



| | |
|-------------------------------------|--|
| Title | Axial resistance of CFA piles in Dublin boulder clay |
| Authors(s) | Gavin, Kenneth, Cadogan, David, Twomey, Lloyd |
| Publication date | 2008-08 |
| Publication information | Gavin, Kenneth, David Cadogan, and Lloyd Twomey. "Axial Resistance of CFA Piles in Dublin Boulder Clay." Institution of Civil Engineers/Thomas Telford Publishing, August 2008. https://doi.org/10.1680/geng.2008.161.4.171 . |
| Publisher | Institution of Civil Engineers/Thomas Telford Publishing |
| Item record/more information | http://hdl.handle.net/10197/2433 |
| Publisher's version (DOI) | 10.1680/geng.2008.161.4.171 |

Downloaded 2026-05-01 23:36:40

The UCD community has made this article openly available. Please share how this access benefits you. Your story matters! (@ucd_oa)



© Some rights reserved. For more information



Kenneth Gavin
 Lecturer, School of Architecture, Landscape & Civil Engineering, University College Dublin, Ireland



David Cadogan
 Lecturer, Department of Civil, Structural and Environmental Engineering, Cork Institute of Technology, Ireland



Lloyd Twomey
 Associate, Geotechnical Group, Arup Consulting Engineers, Dublin, Ireland

Axial resistance of CFA piles in Dublin Boulder Clay

K. Gavin BEng, PhD, CEng, MIEI, D. Cadogan BE (Civil), Eurlng, CEng, MIEI and L. Twomey MSc, EurGeol, PGeo, CGeol, FGS

This paper describes the results of static compression and tension load tests performed on three-instrumented large-diameter continuous flight auger piles installed in Dublin Boulder Clay. The piles developed very high shaft resistance and, in contrast to piles driven into Boulder Clay that exhibit friction fatigue, the shaft distribution was uniform along the pile shaft. This resulted in the normalised average shear resistance being mobilised by a bored pile exceeding that of a pile driven in similar ground conditions. In contrast, the base resistance of the test piles was significantly lower than that of a pile driven in similar ground conditions.

NOTATION

| | |
|-------------------|---|
| D | pile diameter |
| G | shear modulus of soil |
| G_0 | small-strain shear stiffness |
| f_1 | empirical coefficient linking SPT N and s_u |
| K | coefficient of earth pressure |
| K_a | coefficient of active earth pressure |
| K_p | coefficient of passive earth pressure |
| K_m | coefficient of earth pressure mobilised at pile–soil interface |
| K_0 | coefficient of earth pressure at rest |
| N | standard penetration test blow count |
| N_c | bearing capacity factor |
| OCR | overconsolidation ratio |
| q_b | pile end bearing pressure |
| q_s | local shaft resistance |
| q_{smax} | maximum shaft resistance measured during load test |
| s_u | undrained shear strength |
| w_b | pile base displacement |
| w_t | pile-head displacement |
| α | empirical constant linking q_s and s_u |
| β | empirical constant linking q_s and SPT N value |
| δ | interface friction angle |
| δ'_h | horizontal displacement of soil particle at pile–soil interface |
| ϕ_{cv} | constant-volume friction angle |
| ϕ'_p | peak friction angle |
| $\Delta\sigma'_h$ | increase in horizontal stress caused by dilation |
| σ'_v | vertical effective stress |

1. INTRODUCTION

Because of requirements to limit noise and vibration during the construction of city centre developments, non-displacement

piling techniques are very popular. The development of a site near Dublin city centre with complex ground conditions, including near-surface alluvial soils, Dublin Boulder Clay (DBC) and highly fractured limestone, afforded an opportunity to perform full-scale instrumented load tests on continuous flight auger (CFA) piles, which were 762 mm diameter and 11–14 m long. Instrumentation allowed the distribution of shaft resistance to be determined during static load tests on three piles. Shaft resistance developed in the Boulder Clay supported the majority of the load carried by the piles.

Current design practice for routine pile design in Ireland uses the α (total stress) approach to calculate the unit shaft resistance q_s of driven and bored piles in DBC according to

$$q_s = \alpha s_u$$

where α is a reduction factor, and s_u is the undrained strength of the soil in kPa. Although there is little guidance on the choice of an appropriate value for α value, Ciria C504¹ suggests α mobilised by driven piles in Boulder Clay reducing from 1.0 at $s_u = 80$ kPa to a minimum value of 0.45 when s_u exceeds 200 kPa, with values for bored (non-displacement) piles being approximately 10% lower. Because of the high strength of DBC, α values in the range 0.35–0.45 are typically used in Irish design practice. Farrell *et al.*² and Farrell and Lawlor³ report instrumented load test results from driven steel and CFA piles founded in DBC. Farrell and his co-workers investigated the effect of time on the increase in shaft capacity of a driven 273 mm diameter steel pile instrumented with total stress and pore pressure transducers installed in DBC. The pore pressures developed during installation were seen to have returned to near-hydrostatic values prior to a load test performed 17 days after driving. The average s_u value measured along the pile shaft was 450 kPa, and they report α values increasing from 0.21 immediately after installation to 0.55 when the pile was retested after 17 days. Farrell and Lawlor³ report the result of a static compression load test on CFA pile, 600 mm in diameter and 11 m long, installed in DBC. The soil conditions at the site were similar to those in which the driven pile was installed, with the average s_u value along the pile shaft being 360 kPa. The α value mobilised by the CFA pile approached 0.65 at the end of the load test. These limited data suggest that conventional design practice used to calculate shaft resistance is somewhat conservative, and the common assumption that

non-displacement piles develop lower shaft resistance than driven displacement piles may not be true for piles in DBC.

Burland⁴ argues strongly that effective stress approaches should be used to estimate shaft resistance, from

| | |
|---|--------------------------------|
| 2 | $q_s = K\sigma'_v \tan \delta$ |
|---|--------------------------------|

where K is the earth pressure coefficient, σ'_v is the in situ vertical effective stress, and δ is the pile-soil interface friction angle. A common difficulty with the application of equation (2) is the choice of an appropriate K value for design. Instrumented pile test results on displacement piles⁵ show that K depends on the initial soil state, the location relative to the pile tip, and the depth in the ground of the point under consideration. For non-displacement piles, K is often assumed to be equal to K_0 , which can be estimated using the method proposed by Mayne and Kulhawy⁶:

| | |
|---|--|
| 3 | $K_0 = (1 - \sin \phi'_p)$ for normally consolidated soil |
| | $K_0 = (1 - \sin \phi'_p) \text{OCR}^{\sin \phi_p}$ for overconsolidated soil |

where ϕ'_p is the peak friction angle and OCR is the overconsolidation ratio.

Fleming *et al.*⁷ suggest, alternatively, that K is equal to 0.7 for conventional bored piles and 0.9 for CFA piles. CIRIA C504¹ adopts these values for piles in glacial soils.

Because of the high stone content of DBC, it is very difficult to obtain good-quality samples for routine design purposes. The undrained strength is usually correlated to the standard penetration test (SPT) N value using the relationship proposed by Stroud,⁸

| | |
|---|---------------|
| 4 | $s_u = f_1 N$ |
|---|---------------|

where $f_1 = 6$ for low-plasticity clay, and s_u is given in kPa.

