Determination of the Engineering Properties of the Coode Island Silts using a Self Boring Pressuremeter

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1 INTRODUCTION

The eastern approaches of Melbourne's West Gate Bridge are being constructed as an elevated freeway some 2 km long. For the most part this freeway is being supported on large diameter cast in situ reinforced concrete piles. These piles range in diameter from 1.1 to 1.5 m and are socket ed into Silurian mudstone and sandstone. These piles are generally in excess of 40 m in length. The upper 30 m to 35 m is in alluvial deposits ranging from dense gravels to soft silty clays. The upper 10 m to 20 m of these deposits are in the so-called Coode Island Silts, which are in fact predominantly soft to firm silty clays.

These piles must resist the heavy vertical loads from the dead and live loads of the freeway structures as well as lateral loads imposed by thermal stresses in the bridge deck and substantial construction loads resulting from the prestressing of the deck.

The vertical loads are all assumed to be carried in the mudstone. The method of analysis of these pile sockets is described by Williams (1980). The in situ strength and moduli, required for this analysis, have been determined by pressuremeter testing. A significant portion of this testing has been carried out using a high pressure pressuremeter described by Hughes and Ervin (1980).

The lateral resistance of the piles must be provided by the strength of the Coode Island Silts, particularly the material near the surface. Hence the assessment of the stiffness properties as they relate to the lateral behaviour of the piles is of critical importance to the action of the piles. In view of the importance of understanding the action of these piles, full-scale lateral load pile tests were conducted by the Country Roads Board of Victoria, under the direction of Mr. P. McDonald. As part of this pile load testing programme, an extensive series of in situ tests was undertaken. These tests were conducted in two parts:

Part (A) The determination of the in situ, undisturbed, properties of the Coode Island Silts.

Part (B) The determination of the in situ properties of the Coode Island Silts as they relate to the particular pile installation process.

The lateral load pile tests were conducted on four prototype piles, 1.5 m in diameter, which were loaded by pulling them together and jacking them apart. The results of these full-scale tests and the comparisons with the predictions based on the in situ tests are described by McDonald and Scott (1980) The results reported in this paper relate to the determination of the modulus and the undrained strength of the Coode Island silty clays and their stiffness in relation to the behaviour of the lateral loading of the pile.

2 GENERAL SITE CONDITIONS

The general area of the West Gate Bridge site and the location of the West Gate Freeway in South Melbourne are shown in Figure 1. A simplified geological profile along the line of the freeway is shown in Figure 2. Essentially the Silurian age bedrock is overlain by Tertiary and Quaternary age alluvial deposits, of which the near surface deposits are known as the Coode Island Silts.

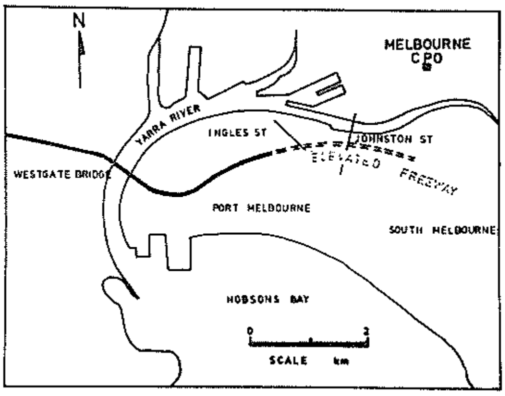


Figure 1 - Location of the West Gate Freeway Project

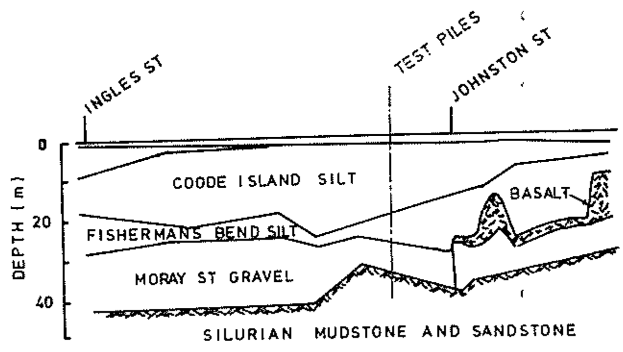


Figure 2 Simplified Geological Section along part of the Freeway

In some areas, generally in the middle of the elevated freeway, some Tertiary age Basalt flows are present within the alluvial deposits. For most of the length of the freeway the lateral resistance to the piles is provided by the soft silty clays in the top 10 metres of the soil. These soils are recent deposits and from Geological evidence they would appear to be normally consolidated. However, the results of both laboratory and field tests indicate they are slightly overconsolidated, probably as a result of dessication, fluctuation in groundwater level and/or aging.

