

The effects of changes in effective stress on a base grouted pile in sand

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Large diameter bored piles with grouted bases have been adopted for the foundations of the majority of the buildings at Canary Wharf in London's Docklands, including a deep basement car park. The piles are founded in the Thanet Sand which forms part of London's aquifer where water levels are reported to be slowly rising. A special pile test has been carried out to determine the effects of both the basement excavation and a possible long term rise in the groundwater level on the performance of base grouted piles. These effects were simulated by injecting water around the pile toe and thus reducing the vertical effective stress.

In this paper the test layout and procedures are described and the results are presented. The test results showed that the reduction in pile base resistance under increased water pressures was less than predicted by conventional design methods based on vertical effective stress. The end bearing resistance was found to be proportional to the mean effective stress.

INTRODUCTION

A deep basement car park is being constructed at Canary Wharf in London's Docklands (Figure 1) as part of the infrastructure for the development. The proposed basement is 8.5m deep and will be founded on base grouted bored piles penetrating into the Thanet Sand (Figure 2).

The Thanet Sand is underlain by the Upper Chalk stratum, both of which form the aquifer beneath London. Simpson, Lance and Wilkinson (1988) have reported that the water level in this aquifer has been depressed by pumping and that in the last 20 years it has steadily been rising due to a reduction in rates of abstraction.

Conventional design methods show that changes in vertical effective stress would affect the bearing capacity and performance of piled foundations. On this site there are two potential causes of change in the vertical effective stress which are the reduction in overburden due to the proposed basement excavation and a possible rise in the groundwater level in the aquifer. Armishaw and Cox (1979) have previously investigated the effects of pore water pressure changes on the bearing capacity and settlement of driven piles in sand and gravel. They found that the reduction in bearing capacity and increase in settlement under increased water pressures was less than would be predicted by conventional design methods. This was considered likely to be due to the locked in prestress effects of driving the piles. In the absence of definitive test data for bored piles it was decided to investigate these effects for the base grouted bored piles proposed at Canary Wharf.

A pile loading test, specifically designed to investigate the effects of the reduction in overburden and any long term rise in groundwater level in the aquifer on the end bearing capacity and performance of base grouted bored piles, was performed at Canary Wharf (Figure 1) between August and November 1987. The above effects were simulated by injecting water under pressure in the aquifer through five injection wells installed around the pile and extending beneath its base. The pore water pressures beneath and around the base of the pile, as well as away from it, were monitored by five standpipe piezometers, so that the effective stress conditions at the pile base could be controlled at any stage of the test. The test was performed under three effective stress conditions, interchanged sequentially during the loading process, to obtain the corresponding load/displacement characteristics.

In order to investigate the performance of the pile base, the shaft resistance was minimised by sleeving the pile over a substantial portion of

