1 Stability and load-displacement behaviour of axially cyclic loaded displacement piles in sands

2 S. Rimoy¹, M. Silva², R. Jardine³

3 ¹ University of Dar es Salaam, Lecturer in the Department of Transportation and Geotechnical
4 Engineering, United Republic of Tanzania, rimoy@udsm.ac.tz

5 ² Federico Santa Maria Technical University, Assistant Professor in the Department of Civil Engineering,
6 Chile, matias.silvai@usm.cl.

7 ³ Imperial College London, Professor in the Department of Civil and Environmental Engineering, United
8 Kingdom, r.jardine@imperial.ac.uk

9 Abstract

10 Uncertainties regarding the axial cyclic behaviour of piles driven in sands led to an extended
11 programme of calibration chamber instrumented pile experiments. Broad trends are identified and
12 interpreted with reference to normalised cyclic loading parameters Q_{cyclic}/Q_T , Q_{mean}/Q_T and N . Cyclic
13 damage is shown to be related to changes in the radial effective stress regime close to the shaft. While
14 stable loading leads to little or no change as cycling continues in the sand masses' effective stress
15 regime, high-level cyclic loading can affect stresses far out into the sand mass. The test systems'
16 chamber-to-pile diameter ratio has a significant impact on outcomes. Piles installed in loose, fine, sand
17 are far more susceptible to cyclic loading than in denser, coarser sand. Little or no change in pile
18 stiffness was seen in tests that remained within the stable cyclic region, even over 10,000 or more
19 cycles. Unstable tests lost their stiffness rapidly and metastable cases showed intermediate
20 behaviours. The permanent deflections developed under cycling depend on the combined influence
21 of Q_{cyclic}/Q_T , Q_{mean}/Q_T and N . While model tests provide many valuable insights into the behaviour of
22 piles driven in sand, they are unable to capture some key features observed in the field.

23 Background

24 The ability to sustain cyclic loading can be critical to structures founded on piles, particularly in
25 offshore energy applications; see Jardine et al (2012) or Andersen et al (2013). However, relatively
26 little information exists to help guide designers, especially in sands. The French National *Sollicitations*
27 *Cycliques sur Pieux* (SOLCYP) programme described by Puech and Garnier (2017) aimed to address this
28 shortfall. Axial cyclic design methodologies were proposed for sands based on monotonic and cyclic
29 field tests on driven and bored piles, drawing on the one-way (OW) tension and two-way (TW) tension
30 and compression cyclic experiments conducted by Jardine and Standing (2012) on seven, 457mm
31 outside diameter, mostly 19m long, open steel piles driven in medium-to-fine dense Dunkirk sand, in

northern France. Relevant parallel studies at the same location include: (i) those by Jardine et al (2006) and Carroll et al (2020) into how ageing affects the axial capacity of open steel driven piles and (ii) the PISA project's multiple tests on driven steel monopiles subjected to monotonic and cyclic lateral loading (see Byrne et al 2017; Zdravkovic et al 2020 and MacAdam et al 2020).

Field cyclic test outcomes

Thirteen cyclic axial tests conducted at Dunkirk led to the interactive stability diagram shown in Figure 1 which delineates how responses to regular cycling depend on the combination of mean pile head load (Q_{mean}) and cyclic amplitude (Q_{cyclic}), relative to the tension shaft capacity (Q_T), and number (N) of cycles. It categorises pile behaviour under cycling as being potentially:

- Stable (S), where pile head axial displacements accumulate very slowly over thousands of cycles and there is no significant loss of shaft capacity,
- Unstable (US), where displacements accumulate relatively rapidly and shaft capacity falls to give failure within 100 cycles, or
- Meta-Stable (MS), where displacements accumulate at moderate rates without stabilising. Capacity losses accrue that lead to failure after 100s or 1000s of cycles.

The normalised cyclic outcomes appeared broadly insensitive to the piles' ages after driving and prior axial loading histories. Rimoy et al. (2013) analysed the piles' cyclic axial load-displacement and stiffness responses, showing that stable cycling led to minimal displacement accumulation and no stiffness loss. However, unstable cycling led to displacement drifts and stiffness reductions that accelerated rapidly as failure was approached, while metastable cycling led to intermediate behaviours. Rimoy et al (2013) illustrated how normalised displacement accumulations varied with the normalised loading parameters through 3-D plots, as shown in Figure 2. Bored and cast in-situ were tested cyclically nearby (at Loon Plage) in a similar sand profile proved to be far more sensitive to load cycling. The stress relief and lack of pre-conditioning cycles implicit in their installation process left them far less able than driven piles to sustain repetitive loading; Puech and Garnier (2017).

Further research requirements and earlier model tests at Grenoble

While the Dunkirk tests provide a uniquely valuable dataset, they could not consider how different sand conditions might affect cyclic performance. It was impossible to measure soil contact stresses on the driven piles' surfaces, or in the sand mass. Extending the tests to cover more than ≈ 1000 cycles was also unfeasible. Cyclic laboratory tests conducted on small model piles, under 1-g conditions, or by multi-g centrifuge techniques (Guefrech et al. 2012, Blanc et al. 2015, Blanc & Thorel 2016, Isorna

et al. 2017) offer a potential means to address such gaps in knowledge, as reported from Chan and Hanna (1980) onwards. However, the practical credibility of such experiments depends on the degree to which they represent field conditions.

Jardine et al (2013a, b), Tsuha et al. (2012) and Rimoy et al (2015) reported experiments with 1m long, 36mm diameter model piles in the 1.20m diameter, 1.50m deep, fully pressurised Grenoble-INP calibration chamber, aiming to study in greater detail the phenomena observed in the field. The experiments were run under strictly controlled environmental conditions, with multiple miniature sensors gauging the local stresses both on the pile shaft and in the dry sand employed: see Jardine et al (2009) and Zhu et al (2009). Crucial differences between field and laboratory behaviour soon became evident. Rimoy et al (2015) noted that the model piles did not display the same marked growth of shaft capacities in the weeks and months that follow installation. Carroll et al (2020) ascribed this to (i) physio-chemical conditions being different in the laboratory and (ii) potential scale and geometrical effects that result from the finite thicknesses of zones of crushed sand grains formed around the walls of open tubular steel piles. Nevertheless, the model tests offered new insights into the effective stress conditions developed during installation, static load testing and under repetitive loading (with up to 1000 cycles) in dense dry silica Fontainebleau NE34 sand.

