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Pile tests to justify higher adhesion factors in London Clay

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The London District Surveyors Association guidance notes for the design of straight shafted bored piles in London Clay recommend an adhesion factor of 0·5 for shaft friction pile design. This value, based on the back-analysis of many maintained load pile tests to failure in London Clay, was selected to cover all design scenarios irrespective of site location, pile construction method, pile geometry and whether or not pile load testing is carried out. This 'one size fits all' approach to pile design is potentially over conservative. For sites where maintained load tests (MLTs) to failure on representative preliminary test piles are carried out, the results can be back-analysed to determine the appropriate adhesion factor for the site, which may then be used to improve the efficiency of the pile design. This paper discusses the pile design philosophy, pile load testing and construction sequence for a large development in Whitechapel, London, where MLT results were used to justify an increased adhesion factor of 0·6. This increased adhesion factor ensured that piles were 'dry' bored and founded above the water-bearing strata of the Lambeth Group, leading to lower project costs and risks.

Notation

A_{base}	pile base area
A_{conc}	pile concrete cross-sectional area
A_{sc}	longitudinal steel cross-sectional area
A_{shaft}	pile shaft area
c_{u}	undrained shear strength
$c_{\mathrm{u\ ave}}$	average undrained shear strength
$c_{\mathrm{u\ base}}$	undrained shear strength at base of pile
E_{b}	soil elastic modulus at base
E_{c}	concrete elastic modulus
f_{cu}	characteristic concrete cube strength
f_{y}	yield strength of steel
k_{e}	friction centroid
k_{s}	lateral earth pressure coefficient
M_{s}	shaft flexibility factor
N_{c}	end bearing capacity factor
N_{ult}	ultimate compressive strength
Q_{b}	ultimate end bearing capacity
Q_{s}	ultimate shaft friction
α	adhesion factor for undrained soil
γ_{b}	bulk density of soil
δ	pile interface friction angle
$\sigma'_{\mathrm{ave\,v}}$	average effective vertical overburden stress
$\varnothing$	pile diameter
ϕ'	angle of friction of soil

1. Introduction

Aldgate Place is a £250 million British Land/Barratt London development in Whitechapel, east London, comprising several multi-storey buildings and publicly accessible open spaces (Figure 1). This large scheme will deliver 463 residential units, 3000 m^2 of commercial/retail space and a new hotel. The project includes four tower blocks and a two-storey basement, which cover 40% of the site. The site is approximately square in shape, spanning 100 m northwest to southeast and 80 m northeast to southwest. It is bounded by Whitechapel High Street to the north, Commercial Street (A13) to the east, Buckle Street to the south and Leman Street to the west. The approximate National Grid reference of the site is TQ 340 813, and it is approximately 900 m north of the River Thames. Figure 2 shows an aerial image of the site.

At 27 storeys, block F is the tallest tower and the foundations comprise 29 large-diameter rotary bored piles ranging from

Figure 1. Artist's impression of completed Aldgate Place development

Figure 2. Aerial image of Aldgate Place

750 mm to 1800 mm in diameter to support safe working loads (SWLs) ranging from 2·5 MN to 15·0 MN. Within the footprint of block F there were approximate 30 existing large-diameter under-ream piles from the previous structure that had occupied the site. It was not possible to reuse any of these piles for various reasons, however they made the site extremely congested and limited the number of new piles that could be installed and their locations. A further constraint was the maximum pile depth, which was limited to 42·5 m to prevent entering the water-bearing zone of the Lambeth Group, which would have led to deterioration of the pile bore and base.

The key aspects of the pile design and specification are summarised as

- axial pile capacity design in accordance with London District Surveyors Association guidance (LDSA, 2009) and BS 8004 (BSI, 1986)
- piles constructed in accordance with the ICE Specification for Piling and Embedded Retaining Walls (SPERW) (ICE, 2007).
- pile design to incorporate a geotechnical 'global' factor of safety (FOS) of 2·0 on shaft and base resistances, subject to successful completion of a preliminary test pile (PTP) to validate design parameters
- PTP undertaken in accordance with SPERW extended proof load test (ICE, 2007).

