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AXIAL LOAD BEHAVIOR OF A DRIVEN CAST-IN-SITU PILE IN SAND

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ABSTRACT

Driven cast-in-situ (DCIS) piles are a popular choice amongst piling contractors due to the ability to readily adjust pile lengths to suit the depth of penetration required. Despite their widespread use, there is a dearth of published data on the axial load behavior of temporary-cased DCIS piles, particularly in cohesionless soils. This paper reports the results of a static compression load test on a 340 mm nominal diameter, 5.75 m long DCIS pile in a dense sand deposit in Shotton, Wales. The test pile was instrumented with vibrating-wire strain gauges at various levels to determine the shaft and base resistance during loading. Analysis of the test results showed that pile behavior was predominantly end-bearing, with the base resistance accounting for approximately 81 % of the total capacity at a displacement of 10 % of the pile diameter. The pile exhibited a stiff stress-displacement response during the initial stages of loading due to the level of pre-stress applied to the soil beneath the base during driving of the steel installation tube. The displacement required to mobilize the shaft resistance was similar to that reported for preformed displacement piles, with a peak local shaft friction of 105 kPa occurring near the base of the pile which diminished with increasing distance from the tip. Finally, the load test results were compared with two popular CPT-based design methods (LCPC and Imperial College methods) for displacement piles in sand. Despite having specific empirical correlations for DCIS piles, the LCPC method significantly under-predicted the capacity of the test pile.

INTRODUCTION

Temporary-cased driven cast-in-situ (DCIS) piles are constructed by top-driving a hollow steel tube using a pile driving hammer, with a sacrificial circular steel shoe placed at the base of the tube prior to driving. The diameter of the shoe is slightly larger in comparison to the driving tube in order to create an annular space between the soil and the tube, thus minimizing shaft resistance during driving. When the required depth of penetration is reached, high-slump concrete is introduced into the tube through either skipping or pumped methods, followed by tube removal. The pile is then left to cure in-situ for a number of days, with the steel shoe remaining at the base. DCIS piles are becoming an increasingly popular choice in comparison to preformed piles due to the ability to readily adjust pile length to suit the depth of penetration required.

Due to the method of installation (i.e. driving), DCIS piles have traditionally been assumed to behave axially in a similar manner to other full-displacement pile types e.g. precast concrete piles and closed-ended steel piles. However, there are

relatively reported few case histories of axial load tests on instrumented DCIS piles to verify this assumption. Neely (1991) developed a database of load tests on expanded-base DCIS piles with concrete compacted shafts (i.e. Franki piles) in sand in order to develop empirical correlations for design. However, as none of the piles in the database were instrumented, the shaft and base resistances were estimated using Chin's (1972) hyperbolic function method which requires a number of assumptions to estimate shaft and base components rather than using direct measurements.

This paper reports the results of a static compression load test conducted on a 340 mm diameter, 5.75 m long, DCIS pile in a uniform alluvial sand deposit in order to gain a better insight into the axial load behaviour of temporary-cased DCIS piles in cohesionless soils. The pile was instrumented with vibrating-wire strain gauges at various levels in order to investigate the variation in the shaft and base resistances during loading. The measured capacity was then compared with those predicted by two CPT-based driven pile design methods.

