The influence of residual loads on the axial load–displacement response of piles in sand

David J.P. Igoe[1], Kenneth G. Gavin[2] and Brendan C. O'Kelly[3]

[1] Research Assistant, School of Civil, Structural and Environmental Engineering, University College Dublin, Newstead Building, Belfield, Dublin 4, Ireland.
Tel: +3531 716 3229, fax: + 3531 7167399, email: david.igoe@ucd.ie

[2] Senior Lecturer, School of Civil, Structural and Environmental Engineering, University College Dublin, Newstead Building, Belfield, Dublin 4, Ireland.
Tel: +3531 716 3222, fax: + 3531 7167399, email: kenneth.gavin@ucd.ie

[3] Associate Professor, Department of Civil, Structural and Environmental Engineering, Trinity College Dublin, Dublin, Ireland.
Tel: +3531 896 2387, email: bokelly@tcd.ie

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ABSTRACT

This paper investigates the effects of residual loads developed during installation on the axial load–displacement response of displacement piles at two sites. At the first site, open- and closed-ended piles were installed by jacking into a loose sand deposit. At the second site, an open-ended pile was driven into medium dense sand. The instrumented piles used were of twin-wall construction, allowing separation of the base and shaft loads during pile installation and load testing, thereby facilitating accurate measurements of the residual stresses. For open-ended piles, the development of residual stresses during installation was found to depend on the degree of soil plugging. The pile jacked into the loose sand became fully plugged, resulting in relatively large end-bearing stresses mobilized during installation. Between jacking strokes (when the pile head was unloaded), large residual shear stresses developed at the pile–soil interface on both the external and internal walls of the pile; hence relatively large end-bearing stresses were able to mobilize. Piles that developed large residual stresses exhibited stiffer load–displacement responses when loaded in compression. However, during first-time loading in tension, the piles exhibited relatively soft stiffness responses which were due to the presence of residual stresses.

Keywords: open-ended pile; plugging; residual load; tension loading

INTRODUCTION

During installation of displacement piles, the jacking or driving energy causes the pile material and the soil beneath the pile tip to compress. Removal of this energy causes a rebound of the affected soil and elastic decompression of the pile. As only small displacements are required to mobilize shaft friction, this rebound can mobilize negative shear stresses, particularly along the upper portion of the pile. Since larger movements are required to unload the pile base, the negative shaft friction mobilized during rebound can lock-in some of the base load. The resulting residual base load (Q_{res}) remaining on the pile is counteracted by the negative residual shear stresses (τ_{res}). The presence of residual loads can significantly affect the pile behavior. For instance, Gavin and Lehane (2007) performed field tests on instrumented open- and closed-ended piles and found that the pile base stiffness in compression was strongly dependent on the magnitude of the residual load present. Alanweh (2005) performed numerical analyses and proposed a method for predicting the load–displacement response of piles driven in sand. He noted that piles which developed significant residual loads during installation exhibited a much softer stiffness response during tension loading compared with piles having no residual loads present. This was attributed to the residual loads effectively pre-loading the pile shaft in tension.

In this paper, the installation and axial load testing of two instrumented open-ended piles and one instrumented closed-ended pile are presented. The open-ended piles were of twin-wall construction which allowed independent measurements of the external shaft friction and base loads (end bearing and internal shaft friction components), thereby allowing accurate assessment of the residual base load. This paper aims to improve the understanding of pile behavior, particularly regarding the effect of residual loads on the pile load–displacement response, as well as adding to the limited number of quality residual load measurements reported in the literature.

EXPERIMENTAL INVESTIGATIONS

Loose Sand Bed, Blessington Test Site

Field tests were performed at the University College Dublin (UCD) geotechnical research site at Blessington, County Wicklow, Ireland. Details of the natural ground conditions comprising glacially-deposited very dense fine sand deposits, with the water table located approximately 13 m below ground level (bgl), have been reported by Gavin and O'Kelly (2007), Gavin et al. (2009), Doherty et al. (2012) and Igoe et al. (2011), among others. These deposits typically have cone penetration test (CPT) end resistance (q_c) values ranging 15–20 MPa and a small strain shear stiffness ranging 100–150 MPa. The loose sand test bed for the present investigation was created by excavating a 10 m long, 2.5 m wide and ~ 6 m deep trench in the very dense sand deposit. The heavily over-consolidated nature of the deposit allowed the vertical trench to be maintained without support until such time that the trench was backfilled with the excavated sand using the raining deposition technique and a minimum drop height of 1.0 m. In the trench, the measured water content increased with depth from ~ 8% at the ground surface to 12% at 4.5 m bgl. Four profiles of the CPT q_c and sleeve friction (f_s) against depth were taken at 2 m horizontal spacing along the backfilled trench 7 days its formation, with another 4 CPT tests performed 7 months later, with the average, maximum and minimum q_c and f_s values presented in Figs. 1a and 1b. These traces indicate that the backfilled sand trench was relatively uniform, with a CPT q_c of ~ 0.8–1.0 MPa between 2 m bgl and the bottom of the trench located at ~ 6 m bgl. The slightly elevated q_c values in the uppermost 2 m reflect the zone affected by trafficking of the truck during the CPT testing. Multiple Analysis of Surface Waves (MASW) tests provided the small-strain shear stiffness (G_0) profile with depth (Fig. 1c).

