Shearing resistance during pile installation in sand

Jit Kheng Lim BEng, MEng, PhD

Research Associate, School of Civil, Environmental and Mining Engineering, The University of Western Australia, Crawley, WA, Australia

Barry Lehane BE, MAI, DIC, PhD

Winthrop Professor of Geotechnical Engineering, School of Civil, Environmental and Mining Engineering, The University of Western Australia, Crawley, WA, Australia

This paper examines the detailed stress changes recorded by surface stress transducers equipped on model piles during jacked installation of various piles at sand sites with different groundwater regimes and sand characteristics. Three separate types of shear stress–displacement characteristics are observed and these are explained using findings from existing laboratory and field-based studies on soil–pile–penetrometer interface friction. The findings advance understanding of factors influencing the shearing resistance developed during displacement pile installation in sand, which is an important consideration for pile driveability assessments.

Notation

C_u	coefficient of uniformity
D	pile diameter
D_{50}	mean particle size
D_r	relative density
$e_{\max}$	maximum void ratio
$e_{\min}$	minimum void ratio
G	operational shear modulus
K	normal stiffness
L	pile length
q_c	cone tip resistance
R	interface surface roughness
R_n	relative roughness
S_r	degree of saturation
u_o	ambient pore pressure
Δy	interface slip dilation
$\Delta \sigma'_n$	changes in radial effective stress during shearing
δ_{cv}	constant volume interface friction angle
σ_r	radial total stress
σ'_r	radial effective stress
τ	shear stress
τ_{peak}	peak shear stress

1. Introduction

The shearing resistance developed during displacement pile installation in sand is an important consideration for pile driveability assessments. Although laboratory interface tests have greatly improved understanding of the interface shearing characteristics, confidence in their direct application to the large displacement and rapid shearing induced during pile installation is limited, owing to the difficulty in replicating the correct boundary conditions in laboratory test set-ups. This paper presents examples of shear stress and radial stress data recorded during jacked pile installation in a variety of different sand types and hence provides added insights into the mechanisms operating at the pile shaft–sand interface.

2. Background

For many decades, the direct shear apparatus (DSA) has been used to study the interface shearing behaviour between soils and construction materials (e.g. Al-Douri and Poulos, 1992; Desai *et al.*, 1985; Jardine *et al.*, 1992; Potyondy, 1961). The limitations of the direct shear device were highlighted by Kishida and Uesugi (1987), who proposed a better alternative known as a simple shear interface apparatus (SSIA). Parametric studies conducted by Uesugi and Kishida (1986), Frost *et al.* (2002), Lings and Dietz (2005) and DeJong and Westgate (2009), among others, using either the DSA or SSIA, found that the major factors influencing the interface behaviour are the sand type and relative roughness (R_n), that is, the ratio of a measure of the interface surface roughness (R) to the mean effective particle size (D_{50}).

Unlike the DSA and SSIA, the ring shear apparatus (RSA) allows for unlimited shear displacements and exhibits smaller end effects; it is therefore claimed to model the large displacement interface behaviour relevant to a displacement pile more realistically. Various types of RSA have been explored by Tika (1999), Kelly (2001), Corfdir *et al.* (2004) and Ho *et al.* (2011) to investigate a range of aspects related to the drained interface shearing resistance in sand. The results showed that particle damage together with a change in interface roughness following large displacement shearing lead to different ultimate interface friction angles from those derived using the DSA. It is important to note that conventional DSA, SSIA and RSA interface tests are generally performed under constant normal load (CNL) conditions and focus mainly on the mobilised interface friction angles.

Boulon and Foray (1986) suggested that the pile shaft interface shearing mechanisms should be intermediate between the constant normal load and constant volume conditions, and can be modelled approximately by imposing a (linear) spring with an appropriate stiffness (K) normal to the interface (termed the

constant normal stiffness (CNS) condition). The CNS test has become more popular after it was realised that the degradation of shaft friction with cycling observed in the field (Murff, 1987; Poulos, 1988; White and Lehane, 2004) could only be replicated in laboratory interface tests if the CNS condition was used in place of the more generally employed CNL mode.

