Insitu Testing of Soft Rocks with Pressuremeter

by

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Introduction

The conventional insitu testing methods like the vane shear test, standard penetration test etc., employed to investigate the insitu properties of soils cannot be used to investigate soft rocks because the material proves to be too strong to undergo testing by such tools; at the same time, the material is usually too weak to provide core samples. Hence the alternatives resorted to are, plate bearing tests, insitu shear tests, flat jack tests etc., which are costly, time consuming and difficult. Under these circumstances, the Menard pressuremeter designed for direct use in exploratory boreholes assumes greater importance in insitu testing of soft rocks because of its simplicity, low cost and versatility.

Earlier Work Done

Analytical work

Pressuremeter has evoked considerable interest among research as well as practising engineers in the recent times. Theoretical as well as experimental investigations to better understand the pressuremeter test results and their correlations with the results of other routine tests are increasingly attempted. Menard (1957, 1963) obtained expressions for the modulus of elasticity, cohesion and friction in terms of the pressuremeter test data assuming an ideally elastic—perfectly plastic material behaviour. Gibson and Anderson's (1961) analyses were also based on an ideal elastic behaviour. Ladanyi (1963) developed expressions for different characteristics taking into account the volume changes due to deviatoric loading. He extended (1972) the theory of Gibson and Anderson (1961) to enable a direct determination of the undrained stress-strain curves from the pressuremeter curves. Palmer (1972) and, Baguelin Jezequel, Le Mec and Le Mehaute (1972) also have derived the undrained stress strain curves from pressuremeter curves.

Experimental work

Gibson and Anderson (1961) reported the values of shear strength obtained from pressuremeter tests to be higher than those obtained from the

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conventional triaxial tests. Because of the disturbances during sampling etc., the conventional triaxial tests might under estimate the strength values. In addition to their reliability, pressuremeter tests are reported to be economical also compared to that of an equivalent investigation (Eisenstein and Morrison, 1973). Using pressuremeter test results and laboratory test results, Lukas and Bussy (1976) found the preconsolidation pressure to be the same as the creep pressure (obtained from pressuremeter tests) and established good correlations between pressuremeter modulus and the elastic modulus computed from conventional oedometer test data.

The results of standard penetration tests, plate bearing tests and pressuremeter tests conducted on marl are reported by Hobbs (1969) and the results show reasonably good correlations. Komornik, Wiseman and Frydman (1969) also found useful correlations between the results of routine tests and pressuremeter tests on sands, clayey sands, clays, sandstone and marl. Dixon (1970) reported that pressuremeter test results were useful in assessing construction support requirements for a surge chamber excavation in addition to the prediction of other properties. Burgess and Eisenstein (1977) studied three case histories of documented foundation behaviour and based on the back analyses of settlements concluded that pressuremeter provides realistic and meaningful values for deformation modulii.

Menard (1975) has discussed in detail the performance of prssuremeter tests and the analyses and interpretation of test data. He has presented charts for estimating bearing capacity factors for different types of foundations and for different types of soils. He has explained not only settlement computations for layered soils but also the evaluation of differential settlements.

Description of Equipment

Basically the pressuremeter equipment is a combination of two components—the probe, which is lowered into the borehole and kept at the required depth of test, and, the pressure-volumeter, which is situated at the surface (schematically shown in Figure 1). The probe consists essentially of a central measuring cell and two guard cells at both the ends of the measuring cell. The guard cells, which are under gas pressure, are used to reduce end effects on the measuring cell and to ensure an essentially radical expansion. The measuring cell is pressurised with water and kept at a slightly higher pressure than the guard cells to ensure the pressing of the same against the bore hole wall. These cells are protected by a shield of overlapping metallic strips to minimise the possibility of membrane punctures.

A system of two concentric tubes connects the probe to the pressure-volumeter; the inner tube connecting the measuring cell to the manometer and the outer one the guard cells to the regulated gas pressure unit. The regulated gas pressure system serves to exert the required pressure for the water line also. Thus, since the pressure in both the tubes are approximately the same, the possible expansion of the inner tube is nullified resulting in an accurate reading of the volume of water injected in. (The probes are available in EX, AX, BX and NX sizes). A detailed description of the pressuremeter can be found in the article by Menard (1975). Compressed carbondioxide is usually used for supplying the pressure, and for high pressure tests compressed air or nitrogen is used.