The extended Grenoble study

This paper reports further model experiments conducted by Rimoy (2013) and Silva (2014) at Grenoble with similar systems that explored the potential influences on cyclic behaviour of:

- 1) Different sands and initial sand states
- 2) Calibration chamber boundary conditions
- 3) Larger numbers of cycles and a wider range of loading regimes
- 4) Installation by driving as well as jacking

The experiments provided both new insights into displacement behaviour and an opportunity to reassess the relative advantages and disadvantages of field versus laboratory testing.

Model piles

The model test piles comprised either:

- Closed-ended, 36mm diameter (D) cone-tipped driven 'DT' piles equipped with axial load cells, or
- The cone-tipped, stainless-steel, 36mm diameter (D) Mini-Imperial College Pile (Mini-ICP) shown in Figure 3, which records local shaft shear and radial stresses accurately through three sets of Surface Stress Transducers (SSTs) as well as axial loads and temperatures.

Careful consideration had to be given to the ratio between pile centreline surface roughnesses (R_{cla}) and diameter D , which affects the axial capacity of model piles markedly; see Jardine et al (2005) or Carroll et al (2018). Adopting the typical $R_{cla} \approx 10\mu\text{m}$ of industrial piles for small diameter model piles would lead to the component of shaft resistance (equal to $\Delta\sigma'_{rd} \tan\delta$) that results from constrained interface dilation dominating capacity. The model piles were therefore air-abraded to lower R_{cla} values of 3 – 4 μm to reduce any unintended model scale effects. The pile tips had the same geometry as standard 36mm diameter CPT cones.

Calibration chamber testing arrangements

Each model pile was installed in a new ≈ 2.7 tonnes mass of air-pluviated sand contained within the test calibration chamber. Fontainebleau NE34 or GA39 sands were employed, whose index properties are listed in Table 1. Detailed characterisation of NE34 has been reported by Yang et al (2010), Aghakouchak (2015), Aghakouchak et al. (2015), Altuhafi et al (2018), Liu (2018), and others; the last three references also considered Dunkirk sand, showing that it has similar mechanical behaviour to NE34. GA39 also has broadly similar behaviour, despite its smaller grain size.

Rimoy (2013) and Silva (2014) describe the systems employed to measure local stresses and loads in the sand masses while maintaining good long-term boundary stress and temperature controls. Multiple miniature stress sensors were deployed, whose marked non-linear cell action interaction effects were recognised and carefully calibrated, following Zhu et al. (2009). The sand masses could be tested dry, or after near-saturation with water following CO_2 replacement, employing initial density indices $I_D \approx 40\%$ and 70% for the GA39 and NE34 sands respectively. Vertical pressures of 150 ± 5 kPa were maintained before pile installation, over extended ageing durations and during load testing.

The experimental chamber-to-pile diameter ($D_{chamber}/D$) ratios was 33.3. Although much lower ratios are frequently adopted, Salgado et al. (1998) argue that field conditions can only be matched in rigid chambers if $D_{chamber}/D > 100$. Check tests were therefore undertaken with a range of 'passive' and 'active' conditions (after Huang & Hsu 2005):

- BC1 – Passive stress-controlled radial boundaries, with three depth-segmented water-filled membranes held under constant applied pressure
- BC3 – Passive radial rigid-wall, as imposed by the steel chamber boundary
- BC5 – Active radial boundaries that replicated 'field conditions' through three depth-segmented water-filled membranes which are isolated to measure the three boundary pressures during installation. The elevated pressures achieved at end of installation were held fixed throughout ageing to compensate for the effects of elastic strain energy stored beyond

the limit of the calibration chamber wall in the field. The membranes could be isolated if required during static or cyclic testing.

Figure 4 presents the tip resistance profiles recorded during continuous monitoring of the jacked Mini-ICPs into the sand masses contained in the calibration chamber. These traces, which are equivalent to CPT qc profiles, show profiles that tend to a steady maximum equivalent qc ≈ 20 MPa in NE34 sand, while the finer GA39 sand profiles developed maxima of ≈ 10 and ≈ 15 MPa in tests ICP05 and ICP10 respectively.

Pile testing methods and programme

This paper reports 43 cyclic and 42 static tests conducted on 5 'DT' and 7 Mini-ICP piles. Table 2 outlines all the installations and loading sequences considered in this paper. Tests on the finer GA39 sand were conducted with a density index $I_D \approx 0.40$, while tests on the NE34 sand with an $I_D \approx 0.70$. Mini-ICPs ICP04 to 11 were jacked to their final base depth of 980 mm in around 50 jack strokes, applied at 2 mm/s, followed by full unloading at 0.01 mm/s to zero pile head load, as in pile driving. The driven piles were installed through a 3m high aluminium tubular frame. A driving mechanism released a 14.8 kg hammer, allowing it to fall through drop-heights that could be varied. The drop-heights were progressively increased as the series progresses, from ≈ 800 mm for piles DT III and DTIV, to more than 2m for DTVI and DTVII. Rimoy (2015) gives further details of the earlier trial piles DTI and DTII.

The static testing was conducted under displacement control at a wide range of ages after installation. Rimoy (2013) shows that the jacked stainless-steel model piles' capacities were insensitive to age in NE34 sand and generally fell close to those predicted with the CPT-based ICP-05 (Jardine et al 2005) procedures. The latter give generally unbiased predictions for field pile tests, covering wide range of pile sizes and sand states, conducted 10 to 30 days after driving; see Jardine et al (2006) and Yang et al (2017). However, the laboratory model piles driven into NE34 sand gave considerably lower tension capacities than expected by ICP-05 predictions, indicating that calibration chamber tests do not reproduce field driving conditions in a fully representative manner. Mini-ICP piles jacked into loose GA39 sand also gave lower than expected tension capacities; installation in loose sand masses appears difficult to match in calibration chambers; Rimoy (2013).

