A review of pile test results and design from a London clay site

Un compte rendu sur les résultats d'essais sur pieux et leur dimensionnement sur un site d'argile de Londres.

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ABSTRACT: A large number of different types of piles have been installed on a well-characterised stiff clay site. The capacity of piles tested soon after installation has been assessed by maintained load tests. A simple total stress design method in regular use in the UK has been used to compare the results from the different types of pile. The results have been used to make comment on the choice of parameters within the design method and the Eurocode factors for pile resistance quoted in the UK National Annex.

RÉSUMÉ : Un grand nombre de différents types de pieux ont été installés sur un site d'argile raide bien caractérisée. La capacité portante des pieux testés peu après leur installation a été évaluée par essais de chargement statique. Une méthode de dimensionnement simple en contrainte totale, en usage au Royaume-Uni, a été utilisée pour comparer les résultats des différents types de pieux. Les résultats ont servis à commenter le choix des paramètres pour la méthode de dimensionnement ainsi que les facteurs de l'Eurocode utilisés pour la résistance des pieux cités dans l'annexe national britannique.

KEYWORDS: Piles, pile tests, pile design.

1 INTRODUCTION.

Pile design in Europe has been changed by the adoption of the Eurocodes, which articulate a number of different options for the evaluation of pile capacity. A traditional UK procedure has been compared for a number of pile types and relevant issues for choice of parameters and resistance factors highlighted.

Three series' of maintained load (ML) pile tests are described and the data presented, on CFA, auger displacement, driven and bored piles. These tests have given an opportunity for comparisons to be made between different pile types on the same site.

2 THE SITE

2.1 *Location and geological setting*

The site is located at Chattenden, northern Kent, approximately 30 miles south-east of central London. The site is underlain by high plasticity London Clay to a depth of at least 44m.

The ground slopes gently at about 1:10. Successive emergent shear surfaces uncovered in trial pits indicate that there has been down-slope movement to depths of 1.5m in the past.

2.2 *The site and soil properties*

The site has been used by BRE as a shrinkable clay and in situ testing trial site since 1987. Dummy foundations – pads, trench fill and piles have been installed and monitored during seasonal and vegetation-induced changes in water content and consequent soil movements. The site has since been used to test the behaviour of piles and pile installation and a number of in situ test procedures.

2.2.1 *Site investigation and soil parameters*

Site investigations carried out over a number of years have given information on index properties, stiffness, shear strength (via in situ and laboratory tests) and in situ stresses. The London Clay is of high plasticity, heavily overconsolidated and anisotropic. Figure 1 shows some of the basic soil properties. Shear strengths range from 40kPa to over 140kPa at 10m depth, with K_0 reducing from 3 near the surface to 2 at depth.

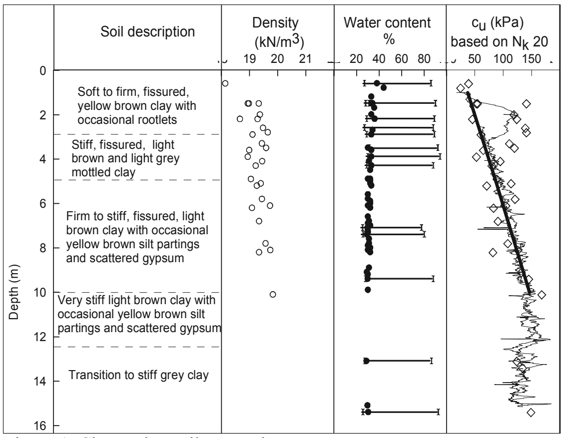


Figure 1. Chattenden soil properties.

A number of CPT profiles were carried out over the piling trial area, which showed that the strata were very uniform laterally and vertically (Figure 1) with occasional claystone bands. In one area there was an increased shear strength over the upper 5m, thought to be associated with the previous presence of trees. Figure 1 shows undrained shear strengths interpreted from the CPT (using N_{kt}=20) and laboratory UU and CU tests on 100mm samples; a general trend line is also shown through all data.

