INTERNATIONAL SOCIETY FOR
SOIL MECHANICS AND
GEOTECHNICAL ENGINEERING

This paper was downloaded from the Online Library of the International Society for Soil Mechanics and Geotechnical Engineering (ISSMGE). The library is available here:

https://www.issmge.org/publications/online-library

This is an open-access database that archives thousands of papers published under the Auspices of the ISSMGE and maintained by the Innovation and Development Committee of ISSMGE.

Deformation properties of soil from in situ and laboratory tests

Déformabilités sol de in situ et essais en laboratoire

A.(Tony) P.Butcher & John J.M.Powell, *BRE Ltd, Watford, Herts, UK*

ABSTRACT: The design of geotechnical structures requires an assessment of the displacements during the life of a structure and hence a knowledge of the ground stiffness and its variation with stress and strain. It is well known that in order to successfully predict movements the anisotropy of stiffness and stress are the most important properties to quantify. This paper compares the data from various test methods and shows that G_0 can vary in any one deposit depending on the test method. The reasons for this are investigated and discussed and the need to combine the data from both seismic and other in situ tests to define the non-linear shear modulus / strain relationship is highlighted.

RÉSUMÉ: La conception géotechniques derrière les des des moulues des structures exige une évaluation les déplacements pendant la vie derrière une d'd'une d'une structure et ainsi une connaissance par rapport de la rigidité et sa variation l'effort et la contrainte. Il est bien connu qu'afin de prévoir avec succès des mouvements l'anisotropie de la rigidité et l'effort soient les propriétés les plus importantes à mesurer. Cet article compare les données de diverses méthodes d'essai et prouve que G_0 peut changer dans n'importe quel un dépôt selon la méthode d'essai. Les raisons de ceci sont étudiées et discuté et la nécessité de combiner les données du séismique et d'autre les essais in situ pour définir le rapport non linéaire de module / contrainte de cisaillement est mise en valeur.

1 INTRODUCTION

The design of geotechnical structures requires an assessment of the likely displacements during the life of the structure and hence knowledge of the ground stiffness and its variations with stress and strain are crucially important. It has been shown by Simpson et al (1996) that in order to successfully predict lateral and vertical movements around excavations and tunnels in overconsolidated soils the anisotropy of stiffness and stress are the most important properties to quantify.

Stiffness is determined either at very small strain (<0.0001%) using shear wave measurements or using resonant column or hollow cylinder at 0.001% to 0.05%, or at large strains (>0.1%) using direct (triaxial, pressuremeter, plate test) methods. Alternatively indirect empirically derived relationships (from CPT, SPT etc) are possible. Stiffness in much of the working range (0.01 – 0.1%) may be obtained by using shear modulus – shear strain degradation curves. Various forms for these curves have been developed, two examples are Hardin & Drnevich (1972) and Vucetic & Dobry (1991).

This paper compares the data from various test methods and shows that G_0, the shear modulus at very small strain, can vary in any one deposit depending on the test method. The reasons for this are discussed and the need to combine data from both the seismic and other in situ tests to define the non-linear shear modulus/strain relationship is highlighted. The variation of the shape of shear modulus/strain relationship with index parameters is illustrated and compared with other models.

2 BACKGROUND

Over a number of years the BRE has been carrying out static and dynamic testing of soils and soft rocks in both the field and the laboratory for the determination of ground properties, including stiffness. This paper uses data from five test sites on a range of soils from stiff heavily overconsolidated aged clays (Gault and London), through stiff glacial clay till (Cowden) to soft alluvial silty clays (Bothkennar and Pentre). The basic soil properties at each site are given in Table 1. The London clay (Chattenden) and Gault clay (Madingley) sites comprise stiff, fissured, heavily over consolidated, aged clays whereas the Cowden site comprises a largely unstructured 'clay sized' rock flour matrix containing rock fragments. The soft alluvial silty clays at the Pentre and Bothkennar sites both have microstructures of thin laminations with some cementation.