Given the reliance on SPT N values for site investigation, direct correlations between shaft resistance and SPT (N) have been proposed:

| | |
|---|-----------------|
| 5 | $q_s = \beta N$ |
|---|-----------------|

Typical β values are 1.0 for bored piles and 2.0 for driven piles when the unit shaft resistance has units of kPa. However, Robert⁹ compiled a large database of bored and driven piles in sand, and concluded that there was no systematic difference between the shaft resistance mobilised by bored or driven piles, with $\beta = 1.9$ giving the best fit to the available data.

The base resistance of piles founded in DBC is estimated using the bearing capacity equation

| | |
|---|-----------------|
| 6 | $q_b = N_c s_u$ |
|---|-----------------|

where N_c is a bearing capacity factor. Farrell *et al.*² report a value for N_c of 55 from a load test on a driven pile, and N_c of 9 is assumed for non-displacement piles.

Recent instrumented pile test programmes, such as those reported by Jardine *et al.*¹⁰ and others, have resulted in the development of effective stress design methods for displacement piles. There has been a dearth of parallel research for non-displacement piles. As a result, designers still rely on a wide range of empirical design techniques to calculate the axial resistance of single piles in clay: in particular, total stress methods such as the α method remain popular despite their generally poor reliability.¹⁰ This paper presents the results of maintained load tests performed on three full-scale instrumented CFA piles installed in DBC. The test results are then compared with other instrumented pile tests in DBC reported in the literature.

2. GROUND CONDITIONS

2.1. Engineering properties of DBC

As noted by Long and Menkiti,¹¹ DBC is the primary superficial deposit overlying bedrock in the greater Dublin area. It is classified into four distinct major units: Upper Brown, Upper Black, Lower Brown and Lower Black Boulder Clay. All four units are highly overconsolidated, and are dense/hard in situ with low plasticity index (10–15%). The Lower Brown and Black Boulder Clay were identified only in areas north of the city centre at depths in excess of 10 m below ground level. The pile tests discussed in this paper, in common with the majority of structures in the Dublin area, are founded in the upper brown and upper black DBC.

Triaxial compression tests on rotary-cored samples of DBC¹² reveal strong dilation at low stress level (mean stress below 150 kPa) with peak friction angles up to 43° and constant-volume friction angles ϕ'_{cv} of approximately 34°. Although the material is referred to as Boulder Clay, its clay mineral content is very low, and the fines content (of up to 35% by weight) is composed largely of rock flour. The field permeability of 10⁻⁹ to 10⁻¹⁰ m/s and the absence of clay minerals caused Lehane and Simpson¹³ to conclude that the engineering behaviour of DBC is characteristic of very low-permeability dense sand. Additional information on engineering parameters of the DBC is contained in Lehane and Simpson¹³ and Long and Menkiti.¹¹

2.2. Ground conditions at test site

Ground conditions at the site, which is located approximately 2 km south of Dublin city centre (see Fig. 1), comprise 2–4 m of made ground overlying a layer of alluvial gravel (with silt pockets), which is in turn underlain by DBC. The Boulder Clay, which overlies limestone bedrock, was typically encountered at –2 mOD, and consists of a thin layer (generally < 1 m) of Upper Brown Boulder Clay, overlying Upper Black Boulder Clay, which typically extends to –6 mOD. However, a site investigation for a previous building in the area suggested that the rock level was greater than 20 m over a part of the central site area. A detailed desk study for the current development identified an ancient river channel (a v-shaped valley), infilled

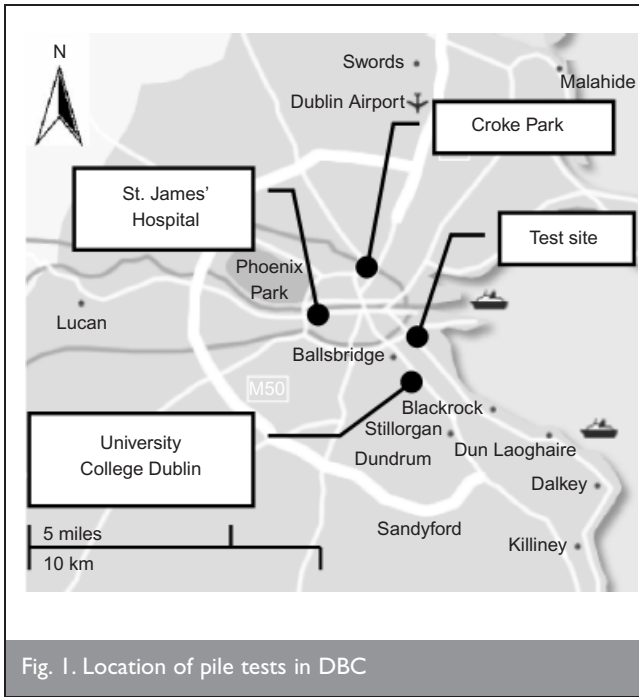


Fig. 1. Location of pile tests in DBC

with DBC, cutting into the bedrock in this area. The groundwater table is at 0 mOD. The primary aim of the site investigation for the current development was delineation of the buried valley feature. In this respect, shell and auger boreholes to refusal were undertaken with follow-on Geobore S rotary coring (see Fig. 2).

Ground level at the site is relatively flat at approximately +4 mOD. The uncorrected SPT N value to depth profile shown in Fig. 3(a) distinguishes the layers based on the soil description. The silt pockets within the alluvial gravel are soft to firm in situ with N values ranging from 5 to 17. The N values of the normally consolidated, alluvial gravel increase from 10 at +1 mOD to 40 at -2 mOD. The material is highly variable, and is generally considered to be unsuitable as a founding stratum owing to the presence of the silt lenses. These are shown in Fig. 3(a) to extend up to -3 mOD in places.

The SPT N values in the DBC were in the range 40–80, with the lower values being measured in the brown DBC, whereas values in the black DBC averaged $N = 75$. No measurements of

undrained shear strength were made, as samples from the shell and auger boreholes were unsuitable, and rotary-cored samples were all logged to identify the soil-rock interface. Measurements of the water content and liquid and plastic limits shown in Fig. 3(b) reveal liquidity indices below zero, and are typical of those reported by others. Two profiles of the small-strain shear stiffness were measured at the test using the multichannel analysis of surface waves (MASW) technique. The G_0 values of 300 MPa measured in the brown DBC, rising to 550–850 MPa in the black DBC, are similar to measurements reported at various DBC sites by Long and Menkiti.¹¹

The middle Carboniferous 'Calp' limestone underlying much of Dublin is generally strong, and is typically heavily fractured only within 1–2 m of the overlying glacial soils. The limestone is a 'muddy' Argillaceous deposit, and is typically not prone to large-scale solution weathering. Outside the buried valley feature the rock was described as strong, highly fractured, and relatively unweathered. Total core recovery of 100% was achieved, and the rock quality designation was typically below 30%. In the centre of the site the rotary coreholes were continued to 20 m below ground level without encountering limestone.

