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Modulus and Shear Strength Values Measured in the Pressuremeter Test Compared with Results of Other In-situ Tests

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SUMMARY The authors have performed pressuremeter testing in a range of soil and rock types. Tests were performed either with a self-boring pressuremeter (the so-called Camkometer) or with a high-pressure insertion-type pressuremeter. At a number of sites, the results of other in situ tests were available for comparison. These tests include Standard Penetration Tests, electric friction-cone penetration tests and dynamic cone penetration tests. Values of shear modulus and shear strength (in cohesive soils) have been correlated with SPT and dynamic cone test blowcounts and with static cone resistance (q_c) values. The correlations obtained are compared with various published correlations.

1 INTRODUCTION

Penetration tests of various types are a simple and widely used method of obtaining design information in site investigations. However, these tests do not provide either strength or deformation parameters directly; in order to obtain such parameters, empirical correlation factors must be used. In general, although these correlation factors have been derived on the basis of extensive experience, they must be used with considerable caution when applied in areas in which they have not previously been proven.

The pressuremeter test appears to be an ideal means of measuring both strength and deformation parameters directly. This has been especially true since the introduction of self-boring pressuremeters and improved methods of data interpretation. Thus by performing pressuremeter tests in conjunction with various penetration tests, it should be possible to determine the relationships between the parameters obtained.

The authors have undertaken pressuremeter testing at a number of sites. Tests have been performed using either a self-boring pressuremeter (the so-called Camkometer) in sands and clays, or a high-pressure pressuremeter (Jewell and Fahey, 1984) in rock. This paper compares the results of these tests with the results of penetration tests at the same locations.

2 PRESSUREMETER TESTING

The two types of pressuremeter used in the study are adequately described elsewhere; refer Wroth and Hughes (1973) and Jewell and Fahey (1984) for descriptions of the Camkometer and high-pressure pressuremeter, respectively. In general, the Camkometer was used where possible - ie. until the strength or hardness of the material made self-boring of the instrument impossible. In such materials, the high-pressure pressuremeter was used in N-sized diamond-cored bore holes. No differentiation is made between parameters derived from the two pressuremeters in the correlations which follow.

The pressuremeter test provides a measurement of the shear strength (c_u) in cohesive soils and of the shear modulus (G) in both cohesive and cohesionless soils.

2.1 Shear Modulus

When the pressuremeter test data are plotted as pressure ψ versus "cavity strain" ε (defined as the change in radius divided by the current radius), the shear modulus can be obtained from the gradient of any purely elastic phase of the test using the relationship.

$$G = \tfrac{1}{2}\, d\psi/d\varepsilon$$

Such "elastic" phases of the test occur during initial loading until the material adjacent to the pressuremeter reaches the plastic state, and also if an unloading-reloading loop is carried out at any stage of the test (refer to Figure 1). In an ideal test in an ideal elastic-plastic material, values of G derived from initial loading (G_I) or from an unloading-reloading loop (G_{UR}) would be identical. However, in practice, this is not the case, with G_I being generally significantly less than G_{UR}; ratios of G_{UR} to G_I of between 1 and 7 have been observed.

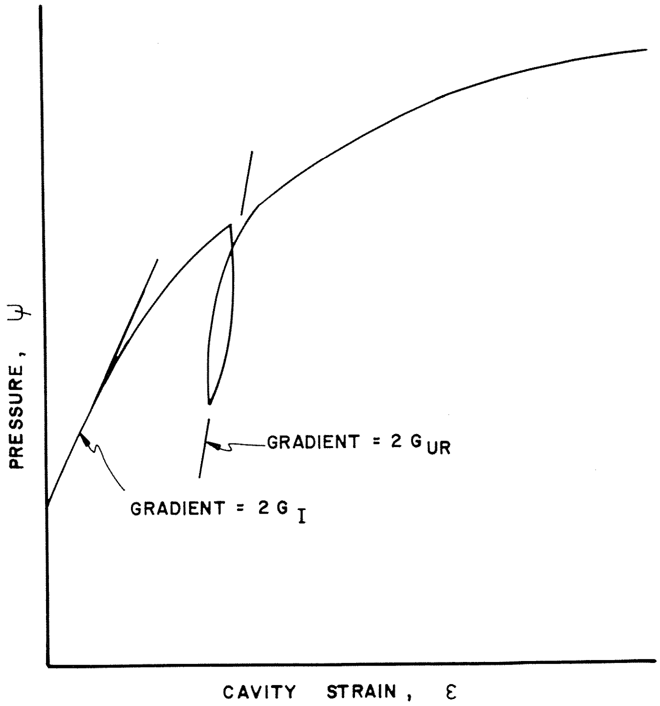

Figure 1 Typical pressuremeter test curve

We have decided in this paper to deal only with values of G_{UR}, mainly because:

