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Base Resistance of an Open-Ended Pile Installed in Medium Dense Sand

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Abstract

The paper presents the results from the installation and load testing of an instrumented, open-ended model pile in a loose to medium dense sand at Donabate, County Dublin. The model pile had an external diameter of 168 mm, a wall thickness of 9.0 mm, and its twin wall construction allowed the continuous measurement of the internal and external shaft loads, the radial effective stress and the internal soil column length (plug length) throughout testing. The tests were performed to study the factors that affect plug formation and the effects of soil plugging on the shaft and base resistances mobilised during installation. The onset of soil plugging (quantified by the incremental filling ratio, IFR) coincided with a drop in the CPT q_c resistance value suggesting that plugging depended on differences between the soil states at the pile base and the soil in the pile plug. The installation load appeared to be a function of the values of q_c and IFR with the resistance decreasing as the IFR reduced and increasing as the pile moved through a weaker layer. The majority of the installation load was resisted by the pile base through a combination of the stresses mobilised beneath the pile annulus and the shear stresses mobilised in the soil plug. The annular stresses were approximately equal to the CPT q_c base resistance. A comparison of the measured values of the base resistance with that predicted by a method which uses q_c and IFR values as input parameters showed good agreement. Hence, the pile installation resistance and axial load capacity could be predicted with confidence if IFR could be predicted prior to pile installation.

Keywords: Open-ended; Pile; Installation; Sand

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INTRODUCTION

The axial resistance of piled foundations is derived from the end bearing stresses developed at the pile toe and the shaft frictional resistance developed between the pile shaft and the surrounding soil. For an open-ended pile (where a soil plug forms inside the pile), the unit base resistance q_b , is a combination of the stresses developed around the pile annulus q_{ann} , and the stresses developed by the pile plug q_{plug} , and is given as:

$$(1) \quad q_b = \frac{q_{plug} R_i^2 + q_{ann} 2Rt}{R^2}$$

where R is the external radius, R_i is the internal radius, and t is the wall thickness of the pile.

Model pile tests [1] have shown that the annular resistance that develops at the base of pipe piles q_{ann} , was approximately equal to the Cone Penetration Test (CPT) end resistance (q_c). The plug resistance q_{plug} , was found to be a function of both q_c and incremental filling ratio (i.e. rate of change of the plug height with respect to the pile penetration). The ratio q_{plug}/q_c was found to have varied linearly from a minimum value of 0.15 when the pile had been installed in a fully coring mode (IFR=1) to unity when the pipe pile had been driven for a distance of 6–7 pile diameters in a fully plugged mode (IFR=0). The mode of penetration during installation not only affected the base capacity of the open-ended pile (as q_b is a function of q_{plug}), but also affected the displacement and densification of the ground surrounding the pile, in turn affecting the shaft capacity. Field tests by Lehane [2] and others have shown that the peak shaft resistance q_s , of a pile is given by:

$$(2) \quad q_s = \sigma'_{hp} \tan \delta_f$$

$$(3) \quad \sigma'_{hp} = \sigma'_{hs} + \Delta \sigma'_{rd}$$

where δ_i is the interface friction angle and σ'_{hp} is the horizontal effective stress at failure, which comprises two components (Eq. 3): the stationary horizontal effective stress σ'_{hs} , and a dilational component $\Delta\sigma'_{rd}$.

By using cavity expansion approaches [3], it has been shown that the stress developed due to dilation was inversely proportional to the pile diameter and that while it may have a significant effect on the capacity of model piles, its effects are almost negligible for full-scale piles.