3 THE PILES

3.1 *Piling trials*

Trials into the control of CFA drilling, pile types, ageing of bored piles, pile improvements for reuse and pile testing methodologies have been carried out. A general description is indicated in Table 1.

Table 1. Piling Trials.

Trial	*Dates*	*Piles*
TOPIC	2001	CFA, Bored, Displacement
RuFUS	2001-2006	CFA, Bored
RaPPER	2007-2009	CFA, Driven

3.2 *Pile Installation*

A number of different pile types (commercial and developmental) were installed at the site over 10 years. This paper focuses only on those types in current use, tested at an age of up to 5 months, but mostly 2.5-3.5 months, by static incremental maintained load testing (ML). Other papers describe piles tested for other purposes (Skinner et al 2003, Fernie et al 2006, Powell and Brown 2006, Powell and Skinner 2006, Butcher et al 2008, Brown and Powell 2012) Tables 1 and 2 show the piles used in this study.

Table 2. The test piles.

Pile	Date	Dia. (mm)	Effective Length (m)
CFA (T33)	2001	300	10
CFA (T34)	2001	300	10
CFA (T14)	2001	300	10
CFA (T13)	2001	400	7
CFA (T15)	2001	400	7
CFA (MC1)	2007	450	9.5*
CFA (MC2)	2007	450	9.5*
Bored (T16)	2001	300	10
Bored (T40)	2001	300	7
Bored (T46)	2001	300	5.8**
Bored (T47)	2001	300	5.8**
Screw Dispt (T30)	2001	300/600	7
Displacement (T35)	2001	300	9
Displacement (T36)	2001	300	7
Driven (TP1)	2007	275	10
Driven (TP2)	2007	275	10

* 1.5-11 m, ** 5.2-10.m

3.3 *Pile testing*

The Topic and RuFUS pile tests were undertaken using a combination of BRE load frames and a remotely operated hydraulic loading and control system The loading system utilised closed loop control of the hydraulic jack, monitoring displacement transducers and a load cell. Load was applied in incremental steps; increments of 25kN were each held for a minimum 1 hour and until the settlement rate reduced to 0.1mm/ hr. Using this procedure it was hoped that the load at which rupture of the skin friction occurred would be approached relatively slowly. Tests were terminated once failure was clearly defined, generally indicted by runaway displacement.

For the RaPPER programme the test equipment was similar to that described above but operated on site. The test method used complied with the ICE Specification for Piling and Embedded Retaining Walls 2nd ed. (ICE 2007); the procedure was similar to that above but using 125kN increments throughout firstly up to 500kN, then an unload/reload loop before continuing until failure was established. The increments during loading were maintained for a minimum 30mins and until the rate of settlement reduced to 0.1mm/hr. This criteria works well until failure is approached.

In the RaPPER project testing was also conducted by constant rate of penetration, dynamic and rapid load or statnamic means and is described elsewhere (e.g. Butcher et al, 2008, Brown and Powell 2012).

3.4 *Test Results*

3.4.1 *Definition of shaft failure capacity*

The majority of the CFA and bored piles, at 300 to 450mm were anticipated to be essentially friction piles. These piles had relatively high length to diameter ratios; additionally no attempt was made to clean the bases of the bored piles. As failure was achieved at relatively small displacements, typically less than 5/6mm this would appear to be a reasonable assumption. In these cases, capacity was taken to be the maximum 'stable' load achieved. Given the tendency in brittle London clay for the pile to 'shed' load down its length as failure is initiated in the upper parts then a 'stable' load was taken to be either the last increment applied if this was maintained for some time before significant displacements occurred or the next to last increment if the pile failed rapidly soon after application. The interpretation of the failure load increment was more difficult for larger increments. The screw displacement piles, at 600mm external diameter, and the driven piles were expected to demonstrate rather more base capacity. For the driven piles capacity was taken to be again the maximum 'stable' load but with an allowance for base capacity based on eqn (2). For the larger 'displacement' piles failure was taken as the load at which significant creep started to occur under load and this was also checked based on Chin and Fleming constructions.