- for most soils, the value of G_{UR} appears from limited evidence to be more appropriate than G_I in predicting ground deformations especially in sands and stiff clays.
- the value of G_I depends very much on the disturbance caused during pressuremeter insertion whereas G_{UR} is practically unaffected by disturbance (Fahey, 1980).
- the initial part of the pressure-expansion curve is frequently curved rather than straight, so that determining G_I is to some extent subjective.
- an unload-reload loop can frequently exhibit hysteresis (as illustrated in Figure 1), but the gradient of the loop can be determined unambiguously by a least-squares regression analysis.

Values of elastic modulus E can be determined from the shear modulus G by choosing an appropriate value of Poission's ratio μ:

$$E = 2(1+\mu)G$$

Since Poission's ratio varies between 0.5 for undrained deformation to about 0.25 for drained deformation, then

$$E' \geq 2.5G \text{ and } E_u \leq 3G$$

2.2 Undrained Shear Strength (c_u)

Shear strengths reported in this paper were derived from the pressuremeter tests using a method based on that proposed by Gibson and Anderson (1961). Basic assumptions in this method are that the soil can be represented by a linear-elastic, perfectly-plastic soil model, with deformation occurring under zero volume change conditions. When the correct reference state for strain is chosen (ie. the point at which the applied pressure is equal to the in situ horizontal stress), the analysis shows that

$$\psi = \psi_\ell + c_u \, \ell n \, (\Delta V/V)$$

where ψ_ℓ is the limit pressure and ΔV and V are the change in volume, and the current volume, respectively, of the pressuremeter. Thus a plot of pressure ψ against $\ell n \, (\Delta V/V)$ would have a gradient equal to c_u.

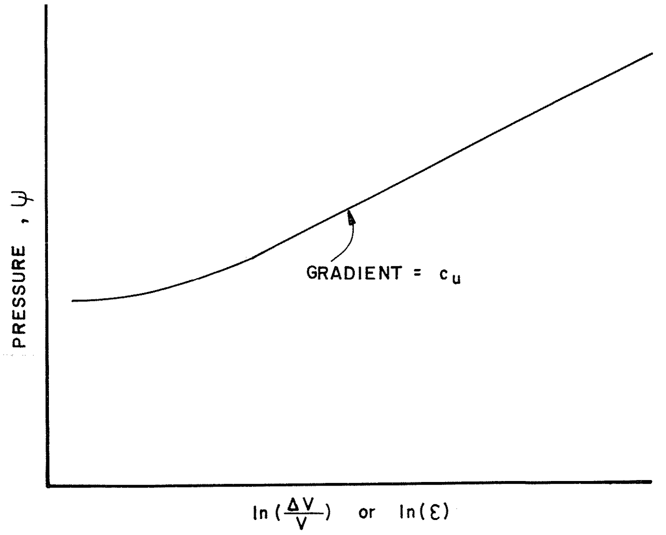

Figure 2 Obtaining c_u from pressuremeter test

Since $\Delta V/V \approx 2\varepsilon$ for small strains, the shear strength can also be obtained from a plot of ψ against $\ell n \, \varepsilon$ (refer Figure 2).

3 PENETRATION TESTS

The penetration tests discussed in this paper were of three types:

- electric friction-cone penetration tests with measurement of cone resistance q_c and sleeve friction f_s (AS1289. F5.1-1977)
- Standard Penetration Tests; results quoted as N, the number of blows required to drive the sampler from 150 to 450 mm penetration (AS1289. F3.1-1977)
- continuous dynamic cone penetration tests, ie. driving a solid 50 mm diameter 60° cone with an S.P.T. hammer; results quoted as NC, the number of blows required for each 300 mm of penetration.

None of these tests were performed by or in the presence of the authors, so the results are taken on their face value. In some cases the dynamic cone tests were performed continuously from the surface, whereas in others they were performed out of cased bore holes, with the casing being advanced periodically to prevent build-up of skin-friction resistance on the rods.