The average moisture content for the silty clays of the Coode Island Silts range from 49-82% and the Liquid limit and Plastic index range from 58-103 and 35-80 respectively.

Further, although the material in the top 20 m is described as Coode Island Silts it also contains shell beds and lenses of silt and sand.

3 SITE INVESTIGATION AT TEST PILE SITE

Part (A) The determination of the in situ, undisturbed properties of the Coode Island Silts

This investigation was done using the electrical friction cone penetrometer developed at the Country Roads Board by Holden (1974), a Ménard pressuremeter and a self boring pressuremeter.

The penetrometer does not give a direct measure of either the shear strength or the modulus, however, it does give a measure of the consistency of the material. The results of a typical penetration test are shown in Figure 3. The average cone resistance increases almost linearly until a depth of 14 to about 20 m is reached. The interfingering of numerous sand or shell lenses shows up clearly as spikes on the penetration records. Even though the penetrometer does not give a direct measure of the modulus or the shear strength, numerous authors have made correlations with the penetration resistance Most of these have been summarised by Sangleret (1972)

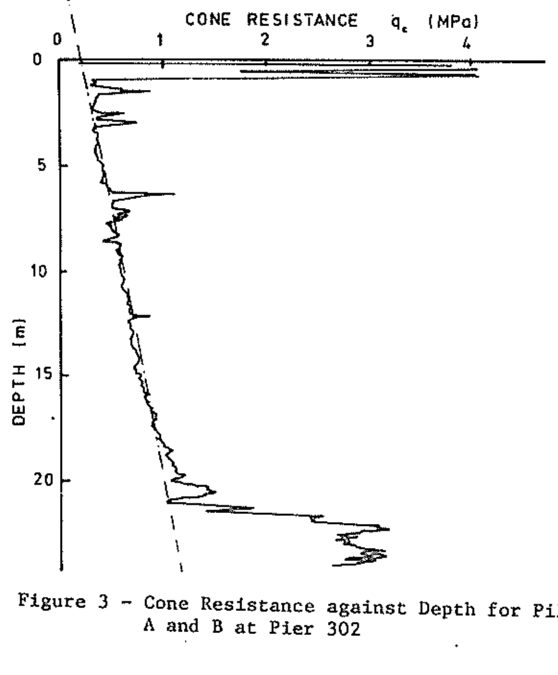


Figure 3 - Cone Resistance against Depth for Piles A and B at Pier 302

For a normally consolidated clay - the generally accepted correlation is:

$$c_u = q_c/10 \text{ to } q_u/15$$

and

$$E = 150 \text{ to } 400\ c_u$$

Using the above correlations with the cone resistances shown in Figure 3 gives the shear strength and modulus graphs in Figure 9a and b. It is interesting to note that the average increase in cone resistance with depth in the upper 12 m for all the tests, which were about 50 m apart, is very similar.

In contrast with the penetrometer, the Ménard-pressure meter is designed to give a direct measure of the modulus and shear strength. The Ménard pressuremeter (Ménard 1957), which is placed down a pre-drilled hole, is expanded radially against the sides of the borehole during the test. A typical result is illustrated in Figure 4.

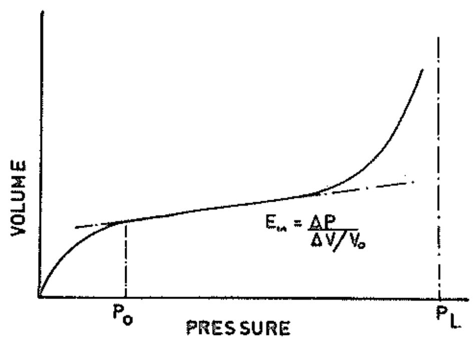


Figure 4 - Typical Ménard Pressuremeter Curve

The modulus is obtained from the flat section of the curve and the shear strength is conventionally evaluated from the difference in pressure between the limiting pressure P_L and the in situ pressure P_o

$$c_u = (P_L - P_o)/5.5$$

The results of the Youngs modulus and undrained shear strength obtained from the Ménard test are shown in Figures 5a and b.

