Comparison of Shear Wave Velocity Measured In Situ and on Block Samples of a Marine Clay

Melissa M. Landon, University of Massachusetts Amherst, Amherst MA, USA
Don J. DeGroot, University of Massachusetts Amherst, Amherst MA, USA
Joseph Jakubowski, University of Massachusetts Amherst, Amherst MA, USA

ABSTRACT
This paper presents results of shear wave velocity (V_s) measurements conducted in situ and on block samples of a marine clay. The tests were conducted at a research site located in Newbury, Massachusetts that contains a 12 m layer of Boston Blue Clay. In situ measurements of V_s were conducted using a seismic piezocone (SCPTU) and Sherbrooke block sample measurements were made in the field immediately after sampling using portable bender element equipment. On average the block sample V_s values were 25% less than that of the SCPTU values whereas specimens remoulded at the natural water content had V_s values 80% lower than the SCPTU. The results suggest the potential for the development of a methodology using the SCPTU and a portable bender element system to assess sample quality in the field immediately after sample collection.

RÉSUMÉ
Cet article présente des résultats des measures de la vitesse de la vague de cisaillement (V_s) conduites in situ et sur des échantillons en bloc d'un argile marin. Les essais ont été effectués à un emplacement de recherches situé dans Newbury, Massachusetts où se trouve une couche de 12 m d'argile de Boston Blue Clay. Mesures in situ de V_s ont été conduits en utilisant un piezocone séismique (SCPTU) et des mesures d'échantillon de bloc de Sherbrooke ont été faites dans le domaine juste après le prélèvement à l'aide de l'équipement portatif d'élément de cintreuse. En moyenne les valeurs V_s de l'échantillon en bloc étaient 25% moins que cela des valeurs de SCPTU tandis que les spécimens remamiés à la teneur en eau normale a eu des valeurs de V_s de 80% plus bas que le SCPTU. Les résultats suggèrent le potentiel pour développer une méthodologie en utilisant le SCPTU et un système portatif d'élément de cintreuse pour évaluer la qualité d'échantillon dans le domaine juste après la collection témoin.

1. INTRODUCTION

This paper presents results of an investigation of the relationship between shear wave velocity V_s measured in situ and on Sherbrooke block samples of Boston Blue Clay. The long term objective of the research is to investigate the potential for using non-destructive techniques to assess sample quality in the field immediately after sampling. Such non-destructive methods include measurement of shear wave velocity and soil suction (i.e., sampling effective stress). This paper focuses on results obtained using measurement of V_s in situ using the seismic piezocone and on high quality block samples using portable bender element equipment.

2. SITE DESCRIPTION

Seismic piezocone tests and Sherbrooke block sampling were performed at the Newbury site, located approximately 55 km north of Boston, MA. This site was chosen because of the presence of a thick layer (approximately 12 m) of Boston Blue Clay (BBC) close to the ground surface that contains a stiff overconsolidated crust over a soft lightly overconsolidated zone. BBC is a glacial marine deposit that exists in the greater Boston area due to deposition from glacial melt water (Kenney 1964). The advantage of testing the BBC is that it is well characterized within the geotechnical engineering community in the northeastern United States.

Detailed stratigraphy information is reported by Paikowsky and Hart (1998) for the Newbury site. The subsurface consists of about 2.5 m of miscellaneous fill overlaying a thin 0.5 m thick organic layer (0 to 3 m). From a depth of 3 m to 5.5 m there is a 2.5 m layer of overconsolidated BBC, and from 5.5 m to 16 m, there is a 10 m layer of low overconsolidation ratio (OCR) BBC. At depths greater than 16 m, the soil consists of interbedded silt, sand, and clay. Bedrock is at about 35.5 m. The groundwater table at the time of sampling was located 1.7 m below ground level, which is approximately 3.7 m above mean sea level.

3. IN SITU TESTING AND SOIL SAMPLING

3.1 Seismic Piezocone Penetration

Seismic piezocone (SCPTU) tests were performed in general accordance with American Society for Testing and Materials (ASTM) D5778 *Standard Test Method for Performing Electronic Friction Cone and Piezocone Penetration Testing of Soils*. Continuous measurements of tip resistance, sleeve friction, and pore pressure were obtained for site characterization. Downhole seismic soundings were performed at one-meter intervals over the depth of investigation. Jakubowski (2004) presents detailed information on the methods and results of SCPTU testing at the Newbury site.