4 TEST SITES

The results presented were obtained from seven sites, five of which are in the Perth Metropolitan area. For contractual reasons, we are not at liberty to identify the sites. However, the following is the most relevant information about each site.

4.1 Site A

Soil stratigraphy consisted of five to seven metres of loose sand (hydraulic fill) overlying very stiff clays (weathered granite). Camkometer tests were performed only in the weathered granite. Results of electric friction-cone tests were available for comparison. The site is located in the south of Western Australia.

4.2 Site BA

The site is located on the flood plain of the Swan River east of Perth. Results quoted relate to a layer of soft to firm silty clay which extends from underneath a dessicated crust to depths of about 12-14 m. Camkometer tests and electric friction-cone tests were compared for this site.

4.3 Site BU

The site is located in the Guildford Formation to the east of Perth. Stratigraphy consists of interbedded layers of sands and clays. Camkometer tests, Standard Penetration Tests, electric friction-cone tests and dynamic cone tests were performed.

4.4 Site D

The site is located to the north of Perth. Camkometer tests, Standard Penetration Tests, electric friction-cone tests and dynamic cone tests were performed. Tests reported were performed at depths between about 3 m and 10 m in loose to dense sand (Safety Bay Sand).

4.5 Site E

The site is located east of Perth in the Bassendean Sand. Camkometer tests, Standard Penetration Tests, electric friction-cone penetration tests and dynamic cone tests were performed.

4.6 Site M

The site is located about 80 km south of Perth. Stratigraphy consists of layers of sand and clay overlying siltstone (South Perth Shale). Camkometer tests were performed in the upper strata, with high-pressure pressuremeter tests in the siltstone. Both SPTs and continuous dynamic cone tests were available for comparison. Tests were undertaken in two bore holes sufficiently far apart to warrant treating them as two separate sites; these are identified as Sites M3 and M5.

4.7 Site S

The site is located to the south of Perth. Testing was limited to high-pressure pressuremeter tests in siltstone; thus data from this site are used only in the correlation of pressuremeter-derived values of G with c_u.

5 RESULTS

The test results from the various sites are presented as plots of one of the pressuremeter-derived parameters (G or c_u) versus the appropriate penetration test parameter (SPT blowcount, N; dynamic cone blowcount, NC; or electric friction-cone resistance, q_c). Values of G and c_u are also compared for the various sites. No differentiation is made between cohesive and cohesionless soils in the plots of G versus any of the penetration test results. Because of the limited amount of data and the scatter in that data, no attempt has been made to analyse it by statistical methods (ie. to obtain best fit lines, correlation coefficients etc).

5.1 Shear Modulus (G) and Shear Strength (c_u)

Values of shear modulus (G) are plotted against shear strength (c_u) in Figures 3(a) and 3(b). Soil types range from soft clay (Site BA) to siltstone (Sites M3, M5 and S). These figures indicate that there is a reasonable correlation between G and c_u, with a sensible mean and lower bound respectively of $G = 100\ c_u$ and $G = 150\ c_u$. These values are in keeping with reported ratios for stiff clays especially when the value of modulus used has been obtained from back-analysis of the performance of structures (d'Appolonia et al, 1971, Wroth et al, 1979, Butler, 1974) or from carefully-performed laboratory tests on samples trimmed from undisturbed block samples (Ward et al, 1959) or from pressuremeter or plate bearing tests (Marsland 1971, Windle, 1976).

5.2 Shear Modulus (G) and Cone Resistance (q_c)

Values of shear modulus (G) are plotted against cone resistance (q_c) in Figure 4. Though a large amount of scatter is evident, a number of significant observations can be made:

- for Sites A, BA and BU, which consist of tests only in cohesive soils, most of the G values are greater than 10 q_c.
- for Sites D and E, (all tests in sand) the ratio of G to q_c ranges from about 3 to greater than 10, with one value as low as 2.

