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Instrumentation and axial load testing of displacement piles

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Despite the fact that results of many instrumented pile load tests have been reported in the literature, it is difficult to find well-documented instrumentation procedures that can be used when planning a load testing programme. A load test programme designed to investigate various aspects of the design and behaviour of driven steel piles is discussed in the present paper. Although the literature contains information on load testing of instrumented piles driven in either sand or clay, limited information is available regarding their axial load response in transitional soils (soils composed of various amounts of clay, silt and sand). Results are presented for fully instrumented axial load tests performed on an H pile and a closed-ended pipe pile driven into a multilayered soil profile consisting of transitional soils. In addition, the load testing planning, the instrumentation of the piles, the testing methods and the interpretation of the pile testing data are discussed in detail in the context of this and other load testing programmes described in the literature, in order to illustrate the various steps.

Notation

A_b	area of pile base
A_{si}	pile shaft area interfacing with layer i
A_1, A_2	fitting parameters for the equation to estimate q_{bl} when the pile base is embedded in a strong layer overlying a weak layer
B_o	outer pile diameter
C_1, C_2	slope and intercept of the axial load–settlement curve in the $1/Q$ against w space
H_t	distance from the pile base to the interface of the weak/strong layers
L	pile length
n	number of soil layers
Q	axial load applied to the pile head
Q_{sL}	limit shaft resistance
$Q_{bL,ult}$	ultimate base resistance
Q_{ult}	ultimate axial load-bearing capacity
$q_{b,ult}$	ultimate unit base resistance
$q_{c,strong}$	cone resistance of strong layer
$q_{c,weak}$	cone resistance of weak layer
q_{sLi}	limit unit shaft resistance for soil layer i
w	settlement at the pile head corresponding to the axial load Q
z	pile depth

1. Introduction

Steel H piles and steel pipe piles are used worldwide for the foundations of a variety of structures. These piles are manufac-

tured in a wide range of dimensions and lengths, providing for both small and large axial and lateral load capacities. Steel piles offer significant advantages over other types of piles as they are easy to handle, splice and drive into the ground. The possibility of applying axial loads soon after installation can also be advantageous, particularly for underpinning works. Despite the fact that driven piles are often used in practice because of the many advantages they offer, there are still knowledge gaps in their design, particularly when they are installed in non-textbook soils. In addition, although results of instrumented pile load tests are available in the literature, it is difficult to find well-documented discussions of instrumentation procedures that can be referred to when planning a load testing programme.

Before a load testing programme is undertaken, a thorough review of the literature is always recommended so that mistakes and problems faced by previous researchers can be avoided at the planning stages. It is not uncommon for load tests to fail to achieve their goals because of faulty planning.

The type and number of instruments used in a given pile load test programme depend on its goals. For example, in the case of verification or proof load tests, only an axial load cell and an axial displacement instrument, installed at the pile head, may be sufficient. However, if investigation of other aspects of the pile–soil interaction problem is required, then other instruments are needed as well. Figure 1 summarises the measurements and instruments typically used in pile load test programmes.

Figure 1. Measurements and instruments in pile load test programmes (ALC: axial load cell; ADI: axial displacement instrument; SG: strain gauge; PPT: pore pressure transducer; SD: surveying device; ACC: accelerometer; RSC: radial stress cell; OC: Osterberg cell; SFC: shaft friction cell)

Axial load tests have been performed on instrumented full-scale piles mainly for determining the load–settlement and the load–transfer relationships and for comparing measured data with corresponding values predicted by pile design methods. In this paper, a detailed description is presented of the procedures required for successful pile instrumentation and data acquisition and the results of four high-quality instrumented pile load tests performed on steel piles driven in a mixed soil profile.

2. Pile load response

The ultimate axial load-bearing capacity Q_{ult} of a single pile is expressed as the sum of the ultimate base resistance $Q_{b,ult}$ and the limit shaft resistance Q_{sL}

$$1. \quad Q_{ult} = Q_{b,ult} + Q_{sL} = (q_{b,ult} A_b) + \sum_{i=1}^n q_{sLi} A_{si}$$

where $q_{b,ult}$ is the ultimate unit base resistance; q_{sLi} is the limit unit shaft resistance for soil layer i ; A_b is the area of pile base; A_{si} is the pile shaft area interfacing with layer i ; n is the number of soil layers.

Only small pile displacements (typically of the order of 1% of the pile diameter) are required for complete mobilisation of Q_{sL} . The value of Q_{ult} adopted in practice based on pile load test results usually corresponds to a pile displacement equal to 10% of the pile diameter, so Q_{sL} is fully mobilised when the applied axial load $Q = Q_{ult}$. Both Q_{sL} and $Q_{b,ult}$ are affected by

- (a) the degree of soil displacement during pile installation
- (b) soil type and stress state
- (c) installation method

- (d) degree of ‘soil plugging’ developed during installation
- (e) pile size and shape
- (f) the presence of residual loads after installation
- (g) set-up and other time-related effects.

