Cyclic Undrained Stiffness of Stiff Clay and Volcanic Ash

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SUMMARY: This paper will present results of strain and stress controlled cyclic undrained triaxial tests on specimens of Auckland clays and on volcanic ash specimens from Tauranga. The change in shear modulus and damping with strain amplitude over a range of ±0.1% to ±2% will be presented. Most of the tests were done at frequencies of 0.2 Hertz. In addition monitoring of the pore pressure change, which were measured at mid-height of the specimen, will be discussed.

1. INTRODUCTION

The cyclic undrained stiffness of soils is important in the prediction of the response of a site to earthquake excitation. It is also important for consideration of dynamic soil structure interaction both for footing and raft foundations and for the cyclic lateral stiffness of pile foundations. It is well established that the cyclic behaviour is very dependent on shear strain amplitude (γ). Two parameters are commonly used to characterise the soil properties, these are apparent shear modulus (G) and equivalent viscous damping ratio (D) both of which are defined in Fig. 1. As the shear strain amplitude increases the apparent shear modulus decreases and the damping ratio increases. It has long been established that the values of these properties are independent of cyclic frequency, at least in the range of interest of interest in earthquake engineering, Taylor (1971).

This paper presents some cyclic stiffness and damping data for clays from two sites in Auckland and a volcanic ash from a site in Tauranga which demonstrate the effect of strain amplitude. The intention of the tests reported was to investigate the rate of decrease in G with increasing strain amplitude and the effect of the number of cycles. It is concluded that degradation is unlikely to be significant for the Auckland clays but that the volcanic ash behaves more like a sand than a cohesive soil in that it exhibits a substantial increase in pore water pressure and a degradation of stiffness during undrained cyclic loading.

The current understanding of the undrained cyclic properties of soil has evolved over the last two or three decades. An early source of information is the report of Seed and Idriss (1970). Using data available in the late 60's this proposed that one G - γ relation and one D - γ relation are representative of the behaviour of clays, a similar pair of relations was thought to be appropriate to the properties of sand. The current understanding is a little more complex in that the G - γ relation for clays is now thought to be a function of plasticity index, Sun et al (1988), the form of their relation is presented in Fig. 2. These curves have become important in understanding site effects which were a most significant part of the response of the local soils to the Mexico City earthquake in 1985 and the soils in Oakland during the Loma Prieta earthquake of 1989. Damping ratio values for clays may also be a function of plasticity index but to date no definite proposal has emerged and the scatter of the data is about the same as presented by Seed and Idriss in 1970, this range of damping values is presented in Fig. 3.

The ordinate in Fig. 2 is the ratio G/G_{max}. The G_{max} used to normalise the strain dependent shear modulus is the value observed at very small strain amplitudes. This is the shear modulus associated with the passage of elastic shear waves, it has been found that for shear strains of the order of about 10^{-4} to 10^{-3} % most soils behave elastically. It is well known that G_{max} is a function of the void ratio of the soil and the square root of the mean principal effective stress. This square root relation will be used in the paper to correct for the change in stiffness of the volcanic ash as the pore water pressure increases.

Apparent Shear Modulus: G = slope AA'

Equivalent Viscous Damping Ratio:

D = Area of loop / (4π area of Δ OAB)

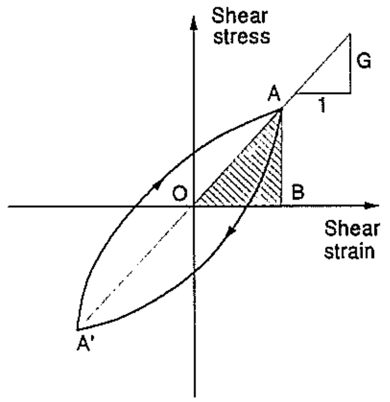

Figure 1 Definitions of the apparent shear modulus and equivalent viscous damping ratio.

