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Advances in understanding of base grouted pile performance in very dense sand

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Abstract

At Canary Wharf in London, a number of buildings are under construction which are supported on large diameter piles with base grouting founded in the Thanet Sand stratum. Most of these buildings are being built within large cofferdams on land reclaimed from the docks that surround the development. Existing design methods, which do not take into account the relatively high horizontal stresses that act in the ground in this situation, were found to result in conservative designs. Following the loading of a heavily instrumented test pile to failure, a revised design approach was proposed which relates shaft stress to the horizontal stresses in the ground. The design method also explicitly considers pile head movements and uses limits compatible with the structural design for the derivation of the pile design parameters and factor of safety.

Introduction

Canary Wharf has been a very important centre of business for London both in the recent past and historically. During the past 12 years the focus has been towards redeveloping the area as a financial and commercial centre. Canary Wharf is currently Europe's largest commercial development consisting of approximately 30 individual sites of medium and high rise structures. As part of the Phase 2 development currently under way, two new towers similar in scale to the existing One Canada Square tower are being constructed for HSBC and Citigroup.

As with any new development of significant scale, preliminary pile tests are necessary to fully evaluate and understand the particular site conditions. Many such tests have already been undertaken at Canary Wharf. However, none have fully tested base grouted piles to failure, neither have previous pile tests been undertaken from a reduced level and lower overburden due to installation of piles from the base of a cofferdam. In order to correct this, an ambitious preliminary pile testing scheme was proposed. This related in particular to the HSBC site, but would subsequently provide a more economical solution for other structures to be constructed within the Phase 2 works.

In order to fully understand the performance of base grouted piles founded in very dense over-consolidated sands such as the Thanet Sand, it was necessary to carry out load testing to a level sufficient to mobilise a significant proportion of the end bearing capacity. To do this would require loading close to the structural limit of the pile.

This paper presents results of load testing a fully instrumented test pile founded approximately 5m into the Thanet Sand. The test pile was 900mm diameter and was sleeved though the Terrace Gravel. Reaction for the test was provided by six 1500mm diameter anchor piles.

The Site

Canary Wharf is located approximately 5km east of the City of London at the northern edge of the Isle of Dogs, Figure 1.

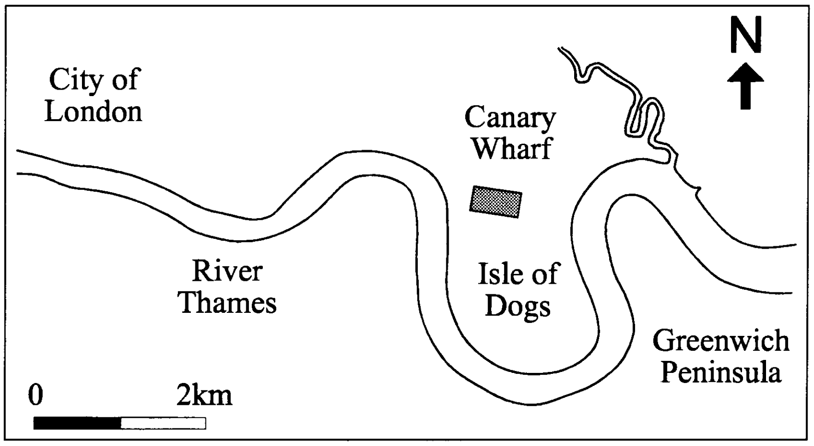

Fig 1 : Key Plan

Canary Wharf Limited are currently proceeding with phase 2 of the development which includes construction of a new headquarters for HSBC. This site which is shown on Figure 2 is to the east of the existing Canary Wharf development and is located between the old West India Import and Export Docks. The HSBC tower is being constructed in the north cofferdam, which was constructed during the Phase 1 development. Originally this formed part of the West India Import Dock. Development of the area for offices started in 1988.