Table 1: Sites, soil types and basic soil properties

Site	Soil	c_u (kPa)	I_p	OCR	Reference
Chattenden	London clay	60 – 140	52 - 67	16-40	Hope (1993), Crilly et al (1992)
Madingley	Gault clay	90 – 160	50 - 55	20-50	Butcher & Lord (1993)
Cowden	Glacial till	105 – 270	14 - 20	1.5- 5	Marsland & Powell (1985)
Pentre	Soft silty clay	30 – 45	13 - 20	1.5-2.5	Lambson et al (1993)
Bothkennar	Soft silty clay	20 – 60	34 - 50	1.2-1.3	Hight et al (1992)

3 LABORATORY AND FIELD MEASUREMENTS

The methods used in this paper to determine stiffness were:

- Laboratory:
  resonant column tests for small strain stiffness and piezoceramic bender element tests for very small strain stiffness.
- Field:
  cross-hole, down hole and seismic cone shear wave for very small strain stiffness and cone pressuremeter tests (CPM) for large strain stiffness.

The laboratory tests were carried out by the Norwegian Geotechnical Institute (NGI) on samples taken by BRE. The stiff overconsolidated clays were sampled using a pushed thin walled tube sampler, the soft clay at Pentre was sampled using a pushed thin walled tube piston sampler and the specimens from Bothkennar were prepared from blocks cut with a Sherbrooke sampler.

The samples for resonant column testing also had piezo-

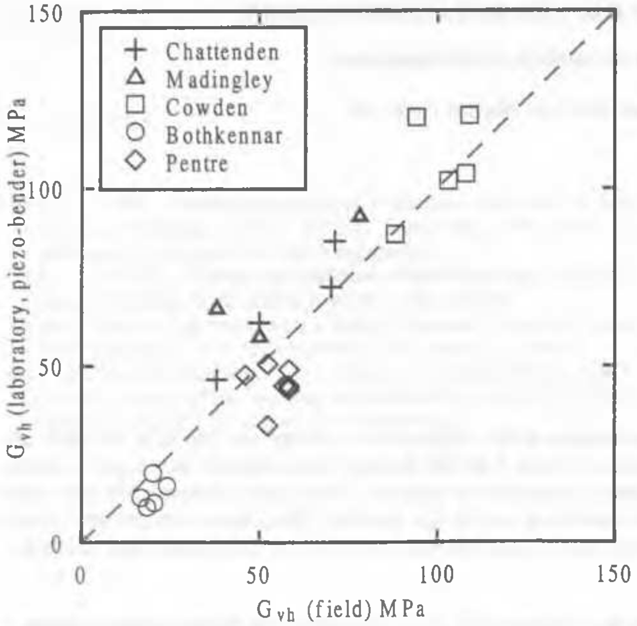

Figure 1: Comparison of stiffness derived from field and laboratory shear wave measurements

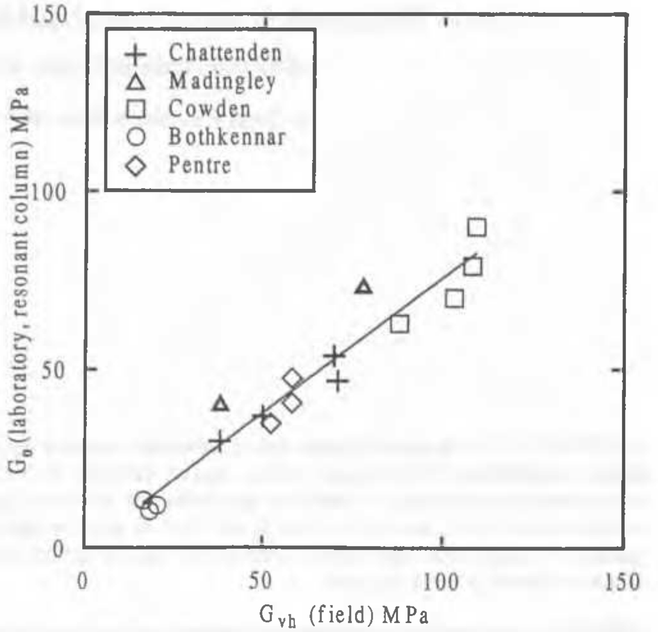

Figure 2: Comparison of stiffness from field shear wave measurements and laboratory resonant column tests.

ceramic bender elements installed in their top and base to measure vertically propagating horizontally polarised (V_{vh}) shear waves. (N.B. The notation used in this paper for shear wave velocity (V_s) and Shear Modulus (G) includes subscripts to denote the directions of propagation and polarisation. For example V_{hv} is the velocity of a horizontally propagating, vertically polarised shear wave and the shear modulus derived from this shear wave velocity, $G=• .V_s^2$ where • is the bulk density, will also use the notation G_{hv}).