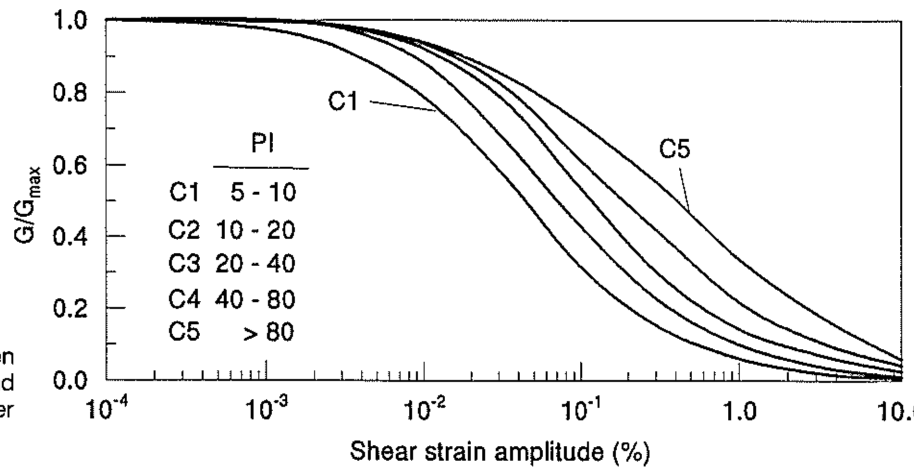

Figure 2 Relation between apparent shear modulus and strain amplitude for clays (after Sun et al (1988)).

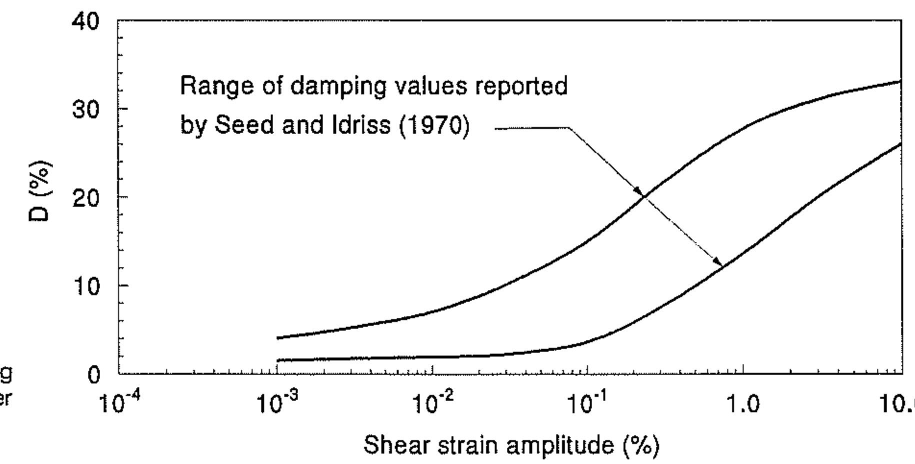

Figure 3 Range of damping values observed for clays (after Seed and Idriss (1970)).

2. TEST PROCEDURE

The soil specimens were cut from block samples 200mm high and by 203mm in diameter. These were recovered from the site by pushing a steel sampling tube into a prepared surface at the site. The tubes are fitted with a removable cutting shoe and driving head. After pushing into the soil the sampling tubes are recovered by hand digging, the cutting shoe and the driving head removed, and the ends sealed with rubber sheeting held in place with clamped endcaps. The samples are stored in the laboratory until the time of testing when the block is extruded from the tube in a hydraulic press and the sample cut into four with a wire saw. In this way four 75mm diameter by 150mm tall specimens are obtained from each sample.

A back pressure of 700 kPa was used to ensure saturation of the specimens. The frequency of cycling was 0.2 Hz. The majority of the tests were strain controlled but a few stress controlled tests are also reported. The strain controlled apparatus was originally designed by Taylor and Bacchus (1969). The pore pressure response in the soil was monitored during the tests using a miniature pressure transducer mounted at mid-height of the specimen.

Strain control was applied externally to the triaxial cell so any bedding errors would introduce errors in the measurement of the shear modulus. Consolidation of the specimen was assumed to take care of any such potential problems. The data was gathered using a personal computer (AT clone) fitted with an analogue to digital card. This was capable of recording four channels of data at a rate of several thousand cycles per second. For each loading cycle 30 data sets were recorded and each phase of the test continued typically for 40 cycles. The computer software was able to record the transducer readings as well as plot the load deformation loops to the computer screen during the tests. Subsequent to the test more detailed processing of the data was done. In finding the apparent shear modulus for each loop, which from Fig. 1 is a secant modulus, the end points need to be identified. Some refinement was applied to the data at the turning points before the G and D values were calculated.