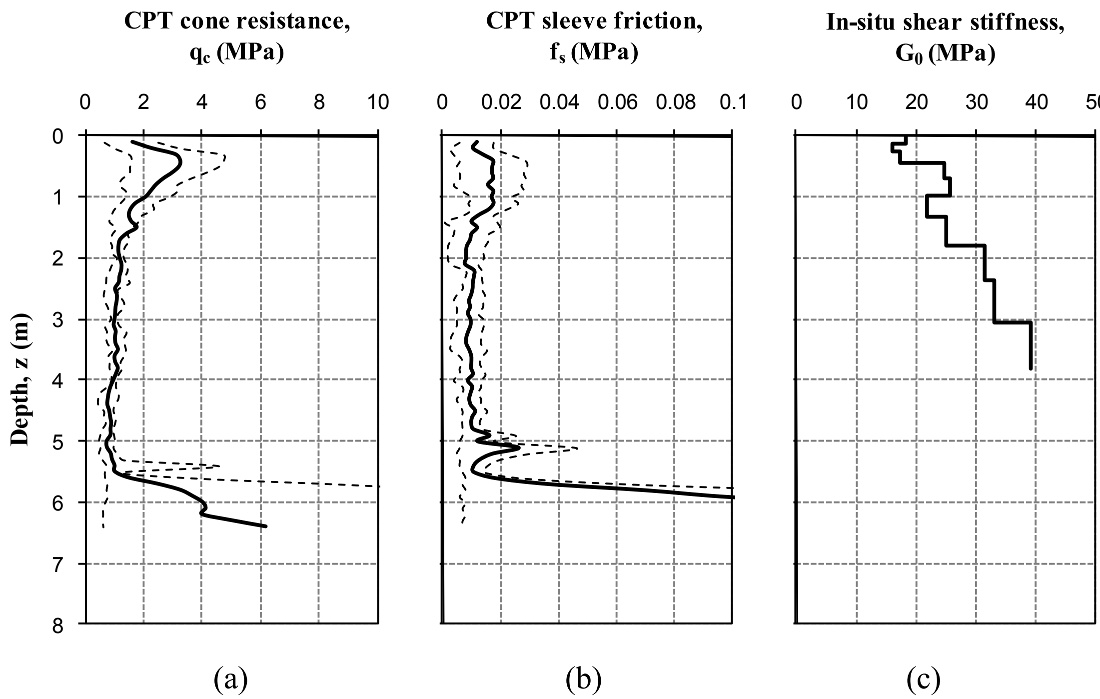

Figure 1 Blessington (a) CPT cone resistance q_c; (b) sleeve friction f_s; (c) in-situ shear stiffness G_o as determined by MASW

Laboratory testing was performed to supplement the in-situ tests and establish basic soil properties. The Blessington sand was classified as fine to medium grained, with a mean particle size (D_{50}) ranging 0.1–0.15 mm. Shear box tests indicated a constant-volume friction angle of 36°, with an interface friction angle of ~30° measured for a stainless steel interface.

Mortarstown Test Site

Field testing was also performed at the Mortarstown test site, County Carlow, Ireland. Details of the site have been described by Igoe et al. (2013). A 12 m deep cable-percussion borehole drilled approximately 3 m from the test pile indicated a gravelly clayey silt layer to 1.2 m bgl overlying a medium dense sand (D_{50} = 0.22 mm) layer to 12 m bgl, with the groundwater table located at ~ 6 m bgl. The sand layer was inter-bedded by medium dense gravel from 6.2 to 7.2 m bgl. A second borehole installed ~ 10 m from the test pile indicated a sandy gravelly clay layer to 2.3 m bgl overlying gravel to 10 m bgl, illustrating the variability of the ground conditions across the test site. A 300 mm deep gravel platform was placed on the ground surface to facilitate the piling works.

In-situ tests at the site included standard penetration tests (SPTs), three CPTs and MASW tests (Fig. 2). The tests CPT1, CPT2 and CPT3 were performed 2, 4 and 8 m away from the pile location respectively. The SPT N-values typically ranged 15–20 (Fig. 2d), with occasional higher N values in Irish glacial deposits indicative of the presence of cobbles. The CPT q_c and f_s values were highly variable in the gravelly clayey silt layer. For the underlying sand layer, the CPT1 q_c values remained consistent at ~ 8 MPa between 1.2 and 5 m bgl, increasing to 12–20 MPa at greater depth. The MASW G_0 values ranged 120–190 MPa between 1.2 and 5.5 m bgl, below which G_0 increased sharply to greater than 300 MPa.

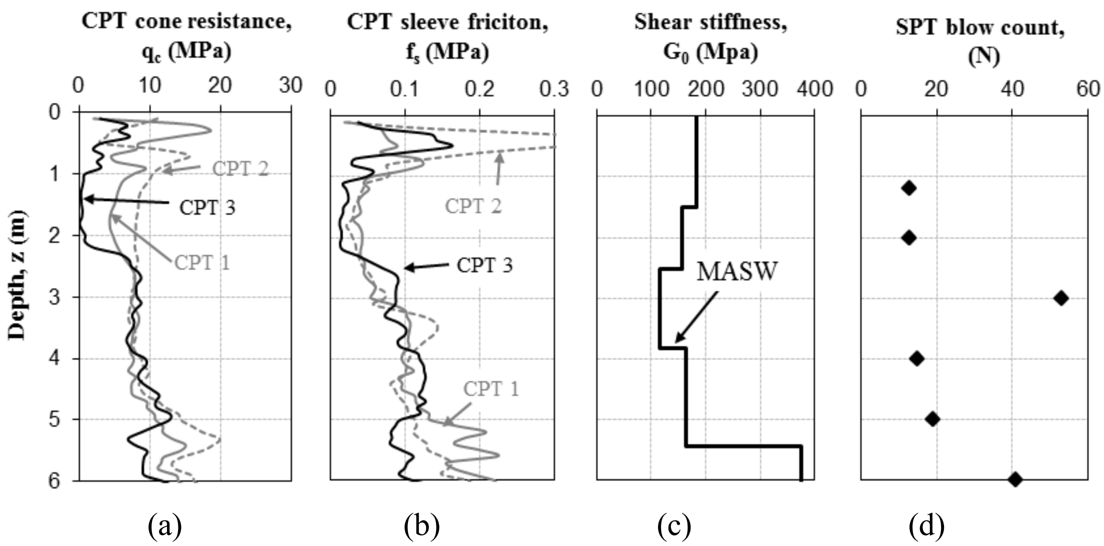

Fig. 2 Mortarstown (a) CPT cone resistance q_c; (b) CPT sleeve friction f_s; (c) in-situ shear stiffness G_0 as determined by MASW; (d) SPT blow count.

