The Response of Single Piles in Clay to Axial Load

La Rèponse des Pieux Isolès, Situès dans l'Argile, à une Charge Axiale

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SYNOPSIS The results of 23 axial, vertical load tests on 12 mm diameter model piles of different lengths installed in a large, homogeneous remoulded London clay bed are presented. Data, measured during installation by jacking at a constant rate, is provided on total loads, pile base loads and radial stresses at the toe.

The main contributions of the paper are precise vertical displacement measurements on the piles at working load which show completely linear behaviour up to approximately half ultimate load. Elastic analyses have been used to interpret the vertical stiffnesses all of which were consistent with a single value of the soil shear modulus (G). Empirical relationships between the G value and the undrained cohesion (c_u) of the London clay are discussed in some detail.

INTRODUCTION

It has been known for many years that as piles are incorporated into larger and larger groups so the displacement of the groups under working load rather than their ultimate load capacity becomes the governing design criterion. Nevertheless piling research has mainly concentrated on the ultimate load problem and predictions of working load displacements of pile groups are still largely empirical. A number of papers have been published recently (Davis and Poulos 1968; Butterfield & Banerjee 1970, 1971; Poulos 1972; Banerjee 1975, 1976) presenting rigorous linear elastic analyses of pile-soil systems with the implication that they may be relevant to the prediction of the working load stiffness of real pile groups, the load sharing between piles in a group and possibly even the stress distributions across the pile soil interfaces (Ottaviani 1975). The results presented in this paper are the first to be published from a series of very precise tests on model piles (diameter D = 12 mm, and length to diameter ratios L/D = 20, 30, 40) jacked at constant velocity into a large bed of remoulded London clay with the express intention of measuring the conventional ultimate load parameters and interface stresses during installation and subsequently their working load stiffness both singly and in groups (Ghosh 1976). The general philosophy behind the test programme was to determine a pseudo-elastic shear modulus (G) for the soil, which, for an assumed Poisson's ratio = 0.5, produced the best fit of a linear elastic analysis to the measured vertical stiffness over a range of single pile geometries. Pile groups were then to be investigated similarly and the work extended to include quite general inclined loads and applied moments (Abdrabbo 1976), with installation data and working load stiffnesses measured in each case. All 6 independent components of the pile group planar stiffness matrix could then be interpreted via elastic analyses and the best fit G value found. By this means the validity, at model scale, of determining a soil modulus from a single pile test and its use to predict the stiffness matrix etc. for a general pile group could be tested. If such an inexpensive model scale investigation produces encouraging results then relatively few confirmatory field scale tests may suffice to establish their practical utility also.

This paper presents all the data obtained from some 23 single piles tested under vertical axial loads.

EXPERIMENTAL DETAILS

The steel piles (D = 12 mm) were installed at a constant rate of penetration (6.5 mm/min) to their full depth. They each had an axial load cell at their head and the maximum recorded value (Q_u) during installation was taken as the ultimate load capacity of the pile. The working load (Q) was then defined arbitrarily as $0.5 \times Q_u$. A further load cell at the pile toe measured a nominal radial interface total stress (σ_r) by detecting the mean hoop stress in the pile wall at approximately 12 mm above the pile tip. A base load was also measured at this level which, when corrected for the average interface shear stresses over the 12 mm length, gave the "true" base load (Q_b). Rather than manufacture a small clay bed individually for each test, a process which can lead to excessive scatter of results (Whitaker 1957), in addition to unwanted boundary restraints on the system, a large remoulded London clay bed (1.5 m × 1.5 m × 1.4 m deep) was prepared and used for all tests with the sites judiciously distributed over the bed. The effect of variation of the pile length to bin depth ratio between tests was allowed for in the elastic analyses but no sidewall corrections were necessary. The bed was prepared by hand-ramming small clay cakes into the bin, simulating the action of a "sheeps-foot" roller. The clay all came from one small zone of a Hampshire brickworks clay pit, the clay was pugged and extruded into "green" bricks at the works and each brick cut into eight slices before being rammed into the clay bed. 77 moisture content samples taken throughout the bed had a mean moisture content $w = 25\%$ (standard deviation, $\bar{\sigma} = 0.58\%$) after it had matured under a sealed surface for about one year.

