Measurement of stresses around closed-ended displacement piles in sand

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Calibration chamber experiments are reported that investigate the evolution of stresses around closed-ended, highly instrumented, model displacement piles during simulated driving into a heavily instrumented sand mass. The soil stresses are shown to vary spatially relative to the pile tip location. As well as showing considerable radial variation, the stresses developed at any given depth build sharply as the tip approaches, and reduce rapidly as it passes. Clear differences are evident between the behaviours seen close to the shaft during alternate penetration and pause periods. Load-cycling effects are most significant close to the shaft, where the local stress paths indicate a tendency for constrained ‘dilatant’ behaviour, with radial stresses increasing, during loading. In contrast, markedly ‘contractant’ radial stress reductions are evident on unloading.

KEYWORDS: model test; piles; sands; stress analysis; stress path

INTRODUCTION

Establishing the stress conditions around piles driven in sand is crucial to understanding processes such as group interaction (Chow, 1997) or capacity growth with time (Chow *et al.*, 1998; Jardine *et al.*, 2006). Driving involves extreme stresses and strains, particle breakage and load cycling, creep and localised interface shear processes that cannot be modelled fully at present (Liyanapathirana *et al.*, 2000; Sheng *et al.*, 2005; White *et al.*, 2005; Henke & Grabe, 2006). Experiments designed to explore the phenomena and provide benchmarks for analysis are described below, following studies by Nauroy & Le Tirant (1983), Allard (1990), Foray *et al.* (1993), Lehane & Gavin (2001) and Gavin & Lehane (2003).

CALIBRATION CHAMBER TESTS

Jardine *et al.* (2009) describe the arrangements for experiments with 36 mm diameter model piles in the 1.2 m internal diameter (ID) INPG calibration chamber shown in Fig. 1, which gave a pile-to-chamber diameter ratio of 33.3. The 1.5 m stroke, Rosier electro-thrust electric cylinder jack that advanced the pile through a roller assembly into the pressurised sand mass could be programmed with LabVIEW (National Instruments, 2003) under load (monotonic or cyclic, up to 45 kN) or displacement (from 0.01 to 10 mm/s) control. Each test involved a fresh sand mass that contained dozens of pre-installed soil stress cells. The sand vertical loading applied after placement was maintained during and after pile installation. Reported below are two cone penetration test (CPT) ‘pile’ trials and three tests with a reduced-scale version of the ICP pile developed to measure shaft surface stresses, axial loads and temperatures accurately in field tests (Bond *et al.*, 1991).

Boundary conditions can influence CPT test results considerably (Salgado *et al.*, 1998; Huang & Hsu, 2004). The chamber configurations, surcharge levels, instrument arrays and jacking styles applied are outlined in Table 1 and in the Appendix. A rigid outer radial boundary was applied throughout, and a 2 mm thick latex sheet, lubricated with silicone grease, reduced outer wall friction in the Mini-ICP experiments. The first four tests surcharged the sand mass through a ‘standard’ top membrane having a central 200 mm ID. A base-pressurised membrane was also fitted for the first three experiments. However, the ‘standard’ membranes appeared to induce undesirable stress non-uniformity, as noted in other tests by Eiksund (1994). Elastic-plastic finite-element (FE) analyses identified centreline stress anomalies propagating down to ~350 mm, and indicated that uniformity could be improved by reducing the membrane ID to 50 mm, which was adopted from Mini-ICP3 onwards. CPT check tests, shown in Fig. 2, proved that both membrane designs gave quasi-constant sections with $q_c = 21 \pm 2$ MPa (under 150 kPa) and friction ratios, f_r , around 1%. While the steady q_c was achieved at shallower critical depth with the 50 mm ID membrane, Mini-ICP3 identified a new shallow-depth stress concentration with this design; fitting a circumferential collar to oppose inward membrane spreading helped reduce this imperfection.

Turning to the effects of (a) vertical loading and (b) the stress sensors’ presence, Fig. 3(a) shows the spread of q_c profiles normalised by $(\sigma'_{z0}/P_a)^{0.5}$ to eliminate the effects of slight surcharge differences (Lunne *et al.*, 1997). Maxima with $17.6 < q_c/(\sigma'_{z0}/P_a)^{0.5} < 23.0$ developed over the quasi-constant sections at the indicated reference level. The maxima correlate imperfectly with the number of stress sensors installed in the sand mass. Adding one layer of 12 stress sensors appears to boost q_c by around 15%, rising to 17% when three such layers are placed. The effects appear to apply over depth intervals around ± 7 pile diameters above and below each sensor layer. Fig. 3(b) goes on to show a further, more localised, instrument layer influence on side friction that boosts f_s by around 8% over an interval around 50 mm above and below the instruments.