3. PILE DETAILS

Three instrumented CFA piles were installed at the site using a Soilmec R622 model high-torque piling rig with a 762 mm diameter auger. Approximate locations of the test piles TP1 to TP3 relative to the buried valley feature are shown in Fig. 2. Piles TP1 and TP2 were loaded in compression, and pile TP3 was loaded in tension. The depths at which the torque limit on the CFA rig was met during drilling were -8 mOD for TP1, -9.8 mOD for TP2 and -7 mOD for TP3. Pile TP3 was the only one to encounter rock, with the lower 1.35 m of the pile shaft being drilled through fractured limestone.

The compression piles (TP1 and TP2) were reinforced with six T20 bars, and the tension pile was constructed using four full-length 36 mm diameter high tensile Dywidag bars, which were cast into the head of the pile. Vibrating-wire-type (Gage Technique TES/S-J/T) embedment strain gauges were attached to the steel reinforcement cage in groups of four at fixed depths of -1, -3.5, -5.5 and -7.5 mOD on the compression piles, and 0, -2.5, -4.5 and -6.5 mOD on the tension pile.

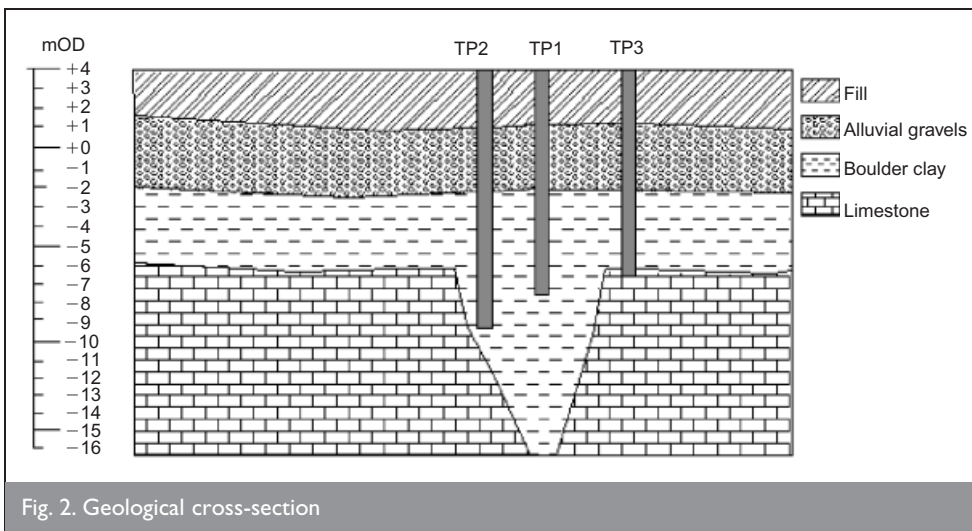


Fig. 2. Geological cross-section

4. STATIC LOAD TEST RESULTS

4.1. Procedure

The load tests were performed a minimum of 20 days after the piles had been constructed, and were undertaken in accordance with the Institution of Civil Engineers' piling specification.¹⁴ All piles were subjected to maintained load tests. The two compression piles were tested to a maximum load of 3750 kN (250% of their design load),

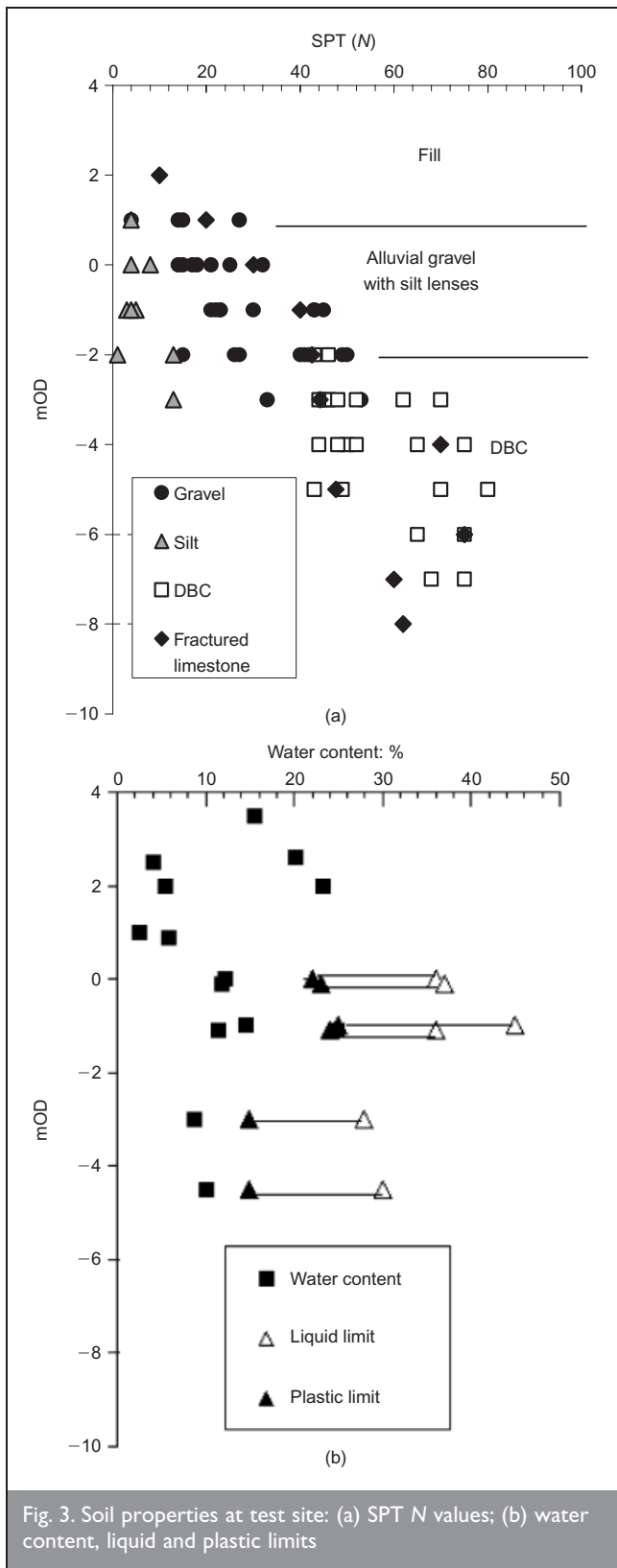


Fig. 3. Soil properties at test site: (a) SPT N values; (b) water content, liquid and plastic limits

and the tension pile was loaded to a maximum load of 3000 kN. The jacking system was attached to a hydraulic powerpack, which in turn was linked to a data logger/controller unit. This unit was managed and operated by software that allowed the applied load to be regulated automatically for each test, leading to the precise application of the test loads at the correct interval, and to accurate logging of the applied load and pile displacement.¹⁵ The compression piles were loaded and unloaded in three cycles, and the tension

pile underwent four load cycles. Each test was scheduled for a minimum of 30 h. The individual minimum time periods relevant to the application of each increment of load were extended until the rate of settlement was less than 0.1 mm/h.

