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Evaluation of local friction and pore water pressure evolution along instrumented probes in saturated clay for large numbers of cycles

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Abstract

In order to investigate local friction mobilization along piles submitted to cyclic axial loadings, a calibration chamber experiment is presented based on the implementation of instrumented probes in specimens of saturated clay. Typical results obtained with a piezo-probe are presented, allowing to measure, on top of tip resistance and local friction, the local pore water pressure (PWP) developed on the probe shaft. Specific piezometers installed in the clay specimen allow, in addition, to obtain a good picture of the PWP field around the probe during installation and loading.

After installation of the probe, a succession of monotonic and cyclic displacement-controlled loading phases are applied. Upon cyclic loading of the piezo-probe up to very large number of cycles (10^5 cycles), an initial degradation of local friction is observed, followed by a subsequent reinforcement, which keeps going until the end of the cyclic sequence. The friction evolution is related to the evolution of the PWP measured during cyclic loading. In particular, the influence of the cyclic loading sequence on the post-cyclic static friction resistance is evaluated. A comparison is finally made with the results obtained with another type of probe, showing a good consistency between both types of results.

Key words:

Pile behaviour, local shaft friction, excess pore water pressure, piezocone, cyclic loading, saturated clay, physical modelling, calibration chamber, instrumented probe

1 – INTRODUCTION

The design of piles submitted to complex transitory loadings like environmental loadings (wind action, wave action, earthquakes, etc.) and/or industrial loadings (rotating and vibrating machines) constitutes an important issue in geotechnical engineering related to the construction of important civil engineering structures like offshore oil production platforms or large size offshore wind turbines founded on monopiles.

In terms of shaft friction capacity, the understanding and evaluation of local friction evolution along pile shafts submitted to axial cyclic loading constitutes an important issue, which appears very important for the improvement of the design of piles submitted to such type of loading.

Concerning this issue, experimental research based on physical modelling approach and development of laboratory experiments appears to be very useful to better understand the mechanisms involved in friction mobilization under cyclic loading. This type of experiments allows to better understand, under well-controlled lab conditions, the mechanisms involved in the mobilization of local friction under cyclic loading.

This type of experiments have been widely used since the beginning of the years 80', very often in relation with the offshore oil production industry and results have been presented in the literature, based on the use of various types of probes representing sections of model piles, tested in soil specimens under various boundary and confinement conditions. Both sands and clays have been studied, most of the time for small to medium numbers of cycles, say less than 10^4 cycles.

Concerning sands, interesting results have been published by Chan and Hanna (1980), Lee and Poulos (1990), Al-Douri and Poulos (1995), Chin and Poulos (1996), Le Kouby et al. (2004), Lehane and White (2004), Tsuha et al. (2012). In most studies, the authors find a significant degradation of the mobilized local friction during application of the cycles.

Concerning saturated clays, interesting results have been published by Poulos (1981a), Matlock et al. (1982), Goulois et al. (1985) or Procter and Khaffaf (1987). More specifically, Poulos (1981a) presents the results of small-scale laboratory tests carried out on a model pile section (20 mm in diameter) loaded in reconstituted saturated clay specimens (152 mm in diameter), up to maximum number of 1000 cycles, showing that two-way displacement-controlled cycles cause significant reduction in pile-shaft skin friction which may result in failure of the model pile section. Poulos notes the significant influence of the cyclic displacement amplitude on the level of friction degradation.

Based on the results of axial cyclic displacement-controlled load tests carried out on a model pile section (2.5 cm in diameter) inserted into a confined soft clay specimen reconstituted in a calibration chamber (76.2 cm diameter specimens), Matlock et al. (1982) point out a significant reduction in mobilized friction during the application of the cycles (maximum number of 300 cycles).

Concerning the case of very large numbers of cycles (several 10^4 to 10^6 cycles, corresponding to fatigue behavior), very few if no publications are available. For sands, Bekki et al. (2013) have presented results showing the evolution of local friction along an instrumented pile-probe jacked into a sand specimen reconstituted in a calibration chamber, for large numbers of displacement-controlled cycles (10^5 cycles). The results obtained show that after an initial degradation phase of the local skin friction (cyclic strain-softening), a reinforcement phase

(cyclic strain-hardening) develops up to very high number of cycles (10^5). However, for the case of fine-grained soils, typically saturated clays, and large numbers of cycles, no publication has been found on the subject in the literature.

