Laboratory measurements of small strain stiffness of a soft rock

Mesures en laboratoire de la rigidité de petite tension d'une pierre molle

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ABSTRACT: Tests have been carried out on natural and destructured soft rock at high pressures in a triaxial cell using bender elements to determine the small strain shear modulus G_o. The results demonstrate that the general framework which relates G_o to the current stress and overconsolidation ratio previously established for fine grained soils applies as well for the soft rock investigated.

RESUME: Des essais ont été poursuivi sur des pierres mous naturelles et décomposées à houte pressions dans une machine triaxial utilisant "bender elements" afin de déterminer la rigidité de petite tension G_0. Les résultates ont démontré que la théorie générale relative à G_0 par rapport à l'état actuel et établie dans le passé sur des sols de fins graviers est aussi applicable sur des pierres mous étudiées.
Laboratory Measurements of Small Strain Stiffness of a Soft Rock.

1 INTRODUCTION

The shear modulus for very small strains, G_o is now recognised as an important parameter. It defines the maximum stiffness for non-linear stress-strain behaviour and it strongly influences calculations of ground movements around engineering structures. Values for G_o may be determined in laboratory and in situ tests from measurements of the velocities of shear waves.

In laboratory tests on soils shear waves are commonly generated and detected using bender elements (Shirley and Hampton, 1977). The techniques for use of bender elements in triaxial and oedometer tests on soils and for interpretation of the results have been well established (Jovičić et al, 1996) and are now used routinely in many laboratories. For tests on rocks, shear plates (Brignoli et al., 1997) are used instead of bender elements because of the difficulty in installing bender elements into rock samples. However, there is less coupling between shear plates and the sample than between bender elements and the sample with the consequence that shear plates become ineffective for tests on soils and soft rocks.

For fine grained soils the relationship between G_o and the current state of the soil can be expressed by:

$$\frac{G_o}{p_r} = A\left(\frac{p'}{p_r}\right)^n R_o^m \qquad (1)$$

where p_r is a reference pressure, usually taken as p_r = 1kPa, required to make equation (1) non-dimensional: $R_o = p'_y/p'$ is the apparent overconsolidation ratio where p'_y is the stress at the intersection of the current swelling and recompression line with the normal compression line: A, n and m are material parameters (Viggiani and Atkinson, 1995a). For coarse grained soils it is difficult to determine a value for p'_y and G_o is commonly related to effective stress and a function of the voids ratio (Hardin and Black, 1966).

Tests on bonded and unbonded sands and soft rocks have shown that the strengths and stiffnesses of these materials generally conform to the overall framework established for fine grained soils (Coop and Lee, 1993; Coop and Atkinson, 1993). At high pressures, sands and soft rocks reach well- defined normal compression lines and it is then possible to define the state of a coarse grained soil in terms of its apparent overconsolidation ratio which measures the distance of the current state from the normal compression line. To investigate further the small strain stiffness of sands and soft rocks, tests were carried out in the high pressure triaxial cell described by Coop and Lee (1993) fitted with bender elements to measure G_o.

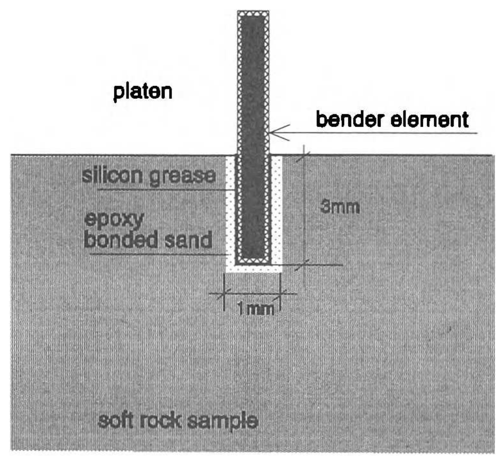

Figure 1. Installation of bender element into a sample of soft rock.

2 EQUIPMENT AND TEST PROCEDURES.

Tests were carried out on 50mm dia. samples of natural Greensand and on the same material after it had been destructured. Values of G_o were measured at confining pressures up to 70MPa.

For bender element tests on soils in a triaxial or oedometer apparatus it is usually possible simply to push the bender elements into the top and bottom faces of the sample without damage to either the bender elements or the sample. For very stiff soils and soft rocks, however, this technique is not possible and it is necessary to excavate slots in the ends of the sample to receive the bender elements. In this case it is important to seal the bender elements into the slots so that they effectively transmit and receive the shear waves.