Lehane [2] also reported data from the installation of the 102 mm diameter and 6 m long, steel, closed-ended Imperial College Pile in a loose to medium dense sand deposit at Labenne (south-west France). The pile had been instrumented with horizontal stress sensors at three locations along the pile shaft, each location identified in terms of the distance (h) from the pile base normalized by the pile radius (R). These tests showed that the σ'_{hs} profiles at a given level on the pile were found to have mirrored the q_c profiles, suggesting that σ'_{hs} was controlled by the *in-situ* sand state. Critically, the σ'_{hs} values at a given depth decreased with increasing distance h . Chow [4] reported data from the installation of the same pile in a dense sand deposit at Dunkirk (France) for which the σ'_{hs} values were 4 to 5 times greater than the values measured at the Labenne site. The data from both sites indicated that the value of σ'_{hs} mobilized at a given location along the pile shaft was almost directly proportional to the CPT q_c resistance value and depended on the distance (h) of that point above the pile base. Subsequently, Jardine et al. [5] proposed the following best-fit relationship (known as the Imperial College design approach):

$$(4) \quad \sigma'_{hs} = 0.029 q_c (\sigma'_v / P_{atm})^{0.12} (h/R)^{-0.38}$$

where P_{atm} is the atmospheric pressure and σ'_v is the vertical effective stress.

In an attempt to account for the reduced horizontal stress caused by the installation of open-ended piles, Jardine et al. [5] also recommended replacing the R term in Eq. 4 with a modified radius term R^* , given as:

$$(5) \quad R^* = (R^2 - R_i^2)^{1/2}$$

This expression assumes that the pile is fully coring during installation and hence does not account for the beneficial affects of partial plugging on the shaft resistance that was subsequently demonstrated by Gavin and Lehane [1].

Given the lack of field measurements on pipe piles installed in sand, a series of experiments were performed to assess the effects of partial plugging on the axial capacity of open-ended piles. Specifically, this paper concentrates on the effects of partial plugging on the pile base resistance.

DESCRIPTION OF EXPERIMENTS

Site Conditions

The pile test was performed in a loose to medium dense sand deposit at Donabate, North County Dublin. A Shell and Auger borehole drilled at the site revealed a 2.6-m thick deposit of medium dense sand, underlain by a thin peat layer between about 2.6 and 2.8 mbgl, over 1.5 m of medium dense gravel layer of unproved thickness. The ground water table was located at the base of the sand layer. Three CPT tests were conducted from the bottom of 0.3-m deep starter holes in the area of the pile installation. Profiles of the CPT end resistance (q_c) and the sleeve friction (f_s) are shown in Fig. 1. The profiles indicated highly variable q_c values from ground level to 1.0 mbgl. To avoid this layer, the pile was installed in a 1.0-m deep borehole. At this depth, the q_c resistance in the sand layer gradually increased from 5 MPa with depth, reaching about 8 MPa at the depth at which the peat layer started to affect the q_c values (2.1–2.5 mbgl). The q_c values reduced to about 1.2 MPa within the peat layer and subsequently increased rapidly to about 12–15 MPa in the medium dense gravel. The sleeve friction

measurements were more variable than the q_c measurements although the profiles at all three locations were consistent (Fig. 1b).

