Soil strength from geophysical measurements for soft clays

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ABSTRACT

Knowledge of seabed soils is essential if offshore and nearshore structures are to be safely designed and properly built. A large part of the commercial and operational risk involved relates to uncertainties about the soil properties at the site. It is therefore important to perform sufficient investigation to evaluate these risks thoroughly. Geophysical surveys are required to understand the nature and characteristics of the seabed. Site specific correlations between soil strength and various geophysical measurements can be developed, but a controlled laboratory study is required to highlight variability in these correlations for a range of geotechnical material.

This work presents the development of a framework for correlating sediment strength, undrained shear strength, for soft clays to geophysical measurements, primarily shear wave and body wave velocities. Small strain measurements using elastic waves provide valuable soil information without altering the soil fabric. The small strain shear modulus (Gmax) is an indicator of many soil properties such as density, soil stiffness, sample disturbance, and can be calculated using the shear wave velocity (Vs) values measured by bender elements. Influence of variables such as soil density, confining stress, and stress history on shear modulus are also examined.

KEY WORDS: shear modulus, shear strength, soft clay, bender elements

INTRODUCTION

Knowledge of seabed soils is essential if offshore and nearshore structures are to be safely designed and properly built. A large part of the commercial and operational risk involved in these works relates to uncertainties about the properties of the soil at the site. It is therefore necessary to perform sufficient investigation to evaluate these risks thoroughly.

Many geophysical techniques are available to the engineer to perform such investigations. Several sensors currently provide geophysical information without requiring direct contact with the seafloor. Sub-bottom profiling, swath bathymetry, electro-resistivity, seismic refraction, and electromagnetic sensors are all examples of these techniques, several of which are frequently utilized with Autonomous Underwater Vehicles (AUV) and Remotely Operated Vehicles (ROV). As site specific correlations between soil strength and various geophysical measurements can be achieved, a controlled laboratory study is required to develop a framework for correlating sediment strength; undrained shear strength for clays to geophysical measurements, primarily shear wave and body wave velocities.

This work aims to quantify the magnitude and material uncertainty when characterizing the engineering properties (void ratio, porosity, water content, density, strength) of soft clay using geophysical methods. Shallow sediments or sediments within 7 m of the seafloor are targeted for this research.

BACKGROUND

Site investigations for offshore structures, nearshore structures and dredging works are necessary to acquire data that will facilitate successful foundation design, site or route selection, choice of foundation type, dimensioning, installation and operational integrity of the proposed structure. A geophysical survey is required to understand the nature or characteristics of the seabed. Typically, a combination of techniques such as echosounding, side scan sonar, reflection seismic systems and electrical resistivity systems are used.

Along with geophysical surveys, it is also necessary to define site specific geotechnical data such as strength parameters, consolidation characteristics, permeability etc all of which are usually carried out by either insitu tests (traditionally Cone Penetrometers) or offshore and onshore laboratory tests on cored samples. Profiles of triaxial compression/extension and undrained shear strength values are determined on representative samples, for offshore foundation design in soft clays (Lunne, 1976; Andersen, 2005).

Soil strength can be related to geophysical measurements using the small strain shear modulus (G_o) which is an indicator of many soil properties. The shear modulus can be computed from the soil density (ρ) and measured shear wave velocity (V_s) through the soil using the equation 1.

$$G_o = V_s^2 \rho \quad (1)$$

The small strain shear modulus or initial shear modulus, G_{max} characterizes stiffness when soil response is linear elastic and its importance in prediction of soil response and soil structure development has been studied extensively (Youn, 2008; Dyvik, 1985).

Several factors affect small strain shear modulus: strain level, effective stress state, OCR, void ratio, soil macro- and micro-structure, cyclic behavior, damping, consolidation and ageing (Lee, 2005; Leong 2009; Landon, 2007). It is well established that G_o is a function of soil matrix, and since waves transmitted thorough solid media travel faster, lower soil void ratios lead to higher shear modulus values.