Conventional laboratory tests on the clay provided the following parameters, $w_L = 65\%$, $w_p = 23\%$, $I_p = 42\%$, $I_L = 4.7\%$, $G_s = 2.72$, $c_u = 100.0$ kN/m^2 ($\bar{\sigma} = 8$ kN/m^2) and a series of consolidated undrained triaxial tests the c_u versus w data shown in Fig. 1(a). Oedometer tests gave $C_c = 0.36$, $C_s = 0.079$ and the m_v, C_v values for the range of increasing consolidation pressures shown in Fig. 1(b).

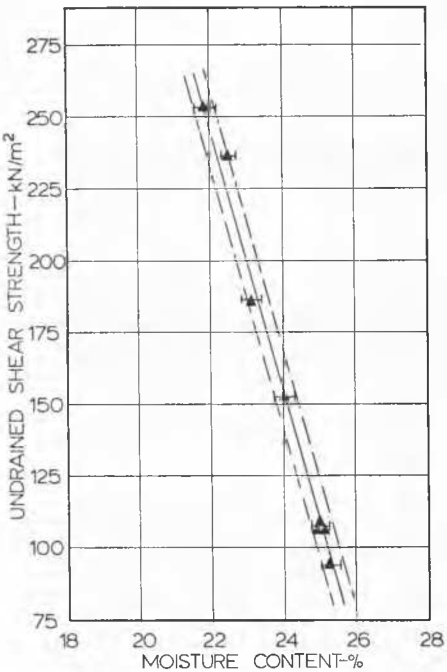

Fig. 1(a) w-c_u relationship from consolidated undrained 38 mm triaxial tests.

For all the load tests the total load was measured by an independent load cell and recorded on punched tape via a data logger and digital voltmeter with a sensitivity of 1 μV. The output of all the axial load cells used ranged between 1.5 and 2.0 μV/N with the radial stress sensitivity rather less satisfactory at only 0.3 μV/kN/m^2.

For the working load tests the pile penetration rate was reduced from 6.5 mm/min to 6.5×10^{-3} mm/min since the vertical displacements at which the nominal working loads developed on the piles were only of the order of 50 μm. These were measured by two displacement transducers, sensitivity 67 μV/μm per volt excitation, mounted symmetrically about the pile.

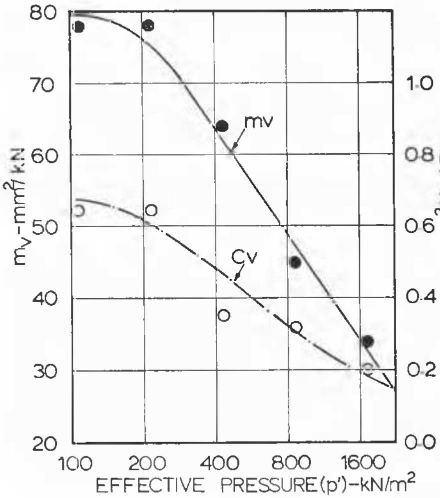

Fig. 1(b) Oedometer test results.

INSTALLATION MEASUREMENTS

Typical total load and base load versus penetration results are shown in Fig. 2 and similar measurements, together with the fully driven mean σ_r values, from 23 such tests are summarised in Table I. The conventional relationship for predicting Q_u in terms of an adhesion factor α, an undrained bearing capacity factor N_c, and L/D can be written as

$$\frac{Q_u}{\pi D^2 c_u} = \alpha(L/D) + \frac{N_c}{4} + \frac{\gamma L}{c_u} \qquad - (1)$$

in which, at model scale, the last term is negligible.