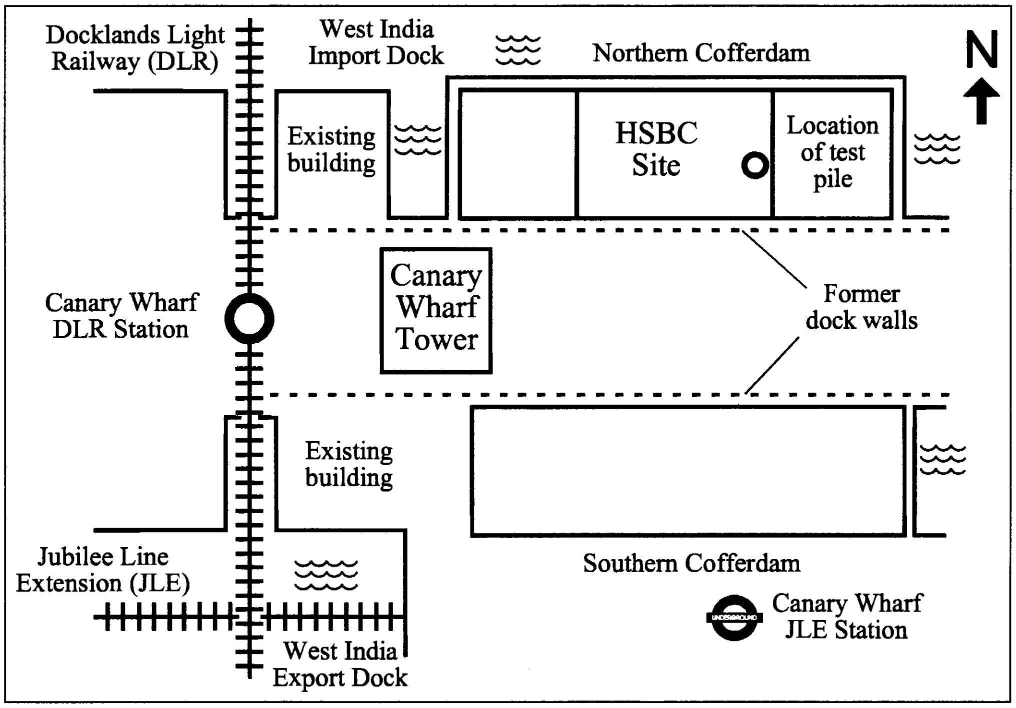

Fig 2 : Site Location Plan

Historical Background

The Canary Wharf area has been subject to much development throughout the last 200 years, for the most part being part of a major port. The site was first developed from 1802 when it was selected for the location of the West India Dock, with the Import and Export Docks opening in 1802 and 1806 respectively.

Ground Conditions

Ground conditions at the site comprise Terrace Gravel, Lambeth Beds Clay, Lower Lambeth Beds Sand and Thanet Sand overlaying Chalk. Figure 3 summarises the soil strata obtained from a borehole put down next to the test pile location and also shows details of the test pile.

