INTERFACE LOAD TRANSFER DEGRADATION DURING CYCLIC LOADING: A MICROSCALE INVESTIGATION

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ABSTRACT

The shaft capacity of piles in sand subjected to cyclic (wave) loading has been observed to decrease significantly with loading cycles (Poulos, 1989). A number of researchers (Boulon and Foray, 1986; Tabucanon et al., 1995; Shahrour et al., 1999) have replicated the characteristics of the load transfer degradation behavior in the laboratory through cyclic interface shear testing with a constant normal stiffness confinement condition (Vesic, 1972). However, no consensus currently exists as to the primary microscale mechanisms that govern cyclic interface shear behavior and load transfer degradation.

A research program was undertaken to quantify the contribution of soil properties, cementation, confinement condition, and displacement mode, in load transfer degradation. Monotonic and cyclic interface shear tests were performed using a modified interface direct shear device with a Perspex side window. The specimen particle displacement fields were quantified during selected cycles by capturing high resolution digital images (1600×1200 pixels) and using Particle Image Velocimetry (White et al., 2001a). Results indicate that the confinement condition, which is intended to replicate the elastic response of the far-field soil, is of primary importance as it allows for normal stress relaxation with soil contraction adjacent to the interface. The displacement magnitude, particle characteristics, and particle-particle cementation were also observed to affect the magnitude and rate of degradation. It is anticipated that these findings will provide a fundamental rationale to identify field conditions where shear stress degradation is likely to occur and a basis from which more rigorous models may be developed.

Key words: CNS, constant normal stiffness, cyclic degradation, cyclic shear, interface shear, load transfer, particle image velocimetry, particle tracking, planar deformations (IGC: E12)

INTRODUCTION AND BACKGROUND

The design of pile foundations to support offshore platforms in sand has traditionally been based on the estimated peak static load. However, the actual loading of offshore platforms consists of variable cyclic loading, which results in a significant degradation of shaft capacity. Shaft capacity degradation with cyclic loading has been observed in the field (Chan and Hanna, 1980; Poulos, 1989; Kraft, 1990; Turner and Kulhawy, 1990) and replicated with scaled pile and rod shear tests in the laboratory (Jewell and Randolph, 1989; Fioravante et al., 1997; Ong et al., 1999). However, accurate prediction of the cyclic degradation of shaft capacity has proven difficult. Various static pile analysis methods have been modified to account for the degradation in load capacity and the accumulation of permanent displacement with cyclic loading. These methods consist of load transfer (Randolph and Jewell, 1989; Randolph, 1994) and con-

tinuum (Boulon et al., 1980; Poulos, 1983; Poulos, 1990) based approaches. Generally they are developed based on experience and performance trends rather than fundamental soil properties (Tabucanon, 1997). It is anticipated that quantification of the particulate level mechanisms driving cyclic degradation would enable improved prediction of pile performance under cyclic loading.

Cyclic degradation in sands is attributed to a decrease in the normal stress due to cumulative contraction of the soil within the shear zone contacting the interface. However, reliable prediction requires knowledge of the thickness of the shear zone, and the influence of soil type and loading conditions on the contraction behavior. The characteristics of cyclic degradation have been replicated in the laboratory through cyclic interface direct shear tests with a constant normal stiffness (CNS) confinement condition. Originally developed to account for contraction/dilation of rock joints and rock-socketed piles (Obert et al., 1976; Johnston et al., 1987), the CNS con-

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Written discussions on this paper should be submitted before March 1, 2004 to the Japanese Geotechnical Society, Sugayama Bldg. 4F, Kanda Awaji-cho 2-23, Chiyoda-ku, Tokyo 101-0063, Japan. Upon request the closing date may be extended one month.

dition incorporates an initial normal stress coupled with a spring that reflects the elastic far-field stiffness around a cylindrical pile (Vesic, 1972). The normal stress at any point during CNS shearing is determined by:

$$\sigma_n = \sigma_{no} + \delta_v K = \sigma_{no} + \delta_v (4G_0/d) \quad (1)$$

where σ_n is the current normal stress, σ_{no} is the initial normal stress, δ_v is the deformation of the sample normal to the interface (i.e. reflecting contraction/dilation), K is the spring stiffness (i.e. reflecting the elastic response of the far-field), G_0 is the small strain shear stiffness of the soil, and d is the pile diameter (Johnston et al., 1987). The limiting conditions of CNS where K is equal to 0 and ∞ reflect constant normal load (CNL) and constant normal height (CNH) conditions, respectively.

Adaptation of CNS confinement to soil-pile interface laboratory research was first applied using a mechanical spring (Boulon and Foray, 1986). Notable subsequent research includes: a method to predict interface behavior under CNS confinement from CNL test results (Airey et al., 1992), the quantification of the influence of σ_{no} , K , soil type, interface surface roughness, and loading cycle on the rate of cyclic degradation (Tabucannon et al., 1995), susceptibility of loose sands to rapid cyclic degradation (Shahrour et al., 1999), and cyclic degradation following monotonic shear (Tabucannon, 1997; Kelly, 2001). This research has been based on global measurements obtained externally of the test specimen and therefore has not provided an understanding of the driving mechanisms at the microscale.

Various experimental techniques have been used to measure planar deformation fields in geotechnical element and model tests. These include X-ray (Roscoe et al., 1963; Phillips, 1991), photographic (e.g. Allersma et al., 1994) and stereophotogrammetric methods (e.g. Andrawes and Butterfield, 1973) as well as computer-based automated target tracking systems (Taylor et al., 1998; Paikowsky and Xi, 2000). These experimental techniques rely on tracking artificial targets within the model. The reliance on target markers is a limitation as markers can influence the specimen behavior and become obscured by soil during testing. The performance of the above techniques can be assessed by considering the errors associated with precision (the random difference between multiple measurements of the same quantity) and accuracy (the systematic difference between a measured quantity and the true value). As summarized by White (2002), the measurement precision achieved using these techniques ranges from 1/500th of the field of view (FOV) (Allersma et al., 1994) to 1/11000th of the FOV (Andrawes et al., 1973). The accuracy is often not rigorously established. Taylor et al. (1998) achieved an accuracy of 1/5600th of the FOV using close range photogrammetry.