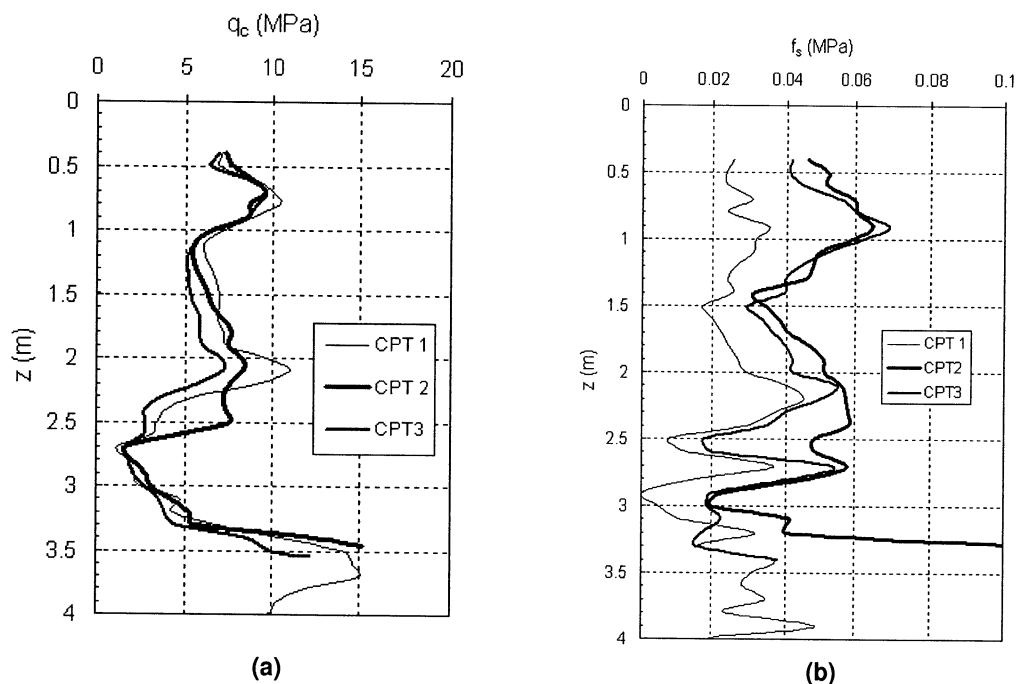


Figure 1. (a) CPT q_c and (b) f_s profiles at the Donabate test site.

Model Pile

An open-ended, stainless steel, tubular model pile with an external diameter of 168 mm and a wall thickness of 9 mm was used for the pile test. The pile was 2.0-m in length and had been constructed using two separate steel tubes, 168 and 154-mm in diameter, which slide over one another. The tubes were connected at the pile cap and were sealed at the base where a specially fabricated pile annulus was attached to the inner tube.

The pile incorporated three levels of sensors, each of which measured the total earth pressures and the pore pressures. The sensors used were Kyowa PS-5KA miniature pressure transducers, 6.0-mm in diameter and a capacity of 500 kPa. The total stress was directly measured using a sensor mounted flush with the pile surface. The pore pressure units comprised a porous ceramic disc mounted flush with the pile surface and in front of a pressure transducer. The void in between the ceramic disc and the pressure transducer was saturated so that only fluid pressure acted on the sensing face. The total earth pressure and pore pressure sensors were mounted diametrically opposite at $h/D = 1.5, 5.5$ and 10.0 from the pile base, where h/D is the distance from the pile base normalized by the external pile diameter. Electrical-resistance strain gauges were glued to the walls of the inner and outer tubes at different locations along the pile length. These allowed the distribution of the load acting along the external shaft, as well as the annulus and internal plug loads, to be determined separately throughout testing.

Test Program

The test pile was pushed to a final depth of 2.9 mbgl using a 20-tonne capacity CPT truck. The pile was jacked at a rate of 20 mm/s and fully unloaded after each 100 mm jacking stroke to monitor the development of the pile plug. After installation, two displacement transducers were attached to an independent reference beam to allow the vertical displacement of the pile

head to be monitored during subsequent load testing. A series of static and cyclic load tests were performed in both compression and tension. For the purposes of this paper only the pile installation stage will be discussed.

PILE INSTALLATION

Plug Development

Soil core development in pipe piles is a complex process and is thought to depend on the pile diameter and wall thickness, pile-wall roughness, mode of pile installation and soil density amongst other factors. Measurements of the plug development made by Paikowsky and Whitman [6] suggested that full plugging occurred after penetrations of 10–20 pile diameters (D). A profile of the IFR values measured during the installation is shown in Fig. 2a. The pile was fully coring (IFR $\cong$ 100%) to a depth of about 1.7 mbgl. At this depth, partial plugging began and the pile had become fully plugged at 2.2 mbgl, and remained so until the final penetration depth of 2.9 mbgl had been reached. The early development of full plugging on the pile after a penetration of only about 6D was unusual. During installation, if the resistance of the ground beneath the pile base exceeds that of the shear resistance of the soil in the plug, the plug will fail and so movement will occur. It is rational to assume that the IFR value would be affected if the pile moved through layers with different shear strengths. The thin layer of peat at about 2.6mbgl, indicated by the site investigation, was seen to affect the CPT end resistance (Fig. 1) when the cone had penetrated 200–300 mm (5–9 cone diameters) above the peat layer. It was possible that this weak zone also affected the IFR value recorded for the pile as plugging began when the pile tip was located 5D above the peat layer.

