ESTIMATION OF THE STIFFNESS OF WEATHERED AND FRACTURED ROCK

J.M.O. Hughes*

May, 1998

ABSTRACT

Large structures, particularly bridge foundations, are often founded on weathered and fractured rocks which are unlikely to collapse under load. However movement, both vertical and lateral, can be a concern in the design of the structure. Further, in large structures, the dynamic forces caused by earthquake motion can be influenced by the elastic stiffness of the foundation. Even with the best coring, with full core recovery, estimation of the mass stiffness from a core sample comprised of a collection of very hard rock pieces -- often with significant weathering between the joints -- is a major challenge. However, in many situations, the wall of the hole left after coring will be in a relatively smooth condition, even though the core may only be partially recovered. Some indication of the stiffness of the mass of the material can be obtained by testing the wall of the core, rather than the core itself.

INTRODUCTION

If the material below a foundation is homogeneous, and a complete undisturbed sample can be obtained either by driving a tube or by coring, then conventional geotechnical testing methods will provide sufficient insight into the appropriate material properties.

However, in weathered and fractured rock or in gravelly clays, the materials are very heterogeneous. Some indication of the properties of the various components of the mix can be determined from conventional testing techniques. For instance, compression tests can be completed on intact sections of the rock core to determine the shear strength and the intact rock stiffness (Figure 1). Shear and consolidation tests can be conducted on the fill material. Even if complete cores can be obtained and the properties of the components determined, estimation of the material stiffness is, in many cases, a matter of experience and judgement. Seismic shear wave and P-wave results taken "down hole" may assist in identifying material type boundaries such as the depth of weathering, but they tend to offer little assistance in determining the stiffness under the working load conditions.

---

* Hughes Insitu Engineering Inc., 804-938 Howe Street, Vancouver, B.C. V6Z 1N9
Tel. (604) 331-4451 Fax (604) 331-4452

In many materials that are very heterogenous in nature and are difficult to sample, where the core itself is poor or non-existent, the side wall of the hole from which the core has been cut may in fact be relatively intact and smooth, particularly if careful attention is given to cutting the hole with the intent of producing a smooth wall. If this can be achieved, there is the possibility of completing a loading test on the wall. The resulting load deformation curve will be on the wall of the material in its in-situ state. Hence, the resulting data will reflect the behaviour of in-situ material, the disturbed material adjacent to the wall, and that undisturbed material some distance out.

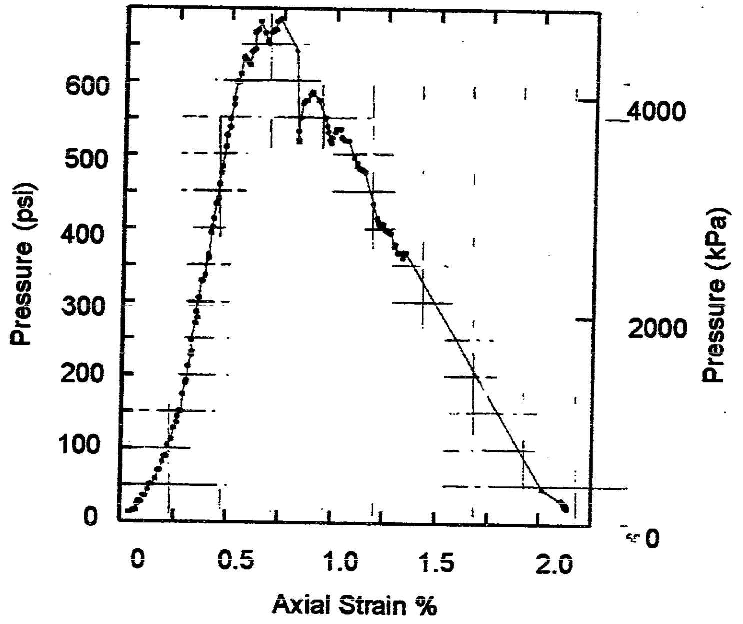
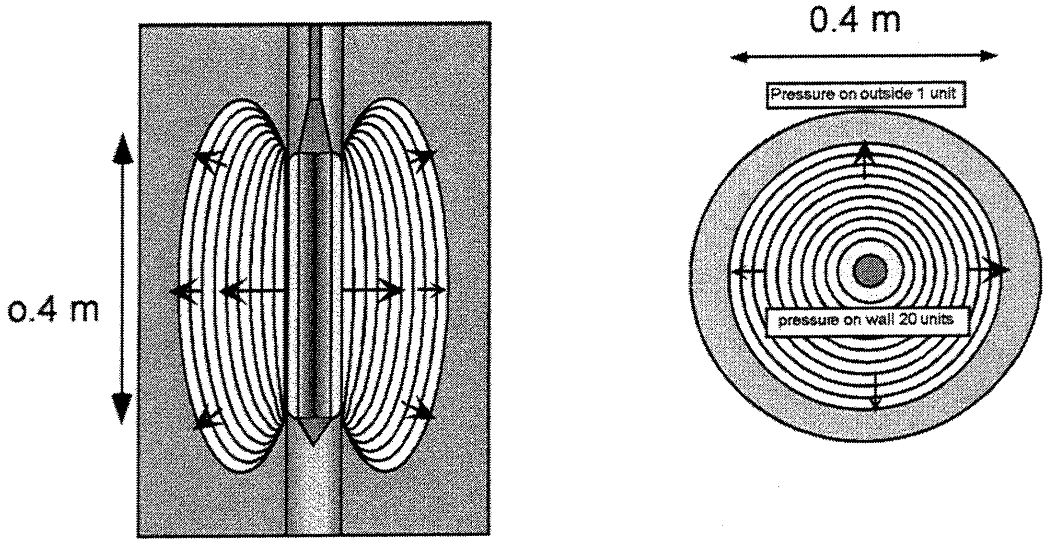