Both soil type and stress state influence pile behaviour significantly. For sand, available data by Lehane *et al.* (1993) and analyses by Salgado (2006a, 2006b); Loukidis and Salgado (2008) and Basu *et al.* (2011) suggest that q_{sL} depends on the mobilisation of critical-state shear strength in a very narrow shear band surrounding the pile. Changes in radial effective stresses caused by pile installation at a sand element at depth z down a pile of length L depend on z , the relative density of the element, and the ratio of the vertical distance $L-z$ from the element to the base of the pile to the pile radius. However, changes in radial effective stresses during axial loading of the pile depend mainly on local interface dilation and principal stress rotation. Both of these factors affect the value of q_{sL} that is in fact mobilised. For clay, q_{sL} depends instead on the mobilisation of residual shear strength (Coop and Wroth, 1989; Salgado, 2006b; Chakraborty *et al.*, 2012).

Residual loads, which are compressive axial loads that are locked in the pile as a result of pile driving, need to be carefully evaluated when interpreting pile load test results (Alawneh and Malkawi, 2000; Briaud and Tucker, 1984). During driving, the pile undergoes elastic compression. After driving, the pile tries to rebound to recover its initial length, moving upwards. The soil below the pile base, compressed during driving, also tends to push the pile upwards. However, because interface shear resistance is mobilised between the pile shaft and the surrounding soil as the pile rebounds, q_{sL} reverses direction (from an upward direction to a downward direction), at least in the upper portion of the pile. Therefore, measurement of the residual load distribu-

tion with depth is required for an accurate estimation of the unit base and shaft resistances from pile load test results.

2.1 Instrumented piles

The instrumentation selected depends on the objectives of the testing programme (see Figure 1). Instrumented full-scale piles, typically installed by driving, found in the literature usually have a diameter larger than 250 mm. Full-scale load testing is preferred over model pile testing because full-scale piles represent field conditions. Model piles offer the following advantages

- (a) ease of fabrication, handling, attachment of both basic and special-purpose instruments and installation
- (b) possibility of pile extraction at the end of each test for reuse
- (c) preliminary experimental validation of pile prediction methods, before moving on to full-scale tests
- (d) preliminary assessment of the performance of sensors, before using them in full-scale piles.

However, model pile testing cannot completely replace full-scale pile testing because scale effects may affect the interpretation of the acquired data. These scale effects are attributable either to the ratio of pile diameter to particle size (which may make tests in coarse-grained soils with large particle size compared to pile diameter not representative of field conditions) or to depth effects (which may make the stress level along the model piles too low for direct application to field calculations). In addition, model piles are usually jacked (they are seldom driven) into the ground

to avoid exposing special-purpose transducers, which are sometimes difficult to calibrate, to impact loads that can potentially cause their malfunction during testing. Table 1 outlines the details of some instrumented model piles available in the literature. Table 2 summarises the details of some instrumented full-scale piles described in the literature during the past decade.

2.2 Residual loads

Residual loads affect the actual distribution of q_{sL} with depth (Fellenius, 2002a, 2002b). These loads are usually more difficult to measure in driven piles than in jacked piles. They are measured by zeroing all strain gauges immediately before pile driving and taking gauge readings at the end of driving. However, it is not easy to measure the residual load after pile driving (Hajduk and Paikowsky, 2000; O'Neill *et al.*, 1982). In fact, strain gauges are often re-zeroed before axial load testing to correct read-out drifts that develop any time between pile installation and testing (read-out drifts of vibrating-wire strain gauges are possibly caused by micro slippages at the wire grips due to inertial forces during driving). Both vibrating-wire and electrical-resistance strain gauges have been successfully used for measuring pile load transfer, provided that careful attention is given to adequate gauge installation.

