The Design of Piles and Cylinder Foundations in Stiff, Fissured Clay

Pieux et fondations cylindriques sur l'argile raide fissurée

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SUMMARY

This paper describes tests carried out in the City of London to determine means of estimating the safe working loads on piles and foundation cylinders in a stiff fissured clay. The tests consisted of loading a 4.5-ft-diam bearing plate and a 6-ft-long section of 6.0-ft external diameter grouted lining to failure at four levels in the London Clay. From the results, design criteria for piles are established in terms of the load-settlement characteristics as well as the ultimate load capacity. The load-settlement relationships of the shaft and the base are shown to be markedly different; empirical formulae are established to express these relationships and the behaviour of the pile as a whole in terms of the elastic properties of the clay and the pile dimensions. The ultimate load is shown to be a function of the lower limit of the undrained shear strength.

SOMMAIRE

On décrit les essais effectués à Londres pour déterminer les moyens d'estimer les charges de service admissibles des pieux et fondations cylindriques construits sur l'argile raide fissurée. Des essais de charge sur plaques de 4.5 pieds de diamètre et sur revêtements coulés de 6.0 pieds de longueur et de 6.0 pieds de diamètre ont été effectués à quatre niveaux. Des résultats sont déduits des critères de construction pour pieux en fonction des caractéristiques de tassement ainsi que la capacité portante. On démontre que les relations entre le tassement et la charge du cylindre et de la plaque de support sont nettement différents: des formules empiriques sont établies pour exprimer ces relations et le comportement du pieu en fonction des propriétés élastiques de l'argile et des dimensions du pieu. La capacité portante est en fonction de la limite inférieure de la résistance au cisaillement à teneur en eau constante.

LONDON CLAY is a highly overconsolidated, stiff fissured clay which has been found to be suited to the construction of bored piles and mechanically augured foundation cylinders. These piles and cylinders are normally constructed by placing concrete into the unlined borehole in the clay. Holes up to 8 ft in diameter and 100 ft deep have been bored without lining and left open several days without difficulty. The larger foundation cylinders are frequently provided with mechanically formed enlarged bases up to 15 ft in diameter.

For more than a decade widely different views (Meyerhof and Murdock, 1953; Golder and Leonard, 1954; Skempton, 1959; Frischmann and Fleming, 1962) have been put forward on how the safe working loads on bored piles in London Clay should be estimated from the results of laboratory measurements of undrained shear strength. The question has become more acute with the advent of large cylinder foundations because their load-carrying capacity is so high that it is difficult and expensive to test full-size samples in order to prove the working load on each job, as is usually done with bored piles. It is therefore important to have a method of deriving a reliable safe estimate of the load-carrying capacity of a large cylinder. To understand the mode of action of these cylinders the authors' firm has made a number of large-scale field tests on the sites of various building projects in London. The most important of these are the tests described in this paper.

The previous tests carried out by the firm indicated that the ultimate load-carrying capacity of an enlarged base is not reached until the settlement is several times that which can be tolerated in any normal structure, whilst the full adhesion on the shaft is developed when the settlement is less than half an inch. It appeared from these tests that the criterion of the working load might be the settlement at the working condition rather than the ultimate load-carrying capacity. It appeared also that, even when the base is enlarged, most of the load at working load is carried by the adhesion of the shaft, although the maximum adhesion that could be reliably developed between the concrete shaft and the clay was uncertain.

The experiments described in this paper were carried out in this context, the objects being to obtain load-settlement relationships for the base and sides of a cylinder, and to relate the ultimate shaft adhesion and ultimate bearing capacity to the appropriate value of the undrained shear strength.

TEST PROCEDURE

From December, 1962, to May, 1963, a programme of *in-situ* testing was carried out on a site in the Great Saint Helens district of the City of London. The tests were designed to study the relationships between load and movements for shaft linings and bearing plates in London Clay. A lined shaft of 5 ft ID and 6 ft OD was dug by hand and lined with precast concrete segments. There were five segments to each ring, and the rings were 2 ft high. Each ring of segments was assembled with bolts as soon as the excavation had been removed, and the space behind the segments was immediately filled with Portland cement grout. At this stage it was not possible to apply any appreciable pressure to the grout. The shaft extended from an existing basement to 9 ft into the Woolwich and Reading beds, a