This paper reports on a maintained load test (MLT) to geotechnical failure at ~10 MN carried out on a 750 mm diameter × 37·0 m deep rotary bored PTP at Aldgate Place and the back-analysis of the results to justify an increased adhesion factor (α) of 0·6. This was then used to optimise the design of the main works piles.

2. Ground conditions

A site investigation was undertaken by RSA Geotechnics Limited between September and October 2013, which included

- four light cable percussion boreholes to 40 m depth (BH2 to BH5) well distributed across the site
- one light cable percussion borehole to 55 m depth (BH7)
- the installation of combined groundwater and gas monitoring wells within selected boreholes
- in situ standard penetration tests (SPTs) and laboratory undrained triaxial tests (100 mm) and Atterberg limit tests

The site investigation borehole layout plan is shown in Figure 3. The ground conditions are typical for London, comprising made ground (MG) overlying river terrace deposits (RTD), London Clay (LC) and the Lambeth Group (LG). The boreholes are plotted on a chart against elevation (mOD) in Figure 4. The design ground profile is preserved in Table 1.

Made ground is present between 3·9 mbgl (metres below ground level) and 6·3 mbgl (from piling platform level (PPL)) and typically comprises silty sand and gravel or gravelly sand, with subordinate inclusions of brick, block and concrete, and occasional ash, wood, clinker, tile, plastic, glass, shell, cable and textile. No axial pile capacity was taken from this stratum in the PTP design calculations because a permanent slip liner was installed and hence any shaft friction would be relatively negligible. The RTD extend to between 8·7 mbgl and 10·4 mbgl and comprise dense to very dense, brown, fine to coarse sand and gravel. SPT N values are generally shown to increase with depth, with values ranging from 29 to >50.