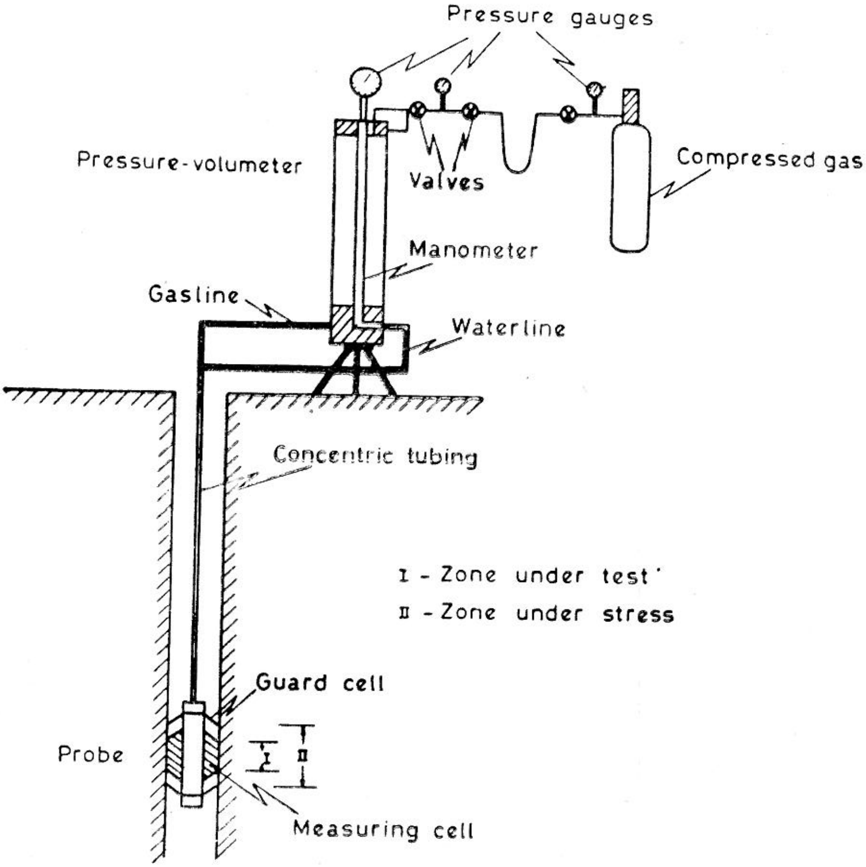

FIGURE 1

Testing Procedure and Tests Performed

Testing procedure

After lowering the probe to the required depth in the bore hole, pressure is applied in increments till failure and the corresponding volume changes noted at intervals of 15, 30 and 60 seconds for each pressure increment. The increments are to be so chosen that enough number of readings (at least about 10) are obtained to plot the curves. If cyclic characteristics are to be obtained, care should be taken to see that the maximum pressure is well below the failure level in all the cycles except the last one.

Tests performed

In the present investigation a total of 15 tests were done in six bore holes of BX size. Bentonite was used as the stabilizing fluid during drilling. The strata tested is weathered to highly weathered granite. Out of the 15 tests, it was not possible to reach failure in 6 tests for want of gas pressure. Excepting one test where unloading was done, for all the tests only loading was done. These tests form part of a detailed site investigation