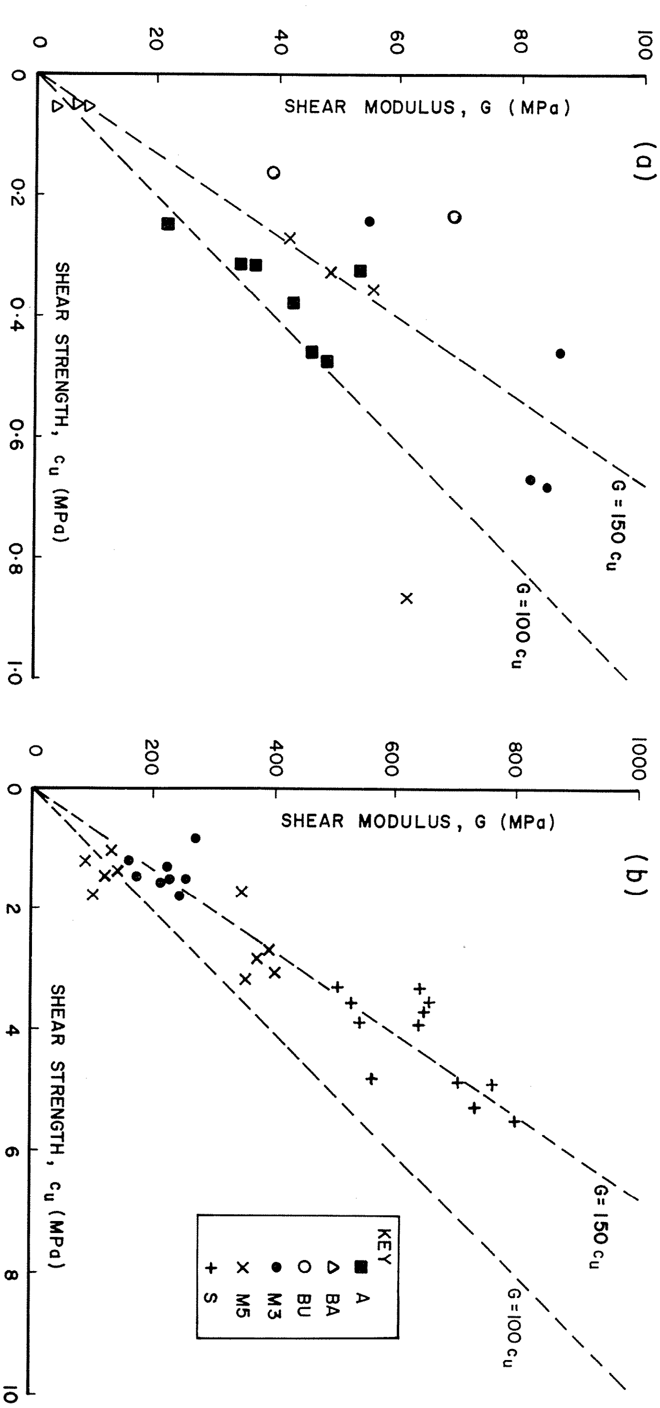

Figure 3 Plots of G versus c_u

- the relationship $G = 1.15q_c$ (ie. $E = 3q_c$), which represents the mean relationship for sand suggested by Schmertmann et al (1978), is shown on the figure; this indicates that Schmertmann's relationship would severly underpredict G or E from q_c values (and hence would overpredict settlements) for these sites which is in line with general experience for sands of the Perth area.

A comparison between the results of plate bearing tests and cone tests in clays and sandy clays at 97 sites was reported by Trofimenkov (1979). He shows a good correlation between E and q_c which he expressed as $E = 7.8q_c + 20\text{kgfcm}^{-2}$. This is incorporated into USSR building codes as $E = 7q_c$ ($G \approx 2.3q_c$) which, while higher than the Schmertmann relationship for sand, is still much lower than the values for clay (Sites A, BA and BU) shown on Figure 4.