UCD Open-Ended Model Pile

The University College Dublin open-ended model pile was used for the Blessington tests. This pile is of stainless steel construction, with an external diameter (D) of 168 mm and 9 mm wall thickness, with an internal diameter of 150mm. Its lower instrumented section, 2.0 m in length (Fig. 3a), was constructed using the twin-walled technique, where two strain-gaged pipes having slightly different diameters are joined at the top, thereby allowing independent measurements of the internal and external shaft resistances and also of the base load developed on the pile annulus. Three instrumented units housing radial total stress and pore pressure sensors were located at h/D = 1.5, 5.5 and 10.5: where h is the distance measured from the pile tip (see Fig. 3a). In addition, a total stress sensor mounted flush in the pile annulus allowed independent measurements of the annular and plug stresses. This arrangement also facilitated measurement of the residual load acting on the pile annulus for the present investigation. 1.0 m long extension sections machined from the same pipe material were used to allow pile installation to 6 m bgl. Full details of the design, construction and calibration of the UCD open-ended model pile are presented in Igoe et al. (2010a, b). This pile can also be installed in a closed-end mode by fitting a specially designed adaptor plate over the annulus, covering the pile base area. Additional strain gages fitted to the inner wall surface of the pile's inner tube in a full-bridge configuration helped determine the residual load and compensate for bending effects.

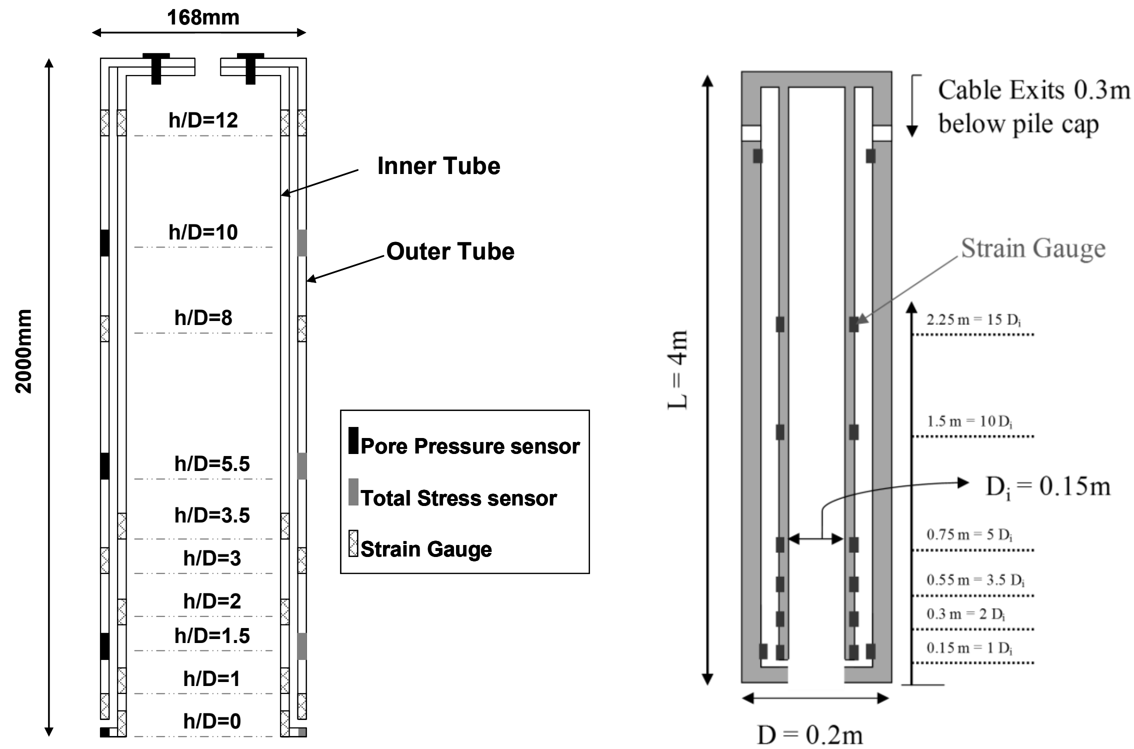

Fig. 3: Open-ended pile constructions and instrumentation layouts: (a) UCD model pile (Igoe et al., 2010a, b); (b) Mortarstown pile (IP-240).

Mortarstown Open-Ended Pile

The Mortarstown instrumented pile (IP-240) was also constructed using a twin-wall design. Its outer tube was a 4 m long heavily rusted steel pile, with an external diameter of 200 mm and 10 mm wall thickness (Fig. 3b). Its stainless steel inner tube had an internal diameter of 150 mm and 2 mm wall thickness. The inner and outer tubes were welded to a top plate, forming a rigid connection. A 25 mm thick annular plate was welded to the outer tube at the pile toe end. A 2 mm clear gap between the annular plate and the inner tube (sealed using a flowable rubber compound) prevented any load transfer between the plug formed during driving and the rest of the pile. The pile was instrumented with uniaxial foil strain gages as shown in Fig. 3b. Due to heavy rusting of the outer tube, it was necessary to grind flat areas onto the pile wall surface to ensure full bonding of the strain gages. Difficulties in grinding its internal wall surface meant that that the gages could only be placed near the ends of the tube. Three gages were attached, 120º apart, directly above the annulus and directly below the cable exit holes, allowing accurate separation of the annular and outer shaft loads. The inner tube had 6 levels of three strain gages to provide a breakdown of the internal shear stresses and plug load. The wiring from the gages was housed in the void between the two tubes, exiting at the side of the pile, 300 mm below the top plate, to prevent damage from excess heat transfer during pile fabrication (i.e. welding of the tubes to the top plate). The cable exit holes were sealed using a flowable rubber compound. The plug length formation during was monitored using a plumb line accessed through a hole in the top plate.

Blessington Testing Program

The UCD instrumented pile was installed both open- and closed-ended in two separate installations. The open-ended installation (OE1) involved jacking the pile from the bottom of a 1.9 m deep starter hole at a rate of 20 mm/s to a final depth of 5.9 m bgl in the loose sand trench (Fig. 4), using a 20-tonne CPT truck as reaction. The installation was paused and the pilehead completely unloaded after each 100 mm of penetration to monitor the plug development and residual stresses mobilized during installation. Static compression and tension load tests were performed immediately following installation. The tension load tests were used to verify the residual base loads measured throughout the installation and testing.

