

Advanced laboratory testing in research and practice: the 2nd Bishop Lecture

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This paper demonstrates the special capabilities and practical value of advanced laboratory testing, focusing on its application in advancing the understanding and prediction of how driven piles function and perform in sand. Emphasis is placed on integrating laboratory research with analysis and field observations, drawing principally on work by the author, his colleagues and research group. The laboratory studies include highly instrumented static and cyclic stress-path triaxial experiments, hollow cylinder and ring-shear interface tests and micro-mechanical research. Soil element testing is combined with model studies in large laboratory calibration chambers, full-scale field investigations and numerical simulations to help advance fundamental methods for predicting pile behaviour that have important implications and applications, particularly in offshore engineering.

Notation

C_c	gradient in oedometer e -log σ'_v relationship
C_u	coefficient of uniformity = d_{90}/d_{10}
D	diameter; D_{chamber} and D_{pile} are the calibration chamber and pile diameters, respectively
d_{10}, d_{50}, d_{90}	particle diameters at 10%, 50% and 90% points, respectively, on particle-size distribution
E	Young's modulus, with subscripts and superscripts for different modes
e_0	initial void ratio
$e_{\text{max}}, e_{\text{min}}$	void ratios at the loosest and densest states, respectively
G	shear modulus, with subscripts and superscripts for different modes
H	height above the pile tip (positive) or depth below the pile tip (negative)
k	axial stiffness of pile
K	σ'_3/σ'_1 effective stress ratio
K_0	σ'_r/σ'_z under purely vertical loading with no radial strain
K_A	Rankine coefficient of active earth pressure
K_{CNS}	normal stiffness applied in the constant normal stiffness shear test
L_p	pile penetration depth
p'	mean effective stress
p_A	atmospheric pressure
Q	pile head loads, with subscripts and superscripts for different modes
q	deviator stress
q_c	cone penetration test cone resistance
R	pile radius
r	radius of point from pile axis
R^*	equivalent radius of an open-ended pile

R_0	outer radius of an open-ended pile
R_i	inner radius of an open-ended pile
T_{rz}	local shaft shear resistance on pile
Z	depth below the sand surface
z_{tip}	depth of pile tip below the sand surface
Δ	axial displacement
Ψ	angle of dilation
δ'	effective angle of interface shearing resistance
ε	strain, with subscripts and superscripts for different modes
$\sigma'_1, \sigma'_2, \sigma'_3$	major, intermediate and minor principal effective stresses, respectively
σ'_r	effective radial stress; $\sigma'_{rm}, \sigma'_{rs}, \sigma'_{rmax}$ are the moving, stationary and maximum values, respectively
σ'_{rc}	equalised radial effective stress
σ'_z	effective vertical stress; $\sigma'_{zm}, \sigma'_{zs}, \sigma'_{zmax}$ are the moving, stationary and maximum values, respectively; σ'_{z0} is the free-field vertical effective stress
σ'_θ	effective circumferential stress; $\sigma'_{\theta m}, \sigma'_{\theta s}, \sigma'_{\theta max}$ are the moving, stationary and maximum values, respectively
ϕ'	effective angle of shearing resistance; ϕ'_{cs} is the critical state value

Introduction

The Bishop Lecture was inaugurated by Technical Committee TC-101 (formerly TC-29) of the International Society for Soil Mechanics and Geotechnical Engineering, honouring the legacy of Professor Alan Bishop (1920–1988), the leading figure of his generation in geotechnical laboratory experiments and equipment design. Bishop was well known for his meticulous attention to detail, analytical rigour

and application of fundamental research in civil engineering practice. His contributions to soil sampling and testing were summarised in the last major keynote he gave, at the Stockholm international conference on soil mechanics and foundation engineering (Bishop, 1981). Similarly admirable attributes were clear in the first Bishop Lecture presented by Tatsuoka (2011), making the invitation to deliver the second lecture both a considerable challenge and a poignant honour for this former student of Bishop and Skempton. The lives, work and archived papers of the latter two pioneers are described together in a website hosted by Imperial College: <http://www.cv.ic.ac.uk/SkemArchive/index.htm>.

The key aim here is to demonstrate the special capabilities and practical value of advanced laboratory testing, mirroring Bishop's work and TC-101's intent in the international symposia it convened in Hokkaido 1994, London 1997, Torino 1999, Lyon 2003, Atlanta 2008 and Seoul 2011. The focus here is on the mechanics of piles driven in sand, a practical problem that was thought fully resistant to 'theoretical refinement' by Terzaghi and Peck (1967). The illustration draws principally on work by the author, his colleagues and research group. In keeping with Bishop's approach, emphasis is placed on integrating laboratory research, analysis and field observation.

The selected topic is significant industrially. Pile stiffness, capacity, cyclic response and long-term behaviour can be critically important to, for example, wind-turbine foundations. However, the key geomechanics issues are complex and cannot be addressed fully or reliably with currently available conventional design tools. Database studies and prediction competitions have quantified the significant biases and scatters associated with conventional practice. The coefficients of variation (CoVs) established by contrasting axial capacity predictions with field tests typically fall around 0.5 to 0.7. Some methods' predictions scatter around half the measurements, while others' tend to double the test values (Briaud and Tucker, 1988). The capacity CoVs can be halved and biases largely eliminated by applying modern 'offshore' methods (Jardine *et al.*, 2005b; Lehane *et al.*, 2005). But displacement predictions remain unreliable under axial, lateral or moment loads. It is also unclear how cyclic or extended loading should be considered (Jardine *et al.*, 2012; Kallehave *et al.*, 2012). Improving understanding and predictive ability will benefit a broad range of applications, especially in offshore energy developments.

The author's research with displacement piles in sand started with highly instrumented field model piles at Labenne (southwest France; Lehane *et al.*, 1993) and Dunkerque (northern France; Chow, 1997), where full-scale testing followed. Some of the full-scale test results are reviewed below before considering new research prompted by some surprising and significant results.

The Dunkerque profile comprises medium-dense fine-to-medium clean silica Holocene marine sand overlain by hydraulic sand fill. Jardine *et al.* (2006), Jardine and Standing (2012) and Rimoy *et al.* (2013) give details of the geotechnical profiles, pile driving records and testing methods. Static and cyclic axial loading tests

were conducted on multiple piles, including six 19.3-m-long 457-mm-o.d. driven steel pipe piles: R1 to R6. Static axial testing involved a maintained-load (ML) procedure where load (Q) was applied initially in 200-kN steps that reduced as the tests progressed. Loads were held constant until creep rates slowed to pre-set limits; the piles took between several hours and 1.5 d to reach failure. More rapid ML tension tests that achieved failure with an hour were also conducted after cyclic loading experiments. Testing rate was found to affect displacements but to have little influence on shaft capacity. The cyclic tests were controlled to deliver approximately sine-wave load variations at ≈ 1 cycle/min.

The static testing investigated, among other factors, the effects of pile age after driving. Figure 1 presents tension tests on three identical piles that were aged for 9 to 235 d before being failed for the first time. The following are noted.

- The load displacement ($Q - \delta$) curves are practically identical up to $Q \approx 1$ MN but then diverge to show marked increases in Q_{ult} (the ultimate load shaft capacity) with age.
- Creep displacements ($d\delta/dt$ when $dQ/dt = 0$) were negligible until $Q > 1$ MN after which creep became progressively more important, finally dominating as failure approached.

Load-displacement behaviour was highly non-linear. The overall pile head secant stiffnesses $k = Q/\delta$ all fell as loading continued with no discernible 'linear-elastic' plateau. This feature is highlighted in Figure 2 with data from 'first time' tension tests on five 'R' piles. The pile stiffnesses, k_i , are normalised by k_{Ref} , the value developed under Q_{Ref} – the first (200 kN) load step. The loads Q are normalised by Q_{Ref} .

An objective assessment was made of how well the Dunkerque pile tests could be predicted by well-qualified engineers by

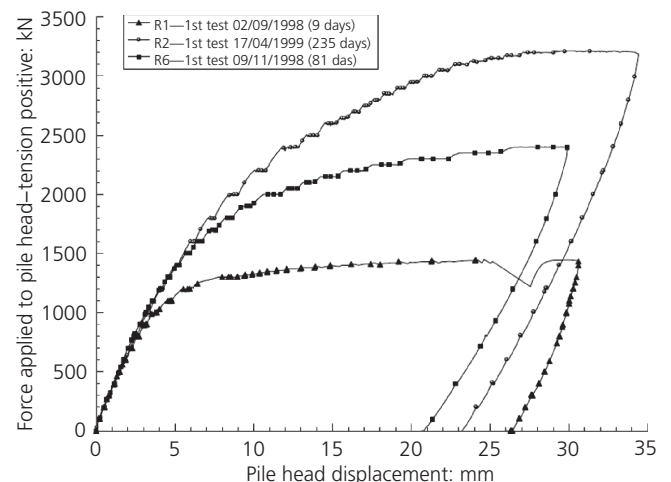


Figure 1. Load-displacement curves from first-time tension failures on Dunkerque piles R1, R2 and R6 (Jardine *et al.*, 2006)

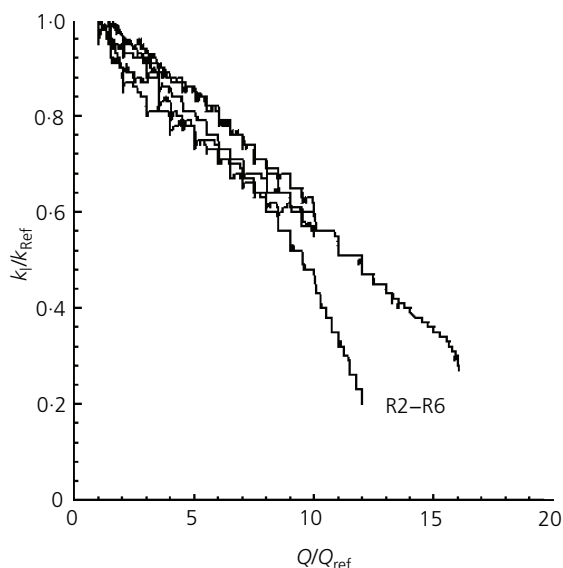


Figure 2. Stiffness load-factor curves from first-time tests at Dunkerque conducted (except R6) around 80 days after driving (Rimoy *et al.*, 2013)

inviting entries to an open competition that concentrated on the static and cyclic tests conducted ≈ 80 d after driving (Jardine *et al.*, 2001a). Over 30 (many prominent) international practitioners and academics took part, sending in a wide spread of predictions. The

axial capacity estimates confirmed the expected CoV of 0.6, as well as significant bias; the stiffness predictions were similarly spread.

No competitor was prepared to predict the cyclic test outcomes; some indicated that cycling should have no effect in clean sand. Figure 3 illustrates the field outcomes in a cyclic failure interaction diagram. The conditions under which 13 tests ended in failure and one developed a fully stable response are summarised by plotting the normalised cyclic load amplitude $Q_{cyc}/Q_{max\ static}$ against the average mid-cycle load $Q_{mean}/Q_{max\ static}$, where $Q_{max\ static} = Q_T$ current tension capacity. If cycling and testing rate had no effect, then failures should lie on the ‘top-left to bottom-right’ diagonal static capacity line: $Q_{cyc} + Q_{mean} = Q_T$ in Figure 3. However, the cyclic test failure points all fell well below this limit, proving a negative impact that grew directly with Q_{cyc}/Q_{mean} . High-level two-way (tension and compression) cycling could halve shaft capacity within a few tens of cycles.

Rimoy *et al.* (2013) discuss the piles’ permanent displacement and cyclic stiffness trends, noting also that their non-linear cyclic stiffnesses depended primarily on Q_{cyc}/Q_T and did not vary greatly with the number of cycles (N) until failure approached. The permanent displacement trends were more complex, depending also on Q_{mean}/Q_T and N . Interactions were seen between the piles’ ageing and cyclic behaviours: low-level cycling accelerated capacity growth, while high-level cycling slowed or reversed the beneficial capacity trend.

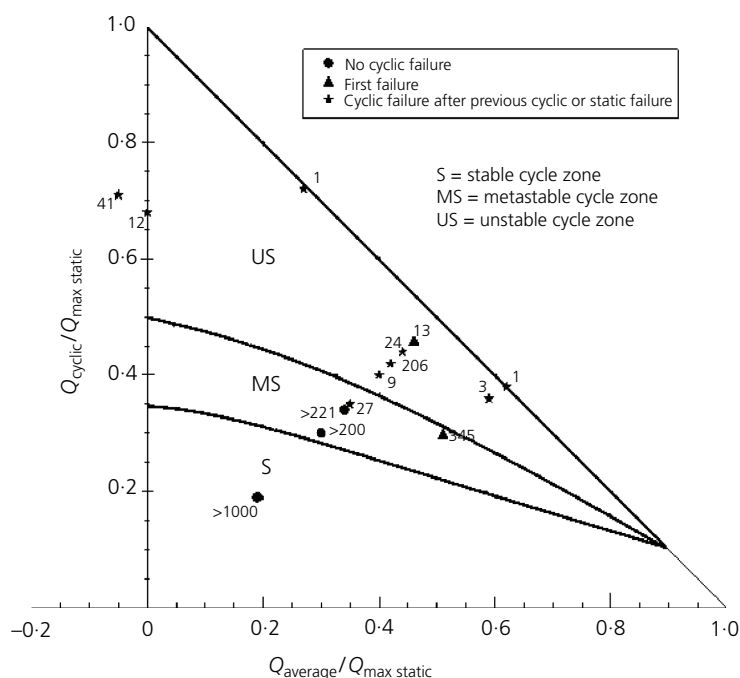


Figure 3. Axial cyclic interaction diagram for full-scale cyclic tests on piles driven at Dunkerque (Jardine and Standing, 2012)

Sand	Specific gravity (G_s)	d_{10} : mm	d_{50} : mm	d_{90} : mm	C_u	e_{max}	e_{min}
Dunkerque	2.65	0.188	0.276	0.426	2.27	0.97	0.51
NE 34	2.65	0.150	0.210	0.230	1.53	0.90	0.51
HRS	2.66	0.190	0.283	0.312	1.64	0.85	0.55
TVS	2.66	0.160	0.250	0.265	1.67	0.85	0.55

Table 1. Index properties of silica sands employed in laboratory studies

Eight research themes are considered below that addressed the shortfalls in understanding as revealed by the Dunkerque tests

1. characterising the sands' true stress–strain relationships, correlating advanced laboratory and in situ measurements
2. checking, through finite-element (FE) modelling, whether laboratory-based non-linear predictive approaches led to better matches with full-scale behaviour
3. stress-path laboratory testing programmes that investigated creep and ageing trends
4. studying the stress conditions imposed by pile installation through highly instrumented calibration chamber (CC) tests
5. grain-crushing and interface-shear zone studies involving high-pressure triaxial, ring-shear and laser particle analysis
6. quantitative checking against advanced numerical analyses
7. model-pile CC cyclic loading experiments
8. cyclic soil element tests to replicate pile loading conditions.

A common theme is that sands show strong non-linearity, plasticity and time dependency from very small strains and have markedly anisotropic properties. It is argued that their overall responses can be understood within a critical state soil mechanics framework, provided that the above features are accommodated and the importance of particle breakage is recognised, especially under high pressures and within abrading shear bands. Space constraints limit the details that can be reported for the various studies cited or the reviews that can be made of research by other groups. However, PhD theses and co-authored articles are cited to cover the main omissions.

Characterising stress–strain behaviour

Bishop recognised at an early stage that geotechnical stress–strain measurements are constrained heavily by equipment capabilities. ISSMGE Technical Committee 29 (now TC-101) was set up to coordinate advanced laboratory developments, leading to a review of apparatus, sensors and testing strategies by Tatsuoka *et al.* (1999). The hydraulic stress-path cells and hollow cylinder apparatus (HCA) advocated by Bishop and Wesley (1974) and Bishop (1981) allow in situ stress conditions to be imposed and studies made of shear strength anisotropy (see e.g. Hight *et al.*, 1983; Shibuya *et al.*, 2003a, 2003b). Burland and Symes (1982) and Jardine *et al.* (1984) went on to show that end-bedding, sample tilting and compliance caused very large errors in conventional geotechnical strain measurements that often led to completely misleading soil stiffness characteristics. Local

strain sensors or dynamic non-destructive techniques are required to obtain representative data (see Tatsuoka *et al.*, 1999).

Laboratory research with such equipment that contributed to the first phase of research that advanced the 'Dunkerque agenda' included the PhD studies of Porovic (1995), who worked with a resonant column (RC)-equipped HCA, and of Kuwano (1999), who developed dual-axis bender elements and enhanced resolution local strain sensors for stress-path triaxial tests. Porovic worked mainly with Ham River sand (HRS), a silica sand graded from Thames Valley gravels that has been tested since Bishop's arrival at Imperial College and is now known generically as Thames Valley sand (TVS) (Takahashi and Jardine, 2007). Kuwano studied Dunkerque sand, spherical glass ballotini (GB) and HRS; Connolly (1998) undertook RC and HCA experiments on Dunkerque sand. The sands were tested saturated after pluviation to the desired initial void ratios; Table 1 and Figure 4 summarise their index properties. Figures 5–7 illustrate the apparatus employed in this first period of 'sand' research. Studies with the TVS and French Fontainebleau NE 34 sands are considered later in the paper.