Test sand

Fine NE34 Fontainebleau sand, with the index properties given in Table 2, was selected to reduce particle-scale effects. Yang *et al.* (2010) gave full details of the

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Fig. 1. Schematic diagram of Mini-ICP1 test showing one example instrument layout

Fig. 2. Cone resistance q_c profiles for alternative top-membrane designs

mineralogy, grain shapes and sizes. They also reported peak and critical state ϕ' values of 35° and 33° from direct shear tests on samples with $e_0 = 0.62$. Interface ring-shear tests conducted against steel surfaces prepared to match the piles developed shear zones comprising fractured grains and crushing products. After several metres of shear displacement under $100 < \sigma'_h < 800$ kPa, their ultimate δ'_{cv} values

were 25 – 27° . High-pressure triaxial compression tests conducted by Altuhafi & Jardine (2011) to match the pile test conditions indicated $\phi'_{cs} = 30^\circ$ in the tip-crushing zone, where $\sigma'_1 > 20$ MPa. Samples that were sheared under these high pressures before being retested under lower stresses ($300 < p'_f < 800$ kPa) developed strong dilation with $\phi'_{peak} = 42^\circ$ and $\phi'_{cs} = 33^\circ$.

The sand was placed in the chamber by air pluviation from a full-aperture hopper with a drop height of 500 mm, leading to an average filling rate of 0.225 mm/s. An average $e_0 = 0.62$ ($D_r = 72\%$) was evaluated in a check test. Pluviation was halted at up to three stages to allow soil stress sensors to be placed, which might induce local non-uniformity in e_0 , particularly around the shaft. The pluviation settings were kept constant, and checks confirmed that filling rates varied by less than 0.015 mm/s throughout the programme. Minor drop-height variations are considered unlikely to have affected e_0 significantly (Vaid & Negussey, 1984).

Soil stress sensors

Dozens of strain-gauged vertical, radial and circumferential soil stress cells were deployed at locations between $2R$ and $20R$ from the pile axis (where R = pile radius), in patterns designed to minimise mutual interference. The Appendix lists the arrangements for each individual test, and Fig. 4 shows a typical plan view of a single instrument layer. Zhu *et al.* (2009) describe full details of the disc-shaped cells, which had capacities of 500 kPa to 7 MPa and diameters of 6–6.5 mm, and were 0.6–1.4 mm thick. Their faces were oriented orthogonal to the normal stress to be measured. As noted above, the cells and cables appear to reinforce the sand mass sufficiently to raise local q_c values by 15–17%, making it vital to record

Table 1. Programme summary

Test	Preparation date	Installation date	Regulated temperature: °C	Boundary conditions			Soil stress sensors	Pile installation				
				Surcharge pressure: kPa	Pressure membrane	Lateral latex		Depth: mm	Stroke: mm	Jacking rate: mm/s	Installation style	Maximum force: kN
CPT1	27 Jul. 2007	27 Jul. 2007	N/A*	200	200 mm ID top and base	No	12(1)	1000	10	1	Partial unloading	48.7
CPT2	21 Sep. 2007	21 Sep. 2007	N/A	163	200 mm ID top and base	No	12(1)	1020	10	1	Full unloading	34.0
Mini-ICP1	26 Nov. 2007	26–27 Nov. 2007	16	150	200 mm ID top and base	Yes	36(3)†	920	10	0.5	Full unloading	34.4
Mini-ICP2	30 Apr. 2008	20 May 2008	18	152	200 mm ID top only	Yes	40(2)	990	5	0.5	Full unloading	37.7
Mini-ICP3	28 Apr. 2009	29 May 2009	18	152	50 mm ID top only	Yes	36(3)	990	20	0.5	Full unloading	37.9

* N/A: temperature not regulated.

† 36(3): signifies 36 cells installed on three levels.

Fig. 3. (a) Cone resistance traces applied in test analysis (see the Appendix); (b) sleeve friction f_s of CPT2 test

local q_c during installation. The CPT sleeve friction data seen in Fig. 3(b) suggest further local effects in congested areas close to the shaft, where sand density might also vary. As discussed later, interactions were noted between the soil sensors and pile surface stress transducer (SST) measurements in Mini-ICP1; the instrument levels were revised in subsequent tests to avoid leaving the SSTs within the soil sensors' local zones of influence at the end of installation.

Mini-ICP

The design and calibration of the stainless steel 36 mm diameter Mini-ICP were described by Jardine *et al.* (2009). Tests Mini-ICP1 and Mini-ICP2 adopted the configuration

Table 2. Index properties of NE34 Fontainebleau sand

Grain shape	SiO ₂ : %	Specific gravity, G_s	d_{10} : mm	d_{50} : mm	d_{60} : mm	Coefficient of uniformity, C_u	$e_{\max}$	$e_{\min}$
Sub-angular	99.70	2.65	0.15	0.21	0.23	1.53	0.90	0.51

Fig. 4. Plan layout of soil stress sensors at one level in Mini-ICP2; Appendix gives details of layout for each test

shown in Fig. 5. The shaft was air-abraded to a mean roughness, R_{CLA} , of 3–4 μm before each test. The tip comprises a 60° cone that aided positioning, limited soil sensor damage, and allowed comparison with pilot CPT

Fig. 5. Configuration for Mini-ICP1 and Mini-ICP2

tests. The instrument clusters are orientated at 120° intervals around the shaft to minimise shaft-bending effects. Each comprises: an axial load cell (ALC), which helps to identify average shaft friction, f_s ; an SST that measures local radial total stress (σ_r) and shear stress (τ_{rz}); a temperature sensor; and MEMS inclinometers. The instruments are identified by their normalised distance h/R above the tip. A tip load cell was added to Mini-ICP3 to help isolate base resistance; earlier tests relied on check CPT trials combined with extrapolations made from the higher ALC and SST measurements.