The samples in the laboratory were reconsolidated to their assumed in situ effective stress by setting the cell pressure to the in situ lateral stress and then slowly changing the vertical effective stress until the in situ stress state was reached. Shear wave measurements were then made using the piezo-ceramic bender elements and the shear strain cycled with increasing strain as in a standard resonant column test.

The resonant column test shears the specimen horizontally in a vertical cylinder and could be taken as equivalent to the mechanism for the transmission of a horizontally polarised vertically propagating shear wave. The stiffness (G_0) from the resonant column test can therefore be taken to be comparable to the stiffness, G_{vh} (calculated from the vertically propagating, horizontally polarised shear wave velocity V_{vh}) though the strain rate and amplitude are greater than for either the laboratory or field shear wave measurements. The resonant column test can however change the shear strain (over a limited range) at which the measurements are made so allowing stiffness values at up to 0.1% shear strain to be determined. In this case the values will be termed G_{RC}.

The field measurements of shear wave velocities were made using BRE equipment and techniques (Butcher & Powell, 1996). The cross-hole measurements were all made between three co-linear lined boreholes with known verticality and the down-hole measurements from a surface source to either a seismic cone or receivers in a lined borehole.

The direct in situ field measurements of shear modulus presented here were made with the CPM using the testing procedures detailed in Powell & Shields (1995). Each test contained between one and three unload-reload loops. Unload-reload stiffnesses were calculated from the loops as secant values over the whole loop or as secants at various strains up or down the loop. When converting the cavity strains from the CPM tests to equivalent triaxial shear strains the method of Jardine (1991) was adopted.

4 COMPARISON OF LABORATORY AND FIELD MEASUREMENTS AT THE SAME STRAIN LEVEL

The field downhole shear wave measurements and those made using the piezo-ceramic bender elements in the laboratory specimens both use vertically propagating horizontally polarised shear waves. The difference between the field and laboratory measurements is their frequency and amplitude, the laboratory shear waves having frequencies two orders of magnitude greater than those used in the field and with significantly lower amplitudes.

Figure 1 shows G_{vh} field against G_{vh} laboratory (piezo-bender) for all the sites and shows that the normally consolidated soils give laboratory values slightly lower than the field derived values whereas the heavily over consolidated soils gave laboratory values higher than the field values. This Figure includes three data points from earlier piezo-bender tests in consolidated drained triaxial tests. Cowden data fall between these two groups with the field and laboratory generally agreeing.

The lower laboratory G_{vh} values for the normally consolidated soils may be due to sample disturbance whereas the higher values for the stiff fissured overconsolidated clays maybe related to the scale effects of the macro structure as the laboratory samples are less likely to include a representative macro structure. The macro structure of the overconsolidated glacial till at Cowden does not include the well defined fissures found in the Gault and London clays and may explain why the field and laboratory values agree. The changing levels of OCR may also be a factor.

Figure 2 compares the resonant column G_o with the field G_{vh} and shows that the values from the resonant column test increasingly under estimate the field stiffness as stiffness increases. This data shows a more consistent relationship (the line shown is a least squares fit) than that from the piezo-bender measurements (Figure 1) but with the Madingley data points above the best fit line and close to parity. The difference is possibly a strain rate effect but the pattern of data is significantly different from that in Figure 1 except for the soft normally consolidated soils which gave laboratory modulus values lower than the field values.

Figure 3 compares the stiffnessess from resonant column and piezobender tests on the same specimens. For the soft normally consolidated soils the Figure shows good agreement between the two laboratory methods and is also in agreement with findings reported by Dyvik & Madshus (1985). However, the resonant