The piezocone had a standard 60° apex angle, projected area of 10cm^2, and a friction sleeve with a surface area of 150 cm^2. The pore pressure filter was located just behind the shoulder, i.e., the u_2 position. The cone was equipped with a friction reducer located 57.8 cm from the cone tip along the shaft, where the shaft diameter increased from 36.6 mm to 41 mm over a distance of 3.7 cm. CPTU profiling was performed by pushing the cone in one meter intervals at the standard rate of 2 cm/s while the data acquisition system continually recorded tip resistance, sleeve friction, and pore pressure at a rate of 2 readings per second. A laptop computer was used to visualize the profiling data in real time. At the end of each push interval the data acquisition program was suspended and shear wave velocity measurements were made.

A 1.5 m long steel beam with four 10 cm sharpened steel fins, perpendicular to the beam, that protruded into the soil was used to transmit shear waves from the ground surface to depth where the downhole cone geophones recorded the shear wave. During testing, the beam was placed approximately one meter from the cone hole and a normal force was applied by the drill rig outrigger. A 9.1 kg hammer, hung from a 0.61 m tall vertical stem attached perpendicularly to the beam, was lifted parallel to the ground to a height of 0.61 m and released to impact a steel plate on the beam and generate a shear wave. Two geophones were attached to the beam and oriented in the direction of the impact. They were used to trigger the data acquisition system to record the response of the two downhole geophones, which were located 75 cm from the cone tip and oriented 90° relative to each other in the horizontal plane. Jakubowski (2004) used cross-correlation of the shear wave at consecutive depths to determine the time delay between the two waves and divided this time by the distance between the two depths to obtain shear wave velocity at the midpoint of the two depths.

3.2 Sherbrooke Block Sampling

A commercial drill rig was used to obtain six Sherbrooke blocks between 4.6 and 9.6 m. Sampling was performed according to the methods described by Lefebvre and Poulin (1979) and DeGroot et al. (2003). The borehole was prepared by excavating to a depth of 2.6 m with a backhoe and installing a 53 cm diameter pipe casing below the water table into the clay stratum. The casing was stabilized by backfilling the borehole around the outside of the casing. A 50.8 cm diameter pilot hole was augered to a depth of 4.3 m, and filled to the surface with a barite-bentonite mixture to prevent excessive bottom heave and to facilitate transport of soil cuttings away from the sampler. The bottom of the borehole was cleaned of soil cuttings to a flat surface using a flat bottom bucket auger. The sampler was then lowered into the borehole with the cutting blades in the open position until it was approximately 15 cm from the bottom of the borehole. Water was pumped through the drill string to the block sampler drilling fluid jets. The sampler was rotated at approximately 15 revolutions per minute during final lowering to the bottom of the borehole. The sampler was advanced for approximately 33 cm at a rate of 2 to 3 cm/min with continuous rotation. The block sampler fluid jets removed soil cuttings from the vicinity of the carving tools. It was observed that slower rotation and penetration rates produced smaller diameter blocks, as the drilling mud had more of an opportunity to erode the soil at a particular depth. Smaller diameter block samples were also obtained if the drill string was not concentric, allowing the sampler to wobble and "grind" down the sides of the sample. The latter is believed to be more significant in reduction of block sample diameter. Penetration rates were slower for the shallower, stiffer samples, and the average sampling time was between 15-20 minutes. On average, penetration and rotation was between 6-12 minutes for the deeper, softer samples.

After the sampler reached the desired depth, a donut drop hammer was lowered down the drill string to trigger the bottom cutting blades. The sampler was rotated without advancement with the fluid jets active for five minutes, allowing the cutting blades to cut the bottom of the sample from the soil deposit. The cutting blades also served as basal support for the soil block while the sampler was lifted out of the borehole. After removal from the sampler, each block sample was cleaned, measured and then field bender element tested to determine shear wave velocity as described in Section 4.2. Each sample was sealed with a 50:50 mixture of paraffin wax and petroleum jelly (LaRochelle et al. 1986), secured to a plywood board coated with two layers of plastic wrap dipped in the wax mixture, and packed in a special wooden box (DeGroot et al. 2003) to stabilize the sample for transportation.

4. BLOCK SAMPLE TESTING

4.1 Soil Characterization

Soil characterization and index testing was performed for each sample and included grain size distribution, Atterberg limits, and specific gravity tests. Laboratory bender element tests and constant rate of strain (CRS) one-dimensional consolidation tests were performed for each block sample. The major purpose of the CRS tests with respect to the content of this paper is the assessment of sample quality using the measure of vertical strain during reconsolidation to the in situ vertical effective stress.