The Q_u values from Table I are plotted in this form in Fig. 3 which, within the experimental scatter, substantiates equation (1) with $\alpha = 0.45$ and $N_c \simeq 11.5$. This N_c value is also that calculated independently from the mean Q_b value in Table I ($\bar{\sigma} = 12.5\%$). Whereas it is commonly assumed that $N_c = 9$, higher values have been reported recently by a number of investigators also working in London clays. Cooke & Price (1973) obtained $N_c = 14$ at the end of CRP installation (5 mm/min) of a steel pile 168 mm diameter and 3.5 mm long which increased to $N_c = 16.4$ when finally tested (CRP = 0.125 mm/min) some 3 weeks later. Butterfield & Johnston (1973) found $N_c \simeq 18$, again during CRP installation (21 mm/min) of a 100 mm diameter steel pile also 3.5 m long. It should be noted that the present piles were installed in about one hour which for 12 mm diameter, means that they were modelling a very slow installation process indeed at field scale and therefore both here and during the subsequent working load tests drained conditions probably developed around them.

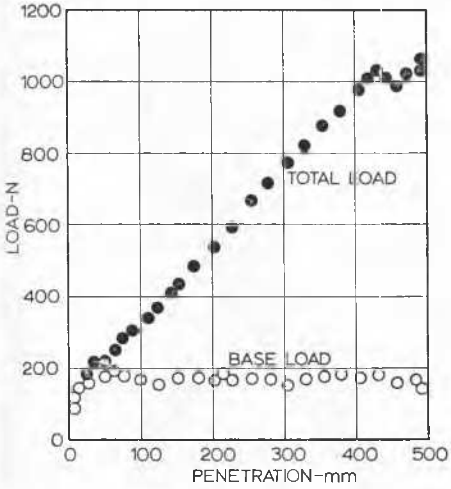

Fig. 2 Total load and base load during pile installation L/D = 40.

Fig. 4 shows a series of measurements of shaft load versus pile penetration on which are superimposed lines corresponding to various α values. Apart from a suggestion that higher α values may be applicable over the first few diameters of pile penetration an average α value rather less than 0.5 fits the majority of the data.

A typical plot of σ_r against pile penetration is shown in Fig. 5. The mean radial stress is seen to increase steadily with depth from about 4.2 c_u to ?.2 c_u when fully driven. From Table I the grand mean of the maximum σ_r values recorded was 454 kN/m^2 ($\bar{\sigma}$ = 17.5%) which corresponded to about 4.5 times the initial c_u values for the clay bed. These values are in reasonable agreement with more precise measurements reported by Butterfield & Johnston (1973) which ranged between 4 and 8 times c_u at the pile toe and with an analytical prediction, for undrained behaviour of 7 c_u (β = 150) published by Butterfield & Banerjee (1970).

From the local interface shear stresses, $\tau = \alpha c_u$, and the mean pile toe σ_r values the ratio $\tau/\sigma_r = \tan\delta$ can be calculated. The mean interface friction angle (δ) near the toe was found to be 5.7° ($\bar{\sigma} \simeq$ 1°) compared to the average value of δ = 10° (± 2°) for the 100 mm pile tests mentioned above.

When the piles were unloaded they remained in compression with residual base loads of 0.46 Q_b ($\bar{\sigma}$ = 10%) and residual radial stresses of 0.76 $\bar{\sigma}_r$ ($\bar{\sigma}$ = 13%). Cooke & Price (1973) also reported residual base loads of about 50% Q_b on their piles.

All the foregoing data serves to establish that the pile installation process and the clay bed generated highly reproducible results which conformed to the expected pattern of behaviour. The subsequent exploration of the response of the piles to axial working loads could therefore be expected to yield meaningful results.

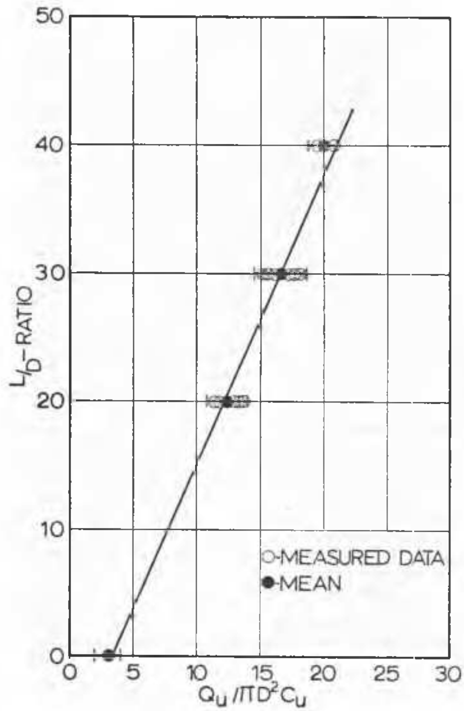

Fig. 3 Relationship between ultimate load Q_u and L/D ratios in non-dimensional form.