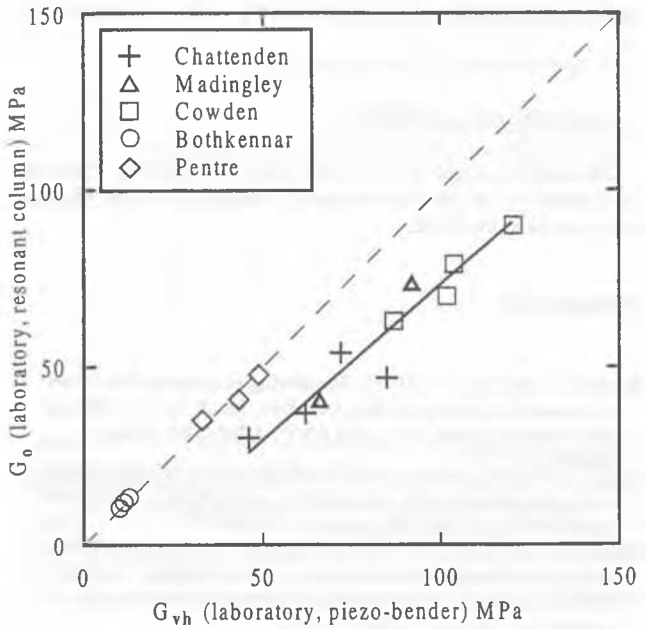

Figure 3: Comparison of G from laboratory tests

column gave consistently lower values than the piezo-benders in the stiff overconsolidated soils.

These differences between the measured stiffnesses in the laboratory are, probably related to strain rate; the strain rate having a less significant effect in the soft normally consolidated soils.

5 SHEAR MODULUS/STRAIN RELATIONSHIPS

Shear modulus – shear strain relationships are used to estimate the behaviour of soils at working strain levels when the measurements have been made either at very small strain levels or at relatively large strain levels. The most popular relationship for clay soils is that developed by Hardin & Drnevich (1972); it uses an empirical relationship controlled by G_o and the undrained shear strength. The shape of the degradation of G/G_o uses a hyperbolic relationship obtained from the analysis of a large database of soil properties. A second model by Vucetic & Dobry (1991) investigated the effect of plasticity index (I_p) on the rate of degradation of modulus with increasing strain.

Figure 4 shows the laboratory data from the five sites plotted on the Vucetic & Dobry curves; the values of I_p in the legend are those of the particular specimens tested. The plot shows that the measured values fall essentially in two groups with the higher I_p soils falling in one group above the lower I_p soils. However, they all fall below the corresponding Vucetic & Dobry curves especially at shear strains of 0.01% and greater. In practice, therefore, direct use of the Vucetic & Dobry curves would tend to over estimate the G/G_o values and resultant stiffness.

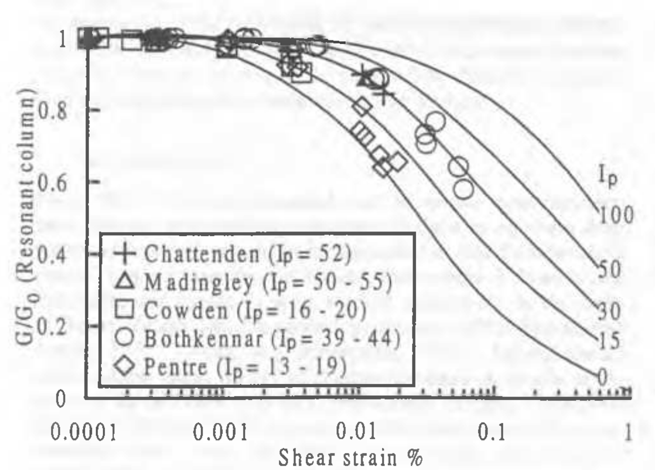

Figure 4: Resonant column data with Vucetic & Dobry (1991) curves

6 COMPARISON OF LABORATORY AND FIELD MEASUREMENTS AT DIFFERENT STRAIN LEVELS.

Figure 5 shows resonant column data, over a range of strains, from the (normally consolidated) Pentre site. Also included are lines for Vucetic & Dobry (1991), using an I_p of 15, and Hardin & Drnevich (1972), using G_{vh} from the in situ geophysics tests and shear strength from laboratory tests by Lambson et al (1993). At Pentre the field measurements of shear wave velocities gave values of G_{vh} and G_{hh} that were equal at most depths.

The CPM gives shear modulus values that would be expected to be G_{hh}. The shear modulus values calculated from the CPM have been normalised by both G_{vh} and G_{hh} from the in situ geophysics tests whilst the resonant column data have been normalised by the initial very small strain resonant column measurements.

