Field experiments at three sites to investigate the effects of age on steel piles driven in sand

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INTRODUCTION
Studies of the reliability of conventional design methods (including the internationally applied API (2011) main text method) predictions for the axial capacities of large piles driven in sand have revealed wide scatter and significant bias; see, for example, Tang et al. (1990) or Jardine & Chow (1996). Alternative methods that offer better reliability include the Fugro-05 (Kolk et al., 2005), ICP-05 (Jardine et al., 2005), NGI-05 (Claussen et al., 2005) and UWA-05 (Lehane et al., 2005) approaches. All four, which are now cited in API's commentary, employ site-specific CPT profiling to characterise the sand state and recognise the effect of the relative pile tip depth (h) on shaft capacity. Independent database studies by Yang et al. (2017) and Lehane et al. (2017) involving high-quality load tests on piles with outside diameters exceeding 0.2 m gave broadly similar results, showing that the ‘full’ ICP-05 and UWA-05 methods for sands offer the lowest degrees of predictive scatter and bias. Field pile driving monitoring has also confirmed that ‘full’ ICP-05 predictions are representative of the shaft capacities developed by large offshore piles within a few days of driving; see Overy (2007) or Jardine et al. (2015).

However, most field load tests are conducted relatively soon after driving and axial capacities change with time. Schmertmann (1991), Åstedt et al. (1992), Chow et al. (1998), Bea et al. (1999), Axelson (2000), Jardine et al. (2006), Gavin et al. (2013), Karlsrud et al. (2014), Lim & Lehane (2014), Gavin et al. (2015) and Rimoy et al. (2015), among others, have reported marked growth over weeks and months after driving in sands, although the data are often widely scattered and the processes that control ageing remain uncertain. Ageing appears to benefit shaft capacity primarily (Rimoy et al., 2015) and its effects are clearest in ‘first-time’ tension tests to failure. Jardine et al. (2006), Karlsrud et al. (2014), Rimoy et al. (2015) and Gavin et al. (2015) reported from ‘first-time’ tension tests on open-ended steel piles (340 mm < OD < 508 mm (OD, outside diameter)) at three sand sites the systematic trend shown in Fig. 1. Capacity growth took place in the first 8 months after installation before reaching an upper limiting capacity, around 2.5 times higher than the ICP-05 predictions. The ICP-05 capacities were exceeded within 2 weeks of driving.

Rimoy & Jardine (2015) and Rimoy et al. (2015) collated ageing data from tests on 103 industrial (0.2 < OD < 1.3 m) piles conducted at various ages after driving in sand, as summarised in the Appendix. Most involved multiple re-tests on individual piles after relatively short pauses, so promoting scatter and systematically slower capacity growth trends than are seen in first-time tests (Jardine et al., 2006). The compression capacities include base capacity components that may be of similar, or greater, magnitude to the shaft resistances and may grow as ever larger tip settlements accumulate through re-testing. These factors and a lack of cone penetration test (CPT) and other site information make the trends harder to interpret than the research outcomes presented in Fig. 1. However, the scatter diagrams presented in Figs 2(a) and 2(b) from datasets of compression tests in Rimoy et al. (2015) indicate similar overall set-up trends for...
Chow et al. (1998) proposed three mechanisms to explain the effects of age on shaft capacity:

(a) stress redistribution leading to higher stationary radial effective stresses σw, acting on the shafts
(b) gains under axial loading of the dilative shaft radial stress Δσrd, capacity component, as demonstrated in instrumented field tests by Lehane et al. (1993) and Chow (1997)
(c) physicochemical processes involving the soil and shaft.

Gavin et al. (2013) noted that stress redistribution after driving caused the radial effective stress on their instrumented piles to reduce with time. They concluded that the primary mechanism contributing to the ageing of their 340 mm OD steel piles was increased dilative response with time (mechanism (b)). A secondary effect involved sand particle bonding to the lower pile shaft, leading to changes in shaft roughness and migration of the shear failure surface into the sand mass (mechanism (c)). In contrast Chow et al. (1998) and Jardine et al. (2006) noted the importance of increases in radial effective stresses acting on the pile shaft (mechanism (a)) to gains in pile capacity.

Jardine & Standing (2012) reported further that low-level cyclic loading enhances capacity. White & Zhao (2006) investigated the impact of environmental or seasonal cycles on pile ageing. They reported that the set-up rates of model mild steel piles increased when water depth was cycled, although stainless steel piles showed no gains with time. Other work suggests that the installation process might also be significant. Lim & Lehane (2014) noted considerably less set-up and ‘friction fatigue’ with small-diameter jacked piles than with piles driven at the same site and argued that ageing involves a recovery process that leads to a stable final upper-bound outcome, as seen in the tests summarised in Fig. 1.

Rimoy et al. (2015) report intensive long-term calibration chamber testing on 36 mm OD, closed-ended, jacked and driven piles in medium-dense silica sand. Their experiments aimed to study the ageing process under closely controlled conditions and included comprehensive measurement of the normal stresses developed on pile shafts and in the sand mass. However, the model piles developed far less set-up than the industrial piles reported in Figs 1 and 2. Imposed cycles of environmental stress change also had little influence on pile capacity.

Tsuha et al. (2012) found that low-level cyclic axial loading improved the same model piles’ tension capacities, whereas high-level cycling or hard driving severely damaged capacity, as with industrial piles (Jardine & Standing, 2012). Rimoy et al. (2015) suggested that interrelated cyclic and ageing stress redistribution mechanisms exist that are affected by the bands of fractured and compacted sand that form around steel displacement piles when qC > 8 MPa. Yang et al. (2010) found bands of adhered sand ⊅ 5 to 20D50 thick around their 36 mm model piles whose width grew with relative tip depth, h, and amounted to 0·5 to 1·5 mm, averaging around 0·6D50. Rimoy et al. (2015) argue that the arching mechanism on the outer pile wall may be affected by the ratio of D/D50 and that the stress redistribution mechanism may not apply as effectively to small-diameter closed-ended piles.

Growth of either shaft roughness or sand stiffness with age could increase the dilatant Δσrd component of shaft capacity that is captured by the ICP-05 design approach and varies (as proposed by Boulon & Foray (1986)) with 1/D. Outward radial expansion of the shaft through corrosion reactions could also raise σw while cementing of the fractured sand zone by iron compounds could increase the constant volume interface friction angles, δcv, and could lead, ultimately, to...
soil–soil shear strength controlling the long-term shaft resistance.

This paper reports experiments with 51 micro-piles (48 < D < 60 mm) driven and tested statically at various ages in tension at the Larvik (Norway), Dunkirk (France) and Blessington (Ireland) sites where the earlier experiments summarised in Fig. 1 were conducted. Mild (MS), stainless (SS) and galvanised (GS) steel piles were driven above and below the water table under the broadly similar climatic conditions outlined in Table 1. Some piles were installed with smooth and un-corroded surfaces, while others were pre-corroded or pre-driven to modify their surfaces. The experiments investigated how pile material, roughness and diameter affected ageing at three well-characterised research sites with different sand profiles. The piles were left undisturbed during the ageing period. Daily to seasonal temperature changes and possibly pore water suction fluctuations above the water table at Dunkirk and at Blessington were the only environmental variations experienced over the ageing periods. The central questions investigated are listed below.

(a) How influential to ageing are any physicochemical processes associated with the pile material, the soil and groundwater?

(b) Do other pile-specific factors such as scale, shaft roughness, installation process or environmental site conditions affect the outcomes?

(c) Does any upper shaft capacity limit, such as ≈ 2·5 times the ICP-05 medium-term prediction apply, as indicated by Fig. 1, irrespective of any continuing active ageing processes?