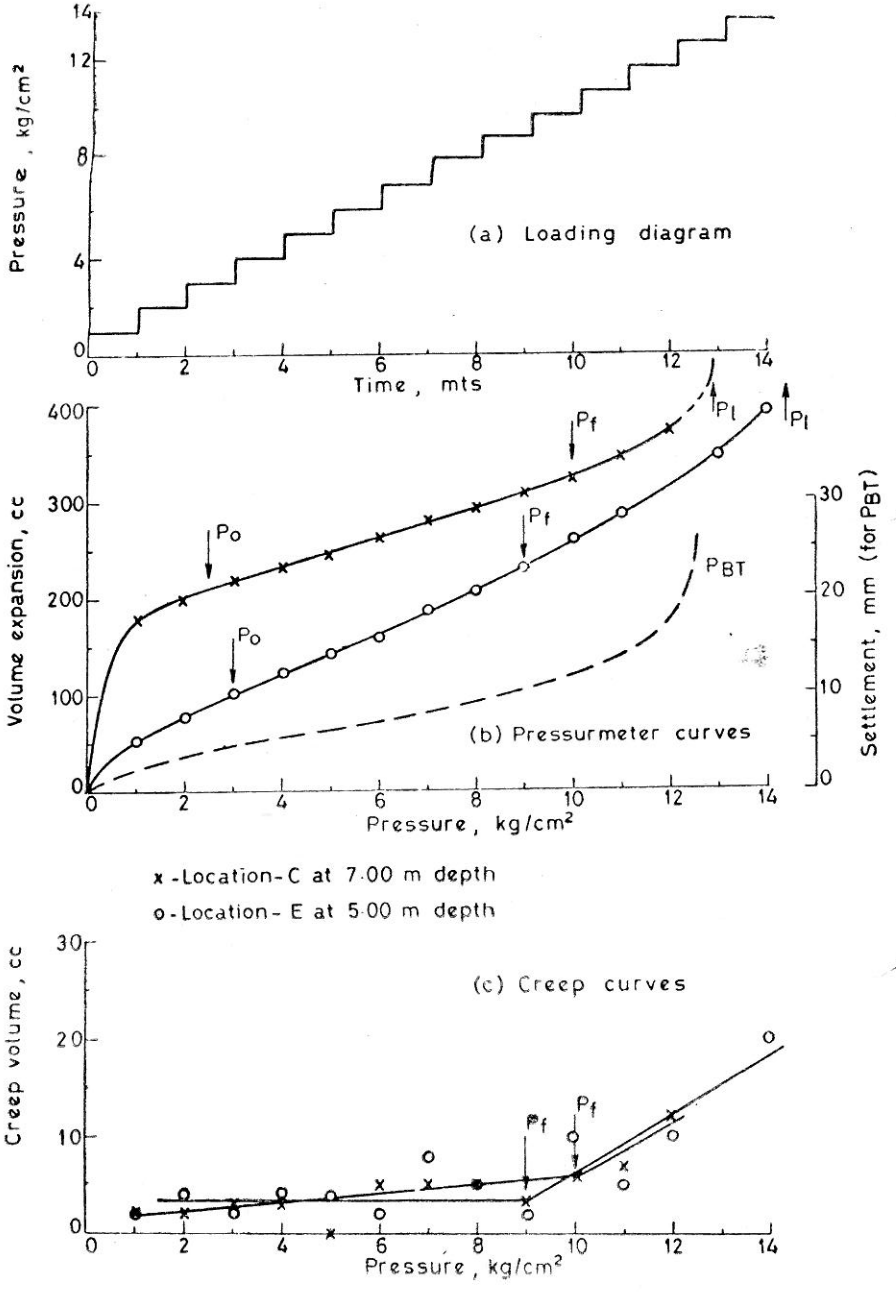

FIGURE 2

carried out which consisted of geophysical surveys, field tests and laboratory tests. The results of pressuremeter tests and only such results of other tests used for corroboration/correlation are presented in this paper.

Interpretation of Test Results

The plot pressure versus volume change (at 60 seconds) is called the *pressuremeter curve* (Figure 2*b*) and that of pressure versus volume change between 30 and 60 seconds called the *creep curve* (Figure 2*c*). The pressuremeter curve normally exhibits three phases; a curved section reflecting the recompression of the material, a near linear portion indicating the elastic range and a nonlinear portion showing the plastic range leading to failure. The end of the initial curved portion (or the beginning of the elastic range) is called the *initial pressure*, P_o; the pressure corresponding to the end of elastic range (or the beginning of plastic range) is called the *creep pressure* P_f; and the failure pressure is called the *limit pressure* P_l. The creep curve shows the tendency of the material to deform with time and it can be also used to find out of P_o, P_f and P_l (Figures 2*c* and 3*b*).

