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Strain-Rate Effects on the Undrained Shear Strength of Waitemata Residual Clay

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Summary The main part of the work reported in this paper was an investigation into the effect of strain-rate on the strength and deformation parameters of a residual Waitemata clay. The motivation for the work was the observation that the measured vertical capacities of small diameter driven timber piles in Auckland clays were less than those predicted by the Hiley formula. Unconsolidated-undrained triaxial compression tests were carried out on twenty two undisturbed samples. Two values of axial strain rate were applied: 20% per second and 0.027% per second. With this 750-fold increase in the rate of strain, an increase was observed in both the peak and large strain undrained shear strengths of about 70% and 85% respectively. Differences in stress-strain behaviour were also noted between the two sets of data: the rapid UU tests reached the peak shear resistance at larger axial strains than the slow tests and did not exhibit as significant a post peak reduction in shear stress.

1. INTRODUCTION

The information reported in this paper is the outcome of two graduate student projects. The first project, Fitch (1996), investigated the driving of piles, mainly for domestic construction, around Auckland. It also reviewed an earlier series of field tests on the capacity of small diameter driven timber piles in Auckland clay soils. The conclusions from this work were: (i) that the final set of the piles is related to the undrained shear strength of the clay, and (ii) that the Hiley formula overpredicts the vertical capacity of driven piles in these soils. The second project, Ahmed-Zeki (1996), investigated one possible source of this discrepancy in pile capacity and compared the unconsolidated undrained shear strength of residual Waitemata clay at very rapid rates of strain with that at more conventional strain rates.

It was found that increasing the deformation rate of the clay by a factor of 750 increased the peak undrained shear strength by about 70%. Such an increase indicates that the rate of shearing during pile driving will be able to mobilise more shear resistance in the soil than is available during static loading, consequently it is not surprising that the Hiley formula overpredicts the pile capacity. (This simple approach does, of course, overlook the complex dynamic nature of the loading applied to soil adjacent to a pile during driving.)

2. DRIVEN PILES IN AUCKLAND CLAYS

Fitch had access to data on the driving of about 1000 small diameter piles, mostly timber, for domestic and light industrial construction around Auckland. In all about 30 different sites were involved. In most of the cases the piles were in clay derived from in situ weathering of Waitemata Group soft rocks which are tertiary age sandstones and siltstones very common around Auckland. Intact cores of the unweathered material, about 50 mm in diameter, have unconfined compressive strengths of several hundred kPa to a few MPa. Careful examination of the data lead to the selection of the records from the driving of 31 piles. For these piles there were good driving records and also good vane shear strength data for the soil profile through which the piles were driven. As the diameter of the piles varied and the driving rigs also differed, the sets were normalised to account for diameter and the energy delivered to the pile.

NZ Standard NZS:3604 (1990) covers light timber construction and includes the driving of small diameter piles. It is expressed in terms of piles 140 mm in diameter driven with a hammer which delivers 4800 Joules per blow. As there are energy losses in the driving process, both because the hammer does not deliver the required amount of energy and because of conditions at the top of the pile, the NZS:3604 document assumes that the driving efficiency is such that a net energy of 3330 J is actually delivered to the pile from each blow. The diameter is allowed for by normalising the energy delivered to the pile in proportion to the base area of the pile relative to the base area of a 140 mm diameter pile. The measured set is thus normalised to account for both of these factors. The normalised set so obtained is the set expected if 3330 J was actually delivered to a 140 mm diameter pile.

Two sets of data were evaluated. The driving data obtained by Fitch could be linked to the undrained shear strength of the clay through which the pile was driven, and particularly the strength at the depth of the pile tip. The undrained shear strength of the clay was obtained from a small diameter Pilcon shear vane, commonly used around Auckland for routine investigations. As these piles were part of buildings under construction it was not possible to measure the vertical capacity directly. A second set of data, Lapish and Melville-Smith (1974), reports on field loading tests on driven piles in Pleistocene clays in the South Auckland region. These piles were all 140 mm in diameter and were driven to a depth of 1.2 m. Subsequently the piles were tested to failure under vertical loading. At the time of writing, shear strength data for the Pleistocene soil was not available, so, as often happens in geotechnical engineering, one has to make do with less than complete information. As the piles were not deep, the capacity which includes both the base and shaft resistance, was used to estimate the average undrained shear strength - the shaft adhesion was assumed to be $0.5s_u$.