The third instrument which was used extensively at this site was a self boring pressuremeter. The instrument used here is a modified version of the self-boring pressuremeter (Camkometer) which was developed at Cambridge in the early 70's. (Hughes 1973). This instrument essentially consists of a thick walled tube which is slowly jacked into the ground.(Figure 6) The material displaced by the instrument is removed by the action of a central rotating cutter; water or drilling mud is pumped down the central rotating cutter rod and the mud and cuttings return to the surface in the annular space between the cutter rod and the body of the instrument.

Using this technique the instrument can be placed in the ground with the soil surrounding the instru ent suffering very little disturbance.

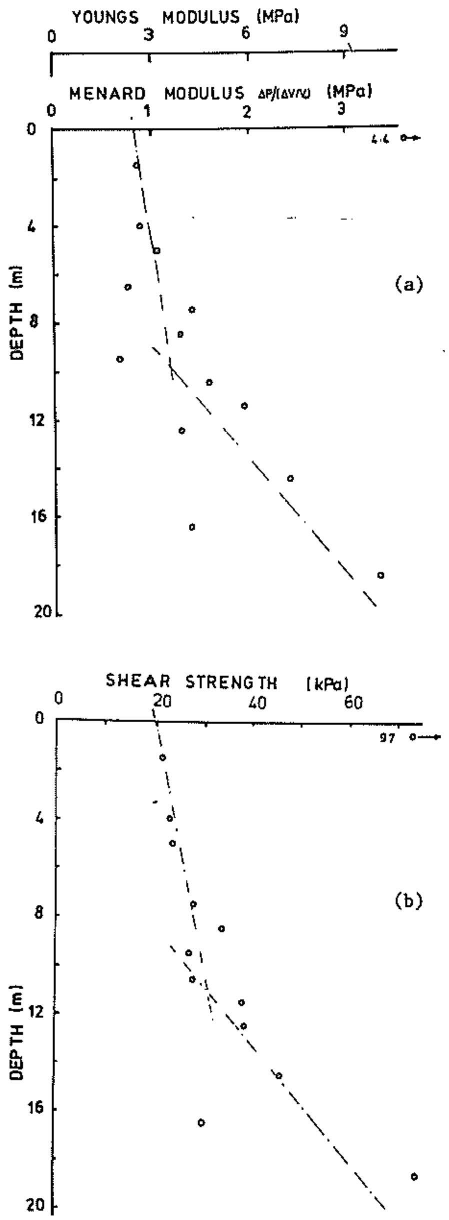


Figure 5 - (a) Modulus and (b) Shear Strength obtained from a Ménard Pressuremeter

Once the instrument is in place the membrane on the outside of the thick-walled tube can be expanded against the undisturbed soil (Figure 6). The pressure required to expand the membrane and the radial displacement of the membrane are recorded electrically with transducers inside the instrument.

The instrument used at the West Gate test site was developed by the first author in conjunction with Coffey and Partners for commercial use, where ease of operation is essential. The instrument can be placed in the ground using virtually any rotary drilling rig.

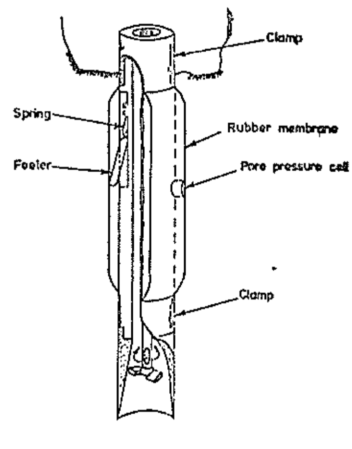


Figure 6 - Self-Boring Pressurementer (Camkometer)

A typical result of a self-boring pressuremeter test is shown in Figure 7. Several features of the behaviour of the soil can be determined from this test. In particular, the in situ lateral stress, the Young's modulus and the undrained stress/strain curve. The in situ lateral stress is considered to be the pressure at which the membrane expands from the body of the instrument. In the test shown on Figure 7 this would correspond to 130 kPa. The in situ Young's modulus is more difficult to determine because the soil in almost all cases is non linear hence the modulus shown is the secant modulus at 1% radial displacement. Clearly if a larger displacement was considered a reduced modulus would be obtained.