There are a number of empirical equations which relate G_o with known soil parameters such as void ratio, OCR and stress state for deep sediments (Hardin, 1963; Hardin, 1968; Marcuson, 1972; Houlsby, 1991; Jamiolkowski, 1994; Shibuya, 1997). There is however, a lack of information in the literature about the small strain properties of shallow saturated sediments in the top 7~m of the seafloor. The correlation of stiffness, G_o with soil strength is not well understood in these low confining pressures. This information is vastly useful to interpolate between, and extrapolate from, borehole data for applications such as pipelines which traverse across large areas of the seafloor and require detailed information about shallow sediments. This is also applicable to offshore wind farms which require several separately founded structures over a large area.

A common laboratory method for determining G_o is through the use of piezo-ceramic plates, known as bender elements. Bender elements have been used to measure shear wave velocity in soils starting the 1970's (Shirley, 1978; Dyvik, 1985; Agarwal, 1991; Viggiani, 1995; Santamarina, 2001; Pennington, 2001; Landon, 2007). Piezoelectric plates generate a voltage when it is mechanically stressed and oscillate when it is excited by a voltage source.

Based on this principle the bender elements are placed on the two ends of the triaxial sample, a shear wave or compression wave pulse is generated by bender elements at one end (bottom) and this wave propagates along the specimen length; a receiver element at the other end (top) of the sample picks up the wave and generates an output voltage. The bender elements provide a more direct and non-destructive measure of shear wave velocity of a soil. Another advantage is that the shear wave velocity can be monitored in conjunction with other soil parameters.

Many methods exist for the determination of the travel time of the shear and compression wave such as: travel time by direct arrival, travel time between characteristic points, travel time by cross-correlation, travel time using multiple arrivals, wavelet analysis, phase detection analysis (Brignoli, 1992; Viggianni, 1995; Jovicic, 1996; Arulnathan, 1998; Bonal et al., 2012; Airey, 2013). Cross correlation was found to give the most accurate and consistent results and was adopted for this research.

METHODOLOGY

Testing Equipment

The triaxial testing system used for this research was the GEOTAC TruePath system, which consists of the axial load frame, cell and pore pressure-volume flow pumps, instrumentation, data acquisition and control hardware and software. The axial load frame has a capacity of 4.45 kN (1,000 lb), and also provides deformation control with a calibrated screwjack. The position of the platen is recorded when the test is started and the change in platen position used to calculate deformation. The pore pressure-volume pump continuously measures the volume change in the sample during consolidation and by comparing axial and volumetric deformations, a feedback loop can automatically enforce K_o conditions (zero lateral strain) by varying the cell pressure and vertical load (Murali, 2014). Combinations of vertical and volumetric deformation rates can be used to control strain paths (Bishop and Wesley, 1975; Germaine and Ladd, 1988; Berre and Bjerrum, 1973).

Two sets of caps for the triaxial samples were equipped with piezoelectric transducers. The systems were manufactured by GCTS Testing Systems and fit the GEOTAC setup with some modification. Bender elements and p-crystals are installed in one set of caps, while both p- and s-crystals are installed on the other set. The bender elements protrude from the caps, while the s-crystals are mounted under the surface of the cap. Since the water lines align with ports in the bottom of the cell undrained testing can be carried out in the system.

Bender element are manufactured from lead zirconate titanate. Characteristics for the bender elements are:

- Capacitance = 4 E-10 farad
- Static Voltage = 35 Volt/Newton (full length)
- Max. voltage = 500 Volts
- Resistivity 1E8 ohm-m Curie Temp = 495 C

The system is completed by a Tectronix arbitrary function generator AFG320, a Tectronix oscilloscope TD3014B and TDS3GV and a Piezosystems PiezoLinear Amplifier as shown in Fig. 1.