The test procedure for each specimen, after back pressure saturation and consolidation, was to start with the smallest strain amplitude desired (in the case of a strain controlled test) or the smallest cyclic shear stress (for a stress controlled test), and apply the first 40 or so cycles and pause to process the data etc. The next strain amplitude was set and another series of 40 or so cycles then applied. Generally there was no reconsolidation of the specimens between tests at different strain amplitudes. The only exceptions being at the end of the testing of the volcanic ash where the strain amplitude was decreased for the last two loading phases and consolidation to dissipate the excess pore water pressure was allowed prior to the final loading phase.

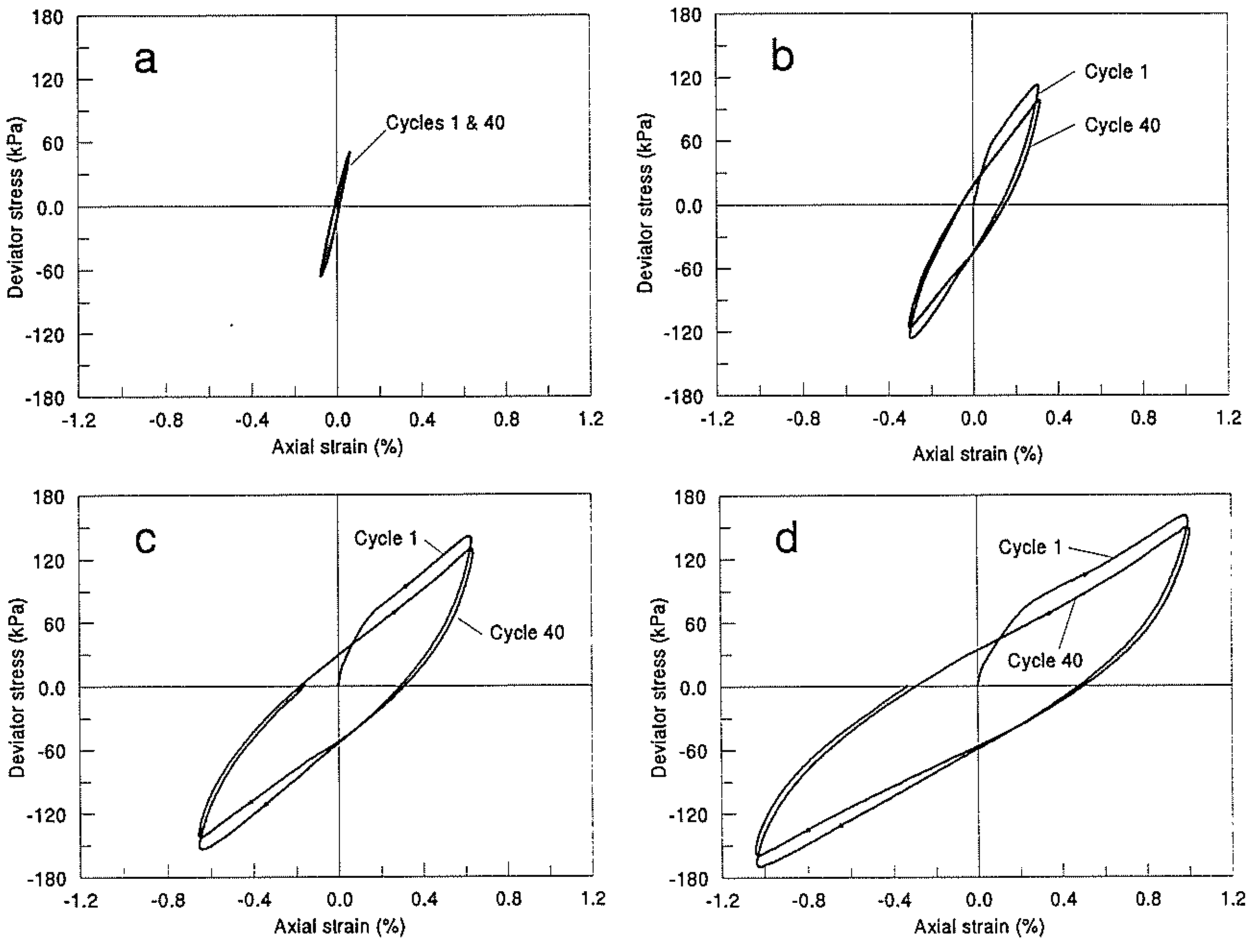

Figure 4 Stress strain loops from strain controlled tests on Auckland clay. (a) strain amplitude 0.07%, (b) strain amplitude 0.32%, (c) strain amplitude 0.64%, (d) strain amplitude 1.03%.

3. AUCKLAND CLAYS

Two clay samples from Auckland were tested, one from Sunset Rd. on the North Shore and the other from a subdivision at Manurewa. The water content of the specimens after back pressure saturation and hydrostatic consolidation to effective confining pressures in the range 100 to 200kPa was about 40%. The plasticity index of the clays was in the 30 - 40% range and the liquidity index was about 0.1. The undrained shear strength of the clays, determined with a laboratory vane, was about 75 to 100kPa. Strengths of this order and the liquidity index suggest that the clay would behave as a heavily overconsolidated material. Clays with roughly this range of properties are one of the characteristic materials found in the Auckland area.