DESCRIPTION OF TEST SITES

Karlsrud et al. (2014) established the Norfolk Geotechnical Institute’s (NGI’s) test site, in the Larvik municipality (Norway) around 110 km southwest of Oslo, in the Numedalslågen estuary, which has a small tidal range. The ground surface is 2·4 m above sea level; 2 m of made ground overlies loose-to-medium-dense fluvial silty sands and silt layers down to at least 22 m. The grain size distributions applying over the study depth range indicate 5 to 20% silt, as presented in Fig. 3. Radiocarbon dating indicates deposition 2600 to 1200 before present (bp), while X-ray diffraction (XRD) testing on a 7·9 m deep sample indicates 25% quartz, 37% feldspars and 38% plagioclase and no clay minerals (NGI, 2009).

Nine CPT tests within the 15 m by 30 m test area showed piezocone $q_c$-min and $q_c$-avg consistently around 1 and 2-1 MPa, respectively, over the 3-5 to 7 m depth range, while $q_c$-max showed greater variation (2-8 to 5·5 MPa); see Figs 4(a) and 5. Piezocone excess pore pressures measured at the $u_r$ position were generally positive and often around 30 kPa (and in some cases 120 kPa); see Fig. 4(d). Samples from the boreholes identified in Fig. 5 showed the profile becoming siltier at depth; the CPTu results over 20 m depth led Lehane et al. (2017) to exclude the larger Larvik piles from their database of tests in free-draining silica sands. Ring shear interface and triaxial tests run at Imperial College on borehole samples gave the mechanical parameters listed in Table 2, while the chemical testing results in Table 3 indicate acidic groundwater conditions.

The Dunkirk Port-Ouest site described by Jardine et al. (2006), near Gravelines, Northern France, consists of dense-to-very-dense marine sand under hydraulic fill derived from the marine sand. The PISA project (Byrne et al., 2015) provided a 25 by 15 m area in which 50 mm dia. micro-piles were driven to ≈ 2 m, 2-3 m above the water table, within the hydraulic fill; see Fig. 6. The test locations were not affected by sea tides. Grain size distributions are shown in Fig. 3; other soil mechanical parameters are summarised in Table 2 and chemical testing is reported in Table 3. The sub-angular particles comprise ≈ 85% silica plus calcium carbonate (CaCO₃) shell fragments and other minerals that leave the soil slightly alkaline (Chow, 1997). Ring shear interface and triaxial tests run at Imperial College gave the mechanical parameters listed in Table 2; higher $\phi'$ can be expected at $D_s = 100\%$ (see Kuwano (1999)). CPT and seismic CPT test investigations performed for the PISA project (Zdravković et al., 2019) and this ageing study provided the representative CPT profiles plotted in Fig. 4(b). CPT tests 01, 03 and 04, show $q_c$ increasing from zero to up to 40 MPa at 1·5 m depth before reducing significantly. The 19 Panda2® dynamic penetrometer tests conducted as shown in Fig. 6 are not reported in detail but indicated denser conditions in the central test area. Emerson et al. (2008) noted that a surface failure mechanism affects the tip resistances developed in shallow CPT tests until a critical depth $z_{crit}$ is exceeded. They propose an expression for $z_{crit}$ that involves CPT diameter, tip resistance and sand unit weight which indicates that near-surface effects probably

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**Table 1. Meteorological data published for the locations nearest the three sites**

| Site                  | Rainfall (annual): mm | Temperature Monthly averages | Humidity Monthly averages |
|-----------------------|-----------------------|-----------------------------|---------------------------|
|                       |                       | Min.: °C | Max.: °C | Min.: % | Max.: % |
| Blessington (Dublin, Ireland) | 734                  | 5       | 16   | 73   | 83   |
| Dunkirk (France)    | 710                   | 4       | 18   | 78   | 84   |
| Larvik (Norway)     | 763                   | 2       | 16   | 60   | 85   |

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**Fig. 3. Grain size distribution for Larvik, Dunkirk and Blessington**

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*Figures and tables are not included in this text snippet.*
reduced the $q_c$ values down to 1.5 m depth at Dunkirk. Shallower $z_{cr}^c$ depths are indicated for Larvik, which do not affect the CPT profiles applying over the Larvik test piles’ 3.8 to 6.2 m contact depth range.

The seismic CPT traces plotted on Fig. 4(f) indicate relatively high $G_{max}$ values. Piezocone soundings conducted adjacent to the micro-pile indicated $u_2$ pore pressures that were generally zero or negative (down to −15 kPa) over the depth range of interest, although positive pressures of up to 40 kPa were seen over intervals where fines contents were higher; see Fig. 4(e). Suctions of ≈ 10 kPa that may have varied seasonally were found in ‘hanging water’ laboratory measurements (see Dane & Hopmans (2002)) on block samples taken in winter adjacent to the piles. The suction profile interpreted for the PISA programme and shown on Fig. 4(f) was taken as representative in the analyses that follow.

The Blessington site is located 25 km southwest of Dublin city. As documented by Gavin & O’Kelly (2007), Gavin et al. (2009) and Doherty et al. (2012), glacial deposition coupled with recent quarrying has left the test location in an overconsolidated state. The water table is ≈ 13 m deep and ground conditions comprise very dense ($D_r \approx 100\%$) glacially deposited medium-to-fine sand composed of quartz and hard limestone (CaCO₃) grains which impart alkalinity to the sand. Fig. 3 shows the sand’s average grading curve; other
mechanical soil properties are summarised in Table 2 while Table 3 provides chemical test results. The pile layout is shown in Fig. 7. Eight CPT tests performed within 5 m of the micro-piles (Prendergast et al., 2015) showed average $q_c$ increasing from 10 MPa to 15 MPa at 3 m, see Fig. 4(c). The Blessington test piles were driven from 1 m deep starter holes and the Emerson et al. (2008) analysis indicates that the CPT profiles should be mainly free of any shallow-penetration effects. Given the 13 m water table depth, piezocone tests were not undertaken. Water contents are $\approx 11\%$ over the first 2 m, where 10 to 12 kPa suctions were measured in situ with Decagon T4 tensimeters with filter lengths between 0-30 m and 1-15 m. The suctions may vary seasonally, but there are no other significant environmental cyclic actions to consider. Fig. 4(g) displays the $G_{\max}$ profile interpreted from multi-channel analysis of surface waves (MASW) reported by Prendergast et al. (2015) and the interpreted in situ suctions.

**EXPERIMENTAL PROGRAMME**

**Piles and installation**

Details of the micro-piles tested in this study are summarised in Table 4, while Tables 5–7 report the details of plug formation at the end of pile installation as the final plug length ratios (PLRs), defined as the internal plug length divided by the pile embedment depth. Seven 50 mm OD mild steel (MS) and two 48-2 mm OD stainless steel (SS) open-ended tubular piles were driven at Larvik to 6-2 m tip depth, well below the water table. The piles were driven through cased holes pre-bored to 3-8 m depth (see Fig. 5), giving an embedment length of 2-4 m.

**Table 2. Summary of soil parameters and ground conditions for three test sites**

| Unit | Larvik* | Dunkirk | Blessington |
|------|---------|---------|-------------|
| Water table BGL (m) | 2-2 | 5-4 | 13 |
| Description | Loose to medium dense silty fine to medium sand with some silt layers | Dense to very dense sand | Dense, medium to fine sand |
| Origin | Fluvial | Marine hydraulic fill | Glacial |
| Unit weight ($\gamma_{\text{max}}$) | 18-9–19-6 kN/m$^3$ | 17-1$^\dagger$ | 20-0$^\dagger$ |
| Water content | 26-5 | 5-7 | 10$^\ddagger$ |
| Relative density ($D_r$) | $\approx 20$ | 100 | 100 |
| Saturation ($S_r$) | $100^\dagger$ | 25-40$^\dagger\dagger$ | 60$^\dagger\ddagger$ |
| $D_{50}$ (mm) | 0-37-0-8 | 0-4 | 0-6 |
| $D_{10}$ (mm) | 0-16-0-58 | 0-26 | 0-1-0-15$^\ddagger$ |
| Fines < 0-063 mm | 6-20% | 0 | 5-10 |
| Effective peak, $\phi'_p$ (triaxial) and test conditions | $36, \sigma'_1 = 200$ kPa$^g$ $c_0 = 0-80$ $D_r = 45\%$ (estimated) | $37, \sigma'_1 = 200$ kPa$^h$ $c_0 = 0-64$ $D_r = 75\%$ | $42, \sigma'_1 = 200$ kPa$^i$ $c_0 = 0-59$ $D_r = 100\%$ (estimated) |
| Constant volume, $\phi'_v$ (triaxial) | $35, \sigma'_1 = 200$ kPa$^g$ | $32, \sigma'_1 = 200$ kPa$^h$ | $35, \sigma'_1 = 200$ kPa$^i$ |
| Constant volume interface shear $\delta_{\text{iv}}$ | $27-8, \sigma'_s = 200$ kPa$^g$ | $27-5, \sigma'_s = 200$ kPa$^h$ | $29-4, \sigma'_s = 200$ kPa$^i$ |
| and test conditions | | | |
| Overconsolidation ratio (OCR) | 1 | 1 | 15 at 1 m$^3$ |
| Average $q_c$ (MPa) | 2 | 30 | 5 at 3 m$^3$ |
| Average $f_c$ (MPa) | 0-02 | 0-1–0-3 | 15$^\dagger$ |
| Small-strain stiffness value, $G_{\max}$ (kN/m$^2$) | — | 50–130$^\ddagger$ | 50–150$^\ddagger$ |