Cyclic testing was achieved by imposing smooth sinusoidal load or displacement waveforms at the pile head, as illustrated in Figure 5. Rates between 0.5 and 8.5 cycles per minute were employed which spanned the typical median offshore foundation loading period of ≈ 10 s (or 6 cycles per minute). Most of the cyclic tests were conducted under load control, following the Q_{mean}/Q_T and Q_{cyclic}/Q_T parameters in Table 2 and 3. The cyclic tests' codes signify: first their Test Series numeral, then an extension

denoting the mode (One-Way or Two-Way), followed by the number of tests performed on the pile up to, and including, that stage. Reference shaft capacities (Q_T) were measured through tension testing before most cyclic tests. Tension tests were adopted because they require smaller displacements (usually far less than $D/10$) than compression tests to reach peak resistance and are simpler to interpret. It is recognised that prior testing on any given pile affected the responses observed in any subsequent test and the reference static tension tests served to track the impact of prior experiments on axial shaft capacity. Tension loads are reported as having a negative sign. High level Two-Way cycling could not be sustained over many cycles without uncontrollable failure, so displacement control was employed for such cases. The loads sustained by the pile shafts varied as the pile shafts lost their resistance, leading to declining load paths in Q_{mean}/Q_T and Q_{cyclic}/Q_T space, as noted in Tables 2 and 3.

Axial cyclic loading tests results

Classification criteria

The overall test outcomes are summarised through cyclic stability diagrams, which consider how the number of cycles N sustained before failure varied with loading conditions. Cyclic failure was defined in terms of the accumulated cyclic displacements a normalised by the pile diameter D (mm), and the derivative $d(a/D)/dN$.

Stable (S) tests sustained at least 1000 cycles, displaced by less than $0.01D$ at rates less than or equal to $3D \times 10^{-6}$ mm/cycle and showed no loss in operational static shaft capacity.

Unstable (US) tests accumulated total displacements greater than $0.1D$ at rates greater than $3D \times 10^{-4}$ mm/cycle over significant durations, showed significant shaft capacity degradation and generally failed within 100 cycles.

Metastable (MS) tests showed intermediate displacements, displacement rates and modest shaft capacity degradation that led to failure in hundreds of cycles. While small numbers of such cycles may not affect capacity or serviceability, large numbers cannot be sustained safely.

Outcomes for piles jacked into dry NE34 sand

Figure 6a considers the new tests on dry NE34 sand. These include a test taken to 7,000 cycles on ICP04 (under BC3 conditions) as well tests on ICP07, ICP08 and ICP11 after their installation with BC5 conditions. The outcomes, categorised as Stable, Metastable or Unstable, manifest a similar pattern to the Dunkirk field tests summarised in Figure 1. The displacement controlled ICP08 test provides an interesting example in this plot. The N values annotated at five stages of the loading path followed by

191 this experiment trace out how axial capacity degraded over several thousand cycles. After undergoing
192 immediate shaft failure during its first cycle, ICP08 showed sharp reductions in capacity and stiffness
193 as its load path migrated down to levels at which load-controlled tests had been fully stable. The pile
194 was mobilising less than 10% of its initial capacity when cycling ended after 20,002 cycles and a post-
195 cycling tension test proved that it has lost $\approx 70\%$ of its initial shaft capacity.

196 *Impact of sand grain size and relative density*

197 Sand density and grain size effects were investigated next in installations ICP05 and ICP10, which
198 imposed BC3 and BC5 conditions respectively. While it had been intended to achieve a similar I_D to
199 that of the NE34 (≈ 0.70) test with the finer GA39 by a modified air-pluviation procedure, this proved
200 difficult, and the final I_D achieved was ≈ 0.40 . Tables 2 and 3 shows that the looser and finer sand
201 masses led to far less favourable monotonic and normalised cyclic capacities. The nine load-controlled
202 tests plotted in Figure 6c for GA39 sand show normalised boundaries that fall well below those
203 manifested by denser NE34 sand.

204 *Interface effective stress paths from Mini-ICP tests in dry sand*

205 The new cyclic loading tests undertaken in the ICP05 to 11 experiments revealed similar styles of
206 effective stress responses applied with: (i) a range of calibration chamber boundary conditions, (ii) wet
207 NE34 sand, (iii) finer, looser, dry GA39 sand, (iv) in many tests, after far larger numbers of cycles. Figure
208 7 illustrates these trends with SST data recorded at three levels of the pile shafts in dry NE34 sand
209 tests employing BC5 conditions. Figure 7a) illustrates a stable test (ICP07-OW1) which confirms that
210 the trends identified by Tsuha et al (2012) are not affected significantly by changing the boundary
211 conditions: the local stress loops remain immobile and do not engage the interface shear criterion.
212 Figures 7b) and c) illustrate the impact of the unstable, extreme two-way applied conditions in the
213 earlier discussed ICP08 TW1 experiment, considering in Figure 7b) both the first 10 and the final 20
214 cycles of this 20,002 cycles experiment. As in Tsuha et al's (2012) tests, the $\pm 0.5\text{mm}$ movements (or
215 $\pm 2.8\%$ of the pile diameter D) imposed were sufficient to cause all points on the shaft to engage the
216 interface shear failure envelope, even though larger displacements (up to $D/10$) would be required to
217 achieve static failure after dilation to 'critical states'; see Jardine et al (2013a, b). The latter test
218 indicated a higher interface shearing angle $\approx 30^\circ$, possibly reflecting an inadvertently rougher surface
219 texture with this pile.