Piles are often instrumented with vibrating-wire strain gauges because of their known long-term stability under static loading, which is particularly important for pile set-up investigations. However, these gauges show a relatively slow response time. For

References	Soil type	Pile installation	PED: mm	PIL: m	ALC levels	ADI levels	SG levels	PPT levels	RSC levels	SFC levels
Cooke <i>et al.</i> (1979)	OC clay	Jacking	168.0	5.00	1 head 3 shaft 1 base	1 head	—	—	—	—
Karlsrud and Haugen (1985)	OC clay	Jacking	152.4	5.15	1 head	1 head	6 shaft	4 shaft	4 shaft	—
Jardine and Bond (1989)	OC clay	Jacking	102.0	2.60	1 head	1 head	—	2 shaft	2 shaft	2 shaft
Lehane <i>et al.</i> (1993)	Sand				2 shaft			1 base	1 base	1 base
Lehane and Jardine (1994)	Soft clay				1 base					
Coop and Wroth (1989)	OC clay	Jacking	80.0	1.135	1 head 3 shaft	1 head	—	2 shaft 1 base	2 shaft	2 friction sleeves
Paikowsky and Hart (2000)	NC clay	Driving	76.2	2.87	2 shaft 1 base	1 shaft at slip joint	—	1 shaft	1 shaft	2 friction sleeves
Lee <i>et al.</i> (2011)	Soft clay				1 head	2 head	9 shaft	—	—	—
	Fine silica sand	Driving	30.0	1.20	1 head 1 base					

PED: pile external diameter; PIL: pile instrumented length; ALC: axial load cell; ADI: axial displacement instrument; SG: strain gauge; PPT: pore pressure transducer; RSC: radial stress cell; SFC: shaft friction cell; OC: overconsolidated; NC: normally consolidated.

Table 1. A summary of some axial load tests on instrumented model piles described in the literature

References	Soil type	Pile type/ installation	ED: mm	L: m	ALC levels	ADI levels	ERSG levels	VWSG levels	RSC levels	SFC levels	PPT levels	ACC levels
Paik <i>et al.</i> (2003)	Gravelly sand	CE pipe/driving	356	8.24	1 head	1 head	PDA 9 shaft	—	—	—	—	PDA
Paik <i>et al.</i> (2003)	Gravelly sand	OE pipe/driving	356	8.24	1 head	1 head	PDA 9 shaft 10 inner wall	—	—	—	—	PDA
Paikowsky and Hajduk (2003)	Clayey soils	CE pipe/driving	324	31.4	1 head	1 head	PDA 2 shaft TT	14 shaft	2 shaft	—	16 shaft	PDA 2 shaft
Paikowsky and Hajduk (2003)	Clayey soils	CE pipe/driving	324	24.4	1 head	1 head	PDA 2 shaft TT	10 shaft	2 shaft	—	11 shaft	PDA 2 shaft
Yu (2004)	Silty sand	H/driving	b_f : 326 d : 338	33.2–55.4	1 head	1 head	—	9–16 shaft	—	—	—	—
Yang <i>et al.</i> (2006a)	Silty sand	H/jacking	b_f : 321 d : 328	25.8/41.5	1 head	1 head	—	9–15 shaft	—	—	—	—
Yang <i>et al.</i> (2006b)	Silty sand	H/jacking	b_f : 321 d : 328	35.5–40.5	1 head	1 head	13–14 shaft	—	—	—	—	—
Yu (2004)	Silty sand	H/jacking	b_f : 326 d : 338	40.9	1 head	1 head	11 shaft	—	—	—	—	—
Current work	Mixed soils	CE pipe/driving	356	17.4	1 head	1 head	PDA	17 shaft	—	—	—	PDA
Current work	Mixed soils	H/driving	b_f : 310 d : 308	17.4	1 head	1 head	PDA	12 shaft	—	—	—	PDA

ED: pile external dimension; L: pile length; ALC: axial load cell; ADI: axial displacement instrument; ERSG: electrical-resistance strain gauge; VWSG: vibrating-wire strain gauge; RSC: radial stress cell; SFC: shaft friction cell; PPT: pore pressure transducer; ACC: accelerometer; PDA: pile driving analyser; CE: closed-ended; OE: open-ended; TT: telltale; b_f : flange width of H pile; d : depth of H pile.

Table 2. A summary of some axial load tests on instrumented H and pipe piles described in the literature

measurements during pile driving, faster-response electrical-resistance strain gauges are often preferred, together with accelerometers. Furthermore, because the masses of electrical-resistance strain gauges are very small compared to those of vibrating-wire strain gauges, electrical-resistance strain gauges are less sensitive to the inertial forces resulting from pile driving and, hence, are less likely to drift after pile driving. This means that electrical-resistance strain gauges are more suitable for measuring residual loads than vibrating-wire strain gauges, which are subject to possibly significant drifting; however, electrical-resistance strain gauges do not normally survive in the long term because they are more sensitive to humidity and other weather-related effects. Therefore, pile instrumentation combining both electrical-resistance gauges, whose most important goal would be that of obtaining residual loads after driving, and vibrating-wire strain gauges, whose major goal would be that of long-term monitoring, may be a better option. This may also be a more cost-effective solution because the cost of acquisition and installation of a reasonable number of electrical-resistance strain gauges is lower than that of vibrating-wire strain gauges.