4.2 Shear Wave Velocity

Vertical shear wave velocity was measured in the field immediately following sampling and subsequently in the laboratory on each block sample using portable piezoceramic bender element equipment developed at the University of Massachusetts Amherst (UMass Amherst; Landon 2004). The system includes a pair of bender element platens (Figure 1) fabricated in the UMass Geotechnical Laboratories, a Wavetek 29 programmable digital direct sampling (DDS) 10 MHz function generator, a portable Pico ADC 212/100 high speed, high precision, two channel PC based oscilloscope with 12 bit resolution and 5 GS/s sampling rate, and a laptop computer. Following Dyvik and Madshus (1985) and Pennington (1999), the transmission element was wired in a parallel configuration and the receiving element was wired in a series configuration. Both elements were coated in epoxy for water proofing and grounded electronically. For each test, bender element platens were oriented so the shear wave was propagated vertically and polarized horizontally, i.e., V_{vh}. The transmitting bender element was excited with a sine wave from the function generator that was varied between 1 and 7 kHz to produce a received wave with very little near field effect and that had a visually distinct shear wave arrival. The optimum input frequency was dependent on soil stiffness. Both transmitted and received waves were recorded and saved to a laptop computer to determine travel time. The sample length in the direction of shear wave polarization was recorded for shear wave velocity calculation.

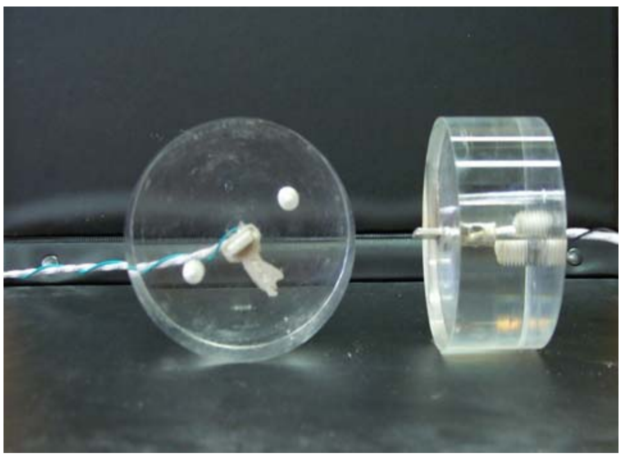

Figure 1 Portable 75 mm diameter acrylic platens containing bender elements.

Figure 2 plots an example of transmitted and received bender element traces for a Sherbrooke block sample. The averaging function of the PICO ADC oscilloscope was used to record the received signal, however, it was not necessary to filter or amplify the received traces. Interpretation of the shear wave travel time was performed manually by analyzing the time difference between the initiation of the transmitted signal and the arrival of the shear wave. The arrival of the shear wave was scaled from the plotted data to the nearest 0.02 ms or less, which represents a minimum accuracy of about 1% for a 270 mm length specimen, which was the average length for the block samples tested. Shear wave arrival time was determined using the first zero crossover method, an example of which is shown in Figure 2. The shear wave arrival was determined by drawing a horizontal line from the initial portion, or zero reference, of the received wave and identifying the time of the first zero crossover of the positively polarized peak. The start of the rise of the transmitted signal was also identified by scaling, and the travel time of the shear wave was calculated as the time of arrival of the received wave minus the start of the transmitted wave and the system calibration time, t_c.

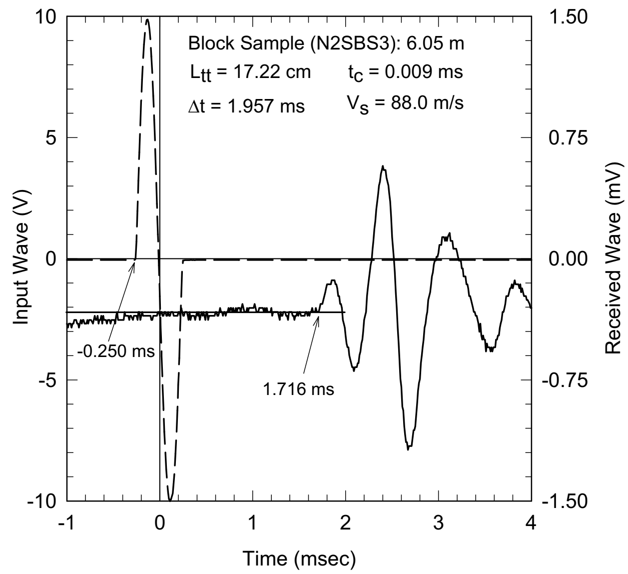

Figure 2: Example of block sample transmitted and received bender element waves.