* NGI (2009).  
† Chow et al. (1998) and Chow (1997).  
‡ Gavin & O’Kelly (2007).  
§ Doherty et al. (2012).  
∥Prendergast et al. (2015).  
*2017–2018 triaxial and interface ring shear tests at Imperial College. Interface shear tests conducted against mild steel interfaces with $R_{\text{CLA}}$ 8 to 13 μm, following Jardine et al. (2005) procedures.  
** After Aghakouchak et al. (2015).  
‡‡ Zdravkovic et al. (2019).  
‡† Nominal as $S_r$ varies with time and depth.
Table 3. Chemical tests on sand samples from the three sites

|               | pH | Sulfate (SO₄): | Carbonate (CO₃): | Total inorganic carbon: | Conductivity: |
|---------------|----|----------------|------------------|------------------------|--------------|
|               |    | mg/kg TS       | % TS             | % TS                   | mS/m         |
| Dunkirk       | 8·6| 1000           | 9·5              | 1·9                    | 8·3          |
| Larvik        | 3·5| 11 900         | 0·1              | 0·02                   | 67·7         |
| Blessington   | 8·0| <1000          | 20·5             | 4·1                    | 8·6          |

Note: TS, total solids.

Fig. 6. Dunkirk site layout

Fig. 7. Blessington site layout

Fig. 8. A.1 shows the visual condition of the Larvik piles prior to installation. The outsides of the SS piles were air-abraded, and the MS piles were deliberately pre-corroded by 4 months’ exposure on site before driving (with the exception of pile 2, which was left exposed for some days). Check measurements on spare MS and SS piles that had been exposed for up to a year on site gave average Rcla roughness values of 8·5 and 9·8 μm, respectively, with a Mitutoyo Surftest-SJ-210-Series-178, that match the typical Rcla ≈ 10 μm of industrial piles (Jardine et al., 2005). The piles were installed by a 0·62 kN weight drop-hammer that could fall between 0·20 and 0·35 m and impart energies between 74 and 130 Joules/blow, assuming 60% efficiency. A removable temporary cage inserted in the cased holes ensured pile verticality during installation.

In total, 31 open-ended piles were driven at Dunkirk, as shown in Fig. 6, in the southeast corner of the PISA site, 20 m from the nearest PISA pile. Multiple roughness measurements with a portable Taylor Hobson Surtronic 25 indicated pre-installation Rcla = 2 ± 1 μm for the 21 fresh MS piles (S1 to S20 and S25) and four stainless steel piles (S21, S23, S27 and S29-Inox) that were driven from ground level in January 2016. An adjustable Sol Solution Grizzly® machine was employed whose maximum energy (475 Joules/blow) and 60 to 70% energy ratio is equivalent to that of a standard penetration test (SPT) hammer. Four of the 21 MS piles were extracted after 4 months and re-driven as ‘pre-corroded’ rougher piles (R1, R2, R3 and R4). Their rougher shafts and sand adhering to their interiors led to lower PLRs than with fresh piles. Two of the four SS piles were extracted and air-abraded on their outsides before re-driving in July 2016 as R15 and R16. While Rcla values were measured before testing for most piles, this was not feasible for MS piles R1 to R4, as their Rcla values were higher than the Surtronic 25 could measure, as well as for SS piles R15 and R16. Visual inspection and tactile checks indicated Rcla values comparable to those of lightly rusted industrial piles (≈ 10 μm) for MS piles R1 to R4, while the re-driven shot-blasted SS piles were gauged to have been little affected by the air-abrasion applied and to have retained Rcla ≈ 2 μm.

Driving took place in January 2016 at Blessington, with the four pairs of 60 mm OD, 4 mm thick MS and galvanised (GS) tube-piles shown in Fig. 7. The MS piles were mildly pre-corroded. The GS piles’ manufacture involved molten zinc dipping, which left a moderately rough surface; Rcla values were estimated as ≈ 10 μm for both pile types. The top metre of sand was augured to avoid contact with superficial material. Handheld metal post drivers were employed until all piles effectively refused after penetrations of 0·5 to 0·7 m. Pile MS1 was damaged and had to be abandoned; a greatly oversize 4 t hammer was employed the next day to achieve the final 2·75 m tip depths. Although the piles were not exhumed, earlier studies by Gavin et al. (2013) at Blessington indicate that crusts of crushed sand are likely to have formed around the shafts on driving that may have undergone chemical modification during ageing. The mild steel piles are likely to have corroded in situ.
Table 4. Properties of piles

|                | Larvik                                      | Dunkirk                                             | Blessington                                          |
|----------------|---------------------------------------------|-----------------------------------------------------|-----------------------------------------------------|
| $R_{ch}$ fresh MS ‡ | $\mu$m —                                 | 1–3§                                                | —                                                   |
| $R_{ch}$ pre-corroded MS | $\mu$m NA§ 9.2§ 8.5§ 7.5∥ | $\approx 10$ after extraction* †∥ | $\approx 10$* §                                    |
| $R_{ch}$ SS/GS   | $\mu$m —                                 | 1–3§                                                | $\approx 1–3$ after extraction* ‖                  |
| $R_{ch}$ air-abraded SS | $\mu$m NA§ 10.3∥ 9.8∥ | $\approx 1–3$ after extraction* ‖                  | —                                                   |
| Grade MS        | — EN 10305-3 (BSI, 2016), E220 + CR2 – S2 | E470 NA                                             | NA                                                  |
| Grade SS/GS     | — AISI 304, 316L                          | 51-MS NA                                            | 60                                                  |
| Outside diameter | mm 50-MS                                  | 48-2-SS                                              | 50-6-SS                                             |
| Wall thickness (t) | mm 2                                      | 8-MS                                                 | 4                                                   |
| D/t ratio       | — 24–25                                   | 6.3–6.7                                              | 15                                                  |
| Contact length average | m 2.4                                    | 1.97                                                 | 1.75                                                 |

Note: MS, mild steel; SS, stainless steel; GS, galvanised steel; NA, not available.

*Estimated.
†Sand adhered to the pile.
‡The 'fresh' interfaces were as delivered to site without any deliberate pre-corrosion or further air abrasion.
∥Measured centre-line average roughness ($R_{ch}$) prior to installation.
††Measured centre-line average roughness ($R_{ch}$) as found on extraction post tension test, including any in situ corrosion.
∥∥Measured centre-line average roughness ($R_{ch}$) as found on spare not installed piles left exposed over 1 year on site.