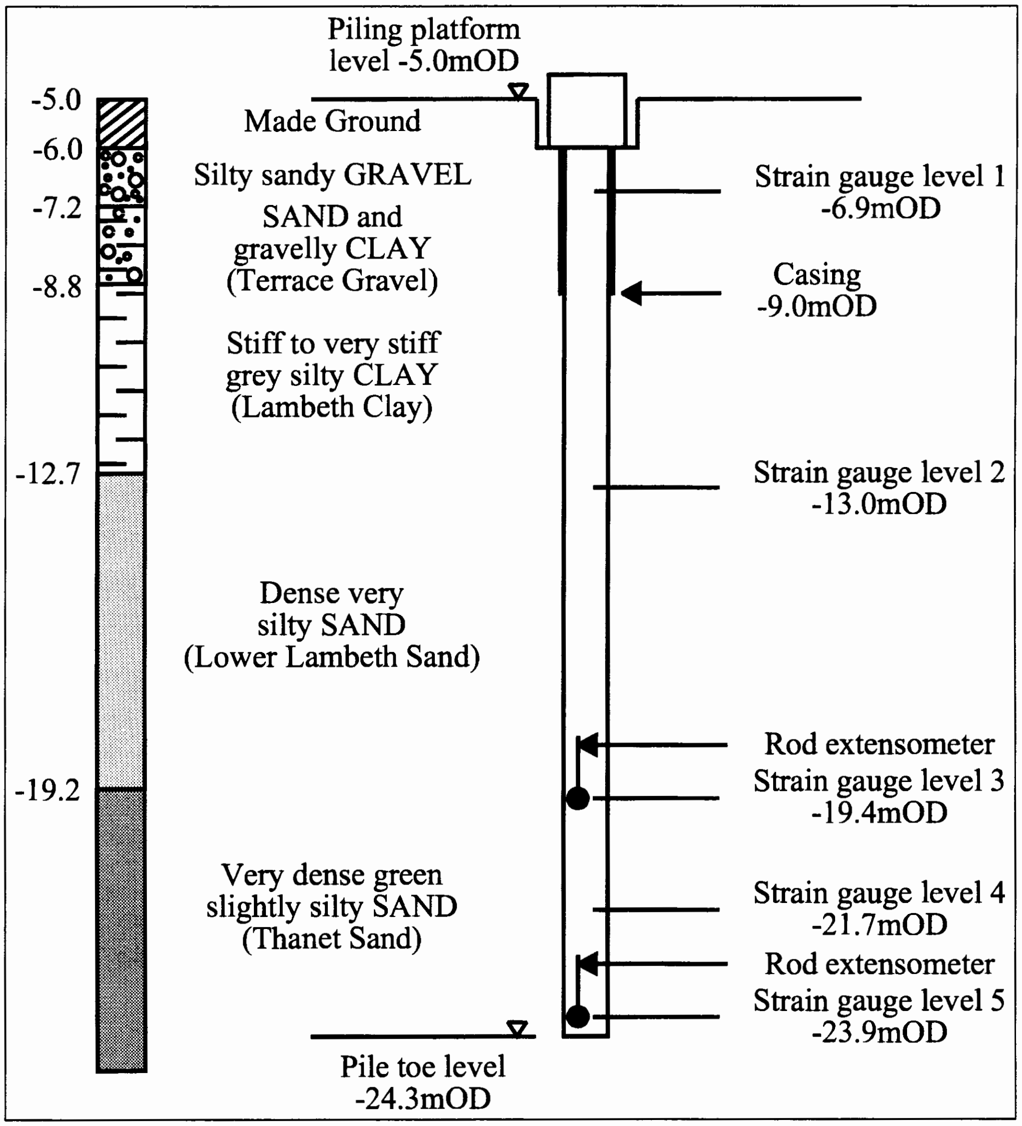

Fig 3 : Ground Conditions and Test Pile

Groundwater

The stratigraphy of the site described above is generally split in terms of the groundwater, into upper and lower aquifers separated by the Lambeth Clay aquitard. In the upper aquifer, groundwater is at a level of approximately –5.5mOD as a result of groundwater lowering within the cofferdam. Due to historical pumping, groundwater in the lower aquifer is also depressed, resulting in under-drainage of the clays, but is generally rising at a rate of a few metres per year.

A piezometer was installed next to the test position to establish the precise ground water conditions during the testing works.

Pile Description

Although contract piles were to be 1500mm diameter and bored up to 8m into the Thanet Sand, it was proposed to test a smaller diameter 900mm pile with a 5m socket as detailed in Figure 3. This would have the dual advantage of reducing the required reaction force but maximising the possibility of achieving an ultimate capacity.

1500mm diameter piles were proposed for the reaction. Because of severe space restrictions, it was necessary to use six working piles. Modifications to the steel reinforcement for the piles were made to accommodate the test reaction loads.

Geotechnical Design

Ground conditions for the pile are shown in Figure 3. At the time of testing, ground water was at about –5.5mOD, with a profile less than hydrostatic due to under-drainage of the clays. The pile design parameters used for the initial pile capacity check are given in Table 1.

Strata Description	Assumed Soil Properties	Pile Design Parameters
Terrace Gravel	ϕ' = 33 degrees	Ignored in design
Lambeth Clay	c_u = 100 kPa	α = 0.4
Lambeth Sand	ϕ' = 33 degrees	K = 0.7 δ = 2/3 ϕ'
Thanet Sand	ϕ' = 36 degrees	K = 0.7 δ = 2/3 ϕ' Nq* = 47

Table 1 : Soil Properties and Design Parameters

Check calculations based on the above parameters gave a typical pile capacity of about 12,600kN. However, parametric studies allowing for variations in the soil parameters and groundwater assumptions suggested a range of ultimate pile capacities up to 21,000kN. This value was therefore selected as the maximum test load.

Reinforcement Design

Structural design was required to ensure that the test pile and anchor piles had sufficient strength to accommodate the maximum applied test load.

Figure 4 shows structural details of the proposed 900mm diameter test pile. The pile was formed using a 14 day 50N/mm^2 concrete and was reinforced with a full length 24T40 cage. Bars were bundled in pairs to increase the clear spacing between bars to allow easier concrete flow into the 75mm cover zone. Shear reinforcement comprised T20 rings at 100mm centres increasing to 200mm centres below 12m depth. Two pairs of grout pipes were provided for base grouting. The grout pipes were also used for sonic logging to confirm full integrity of the test pile shaft.

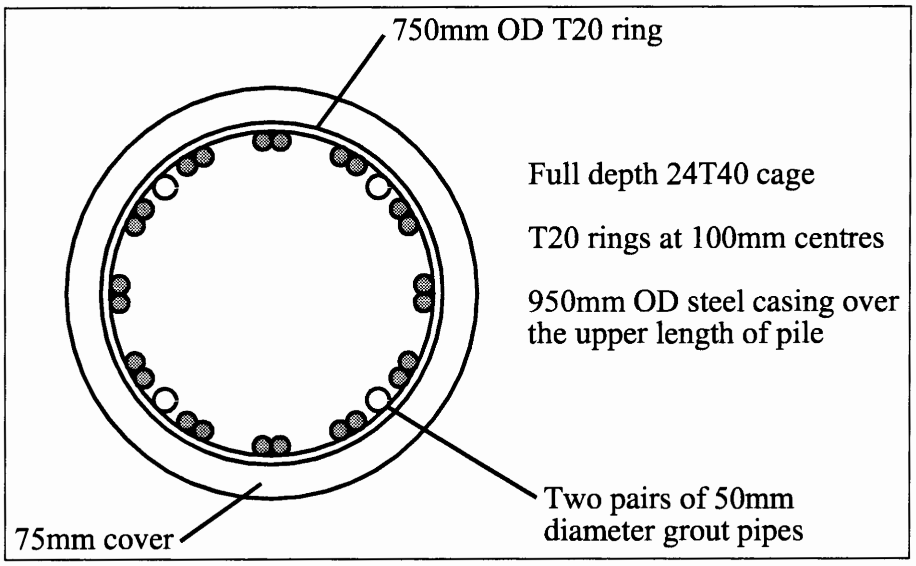

Fig 4 : Structural Details of Test Pile

A 950mm OD 15mm wall thickness casing was installed over the upper 3m length of shaft to provide additional moment capacity during testing.