4.2. Load–displacement response

The overall load–displacement responses of the compression and tension piles are shown in Figs 4(a) and 4(b) respectively. The compression piles exhibited a much stiffer response than the tension pile. The pile-head displacement of both compression piles at the working load of 1500 kN was less than 2 mm. Of the two compression piles, TP2 (the longer pile)

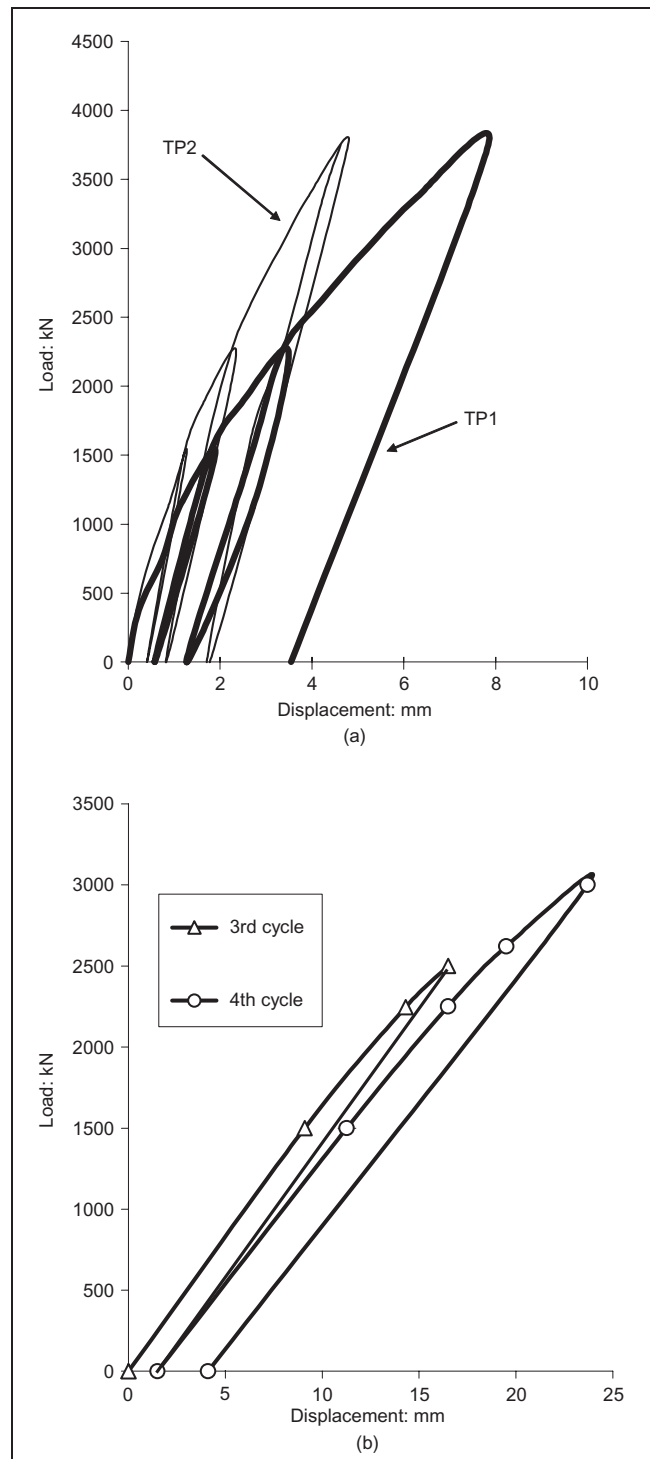


Fig. 4. Load–displacement curves for test piles: (a) compression piles; (b) tension pile

showed the stiffer response, with the pile-head displacement at the ultimate load of 3750 kN being 4.63 mm, compared with 7.84 mm for the shorter pile (TP1). A notable feature of the tests on the compression piles was the extremely stiff response of the piles during reloading portions of the test, where, for example, the overall displacement measured on TP2 during the last reload cycle (when the load increased from 0 to 3750 kN) was 2.85 mm. Although the tension pile was subjected to a total of four load cycles, only the final two load cycles are shown in Fig. 4(b). During the first two load cycles the displacement transducers were attached to the pile head, which was expanded (to a 1 m × 1 m square) at ground level. Cracking of the concrete around the displacement transducers was evident after two load cycles, and the displacement measurements were deemed to be unreliable. The transducers were placed onto steel plates for the last two load cycles.

4.3. Distribution of load in test piles

Assuming that steel and concrete experience equal strain during the application of load to the pile, the load at any level in the pile is given by multiplying the measured strain by the appropriate cross-sectional area and elastic stiffness of the concrete and steel. Whereas the elastic stiffness of steel and the cross-sectional area of steel in the pile were known, and the area of concrete was given by the assumption that the pile diameter was 762 mm, the elastic stiffness of the concrete may vary during a load test. The appropriate small-strain concrete modulus for use was determined based on laboratory core tests of the concrete, and the non-linear stiffness-strain response was quantified from the strain gauge readings using the tangent modulus approach.¹⁶ The raw data were also corrected to account for the effects of creep.¹⁷ Further details of the strain gauge interpretation are contained in Cadogan.¹⁸

The distribution of load during the load tests on the test piles is shown in Fig. 5, where the following can be seen.

- In all cases, the majority of load is carried by shaft resistance mobilised in the DBC (below -2 mOD). Assuming that the load at the tip of the tension pile is zero (see Fig. 5(c)), a significant portion of the tension pile resistance was developed in the fractured limestone within 0.5 m of the pile base.
- The strain gauge measurements allow the local shaft shear resistance (q_s) to be calculated along the pile shaft. The q_s values mobilised during the compression tests are plotted against pile-head displacement (w_t) in Fig. 6(a). The data show that shaft resistance increases with increasing pile-head displacement throughout the tests and, in most cases, had not reached ultimate values at the end of the load test.
- The mobilised shaft resistance response was much softer in the tension test (see Fig. 6(b)). However, the resistance mobilised at large displacements is compatible with values measured at much lower pile-head displacements during the compression tests. While the shear stress mobilised in any soil layer was naturally dependent on the pile-head displacement during the test, and on the direction of loading, shear resistance values of up to 100 kPa were measured in the alluvial gravel, 150–300 kPa in the DBC, and up to 850 kPa in the highly fractured limestone at the end of the load tests.
- If, in the compression test on pile TP1, the load distribution