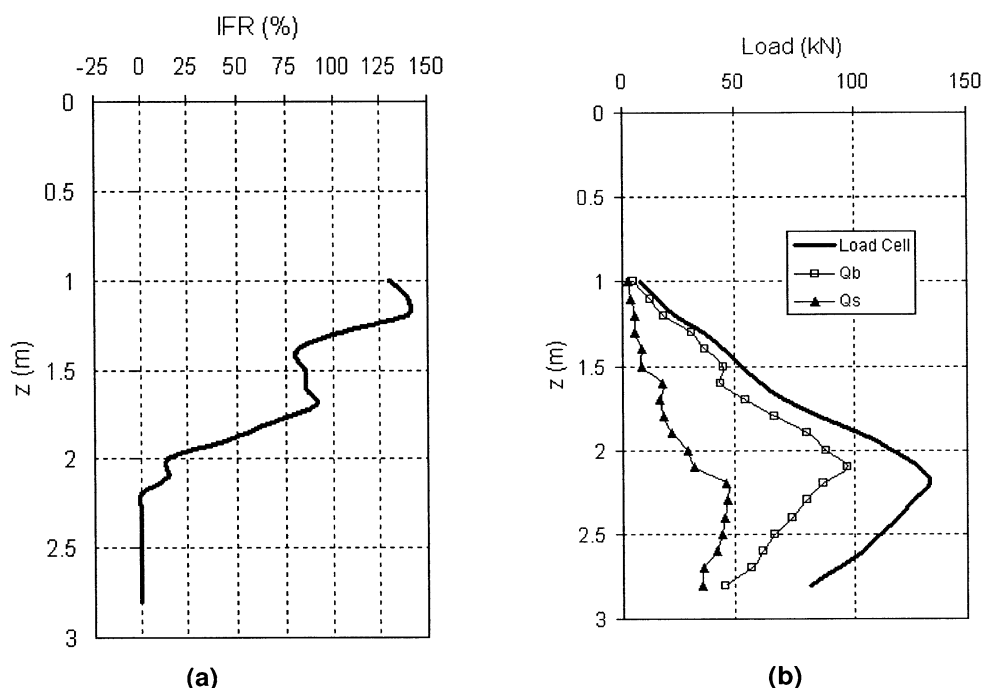


Figure 2. (a) IFR and (b) Total pile resistance IFR measured during installation

Installation Resistance

The total resistance of the pile measured by the load cell, and its components of base resistance (Q_b) and external shaft resistance (Q_s) mobilised during pile installation are shown in Fig. 2b. The pile resistance increased gradually and at about 1.7 mbgl the rate of increase in the base load (and therefore the total resistance) began to accelerate. This increase directly coincided with the decreasing IFR value and occurred when the q_c values increased

marginally with depth, indicating a strong correlation between base resistance and IFR. Increases in shaft resistance were more gradual as the shaft resistance was mobilised along the entire pile length and was therefore less sensitive to localised changes in pile plugging and stress state at the pile base. Despite the fact that the pile had fully plugged, the pile resistance continued to reduce below 2.2 mbgl, mirroring the reduction in CPT q_c values.

Base and Shaft Resistance during Installation

Profiles of the CPT q_c resistance and the annular and plug resistances measured during installation are shown in Fig. 3a. The plug resistance q_{plug} , increased steadily with depth until it became equal in value to q_c at a depth of about 2.8 mbgl, with $q_{ann} \equiv q_c$ throughout installation.