Signal conditioning

Each Mini-ICP cluster had dedicated on-board power regulation, a 24-bit data logger and a microprocessor. They were powered through a common input line, and transmitted their digital information to two external computers through a telemetry cable. Purpose-built ‘mini-pile’ software was installed on one PC, while the second ran the Docklight commercial software as a back-up, logging the raw hexadecimal output. The system was synchronised with an NI Compact FieldPoint programmable automated controller (PAC) that logged the other instruments at 16-bit accuracy. Individual cables connected gauges to the PAC, which embedded the intelligent control and analysis capabilities of NI LabVIEW 8.2. The FieldPoint’s low-pass digital filtering was adjusted to optimise signal-to-noise (S/N) ratios with cut-offs set at 50–60 Hz for all sensors. The jack and membrane pressure control was updated at 350 Hz, and the overall data-sampling frequency was 1 Hz. An independent automatic system regulated the chamber’s temperature.

Potential errors in stress measurements

It is generally acknowledged that accurate soil stress measurement is very difficult to achieve at discrete points; careful transducer calibrations and closely controlled temperatures are essential. The very stiff SST cells incorporated into the Mini-ICP pile minimised cell action under-registration. Jardine *et al.* (2009) calculated upper-bound potential errors of 11%; cross-relating (a) axial load cell transfer trends with local shear stresses and (b) Mini-ICP and laboratory δ' values confirmed generally lower cell-action errors. Instrument cross-sensitivities were also calibrated out carefully; checks also showed that Poisson effects caused by pile axial loads were negligibly small. While the Mini-ICP data are internally consistent, it is acknowledged that measurements can be influenced by chamber boundary conditions, potential sand non-uniformity, differences between individual pluviated sand masses, and the presence of the soil stress cells and their cables. The latter cells developed more significant interactions with the surrounding sand as they underwent multiple loading cycles (up to several MPa) before settling to typically greatly reduced end-of-installation stresses. Intensive calibrations, designed to follow the same patterns, were conducted in a special sand-filled cell. The results illustrated in Fig. 6 show interactions that render the manufacturers' fluid pressure calibrations inappropriate. Zhu *et al.* (2009) detailed the non-linear, stress-history-dependent calibration procedures and models, noting the following.

- Initial 'virgin loading' L curves. Deviations between measurements and power fitting functions ranged between 7 and 40 kPa, depending on capacity and type.
- The hysteretic unloading U curves could be normalised by their maximum loads to provide second backbone functions. Fitting two-part exponential expressions limited typical errors to 15–60 kPa.
- Reloading could be described by linear extensions to the L and U curves. Individual measurements deviated by 10–60 kPa.
- Overall typical errors under the pile test conditions fall in the 50–150 kPa range. The minima and maxima recorded in tests were ~40 kPa and ~5 MPa respectively.
- Errors are likely to be proportionally least significant at high stress locations during steady pile penetration, and most significant at points that have relaxed to low stresses following high stress cycling.
- Cell action errors can be reduced, at the expense of sensitivity, by adopting stiffer devices with capacities far above the anticipated maximum soil stress. Averaging is also beneficial.

The soil stress measurements made in the chamber under ~150 kPa surcharge prior to pile tests demonstrated that the annular top-membrane geometry gave vertical stress reductions over the $2 < r/R < 8$ range at shallow depths. The measurements were broadly in line with FE predictions for the two membrane geometries. After discounting the shallowest, untypical stresses measured in Mini-ICP1, the average vertical stress assessed from multiple comparable measurements was 140 kPa, close to that expected under 150 kPa when account is taken of the membrane's annular shape. The average radial and circumferential stresses were 62 kPa and 64 kPa, giving average $K_0 \approx 0.45$, which is close to an estimate from $K_0 \approx 1 - \sin \phi' = 0.43$, taking $\phi' = 35^\circ$. While the largest deviations from the expected stresses were 25 kPa for radial and circumferential stresses and 45 kPa for vertical stresses, around 70% of the measurements fell within 15% of the mean values.

The data scatter is interpreted as being due mainly to

Fig. 6. Typical calibrations for: (a) 3 MPa capacity cells; (b) 7 MPa capacity cells

- true stress non-uniformity
- temperature variations between sensor placement and the final temperature-controlled, pressurised conditions
- possible positioning errors.