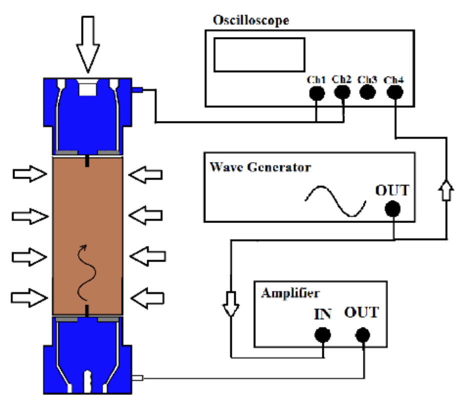

Fig. 1: Schematic of test setup along with triaxial bender caps.

The function generator provided a burst sine output signal at a frequency of 50 kHz. It was found to give the least amount of electrical interference with the other devices in the laboratory. The driving signal was amplified and served as the input for the bottom cap bender element. The receiver signal from the top cap is sampled by the oscilloscope. It was very useful to connect the electrical ground of all components of the measuring system to the metal parts of the cell housing.

Material Characterization

The soil that was tested in this research program was Kentucky Special kaolin. Table 1 presents the results of index properties and specific gravity of the kaolin that was tested. All laboratory tests were conducted in accordance with the American Society for Testing and Materials (ASTM) standard for each test.

Table 1: List of properties of kaolin tested.

Property	Value
Trade name	Kentucky Special kaolin
Manufacturer	Aardvark Clay & Supplies
Specific gravity, G_s	2.6
Liquid limit, LL	61
Plasticity Index, PI	29

Constant Rate of Strain (CRS) consolidation testing was carried out for all clay samples. The specimens were prepared by mixing a ratio of soil and water resulting in an initial water content of approximately 1.5 times the liquid limit of the clay. Since the clays were consolidated from a slurry state, the samples could not be placed in the steel ring according to the conventional methods of cutting an undisturbed sample to size. The clay was placed into the steel ring in stages, as shown in Fig. 2. Once the sample was prepared (stage 4), it was placed into the cell and CRS consolidation tests started.

Fig. 2: Sample preparation for CRS tests.

This test method was used to determine the magnitude and the rate of consolidation of the saturated cohesive soil samples. This was achieved by applying a constant rate of axial strain in compression while the sample is restrained laterally and drained axially to one surface (ASTM Standard D4186, 2006). These clays were tested at strain rates varying from 5 % to 7 % so as to develop a small amount of excess pore pressures during the consolidation. Fig. 3 shows the results of CRS tests on the kaolin and table 2 lists the coefficients of consolidation and compression indices of the clay. This was used to compute the time required for bench consolidation while preparing the triaxial samples.

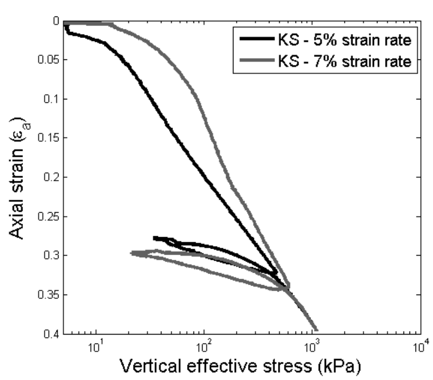

Fig. 3: CRS test results on Kentucky Special (5% and 7% strain rate).

Table 2: Consolidation properties of Kentucky Special clay.

Strain rate (%/hr)	Coefficient of consolidation C_v (ft^2/day)	Compression Index C_c
5 %	0.006	0.543
7 %	0.03	0.708

Sample Preparation

The samples for the triaxial tests were prepared similar to the process used by Germaine (1982) for resedimented Boston blue clay. Dry powdered clay was mixed with water in a soil mixer to produce a slurry. The slurry was thickened by placing it in a 100^{o}C oven and removing it every hour for 5 min to stir it and let it cool. This process of stirring and cooling the soil ensured the soil is only thickened and not dried. Once the slurry had thickened, it was slowly scooped into a 2 in inner diameter split mold fitted with a membrane and closed at the bottom end with a porous stone. This process was carried out slowly to prevent the formation of air pockets and voids. The slurry was incrementally loaded with dead weights until the desired vertical effective stress for each test was reached. Since these samples were prepared from slurry, the time required for consolidation was quite long. Removal of these samples before the time specified resulted in these samples failing when they were installed in the triaxial cell. Once the target stress was achieved, the load was removed. The sample encased in the membrane was then removed from the split mold and treated as a specimen for testing. Table 3 shows the initial water content, initial void ratio and unit weight for the clay.