Kuwano and Jardine (1998, 2002a, 2002b) noted the high sensor resolution and stability required to track sands' stress–strain responses from their (very limited) pseudo-elastic ranges through to ultimate (large strain) failure. Even when the standard deviations in strain measurements fall below 10^{-6} , and those for stresses below

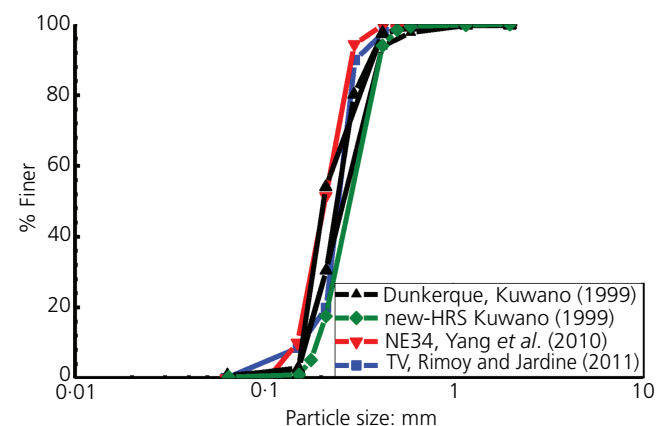


Figure 4. Summary of particle-size distributions for granular media employed in reported laboratory research

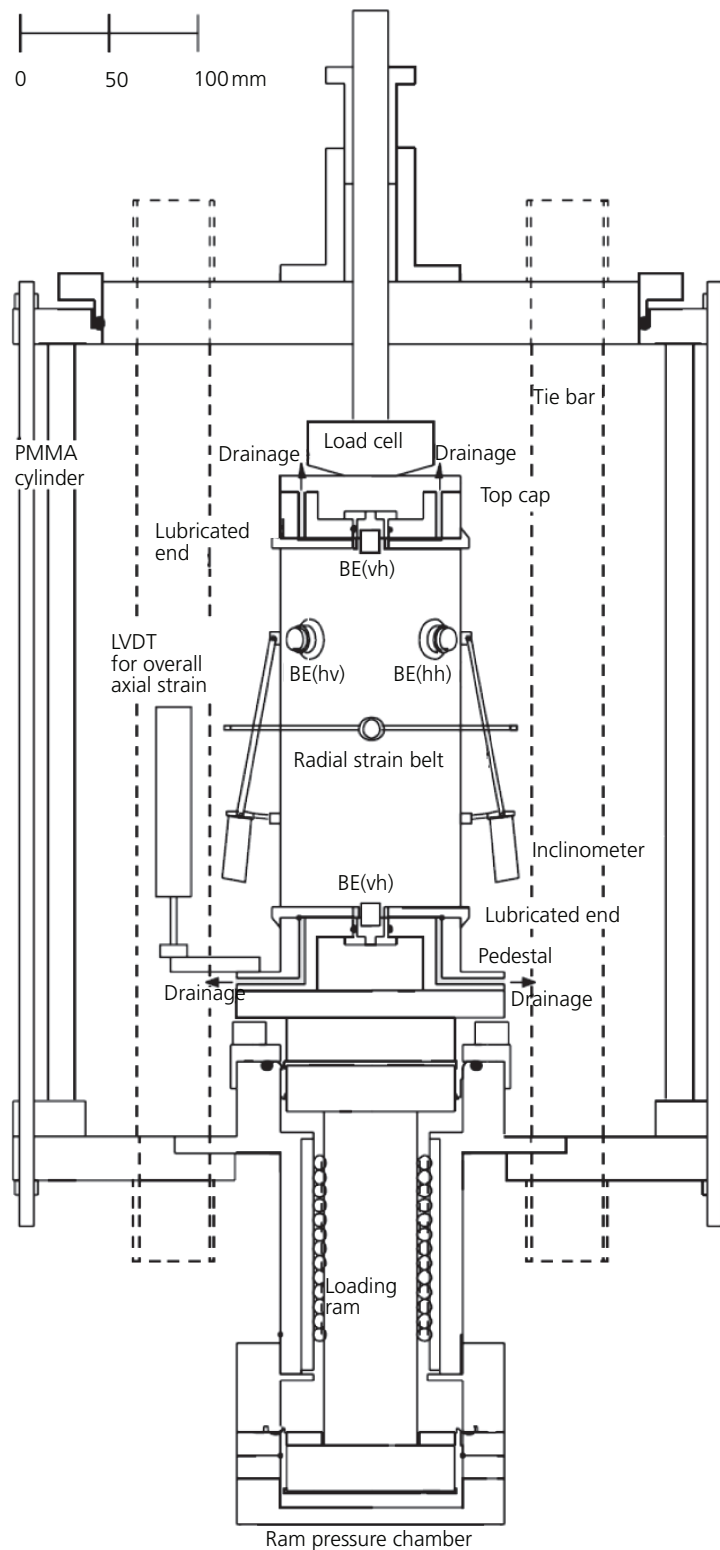


Figure 5. Automated hydraulic stress path triaxial cell for 100-mm-OD specimens employed to investigate non-linear, anisotropic, pressure and time-dependent stiffness of sands

(Kuwano and Jardine, 1998, 2002a). BE, bender element; LVDT, linear variable differential transformer; PMMA, poly(methyl methacrylate)

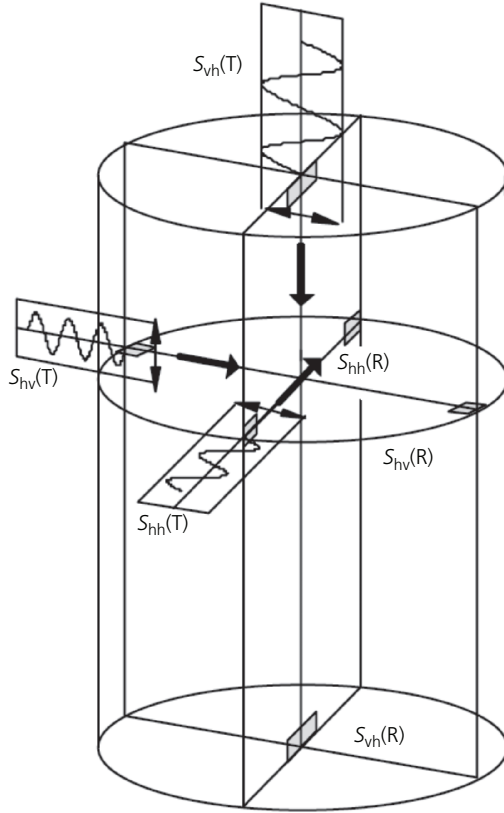


Figure 6. Bender element configuration to investigate stiffness of sands (Kuwano and Jardine, 1998, 2002a)

0.05 kPa, multiple readings and averaging are required to establish initial stiffness trends. Highly flexible stress-path control systems are also essential.

Kuwano and Jardine (2007) emphasise that behaviour can only be considered elastic within a very limited kinematic hardening (Y_1) true yield surface that is dragged with the current effective stress point, growing and shrinking with p' and changing in shape with proximity to the outer Y_3 surface (Jardine, 1992). The last one corresponds to the yield surface recognised in classical critical state soil mechanics. Behaviour within the true Y_1 yield surface is highly anisotropic, following patterns that evolve if K , the ratio of the radial to vertical effective stress ($K = \sigma'_r/\sigma'_z$), changes. Plastic straining commences once the Y_1 surface is engaged and becomes progressively more important as straining continues along any monotonic path. An intermediate kinematic Y_2 surface was identified that marks (a) potential changes in strain increment directions, (b) the onset of marked strain-rate or time dependency and (c) a threshold condition in cyclic tests (as noted by Vucetic (1994)) beyond which permanent strains (or p' reductions in constant volume tests) accumulate significantly.

The Y_3 surface is generally anisotropic. For example, the marked undrained shear strength anisotropy of sands has been identified in

earlier HCA studies (Menkiti, 1995; Porovic, 1995; Shibuya *et al.*, 2003a, 2003b) on HRS. The surface can be difficult to define under drained conditions where volumetric strains dominate. Kuwano and Jardine (2007) suggested that its evolution could be mapped by tracking the incremental ratios of plastic to total strains. They also suggested that the phase transformation process (identified by Ishihara *et al.* (1975), in which specimens that are already yielding under shear in a contractant style could switch abruptly to follow a dilatant pattern) could be considered as a further (Y_4) stage of progressive yielding. Jardine *et al.* (2001b) argue that the above inelastic features can be explained by micro-mechanical grain contact yielding/slipping and force chain buckling processes. The breakage of grains, which becomes important under high pressures, has also been referred to as yielding (see Bandini and Coop, 2011; Muir-Wood, 2008).

HCA testing is necessary to investigate stiffness anisotropy post- Y_1 yielding (Zdravkovic and Jardine, 1997). However, cross-anisotropic elastic parameter sets can be obtained within Y_1 by assuming rate independence and combining very small strain axial and radial stress probing experiments with multi-axis shear wave measurements. Kuwano (1999) undertook hundreds of such tests under a wide range of stress conditions, confirming the elastic stiffness (Equations 1–5). Ageing periods were imposed in all tests before making any change in stress-path direction to ensure that residual creep rates reduced to low proportions (typically $<1/100$) of those that would be developed in the next test stage. Note that the function used to normalise for variations in void ratio (e) is $f(e) = (2.17 - e)^2/(1 + e)$.

1. $E_u = f(e) \cdot A_u \cdot (p'/p_r)^{B_u}$
2. $E'_v = f(e) \cdot A_v \cdot (\sigma'_v/p_r)^{C_v}$
3. $E'_h = f(e) \cdot A_h \cdot (\sigma'_h/p_r)^{D_h}$
4. $G_{vh} = f(e) \cdot A_{vh} \cdot (\sigma'_v/p_r)^{C_{vh}} \cdot (\sigma'_h/p_r)^{D_{vh}}$
5. $G_{hh} = f(e) \cdot A_{hh} \cdot (\sigma'_v/p_r)^{C_{hh}} \cdot (\sigma'_h/p_r)^{D_{hh}}$

The terms A_{ij} , B_{ij} , C_{ij} and D_{ij} are non-dimensional material constants and p_r is atmospheric pressure. With Dunkerque sand, the values of B_u and the sum $[C_{ij} + D_{ij}]$ of the exponents applying to Equations 1 to 5 fell between 0.5 and 0.6. The equations are evaluated and plotted against depth in Figure 8 adopting Kuwano's sets of coefficients (A_{ij} , B_{ij} , C_{ij} and D_{ij}) combined with the Dunkerque unit weight profile, water table depth and an estimated $K_0 = 1 - \sin \phi'$ for the normally consolidated sand. A single void ratio (0.61) has been adopted for this illustration that matches the expected mean, although the cone penetration test (CPT) q_c profiles point to significant fluctuations

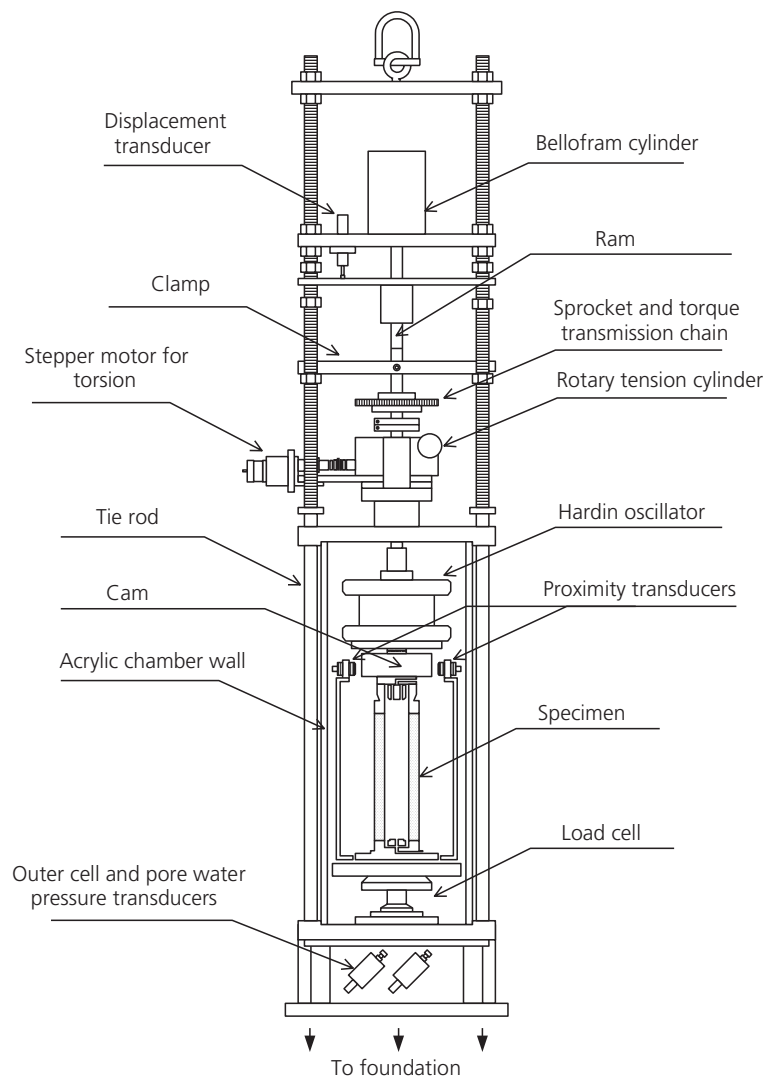


Figure 7. Schematic arrangements of resonant-column HCA system employed to test sands (Nishimura *et al.*, 2007)

with depth in void ratio and state. Also shown is the in situ G_{vh} profile measured with seismic CPT tests and DMT tests conducted by the UK Building Research Establishment (Chow, 1997).

The sand's marked quasi-elastic stiffness anisotropy is clearly evident. Under an overconsolidation ratio (OCR) of 1, K_0 conditions, the E'_v/E'_h ratio is ~ 1.7 , while $E'_v/G_{vh} \sim 3.9$. The pattern of anisotropy varies with OCR and applied K ratio. The field quasi-elastic seismic CPT G_{vh} profile matches that from RC-equipped HCA tests by Connolly (1998) and falls marginally ($\approx 12\%$) above Kuwano's bender element G_{vh} profile.

The Dunkerque HCA and triaxial tests demonstrated how stiffness anisotropy persists after Y_1 yielding and degrades with strain.

Figure 9 illustrates the shear stiffness trends from undrained triaxial compression, triaxial extension, which should converge within the very small strain elastic region, along with torsional shear (HCA) experiments. The stiffnesses are normalised by p' , as the stress level exponent was higher over this range than in the ' Y_1 bubble' and approaches unity at 0.1% . The tests on K_0 consolidated samples were all sheared from $p' = 200$ kPa at OCR = 1. Higher stiffness ratios were developed in other tests conducted at OCR = 2 (Jardine *et al.*, 2005a).

Advanced laboratory testing offers the only means of making such accurate measurements of the non-linear, time-dependent and anisotropic behaviour of geomaterials and how they respond to the general stress paths applied by field foundation loading.