The method adopted for the present tests on natural samples of Greensand is illustrated in Figure 1. After the sample had been trimmed to the required size small slots were cut into each end by hand. The slots were filled with a mixture of destructured Greensand and quick-setting epoxy resin (Araldite) and the bender elements pushed into the slot. The bender elements were lightly coated with silicon grease to facilitate extraction at the end of the test. The mixture was made from approximately equal proportions of sand and epoxy resin by weight. The sand was obtained from the trimmings formed during sample preparation. This gave a mixture which initially had the consistency of a thick paste, which set and cured in a few minutes and, when set, had a stiffness comparable to that of the intact Greensand.

As with all bender element tests it is necessary to use a transmitted waveform and frequency which avoids the near field effect and for which the received signal is clear (Jovičić et al., 1996). For the tests on Greensand the wave form used was a single sine wave pulse with a frequency of about 7.5kHz. Pulses were transmitted at intervals sufficiently long to avoid interference between successive pulses.

Figure 2 shows traces of the transmitted and received signals copied from the oscilloscope output obtained from a typical bender element test on natural Greensand. The travel time of 0.0636ms has been obtained from the first deflections of the transmitted and received signals. The traces shown in Figure 2 are typical of those obtained from all the tests on natural Greensand. They were, in all cases, sufficiently clear for consistent and accurate measurements of shear wave velocity to be made without recourse to numerical analyses of the signals (Viggiani and Atkinson, 1995b).

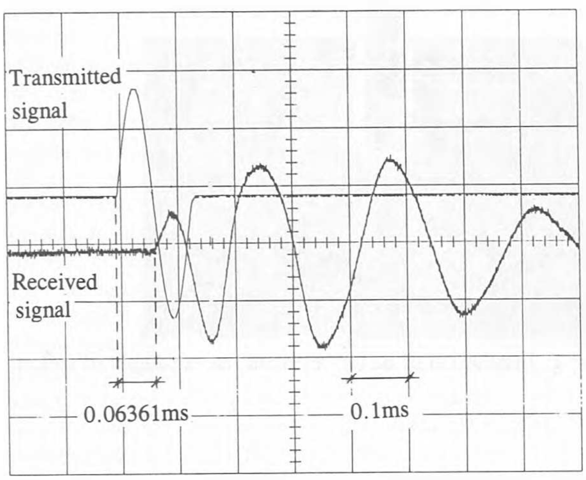

Figure 2. Transmitted and received bender element signals from a test on intact Greensand.

3 LABORATORY TESTS

The Greensand used in the tests is a Cretaceous deposit which outcrops in the Weald basin and to the north-west of London. Typically it is uniformly graded with grain sizes in the range 0.1mm to 0.3mm. The grains are predominantly rounded quartz and the matrix is cemented principally by iron oxide. Block samples were obtained from a quarry near Maidstone in Kent and they have been described in detail by Cuccovillo (1995). The unconfined compressive strength of the natural samples was generally about 1MPa.

Samples of natural material were prepared from the blocks by careful hand trimming in a soil lathe using a metal file: the ends were trimmed using a flat sandpaper block. Destructured samples were prepared from a block sample using a rubber pestle and mortar taking care not to damage the sand grains. Samples for testing were then manufactured by wet compaction and saturated under a back pressure using carbon dioxide flushing (Lee, 1991).

4 INTERPRETATION AND DISCUSSION OF THE RESULTS

Figure 3 shows data for cycles of isotropic compression and swelling for natural and destructured samples: the maximum confining pressure reached in the tests was 70MPa. At effective stresses in excess of about 2MPa the data from the test on the destructured sample define a linear normal compression line with gradient $\lambda = 0.16$. At these high effective stresses the mechanism of compression is due predominantly to fracture and crushing of the grains (Coop and Lee, 1993). Along AB the sample is defined as compacted because it has not yet reached the normal compression line while along CDE and FG the sample has been compressed and swelled. The important difference between a compacted sample and a compressed and swelled sample is that the former has never reached the normal compression line.

Figure 4 shows values of G_0 obtained from the bender element measurements at the states shown in Figure 3 plotted against the effective stress, both plotted to logarithmic scales. The points marked A to G correspond to the points marked in Figure 3.

The stiffnesses of the destructured sample vary with effective stress and specific volume. For states when the sample was near the normal compression line (BCEF in Figure 3) the relationship between log G_0 and log p' in Figure 4 is linear and is given by equation (1) with $A = 4000$ and $n = 0.59$.

For states which are not normally consolidated, such as AB, CDE and FG the relationships between log G_0 and log p' are also nearly linear but with slightly different gradients and, in general, the small strain stiffness at these states is a little larger than at corresponding normally consolidated states at the same effective stress. The differences between stiffnesses at normally consolidated and overconsolidated states in Figure 4 are not great and close inspection is required to notice that the gradient along AB in Figure 4 is a little different to the gradient along BC.