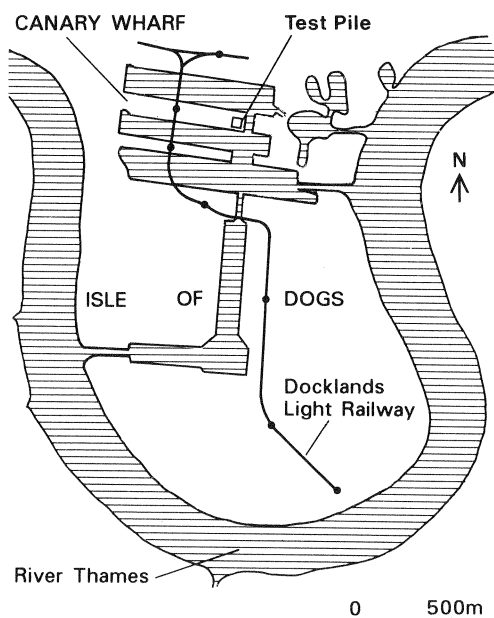


FIGURE 1 SITE LOCATION

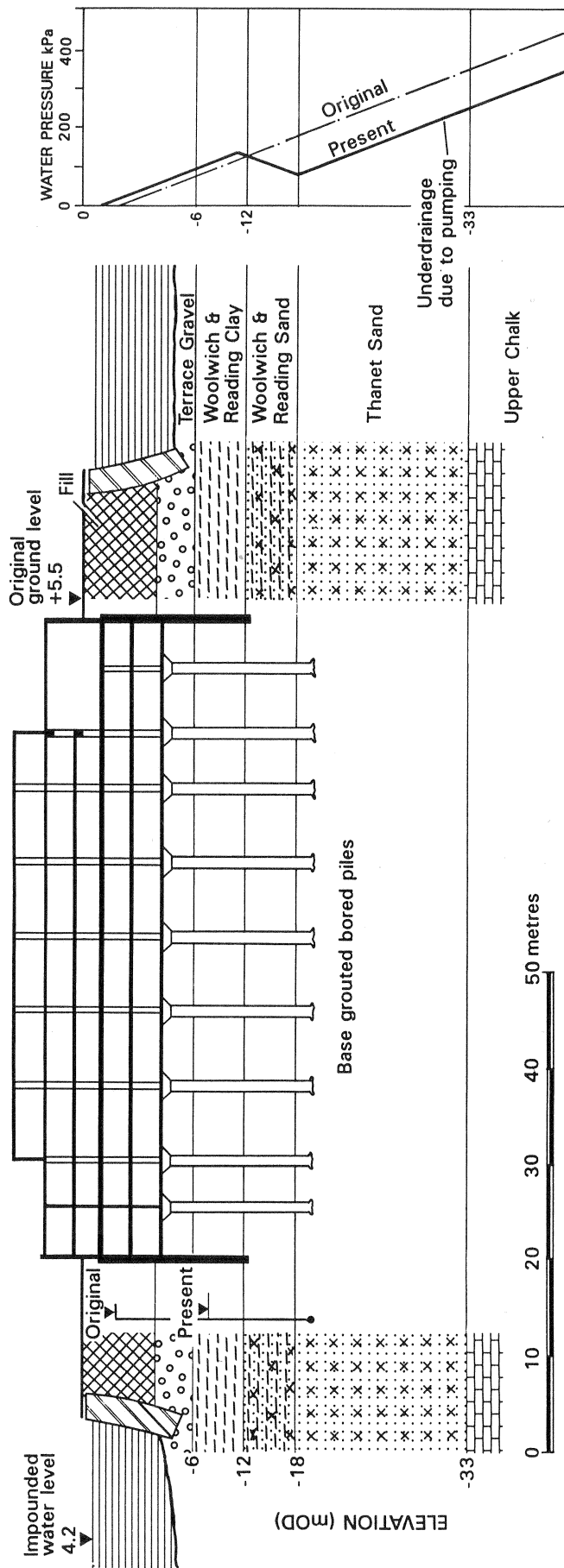


FIGURE 2 SECTION THROUGH BASEMENT CAR PARK

its length. In addition, six vibrating wire strain gauges and six rod extensometers were installed to monitor the pile's base load/displacement response during the test.

In this paper the test layout and procedures are described and the results of the pile loading test are presented. The test results are discussed in relation to the design of the base grouted bored piles under deep basements at Canary Wharf, taking into account the long term effects of rising groundwater level.

THEORETICAL BACKGROUND

Effects of Ground Stress on Pile Performance

The purpose of the pile loading test was to examine the base response of a base grouted pile founded in the Thanet Sand.

The base response, that is the load/displacement behaviour, is dependent on the state of stress in the ground around the pile. This can be described in terms of two components:

$$\begin{aligned}\sigma'_v &= \sigma_v - u, \text{ the vertical effective stress} \\ \sigma'_h &= \sigma_h - u, \text{ the horizontal effective stress}\end{aligned}$$

where σ'_v and σ'_h are the total vertical and horizontal stress respectively and u is the water pressure, which acts equally in all directions.

The ability of the pile base to carry load is known to be dependent on both σ'_v and σ'_h , but the precise form of this dependence is uncertain. In addition, whilst the value of σ'_v is fairly easy to determine, σ'_h is much more difficult. Both the formation of the excavation and a future rise in groundwater level will lead to reductions in σ'_v , σ'_h and, thereby, in the piles' ability to resist load. Only by load testing could this change in the pile's performance be quantified.

Effects of Excavation and Change of Water Pressure on Ground Stress

As a result of an excavation of about 8.5m for the car park, this will cause a reduction in vertical total stress, σ_v . In turn the vertical effective stress will reduce so that:

$$\sigma'_{v \text{ car park}} = \sigma_v - \Delta\sigma_v - u$$

Thus the change in the vertical effective stress is equal to the change in total stress caused by the car park excavation. A reduction in vertical effective stress is accompanied by a proportional reduction in horizontal effective stress.

As a result of the long term rise of water pressure in the Thanet Sand, the water pressure around the pile base will increase by an amount Δu

$$\sigma'_{v \text{ car park long term}} = \sigma_v - \Delta\sigma_v - (u + \Delta u)$$

Again, the horizontal effective stress will reduce in proportion.

The reduction in vertical effective stress around the base is equal to the sum ($\Delta\sigma_v + \Delta u$), the twin effects of the car park excavation and rise in water pressure respectively.

Whilst the vertical effective stress at all stages during and after construction can be estimated simply, the horizontal effective stress cannot. However, for relatively small changes in vertical stress, it is likely that the change in horizontal effective stress is roughly proportional to the change in vertical effective stress. Although the constant of proportionality is not known with precision, so long as the vertical effective stress is reduced, the horizontal effective stress will be reduced in proportion.

Simulation of the Effects of Excavation and Rising Water Levels by Controlling Water Pressures

It is not necessary to simulate the basement excavation by an excavation. Both the effects summarised above could be simulated around the test pile by increasing the water pressure by an amount equal to ($\Delta\sigma + \Delta u$) close to the pile base.

Accordingly, a pile loading test programme was designed so that the change of σ'_v and σ'_h could properly be assessed, and its effect on base response quantified. The reduction in σ'_v is assessed to be about 200kPa, compared with a current value of σ'_v of about 370kPa at the location of the test pile. It was intended to increase the water pressure around the pile base in two steps by 150kPa and 50kPa respectively to simulate the twin effects of basement excavation and water pressure rise. In the event slightly smaller increases in water pressure, of 125kPa and 50kPa, were achieved.

GROUND CONDITIONS

The soil profile at the test pile location consists typically of 6m of Fill and Alluvium and 6m of Terrace Gravel overlying 7m of Woolwich and Reading Clay and 6m of Woolwich and Reading Sand above the Thanet Sand. The Thanet Sand is a very dense uniform silty fine sand with Standard Penetration Test blowcounts in excess of 100. Particle size distribution analyses indicate a silt content of between 5% and 30%.