220 Rimoy (2103) found more significant differences from Tsuha et al's (2012) patterns in his (dry) Mini-
221 ICP tests with the finer, loose, GA39 sand, which developed far lower axial resistances and radial shaft
222 shear stresses than were seen with NE34 sand under the same (150 kPa) vertical calibration chamber

pressure. Figures 8a) and 8b) present ICP05's interface stress paths, as measured at two shaft levels in tests ICP05 OW1 and TW1 under BC3 conditions. While the OW1 experiments' Q_{cyclic}/Q_T , Q_{mean}/Q_T coordinates plot in the denser NE34 sand's 'stable' region, the GA39 stress paths engaged the interface shear envelope the start of cycling and showed marked σ'_{rs} reductions and cyclic displacement growth. Higher interface shear angles ($\delta \approx 34^\circ$) applied because of the higher normalised pile-soil roughness ratio R/d_{50} ; as noted in ring-shear tests by Ho et al (2011). The next (TW1) cyclic test on ICP-05 developed more open effective stress path loops and more significant stationary effective radial stresses, σ'_{rs} drift. Shaft capacity losses of 7% and 22% respectively were confirmed by post-cyclic tension tests, confirming a metastable response. Further cyclic tests (TW2 and OW2) on ICP05 led to metastable and unstable responses respectively, with further modest reductions in σ'_{rs} and shaft capacity. These experiments, and later tests on ICP10 (which was installed in loose GA39 under BC5 boundary conditions) confirmed the higher interface shear angles and provided more information on the model piles' greater susceptibility to cyclic loading, in both absolute normalised terms, when installed in the finer and looser GA39 sand.

237 Piles installed in submerged NE34 sand

Direct comparison between the dry sand model experiments and the Dunkirk field tests might be questioned on several grounds, including the key difference that the field piles developed most of their capacity below the water table. Experiment ICP06 was designed to test whether water influenced the outcomes by installing a Mini-ICP, under BC3 conditions, in near-saturated NE34 sand. A displacement-controlled, TW, test was then conducted, followed by a fully stable OW test to over 1000 cycles, which raised the shaft capacity marginally, as listed in Table 2. The normalised outcomes of both tests are presented in Figure 6b, showing fully compatible trends to Figure 6a: submerging the sand did not affect the normalised cyclic stability patterns.

Rimoy's (2015) investigation of how water affected behaviour identified again asymmetric 'butterfly-wing' local interface stress path patterns over the first ten cycles of his displacement controlled ($\pm 0.5\text{mm}$) TW1 test on ICP06. As before, the paths were constrained by the $\delta = 27^\circ$ interface shearing angle, before progressing with gradually reducing rates of σ'_{rs} loss to greatly diminished final conditions after 12,000 cycles. A post-cyclic tension test confirmed a 71% loss of shaft capacity, close to that observed with dry sand in ICP08.

252 Influence of installation method

The influence of installation procedure was investigated further by driving closed ended 36 mm diameter piles DTIII to DTVII in dry, $I_D \approx 0.70$, NE34 sand under BC1 and BC5 conditions. A total of

255 twenty-one (OW and TW) cyclic loading tests were performed, as listed in Table 3, leading to the
256 results plotted in Figure 6d. The model driven piles' normalised unstable, metastable and stable
257 boundaries plot below those manifested by comparable cyclically jacked piles (Figure 6a and b) and
258 also the Dunkirk field test trends in Figure 1. Possible reasons for this discrepancy are discussed later.

259 Measurements were also made in BC5 tests of the pressures developed in the chamber's three,
260 vertically stacked, outer radial chamber membranes. The membranes were isolated during the TW1
261 test on ICP08 and Figure 9b) shows that an ≈ 10 kPa reduction in σ'_{rs} developed out at $r/R \approx 30$,
262 confirming that either larger chambers or active boundary conditions (such as the BC5 system) are
263 required to capture the field response in a fully representative manner.

264 Boundary pressure measurements made with BC5 conditions during installation showed increases
265 during jacked installation (when the membranes were isolated) that contrasted with radial pressures
266 falling sharply at the chamber wall during equivalent driven installations; see Figure 10a) and 10b).
267 The top membrane's vertical pressure also diminished by $\approx 30\%$ by the end of driving. Balachowski
268 (1995) reports similar reductions lateral membrane pressures when an open-ended pile was driven
269 into a calibration chamber filled with very dense sand. Reduced-scale physical modelling of dynamic
270 pile installation presents considerable challenges because the driving energy that is free to radiate
271 away under field driving conditions may be reflected and conserved within confined test chambers in
272 an unrepresentative manner.

273 *Cyclic stress variations in the sand-pile interface*

274 The Mini-ICP piles' instrumentation allowed the cyclic local shear stress τ_{rz} versus normal radial
275 effective stress σ'_r stress paths to be followed at the sand-pile interface, which was not possible with
276 the piles driven in the field or laboratory. It is also possible to follow the stress paths invoked within
277 the surrounding sand mass. Tsuha et al. (2012) reported on the paths followed at three levels on the
278 shaft in dry ($I_D \approx 0.70$) NE34 sand under BC1 and BC3 conditions, showing that stable tests' stress paths
279 remained within an interface failure envelope, for which $\delta \approx 27^\circ$ in accordance with Yang et al's (2011)
280 ring-shear interface tests. As in the field, stable cycling led to either modest shaft capacity gains, or no
281 loss in tension resistance. However, unstable tests' effective stress paths engaged the $\delta \approx 27^\circ$
282 envelopes from an early stage and developed 'butterfly-wing' shaped paths that collapsed inwards
283 with each unloading stage and resembled those of sands undergoing undrained 'cyclic mobility' in
284 saturated triaxial or simple shear tests, even though the sand was dry. Marked losses in the stationary
285 (or mid-cycle) σ'_{rs} values developed as cycling progressed that led rapidly to failure. The metastable
286 tests' stress paths fell between these limits, with slower reductions in σ'_r that led to shaft failure after
287 100s or 1000s of cycles. Jardine (2020) ascribed these trends to the kinematic conditions applying at

the pile shaft, which maintain near constant volume conditions even though the dry (or wet) sand can drain freely. He also showed how this could be predicted from triaxial or Hollow Cylinder Apparatus experiments that followed the protocols outlined by Aghakouchak et al (2015).

Cyclic stress variations in the sand mass

Tsuha et al (2012) and Jardine et al (2013 a, b) showed how the radial, circumferential and axial stress measurements made within the sand mass could be combined with vertical shear stress distributions inferred from the SST pile instruments to track the 4-dimensional (σ'_z , σ'_r , σ'_θ and τ_{rz}) cylindrical stress system within the sand mass during cyclic pile load tests. Stable tests led to local effective stress paths that inevitably engaged and re-located the sand's elastic (Y_1) kinematic yield surface (KYS) but remained within its second Y_2 KYS, as defined by Jardine (1992), (2013). This system of KYSSs was identified and explored in cyclic laboratory tests on Dunkirk and NE34 sands by Kuwano (1999) and Aghakouchak et al (2015). The radial σ'_r stress changes provoked within the surrounding sand mass by severe two-way cycling are illustrated in Figure 8a) by measurements made at three normalised radial distances (r/R) from the pile axis at a normalised level $h/R = 39.2$ vertically above the tip in test TW1 on ICP08. While the pile shaft experiences large (≈ 130 kPa) final losses in σ'_{rs} , significant (≈ 50 kPa) reductions extended out to 2 to 8 radii from the axis.

