1. Stress Strain

The stress strain behaviour of kaolin at various strain rates are shown in Figure 4. The deviatoric stress is plotted directly against strain and not as a stress ratio since all samples were consolidated to the same effective stress before shearing. From the stress strain curves in Figure 4, it can be seen that the samples display different behaviour depending on strain rate, ranging from strain hardening to clearly identifiable maximum deviator stresses. The test undertaken at the slowest rate of 1%/hr shows no peak deviatoric stress, as the soil work hardens under continued straining. This test displayed the highest overall shear strength. As samples were tested at increasing strain rates from 5%/hr to 300%/hr the overall deviatoric stress decreased, and the samples began to display more clearly defined peak deviatoric stresses. The 300%/hr test developed the lowest overall deviator stress identifying this strain rate as close to that at which transition from drained to undrained behaviour occurred. As the strain rate was increased to 1,800%/hr the sample achieved higher deviator stress than the 300%/hr test. At 18,000%/hr the stress strain curve again displayed a comparatively large increase in peak deviator stress .The 180,000%/hr was the greatest strain rate used, and the soil again displayed an increase of peak deviator stress, although the q_{max} value was lower that that found at 1%/hr.

3.2. Excess pore pressure development

The effect of strain rate on excess pore pressure development measured at sample mid height is shown in Figure 5. The 1%/hr test was not fully drained as excess pore pressures developed in the initial part of the test. However as the test progressed the level of pore pressure decreased due to consolidation and the sample strain hardened as no peak deviator stress developed and the sample did not exhibit any signs of localisation after the test. Increasing excess pore pressures were measured at 5%/hr and 10%/hr. All tests conducted at strain rates of 5%/hr and greater developed clear shear planes, during shearing. Increasing the strain rate to 300%/hr produced a decrease in the excess pore pressure, indicating a change in the trend of pore pressure development with increased strain rate. At strain rates greater than 300%/hr the magnitude of excess pore pressures continually reduced with increasing strain rate.

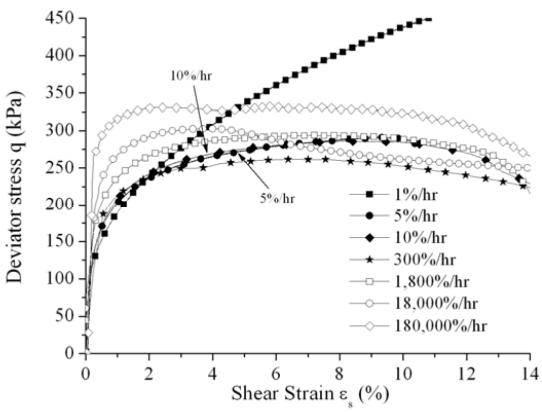

Figure 4. Stress strain relationship for normally consolidated reconstituted Speswhite kaolin at strain rates from 1%/hour to 180,000%/hour.

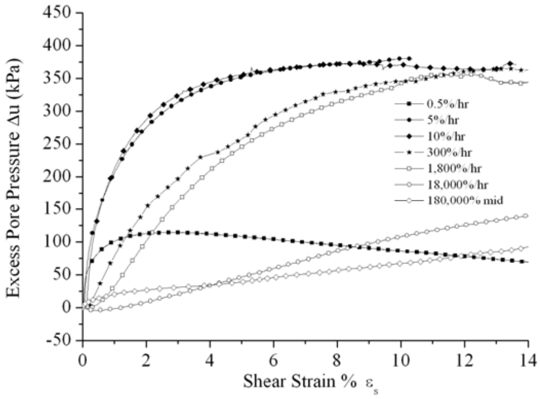

Figure 5. Development of excess pore pressure with strain measured at the sample mid height.

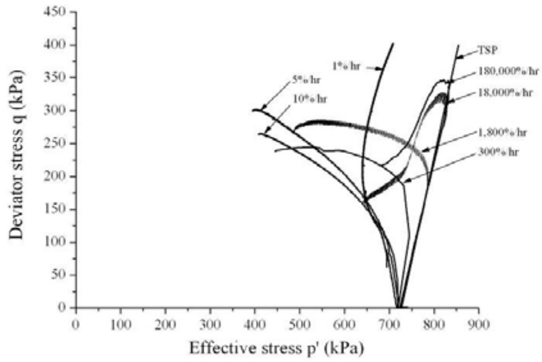

Figure 7. Stress paths for soils sheared at increasing strain rate.