Based on the failure criteria discussed above, the ultimate capacity of each pile is shown in Table 3.

4 PILE DESIGN

4.1 *Design by calculation*

In the UK, it is common to use a total stress method for the calculation of pile capacity in clay soils. For the purposes of the present paper the model for pile capacity has been taken considering undrained behaviour and to be the sum of shaft (Q_{su}) and base (Q_{bu}) where:

$$Q_{su} = \Sigma(q_{su}\ \Delta L\ A_s) \qquad (1)$$

Where; q_{su} is ultimate unit shaft friction; ΔL the appropriate section of pile length; A_s is surface area per unit length of pile

Here : $q_{su} = \alpha\ c_u$

where: α is an empirical factor; c_u is the average shear strength over the length ΔL

and base capacity as: $Q_{bu} = A_b\ N_c\ c_{u\ base}$ (2)

where: A_b is the area of the base of the pile, N_c is the undrained bearing capacity factor generally taken as 9, c_u base is the undrained shear strength at the base of the pile.

This is used to calculate the pile capacity under BSEN1997-1 (7.6.2.3), to which model and partial factors are applied to identify the design pile resistance. Estimates of pile capacity for the different types of piles have been made for all of the piles using the α–c_u method.

4.1.1 *Results – alpha values and soil parameters*

There is an intimate link between selection of a value of alpha (α) and soil strength. One has to ensure that when α values are selected from the literature then the same method of shear strength derivation has to be used (sampling methods, sample sizes and testing). Typically a design line for shear strength has been a 'mean' value and that is what has been adopted here.

All values for α (Table 3) were those based on shear strength profiles from CPTs to the piles correlated to UU triaxial. Although the main test area described was very uniform, the area where the RaPPER piles were located was a little distance away and seems to have undergone desiccation in the upper layers although the CPTs come together below 5m.

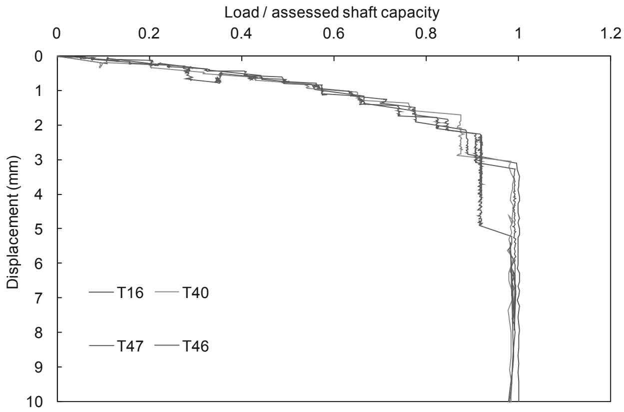


Figure 2. Normalised bored pile test results.

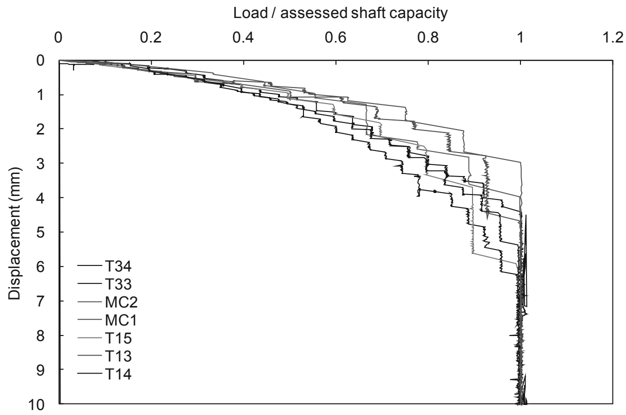


Figure 3. Normalised CFA pile test results.