Cyclic stiffness

Cyclic stiffnesses, $k = (Q_{\max} - Q_{\min})/d_{\text{cyclic}}$, were defined as outlined in Figure 11. The loading and unloading values k_l and k_u generally converged but could show differences when 'viscous' effects delayed the strain peaks and significant strains accumulated during each cycle in unstable tests. Tables 2 and 3 lists the stiffness measured over the first cycle of cycling $k_{l, N=1}$. The loose GA39 sand tests developed similar stiffnesses than the NE34 cases, within which stiffnesses generally reduced as Q_T fell (due to prior testing) and/or the imposed normalised loading levels rose. Figure 11a) displays the latter trends for GA39 and NE34 sand cases by plotting non-dimensional stiffness $k_{l, N=1}$ multiplied by diameter D and divided by Q_T against Q_{cyc}/Q_T for jacked pile tests, differentiating cases with $0 \leq Q_{\text{mean}}/Q_T < 0.3$ from those with $0.3 \leq Q_{\text{mean}}/Q_T \leq 0.6$, while Figure 11b) shows the equivalent for driven piles. Jacked pile tests showed similar non-dimensional stiffness as Dunkirk field tests at significantly lower levels of normalised cyclic loading. On the contrary, driven piles tests present a degradation of $k_{l, N=1}$ as Q_{cyc}/Q_T progresses for similar cyclic amplitudes as the jacked piles.

Figure 13 goes onto show how loading stiffnesses (and therefore cyclic displacements) remained practically constant up to 10,000 cycles in the stable ICP04-OW1, ICP06-OW1, ICP07-OW1, ICP07-TW1 and ICP11-OW1 tests by plotting $k_l/k_{l, N=1}$ against logarithm of N . The equivalent data from the

metastable tests ICP04-TW1, ICP04-OW2, ICP05-OW2, ICP05-TW1, ICP05-TW2, ICP10-OW4, ICP10-TW2 and ICP11-OW2 are also shown. Most of the latter also manifested nearly constant stiffness. However, in some cases, such as ICP04-TW1, stiffness degraded by up to 85% as failure approached. The corresponding unstable cyclic loading tests stiffness trends are illustrated in Figure 14. Test ICP05-OW2, ICP11-OW3 and ICP11-TW1 survived to just 74, 61 and 25 cycles respectively and showed cyclic stiffness degrading after just 10 to 30 cycles. Figures 15 and 16 show how the driven piles' normalised stiffnesses varied with number of cycles, giving broadly similar trends to the jacked piles. Overall, no clear difference could be seen between the stiffness degradation trends developed under BC3 or BC5 conditions in wet-or-dry, loose-or-dense sand masses, by jacked or driven piles.

Permanent accumulated cyclic displacements

The permanent cyclic pile-head displacements accumulated during load-controlled cycling are shown in Figure 17 to 19. Naturally, the displacement-controlled tests did not develop permanent displacements. The segmented reference lines correspond to the stability criteria discussed earlier, which are broadly consistent with the Dunkirk field tests analysed by Rimoy et al. (2013) although some differences exist in the rates at which the cyclic displacements (divided here by diameter D) grew as cyclic failure approached under metastable or unstable conditions. Stable cyclic tests showed almost negligible displacement accumulation rates, independent of the installation method, sand type and density indices.

Figure 2 illustrated the relationships found by Rimoy et al (2013) between the Q_{cyc}/Q_T , Q_{mean}/Q_T and N conditions at which permanent displacements reached 2%D in the full-scale Dunkirk tests. Figures 19 provide a comparison between Dunkirk results and the equivalent surfaces for tests on dry NE34 for displacement levels 0.2D. While reduced-scale model piles showed similar trends to the Dunkirk field test, they are unable to capture pile response at high levels of cyclic amplitude. Figure 20b and 20c shows that model piles reached equivalent normalised displacement levels under lower normalised cyclic amplitudes, confirming their narrower metastable zone and smaller stable region, as identified earlier in Figure 6. Rimoy (2013) and Silva (2014) give further details of, and discussion on, the model piles' permanent displacement trends.

Discussion

The studies described in this paper explored first how cyclic stability diagrams could be constructed from model tests. The tests examined aspects that could not be varied easily in the field, including sand state, grading and large numbers of load cycles, through both displacement and load-controlled arrangements. The experiments also investigated whether the stability outcomes vary between cases

352 where the sand mass is dry or saturated with water, as well as the influence of model test boundary
353 conditions. The multiple stress sensors deployed in the experiments provided new insights into the
354 local conditions developed on the surfaces of jacked Mini-ICP piles, within the surrounding soil mass
355 and on the calibration chamber boundaries during installation and cyclic testing. These observations
356 are valuable to the formulation of representative degradation laws for incorporation into FEM or local
357 (t-z) soil-pile cyclic analyses. Finally, the tests provided information on the highly non-linear stiffness
358 response and permanent displacement growth trends of the model piles under cyclic loading. These
359 outcomes are valuable, given the very sparse evidence that has been reported through field cyclic
360 loading experiments on piles driven in sands.

361 Designers of foundations intended to carry significant levels of cyclic loading require guidance on how
362 to consider repetitive loading cases safely and economically. Jardine et al (2012) and Puech and
363 Garnier (2016) summarised how this might be achieved through the routes incorporated in the
364 'SOLCYP' scheme shown in Figure 20. Reliable cyclic stability diagrams, such as Figure 1, can aid both
365 preliminary screening studies and quantitative global assessments; Jardine and Standing (2012). Such
366 procedures have applied in offshore projects reported by Merritt et al (2012) and Argiolas and Jardine
367 (2017). Figure 20 also identifies pathways for generating cyclic stability diagrams through site-specific
368 cyclic element testing, which may follow specific protocols (Aghakouchak et al 2015, or Jardine 2020),
369 as applied by Rattley et al (2017). Cyclic element testing can also inform or calibrate numerical
370 analyses that employ FEM or local (t-z) soil-pile cyclic tools, such as those reported by Atkins (2000).