3. Pile load testing programme

In the context of the present research, instrumented axial load tests were performed on an H pile and a closed-ended pipe pile, both driven into a multilayered soil profile in Indiana. The objective of these tests was to determine, for each pile, the following

- (a) load–settlement curve
- (b) Q_{ult}
- (c) load–transfer relationship
- (d) Q_{sL} and $Q_{b,ult}$
- (e) q_{sLi} for each soil layer crossed
- (f) $q_{b,ult}$.

The pile load test results are compared with predictions by soil property based and in situ test based design methods in Seo *et al.* (2009) and Kim *et al.* (2009). Pile set-up effects are discussed by Lee *et al.* (2010).

3.1 Site investigation

The test site is located on State Road 49 (on the north side of Oliver Ditch) in Jasper County, Indiana. Four standard penetration tests (SPTs) and four cone penetration tests (CPTs) were performed before driving the piles into the ground. The groundwater level and bedrock were found at a depth of 1 m and 26 m, respectively. Two of the CPTs near the location of the test H pile were terminated at a depth of about 18 m, about 1 m into an extremely dense non-plastic silt layer with an average q_c of 50 MPa. Below this silt layer, there exists a relatively soft, thick clay layer with an average q_c of 1.5 MPa. The soil profile consists of 11 soil layers, starting from the ground surface to the pile base. Figure 2 shows the surface layout of sampling boreholes, in situ tests, test piles, reaction piles and extra piles. Figure 3 shows the soil profile and the results of SPTs and CPTs. Results of laboratory tests performed on soil samples obtained from these boreholes are reported by Seo *et al.* (2009) and Kim *et al.* (2009).

3.2 Pile instrumentation

It is well known that pile capacities are affected by the degree of soil displacement during pile installation. Typically, ultimate pile capacities are greater for displacement piles than for partial or non-displacement piles because of the stiffer response of displacement piles, which can be attributed to the significant amount of soil preloading during their installation (Salgado, 2008). In order to compare the load–settlement and load–transfer behaviours of partial and full displacement piles, an H pile (partial displace-

Figure 2. Layout of pile load tests and in situ tests (MCEP: main closed-ended pipe pile; MHP: main H pile; RHP: reaction H pile; RCEP: reaction closed-ended pipe pile; ECEP: extra closed-ended pipe pile; EHP: extra H pile)

(a)

(b)

Figure 3. Soil profile and results of in situ tests at (a) H pile location, (b) pipe pile location

ment pile) and a closed-ended pipe pile (full displacement pile) were selected in the present study.

The H pile had 310 mm of flange width, 308 mm of web depth and 15 mm of thickness (HP 310 × 110). The spiral-

welded pipe pile, which was closed by a 25.4 mm thick steel plate welded to the base, had 356 mm outer diameter and 12.7 mm wall thickness. Both piles were driven to a depth of 17.4 m within the dense silt layer (i.e. both piles were embedded inside this layer at least 1.5 times the outer

diameter of the pipe pile or the flange width of the H pile). Their final lengths, including the portion above the ground surface, were 18.5 m.

Twenty-four vibrating-wire strain gauges (Geokon Model 4150) were attached to the outside of the two flanges of the H pile, at 12 levels. Thirty-two vibrating-wire strain gauges (also Geokon Model 4150) were attached to the outer surface of the pipe pile, at 16 levels. All gauges were spot-welded to the pile surface, covered with a semicircular plate and sealed with silicone rubber. Care was taken to install these gauges away from the spiral welding joints of the pipe pile, where the pipe wall thickness may be slightly different. To avoid damage during driving, the gauges on both piles were covered with steel angle channels (76 mm wide and 6 mm thick), welded to the pile surface. These channels ran from the base to near the head of the piles. The lower end of each channel was closed by welding a tapered cover to prevent the entrance of soil during pile driving. Gauge cables were routed to the ground surface inside the channels. The cables were wrapped with aluminium tape to protect them from the heat generated by welding. The strain values obtained from the vibrating-wire strain gauges installed above the ground level were used to calculate the Young's modulus values for both piles.

Although other instrumentation techniques – such as electrical-resistance strain gauges – could have been used for measuring axial strains in these test piles, the spot-welded vibrating-wire strain gauge was selected because of its

- (a) small size
- (b) ruggedness
- (c) ease of installation
- (d) moisture resistance
- (e) long-term stability (required for evaluating pile set-up)
- (f) possibility of using long instrument cables without significant loss of accuracy
- (g) ease of connection to a field data acquisition system (with a built-in plucking circuit).