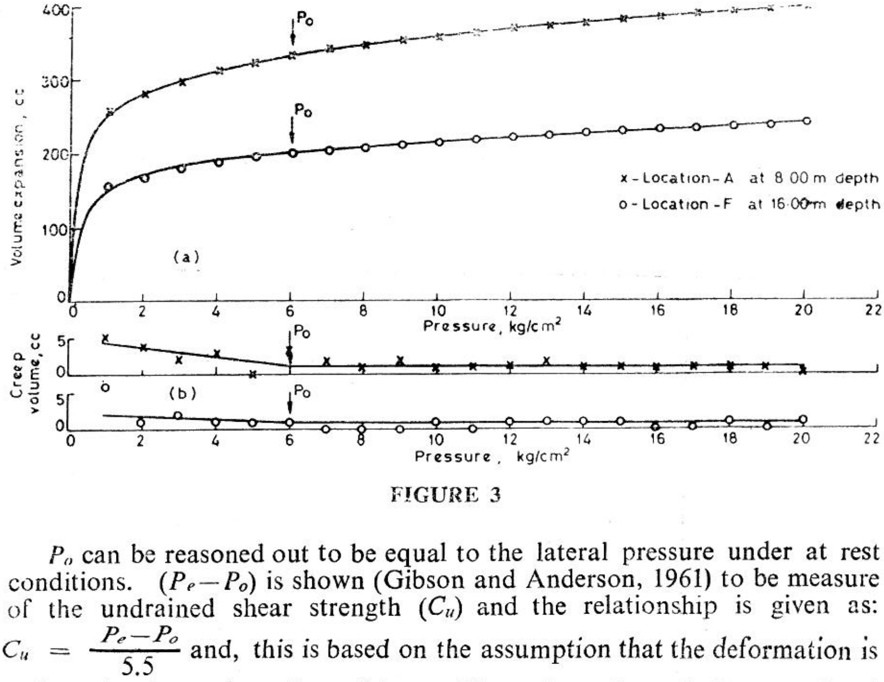

FIGURE 3

P_o can be reasoned out to be equal to the lateral pressure under at rest conditions. ($P_e - P_o$) is shown (Gibson and Anderson, 1961) to be measure of the undrained shear strength (C_u) and the relationship is given as: $C_u = \dfrac{P_e - P_o}{5.5}$ and, this is based on the assumption that the deformation is under 'no volume change' conditions. The values P_o and P_e are related to the ultimate bearing capacity as: $UBC = P_o + (P_e - P_o)\left(1 + 0.4\,\dfrac{B}{L}\right)$ where B and L are the breadth and length of the foundation, (Thorley, Calhoon, Zeman and Watt, 1969).

The pressuremeter modulus, E could be derived (Menard, 1975) from the slope of the linear phase (i.e. between P_o and P_f) as under:

$E = 2\,(1 + \mu)\;V\,\dfrac{\triangle P}{\triangle V}$ where V is the volume of the cavity at the instant when the slope of the linear phase $\dfrac{\triangle P}{\triangle V}$ is measured and is equal to the sum of the initial volume of the measuring cell and the mean additional volume injected; and μ is the Poisson's ratio for the strata tested. This expression is derived considering the radial expansion of a cylindrical cavity in an elastic medium.

If it is not possible to reach failure level during a test, the limit pressure can be found using extrapolation provided the creep pressure has been exceeded during the test. The last few readings of pressure and the corresponding volume change (expressed as a ratio of the volume injected to the original volume) are plotted in a log-log scale and these can be seen to form a straight line. The point at which the extrapolation of the straight line meets a volume ratio of unity corresponds to the limit pressure. The extrapolation technique has stemmed out of the observation that in most of the cases, failure was reached when the volume of the cavity was doubled under pressure.

Discussion of Results

Initial and limit pressures

It can be seen from Table 1 that P_o increases with depth. It is in order since P_o represents the lateral pressure under at rest conditions which normally increases with depth. Similarly, P_l which is a measure of the strength of the strata also increases with depth. As stated earlier it was not possible to obtain P_l in six cases for want of higher gas pressures:

TABLE 1

Sl. No.	*Location*	*Depth of test (m)*	P_o *(kg/cm²)*	*Pf (kg/cm²)*	P_e *(kg/cm²)*
1	A	8.00	6.00	—	—
2	A	5.00	1.00	2.00	3.00
3	B	5 50	3.20	—	—
4	C	7.00	2.50	10.00	12.90
5	C	5.00	2.00	7.00	15.80
6	C	4.00	2.00	6.00	9.00
7	D	12.00	5.00	—	—
8	D	8.70	5.00	—	—
9	D	5.80	4.00	—	—
10	D	4.40	1.00	3.00	4.00
11	E	5.00	3.00	9.00	14.40
12	E	3.70	2.00	8.00	10.00
13	F	16.00	6.00	—	—
14	F	9.00	6.00	10.00	11.00
15	F	4.00	—	—	1.50

prediction of the same using the extrapolation technique is also not possible since P_f was not exceeded in the tests.

Figures 2b and 2c show the typical pressuremeter and creep curves obtained wherein failure was reached whereas Figures 3 and 4 show the same where failure was not reached. It is not possible to define P_f in Figures 3 since as stated earlier even P_f was not reached during testing.