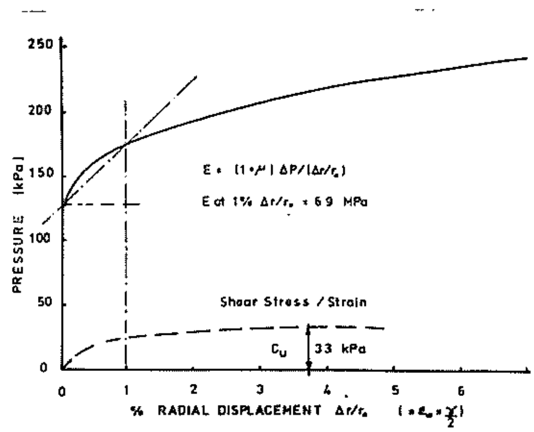
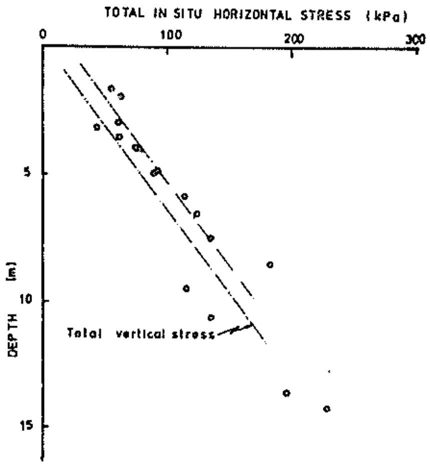


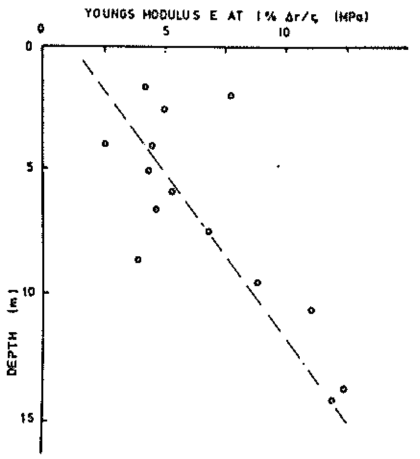
Figure 7 - Typical Pressure Expansion Curve from a Self Boring Pressuremeter

The undrained stress/strain curve is calculated by the method of Palmer (1972). This simple method only requires that the soil deforms radially at constant volume. The stress/strain curve and hence the undrained shear strength is shown on the bottom of Figure 7. The results of the in situ lateral stress, 1% secant modulus and undrained shear strength are shown in Figures 8a, b and c.

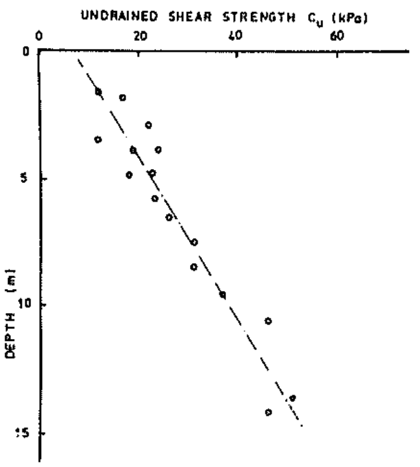
The average modulus and shear strengths for all three methods of in situ test are combined in Figures 9a and b. It would appear that the shear strengths obtained from the various in situ field techniques do not vary too much. However, there is considerable variation in the values and the trend of the moduli obtained from the different techniques. Hence the use of one particular method alone for determining the modulus should be viewed with caution.



(a)

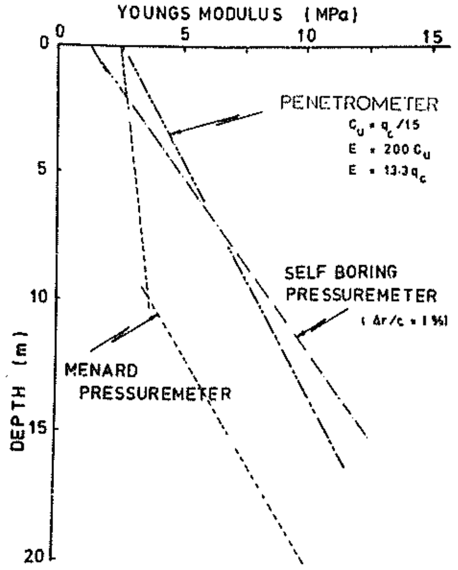


(b)