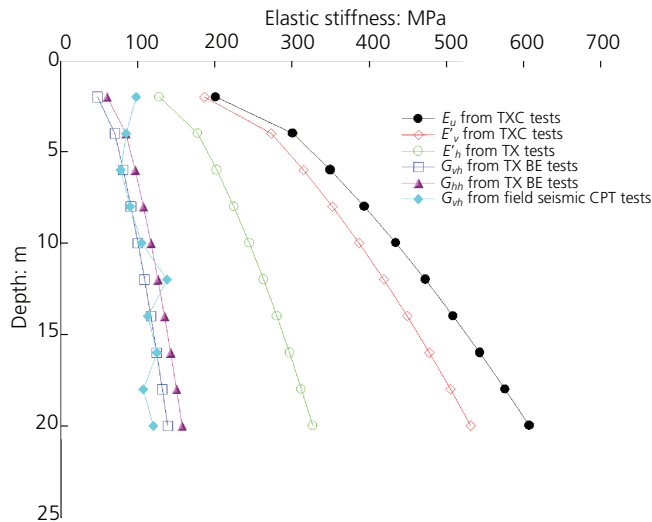


Figure 8. Quasi-elastic stiffness component profiles at Dunkerque. Seismic CPT G_{vh} profile also shown (Jardine *et al.*, 2005a)

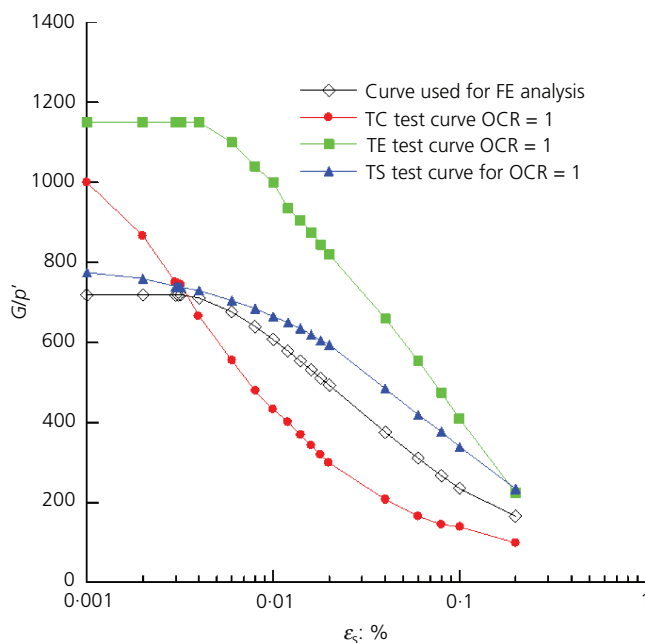


Figure 9. Experimental shear stiffness–shear strain invariant curves with ICFEP analysis curve (Jardine *et al.*, 2005a)

Comparing laboratory-based predictions with field behaviour

The degree of match between laboratory and field stiffness trends was investigated through fully non-linear FE simulations with the code ICFEP (Potts and Zdravkovic, 1999, 2001). Several of the ‘80-day’ Dunkerque tests were modelled. The key aspects emphasised by Jardine *et al.* (2005a) were as follows

- meshing to accommodate eight ‘density’ sub-layers, based on pile-specific CPTs, with bulk unit weights varying above and below the water table from 17.1 to 20 kN/m³
- following triaxial and direct-shear tests by Kuwano (1999), peak ϕ' values ranging between 35° and 32° for the dense-to-loose sand sub-layers, dilation angles $\psi = \phi'/2$ and a single pile–sand interface shear angle $\delta' = 28^\circ$
- non-linear shear and bulk stiffnesses curves fitted to laboratory test data with simple effective stress functions from Jardine and Potts (1988) (after Jardine *et al.*, 1986)
- noting that pile loading imposes vertical shearing on the shaft and axial loading at the base, a normalised ‘dense’ shear stiffness relationship was selected that was biased towards the OCR = 1 torsional shear HCA curve in Figure 9
- a normalised ‘dense’ bulk stiffness–volume strain curve fitted from Kuwano’s swelling/re-compression tests and adjusted to meet K_0 swelling effective stress–path checks
- softer stiffness curves (factored by 0.8) for the thin ‘organic’ loose sub-layers identified from the CPT traces
- effective stress regimes that were simplified to give constant stress ratios σ'_r / σ'_{z0} near the pile shaft within each block (where σ'_{z0} is the undisturbed vertical effective stress) that decayed monotonically out to far-field K_0 values. The shaft shaft radial stresses were derived following the Jardine *et al.* (2005b) procedures, adjusted to account for the piles’ 80-d ages. Estimates for how $\sigma'_\theta / \sigma'_{z0}$ and σ'_z / σ'_{z0} varied at points away from the shaft could only be based on judgement.

Figure 10 compares the non-linear FE analysis with the ‘end-of-increment’ Q – δ envelope curve for pile R6 shown in Figure 1. The pile’s overall capacity was well predicted, as were pile head

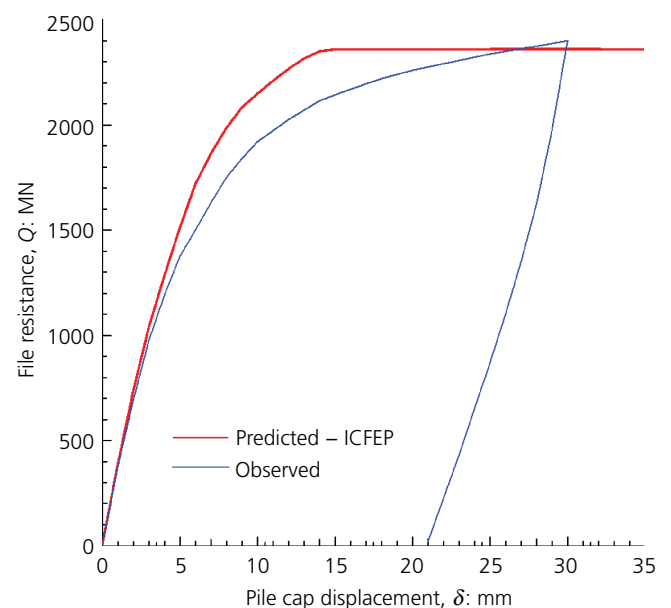


Figure 10. Predicted and (end of load stage) measured load–displacement curves: 80-d test on R6 (Jardine *et al.*, 2005a)

movements up to half Q_T . The approach gave broadly successful numerical predictions for all piles' initial stiffness responses under compression and cyclic loading as well as insights into the shaft shear stress distributions, the strain fields and potential group interaction effects (see Jardine and Potts, 1988). Lateral/moment loading responses and group analyses may be considered through three-dimensional approaches (Potts and Zdravkovic, 2001). Stiffness anisotropy can be addressed within the same non-linear framework (Addenbrooke *et al.*, 1997). However, the time-independent FE analysis could not predict the large creep movements that developed in the field, following a stick-slip pattern, as failure approached. New research was required into several aspects of behaviour

- the time-dependent processes of ageing and creep
- the stress regime set-up in the soil mass by driving
- how cycling affects stiffness, capacity and permanent displacements.

Investigating time-dependent behaviour

Laboratory research designed to investigate the time-dependent behaviour of piles driven in sand is considered below. However, it is noted first that Bishop also recognised the need to consider time effects carefully. Late in his career, he designed elegant triaxial cells that used long, soft, adjustable mechanical springs to provide uninterrupted and easily controlled long-term deviator force actuators. Davies (1975) reports long-term tests on natural clays conducted with several of the cells described by Bishop (1981). Also noted is Tatsuoka's (2011) very thorough exploration of time dependency in his Bishop Lecture.

Sand properties are often considered independent of rate and time. However, long-term field observations reveal that settlements can double or more under shallow foundations on sand through long-term creep (Burland and Burbridge, 1985; Frank, 1994; Jardine *et al.*, 2005a). Kuwano and Jardine (2002a) reviewed the stringent experimental requirements necessary for investigating the creep of sands through triaxial tests: very stable high-resolution, local strain sensors are required, as are high-quality pressure and temperature control systems. Membrane penetration has to be considered carefully; lubricated low-friction sample ends are also recommended.

Kuwano and Jardine illustrated aspects of short-term creep behaviour through tests on saturated HRS and GB specimens prepared at various initial densities. The tests advanced along the drained 'near isotropic' and ' K_0 ' stress paths set out in Figure 11 at mean stress rates dp'/dt of around 100 kPa/h. The paths were punctuated, as indicated, by periods 'C' where samples were allowed to creep under constant stresses for several hours.

Pressure-dependent elastic stiffness functions (Equations 1–5) established from parallel tests were integrated to calculate the contribution of elastic straining $d\epsilon^e$ to the overall total (elastic-plastic) strains $d\epsilon^{ep}$ developed over each test stage. Figure 12

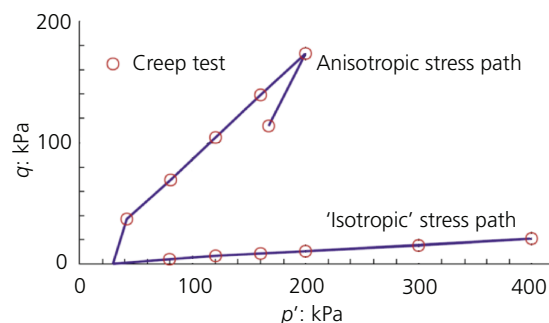


Figure 11. Effective stress paths followed in drained 'creep' stress path tests on HRS and GB specimens (Kuwano and Jardine 2002a)

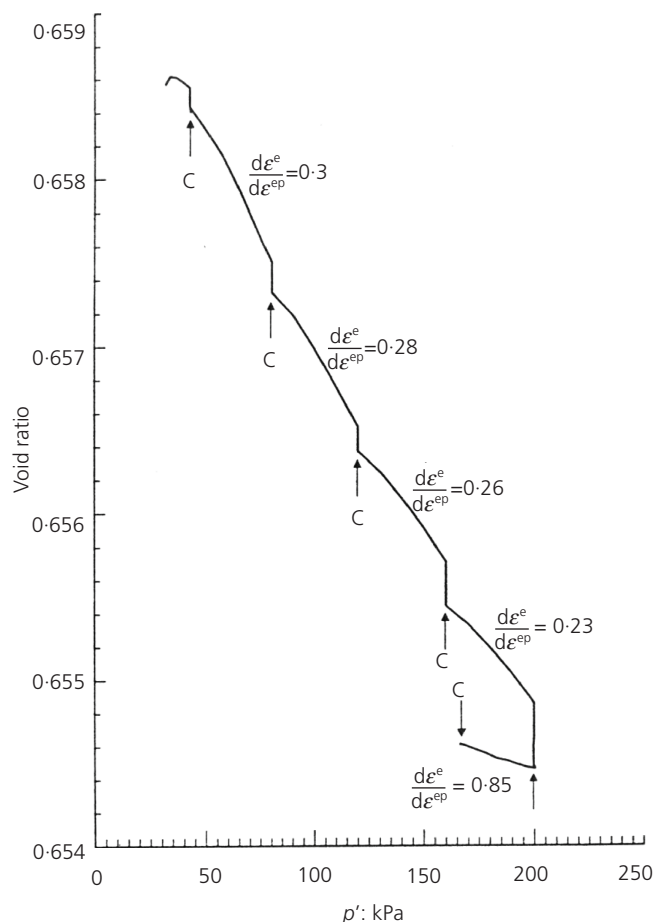


Figure 12. Overall e – p' relationship of K_0 compression tests on medium-dense HRS, showing ratios $d\epsilon^e/d\epsilon^{ep}$ of elastic to plastic strains and time-dependent compression over creep stages (C) (Jardine *et al.*, 2001b)

illustrates the void ratio (e)– p' relationships obtained from the K_0 normally consolidated stage of test H4 on an HRS specimen prepared to the average relative density applying to the Dunkerque field profile. The average $d\epsilon^e/d\epsilon^{ep}$ ratios applying during loading

($dp'/dt > 0$) stages fall from 0.30 to 0.23 as loading continues, indicating an increasingly plastic response. However, the additional plastic strains developed during creep stages (where $dp'/dt = d\epsilon^c/d\epsilon^{cp} = 0$) become progressively more significant as loading continued and contributed the major part of the overall 'consolidation' strains (ϵ_{con}) by the end of the test.

The latter point is emphasised in Figure 13 by plotting the proportion of the overall consolidation strain ϵ_{con} that was due to creep ϵ_{cre} during the pause periods of test H4 and two otherwise identical experiments on loose HRS and medium-dense, nearly spherical, GB. Overall, the relative contribution of creep appears to (a) grow with stress level and grain angularity and (b) fall with initial void ratio, OCR and stress ratio $K = \sigma'_3/\sigma'_1$. Jardine and Kuwano (2002a) also show that creep strain rates decay inversely with time over the first few hours. Jardine *et al.* (2001b) offer observations on the micro-mechanical processes that control the experimental behaviour seen in triaxial and HCA tests.

It is argued later that the kinematic conditions applying close to the shafts of displacement piles impose approximately constant volume conditions. The constant volume creep response is illustrated in Figure 14 by showing first the effective stress path followed by an isotropically normally consolidated medium-dense HRS specimen that was allowed to creep to a stable condition before being sheared undrained in triaxial compression under a constant axial rate of 0.5%/h, punctuated by seven constant stress creep pauses.

Figure 15 presents the strain-time ($\epsilon-t$) responses observed over the undrained creep stages. Note the following: (a) very little creep before the Y_2 surface is engaged (at $q \approx 30$ kPa $\approx 0.15p'$), (b) the post Y_2 family of $\epsilon-t$ curves in which creep rates grow exponentially with q , (c) a marked softening of the stress-strain

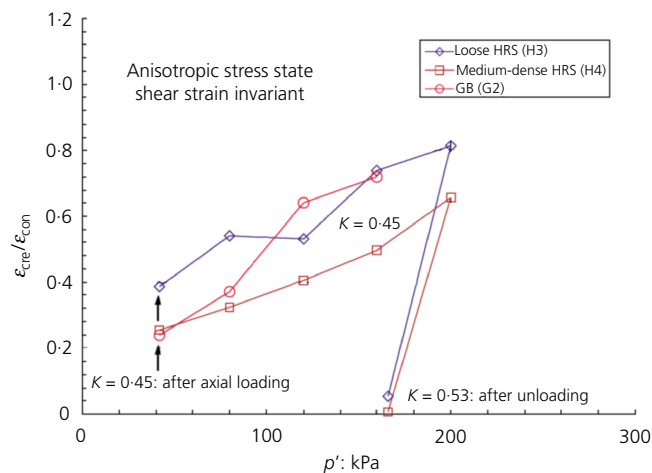


Figure 13. Ratios of creep strains ϵ_{cre} to total consolidation axial strains ϵ_{con} in K_0 compression tests on HRS and GB specimens following the paths shown in Figure 11 (Kuwano and Jardine 2002a)

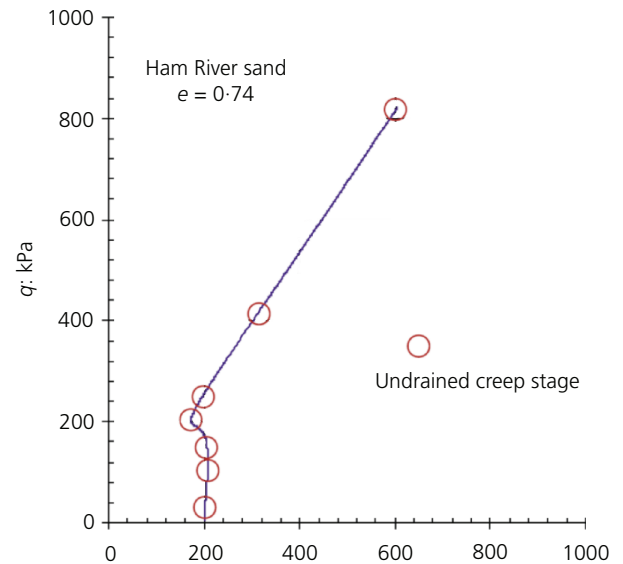


Figure 14. Effective stress paths followed in undrained 'creep' stress-path test H2 on HRS specimen (Kuwano and Jardine 2002a)

response and anti-clockwise effective stress-path rotation at the Y_3 stage (when $q \approx 160$ kPa), (d) the Y_4 phase transformation point (at $q \approx 200$ kPa, $p' \approx 170$ kPa when q/p' approaches $M_{critical}$ state), and (e) a second family of $\epsilon-t$ curves applying post Y_4 showing creep rates that grow slowly as q increases very significantly.

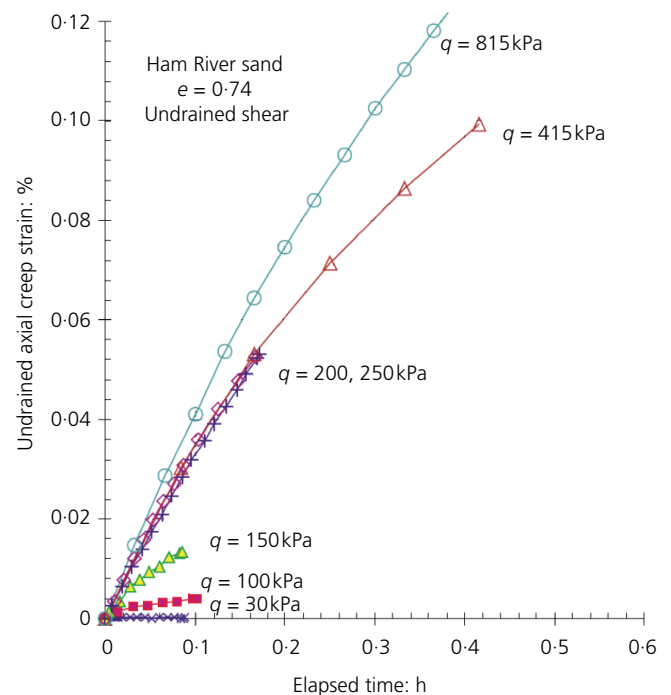


Figure 15. Strain-time paths followed in seven undrained 'creep stages' of stress-path test H2 on HRS specimen identified in Figure 14 (Kuwano and Jardine 2002a)

The triaxial trends bear out the pile load-test trends in Figure 1 for ‘creep yielding’ (noted at $Q \approx 1$ MN with the R piles) followed by creep rates that rise rapidly with each subsequent load step. It is clear that time dependency has an important impact on both laboratory and field pre-failure behaviour.