Table 3: Initial water water content, void ratio and unit weight of KS.

Property	Value
Initial water content, w_i	85 %
Initital void ratio, e_o	2.34
Unit weight, γ	14.95 kN/m^3

Laboratory Testing Program

The laboratory tests carried out in the testing program consisted of K_o consolidated undrained compression triaxial tests (CK_oU) as presented in table 4. Each test consisted of four phases: seating, backpressure, consolidation under K_o conditions and shearing. The steps are briefly described in this section.

Table 4: Tests carried out in this testing program.

Test number	Vertical effective stress, σ'_v (kPa)	Over consolidation ratio, OCR
Test 1	60	1
Test 2	45	1
Test 3	30	2
Test 4	15	4
Test 5	7.5	8

Once the prepared sample was installed in the triaxial chamber, a seating pressure of 5 kPa and a seating load of 6.67 N (1.5 lbf) was applied. Drain lines were flushed multiple times to ensure no air bubbles were trapped in the pore pressure system. Each specimen was back pressured to about 138 kPa (20 psi) for 12 to 15 hrs, after which the B-value was checked. The test proceeded to the consolidation phase if the B-value was greater than 0.95. This step was extremely important as the presence of air pockets in the sample diminished the output signal on the oscilloscope.

The consolidation part of the test was controlled manually. The K_o for this kaolin was first determined by carrying out one test where the consolidation phase was completely automated. The value of K_o for kaolin was found to be 0.78. The K_o values for unloading until OCR = 8 is plotted in Fig. 4. The specimens to be tested at various OCRs were then unloaded manually to the desired value depending on the K_o value.

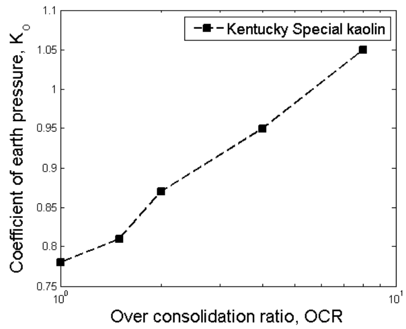

Fig. 4: K_o plotted vs OCR

These tests required the consolidation phase to be carried out manually as the bender and ultrasonic caps could not be immersed in water. Figs. 5 and 6 show examples of volumetric strain vs time and volumetric strain vs vertical effective stress curves during the manual consolidation process. The loads were applied in 3 to 4 increments and allowed to equilibrate after each increment. The K_o value during each of these increments was calculated and the ratio of confining pressure and vertical load was maintained accordingly. The strain rate used was approximately 1% per hour, which resulted in a consolidation time of 4 days. The specimen was assumed to be normally consolidated (NC) at the end of consolidation.

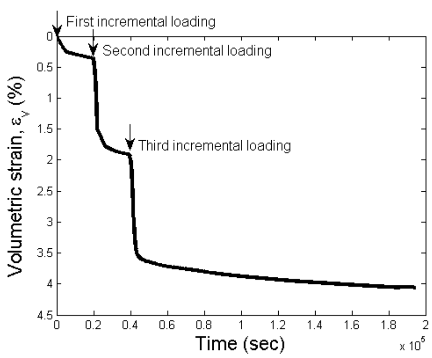

Fig. 5: Volumetric strain vs time showing manual incremental loading

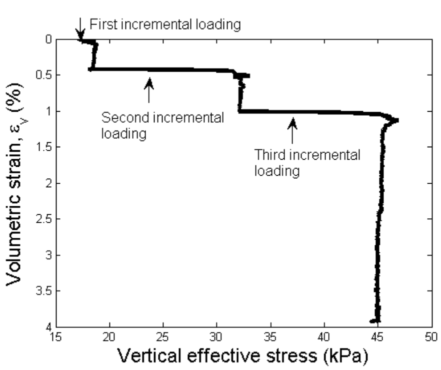

Fig. 6: Volumetric strain vs vertical effective stress

At this point in the test, the specimen was allowed to sit for 24 hours at the final stress state for the sample to reach equilibrium, to allow some secondary compression to take place.