Within this context, the objective of this paper is to present specific results obtained on a laboratory prototype setup associated to a specific experimental procedure (physical modelling approach) aiming at studying the local friction behaviour of soil-pile interface submitted to very large numbers of axial loading cycles, in the case of reconstituted saturated clay. Measurement of pore pressure developed during testing on the instrumented probe used and within the soil specimen appears to be very interesting and helpful to better understand the friction evolution process in saturated clay. This research has been carried out within the framework of the French national research project SOLCYP (Puech et al. 2012)

After a description of the testing setup and of the experimental procedure developed, a typical test is presented, showing the interest of this prototype setup for better understanding the mechanisms of local friction evolution along piles submitted to large to very large numbers of axial loading cycles.

2 – TESTING SETUP AND EXPERIMENTAL PROCEDURE

2.1 The calibration chamber testing setup

The calibration chamber used in this research allows to reconstitute and to put under confinement uniform cylindrical soil specimens, 524 mm in diameter and 700 mm high. Figure 1 presents a 3D drawing showing the complete setup, the chamber itself being part of a loading frame which allows to install and load various types of probes. Figure 2 presents a general view of the experimental setup. The vertical stress is applied to the specimen by applying a water pressure, $p_{conf,v}$, through a large diameter bottom piston and the horizontal stress is applied, independently, through a water pressure, $p_{conf,h}$, which horizontally confines the specimen contained in a neoprene membrane. For sand testing, a comprehensive description of the equipment and associated testing procedure has been given by Bekki et al. (2013) and more details will be found in that paper.

As far as saturated clay testing is concerned, further developments have been made around the calibration chamber, and, in particular, the development of a large size consolidometer for reconstitution of the clay specimen, as well as prototype piezometers for measuring pore water pressures within the soil specimen, and the use of a specific instrumented probe

allowing to measure pore water pressure on the probe itself during loading. These elements are further described below.

2.2 Large size consolidometer for testing saturated fine-grained soils

The consolidometer developed is a rigid wall reservoir equipped with a top and bottom draining plates, which allows to reconstitute 524 mm diameter and 600 to 800 mm high specimens of saturated clay under K_0 conditions (no lateral deformation during the consolidation process), starting from a clay slurry (Fig. 3). The specimen is progressively consolidated by application of increasing force increments with the hydraulic jack visible in Figure 3 on top of the reservoir, until the selected vertical consolidation stress has been reached. A thin layer of silicon grease is applied on the inner wall of the consolidometer with addition of a deformable plastic film in order to minimize side friction during the consolidation process and ease the unmolding process of the specimen after transportation onto the calibration chamber base. Both applied force and vertical displacement of specimen surface are monitored during the consolidation process. More information concerning the consolidation process will be found in Muhammed (2015).

2.3 Instrumented probes used

Two instrumented probes have been used during the course of this work in order to study the evolution of local friction upon large numbers of cycles. The first probe, called friction-probe, without pore water pressure measurement, has been described in Bekki et al. (2013) (Fig. 4-b). This probe has a diameter of 36 mm (cross section of 10 cm²). It allows the measurement of tip resistance on an instrumented cone and local friction on a friction sleeve, 11 cm long with a surface of 124,4 cm², located on the shaft, at a certain distance from the tip (240 mm above the tip), which allows to minimize interactions between tip resistance and sleeve friction. The second probe, with pore water pressure measurement, called piezo-probe, is a piezocone probe which has been adapted and used as a pile-probe, in order to record, on top of local friction and tip resistance, pore pressure during installation and loading phases (Fig. 4-a). We will focus in the following on the measurements made with the piezo-probe.

Figure 4-a shows a simplified cross section together with a view of the piezo-probe. The piezo-probe has a cross section of 10 cm² (diameter of 36 mm). The conical tip (cone apex angle of 60°) is equipped with a 25 kN precision force transducer allowing the measurement of the tip resistance upon loading. The friction sleeve is located on the shaft, 50 mm away

from the tip apex. The friction sleeve is 13.5 cm long (friction sleeve area of 150 cm²) equipped with a ± 25 kN load sensor. The sensors allowing measurement of cone resistance q_p and local friction f_s are mounted on one single measuring body. The pore water pressure sensor (2 MPa measuring range) is located on the shaft, just above the tip (40 mm away from the tip apex), allowing to measure, through a porous ring, the pore water pressure evolution at the soil-probe interface. The pore pressure measuring circuit (water chamber and porous ring) are saturated with glycerine oil, following a specific procedure given by the manufacturer, before each test. The surface of the friction sleeve is specifically threaded in order to ensure a perfectly rough interface with respect to friction mobilization (average roughness $R_a = 0.4 \mu\text{m} \pm 0.25$).