Pile shaft friction degradation was further investigated using CNS conditions in the DSA to examine the influence of relative density, sand type, surface roughness, normal stress and normal stiffness on the monotonic, cyclic and post-cyclic responses (Airey *et al.*, 1992; DeJong *et al.*, 2003, 2006; Mortara *et al.*, 2007, 2010; Porcino *et al.*, 2003; Tabucanon *et al.*, 1995) and was incorporated into the SSIA (Evgin and Fakharian, 1996) and RSA (Kelly, 2001) devices. One striking feature revealed by the CNS–RSA experiments (Kelly, 2001), as indicated in Figure 1, is the shear stress brittleness observed when shearing a calcareous sand against a rough interface to large displacement; this response was not observed in silica sand. Lehane *et al.* (2005) showed that CNS tests will always only approximate the response

at the pile shaft, as the operational normal stiffness reduces as the dilation at the interface increases.

Direct monitoring of shaft stresses on an instrumented model pile is certainly the most desirable approach to improve understanding of the shearing behaviour at a pile shaft. This approach captures the true pile penetration mechanism and soil flow, real boundary conditions, actual particle damage and fines migration, and varying normal stiffness – none of which can be simulated completely satisfactorily in laboratory interface tests. However, the scarcity of an accurate and highly repeatable earth pressure cell equipped on the pile shaft has been a major obstacle.

A multi-sleeve friction attachment for a cone penetrometer was developed at the Georgia Institute of Technology (GT) to examine the relationship between interface shear strength and surface roughness directly through in situ measurement (DeJong and Frost, 2002; Frost and DeJong, 2005). The attachment comprises four individual friction sleeves, each of which can be assigned a different texture (representing the required surface roughness) and located at different positions relative to each other. In keeping with the observations from laboratory interface tests, the results show that interface shearing resistance depends critically on the surface roughness. Their experiments were performed in a single monotonic push in contrast to small incremental driving/jacking stages (associated with shear reversal) characteristic of displacement pile installation.

An instrument, referred to as a surface stress transducer (SST), developed at Imperial College London (ICL) in collaboration with Cambridge Insitu (Bond *et al.*, 1991), is arguably one of the most suited to measuring pile shaft friction characteristics. It allows for a designated surface roughness to be specified and measures both radial stress and shear stress acting on the pile shaft simultaneously. The instrument is the key component in the Imperial College pile (ICP), which has been used with considerable success in various ground conditions around Europe (Bond, 1989; Chow, 1997; Lehane, 1992). The ICP programme, however, only covered two siliceous sand sites (i.e. Labenne and Dunkirk) as well as recent experiments in a laboratory testing chamber (Jardine *et al.*, 2009, 2013), leaving considerable scope to examine pile shaft shearing characteristics of sands under a variety of different conditions.

Field experiments using the SST have recently been undertaken at The University of Western Australia (UWA) to explore sand sites of different mineralogy and groundwater regimes. The programme employed two newly fabricated SSTs with diameters of 65 mm and 135 mm, as well as one of the 100 mm diameter SSTs used on the ICP. Results obtained with these devices at three different sand sites are presented here and their contrasting shaft friction responses are used to provide fresh insights into the mechanisms operating at the pile shaft–sand interface. It is noted, however, that, compared to full-scale driven piles, the model piles employed are smaller in diameter, tested in lower

Figure 1. Effective stress paths from CNS ring shear tests shearing: (a) Sydney silica sand; (b) Barry's Beach calcareous sand against rough interface (after Kelly, 2001)

stress regimes and were installed using an incremental jacking procedure.

3. Test sites

The jacked pile installations with the SST were performed at three normally consolidated sand sites, characteristics of which are described in the following paragraphs.