Figure 1. Unconfined compression test on the rock core for the test shown in Figure 7c

In contrast to determining the material properties on a core sample of, for example, 76 mm in diameter or less, a very large volume of the material can be stressed by loading the wall of the cavity. If, for instance, a radial stress of 20 times the design stress is put on the wall of the cavity, then at a diameter of approximately 400 mm, the radial stress will have dropped to the design stress. Hence, although the stress distribution has varied radially, a very large block of material has been stressed. This is rather analogous to the stress distribution below a test pile. Under test conditions, a large block of material is under load, even though the soil beneath the footing is under a varying stress condition.

Therefore, the volume of material tested by loading the walls of the cavity is very large. It essentially has a "doughnut" shape, being 400 mm high (the typical length of the loaded wall) with an outer diameter of 400 mm and a 76-mm hole in the middle (Figure 2). The volume of soil contained in this "doughnut" is over 20 times the volume of the hole from which the core has been taken. Although the disturbance to the soil adjacent to the wall will have some influence on the data as a result of the drilling, there is the potential of determining the average material properties of a very large mass of soil.



Vertical section

horizontal section

Figure 2. Zone of influence of stress in the rock under a lateral stress on the wall of the borehole

TESTING THE DRILL- HOLE WALL

Conventional View

The pressuremeter test is the common name for the type of test that is performed on the drill-hole wall. This test is usually understood to be a performed by a standardized instrument, with the test conducted in a standardized manner, irrespective of the problem under consideration. The data are interpreted in a standard manner to provide two index numbers:-

- a limit pressure, P_L
- a pressuremeter modulus, E_M.

A typical standard test, comprising about ten points, is shown in Figure 3. The various material properties are then determined by empirical correlations with the two index numbers.

In materials such as stiff clays, in which there is a wealth of data and field performance information, the "Ménard pressuremeter system" works very well. Unfortunately, this vision of the pressuremeter test can be a little restrictive in trying to determine the properties of materials that are very difficult to sample, or those of a heterogeneous nature for which little published data is available. By looking at the test in an alternative manner, further insight can be gained into the type of test that might be conducted, and how the data can be used for determining the properties of these materials.