Nominal design verification load (DVL) for the test pile was specified as 6,000kN. The six anchor piles and load frame were designed to cater for a maximum test load of 21,000kN, equivalent to 3.5 times working load. Pile shaft stresses were checked for all load cycles. Pile stresses were expected to exceed 0.4 fcu for loads greater that 2.5 times working load, reaching a peak shaft stress of 33N/mm^2 at the maximum test load. This was considered acceptable for controlled short term loading. Taking account of both the concrete and steel area suggests a minimum partial load factor of 1.1 at the maximum test load.

Structural analysis was carried out to determine the ultimate bending moment capacity for the test pile section. Computations suggested a minimum moment capacity of about 1480kNm at the maximum test load of 21,000kN.

Instrumentation

During construction, the test pile was fitted with vibrating wire strain gauges and rod extensometers. In order to provide complete information about the load distribution throughout the length of the pile, five levels of strain gauges corresponding to changes in the geology were installed as shown in Figure 3. Four strain gauges were installed at each level. Rod extensometers were installed at levels 3 and 5.

Pile Cap Design

Because of the very high shaft stresses anticipated during the load test, design of the test cap was critical to ensure safe application of the load. The pile cap was cast using a 1550mm OD 15mm wall thickness steel casing to form a cap 1.5m in height. The cap was reinforced using the test pile steel and an additional concentric 20T32 cage with T16 shear rings at 100mm centres.

The test load was applied using two hydraulic jacks bearing against a 1.4m square by 150mm thick steel plate bedded onto the top of the pile cap. Design checks were carried out to ensure that adequate shear and bursting reinforcement was provided. Punching shear, reinforcement bond and bearing stresses were also checked.

Pile Construction

The test pile was constructed from a reduced level within the North Cofferdam at the eastern edge of the HSBC site, Figure 2. The 900mm diameter test pile and six number 1500mm diameter anchor piles were installed using a purpose built RT-3 auger piling rig. Drilling of the Lower Lambeth Beds Sand and underlying Thanet Sand was carried out using bentonite drilling techniques.

Pile Boring

Excavation through the Terrace Gravel required the use of temporary casing installed into the top of the Lambeth Clay. This allowed open bore construction using a bladed auger to the base of the clay, at which point bentonite was introduced into the bore. The pile was then completed using a drilling bucket. During construction, the bore was logged to confirm the expected ground conditions. Before concreting, the bentonite within the bore was cleaned to remove sand and then checked to ensure conformance with the specification criteria for density, pH and rheological properties.

The 900mm diameter test pile was constructed to give a 5.1m socket length in the Thanet sand. The six 1500mm diameter anchor piles were constructed in a similar manner, but were bored about 8m into Thanet Sand.

Concreting

As described above, the reinforcement required to cater for the test load resulted in a very congested cage. A high slump self compacting concrete mix was used in order to ensure maximum concrete density and flow of concrete in and around the reinforcement. The concrete was placed using a tremie pipe, which was surged regularly to improve flow and compaction. Pile concreting records show a 12% total concrete overbreak above the theoretical volume. This would suggest an actual pile diameter of 950mm.

Base Grouting

Two grouting circuits were used to base grout the test pile. Each grouting circuit comprised a pair of 50mm diameter tubes connected to a tube à manchette at the base of the cage. Three days after construction, water was used to hydrofracture the concrete at the base of the pile.

Base grouting was undertaken seven days after pile construction. During grouting, applied pressures and pile head uplift were closely monitored, together with the pile strain gauges. Uplift was measured using dial gauges and LVDTs connected to the data logger used to monitor the strain gauges. Reference beams were also checked using precise levelling techniques.

A total of 231 litres of grout was pumped. This resulted in a maximum pile uplift of 0.35mm under a grout pressure of up to 60bar.

Pile Testing

The pile load test was a modified form of that given by the ICE Piling Specification (1996). A total of five maintained load cycles were specified.

All testing was carried out using a computerised logging and control system to ensure minimal load variation. Load was measured using a fully calibrated load cell. Pile head settlements were measured using LVDTs connected to a data logger, with precise level checks on the pile head and reference beams. Load and settlement measurements were taken at one minute intervals reducing to 5 minute intervals for the latter stage of each maintained load. Strain gauges and extensometers were monitored at 5 minute intervals using a synchronised data logger.