4.3 CRS Consolidation

Laboratory constant rate of strain (CRS) one-dimensional consolidation tests were performed to determine the stress history profile for the site as well as sample quality using the Sample Quality Designation (SQD) method (Terzaghi et al. 1996). The CRS consolidation tests were performed in general accordance with ASTM D4186 *Standard Test Method for One-Dimensional Consolidation Properties of Soils Using Controlled-Strain Loading* and procedures discussed in Sandbeakken et al. (1986). Each specimen had a seating load applied ($0.25\sigma'_{v0}$ for soft samples and $0.5\sigma'_{v0}$ for stiff samples) and back pressure saturated to 200 kPa overnight. Consolidation was performed at a rate of 1% per hour (2.8×10^{-6} s^{-1}) to either a maximum strain of 30% or maximum stress of 95% of the load cell capacity, whichever occurred first. It has been found at the UMass Amherst Geotechnical Engineering Laboratories that a strain rate of 1% per hour typically results in a base pore excess pore pressure ratio (i.e., $\Delta u_b/\sigma_v$) of less than 10% for BBC.

5. TEST RESULTS

5.1 Soil Characterization

Over the depth of sampling the BBC has plastic and liquid limits that range from 24% to 30% and 45% to 49%, respectively, with no trend with depth (Figure 3). Plasticity index ranges from 19 to 21, and liquidity index increases with depth from 0.59 in the crust to 1.26 at 9.8 m. The Unified Soil Classification System plasticity chart classifies the soil as a CL (low plasticity-lean clay) or ML (low plasticity silt) soil. Hydrometer results show an average clay fraction of 62% and silt fraction of 37% for the samples. There is a trace of sand in some of the samples.

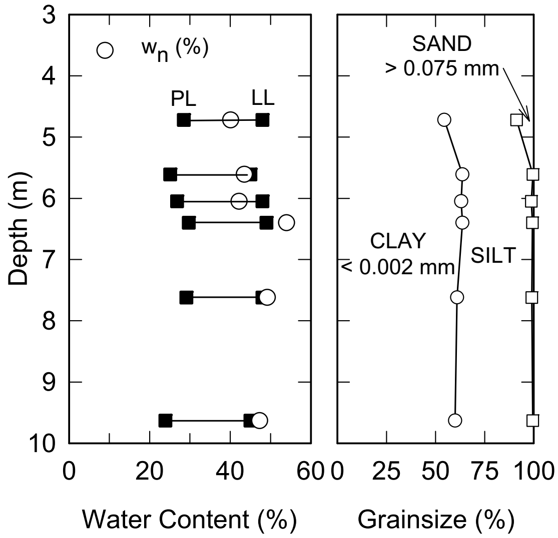

Figure 3: Atterberg limits, natural water content and grain size distribution for Newbury block samples.

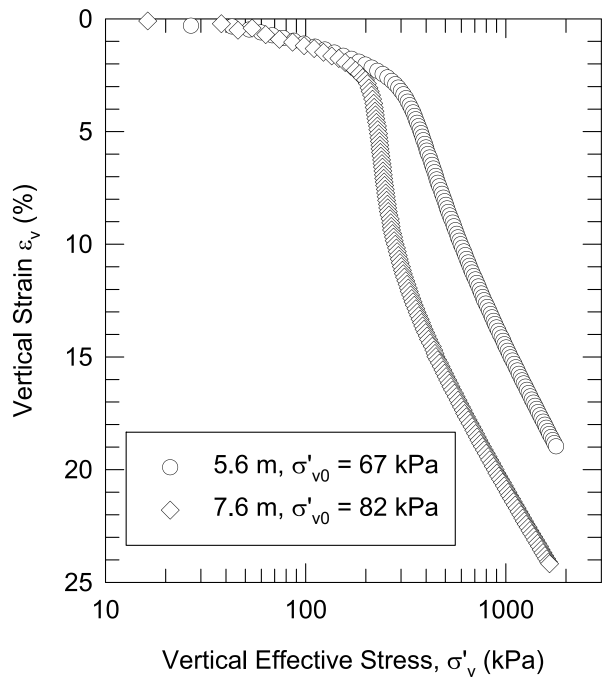

Figure 4 CRS consolidation curves for block samples within and below the BBC crust.

5.2 CRS Consolidation

Table 1 presents a summary of the one-dimensional consolidation results and includes natural water content (w_n), initial void ratio (e_0), total density (ρ_t), in situ σ'_{v0}, and preconsolidation stress (σ'_p) determined with the strain energy method (Becker et al. 1987). Consolidation curves for the samples showed a transition with depth from rounded curves at shallow depths (i.e., in the crust) to curves with a more distinct break at the preconsolidation stress as well as a reversed s-shape in the virgin compression region for samples at greater depths (Figure 4). Preconsolidation stress values range from 662 kPa (4.7 m) to 216 kPa (9.6 m) and these data were used to generate the estimated in situ preconsolidation stress and OCR profiles plotted in Figure 5. OCR decreases rapidly from 14.9 at 4.5 m to 3.0 at 6.3 m, after which the rate of decrease with depth is much less, reaching an OCR of about 2.2 at 10 m. Most likely the high overconsolidation of the soil in the shallow subsurface is primarily a result of desiccation that occurred because of groundwater table fluctuations. Below the crust, the OCR of approximately 2.2 is possibility due to ageing and/or cementation.