371 *Potential differences between model pile outcomes and field behaviour*

372 However, it is crucial to consider how well the model piles represent field behaviour. The axial
373 capacities developed by piles jacked into NE34 sand appeared compatible with design rules that match
374 the field test capacities of open and closed ended piles in a wide range of relative densities, as
375 measured at moderate ages (around 10 days) after driving, although the degree of fit varies with the
376 calibration chamber boundary conditions; see Rimoy (2013).

377 Despite multiple variations being made in boundary conditions, sand states, installation procedures,
378 as well as cycles in the external calibration chamber pressures the jacked and driven model tests could
379 not replicate the marked set up trends reported from the Dunkirk field tests; Rimoy et al (2015). Model
380 tests cannot yet capture all aspects of the relevant physical processes and it is vital that attempts to
381 predict field pile behaviour through 1-g or centrifuge tests must recognise potential uncertainties
382 relating to test systems, such as the potential to conserve pile driving energy in an unrepresentative
383 manner, overly close boundaries, and scaling relationships that may not capture all of the physio-
384 chemical conditions that apply in the field. Model tests provide many valuable insights, but they do

385 not obviate the need for high quality and comprehensive field testing as described by Karlsrud et al
386 (2014), Byrne et al (2017) or Carroll et al (2020).

387 Nevertheless, the NE34 sand model test results match the field findings when normalised by the
388 current pile tension capacities, Q_T . The experiments also provide clear evidence that water has little
389 independent impact on the normalised cyclic loading outcomes. They proved too that finer sands
390 develop higher δ angles and indicated that piles installed in loose sands are likely to have both lower
391 cyclic stiffness and be particularly susceptible to cyclic loading, even in normalised terms.

392 **Conclusions**

393 This paper reviewed current uncertainties regarding the axial cyclic behaviour of piles driven in sands,
394 referring to field tests before reporting and interpreting an extended programme of 43, mostly heavily
395 instrumented, calibration chamber cyclic model pile tests supported by multiple static tests. The
396 programme was designed to explore the potential influences of (i) initial conditions in the sand, (ii)
397 large numbers of cycles and a wide range of loading regimes, (iii) calibration chamber boundary
398 conditions and (iv) pile installation by both driving and jacking. The experiments contribute towards
399 developing design guidance for cyclic axial loading cases and lead towards ten main conclusions:

- 400 1. Wet and dry sands placed at the same density indices follow the broadly same trends under
401 regular cyclic loading when the outcomes are considered in terms of the normalised cyclic loading
402 parameters Q_{cyclic}/Q_T , Q_{mean}/Q_T and N .
- 403 2. Little or no change in cyclic stiffness occurs when loading remains within a well-defined stable
404 region, even over 10,000 or more cycles. In contrast, unstable tests lose stiffness rapidly and fail
405 dramatically after less than 100 cycles. Metastable tests show intermediate responses that
406 depend on the imposed normalised loading levels.
- 407 3. Cyclic damage occurs due to reductions in the radial effective stresses acting on the shaft and
408 failure is controlled by interface failure angles that can be predicted by ring-shear laboratory
409 tests.
- 410 4. Stable loading leads to little or no change in the sand masses' effective stress regime as cycling
411 continues, but the effects of high-level cyclic loading extend out into the surrounding sand mass.
- 412 5. The chamber-to-pile diameter ratio of 33 employed in the present tests led to significant radial
413 stress variations at the chamber wall, whose impact could be reduced by adopting 'active' BC5
414 chamber boundary conditions. Model tests involving closer boundaries are inevitably less
415 representative of field behaviour.

- 416 6. The outcomes from cyclic tests undertaken with a range of conditions could be synthesised, and
417 rational comparisons made between test Series', by employing the normalised loading
418 parameters.
- 419 7. The jacked Mini-ICP calibration chamber tests in NE34 sand led to broadly similar normalised
420 cyclic stability, stiffness and permanent displacement trends to full-scale field tests (conducted
421 at various ages after driving) in Dunkirk sand, which has similar grain sizes, mechanical properties
422 and in-situ I_D levels to the main model test sand.
- 423 8. The calibration chamber tests confirm how the permanent deflections and stiffnesses developed
424 under cycling depend on the loading parameters Q_{cyclic}/Q_T , Q_{mean}/Q_T and N .
- 425 9. However, the model experiments were unable to capture reliably some of the key features
426 observed in the field, including shaft capacity growth over time, the impact of driven rather than
427 jacked installation and potentially the impact of low relative density sand layers. These outcomes
428 may be unintended consequences of the model pile testing arrangements.
- 429 10. While model tests provide many valuable insights into the cyclic behaviour of piles driven in sand,
430 they do not obviate the need for comprehensive, high-quality, field testing.

431

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Tables

Sand	Grain shape	SiO ₂ (%)	G _s	d ₁₀ (mm)	d ₅₀ (mm)	d ₆₀ (mm)	C _u	e _{max}	e _{min}
NE34	Sub-angular, sub-rounded	99.70	2.65	0.150	0.210	0.230	1.53	0.90	0.51
GA39	Sub-angular, sub-rounded	98.60	2.65	0.100	0.127	0.132	1.32	1.01	0.56