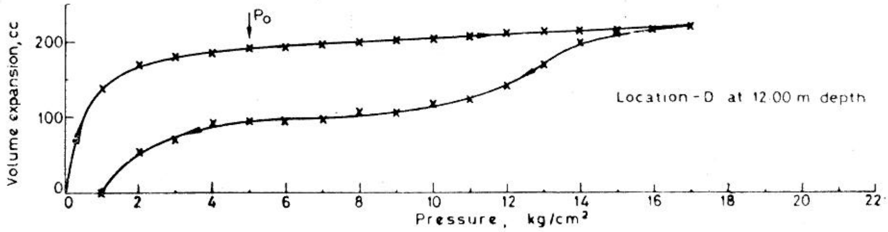

FIGURE 4

Shape of the curves

The shape of the pressuremeter as well as creep curves are representative of the nature of the materials tested. For example, in Figure 2b, P_o to P_f is linear followed by a curve of progressively increasing slope showing the onset of plastic deformation leading to failure after the elastic range is exceeded which is typical of a compact—intact material. But in Figure 3a, the linear portion over some range after P_o is followed by another linear portion of flatter slope. This can be due to the closing of micro-fissures leading to a compact strata offering more resistance to deformation which results in a flatter curve. The loading characteristics obtained from a plate bearing test conducted on a similar material presented in Figure 2b (using dashed lines) shows the similarity between the pressuremeter curves and PBT curves; and it is reasonable since both are, in principle, insitu load tests.

Elastic modulus

Table 2 reports the values of elastic modulus, ultimate bearing capacity and compressive strength computed from pressuremeter test data together with those obtained from plate bearing tests and unconfined compression tests on samples taken from nearby locations and at comparable depths. For computing E, the value of Poisson's ratio has been taken as 0.33 based on seismic refraction studies (Pandian and Raju, 1977). The values of E obtained from PBTs are higher than those derived from pressuremeter tests. This is because of the higher resistance offered by the strata to deformation perpendicular to the horizontal plane (as in the case of plate bearing tests) than that parallel to the horizontal plane (as in the case of pressuremeter tests).

For the only test where rebound was done (location D at 12.00 m depth) the rebound modulus (1005 kg/cm²) is seen to be higher than the compression modulus (712 kg/cm²). The ratio of the robound modulus to compression modulus provides a qualitative measure of the degree of fracturing of the rock. It is said (Dixon, 1970) that the ratio varied from 1.50 to 10.00 and confirmed the visual interpretation of zones of moderately to heavily fractured rocks. The ratio obtained for the test done (1.4132) indicates negligible fracturing and confirms the visual observation made.

TABLE 2

Sl. No.	*Location*	*Depth of test (m)*	*E(kg/cm²)*			*Compressive strength (kg/cm²)*		*UBC (kg/cm²)*	
			I	*II*	*III*	*I*	*III*	*I*	*II*
1	A	8.00	585	1825	—	—	—	—	13.80
2	A	5.00	16	—	—	0.39	—	3.80	—
3	B	5.50	585	735	465	—	5.07	—	11.40
4	C	7.00	139	319	400	2.04	3.44	17.00	—
5	C	5.00	122	—	—	2.71	—	21.30	—
6	C	4.00	78	—	—	1.38	—	11.80	10.00
7	D	12.00	712 (1005)*	—	—	—	—	—	—
8	D	8.70	726	—	—	—	—	—	—
9	D	5.80	270	319	—	—	—	—	10.00
10	D	4.40	45	—	—	0.59	—	5.20	—
11	E	5.00	84	383	430	2.24	10.26	19.00	—
12	E	3.70	69	—	—	1.57	—	13.20	11.60
13	F	16.00	822	—	—	—	—	—	—
14	F	9.00	159	555	—	0.98	—	13.00	10.00
15	F	4.00	13	—	—	—	—	—	—

(I: from pressuremeter tests; II: from plate bearing tests; III: from unconfined compression tests; *Rebound Modulus).

Compressive strength

Unconfined compression tests were done on block sample obtained from trial and test pits. In some places, it was not possible to take the block samples with hand tools because the highly weathered rock was friable: hence samples were taken wherever it was possible (or in otherwords wherever the formation had enough strength to withstand the sampling operations) and tested. This explains the consistently higher compressive strength values exhibited by *ucc* tests (Table 2) compared to those from pressuremeter tests. The same explanation holds good for the higher values of E also obtained from *ucc* tests.

Ultimate bearing capacity

The ultimate bearing capacity values as computed from pressuremeter tests and as obtained from plate bearing tests are reported in Table 2. Even though both are numerically comparable, it can be seen that the ones obtained from PBTs are less in all the cases, though slightly. This may seem to contradict the discussions regarding E; but if it is remembered that for computing E values the linear elastic portion of the curves are used and for the UBC it is the failure level, the reasons are discernable. To amplify, at failure the lines (or planes) of rupture have to pass through with changing directions in plate bearing as well as pressuremeter tests resulting in a strength value which necessarily has to lie in between those obtained perpendicular and parallel to the horizontal planes. (An accurate assessment could be made only with the knowledge of the directional variation of the strength of the strata and the geometry of the rupture lines).

Conclusions

Pressuremeter tests could be reliably used to predict the engineering properties of soils and soft rocks. However, its results need be studied and correlated with the results of other well known tests in detail for better appreciation.

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