Table 1: Block sample CRS consolidation data

Depth (m)	w_n (%)	e_0	ρ_t (Mg/m^3)	σ'_{v0} (kPa)	Strain Energy σ'_p (kPa)
4.72	40.1	1.053	1.835	60.1	662
5.61	43.5	1.129	1.815	66.7	288
6.05	42.2	1.237	1.750	69.9	202
6.40	54.0	1.409	1.719	72.6	222
7.62	49.2	1.264	1.747	81.7	210
9.63	47.3	1.255	1.796	96.6	216

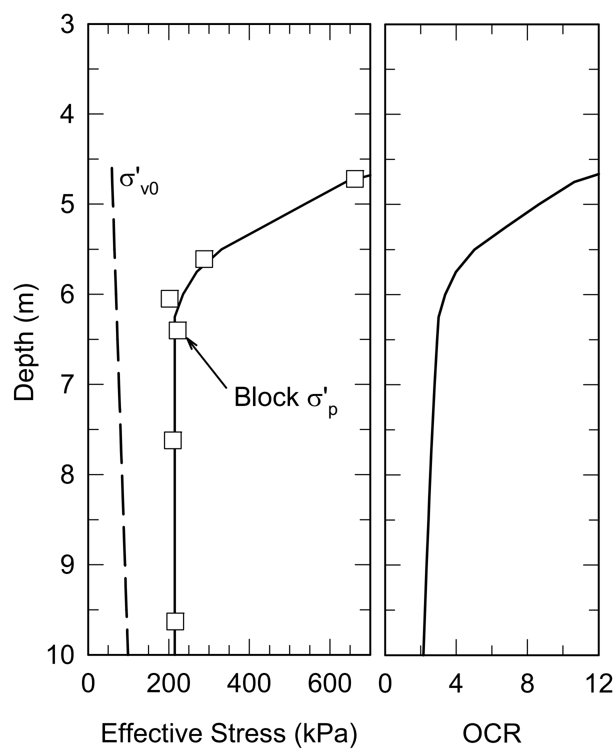

Figure 5: Stress history and overconsolidation ratio for the Newbury BBC site.

Table 2 presents the volumetric strain, ε_{vol}, of the sample specimens measured during reconsolidation in the CRS test to the in situ vertical effective stress, σ'_{v0}, and the corresponding SQD sample quality designations. As defined by Terzaghi et al. (1996) SQD designations range from A (best) to E (worst). As noted in Table 2 sample quality for the BBC block samples were either A or B over the depth of investigation.

Table 2: Newbury block sample quality data.

Depth	Terzaghi et al. (1996)	
(m)	ε_v at σ'_{v0} (%)	SQD
4.72	0.38	A
5.61	0.71	A
6.05	1.26	B
6.40	0.40	A
7.62	0.98	A
9.63	1.24	B

5.3 Cone Penetration Results

Figure 6 plots corrected tip resistance, q_t, and measured pore pressure, u_2 along with the hydrostatic pore pressure, u_0. Between 3.5 m and 5.5 m, q_t values showed a rapid decrease from 2600 kPa to about 500 kPa. High q_t values in this region indicate the presence of a stiff soil, which in this case is the overconsolidated clay crust. Also in this region (3.5 m to 5.5 m), measured pore pressure is mostly negative with an increase to low positive values at 5 m (260 kPa). Negative and low positive pore pressures confirm the presence of a stiff, overconsolidated soil in this region. At depths greater that 5.5 m, q_t values show a slight, almost linear increase with depth, from 500 kPa to 600 kPa at 14.5 m. Measured pore pressure increases from 300 kPa to 600 kPa over this range.

5.4 Shear wave velocity

Figure 7 presents the interpreted shear wave velocity profile obtained from SCPTU testing (V_{SCPTU}) for the Newbury site. Shear wave velocity initially decreases from 180 m/s at 3.4 m to 125 m/s at 5.9 m where it then begins to increase again to 148 m/s at 9.9 m. At depths where soil samples were obtained, V_{SCPTU} decreases from 161 m/s at 4.7 m to 125 m/s at 5.9 m where it then increases to 139 m/s at 9.8 m.

Figure 7 also plots field and laboratory measured V_{vh} for block samples and V_{vh} for remoulded block sample cuttings. Field measured V_{vh} values for block samples range from 96 m/s and 104 m/s and shows a slight increase with depth. Laboratory (i.e., after transport of the samples to UMass Amherst) V_{vh} values for block samples ranged from 85 m/s to 112 m/s and the profile shows some scatter when compared to the field profile, though the average of the laboratory values are similar to field measured values. Scatter in the laboratory profile may be a result of possible disturbance, destructurization, or changes in effective stress state (i.e., suction) that may have occurred during sample handling and transportation.