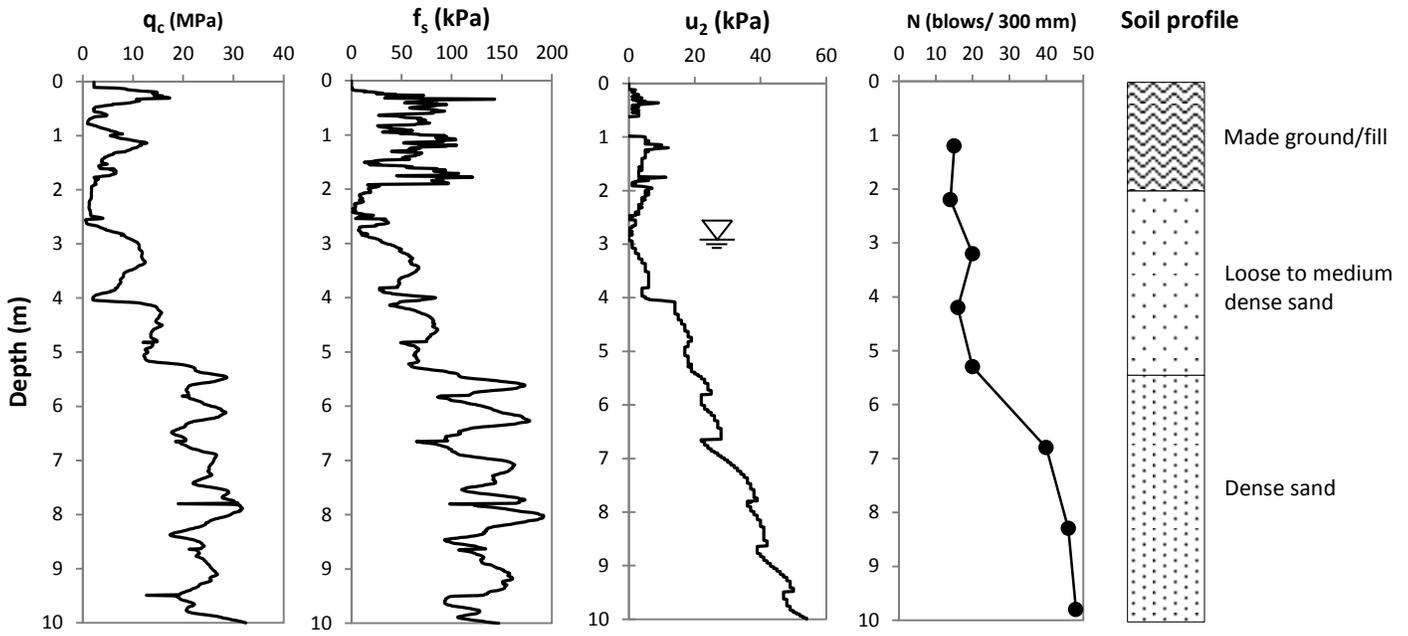


Fig. 1. Measured CPT, SPT and soil profiles at test location

SITE LOCATION AND GROUND CONDITIONS

The pile test was performed at a location approximately 3 km northwest of the village of Shotton in Flintshire, North Wales. The geological profile of the area, described in detail by Nichol and Wilson (2002), comprises of interbedded layers of sandstone, mudstone, siltstone and coal deposited during the Carboniferous Age, which are overlain by glacial till and superficial deposits of alluvial sands and gravels from the nearby River Dee.

A total of 5 no. cone penetration tests (CPTu) were conducted at several locations on the site, including one directly at the location of the pile test, the results of which are shown in Fig. 1. The soil profile inferred from the CPT data consisted of approximately 2 m of made ground (composed of sand, silt and gravel), overlying clean to slightly silty sand. The density of the sand layer increased with depth, becoming very dense at 5 m below ground level (bgl). Samples obtained from a nearby borehole indicated poorly-graded fine sand with a mean particle size D_{50} of 0.16 mm and a uniformity coefficient C_u of 2.2. The water-table was located approximately 3.0 m bgl.

TEST PILE DETAILS, INSTALLATION AND STATIC LOAD TEST

The DCIS test pile was 5.75 m in length, with a nominal shaft diameter of 340 mm. In order to measure the shaft and base resistance during loading, the pile was instrumented with 16 no. vibrating-wire strain gauges at four separate levels (0.3 m,

2.5 m, 4.0 m and 5.5 m bgl), with an array of four gauges placed at each level in order minimize the effect of bending on the measured strains during loading, as well for redundancy purposes. Reinforcement consisted of 4 no. H40 bars, with H10 helical links at 200 mm centers for shear reinforcement.

The pile was installed by top-driving a 323 mm outer diameter hollow steel tube with a 380 mm diameter sacrificial driving shoe at the base using a 5-tonne Junttan HHK5A hydraulic hammer. Upon reaching the required depth, the hammer was retracted and the driving tube was filled with high-slump concrete with a 28 day cube strength of 45 MPa. The hammer was then reattached and several blows were applied to the tube during extraction in order to compact the concrete. The reinforcement (with instrumentation attached) was inserted into the concrete after tube extraction in order to prevent any damage to the gauges.