Particle Image Velocimetry (PIV) is a velocity measuring technique originally developed in experimental fluid mechanics (Adrian, 1991) and recently adapted to obtain planar soil deformation measurements (White et al., 2001a, 2002). PIV removes the need for discrete target

Table 1. Summary of CNS interface shear testing program

Test #	Sand type	Displacement ¹	# cycles	Cementation
T059m	IMDEX 16-30	M, 12 mm	n/a	Uncemented
t074c	IMDEX 16-30	C, ± 1.0 mm	45	Uncemented
t062c	IMDEX 16-30	C, ± 4.0 mm	45	Uncemented
t072c	IMDEX 16-30	C, ± 125 kPa	45	Uncemented
t076c	IMDEX 16-30	C, ± 1.0 mm	45	Cemented
t075c	LeGendre 16-30	C, ± 1.0 mm	45	Uncemented

Confinement condition for all tests: $\sigma_{no} = 100$ kPa, $K = 250$ kPa/mm.

¹M denotes monotonic, C denotes cyclic.

markers by tracking the inherent texture (i.e. spatial variation of brightness) of soil grains through a series of images. Each image is divided into a mesh of PIV patches. The displaced location of each patch in a subsequent image is obtained by determining the location of highest correlation between each patch and a larger search region from a subsequent image. When generating a patch mesh, a balance is required between the precision, which increases with PIV patch size, and the fineness of the derived displacement field, which decreases with patch size. The accuracy of PIV measurements is governed by the method used to convert image-space coordinates into object-space coordinates. White et al. (2001b) describe the use of close range photogrammetry, which allows variations in image scale due to the optical distortion to be removed, to reconstruct planar coordinates to an accuracy of 1/18500th of the FOV.

This paper describes a series of interface shear tests and PIV analyses performed to identify the primary microscale mechanisms driving cyclic degradation and to relate these mechanisms to previous observations based on conventional measurements. An overview of the testing program, interface shear device, image acquisition system, and implementation of PIV is first presented. The results and analysis from both conventional global measurements and PIV analysis are summarized for one monotonic and five cyclic CNS tests. Finally, conclusions regarding the primary microscale mechanisms governing cyclic degradation and their implications on practice are made.

TESTING AND ANALYSIS

Testing Program

The results of one monotonic and five cyclic tests are presented to highlight the influence of interface shearing displacement mode (displacement versus shear stress controlled), sand type (silica versus calcareous/subrounded versus angular), and cementation condition (uncemented versus lightly cemented) (Table 1). A CNS confinement consisting of an initial normal stress, σ_{no} , of 100 kPa and a stiffness, K , of 250 kPa/mm was applied in all tests. A fully rough surface, comprising an aluminum plate with one layer of IMDEX 16-30 sand grains coated with epoxy, was positioned below the soil specimen to ensure complete engagement of the sand with the interface.

Fig. 1. Setup of modified interface direct shear test with image capture for PIV analysis

Though the surface was not observed to degrade during testing due to the hardness of the silica sand-epoxy matrix, a new surface was used for each test. Based on previous findings (Uesugi et al., 1986; Frost et al., 2002) this surface ensures a fully rough interface condition.

Two sands were used during testing. IMDEX 16-30 sand is a subrounded silica sand with a D_{50} of 0.72 mm. LeGrand 16-30 sand is an artificial gradation of a well-graded angular calcareous sand from the Northwest Shelf, Australia, with a D_{50} of 0.83 mm. All samples were prepared by tamping to a medium relative density of 60–65%. Lightly cemented test specimens were prepared by mixing a 1% gypsum – 4% water (relative to the sand weight) solution with dry sand and moist tamping the sand to the required density. The cemented specimen was cured for 12 hours under the initial CNS normal stress ($\sigma_{no} = 100$ kPa) prior to testing to ensure that decementation observed during testing was only due to shearing.

Testing Setup

All tests were performed using a direct shear device modified to enable interface shearing and image capture for PIV analysis (Fig. 1). The interface shear box was 60 mm × 100 mm with a typical specimen height of 20 mm. A 12 mm thick Perspex window extended along the length of one side of the shear box. The top platen was fixed to the loading piston to prevent particle movements in the upper portion of the specimen due to platen rotation. The frame of the upper part of the shear box was constrained with rollers to prevent vertical movement while allowing unrestrained horizontal movement. The rollers were positioned for a maximum interface to shear box gap of about $1/2 D_{50}$. These constraints may result in a higher measured friction angle (Kelly, 2001; Dietz, 2001). However, loss of sand through gaps in the device results in artificial specimen contraction and consequently relaxation of the normal stress for a CNS confinement condition. Secondly, these constraints provide more representative soil displacement behavior since they eliminate soil loss and particle movements due to top platen rotation.

Standard direct shear testing procedures were followed. The interface shear box and the vertical constraint

rollers were assembled and the specimen prepared. Shearing was induced in the sample by displacement of the fully rough surface. A closed loop data acquisition and control program applied the initial normal stress and continuously varied the normal load as a function of the measured top platen displacement. The displacement rate (0.5 mm/min), displacement mode, and the number of cycles were also automated during testing. Global displacement and stress measurements were acquired at every 0.01 mm of horizontal displacement.