During excavation of the test pile the top of the Thanet Sand was encountered at approximately -21.2mOD and it was overlain by an approximately 1.3m thick layer of greenish grey fine silty sand with occasional flint gravel and shell fragments. The strata near the founding level of the test pile are shown on the pile section in Figure 3.

The piezometric water level in the Thanet Sand, at the test pile site, was measured in the standpipe piezometers to be at approximately -5.1mOD. The piezometric level profile in the Thanet Sands rises from about -7.5mOD at the western end of the Canary Wharf site to about -4.0mOD at the eastern end. This is due to underdrainage in the Chalk below the Thanet Sand.

PILE INSTALLATION

The test pile was located at the east end of Canary Wharf. The pile was sleeved over a portion of its length to eliminate shaft resistance over the sleeved length, as shown on Figure 3. The pile was double sleeved using two casings. The annulus between the two casings was filled with bentonite drilling fluid. Three 50mm diameter vertical steel tubes, attached on the inner casing, were used as spacers to prevent excessive horizontal movements of the pile head and potential buckling.

The test pile was augered under bentonite beneath the outer casing (1350mm in diameter, 21.5m long). A flat bottom slurry bucket was used to remove the last 300mm of soil, reaching thus the final pile toe level of -22.3mOD.

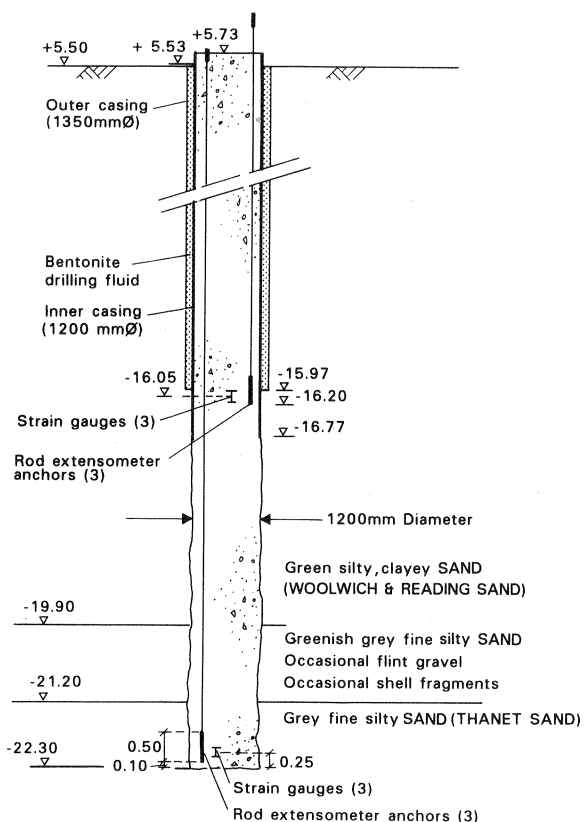


FIGURE 3 TRIAL PILE DETAILS

Four grout circuits were attached to the reinforcing cage prior to its installation, to facilitate grouting of the pile base. The arrangement of the grouting tubes is shown in Figure 4. Twenty four hours after concreting, the tube a manchettes were opened with water under pressure to ensure there was no blockage when the pile was grouted. Eleven days after concreting the pile was base grouted. The grout mix consisted of a cement to water ratio 5:3. The applied maximum grouting pressure was 50 bars and the total injected grout volume was 258 litres. During base grouting the pile uplift was monitored with three level points and a dial gauge on the pile head. The strain gauges and rod extensometers were also monitored during the

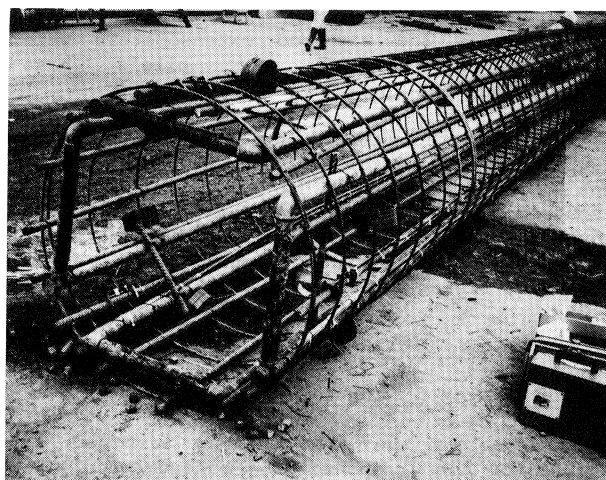


FIGURE 4 ARRANGEMENT OF GROUT TUBES

grouting. The average measured pile top uplift was 1.2mm and the average elastic shortening of the unsleeved length of the pile was 0.4mm (ie. the average pile base uplift was 1.6mm).

Based on the measured average elastic shortening of the pile during base grouting and the Young's modulus of the concrete determined during the pile test, allowing for the age of concrete at the time of grouting, the average base stress during grouting was estimated to be 4300kN/m². Since the maximum applied grouting pressure at the base is estimated to be 5400kN/m² (50 bars plus the additional pressure head from the pump level to the base of the pile), then the maximum pressure could not have been applied to more than $(4300/5400) \times 100 = 80\%$ of the base area at any given time.