As a number of studies on residual load development in cast-in-situ piles have been reported in the past e.g. Fellenius *et al.* (2009), the strain and temperature behaviour was continuously monitored using a data-logger for a period of 14 days after casting in order to investigate if residual stresses developed in the test pile. However, analysis of the strains did not reveal the presence of any residual loads, and the pile was therefore assumed to be in a stress-free state immediately prior to conducting the static load test.

A maintained-load compression test was conducted on the pile

in accordance with the Institution of Civil Engineers Specification for Piling and Embedded Retaining Walls (2007) approximately 14 days after casting. The compression load was applied by jacking the pile against a steel reaction frame which in turn was connected to 6 no. DCIS anchor piles which were installed prior to the test pile. A load cell was used to measure the applied load, with pile displacement monitored by four linear variable differential transducers (LVDT) at the pile cap. The test pile was initially subjected to two loading cycles in increments of 250 kN to maximum loads of 1000 kN and 1500 kN respectively. This was immediately followed by a final cycle in which the pile was reloaded to 1500 kN, followed by 100 kN increments until failure occurred. Each load was held constant until the rate of displacement reduced to 0.2 mm/hour. The test was terminated at an applied load of 2400 kN and a pile head displacement of 53 mm.

RESULTS

The measured load-displacement response at the pile head is shown in Fig. 2, where it is evident that a clear plunging failure was not achieved during the test. However, the load at a displacement corresponding to 10 % of the shaft diameter D_s , was 2147 kN.

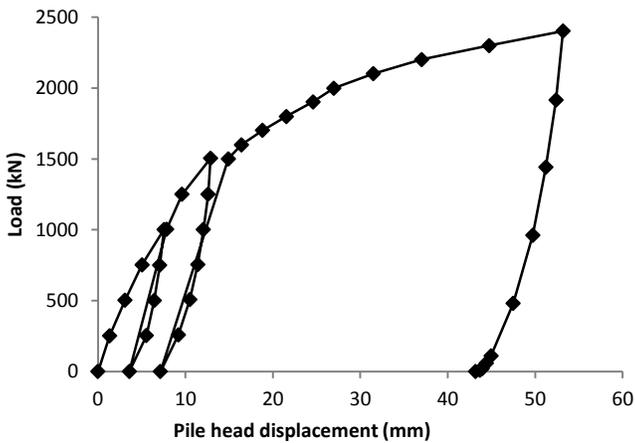


Fig. 2. Measured load-displacement response

The raw strains measured by the gauges were averaged at each level and corrected for the effect of creep in the concrete during the duration of each load hold, as highlighted by Lam and Jefferis (2011). Due to the non-linear variation in the elastic modulus of concrete E_c with strain ε , the secant modulus method was used to determine the pile modulus E_p using the strains measured at 2.5 m (the uppermost gauge level was inadvertently cast into the enlarged pile cap). As a result, E_p varied between 36 GPa and 41 GPa during the load test.

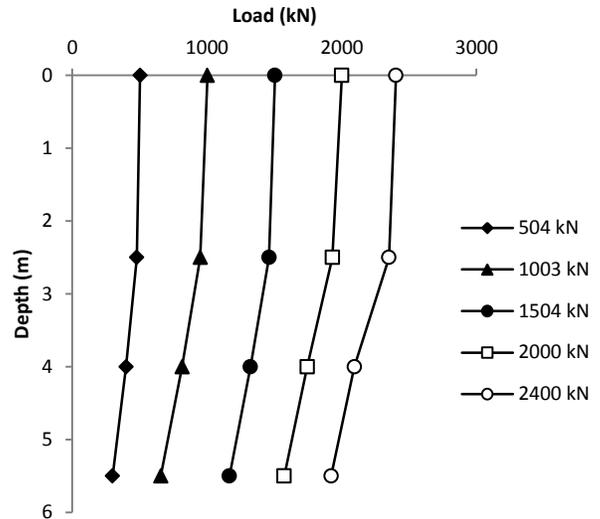


Fig. 3. Load distribution

The derived load distribution during the test is shown in Fig. 3, while Fig. 4 shows the variation in local shaft friction q_s with displacement w_s between each gauge level (the loading cycles have been omitted for clarity purposes). Minimal load was transferred to the made ground layer between the surface and 2.0 m bgl, with the majority of shaft resistance provided by the sand layers below 2.5 m bgl. Despite the variation in density of the sand (as evident from the CPT q_c profile in Fig. 1) below 2.0 m bgl, the measured local shaft friction was broadly similar, with peak q_s values of 90 kPa and 105 kPa in the loose to medium-dense and dense layers respectively. A shaft displacement equivalent to $0.024D_s$ was required to mobilize the peak shaft friction, which is similar to the typical value of $0.02D_s$ for driven piles according to Fleming *et al.* (2008).