The closed-ended installation (CE2) involved jacking the pile from 1.8 to 5.5 m bgl, again using 100mm strokes and a rate of 20 mm/s. Some damage to the pile extension sections occurred prior to the start of this test which prevented their use. CPT cone rods were attached to the head of the 2.0 m long instrumented pile section instead, allowing it to be jacked to its final depth of 5.5 m bgl (see CE2 installation in Fig. 4). In other words, the pile CE2 had an embedded length of 2.0 m, half that of pile OE1. Due to difficulties with the CPT attachments, no static load tests were performed on pile CE2. The pile installation and testing program is summarized in Table 1.

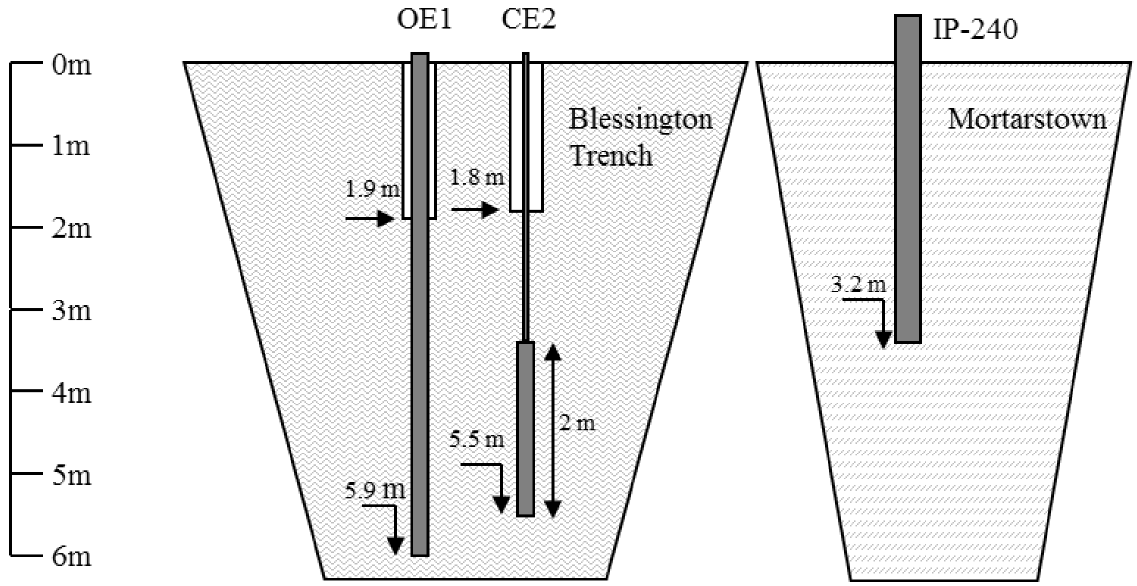

Fig. 4: Schematic of pile installations at the two test sites.

Table 1: Summary of pile testing program.

Pile	**Details**	**Comments**
Blessington OE1	- Jacked installation in 100 mm stokes - Maintained compression load test - CRE tension load test to 13 mm - CRE tension reload to 4 mm	Starting depth = 1.9 m Final depth = 5.9 m Final pile embedment = 4.0 m
Blessington CE2	- Jacked installation in 100 mm stokes - No load tests	Starting depth = 1.8 m Final depth = 5.5 m Final pile embedment = 2.0 m
Mortarstown IP-240	- Driven installation using 0.8 m strokes - Compression test to 180 kN - CRE tension test 1 - CRE tension test 2	Final depth = 3.2m 0.03 mm displacement 28 mm displacement 27 mm displacement

Mortarstown Testing Program

The Mortarstown instrumented pile (IP-240) was driven using a Junttan PM16 piling rig (Fig. 5a) having a 3000 kg ram weight, 0.8 m stoke and capable of producing a maximum driving energy of 24 kN.m. Driving was stopped at 3.2 m bgl to avoid possible damage of the cables from the pile instrumentation. Both compression and tension static load tests were performed using the 20 tonne CPT truck as a reaction. In order to provide sufficient clearance for the truck to locate above the pilehead, it was necessary to raise the ground level locally around the pile by placing a 0.4 m deep gravel layer (Fig. 5b). Delays in the availability of the CPT truck resulted in the pile being load tested 240 days after its installation. A compression load of 180 kN (capacity of the CPT truck) was applied to the pilehead causing a maximum pilehead displacement of only 0.2 mm. When the load was removed, the pilehead displacement was nearly fully recovered, with a net pilehead displacement of 0.03 mm recorded. Immediately after the compression test, two constant rate of extraction (CRE) tension load tests were performed (see Table 1)

Fig. 5 (a) Pile driving and (b) Pile load testing at the Mortarstown site.

EXPERIMENTAL RESULTS

Blessington Pile Installations

Open-ended piles differ from closed-ended piles in the way that they carry the load. Internal shear stresses develop as soil enters an open-ended pile may induce different degrees of plugging. In this case, the base resistance can be taken as the end-bearing developed beneath the pile annulus and soil plug. The development of the internal soil plug is often described by the incremental filling ratio (IFR, as %):

$$IFR = (\Delta L_{plug} \text{ x } 100)/\ \Delta L \qquad [3]$$

where ΔL_{plug} is the change in plug length for a given penetration increment ΔL.

Thus the IFR reduces from 100% when fully coring to zero when fully plugged. The IFR profile measured during installation of open-ended pile OE1 is shown in Fig. 6a. This pile remained fully coring for only the first 100 mm jacking stroke (i.e. from 1.9 to 2.0 m bgl), thereafter experiencing significant plugging. After 12 jacking strokes (i.e. at 3.1 m bgl), the pile became fully plugged, remained so until the final two jacking strokes, where it became unplugged (IFR value increased), as it approached the very dense sand at the bottom of the trench.