Site A: Shenton Park site is located about 5 km from Perth city centre and comprises a sub-rounded to sub-angular, siliceous dune sand underlain by a weakly cemented limestone at a depth of about 10 m. The water table is at the base of the sand layer. The sand maintains a level of saturation of between 10% and 14% (outside treed areas; see [Lehane *et al.* \(2004\)](#)) and contains a small amount of carbonates (less than 5%); these conditions provide the potential for very light cementation and/or suction between particles. The sand is uniformly graded with a mean particle size (D_{50}) of 0.42 mm and coefficient of uniformity (C_u) of about 2.5. Maximum and minimum void ratios ($e_{\max}$ and $e_{\min}$) are 0.79 and 0.44, respectively, and the constant volume friction angle measured under triaxial conditions is 32° ([Schneider, 2007](#)).

Site B: South Perth Esplanade site is a residential re-development area located close to the Swan River in the centre of Perth. The mean water table level is at a depth of 1.5 m and varies from this mean by up to 0.5 m due to tidal effects. The site comprises a well-compacted sand fill in the upper 1.5 m and is underlain by alluvial silica sand. Classification tests on the sub-rounded to sub-angular alluvial sand indicated D_{50} and C_u values of 0.49 mm and 1.9, respectively. Properties of the alluvial sand are very similar to those found at Shenton Park.

Site C: Ledge Point site is adjacent to Ledge Point village, about 100 km north of Perth. The test site comprises coastal dune sands with a calcium carbonate content of 90%. The sand is angular and uniformly graded with D_{50} of 0.25 mm and C_u of about 2. The constant volume friction angle in triaxial compression was 39° and the $e_{\max}$ and $e_{\min}$ were recorded as 1.21 and 0.90 respectively ([Sharma, 2004](#)). The in situ water content varied between 5% and 8%, which is equivalent to a degree of saturation (S_r) of between 10 and 20%.

Data from [Cho and Santamarina \(2001\)](#) suggest that, for the given S_r and D_{50} values, suction pressures within the sands at sites A and C were negligible. [Lehane \(1992\)](#) also showed an absence of any suction pressures in sand with a similar D_{50} and S_r value at Labenne.

Cone penetration tests (CPTs) were performed in close proximity to the test piles at these three sites. Figure 2 compares typical profiles of cone tip resistance (q_c) and corresponding in situ relative densities (D_r) estimated from correlations proposed by [Jamiolkowski *et al.* \(2001\)](#), assuming medium compressibility for sites A and B and high compressibility for site C. It is seen that

the sand strata between 2 m and 2.5 m (where examination is focused in this paper) has an average q_c value of about 3.5 MPa and average D_r values ranging from 30% to 45% (i.e. loose to medium dense).

4. Test piles

The model test piles employed are (reduced-scale) closed-ended steel piles with outer diameters (D) of 65 mm, 100 mm and 135 mm. Each pile is instrumented with a single stainless steel SST, which is fixed at about 0.7 m above the pile tip. The position is approximately ten times the pile diameter ($10D$) above the pile tip for the 65 mm diameter pile, where conditions can be considered relatively stable and free from major end effects ([Campanella and Robertson, 1981](#)). The heights above the pile tip for the 100 mm and 135 mm diameter piles are about $7D$ and $5D$, respectively. The piles were installed to an average embedment length of about 3 m, which resulted in a range of pile slenderness ratios (L/D) comparable to typical prototype piles.

A CPT truck provided the reaction to install the model piles in a series of either 100 mm or 250 mm long jacking strokes at an average jacking speed of about 6 ± 2 mm/s at sites A and C. A faster rate (averaging about 15 mm/s) was employed for the test programme at site B owing to the limited time frame allowed to work within the commercial site. The piles were fully unloaded after each jacking stroke and left for a brief pause period of between 1 and 5 min before jacking recommenced; this incremental procedure was an attempt to mimic the cyclic shearing involved in the pile installation process.

Prior to installation, the model piles were shot-blasted to a centre-line average roughness of approximately $8 \mu\text{m}$, which was seen to have reduced to about $4 \mu\text{m}$ after pile removal. Previous research (e.g. [Ho *et al.*, 2011](#); [Yang *et al.*, 2010](#)) suggests that the changes of both roughness (R) and particle size (D_{50}) during pile installation are most critical for shearing displacement within the first metre and during the first few shearing cycles; these effects diminish as shearing continues. Therefore, despite a shorter total displacement compared to that of the prototype piles, the relative roughness ($R_n = R/D_{50}$) that governs the interface friction angle along the pile shaft has been reasonably modelled in the experiment.