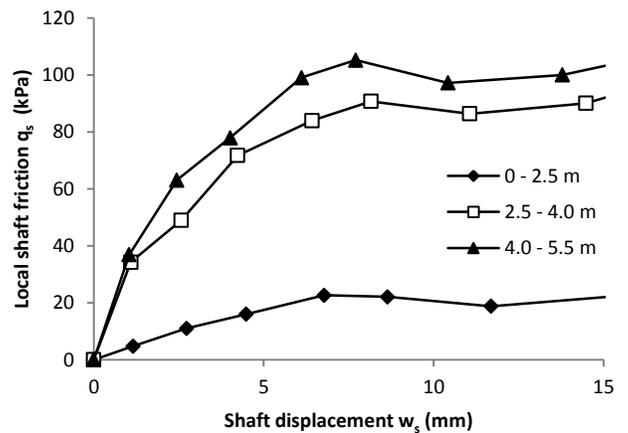


Fig. 4. Variation in local shaft friction with displacement

The peak local shaft friction q_s of a pile in sand can also be expressed using Equation 1, where K is the coefficient of lateral earth pressure, δ is the interface friction angle and σ'_{v0} is the free-field vertical effective stress. As the construction of DCIS piles requires the concrete to be cast in-situ, a rough pile shaft surface is created. This results in shearing occurring within the sand immediately adjacent to the pile-soil interface, and therefore, δ is normally assumed to be equivalent to the constant-volume friction angle φ'_{cv} of the sand (Salgado 2010). As direct shear tests on sand samples were not conducted during the ground investigation stage, φ'_{cv} was unknown and β -coefficients were therefore used to represent the $K \tan \delta$ term in Equation 1.

$$q_s = K \sigma'_{v0} \tan \delta = \beta \sigma'_{v0} \quad (1)$$

The derived β values for the test pile were obtained by dividing the measured peak local q_s values by their corresponding average vertical effective stresses σ'_{v0} , and are summarised in Table 1. The β -coefficients are similar to reported values for concrete-compacted Franki piles in sand by Neely (1990), both of which are greater than the typical range of values of 0.8-1.2 expected for driven piles in dense sand according to the Canadian Foundation Engineering Manual (2006).

Table 1. Summary of measured peak shaft friction and β -coefficients

Depth z (m)	Peak local shaft friction q_s (kPa)	β -coefficient	$(w_s/D_s)_{peak}$ (%)
1.25	22	0.89	1.99
2.25	90	1.86	2.41
4.75	105	1.64	2.26

The load distribution in Fig. 3 was linearly extrapolated to 5.75 m bgl in order to determine the base resistance of the pile during loading. As the pile base was founded in a dense stratum, the axial load behaviour was predominantly end-bearing, with the base resistance accounting for 81 % of the total pile capacity at a displacement equivalent to 10 % of the pile diameter. The variation in base resistance q_b with base displacement w_b is shown in Fig. 5. A stiff linear response was evident up to a normalized displacement w_b/D_b of 0.026, after which the resistance increased at a reduced rate due to the degradation in base stiffness. Such behaviour is typical of displacement piles due to the level of pre-stress induced in the soil beneath the base during driving e.g. Gavin & Lehane (2007).