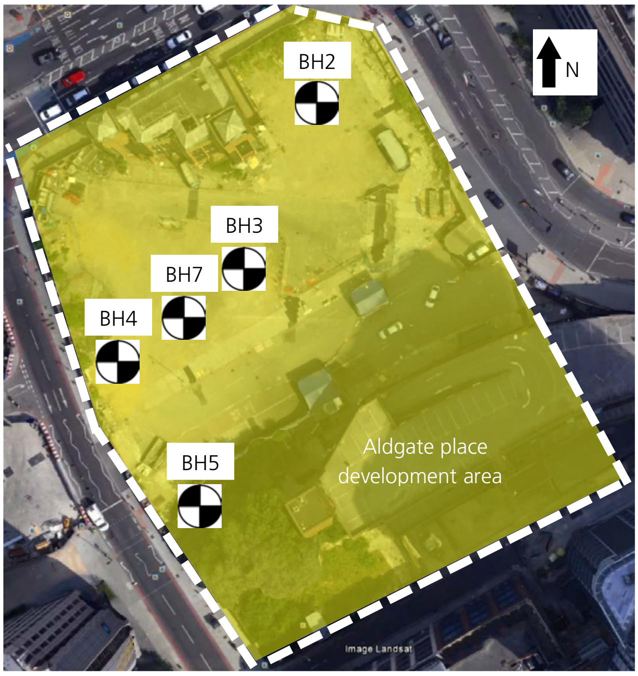

Figure 3. Site investigation borehole layout plan

The LC extends to depths of between 37·0 mbgl and 38·4 mbgl. It is typically described as stiff, becoming very stiff with depth, fissured or extremely closely fissured, silty clay with occasional partings of silt and fine sand, and occasional selenite crystals. A small number of claystone bands were encountered, which were penetrated with a chisel. Laboratory testing classified the LC as having very high plasticity with a high shrinkage potential. Four plasticity index (PI) determinations in the LC gave values ranging from 46% to 52%, with a mean value of 50%. SPT N values are generally shown to increase with depth, with values increasing from 13 to >50, indicating firm to very stiff conditions. A chart of undrained shear strength (c_u) versus elevation and the proposed design line is given in Figure 5. A correlation factor $f_1 = 5{\cdot}0$ was used to convert the SPT N values to estimated undrained shear strength (Stroud, 1974). The high PI values would typically indicate a slightly lower value ($f_1 = 4{\cdot}5$) to be appropriate; however, the chosen value gave a better fit with the triaxial test data. Below −10·0 mOD (metres Ordnance Datum), the correlation between c_u obtained from the triaxial tests and the SPT data becomes weaker and there is a greater scattering in the results. The c_u values determined from SPTs tend to increase above the observed trend exhibited in the LC below −10·0 mOD. A possible explanation for this is loss in hammer blow energy at depth, leading to an increased number of blows and higher SPT N values, or localised claystone bands. The triaxial test results also show a larger range of values at depth, which may be attributed to sample disturbance during extraction from the ground and/or transportation to the laboratory.

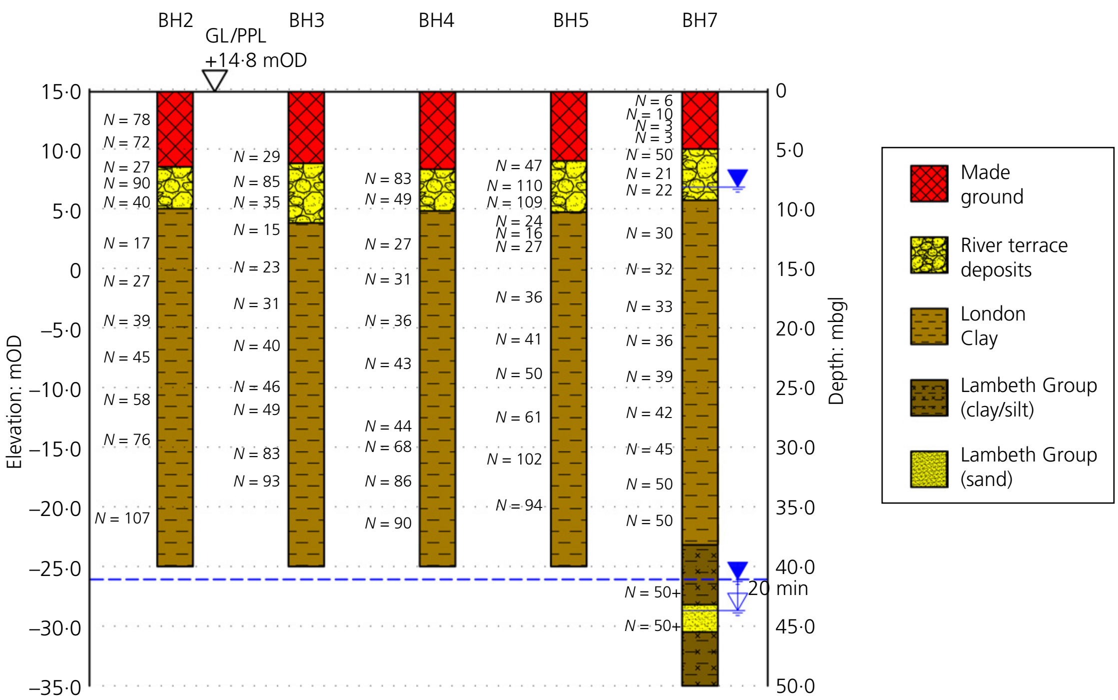

Figure 4. Borehole summary

Stratum	Elevation at top of stratum: mOD	Stratum thickness: m
Made ground (MG) (∼0·5 m thick piling platform included in stratum)	+14·8	6·3
River terrace deposits (RTD)	+8·5	3·5
London Clay (LC)	+5·0	29·0
Lambeth Group (LG)	−24·0	15·0 (base not proven)