As explained above the observed pile sets were normalised for diameter and for the net energy delivered. The net energy is the product of the hammer weight, drop height, hammer efficiency, and blow efficiency. This is normalised with respect to the net energy assumed to be delivered in the NZS:3604 document - 3330 J instead of the nominal 4800 J. The normalised set versus undrained shear strength relationship so obtained is plotted in Fig. 1. This diagram shows the not unexpected result that the set is a function of the shear strength of the clay. The fact that the data for the Pleistocene clays merges into that for the Waitemata clays indicates that the form of the relation is consistent for the range of clay strengths across the Auckland region. There is of course some scatter, part of which might be a consequence of the fact that for the piles in the Pleistocene clays, all of which were 1.2 m in length, the inferred average undrained shear strength at the tip of the pile and along the shaft is plotted, whereas for the Waitemata clays it is the undrained shear strength at the base of the pile which is plotted.

The lengths of the piles in Waitemata clay were typically between 2 and 4.2 m, but three piles were much longer, as annotated in Fig. 1, and for these three the plotted points lie above the remainder of the points for the piles in Waitemata clay. A contributing factor to this positioning of the three points may be the relative pile and hammer masses. For these three the hammer mass was less than the pile mass, while for all the other piles the hammer mass was much greater than that of the pile.

The merging of the two sets of data in Fig. 1 could be improved by the use of a value for the adhesion coefficient greater than 0.5 which would move the points for the Pleistocene data downwards. This possibility is not pursued further here as the main thrust of the paper lies elsewhere.

Figure 2 plots the measured vertical capacity of the piles in the Pleistocene clays and the predictions obtained applying the Hiley formula. This plot shows that the Hiley estimate is an upper bound on the actual pile capacity and, for most cases, this is considerably in excess of the measured capacity. The remainder of the paper investigates one possible factor contributing to this discrepancy.

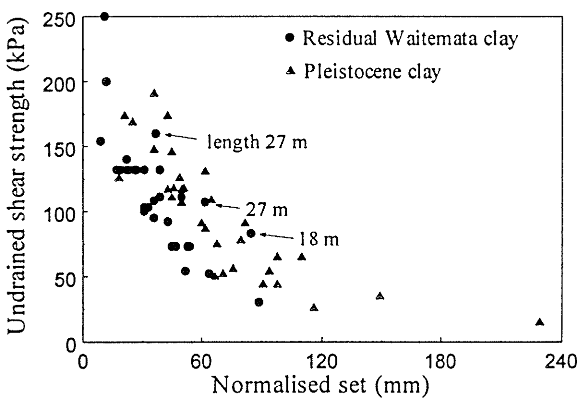

Figure 1 Relationship between normalised set and undrained shear strength for small diameter driven piles in Auckland clays.

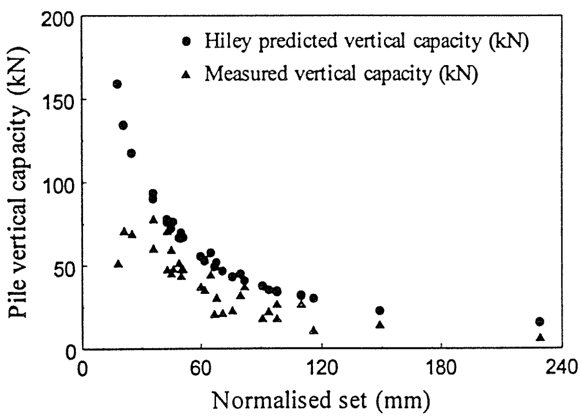

Figure 2 Measured vertical capacity of driven timber piles in Pleistocene clays and the Hiley formula estimates.