Also shown in Figure 4 are the small strain stiffnesses G_0 of the natural sample. At effective stresses up to about 5MPa the small strain stiffness G_0 is nearly independent of the effective stress indicating that the stiffness is controlled by the bonding. At effective stresses above about 5MPa the stiffness increases slowly with effective stress. At very high effective stresses indeed, the stiffness of natural samples might approach the stiffness of destructured samples as increasing effective stress and continuing volumetric strain destructures the natural material.

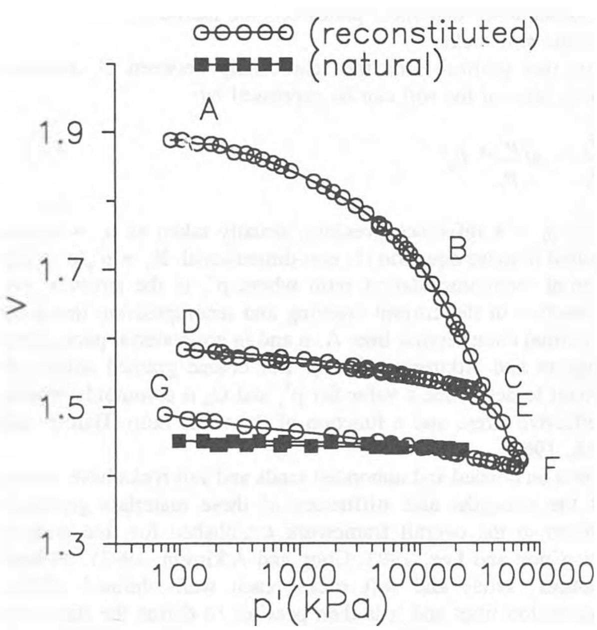

Figure 3. Isotropic compression and swelling of natural and destructured Greensand.

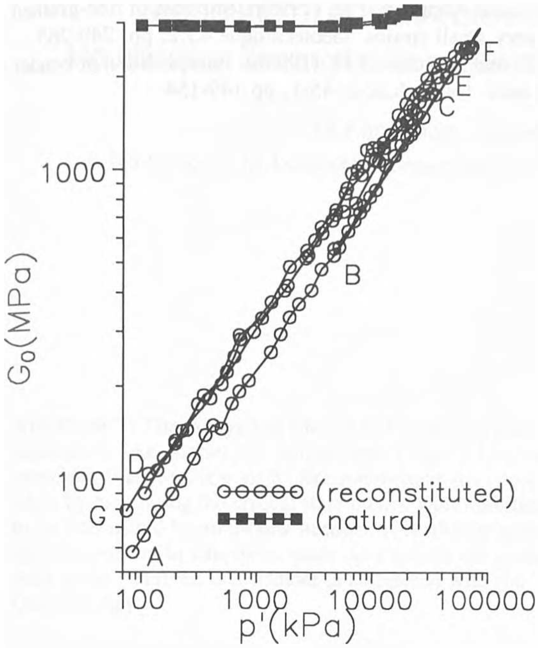

Figure 4. Small strain stiffness of isotropically compressed natural and destructured Greensand.

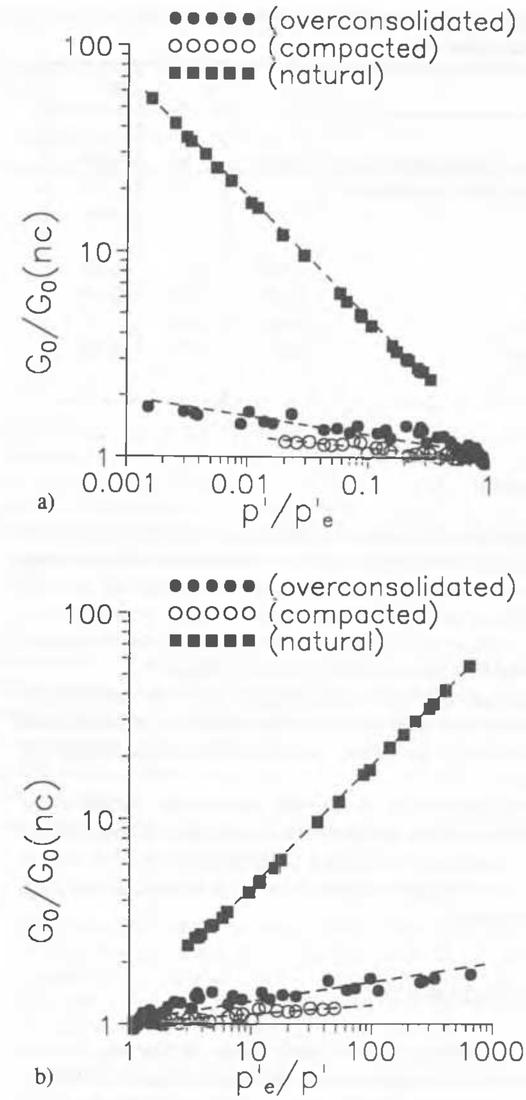

Figure 5. Small strain stiffness of isotropically compressed natural and destructured Greensand.