2.4 Instrumentation of the clay specimen for pore pressure measurement

Specific piezometers have been developed in order to measure pore water pressures (PWP) within the clay specimen during the different testing phases. Six of these piezometers (numbered n°1 to n°6), are manually pushed into the specimen and installed at different locations and different depths within the soil specimen in order to pick up the pore water pressure distribution within the specimen and its evolution during the successive testing phases. Figure 5 shows a 3D drawing and a cross section of the piezometers (Fig. 5-b), as well as the locations within the soil specimen where the pore pressure is measured (locations n°1 to 6 corresponding to pore pressures $u_{r,1}$ to $u_{r,6}$, fig. 5-a). In figure 5, $r_1 = 11.9r_0$ where r_0 is the radius of the cone, $r_2 = 8.8r_0$, and $r_3 = 5.5r_0$. A specific protocol has been developed in order to ensure full saturation of the measuring circuit of each piezometer. After installation of the piezometers, the specimen is left alone for complete dissipation of the excess pore water pressure (EPWP) generated during this phase. The saturation of piezometers is checked by observing the excess pore water pressure generation and dissipation during the installation of each piezometer. A good time response of the system is obtained accounting for a good saturation of the measuring circuits.

2.5 Testing procedure

The soil specimen is first reconstituted and pre-consolidated in the consolidometer by applying successive increasing load increments up to the maximum value selected. The initial water content of the clay slurry is adjusted to about 87% (1.5 times the liquid limit of the clay). This process allows to reconstitute, under K_0 conditions, saturated uniform clay

specimens 524 mm in diameter and 600 to 800 mm in height. After reconstitution, the clay specimen is transported onto the calibration chamber setup and adjusted on the bottom piston of the chamber. After removal of the two part consolidation mold, a lateral confinement rubber membrane is adjusted around the specimen. The lateral confinement cell and the top cover of the calibration chamber are then adjusted and tightened. Final consolidation of the specimen is achieved by adjusting both vertical and horizontal confinement pressures. This allows to follow an anisotropic consolidation stress path close to the state of stress initially applied in the consolidometer. The specimen is then left alone for about 24 hours for complete dissipation of excess pore water pressure and final consolidation in the calibration chamber. Finally, the six piezometers (Figure 5-a) are installed in the clay specimen at different radial distances and depths from the vertical axis of the specimen.

3 – DESCRIPTION OF A TYPICAL PIEZO-PROBE TEST

3.1 Type of clay used

A reference industrial clay, a kaolinite called “Speswhite”, has been used for the test presented in this paper. Table 1 gives some of the physical properties of this clay, as determined by the authors. Table 2 presents basic Cam-clay model parameters for this specific clay, as also identified by the authors. Figure 6 gives the particle size distribution curves of Speswhite kaolinite.

3.2 Clay specimen characteristics

The clay specimen has been consolidated to a final vertical effective stress σ'_{v0} of 250 kPa and a final effective horizontal stress σ'_{h0} of 150 kPa corresponding to an estimated value of K_0 equal to 0.60. The initial consolidation of the specimen has been achieved in the consolidometer in five steps, corresponding to the application of successive vertical stresses of 5, 15, 45, 125 and 250 kPa. Figure 7 shows the corresponding settlement and vertical deformation versus time curves corresponding to the five successive load steps applied.

3.3 Installation of the piezo-probe

The piezo-probe was jacked into the clay specimen with the long stroke hydraulic jack at a constant displacement rate of 1 mm/s. The installation was performed in a single continuous step down to a depth of 420 mm, after which a dissipation test was conducted. The final configuration is such that the piezocone shaft is vertically centred within the clay specimen.