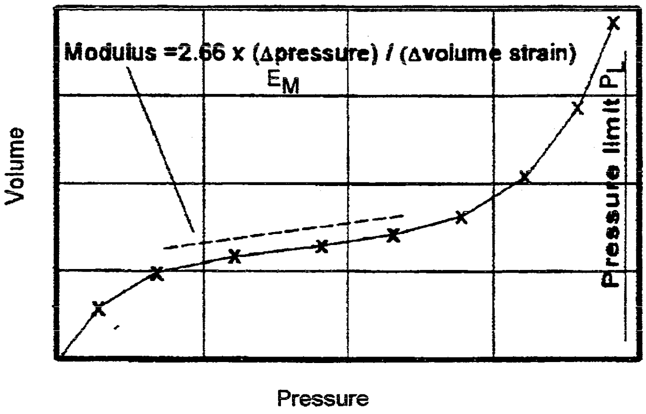


Figure 3. Standard Ménard Pressuremeter Test

Alternative View

The cavity wall is loaded radially with an "instrumented packer", commonly known as a rock dilatometer, or pressuremeter. The packer provides a means of loading the borehole wall radially, by applying a pressure inside the flexible packer. If the pressures and displacements are known, then the radial movement of the wall can be evaluated for a given pressure. The resulting "boundary value" information does not have to conform to any standard test. It can result from any loading pattern that is appropriate for the particular problem under consideration. For tests in very stiff material, particularly if the tests are at some depth, the standard pressuremeter in which displacements are measured by changes in fluid levels at the surface may not provide sufficient accuracy. Instruments in which the sensors are electronic and inside the probe are probably more appropriate.

Figure 4 is an example of a dilatometer with a pressure range of 20 MPa with electronic sensors. Instruments of this type have been used to depths of over 700 m. As the data from such instruments can easily be captured electronically, there is no restriction on the number of data points obtained. In practice, several hundred points are collected for each test.

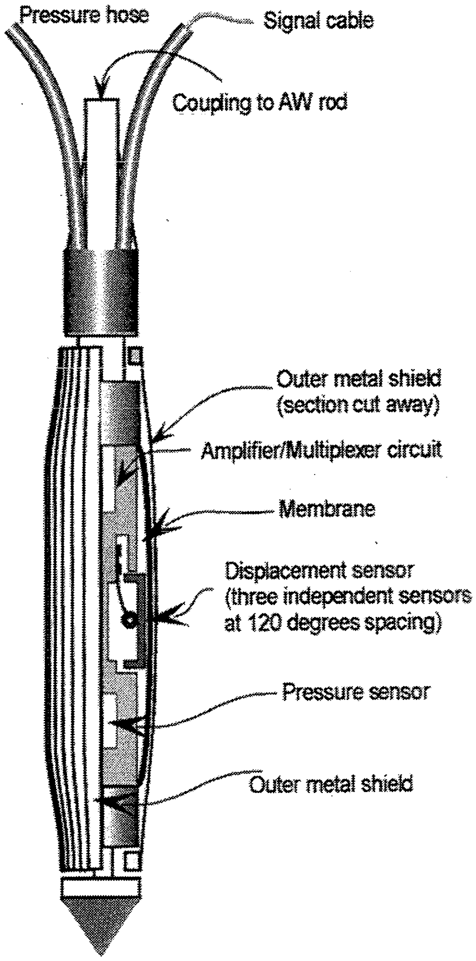


Figure 4. High-pressure dilatometer

The boundary value information can be used in four ways:

- The two index parameters P_L and E_M can be calculated and used empirically to determine the behaviour of the material. (Briaud 1992)
- The material parameters can be directly determined from the boundary value information by assuming that the material deforms according to a particular model. (Hughes 1997 and Roy 1997)
- The boundary value information can be used to check on the suitability of a particular complex material model. (Salgado and Byrne 1990)
- A qualitative indication of the behaviour under a particular loading situation can be obtained.

In general, to use the boundary value data for the first method, the test is restricted to be performed in a standard manner. However, in the other methods, the test can be performed in the manner which is more appropriate to the data required.

The object is not to conduct a standard test, but to obtain boundary value information that is as relevant as possible to the problem under consideration. For example, in a situation where piles are laterally-loaded under earthquake conditions, where large cyclic lateral loading is the major concern, then a qualitative indication of the behaviour of the material can be obtained by conducting large cyclic load/reload on the wall of the borehole. Figure 5 is an example of large-cycle lateral loading tests on fractured rocks for the bridge at Point Orford, Oregon. From a visual inspection of the data, it is clear that substantial movement occurs with each cycle of load. Hence, from a design point of view, the mass stiffness -- i.e. the secant stiffness from zero strain to the peak of successive unload cycles -- degrades with the number of cycles.