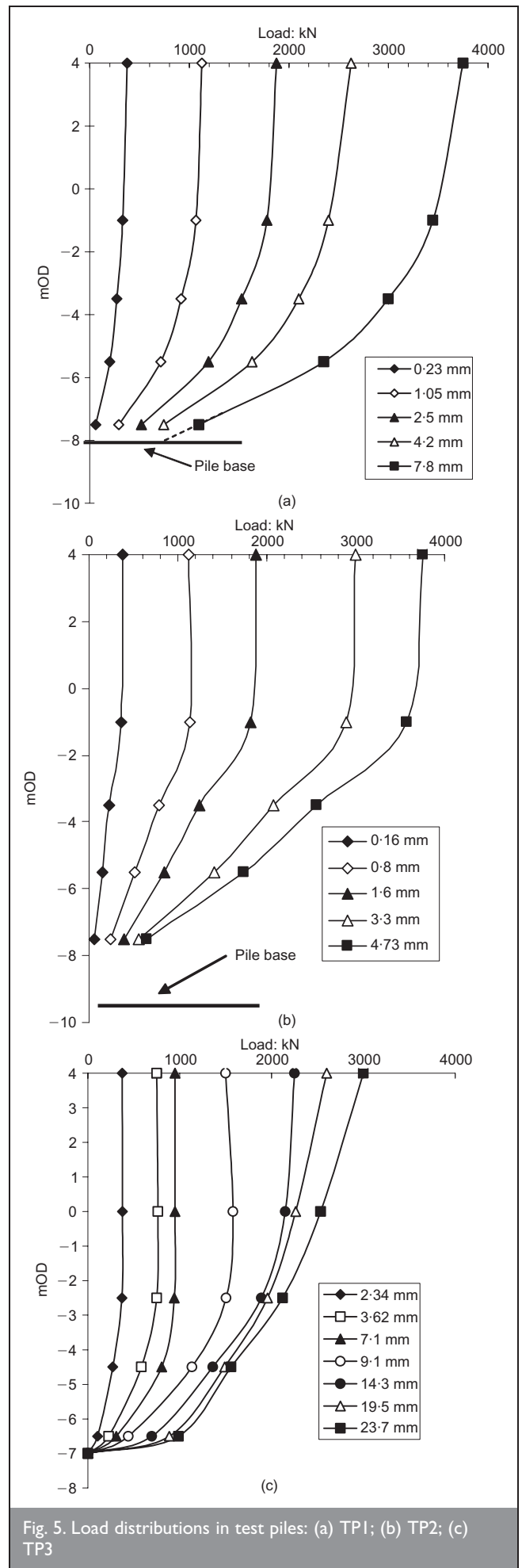


Fig. 5. Load distributions in test piles: (a) TP1; (b) TP2; (c) TP3

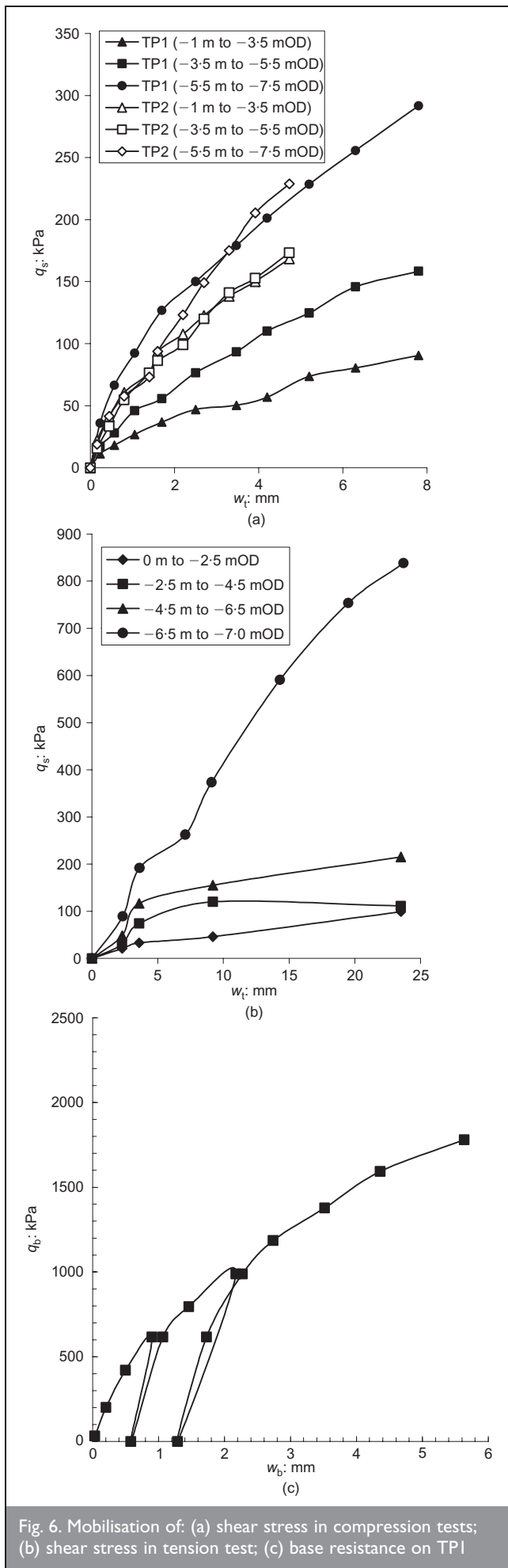


Fig. 6. Mobilisation of: (a) shear stress in compression tests; (b) shear stress in tension test; (c) base resistance on TPI

below the last set of strain gauges (which are close to the pile base) is assumed to be uniform (see dotted lines in Fig. 5(a)), the base load can be obtained. The inferred base resistance mobilised by pile TP1 is plotted in Fig. 6(c) against the pile base settlement (w_b), which is calculated by subtracting the elastic compression of the pile from the pile-head displacement. Borghi *et al.*¹⁹ and Delpak *et al.*²⁰ have shown significantly increased shaft resistance near the tip of piles, caused by shaft-tip interaction, which suggests that the assumption of uniform shear stress distribution near the pile tip is questionable. Borghi *et al.*¹⁹ present measurements made on driven piles, which generate large end-bearing stresses during installation and large residual stresses near the pile tip. Delpak *et al.*²⁰ report tests on bored piles installed through overburden and founded in rock. In the case of the non-displacement compression test piles described herein, installed in a soil layer where the soil properties in the vicinity of the pile tip are relatively uniform, no major effect of base interaction would be expected, and the assumption of uniform shaft resistance over the lower shaft is reasonable for this pile. Owing to the deeper penetration of TP2, the lowest set of strain gauges (at -7.5 mOD) was 2.3 m from the base of the pile. The load distribution curves for this pile, and the relatively small pile-head displacement achieved in the static load test, suggest that the shaft resistance near the pile tip and the mobilised base resistance may have been low. In contrast, the pile tested in tension, TP3, which was drilled into the fractured limestone and exhibited large pile head movement, developed very high shaft resistance near the pile tip.

5. DISCUSSION

5.1. Mobilised shaft resistance

Effective stress design methods for displacement piles have been developed largely through the use of surface-mounted stress sensors on the shaft of instrumented piles, which have allowed the measurement of radial effective stress during installation and loading. Tests on piles in sand⁵ have shown that dilation at the pile-soil interface during loading can lead to large increases in shaft capacity. Jardine *et al.*¹⁰ and others have shown that cavity expansion theory can be used to estimate the increase in horizontal stress ($\Delta\sigma'_h$) caused by dilation, from

7

$$\Delta\sigma'_h = 2G\delta_h/D$$

where G is the shear modulus of the soil, δ_h is the horizontal displacement of a soil particle at the pile-soil interface, and D is the pile diameter. They show that dilation effects can dominate the shaft capacity of model (small diameter) piles installed in sand, while contributing less than 5% of the total shaft capacity of piles with $D > 300$ mm. Similar phenomena were noted during partially drained loading of small-diameter piles in clay by Gallagher.²¹ Given the large diameter of the test pile described herein, and the strong tendency for the effects of dilation measured in triaxial tests on DBC to reduce when the mean stress exceeds 150 kPa, it appears reasonable to assume that the maximum coefficient of earth pressure (K_m)

mobilised during the load test can be inferred (assuming there is no interface dilation) from

$$K_m = \frac{q_{smax} / \tan \delta}{\sigma'_v}$$

where q_{smax} is the maximum shear resistance measured, δ is the constant-volume interface shear angle, and σ'_v is the vertical effective stress.