5. Effect of stress level

The test piles were installed to relatively shallow depths (up to 4 m) and therefore the effect of lower stress levels on the experimental results is worthy of consideration. The shearing behaviour at a pile shaft has been shown by [Boulon and Foray \(1986\)](#) and many others to behave more closely to the constant normal stiffness (CNS) condition, for which the normal stiffness (K) can be expressed as

$$1. \quad K = \frac{\Delta \sigma'_n}{\Delta y} = \frac{4G}{D}$$

Figure 2. Comparison of CPT end resistances (q_c) and estimated relative density (D_r) using correlation by Jamiolkowski *et al.* (2001) for three investigated sand sites

where $\Delta\sigma'_n$ is the changes in radial effective stress during shearing due to interface slip dilation of Δy , G is the operational shear modulus of the sand mass confining the pile shaft during the shearing and D is the pile diameter.

Under the constrained conditions adjacent to a pile, the sand stiffness (and thus the normal stiffness) at a given strain level is larger at higher stress levels. Therefore, as shown in Lehane *et al.* (2005), although the level of dilation is lower at high stress levels, the product of the operational shear stiffness and the interface slip dilation can be greater or less than the corresponding product at low stress levels. In addition, the strongly dilative response at high stress levels observed by Kelly (2001) and presented in Figure 1 (discussed later) does not support the contention that dilation effects are small at high stress levels. Therefore, given the inverse dependence on diameter shown in Equation 1, the combination of the smaller diameters of the model piles employed in the experiments and the lower stress regime is expected to lead to a broadly similar normal stiffness condition to full-scale piles.

6. Results and discussion

Despite having comparable q_c values and stress levels in the focus area between 2 m and 2.5 m (see Figure 2), the installation records at the three sites displayed different trends. Representative cases showing profiles of radial total stress and shear stress recorded during installation are presented on Figures 3 and 4, which show data obtained with the 65 mm diameter piles at sites A and B and the 100 mm diameter piles at sites A and C. The traces shown correspond to the measured stresses while the piles were moving and when the piles were stationary in between the jacking stages. It is seen that the shaft stresses at site A remained about constant during shearing, whereas both radial and shear stresses at sites B and C reduced as shearing continued. The reductions were recovered (to different degrees) during subsequent jacking strokes following shear reversal in the unloading–reloading processes.

The variations of shear stress (τ) with radial total stress (σ_r) and radial effective stress (σ'_r) (referred to as TSP and ESP, respectively) recorded for a typical loading stage at the three test sites

Figure 3. Profiles of radial and shear stresses during installation of 65 mm diameter model piles at (a) site A and (b) site B

are presented in Figure 5. At all sites, after a small reduction in σ_r , continued shearing results in an increment of radial stress accompanying an increment of shear stress. This behaviour reflects the effects of constrained dilation during interface shearing, such as those observed during load tests in previous ICP experiments (e.g. Lehane *et al.*, 1993) and those seen in CNS interface shear tests. At site A, where radial effective stresses (σ'_r) can be assumed equal to radial total stresses, the sand remains at critical state conditions during continued large displacement shearing (i.e. constant τ and σ'_r when the maximum shear stress is attained). In contrast, the shear stresses reduced considerably at sites B and C after the peak values were reached.

At site B, the higher penetration rate (about four times faster than at site A) and the fully saturated conditions suggest that negative pore pressures were generated during the start of the jacking phase and that these then dissipated as shearing continued, resulting in lower effective stresses. The tendency for shear stresses to stabilise towards the end of each jacking stage coupled with the recovery in shear stresses that can be developed during each subsequent jacking stage (as seen on Figure 3(b)) is consistent with the development of transient excess pore pres-

ures. Yang *et al.* (2010) provide evidence indicating a greater tendency for pore pressure generation in the dilative shear band adjacent to the pile shaft–sand interface following displacement pile installation, due to grain crushing and a reduced sand permeability. The effective stress path (τ and σ'_r) estimated using the interface friction angle ($\delta_{cv} = \tan^{-1}[\tau/\sigma'_r]$) of 25° (which was also found applicable at site A) is shown on Figure 5(b). This path suggests that pore pressures reduced by about 20 kPa below ambient pressures when the peak shear stress was developed. Such brittleness was also shown by SSTs below the water table in compression tests at Labenne (Lehane, 1992) when the pile accelerated under load control when peak capacity was mobilised.