WORKING LOAD MEASUREMENTS

The results of 10 working load stiffness measurements are summarised in Table II in which, for each test, the upper row of the figures refers to a first loading approximately 1 hour after installation and the lower row to a second loading about an hour later. Typical load penetration curves are shown in Figs 6 and 7, for L/D = 30 and 40 respectively and loads up to approximately 50% of Q_u. These all show high linearity, with coefficients of correlation (r) greater than 0.98 in every case. When unloaded the piles rebounded fully without any measurable permanent displacement. The measured vertical stiffnesses (Q/W) from Table II are plotted in Fig. 8 against the three L/D ratios (20, 30, 40) investigated. The curves shown on this figure are those obtained from elastic analyses of a comparable pile system. The full lines are taken from Butterfield & Banerjee (1971) corrected for the effect of a rigid basal layer at the level of the tank bottom (Banerjee 1970), the chain dotted line shows Poulos (1972) elastic analysis (for G = 15.3 MN/m^2 and ν = 0.5) and the heavy dashed lines are the mean best fit curves through the experimental results

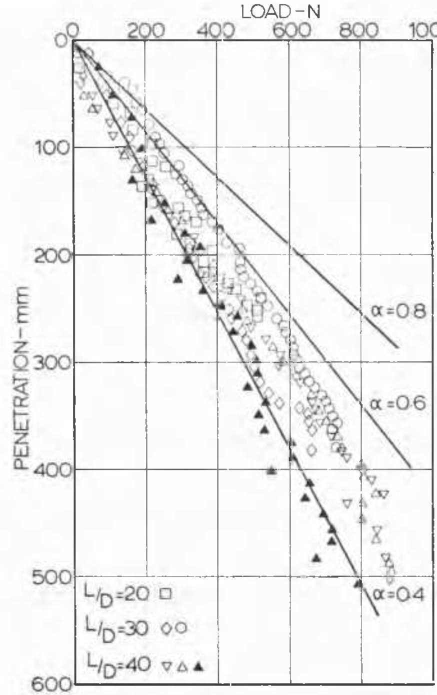

Fig. 4 Pile shaft resistance versus penetration.

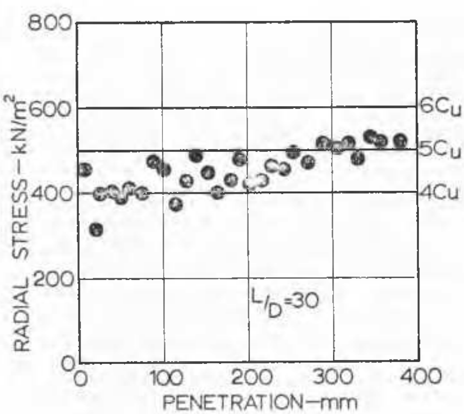

Fig. 5 Total radial stress at pile toe versus penetration

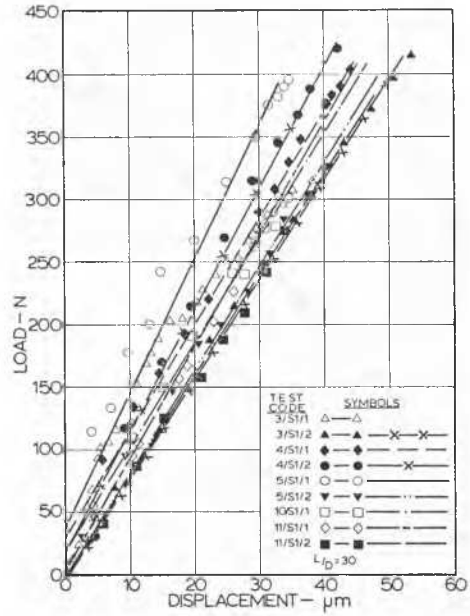

Fig. 6 Representative load displacement curves L/D = 30.