The last potential difficulty was addressed by using positioning diode lasers mounted on a horizontal bar fitted to the chamber top. One vertical beam projected the chamber's axis (tracing the line of pile penetration) while a parallel beam projected the desired sensor location. The 'sensor' beam was aligned for each instrument by referring to a circular inscribed template that fitted the chamber interior tightly and mapped the radial distances from the axis and the diamet-

rical directions. Instrument levels were confirmed by measuring vertical distances up to the horizontal bar. The cells' initial positions were known to within 1 mm, or $0.05R$. The radial distances between each sensor and the perimeters of (a) the pile and (b) the chamber internal wall were re-measured during post-testing excavation. Sensors in the $r \leq 3R$ range experienced relatively small changes in position (radial displacements up to 3 mm, vertical < 2 mm) due to pile installation and testing; movements at $r \geq 5R$ were too small to measure (< 1 mm) with the means deployed. While angular orientations were hard to measure, horizontal checks on vertical stress cells with a sensitive small spirit level indicated final inclinations $< 10^\circ$, which appeared to be representative for all three axes. Measurement errors resulting from unmeasured rotations grow with the applied stress ratio. Mohr circle analysis indicated a maximum error of $\sim 8\%$ for a 10° rotation or misalignment. The calculation considered $\sigma'_1/\sigma'_3 \approx 6$ as an upper limit, corresponding to a failure state with $\phi' = 45^\circ$, the maximum angle that could be expected with heavily pre-loaded and crushed sand (Altuhafi & Jardine, 2011). The stress ratios and potential errors would be far lower in most cases.

Programme of experiments

Table 1 and the Appendix summarise the detailed experimental arrangements of each test, and Fig. 3(a) reports the respective CPT profiles. The levels at which the soil stress measurements were made in the soil mass are indicated also in Figs 7 and Figs 9–15. Installation jack stroke lengths varied between 5 mm and 20 mm. While pile head loads were not released between strokes in CPT1 (see Fig. 7), they were reduced to zero between strokes in all other installations. Penetration rates were slowed to aid logging and control in the Mini-ICP installations; creep and ageing periods were also imposed that increased penetration tip resistances. Ageing periods were imposed after installation to permit potential shaft capacity changes with time, as seen in field tests by Jardine *et al.* (2006). Monitoring was continuous, and good control was maintained over surcharge pressures and temperatures throughout (Jardine *et al.*, 2009). Mini-ICP3 had the best overall test configuration. Unfortu-

nately, electrical connections were lost with the on-pile instruments (but not the soil stress sensors) during the latter part of its installation. While data from all five installations are used later in a synthesised interpretation, the focus below is on Mini-ICP1 and Mini-ICP2 to illustrate general aspects of the experiments.

STRESSES MEASURED ON THE MINI-ICPs

The axial forces and soil stresses developed on the piles varied cyclically as (a) the tip penetrated and (b) the head load was released before the next stroke. Below, the 'moving' (suffix 'm') data recorded during steady penetration (towards the end of each 'push') are distinguished from the temporarily stationary (suffix 's') zero load stage measurements.

Variation of axial load and surface stresses within a cycle

A typical jacking cycle is illustrated in Fig. 8, showing variations with time of (a) local shaft stresses and (b) axial loads and pile head displacements, as well as illustrating in (c) the local stress paths.

- Local axial load and shaft shear stress increase as the pile penetrates, tending to steady values after ~ 8 mm.
- Shaft shear stresses fall during unloading and become negative to counteract the locked-in toe load (≈ 1 kN).
- The radial stresses initially reduce, but then climb during penetration as the effective stress paths approach the limiting shaft interface δ' line. This stress path rotation is interpreted as indicative of constrained dilation, and is seen as a local phase transformation (PT) process. The constrained dilation continues until a constant 'critical state' is reached at ~ 8 mm, which is maintained until the axial load is released.
- The local shaft effective stresses reduce on unloading, indicating a tendency towards constrained contractive behaviour, until the path approaches a second PT point associated with reverse (upward or tensile) failure. The path then climbs the tension failure line, and terminates slightly to the right of the start point.
- Similar δ' values apply at the downward and upward PT points (around 21°) and respective shaft failure conditions ($27\text{--}28^\circ$).

While the pile head experiences purely compressive loading, the shaft experiences fully two-way shear cycling. The PT points mark changes in the dilatancy response, and are analogous to those noted in undrained triaxial tests or constant normal stiffness interface shear tests (Ishihara *et al.*, 1975; Boulon & Foray, 1986).

Axial loads and base resistance

The leading ALCs' axial force variations with penetration depth, L_p , are illustrated in Fig. 9, showing (a) steady penetration data and (b) stationary profiles along with the respective depths of the soil stress sensor layers. Eliminating the base membrane (see Table 1) led to less variation towards the end of jacking in Mini-ICP2, and the full set of profiles from this test is shown in Fig. 10. The traces are affected by the top-membrane geometry down to $L_p \approx 350$ mm. Note also that because the lowest ALC includes a significant shaft resistance component, the base ($h/R = 1.7$) trace in Fig. 10(a) was projected from a parallel CPT test. Modifying the top membrane and adding a base ALC made this easier in later tests.

The Q_m loads developed peaks at 550 mm and 990 mm, reflecting possible variations in sand state, chamber boundary or soil sensor effects as discussed earlier, while the Q_s values remained small at even the leading cluster until $L_p > 400$ mm.