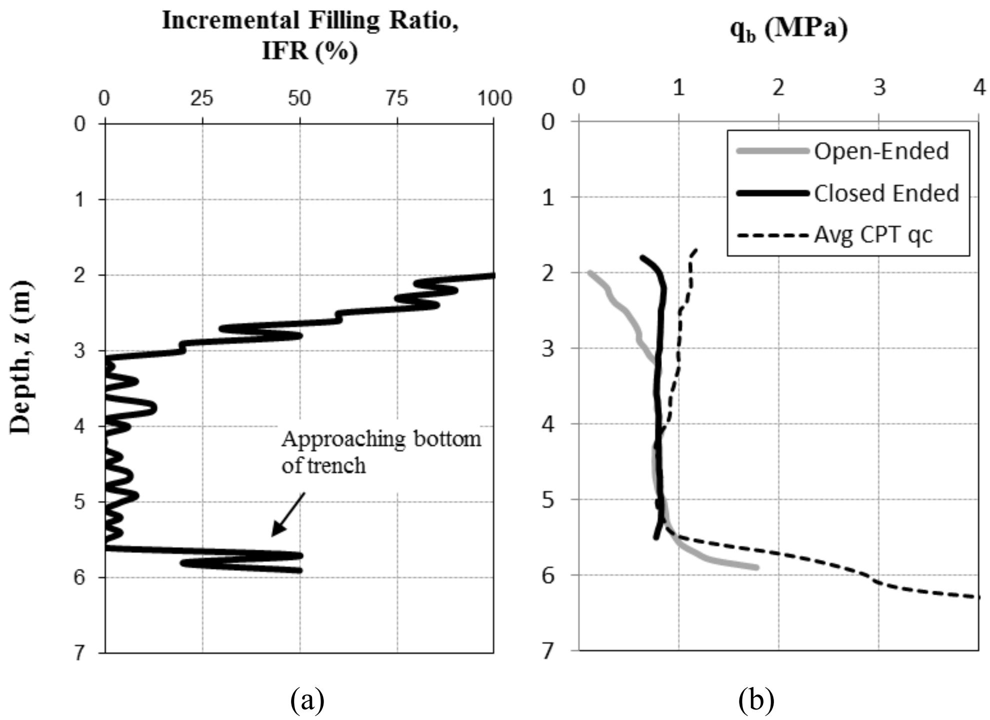

Fig. 6 Pile installations at Blessington: (a) incremental filling ratio for pile OE1; (b) unit base resistance.

One of the key aspects of the pile load tests was to allow accurate measurement of the load distribution between the shaft and base of the pile using its twin walled design. The internal and external strain gages were monitored throughout the pile installations, allowing a consistent profile of the shaft and base loads to be derived. The unit base resistance (q_b) mobilized during the installation of the open- and closed-ended piles are shown in Fig. 6b. These are compared with the measured CPT q_c profiles, averaged using the Dutch averaging technique (as recommended by White and Bolton (2005)) to account for the difference in diameters of the pile and CPT cone. For the open-ended pile, the q_b value increased steadily between 2.0 and 3.2 m bgl, corresponding to the depth range over which IFR reduced and the onset of significant plugging. Once the pile was fully plugged, the q_b value remained approximately constant and only began to increase again at ~ 5.5 m bgl, as the pile approached the very dense sand at the bottom of the trench. In contrast, the q_b value for the closed-ended pile became approximately constant after 0.2 m penetration (i.e. at 2 m bgl), remaining steady throughout the installation and approximating the base load developed by the plugged open-ended pile up to 5.5 m bgl. The q_b values mobilized by both piles were slightly lower than the CPT q_c during the initial stages of installation, becoming approximately equal below 4.0 m bgl; i.e. after ~ 2.2 m penetration equivalent to 13 D.

The load distribution measured during installation of the open- and closed-ended piles are shown in Fig. 7. For the open-ended pile, Fig. 7a shows that the installation shaft resistance (Q_{shaft}) remained low until the pile had penetrated more than 1.7 m (~ 3.6 m bgl). Beyond this region, Q_{shaft} increased continuously as the pile shaft extended into the zone where the pile had remained fully plugged, reaching ~ 10 kN at the end of installation. Figure 7b shows that for the closed-ended pile, the Q_{shaft} value increased continuously until 3.8 m bgl, after which the 2.0 m long instrumented pile section was fully embedded (refer to CE2 in Fig. 4), after which the Q_{shaft} value remained constant at ~ 6 kN.

For the closed-ended pile (Fig. 7b), the residual base load (Q_{res}) approximated the shaft resistance throughout the installation; i.e. full installation shaft resistance mobilized in residual tension. For the open-ended pile, the Q_{res} value remained small until the pile became heavily plugged (IFR<50%) below ~ 2.6 m bgl, thereafter increasing steadily. Below 2.6 m bgl, Q_{res} ranged between 75% and 100% of Q_s for the remainder of the installation depth. It is suggested that the very high Q_{res}/Q_{shaft} ratio values are linked to the very high ratio of base to shaft resistances mobilized during installation. As the base resistance is significantly greater than that of the shaft, only a fraction of the base load needs to be 'locked in' to fully mobilize the shaft capacity in residual.