for first (A) and second (B) loadings. The difference in mean stiffness between A and B is only about 5% and from Fig. 8 an elastic shear modulus G ≈ 14 MN/m^2 is seen to provide a good interpretation of the experimental results (ν = 0.5). Alternatively by averaging the Table II values G = 15.4 MN/m^2 ($\bar{\sigma}$ = 11.3%) is obtained. Cooke & Price (1973) using a similar,homogeneous modulus, fitting technique obtained G ≈ 12 MN/m^2 in a softer clay in which c_u varied linearly between 35 and 65 kN/m^2 from ground surface to pile tip depth.

The model tests therefore established the linearity, under vertical centric working loads, of a single pile system in remoulded London clay and showed that a consistent interpretation of the stiffness over a range of L/D ratios is provided by elastic analysis with, in this case, G ≈ 15 MN/m^2 and ν = 0.5 for the soil. The extrapolation of this result to full scale piles in clay should be treated with caution however since at field scale an elastic model with a soil modulus increasing linearly with depth is probably more appropriate. The analysis of piles and pile groups embedded in such a "Gibson" material is now feasible (Banerjee & Butterfield 1976).

Any changes which occurred in the pile base loads and mean radial stresses during the working load tests were also recorded. The scatter in these measurements was high but the residual base loads increased typically from 5 × c_u to around 6 × c_u (i.e $\Delta Q_b/Q$ ranged from 12% to 14% whereas elastic analysis predicted values from 5% to 10% for long and

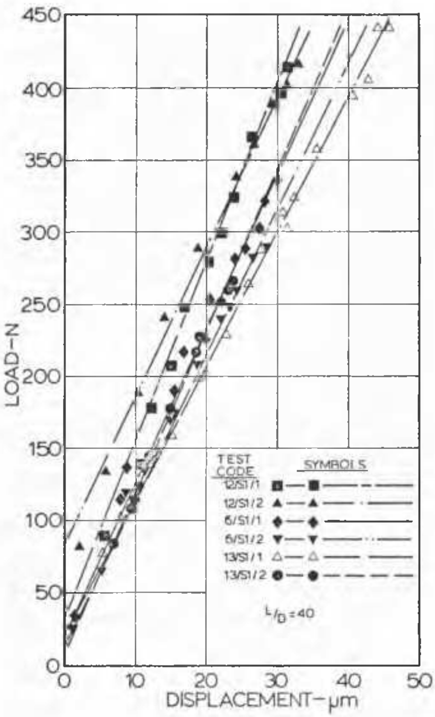

Fig. 7 Representative load displacement curves L/D = 40.

short piles respectively). The average change in σ_r was also very small, and tensile, reducing from its residual value of approximately 3.3 c_u to about 3.0 c_u.

EMPIRICAL ASSESSMENT OF THE SHEAR MODULUS

As suggested earlier an actual single pile test is probably the most realistic way of determining a G value for use in analyses of groups of piles. This has now been substantiated at model scale, by large numbers of group tests under a wide range of loading conditions (Ghosh, Abdrabbo 1976). However it should be appreciated that in a field situation such a single modulus is very much an averaged value which is unlikely to agree closely with more local moduli determined from either plate tests or, in particular, tests on small soil samples. For example although Cooke & Price (1973) found an overall best fit homogeneous G to be 12 MN/m^2, by measuring local shear stresses and strains around their pile they concluded that, at $Q = 0.35Q_u$, G near the pile-soil interface ranged between 2.3 and 3.7 MN/m^2 from top to bottom of the pile whereas at $Q = 0.57Q_u$ the corresponding values were approximately 3.7 and 4.7 MN/m^2.