Fig. 7. Jacking forces during installation, CPT1 (M, middle level of sensor C1)

Fig. 8. Local response at leading instrument cluster during 52nd jacking cycle of Mini-ICP1 (tip depth 520 mm). Variations with time of: (a) shaft stresses; (b) local axial load and displacement; (c) local stress paths

The residual tip pressures grew linearly from this tip depth to reach around 22% of q_c at the end of jacking. Differentiating the axial load profiles with respect to depth provides traces for the back-up average local shaft friction, f_s . Examples are presented in Fig. 11 for the leading–following (mid $h/R = 18.4$) and following–trailing clusters (mid $h/R = 35.9$) of Mini-ICP1 and Mini-ICP2. While the two tests are broadly comparable, Mini-ICP2 gave higher f_s values during pushes, possibly as a result of the different jack strokes and/or stress sensor layouts. Comparison of Fig. 11(b) with Fig. 11(a) suggests f_s reducing sharply with increasing h/R , as in field ICP tests in sand (Lehane *et al.*, 1993; Chow, 1997).

Fig. 9. Loads at leading ALC at ends of each (a) push and (b) pause, with soil instrument positions shown (T, top level of sensor; M, middle level of sensor; B, bottom level of sensor; M1, Mini-ICP1; M2, Mini-ICP2; for leading ALC position see Fig. 5 and Appendix)

Radial stresses

Summary traces for Mini-ICP1 and Mini-ICP2 are presented in Fig. 12, along with the respective depths of the soil stress sensor layers that cover the full range of penetrations, plotting radial effective stress against sensor depth. Normalised plots are given in Fig. 13 that apply the q_c traces given in Fig. 3(a). The measurements made at $z < 350$ mm lead to σ'_r/q_c values that fall below those seen in field tests, and fail to show the expected stress reductions with h/R . Noting also that the improved Mini-ICP3 did not repeat this pattern, the shallow data are considered artefacts of the ‘standard’ top-membrane geometry, and are excluded from Fig. 13. Points of note at $z > 350$ mm include the following.

(a) A trend for the leading SST to develop higher stresses

Fig. 10. Loads at all ALCs at ends of each (a) push and (b) pause in Mini-ICP2, with soil instrument positions shown (for ALCs' positions see Fig. 5 and Appendix)

under both moving and stationary conditions, confirming a strong reduction of σ'_r with h/R .

- (b) Unexpected fluctuations in σ'_r with depth for both h/R ratios. In particular, the leading cells tend to fall at depths below the $z = 550$ mm stress sensor array. While not fully explained, these variations are unlikely to reflect cell action. They are more probably related to the test boundary conditions, variations in sand state, or the effects of instrument cables.

Shear stresses

Figure 14 presents the corresponding local shear stress measurements. Measurements at $z < 350$ mm are again considered unrepresentative products of the top-membrane geometry. Field trends such as the influence of h/R on shear stresses are clearer at greater depths, particularly during the pause periods. Fig. 14(a) also presents the back-up f_s values

Fig. 11. Profiles with penetration depth of average shaft friction f_s between (a) leading–following (mid $h/R = 18.4$) and (b) following–trailing (mid $h/R = 35.9$) clusters

at the following SST level derived from the ALCs at the end of each push or pause in Mini-ICP1, showing stresses that are generally compatible with the local SST data, except at the 190 mm and 550 mm soil stress sensor array depths.

As noted earlier, the local stresses on the shaft register local effects within ± 50 mm of the stress sensor arrays that have less influence on the ALC measurements. The following and trailing SST τ_{rz} measurements made within ± 50 mm of the top and middle sensor arrays were therefore considered less reliable than the f_s profiles. Corrections were made over the same depths to the shaft σ'_r values (developed at failure) by taking $\sigma'_r = \tau_{rz}/\tan \delta'$ and applying the mean δ' value.

Corrected σ'_{rs}/q_c and τ_{rz}/q_c profiles are given in Fig. 15, covering the end of each pause in Mini-ICP1. The traces have steadier trends with depth than those in Fig. 14, giving σ'_{rs}/q_c of around 1.2% and 0.7% at $h/R = 6.7$ and 21.7 respectively, while Mini-ICP2 gave slightly higher values (1.75–0.9%; see Fig. 13). These values fall near the σ'_{rs}/q_c ranges anticipated (1.4% and 0.9%) at the same h/R values and for closed driven field piles by the ICP design approach (Jardine *et al.*, 2005). While the experimental boundary conditions are different from the field pile case, where σ'_{r0} varies strongly between pile head and tip, it is encouraging to find broad agreement.