Some typical stress strain loops for the Auckland clays are plotted (to the same scale) in Fig. 4a to d. The shear stress plotted is the deviator stress, $\sigma_1 - \sigma_3$, and the strain is the axial strain of the specimen (using this set of axes the slopes of the lines between the ends of the loops in Figs. 4 and 7 are three times the value of G). It is apparent from Fig. 4 how the apparent shear modulus decreases with increasing strain amplitude whilst the damping ratio increases. In addition it is evident that there is some degradation of the shear modulus as the number of cycles increases. This is barely perceptible for the small cyclic shear strains in Fig. 4a but becomes more apparent as the strain amplitude increases. At the end of some several hundred cycles of undrained cyclic loading a small increase in pore water pressure, of the order of 20kPa, was noted. This is not altogether surprising as static undrained shearing of a stiff clay would not generate large increases in pore water pressure. Since the number of cycles of undrained loading applied to the clay specimens was much greater than that expected in an earthquake and, as the rise in pore water pressure was small, the slight degradation in stiffness can be considered as a fatigue phenomenon.

In Fig. 5 the effect of strain amplitude on the apparent shear modulus for the first few cycles of each loading stage is plotted along with part of the C2 and C3 clay curves from Fig. 2. If the apparent shear modulus at the end of the 40 cycles for each loading stage is plotted the points lie just beneath those in the figure. As has been explained above the curves in Fig. 2 are normalised with respect to the small strain shear modulus of the soil, a value that was not determined for the Auckland clays. The assumption made to circumvent the lack of this piece of information was to take the shear modulus result for the smallest strain amplitude and attach it to the C2 clay curve. These points are labelled starting points in the figure. Having made this assumption it is of interest that the apparent shear modulus plots for the Auckland clays