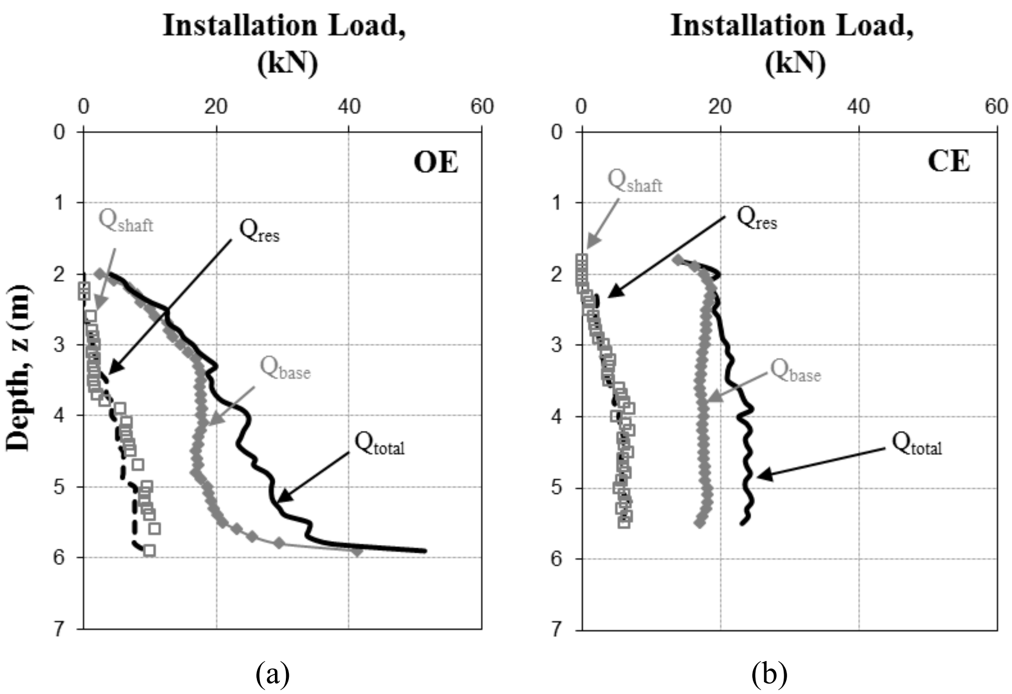

Fig. 7 Pile installation loads: (a) open ended; (b) closed ended.

Blessington Load Tests (Pile OE1)

Figure 8a shows the load–displacement curves describing separately the total, base and shaft loads mobilized during the maintained load compression test followed by the two CRE tension tests performed on the open-ended pile. For the compression test, the load was increased in increments until a pile head displacement of 26 mm (~ 15% D) after which it was unloaded. The following are noted:

- A maximum total load of 55 kN was measured, comprising 45 and 10 kN in base and shaft resistance respectively.
- The base and shaft stiffness were initially quite high but degraded significantly after a pile head displacement of 2–3 mm. The continuing increase in shaft load with displacement during the compression test may be caused by the lowermost 0.2 m length of the pile which was located in the zone where high base stresses were noted during installation (i.e. below 5.7 m bgl, see Fig. 7a).
- During unloading, the pile head rebounded by ~ 2 mm, which was sufficient to mobilize a residual base load of 8.0 kN. Referring to the shaft unload curve, the shaft capacity was fully mobilized in residual tension after ~ 1 mm of rebound, but the pile continued to displace upward until its base had unloaded sufficiently to match the tension capacity of the pile; i.e. until the base and shaft loads were in equilibrium.
- The full mobilization of the shaft capacity in residual tension may be explained by the very high ratio of base to shaft load noted at the end of installation (i.e. as the pile toe approached the very dense sand below the bottom of the trench) while the pile shaft remained primarily embedded in the loose trench sand.

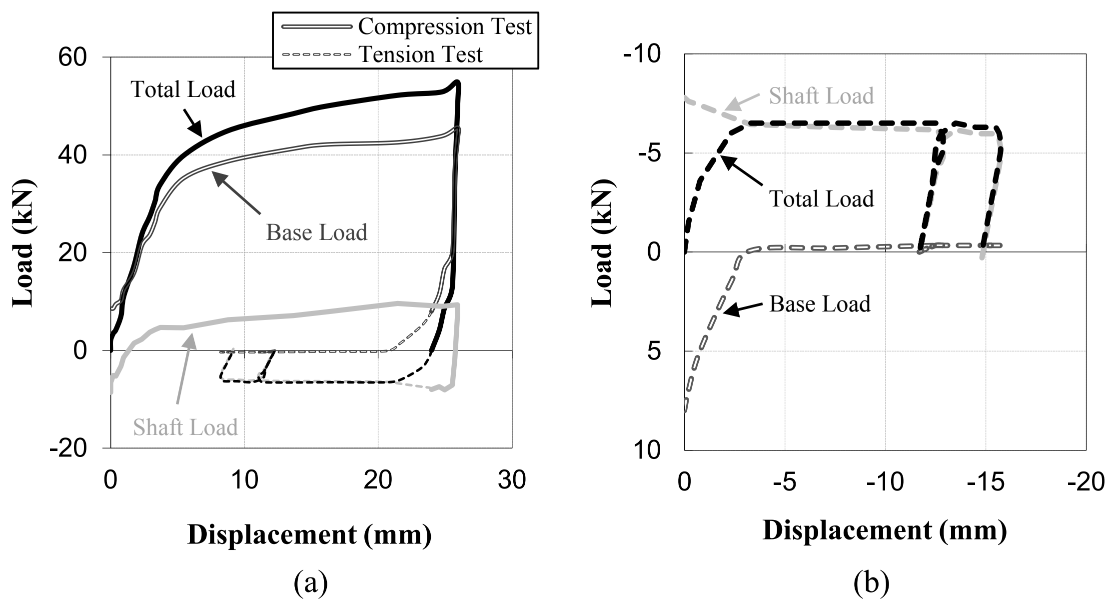

Fig. 8 Open-ended pile: (a) compression and tension load tests; (b) tension load tests only.

The results of the two tension load tests are shown more clearly in Fig. 9b. During the first test, the pile was extracted at a constant rate until a pile head displacement of 13 mm was achieved before fully unloading. The following are noted:

- The maximum pile head load measured was 6.5 kN, compared with the 8.0 kN residual tension shaft load mobilized pre-test. This behavior is thought to be caused by the rotation of the principal stresses and reduction in the mean stress level near the pile toe as a result of the removal of the residual base load, which essentially ‘unlocked’ the highly stressed zone near the pile toe.

- By approximately 3 mm upward displacement (~ 5 mm including the rebound at the end of the compression test), as the residual base load was completely removed, the shaft load reduced from its initial value of 8.0 kN to 6.5 kN, thereafter remaining steady until unloading.