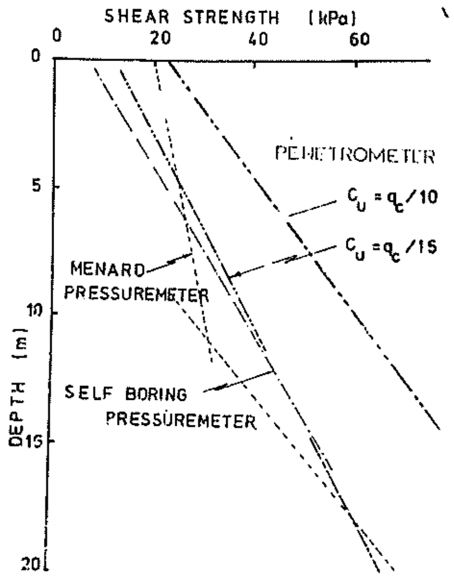


(c)

Figure 8 - (a) Results of the In Situ Lateral Stress (b) 1% Secant Modulus and (c) the Undrained Shear Strength for Tests at Pier 302 from a Self-Boring Pressuremeter



(a)



(b)

Figure 9 - (a) Average Moduli and (b) Shear Strengths obtained from a Penetrometer, Ménard Pressuremeter and a SBP

Part (B) The determination of the in situ properties of the Coode Island silty clays as they relate to the particular pile installation process

The piles at this site were constructed by driving casing through the upper layers of soft materials then into the top of the mudstone., The steel casing had a reinforcing shoe which was 25 mm proud of the casing. This construction enabled the inside diameter of the casing to remain constant for ease of construction of the pile sockets, however, it did mean that, in theory at least, a void could be left around the outside of the casing into which the soft soils would flow as shown in Figure 10. In view of the fact that the 25 mm radial gap was large in terms of the shear stresses which would be imposed on the soil as it moved back, it was considered that this effect should be examined in more detail.

(The 25 mm gap is equivalent to a radial displacement of about 3%. If the soil closes back in an undrained state it would experience a 6% shear strain.)

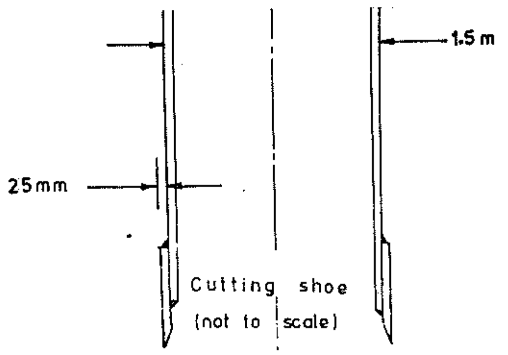


Figure 10 - Details of the Cutting Shoe on the Bottom of the Casing

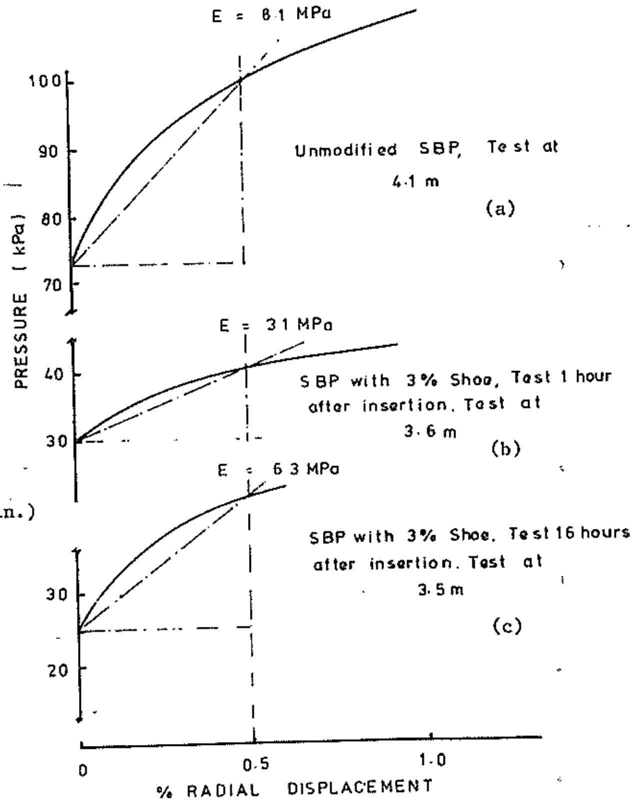


Figure 11 - (b) and (c) Pressure Expansion Curves obtained from a Modified SBP (3% Shoe) Compared with the Unmodified Instrument, (a) for tests at 3.5 - 4 m depth

The self-boring pressuremeter used in the previous study was modified by constructing an instrument with an enlarged cutting shoe which would directly model the geometry of the casing. Two model studies were undertaken. In the first study a 3% enlarged shoe was used and in the second a 6% shoe.