Table 1: Index properties of NE34 Fontainebleau and GA39 Nemours sand

No.	Test Series ^{1,2,3}	Test code	Jack load (kN)	N cycles	Rate (N/min)	Referential Q _T (kN)	Q _{cyclic} /Q _T	Q _{mean} /Q _T	Description	Post cycling tensile capacity (kN)	K _I , N=1 (kN/mm)
Dry NE34 sand with BC3 boundary condition and density indices I _D ≈ 70%											
1	ICP04	OW1	-3.5 – 0	7000	0.5	-11.11	0.15	0.15	Stable	-13.22	59.47
2		TW1	-4.0 –4.0	600 (N _f =580)	3.0	-13.22	0.23	0.06	Meta-stable	-5.39	38.50
3		OW2	-4.6 – -2.3	50	6.0	-5.39	0.21	0.63	Meta-stable	-6.00	31.91
Wet NE34 sand with BC3 boundary condition and density indices I _D ≈ 70%											
4	ICP06	TW1	Varies ²	12022	~3.0	-15.49	Varies	Varies	Unstable	-4.45	--
5		OW1	-1.0 – 0	1000	8.5	-4.45	0.11	0.11	Stable	-4.69	64.12
Dry NE34 sand with BC5 boundary condition and density indices I _D ≈ 70%											
6	ICP07	OW1	-7.2 – -4.8	4040	2.5	-13.16	0.09	0.46	Stable	-13.76	103.79
7		TW1	-1.2 – 1.2	4760	2.5	-13.76	0.07	0.02	Stable	-13.68	124.93
8	ICP08	TW1	Varies ²	20002	~4.0	-11.50	Varies	Varies	Unstable	-3.47	--
9	ICP11	OW1	-6.47- -2.58	2500	2.15	-13.19	0.14	0.34	Stable	-15.97	67.07
10		OW2	-8.73- -2.32	1163 (N _f =1163)	1.78	-15.97	0.20	0.34	Metastable	-15.17	67.35
11		OW3	-10.5 - -1.59	61 (N _f =61)	0.61	-15.17	0.29	0.40	Unstable	-13.56	56.59
12		TW1	-8.11- 2.65	25 (N _f =25)	1.19	-13.56	0.39	0.21	Unstable	-12.91	37.77
Dry GA39 sand with BC3 boundary condition and density indices I _D ≈ 40%											
13	ICP05	OW1	-2.2 – -0.6	1050	5.0	-3.77	0.22	0.38	Meta-stable	-3.50	54.97
14		TW1	-1.3 – 0.5	300	3.5	-3.50	0.26	0.11	Meta-stable	-2.74	22.48
15		TW2	-1.1 – 0.2	1000	6.0	-2.74	0.24	0.17	Meta-stable	-2.60	55.71
16		OW2	-1.9 – -0.2	74 (N _f =74)	4.0	-2.60	0.33	0.40	Unstable	-2.30	30.7

No.	Test Series ^{1,2,3}	Test code	Jack load (kN)	N cycles	Rate (N/min)	Referential Q_T (kN)	Q_{cyclic}/Q_T	Q_{mean}/Q_T	Description	Post cycling tensile capacity (kN)	$k_l, N=1$ (kN/mm)
<i>Dry GA39 sand with BC5 boundary condition and density indices $I_D \approx 40\%$</i>											
17	ICP10	OW1	-1.7 – 0	1000	4.92	-7.35	0.11	0.12	Stable	-7.68	61.56
18		OW2	-3.0 – 0	1000	4.29	-7.68	0.19	0.20	Stable	-8.20	50.56
19		OW3	-4.25 – -2.0	1000	6.94	-8.20	0.13	0.39	Stable	-6.16	68.50
20		OW4	-3.4 – 0	500 ($N_f=587^3$)	3.96	-6.16	0.27	0.28	Metastable	-4.25	40.37
21		TW1	-2.4 - -1.3	2 ($N_f=1$)	2	-4.25	0.43	0.14	Unstable	-2.95	2.50
22		TW2	-0.87 - -0.9	250	4.71		0.21	0.00	Metastable		6.58

¹ Tests ICP 01 to ICP03 are summarised by Tsuha et al. 2012.

² Tests conducted under displacement control with cyclic amplitude of +0.5 to -0.5mm.

³ Estimated number of cycles at failure

Table 2: Axial cyclic loading test series using Mini-ICP.

No.	Test series	Test code	Jack load (kN)	N cycles	Rate (N/min)	Referential Q _T (kN)	Q _{cyclic} /Q _T	Q _{mean} /Q _T	Description	Post cycling tensile capacity (kN)	k _I , N=1 (kN/mm)
Dry NE34 with BC1 boundary condition and density indices I _D ≈ 70%											
5	DTV	OW1	-1.57 – 0	1800	4.9	-7.38	0.10	0.11	Stable	-7.75	57.63
6		OW2	-2.84 – 0.53	1000	3.0	-7.75	0.15	0.22	Stable	-7.60	88.46
7		OW3	-3.75 - -2.35	1000	4.6	-7.60	0.09	0.40	Stable	-7.32	291.78
8		OW4	-4.94 - -0.75	71 (N _f =71)	2,2	-7.32	0.29	0.39	Unstable	-6.86	90.91
9		TW1	-2.22 – 0.74	670	2,6	-6.86	0.22	0.11	Meta-stable	-6.66	33.13
10		TW2	-2.85 – 2.13	23 (N _f =23)	0,82	-6.66	0.38	0.05	Unstable	-4.10	4.43
11		OW5	-1.15 - 0	1500	4.4	-4.10	0.13	0.15	Meta-Stable	-5.55	50.71
Dry NE34 with BC5 boundary condition and density indices I _D ≈ 70%											
1	DTIII	OW1	-3.1 – 0	1000	3.7	-7.40	0.21	0.21	Meta-Stable	-8.70	70.24
2		OW2	-2.3 – 0	1000	5.0	-8.70	0.14	0.13	Meta-Stable	-8.30	94.71
3	DTIV	OW1	-1.6 – 0	1000	4.8	-6.98	0.12	0.12	Stable	-7.81	63.07
4		OW2	-3.0 - 0	410 (N _f =410)	3.2	-7.81	0.20	0.20	Meta-stable	-7.69	66.13
12	DTVI ¹	OW1	-2.35 – -1.19	2000	3.9	-6.79	0.09	0.26	Stable	-7.24	105.27
13		OW2	-4.65 – -2.55	1500	3.3	-7.24	0.15	0.50	Meta-Stable	-7-27	104.80
14		OW3	-3.61 – -0.75	616 (N _f =616)	3.0	-7-27	0.20	0.30	Meta-stable	-7.13	92.89
15		OW4	-5.55 – -0.75	22 (N _f =22)	1.7	-7.13	0.34	0.44	Unstable	-7.00	25.16
16		TW1	-3.09 – 0.94	44 (N _f =44)	1.9	-7.00	0.26	0.18	Unstable	-6.39	45.56
17		TW2	-3.73 – 2.41	11 (N _f =11)	0.6	-6.39	0.39	0.14	Unstable	-4.44	5.05
18	DTVII ²	OW1	-5.30 – -3.82	2700	3.3	-7.54	0.10	0.60	Stable	-8.24	71.02