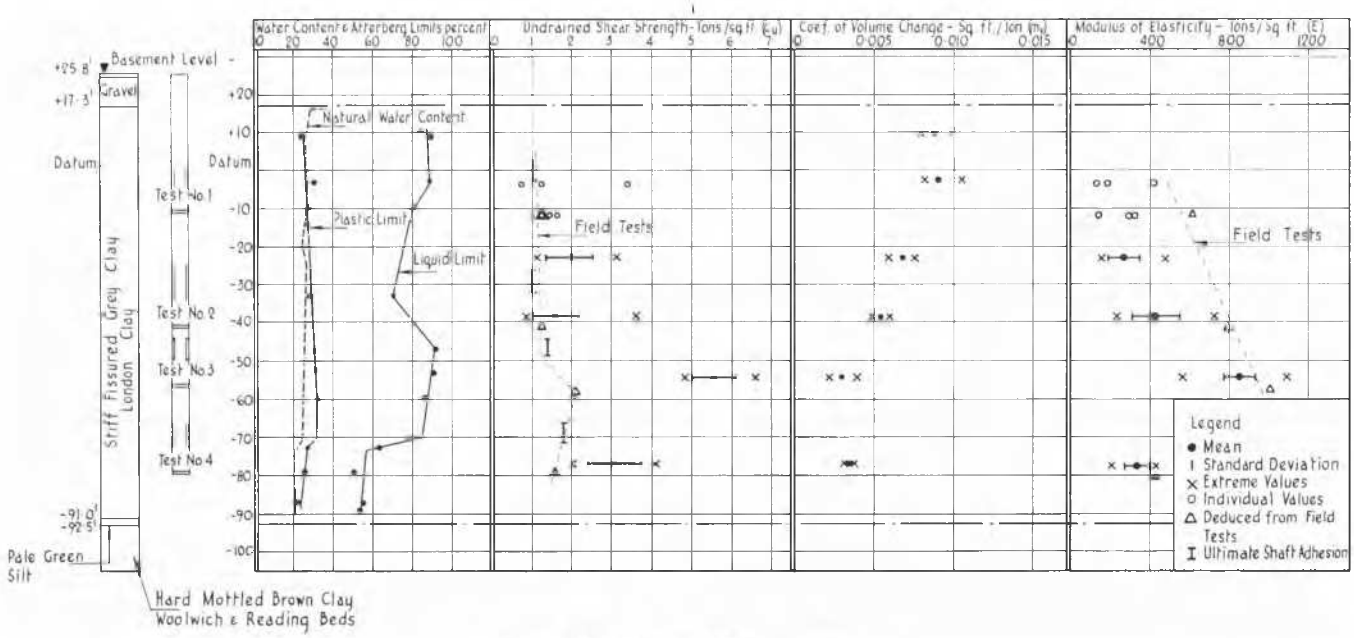

FIG. 1. Soil Profile and Properties.

depth of 124 ft. Care was taken to seal off the water in the gravel overlying the London clay before the shaft was sunk into the clay. During the excavation there was no visible water on the sides of the shaft, excepting a small amount from a fissure that was encountered at 26 ft below Ordnance datum. This seepage was completely sealed when the lining was built and grouted. Loading tests were carried out at four levels in the London Clay.

In Phase One of each test a concrete block, 4.5 ft in diameter, was jacked into the ground at the base of the shaft, the reaction of the jack being transferred *via* spreader beams to the concrete shaft lining above. The load transmitted to the block was measured independently of the jacking system by a hydraulic load cell; the settlement of the block was measured by dial gauges as the movement of three equilaterally spaced wires, stretched under a spring tension of about 40 lb, relative to a reference frame fixed at the top of the shaft. Each load increment was held for about 12 hours.

At the completion of the first phase the jack was released and a ring of shaft lining was removed to leave three rings (6 ft) of the shaft lining for the jack to push against. These three rings were loaded by jacking them against the base and the relationship between the load on them and their movement was measured by the same techniques as in Phase One. In Phase Two each load increment was held for about two hours.