3. EFFECT OF RATE OF STRAIN ON UNDRAINED SHEAR STRENGTH

Several investigators have found that cohesive soil behaviour is dependant on the rate of loading. For clays with over-consolidation ratios of 8 or more, Sheahan *et al.* (1996) found that the peak shear strength effective stress failure envelope is not affected by strain-rate and that any increase in the undrained shear strength is a consequence of lower shear-induced pore pressures. For low overconsolidation ratio clays they found that increases in s_u are caused by both lower shear induced pore pressures and increases in the mobilized friction angle. Based on the findings obtained from conventional triaxial tests for various clays, Kulhawy and Mayne (1990) state that increasing strain-rate causes an increase in the undrained shear strength s_u by about 10% for every 10-fold increase of strain-rate.

Ladanyi (1977), in his comparison of the results of deep cone penetration tests with the results of full-scale load tests on piles in saturated clays, reaches a similar conclusion to that which motivated the work reported herein. He emphasises that the cone test (rate of penetration 20 mm second) mobilises an undrained shear strength which is 20 to 40% higher than that apparent under pile static load to failure tests.

Penumadu and Chameau (1997) found from pressuremeter test results on normally consolidated soil that a five-fold increase in the rate of probe expansion results in a large increase in the deduced initial shear modulus by 240-350%. For overconsolidated specimens (OCR=5), similar increases in rate of probe expansion showed less pronounced effects compared with normally consolidated specimens. In analyzing one-dimensional loading-rate effects, Farr (1990) observed a gradual rather than a dramatic increase in the constrained modulus during loading as the time to peak pressure decreases.

The main objective of this paper is to compare the response of Waitemata residual clay to rapid strain rates in comparison to the response under quasi-static strain rates. Even though the rapid strain rate tests do not involve cyclic loading it is assumed, nevertheless, that they provide information of relevance to the capacity of the piles during driving.

4. LABORATORY TESTING

4.1 Soil and Sample details

Unconsolidated undrained triaxial tests were performed on specimens, trimmed from block samples, at the natural water content and degree of saturation (close to full saturation). The effects of load rate on the undrained shear strength and undrained modulus were determined.

Hand-carved block samples were obtained from a newly excavated area to a depth of about 2.5 metres below original ground surface. This area is located on the north-western side of Greville Road, Albany - Auckland, where the surficial materials are residual soils derived from the weathering of Waitemata Group sandstones and siltstones.

The soil was typically described as: stiff, very plastic, silty clay, mottled brownish yellow in colour (although sometimes light grey), with occasional limonite staining and frequent rootlet inclusions.

As the soil is derived from the in situ weathering of the highly jointed Waitemata Group soft rocks some relict jointing is present in the clay. For a few of the specimens tested this jointing dominated the response of the soil. In particular, two specimens showed a continuous increase in load, rather than passing through a peak, and both were observed to fail along relict joints which were noticed during specimen preparation. One such example is clearly evident among the curves plotted in Fig. 5.

A range of Atterberg Limit tests and water content evaluations done by Meyer (1997) on specimens recovered from the same area show that the soil falls in the CH group according to the Unified Soil Classification System. The liquid limit values ranged from 50 to 66% with an average of 60%; the plastic limit values were between 28 and 36% with an average of 32%, and the resulting liquidity indexes ranged between 0.3 and 0.5 with an average of 0.4. For Meyer's specimens the water contents were between 38% and 49%; with an average of 43%, which is 8 percent less than the average water content (determined after testing) for the triaxial specimens in the programme reported in this paper.

4.2 Specimen Preparation

Rectangular prisms were cut from the block samples with the top-bottom direction always maintained vertical, and their ends made as plane as possible. The prism was then placed in a soil lathe, and trimmed using a wire saw until a cylindrical specimen 75 mm in diameter was obtained. Subsequently the specimen was removed, placed in a split mould and cut to 150 mm length, and the ends made plane and perpendicular to the longitudinal axis of the specimen.

4.3 Testing Equipment

Testing was carried out using the Materials Testing

Machine MTS 810 electro-hydraulic closed-loop system which has the capacity of high-velocity compression or tension, and high frequency fatigue testing. The system comprises an actuator, load frame, hydraulic power supply, and a control console. The displacement, cell pressure, and axial load transducers were logged with an A/D card installed in a personal computer. In addition, an X-Y plotter in the MTS console gave a direct indication during the test of load versus displacement.