The operation of the vibrating-wire type of strain gauge is based on changes of the natural frequency of a tensioned wire. A small relative movement of the fixed ends of the wire, which is vibrated by the input pluck pulse, will alter the tension of the wire and hence cause a change in its natural frequency. This change in natural frequency is measured by means of an electromagnetic coil positioned next to the wire. Although vibrating-wire strain gauges, in contrast with electric-resistance strain gauges, are durable and have minimal lead-wire effect, they are very sensitive to wire tension and have limited range of measurable strains (usually up to 3000 microstrains). Therefore, much care is needed in their installation. As illustrated in Figure 4, the steps followed in the installation of the gauges were as described below.

- (a) A 150 mm × 100 mm surface was prepared by grinding it first with a hand grinder to remove rust from the pile surface,

sanding it with sand paper to obtain a flat, clean and smooth surface, degreasing it with acetone and, finally, wiping it with a soft cloth.

- (b) The gauge was carefully positioned at the desired location on the pile and aligned with the pile axis.
- (c) The gauge cable was anchored to the pile surface in a zigzag pattern to relieve any tension in the connection between the gauge and the cable, and to reduce tension in the cable induced by inertial forces generated during pile driving.
- (d) The protective cover of the gauge was removed and its reading was checked.
- (e) The gauge mounting pads were spot welded to the pile surface.
- (f) Cyanoacrylate adhesive was spread over the mounting pads to protect the gauges from corrosion (Figure 4(a)).
- (g) The stability of gauge readings was rechecked by gently tapping the mounting pads to relieve any local stresses induced by the welding procedure.
- (h) The cable-to-lead-wire junction was firmly secured to the steel surface by spot welding steel strips around the cable, leaving some slack in the lead wires (Figure 4(b)).
- (i) A stainless steel semicircular cover was placed over the gauge and secured with two spot-welded steel strips.
- (j) The installed gauge was waterproofed with silicone rubber (Figure 4(c)).

Both test piles were fabricated in two segments to facilitate the attachment of instruments in the laboratory, the safe transportation of the piles to the test site and the lifting of each pile segment by a rig before driving. After the lower pile segment was driven into the ground, the strain gauge cables were identified and rewired at the pile junction; the cables were then inserted into the angle channel of the upper segment of the pile. The upper pile segment was lifted by the rig, aligned with the lower pile segment and butt welded to its top. The pile was then driven to its final depth. All gauge cables were connected to a field data logger (Geokon Micro-10, model 8020) through three multiplexers (Geokon model 8032). The measurement and data acquisition system was protected against harsh environment and lightning damage.

All strain gauges were zeroed before pile driving and before each axial load test. However, readings taken immediately after driving had a random pattern, unrelated to the expected distribution of residual load along the pile shaft. Harsh impact and vibration during driving may have affected the initial tension of the vibrating wires. According to Hajduk and Paikowsky (2000), vibrating-wire strain gauges are prone to shifts in the zero readings (i.e. zero drift) under dynamic loading because of relaxation of the gauge wire tension. Another possible reason for the discrepancy is the possibility of residual stresses in the pile due to the fabrication process, a fact that is generally not recognised (Akutagawa *et al.*, 2005). Seo *et al.* (2009) and Kim *et al.* (2009) estimated the residual loads using the prediction method of Darrag and Lovell (1989) in the interpretation of the

Figure 4. Installation of strain gauges: (a) applying a cyanoacrylate adhesive over the welded mounting pad; (b) securing cable tightly to the surface by welding thin stainless steel strips; (c) waterproofing with silicone rubber; (d) view of all installed gauges; (e) welding of a steel angle channel to cover all gauges and cables; (f) welding of a tapered cover to the lower end of the channel

results of the axial load tests, and the estimated residual loads were small compared with the loads carried by the piles.

The details of the pile instrumentation and the location of the strain gauges along the length of each pile are shown in Figure 5. The large number of strain gauges in these piles allowed for a better correlation between q_{sL} and soil type along the pile shaft. This dense gauge layout provided enough redundancy to compensate for eventual gauge losses during pile driving and to ensure reliable strain measurements without installing backup instruments (such as rod extensometers). Four strain gauges did not survive driving and load testing.

3.3 Pile loading

Figure 6 shows the pile axial load testing layout. A 500 mm wide and 50.8 mm thick square steel plate was welded to the head of each test pile for distributing the applied axial load evenly. A

Figure 6. View of axial pile load testing layout

Figure 5. Instrumentation details: (a) H pile; (b) closed-ended pipe pile (dimensions in mm)

hydraulic jack was placed over this plate. A 400 mm wide and 76.2 mm thick square steel plate was placed over the jack ram. A load cell was then placed over this steel plate. Another 400 mm wide and 76.2 mm thick square steel plate and a spherical seat were inserted between the load cell and the reaction beam situated above it to maintain the verticality of the load. All these components were carefully aligned and levelled before pile testing. The settlement of each test pile was measured by two dial gauges (one on each side of the pile shaft) attached to two reference beams by magnetic clamps. The spindle of each dial gauge rested on a steel bracket welded to the pile shaft. The supports of each reference beam were placed at least 6.8 pile diameters away from the test piles.