TEST RESULTS

The soil profile, the levels at which the tests were conducted, and the soil properties as measured in the laboratory and deduced from the field tests are shown in Fig. 1. The results of the loading tests are shown in Figs. 2 and 3. It was found that there was a distinct change of slope in the plot of settlement against the square root of the time; this change of slope was taken to indicate the end of the "immediate" settlement. It was thus possible, as has been done in Figs. 2 and 3, to separate the "immediate" settlement from the "consolidation" settlement. In this series of tests the change in slope occurred after about 10 min for the base and 2 min

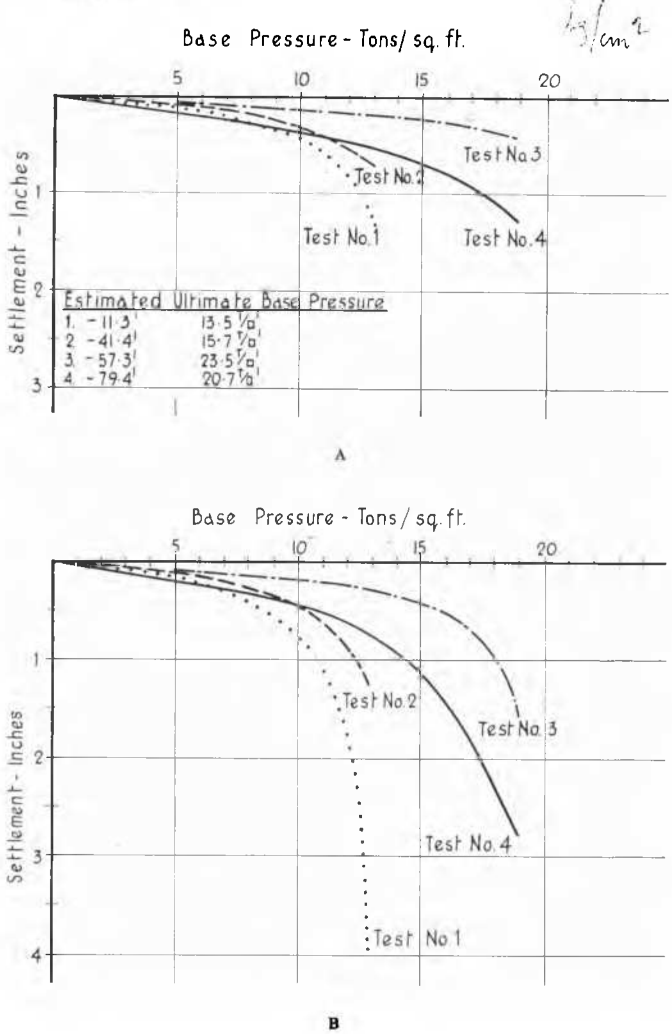

FIG. 2. A, Immediate (10-min) settlement *versus* base pressure. B, Long-term (12-hr) settlement *versus* base pressure.

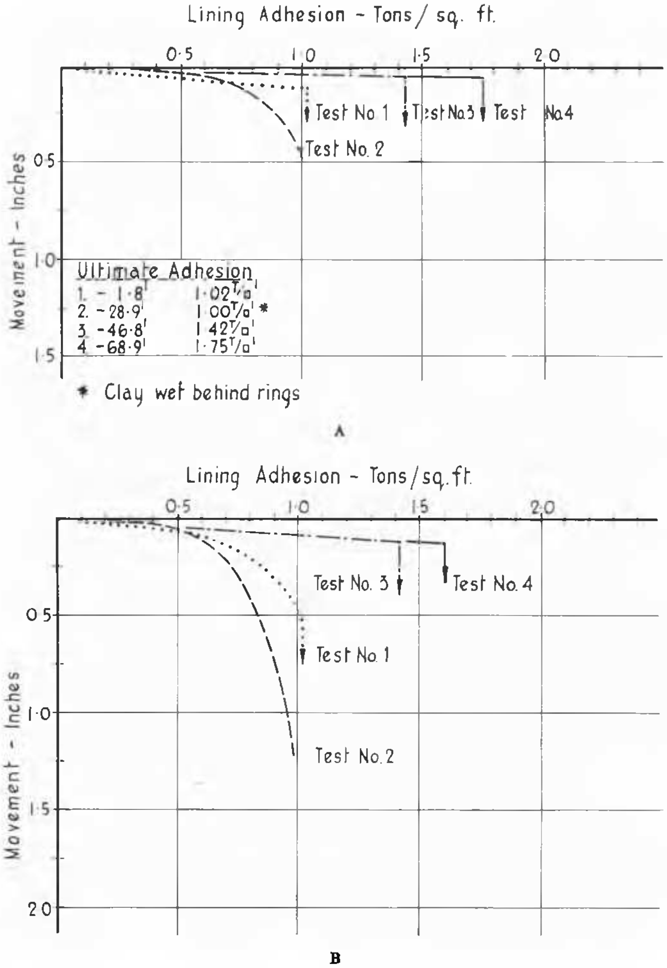

FIG. 3. A, Immediate (2-min) movement *versus* shaft adhesion. B, Long-term (2-hr) movement *versus* shaft adhesion.

for the lining and so the movement after 10 min and 2 min respectively has been taken as the "immediate" settlement.