The more crushable and angular calcareous sand at site C compared to the siliceous sands at sites A and B suggests that the post-peak drop in shear stress evident on Figure 4(b) and Figure 5(c) arose due to contraction following particle breakage as shearing continued (e.g. Coop and Lee, 1993; Luzzani and Coop, 2002); such contraction leads to a drop in radial effective stress acting on the pile shaft and hence a reduction in shear stress. This permanent change in fabric is consistent with the progressive

Figure 4. Profiles of radial and shear stresses during installation of 100 mm diameter model piles at (a) site A and (b) site C

reduction in installation shear stresses shown on Figure 4(b). Further evidence of this effect is seen on Figure 1 (Kelly, 2001), which shows considerable brittleness of shear stresses in large-scale CNS ring shear tests on Barry's Beach calcareous sand, but no brittleness on the Sydney silica sand. CNS direct shear interface tests conducted at low stress levels on the Ledge Point sand (Lehane *et al.*, 2012) also revealed post-dilative contraction after initial dilation.

The measured local shear stresses were normalised by their corresponding peak shear stresses (τ_{peak}) for a range of different (and typical) jacking stages and are presented in Figure 6. It is evident that the shear stresses for the model piles at site A increased to a certain plateau, which were then maintained as shearing continued. At site B, shear stress brittleness was observed during jacking into a water-bearing sand stratum. A similar trend to that seen at site B was recorded at site C when installing a model pile into a calcareous sand layer above the water table. It should be noted that the shear displacements in Figure 6 (also in Figure 3 and Figure 4) are crude estimations in the absence of displacement measurements during penetra-

tion, which do not represent the initial curve stiffness accurately.

7. Conclusion

Interpretation of the data observed during jacked installation of instrumented displacement piles at three sand sites indicated the following trends.

- Shear stresses developed on the shafts of piles are directly related to the radial effective stress and interface friction angle, by way of Coulomb's friction law.
- No post-peak reduction in shaft shear stress occurs during jacked pile installation in dry silica sands. Post-peak reductions in shear stress can, however, arise when the silica sands are saturated and installation is rapid. Such reductions are due to the dissipation of negative excess pore pressures and do not reflect any significant change in the fabric of the sand at the pile shaft.
- Actual brittleness (i.e. structural damage) occurs in compressible sands sheared against a rough interface, which results in particle breakage and associated contraction with a

Figure 5. Stress paths for a typical shearing stage during pile installation at (a) site A, (b) site B and (c) site C

consequent reduction in radial and shear stresses on the pile shafts during shearing.

Acknowledgements

The authors thank Professor Richard Jardine of Imperial College London (ICL) for providing the 100 mm diameter surface stress transducer (SST). The contributions of previous undergraduate students at the University of Western Australia, Daniel Brook and Thomas Pine, are gratefully acknowledged. The first author would like to acknowledge the scholarship given by the Malaysian government through Universiti Teknologi MARA (UiTM). This research project was funded by an Australian Research Council Discovery grant.

REFERENCES

- Airey DW, Al-Douri RH and Poulos HG (1992) Estimation of pile friction degradation from shearbox tests. *ASTM Geotechnical Testing Journal* **15**(4): 388–392.
- Al-Douri RH and Poulos HG (1992) Static and cyclic direct shear tests on carbonate sands. *ASTM Geotechnical Testing Journal* **15**(2): 138–157.
- Bond AJ (1989) *Behaviour of Displacement Piles in Overconsolidated Clays*. PhD thesis, University of London (Imperial College), London, UK.
- Bond A, Jardine R and Dalton J (1991) Design and performance of the Imperial College instrumented pile. *ASTM Geotechnical Testing Journal* **14**(4): 413–424.
- Boulon M and Foray P (1986) Physical and numerical simulation of lateral shaft friction along offshore piles in sand. *Proceedings of the 3rd International Conference on Numerical Methods in Offshore Piling, Nantes, France*, pp. 127–147.
- Campanella RG and Robertson PK (1981) Applied cone research. *Proceedings of Symposium on Cone Penetration Testing and Experience*. ASCE, Reston, VA, USA, pp. 343–362.