Although it is clear from Fig. 6(a) that the ultimate shaft resistance of the piles was not mobilised during the load tests, it is of interest to compare the maximum shaft resistance values mobilised during the tests with the coefficient of earth pressure (K) values predicted using conventional techniques.

K_m values inferred from equation (8) for the test piles are compared in Fig. 7(a) with K_0 values predicted using equation (3), values measured from high-pressure dilatometer tests at the Dublin Port Tunnel site,¹¹ the CIRIA C504 recommendations for glacial till,¹ and K_0 values derived using a finite element model of DBC.¹³ The inferred K_m values (shown by discrete solid symbols) are in the range 1.9 to 3.2. They are higher than the CIRIA C504 recommended values; however, K_0 values estimated from a range of sources and measured using in situ tests are seen to be highly variable. Such variability results in significant uncertainty for designers in the application of effective stress methods in the form of equation (2).

5.2. Comparison with instrumented pile tests in DBC

In this section, the results of the load tests on the CFA piles at the test site are compared with an instrumented pile test on a CFA pile, 600 mm in diameter and 11 m long, installed at the St James's Hospital site by Farrell and Lawlor³ (see Fig. 1), designated J1 in the following, and a driven steel closed-ended pile, 273 mm diameter and 6.4 m long, installed at Croke Park by Farrell *et al.*,² designated CP1. Both piles were installed through fill and Upper Brown Boulder Clay into the Upper Black Boulder Clay, and were subjected to static load tests where the pile-head displacement at the end of the test exceeded 2.5% of the pile diameter. The shaft resistance would therefore be assumed to be fully mobilised. Further details of the test piles and the site conditions are given in Table 1.

The distribution of K_m values (inferred using equation (8)) along the pile shafts is considered by plotting the values against distance from the pile base (h) normalised by pile diameter (D) in Fig. 7(b). K_m values on the CFA pile, J1, that approach the passive pressure limit (K_p) of the soil are significantly higher than values measured on TP1 and TP2, although, as noted, the relative pile head movements were much greater on J1. With the exception of the value measured closest to the tip of pile J1, K_m values were relatively uniformly distributed along the shaft of the CFA piles. A clear trend for K_m to reduce with increasing distance (h/D) from the tip of the driven pile is evident.

Farrell *et al.*² also noted that the shaft resistance mobilised on pile CP1 decreased significantly with increasing distance from the pile base. Similar shear stress distributions are noted on instrumented displacement piles in sand and clay,^{5,22} and have

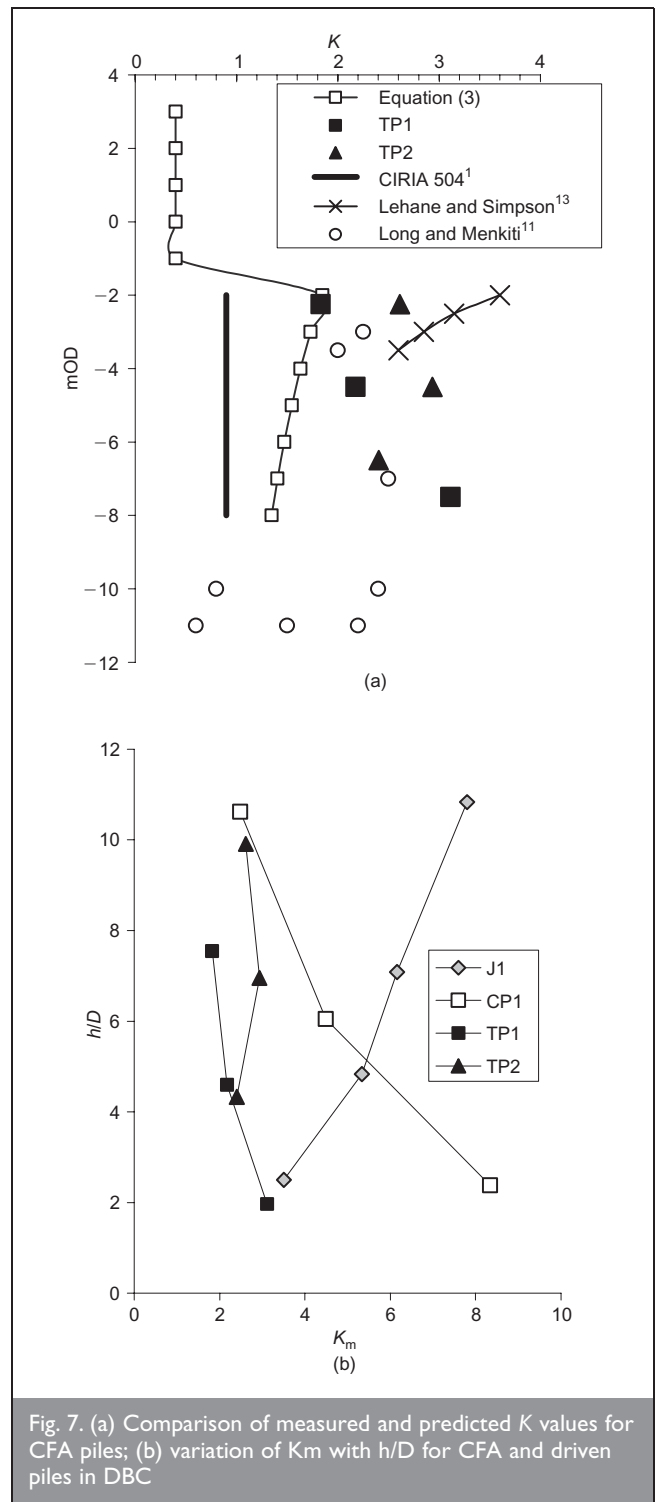


Fig. 7. (a) Comparison of measured and predicted K values for CFA piles; (b) variation of K_m with h/D for CFA and driven piles in DBC

been incorporated into pile design methods such as the Imperial College method.¹⁰ Gavin and O'Kelly²³ show that the large end-bearing stresses developed during displacement pile installation cause an increase in horizontal stress near the pile base, which in a given soil layer reduces as the pile base is driven further downwards and also as the number of load cycles (caused by pile-driving) increases. The latter effect, whereby horizontal stresses measured on instrumented piles reduce with increasing load cycles, is commonly known as friction fatigue. Although CFA piles do not benefit from the enhanced in situ stress conditions experienced by a displacement pile during installation, they are also unaffected by stress reductions due to cyclic loading. This potentially explains the relatively uniform distribution of K_m along the

| Test | Reference | Pile type | D: mm | Length: m | Average s_u in DBC along pile shaft: kPa | Test type | Equalisation time: days |
|--------|------------------------------------|--------------|-------|-----------|--|-------------|-------------------------|
| TP1 | This paper | CFA | 762 | 12 | 420 | Compression | 24 |
| TP2 | | CFA | 762 | 13.8 | 420 | Compression | 20 |
| TP3 | | CFA | 762 | 11 | 420 | Tension | 26 |
| J1 | Farrell and Lawlor ³ | CFA | 600 | 11 | 360 | Compression | 5 |
| CPI-1C | Farrell <i>et al.</i> ² | Driven steel | 273 | 6.4 | 450 | Compression | 0.1 |
| CPI-2C | | Driven steel | 273 | 6.4 | 450 | Compression | 1.8 |
| CPI-3C | | Driven steel | 273 | 6.4 | 450 | Compression | 17 |

Table 1. Summary of instrumented pile tests in DBC

shaft of the CFA piles. The actual distribution of horizontal stress along the pile shaft will of course depend on the geological and loading history of the deposit.