Longer-term triaxial stress-path experiments designed to investigate the interactions between pile ageing and low-level cyclic loading noted by Jardine *et al.* (2006) are considered next. Rimoy and Jardine (2011) report suites of tests conducted on medium-dense TVS sand (see Figure 4 and Table 1) in the advanced hydraulic stress-path cell system illustrated in Figure 16.

Figure 17 sets out the effective stress paths followed by Rimoy and Jardine (2011), indicating the pause points at which drained creep straining was observed for 2- to 4-d durations under constant

stresses – either in an undisturbed ‘true’ state or in combination with low-level drained cyclic loading.

Figures 18 and 19 show the volumetric and shear strain invariant responses observed during ‘true’ creep at three p' levels, showing stable and consistent trends. While the invariant shear strain increased monotonically with time and p' level, the volumetric trends reversed when ε_s exceeded $\approx 0.015\%$ after several hours and diverged strongly from the initially near K_0 pattern, where $d\varepsilon_s/d\varepsilon_{vol} = 1$ and $d\varepsilon_s/d\varepsilon_{vol} = 2/3$ for zero radial strains. Monotonically continuing shear distortion led to sharp rotation of strain increment directions, eventually establishing a steady trend for $d\varepsilon_s/d\varepsilon_{vol} \approx -1$.

This interesting kinematic yielding trend, which was not apparent in the shorter-duration creep tests investigated by Kuwano (1999), can be seen as the (stationary) effective stress point engaging

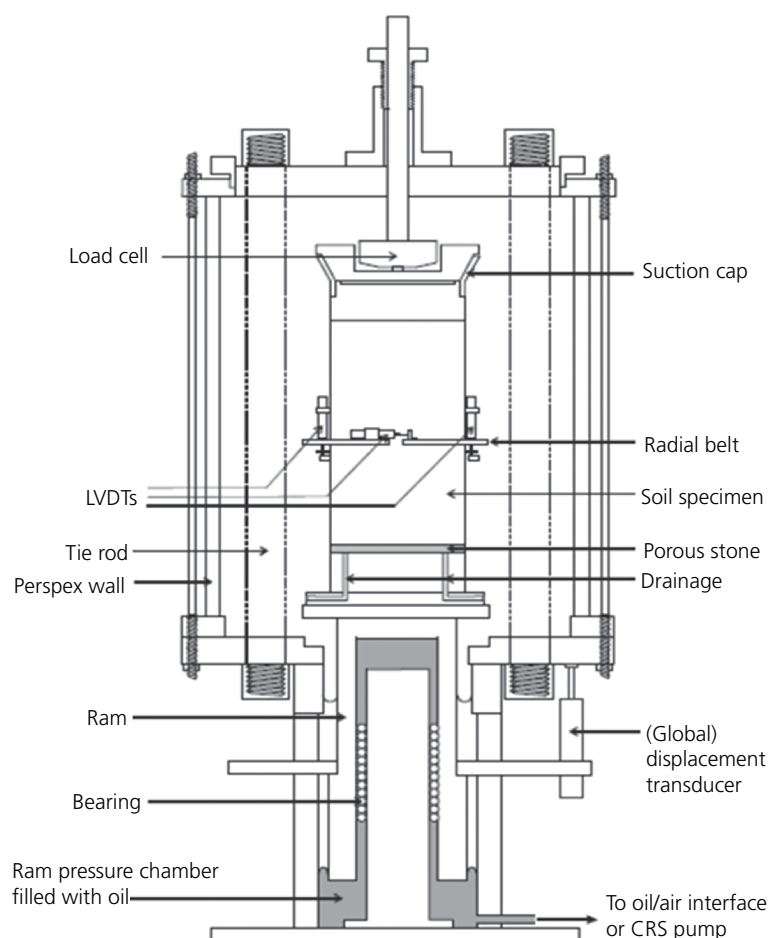


Figure 16. Advanced IC automated hydraulic stress-path triaxial apparatus and instrumentation for 100-mm-OD specimens described by Gasparre *et al.* (2007) and employed by Rimoy and Jardine (2011). LVDT, linear variable differential transformer

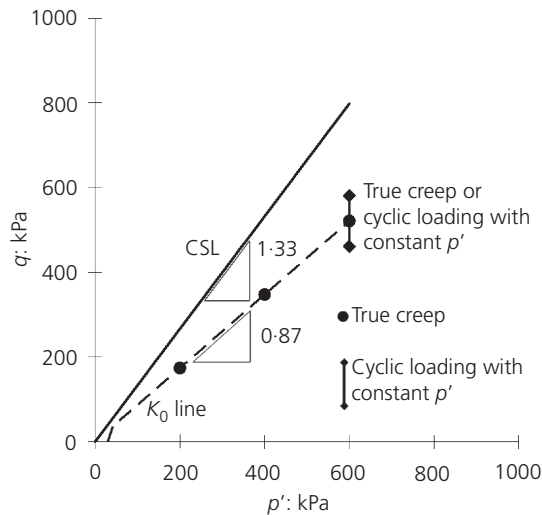


Figure 17. Effective stress paths followed in creep-cyclic interaction triaxial tests on TVS specimens after Rimoy and Jardine (2011)

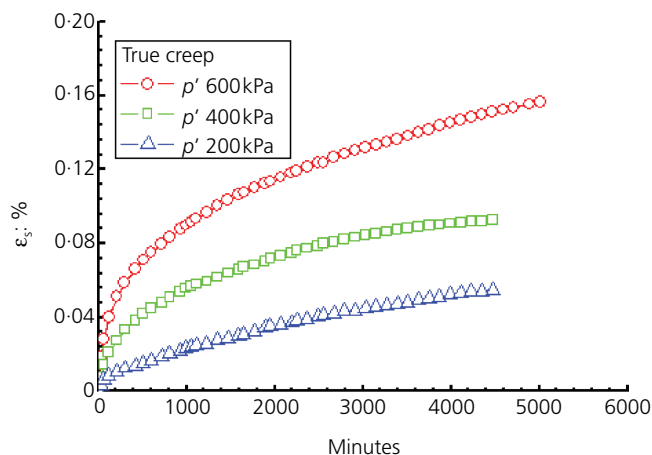


Figure 18. Shear strain invariant-time trends followed in 'true creep' stages of stress-path triaxial tests on TVS specimens (Rimoy and Jardine, 2011)

a kinematic yield surface that is moving with respect to time or strain rate. Given the final strain increment direction, it appears that the Y_2 'bubble' has moved rightwards with time and the fixed effective stress point has engaged its leftward limit. Under strain-controlled K_0 conditions, any radial dilation has to be suppressed, leading to radial effective stresses and increases in K_0 . Bowman and Soga (2005) noted similar features in independent experiments, speculating that this feature might play a significant role in pile capacity growth with age.

Rimoy and Jardine (2011) also explored the interactions between creep and low-level cyclic loading. Figure 20 plots the ε_s - t trends

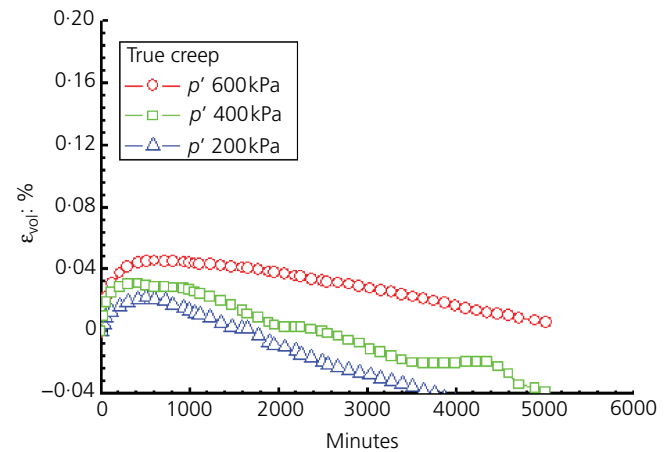


Figure 19. Volume strain-time trends followed in 'true creep' stages of stress-path triaxial tests on TVS specimens (Rimoy and Jardine, 2011)

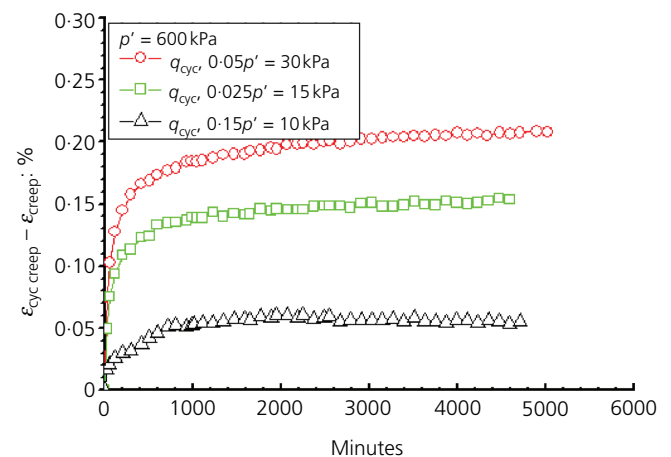


Figure 20. Shear strain invariant-time trends from cyclic stress-path tests on TVS specimens conducted at 1 cycle/min (Rimoy and Jardine, 2011)

from tests where the deviator stresses q were varied by one cycle per minute (as in the Dunkerque pile tests) while keeping p' constant. The cycling commenced as soon as the stress path arrived at the desired p' level with (half peak-to-trough) amplitudes q_{cyc} equal to 5%, 10% and 15% of p' . The cyclic tests showed augmented rates of permanent strain development, which in the $q_{cyc} = 0.15p'$ test doubled those seen in the 'true creep' experiment. Other experiments showed that prior drained ageing (creep) or overconsolidation slows permanent strain development.

More complex interactions are revealed by plotting ε_s against ε_{vol} in Figure 21. It can be seen that cyclic loading retards the shift from contractive-to-dilative volumetric response. The time-dependent Y_2 point is pushed forward in terms of both creep duration and

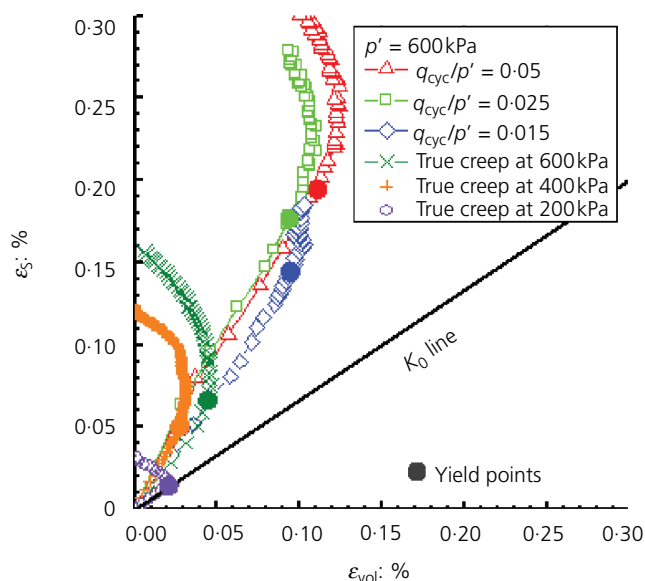


Figure 21. Shear strain invariant-volume strain trends followed in creep-cyclic interaction stress-path triaxial tests on TVS specimens (Rimoy and Jardine, 2011)

shear strain developed. Low-level cyclic loading does not simply accelerate creep. It also holds back and probably expands the time-dependent kinematic Y_2 surface. It is interesting that low-level cycling enhances pile capacity growth, suggesting that the

delayed dilation mechanism may be playing a more complex role than had been appreciated in pile axial capacity growth with time. The laboratory tests provide critical data against which new time-dependent and kinematic yielding models may be tested.

Establishing the stress conditions developed around laboratory model displacement piles

The laboratory element testing described above reveals highly non-linear, anisotropic, time-dependent and inelastic stress-strain behaviour. These features depend critically on the samples' effective stress states and stress histories. However, the lack of knowledge regarding the effective stress regime setup in the surrounding sand mass when piles are driven called for further research. Calibration chamber experiments offered the promise of new insights that would help to link laboratory element tests and field pile behaviour.

Laboratory CCs were developed originally to aid field standard penetration test and CPT interpretation in sands. Multiple test series have been conducted on uniform (well-characterised) sand masses under controlled pressure or displacement boundary conditions (see e.g. Baldi *et al.*, 1986; Huang and Hsu, 2005). Laboratory CCs also provide scope for measuring stresses in soil masses around model piles (during and after installation) and also allow 'post mortem' sand sampling; these activities are far more difficult to perform in field tests.

Joint research with Professor Foray's group at the Institut National Polytechnique de Grenoble (INPG) has included a comprehensive

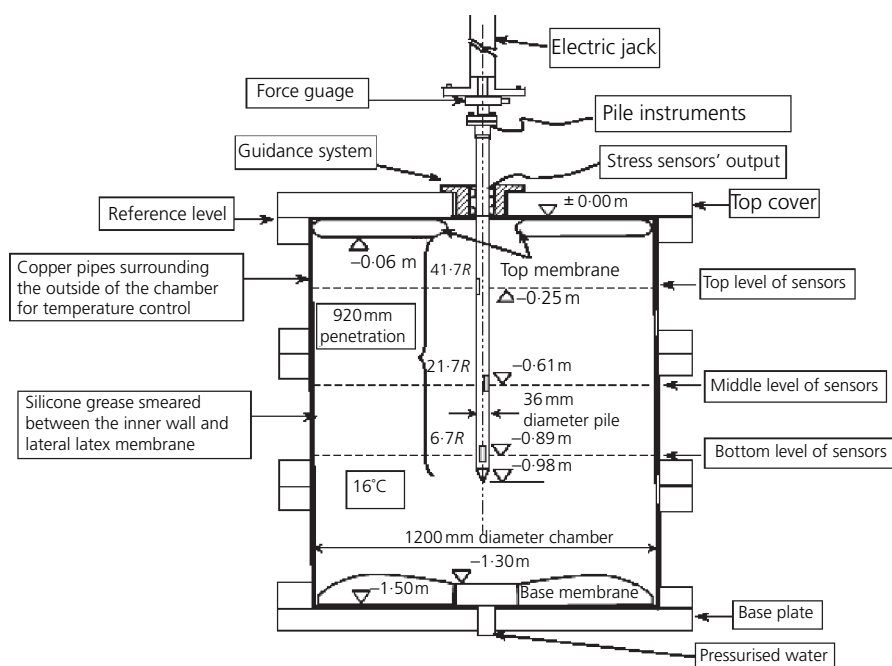


Figure 22. Schematic arrangements for fully instrumented environmentally controlled calibration chamber mini-ICP tests (Jardine *et al.*, 2009)

study of the stresses developed around closed-ended displacement piles. Cone-ended stainless-steel, moderately rough ($R_{CLA} \approx 3 \mu\text{m}$) piles with 18-mm-radii R (the same as a standard CPT probe) (mini-Imperial College Pile (ICP)) were penetrated 1 m into dry, pressurised and highly instrumented medium-dense Fontainebleau NE 34 silica sand. NE 34 has the index properties shown in Figure 4 and Table 1 and is broadly comparable to the earlier discussed Dunkerque, HRS and TVS sands. Jardine *et al.* (2009) detail the general experimental arrangements outlined in Figure 22. Cyclic jacking, with full unloading between strokes, was imposed to simulate pile driving installation.

The mini-ICP instrumentation included reduced-scale surface stress transducers that measure radial and shear shaft stresses at radial

distances $r/R = 1$ from the pile axis at three levels, as shown in Figure 23. Measurements were also made of σ'_z , σ'_θ and σ'_r at two to three levels in the sand mass at radial distances between 2 and 20R from the pile axis using miniature soil sensors. Zhu *et al.* (2009) focus on the sensors' calibrations and performance, emphasizing the care needed to address non-linear and hysteretic cell action.

Upper annular membranes were used to apply a surcharge pressure of $\sigma'_{z0} \approx 150 \text{ kPa}$ to the sand mass. Separate CPT tests established q_c profiles for various boundary conditions. As shown in Figure 24, two alternative membrane designs gave quasi-constant CPT trace sections with $q_c = 21 \pm 2 \text{ MPa}$, although this was achieved at a shallower depth with the smaller-i.d. membrane. Also shown is the q_c profile predicted by Zhang *et al.* (2013) that is discussed later.