Figure 5 shows that both the shear strain/shear modulus models appear to over estimate the G/G_o values at any but the smallest strain. However, the resonant column and the CPM data describe a curve indicating a faster rate of degradation of shear modulus with increasing strain than either of the models would predict. The CPM data, while giving lower modulus ratios than the models suggest, does give data that follows on from the modulus degradation of the resonant column data.

Figure 6 shows the same sets of data for the (heavily overconsolidated) clay at Madingley. In this case the in situ G_{hh} values are significantly higher than the G_{vh} values (see Butcher & Powell 1995) and so Hardin & Drnevich curves using both are included. It can be seen that the normalisation in the Hardin & Drnevich model has a significant effect on the rate of degradation of the curve, the predicted rate being much quicker for the G_{hh} than the G_{vh}. The Vucetic and Dobry model makes no such distinction. In Figure 6 the modulus ratio values from the CPM form two distinct bands both with significantly lower values than the corresponding Hardin & Drnevich and Vucetic & Dobry curves. The CPM G/G_{hh} values give a much greater shear modulus degradation with increasing shear strain than the G/G_{hv} which in turn are steeper than Hardin & Drnevich might suggest. Again the predicted stiffnesses from the models would be higher than those measured.

It should be noted that the position of the CPM data would be affected by the correctness of the model for converting cavity strain to equivalent shear strain.

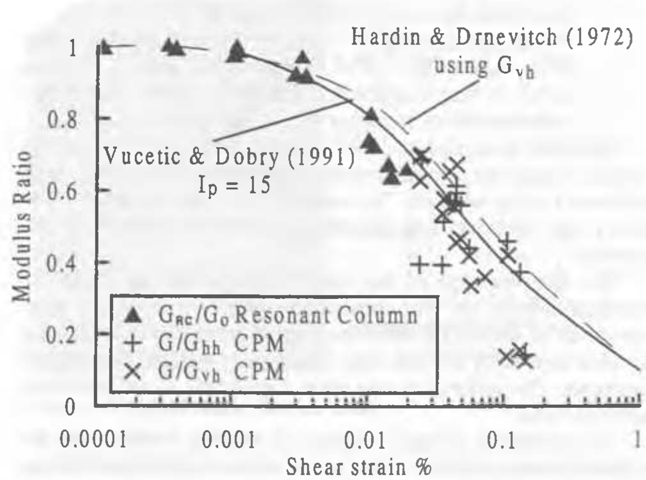

Figure 5: Resonant column and CPM data from Pentre site

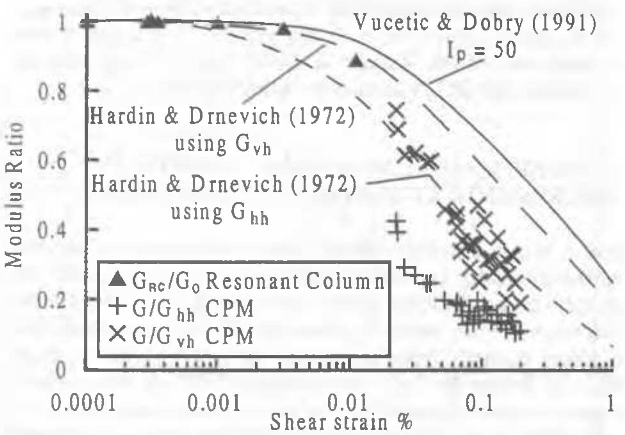

Figure 6: Resonant column data from the Madingley site

7 DISCUSSION AND CONCLUSIONS

This study has highlighted concerns over the measurement and assessment of shear modulus from established laboratory and field tests.

The laboratory data shows:

- Laboratory measurements of shear waves (piezobender tests) gave higher G values than field measurements in stiff fissured soils but lower values in the soft alluvial soils
- Resonant column tests gave G values on average 25% lower than those derived from field shear wave measurements
- Resonant column tests gave G values consistently lower than piezo-bender tests on the same specimens of stiff overconsolidated soils, highlighting concerns over comparability of data from these tests (probably related to a greater effect of strain rate on overconsolidated soils than on normally consolidated soils).
- Assessments of G at working strains, based on the use of index properties to control the shear strain-shear modulus degradation curve, would overestimate the shear modulus.