Nevertheless the direct prediction of group behaviour in the absence of reliable single pile test data is a common practical problem. Therefore a further simplification often made for preliminary design purposes is to relate G empirically to soil parameters more readily available from standard tests. The most common of these assumes G and c_u to be linearly related by $G = \beta c_u$. The model test results reported here suggest that $\beta \approx 150$ which is in general agreement with field scale pile values summarised by Poulos (1972). In Cooke & Price's test c_u varied from 35 kN/m^2 to 65 MN/m^2 along the pile length which, at $Q = 0.57Q_u$, suggests local β values of 105 and 72 respectively.

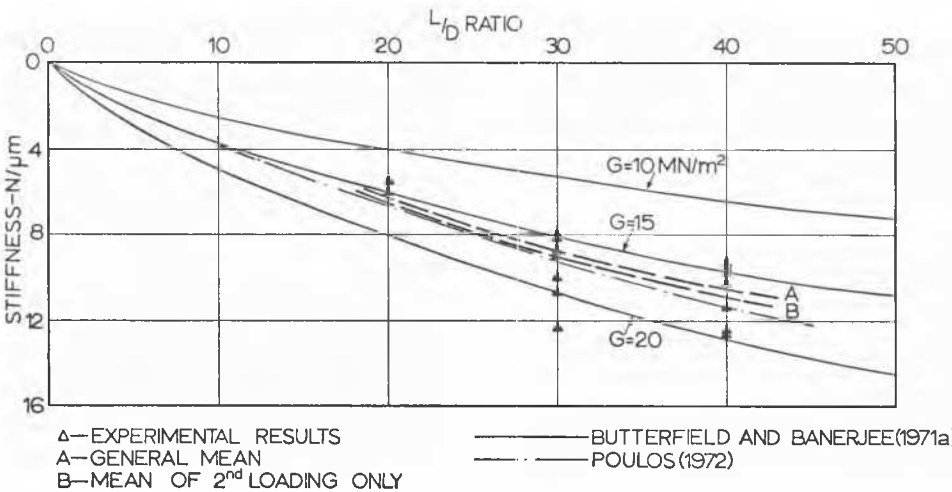

Fig. 8 Stiffness versus L/D ratio. Experimental values and results of elastic analyses.

TABLE I

Test No.	L/D ratio	Q_u N	Q_b N	Q_s N	σ_r kN/m^2
1	40	978	133	845	517
2	40	1058	165	894	345
3	40	974	142	832	414
Means		1003	147	857	425
1**	30	801	169	632	545
2**	30	925	165	760	372
3**	30	778	125	621	303
4	30	845	178	667	-
5	30	889	155	734	551
6	30	1108	(205)	(903)	517
7	30	800	142	658	558
8	30	818	126	694	517
9	30	667	(111)	556	(152)
10	30	867	120	747	448
Means		850	148	674	476
1*	20	614	151	463	469
2*	20	663	151	512	400
3*	20	583	147	436	338
5*	20	725	129	574	517
6*	20	716	169	547	483
7*	20	547	116	431	352
8*	20	596	138	458	496
10*	20	618	(111)	507	434
11	20	(471)	166	(307)	496
12	20	707	157	552	(896)
Means		641	147	479	443
Grand mean values			147N		454 kN/m^2
Standard deviations			18.4 N		79.2 kN/m^2

Table I: Pile ultimate loads, base and shaft loads and total radial stresses near the toe during installation

Tests marked * or ** are intermediate measurements on the longer piles and results in parentheses have been omitted from mean values.

If the mean c_u (50 kN/m^2) and G values (12 MN/m^2) are used then β = 240. This is clearly an overestimate of any "mean" β since the stiffer soil located directly beneath their pile base must be reflected in the average G value measured. Such relationships are further complicated by consolidation of the soil immediately around the pile shaft. In the model tests moisture content measurements immediately following pile extraction showed that w adjacent to the pile-soil interface had decreased by from 1% to 1.5% below the initial bed value. This implies, from Fig. 1(a) a local increase of c_u of almost 100%.