3.3. Total volume change

All the tests discussed in this paper were conducted whilst permitting drainage. Figure 6 presents results of from all tests conducted for this paper in v-p' space. The largest change in volume was found at the slowest strain rate of 1%/hr. The overall change in volume decreases from the level of volume change at 1%/hr as the strain rate increases. The volume change at a strain rate of 300%/hour was very small indicating the soil was almost fully undrained. When the strain rate was increased further beyond 300%/hour there was no volume change and response of the soil was fully undrained. Where the soil response was undrained, lower excess pore pressures were measured with increasing strain rate, and when these tests are represented in v-p' space it appears that they move to the right of the normal consolidation line (NCL).

3.4. Stress paths

The stress paths as shown in Figure 7 show the variability of response to increased strain rate. The stress paths are constructed using the pore pressure

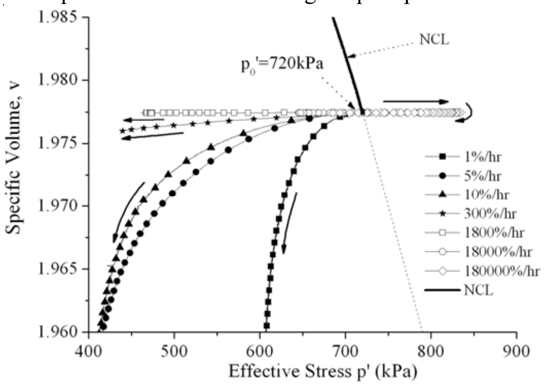

Figure 6. Change in Specific Volume with change in effective stress for tests at increasing strain rate.

measured at sample mid height. From the Figure 7, it can be seen that the 1%/hr, 5%/r 10%/hr all initially behaved in a partially drained manner. Tests conducted at strain rates greater than and including 300%/hr initially move to the right, following the total stress path. Similar trends were found in undrained triaxial studies at increased strain rate (Sheahan *et al*, 1996, Balderas-Meca, 2004). It was reported in these studies that increased strain rate led to a greater suppression of pore pressure, and thus an increase in the mean effective stress p'.

4. PORE PRESSURE RESPONSE

Based upon the nature of the stress paths and the recorded pore pressures with increasing strain rate investigation of the uniformity of pore pressure response was undertaken. The non-uniformity of pore pressure development in a triaxial sample where drainage was allowed has been investigated both numerically and experimentally in several studies (Carter, 1982, Atkinson *et al*, 1985, Liyanapathirana *et al*, 2005). However little is known on the non uniformity of pore pressures within undrained samples tested at greatly increasing strain rate. For the tests conducted in this study where the soil response can be described as fully undrained, pore pressures appear to reduce with increasing strain rate.

Figure 8. The mid height pore pressure transducer attachment used for 180,000% repeat test.

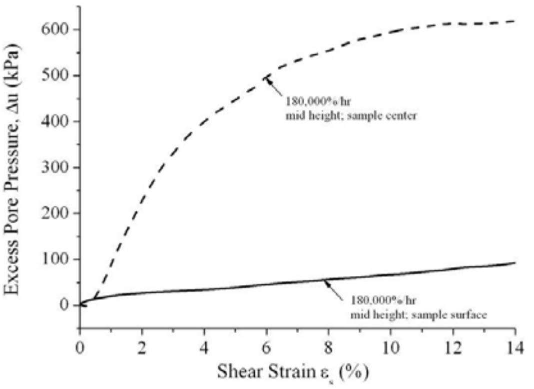

Figure 9. Comparison of excess pore pressure development measured at sample mid height centre and surface.

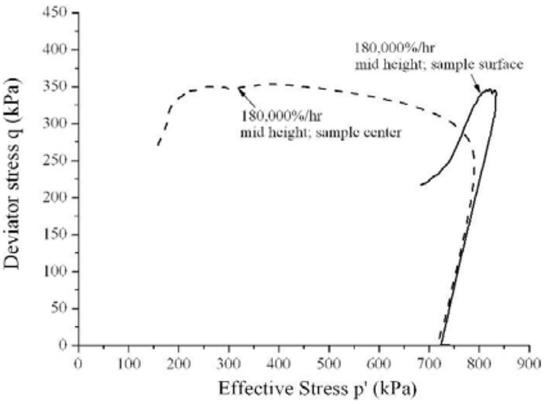

Figure 10. Stress paths comparing effect of mid height sample centre and sample surface pore pressure measurement.