A Nikon 995 digital camera equipped with a tether control and mounted on a bracket at a lens to image plane distance of about 10 cm was used to acquire digital images through the Perspex window. 1600 × 1200 pixel images with a typical field of view of 35.0 mm by 26.25 mm captured the central 1/3 of the 100 mm specimen in addition to the top platen and interface plate. Images were obtained throughout the monotonic test (t059m) and during cycles 1–5, 21–25, and 41–45 of the displacement controlled cyclic tests at 0.1 mm interface displacement intervals. For the shear stress controlled test (t072c) images were captured at 0.1 mm increments and at the direction reversals that occurred when ± 125 kPa was reached.

As discussed in the test results, minimal damage to the Perspex window was observed during all tests except for t062c, due to the large interface displacements. The Perspex window was replaced whenever abrasive damage on the Perspex window distorted the acquired images. Since the soil-perspex friction angle is significantly lower than the soil-soil friction angle, there is no tendency for shearing to occur along planes parallel to the window, within the soil sample, rather than at the soil-perspex interface. In addition, individual particles were observed to slide behind the Perspex window, rather than roll, indicating that the soil-perspex interface was not inducing particle rearrangement or dilation of the soil in contact with the window. Therefore it is assumed that behavior observed at the window is representative of the full cross-section of the sample.

It is acknowledged that the end boundaries in conventional direct shear tests can influence the development of a failure zone. At each end, the horizontal movement of the sample, from the platen down to the $1/2 D_{50}$ gap above the interface, must be zero, for compatibility with the end walls. It is noted that other interface element tests using simple shear or ring shear apparatus do not offer this restriction. To minimize the influence of the end walls on the PIV data, images were only captured in the central 1/3 of the specimen where the end boundary influence is minimal. In this central region the specimen behavior is more closely representative of field conditions adjacent to a pile shaft. As discussed later, horizontal movement was observed in the upper central zone of the sample, indicating that the ends of the sample undergo compression in order to comply with the restraint imposed by the end walls. Analysis of the PIV data showed that shearing was uniform across the lateral extent of the image.

Fig. 2. Plot of 1600×1200 image with white control markers, a $1058 \times 30 \times 30$ pixel 'top' patch mesh, and a $170 \times 40 \times 40$ pixel 'bottom' patch mesh

PIV Analysis

PIV Mesh

The soil region captured in an image was divided into a mesh of typically 1000 PIV patches (Fig. 2). A patch size of 30×30 pixels was used in the 'top' zone of the sample, extending from the top platen to within approximately 5 mm of the interface. A larger patch size of 40×40 pixels was used in the 'bottom' zone of the sample (within 5 mm of the interface) to enable improved tracking within the turbulent shear zone. The corresponding measurement precision (RMS error) for the 30×30 and 40×40 patch sizes was 0.02 and 0.015 pixels, respectively (White, 2002). With an image resolution of 1600×1200 pixels, the normalized measurement precision was $1/80000$ th– $1/107000$ th of the FOV over a single measurement step.

Image-Space to Object-Space Transformation

To perform image-space to object-space transformation white control points were machined into the back of the Perspex window (Figs. 1, 2). By comparing the separation of adjacent control points in image-space and object-space, the variation in image scale due to optical distortion created by the camera lens system, refraction through the viewing window, and non-coplanarity of the image and control point planes was calculated. In a typical image, the image scale varied by ± 0.3 pixels/mm around an average value of 45.4 pixels/mm. This variation in image scale leads to a locational accuracy of $\pm 0.6\%$. Image scale variations also arise since the observed soil grains do not comprise a planar surface. Grains contacting the Perspex are closer to the camera than recessed grains visible through voids. The effective projective center of the camera was calculated to be 38 mm from the inner face of the Perspex. A second layer of grains positioned $1/2 D_{50}$ behind the window (i.e. ≈ 0.36 mm) leads to a variation in image scale of approximately 1.1%; significantly greater than that created by other sources of image distortion. Therefore, the use of close range photogrammetry to deduce the variation in image

Fig. 3. Test t059m (a) Shear stress vs. horizontal displacement, (b) Vertical displacement vs. horizontal displacement and (c) Vertical position vs. total horizontal displacement

scale due to optical distortion is unwarranted as these errors are smaller than those created by the difference in camera-to-object distance for different soil grains. In the analysis herein, the image-space to object-space conversion has been made by assuming a constant image scale of 45.4 pixels/mm.

Overall Measurement Error

Since the image scale remains constant throughout each image sequence, no systematic difference exists between the coordinates of a soil element in subsequent images. Instead, the error in the measured displacement of a soil element derives from the precision of the measurement system, not the accuracy. Therefore, the error in each soil element displacement step is very small and equal to the PIV precision described previously. This error corresponds to a soil displacement of less than $0.5 \mu\text{m}$, noting that the FOV is 35 mm wide. The error in the absolute location of each soil element is comparatively large ($\pm 0.6\%$). In subsequent figures depicting initial soil element (patch) location relative to the interface (e.g. y-axis in Figs. 3, 4), this error corresponds to $\pm 120 \mu\text{m}$ over the 20 mm sample height. However, the displacements of patches between subsequent images and the average displacements within a patch layer (e.g. x-axis in Figs. 3, 4) are governed by the precision rather than the accuracy. Therefore observations made with regard to patch and average patch displacements have considerably less error, though not as low as the system precision ($0.5 \mu\text{m}$) due to random walk errors associated with cumulative patch displacements.