5.3 Shear Strength (c_u) and Cone Resistance (q_c)

Values of shear strength (c_u) are plotted against

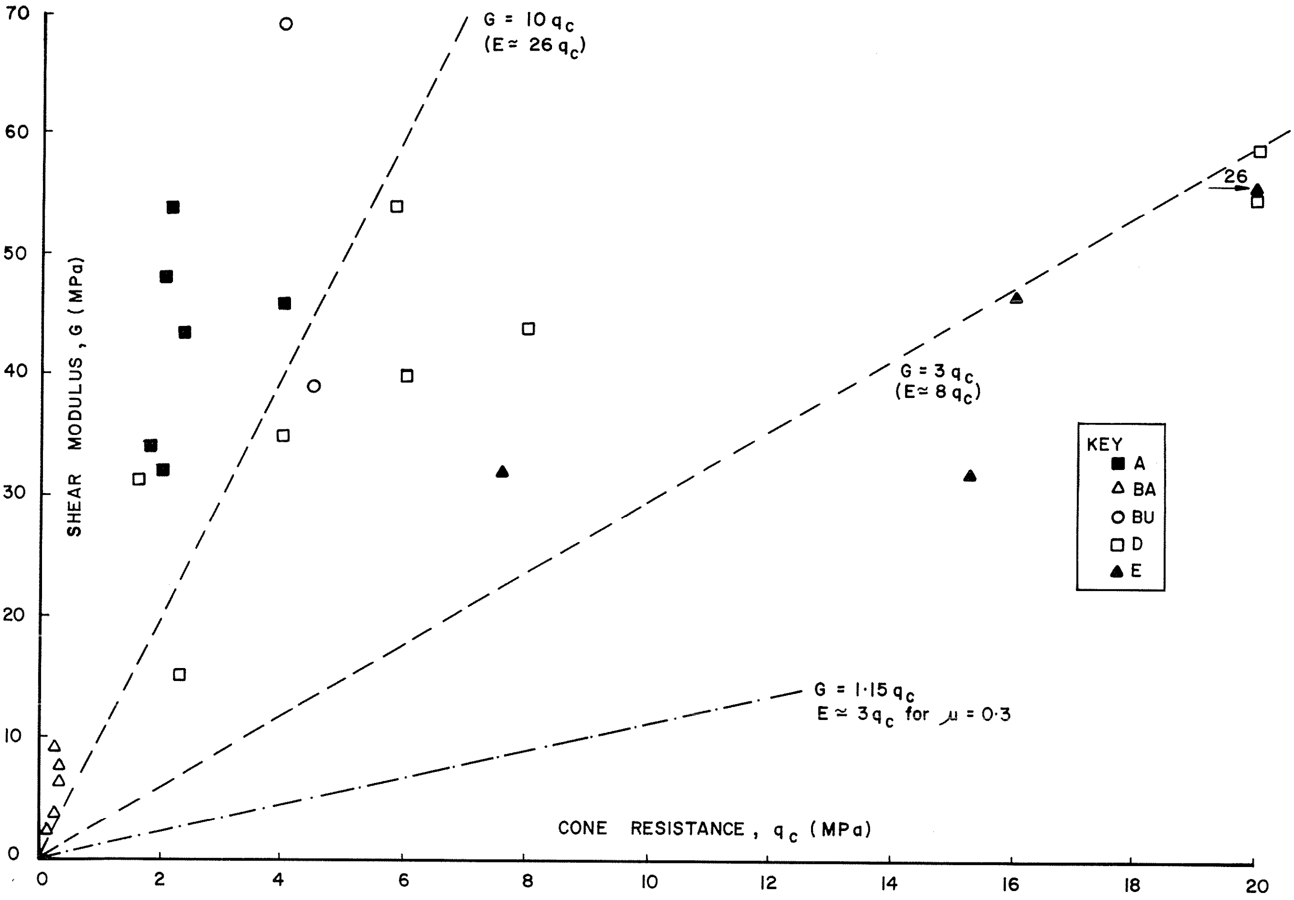

Figure 4 Plot of G versus q_c

cone resistance (q_c) on Figure 5. This figure indicates that for Sites A and BA, the ratio of q_c to c_u is about 6, while for Site BU it is much higher (20 to 30).

The relationship between q_c and c_u is usually expressed as a bearing capacity factor N_c ie. $q_c = N_c\ c_u$. Values of N_c between 5 and 70 (Tavenas and Leroueil, 1979) and between about 10 and 30 (Marsland, 1979) have been reported. The value of N_c of 6 shown in Figure 5 is thus at the lower end of the range of reported values. Note however that in this case the shear strength measurements were obtained with the pressuremeter (Camkometer) which gives values higher than conventional laboratory methods or other in situ methods.

5.4 Shear Strength (c_u) and SPT Blowcount (N)

Values of shear strength (c_u) are plotted against S.P.T. blowcount (N) in Figure 6. A large amount of scatter is evident in the data. However, it can be seen that the relationship proposed by Stroud (1974), ie. c_u = 5N(kPa), is a sensible lower bound to the data. Thus there may be benefit in performing Standard Penetration Tests even in soft/weak rock to provide a lower-bound estimate of the shear strength, though the N-values obtained may be very much higher than the value of 60 at which the Australian Standard recommends the test should be terminated.