After the consolidation phase, all specimens were sheared in compression at a strain rate of 5%/hr. During this phase s and p wave velocity readings were recorded at specific strain intervals.

TESTING RESULTS

Strength Results

Fig. 7 shows the stress strain curves for both the normally consolidated and over consolidated samples of kaolin tested. As seen in the figure all the normally consolidated samples fail at very low axial strains (<0.2%) and the over consolidated samples fail after 1% axial strain. Fig. 8 shows the excess pore pressure plotted vs axial strain. As expected the normally consolidated samples develop higher excess pore pressure and samples tested at OCR equal to 4 and 8 develop negative excess pore pressure.

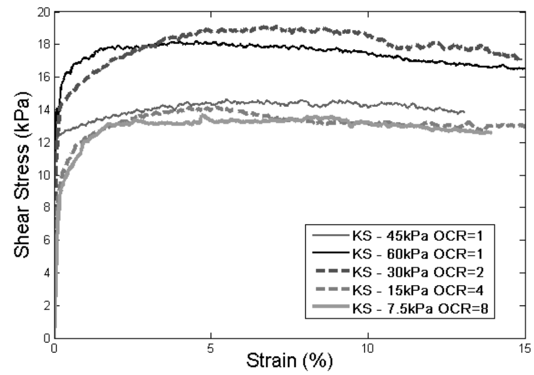

Fig. 7: Stress strain curves for KS clay

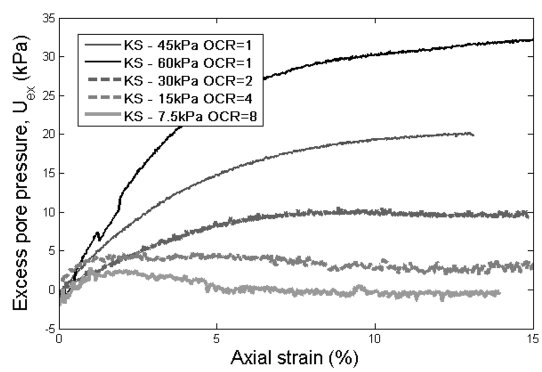

Fig. 8: Excess pore pressure generated for KS clay

Figure 9 shows the effective stress paths normalized with the maximum vertical consolidation stress (σ'_v) for OCR = 1, 2, 4 and 8 for Kentucky Special kaolin. As seen from the plot, the effective stress failure envelope has a slope of 24º.

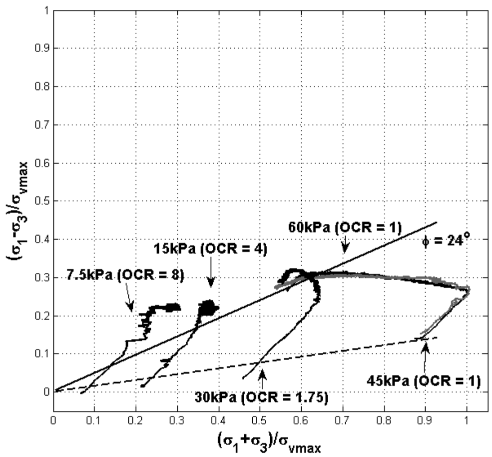

Fig. 9: Effective stress path for KS clay

Bender Element Results

Figure 10 shows the variation of the shear modulus and bulk modulus with respect to axial strain during the shear phase for each sample of Kentucky Special clay tested. As expected the shear modulus values decrease with decreasing confining pressure.