The second CRE tension test was performed immediately after the first; with the pile reloaded in tension until a pile head displacement of 4 mm was achieved and then fully unloaded (Fig. 9). The relatively soft stiffness during first-time tension loading can be explained by the residual base load effectively preloading the pile shaft, with the uplift of the pile only serving to remove the residual base load. In this respect, the pile head tension load–displacement response was primarily dependent on the base unloading stiffness, rather than that of the shaft. The reloading test showed a stiffer response, whereby a peak load was reached after ~ 1.8 mm, followed by a brittle response and slight drop in load.

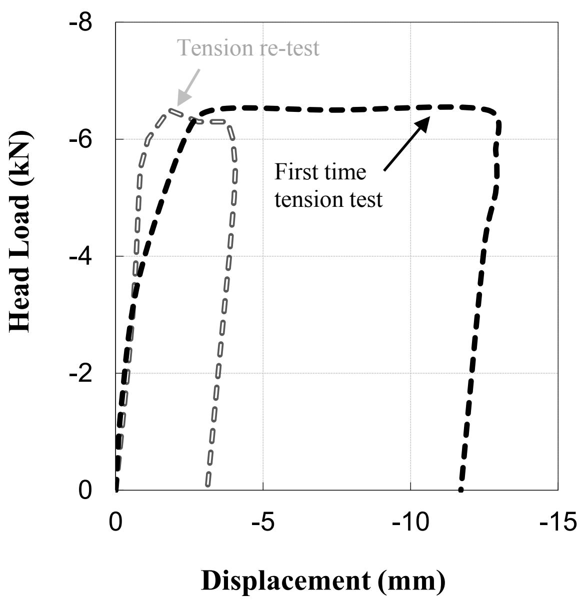

Fig. 9 Comparison of first-time tension loading and re-loading for open-ended pile OE1

Mortarstown Instrumented Pile (IP-240)

Figure 10 shows the piling rig blow count and the development of the soil plug monitored during installation of the Mortarstown pile. The pile was installed in 54 blows and was fully coring to 0.5 m bgl, below which the IFR value consistently reduced with depth, reaching a final IFR of ~ 25% at the end of driving.

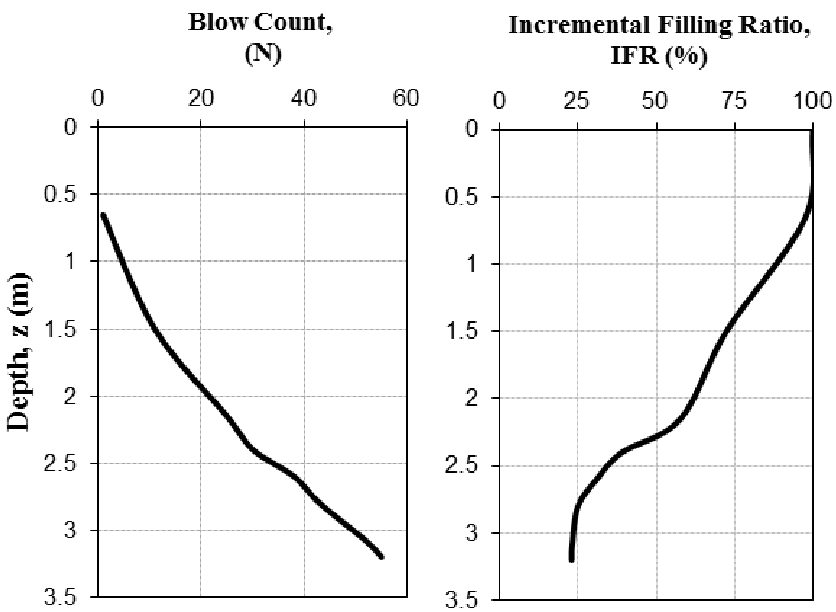

Fig. 10 Installation of Mortarstown IP-240 pile: (a) piling rig blow count; (b) incremental filling ratio.

240 days after its installation, the pile was loaded in compression although a permanent displacement of only 0.03 mm was achieved from the maximum 180 kN compression load that could be provided by the weight of the CPT truck. Two CRE tension tests were performed immediately afterwards, with the load–displacement responses shown in Fig. 11. During the first tension test, the pile was continuously jacked to a pilehead displacement of 28 mm before unloading. The pile was immediately re-tested in tension to a further 27 mm of pilehead displacement. Referring to Fig. 11, the following are noted:

- At the start of the first tension test, the pile had a residual compressive base load of ~ 40 kN, balanced by a residual tension shaft load of 40 kN. The total pile load was seen to increase continuously, reaching a maximum of 120 kN for a pile head displacement of 22 mm.
- The second tension test, performed immediately after the first, produced a maximum total pile load of 110 kN, mobilized after an additional upward displacement of 20 mm.
- The residual base load was completely removed after ~ 15 mm of upward displacement after which a small (apparent) base load was mobilized owing to the self-weights of the pile and soil plug.

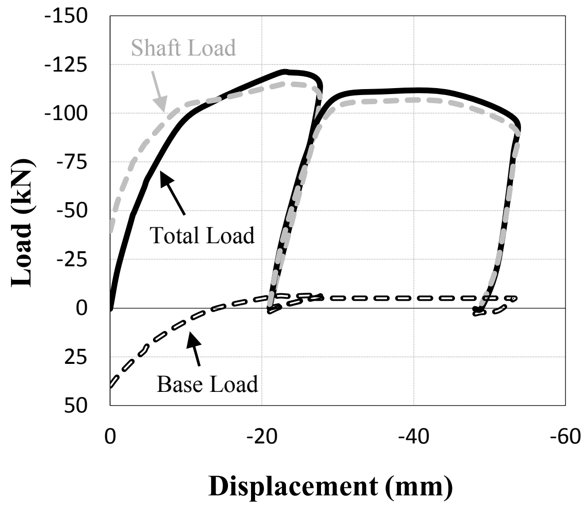

Fig. 11 Tension load tests on Mortarstown IP-240 pile

Referring to Fig. 12a, the total load–displacement response for the tension re-test indicated greater stiffness when compared against the first-time tension test results. However, a comparison of the shaft load response from both tests (Fig. 12b) indicates that for low displacements, the first-time test mobilized greater shaft load (on account of the residual load present), with both tension tests mobilizing similar shaft loads for pile head displacements greater than 6 mm. Hence the residual base load has a significant influence on the tension stiffness during the first-time test.