RESULTS

Monotonic, 12 mm Displacement, Silica Sand, Uncemented (t059m)

In addition to performing cyclic tests, monotonic tests were performed to verify device performance and determine the applicability of PIV analysis to interface shear analysis. Conventional global measurement plots of shear stress and vertical displacement versus horizontal displacement for test t059m are shown in Fig. 3(a) and 3(b). The shear stress increases rapidly to about 160 kPa and then remains relatively constant, continuing to increase slightly with displacement. Rapid dilation is observed within the first 3.0 mm of displacement with continued minor dilation at larger displacements. In terms of stress ratio ($\mu = \tau/\sigma_n$), a distinct peak friction coefficient of 1.20 occurs at about 1 mm and a constant friction coefficient of approximately 0.85 is observed for displacements greater than about 3 mm.

PIV analysis was performed following the methods outlined previously, yielding a continuous vertical and horizontal displacement history across the entire image area. Representative behavior of PIV patches relative to their distance from the interface was obtained through averaging the displacement components within each row of patches. The horizontal displacement profile from the images corresponding to 1.0 mm interface displacement intervals are plotted in Fig. 3(c). The maximum displacement occurs adjacent to the interface and decreases with distance from the interface. For all tests the shear zone was defined as the specimen region that shows a significant gradient of horizontal displacement (i.e. shear strain).

The thickness of the shear zone, determined from the sharp change in gradient in Fig. 3(c), following 1 and 12 mm of interface displacement was approximately 3.4 and 5.3 mm (Table 2), respectively. This indicates that the interface shear zone thickness increases rapidly with initial interface displacement (1 mm) and then increases gradually with subsequent displacement. It is noted that this value is similar to shear zone thicknesses against rough interfaces observed by DeJong and Frost (2002) using other measurement techniques.

A similar plot of total displacement perpendicular to the interface versus interface displacement reveals the origin of the global dilation shown in Fig. 3(b). The verti-

cal displacement within the shear zone increases linearly from 0 mm at the interface to a vertical displacement equal to that of the top platen at the upper limit of the shear zone. This indicates a uniform dilation rate within the shear zone.

Soil in the region above the shear zone region shows uniform vertical displacement, indicating negligible vertical strain (dilation) in the upper part of the sample. Uniform horizontal movement of the upper part of the sample is observed (Fig. 3(b)) due to compression and extension of the end regions (outside the image).

The shear and volumetric strains, indicated by the displacement gradients parallel (horizontal) and perpendicular (vertical) to the interface, respectively, are relatively uniform within the shear zone. The average shear strain in the lower 3.5 mm of the shear zone increases linearly from 0.11 to 2.58 as the interface displacement advances from 1 mm to 12 mm, respectively (Table 2). Assuming that the constant shear strain present between 0.5 mm and 3.5 mm from the interface extends to the interface (0.0 mm), the total displacement of the soil adjacent to the interface at each displacement interval was estimated. The degree of interface slip was defined as the relative displacement between the rough interface surface and the soil displacement adjacent to the interface. This value was computed as the percent difference between the actual interface displacement (measured globally) and the estimated total displacement of soil adjacent to the interface. The percent slip at the interface was relatively constant for all displacement increments, averaging 9.3% and varying irregularly from 7.3% to 11.7% (Table 2).

The volumetric strain reflects dilation of the shear zone, increasing from 0.03 to 0.08 between interface displacements of 1 and 3 mm. At larger interface displacements the volumetric strain remains relatively constant (between 0.06 and 0.08) indicating the establishment of a constant volume condition within the interface shear zone. The observation of uniform volumetric strains occurring within the shear zone provides insight into the mechanism driving dilation and therefore increased shear stress during monotonic displacement. PIV results indicate that the observed global dilation is the result of the uniform dilative volumetric strain throughout the shear zone with minimal dilation between the interface and contacting particles. The total dilation of the shear zone determined from PIV is comparable to $1/2D_{50}$ of the

Table 2. Summary of results of CNS interface shear tests

Test #	Shear zone thickness (mm)	Shear strains, γ	Percent slip (%)	Secant shear modulus, G (kPa)	Volumetric strains, ϵ_v
t059m	3.4(@1 mm)–5.4(@12 mm)	0.11(@1 mm)–2.58(@12 mm)	9.3	N/A	0.03(@1 mm)–0.08(@12 mm)
t074c	3.6(c1–5)–5.6(c41–45)	0.38(c1–45)	8.9	713(c1), 519(c5–45)	0.014
t062c	4.6(c1–5)	1.47(c1)–0.60(c5)	20.8(c1)–59.8(c5)	255(c1), 506(c5–45)	0.003–0.02(non-uniform)
t072c	4.0(c1–5)–4.6(c41–45)	0.19(c1)–0.56(c41)	2.9(c1)–17.1(c41)	1321(c1), 496(c21–45)	0.006–0.02(non-uniform)
t076c	3.0(c1–5)–4.0(c21–45)	0.69(c1–5)–0.59(c21–45)	15.8	439(c1), 370(c5–45)	0.006(c1–5)–0.014(c21–45)
t075c	10.0(c1)–6.0(c21–45)	0.33(c1)–0.17(c21–45)	5.4(c1)–47(c41–45)	1001(c1), 689(c5–45)	0.020

Note: c1 denotes cycles 1.

Fig. 4. Test t074c (a) Shear stress vs. horizontal displacement, (b) Vertical displacement vs. horizontal displacement and (c) Vertical position vs. total horizontal displacement

IMDEX sand (≈ 0.36 mm). This quantitative analysis refines previous hypotheses (Jardine and Chow, 1996; Chow, 1997) where dilation was viewed as a localized outward movement adjacent to the interface equal in magnitude to the roughness (which in this case is D_{50}).