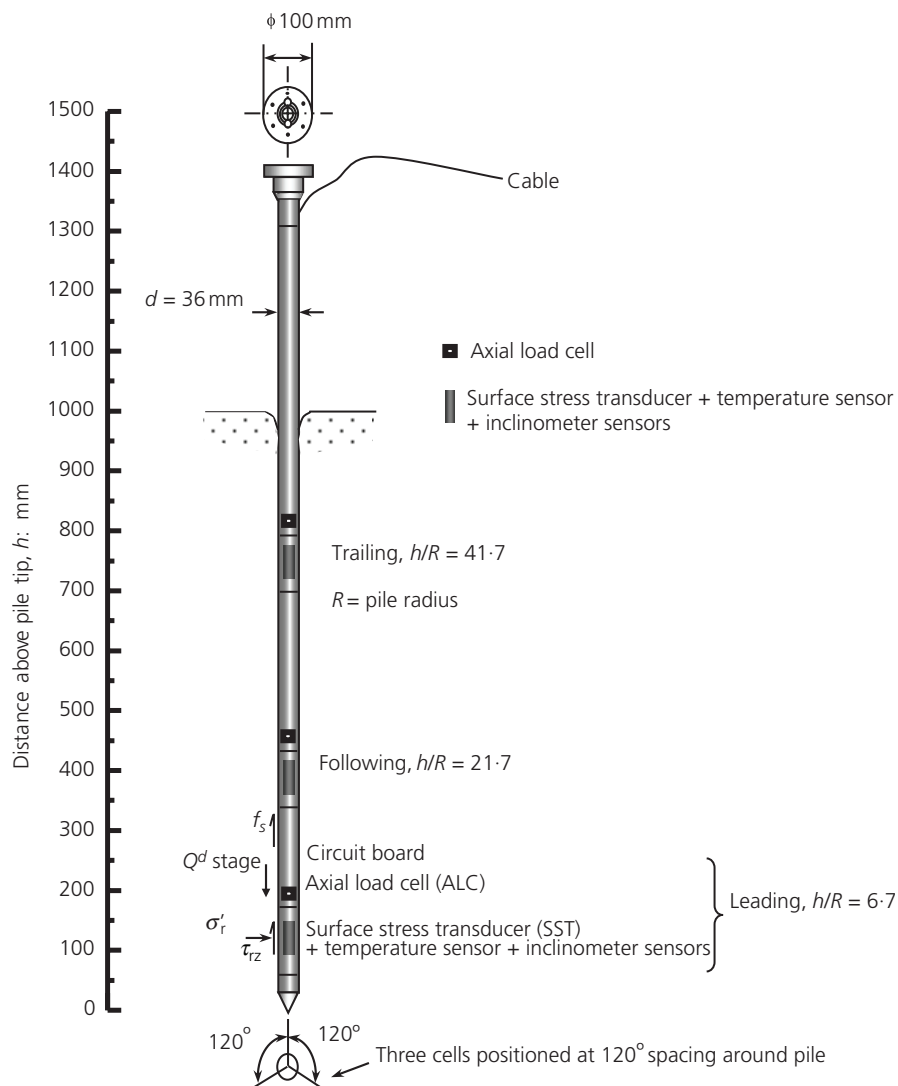


Figure 23. Schematic of laboratory mini-ICP pile with three levels of surface stress transducers, as well as axial load cells, temperature sensors and inclinometers (Jardine *et al.*, 2009)

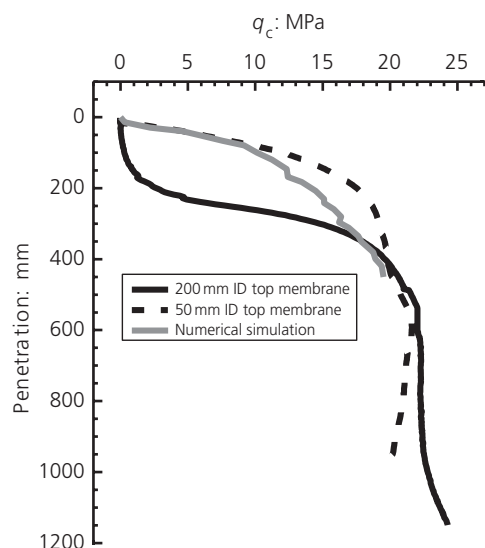


Figure 24. Measured and predicted CPT q_c profiles with alternative CC top membranes (Jardine *et al.*, 2013a; Zhang *et al.*, 2013)

Rimoy (2013) describes more recent experiments with the same equipment, noting that axial capacities from multiple load tests agree encouragingly well with predictions made with the ‘field-calibrated’ capacity approach outlined by Jardine *et al.* (2005b), which gave good results for the Dunkerque field tests.

Jardine *et al.* (2013a, 2013b) report and interpret the measurements made during installation, referring to these as the ‘mini-ICP data set’. Pile penetration invoked extreme stress changes in all three normal stress components and significant stress changes out to $r/R > 33$. Synthesising thousands of stress measurements led to contour plots for the stress components including the radial stress set given in Figure 25 derived for ‘moving’ steady penetration (σ'_{rm}) stages. The results are normalised for local q_c and plotted with cylindrical coordinates defined relative to the pile tip. Normalised vertical distances (h/R) above are positive; points below have negative h/R . Separate plots were derived for ‘stationary’ pause radial stresses (σ'_{rs} points) recorded when the pile head was unloaded fully. Moving and stationary contour sets were also reported for the vertical (σ'_z) and hoop (σ'_θ) stresses.

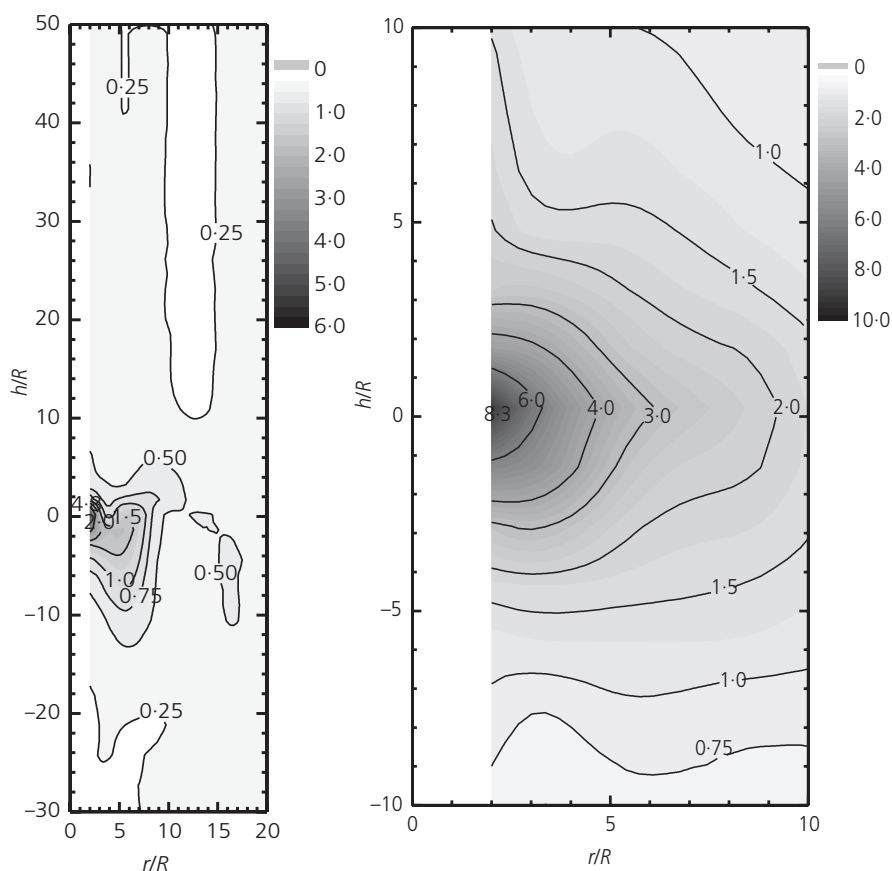


Figure 25. Contoured radial stresses around a penetrating conically tipped pile (normalised by CPT q_c and shown in %) as measured in laboratory CC tests (Jardine *et al.*, 2013b)

The contour plots indicate intense stress concentrations emanating from the pile tip. Radial stress maxima exceeding 15% q_c were observed at $h/R \sim 0.5$, $r/R = 2$, during penetration, while the ‘zero-load’ stationary values were two to three times smaller. Yang *et al.* (2010) describe how an active failure develops beneath the advancing tip where, on average, $\sigma'_{zm}/q_c = 1$, $\sigma'_{rm} = \sigma'_{\theta m} = K_A \sigma'_{zm}$ and $K_A = \tan^2(45 + \phi'/2)$. Close analysis of the ‘moving’ and ‘stationary’ stress measurements shows the greatest divergence near the tip ($-5 < h/R < 3$) where substantial differences extend to $r/R = 10$. Variation is mainly restricted to the $r/R < 2$ region at higher levels on the shaft.

The most reliable observations of how stresses vary with r/R (at set h/R values) were developed from the end-of-installation measurements. The stationary σ'_r and σ'_θ profiles interpreted by Jardine *et al.* (2013b) for four h/R values are presented in Figures 26 and 27. Note that the final radial stresses develop maxima away

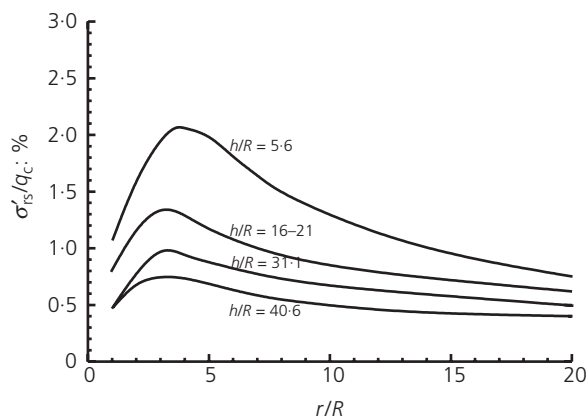


Figure 26. Radial profiles of radial stresses measured around a model pile after installation in a laboratory CC (normalised by CPT q_c and shown in %) (Jardine *et al.*, 2013b)

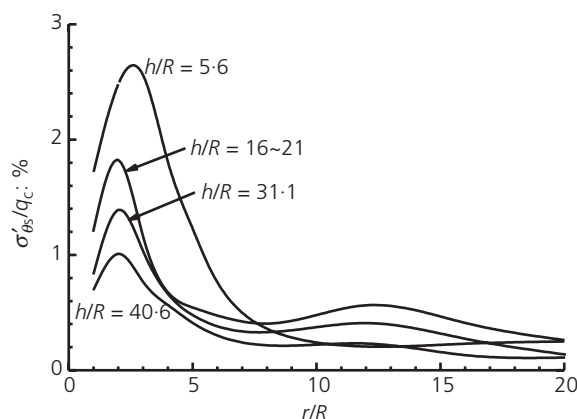


Figure 27. Radial profiles of hoop stresses around a model pile after installation (normalised by CPT q_c and shown in %) (Jardine *et al.*, 2013b)

from the shaft, between $2 < r/R < 4$; σ'_θ must vary steeply with r/R to maintain equilibrium and give $\sigma'_\theta > \sigma'_r$ close to the shaft.

The above effective stress profiles, taken in combination with the time-dependent behaviour discussed above (‘Investigating time-dependent behaviour’), have the potential to explain the marked field capacity–time trends illustrated in Figure 1 by the Dunkerque tension pile loading tests.

Laboratory testing and fabric studies to investigate particle crushing and interface shear processes

The CC model pile tests also revealed the important micro-mechanical features illustrated schematically in Figure 28. Post-mortem sampling revealed a clearly differentiated grey-coloured interface shear band (zone 1) around the shaft, as shown in Figure 29. The following paragraphs report the insights provided by laboratory studies into the breakage phenomena. Their influence on the stress regime developed around the penetrating pile is considered later.

Yang *et al.* (2010) describe how the three concentric micro-fabric zones were defined, their diameters measured and samples comprising only a few grams analysed with a QicPic laser-based imaging system. The last one can resolve particles with sizes between a few micrometres and several millimetres. Care is needed to relate the various optical definitions of grain size with sieve analyses, and the Feret minimum optical measurement correlated best. The grey zone 1 band contained the highest fraction of modified, partially crushed sand. Fracture commenced beneath the active pile tip area once $q_c > 5$ MPa. The high-pressure oedometer test on NE 34 sand illustrated in Figure 30 indicates that large-scale breakage is delayed until $\sigma'_z > 10$ MPa under K_0 conditions.

Yang *et al.* (2010) tested material taken from the zone 1 shear zone, finding that breakage reduced the minimum void ratio e_{min} very considerably but had less effect on e_{max} . The sand was densified in the shear zone and manifested a higher relative density in relation to its modified limits. The original (intact) and modified (partially crushed) e_{min} and e_{max} values are shown in Figure 30 for reference. Although not demonstrated here, the experiments reported by Altuhafi and Jardine (2011) support the view that a family of critical state lines evolve as breakage progresses under high-pressure shearing that is also strain-rate dependent. Stable unique critical states do not appear feasible under such conditions (Bandini and Coop, 2011; Muir-Wood, 2008).

Once produced, the crushed material is smeared over the advancing pile shaft, giving an initial zone 1 thickness ≈ 0.5 mm, which grew to ≈ 1.5 mm at any given soil depth as the tip advanced and the cyclic interface shearing caused by jacking promoted further shear abrasion.

Figure 31 displays the progressively increasing breakage from the fresh sand through zones 3 and 2 to the interface zone 1, where about 20% of the sand comprises fragments finer than the smallest grains present in the parent NE 34. Image analysis showed that the

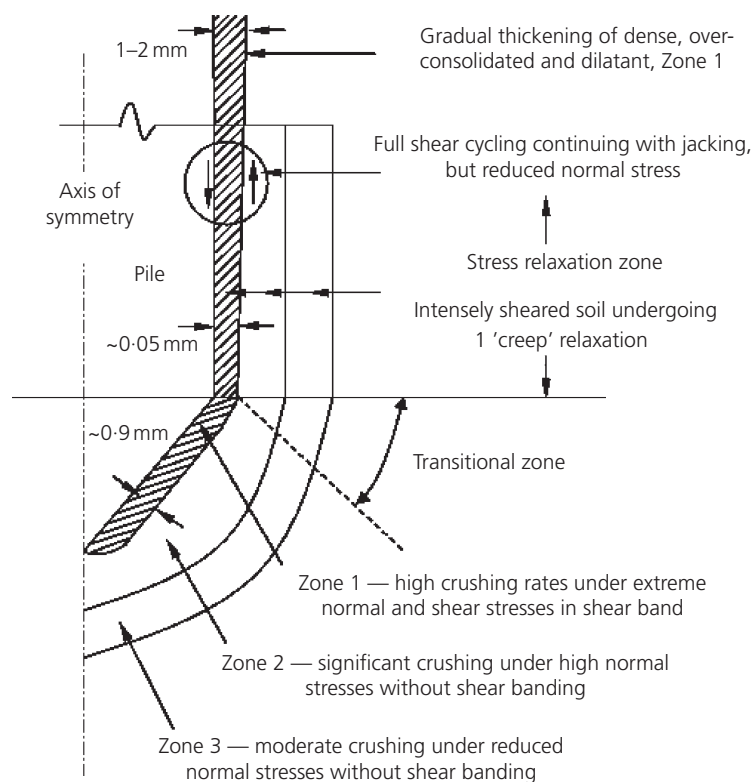


Figure 28. Schematic of crushing and interface shearing zones developed around laboratory model piles (Yang *et al.*, 2010)

zone 1 sand has similar sphericity and convexity to fresh NE 34, while diffraction analyses showed quartz contents (99.6%) just 0.1% lower than for intact NE 34.

The pile surface was also modified. Multiple Rank Hobson Talysurf measurements showed that the maximum surface roughness declined from around 33 to 22 μm , while the centre line average values fell from 3.8 to 2.8 μm . The abraded 1- μm thickness of stainless steel would have contributed less than 1/1000th of the average thickness ($\approx 1\text{ mm}$) of the interface shear zone, which is compatible with the very slightly (0.1%) lower quartz content of the zone 1 material.

Parallel interface ring-shear experiments were conducted with a modified version of the Bishop *et al.* (1971) equipment, shearing NE 34 against surfaces identical to the pile shaft, at normal stresses up to 800 kPa. These tests also developed grey 'zone 1' shear bands, as illustrated in Figure 32, although the bands were thinner and had lower percentages of broken grains than those adjacent to the model piles. Ring-shear tests employing the lower interface configuration shown in Figure 33 did not reproduce the high-pressure pile tip breakage conditions, but led to closely comparable $\delta = \tan^{-1}(\tau_{zh}/\sigma'_z)$ angles to the pile tests that were practically independent of stress level over $100 < \sigma'_z < 800\text{ kPa}$.

Ho *et al.* (2011) extended the study, covering a wider range of gradings with seven silica sands and silts (including NE 34 and

TVS) in ring-shear tests involving interfaces positioned both above and below the sand samples. Their sweep of δ angles against d_{50} is shown in Figure 34 where the upper plot (a) shows trends after shearing to 50 mm, while the lower plot (b) indicates those after 8 m of shear displacement. Also shown are the 'critical state' trends suggested by Jardine *et al.* (1992) from low-displacement (5 mm) direct-shear interface tests and by CUR (2001) from cyclic shear box interface tests.

It is clear that the angles previously interpreted as stable 'critical state' values in fact vary with the test conditions.