Table 5. Peak pile capacities for Larvik

| Pile ID | Time: days | Capacity: kN | Pile     | Driving energy | Plug length ratio (PLR) |
|---------|------------|--------------|----------|----------------|-------------------------|
| P05     | 1          | 2.60         | MS*      | Variable       | 0.29                    |
| P04     | 14         | 3.71         | MS*      | Variable       | 0.52                    |
| P12     | 14         | 4.26         | MS*      | Variable       | 0.32                    |
| P02     | 78         | 3.94         | MS*      | Variable       | 0.42                    |
| P07     | 189        | 4.74         | MS*      | Variable       | 0.43                    |
| P03     | 313        | 5.45         | MS*      | Variable       | 0.40                    |
| P06     | 315        | 5.96         | MS*      | Variable       | 0.42                    |
| P08     | 696        | 5.45         | MS*      | Variable       | 0.54                    |
| P09     | 21         | 2.66         | SS†       | Variable       | 0.60                    |
| P10     | 314        | 2.43         | SS†       | Variable       | 0.54                    |

*Pre-corroded.
†Air-abraded. Failure defined at peak.

Testing arrangements

Equipment and procedures varied between sites. The Larvik piles’ first-time static tension capacities were measured 1, 14, 21, 78, 189, 315 and 696 days after driving with the system shown in Fig. 9. An extension rod connected the pile head, through a load cell, to a suspended hydraulic cylinder actuated by a GDS ADVPC advanced pressure/volume controller. Jack pressures were raised gradually and held constant for 20 min over load stages. A SignalExpress logging system recorded time, load and vertical pile movements relative to the pre-installed casings, which were assumed unaffected by loading. Tests ended when clear load–displacement plateaux developed, and displacements exceeded 7 mm (14% of the piles’ OD).

The Dunkirk piles were subject to static tension tests after 0–1, 1, 14, 28, 90, 175, 272 and 315 days’ ageing. The reaction frame comprised four steel beams that transferred load to timber foundations; see Figs 10(a) and 10(b). An Enerpac manual hydraulic jack acted through a load cell. Load increments were set initially at 10% of the estimated medium-term tension capacity (that reduced as failure approached) and maintained for 15 min. Tests continued until displacements reached 15 to 30 mm (30 to 60% of pile OD) as measured by two linear variable differential transducers (LVDTs) supported on vertical stands set 0.7 m on either side of the pile axis (Fig. 10(a)). A third LVDT measured the loading system deformation and the corrected mean LVDT displacement was applied in test analysis. Strong winds affected displacement measurements in some tests.

The Blessington static load tension tests were carried out 2, 21 and 78 days after driving. The 121 day MS1 pile was abandoned and equipment malfunctioned in the 121 day GS pile test. Testing arrangements were similar to those at Dunkirk, see Fig. 11. Displacement measurements relied on a reference beam and three magnetically clamped LVDTs. An Enerpac manual hydraulic jack applied $\approx 5$ kN load increments, while the LVDTs and the load cell outputs were recorded by a Campbell Scientific data logger at 0.1 s intervals and transferred to a computer. Load steps were held constant for 30 s and tests terminated when it was no longer possible to maintain loads by jacking. The final displacements exceeded 20 mm (33% of pile OD). Most tests ended within 10 min, while those at Larvik and Dunkirk extended several times longer.

FIELD BEHAVIOUR

Installation

The Larvik piles’ driving drop-weight heights increased from 0.2 m to 0.35 m as penetration advanced, giving the blow-count envelopes and means in Fig. 12(a). The final PLRs ranged from 0.29 to 0.60, with SS piles showing slightly higher ratios than the pre-corroded MS piles; see Table 5.

At Dunkirk the Sol Solution ‘Grizzly’ machine drove piles S1 to S9 at its maximum (SPT) energy level. Lower ratings (from 21% up to 100%) were applied to other piles to achieve consistent penetration rates and Fig. 12(b) presents the
The Blessington piles required 70% energy rating. Failure defined at peak.

### Table 7. Peak pile capacities for Blessington

| Pile ID | Time: days | Capacity: kN | Pile | Driving energy | Plug length ratio (PLR) |
|---------|------------|--------------|------|----------------|------------------------|
| S1      | 0-1        | 28-6         | MS*  | Variable       | 0-35                   |
| S2      | 2          | 29-7         | MS*  | SPT            | 0-52                   |
| S3      | 16         | 62-6         | MS*  | SPT            | 0-43                   |
| S4      | 16         | 59-3         | MS*  | SPT            | 0-36                   |
| S5      | 30         | 56-7         | MS*  | SPT            | 0-38                   |
| S6      | 92         | 67-4         | MS*  | SPT            | 0-38                   |
| S7+     | 93         | 64-2         | MS*  | SPT            | 0-41                   |
| S8      | 175        | 70-6         | MS*  | SPT            | 0-43                   |
| S9      | 274        | 71-1         | MS*  | SPT            | 0-38                   |
| S10B    | 315        | 71-7         | MS*  | Variable       | 0-46                   |
| S11     | 315        | 67-6         | MS*  | Variable       | 0-50                   |
| S12     | 273        | 79-6         | MS*  | Variable       | 0-49                   |
| S13+    | 273        | 76-7         | MS*  | Variable       | 0-40                   |
| S14+    | 176        | 73-9         | MS*  | Variable       | 0-48                   |
| S15     | 174        | 78-2         | MS*  | Variable       | 0-49                   |
| S16     | 90         | 71-6         | MS*  | Variable       | 0-40                   |
| S17     | 28         | 66-5         | MS*  | Variable       | 0-42                   |
| S18     | 14         | 67-8         | MS*  | Variable       | 0-43                   |
| S19     | 1          | 32-8         | MS*  | Variable       | 0-38                   |
| S20     | 0-1        | 31-2         | MS*  | Variable       | 0-36                   |
| R1      | 1          | 50-2         | MS§  | Variable       | 0-22                   |
| R2      | 85         | 89-0         | MS§  | Variable       | 0-21                   |
| R3      | 183        | 66-3         | MS§  | Variable       | 0-24                   |
| R4      | 223        | 76-3         | MS§  | Variable       | 0-23                   |
| S21-Inox| 0-1        | 25-9         | SS   | Variable       | 0-38                   |
| S23-Inox| 14        | 25-8         | SS   | Variable       | 0-44                   |
| S27-Inox| 28        | 24-6         | SS   | Variable       | 0-43                   |
| S29-Inox| 91        | 27-6         | SS   | Variable       | 0-44                   |
| R5      | 1          | 32-3         | SS§  | Variable       | 0-38                   |
| R6      | 99         | 25-6         | SS§  | Variable       | 0-38                   |

*Fresh.
†Pre-corroded.
‡Air-abraded.
§Pile was used previously in this study as fresh MS or SS. SPT: as standard penetration test, delivering up to 475 Joules/blow with 60–70% energy rating. Failure defined at peak.

### Table 7. Pile capacities for Blessington

| Pile ID | Time: days | Capacity: kN | Pile | Driving energy | Plug length ratio (PLR) |
|---------|------------|--------------|------|----------------|------------------------|
| MS4     | 2          | 54-3         | MS*  | Variable       | 0-35                   |
| MS3     | 21         | 55-8         | MS*  | Variable       | 0-39                   |
| MS2     | 78         | 50-3         | MS*  | Variable       | 0-34                   |
| MS1     | 121        | —            | MS*  | Variable       | 0-41                   |
| GS4     | 2          | 52-8         | GS   | Variable       | 0-21                   |
| GS3     | 21         | 50-3         | GS   | Variable       | 0-22                   |
| GS2     | 78         | 56-8         | GS   | Variable       | 0-23                   |
| GS1B    | 121        | —            | GS   | Variable       | 0-19                   |

*Pre-corroded. Failure defined at peak.

The Blessington piles were shot-blasted SS piles, which was confirmed to result in a 70% energy rating. The pre-corroded MS piles had smoother surfaces than the other piles, and the pre-corroded MS Blessington piles matched better the Larvik, Blessington and typical industrial pile conditions. Capacities are based on maximum recorded loads corrected for pile and plug weights; no reverse end bearing is considered.