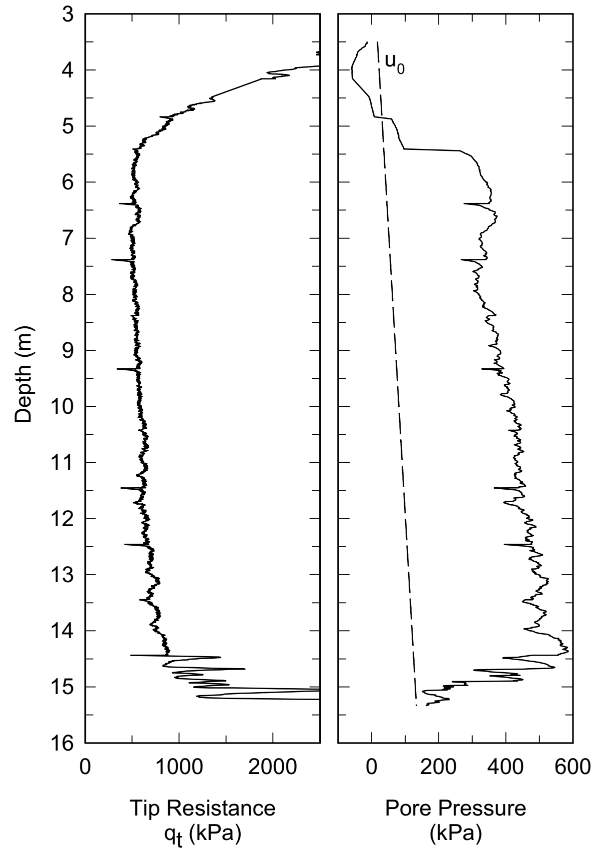

Figure 6: Piezocone penetration results from Newbury.

In both cases, V_{vh} values are lower than V_{SCPTU} values at similar depths and the field and laboratory profiles do not have shapes similar to the V_{SCPTU} profile. The SCPTU V_{vh} values are for the soil at in situ stress conditions (i.e., mean stress $\sigma'_m = 1/3(\sigma'_{v0} + 2\sigma'_{h0})$) whereas the block samples were at an unknown isotropic stress state, i.e., σ'_s = sampling effective stress = sample suction. Values of σ'_s should be expected to be much lower than σ'_m due to stress relief during sampling and possible sample disturbance (Ladd and Lambe 1963, Ladd and DeGroot 2003). Hight et al. (2003) present a method to analyze in situ and sample V_{vh} data that takes into account the effective stress state in situ and within samples but the method requires measurement of σ'_s.

V_{vh} values for block sample cuttings remoulded at their natural water content were much lower than in situ and intact block sample values. Remoulded samples represent the probable lower bound of obtainable V_{vh} values. As plotted in Figure 7, V_{vh} for remoulded specimens decreases with depth from 43 to 18 m/s and shows a notable decrease from 4.7 to 6.0 m, after which values remain nearly constant. For reference, above 6.4 m, the w_n for the remolded samples is in the plastic range

(LI < 1) and at depths below 6.4 m, w_n is greater than the LL (Liquidity Index, LI > 1). V_{vh} values for remoulded BBC specimens are nearly constant at water contents greater than the liquid limit.

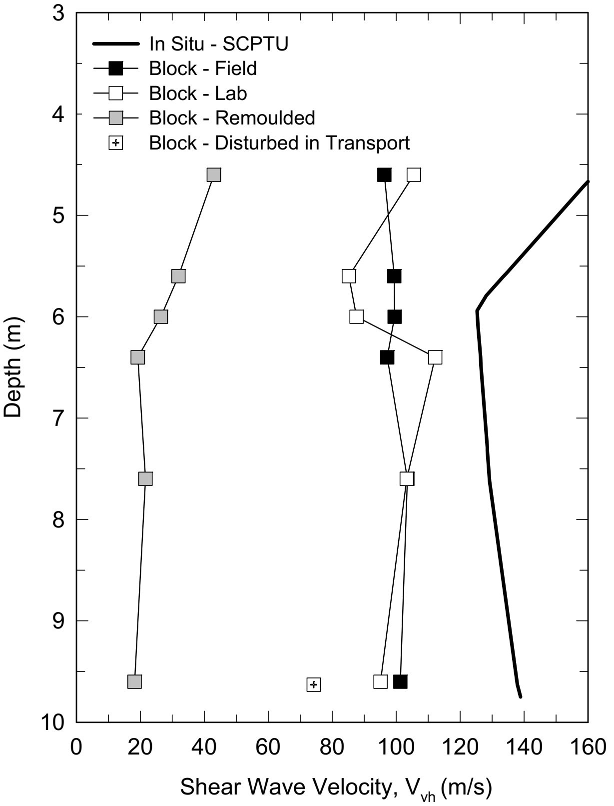

Figure 7: Newbury in situ and block sample shear wave velocity.