The field data shows:

- Use of shear moduli from the CPM would tend to give significantly lower G at working strains than those estimated from existing methods based on index properties or G_0 and c_u. (This assumes the conversion used for cavity strain to shear strain is applicable. The conversion may over correct so giving a lower strain level).
- The CPM shows a believable degradation curve for both Pentre and Madingley but it gives significantly lower G/G_0 values. The CPM values normalised by the G_{hh}, which is more logical for the pressuremeter, gave significantly lower modulus at working strains.

The work presented has shown that there is still much to understand about the interrelationship of measurements made from different testing methods. In particular the comparison of laboratory and field data has not shown a consistent pattern of behaviour.

The determination of the small strain behaviour using the resonant column test will consistently give the lowest, i.e. most conservative, values of stiffness at small strains when compared to other laboratory and the field values from seismic wave measurements. The differences are more significant in stiff overconsolidated soils.

The estimation of shear modulus at working strains using established models and the small strain values will tend to over estimate the modulus values at working strains.

The use of CPM data to estimate shear modulus at working strains will give lower design values than those from the shear modulus-shear strain models particularly when normalised by G_{hh}.

8 AKNOWLEDGEMENTS

The authors would like to thank their colleagues at BRE and the Department of the Environment, Transport and the Regions who sponsored the work.

REFERENCES

Butcher A.P. & Lord J.A. (1993). Engineering properties of the Gault clay in and around Cambridge, UK. *Proc. Int. Symp. Geotechnical Engineering of Hard Soils - Soft Rocks*, Athens, 20-23 Sept. Balkema, 405-416.

Butcher A.P. & Powell J.J.M. (1995). The effect of geological history on the dynamic measurement of the stiffness of the ground. *Proc. XI ECSMFE*, Copenhagen, May 1995. 1: 27-36.

Butcher A.P. & Powell J.J.M. (1996). Practical considerations for field geophysical techniques used to assess ground stiffness. *Proc. Int. Conf. on Advances in Site Investigation Practice*, ICE London, March 1995. Thomas Telford, 701-714.

Crilly M.S., Driscoll R.M.C. & Chandler R.J. (1992). Seasonal ground and water movement observations from an expansive clay site in the UK. *7th Int Conf on Expansive Soils*, Dallas Texas, 313-318.

Dyvik R & Madshus C (1985). Laboratory Measurements of Gmax Using Bender Elements. *ASCE Convention on Advances in the Art of Testing Soils Under Cyclic Conditions*, 186-196.

Hardin B.O. & Drnevich V.P (1972). Shear Modulus and Damping in Soils - Measurement and Parameter Effects. *ASCE, JSMFD*, 98(6): 603-624.

Hight, D.W., Bond, A.J. & Legge, J.D. (1992): Characterisation of Bothkennar clay: an overview. *Géotechnique* 42(2): 303-348.

Hope, V.S. (1993). Applications of seismic transmission tomography in civil engineering. *PhD Thesis, University of Surrey*, Guildford, UK.

Jardine, R.J. (1991). Discussion 'Strain dependent moduli and pressuremeter tests' *Géotechnique*, 41(4): 621 - 626.

Lambson, M.D., Clare, D.G., Senner, D.W.F., & Semple, R.M. (1993). Investigation and interpretation of Pentre and Tilbrooke Grange soil conditions. *Large scale pile tests in clay*, Thomas Telford, London, 134 – 196.

Marsland A. & Powell J.J.M. (1985). Field and laboratory investigation of the clay tills at the Building Research Establishment test site at Cowden, Holderness. *Proc Int Conf on Construction in Glacial Tills and Boulder Clays*, Edinburgh, 12-14 March 1985, 147-168.

Powell, J.J.M. & Shields, C.H. (1995): Field Studies of the Full Displacement Pressuremeter in clays. *Proc. 4th. Int. Symp. on Pressuremeters (ISP 4)*. Sherbrooke, Canada, May 1995.. 239-248

Simpson B, Atkinson JH & Jovicic V (1996). The Influence of Anisotropy on Calculations of Ground Settlements above Tunnels. *Proc. Geotechnical Aspects of underground Construction in Soft Ground*, The City University, London, 591-595.

Vucetic M & Dobry R (1991). Effect of Soil Plasticity on Cyclic Response. *ASCE, JGED*, 117(1): 89-107.