The pre-corroded MS piles driven at Larvik showed, after 315 to 696 days, upper limit capacities ≈2.2 times the 1 day reference value. Their set-up factor fell below the equivalent ratio (≈2-9) found from the 508 mm OD piles after 200 days; see Fig. 1. In contrast to the MS piles, the SS Larvik piles’ capacities remained at the pre-corroded MS pile’s day 1 capacity. The apparent test scatter reflects the tendencies of the ‘east-end’ piles 12 and 6 to plot above the ‘west-end’ piles 3, 4 and 5 because the mean CPT qe values tend to increase by ≈15% west to east across the test area; the relatively low capacity of pile 2 reflects its lower period of on-site exposure, pre-corrosion and roughening before driving.

The Dunkirk tests investigated capacity variations between the fresh MS, pre-corroded MS, initially smooth SS and air-abraded SS piles. As shown in Fig. 14(b), the fresh MS and the SS piles did not set-up significantly after the first day of driving. However, the capacity of a pre-corroded MS pile tested at 1 day (R1) was 1.5 to 1.8 times higher than that of a relatively smooth fresh MS pile at similar age (S1, S2, S19, S20). Increasing the shaft roughness boosts initial static capacity (and reduces driving PLR) significantly; see Tables 4 and 6. The pre-corroded MS and fresh MS Dunkirk piles all developed long-term capacity growth. Despite scatter, the trend lines show steeper (semi-logarithmic) medium-term gains for the fresh MS piles.
It is interesting that the pre-corroded and fresh MS piles both tend towards maxima around 1.4 to 1.6 times the 1 day reference (pre-corroded) capacity. However, the latter ratios are lower than the $C_{25}^{2}$ ratio seen with the Larvik micro-piles and far below the 2.9 to 3.7 gains seen with larger piles at Larvik and Dunkirk, respectively: see Fig. 1. As at Larvik, the Dunkirk SS piles showed no set-up. The SS piles delivered similar capacities after undergoing air-abrasion, confirming that this treatment had not modified their relatively hard surfaces as significantly as had been intended.

The equivalent trends for Blessington are presented in Fig. 14(c). It is noteworthy that the MS or GS piles driven with the 4 t hammer developed practically the same capacities and, unlike the larger piles illustrated in Fig. 1, no set-up.

**Average shaft shear resistances**

It is instructive to consider the piles’ average failure shaft shear resistance ($r_{avg}$) variations with time in Figs 15(a)–15(c). Also plotted in Fig. 15 are the $r_{avg}$ average shaft shear resistances from the larger piles reported in Fig. 1 and the average sleeve friction resistance ($f_s$) over the embedment interval from the closest CPTs, which vary from $\approx 20$ kPa for Larvik to $\approx 150$ kPa at Dunkirk and Blessington. Despite the piles’ open ends and the geometrical ($h/R^*$) or ‘friction fatigue’ factors identified from
instrumented field tests in sands, the pre-corroded rough micro-piles’ $r_{\text{avg}}$ resistances exceed or match $f_s$ from day 1 at Dunkirk and Blessington, but only climb towards 0.75 $f_s$ after a year at Larvik.

The Larvik MS micro-piles developed markedly lower average resistances at all ages than equivalent 508 mm diameter piles, which may reflect partially the higher CPT $q_c$ values applying over the larger piles’ shafts. The opposite applied at Dunkirk where the MS micro-piles developed higher resistances at all ages, despite their shallow depths.

It is interesting that the Blessington micro-piles achieved, in a less variable $q_c$ profile, short-term shaft capacities that matched the long-term upper limit achieved by the large piles and also the average $f_s$ values developed in the monotonically jacked CPT tests. The shaft ageing outcomes found with these relatively long ($L/D = 29.2$) micro-piles are compatible
with the hypothesis posed in the introduction that upper limits may apply that cap the shaft resistance, and therefore the radial effective stresses that can develop, despite further corrosion, shaft roughening or any otherwise potentially beneficial ongoing ageing process. The only special feature of the piles that may have led to their limiting capacities being reached very early after driving was the much higher driving energy, and therefore large penetration for a given blow, imparted by the 4 t hammer.

Capacity normalisation

Further insights can be gained by normalising the measured capacities with capacity predictions from design procedures that account for site and pile characteristics. While the Larvik piles were submerged, even small suctions could be important to the Dunkirk and Blessington micro-piles and their $\sigma_{\text{fs}}$ profiles have been assessed by treating the profiles as if saturated and adding the measured suctions to the total vertical stresses.

The API (2011) main text method assumes that local $\tau_f = \beta \sigma_{\text{fs}}$, where $\sigma_{\text{fs}}$ is the free-field effective stress. An upper-bound $\tau_f$ applies to API (2011) that depends with $\beta$ on in situ relative density and grain size. API (2011) is not applicable to loose sands, so the previous API (1993) version has been applied to the Larvik pile cases. The NGI-05 method uses $q_c$ as the main soil input parameter to determine local relative density, which together with $\sigma_{\text{fs}}$ controls shaft resistance. The NGI-05 and API main text methods do not link $\tau_f$ to pile diameter, $G$ or shaft roughness.

The alternative ICP-05 method was developed from field experiments with 102 mm dia. instrumented jacked piles, installed to depths of up to 6 m in loose dune sand (Løhne et al., 1993) and dense marine sand (Chow, 1997). Local stress sensor measurements of shear and radial stresses plus pore pressures, combined with experiments on larger instrumented open pipe-piles showed that the shaft failures achieved in tension tests conducted within days of installation could be matched by three equations

$$\tau_f = \sigma_{\text{fs}} \tan(\delta_c)$$  \hspace{1cm} (1)

with

$$\sigma_{\text{fs}} \approx (0.8\sigma'_c + \Delta\sigma_{\text{fd}})$$  \hspace{1cm} (2)

and

$$\sigma_c \approx 0.029q_c^{0.13}(P_{\max})^{0.13}(h/R)^{-0.38}$$  \hspace{1cm} (3)

Where $\tau_f$ is the local shear stress at failure; $\delta_c$ is the ultimate interface shearing angle; $h$ is the depth of the pile tip below the point in question; and $R^* = (R_c^2 - R_a^2)^{0.5}$. Interface ring-shear tests show that $\delta_c$ reduces with sand $D_{50}$ and increases with interface average centre-line roughness $R_{\text{cla}}$; Ho et al. (2011). Similar tests on samples from all three sites with appropriate interfaces and stress levels gave the angles indicated in Table 2. The radial effective stress acting on the pile shaft at failure, $\sigma_{\text{fr}}$ is related to

(a) the equalised shaft radial effective stress, which depends weakly on $\sigma_{\text{fs}}$ and reduces with normalised pile tip depth, $h/R$.

(b) the change in shaft stress $\Delta\sigma_{\text{fd}}$, due to constrained outward radial movements related to dilation at the interface. The reviews of Løhne (1992) and Chow (1997) of available field and laboratory model test data for sands indicated that equation (4) provides suitable estimates for $\Delta\sigma_{\text{fd}}$ when the dilative radial displacement is taken as equal to the average peak-to-trough roughness, $2R_{\text{cla}}$.

$$\Delta\sigma_{\text{fd}} = 4GR_{\text{cla}}/D$$  \hspace{1cm} (4)

While $\Delta\sigma_{\text{fd}}$ values are hard to evaluate precisely, equation (4) indicates that dilation offers relatively modest contributions (often less than 5%) to the medium-term capacities of large-diameter industrial piles. However, Axelsson (2000) and Gavin et al. (2013, 2015) argue that the ‘dilative’ term contributes far more significantly to aged industrial piles. It also has the potential to dominate the frictional resistance of micro-piles. While Jardine et al. (2005) suggest a default $R_{\text{cla}} \approx 10$ μm estimate for industrial steel piles, the case-specific values assessed for the present study are summarised in Table 4. Radial movements greater than double the initial $R_{\text{cla}}$ could be generated on loading after any corrosion in situ and/or bonding with sand particles. It is also possible that movements lower than $2R_{\text{cla}}$ could apply in any very loose, silty/clayey sands that contract when sheared.