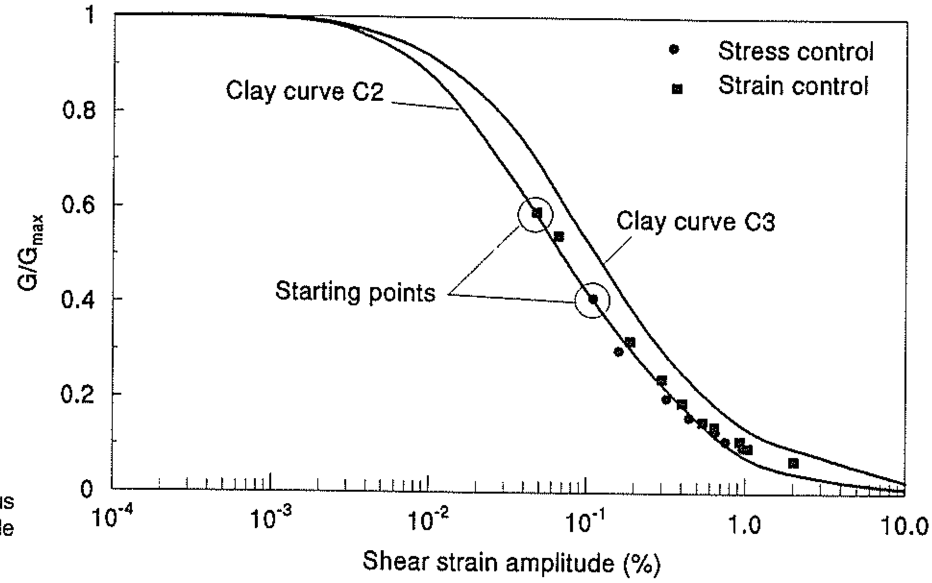

Figure 5 Shear modulus variation with strain amplitude for the Auckland clays.

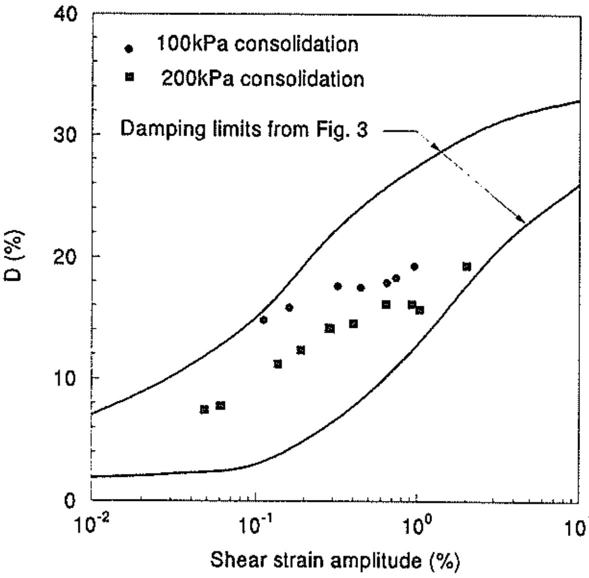

Figure 6 Damping values for the Auckland clays.

follow the trend of the C2 and C3 clay curves which cover the plasticity index of the clay. As the increase in pore pressure was small during the course of the tests no correction was applied to the apparent shear modulus values to account for changes in effective stress.

In Fig. 6 the damping values of the clays are plotted along with part of the range of damping values from Fig. 3. The results for the Auckland clays lie within the range reported by Seed and Idriss (1970).

4. TAURANGA VOLCANIC ASH

A specimen of volcanic ash from Tauranga was subject to cyclic undrained loading. The ash had a natural water content of 62%, a plasticity index of 25%, a liquidity index of 0.6 and an undrained shear strength similar to that of the Auckland clays.

Figure 7 shows the effect of strain amplitude on the cyclic stiffness of the volcanic ash soil. Two sets of results are given for strain controlled tests and two for stress controlled tests. In the strain controlled tests the shear modulus decreases as the number of cycles increases and this is manifested with a decrease in the shear stress at the extremity of the loop, Figs. 7a and 7b give examples of this which are very similar to the behaviour seen in Fig. 4. In the case of stress controlled cycling the decrease in apparent shear modulus with increasing number of cycles is accompanied with an increasing range of shear strain for the loops, Figs. 7c and 7d give examples of this. In the case of the loops plotted in Fig. 7d there is a very rapid degradation in stiffness which is due in part to the large cyclic shear stresses applied to the specimen and also to the generation of pore water pressure discussed below. The specimen for which results are plotted in Fig. 7c and d was consolidated to an effective consolidation pressure of 100kPa whereas the specimen for Fig. 7a and b was initially consolidated to 200kPa.

Unlike the Auckland clays the volcanic ash soils demonstrated a considerable generation of excess pore water pressure during the cyclic loading. After several hundred cycles of undrained loading the pore water pressure had risen to 85% of the initial effective consolidation pressure. In this way the cyclic response of the ash is similar to that of saturated sand. The increase in pore water pressure must have been at least a contributor to the decrease in the shear modulus of the ash. Thus there may be two mechanisms leading to the decrease in the shear modulus of the ash with increasing numbers of cycles - degradation (a type of fatigue as with the clays) and the decrease in effective confining stress because of the rise in pore water pressure.