The distribution of β (see equation (5)) along the pile shafts is considered by plotting the values against h/D in Fig. 8. The CFA piles show no tendency for β to vary with h/D . In contrast, the driven pile shows an apparent reduction in β with increasing h/D . Although there is obvious scatter, as would be expected given the wide variation in mobilised pile displacements and soil conditions along the pile shafts, it would appear from Fig. 8 that a reasonable lower-bound q_s value in kPa for CFA piles in the DBC would be given for glacial till by

$$q_s = 3N \quad (9)$$

The recommended β value in equation (9) equates to an α value of 0.5, given that s_u values for DBC are routinely

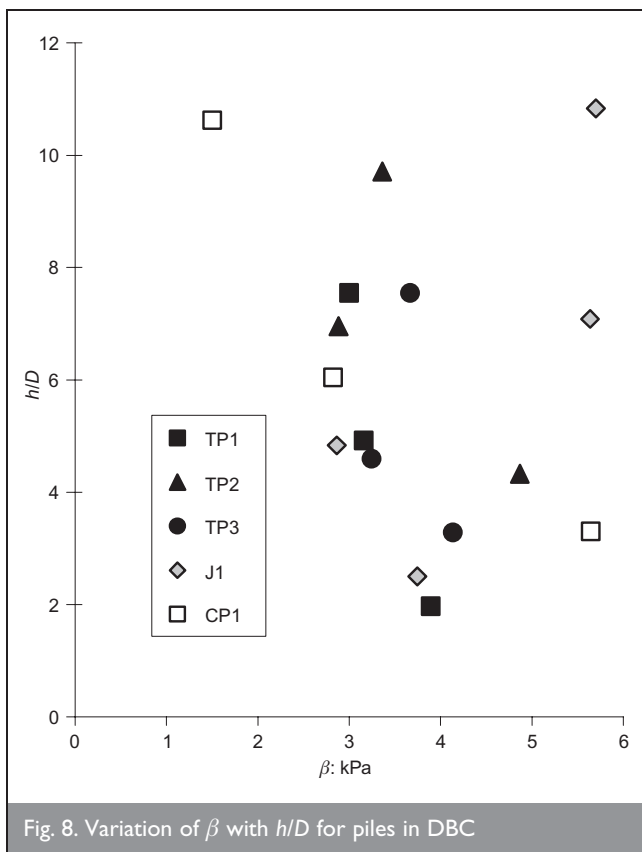


Fig. 8. Variation of β with h/D for piles in DBC

estimated using the correlation given in equation (4). The limited data would suggest that a more conservative value of β of 1.9 should be adopted for the alluvial gravels, in line with the recommendation of Robert.⁹

5.3. Base resistance

Farrell and Lawlor³ noted that the contribution of base resistance to the overall resistance of pile J1 was small. In the tests described in this paper, the very high shaft resistance mobilised during the compression tests meant that pile base displacement was low when the pile test load reached the proof load. The bearing capacity factor $N_c (= q_b/s_u)$ mobilised during the test on TP1 is compared in Fig. 9 with measurements on pile J1 and with the three compression load tests performed on the driven closed-ended pile CP1. The rates of mobilisation of the normalised pile base resistance of the two CFA piles are similar. Although the normalised base displacement of pile TP1 was less than 1% at the end of the test, the data from pile J1 suggest that N_c approaches 9 at a pile base displacement greater than 5% of the pile diameter.

Based on observations from pore pressure sensors on the pile shaft, Farrell *et al.*² suggest that the pile test performed 0.1

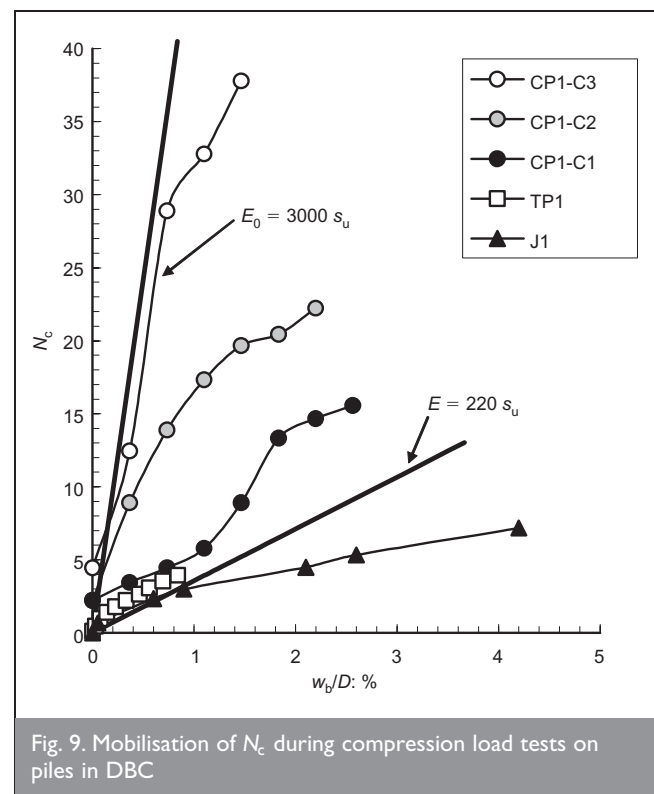


Fig. 9. Mobilisation of N_c during compression load tests on piles in DBC

days after installation (CP1-1C) was undertaken prior to significant pore pressure dissipation, whereas excess pore water pressure established during installation was almost fully dissipated before the pile test at 17 days (CP1-3C). The bearing capacity factor mobilised at relatively low pile base displacement is seen in Fig. 9 to increase from 15 to 38 during the equalisation period. A striking feature of the tests is the increase in pile base stiffness response during the equalisation period. A linear elastic secant soil modulus (E) of 100 MPa is commonly used to predict footing settlements in DBC under working stress conditions:¹¹ this corresponds to an E/s_u ratio of ~ 220 . This stiffness value is seen to provide a reasonable estimate of the footing response during the test undertaken on the driven pile prior to significant pore pressure dissipation occurring (Test 1c) and also the initial response (at normalised pile-head displacement less than 1% of the diameter) of the CFA piles. The response in the test performed on the driven pile where the excess pore pressures had reduced to near-hydrostatic levels (Test 3c) is markedly stiffer. Although no site-specific small-strain stiffness data were available for the Croke Park site, values measured by Long and Mekiti¹¹ at a number of sites and shear wave stiffness values at the Ballsbridge site suggest that E_0/s_u of 3000 is appropriate for DBC at Croke Park. This value of E_0 is seen to provide a good fit to the data from the final load tests for pile base settlements below 0.7% of the pile diameter. Gavin and Lehane²⁴ report similar results for displacement piles in sand. They suggest that the high initial stiffness is caused by a combination of prestress applied at the pile base during installation (and in this case due to previous load tests) and the high residual stresses locked in at the pile tip (evident in Fig. 9). It is apparent from the data in Fig. 9 that CFA piles installed in Dublin Boulder Clay develop much lower ultimate base resistance and exhibit a softer stiffness response than piles driven into similar materials.