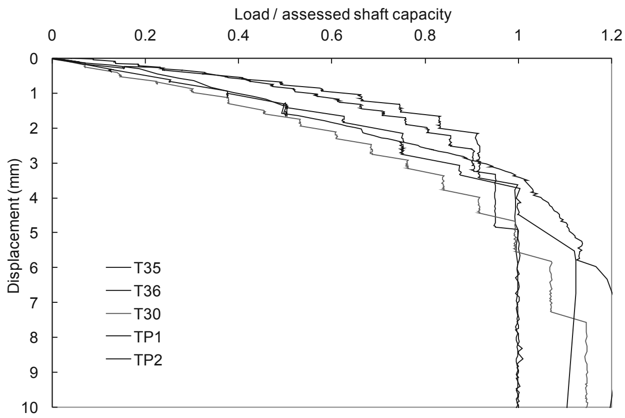


Figure 4. Normalised displacement pile test results.

4.1.2 *Results – bored piles*

The results from the 4 bored piles gave remarkably consistent values for α in the range 0.5 to 0.55. They all gave a very similar behaviour, failing at a pile head movement of about 3mm as shown in normalized plot in Figure 2.

4.1.3 *Results – CFA piles*

The 7 CFA piles showed a significant variation in capacity with control of installation. The 'normal' drilling installation parameters for a clay of this type is around 100mm penetration per revolution. A lower penetration per revolution increases the potential for greater smearing of the bore, whilst the converse is true where higher penetration rates are used.

In the study, rates of 150mm or 120mm, 100mm and 50mm penetration/revolution were used for the different 300mm and 400mm diameter augers with a common pitch of 350mm (Skinner et al 2003). These rates were labelled 'tight', 'normal' and 'loose'. The capacities found showed that there was an increase in α for a higher penetration per revolution (Table 3). The value found could be related to installation (Figure 5). The worst value obtained (TP13) gave an α value close to that of the 'bored' piles! The change of α for CFA piles has potential benefits for challenging sites, but carries a warning that preliminary test piles need to be installed with the same parameters same as working piles, to ensure the design parameters selected are appropriate. The 3 'normal' CFA piles gave α values in the range 0.72-0.75 which are somewhat higher than the typically used value of 0.6. This may reflect the fact that low values have been encountered previously and this may have been a result simply of poor construction control.

They all showed basically similar behaviour, failing at a pile head movement of between 3 and 5mm, those going to 5mm showing a slightly more curved response as shown in Figure 3 normalized plots. For the 300mm diameter CFA piles, the longer piles show the more curved behaviour but these were also loaded in smaller increments and as a result took longer times to failure - which may have allowed more shedding of load down the pile as local failure occurred at shallow depths. The 450mm diameter piles (MC1 and MC2) showed the stiffest behaviour but were loaded in larger increments and so shorter times.

Table 3. Derived Alpha values

Pile type	Static failure load (kN)	Alpha
CFA (T33)	700	.84
CFA (T34)	600	.72
CFA (T14)	625	.75
CFA (T13)	325	.52
CFA (T15)	500	.82
CFA (MC1)	1000	.72
CFA (MC2)	1050	.75
Bored (T16)	450	.54
Bored (T40)	225	.51
Bored (T46)	300	.52
Bored (T47)	310	.54
Screw Dispt (T30)	650*	.72
Displacement (T35)	525	.73
Displacement (T36)	300	.65
Driven (TP1)	1000	1.0
Driven (TP2)	950	.95

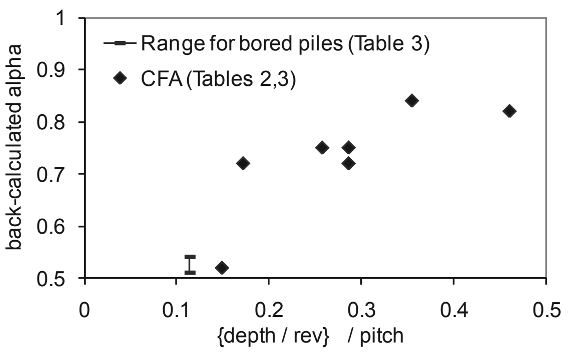
*interpreted shaft only

4.1.4 *Results – displacement piles- auger*

Auger displacement piles have the advantage of minimal spoil generation without the noise disadvantage of driven piles.