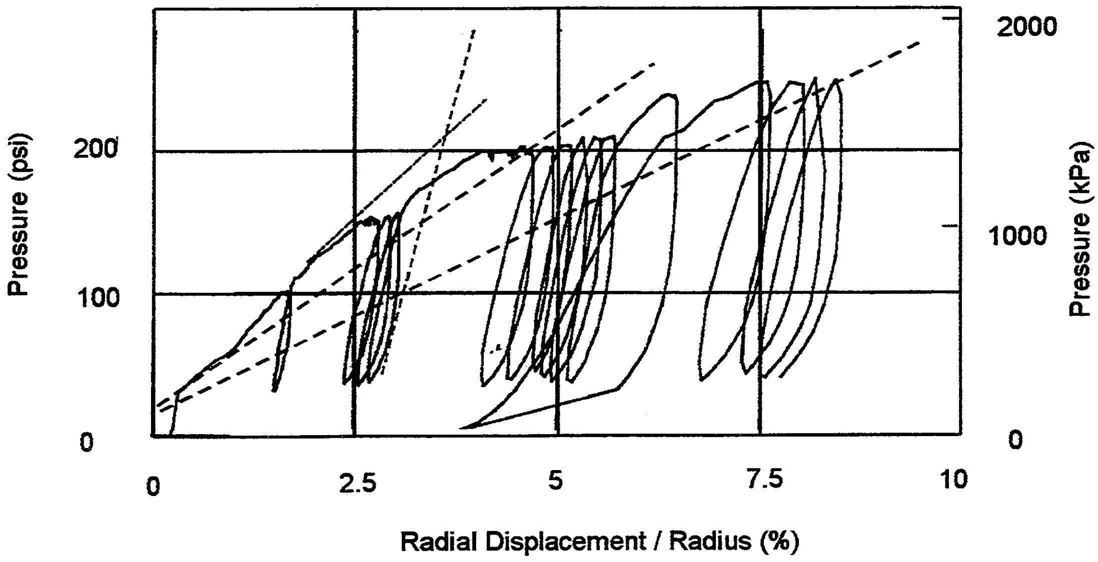
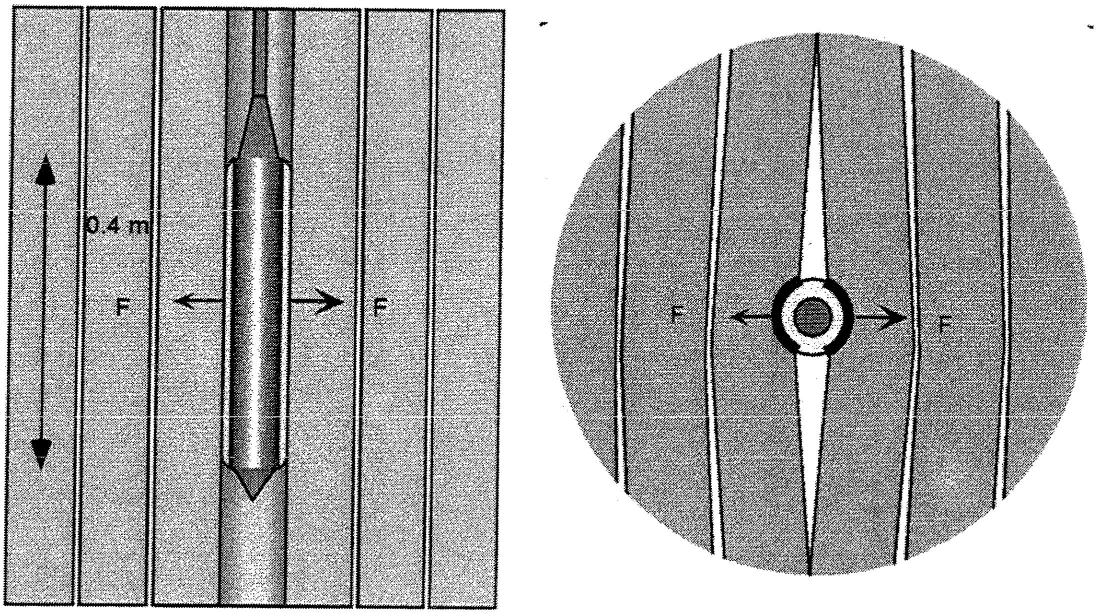


Figure 5. Cyclic boundary value information on fractured rock

PROOF LOADING

In the conventional manner of examining this borehole test, the wall is considered to be stressed radially. However, consider that on the outside of this dilatometer are two imaginary semicircular rigid plates, the length of the packer section. As pressure is applied, these plates will move radially outward against the wall. The plates will then be applying a horizontal force on the rock. If a stress of 20 MPa is applied to the packer, then an equal and opposite force of 600 kN is applied to a 400 mm length of borehole (Figure 6). This is indeed a very large force. Hence, the boundary data pressure/radial displacement can be considered as a load/displacement curve. In a qualitative manner loads or forces are, in general, much more easily understood than stresses. Hence, the test can be visualized as a proof load test of the material.