Table 1. Design ground profile

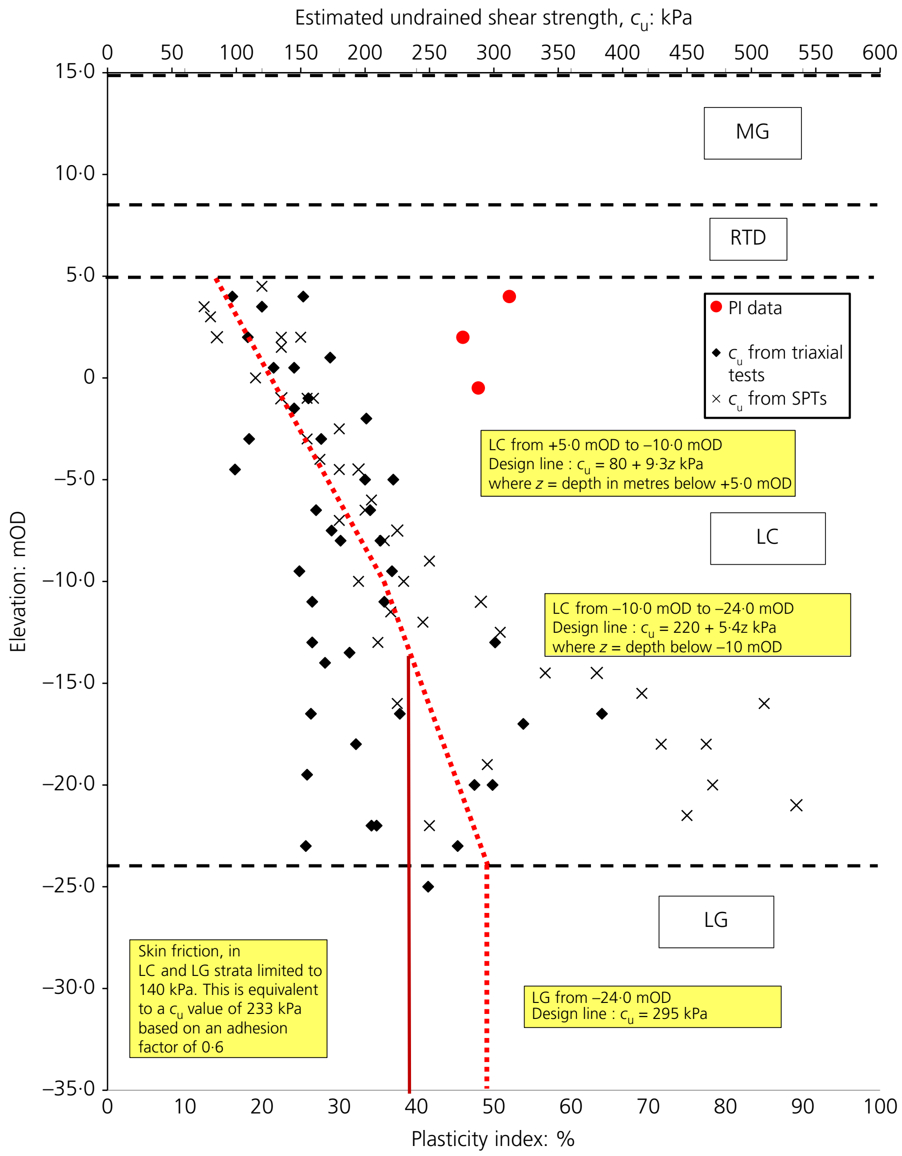

Figure 5. Undrained shear strength (c_u) and plasticity index (PI) plotted against elevation and the proposed design line