In Fig. 8 the variation of the apparent shear modulus with strain amplitude is plotted for the tests on the volcanic ash. Part of clay curve C2 is plotted for comparison and the first strain amplitude result is attached to the curve in the same manner as has been described above for the clay. Also in Fig. 8 the strain controlled results are supplemented with those for the stress controlled tests on the ash and the apparent shear modulus values are "corrected" for the decrease in effective stress caused by

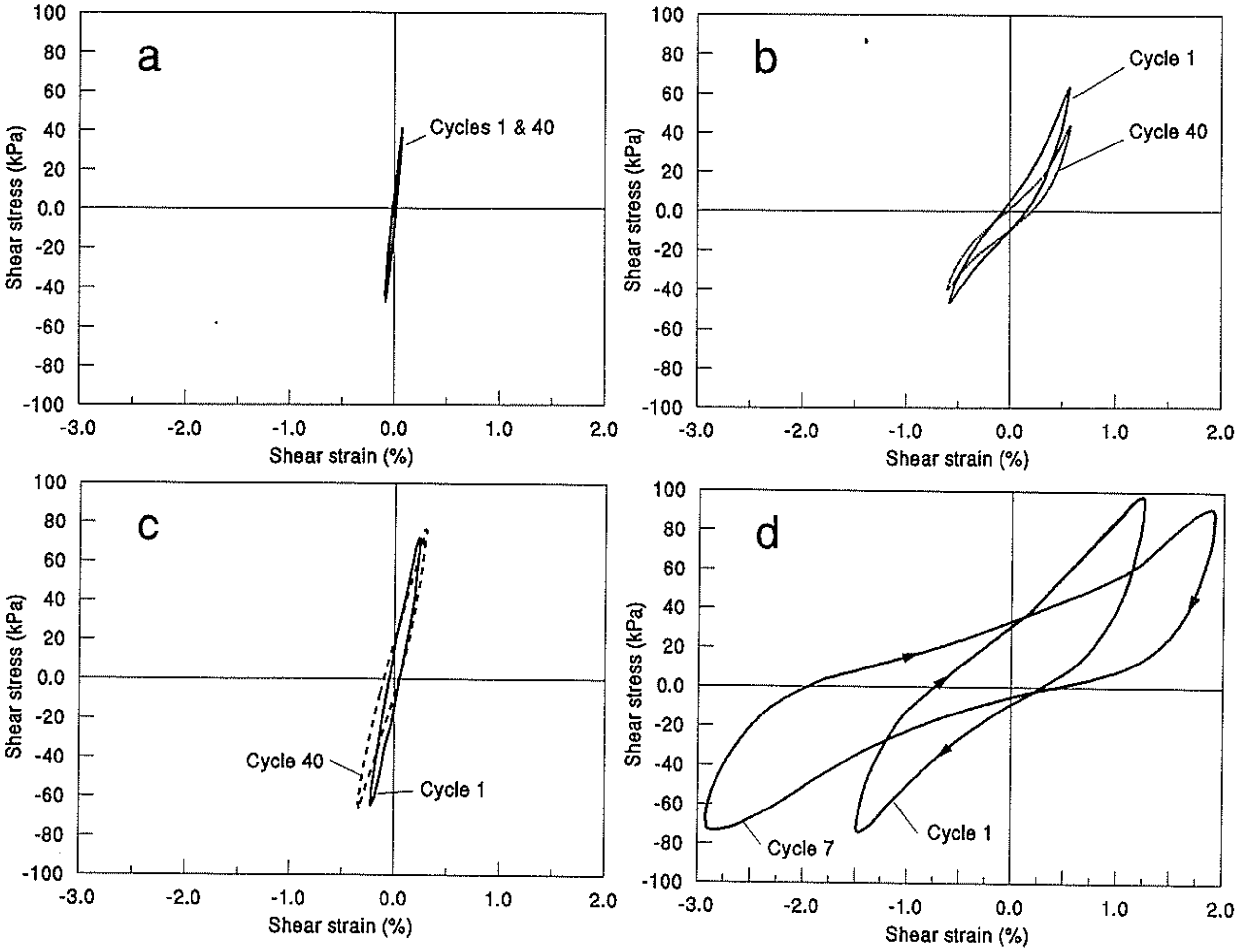

Figure 7 Stress strain loops for tests on volcanic ash. Strain controlled tests at 200kPa consolidation pressure; (a) strain amplitude 0.09%, (b) strain amplitude 0.60%. Stress controlled tests at 100kPa consolidation pressure, (c) and (d).