The instantaneous dilation rate of the shear zone, computed as the ratio of the volumetric ($-\varepsilon_v$) to shear strain (γ) gradients, decreases from 0.28 to 0 within the first 3 mm of displacement. The initial dilation rate (0.28) is significantly higher than maximum global dilation rate of 0.08. The dilation rate then approaches 0 at about 3 mm of interface displacement for both global and PIV analysis. The instantaneous dilation rate determined from PIV is a direct measure from within the shear zone and is computed as $-\varepsilon_v/\gamma$. In contrast, the global dilation rate is a measure of the net response of the specimen, computed as $\Delta y/\Delta x$ over a selected displacement range. In this calcu-

lation, Δx includes $\gamma \times h_i$ (shear zone thickness) within the shear zone in addition to horizontal compression of the specimen (Fig. 3(c)). At small interface displacements, horizontal compression dominates the global horizontal displacement measurement and therefore Δx , significantly decreasing the global dilation rate. While further analysis is needed, this notable difference between the true dilation rate within the shear zone and the global dilation rate questions the appropriateness of the dilation angle conventionally reported from shear box tests.

Cyclic, ± 1 mm Displacement Controlled, Silica Sand, Uncemented (t074c)

Test t074c highlights typical CNS cyclic interface shear behavior at relatively small cyclic displacements for subrounded silica sand in an uncemented structure. An initial increase in shear stress to 150 kPa (corresponding to a friction coefficient of 1.2) during displacement to +1 mm mirrors the monotonic behavior (t059m) (Fig. 4(a)). Following displacement reversal at ± 1 mm, a rapid decrease to 30–40% of the peak shear stress is observed within the first 0.1 mm of displacement. This is followed by a non-linear transition zone over about 0.3 mm of displacement which leads into a region in which the shear stress increases at a constant rate. The shear stress then increases more rapidly as the displacement reversal limit (± 1 mm) is approached. As the number of cycles increases, the initial decrease in shear stress occurs more rapidly, the transition zone narrows, and the shear stress increases more rapidly near the displacement reversal limit. In the global displacement plot (Fig. 4(b)), a characteristic 'saddle' behavior is observed following initial dilation of 0.06 mm. Between displacement reversals, contraction occurs within approximately the first 35% (0.7 mm) followed by dilation over the remaining 65% (1.3 mm) (Fig. 4(b), inset schematic). The rate of contraction is higher than the dilation rate, resulting in cumulative contraction of 0.28 mm over 45 cycles, with a corresponding reduction in normal stress from 100 kPa to 36 kPa.

Summary results from the PIV analysis were generated by averaging displacements within each patch row. The average horizontal displacements between the limit reversals (+1.0 mm to -1.0 mm) in cycles 1, 5, 21, 25, 41, and 45 are plotted in Fig. 4(c). The shear strain within the lower portion of the shear zone (0.0 mm to 3.57 mm) is consistent between all cycles and averages 0.38 (0.34 to 0.44) for the cycles plotted (Table 2). The secant shear modulus, G , defined as the total change in shear stress per cycle divided by the shear strain within the shear zone, was calculated for cycles 1, 5, 21, 25, 41, and 45 (Fig. 5). For cycle 1, G is equal to 713 kPa, after which it remains relatively constant (519 kPa) with continued cycling (Table 2).

The percent slip is similar to t059m and averages 8.9% throughout cycling. The volumetric strain was relatively constant and average 0.014. The shear zone thickness increases from 3.6 mm in cycle 1 to 5.0 mm and 5.6 mm in cycles 21 and 41, respectively. The shear zone thick-

nesses for cycle 1 and a monotonic displacement of 1 mm (t059m) are similar. Above the shear zone, the observed soil moves horizontally, reflecting compression of the sample against the end walls of the box. The amplitude of this horizontal movement decreases from 0.40 mm to 0.23 mm between cycles 1 and 45 as the peak shear stress, and hence the horizontal compression of the sample reduces. The average total vertical displacement remained relatively constant for all cycles at about 0.14 mm, which agrees with global dilation measurements.

The characteristic displacement behavior within three regions of the specimen for each cycle interval is shown in Fig. 6 where the horizontal coordinate is relative to the initial position and the vertical coordinate is relative to

the interface. Patches T391 (14.7 mm), B51 (2.7 mm), and B119 (1.0 mm) correspond to the upper part of the sample outside the shear zone, the border of the shear zone, and within the shear zone respectively. The dilation associated with initial shearing to +1.0 mm followed by the 'saddle' contractive-dilative behavior in subsequent cycles is clearly evident in T391. In contrast, patch B119 is characterized by horizontal random movement with minor vertical displacements due to proximity to the rough interface. The horizontal displacement range of about 1.4 mm corresponds with the average trends. In cycles 1–5 the displacement behavior of patch B51, which lies initially 2.7 mm from the rough interface, is similar to T391 but with larger horizontal displacements (0.6 mm). In the later cycles behavior is more random, containing increased horizontal displacements (0.8 mm), and a displacement pattern more similar to patch B119. This transition is indicative of the widening of the shear zone to include patch B51, and matches the observed increase in shear band thickness (Fig. 4(c)).

Cyclic, ± 4 mm Displacement Controlled, Silica Sand, Uncemented (t062c)

Increased displacement of ± 4 mm in t062c results in notably different behavior than t074c as evident in Fig. 7. The shear stress behavior (Fig. 7(a)) is similar to that observed initially in t074c and t059m with peak and residual friction coefficients of 1.2 and 0.9, respectively. Initial dilation (0.4 mm) consistent with t059m occurs in the first portion of cycle 1, followed by a cumulative contraction of 0.2 mm. The asymmetric behavior evident in the global contraction/dilation plot (Fig. 7(b)) is not evident in the PIV data (patch T391 for cycles 1–5, Fig. 8).

* Note: calculated using estimated shear strain values for t062c cycles 21, 25, 41, 45 and t076c cycles 41, 45.