Stratum	Bulk density, γ_b: kN/m^3	Undrained cohesion, c_u: kPa	Adhesion factor, α	Angle of internal friction, ϕ': degrees	Lateral earth pressure coefficient, k_s	Bearing capacity factor, N_c
MG	18·0	0	N/A	N/A	N/A	N/A
RTD	20·0	N/A	N/A	37	0·8	N/A
LC	20·0	5·0 mOD to −10·0 mOD: 80 + 9·3/m −10·0 mOD to −24·0 mOD: 220 + 5·4/m	0·60 (assumed to determine maximum test load)	N/A	N/A	9
LG	20·0	295		N/A	N/A	9

Table 2. Axial bearing pile design parameters

The LG was encountered beneath the LC in all boreholes and is described as very stiff, friable, mottled brown, grey, grey–brown, blue–grey and orange–brown silty clay. The deepest borehole (BH7) proved the LG to 50 m depth. Triaxial test results within the upper and lower silt lenses gave c_u values of 58 kPa and 91 kPa, respectively, which are below the proposed design line; however, this is likely explained by sample disturbance along silt partings. Due to lack of SPT/triaxial test data in this layer it was assumed that the in situ shear strength was at least equivalent to the LC above.

The identification of groundwater seepages within the RTD during drilling was obscured by the need to add water to penetrate these dense granular deposits. No seepages were recorded within the LC. Long-term monitoring data suggest that the groundwater level is towards the base of the RTD, and a design groundwater level of +5·70 mOD (i.e. 0·7 m above the base of the RTD) was used in the design.

3. Pile design

The geotechnical design parameters detailed in Table 2 are based upon exploratory borehole records, in situ and laboratory test data, technical literature and experience.

The ultimate shaft friction (Q_s) in the cohesionless RTD was calculated based on an effective stress approach, taking $\phi' = \delta$

1. $$Q_s = A_{\text{shaft}} \cdot k_s \cdot \sigma'_{\text{ave v}} \cdot \tan\delta$$

The Q_s in the cohesive LC and LG was calculated based on a total stress approach, where

2. $$Q_s = A_{\text{shaft}} \cdot \alpha \cdot c_{\text{u ave}}$$

The theoretical ultimate shaft friction was limited to a maximum of 140 kPa, with a limiting mean value of 110 kPa through the cohesive LC and LG, in accordance with LDSA (2009).

It was a specific requirement that all rotary bored piles must terminate in the dry stable LC/LG above the water-bearing granular layer at 43·5 m depth, encountered in BH7 and confirmed by a deeper anchor pile trial bore.

The theoretical ultimate end bearing capacity (Q_b) was calculated based on the undrained shear strength and the end bearing capacity factor $N_c = 9$ as

3. $$Q_b = A_{\text{base}} \cdot N_c \cdot c_{\text{u base}}$$

The piles were designed in accordance with BS 8004 (BSI, 1986) and LDSA (2009) (as specified by the engineer) with an overall (global) geotechnical FOS of 2·0, subject to completion of a satisfactory PTP. A further serviceability design check was made to ensure that there was a minimum FOS of 1·2 on the ultimate shaft capacity alone, which should ensure minimal pile settlement at SWL.

A particularly important part of the design was to make sure that the pile concrete was poured as soon as possible after completion of boring to depth (i.e. within 2 h maximum) in order to mitigate deterioration of the pile shaft and base.

The final pile lengths were in the range 25·0 m to 42·5 m depth below piling platform level (i.e. just above the water-bearing zone of the LG), with a 'global' FOS of 2·0 and an adhesion factor of 0·6.