4.4 Testing Procedures

Unconsolidated-undrained triaxial compression tests were performed at two values of rate of axial compression: 30 mm per second and 0.04 mm per second. For a 150 mm tall specimen, the corresponding strain rates are 20% per second and 0.027% per second, respectively. Considering the 2.5 metres depth of overburden at the location of the samples, a cell pressure of 50 kPa was applied in all the tests. As K_0 for the residual clay is unknown and the soil behaves as if it was moderately to heavily overconsolidated, a K_0 value of 1.0 was assumed in adopting the 50kPa cell pressure.

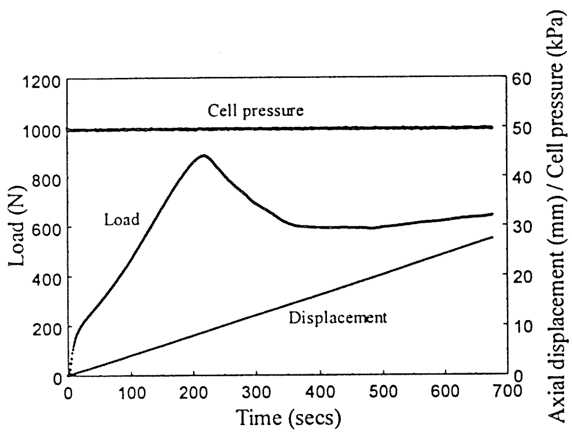

Figure 3 Measured values vs time, slow strain rate test.

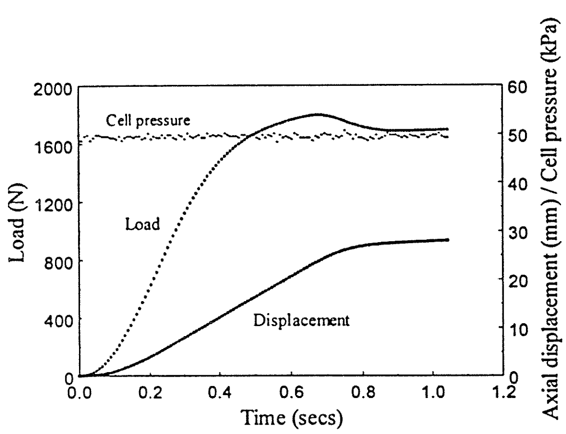

Figure 4 Measured values vs time, rapid strain rate test.

Testing continued to about 18.5% axial strain with load and displacement readings were recorded at pre-assigned time intervals. The cell pressure was also monitored to ensure that there was no change during the loading, particularly during the rapid loading. A time interval of one second was chosen for the slow strain-rate tests, and 0.005 second for the fast strain-rate tests. Figures 3 and 4 show measured values for typical tests plotted against time in the slow and high strain-rates modes. Specimens tested under slow rate of strain reached maximum shear stresses in 225 seconds on average (range 195 - 263). In the fast strain-rate mode, the average time-to-failure was 0.64 seconds (range 0.54 - 1.0).

5. TESTS RESULTS AND DISCUSSION

5.1 General

The area correction detailed in paragraph 6.2.1.6c, NZS 4402:1986, Test 6.2.1 was applied to the data. The restraining effect of the rubber membrane was considered insignificant and thus neglected. Any seating errors apparent when the stress-strain curve was plotted were corrected by extrapolating the low strain linear part of the curve back to the strain axis, so defining a new origin for the strains.

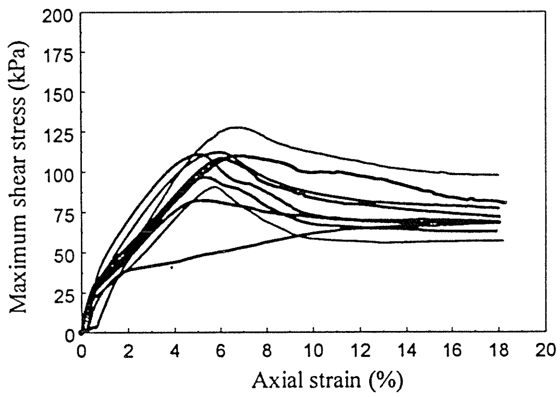

Figure 5 Maximum shear stress vs axial strain, slow strain rate tests.