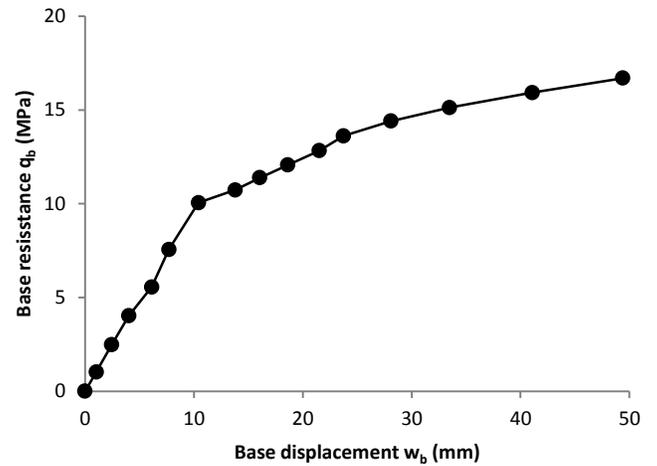


Fig. 5. Variation in base resistance with displacement

The base resistance q_b is also defined using Equation 2, where N_q is the bearing capacity factor and σ'_{v0} is the free-field vertical effective stress at the base of the pile. Displacements in excess of the pile diameter are typically required to achieve a true plunging failure in sand and q_b is often defined at the load corresponding to a displacement of 10 % of the base diameter D_b . Using this criterion, the base resistance $q_{b0.1D}$ for the test pile was 15.17 MPa and the corresponding N_q value using Equation 2 was 204, which is significantly greater in comparison to the database of N_q values at 10% D_b for preformed driven piles in sand compiled by Chow (1997).

$$q_b = N_q \sigma'_{v0} \quad (2)$$

The magnitude of displacement required to mobilize both the shaft and base resistance, together with the back-calculated β and N_q parameters, demonstrated that the axial load behavior of the DCIS test pile was similar to that which would be expected of a full-displacement driven preformed pile in sand, despite the fact that the pile was cast and cured in-situ. Therefore, the appropriateness of driven-based design methods for estimating the capacity of temporary-cased DCIS piles in sand is now investigated.

COMPARISON WITH CPT-BASED PILE DESIGN METHODS

As the behavior of a pile during loading is analogous to that of a cone penetrometer in a CPT test, a number of CPT-based methods for predicting the axial capacity of piles have been developed. The LCPC method (Bustamante & Ganeselli 1982) uses empirically-developed coefficients to relate the measured q_c profile to pile capacity and is of particular interest in this paper as it provides specific coefficients for DCIS piles (as well as other pile types e.g. bored). The local shaft friction q_s is estimated using Equation 3, where the value of α ranges from 150 to 300 for DCIS piles in sand, depending on the

average cone resistance in the respective layer. A filtering procedure is applied to the q_c profile, the details of which are described by Bustamante & Gianceselli (1982), prior to estimating the base resistance using Equation 4, where a base coefficient $k_c = 0.4$ is stipulated for DCIS piles.

$$q_s = q_c / \alpha \quad (3)$$

$$q_b = q_c k_c \quad (4)$$

The results of load tests on highly-instrumented closed-ended steel piles by Lehane (1992) and Chow (1997) have enabled a better understanding of the factors influencing driven pile behavior during installation and loading in sand, and have led to a new CPT-based prediction method for offshore piles, commonly known as the Imperial College ICP-05 method (Jardine *et al.* 2005). The local shaft friction at failure q_s is given by Equation 5, where σ'_{hc} is the lateral effective stress after installation, $\Delta\sigma'_{hd}$ is the increase in lateral effective stress due to dilation effects during loading and δ is the interface friction angle. Lehane (1992) showed that the σ'_{hc} profile along the pile shaft was closely related to the corresponding q_c profile and can therefore be estimated using Equation 6, where the h/R term represents the decay in σ'_{hc} during driving (referred to as 'friction fatigue') at a distance h from the pile base, normalized by the pile radius R . The dilation-related increase in lateral stress $\Delta\sigma'_{hd}$ is calculated using Equation 7 where G is the soil shear modulus and Δr represents the horizontal displacement of the soil at the pile-soil interface (≈ 0.02 mm). The base resistance q_b is given by Equation 8, where q_{cavg} is the average cone resistance over a distance of $1.5 D_b$ above and below the base and D_{CPT} is the diameter of the cone penetrometer (≈ 36 mm).

$$q_s = (\sigma'_{hc} + \Delta\sigma'_{hd}) \tan \delta \quad (5)$$

$$\sigma'_{hc} = 0.029 q_c (\sigma'_{v0} / \sigma_{am})^{0.13} (h/R)^{-0.38} \quad (6)$$

$$\Delta\sigma'_{hd} = 2G\Delta r / R \quad (7)$$

$$q_b = q_c [1 - 0.5 \log(D_b / D_{CPT})] \quad (8)$$

The construction of the test pile on the location of the CPT test enabled a direct comparison between the measured capacity from the load test and the capacities predicted by the LCPC and ICP-05 methods. The LCPC method was chosen in order to assess whether the correlation factors for DCIS piles were realistic and the ICP-05 method was also selected as it accounts for the effects of interface dilation and friction fatigue during driving.