- The lower interface arrangement led, with $d_{50} > 0.2\text{ mm}$ of sands, to lesser δ angles after 50-mm displacements than equivalent upper interface tests, where fine fragments can fall from above into void spaces beneath the shear zone.
- Lower interface ring-shear tests gave similar trends at 50-mm displacement to (5 mm) direct-shear interface tests.
- Fragments appear to choke available void spaces after large displacements (8 m), preventing lower friction angles persisting with coarser sands and upper interfaces. The ring-shear trends converge, but do not conform fully to the uniform $\delta = 29^\circ$ CUR (2001) recommendation.

The CC model studies reported above ('Establishing the stress conditions developed around laboratory model displacement files')

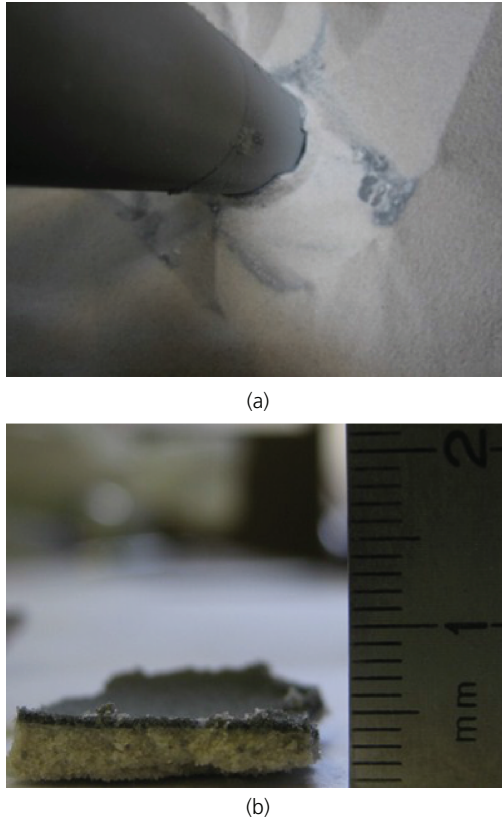


Figure 29. Photographs of interface shear zone developed around a laboratory model pile: (a) top view from above and (b) side view of shear zone material (Yang *et al.*, 2010)

testified to the extreme stresses developed beneath advancing pile tips. Stresses rose and fell around the shaft (at any given depth) by almost two orders of magnitude as the tip penetrated to greater depths. Such changes in stress level, combined with particle breakage, affect the sand's constitutive behaviour. Altuhafi and Jardine (2011) conducted tests to investigate these features using the high-pressure apparatus shown schematically in Figure 35 to subject medium-dense NE 34 to the effective stress paths set out in Figure 36.

The key test stages were as follows.

- K_0 compression to $p' = 9$ MPa, simulating the pile tip advancing towards the sand element from above.
- Drained compression under constant σ_r' until apparent 'critical states' were reached with $\sigma_r' > 20$ MPa, simulating failure beneath the conical pile tip. Tests that stopped abruptly developed large creep strains. The displacement strain rates therefore were slowed progressively to reduce residual creep effects prior to unloading. The 'critical state' $e-p'$ relationships depend on time.
- Drained unloading to $q = 0$ under constant σ_r' before isotropic unloading to p' values between 150 and 500 kPa (giving OCRs of 40 to 140 in terms of vertical stresses), simulating the sharp unloading experienced as the tip passes.
- Renewed drained shearing to failure at constant σ_r' in compression (or at constant p' in extension) to assess the shear strength and dilatancy of the 'heavily overconsolidated' and partially crushed sand.

The results obtained are illustrated in Figure 37, plotting mobilised angles of shearing resistance ϕ' against axial strain. The upper

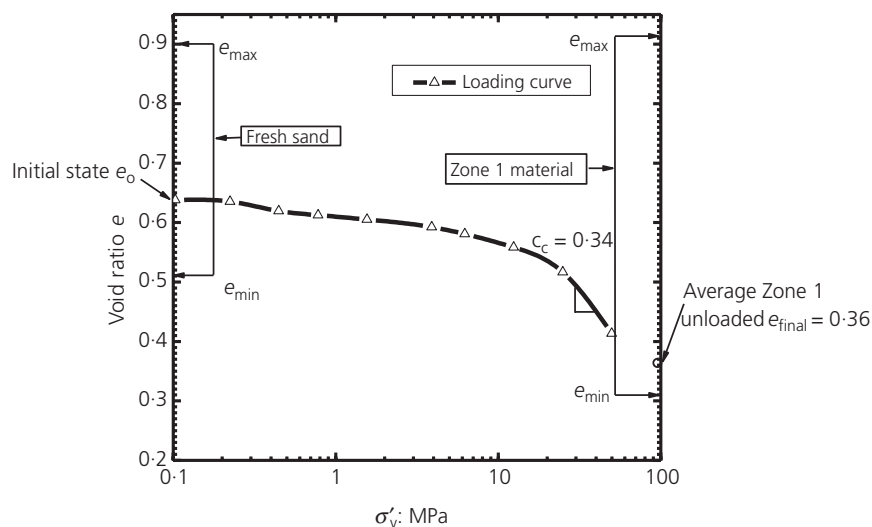


Figure 30. Void ratio–vertical effective stress relationship from high-pressure oedometer test on NE 34 sand, also showing $e_{\min}$ and $e_{\max}$ values of intact sand (left) and Zone 1 material (right) (Yang *et al.*, 2010)

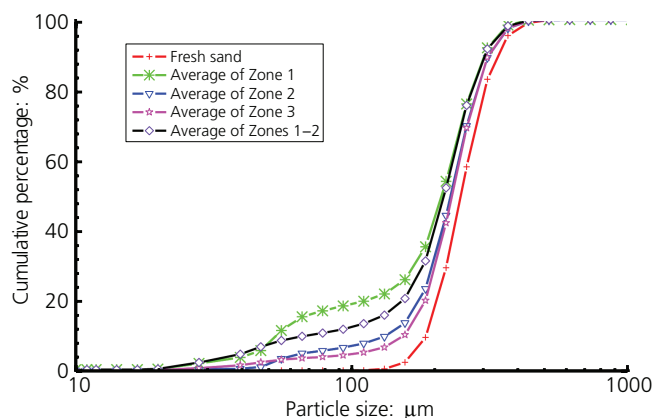


Figure 31. Optical grain-size distributions defined by Feret minima for fresh NE 34 sand and Zones 1 to 3 (Yang *et al.*, 2010)

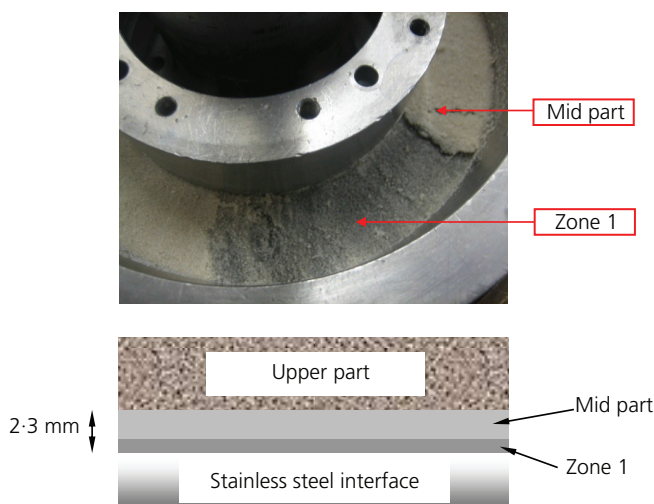


Figure 32. Photograph and scheme of shear zones from interface ring shear tests on NE 34 sand (after Yang *et al.*, 2010)

plot (a) shows the generally ductile–contractant response seen in six similar high-pressure tests, with peak ϕ' only slightly greater than the ‘critical state’ (30°) angle. The lower plot (b) summarises the ‘overconsolidated’ response observed on recompression after unloading. All three ‘overconsolidated’ samples dilated as they sheared, developing peak $\phi' \approx 42^\circ$, well above the ultimate angles (around 33°) developed after large shear strains and diminished dilation.

It is clear that the sand’s behaviour alters radically on unloading as the pile tip advances by a few diameters, changing from being contractant, ductile, highly prone to creep and offering relatively low ϕ' beneath and around the tip, to being dilatant, brittle and able to mobilise far higher peak ϕ' in the mass that surrounds the shaft. These features were critical to the Jardine *et al.* (2013b) interpretation of

the model pile CC stress measurements illustrated in Figures 24–27. Further analysis of the evolving family of ‘critical state’ e – p' curves developed by crushing is underway by Dr Altuhaifi.

Comparison with numerical analyses

Recently published numerical analyses allow further links to be established between the soil element and model pile experiments. Zhang *et al.* (2013) present FE analyses of penetration in sands in which they adopted an arbitrary Lagrangian–Eulerian (ALE) approach to deal with the implicit moving boundary problem and a constitutive model that accounted for grain-size distribution evolving through grain breakage. Their analyses included simulations of the CC model pile tests that applied a ‘breakage’ constitutive model that they calibrated against NE 34 laboratory tests reported by Yang *et al.* (2010) and others.

The Zhang *et al.* (2013) predictions for the mini-ICPs end-bearing characteristics are presented in Figure 24, together with the CC measurements. The agreement is good when considering the same CC upper boundary conditions. Figure 38 compares the breakage pattern identified by Yang *et al.* (2010) around the mini-ICP pile tip with the Zhang *et al.* (2013) contoured predictions for their internal breakage parameter B , which scales linearly between the sand’s initial ($B = 0$) and ultimate ($B = 1.0$) ‘fully crushed’ grading curves. The simulated and experimentally established patterns are similar, with the maximum B predicted as ≈ 0.35 close to the shaft, far from the ‘fully broken’ $B = 1$ limit. The grading curves’ predictions match the Yang *et al.* (2010) measurements well in all three zones, although they do not recover the experimentally observed zone 1 thickness growth with pile tip depth h/R . The latter is thought to develop through the un-modelled process of cyclic interface shear abrasion.

Correspondence with Zhang *et al.* (2013) led to further processing of the stress predictions implicit in their numerical analyses. Interesting comparisons are presented from Yang *et al.* (2014) in Figures 39 and 40, plotting the σ_r' and σ_θ' predictions transmitted by Professor Einav against r/R (Zhang *et al.*, 2013). The stresses are normalised by predicted q_c , as are the experimental equivalents shown in Figures 26 and 27. The overall trends show encouraging quantitative agreement when comparisons are made between predictions and measurements made at h/R values up to 10 (see for example the match between the common curves given for $h/R \approx 6$). Naturally, scope exists to consider further factors such as the effects of stress history on dilatancy and shear strength; creep behaviour; and the extreme cyclic loading that accompanies pile installation and leads to radial stresses continuing to reduce with h/R at ratios >10 .

Laboratory model pile tests to investigate cyclic loading

The mini-ICP CC experiments described above (‘Establishing the stress conditions developed around laboratory model displacement files’) included multiple suites of axial cyclic loading tests with the model piles installed into pressurised medium-dense NE 34 sand.

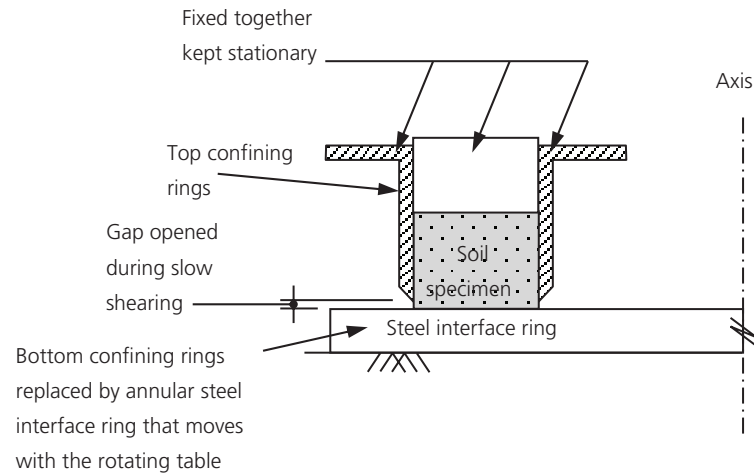


Figure 33. Lower interface configuration for ring shear tests (Ho *et al.*, 2011)

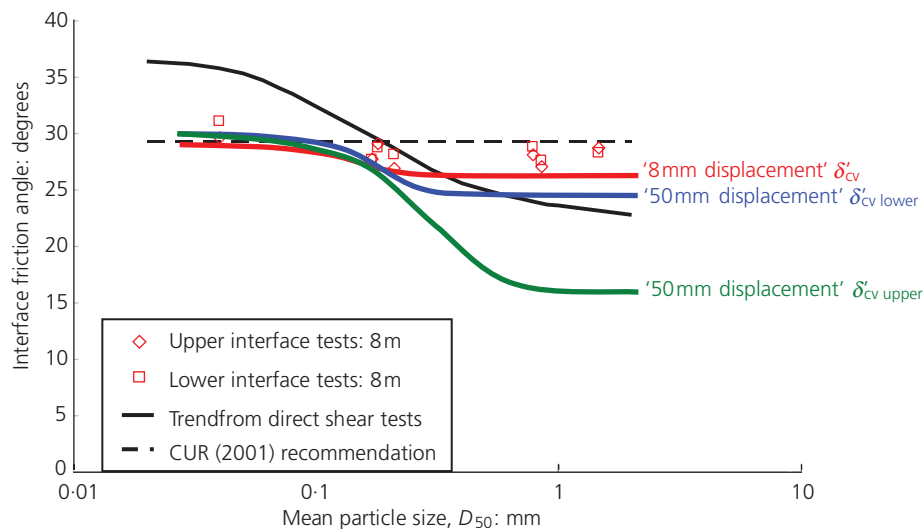


Figure 34. Friction angles from ring shear tests against stainless-steel interfaces with initial CLA roughnesses of 3 to 4 μm . Results after shearing from shear displacements of 50 mm and 8 mm (Ho *et al.*, 2011)

Cycling was found to have a broadly similar effect on axial capacity to that seen in the Dunkerque field tests. Figure 41 presents an overall interactive diagram that compares directly with the field patterns in Figure 3. Tsuha *et al.* (2012) and Rimoy *et al.* (2013) report on the cyclic stiffness and permanent displacement trends. Broadly, they classify responses to cycling as

- stable: capacity increasing slightly, displacements small and stabilising, over 1000 or more cycles
- unstable: reaching failure with 100 cycles
- metastable: falling between these limits.

A particular advantage offered by the laboratory model pile arrangements shown in Figures 22 and 23 was the ability to measure the pile–sand effective stress–path response directly, both at the shaft interface (with the mini-pile's leading, following and trailing surface stress transducers) and within the sand mass by the sand–stress sensor arrays.

Figure 42 illustrates the local interface effective stress paths followed under stable conditions in a 1000-cycle experiment. The patterns resemble those seen in constant normal stiffness (CNS) shear experiments (see e.g. Boulon and Foray, 1986; Dejong *et al.*,

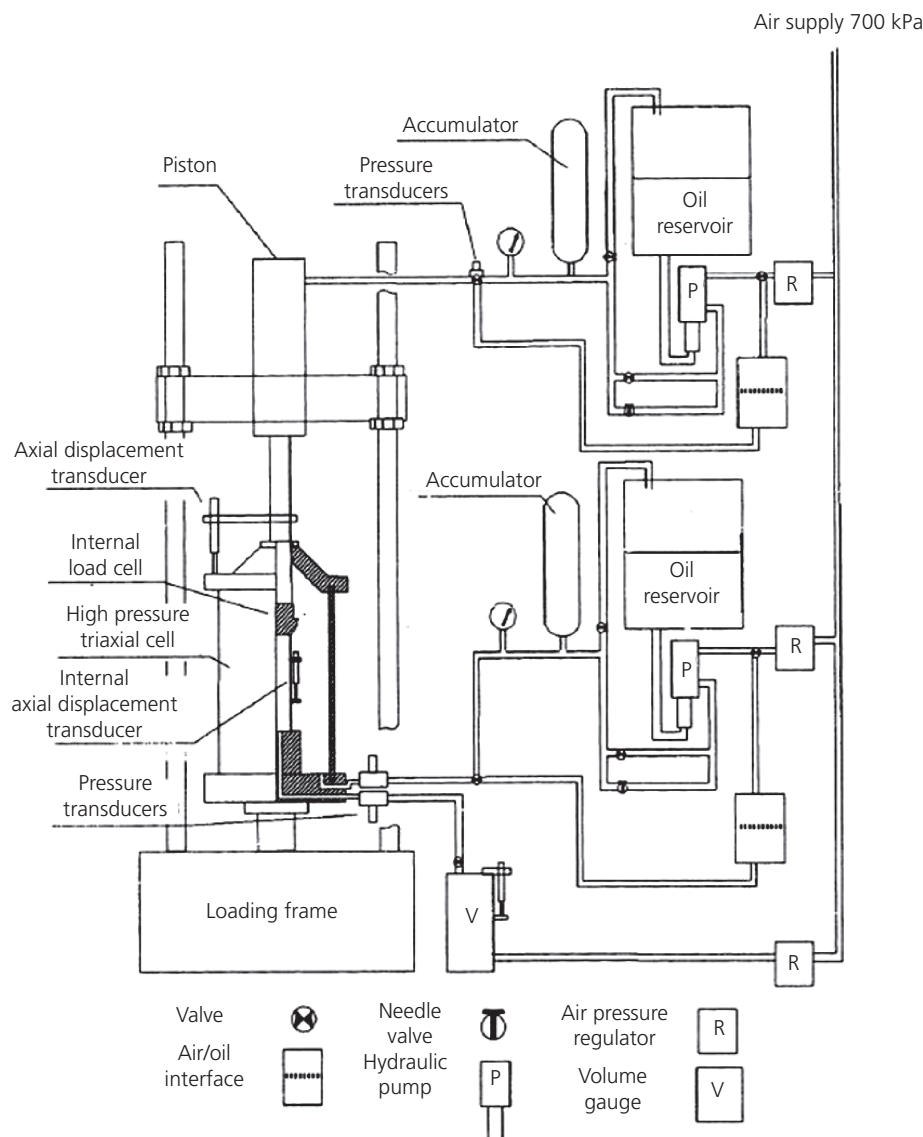


Figure 35. High-pressure triaxial apparatus employed to test crushing NE 34 sand. System described first by Cuccovillo and Coop (1998)

2003), with radial effective stresses increasing under tension loading (that generates negative shaft shear stress) and decreasing under compressive load increments around the relatively rigid mini-ICPs. While the load-displacement response is inelastic (non-linear and hysteretic) under even low-level cycling, the radial effective stress changes and pile head movements induced by each cycle are small.