No.	Test series	Test code	Jack load (kN)	N cycles	Rate (N/min)	Referential Q_T (kN)	Q_{cyclic}/Q_T	Q_{mean}/Q_T	Description	Post cycling tensile capacity (kN)	$k_I, N=1$ (kN/mm)
19		OW2	-7.27 – -2.52	18 ($N_f=18$)	1.8	-8.24	0.29	0.59	Unstable	-8.34	15.44
20		OW3	-4.97 – -0.07	59 ($N_f=59$)	2.2	-8.34	0.29	0.30	Unstable	-7.96	36.57
21		TW1	-5.83 – 0.51	14 ($N_f=14$)	1.5	-7.96	0.38	0.35	Unstable	-8.01	25.30

¹ Lateral membranes' pressure were modified continuously during pile installation to minimize lateral volume variations.

² A pre imposed lateral confinement profile was imposed bases on jacked tests ICP07 and ICP08 lateral membranes responses.

Table 3: Axial cyclic loading test series using driven piles

Fig. 1: Axial cyclic interaction diagram for the full-scale pile tests in Dunkirk silica marine sands (after Jardine & Standing 2012)

Fig. 2: 3D plot for permanent accumulated cyclic displacements equivalent to 2% diameter (D) for Dunkirk field cyclic loading tests, after Rimoy et al. (2013).

Fig. 3: Calibration Chamber and Mini-ICP arrangements of tests ICP07 and 08; vertical section after Rimoy (2013). Pile sensors $h=9R$ above tip are 'Leading', at $26R$ are 'Following', and at $46R$ are 'Trailing'.

Figure 4: Pile end-bearing resistances for Mini-ICP pile jacked tests in this study

Fig. 5: Example of axial cyclic loading load patterns illustrated from load controlled test ICP05-OW2

Fig. 6a: Axial cyclic loading interaction diagram with the Mini-ICP in dry NE34 sand

Fig. 6b: Axial cyclic loading interaction diagram for ICP06 with Mini-ICP in wet NE34 sand

Fig. 6c: Axial cyclic loading interaction diagram with the Mini-ICP in dry GA39 sand

Fig. 6d: Axial cyclic loading interaction diagram for Driven tests in dry NE34 sand

Fig. 7a Interface effective stress paths from stable one-way cyclic loading test ICP07-OW1 in dry NE34 sand under BC5 boundary conditions

Fig. 7b: Interface effective stress paths from unstable two-way cyclic loading test ICP08-TW1 in dry NE34 sand under BC5 boundary conditions first 10 cycles.

Fig. 7c: Interface effective stress paths from unstable two-way cyclic loading test ICP08-TW1 in dry NE34 sand under BC5 boundary conditions final 20 cycles

Fig. 8a: Interface effective stress paths from ICP05-OW1 one-way cyclic test on loose GA39 sand, BC3 boundary conditions, failed after 1050 cycles.

Fig. 8b: Interface effective stress paths from ICP05-TW1 two-way cyclic test on loose GA39 sand, BC3 boundary conditions, failed after 300 cycles.

Fig. 9a: Sand mass radial stresses against the number of cycles applied in unstable two-way cyclic loading test ICP08-TW1 in dry NE34 sand under $\pm 0.5\text{mm}$ displacement control under with BC5 boundary conditions

Fig. 9b: Radial stresses at the calibration chamber boundary against the number of cycles applied in unstable two-way cyclic loading test ICP08-TW1 in dry NE34 sand under $\pm 0.5\text{mm}$ displacement control under with BC5 boundary conditions

Fig. 10a: Lateral boundary stresses during installations in Mini-ICP test ICP08 with boundary condition BC5.

Fig. 10b: Lateral boundary stresses during installations in driven test DTIII with boundary condition BC5.

Fig. 11: Displacement and stiffness parameters employed in the analyses

Fig. 12a: Non-dimensional stiffness $k_{l,N=1} D/Q_T$ against Q_{cyc}/Q_T for jacked pile tests

Fig. 12b: Non-dimensional stiffness $k_{l,N=1} D/Q_T$ against Q_{cyc}/Q_T for driven pile tests

Fig. 13: Normalised axial cyclic pile stiffness versus number of cycles applied in stable and metastable cyclic loading tests for the ICP pile

Fig. 14: Normalised axial cyclic pile stiffness versus number of cycles applied in unstable cyclic loading tests for the ICP pile

Fig. 15: Normalised axial cyclic pile stiffness versus number of cycles applied in stable and metastable cyclic loading tests for driven piles under BC5 boundary conditions

Fig. 16: Normalised axial cyclic pile stiffness versus number of cycles applied for in unstable cyclic loading tests for driven piles tests under BC5 boundary conditions

Fig. 17: Permanent accumulated cyclic displacement during cyclic loading tests in dry NE34 sand using the ICP pile under BC5 boundary conditions

Fig. 18: Permanent accumulated cyclic displacement during cyclic loading tests in dry GA39 sand using the ICP pile under BC5 boundary conditions

Fig. 19: Permanent accumulated cyclic displacement during stable and metastable cyclic loading tests in dry NE34 sand using driven piles under BC5 boundary conditions

Fig. 20a: 3D plot for permanent accumulated cyclic displacements equivalent to 0.2%D for Dunkirk field cyclic loading tests, Rimoy et al. 2013.

Fig. 20b: 3D plot for permanent accumulated cyclic displacements equivalent to 0.2%D for ICP pile tests in dry NE34 sand including data from Tsuha et al. 2012

Fig. 20c: 3D plot for permanent accumulated cyclic displacements equivalent to 0.2%D for driven pile tests in dry NE34 sand.

Fig. 21: 'SOLCYP' schematic workflow procedure for assessing pile response to axial cyclic loading; after Jardine et al (2012).