To account for scale effects between the cone penetrometer and full-scale piles [7] suggested using a mean CPT q_c value q_{cmean} , obtained by averaging over a distance of $1.5D$ above and below the pile tip. The unit plug resistance, normalised by the q_{cmean} value, was plotted against the IFR value in Fig. 3b. It can be seen that whilst the pile was operating in coring or partially plugged mode ($IFR > 0$), the q_{plug}/q_{cmean} values were generally between 0.1 and 0.3. As the pile plugged ($IFR=0$), the q_{plug}/q_{cmean} values steadily rose.

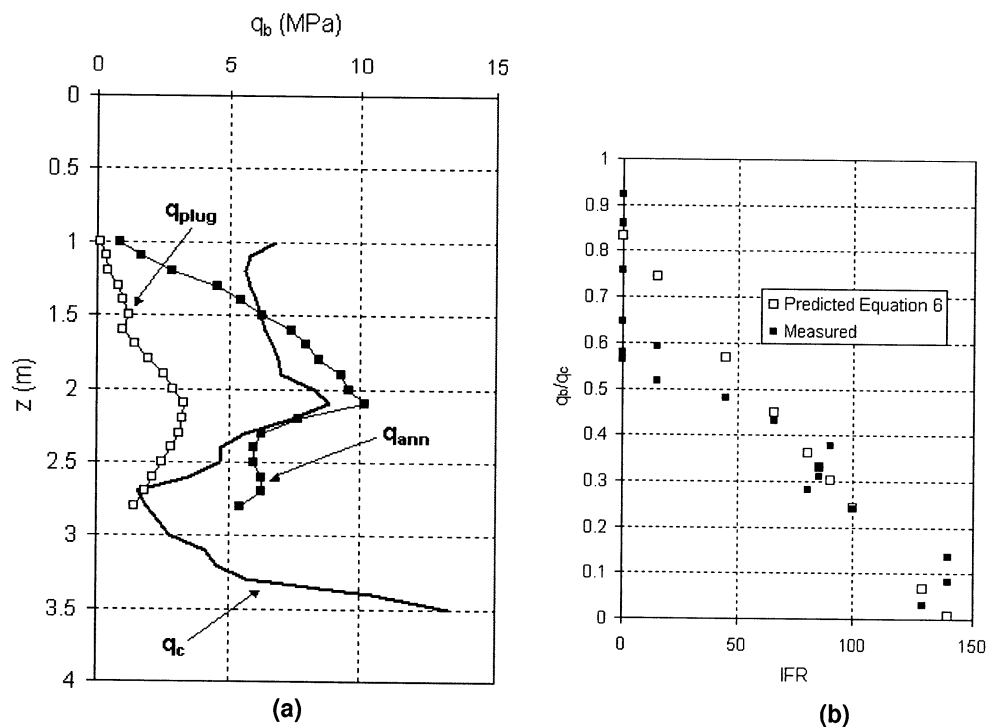


Figure 3. Unit base resistance mobilised during installation.

Model pile tests by Gavin and Lehane [8] indicated that the base stress mobilised at displacements of up to one pile diameter (referred to here as q_{bult}) may be estimated as:

- [6a] $q_{bult} = q_c$ for closed-ended piles.
 [6b] $q_{bult} = (q_{plug} \times A_{ann}) + (q_{ann} \times A_{ann})$ for pipe piles.

where $q_{ann} = q_c$ and $q_{plug} = (0.8 - 0.7IFR)(q_c)$ for $IFR \leq 1$.

Predictions made using Eq. 6 are compared with the unit base resistance measured during the pile installation in Fig. 3b. It is evident that the measured normalized unit base resistance value agreed closely with that predicted by Gavin and Lehane [8]. The mean external shaft

resistance mobilised during installation was compared with the mean CPT f_s value, Fig. 4a. q_{sav} equalled about 40–50 kPa throughout installation and was similar to the mean f_s value suggesting that q_{sav} was independent of IFR. However, whilst q_{sav} was averaged over the whole pile length, f_s was measured locally at the pile toe. Although the data was characterised by scatter, when the q_{sav} values in Fig. 4b were normalised by the mean CPT q_c resistance developed over the pile length (q_{cav}), there was a trend for q_{sav}/q_{cav} to increase as IFR reduced. Typical values of q_{sav}/q_{cav} for closed-ended piles are between 0.008 and 0.010, and the measured values reached these values after penetration occurred in the plugged mode. However, it is important to consider scale effects highlighted in Eq. 3 in the interpretation of shaft resistance trends from model pile tests.