Using the measured data from the CPT test, the average q_c value between each gauge level was used to calculate the shaft friction for each method. As no shear tests were conducted on

sand samples, an assumed constant-volume friction angle $\phi'_{cv} = 33^\circ$ was used to represent the interface friction angle δ in Equation 5, based on the investigation of strength and dilatancy characteristics of sand by Bolton (1986). Figure 6 shows a comparison of the measured peak q_s values with the predicted local shaft friction profiles using the LCPC and ICP-05 methods. Despite having specific coefficients for DCIS piles, the LCPC method significantly under-estimated the shaft friction of the test pile, while an improved estimate was provided by the ICP-05 method in comparison, particularly in the dense layer near the base. However, the measured peak shaft friction in the loose to medium-dense sand layer between 2.5 m and 5.0 m bgl was considerably greater than the predicted values by both CPT methods. It is probable that the driving of the steel tube during installation resulted in a considerable increase in density (and hence q_c) of the sand in this layer, which in turn would lead to a higher prediction of shaft resistance.

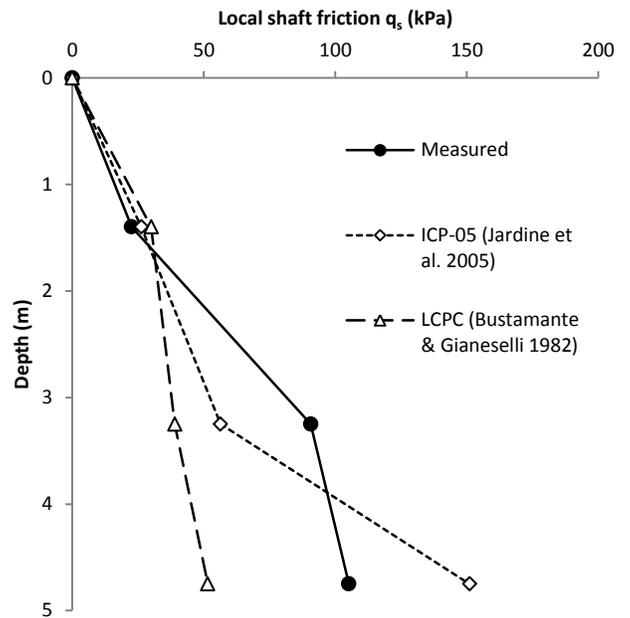


Fig. 6. Comparison of measured and predicted local shaft friction profiles

A summary of the measured and predicted base resistances by the LCPC and ICP-05 methods is provided in Table 2. The predicted base resistance by both CPT methods was significantly smaller in comparison to the measured resistance in the test. Normalizing the measured $q_{b0.1D}$ by the corresponding q_{cavg} (averaged over a distance of $1.5 D_b$ above and below the base) yielded a value of 0.65, which is in reasonable agreement with the relationship for full-displacement piles of $q_{b0.1D}/q_c = 0.6$ proposed by Lehane *et al.* (2007).

Table 2. Summary of measured and predicted base resistances

Method	$q_{b,0.1D}$ (MPa)
LCPC	9.5
ICP-05	11.8
Measured	15.17

CONCLUSIONS

A static load test was performed on an instrumented temporary-cased DCIS pile in a uniform sand deposit. Analysis of the pile behavior during loading demonstrated that the pile behaved in an end-bearing manner during loading, with both shaft and base resistances mobilizing at displacements typically expected of full-displacement driven preformed piles. As the test pile was constructed on the location of a previously-conducted CPT test, the measured pile capacity was compared with two popular CPT-based design methods (LCPC and ICP-05). Despite having coefficients for DCIS piles, the LCPC method significantly under-estimated the shaft resistance of the test pile, whereas an improved estimation of shaft resistance was obtained from the ICP-05 method. However, both methods under-predicted the base resistance by as much as 40 %. Based on the results of the load test, it is tentatively concluded that the axial load behavior of a temporary-cased driven cast-in-situ pile in sand is similar to that of a full-displacement preformed pile.

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