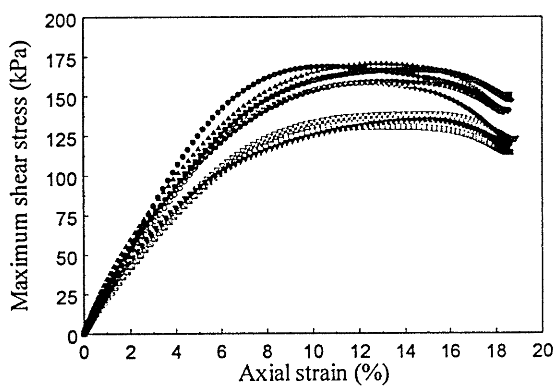

Figure 6 Maximum shear stress vs axial strain, rapid stain rate tests.

5.2 Maximum Shear Stress vs Axial Strain

In Figs. 5 and 6 the maximum shear stress, $(\sigma_1 - \sigma_3)/2$ where σ_1 is the major principal stress and σ_3 the minor principal stress, is plotted against axial strain for the UU tests. Most of the specimens in the slow strain-rate tests showed a pronounced peak, with a post-peak drop off reaching a near constant value at axial strains of about 12 to 15%. The resulting peak s_u values ranged from 62 to 127 kPa with an average of 100 kPa, while the large strain undrained shear strength values were between 57 and 97 kPa, averaging 72 kPa. The peak shear strengths occurred at a range of axial strains from 5.1% to 7.0%, resulting in an average of 5.8%. The corresponding peak strength values for the fast strain-rate mode ranged between 131 and 170 kPa with an average of 153 kPa, occurring at axial strains between 10.7% and 15.1% with an average of 13.2%. The large strain undrained shear strengths were between 114 and 150 kPa, averaging 132 kPa. When comparing group averages at the two rates of strain, it is observed that a 70% increase in the peak shear strength has occurred and 85% increase in the large strain strength as a result of the increase in the rate of strain.

5.3 Undrained Shear Strength vs Water Content

The peak undrained shear strength values for the two rates of strain are plotted against specimen water contents (determined after testing) in Fig. 7. The water contents were determined after the test, the minimum was 42.2% and the maximum 53.7%. Figure 7 shows that for the range of water contents involved there is no clear relationship between undrained shear strength and water content, other than the suggestion that s_u is independent of water content for the range encountered in the specimens. Figure 7 also shows that the there is considerable scatter in the water contents of the individual specimens. Scatter is also apparent in the undrained shear strengths plotted in Figs. 5 and 6. Such variations are typical of Auckland residual clays

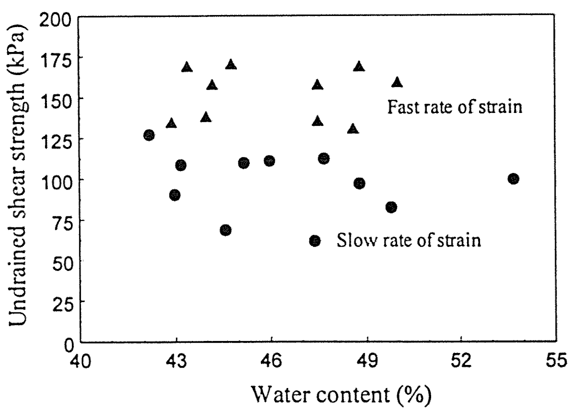

Figure 7 Undrained shear strength vs water content.

5.4 Stress-Strain Behaviour

Examination of the stress strain curves presented in Figs. 5 and 6 reveals some interesting differences between the stress-strain behaviour of the clay at rapid and slow rates of strain. These can be summarised under the following three headings:

(i) The stiffness for axial strains up to 2% is about the same for both series of tests. For the slow rate of strain tests the shear stresses developed at 2% axial strain ranged from 39 to 69 kPa, with an average of 50 kPa, whilst for the fast rate of strain tests the range of shear stresses reached at 2% axial strain was between 43 and 59 kPa.

(ii) The fast strain rate tests reached the peak shear stress at a larger strains than the slow tests - 11 to 15% in contrast to 5 to 7%.