A portion of the deepest block sample was cut from the sample prior to field bender element testing and intentionally transported in an unprotected state (i.e. only partially sealed and transported in a bucket). It was then tested in the laboratory and V_{vh} was determined to be 74 m/s. This value is 20% lower than the intact and carefully sealed and transported block sample from the same depth (9.6 m). This reduction in V_{vh} reinforces the importance of proper sample care and handling during the time period between sampling and laboratory testing for engineering parameters.

6. ASSESSMENT OF SAMPLE QUALITY

The reduction in shear wave velocity for the block samples, with respect to the in situ shear wave velocity is thought to be almost entirely a result of stress relief disturbance. It is not expected that the carved block samples experience the degree of mechanical disturbance that occurs during tube sampling. This was confirmed by Shiwakoti et al. (2000), who reconsolidated specimens from Laval block samples in the laboratory to the anisotropic in situ stress state and found that shear modulus determined from bender element tests compared well with in situ shear wave velocity values obtained with a seismic piezocone. This confirms that confining stress is important to the magnitude of shear wave velocity. However, reconsolidation requires the trimming and set-up of a laboratory triaxial specimen. The objective of this project was to investigate the possibility of developing a non-destructive field method of assessing sample quality. Therefore the block sample measurements were intentionally made directly on the samples without any confining stress.

The remoulded block sample cuttings represent the most disturbed state (i.e. stress relief followed by complete destructuration of the soil during remoulding) and therefore the shear wave velocity represents the probable lower bound for the soil at a particular depth.

Figure 8 plots sample quality with block sample V_{vh} normalized by in situ shear wave velocity, V_{vh}/V_{SCPTU}. These data indicate that for this particular clay and sample depths, very good to excellent quality block samples (as judged by the SQD ratings of A and B) have V_{vh} values in the range of 70 to 80% of the in situ V_{vh} values from downhole SCPTU testing. Completely destructurized samples have V_{vh} values in the range of 10 to 20% of V_{SCPTU} from SCPTU.

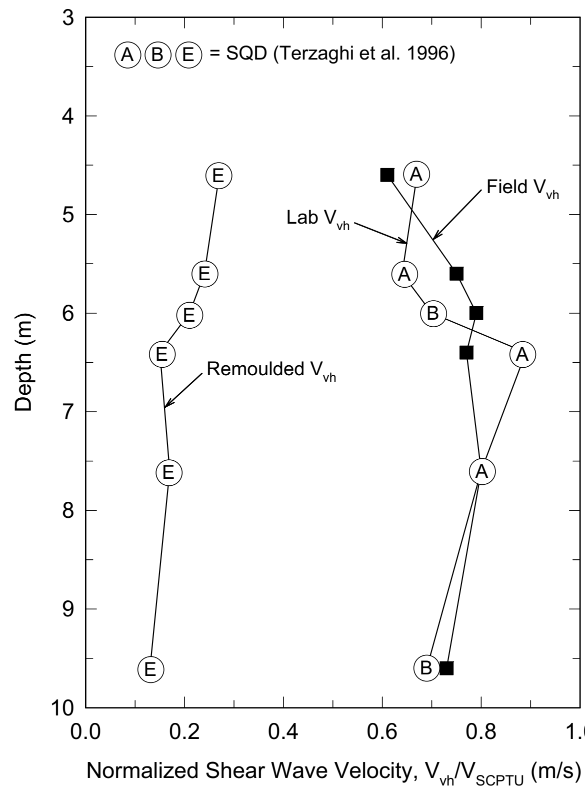

Figure 8: Normalized shear wave velocity for Newbury block samples and sample quality.

These results suggest that as more data such as that presented in Figure 8 are collected for different clays, a system could be developed to evaluate sample quality based on field V_{vh} measurements. In this project, the measurement of V_{vh} for the block samples was relatively simple and quick to perform using the portable bender element system. The equipment can also be used to measure in the field V_{vh} for tube samples by either cutting or extruding a small section of soil from a tube. Such measurements coupled with SCPTU V_{vh} data would provide the potential for evaluation of sample quality in the field immediately after a sample is collected.