Figure 6. Typical profiles of normalised shear stress plotted against shear displacement during installation jacking

- Cho GC and Santamarina JC (2001) Unsaturated particulate materials – Particle-level studies. *Journal of Geotechnical and Geoenvironmental Engineering* **127**(1): 84–96.
- Chow FC (1997) *Investigations into the Behaviour of Displacement Piles for Offshore Structures*. PhD thesis, University of London (Imperial College), London, UK.
- Coop MR and Lee IK (1993) The behaviour of granular soils at elevated stresses. In *Predictive Soil Mechanics*. Thomas Telford, London, UK, pp. 186–198.
- Corfdir A, Lerat P and Vardoulakis I (2004) A cylinder shear apparatus. *ASTM Geotechnical Testing Journal* **27**(5): 447–455.
- DeJong JT and Frost JD (2002) A multisleeve friction attachment for the cone penetrometer. *ASTM Geotechnical Testing Journal* **25**(2): 111–127.
- DeJong JT and Westgate ZJ (2009) Role of initial state, material properties, and confinement condition on local and global soil-structure interface behavior. *Journal of Geotechnical and Geoenvironmental Engineering* **135**(11): 1646–1660.
- DeJong JT, Randolph MF and White DJ (2003) Interface load transfer degradation during cyclic loading: a microscale investigation. *Soils and Foundations* **43**(4): 81–93.
- DeJong JT, White DJ and Randolph MF (2006) Microscale observation and modeling of soil-structure interface behavior using particle image velocimetry. *Soils and Foundations* **46**(1): 15–28.
- Desai CS, Drumm EC and Zaman MM (1985) Cyclic testing and modeling of interfaces. *Journal of Geotechnical Engineering* **111**(6): 793–815.
- Evgin E and Fakharian K (1996) Effect of stress paths on the behaviour of sand steel interfaces. *Canadian Geotechnical Journal* **33**(6): 853–865.
- Frost JD and DeJong JT (2005) In situ assessment of role of surface roughness on interface response. *Journal of Geotechnical and Geoenvironmental Engineering* **131**(4): 498–511.
- Frost JD, DeJong JT and Recalde M (2002) Shear failure behavior of granular-continuum interfaces. *Engineering Fracture Mechanics* **69**(17): 2029–2048.
- Ho TYK, Jardine RJ and Anh-Minh N (2011) Large-displacement interface shear between steel and granular media. *Géotechnique* **61**(3): 221–234.
- Jamiolkowski M, Lopresti DCF and Manassero M (2001) Evaluation of relative density and shear strength of sands from CPT and DMT. In *Soil Behavior and Soft Ground Construction*. ASCE, Reston, VA, USA, GSP 119, pp. 201–238.
- Jardine RJ, Lehane BM and Everton SJ (1992) Friction coefficients for piles in sands and silts. *Proceedings of Conference on Offshore Site Investigation and Foundation Behaviour*, London, UK, pp. 661–677.
- Jardine RJ, Zhu B, Foray P and Dalton CP (2009) Experimental arrangements for investigation of soil stresses developed around a displacement pile. *Soils and Foundations* **49**(5): 661–673.
- Jardine RJ, Zhu BT, Foray P and Yang ZX (2013) Measurement of stresses around closed-ended displacement piles in sand. *Géotechnique* **63**(1): 1–17.
- Kelly R (2001) *Development of a Large Diameter Ring Shear Apparatus and its Use*. PhD thesis, The University of Sydney, Sydney, Australia.
- Kishida H and Uesugi M (1987) Tests of the interface between sand and steel in the simple shear apparatus. *Géotechnique* **37**(1): 45–52.
- Lehane BM (1992) *Experimental Investigation of Pile Behaviour using Instrumented Field Piles*. PhD thesis, University of London (Imperial College), London, UK.
- Lehane BM, Jardine RJ, Bond AJ and Frank R (1993) Mechanisms of shaft friction in sand from instrumented pile tests. *Journal of Geotechnical Engineering* **119**(1): 19–35.
- Lehane BM, Ismail MA and Fahey M (2004) Seasonal dependence of in situ test parameters in sand above the water table. *Géotechnique* **54**(3): 215–218.
- Lehane BM, Gaudin C and Schneider JA (2005) Scale effects on tension capacity for rough piles buried in dense sand. *Géotechnique* **55**(10): 709–720.
- Lehane BM, Schneider JA, Lim JK and Mortara G (2012) Shaft friction from instrumented displacement piles in an uncemented calcareous sand. *Journal of Geotechnical and Geoenvironmental Engineering* **138**(11): 1357–1368.
- Lings ML and Dietz MS (2005) The peak strength of sand-steel interfaces and the role of dilation. *Soils and Foundations* **45**(6): 1–14.
- Luzzani L and Coop MR (2002) On the relationship between particle breakage and the critical state of sands. *Soils and Foundations* **42**(2): 71–82.
- Mortara G, Mangiola A and Ghionna VN (2007) Cyclic shear stress degradation and post-cyclic behaviour from sand-steel interface direct shear tests. *Canadian Geotechnical Journal* **44**(7): 739–752.
- Mortara G, Ferrara D and Fotia G (2010) Simple model for the cyclic behavior of smooth sand-steel interfaces. *Journal of Geotechnical and Geoenvironmental Engineering* **136**(7): 1004–1009.
- Murff JD (1987) Pile capacity in calcareous sands: State of the art. *Journal of Geotechnical Engineering* **113**(5): 490–507.
- Porcino D, Fioravante V, Ghionna VN and Pedroni S (2003) Interface behavior of sands from constant normal stiffness direct shear tests. *ASTM Geotechnical Testing Journal* **26**(3): 289–301.
- Potyondy JG (1961) Skin friction between various soils and construction materials. *Géotechnique* **11**(4): 339–353.
- Poulos HG (1988) The mechanics of calcareous sediments. *Proceedings of the 5th Australian–New Zealand Geomechanics Conference*, Sydney, Australia, pp. 8–41.
- Schneider JA (2007) *Analysis of Piezocone Data for Displacement Pile Design*. PhD thesis, School of Civil and Resource Engineering, The University of Western Australia, Perth, Australia.
- Sharma SS (2004) *Characterisation of Cyclic Behaviour of Calcite Cemented Calcareous Soils*. PhD thesis, School of