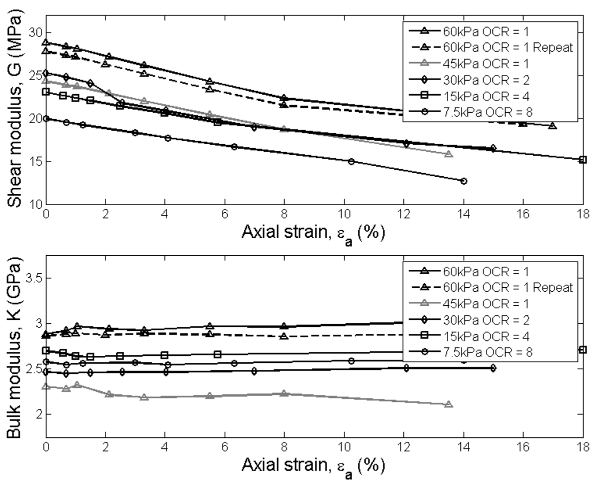

Fig. 10: Shear modulus and bulk modulus for KS clay

The bulk modulus remains relatively constant during the entire shearing process.

GEOPHYSICAL CORRELATIONS

This section provides an overview of how the geophysical data measured was co-related to the measured shear strength values. This document also details how the geophysical data was co-related to the mean stress, over consolidation ratio (OCR) and how the data from this project compared to past available research. The shear modulus (G_o) was obtained from the shear wave velocity reading measured at the completion of the consolidation phase just before the shearing phase. The density for each sample was calculated based on the void ratio of the sample just before testing.

Mean Stress

Hardin and Black (1963) started the research on the effects of confining stress and void ratio on G_o. Their experimental results on sand gave rise to the following empirical equation:

$$Go = A\, f(e)\sigma^n \quad (2)$$

Where A and n were constants varying for each soil type, f(e) was a function depended on the void ratio of the sample. They also applied the same equation to determine the vibration modulus of a normally consolidated clay (edgar kaolin). Hardin (1978) expressed the G_o values of soils subject to isotropic consolidation with the following equation:

$$Go = A\, f(e)\sigma_c^n p_r^{(1-n)} OCR^k \quad (3)$$

Where A is an empirical constant, f(e) is a function of void ratio, σ_c is the confining stress and p_r is a reference stress and OCR is over consolidation ratio.

Viggiani (1995b) obtained the soil parameters A and n for reconstituted samples of Specswhite kaolin by varying the over consolidation ratio and mean stress. Due to the type of tests carried out with a similar experimental setup it was easy to compare the data of Specswhite kaolin with the data of Kentucky Special kaolin obtained from this project.
Fig. 11 shows a log-log plot of normalized mean effective stress and normalized shear modulus for Kentucky Special kaolin samples along with the results of Specswhite kaolin (Viggiani, 1995b). The reference pressure was taken as 1 kPa to be consistent with the paper. The normally consolidated kaolin data points fall close to a perfectly straight line given by equation (4) after Viggiani (1995b). The over consolidated samples show a trend of modulus increasing with increase in OCR.

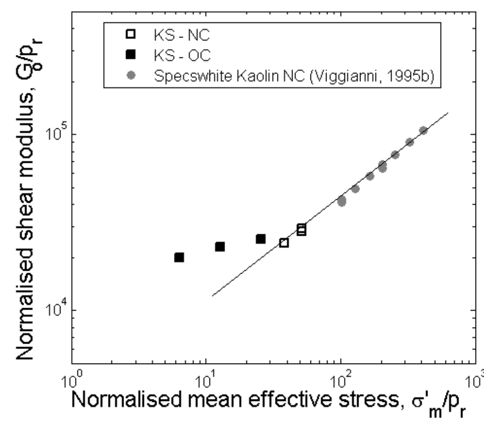

Fig. 11: Normalized mean stress plotted vs normalized shear modulus.

$$\frac{Go}{pr} = A\left(\frac{\sigma_m}{pr}\right)^n \quad (4)$$

The values of A and n are given in table 5 comparing the data of specswhite kaolin to Kentucky Special kaolin. The values of A and n depend on the value taken for the reference pressure (Viggiani, 1995b).