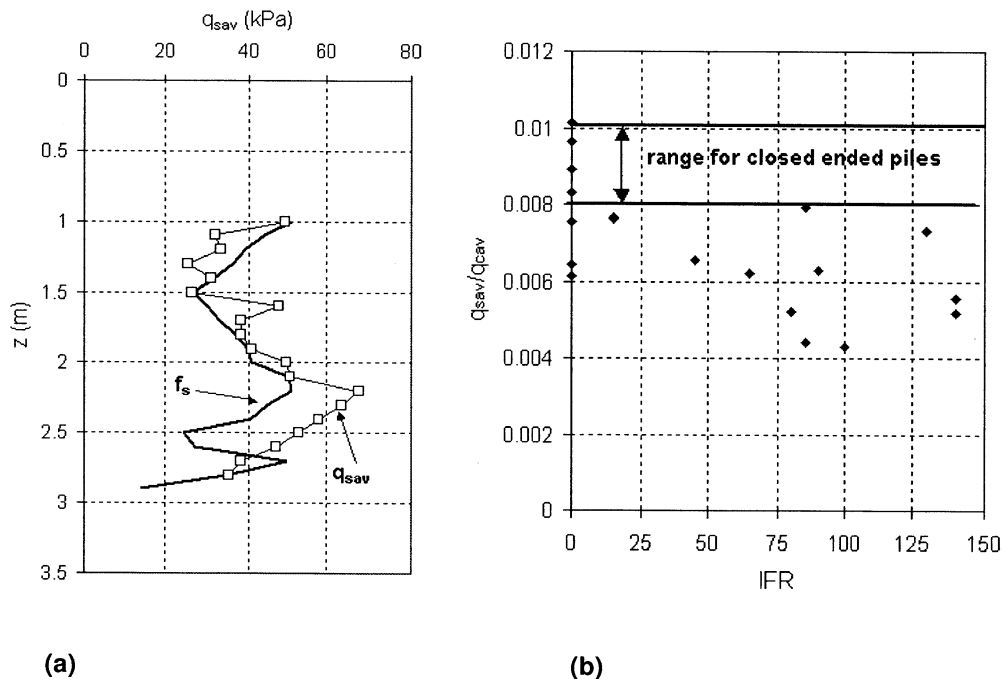


Figure 4. Shaft resistance during installation

DISCUSSION AND CONCLUSION

The paper presented the findings of an open-ended, model pile test conducted at a site in Donabate, County Dublin. The test was performed to assess the factors that affect plug formation and the effects of soil plugging on the base and shaft resistances mobilised during installation. The test results revealed that the incremental filling ratio (IFR) was a function of the CPT q_c resistance and that the IFR value decreased as q_c was reduced. It was found that the jacking load required to install the model pile was dependant on the values of q_c and IFR. The base resistance was seen to take 60–80% of the installation load, the majority of which was taken by the pile plug. The annular resistance was found to be about 3 to 4 times higher than the plug resistance although the plug acted over a much larger area. Values of the plug resistance q_{plug} , normalized by the q_c values, were found to increase as IFR reduced, and q_{plug} continued to increase as the pile penetrated in fully plugged mode (IFR=0). The measured unit base resistance normalized by the q_c value was found to agree closely with that predicted by a recently published design approach. The mean shaft resistance q_{sav} , normalized by the mean q_{cav} value, was also found to increase as IFR was reduced. It was evident that the base and shaft resistances were both dependant on the mode of penetration, defined by the IFR value, as well as the CPT q_c resistance values.

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