4. Preliminary test pile – design, construction and setup

The PTP was a 750 mm diameter × 37·0 m deep rotary bored pile, which was representative of the majority of works piles across the site. The theoretical ultimate geotechnical capacity of the pile was calculated to be 9·0 MN, of which 7·9 MN was provided by shaft friction (using an adhesion factor of 0·6 in the LC) and 1·1 MN provided in end bearing. Therefore, the equivalent SWL of the PTP was 4·5 MN based on a geotechnical FOS of 2·0. The proposed maximum test load was 10 MN (2·22 × SWL), which exceeds the theoretical ultimate

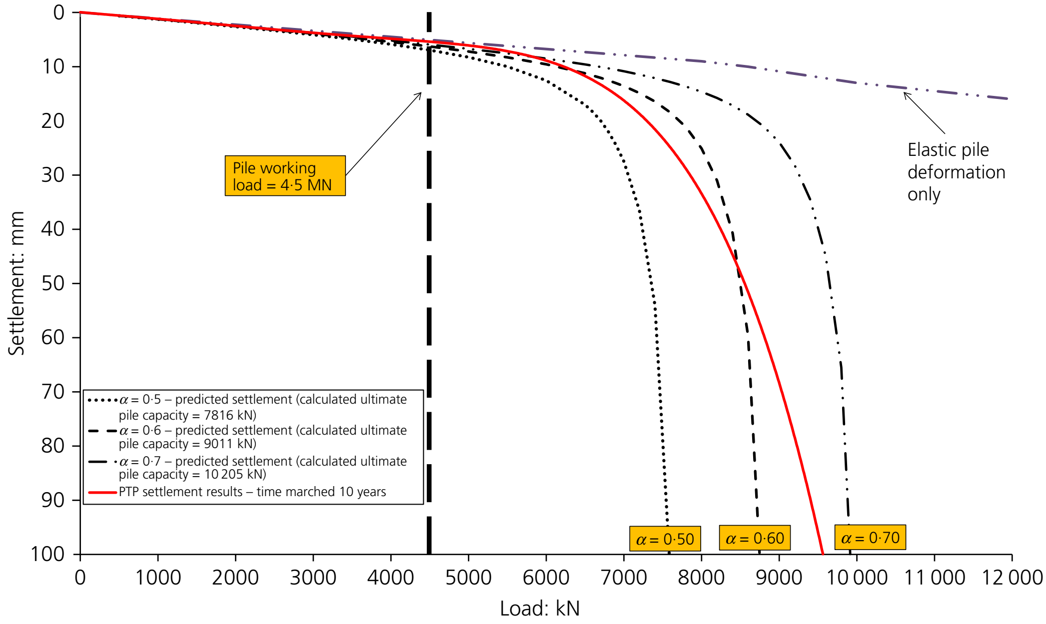

Figure 6. Loadtest-predicted settlement under load with London Clay adhesion factor (α) varied and PTP settlement data time-marched 10 years

geotechnical capacity of the pile and therefore 'failure' of the PTP should have occurred. There are several definitions of pile failure; one of the most common methods was selected here, which is when the pile head settlement had reached 10% of the pile diameter (i.e. 75 mm).

The PTP was heavily reinforced over its upper 12 m with (12 × B32) longitudinal bars inside B20 circular links at 250 mm centres to prevent premature structural failure. Then, from 12 m to 20 m depth, the cage was reduced to (12 × B25) longitudinal bars inside B16 links at 250 mm centres. The remaining 17 m of the pile was unreinforced because it was confined within very stiff clay and much of the applied force had already been transferred into the ground. The pile was formed using C32/40 grade concrete. The ultimate compressive structural capacity of the PTP (N_{ult}) was calculated in accordance with BS 8110-1 (BSI, 1997) as

$$4. \quad N_{\mathrm{ult}} = 0{\cdot}40 f_{\mathrm{cu}} \cdot A_{\mathrm{conc}} + 0{\cdot}75 A_{\mathrm{sc}} \cdot f_{\mathrm{y}}$$

N_{ult} was calculated to be 10·5 MN over the upper 12 m, 9·2 MN from 12–20 m depth and 7·1 MN for the unreinforced concrete section below 20 m. At the maximum test load of 10 MN, there was sufficient load transfer into the ground through shaft friction to ensure that the PTP was not structurally overstressed at any depth.