Fig. 5. Calculated secant shear modulus values for all cyclic tests

Fig. 6. Displacement paths for selected patches in Test t074c plotted as horizontal displacement relative to initial position versus distance from interface

Fig. 7. Test t062c (a) Shear stress vs. horizontal displacement, (b) Vertical displacement vs. horizontal displacement and (c) Vertical position vs. total horizontal displacement

Following an initial dilation of 0.42 mm, symmetric contractive/dilative behavior (Fig. 4(b), inset schematic) is evident with average total horizontal and vertical displacements of 0.49 mm and 0.15 mm, respectively.

Figure 7(c) shows the variation in horizontal movement with distance from the interface during cycles 1–5. The average shear strain within 3.5 mm of the interface decreases rapidly from 1.47 to 0.60 between cycles 1 and 5 (Table 2). The percent slip at the interface steadily increases from 20.8% to 59.8% in cycles 1–5. This range of values is notably larger than the slip observed in the previous tests (about 9%). Within the first five cycles the shear zone thickness remains relatively constant at 4.6 mm. Vertical and horizontal displacements further than 3 mm from the interface for cycles 21–25 and 41–45 exhibit behavior similar to cycles 1 to 5. PIV analysis was not feasible within 3 mm of the interface due to Perspex

Fig. 8. Displacement path for Patch T391 in test t062c plotted as horizontal displacement relative to initial position versus distance from interface

window damage from extensive surface abrasion. Quantitative inspections of captured images indicate an average horizontal movement of about 3 mm within each displacement reversal for the higher cycle intervals. Therefore, a constant cyclic shear strain amplitude of about 0.60 is achieved following 5 cycles. The secant shear modulus, G , rises from 255 kPa in cycle 1 to a value of around 500 kPa (comparable to the other tests on uncemented silica sand) for cycles 5 onwards, which is consistent with the increase in interface slip.

The volumetric strain within the shear zone was non-uniform with cycling and varied from 0.003 to 0.02 due to establishment of a steady state condition within each displacement reversal. However, an average cyclic vertical displacement range of 0.17 mm was observed above the shear zone.

Cyclic, ± 125 kPa Stress Controlled, Silica Sand, Uncemented (t072c)

Shear stress controlled cycling is arguably more reflective of in-situ loading conditions. Test t072c was performed with a stress controlled loading of ± 125 kPa (Fig. 9). The shear stress increases rapidly to 125 kPa at 0.49 mm, where the first stress reversal occurs (Fig. 9(a)). The total horizontal displacement of the interface increases from 1.01 mm to in cycle 1 to 1.36 mm, 2.27, and 2.90 in cycles 5, 21, and 41, respectively. Similarly, the total vertical displacement increases from 0.078 mm in cycle 1 to 0.110 mm, 0.184 mm, and 0.196 mm in cycles 5, 21, and 41, respectively (Fig. 9(b)). A cumulative contraction of 0.16 mm at the end of 45 cycles resulted in a decrease in the normal stress to 60 kPa.

The maximum average displacement of the patch row adjacent to the interface increases consistent with the global measurements discussed previously. The shear strain within the lower linear portion of the shear zone (<3.8 mm) for cycles 1, 5, 21, and 41 was 0.19, 0.26, 0.46, and 0.56, respectively (Fig. 9(c)). The percent slip also increased, being 2.9%, 3.5%, 9.7%, and 17.1% for cycles 1, 5, 21, and 45, respectively. In this stress controlled test the combined effect of increasing shear strain and percent slip with increasing number of cycles is

Fig. 9. Test t072c (a) Shear stress vs. horizontal displacement, (b) Vertical displacement vs. horizontal displacement and (c) Vertical position vs. total horizontal displacement

reflected in a reduction of G . G is initially larger than for the displacement controlled tests ($G = 1321$ kPa in cycle 1), but converges with the other tests on uncemented silica sand after 20 cycles (G averages 496 kPa for cycles 21–45) (Fig. 5).

The shear zone thickness increases slightly during cycling from about 4.0 to 4.6 mm. The average cyclic vertical movement of the upper part of the sample increased from about 0.04 mm in cycle 1 to 0.13 mm in cycles 41 to 45. Average volumetric strains within the shear zone varied considerably with cycling, from 0.006 to 0.014 and generally increasing with increased interface displacement.

Cyclic loading between limits of shear stress results in notably different behavior than the displacement controlled loading conditions examined previously. The measured coefficient of friction at the end of each cycle

remained approximately constant throughout the test. Therefore, the shear-stress controlled cycles correspond to cycles between limits of normal stress, which in turn are limits of sample height (or volume) in CNS conditions. Since each stress reversal leads to a contraction, this loss of volume must be recovered by equal dilation in order for the normal stress (and hence the limiting shear stress) to be recovered. It follows that the displacement amplitude of each cycle depends on the movement required for sufficient dilation to balance the initial contraction. With increasing cycles, the magnitude of the contractive phase increases. The dilation rate remains roughly constant, resulting in increased horizontal cyclic amplitude.

Cyclic, ± 1 mm Displacement Controlled, Silica Sand, Cemented (t076c)

Test t076c consisted of a lightly cemented specimen that was prepared with 1% gypsum and cured from 12 hours under an initial normal stress of 100 kPa prior to shearing. The cemented specimen exhibited a high initial shear stiffness (G_0) of 1301 kPa/mm compared to 537 kPa/mm for the parallel uncemented test (t074c) (Fig. 10(a)). A peak coefficient of friction of 1.5 was observed at 0.15 mm followed by linear degradation to a coefficient of 1.0 at +1 mm of displacement. Softening beyond the initial peak reflects the onset of particle-interface and particle-particle contact decementation adjacent to the interface required for the development of the shear zone. The maximum shear stress reached at ± 1 mm decreased from 132 to 110 kPa, between cycles 1 and 45. An initial dilation of 0.2 mm was followed by a cumulative contraction of 0.28 mm and a final normal stress of 65 kPa after 45 cycles (Fig. 10(b)). A characteristic S-shaped displacement pattern, indicated by the schematic in Fig. 4(b), develops within the first 5 cycles.