The operational (in situ) sand shear stiffness $G$ is also difficult to select, especially for micro-piles driven above the water table. The expressions of Jardine et al. (2005) for $G_{\text{max}}$ as a function of $q_c$ were applied at Larvik, while the seismic
CPT and MASW measurements made at Dunkirk and Blessington provided direct information on the undisturbed in situ $G_{\text{max}}$ profile. The operational values would be raised by the effective stress changes generated by pile installation (particularly at Dunkirk and Blessington), and would be affected by anisotropy or reduced if the response to shaft loading to failure is non-linear; Jardine et al. (2013). The radial cavity strain ($\delta_{\text{cr}}$) required at the shaft for the sand to unlock from the shaft and allow failure to occur is $4R_{\text{cr}}/D$, which amounts to $\approx 0.08\%$ for the rougher micro-piles and exceeds the linear range of most sands. Assuming similar roughness values, the corresponding strains would be up to 10 times lower for the larger piles tested earlier at the same sites.

Fig. 13. (a) Tension load plotted against displacement for selected tests with 10% pile diameter marker at Larvik. (b) Tension load plotted against displacement for selected tests with 10% pile diameter marker at Dunkirk. (c) Tension load plotted against displacement for selected tests with 10% pile diameter marker at Blessington.
The ICP-05 approach allows estimates to be made for how driven pile installation and interface dilation affect shaft radial effective stresses. However, the micro-piles are smaller, and involve lower initial effective stresses, than all previous field evaluations of the method and the predictions are inevitably subject to uncertainty.

Tables 2 and 8 list the input parameters considered appropriate for the micro-piles at various test stages and the tension capacities ($Q_{s,t}$) calculated for all three sites. Table 9 lists the corresponding average capacities as measured ($Q_{sm}$) after 1, 85–100 and 175–315 days and gives $Q_{sm}/Q_{sc}$ ratios for the various cases. Figs 16(a)–16(c) plot the $Q_{sm}/Q_{sc}$ ratio for each pile tested at the three sites, including the IAC curve from Fig. 1.

The loose Larvik sand site is considered first, where 5 to 20% silt contents led to positive excess piezocene pore pressures at some depths (see Fig. 4). The early age micro-pile capacities are over-predicted by API (1993) and marginally under-predicted by NGI-05. As noted earlier, the Larvik MS micro-piles showed marked set-up over the months after driving, but to a lesser degree than the large piles shown in Fig. 1. ICP-05 predictions made with the default $G$-$q_c$ function greatly over-predict the micro-piles’ initial resistances, giving $Q_{sm}/Q_{sc} = 0.23$, in contrast with the method’s more representative prediction for the 508 mm Larvik piles short- to medium-term capacities; see Figs 16(a) and 1. The $\Delta q_{rd}$ component (see equation (4)) provides 85% of the micro-pile capacity predicted with ICP-05 (see Table 8) and less than 10% for the 508 mm dia. cases, so the over-prediction for Larvik must relate primarily to the dilatant term. Although earlier field tests demonstrated highly significant constrained dilation in clean loose sands (Lehane et al., 1993), a 91% reduction is required in the $\Delta q_{rd}$ component given by equation (4) to match the micro-pile capacity seen in the 1 to 2 day tests performed in the loose, silty and contractive Larvik sands.

Moving to the Dunkirk dense sand site, it is recalled first that applying the criteria proposed by Emerson et al. (2008) indicates that the Dunkirk CPT traces may have been subject to near-surface effects down to depths of 1.5 m; the latter may also have affected the pile test outcomes. However, the micro-piles’ capacities were 9.9 times greater at day 1 than expected by API (2011) and 1.6 to 1.8 times greater than estimated by ICP-05 or NGI-05. The SS micro-piles also showed higher capacities than expected, although their
$Q_{sw}/Q_{sc}$ ratios remained fixed at $\approx 1.5$ over time. ICP-05 calculations run for closed-ended conditions increased shaft capacity by just 6%, indicating that the micro-piles’ relatively low PLRs were not the main cause of the discrepancies. Interface constrained dilation and the diameter-dependent $\Delta\sigma_{\text{rd}}$ term (equation (4)) is expected to contribute around half of the shaft capacity and higher than undisturbed in situ operational shear stiffness or shaft roughness again appear to be more likely contributors to the ICP-05’s initial under-prediction. The Dunkirk MS micro-piles’ capacities grew with age although, as at Larvik, their final relative set-up factors were lower than those earlier with larger (457 mm OD) piles shown in Fig. 1. The relatively large relative displacements (see Fig. 13) required to reach micro-pile shaft

Table 8. Parameters and pile tension capacity predictions based on average of all CPTs

| Site            | $\delta_{cv}$: deg | $\beta$ (API) | $D_{50}$: mm | $R_{	ext{dil}}$: $\mu$m | ICP$^\dagger$: kN | API: kN | NGI: kN |
|-----------------|---------------------|----------------|--------------|--------------------------|-------------------|---------|---------|
| Larvik          | 27.8                | 0.21           | 0.25         | 8.5                      | 11.2 (85%)        | 4.4§    | 2.2     |
| Dunkirk*        | 27.5                | 0.56           | 0.26         | 10.0                     | 30.1 (47%)        | 5.1     | 27.9    |
| Dunkirk†        | 27.5                | 0.56           | 0.26         | 2.0                      | 18.9 (15%)        | 5.1     | 27.9    |
| Blessington     | 29.4                | 0.46           | 0.15         | 10.0                     | 27.0 (52%)        | 7.2     | 27.5    |

*Pre-corroded MS piles and fresh MS > 10 days.
†Fresh MS < 10 day, air-abraded SS and SS.
‡ICP: values in parentheses correspond to ratio of the capacity component term $\Delta\sigma_{\text{rd}} \times \tan (\delta_{cv})$ to the total calculated capacity.
§API: The Larvik analysis applies API (1993) as API (2011) does not apply to loose sands.
failure are consistent with constrained dilation (equation (4)) playing a progressively more significant role over time at Dunkirk. The ultimate capacities of the MS piles lead to ICP-05 \( Q_{sm}/Q_{sc} \) ratios that scatter around 2.1 to 2.6, similar to the larger piles’ trend, as shown in Fig. 16(b). The roughness of the fresh MS piles is assumed to change with time and \( R_{cla} \) is assumed = 10 \( \mu \)m after 14 days in-place of the initial 2 \( \mu \)m. 

The Blessington micro-piles’ capacities amounted to 7.7 times the conventional API (2011) estimate and double the NGI-05 estimate. They are \( \approx 2.1 \) those expected from ICP-05 and remain unchanged with age, falling 15% below the long-term ICP \( Q_{sm}/Q_{sc} \) ratio of 2.5 given by the larger piles in Fig. 16(c). If the long-term capacity is subject to an upper limit, as postulated by Jardine et al. (2006), Lim & Lehane (2014), Rimoy et al. (2015) and Gavin et al. (2015), this limit appears to have been reached shortly after driving and to have remained unaffected by any subsequent physicochemical or other ageing process. The reasons for this outcome remain open to speculation. However, as noted earlier, the energy used to drive these piles by the 4 t hammer advanced the Blessington micro-piles rapidly by a full diameter per blow, in a manner similar to pile jacking. Gavin & O’Kelly (2007) showed that pile installation resistance at this site is strongly affected by the installation method, with piles installed with long jacking strokes developing shaft capacities close to the \( f_s \) values measured in the CPT test. The large axial displacements (up to 30% of OD) required to reach tension failure are compatible with the sand’s high \( \delta_c \) values (see Table 2) and the suggestion that the interface dilation components (equation (4)) of shaft resistance were greater than expected from the ICP-05 calculations.