The effective stress paths appear to match, approximately, the Y_2 criteria described above ('Characterising stress-strain behaviour') and traced by Kuwano and Jardine (2007) in small strain triaxial probing tests. Rather than remain exactly static, the radial stresses reduced, albeit at very slow rates, over time, indicating a tendency towards contraction and migration towards the interface shear

failure criterion angles established by Yang *et al.* (2010) through interface ring-shear tests or those shown in Figure 34 from Ho *et al.* (2011). The continuing rates of radial stress reduction might also be related to very slow rates of continuing interface surface abrasion and particle modification.

Multiple static tension tests on the mini-ICPs showed shaft capacities increasing (by up to 20%) as a result of stable cycling, mainly due to changes in loading stress-path geometry that gave a less contractive response under static loading. The Dunkerque field tests also showed tension capacity increasing after a stable 1000-cycle test (Jardine and Standing, 2012). Figures 43 and 44 demonstrate the contrasting responses seen in metastable tests under

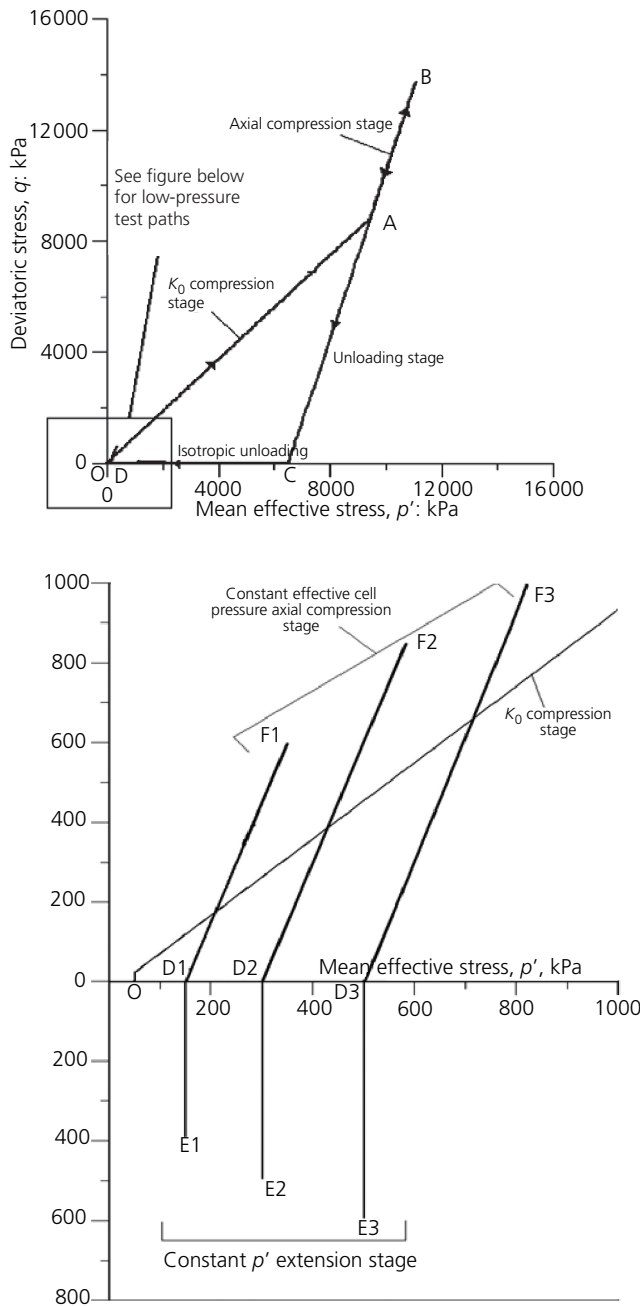


Figure 36. Effective stress paths followed in high/low-pressure triaxial tests on NE 34 sand, showing high-pressure stages (top) and overconsolidated low-pressure stages (below) (Altuhafi and Jardine 2011)

one-way (OW) and two-way (TW) loading, respectively. All paths approach the interface failure envelope as cycling continues, either asymmetrically under OW loading or more symmetrically in the TW test. The milder OW test shows a similar pattern to the stable test shown in Figure 40, except that it migrates more rapidly and engages the critical $\delta = 27^\circ$ failure line, leading to the onset of local slip after several hundred load-controlled cycles. The more severe

TW test progressed further and developed a full failure system with a 'butterfly-wing' effective stress-path pattern resulting from slip displacements that generated dilatant loading stages followed by sharply contractant unloading stages.

Close examination reveals the top-down progressive failure process described by Jardine (1991, 1994). The points where behaviour switches from contractant to dilatant fall on an interface phase transformation line analogous to that noted by Ishihara *et al.* (1975).

Tsuha *et al.* (2012) report on the similarly inelastic cyclic local effective stress responses measured by the multiple cells positioned in the surrounding sand mass, relating these to the sand mass failure criteria established by the experiments outlined in Figure 37.

Laboratory element tests to investigate cyclic loading processes

Predictions can be made through cyclic soil element testing of how cyclic pile head loading affects the local shear stresses τ_{rz} available on the shaft and shear strains in the surrounding soil (Jardine, 1991, 1994). Considering the conditions applying close to axially loaded shafts, as in Figure 46, the hoop strain ϵ_θ must be zero due to symmetry. Also, ϵ_z must be small if the pile does not slip against the shaft and the pile is relatively stiff. The only significant normal strain components are radial (ϵ_r), and these are constrained by the radial stiffness of the surrounding sand mass.

The changes in local radial stress, $\delta\sigma'_r$, developed on the shaft in response to $\Delta\tau_{rz}$ increments that cause dilative or contractive radial displacements Δr at the interface can be related to the shear stiffness of the surrounding sand by the elastic cavity expansion expression given as Equation 6 (Boulon and Foray, 1986). Jardine *et al.* (2005b) suggest that Δr is approximately equal to the peak-to-trough centreline average roughness of the pile surface under static loading to failure. Provided that strains remain very small and the shear stiffness is linear, Equation 6 implies a constant normal stiffness (CNS) interface shear boundary condition, where K_{CNS} is the interface's global radial stiffness value.

$$6. \quad \delta\sigma'_r / \delta r = 2G/R = K_{CNS}$$

Laboratory shear tests can be conducted under CNS conditions (Boulon and Foray, 1986; Dejong *et al.*, 2003) to mimic the pile loading boundary conditions and observe the near-shaft cyclic soil response. Suitable mixed boundary conditions can be devised for simple shear, triaxial or HCA tests. However, sands' shear stiffnesses are non-linear, pressure dependent and anisotropic. Also, K_{CNS} varies with $1/R$, making it hard to define meaningful single CNS values. Constant volume tests in simple shear, triaxial or HCA cells provide upper limit, infinite, CNS conditions that can be met by cycling saturated samples under undrained conditions. More sophisticated controls can be imposed if reliable information is available about the interface stress and strain boundary conditions.

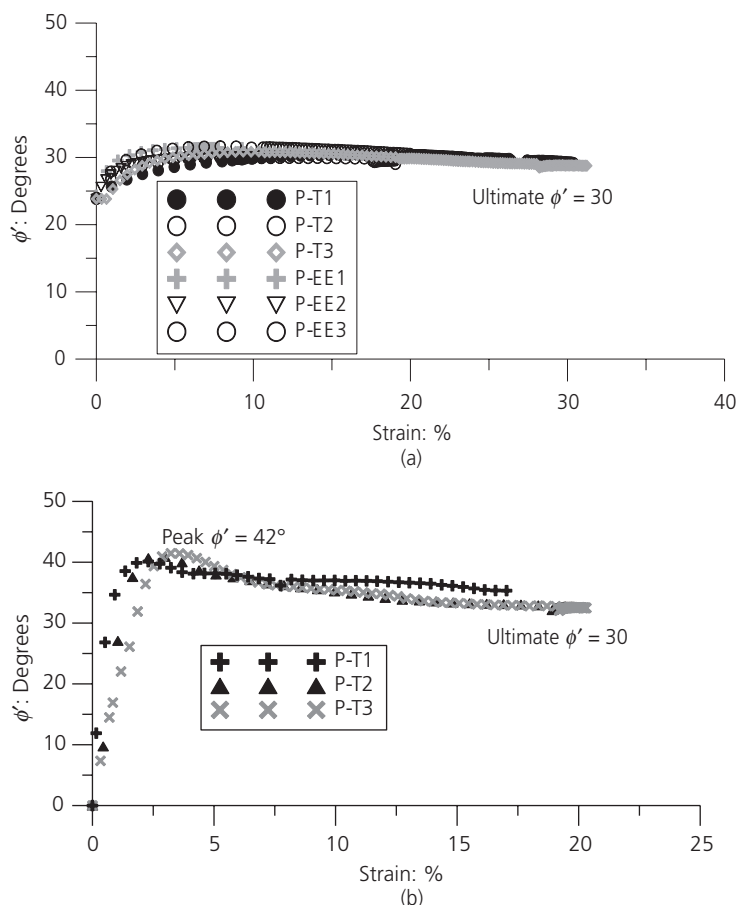


Figure 37. Mobilised ϕ' values plotted against axial strain for both high-pressure (a) and low-pressure (b) test stages of triaxial tests on NE 34 sand (Altuhafi and Jardine 2011)

Constant volume or CNS simple shear (SS) tests provide conditions analogous to those near pile shafts (Randolph and Wroth, 1981). However, conventional simple shear tests cannot provide a full description of the sample's stress state: neither invariant effective stress paths nor Mohr circles of stress can be drawn. Shen (2013) presents new discrete-element method-based simple shear simulations. His analyses, which did not require any assumption of idealised co-axial (or other) plasticity in the sand, emphasise the differences between the true internal stress variables and the 'average' stresses deduced from boundary measurements. He also highlights the impact of apparatus details on the parameters interpreted by alternative simple shear failure hypotheses.

Shibuya and Hight (1987), Menkiti (1995), Nishimura (2006) and Anh-Minh *et al.* (2011) outline the principles and technicalities of conducting SS tests with HCA equipment. While HCAs are subject to sample curvature effects that have to be considered (Hight *et al.*, 1983), their annular geometry automatically provides the complementary shear stresses and so reduces stress non-uniformity. They also allow the full stress and strain tensors to be defined and permit detailed assessments of the effects of anisotropy, variable

b values (reflecting σ_2 ratios or Lode angles) and principal stress axis rotation.

Undrained triaxial experiments can also provide useful information. The shear stress changes $\Delta\tau_{xz}$ developed on the pile shaft and changes to triaxial deviator stress $\Delta q = \Delta(\sigma_1 - \sigma_3)$ can be inter-related by assuming an isotropic soil response and applying general stress invariants, or by simply noting that, in a Mohr circle analysis, increments of pure shear shaft loading $\Delta\tau_{xz}$ have an equivalent effect to an increment Δq that is numerically twice as large. In this simplified view, the changes to mean effective stress, $\Delta p'$, observed under cyclic loading in the triaxial cell can be seen as implying approximately equivalent proportional $\Delta\sigma'_r$ changes at points close to the shaft.

Sim *et al.* (2013) emphasise the need for very stable high-resolution test equipment and stable environments for such tests. This applies particularly to long-duration, low-level cycling tests where p' drift rates and changes in cyclic stiffness/permanent strain development may be slow. Sim *et al.* also report cyclic experiments on Dunkerque and NE 34 sands designed to help interpret the field and laboratory CC model pile tests. Their ongoing research programme is investigating the

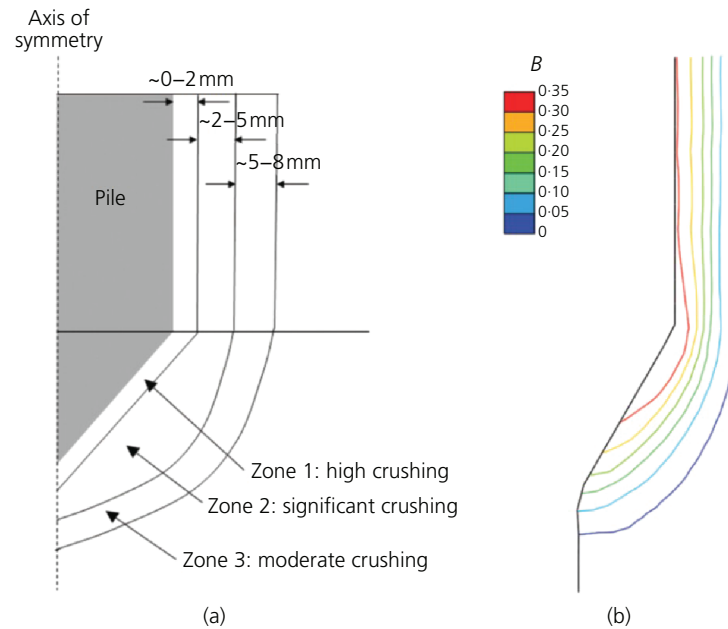


Figure 38. Comparison between (a) the interpretation of Yang *et al.* (2010) of breakage around penetrating mini-ICP model piles and (b) simulation breakage parameter B contours for same tests (Zhang *et al.*, 2013)

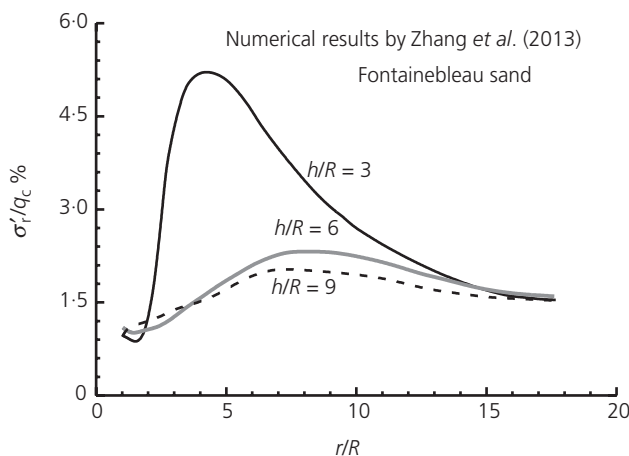


Figure 39. Radial profiles of σ'_r/q_c from the analysis of Zhang *et al.* (2013) of mini-ICP pile in NE 34 sand

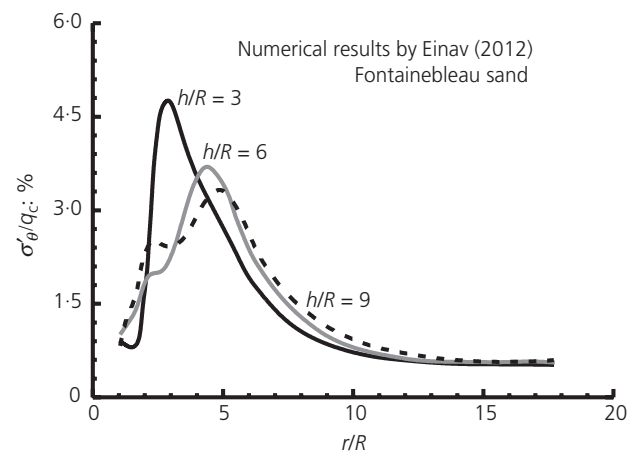


Figure 40. Radial profiles of σ'_θ/q_c from the analysis of Zhang *et al.* (2013) of mini-ICP pile in NE 34 sand

- differences between HCA SS and triaxial responses
- effects of pile installation stress history, including the 'overconsolidation' that takes place as the tip passes, and the effects of the shearing cycles imposed by jacking or driving
- sequence in which different cyclic load packets are applied, assessing the applicability of Miner's rule
- varying sand types and initial sand states.