Within the auger displacement pile category two pile types were tested, straight forward parallel sided piles with all soil displaced laterally and 'screw' displacement piles where a screw thread is cut out from the central shaft thereby giving a larger overall diameter to the pile, still with no soil removal and using less concrete than a CFA or bored pile of the same diameter. The normalised results are shown in Figure 4.

The displacement piles (T35 and T36) gave α values of 0.65-0.72 while the screw displacement pile gave an α of 0.72 (based on the external diameter of the screw thread). The screw pile had a slightly softer response to loading but with the much larger potential base area and the different potential shaft failure modes this is not surprising. In the literature the diameter used to back calculate skin friction values is sometimes taken as a mean value between the central core and maximum thread diameter and care should be taken when comparing values as to the diameter used. The screw pile type tested (Atlas) showed capacity equivalent to a 600mm diameter CFA pile, with significantly lower concrete consumption.



4.1.5 *Results – displacement piles- driven*
The square section driven piles (TP1 and TP2) gave an α of around 1.0 in Table 3. The normalized plots shown in Figure 5 are linear but with a slightly softer response than the bored piles but reaching capacity at a similar displacement of 3-4mm.

4.1.6 *Results – range of alpha for typical piles*
Various sources give values for α for different pile types (e.g. Burland et al 2012) and selected relevant values are shown below in Table 4. Some sources vary the α value so that it decreases after a threshold with increasing shear strength, which effectively creates a maximum value for the achievable shear stress. Others vary α with variations in the c_u/σ'_{vo}. The values quoted in Table 3 reflect the soil conditions and pile lengths and diameters on the test site.

Table 4. Typical values of alpha for London Clay (similar L/D)

Pile type	Range of alpha (α)
Bored	0.45-0.5
CFA	0.6
Driven	0.8

The tests on the various pile types reported here show:

- Bored piles: tests on piles installed in well controlled conditions were at the upper range of the typical α values;
- CFA piles: showed variation dependent on pile installation, and α values varying from those close to bored piles on this site to values much higher than ones typically quoted for CFA; on average values for 'typical' CFA piles on this site were some 30% higher than values normally quoted;
- Auger displacement piles: showed α values similar or slightly lower than the bulk of the CFA results, when an appropriate diameter was selected. For screw displacement piles this was the outer diameter;
- Driven piles: showed very high α values, significantly above those typically quoted.

4.1.7 *Results – range of results compared with Eurocode 7 UK National Annex*
The UK National Annex quotes resistance factors to be applied to the shaft capacity, for various pile types. These values are summarised in Table 5.

The variation in R4 values between pile types could be taken to imply a difference in anticipated variability in capacity. Based on the results found in these studies, a far greater variability is to be expected from CFA piles than bored piles. However these piles were constructed under 'supervision' and so should be well controlled and reflect the inherent variability of the construction methods and what can be achieved.

No comment can be made in this study as to the effects of time to concreting for bored piles, test methodologies or driven piling, as the database is too small.

Table 5. R4 values for shaft resistance only.

Pile type	R4 without load tests	R4 with load tests
Bored	1.6	1.4
CFA	1.6	1.4
Driven	1.5	1.3

5 CONCLUSION

Total stress estimates for ultimate capacity in clay soils are common in the UK. This paper shows pile tests on different pile types and the α value associated with them on one uniform site.

O'Brien and Bown (2008) show, based on a large database of pile tests, that the α-c_u approach is unreliable. In this study, all the quoted sources of variation (shear strength, test methodology, failure definition) other than installation have been reduced as far as possible. Where the results shown here are compared with other data, these other sources of varibility must be considered.

All the α found in this study are higher than the literature for relevant pile and soil types. While this might be partly a function of shear strength, the selected values for shear strength are in accordance with tests on 100mm diameter samples, and the higher capacity can better be explained by greater control.

The testing reported here shows greater variability was found for CFA compared with bored piles, not necessarily implied by the R4 factors. Displacement pile capacity was similar to a CFA pile of relevant diameter (here the outer diameter). In addition under these conditions the driven piles were seen to be very effective.

6 ACKNOWLEDGEMENTS

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