The test setup comprised a load cell and load transfer frame with four anchor piles at 4·6∅ away from the test pile, rated up to a maximum test load of 10 MN. The anchor piles were 750 mm in diameter with a design length of 28·5 m to provide a geotechnical FOS of 2·0 in tension at the maximum test load of 10 MN. Each anchor pile was reinforced to the full depth with four 36 mm diameter high-yield prestressing threadbars, providing an allowable tensile capacity of 2·5 MN per anchor pile.

5. Pile test results and discussion

The pile test was carried out in (ESG, 2014) accordance with SPERW (ICE, 2007) and was subject to load cycles up to 100%, 150%, 200% and 222% of the SWL (see Figure 6). On the third cycle, the pile was loaded up to 9 MN, where an initial pile head settlement of 40·6 mm was recorded. This load was then held for 40 min, during which time the pile settlement crept to 48·0 mm. On the fourth cycle, the pile was loaded to 10 MN and held for 80 min. During this period, the recorded settlement increased from 67 mm to 77 mm (at a diminishing rate, which dropped from 11·41 mm/h to 4·41 mm/h over the last hour). At this point, geotechnical failure was deemed to have occurred and the PTP was unloaded.

When load is applied to a pile it will settle due to a combination of its own internal elastic compression and also due to the elastic deformation and/or creep consolidation of the surrounding soil. Provided that the ultimate capacity of the pile has not been exceeded, the settlement rate will diminish and converge towards zero over time, as the settlement approaches a steady-state value. At higher loads it could take many months or years for the steady-state settlement to be reached. During a standard SPERW (ICE, 2007) MLT there are two 6 h hold periods at $1{\cdot}0 \times$ SWL and $1{\cdot}5 \times$ SWL. The hold periods at higher loads are shorter and are often insufficient for the pile to achieve the required acceptance criteria of $<0{\cdot}10$ mm/h and reducing for <10 mm pile head settlement and $<0{\cdot}24$ mm/h and reducing for pile head settlements of >24 mm. However, modern testing equipment provides very accurate load versus settlement readings to 0·01 mm accuracy at 1–5 min intervals. It is therefore possible to ‘time-march’ these data on a simple spreadsheet to give predicted settlements after 10 years. Any settlement after this length of time will be negligible. At higher loads in clay strata it is important to time-march data in order to assess the ‘loading creep’ and give a realistic long-term load versus settlement relationship. Table 3 presents the time-marched data for the MLT.

The computer program Loadtest, which adopts Fleming’s method of single pile settlement prediction (Fleming, 1992), was used to determine the theoretical load versus predicted settlement curves for the PTP based on three different adhesion factors by adjusting the input Q_s value using $\alpha = 0{\cdot}5$ ($Q_s = 6681$ kN), $\alpha = 0{\cdot}6$ ($Q_s = 7876$ kN) and $\alpha = 0{\cdot}7$ ($Q_s = 9070$ kN). Table 4 lists the Loadtest input parameters, which remained constant in each analysis.

The settlement data from the MLT and the three Loadtest-predicted load versus settlement curves (for $\alpha = 0{\cdot}5$, $\alpha = 0{\cdot}6$ and $\alpha = 0{\cdot}7$) are plotted on the same graph in Figure 6. It can be seen from the graph that, from zero load up to the pile working load of 4·5 MN, all the curves exhibit similar settlement profiles, which over this range is mostly elastic deformation of the pile itself. Beyond the working load, the three Loadtest-predicted curves follow distinctly separate hyperbolic curves as the load approaches the theoretical ultimate capacities of the pile for adhesion factors of 0·5, 0·6 and 0·7, respectively.