To gain a better understanding of the pore pressure response in the sample during high speed tests a hollow aluminium tube was designed to attach to the mid height pore pressure transducer and protrude from the face of the transducer to the centre of the sample (Figure 8). The objective of this was to measure pore pressures that were generated directly at the centre of the sample. The dimensions of the hollow section of the tube was 40mm in length, 6.9mm O.D., 1mm wall thickness, with an increased diameter region to house the transducer (9.8mm O.D. and 10mm in length). The test where this attachment was used was a repeat of the 180,000%/hr test which resulted in an almost identical stress-strain curve as that for the original 180,000%/hr test with surface mid-height pore pressure measurement. The comparison of surface and central sample pore pressure measurement are shown in Figure 9.

The test conducted with the pore pressure transducer attachment (sample centre) showed much greater excess pore pressure generation (Figure 9). Above 0.5% strain the extended pore pressure transducer showed a relatively rapid increase in pore pressure (approx 100kPa per strain increment) with increasing strain up to a maximum of 618kPa. In contrast to this the pore pressures measured by the surface pore pressure transducer increased (at a rate of approx 10kPa per strain increment) to a maximum pore pressure of 91kPa during the test. The stress path (Figure 10) plotted using data from the pore pressure transducer attachment showed similar effective stress values in the initial part of the test however q_{max} was achieved at a lower p', and the effective stress reduced far beyond that found using the normal sample surface pore pressure transducer.

The results from the tests shown in Figures 9 and 10 suggest the reason lower pore pressures are measured at increased strain rate may be due to increased non-uniformity of pore pressures within the triaxial sample with increased strain rate and not as a result of dilation induced pore pressure suppression. This makes interpretation of results in terms of effective stress difficult.

5. ANALYSIS OF THE RATE EFFECT

Figure 11 shows a summary of the normalised deviator stresses obtained from the triaxial tests over the complete range of strain rates tested. The results from the tests are also compared at various strain levels. The deviator stress has been normalised by a reference value of deviator stress (q_{ref}) to derive the rate effect (q/q_{ref}). The reference values of q_{ref} are taken here as the results from the 300%/hr test as this test achieved the lowest overall deviatoric stress. Figure 11 compares q/q_{ref} taken at 2%, 5% and 10% strain. It can be seen that the rate effect in the partially drained domain is heavily dependant on the level of strain. At 2% strain, the tests at strain rates from 1%/hr to 300%/hr attain similar levels of deviator stress and the difference in rate effect is negligible. In the partially drained domain the rate effect continually increases with strain level. In the undrained domain the rate effect shows a clear increase in deviator stress with increasing strain rate, and also the rate effect appears to reduce with increasing strain level, however the differences at 5% and 10% strain are small.

Results from several studies using a CPT at increasing rates of penetration have produced similar shaped 'backbone' curves to those shown in Figure 1 (Bemben and Meyers, 1974, Roy and Woodward 1982, Randolph and Hope 2004, Lehane *et al*, 2009). Randolph and Hope (2004) fitted a hyperbolic function shown in equation 2, to CPT tests conducted in kaolin from Figure 1.

$$\frac{q}{q_{ref}} = \left(1+\frac{b}{1+cV^{d}}\right)\left\{1+\frac{\lambda}{\ln(10)}\left[\sinh^{-1}(V/V_0)-\sinh^{-1}\left(V_{ref}/V_0\right)\right]\right\} \quad (2)$$

Where b, c and d are curve fit parameters, λ is the fractional increase per log cycle increase in normalised velocity V. V_0 is the value of V where the rate correction starts to reduce to zero, and V_{ref} is where the rate correction term passes through unity. Randolph and Hope (2004) mentioned that the exponent d is a key parameter of the equation, which signifies the breadth of the transition zone between drained to undrained behaviour. Equation (2) was fitted to the results at each level of strain in Figure 11 and the parameters are shown in table 2. As shown in Figure 12, equation (2) fits both the partially drained and undrained domain well. Table 2 shows that with increasing strain the curve fit parameter d increases as the drainage effects are more influential at higher strain levels. It is shown that the strain rate effect parameter λ is larger than the 10% per log cycle increase in strain rate as recommended by Kulhawy and Mayne (1990). The undrained rate parameter λ decreases with increasing strain level, although the difference between 5% and 10% strain is very small. The rate effect achieved in the partially drained domain is also significantly larger than that achieved in the undrained domain, almost double the maximum rate effect measured in the undrained domain.