The maximum applied grouting pressure of 50 bars did not apparently cause any fracturing of the ground, judging from the amount and rate of grout intake. The pumped volume of 258 litres indicates that if all the grout remained beneath the base it would have an average thickness of approximately 228mm. This would require an unrealistic amount of compression of the ground beneath the pile base (the base uplift was only 1.6mm). Consequently the grout possibly moves outside the base by compressing the ground, by working its way through weak zones or even by moving upwards along the pile shaft/ground interface.

WELL AND PIEZOMETER INSTALLATION

Four injection wells for pumping water under pressure into the aquifer were installed around the test pile, at a distance of 4.0m from its centre, as shown on Figure 5. Well C1 was replaced by Well C2, which was located 2.7m away from the centre of the pile. The five injection wells extended 7.0m into the Thanet Sands.

Three standpipe piezometers (1 to 3) were installed around the test pile at 120° angles, at a distance of 1.5m from its centre, in order to monitor the pore water pressures around the pile base (see Figure 5). Another two standpipe piezometers (4 and 5) were installed at 4.15m and 19.80m away from the centre of the pile, to

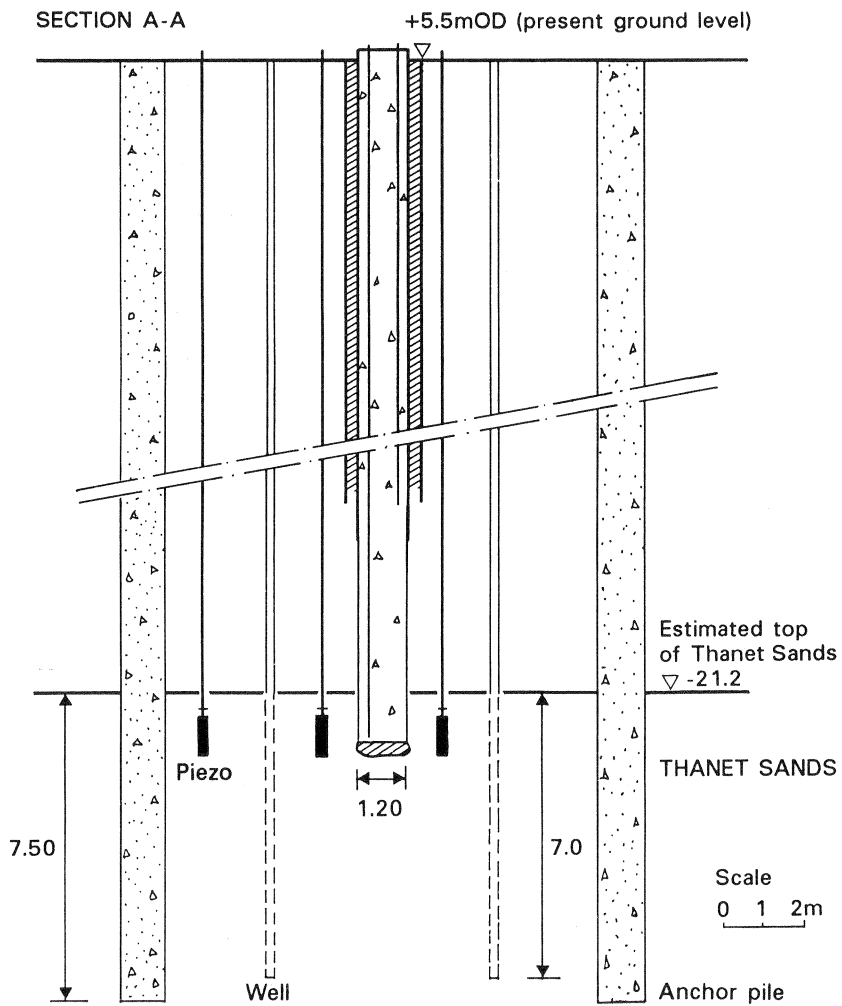
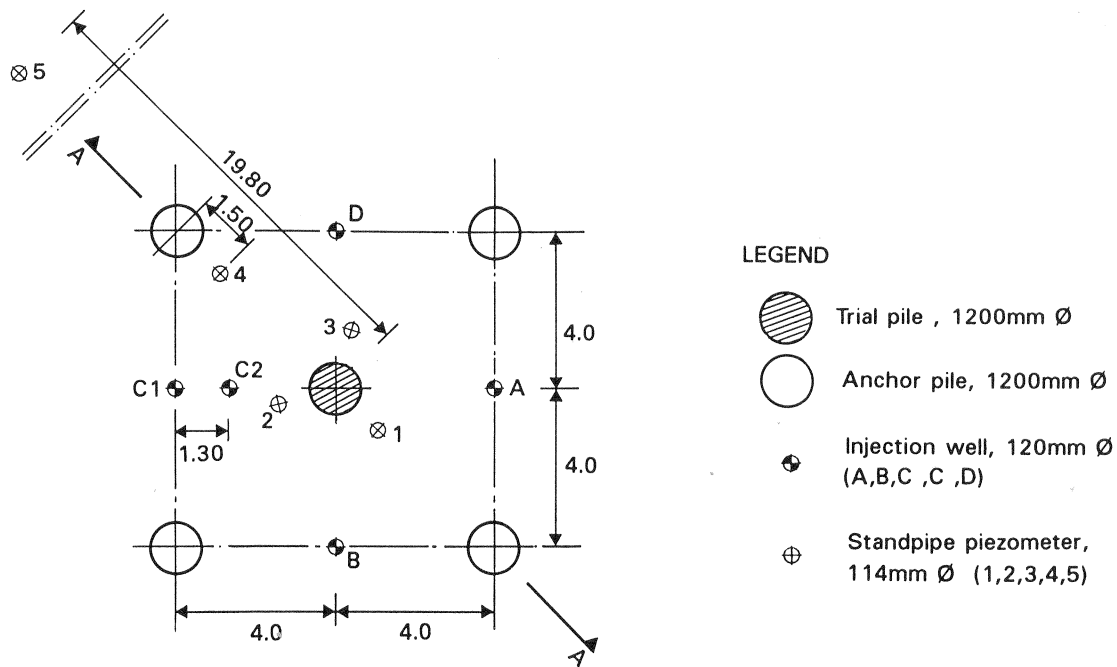