The time-marched MLT load versus settlement curve closely matches the $\alpha = 0{\cdot}6$ and $\alpha = 0{\cdot}7$ predicted settlement lines up to $\sim 1{\cdot}5 \times$ working load; beyond this point, the data show closest correlation to the $\alpha = 0{\cdot}6$ curve, which is therefore deemed appropriate. Furthermore, it can be seen that the $\alpha = 0{\cdot}5$ curve is overly conservative. These piles are primarily shaft friction piles and therefore increasing the shaft friction adhesion factor from 0·5 to 0·6 provides significant pile design optimisation. It should also be noted that the end bearing capacity of the test pile appears to continue to gradually increase with pile settlement with the time-marched data; this could be due to the influence of the underlying granular LG and/or a strain-related hardening mechanism.

6. Conclusions

This paper has discussed the design philosophy, construction sequence, load testing and analysis of a preliminary test pile (PTP) at a large development in Whitechapel, London, to justify the subsequent use of an increased adhesion factor of 0·6 in the pile design. This saved up to 5 m depth on the main works piles, ensured that all piles were founded above the water-bearing sand horizons of the Lambeth Group and allowed ‘dry’ bored construction methods to be employed.

The key conclusions are as follows.

- The adhesion factor $\alpha = 0{\cdot}5$ specified by LDSA (2009) is potentially overly conservative as it is a ‘one size fits all’ approach to pile design in London Clay (LC). On sites where PTPs are carried out, the test results can be back-analysed to determine a site-specific adhesion factor that is influenced by the pile geometry, construction methods and soil properties. For shaft friction controlled piles (i.e. straight shafted bored piles in LC) the adhesion factor has a significant influence on the ultimate pile capacity. Therefore, back-analysis of a PTP to failure to determine this value allows full optimisation of the pile

Load: kN	0	1125	2250	3375	4500	5625	6750	7875	9000	10 000
Settlement: mm	0	1·26	2·69	4·32	5·46	7·23	12·26	27·99	58·57	120·67

Table 3. Pile head settlement measured during MLT (time-marched 10 years)

Pile shaft diameter: m	Pile base diameter: m	Shaft free length: m	Shaft friction length: m	Soil elastic modulus at base, E_b: kN/m^2	Concrete modulus, E_c: kN/m^2	Friction centroid, k_e	Shaft flexibility factor, M_s
0·75	0·75	0·50	37·0	100 000	3·60E+07	0·45	0·0015

Table 4. Loadtest program input parameters

design, with associated reductions in cost, programme and risk.

- 'Working' pile load tests to 1·5 times the safe working load generally only validate workmanship quality and serviceability performance rather than being able to prove and challenge the design philosophy and parameters. It would be impossible to use load–settlement test data up to the working load to determine an appropriate adhesion value or predict the ultimate geotechnical capacity of a pile. It is only when the test pile is loaded to its ultimate geotechnical capacity that the shape of the load–settlement curve is defined, allowing it to be matched to the most appropriate predicted settlement curve. Therefore, in order for this method to be adopted, a site-specific PTP loaded to full geotechnical failure must be undertaken.
- It is widely accepted that construction methods have a significant effect on the adhesion achieved in clay strata. The 0·5 adhesion factor specified by LDSA (2009) is based on the pile bore being open for a maximum of 12 h. For the site considered here, rotary bored piles were drilled quickly and then concreted within 2 h of completion of drilling to mitigate deterioration of the pile shaft and base. It is believed that this methodology was a major contributing factor in achieving an adhesion factor of 0·6 in the LC. Future research would benefit from installing more than one PTP on a site, perhaps using different construction methods (e.g. rotary and continuous flight auger piles) so that the results can be compared and the effects evaluated.
- Where PTP test results are used to determine a site-specific adhesion value, it must be ensured that the works piles are constructed using the same construction methods and quality controls. This PTP was rotary bored, hence the higher adhesion value was only appropriate for the rotary bored works piles on this site.
- PTP test data should be 'time-marched' at least 10 years, especially when the rate of pile head settlement has not reduced to within the SPERW (ICE, 2007) acceptance criteria at higher loads (which is likely in cohesive strata).

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