There was no appreciable tilting of the block and the movement of the lining was uniform. When rings of lining were cut out for the second phase of the tests there was no evidence that wetting or softening of the clay had occurred during the grouting. In Tests 1, 3, and 4 the failure of the lining was sudden and it was impossible to maintain the load once failure had occurred; in Test 1, the failure appeared to be along the fissures in the clay adjacent to the lining. In Test 2, because of a silty seam containing running water, the clay behind the section of lining pushed had a water content about 3 per cent higher than the average for this depth; it is seen in Fig. 3 that the mode of failure for this section was markedly different from the other lining tests.

ANALYSIS OF RESULTS

Base

The base movement to failure was large. In terms of "immediate" settlement, failure occurred at about 5 per cent of the base diameter. The load-settlement relationship was roughly parabolic although it could be assumed to be linear up to about one-third of the ultimate load.

Taking N_c, the bearing capacity factor in the relationship $q_{ult} = c_u N_c + p_o$ (where q_{ult} is the ultimate bearing capacity, c_u the undrained shear strength, and p_o the overburden pressure), to be 9 (Bishop, Hill and Mott, 1945; Gibson, 1950; Wilson, 1950; and Meyerhof, 1951) the undrained shear strength of the clay can be deduced from the ultimate bearing pressure in the tests as: Test 1 at elevation −11.3 ft, 1.2 tons/sq.ft.; Test 2 at elevation −41.4 ft, 1.3 tons/sq.ft.; Test 3 at elevation −57.3 ft, 2.1 tons/sq.ft.; Test 4 at elevation −79.4 ft, 1.6 tons/sq.ft.

Shaft

The shaft-lining movement to failure was very small. Expressed as a percentage of the shaft diameter it was in Tests 1 to 4, 0.1, 0.6, 0.1, and 0.05 per cent respectively. Failure was sudden, and up to failure, except when the clay behind the lining was wet, there was an approximately linear relationship between immediate movement and adhesion.

The elastic stress distribution around a typical test section of the lining was analysed by the finite difference method, using an approach similar to that of Wilson (1948). This analysis showed that the shear stress along the length of the lining being pushed was substantially uniform. The ratio of settlement to adhesion obtained from the short length of lining tested is therefore applicable to an entire pile shaft.

RELATIONSHIP BETWEEN UNDRAINED SHEAR STRENGTH MEASURED IN THE LABORATORY AND IN THE FIELD

The shear strength of the clay was measured in the laboratory by unconsolidated undrained (UU) tests at a cell pressure equal to the overburden pressure. It can also be deduced, as indicated above, from the plate bearing tests. There was a wide scatter in the laboratory results and it is important to establish which value measured in the laboratory is appropriate to field behaviour. For the laboratory testing, one foot cube block samples were taken at 15-ft intervals as the shaft was dug. The blocks were cut from the clay by means of an electric chain saw and were immediately coated with paraffin wax. The large blocks were then cut up, again by saw, into 4 by 2 by 2 in. blocks and stored in wax. Triaxial samples, 1½ in. in diam, were trimmed from these blocks on a soil lathe. It was possible to test about 15 triaxial samples from each block. The wide scatter in the results can be seen from Fig. 1.

This wide scatter is unlikely to have been caused by sampling disturbance because of the extreme care taken but is probably some inherent characteristic of the clay. London Clay is overconsolidated and is hard and highly fissured. The wide scatter is explicable in terms of the fissures; the lower strength measurements in the UU test being the strength when failure occurs along a natural fissure and the higher ones when an intact non-fissured sample is sheared or when the direction of the fissures is not similar to that of the failure plane.