6. CONCLUSIONS

The instrumented pile test results indicate that CFA piles are a very efficient pile solution in glacial till, where, depending on the pile geometry, the average shaft resistance may be higher than that mobilised by driven piles. A physical explanation for the enhanced shaft resistance of CFA piles is that the piles are unaffected by the phenomenon of friction fatigue (reduction in horizontal stress due to cyclic loading), which occurs during the installation of driven piles. Piles installed in DBC were seen to develop very high shaft resistance, with the horizontal earth pressure inferred from shear stress measurements made along the pile shaft, K_m , approaching K_p for CFA piles.

The base resistance mobilised during instrumented load tests on CFA piles was low. Because of the beneficial effects of prestressing of the pile base during installation, the contribution of pile base resistance to the axial capacity of a driven pile would be expected to be much more significant. This effect is reflected in the much lower bearing capacity factor inferred for the base resistance mobilised by CFA piles ($N_c = 9$) when compared with driven piles ($N_c \geq 38$).

ACKNOWLEDGEMENTS

The authors wish to acknowledge the following people, who were critical to the successful completion of the project: Peter Flynn, Associate Director of Arup, who was the project director and initiated discussion on performing instrumented pile tests

at the site; Philip Daynes, Eric Lynas, Pat McCann and Conor Smith of Cementation, the piling contractors on the project, who constructed the piles and installed and monitored all instrumentation; and Joe Blount, Allied Irish Banks, for allowing publication of the results. Thanks are also due to the reviewers for helpful comments on the original draft. The paper was written while the first author was working as a research associate at the Urban Institute at University College Dublin; financial assistance provided by the Institute is gratefully acknowledged.

REFERENCES

1. TRENTER N. A. *Engineering in glacial tills*. Construction Industry Research and Information Association, London, 1999, Report No. C504.
2. FARRELL E. R., LEHANE B. M. and LOOBY M. Instrumented driven pile in Dublin Boulder Clay. *Proceedings of the Institution of Civil Engineers—Geotechnical Engineering*, 1998, 131, No. 4, 233–241.
3. FARRELL E. R. and LAWLOR M. CFA pile behaviour in very stiff lodgement till. *Proceedings of the Institution of Civil Engineers, Geotechnical Engineering*, 2008, 161, No. 1, 49–57.
4. BURLAND J. Closing address. *Proceedings of the Conference on Large-Scale Piles Tests in Clay*, London, 1993, pp. 590–595.
5. LEHANE B. M., JARDINE R. J., BOND A. J. and FRANK R. Mechanisms of shaft friction in sand from instrumented pile tests. *Proceedings of the American Society of Civil Engineers—Journal of Geotechnical Engineering*, 1993, 119, No. 1, 19–35.
6. MAYNE P. and KULHAWY F. K_0 –OCR relationships in soil. *Proceedings of the American Society of Civil Engineers—Journal of Geotechnical Engineering*, 1982, 108, No. 6, 851–872.
7. FLEMING W. G. K., WELTMAN A. J., RANDOLPH M. F. and ELSON W. K. *Piling Engineering*. Surrey University Press, 1992.
8. STROUD M. A. The SPT test: its application and interpretation. *Proceedings of the Institution of Civil Engineers Conference on Penetration Testing*, Birmingham, 1989, pp. 29–49.
9. ROBERT Y. A few comments on pile design. *Canadian Geotechnical Journal*, 1997, 34, No. 4, 560–567.
10. JARDINE R. J., CHOW F. C., OVERY R. and STANDING J. *ICP Design Methods for Driven Piles in Sands and Clays*. Thomas Telford, London, 2005.
11. LONG M. and MENKITI C. Geotechnical characteristics of Dublin Glacial Till. *Géotechnique*, 2007, 57, No. 7, 595–611.
12. LEHANE B. M. and FAULKNER A. Stiffness and strength characteristics of a hard lodgement till. *Proceedings of the 2nd International Symposium on the Geotechnics of Hard Soils and Soft Rocks, Naples*, 1998, 2, 637–646.
13. LEHANE B. M. and SIMPSON B. Modelling the behaviour of a glacial till using a 3-D BRICK soil model. *Canadian Geotechnical Journal*, 2000, 37, No. 5, 1078–1088.
14. INSTITUTION OF CIVIL ENGINEERS. *Specification for Piling and Embedded Retaining Walls*, 1st edn. Thomas Telford, London, 1996.
15. ENGLAND M. Easy static load tests: expert results.

- Proceedings of the 9th International Conference on Piling and Deep Foundations, Nice, 2002, 657–662.*
16. FELLINIUS B. H. From strain to load in an instrumented pile. *Geotechnical News Magazine*, 19, No. 1. 35–38.
 17. LEHANE B. M., PENNINGTON D. and CLARK S. Jacked end-bearing piles in the soft alluvial sediments of Perth. *Australian Geomechanics Journal*, 2003, 38, No. 3, 123–134.
 18. CADOGAN D. *Field Experiments on Non-Displacement Piles*. PhD thesis in preparation, School of Architecture, Landscape and Civil Engineering, University College Dublin, 2008.
 19. BORCHI X., WHITE D. J., BOLTON M. D. and SPRINGMAN S. Empirical pile design based on cone penetrometer data: an explanation for the reduction of unit base resistance between CPTs and piles. *Proceedings of the 5th International Conference on Deep Foundations Practice, Singapore, 2001, 125–132.*
 20. DELPAK R., ROBINSON R. B. and OMER J. R. Assessment of the performance of large diameter bored cast in-situ piles formed in Mercia Mudstone. *Proceedings of the CIRIA Conference on the Engineering Properties of the Mercia Mudstone Group, Derby, 1998, pp. 36–63.*
 21. GALLAGHER D. *An Experimental Investigation of Open and Closed Ended Piles in Belfast Soft Clay*. PhD thesis, University College Dublin, 2006.
 22. BOND A. and JARDINE R. J. Effects of installing displacement piles in a high OCR clay. *Géotechnique*, 1991, 41, No. 3, 341–363.
 23. GAVIN K. G. and O'KELLY B. Effect of friction fatigue on piles in dense sand. *Proceedings of the American Society of Civil Engineers—Journal of Geotechnical and Geoenvironmental Engineering*, 2007, 133, No. 1, 63–71.
 24. GAVIN K. G. and LEHANE B. M. Base load–displacement response of piles in sand. *Canadian Geotechnical Journal*, 2007, 44, No. 9, 1019–1052.

What do you think?

To comment on this paper, please email up to 500 words to the editor at journals@ice.org.uk

Proceedings journals rely entirely on contributions sent in by civil engineers and related professionals, academics and students. Papers should be 2000–5000 words long, with adequate illustrations and references. Please visit www.thomastelford.com/journals for author guidelines and further details.