Figure 5(a) shows the variation of small strain stiffness of Greensand with effective stress normalised with respect to the equivalent pressure p'_e and G_o normalised by dividing by G_o(nc) which is the stiffness of a normally consolidated sample at the same effective stress. The equivalent pressure p'_e is the pressure on the normal compression line at the same specific volume. The value of p'/p'_e describes the distance of the current state from the normal compression line and, since the gradients of the swelling and recompression lines in Figure 4 are relatively shallow, it is approximately equal to the reciprocal of the apparent overconsolidation ratio. The data in Figure 5(a) are for both natural samples and for destructured samples which have reached their current states either by compaction or by compression and swelling.

The data in Figure 5(a) show that all samples are approaching the same state for normally compressed samples where $G_o = G_o$(nc) when $p' = p'_e$. At other states, however, the stiffnesses of the natural samples are considerably greater than the stiffnesses of destructured samples. The stiffnesses of the compacted destructured samples are a little smaller than the stiffnesses of the compressed and swelled destructured samples at the same state. The higher stiffnesses of the natural sample can be attributed to bonding. The reduction of G_o/G_o(nc) for the natural sample is due more to an increase in G_o(nc) with increasing effective stress than to the small changes of G_o shown in Figure 4.

Figure 5(b) shows the same data as in Figure 5(a) replotted as p'_e/p' which is approximately the same as the overconsolidation ratio R_o. This demonstrates that G_o increases with the logarithm of overconsolidation ratio as given by equation (1). From the gradients of the lines the parameter m in equation (1) is 0.61 for natural Greensand, 0.09 for destructured, compressed and swelled samples and 0.04 for destructured and compacted samples.

The data shown in Figures 4 and 5 demonstrate that the small strain shear modulus G_o for natural and destructured Greensand is related to the state and history by the same equation (1) which was found by Viggiani and Atkinson (1995a) for fine grained soils. Table 1 summarises the values for the parameters in equation (1) for Greensand and for the soils tested by Viggiani and Atkinson (1995a).

Generally the value of A is smallest for plastic clays and increases with plasticity and with increasing grain size. The value A = 4000 for destructured Greensand is the largest recorded. The value of the exponent n reduces with decreasing plasticity of fine grained soils and with increasing grain size in coarse grained soils. The value n = 0.59 for Greensand is the smallest recorded. The value of the exponent m which describes the increase of G_o with overconsolidation is more variable. For reconstituted fine and medium grained soils the value of m is consistently about 0.2. For natural Greensand m = 0.61 while for destructured Greensand m < 0.1.

With $R_o = p'_y/p'$ equation (1) becomes:

$$\frac{G_o}{p_r} = A\left(\frac{p'}{p_r}\right)^n\left(\frac{p'_y}{p'}\right)^m \tag{2}$$

For a set of states on a particular swelling and recompression line the value of p'_y remains a constant. For the natural sample G_o was found to be approximately constant, as shown in Figure 4, and hence, for natural Greensand n = m as found. This result demonstrates that equation (1) is sufficient to describe the variation of G_0 with state without adding a function of specific volume.

Table 1: Values of the parameters in equation (1) for Greensand and other soils.

	A	n	m
Greensand:			
destructured and compressed	4000	0.59	0.09
destructured and compacted			0.04
natural			0.61
Slate dust	2500	0.72	0.21
Kaolin clay	2000	0.65	0.19
Fucino Clay	500	0.84	
London Clay	400	0.75	0.24

CONCLUSIONS

1. Measurements of G_0 have been made successfully on samples of soft rock using bender elements in a triaxial cell with confining pressures up to 70MPa and at overconsolidation ratios up to 500.
2. The stiffness of natural Greensand is larger than that of a destructured sample at the same effective stress. The difference can be attributed to bonding in the natural sample.
3. The relationships between G_0, stress and overconsolidation previously established for unbonded fine grained soils were found to apply also for both the natural and the destructured Greensand samples tested.
4. The analyses which reveal the similarities between the behaviour of the bonded and unbonded materials require tests on samples with states on the normal compression line. For coarse grained soils this usually requires tests to be carried out at high confining pressures.

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