Figure 7(a) shows a general view of the reaction system. It was designed in such a way that the reaction frame could be reassembled from one test pile location to the other. The reaction beam was a wide-flange I steel beam, 4.2 m long and 406 mm deep. Both flanges of this beam were heavily stiffened by welding steel plates along its whole length. Vertical web stiffeners were also welded to this beam, particularly above the pile–jack–load cell assembly. The reaction beam was attached to a heavy reaction frame assembled with steel plate girders to distribute the applied load among six reaction piles (Figure 7(b)). The weight of the reaction frame was supported by a stack of concrete beams bearing on the ground surface. Vertical threadbars connected to each reaction pile were tied to the reaction frame (Figure 7(c)).

As shown in the test layout of Figure 2, one closed-ended pipe pile (designated as RCEP) and five H piles (designated as RHP) were used as reaction piles for the instrumented test H pile (designated as MHP). To compare the pile capacities obtained from dynamic restrike tests with those obtained from the static axial load tests, one of the reaction H piles (RHP-6) was driven to the same depth as the instrumented H test pile. All other reaction H piles were driven to 24.4 m to make sure the necessary

Figure 7. View of reaction pile system: (a) general view; (b) stiffened reaction beams and frames; (c) connection of reaction beam to reaction pile with threaded bars

reaction capacity was available. The head of each H reaction pile was stiffened with a pair of steel plates welded between the flanges. Threaded couplers were then welded to the sides of the pile web for connecting the threadbars used as tie rods. A similar set of reaction piles was used for the test pipe pile (designated as MCEP). One extra H pile (designated as EHP) and one extra closed-ended pipe pile (designated as ECEP) were driven to the same depth as the main test piles; both extra piles were used for restrike testing over time. A restrike test was performed on the test H pile (MHP) 126 days after end of driving, but no restrike test was done on the test pipe pile (MCEP). Useful data were obtained from the reaction and extra piles.

Two static axial load tests were performed on the main closed-ended pipe pile 50 and 90 days after driving. For the test H pile, the load tests were performed 63 and 99 days after driving. The different schedules were because of the time needed for moving the reaction frame from one test pile location to the other. The axial load increment was equal to 178 kN, and, for each loading step, the settlement was recorded at 1, 2, 5, 10, 20, 30, 40, 60, 80, 100 and 120 min. The load at each loading step was maintained until the settlement rate from two consecutive settlement readings at the pile head was less than 0.5 mm/h. In instances where it took longer than 2 h for the settlement rate to satisfy the criterion of 0.5 mm/h, the next load increment was only applied after the difference in settlement rates between the current and previous records was less than 5%. The load increment was reduced as the load applied at the pile head approached the plunging or limit load. After reaching the plunging load, the pile was unloaded in 356 kN load steps. The data acquisition system recorded the strains every 2 min during the load test. The strains obtained from the two strain gauges installed on opposite sides of each pile were averaged to determine the corresponding axial load carried by the pile at each level.

All test, reaction and extra piles were driven with an ICE-42S single-acting diesel hammer, which has a ram weight of 18.2 kN

with a maximum hammer stroke of 3.12 m and a rated maximum driving energy of 56.8 kNm. Dynamic monitoring with the pile driving analyser (PDA) was carried out during the driving of these piles. Restrike tests were performed on the reaction and extra piles for monitoring the increase in pile capacity with time. The detailed results of the dynamic tests and set-up effects are presented in Lee *et al.* (2010).

4. Pile load test results

The hyperbolic method proposed by Chin (1970) may be used in practice to interpret pile load test results if the pile load test is extended to sufficiently large deflections. This method is based on the assumption that the load–settlement relation is hyperbolic

$$2. \quad \frac{w}{Q} = C_1 w + C_2$$

where Q is the axial load applied to the pile head; w is the settlement at the pile head corresponding to the axial load Q ; C_1 and C_2 are the slope and intercept of the axial load–settlement curve in the $1/Q$ against w space. The limit axial load capacity of the pile is equal to $1/C_1$. Figure 8 shows the applied load plotted against pile head settlement curves obtained from the first and second static load tests performed on the H and closed-ended pipe piles. The limit load of the H pile estimated using Chin's method for the first and second load tests is 2282 kN. On the other hand, the limit loads of the pipe pile estimated using Chin's method for the first and second load tests are 1678 kN and 1744 kN, respectively. Table 3 summarises the measured capacities at the end of the static load tests for both piles, as well as the ultimate capacities obtained using the 10% relative settlement criterion and the limit capacities estimated using Chin's method. Note that CAPWAP (Case Pile Wave Analysis Programme)-predicted capacities from PDA tests are also presented in Table 3.