FIGURE 5 LOADING TEST ARRANGEMENT

monitor the excess water head dissipation away from it (see Figure 5). Their response zone was located at the pile base level.

All the injection wells were tested individually and in groups before the pile load test. Generally a 50% head loss was observed between the wells and the piezometers around the pile. It was also observed that generally 85% to 95% of the target excess water pressure around the pile was achieved in less than 15 minutes. A minimum time interval of two hours was required for pore water pressures to reach equilibrium around the pile.

The calculated horizontal permeability of the Thanet Sands from the well test results is of the order of 10^{-5} m/sec. The above permeability seems rather high for a fine silty sand, with a silt content between 5% to 30%. However, if thin horizontal layers of clean sand exist within the Thanet Sands, then the above value is quite reasonable.

LOAD TEST ARRANGEMENT

The required reaction was provided by four anchor piles, 1.2m in diameter, extending 34.0m to 34.5m below ground level (ie. between -29.5mOD and -29.0mOD). The anchor piles were constructed under bentonite and were installed at a distance of approximately 5 pile diameters away from the centre of the test pile. The test load was applied to the test pile using 4 hydraulic jacks of 6.5MN capacity each. The maximum travel of the jacks' ram was 120mm. The load was measured using 4 strain gauge load cells incorporated in the jacks.

The instrumentation consisted of:

- 4 dial gauges of 50mm travel graduated in 0.01mm divisions, for measuring pile head displacements.
- 6 plastic rod extensometers.

Three of the extensometers extended to the bottom of the sleeved length of the pile (top of the anchor at -15.7mOD - see Figure 3) and the other 3 extended to the pile base (top of the anchor at -21.7mOD - see Figure 3).

- 6 vibrating wire strain gauges attached on the reinforcement.

Three of the strain gauges were installed at the bottom of the sleeved length of the pile (at -16.05mOD - see Figure 3) and the other three 250mm above the base of the pile (at -22.05mOD - see Figure 3). The strain gauges were placed at 120° around the pile centre.

- 11 levelling points. Two precise levels with micrometer attachments (readings to 0.1mm) were used. The levelling points consisted of:
 - 2 levelling points on the pile head
 - 1 levelling point on the outer casing
 - 4 levelling points on the anchor piles, one per pile
 - 4 levelling points on the reference frame.

- Air temperature was also measured throughout the pile test with a minimum/maximum thermometer.

PILE LOADING TEST PROCEDURE

A loading test procedure was established such that the load/displacement characteristics of the test pile under three different effective stress conditions could be obtained.

The test pile was loaded in steps under present effective stress conditions. At regular intervals, the pore water pressure around the pile base was increased in two steps by 175kPa (first by 125kPa and then by 50kPa), by injecting water through the wells. In the early stages of the test, when settlements were small, the above procedure of water injection was carried out while the load on the top of the pile was maintained constant and pile top settlements were monitored. In later stages of the test however, when pile settlements became large, the top load was not maintained, but was allowed to drop, while the pore pressures were increased and the pile settled.

The first water pressure increase at 10,000kN was designed to give information on the long term settlements at service load of base grouted piles bearing in the Thanet Sands, as a result of a rise in the groundwater level in the aquifer.

Each increment of load, including the zero load, as well as each water injection step, was maintained until the rate of settlement did not exceed 0.05mm in half an hour measured over a half hour interval. Each decrement of load was held for between 15 to 60 minutes.

The loading test was continued 24 hours a day for a total duration of 17 days and 12 hours.

TEST RESULTS

The settlements recorded by the 4 dial gauges were corrected by an amount equal to the average of the reference frame movement. The corrected load/displacement path is plotted on Figure 6.

The corrected pile head displacements at the nominal service load of 10,000kN for the three water pressures are summarised in Table 1 below:

Table 1 Pile settlement at service load

Excess Water Pressure (kPa)	Corrected Pile Head Settlement (mm)
0 (present condition)	9.8
125 (basement)	10.6
175 (basement + rise in water level)	11.3

The additional pile settlement due to the long term rise in the water level in the aquifer was therefore found to be less than 1mm.