The scatter of the data shown in Figure 6 is typical of the scatter observed in similar plots reported in the literature; for example Meigh (1979) shows that the ratio of c_u to N for various soft rocks (mainly mudstones and chalk) varies from less than Stroud's value of 5 to greater than 35.

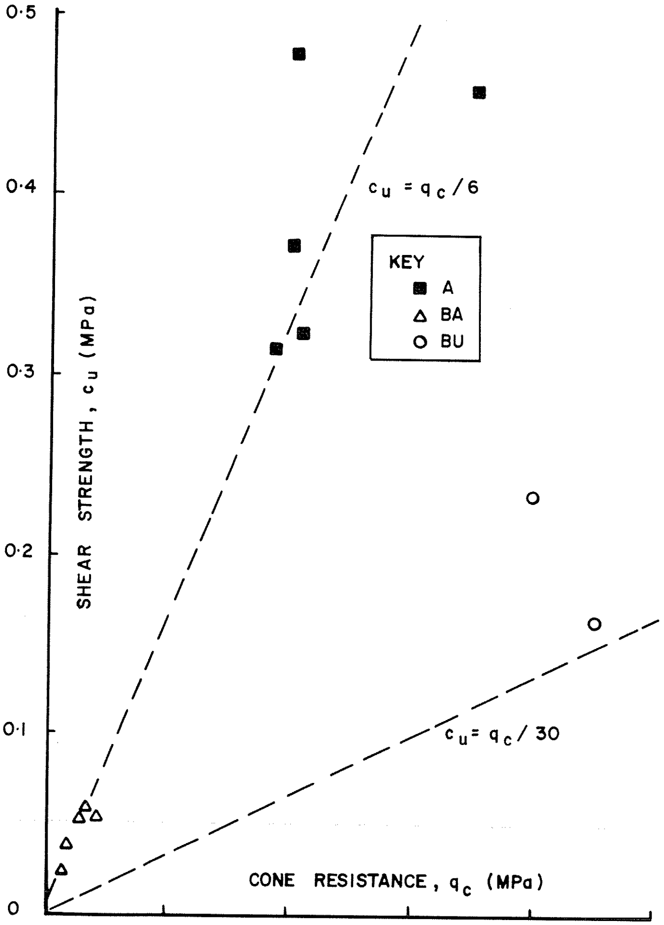

Figure 5 Plot of c_u versus q_c

5.5 Shear Modulus (G) and SPT Blowcount (N)

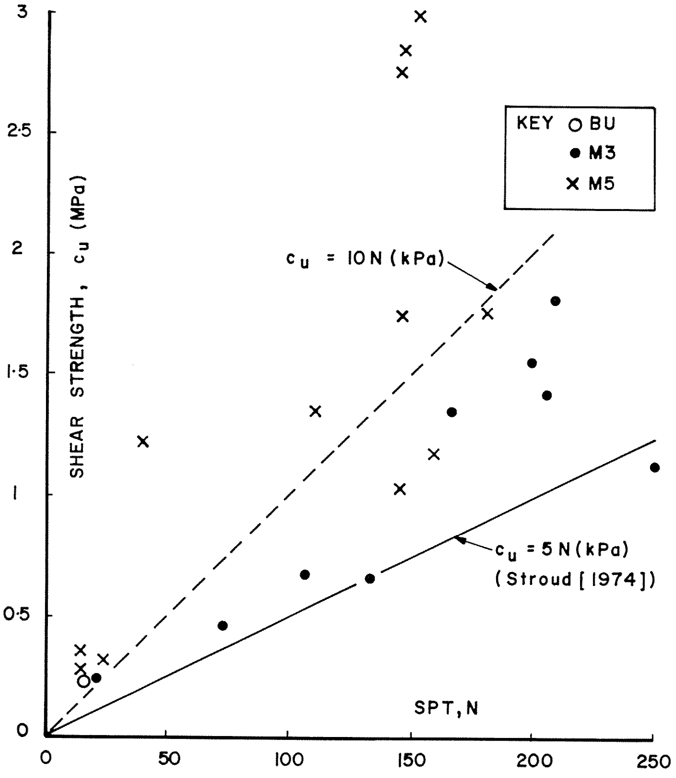

Figure 6 Plot of c_u versus N

Two plots of shear modulus (G) versus SPT blowcount (N) are shown in Figure 7. Though there is considerable scatter, a best fit could be taken as G = 2N (MPa) for N values less than 50, with G = N(MPa) being a lower bound (Figure 7(a)). All of the data in Figure 7(b) comes from two locations about 100 m apart at Site M. This figure suggests that for Site M3, the relationship G = N(MPa) is appropriate, while for Site M5, G = 2N(MPa) gives a better estimate.