In the first study the instrument was drilled into the ground then either within a few minutes or within 16 hours of insertion the membrane was expanded. This study was to examine the behaviour of the soil on the "front" face of the pile (i.e. the side which is pushed towards the soil). In the second series of tests the instrument was drilled in with the 6% shoe and with the membrane expanded to 3%. After insertion the membrane was allowed to collapse under controlled conditions, thus modelling the behaviour of the "back" face of the pile.

The results of three tests done at about the same depth (around 3.5 - 4 m), but in different holes, are shown in Figure 11a, b and c. The first figure represents the initial in situ undisturbed properties as discussed in Part (A), the second figure represents the pressure expansion curve obtained using the 3% shoe and expanding the instrument within an hour of insertion. The soil has moved back on to the instrument; however, it is much softer. The third figure is the pressure expansion curve obtained about 16hrs after insertion. Clearly the soil has moved back on the instrument and become much stiffer, almost back to the undisturbed level. However, it is interesting to note that the in situ lateral stress has not been restored in that time. The same pattern of behaviour has also occurred in the results from other depths. Unfortunately only a limited series of testing was done, and further it was not possible to run the tests over a longer period, nevertheless, the trends would appear to show that

(a) the soil quickly moves back against the pile;

(b) the stiffness of the soil is restored within a reasonably limited period of time, although the time taken for the soil to "flow" back about the 1.5 m caisson could well be different from that taken with the 75 mm diameter self-boring pressuremeter;

(c) the in situ lateral stress does not seem to be restored, at least over the above time period.

In the second series of tests in which a 6% oversize shoe was used, the membrane was generally collapsed under controlled conditions within a short time after insertion. These results indicate that the soil had moved back against the membrane but was now considerably softer and of low modulus. Figure 12 shows the results obtained at 5.7 m.

In two tests the membrane was left inflated to 3% overnight and then allowed to collapse. From these tests a considerably higher modulus was observed than for the "immediate" tests.

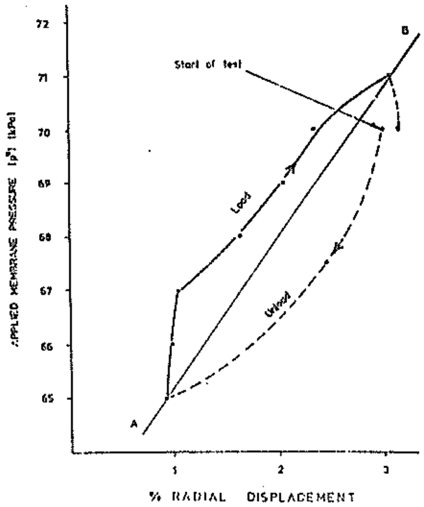


Figure 12 - Pressure Collapse Curves for a 6% to Model the Soil Behaviour on the "Back" Face of the Pile

In conjunction with the above tests, creep or consolidation tests were run in which the pressure in the membrane was kept constant and the strain recorded with time. Two distinct types of results occurred as shown in Figure 13. If the pressure applied was greater than the in situ pressure then the pressure expansion curve followed curve A whereas if the pressure applied was less the in situ pressure expansion curve followed curve B

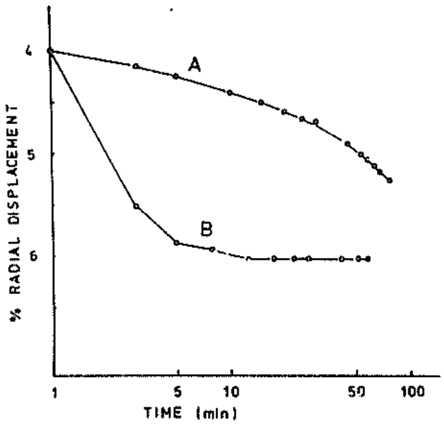


Figure 13 - Consolidation Curves

4 CONCLUSION

The results reported in this paper are all from commercial testing and therefore are perhaps not as precise as those which could be obtained under careful research procedures. Nevertheless the results show that the determination of the modulus of the soil obtained from self-boring pressuremeter tests is promising. Further more for the detailed examination of the behaviour of the actual caisson geometry, a self-boring pressuremeter can be a particularly powerful tool since the boundary conditions are more clearly understood.

5 ACKNOWLEDGMENT

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