### Table 9. Tension capacity analysis for MS piles: measured-to-calculated ratios for three methods

| Days | Capacity measured* kN | NGI-05 | API | ICP-05 |
|------|------------------------|--------|-----|--------|
|      | Larvik                 | Dunk   | Bless | Larvik | Dunk   | Bless |
| Pre-corroded |                  | 2.6    | 50  | 0.23  | 1.67  | 1.75 |
| Fresh   |                      | 2.1    | 33  | 0.38  | 0.95  | 1.75 |
| Pre-corroded |                  | 4.2    | 89  | 0.56  | 2.96  | 2.87 |
| Fresh   |                      | 5.4    | 86  | 0.48  | 2.36  | 2.35 |
| Pre-corroded |                  | 6.4    | 71  | 0.64  | 1.23  | 1.23 |
| Fresh   |                      | 6.4    | 74  | 0.64  | 1.23  | 1.23 |
| Pre-corroded |                  | 9.4    | 71  | 0.64  | 1.23  | 1.23 |
| Fresh   |                      | 9.4    | 74  | 0.64  | 1.23  | 1.23 |

*Cumulative values are presented for the range of days considered for each set of pile types and site.
crushing does not appear to have been a necessary condition for the physicochemical ageing processes to apply to the micro-piles or earlier larger (508 mm) diameter piles. However, the early-age tests on pre-corroded micro-piles proved that surface roughening provides a significant part of the capacity growth, principally through enhanced dilation. The pre-corroded MS piles consequently showed less capacity growth over time at Dunkirk than initially smooth MS piles, as both tended to similar upper limits.

Corrosion reactions could also cause additional growth in static radial stresses due to expansion of the pile volume as corrosion products crystallise at the shaft. Modification of the shaft, as shown in Fig. 8, and iron compound cementing also provide a marginally higher effective diameter, probably augmented dilation and $\delta_{cv}$ angles that may approach the peak or critical state soil–soil $\phi'_{cs}$ values. A shift to $\phi'_{cs}$ would offer a $\tan\phi'_{cs}/\tan\delta_{cv}$ capacity contribution which might provide an additional $\approx 25$–35% in silica sands for piles of any diameter.

**Enhanced dilation under loading**

Instrumented field tests show that constrained dilation under loading contributes to shaft capacity in sands. Axelsson (2000) and Gavin et al. (2013) observed that it contributed to capacity growth over time with industrial concrete and steel driven piles. The shapes of the Dunkirk MS micro-piles’ load–displacement curves also indicate that interface dilation became more important with time. However, no set-up or displacement to failure growth applied to the SS and GS micro-piles, so physicochemical processes involving mild steel (or concrete) are required to generate any additional radial movements that might contribute to raising the shaft stresses in combination with any shift from the interface shear angle rising from $\delta_{cv}$ towards $\phi'_{cs}$.

**Evidence obtained by comparing piles with different diameters**

Set-up appears to be diameter dependent: the micro-piles all developed lower relative gains than the larger piles tested earlier at the same site. The corrosion reactions observed around the pile shafts can be expected to advance at rates that are independent of the piles’ diameters. Micro-piles and larger industrial piles can therefore be expected to show similar absolute gains in effective diameter and absolute additional radial movements due to dilation when loaded to failure. The relative impact of these changes on shaft capacity should all diminish with increasing pile diameter, as indicated by equation (4), rather than follow the opposite trend seen in the field experiments. Recalling from Fig. 2(a) that industrial concrete piles also set-up and that capacity gains have been observed over relatively short times with offshore piles driven in sands to penetrations where the
oxygen supply is severely limited (Jardine et al., 2015), it appears that steel corrosion cannot be the only process at work. Other factors must be operating whose impact is greater with larger diameter piles.

Radial stress redistribution

It remains highly challenging to undertake analyses of installation in sand that explore the generated in situ stress fields quantitatively. However, Yang et al. (2014) and Zhang et al. (2014) show that an arching stress field can be expected to develop in sands around even monotonically penetrating piles. The SS and GS micro-pile experiments confirm the postulate of Rimoy et al. (2015) that radial stress redistribution does not contribute significantly to micro-pile ageing. However, the redistribution mechanism may be more effective around larger open-ended driven piles which involve higher: (a) initial stresses, (b) ratios of diameter $D$ to crushed-sand bandwidth, (c) $D$ to wall thickness, $t$, ratios, (d) final PLRs and (e) total blow-counts. The grain crushing, dynamic load cycling and shear band formation that take place when industrial piles with high $D/t$ ratios are driven may all accentuate arching around the shaft and provide greater scope for stress redistribution and capacity growth over time and contribute to the set-up observed shortly after driving (insignificant water depths) for large offshore piles in dense sands.

Pile driving technique

It has been argued that the lack of set-up, even by physicochemical processes, of the MS Blessington piles was related to employing an oversized hammer over their main drive lengths, which led to installation conditions similar to pile-jacking, with penetrations of one full diameter per blow, and contrasted strongly with the conventional driving of the 340 mm OD piles, which manifested the strong set-up seen in Fig. 1. Other tests at Blessington by Chatta (2006) on 73 mm OD closed-ended piles installed with both 1 m and 50 mm jack strokes and tests with 75, 100 and 114 mm OD open-ended piles driven (with high blow-counts) by an SPT hammer (Gavin et al., 2003) confirm that the installation procedure affects both PLRs and short-term capacity. Hard driving reduces the radial effective stresses around the shaft and leaves greater scope for capacity growth than the micro-piles' high-energy installation, which delivered unusually high short-term capacities that fell close to the upper limits suggested by Figs 1 and 16.

Possible upper limit to aged shaft capacities

The tests at all three sites support the hypothesis that an upper limit applies to aged shaft capacity which amounts to 2:1 to 2:5 times the ICP-05 predictions, irrespective of any ongoing ageing process.

CONCLUSIONS

Fifty-one co-ordinated static first-time tension tests have been reported, along with site investigations, on open-ended steel micro-piles failed at ages between 0-1 and 696 days, at sand sites covering loose to dense conditions and varying silt contents. Fresh, smooth and pre-corroded, rough, mild steel, stainless and galvanised steel piles were considered. Integration with earlier larger diameter pile tests and an independent database study allowed investigation of how pile diameter, steel corrodibility, surface roughness and ground conditions affect ageing behaviour. The influence of driving procedure was also considered. Reflecting the key questions posed in the introduction, three groups of conclusions are drawn.

Group 1: pile material, roughness and physicochemical processes were found to be highly influential to ageing. (a) Physicochemical processes dominated the marked set-up shown by mild steel micro-piles in submerged silty, loose (Larvik) sand and clean, dense unsaturated (Dunkirk) sand. (b) No set-up was seen with non-corrodible stainless or galvanised steel piles, so no independent ageing process, such as radial stress re-distribution, growth of shear stiffness, dilation due to creep or enhanced grain interlocking affected the micro-piles significantly in the absence of physicochemical effects. (c) The physicochemical processes identified as potentially affecting ageing include: (i) pile surface roughness increasing through redox reactions; (ii) bonding between sand and corroding shaft; (iii) marginally higher effective diameters; (iv) interface shear angles rising towards the $\phi_s$ limit; and (v) growth in static radial stress $\sigma_r$ due to radial expansion of the corroding steel.

(d) However, the database studies outlined in Fig. 2 and the Appendix show that concrete and steel piles with diameters between 0·2 and 1·3 m gain capacity at similar rates over time. Steel corrosion reactions cannot be the only mechanism that leads to set-up with larger piles driven in sands.

(e) Pile roughness and constrained dilation had a critical effect on the micro-piles’ capacities. Group 2: four further key points emerged concerning the specific piles and sites investigated. (a) The mild steel micro-piles developed lower set-up factors than the larger industrial piles tested previously at the Larvik, Dunkirk and Blessington test sites. (b) The lack of set-up shown by stainless steel piles proved that seasonal variations in temperature and/or possibly pore pressures had no independent influence on pile ageing, which also appeared largely independent of the sites’ initial geochemical conditions.

(c) The micro-pile shaft capacity measurements indicate that the ICP-05 dilation term expressed in equations (2) and (4) gives only an approximate indication of the observed field behaviour. While the impact of dilation is minor for large piles at early ages after driving, the discrepancies are important with micro-piles. The ICP-05 dilation term should be set to zero when assessing short- to medium-term shaft capacities in loose, silty sands that manifest positive piezocene pore pressures. Equally, applying equation (4) as recommended in ICP-05 appears to underestimate significantly the contribution that constrained dilation makes to micro-pile shaft capacity in dense sands.