(iii) The slow strain rate tests reached a peak stress and then the shear resistance decreased so defining a relatively sharp peak. However, the rapid stain rate tests rise to a maximum shear stress which then remains relatively constant whilst the strain increases by several per cent. Thus the undrained behaviour of the clay under rapid loading could be idealised as elastic-perfectly plastic.

(iv) Examination of Fig. 5 suggests that for several of the slow strain rate stress-strain curves there is a decrease in stiffness at an axial strain of 1% or so. This impression becomes clearer when the individual stress-strain curves are plotted. In contrast the fast strain rate stress-strain curves in Fig. 6 show no such tendency.

6. CONCLUSIONS

The results of the testing programme demonstrate that an increase in the rate of strain produces an increase in the undrained shear strength of Waitemata residual clay. With the 750-fold increase in the strain-rate, the increase in the peak and large strain undrained shear strengths were 70% and 85%, respectively. Strength differences of this order are clearly capable of contributing to the explanation for the overprediction by the Hiley formula of the capacity of driven piles in Auckland clays.

For axial strains up to about 2% the undrained moduli for both rapid and slow rates of strain were very similar. The stress-strain curves for the slow tests showed a decrease in stiffness at about 1% axial strain which was not evident in the stress-strain curves for the rapid strain rate tests.

A further interesting aspect of the results is the difference in overall shape of the stress-strain curves. A clear peak occurs in the stress-strain curves for the slow strain rate tests at about 5 to 7% axial strain. In the rapid strain rate tests the maximum shear stress is reached at a larger strain but then followed with a plateau where the resistance remains approximately constant till a strain of about 16% is reached.

The water content of the specimens tested ranged between 42 and 54%. When plotted against water content the peak undrained shear strengths exhibited considerable scatter and no clear relationship was evident. Such scatter in both water content and undrained shear strength is typical of the residual clays around Auckland derived by in situ weathering from Waitmata Group soft rocks.

7. ACKNOWLEDGEMENT

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8. REFERENCES

Ahmed-Zeki, A. S. (1996) *The effects of rate of strain on the undrained shear strength of a residual Waitmata cohesive soil*, Project X report submitted in partial fulfilment for the requirements for the degree of Master of Engineering Studies, Department of Civil and Resource Engineering, University of Auckland.

Farr, J V. (1990). *One-Dimensional Loading-Rate Effects*, JGED, ASCE,Vol. 116, No. 1, pp. 119-135.

Fitch, N. R. (1996) *Experimentally derived relationship between undrained shear strength and driven pile sets*, Project D report submitted in partial fulfilment for the requirements for the degree of Master of Engineering, Department of Civil and Resource Engineering, University of Auckland.

Kulhawy, F. H. and Mayne, P. W. (1990). *Manual on Estimating Soil Properties for Foundation Design*, Rep. No. EL-6800, Electric Power Res. Inst., Palo Alto, Calif.

Ladanyi, B. (1977) Discussion on *Undrained shear stress behaviour of clays*, JGED, Vol. 103 GT 8, pp. 933-934.

Lapsih, E. B. and Melville-Smith, R. W. (1974) *Technical report in support of draft specification for driven short timber pile foundations for light timber frame construction*, CLC Consulting Group, Auckland.

Meyer, V. (1997). *Low Stress Behaviour of Waitemata Clay*, Ph.D thesis, Dept. Civil & Resource Eng., University of Auckland.

NZS 4402 1986. *Methods of Testing Soils for Civil Engineering Purposes*, Tests 6.1.1 & 6.2.1, Standards Association of New Zealand, Wellington.

NZS:3604 1990. *Code of practice for light timber frame structures not requiring specific design*, Standards Association of New Zealand, Wellington.

Penumadu, D. and Chameau, J-L. (1997). *Strain Rate Effects in Model Pressuremeter Testing*, JGGED, ASCE, Vol. 123, No. 11, pp. 1051-1059.

Schmertman. J. H. (1975). *Measurement of In-situ Shear Strength. State of the art Report*, Proc. ASCE Conf. On In-situ Measurement of Soil Parameters 2, pp. 56-137.

Sheahan, T. C., Ladd, C. C. and Germaine, J. T. (1996). *Rate-Dependant Undrained Shear Behavior of Saturated Clay*, JGGED, ASCE, Vol. 122, No. 2, February, pp. 99-108.