Table 2: Comparing coefficients A and n values with Viggiani (1995b)

Clay type	A	n	Reference
Specswhite kaolin	2088	0.653	Viggianni(1995)
Kentucky Special kaolin	2177	0.657	this research

The influence of void ratio, f(e) as expressed in equation 3 was not very prominent as all the samples were tested at a similar void ratio and thus no variation was observed since the effect was included in computing the coefficient A.

Strength

The shear strength values (S_u) were obtained from each samples stress strain curve during shearing. Failure was considered to be the maximum value for each sample from its stress strain curve. Fig. 12 shows the shear strength vs shear modulus (G_o) both normalized by vertical effective stress.

From the figure it can be seen that the normally consolidated specimens lie in a cluster around the same region and the over consolidated data points fall on a straight line. Similar to SHANSEP given by equation 5 (Wroth, 1984), Houlsby and Wroth (1991) proposed an equation (eq. 6) using initial shear modulus in place of shear strength.

$$(Su/\sigma'_v)_{oc} = (Su/\sigma'_v)_{nc}\ OCR^{m} \quad (5)$$

$$(Go/\sigma'_v)_{oc} = (Go/\sigma'_v)_{nc}\ OCR^{m1} \quad (6)$$

Where G_o is the initial shear modulus, σ'_v is the current vertical effective stress, OCR is over consolidation ratio. From Fig. 12, it appears that the exponent of OCR in both equations 5 and 6, m and m1 are similar if not identical suggesting that OCR affects both shear modulus and shear strength in a similar manner.

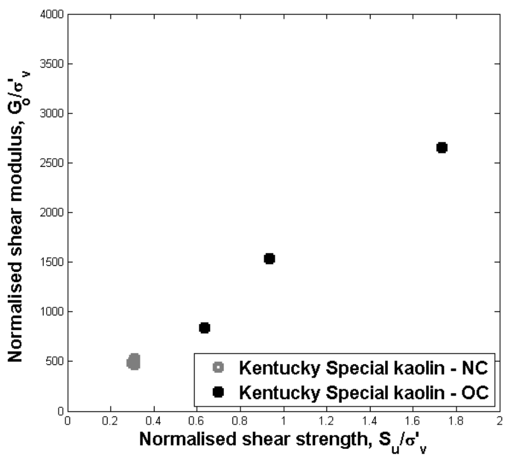

Fig. 12: Normalized S_u plotted vs normalized G_o.

FINAL REMARKS

The work described in this paper is experimental and consists of automated triaxial tests carried out on soft kaolin clay with measurement of shear wave and compression wave velocities using bender elements during testing. The principal purpose of this work was to examine the variation of G_o with the undrained shear strength of kaolin at low confining pressures. Additionally the data from this research was compared with existing data in the literature for parameters on G_o varying with mean stress, OCR and void ratio.

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REFERENCES

Airey, D. and Mohsin, A. K. M. (2013). “Evaluation of shear wave velocity from bender elements using cross-correlation.” *Geotechnical Testing Journal*, 36(4), 1–9.

Arulnathan, R., Boulanger, R. W., and Riemer, M. F. (1998). “Analysis of bender element tests.” *Geotech. Test. J.*, 21-2, 120131.

ASTM Standard D4186 (2006). Standard Test Method for One-Dimensional Consolidation Properties of Saturated Cohesive Soils Using Controlled-Strain Loading. *ASTM International*.

Berre, T. and Bjerrum, L. (1973). “Shear strength of normally consolidated clays.” *Proc. 8th Intl. Conf. on Soil Mech. and Found. Eng.*, Moscow, 1, 39–49.

Bishop, A. W. and Wesley, L. D. (1975). “A hydraulic triaxial apparatus for controlled stress path testing.” *Geotechnique*, 25 Issue: 4, 657–670.

Bonal, J., D. S. and McNally, C. (2012). “Wavelet analysis of bender element signals.” *Geotechnique*, 62(3), 243–252.

Dyvik, R. and Madshus, C. (1985). “Lab measurements of gmax using bender element.” *Proc., ASCE Convention on Advances in the Art of Testing Soils under Cyclic Conditions*, 186196.