Civil and Resource Engineering, The University of Western Australia, Perth, Australia.

Tabucanon JT, Airey DW and Poulos HG (1995) Pile skin friction in sands from constant normal stiffness tests. *ASTM Geotechnical Testing Journal* **18**(3): 350–364.

Tika TE (1999) Ring shear tests on a carbonate sandy silt. *ASTM Geotechnical Testing Journal* **22**(4): 342–355.

Uesugi M and Kishida H (1986) Influential factors of friction

between steel and dry sands. *Soils and Foundations* **26**(2): 33–46.

White DJ and Lehane BM (2004) Friction fatigue on displacement piles in sand. *Géotechnique* **54**(10): 645–658.

Yang ZX, Jardine RJ, Zhu BT, Foray P and Tsuha CHC (2010) Sand grain crushing and interface shearing during displacement pile installation in sand. *Géotechnique* **60**(6): 469–482.

WHAT DO YOU THINK?

To discuss this paper, please email up to 500 words to the editor at journals@ice.org.uk. Your contribution will be forwarded to the author(s) for a reply and, if considered appropriate by the editorial panel, will be published as a discussion in a future issue of the journal.

Proceedings journals rely entirely on contributions sent in by civil engineering professionals, academics and students. Papers should be 2000–5000 words long (briefing papers should be 1000–2000 words long), with adequate illustrations and references. You can submit your paper online via www.icevirtuallibrary.com/content/journals, where you will also find detailed author guidelines.