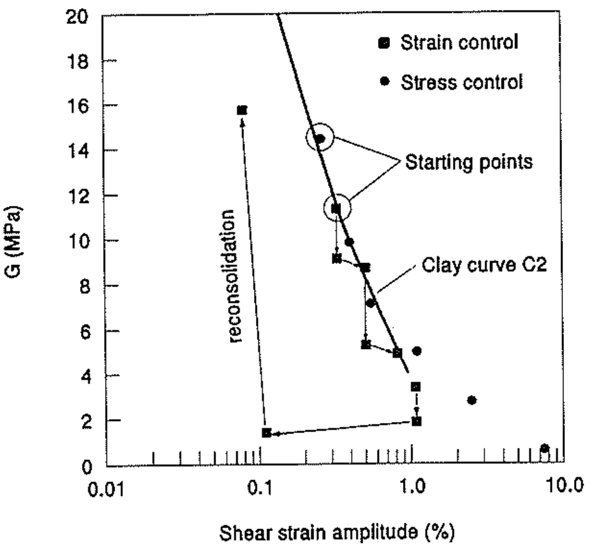

Figure 8 Shear modulus variation with strain amplitude for the volcanic ash, G_{max} corrected to account for the decrease in effective stress.

the increase in pore water pressure. This is done with the square root relation between small strain shear modulus and effective confining pressure mentioned in the introduction. At the end of one of the tests on the ash the effect of decreasing the strain amplitude was investigated. Firstly a decrease to 0.11% , whilst the large excess pore pressure is still within the soil, and then a further reduction to 0.08% after reconsolidation back to an effective stress of 200kPa. It is apparent that the reconsolidation does not allow the ash to regain the stiffness at the commencement of the cyclic loading. Thus several hundred cycles of undrained cyclic loading of the ash causes a permanent degradation in the shear stiffness of the soil.

Figure 9 plots the damping data for the volcanic ash. It is apparent that for the early stages of the cyclic loading the damping behaviour is within the ranges shown in Fig. 3 but as the number of load cycles increases and the strain amplitude is increased the damping does not increase as expected. Possibly this is a consequence of degradation of the internal structure of the ash reflected by the degradation in shear stiffness and increase in pore water pressure.

5. DISCUSSION

The cyclic loading results for the Auckland clays indicate that for the larger strain range the G - γ and D - γ curves in Figs. 2 and 3 are applicable. Although an assumption was needed to locate the results on the particular curve in

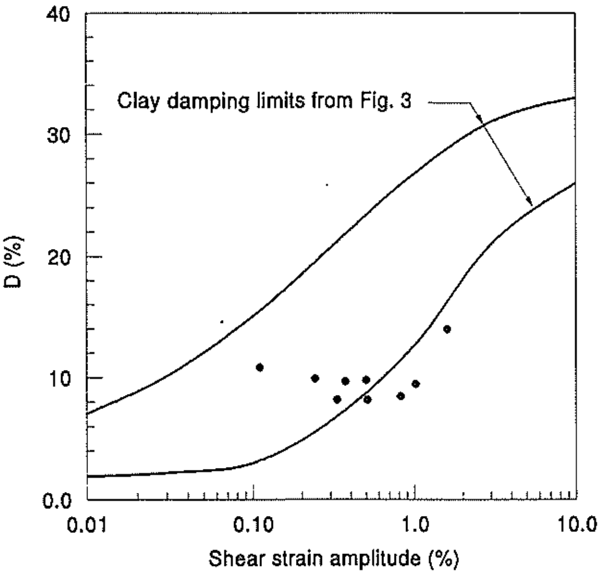

Figure 9 Damping values for the volcanic ash.