Stress paths during installation

The local effective stress variations experienced during a single jacking cycle were discussed earlier and presented in Fig. 8(c). Fig. 16 reports the envelopes provided by the

Fig. 12. Radial effective stresses recorded by SSTs at end of (a), (c) each push and (b), (d) each pause in: (a), (b) Mini-ICP1; (c), (d) Mini-ICP2

moving and stationary leading and following SSTs in Mini-ICP1 and Mini-ICP2. The leading SSTs recorded similar trends during their push stages, indicating a Coulomb relationship with a best-fit 'critical state' $\delta' = 26.7^\circ$. During pauses the upward (tensile) shear stresses developed at the leading position in Mini-ICP1 fell slightly below the trend line, suggesting that the shaft may not have undergone full reverse interface failure during unloading. Similar trends applied in Mini-ICP2, with the leading stationary data falling further below the penetration trend line during the later stages of installation, possibly because of the smaller penetration increments (5 mm per stroke) applied in this test. The following cluster measurements scattered around the same $\delta' \approx 27^\circ$ trend in Mini-ICP1, but there was greater variation at this position in Mini-ICP2. To save confusion, the Mini-ICP2 stress paths plotted for the following position are corrected by adopting the back-up ALC f_s data and calculating σ_r' by assuming local failure with $\delta' = 26.7^\circ$.

Yang *et al.* (2010) investigated the underlying physical processes, including

- (a) pile surface abrasion
- (b) compaction, particle breakage and re-orientation within a well-defined interface shear zone that develops under the tip and adheres to the shaft
- (c) the relationship between the pile's interface behaviour and that in ring-shear tests, which give practically identical δ' values.

They showed that the interface shear zone is denser, and contains fractured grains. Ring-shear tests show thicknesses that grow with the severity of shear loading and scale-up with initial sand particle diameter (Ho *et al.*, 2011).

STRESSES IN THE SAND MASS

As outlined in Table 1, measurements were made in five installations of the effective vertical, radial and circumferential soil stresses σ_z' , σ_r' and σ_θ' , taking the cells' zero datum values as those at the time of initial placement. Extreme strains and stresses developed close to the pile

Fig. 13. Normalised radial effective stresses recorded by SSTs at end of (a), (c) each push and (b), (d) each pause in: (a), (b) Mini-ICP1; (c), (d) Mini-ICP2

axis; some instrument cables parted in each test, and several devices were overloaded (as indicated by signal failures or check calibrations after testing). The Appendix identifies the failure rates; Mini-ICP2 suffered the least, and with this test the main features carried forward for detailed interpretation are illustrated. Several of the key findings confirm and expand on observations made earlier by Allard (1990) and Gavin & Lehane (2003). The two CPT 'pile' tests provide new insights into how installation cycles affect the sand stresses.

Vertical stress

Vertical stress (σ'_z) trends are illustrated in Fig. 17, normalising by local q_c (22.3 MPa) the measurements made at two radial positions ($r/R = 3$ and 8) at a fixed depth below ground level ($z = 700$ mm) where the vertical stress was nominally uniform initially under the ~ 150 kPa sur-

charge. The ratios seen at penetration depths $L_p = (h + z)$ down to 990 mm are plotted against h/R . Note that the sections of such plots with $h/R < 0$ describe how the stresses (at any fixed instrument depth below ground level) develop as penetration brings the pile tip closer to the instrument from above. The $h/R > 0$ sections show how stresses decline as the tip passes and progresses to greater relative depths (h) below the instrument. As noted earlier, the absolute movements experienced by the instruments are relatively small compared with the pile displacements.

The early stages of penetration show σ'_z/q_c decreasing slightly (suggesting axial extension) and reaching minima at $h/R \approx -25$ before rising to sharply defined maxima at $h/R = -9$ and ~ -4 (for $r/R = 8$ and 3 respectively) that are clearer close to the shaft. The maxima seen in the moving σ'_{zm}/q_c profiles are about 1.6 and 3 times the corresponding stationary σ'_{zs}/q_c ratios at $r/R = 8$ and 3 . Marked reductions took place as penetration continued and h/R increased,

Fig. 14. Shear stresses recorded by SSTs at the ends of each push and pause in: (a) Mini-ICP1; (b) Mini-ICP2

particularly at $r/R = 3$. The more radially distant position ($r/R = 8$) appeared to show higher stresses once $h/R > 2$.

Radial stress

Figure 18 presents similar σ'_r/q_c profiles for sensors placed at $z = 700$ mm, $r = 3R$ and $8R$. The initial radial stresses are compatible with the surcharge and $K_0 \approx 0.45$. As with the vertical stresses, σ'_r decreased slightly until $h/R > -20$ and then grew until maxima developed at $h/R = -2.5$ to 0 (with the lower value applying to the higher r/R). The moving maxima are 95% and 45% higher than the stationary maxima at $r/R = 3$ and $r/R = 8$. Stresses fell sharply as penetration continued, becoming less marked once $h/R > 10$ and showing less difference between the stationary and moving measurements.

Circumferential stress

Figure 19 presents the corresponding circumferential (σ'_θ) stress trends, plotting measurements made at $r = 3R$ and $8R$ against h/R for the same 700 mm depth level; σ'_θ decreased with initial penetration (until $h/R = -30$) before building up sharply and developing well-defined maxima at $h/R = -4$ to 0 . Steep falls took place with continuing penetration, before reducing to final values where $\sigma'_\theta < K_0 \sigma'_{z0}$. The differences between the moving and stationary maxima are significant (30–60%), but less marked than with σ'_r or σ'_z .