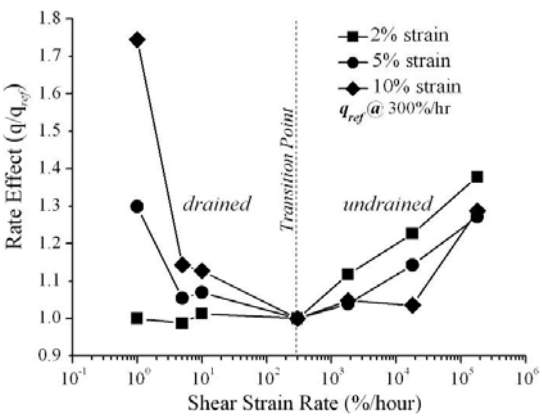

Figure 11. Strain rate effect for kaolin compared at 2%, 5%, and 10% strain.

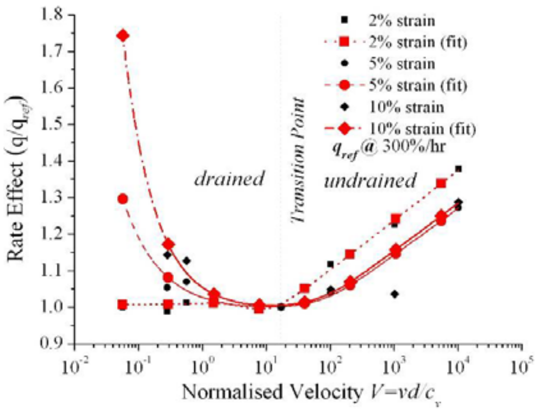

Figure 12. Comparison of real data to curve fit results.

Parameter	*2% Strain*	*5% Strain*	*10% Strain*
b	0.06	0.2	0.8
c	1	2.2	30
d	0.5	1.2	2.2
λ	0.17	0.11	0.11
V_0	60	60	60
V_{ref}	17	17	17

Table 2 Curve fit parameters for equation (2).

6. CONCLUSIONS

This paper presents results from 7 triaxial compression tests on reconstituted, normally consolidated kaolin samples sheared at axial strain rates from 1%/hr to 180,000%/hr. Over this range of strain rates the response of a soil varies from fully drained to fully undrained conditions.

The highest overall strength was achieved at the slowest strain rate, where volume change was the greatest during shearing. Increasing the strain rate between the range of 1%/hr to 300%/hr caused a reduction in volume change, resulting in a decrease of strength. When the strain rate was increased beyond 300%/hr the soil response became fully undrained. When undrained an increase of strain rate resulted in an increase of strength but the undrained strength increase was significantly lower than that as a result of volume change even at the highest strain rate. Thus strength increases due to rate effects for drained and partially drained situations may significantly exceed those associated with the high speed undrained events.

Results of pore pressure measurement at sample mid height show that increasing strain rate leads to higher pore pressure in the partially drained domain, and to lower pore pressure development in the partially drained to undrained domain. The lower pore pressure measured in the partially drained to undrained domain contributes to an apparent increased effective stress p'.

A thin walled hollow aluminium attachment was used to measure pore pressures in the centre of a sample. It was shown that higher pore pressures are generated at the centre of a sample tested at a high strain rate. This test appeared to show that very large pore pressures were generated at the centre of the sample which tended to p_0'. This suggests that there may be significant pore pressure non-uniformities associated with high strain rate testing which make it difficult to interpret results in terms of effective stresses.

Values of deviator stress values were normalised at 2%, 5% and 10% strain by the corresponding value of deviator stress from the 300%/hr test. The resulting 'backbone' curves were used to analyse the rate effect with an equation developed by Randolph and Hope (2004). It is shown that in the partially drained domain strength decreases with increased strain rate, and also the rate effect is strain dependant, with the largest rate effect at the highest strain, indicating that soil properties related to consolidation have greatest influence over the rate effect in this domain. In the undrained domain the strain rate effect ranges between 11-17% per log cycle increase of strain rate depending on strain. The strain rate effect reduces with increasing strain however the difference in rate effect between 5% and 10% strain are small. The undrained strain rate effect measured here is shown to be larger than that recommended in the literature. From comparison with other strain rate dependant tests in the literature it is also clear that the strain rate effect also varies with soil type.

ACKNOWLEDGEMENTS

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