Parry (1971) proposed determining settlements in sand directly from SPT N-values, thereby eliminating the intermediate step of estimating modulus values. Based on elastic theory, his relationship between N and settlement is equivalent to assuming a ratio of E to N of about 4 to 5 (MPa), ie. a ratio of G to N of about 1.5 to 1.9 (MPa). When compared with the data for the "sand" sites in Figure 7(a), this relationship appears reasonable, though conservative in many cases.

5.6 Shear Modulus (G) and Dynamic Cone Test Blowcount (NC)

Values of shear modulus (G) are plotted against dynamic cone test blowcounts (NC) in Figure 8. Most of the values lie within the limits G = 3NC(MPa) and G = 6NC(MPa), with two values as low as G ≃ 2NC(MPa). Thus the ratio of G to NC appears to be significantly greater than G to SPT blowcount N shown in Figure 7(a), which implies that N-values are higher than NC values at the same sites. In fact, at one site (Site D) where SPT and dynamic cone results in loose to dense sands were compared in detail, it was found that there was very poor correlation between the results of the two tests (correlation coefficient ≃ 0.4), with the SPT values being significantly higher than the dynamic cone test results.

The poor correlation between SPT and dynamic cone test results at this site illustrates the difficulty of correlating either of these tests with basic engineering soil parameters such as shear modulus. While a theoretical dependence of blowcount on shear modulus is tenuous, there is little doubt that the SPT and dynamic cone blowcounts should be equal, or at least closely correlated.