To give reassurance that the test reaction load was not damaging the anchor piles, one pile was also monitored during the test for uplift, with a load cell measuring the induced tension. This information provided valuable data for the design of tension piles.

Load Test Results

Figure 5 gives the measured load versus head settlement relationship for the test pile. A summary is given in Table 2.

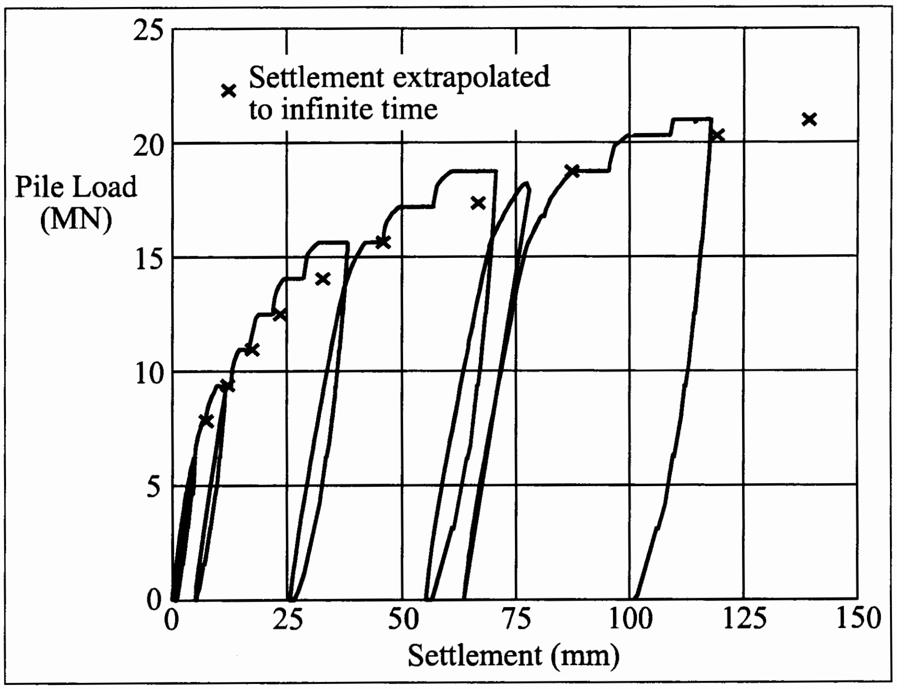

Fig 5 : Measured Load Settlement Behaviour

Load Cycle	Maximum Test Load	Measured Pile Head Settlement
Cycle 1	6,250kN	4.98mm
Cycle 2	9,400kN	11.55mm
Cycle 3	15,650kN	38.31mm
Cycle 4	18,750kN	70.73mm
Cycle 5	21,000kN	117.94mm

Table 2 : Measured Test Results

Figure 6 shows the results of a CEMSET extrapolation fitted to the measured load versus head settlement relationship for the test pile. This extrapolation suggests an ultimate pile capacity equal to about 25,000kN, with the shaft component equal to about 7,000kN.