Effect of jacking style on radial stresses

The effects of jacking style were investigated in the two CPT 'pile' tests. Jack loads were maintained between strokes

Fig. 15. Corrected stresses against measurement depth at end of each pause in Mini-ICP1: (a) radial effective stress; (b) shear stress

in CPT1, while CPT2 (and the Mini-ICP tests) employed full unloading between each stroke (see Table 1). The parallel sets of measurements illustrated in Fig. 20 for gauges set at $r/R = 4.9$ indicated that the jacking style had relatively little effect on the $\sigma'_r/q_c - h/R$ trends, especially during pauses. A lower maximum developed in CPT2 during pushing (~20% less), with further variations at $h/R > 0$, but the general decay in σ'_r with h/R does not appear to be strongly affected by installation cycles.

The two Mini-ICP tests provide additional insights. The 50% shorter jack strokes imposed in Mini-ICP2 led to twice as many cycles as in Mini-ICP1. Figs 21(a) and 21(b) present the normalised moving horizontal stress, σ'_{rm}/q_c , measured closer to the shaft (with $h/R > 0$) at depths of $z = 550$ mm and 700 mm at $r = 2R$ and $3R$ respectively, and

Fig. 16. Stress paths recorded by SSTs at the end of (a), (c) each push and (b), (d) each pause in: (a), (b) Mini-ICP1; (c), (d) Mini-ICP2 (following traces in (c) and (d) interpreted from back-up f_s data from ALCs combined with $\delta' = 26.7^\circ$)

Fig. 17. Typical vertical stress during Mini-ICP2 installation at end of each: (a) push (σ'_{zm}); (b) pause (σ'_{zs})

Fig. 18. Typical radial stress during Mini-ICP2 installation at end of each: (a) push (σ'_{rm}); (b) pause (σ'_{rs})

Figs 22(a) and 22(b) show the normalised stationary horizontal stress, σ'_{rs}/q_c , at the same locations. The datasets have similar initial σ'_{rs}/q_c maxima as the pile tip passes, and fall to similar final low σ'_i/q_c ratios, but the decay curves applying between these limits converge better when plotted against h/R than against N , indicating a closer correlation with relative pile tip depth than with number of cycles. While stronger dependence on N might be expected at the pile/soil interface, where intense two-way cycling applies (see Fig. 8(c)), the on-pile measurements presented in Figs 13 and 15 indicate that doubling N did not lead to steeper radial stresses reductions for fixed h/R ratios.

SUMMARY AND CONCLUSIONS

Local stress measurements have been presented from calibration chamber tests with closed-ended model piles,

Fig. 19. Typical circumferential stresses during Mini-ICP2 installation at end of each: (a) push ($\sigma'_{\theta m}$); (b) pause ($\sigma'_{\theta s}$)

supported by associated soil element testing. The stress measurements are highly challenging, and care has been taken to address multiple potential sources of error. While some imperfections could not be eradicated, the data generally are self-consistent and repeatable. The pile's behaviour appears to be broadly compatible with field trends.

The paper focused first on the measurement techniques and the potential difficulties with sensors, chamber boundary conditions, cables and sand state. The discussion of the pile and CPT experiments has focused on the internal consistency of the data, the effects of load cycles imposed by installation, the normalisation of stresses, and key differences between steady penetration and stationary pause stages. Key conclusions are as follows.

- (a) Pile and soil stresses are closely related to the CPT tip resistance.

Fig. 20. Effect of installation method on radial stresses (CPT1 and CPT2) at the end of each: (a) push; (b) pause

Fig. 21. Comparative influences on moving radial stresses of: (a), (b) number of cycles, N ; (c), (d) relative pile-tip depth, h/R

Fig. 22. Comparative influences on stationary radial stresses of: (a), (b) number of cycles, N ; (c), (d) relative pile-tip depth, h/R

- (b) Shaft failure is governed by the Coulomb law, matching interface ring-shear test results.
- (c) Stresses recorded at given depths in the sand mass rise sharply as the pile tip approaches, and decline sharply as it penetrates below to greater depth. They also vary with radial distance from the pile axis.
- (d) Stress patterns can be defined relative to the moving pile tip, with normalised axial coordinates r/R and h/R .
- (e) Stresses vary greatly between 'push' and 'pause' stages.
- (f) Stresses developed at points away from the shaft, at the end of installation, appear to be only weakly dependent on the total number of jacking cycles.
- (g) Installation imposes two-way cyclic failure, with both contractant and dilative phases of interface shear developing during each stroke.

- (h) Particle crushing and shear band formation processes have a key bearing on the phenomena observed.

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APPENDIX: ARRANGEMENTS OF SOIL STRESS SENSORS AND INSTRUMENTS ON THE MINI-ICP

Table 3 presents the detailed summary of sensors used in the tests, Table 4 gives the typical locations of instruments along the pile shaft, and Table 5 sets out the key aspects of the conditions applying to the five tests, and the rationale for adopting each CPT profile.