The shear strengths deduced from the ultimate bearing capacity, assuming that $N_c = 9$, are also plotted in Fig. 1. These results are close to the lower limit of the laboratory test results, showing that it is the "fissured" and not the "intact" strength that controls the ultimate bearing capacity. This is because the volume of clay involved in a base failure is sufficiently large to contain many fissures, some of which will coincide with the failure surface.

In previous work it has been assumed that the maximum adhesion of London Clay on the sides of the shaft of a pile or a cylinder is less than the undrained shear strength of the clay and the factor α has been ascribed to the ratio of the maximum adhesion to the average shear strength of the clay. The maximum adhesions obtained on the sections of shaft lining tested have also been plotted in Fig. 1, and they also lie close to the lower limit of the laboratory tests. It therefore appears that with a grouted lining, when the clay is dry, the adhesion on the pile shaft can be assumed to equal the fissured strength of the clay, and that it is not a function of the average of all the UU test results at that level.

DESIGN OF CYLINDER PILES

Every structure must satisfy two criteria. Firstly there must be an acceptable load factor against failure, that is indefinite movement, and secondly the deflections under working conditions must be acceptable. The results of these and other tests (Cooke and Whitaker, 1961; Frischmann and Fleming, 1962; Walker, 1963) show that the full adhesion on the sides of the largest practicable cylinder shafts is developed by displacements small enough to be acceptable in most structures. Thus, when the bearing capacity of the pile is mainly due to shaft adhesion, it is sufficient to apply a load factor to the calculated ultimate load, and the working settlement will automatically be small. On the other hand, enlarged bases can be constructed mechanically up to 18 ft in diam, and the tests suggest that so large a base will not reach its ultimate bearing capacity until it has settled some 11 in. This greatly exceeds the settlement that can be tolerated in most structures, so that consideration must be given to the settlement at working load in cases when an appreciable part of the bearing capacity is contributed by the base.

It is necessary therefore to derive or establish empirically the relationships between the settlement and the load in terms of the elastic properties of the soil and the pile dimensions, as well as to estimate the ultimate load capacity of the pile.

Load-"Immediate" Settlement

No theoretical load-settlement relationship has been established for a rigid base but analysis of the contact stresses under a rigid base shows that, in theory, the stress at the edge is infinity. Thus plastic zones develop at the edge and as the load is increased their extent increases. The development of these plastic zones will become dominant in determining the load-settlement characteristics. Therefore the empirical deduction from the tests that the settlement, ρ, varies as the base pressure, p_b, to the power of 3/2 seems appropriate.

This argument suggests for the base the empirical relationship,

$$\rho = (K_b R_b p_b / E_b)^{3/2} \tag{1}$$

where K_b is a constant, R_b is the base radius, and E_b is Young's modulus under the base.

For the shaft there appears from the test results to be a linear settlement-adhesion relationship to near failure. So for $p_s < p_{s\,ult}$,

$$\rho = K_s p_s R_s / E \tag{2}$$

where K_s is a constant, p_s is the shaft adhesion, and R_s is the shaft radius.

Since, in the working range, the load-settlement curves are almost linear, the settlement is assumed to be proportional to the radius, as it would be in an ideal elastic medium. To determine, from these equations relating the settlement to the shaft load and the base load acting independently, the behaviour of the pile as a whole the load carried by the shaft has been added to the load carried by the base at equal values of the settlement. This leads, ignoring secondary effects such as the shortening of the pile, to the following expression for the load, P, to produce a given settlement, ρ:

$$P = \frac{\rho\pi}{K_s}\int_0^{l_s} E_D dD + \pi R_b^2 \left[\frac{E_b \rho}{K_b R_b}\right]^{2/3}. \tag{3}$$

In the foregoing, E_D is Young's modulus at depth D and l_s is the length of pile shaft in the clay, allowing for any part of the shaft that is masked by the base.