The load–transfer curves for the first and second static load tests performed on the H pile and the closed-ended pipe pile are shown

Figure 8. Load–settlement curves at the pile head

in Figures 9(a) and 9(b), respectively. The load–transfer curves of both load tests were extrapolated to a depth of 17.4 m (the last strain gauges were located at a depth of 16.9 m). There might be a slight error in this extrapolation for the H pile because the slope of the extrapolated line depends on the degree of pile plugging, which is hard to ascertain in practice for H piles. However, the load–transfer curve was extrapolated only for the last 0.5 m, and thus any error introduced by this procedure is small. Figure 10 shows the measured limit unit shaft resistances for the last loading step of both load tests. The q_{sL} values for the H pile shown in this figure were calculated by dividing the difference between loads calculated along the shaft at the end of the test from adjacent strain gauge readings by the distance between them and by the rectangular perimeter (1236 mm) of the H pile. Figures 9 and 10 show that almost all of the shaft resistances of the H pile and the closed-ended pipe pile are provided by the soils located below 14 m depth; in particular, the shaft resistances of these piles increase significantly below 16.4 m.

As seen in Table 3, the shaft capacities of both piles are quite comparable (the closed-ended pipe pile shows slightly larger shaft capacity). This small difference in shaft capacities is a somewhat unexpected result because full displacement piles (e.g. closed-ended pipe piles) have typically 20% larger shaft capacities than H piles (Salgado, 2008). This may be due to the complex plugging behaviour of the H pile driven in mixed soils, but more studies are required to better understand plugging of H piles. On the other hand, the base capacity of the H pile is almost twice as large as that of the closed-ended pipe pile. This unexpected result indicates that the bearing layer was likely too thin at the location of the pipe pile, being therefore unable to develop the base resistance that would be expected for the material of which it is composed. It is possible that a punching failure may have

occurred below the base of the closed-ended pipe pile. Owing to horizontal variability in the soil profile, the dense silt layer below the pile base in the location of the closed-ended pipe pile was not as thick as at the H pile location (the thickness of the dense non-plastic silt layer below the base of the pile was 1.0 m for the H pile and 0.7 m for the closed-ended pipe pile, respectively). Although the cone could not penetrate the dense silt layer at the H pile location (Figure 3(a)), it went through the dense silt layer and reached the weak silty clay layer below it at the closed-ended pipe pile location (Figure 3(b)). This suggests that the thickness of the dense silt layer below the base of the closed-ended pipe pile may have been even less than 0.7 m (less than two pile diameters, a number typically considered an absolute minimum for reliance on a significant fraction of the base resistance in a given layer).

Another factor that contributed to the difference in the base capacities is the different base areas of the piles and thus the different influence zones below them. Based on spherical cavity expansion analyses and centrifuge test results, Xu and Lehane (2008) indicated that the influence depth is larger than 1.5 times the outer pile diameter below the pile base when a soft layer is present below a stiff layer. Their results show that the limit unit base resistance q_{bL} of a closed-ended pipe pile embedded in a strong layer overlying a weak layer depends on the relative resistances of these layers and the distance from the pile base to the weak layer. Xu and Lehane (2008) suggested the following equation to estimate q_{bL} when the pile base is embedded in a strong layer overlying a weak layer

$$3. \quad \frac{q_{bL}}{q_{c,strong}} = \frac{q_{c,weak}}{q_{c,strong}} + \left(1 - \frac{q_{c,weak}}{q_{c,strong}}\right) \exp \left[-\exp \left(A_1 + A_2 \frac{H_t}{B_o} \right) \right]$$

where $q_{c,weak}$ and $q_{c,strong}$ are the cone resistance of weak and strong layers, respectively; H_t is the distance from the pile base to the interface of the weak/strong layers; B_o is the outer pile diameter; and

$$4. \quad A_1 = -0.22 \ln \left(\frac{q_{c,weak}}{q_{c,strong}} \right) + 0.11 \leq 1.5$$

$$5. \quad A_2 = -0.11 \ln \left(\frac{q_{c,weak}}{q_{c,strong}} \right) - 0.79 \leq -0.2$$

The equivalent diameter of the H pile is 0.349 m (this value was calculated by equating the gross cross-sectional area of the H pile – that is, the flange width multiplied by depth – to an equivalent circular area). The distance from the base of the H pile and the