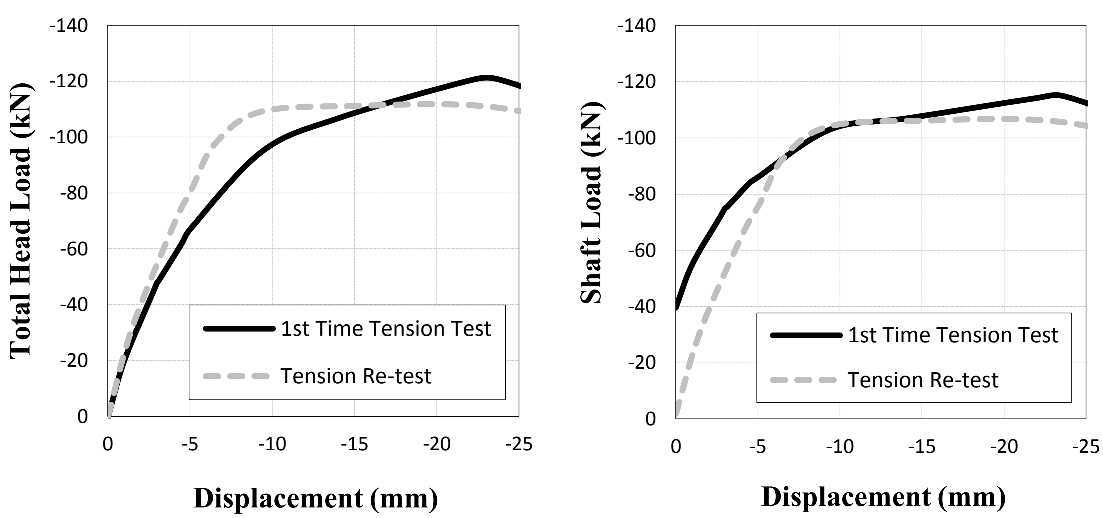

Fig. 12 Load–displacement responses for tension tests: (a) total pile head load; (b) shaft resistance.

DISCUSSION

The axial load–displacement response of a pile is controlled by the pile stiffness as well as the shape of the load transfer curves for the soil around the pile (Randolph 2003). The presence of large residual loads on the pile can result in the shaft being preloaded in tension, thereby reducing the operative stiffness of the load transfer curves when a tension load is applied to the pile head. The field tests presented in this paper for D = 168 and 200 mm have shown that the presence of large residual loads, mobilized during pile installation, can have a significant effect on the first-time load–displacement response under tension loading, particularly for open-ended piles. A final case history of tension load tests on a 7.0 m long, 340 mm diameter open-ended steel pile driven into the very dense sand deposit at the Blessington site is presented in Fig. 13. The pile exhibited similar load–displacement behavior to previous tests for the Blessington and Mortarstown sites reported in the paper, with the tension re-test (which had no residual base load present) showing a stiffer and more brittle response than the first-time tension test performed immediately beforehand. However, it should be noted that for the majority of large diameter, slender open-ended piles used in major infrastructural and offshore projects, the formation of a soil plug during pile installation is less likely to occur. Under these circumstances, the influence of residual loads is expected to be minor compared with the cases reported in this paper.

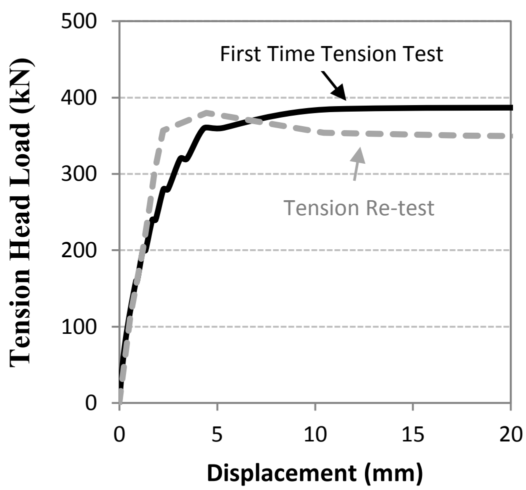

Fig. 13 Tension load tests on 340 mm diameter open-ended pile installed in very dense Blessington sand.

CONCLUSIONS

The pile tests described in this paper provide an insight into residual load behavior for open- and closed-ended jacked and driven piles in sand. The key findings are summarized as follows:

- During installation of the Blessington jacked open- and closed-ended piles, it was noted that, upon unloading, a very high proportion of the shaft resistance was mobilized in negative residual shaft friction. This was attributed to the very high ratio of base to shaft loads recorded during installation.
- At the end of installation, the negative residual shaft load exceeded the pile pull-out capacity. During first-time tension loading, the shaft load reduced as the tension load was applied to the pile head which displaced upward. This behavior was attributed to the rotation of the principal stresses and reduction in the mean stress level near the pile toe, which occurred on account of the removal of the residual base load, essentially 'unlocking' the highly stressed zone near the pile toe.
- A tension re-test performed directly after first-time tension loading showed a significantly stiffer pile head load–displacement response. The tension re-test also exhibited a brittle response, with a peak tension load achieved for a small pile head displacement. The authors suggest that the load–displacement response of first-time tension tests can be highly influenced by the residual base load, effectively preloading the shaft with tensile residual shear stresses. During first-time tension loading, the residual base load acts in addition to the applied tension load to displace the pile upward, masking the true shaft load–displacement response. Tension re-tests, in which the residual base loads have been removed during the preceding tension test, were found to be only dependent on the shaft load response, exhibiting stiffer and more brittle behavior.
- The difference in stiffness between first time tension tests and tension re-tests is not solely dependent on the residual stress. The stiffer response noted during re-tests are also influenced by particle and principal stress re-orientation to withstand the applied stresses during the preceding tests.
- Tests on the Mortarstown driven open-ended pile corroborate the findings from the Blessington jacked pile tests, highlighting the influence of residual loads on the tension load–displacement response of piles in sand.

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