Substituting in these equations the results of the tests using the values of E deduced from the field tests the values of K_s and K_b can be established. This has been done in Table I. K_s is a dimensionless constant. The equation for the base is not dimensionally viable, so K_b has been established on the

TABLE I. CALCULATION OF K_b AND K_s

		Base						Shaft					
Test Location	Test	Level below top of clay (ft.)	Young's modulus (tons/sq.ft.)	Radius (ft.)	Pressure (tons/sq.ft.)	Settlement (in.)	K_b	Level below top of clay (ft.)	Young's modulus (tons/sq.ft.)	Radius (ft.)	Adhesion (tons/sq.ft.)	Movement (in.)	K_s
Great Saint Helens	1	28	620	2.25	5.0	0.12	2.2	19	580	3.00	1.0	0.10	1.60
					10.0	0.43	2.3						
	2	58	780	2.25	5.0	0.07	1.6	46	720	3.00	0.6	0.05	1.75*
					10.0	0.31	2.6						
	3	74	920	2.25	5.0	0.06	1.6	64	820	3.00	1.5	0.05	0.75
					10.0	0.13	1.2						
					15.0	0.22	1.2						
	4	96	500	2.25	5.0	0.18	2.6	86	1000	3.00	1.5	0.05	0.90
					10.0	0.36	2.0						
					15.0	0.67	2.0						
				MEAN			2.0±0.6			MEAN			1.10±0.5
Piccadilly Circus (after Walker, 1963)		58	800	2.17	4.1	0.10	4.1	44	600	1.17	0.5	0.06	5.0
					8.1	0.23	3.6						
					12.2	0.45	3.9						
					16.3	0.80	4.5						
St. Giles Circus (after Frishman & Fleming, 1962)		50	450	3.00	2.3	0.11	4.7	23	300	1.50	0.36	0.20	4.2
					4.7	0.32	4.8						
					7.0	0.54	4.5						
					9.4	0.81	4.3						
					11.7	1.19	4.7						

*Clay wet behind lining.

basis of ρ being measured in inches, R_b in feet, and p_b and E_b in tons per square foot. Also in Table I are shown the values of K_s and K_b determined from two previous tests on unlined piles in London clay, one at Piccadilly Circus (Walker, 1963) and the other at St. Giles Circus (Frischmann and Fleming, 1962). The problem in comparing the results of the different tests is accurately estimating the value of Young's modulus. In Fig. 1 the secant modulus at 50 per cent of the maximum deviator stress in the UU tests and the values of Young's modulus deduced from the field tests have been plotted. The field values lie close to the upper limit of the laboratory results, and they are of the same order as values previously deduced from measurements of the heave in large excavations (Measor and Williams, 1962). This suggests that settlement calculations should be based on the upper limit of laboratory measurements of the modulus.

The difference between the values of K_s and K_b for the Great Saint Helens tests and those for the others emphasizes the difference between hand-dug and machine-augured cylinders.

Ultimate Load

The ultimate load carrying capacity is given by:

$$P_{\text{ult}} = 2\pi R_s \int_0^{l_s} \alpha c_{u_D} dD + \pi R_b 9 c_{u_b}$$

where c_{u_b} is the lower limit of the undrained shear strength of the clay at the level of the base and c_{u_D} is the lower limit of the undrained shear strength of the clay at depth D.

It must be noted that this formula differs from that of previous publications in this field, in that it is based on the lower limit of results of UU tests. Most previous formulae have been based on the mean undrained shear strength and α as applied to the mean strength has been assigned values varying from 0.4 to 0.7.

α, as applied to the minimum undrained shear strength, has been shown in the tests described in this paper on a hand-dug shaft with a grouted lining to be 1.0. Since few of the papers describing ultimate load tests on piles in stiff fissured clays give sufficient strength measurements for the lower limit of the undrained shear strength to be established, more tests will be required before definite values can be ascribed to α as defined above. However, Golder and Leonard (1954) gave sufficient information when describing tests on two *in-situ* piles constructed by boring unlined holes into clay and filling them with concrete. An analysis of the ultimate capacity of their piles B5 and B33 gives values for α of 1.0 and 1.15 respectively. Analysis of data supplementary to that published by Walker (1963), shows that for that test, as applied to the minimum strength, α equals 0.85.

CONCLUSIONS

The tests described in this paper show that in stiff fissured clays:

1. Piles and cylinder foundations must satisfy two design criteria: there must be an adequate factor of safety against ultimate failure and the settlement under working load must be tolerable.
2. The ultimate bearing capacity is a function of the lower limit of the undrained shear strength.
3. For a hand-dug shaft with a grouted lining the ultimate adhesion is equal to the fissured strength. More tests will be required to determine the relationship for other types of shaft but α should probably have a value approaching 1.0.
4. The load-settlement characteristics of the shaft and the base are markedly different.
5. The load-settlement relationship of a cylinder is a function of the upper limit of the values of the secant modulus of elasticity measured in the UU test.

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