Vertical section

Horizontal section

Force (F) = vertical cross section of pressuremeter * maximum pressure

Figure 6. Proof loading of rock

DETERMINATION OF STIFFNESS

In weathered rocks, which are essentially an assemblage of rock pieces, two distinct moduli can be observed: an average modulus that would have been identified by the Ménard modulus E_m, and a much steeper modulus defined from a unload-reload loop. The ratio between these moduli is some function of the rock fractures, orientation and the weathering. Figures 7a, 7b and 7c are three tests on rock which has been described as a heavily-fractured sandstone. All of the samples have an RQD (rock quality designation) of zero: over the pressuremeter test length, there is no piece of core longer than 100 mm. In each case there was 100% recovery. The only difference in the engineering geological description of the three rocks was in the state of weathering on the joints. The first two tests were described as having "fresh" fractures, whereas the third was described as "weathered".

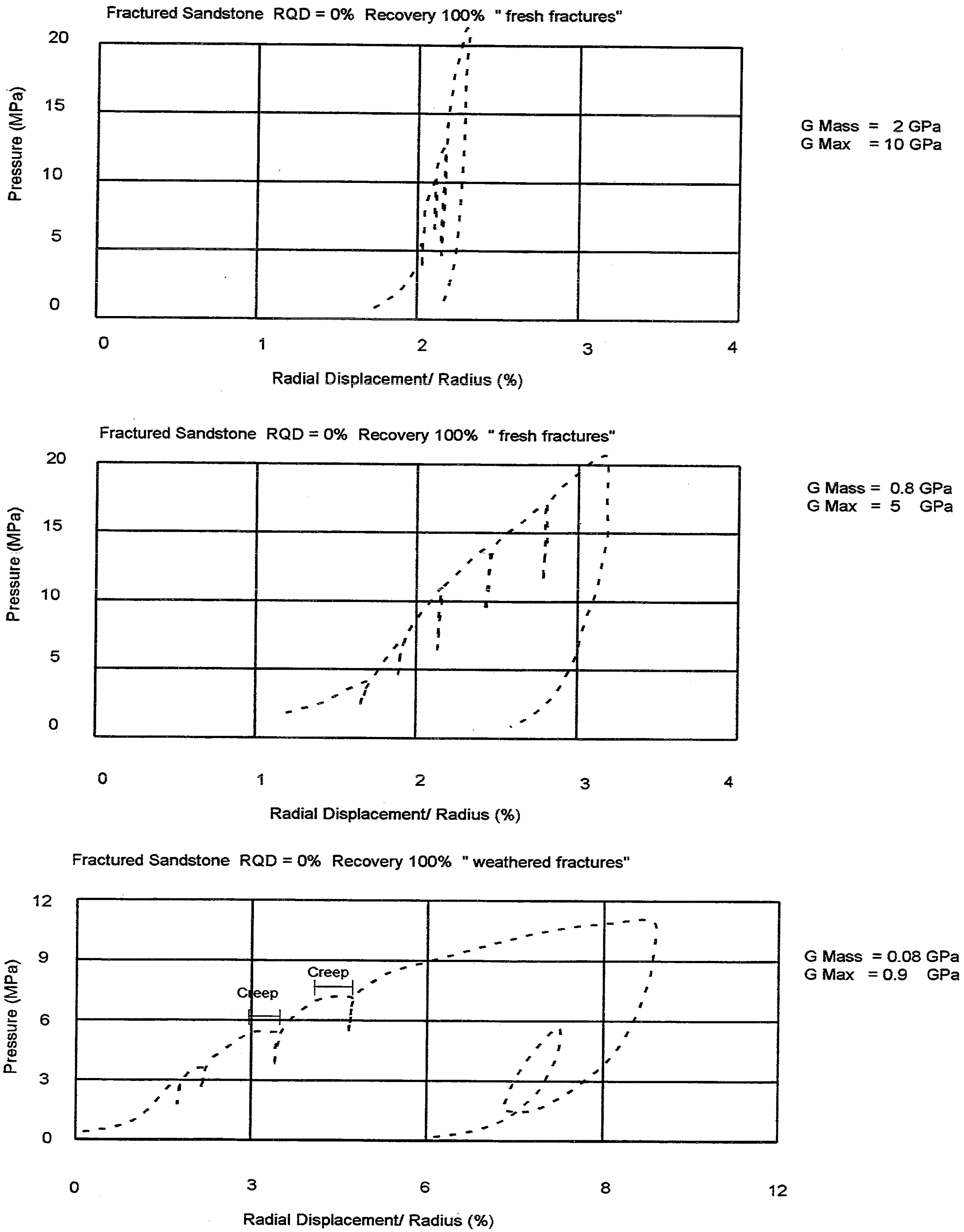
From a visual inspection alone, it is clear that each of the tests behaves in a different manner. (Note the different scales for Figure 7c). The average mass modulus varies from 2.0 GPa to less than 0.1 GPa. The slope of the unload-reload loops varies over a large range of between 10 GPa and 0.9 GPa. What is of further qualitative interest is the creep which occurs under load in the weathered rock test. Before the unload-reload loops in Figure 7c, the pressure was held for five minutes, during which time significant radial movement occurred. In contrast, no creep movement was observed in the tests on fresh fractured rocks.