Sources of capacities	Closed-ended pipe pile (MCEP)						H pile (MHP)					
	First load test (50 days after pile driving)			Second load test (90 days after pile driving)			First load test (63 days after pile driving)			Second load test (99 days after pile driving)		
	Total: kN	Shaft: kN	Base: kN	Total: kN	Shaft: kN	Base: kN	Total: kN	Shaft: kN	Base: kN	Total: kN	Shaft: kN	Base: kN
Load at the end of the static load test	1646	1158	488	1690	1201	489	2092	1053	1039	2181	1156	1025
Ultimate load for a settlement of 10% of the pile diameter	1345	946	399	1500	1066	434	1839	933	906	1851	958	893
Limit load estimated from Chin's criterion	1678	1158	520	1744	1201	543	2282	1053	1229	2282	1156	1126
CAPWAP predictions from EOD (end of driving)												
CAPWAP predictions from BOR (beginning of restrike) ^a												

^a Restrike tests on RCEP-1 and MHP were performed 127 and 126 days after end of driving, respectively.
^b A restrike test on the main closed-ended pipe pile (MCEP) was not performed. These values were obtained from restrike PDA test on RCEP-1, which was driven to the same depth as MCEP. Total, shaft, and base capacities of RCEP-1 at EOD predicted from CAPWAP analyses were 961, 290, and 671 kN, respectively.

Table 3. Summary of measured and estimated load capacities

Figure 9. Load-transfer curves: (a) H pile; (b) closed-ended pipe pile

closed-ended pipe pile to the top of the clay layer underneath the very dense silt layer in which the pile was embedded was assumed to be equal to 1 m and 0.7 m, respectively. This corresponds to 2.9 times the equivalent circular H pile diameter (0.349 m) and 2 times the outer diameter of the pipe pile. Using $q_c = 50$ MPa for the dense silt layer and $q_c = 1.5$ MPa for the soft clay layer, the estimated q_{bL} values at the H pile and the pipe pile base according to Equation 3 are 24.3 MPa and 17.8 MPa, respectively. Note that the equivalent outer diameter of the H pile was calculated assuming that its gross rectangular area was operative at the pile base. This assumption is only valid when the H pile is fully plugged. In reality, the H pile base was most likely partially plugged and hence the operative base area may be in

Figure 10. Distribution of limit unit shaft resistances

between the actual H pile cross-sectional area and the gross rectangular area. This indicates that q_{bL} may have been higher than 24.3 MPa for the H pile because the equivalent outer diameter B_o would have been smaller than 0.349 m for partially plugged conditions. On the other hand, CPT results indicated that the thickness of the dense silt layer below the base of the closed-ended pipe pile may have been even less than 0.7 m, as mentioned previously. If this is indeed the case, q_{bL} of the closed-ended pipe pile would have been smaller than 17.8 MPa. This may partially explain why the base capacity of the H pile is almost twice as large as that of the closed-ended pipe pile.

5. Summary and conclusion

The literature contains a limited number of well-documented cases of instrumented pile load tests in which loading was extended to large enough displacements, load was applied at rates that may be related to situations of practical interest and for which also complete soil characterisation was done at the location of the pile load test. This has an important consequence: researchers attempting to develop new analyses or methods of design have little to no access to quality data to validate their research. One of the aims of this paper was to show very specifically what must be done to obtain information that is complete and that will thus be useful for researchers in the future.

The planning of a pile load testing programme, the principles of operation of the vibrating-wire strain gauges and the procedures required for their successful installation on steel piles have been discussed in detail. In this discussion, the results were used from static instrumented axial load tests performed on an H pile and a closed-ended pipe pile, both driven into a multilayered soil profile. For the test piles, several levels of strain gauges were

attached along their length in order to achieve accurate measurements of the load transferred to each soil layer. The vertical spacing between adjacent strain gauges was reduced near the base of each pile, as a high rate of load transfer was expected to occur at this depth (a very dense silty layer is present at the base of the piles). Ultimate loads corresponding to a settlement of 10% of the pile diameter (1839 kN for the H pile and 1345 kN for the closed-ended pipe pile according to the first loading test) and limit loads according to Chin's criterion (2282 kN for the H pile and 1678 kN for the closed-ended pipe pile according to the first loading test) were determined for both piles. The measured shaft capacities of the closed-ended pipe pile and the H pile were very similar (the closed-ended pipe pile showed a slightly larger shaft capacity). The measured load-transfer curves showed that a substantial portion of Q_{sL} was mobilised in the lower third of each pile. However, the base capacity of the H pile was about twice that of the closed-ended pipe pile. This was attributed to the different influence zones below the base of the piles and the smaller thickness of the bearing layer at the location of the pipe pile, which allowed less base resistance to develop at the location of the closed-ended pipe pile.

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