Figure 47 illustrates the leftward effective stress-path drifts developed in undrained cyclic triaxial tests with paired tests on medium-dense Fontainebleau and Dunkerque samples conducted after K_0 consolidation to 800 kPa and unloading to $\text{OCR} = 4$, to simulate pile installation for points positioned $2 < r/R < 3$ from a pile shaft. A total of 1500 $q_{\text{cyclic}} = 0.20p'$ stress-controlled cycles were then applied at 1/min. The stress paths evidently engaged

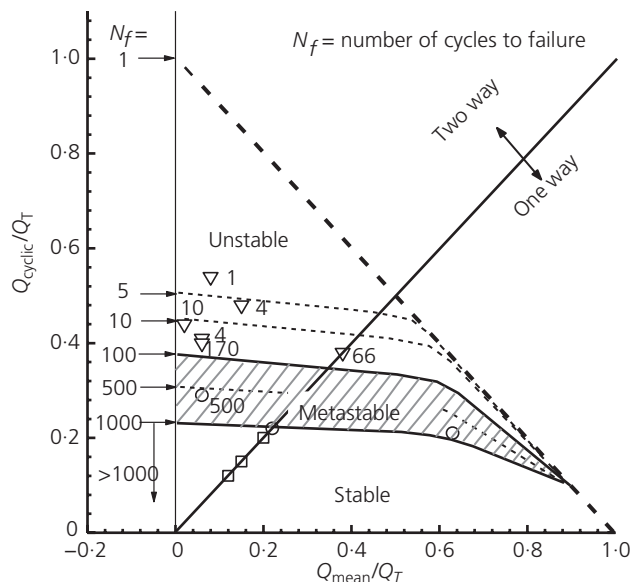


Figure 41. Effects on shaft capacity of cyclic loading. Interactive stability diagram from mini-ICP CC tests (Tsuha *et al.*, 2012)

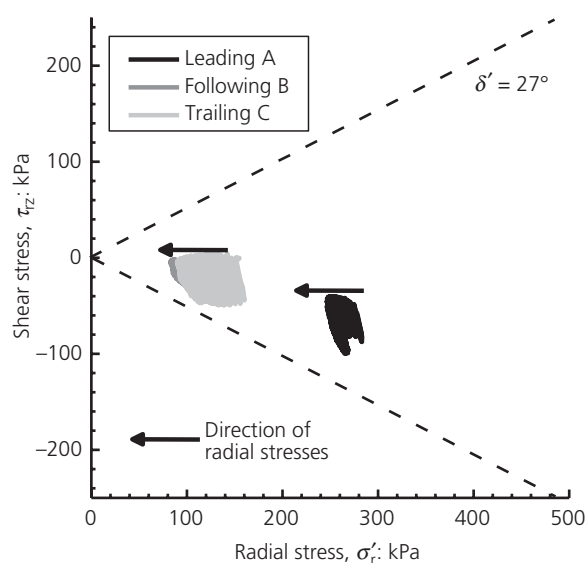


Figure 42. Interface shear $\tau_{rz} - \sigma'_r$ effective stress paths: stable cyclic test ICP4-OW1 (Tsuha *et al.*, 2012)

the samples' Y_2 surfaces. Slow migration led to final mean effective stress reductions of 30% and 40% overall for NE 34 and Dunkerque samples, respectively, under the stringent constant volume conditions imposed. It is interesting that the effective stress paths remained within the mini-ICPs $\tau/\sigma'_n < \tan \delta'$ interface shear envelope ($\delta' = 27^\circ$ when shearing against NE 34 or Dunkerque sand; see Figures 34 and 42–45), implying that, while shaft failure would not be expected to reduce in an equivalent cyclic pile test, the pile shaft would not fail within 1500 cycles.

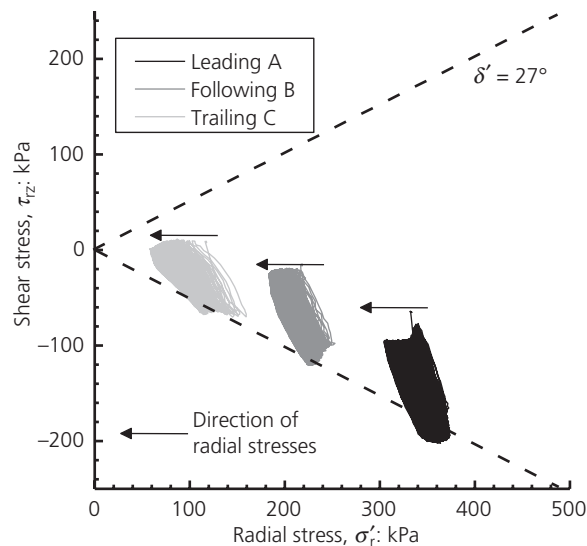


Figure 43. Interface shear $\tau_{rz} - \sigma'_r$ effective stress paths: metastable cyclic test ICP2-OW3 (Tsuha *et al.*, 2012)

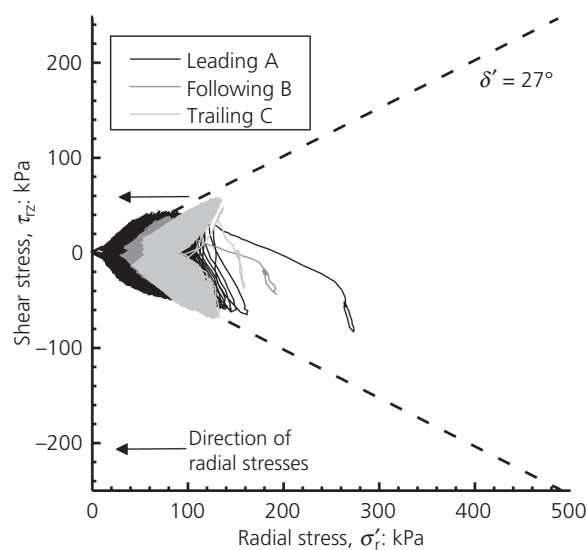


Figure 44. Interface shear $\tau_{rz} - \sigma'_r$ effective stress paths: metastable becoming unstable cyclic loading test ICP4-TW1 (Tsuha *et al.*, 2012)

Jardine *et al.* (2005b, 2012) offer guidance on how to apply such laboratory testing to estimate the axial response of offshore piles under storm cyclic loading. Referring to the flowchart given in Figure 48, the first essential step is careful characterisation (applying rainfall analysis methods) of the storm loads to establish equivalent batches of uniform cycles. Initial screening checks are then recommended with experimentally derived (or appropriately validated theoretical) published cyclic failure interaction diagrams (such as those in Figure 3 or 41). If further analysis is warranted, laboratory or field test data can be applied in site-specific and storm-specific calculations that

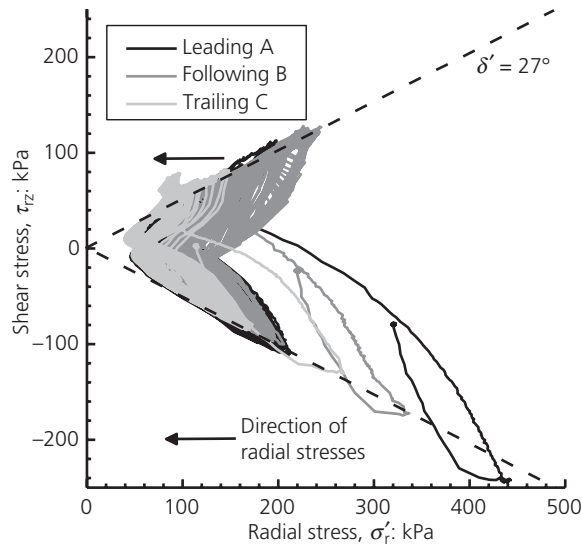


Figure 45. Interface shear $\tau_{rz} - \sigma'_r$ effective stress paths: unstable cyclic test ICP2-TW1 (Tsuha *et al.*, 2012)

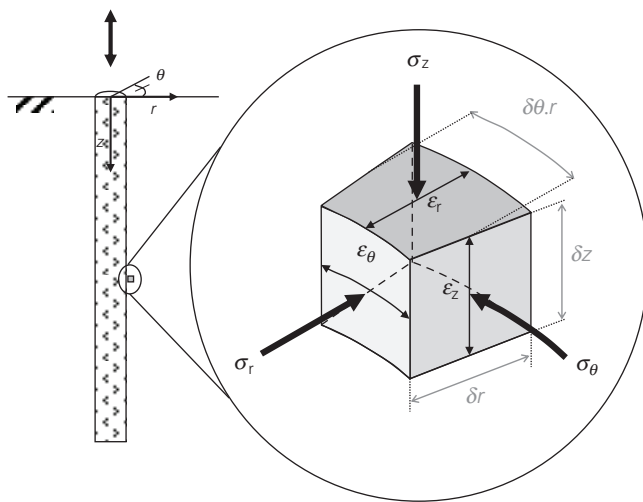


Figure 46. Soil element adjacent to a pile shaft (Sim *et al.*, 2013)

follow either a local (T–Z, the left-hand path in Figure 48) or a global (the right-hand route in Figure 48) assessments procedure. The global approach is most applicable when soil conditions are relatively uniform and progressive top-down failure is not a major concern.

Jardine *et al.* (2012) describe several approaches for such calculations. These include the simple ‘ABC’ formulation given by Jardine *et al.* (2005b). Calibration of the latter approach against both laboratory tests and the Dunkerque field experiments indicated encouraging agreement (Jardine and Standing, 2012). Recent practical applications include a fleet of 40 wind turbines at Borkum West II (German North Sea), which employ a tripod design that relies on three 2.48-m-dia. piles per turbine driven in (mainly) very dense sands (Merritt

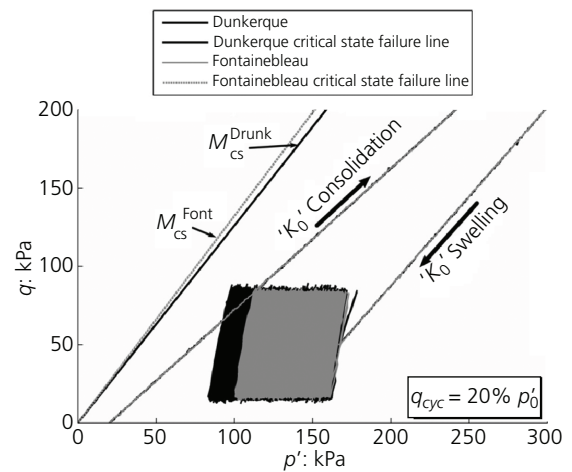


Figure 47. Leftward migration of effective stress paths over 1500 undrained $q_{\text{cyclic}} = 0.2 p'_0$ cycles. Triaxial tests on Dunkerque and NE 34 sands from $p'_0 = 150$ kPa, OCR = 4 (Sim *et al.*, 2013)

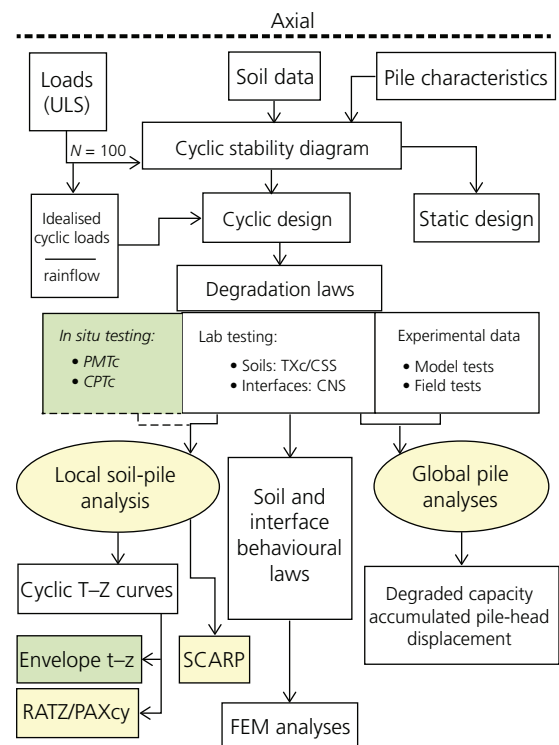


Figure 48. Flowchart outlining approaches for assessing cyclic loading effects in driven pile design: after Jardine *et al.* (2012)

et al., 2012). Another application of the laboratory-derived ABC approach involved manned oil platforms founded on pile groups driven in very hard sandy glacial tills (Jardine *et al.*, 2012).

The fully analytical cyclic assessment route shown as the central path through Figure 48 may also be followed. Laboratory testing can

provide the detailed information required for modelling the sands' complex behaviour including stiffness and shear strength anisotropy; non-linearity and progressive yielding; grain crushing; time effects/creep; and cyclic loading responses. Similarly, the laboratory and field model pile stress measurements can guide the specification (or modelling) of the effective stress regime set up around the driven piles and show how this may change under static/cyclic loading conditions. The stage is now set for numerical modelling that can capture field behaviour far more accurately than was previously possible.

Summary and conclusions

The key aim of the lecture was to demonstrate the special capabilities and practical value of the advanced laboratory testing promoted by Bishop and TC-101. New insights have been offered through static and cyclic experiments with the apparatus and techniques they advocated, including highly instrumented stress-path and high-pressure triaxial tests as well as hollow cylinder, ring-shear interface and micro-mechanical experiments. Emphasis has been placed also on integrating laboratory research, field observations, numerical analysis and CC model pile studies to advance understanding and prediction of the complex behaviour of driven piles in sands.

The experiments investigated sand behaviour under a wide range of conditions. Aspects highlighted for consideration in ongoing and future constitutive modelling include the following.

- Sands show strong non-linearity, marked inelasticity and time dependency seen from small-to-large strains.
- Sands have markedly anisotropic behaviour within the large-scale classical critical state soil mechanics (Y_3) yield surface. Sands also show phase transformation (Y_4) over a wide range of states. These features may occur either in soil continua or during shearing against interfaces.
- Behaviour can only be considered elastic within a very limited kinematic true (Y_1) yield surface that is dragged with the current effective stress point, growing and shrinking with the mean effective stress p' and changing in shape with proximity to the outer Y_3 surface; stiffness is anisotropic within Y_1 , following patterns that evolve with $K = \sigma'_t / \sigma'_z$.
- Plastic straining commences once Y_1 is engaged and becomes progressively more important as straining continues along any monotonic path.
- An intermediate Y_2 kinematic surface may be identified in either continuum or interface shear tests that signifies (a) potentially marked changes in strain increment directions, (b) the onset of important strain-rate or time dependency and (c) a threshold beyond which permanent strains (and mean effective stress reductions in constant volume tests) accumulate significantly in cyclic tests.
- Creep tests and experiments that combine drained creep and low-level cycling show that the Y_2 process is both time dependent and affected by cyclic perturbations.
- Undrained cyclic tests taken to large numbers of cycles tend to show continuous rates of p' reduction, even under

relatively small strain cycles. These trends may be modified considerably by overconsolidation, ageing or pre-cycling.

- Particle breakage develops under large displacement interface shearing as well as high-pressure compression and triaxial conditions. Breakage leads to continuous evolution of the index properties and critical state e - p' relationships.

Conclusions regarding piles driven in sand include the following.

1. Conventional approaches for capacity and load-displacement assessment have generally poor accuracy and reliability.
2. It is possible to improve predictions considerably through numerical analyses that capture the observations made with advanced laboratory stress-strain and interface shear tests.
3. Such predictions rely critically on assumptions regarding the stresses set up around the piles during and after installation.
4. Laboratory and field tests highlight the importance of plastic and time-dependent straining, which becomes progressively more important as stress and strain levels rise.
5. The CC model pile tests demonstrate key physical features of the pile-soil mechanics, including the extreme stress changes and grain breakage experienced during installation. Micro-mechanical laboratory analysis and high-pressure triaxial and ring-shear tests allow the properties of the modified material to be studied in detail.
6. Laboratory model pile experiments demonstrate that radial stress maxima develop at some distance from the pile shafts. This feature can also be predicted analytically in studies that address grain breakage. Taken together with the creep trends discussed above, this feature offers a mechanism for the growth in shaft capacity of piles driven in sand over time.
7. Axial cyclic pile tests show broadly similar modes of stable, metastable and unstable behaviour in full-scale field tests and model experiments in CCs.
8. Local stress measurements made on the ICP and mini-ICP piles give profound insights into the mechanisms of cyclic degradation, demonstrating features of kinematic yielding and interface shear failure that can be tracked in triaxial, HCA and ring-shear laboratory experiments.

Advanced laboratory testing is vital to advancing all difficult geotechnical engineering problems where the outcomes depend critically on the detailed constitutive behaviour of the ground. Tatsuoka (2011), for example, described advanced testing directed towards the performance of large bridge foundations and the compaction of reinforced earth retaining wall backfills, while Kovacevic *et al.* (2012) describe novel analyses of very large submarine slope failures that employed models derived also from detailed and advanced laboratory studies.

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