In the above discussion the slope of the curves has been defined by a shear modulus value. This modulus has been calculated from the slope of the curve, assuming that the material surrounding the dilatometer behaves as a homogeneous linear elastic material. From a mathematical point of view, this assemblage of rocks is unlikely to behave in such a fashion. However, with this simple assumption, some indication of the lateral in-situ modulus can be determined. The vertical modulus will not necessarily have the same value. It will doubtless be some function of the fracture pattern and orientation.

However what is important is that the above data have been obtained on a very large sample of the rock: a "doughnut" shape, 0.4 m diameter with a height of 0.4 m. Further, when looking at the data as a proof load on the soil at a stress of 10 MPa, i.e. 300 kN lateral load, it is clear that the wall in Figure 7a moves 0.15 mm radially outward, that in Figure 7b 0.45 mm, and that in Figure 7c 5.3 mm.

CONCLUSION

In view of the nature of the material, the wall-displacement boundary-value curve is a result of a complex interaction of the joint spacing, orientation, weathering, failure properties of the rock and the weathering material. However, from a qualitative examination of the curve, the gross material behaviour can be clearly seen. By assuming that the material deforms in a simplistic manner, a shear modulus can be determined for the mass of the material in loading and unloading. As the test stresses a very large block of soil, the average mass in-situ property may well be more representative than that obtained from core samples.



Figures 7a, 7b and 7c. "Boundary value" (pressure displacement) data on tests in fractured sandstone

ACKNOWLEDGMENTS

The data presented in this paper have been taken from projects directed by the following Companies:

- Shannon & Wilson Inc., Boston, MA
- Golder Associates, Seattle, WA
- GeoSyntec Consultants, Los Angeles, CA

The concepts have been developed while working on other projects with the following Companies:

- Earth Mechanics, Inc., Los Angeles, CA
- JMA, (Jacobs Engineering Group, Mott Macdonald-Hatch, ACG Environments), Los Angeles, CA
- Earth Technology Inc., Los Angeles, CA
- Fugro West, Los Angeles, CA
- CH2M Hill, San Francisco, CA
- The California Department of Transportation
- Shannon & Wilson, Inc., Seattle, WA

The support from all of these groups is very much appreciated.

REFERENCES

Briaud, J.L. 1992. The Pressuremeter. A.A. Balkema, Rotterdam.

Hughes, J.M.O. 1997. Pressuremeter testing in weathered rock. 50th Canadian Geotechnical Conference, Ottawa, Ontario, October 20-22, pp. 681-686.

Roy, D. 1997. Deformation behaviour of granular deposits from self-boring pressuremeter. Ph.D. Thesis, University of British Columbia.

Salgado, F.M., and Byrne, P.M. 1990. Finite element analysis of pressuremeter chamber tests in sands. Proceedings of the 3rd International Symposium on Pressuremeters, April 2-6, Oxford University, pp. 209-220.