Table 3. Summary of depth and radial (r) locations of soil sensors

Test	Depth below sand surface: mm	Radial sensor positions, r/R	Vertical sensor positions, r/R	Circumferential sensor positions, r/R
CPT1	660	2(F*), 3, 5, 8(F), 20	3, 5, 8	2, 3(F), 5, 8
CPT2	735	2, 3, 5, 8, 20	3, 5, 8(F)	2(F), 3(F), 5(F), 8(F)
Mini-ICP1	Top: 190 Middle: 550 Bottom: 830	2, 3(F), 5, 8, 20 2, 3, 5, 8, 20 2, 3(F), 5, 8, 20(F)	3(F), 5, 8(F) 3, 5, 8 3, 5(F), 8	2, 3(F), 5, 8(F) 2, 3(F), 5, 8(F) 2(F), 3(F), 5, 8(F)
Mini-ICP2	Top: 430 Bottom: 700	2, 3, 5, 8, 12, 16, 20 2, 3, 5, 8, 12, 16, 20	3, 5(F), 8(F), 12(F), 16, 20 3, 5, 8, 12, 16, 20	2(F), 3, 5, 8, 12, 16, 20 2, 3, 5, 8, 12, 16, 20
Mini-ICP3	Top: 270 Middle: 460 Bottom: 730	2(F), 3, 5, 8 2, 3, 5, 8 2(F), 3, 5, 8(F)	2, 3(F), 5, 8 2, 3(F), 5(F), 8 2(F), 3, 5(F), 8	2, 3, 5, 8(F) 2(F), 3, 5, 8 2(F), 3, 5, 8(F)

Pile radius $R = 18$ mm.

*F = failure, attrition rate = 32%.

Table 4. Typical location of instruments along pile shaft

Cluster	Instrument	Height above pile tip, * h : mm	Normalised distance, † h/R	Typical final depth below sand surface for CPT2: mm
Trailing	ALC	827	45.9	193
	SST	750	41.7	270
Following	ALC	467	25.9	553
	SST	390	21.7	630
Leading	ALC	197	10.9	823
	SST	120	6.7	900

* All distances measured from the cone tip (angle of cone = 60°; distance of cone shoulder from the tip = 31 mm) to the sensor's centre.

† Pile radius $R = 18$ mm.

Table 5. CPT profiles adopted for pile test interpretation

CPT profile	Pile test boundary conditions	Source for q_c profile applied for pile test interpretation
CPT1: Profile A CPT2: Profile B Mini-ICP1: Profile C	With top and bottom membrane; no lateral latex sheet With top and bottom membrane; no lateral latex sheet With top and bottom membrane and lateral latex sheet	CPT1 CPT2 Represents test conditions by combining: top level: CPT profile B, middle level: CPT profile D and bottom level: CPT profile D, modified to capture base membrane presence
Mini-ICP2: Profile D Mini-ICP3: Profile E	No bottom membrane; extended pre-installation ageing No bottom membrane; modified top membrane; extended pre-installation ageing	CPT test with the same boundary as Mini-ICP2 CPT test with the same boundary as Mini-ICP3

NOTATION

C_u coefficient of uniformity
 D_r relative density
 d pile diameter
 d_{10} soil particle diameter that 10% of all soil particles are finer (smaller) by weight
 d_{50} soil particle diameter that 50% of all soil particles are finer (smaller) by weight
 d_{60} soil particle diameter that 60% of all soil particles are finer (smaller) by weight
 $e_{\max}$ maximum void ratio
 $e_{\min}$ minimum void ratio
 e_0 initial void ratio
 f_r CPT friction ratio
 f_s shaft friction

G_s specific gravity
 h distance above pile tip
 K_0 coefficient of earth pressure at rest
 L_p penetration depth
 p_a atmospheric pressure
 p_f^i mean effective stress at failure
 Q_{Lm} leading axial load when moving
 Q_{Ls} leading axial load when stationary
 Q_m axial load when moving
 Q_s axial load when stationary
 Q_t jack load
 q_c cone resistance
 R pile radius
 R_{CLA} roughness
 r radius from pile axis

V_{in}	transducer excitation voltage
V_{out}	transducer output voltage
z	depth
δ'	interface angle of shearing resistance
δ'_{cv}	δ' value at critical state
σ	soil stress
σ'_n	normal effective stress
σ'_r	radial stress
σ'_r	effective radial stress
σ'_{rm}	moving radial stress
σ'_{rs}	stationary radial stress
σ'_z	effective vertical stress
σ'_{zm}	moving vertical effective stress
σ'_{zs}	stationary effective vertical stress
σ'_{z0}	initial effective vertical stress
σ'_θ	effective circumferential stress
$\sigma'_{\theta m}$	moving circumferential stress
$\sigma'_{\theta s}$	stationary circumferential stress
σ'_1	major principal effective stress
σ'_3	minor principal effective stress
τ_{rz}	vertical shear stress
$\tau_{rz,m}$	moving vertical shear stress
$\tau_{rz,s}$	stationary vertical shear stress
ϕ'	effective angle of shearing resistance
ϕ'_{cs}	ϕ' value at critical state
ϕ'_{peak}	ϕ' value at peak resistance

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