Wroth (1971) analysed a number of very precise (38 mm) triaxial tests on stiff undisturbed London clay samples and deduced a β value of 50. However he used the secant stiffness of his triaxial tests at 1% axial strain which implies very much higher shear strains than the average value relevant to a working load pile test. Again his β referred to a specific homogeneous sample c_u value whereas, as mentioned above, c_u will always vary in an unknown and complex way around any pile.

Marsland (1973), in a most important paper, discussed elastic moduli for London clay deduced from triaxial tests on 38 mm to 125 mm samples and also 865 mm diameter plate bearing tests in boreholes on the site used later by Cooke & Price for their pile test. From his shallower depth plate tests, on first loading, he found G values ranging from 15 to 20 MN/m^2 (ν = 0.5) with the latter figure applicable to tests in which the plate was meticulously hand bedded and loaded within three hours of completing the borehole and the former bedded on a machine finished base. These moduli were from 1.8 to 4.8 times greater than those he deduced from triaxial tests. Marsland also found the modulus to be strongly dependent upon the time which elapsed between borehole preparation and performance of the plate test (a three or four fold reduction when the time increased from 3 hours to 60 hours).

TABLE II

L/D ratio	Applied Load Q(N)	Head displacement W(μm)	Vertical stiffness (N/μm)	Soil shear modulus G (MN/m^2)
40	416	31.7	12.40	18.0
	415	32.3	12.55	18.3
40	476	50.5	9.32	13.6
	569	66.5	9.50	13.8
40	373	13.4	9.72	14.2
	290	28.3	10.24	15.0
40	441	44.5	9.28	13.5
	274	22.9	11.30	16.5
Means			10.52	15.4
30	423	52.0	8.13	14.4
	416	42.9	8.90	15.7
30	349	38.4	8.18	14.5
	356	28.8	(12.35)	(21.9)
30	417	37.9	10.00	17.6
	396	34.3	10.70	18.9
30	334	40.0	8.60	15.2
	276	32.0	8.60	15.2
30	296	33.7	8.80	15.6
	307	47.5	8.20	14.5
Means	243	46.0	5.4	12.9
20	243	46.0	5.4	12.9
	240	39.0	6.0	14.2
Grand mean G values			1st loading 2nd loading	15.0 MN/m^2 15.8 MN/m^2
Standard deviations				1.70 MN/m^2 1.79 MN/m^2

Table II: Loads, displacements, stiffnesses and best fit shear moduli from working load tests; 1st and 2nd loadings

The assessment of G and/or β is therefore far from a simple or precise procedure and at present the single in-situ pile test appears to be much the most plausible way of obtaining relevant global data.

CONCLUSIONS

1. The load capacity (Q_u) determined from 23 slow speed CRP tests on isolated 12 mm diameter model piles in a homogeneous bed of remoulded London clay could be interpreted consistently in conventional terms by using α = 0.45 and N_c = 11.5. Approximate measurements of radial total stress (σ_r) near the pile toe gave an average value of $4.5c_u$ at full pile penetration.

   On unloading residual base loads of around $5c_u$ remained under the piles and the residual σ_r values were just over $3c_u$.

2. Axial centric working load (Q) tests (at 0.5 Q_u) on 10 such piles showed highly linear behaviour on loading (r = 0.98) and unloading. The loading stiffness measurements were interpreted in terms of a homogeneous elastic system, with $\nu = 0.5$, which gave a best fit soil shear modulus G = 15.4 MN/m^2 ($\bar{\sigma}$ = 1.75 kN/m^2) from 20 measurements.

   During the working load tests the base loads increased by between 12% and 14% of Q, for L/D = 20 and 40 respectively and the radial toe stresses decreased on average by about 10% only.

3. If the mean soil shear modulus is related empirically to the mean initial clay bed c_u value by $G = \beta\ c_u$ then all the tests support $\beta = 154$ ($\bar{\sigma}$ = 17.5). This is in general agreement with other published values also obtained directly by elastic interpretation of pile load tests.

   The use of global G or β values is necessarily a gross approximation to the detailed response of different soil elements within the pile system. It is therefore suggested that neither tests on soil elements nor in-situ tests which do not simulate pile loading paths on the soil may be suitable for predicting parameters relevant to simple elastic stiffness analyses of pile systems.

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