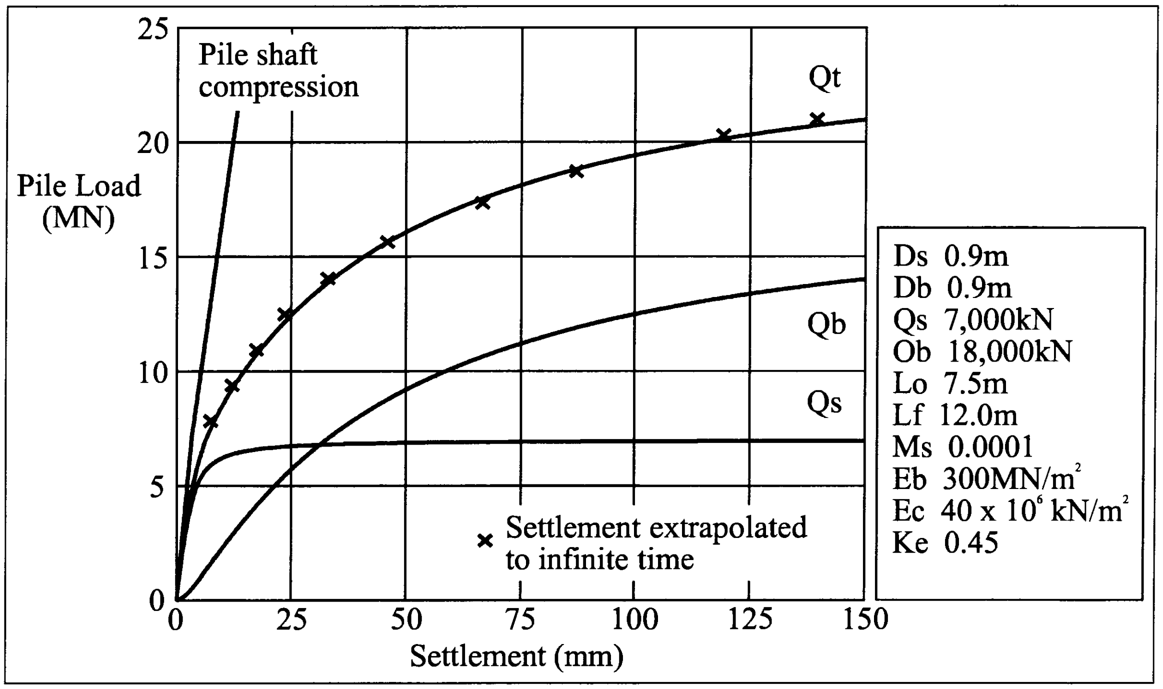

Fig 6 : CEMSET Load Settlement Extrapolation

The variation of load along the pile shaft for each loading stage is shown in Figure 7. At peak load, strain gauge measurements suggest a total shaft capacity of about 6,000kN, comparable with the CEMSET extrapolation. This was computed as the difference between the applied head and base load derived from strain gauges located close to the pile toe.

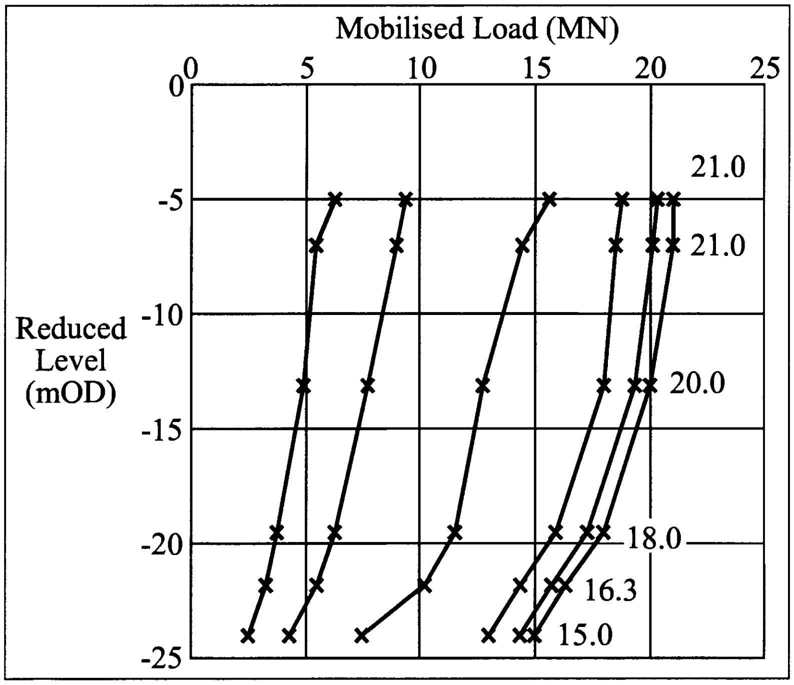

Fig 7 : Load Distribution Along Pile Shaft

Mobilised Capacities

Although the pile was not fully taken to failure, the excess capacity not tested was small and was ignored when design parameters were being derived from the results.

The distribution of load down the shaft and on the base is shown in Figure 7. A summary is given in Table 3 for the condition where no limit is placed on the failure load and also where failure is arbitrarily defined as the load at which head settlement equals 10% of the pile base diameter.

Strata Description		Failure Load (Maximum load reached)	Failure Load (10% of pile diameter)
Terrace Gravel	SHAFT	Assumed low	Assumed low
Lambeth Clay		1,000kN	1,000kN
Lambeth Sand		2,000kN	2,000kN
Thanet Sand		3,000kN	3,000kN
	BASE	15,000kN	13,500kN
Total		21,000kN	19,500kN

Table 3 : Interpreted Load Distribution

These loads can be related to conventional shaft design parameters, as in Table 4. These values are based on a shaft diameter of 950mm, established from the recorded concrete overbreak record. The shaft capacity in the Thanet Sand is that measured between the strain gauges at its surface and those closest to its base. The strain gauges at the base were actually about 400mm above the base so the shaft capacity measured was only for a 4.5m length.

Strata Description	Failure Stress	Design Approach	Design Parameters	Back Analysed Parameters
Lambeth Clay	84kPa	$\alpha.c_u$	c_u = 100kPa	α = 0.8
Lambeth Sand	105kPa	$\sigma_v'K\tan\delta$	σ_v' = 155kPa $\delta = \phi'$ = 33°	β = 0.68 K = 1.05
Thanet Sand	223kPa	$\sigma_v'K\tan\delta$	σ_v' = 210kPa $\delta = \phi'$ = 36°	β = 1.06 K = 1.46

Table 4 : Back-Analysed Parameters

The method of calculating base capacities in these ground conditions was proposed by Troughton and Platis (1989). Instead of relating the base capacity to effective vertical stress using values of Nq from Berezantsev *et al* (1961), they used Vesic's (1977) approach which relates the ultimate base capacity to the mean effective stress in the ground σ_m', calculated for the test conditions as 346kPa. The measured base load of 15,000kN equates to a base stress of 23,580kPa. This suggests a Vesic Nq* value of 68.