7. CONCLUSIONS

Shear wave velocity measurements were made in situ using the seismic piezocone and on Sherbrooke block samples of a marine clay using a portable bender element system immediately after sampling. The bender element system worked well and was a relatively easy and quick test to perform. Laboratory CRS consolidation tests indicated that the block samples were of very good to excellent quality based on A and B ratings using the Terzaghi et al. (1996) SQD method. Shear wave velocity measured on the block samples was on average 25% lower than the seismic CPTU shear wave velocity, which is thought to be a mainly a result of sampling stress relief. Shear wave velocity for remoulded specimens was significantly lower than in situ shear wave velocity (an average of 80% lower), and these represented the extreme of mechanical disturbance. The results of this work suggest that a non-destruction field procedure for evaluating sample disturbance using shear wave velocity has potential, though more experience and data need to be collected for a variety of clays and sampling methods.

8. ACKNOWLEDGEMENTS

This work is part of a project supported in part by the U.S. National Science Foundation under Grant No. CMS-0219480 titled “Collaborative Research: International Development of Non-Destructive Tools to Quantify Sample Disturbance and its Correlation to Measured Behavior”. The authors also gratefully acknowledge Dr. Rune Dyvik, Norwegian Geotechnical Institute; Dr. J. Carlos Santamarina, Georgia Institute of Technology; Dr. Chris Baxter, University of Rhode Island; and Dr. Mostafa Ismail, Centre for Offshore Foundation Systems, Australia, for their assistance during development of the bender element system.

9. REFERENCES

American Society of Testing and Materials (ASTM). 2003. Annual Book of Standards, Vol. 4.08, West Conshohocken, PA, USA.

Becker, D.E., Crooks, J.H.A., Been, K., and Jeffries, M.G. 1987. Work as a criterion for determining in situ and yield stresses in clays. Canadian Geotechnical Journal, 24(4): 549-564.

DeGroot, D.J., Lunne, T., Sheahan, T.C., and Ryan, R.M. 2003. Experience with downhole block sampling in clays with conventional drilling equipment. Proc. 12th Panamerican Conf. on Soil Mechanics and Geotech. Eng., MIT, Vol. 1, 521-526.

Dyvik, R. and Madshus, C. 1985. Lab measurements of G_{max} using bender elements. Proc. Advances in the Art of Testing Soils Under Cyclic Conditions, ASCE, 186-187.

Hight, D.W., McMillan, F., Powell, J.J.M., Jardine, R.J., and Allenou, C.P. 2003. Some characteristics of London Clay. Proc. Int. Workshop on Characterisation and Engineering Properties of Natural Soils, Tan et al. (Eds.), Singapore, 851-908.

Jakubowski, J. 2004. Use of the seismic CPTU for site characterization in soft clays. Masters of Science Thesis, University of Massachusetts Amherst, Amherst, MA.

Kenney, T.C. 1964. Sea-Level movements and the geologic histories of the post-glacial marine soils at Boston, Nicolet, Ottawa, and Oslo. Géotechnique, 14(3): 203-230.

Landon, M. M. 2004. Field quantification of sample disturbance of a marine clay using bender elements. Masters of Science Thesis, University of Massachusetts Amherst, Amherst, MA.

Ladd, C.C. and Lambe, T.W. 1963. The strength of undisturbed clay determined from undrained tests. Symp. on Laboratory Shear Testing of Soils, ASTM, 342-371.

Ladd, C.C. and DeGroot, D.J. 2003. Recommended practice for soft ground site charaterization: Arthur Casagrande Lecture. Proc. 12th Panamerican Conf. on Soil Mechanics and Geotech. Eng., MIT, Vol. 1, 3-57.

La Rochelle, P., Leroueil, S., and Tavenas, F. 1986. Technique for long-term storage of clay samples. Canadian Geotechnical Journal, 23(4): 602-605.

Lefebvre, G. and Poulin, C. 1979. A new method of sampling in sensitive clay. Canadian Geotechnical Journal, 16(1): 226-233.

Paikowsky, S.G. and Hart, L.J. 1998. Development and field testing of multiple deployment model pile (MDMP). University of Massachusetts Lowell, Lowell, MA.

Pennington, D.S. 1999. The anisotropic small strain stiffness of cambridge Gault Clay. PhD Thesis, University of Bristol, UK.

Sandbaekken, T., Berre, T., and Lacasse, S. 1986. Oedometer testing at the Norwegian Geotechnical Institute. Consolidation of Soils: Testing and Evaluation, ASTM STP 892, 329-353.

Shiwakoti, D.R., Tanaka, H., Tanaka, M., and Mishima, O. 2000. A study on small strain shear modulus of undisturbed soft marine clays. Proceedings of the 10th International Offshore and Polar Engineering Conference, Seattle, USA, 455-460.

Terzaghi, K., Peck, R.B., and Mesri, G. 1996. Soil Mechanics in Engineering Practice. J. Wiley & Sons, New York.