The individual load/displacement curves for the three water pressures that were created around the pile base during the test are plotted in Figure 7. The curve for each water pressure has been deduced from Figure 6, by subtracting the settlements which occurred whilst the pile base was subject to higher water pressure. It can be seen in Figure 7 that the derived curves pass approximately through the sets of data points obtained at constant water pressure (such as ABC

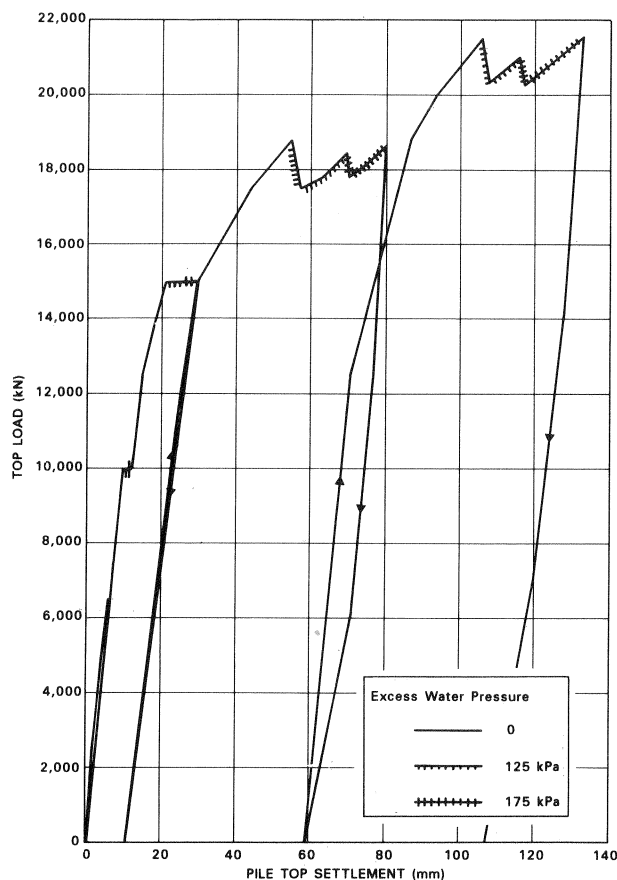


FIGURE 6 LOAD VS. CORRECTED TOP SETTLEMENT PATH in Figure 7). Without the correction described here, this was not the case.

ANALYSIS OF RESULTS

In order to derive the base resistance from the results of the test pile it is necessary to determine the shaft resistance mobilised during the test. The shaft resistance (Q_s) can be calculated by deriving the base resistance from the base strain gauges and subtracting this from the applied load as follows:

$$Q_s = Q - q_b \cdot A_b$$

where Q = applied load

$q_b = \epsilon_b \cdot E_{RC}$ = base stress

ϵ_b = average base strain from base strain gauges

E_{RC} = modulus of elasticity of unsleeved length of pile, deduced from the strain gauge data at the bottom of the double sleeved length of the pile.

The Young's moduli of the sleeved and unsleeved lengths of the pile were calculated from the rod extensometer and strain gauge measurements and are summarised in Table 2 below:

Table 2 Pile Young's Modulus

	Rod Extensometers	Strain Gauges
Sleeved length, E (kN/mm ²)	36.2	36.5
Unsleeved length, E_{RC} (kN/mm ²)	31.5	31.8

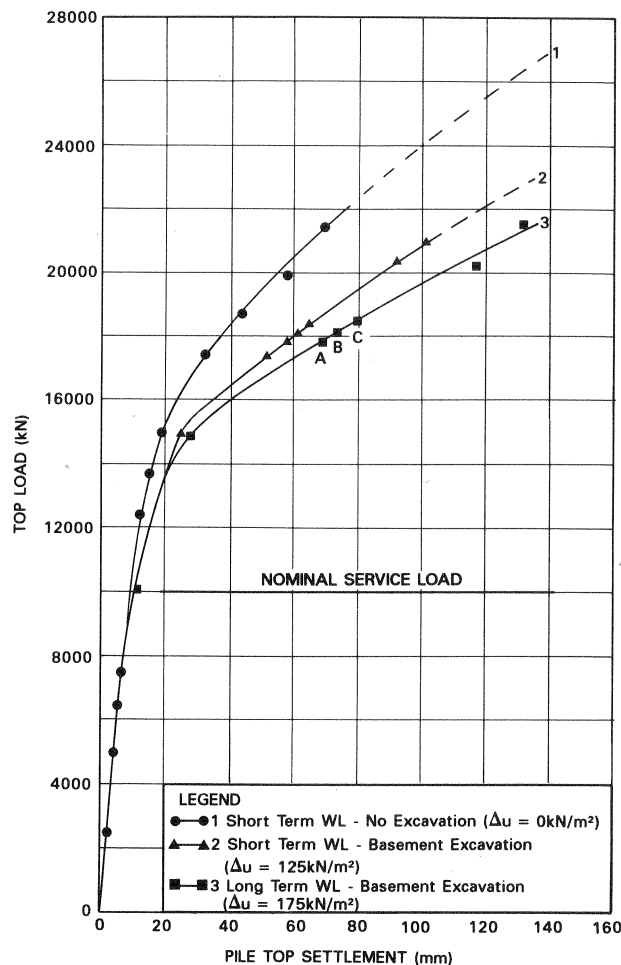


FIGURE 7 REVISED LOADS VS. TOP SETTLEMENT CURVES FOR VARIOUS EFFECTIVE STRESS CONDITIONS

The final shaft resistance of the soil over the unsleeved length of the pile was estimated to be 5,000kN under present effective stress conditions and to fully develop at a pile top load of 13,750kN. The ultimate shaft resistances for the reduced effective stress conditions have been estimated by assuming that the horizontal effective stress changes by:

$$\Delta\sigma'_h = \frac{\nu}{1 - \nu} \cdot \Delta\sigma'_v \text{ (no lateral strain assumed)}$$

where ν = Poisson's ratio

The estimated values of ultimate shaft resistance for the various excess water pressures are summarised in Table 3 below:

Table 3 Ultimate shaft resistance

Excess Water Pressure (kPa)	Ultimate Shaft Resistance (kN)
0 (present condition)	5,000
125 (basement)	4,535
175 (basement + rise in water level)	4,350

In Figure 8, the estimated base stress is plotted against base settlement for the three effective stress conditions. A linear increase of Q_s with pile top load has been assumed. The base settlement has been estimated using the Young's

modulus of the concrete and assuming a uniform development of shaft resistance over the unsleeved length of the pile.

From Figure 8 the ultimate base stress can be defined as the base stress at a base settlement equal to 10% of the pile diameter, ie. 120mm. The estimated ultimate base stresses ($ultq_b$) for the three effective stress conditions, and the corresponding effective overburden pressures (σ'_v) at the base, are summarised in Table 4 below:

Table 4 Ultimate base resistance

Excess Water Pressure (kPa)	σ'_v (kPa)	$ultq_b$ (kPa)
0 (present condition)	371	19,400
125 (basement)	246	16,450
175 (basement + rise in water level)	196	15,250

The variation of ultimate base resistance with vertical effective stress is plotted in Figure 9.

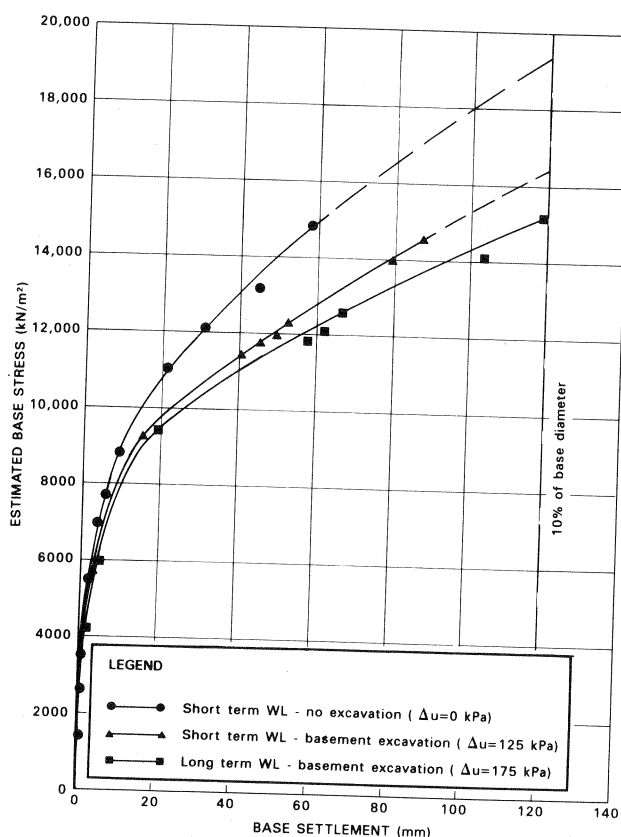


FIGURE 8 PILE END BEARING BEHAVIOUR FOR VARIOUS EFFECTIVE STRESS CONDITIONS

Over the vertical effective stress range of the test the two are related linearly:

$$\Delta q_b = 23.7 \Delta \sigma'_v$$

Although the relationship is linear the line does not pass through the origin, which is contrary to conventional design methods based on vertical effective stress. The ultimate end bearing

capacity of piles founded in sand is usually expressed in terms of vertical effective stress and the bearing capacity factor N_q as follows:

$$ultq_b = N_q \cdot \sigma'_v$$

Values of N_q derived by Berezhantzev et al (1961) are widely used and give a value of approximately 60 for an effective friction angle of 36° , which corresponds to the peak friction angle for the Thanet Sands determined from shear box tests on recompacted samples from the site. The above relationship is also plotted in Figure 9. It can be seen from Figure 9 that a conventional design approach would overestimate the end bearing capacity under present effective stress conditions and it would underestimate it for reduced effective overburden pressures (ie. after basement excavation and a long term water level rise in the aquifer).