The shear zone is narrower than in the uncemented material (t074c), increasing from about 3.0 mm in cycles 1–5 to 4.6 mm in cycles 21–25 and 41–45 (Fig. 10(c)). High shear strains are concentrated within the lower portion of the shear zone (within 2 mm of the interface) and decrease slightly from 0.69 to 0.59 for cycles 1–5 and 21–25, respectively, due to the gradual increase in the shear zone thickness. Volumetric strains at the interface were also clearly defined and uniform within the shear zone, increasing from 0.006 to 0.014 between cycles 1–5 and 21–45 as complete decementation of particles within the shear zone was reached. The percent slip was similar for cycles 1–5 and 21–25, averaging 15.8% and varying between 11% and 21%. This is about 6% higher than in the uncemented test (t074c) (Table 2). For a cemented structure, slip within between the interface and the decemented material appears more favorable than further decementation of the intact material away from the interface. The transition from the shear zone to the upper part of the sample is pronounced (Fig. 10(c)), with the cementation in the upper part likely to remain intact. The smaller shear band of decemented soil has a lower secant shear modulus (G) compared to the uncemented test

Fig. 10. Test t076c (a) Shear stress vs. horizontal displacement, (b) Vertical displacement vs. horizontal displacement and (c) Vertical position vs. total horizontal displacement

(t074c), reducing from 439 kPa in cycle 1 to an average value of 370 kPa (Fig. 5).

Cyclic, ± 1 mm Displacement Controlled, Calcareous Sand, Uncemented (t075c)

The final cyclic test presented herein differs from t074c in that angular, calcareous LeGendre sand with a similar gradation was used. Visual inspections of compiled images and the specimen after testing indicates crushing within the shear zone due to the low hardness of calcareous sands and a higher degree of interlocking due to the particle angularity, respectively.

The initial shearing of the LeGendre sand to $+1.0$ mm resulted in a maximum shear stress of 140 kPa (Fig. 11(a)). Over this displacement the specimen initially contracted by 0.02 mm before dilating to 0.04 mm (Fig. 11(b)). The maximum shear stress achieved at

Fig. 11. Test t075c (a) Shear stress vs. horizontal displacement, (b) Vertical displacement vs. horizontal displacement and (c) Vertical position vs. total horizontal displacement

± 1.0 mm decreased to 71 kPa after 45 cycles. However, the maximum stress ratio observed at ± 1.0 mm averaged 1.2 for all cycles due to continuous decrease in the average normal stress per cycle. A total contraction of 0.35 mm resulted in a decrease in the normal stress to 20 kPa at 0 mm horizontal displacement, notably lower than observed with the IMDEX silica sand (t074c). The minimal initial dilation is indicative of increased particle rearrangement due to crushing as angularity would increase particle frustration and inhibit rearrangement.

PIV analysis shows that the shear zone is significantly larger than in silica sand but less sharply defined (Fig. 11). The average shear strains for cycles 1–5, 21–25, and 41–45 within 4 mm of the interface averaged 0.33, 0.22, and 0.17, respectively. The percent slip averaged 5.4%, 32.7%, and 47.0% for cycles 1–5, 21–25, and 41–45, respectively. Increased slip is attributed to increased par-

ticle rearrangement enabled by particle crushing adjacent to the fully rough interface. It is noted that the IMDEX silica sand epoxied to the interface surface is significantly harder than LeGendre sand and will not wear. Unlike previous observations, the upper bound of the shear zone decreases, from about 10 mm to 6 mm between cycles 1 and 21 beyond which it remains relatively constant. The thickness of the shear zone, and the horizontal movement due to compression of the ends of the shear box, are both higher than in the silica sand tests. The large shear zone in cycle 1 (10 mm) is attributed to higher shearing adjacent to the interface (minimal slip of 5.4%), transmission of displacements further within the specimen due to interlocking of the particle structure, and minimal initial grain crushing. Following the initial displacement reversals the upper zone rearranged into a more dense packing after which it exhibited behavior similar to that observed in previous tests.

The increase in slip with cycling due to particle crushing at the interface affects G (Fig. 5). As discussed previously, increased slip due to particle crushing and a larger shear zone results in a decrease in the shear strain within the shear zone, for a given displacement. Simultaneously, sample contraction results in a 50% reduction in the maximum shear stress achieved at displacement reversals. This results in a stable value of G for cycles 5 through 45, a trend similar to previous tests (Table 2). However, the magnitude of G values in test t075c (calcareous sand) is, on average, 35% larger than for test t074c (silica sand) (Fig. 5).

Average total vertical displacements of the upper region within each displacement reversal increased from about 0.10 to 0.13 mm with slightly higher displacements adjacent to the top platen indicating possible crushing due to continuous changes in the normal stress. The volumetric strains are similar between all cycles, decreasing from about 0.02 adjacent to the interface to 0.0 at about 5 mm from the interface.

The presence of angular calcareous sand has a notable effect on the global interface behavior and the micro-mechanical behavior compared to subrounded silica sand. In carbonate sand a larger shear zone and particle crushing close to the interface coupled with increased slip at the interface with cycling resulted in accelerated degradation compared with silica sand.

CONCLUSIONS

Analysis of conventional global measurements and PIV element (patch) behavior for one monotonic and five cyclic CNS interface shear tests has provided new insight into interface behavior (Table 2). Noteworthy observations at both particulate and specimen (global) scales have been made:

- Three distinct regions of deformation may be identified within the interface shear apparatus; slip between the interface and the shear zone, the shear zone itself, and a region above the shear zone that undergoes horizontal compression against the end walls of the apparatus.