Factor of Safety

Following the philosophy of the LDSA design note for bored piles in clay (LDSA, 1996), it had been agreed that the successful execution of a preliminary pile test would allow the factor of safety to be reduced to 2.25.

Design to Limit Movement

The normalised load displacement curve is shown on Figure 8. Load P has been normalised by an assumed failure load P_{ult} of 21,000kN. Settlement Δ has been normalised by the specified pile diameter D of 900mm.

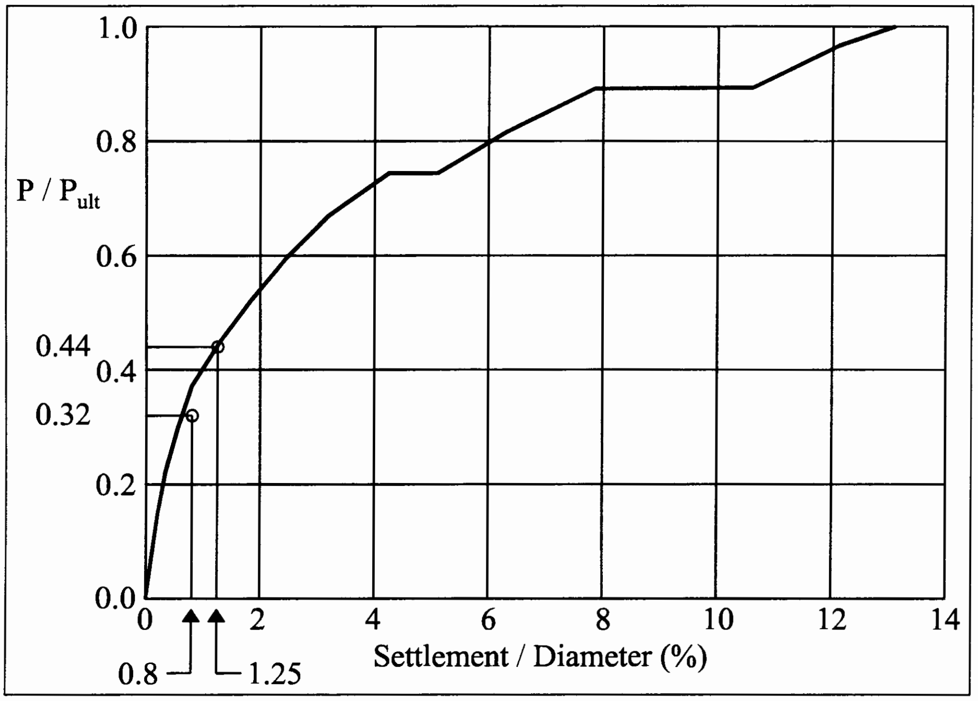

Fig 8 : Normalised Load Settlement Behaviour

Use of a factor of safety of 2.25 yields a P/P_{ult} ratio of 0.44. At this ratio, the corresponding Δ/D ratio is 1.25%, see Figure 8. Applying the design to the 1.5m diameter piles which were proposed, would give a typical settlement of an individual pile at working load of 19mm, which would lead to an expected range of 15 to 24mm after allowing for a 25% range in pile performance. This was thought too large. Additionally, raft analyses had shown that for the larger buildings which were supported on dense forests of 1.5m diameter piles spaced at 2.5 diameter centres, the maximum settlement in the centre of the raft would be proportionately much larger than that of an individual pile. Such raft displacements were also thought to be of concern.

It was therefore decided to target the settlement of an individual pile at 0.8% of Δ/D at working load. This corresponds to a P/P_{ult} ratio of about 0.32, see Figure 8. The expected settlement of a pile at 1.5 times working load ($P/P_{ult} = 0.48$) was taken as 1.6%.

This approach involved an increase in load capacity. Therefore the expected settlements of individual piles would be greater than piles using the original design basis. The specified performance criteria for piles tested in contract load tests were therefore increased to 1.0% of Δ/D for a load equal to working load and 2.0% for a load equal to 1.5 times working load. These values allow for 25% variation in pile performance.

Proposed Design Approach

An immediate benefit of the extra capacity proved in the test was to allow piles to start below the temporary casing 18 hours before concreting instead of 12 hours as previously specified.

This was allowed for by reducing the α value for the Lambeth Clay from 0.4 to 0.2. The 12 hour limitation remained in place for deeper strata.

As the Lambeth Clay was partially made up of a series of intermittent conglomerate bands that were present at the test location, it was considered imprudent to try to increase the calculated capacity of this layer.