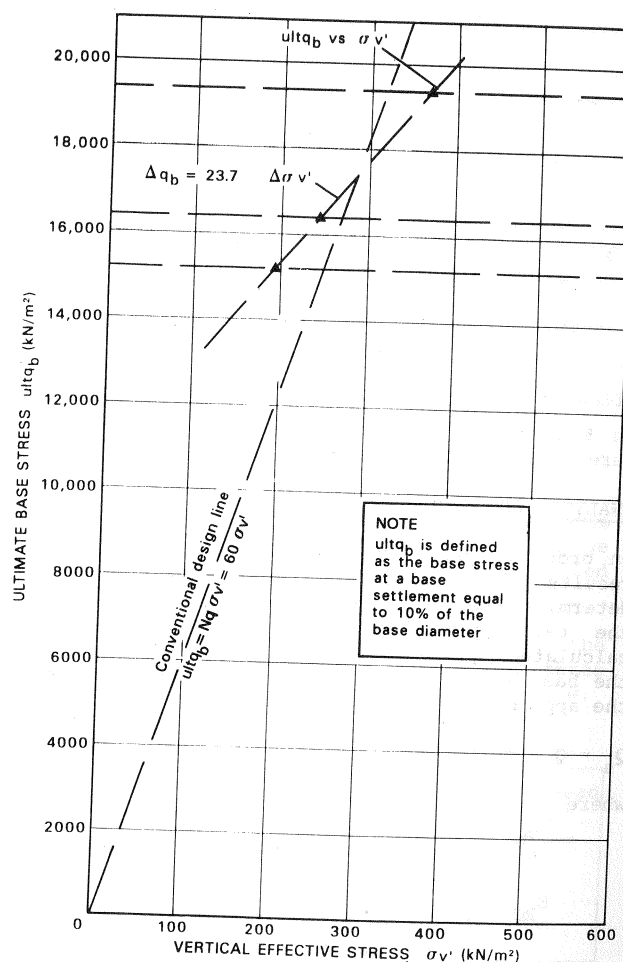


FIGURE 9 VARIATION OF ULTIMATE BASE STRESS WITH VERTICAL EFFECTIVE STRESS

An alternative design approach has been proposed by Vesic (1977). He considers the expansion of a cylindrical cavity around a highly compressed wedge to determine the ultimate end bearing capacity, taking into account the volumetric strain in the plastic compression zone. Based on the above assumption he has related the end bearing capacity of sands not to the vertical but rather to the mean effective stress around the pile base, ie.

$$ultq_b = N_q^* \sigma'_m$$

$$\text{where } \sigma'_m = \frac{\sigma'_v + 2\sigma'_h}{3} = \frac{\sigma'_v + 2k_o \sigma'_v}{3} = \frac{1+2k_o}{3} \sigma'_v$$

σ'_h = horizontal effective stress

$k_o = \sigma'_h / \sigma'_v$ = the coefficient of earth pressure at rest

N_q^* = alternative bearing capacity factor

The ultimate base stresses obtained from the test are plotted against σ'_m in Figure 10. Assuming a k_o value of 1.15, the straight line passing through the three points passes also through the origin, giving an N_q^* value of 47. The value of $k_o = 1.15$ compares well with the value of $k = 1.2$ that was deduced from load tests on small displacement open ended steel tube piles in the Thanet Sands at Canary Wharf, for which $k \approx k_o$.

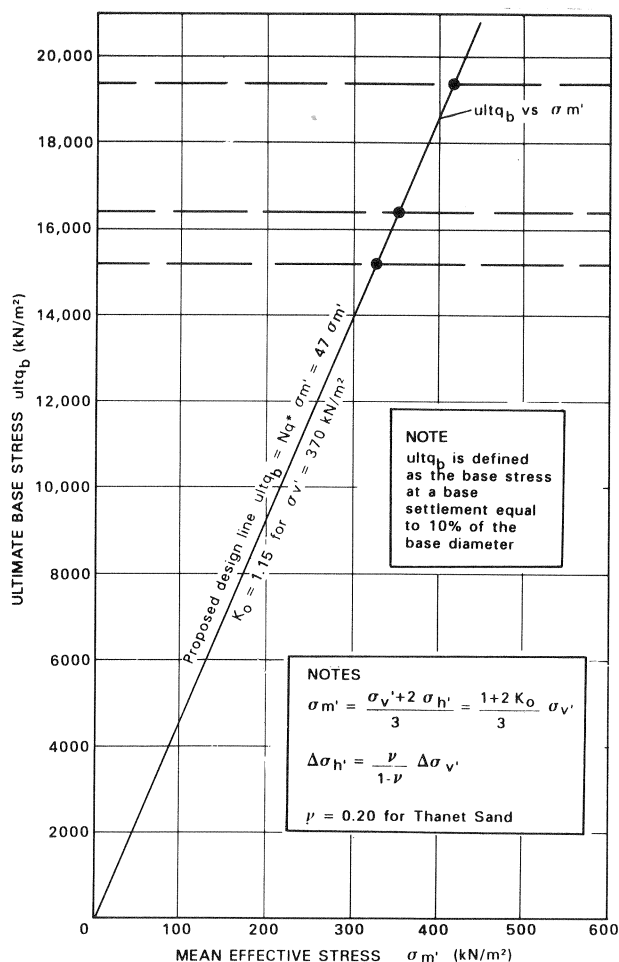


FIGURE 10 VARIATION OF ULTIMATE BASE STRESS WITH MEAN EFFECTIVE STRESS

SUMMARY AND CONCLUSIONS

A pile loading test on a base grouted, sleeved pile was performed at Canary Wharf to investigate the end bearing behaviour of such piles bearing in the Thanet Sand under variable effective stress conditions.

During the test the effective stress at the pile base was controlled by injecting water under pressure around the pile base.

The additional pile settlement at service load due to the long term rise in the water level in the aquifer was less than 1mm.

Conventional design methods of predicting the ultimate base resistance of piles in sand are based on vertical effective stress. These methods would overestimate the ultimate base resistance of the test pile under present conditions and underestimate it for reduced effective stresses after basement excavation or a long term water level rise in the aquifer.

The ultimate base resistance of the test pile was found to be proportional to the mean effective stress as shown on Figure 10.

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