Germaine, J. T. (1982). “Development of the directional shear cell for measuring cross-anisotropic clay properties.” M.S. thesis, Massachussets Institute of Technology, Cambridge, MA.

Germaine, J. T. and Ladd, C. (1988). “Triaxial testing of saturated cohesive soils.” *Advanced triaxial testing of soil and rock*, ASTMSTP 977, ASTMSTP 977, 421–459.

Hardin, B. O. (1978). “The nature of stress strain behavior of soils.” Proc. Geotech. Eng. Div. *Speciality Conf. on Earthquake Eng. and Soil Dynamics*, Vol. 1, Pasadena, 3–90.

Hardin, B. O. and Black, W. L. (1969). “Vibration modulus of a normally consolidated clay.” *Journal of the SMF Div. Proc.*, ASCE, 95, 1531–1537.

Hardin, B. O. and Richart, F. E. J. (1963). “Elastic wave velocity in granular soils.” *Journal of soil mechanics and foundation division*, ASCE, 89(1), 33–65.

Houlsby, G. T. and Wroth, C. P. (1991). “The variation of shear modulus of a clay with pressure and over consolidation ratio.” *Soils Fdns*, 31 No. 3, 138–143.

Jamiolkowski, M., L. R. H. and Lo Presti, D. C. F. (1994). “Remarks on the stiffness at small strain of six italian clays.” *Pre-failure deformation of geomaterials*, Vol. 2, 817–836.

Jamiolkowski, M., L. S. and LoPresti, D. C. F. (1991). “Design parameters from theory to practice.” *Proc., Int. Conf. on Geotechnical Engineering for Coastal Development: Geo-Coast 1991*.

Landon, M.M., D. D. S. T. (2007). “Nondestructive sample quality assessment of a soft clay using shear wave velocity.” *Journal of Geotechnical and Geoenvironmental Engineering*, 133, No. 4, 424–432.

Lee, J.-S. and Santamarina, J. C. (2005). “Bender elements: Performance and signal interpretation.” *J. Geotech. Geoenviron. Eng.*, 131(9), 1063–1070.

Leong, E. C., C. J. and Rahardjo, H. (2009). “Measuring shear and compression wave velocities of soil using bender-extender elements.” *Can. Geoetech. J.*, 46, 792–812.

Lunne, T. O., Eide, J., and de Ruiter, J. (1976). “Correlations between cone resistance and vane shear strength in some scandinavian soft to medium stiff clays.” *Can. Geotech. J.*, 13-4, 430– 441.

Marcuson, W. G. and Wahls, H. E. (1972). “Time effects on dynamic shear modulus of clays.” *Proc. Am. Soc. Civ. Engrs*, Vol. 97, No. SM1, 1359–1373.

Murali, M. and Biscontin, G. (2014). “Geotechnical characterization of gulf of mexico clay.” *Proceedings of the ASME 2014 33rd International Conference on Ocean, Offshore and Arctic Engineering*.

Pennington, D., Nash, D., and Lings, M. (2001). “Horizontally mounted

bender elements for measuring anisotropic shear moduli in triaxial clay specimens." *Geotechnical Testing Journal*, 24(2), 133144.
Santamarina, J.C., K. K. and Fam, M. (2001). *Soils and Waves*. J. Wiley & Sons, Inc., New York, NY.
Shirley, D. J. and Hampton, L. D. (1978). "Shear wave measurements in laboratory sediments." *Journal of Acoustic Society of America*, 63(2), 607613.
Viggiani, G. and Atkinson, J. (1995a). "Interpretation of bender element tests." *Geotechnique*, 45, 149–154.
Viggiani, G. and Atkinson, J. H. (1995b). "Stiffness of fine grained soil at very small strains." *Geotechnique*, 45 Issue 2, 249–265.
Youn, J.U., Y. and D.S.Kim (2008). "Measurement of small strain shear modulus gmax of dry and saturated sands by bender element, resonant column, and torsional shear tests." *Canadian Geotechnical Journal*, 45(10), 1426–1438.