(d) Short-term capacity and set-up depend on the installation process. A range of tests conducted earlier at Blessington confirmed that heavy driving reduces the radial effective stresses available after installation and creates scope for capacity gains over time. However, employing a greatly oversized hammer for the micro-piles led to capacities that did not change with time and were close at an early age to the upper limit developed after long ageing by conventionally driven piles.

Group 3: considering the third key question as to whether any upper limit applies to shaft capacity, irrespective of any
continuing active ageing processes, the following points can be made.

(a) Upper limits to shaft capacity were found at all three sites, by piles of two scales, that appeared to fall around 2·1 to 2·5 times the capacities estimated by the medium-term ICP-05 procedures when based on best estimates of pile and soil conditions.

(b) While most piles are installed with initial capacities well below this upper limit, a range of ageing processes apply in the field that allow growth over time towards the site-specific maxima, which appear to be controlled by limits to the radial pressures that can be developed around the pile shafts.

(c) Exceptions to the above general trend include:

(i) chemically inert stainless steel micro-piles driven in dense Dunkirk sands, which showed no set-up above their initial capacities that fell around 50% above the ICP estimates; (ii) the micro-piles driven in loose, silt and contractant Larvik sand, whose degrees of interface dilation and therefore shaft capacities fell far below those expected.

**APPENDIX: PILE AGEING IN SILICA SANDS**

See Table 10.

**Table 10. Case studies identified by Rimoy et al. (2015)**

| Reference              | Site location            | Ground description                     | Pile description                        | Load test description |
|------------------------|--------------------------|----------------------------------------|-----------------------------------------|-----------------------|
|                        |                          |                                        | Material (number of piles) | Average length: m | Section/diameter: m | L/D  | Type, direction |
|                        |                          |                                        | Concrete (28) | 11 | Hexagonal 0·305 | 36 | S, C  |
| Tavenas & Audy (1972)  | Quebec city, Canada      | Medium uniform sand                    | Concrete (6) | 21 | Square 0·395 | 53 | D/S, C  |
| Skov & Denver (1988)   | Hamburg, Germany         | Sand and silt                          | Steel (1) | 33·7 | Pipe 0·762 | 44 | D/S, C  |
|                        | Südkai, Hamburg Harbour, | Coarse medium to medium fine sand and   | Concrete (1) | 11 | Square 0·508 | 21 | D/S, C  |
|                        | Germany                  | fine gravel                            | Fine sand | 41·4 | Pipe 0·609 | 68 | D, C  |
| Seidel et al. (1988)   | Australia                | Loose to dense sand                    | Steel (5) | 21·5 | Square 0·688 | 31 | D/S, C  |
| Zai (1988)             | China                    | Saturated silty sand                   | Concrete (5) | 38 | Square with void 0·573 | 66 | D/S, C  |
| DiMaggio (1991)        | Mobile County, Alabama,  | Silty claysy fine sand                 | Concrete (1) | 27·4 | Square 0·402 | 68 | D/S, C  |
|                        | USA                      |                                        | Stein (1) | 25·3 | Closed-ended pipe 0·324 | 78 |  |
| Svinkin et al. (1994)  | Various sites, USA       |                                        | Steel (13) | 19·9 | Open-ended 0·355 tapered to 0·2 over 7·6 m | 99·5 | D/S, C  |
| York et al. (1994)     | JFK international terminal, Jamaica, New York, USA | Organic silty clays and peats underlain by fine to medium glacial sand | Timber (1) | 15·8 | N/M | N/A |  |
|                        | USA                      |                                        | Steel (1) | 20·7 | Pipe | N/A |  |
| Chow et al. (1998)     | Port Autonome de Dunkerque, Dunkerque, France | Marine silica sand | Steel (2) | 16·5 | Open-ended pipe 0·324 | 51 | D/S, C/T |
| Axelsson (2000)        | Fittja Strait, Värby, Stockholmn, Sweden | Loose to medium dense glacial sand, $q_c$ average 21 MPa | Concrete (4) | 17·4 | Square 0·265 | 65·8 | D/S, C  |

Continued
INVESTIGATING THE EFFECTS OF AGE ON STEEL PILES DRIVEN IN SAND

Table 10. Continued

| Reference | Site location | Ground description | Pile description | Load test description |
|-----------|---------------|---------------------|------------------|-----------------------|
| Fellenius & Altaee (2002) | JFK international terminal, Jamaica, New York, USA | Fine to coarse medium dense to dense glacial sand | Steel (1) 18 Open monotube 0-45 | 40 D, C |
| Jardine et al. (2006) | Port Autonome de Dunkerque, Dunkerque, France | Medium to dense marine silica sand | Steel (3) 19-02 Open-ended pipe 0-457 | 41·6 S, T |
| Rimoy (2013) | Red Sea port development | Dense coral granitic gravels & sands with cementation | Steel (13) 31·8 Open-ended pipe 1·219 | 26·1 D/S, C |
| Gavin et al. (2013) | Blessington, Ireland | Dense fine glacial sand: q<sub>f</sub> 10–20 MPa | Steel (4) 7 Open-ended pipe 0·34 | 20·6 S, T |
| Karlsrud et al. (2014) | Larvik, Norway | Loose to medium dense clayey silty fine sand | Steel (7) 21·5 Open-ended pipe 0·508 | 42·3 S, T |
| | Ryggkollen, Norway | Medium dense medium fine to coarse sand with cobbles | Steel (6) 20 Open-ended pipe 0·406 | 49·3 S, T |

*C, compression; T, tension; D, dynamic testing with PDA and analysed by CAPWAP (Rausche et al., 1985); S, static testing; N/M, not mentioned.
†Further comment on case studies: for piles with non-circular cross-sections the diameter of an equivalent circular cross-section base area is used.
‡Further comment on case studies: Tavenas & Audy’s (1972) aged piles capacities were obtained from Fig. 15 of the reference.

NOTATION

- D<sub>0</sub>, pile outer diameter
- D<sub>r</sub>, relative density
- D<sub>90</sub>, size of particle at 50% point on particle distribution curve
- D<sub>90</sub><sup>t</sup>, size of particle at 90% point on particle distribution curve
- f<sub>s</sub>, sleeve friction resistance in cone penetration test (CPT) and CPTu
- G<sub>max</sub>, operational shear modulus
- G<sub>min</sub>, small-strain shear modulus
- h, depth of the pile tip below the point in question along the pile
- Q<sub>c</sub>, compression capacity
- Q<sub>t</sub>, tension shaft capacity
- Q<sub>sc</sub>, calculated tension shaft capacity
- Q<sub>mm</sub>, measured tension shaft capacity
- q<sub>c</sub>, measured cone resistance in CPT and CPTu
- R<sub>ex</sub>, equivalent pile radius for open-ended piles
- R<sub>t</sub><sup>l</sup>, centre-line average roughness
- t, pile wall thickness
- u<sub>2</sub>, pore pressure measured behind the cone in CPTu
- z<sub>crit</sub>, critical depth for shallow CPT
- Δσ<sub>rl</sub>, change in radial shaft stress during loading
- Δσ<sub>v</sub>, constant volume interface shearing angle
- Δσ<sub>fr</sub>, radial cavity strain
- Δσ<sub>fe</sub><sup>r</sup>, equalised shaft radial effective stress on the pile shaft
- Δσ<sub>ef</sub><sup>t</sup>, effective stress on the pile shaft at failure
- σ<sub>ef</sub><sup>si</sup>, in situ effective stress
- σ<sub>avg</sub>, piles’ average shaft shear resistance at failure
- τ<sub>fr</sub>, local shear stress at failure
- φ<sub>cr</sub><sup>v</sup>, constant volume angle of shearing resistance
- φ<sub>p</sub><sup>r</sup>, peak angle of shearing resistance

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