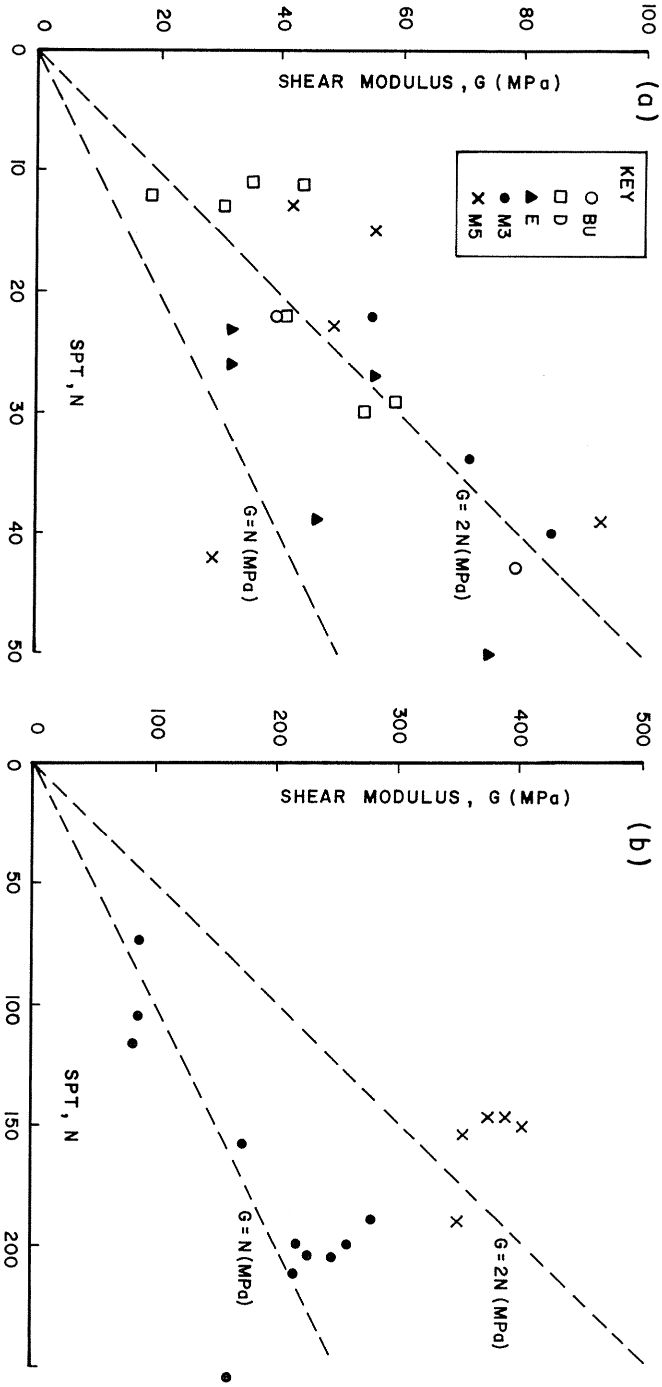

Figure 7 Plots of G versus N

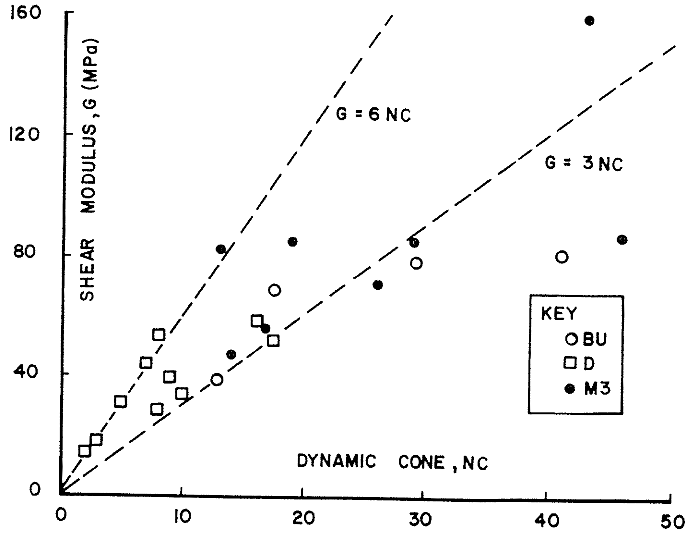

Figure 8 Plot of G versus NC

6 CONCLUSIONS

The values of shear modulus G and shear strength c_u obtained from pressuremeter tests have been compared with results of penetration tests for a number of sites. The following conclusions can be drawn from this work:

- the ratio of the pressuremeter-derived values of G to c_u has a mean value of about 150 with a sensible lower bound of about 100; this appears reasonable when compared with reported G/c_u ratios for other soils.
- the ratio of shear modulus G to cone resistance q_c varied from less than 3 to greater than 10; the ratios for sand were found to be significantly greater than the value of about 1.15 (ie.E/$q_c \simeq 3$) suggested by Schmertmann et al (1978), while the values for clay were greater than the value of 2.3 (ie.E/$q_c = 7$) recommended by Trofimenkov (1979).
- the ratio of cone resistance q_c to shear strength c_u varied from about 6 to 30 with most of the values at the lower end of this range.
- the ratio of shear strength c_u to SPT N-value shows considerable scatter with a mean value of about 10; the value of 5 suggested by Stroud (1974) appears to be a reasonable lower bound to the data presented.
- the ratio of shear modulus G to SPT N-value ranges from less than 1 to more than 3; the G/N ratio of 1.5 to 1.9 for sand derived from the settlement prediction method of Parry (1971) appears to be reasonable in light of the data presented.
- the ratio of shear modulus G to dynamic cone blowcount NC varies from less than 3 to about 6; this ratio is higher than the equivalent G to N (S.P.T.) ratio.

If it is assumed that the value of shear modulus determined from a pressuremeter test is a "fundamental" soil parameter which largely governs the amount of settlement of a structure at working load, then the data presented in this paper suggests that none of the three penetration tests discussed can be expected to form the basis of an accurate method of settlement prediction, since each of the penetration resistance values N, NC and q_c showed only poor correlation with G. However, more work is required to determine if the pressuremeter-derived values of G can lead to accurate settlement predictions in all soil types. It may well be that the values of G will require adjustment by a reduction factor for use in settlement prediction (as is currently advocated for vane shear strengths or Camkometer shear strengths in slope stability calculations in soft clay). These, and other questions can only be resolved by careful monitoring of the behaviour of actual structures in areas in which thorough site investigations using standard and non-standard techniques have been performed.

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