Fig. 5 the slope of the relation follows closely those in Fig. 2, thus even if the location of the curve for the Auckland clays in the G - γ plot is in doubt it is apparent that the rate of decrease in apparent shear modulus with increasing strain amplitude is the same as the curves in Fig. 2. It is of interest that the stiffness and damping do not seem to be greatly affected by the number of cycles of loading even though the total number of load cycles was several hundred by the end of the test and well beyond the number expected in an earthquake. Although there is some degradation in stiffness this is small and occurs at the higher strain ranges. Finally there is a negligible change in pore water pressure in the clay for each cycle. These features of the clay mean that the estimation of site response or calculations of soil structure interaction during earthquake loading can be based on levels of shear strain only and does not require changes to incorporate degradation with the number of cycles or with changes in effective stress following the generation of pore water pressure.

The test results for the volcanic soil indicate a sensitivity to cyclic loading much greater than that of the clay. During the cycling the pore water pressure increases and the stiffness of the ash decreases as can be seen with the stress strain loops in Fig. 7, these changes occur in a few tens of cycles. In this way the volcanic ash has more in common with the cyclic behaviour of a saturated sand than with the cyclic behaviour of the Auckland clay, although, on the basis of soil classification data, one would expect the ash to be more closely allied with the clay. Similarly, at small cyclic strains, the damping in the ash starts with values to be expected from Fig. 3 but the damping does not increase as expected as the strain amplitude increases. Finally when one of the ash specimens was reconsolidated after the pore water pressure had increased to 85% of the effective confining pressure and further cyclic loading conducted at small shear strains the initial stiffness was not recovered. These aspects of the volcanic ash behaviour suggest that the soil has a sensitive structure that is degraded by successive cycles of loading. This means that the estimation of the response of the material to earthquake excitation is, in principle, more complex than that of the Auckland clays. The liquidity index may be a useful parameter for categorising the likely pore pressure response of cohesive soils under cyclic loading. The liquidity index, calculated using the water content of the specimens after back pressure saturation and consolidation, was about 0.1 for the Auckland clays and 0.6 for the Tauranga ash.

The assumption used to locate the initial test result on the G/G_{max} relations presented in Fig. 2 is the most doubtful part of the procedures followed in this paper. A direct measurement of the small strain stiffness of the soil would be a useful addition to the test procedure. At present the installation of bender elements, Dyrik and Madshus (1985), in the top and bottom platens of the triaxial apparatus is being considered. In defence of the of the procedure of simply attaching the data points to the appropriate curve in Fig. 2 the free vibration torsional test results of Larkin and Chan (1992), presented to this conference, support the assumption that the G - γ relation for NZ volcanic soil is very similar to the clay curves. For the Auckland clays similar free vibration torsional tests by Plested (1985) indicate that the form of the curves follows those in Fig. 2.

6. CONCLUSIONS

The following conclusions are reached from the results presented herein:

(i) The stiffness and damping of the Auckland clays was not greatly affected by the number of cycles. This means that earthquake response calculations can be attempted with properties that depend only on shear strain amplitude, i.e. they can be based on a total stress method of calculation.

(ii) For the Tauranga volcanic soil the resistance to cyclic loading was more complex as a rapid build up in pore water pressure, with a consequent loss in stiffness, was observed. Also the damping available did not increase with strain amplitude.

(iii) The results of Fig. 6 suggest that the Auckland clays have damping properties which are adequately described by Fig. 3. Figure 5 shows that the rate of change of apparent shear modulus with strain amplitude for the Auckland clays follows closely that of curves C2 and C3 in Fig. 2. This is an important conclusion as recent earthquake data has coupled enhanced site response, such as occurred at Mexico City in 1985 and in Oakland in 1989, with soils having an extended range of strains over which the behaviour is elastic with a G - γ relation to the right of the C5 curve in Fig. 2.

7. ACKNOWLEDGEMENTS

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Wesley kindly supplied the index data for the soils tested and suggested that the liquidity index might be a useful parameter for categorising the cyclic response of soils.

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