- The thickness of the shear zone varies with test conditions. For all uncemented silica sand tests (t059m, t074c, t062c, t072c) regardless of displacement mode, the shear zone thickness increases with cumulative displacement, from around 3.5 mm ($\approx 5D_{50}$) at 1.0 mm of displacement to around 5.5 mm ($\approx 8D_{50}$) at high cumulative displacements. When lightly cemented (t076c), the shear zone is smaller and more distinct from the upper part of the sample. In contrast, the LeGendre sand shear zone (t075c) is significantly larger, less well-defined, and decreases in thickness with continued cycling.
- Shear strains are constant within the lower portion of the shear zone. The magnitude of shear strain increases with monotonic displacement (t059m) and cyclic displacement magnitude (t062c, t072c). Generally, the magnitude of shear strain within the shear zone decreases with continued cycling regardless of cyclic displacement mode, soil type, or cementation.
- The percent slip is relatively constant throughout monotonic shear (t059m) and uncemented (t074c) and cemented (t076c) silica sand cyclic tests. At large cyclic displacements (t062c) and increasing cyclic displacements (t072c) the percent slip increases with cycling. The percent slip increases most notably for LeGendre sand due to crushing of particles adjacent to the interface.
- The shear zone secant shear modulus, G , varies with loading conditions, soil type and cementation (Fig. 5). For all tests G stabilizes with continued cycling, implying a steady state condition where any reduction in applied shear stress leads to a corresponding decrease in shear strain within the shear band. It is particularly notable that all tests on uncemented silica sand stabilize at similar values of G , regardless of loading method or displacement amplitude.
- Volumetric strains are uniform within the lower portion of the shear zone for monotonic shearing (t059m) and shearing at cyclic displacements up to ± 1.0 mm (t074c, t072c, t076c, t075c). At higher displacements the volumetric strain is non-uniform between cycles due to the continuous particle rearrangement within the shear zone (t062c, t072c).
- The contraction-dilation behavior in both global and PIV patch displacements are symmetric, with contraction and dilation occurring over disproportionate displacement ranges (Fig. 4(b), inset schematic). However, in each cycle the magnitude of contraction generally exceeds the magnitude of dilation, resulting in minor net contraction per cycle. It is this cumulative contraction which leads to a reduction in normal stress and hence shear stress (or friction) during cyclic loading.
- The total specimen dilation reflects volumetric strains within the shear zone rather than being localized between the interface and contacting particles attempting to overcome surface features. The total global dilation measured appears to be independent of specimen thickness, and is instead dependent on shear zone thickness.
- The dilation rate at small displacements during monotonic shearing (t059m) is significantly higher when calcu-

lated from local PIV displacements within the shear zone (0.28) than from conventional global measurements (0.08). This difference arises since global measurements of horizontal displacement include a contribution due to horizontal compression of the upper part of the sample. Further examination of the implications of this observation on the dilation angle determined from direct shear tests is necessary.

From the above microscale conclusions, it is evident that the cyclic degradation of shear stress under CNS confinement arises due to cumulative contraction of the shear band close to the interface. After reversal of the loading direction, the shear band briefly contracts before dilating steadily (Fig. 4 inset). For a given number of displacement-controlled cycles, a larger cyclic amplitude leads to higher shear stress since the dilatant stage outweighs the contractile stage, increasing the normal stress (c.f. t074c, t072c).

The magnitude of the contraction increases with increasing number of cycles. In load-controlled cycling, the increasing contraction is reflected in an increase in displacement amplitude with cycles (and a reduction in the stiffness, G , of the shear band), since a longer phase of dilation is required to recover the volume lost during the contractile phase (e.g. t062c). In displacement controlled cycling the increasing contraction is reflected in a reduction in normal stress and hence shear stress (e.g. t074c, t072c).

Cementation leads to a narrower and more distinct shear band, outside which the cementation is retained and the soil is rigid. This narrower shear band exhibits correspondingly reduced contraction and loss of normal stress (e.g. t074c, t076c).

Carbonate soil exhibits a thicker shear band than silica soil, and increased cumulative contraction, leading to greater degradation of normal and shear stress. In addition, the shear zone decouples from the interface with increasing number of cycles, perhaps due to a layer of fine broken particles forming (e.g. t074c, t075c).

Additional research is currently underway to quantify the effect of initial normal stress (σ_{no}) and stiffness (K) on the conclusions made herein and to assess more comprehensively the roles of cementation, particle angularity, and particle crushing.

These findings have provided initial insight into the primary microscale mechanisms that govern pile shaft degradation under cyclic loading. With additional research, it is anticipated that a framework can be developed to estimate the extent shear stress degradation is likely to occur given the field conditions. Understanding and quantification of the primary microscale mechanisms also provides the opportunity for more rigorous models based on fundamental soil behavior to be developed.

ACKNOWLEDGEMENTS

The funding for this work has been jointly supported through a United States National Science Foundation International Research Fellowship (#INT-0107341) and

the Australian Research Council Center for Offshore Foundations Systems at the University of Western Australia. The PIV analysis was performed in part through the support of Giken Seisakusho Ltd., which is gratefully acknowledged.

NOTATION

CNH	constant normal height
CNL	constant normal load
CNS	constant normal stiffness
d	pile diameter
G	secant shear modulus
G_0	small strain soil shear stiffness
h_t	shear zone thickness
K	normal spring stiffness
Δx	incremental global horizontal displacement
Δy	incremental global vertical displacement
δ_h	horizontal displacement
δ_v	vertical displacement
ε_v	volumetric strain
γ	shear strain
σ_n	normal stress
σ_{no}	initial normal stress

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