A conventional design approach for the Lambeth and Thanet Sands relates ultimate skin friction to the vertical effective stress using an assumed K value (typically 0.7) in the following equation.

$$Q_s = K\sigma_v' \tan\phi\pi DL$$

This approach can be conservative in the over-consolidated soils beneath the HSBC site where the vertical stresses have been reduced by excavation and the horizontal stresses are relatively high. It is these horizontal stresses that control shaft friction.

It was felt that an appropriate basis for pile design was to adopt a method which uses the correct horizontal effective stresses which act on the piles in the ground. This was achieved by separating the design K factor into two components, K_o the ratio between the existing vertical and horizontal stresses, and a factor 'a' to take into account stress relief and disturbance caused by pile installation.

$$K = a.K_o$$

Therefore the conventional design equation can be modified to

$$Q_s = a.K_o\sigma_v' \tan\phi\pi DL \qquad \text{or} \qquad Q_s = a.\sigma_h' \tan\phi.\pi DL$$

To establish suitable values for factor a, it was necessary to estimate the insitu horizontal stress σ_h'. This was done by carrying out an assessment of the stress history of the soil alongside the pile.

The West India Import Dock was excavated in 1802 and was progressively deepened to about –5mOD. When the cofferdams within the dock were installed in 1992, the dock silt was removed and hardcore was placed on top of the natural Terrace Gravel to form a piling platform at –5mOD. Ground level on the adjacent wharf is about +6mOD.

Computations for σ_h' were carried out using a modified form of the approach given by Burland *et al* (1979). The calculated horizontal effective stress σ_h' was limited to avoid passive failure following Bolton and Stewart (1994), using a passive failure criterion set at

$$\sigma_h' \ngtr [1/(1-\sin\phi')]\sigma_v'$$

This value was chosen provisionally as a limit rather than full K_p as the aim of the design was to limit pile displacements. It is expected that future analysis of the pile behaviour using finite elements will allow a higher limit on passive failure to be used.

Values of factor a computed from the test results are shown in Table 5. The interpreted values were then reduced by 25% to give design values. This would account for possible increased disturbance during construction of contract piles.

Strata Description	Calculated σ_h'	Interpreted Friction	Calculated Factor a	Design Factor a	Limiting K used in design $a.K_o$
Lambeth Sand	300kPa	105kPa	0.54	0.4	0.88
Thanet Sand	379kPa	223kPa	0.81	0.6	1.44

Table 5 : Factor a Values Used in Design

The data presented by Troughton and Platis was reviewed in the light of this approach. The range of possible shaft frictions deduced in the Lambeth Sand in their load test was 3730kN to 5000kN which corresponds to values of 0.62 to 0.83, greater than the values of factor a interpreted in the current test.

For end bearing in Thanet Sand, Troughton and Platis reported an Nq* value of 47 for base grouted piles. Due to the constraints placed on the pile design to limit movements, there was little scope to increase this value despite having measured a value of 68 in this pile test.

Calculated Pile Capacity

The capacities calculated using the original and proposed design approaches are summarised in Table 6.

Strata Description	Computed Capacities			Proposed Design Parameters
	Original Design Approach	Measured Loads	Proposed Design Approach	
Lambeth Clay	566kN	1,000kN	283kN	$\alpha = 0.2$
Lambeth Sand	793kN	2,000kN	1,420kN	$a = 0.4$ $\delta = \phi'$
Thanet Sand	915kN	3,000kN	2,376kN	$a = 0.6$ $\delta = \phi'$
SHAFT Q_s	2,274kN	6,000kN	4,361kN	
BASE Q_b	10,340kN	15,000kN	10,340kN	Nq* = 47
TOTAL Q_{ult}	12,614kN	21,000kN	14,701kN	
Design Capacity $Q_{ult}/2.25$	5,606kN	9,333kN	6,534kN	

Table 6 : Pile Capacities Calculated Using the Proposed Design Approach

It should be noted that the calculated design capacity of 6,534kN divided by the measured ultimate capacity of 21,000kN gives a P/P_{ult} ratio of 0.31. This design method therefore complies with the P/P_{ult} ratio of 0.32 that was chosen to control pile movements.

The pre-trial test design capacity of 5,606kN corresponds to a P/P_{ult} value of 0.27. This is equal to $\Delta/D = 0.45\%$ and would indicate that the expected settlement of a typical 1.5m diameter pile would be 7mm and that a settlement limit of 8.5mm should have been applied at working load in a contract load test.

Conclusions

The pile test worked well, indeed the actual capacity was greater than the generous reaction system provided. The successful outcome of the test clearly demonstrated the appropriateness of the proposed design method in the particular situation and allowed a reduced factor of safety to be used.

The data demonstrates that in situations where the ground level has been lowered, horizontal stresses can be used to calculate economical pile lengths. The method used is also remarkable for its explicit consideration of pile movements in the derivation of pile parameters.

Acknowledgements

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