Measurements made on the piezo-probe

The piezo-probe response during installation, in terms of tip resistance, shaft friction, and pore water pressure (PWP) generation versus the tip vertical displacement are shown in Figure 8. After an initial rapid mobilization of the tip resistance (Figure 8-a), a fairly stable tip resistance plateau of about 0.48 MPa is then obtained after about 130 mm of tip penetration, accounting for a good homogeneity of the clay specimen and for the steady state nature of the penetration process. The sleeve friction mobilization (Figure 8-b) starts once the sleeve has penetrated into the soil, to progressively reach a plateau of about 17 kPa after 220 mm of tip penetration. Starting from a residual value of about 75 kPa, the penetration-induced PWP (Figure 8-c) reaches a fairly steady value of 325 kPa after 200 mm penetration into the clay specimen. It should be noted that this EPWP generation, resulting from a very high increase of total stresses, is local and decreases with the radial distance. The increase of total stresses (normal stresses and shear stresses) is due to deep penetration process as the soil is forced outwards to accommodate the volume of the probe. The effective stress field remains anyway positive around the probe. This interpretation is in good agreement with the statements published by Randolph (2003), Gavin and Gallagher (2005), O'Beirne (2016) and Hosseini and Rayhani (2017).

Once probe installation has been achieved, pore pressure measurements are still taken on the different transducers in order to capture the excess PWP evolution with time within the clay specimen. The excess pore water pressure dissipation curve measured on the piezo-probe is shown in Figure 8-d. Pore pressure dissipation takes place just after stopping penetration of the probe, reflecting a good level of saturation of the piezocone filter. It may be observed that the PWP value decreased rapidly and more than 90% of the excess PWP at the piezocone interface was dissipated after four hours.

Pore-water pressure distribution within the clay specimen

Figure 9 presents the evolution of pore water pressures measured on the different piezometers within the soil specimen at different radial distances from the probe. Fig. 9-a presents the evolution of u_{r1} , u_{r2} and u_{r3} , as defined in figure 5 (mid-friction sleeve level). The pore pressure starts building up on piezometer n°3 (u_{r3}), followed by u_{r2} and u_{r1} , which is fairly consistent. The pore pressure u_{r3} starts building up when the probe tip is at about 24 cm vertically from the piezometer which allows to obtain a fairly good evaluation of the size of the influence zone around the piezocone tip. The maximum value reached by u_{r3} is about

75 kPa, which starts decreasing as soon as the installation process has been stopped. The pore water pressures u_{r2} and u_{r1} keep increasing after installation has been stopped due to excess pore water pressure dissipation and redistribution, the values reached by u_{r1} being lower than u_{r2} , itself lower than u_{r3} , which is consistent. Fig. 9-b presents similar data for piezometers n° 4 and n° 6 located at the tip apex level, which are consistent with previously described data. The difference in the shape of the curve obtained for u_{r4} with respect to u_{r3} is related to the sudden stop of the tip when the latter reaches u_{r4} piezometer level (0.42m from the top). For u_{r3} piezometer, the probe first approaches the sensor, then reaches the sensor depth level and finally goes beyond the sensor depth level.

Figure 10 presents, for both mid-friction sleeve level and tip level, the pore water pressure distribution within the clay specimen in terms of isochrones, showing the evolution of the pore water pressure field around the probe during the consolidation process after installation of the probe. Very high PWP gradients may be observed close to the probe. The consolidation process is controlled by the coefficient of consolidation of the clay.

The pore water pressure distribution around the piezo-probe shows that for a r/r_0 ratio of 12, the excess pore water pressure generation is very small, showing that the installation process has little influence beyond that distance. This means that the clay specimen diameter of 524 mm ($r/r_0 = 14.55$) is sufficient to minimize boundary effects.

Based on the pore pressure measurements made on the piezo-probe and within the clay specimen with the piezometers, figure 11 presents a picture of the overall PWP field within the clay specimen around the piezo-probe for different stages of the probe installation process. This evaluation includes both generation and dissipation of excess pore water pressure (measured on the different transducers). This evaluation is based on the measurements made on the different transducers and on the following hypothesis : axisymmetrical hypothesis ; stationarity of the process, meaning that at any stage of the installation process, the PWP field around the probe remains unchanged (except for the very beginning of the installation process).

3.4 Initial monotonic or « static » loading of the piezo-probe

After installation and complete dissipation of the EPWP around the piezo-probe (about 24 hours), static loading tests were performed in order to obtain the local friction mobilisation curve up to failure. The same loading procedure as the one described in Muhammed (2015) for friction-probe testing was implemented in this study, allowing to make subsequent comparisons between piezo-probe and friction-probe results. Two successive displacement-

controlled quasi-static monotonic compression loadings are applied to the piezo-probe up to failure (4 mm total vertical displacement corresponding to about 11% of the piezo-probe diameter). The first static tests is performed at a displacement rate of 30 $\mu\text{m}/\text{min}$, the second one being performed at 300 $\mu\text{m}/\text{min}$, after a resting time of two hours, allowing complete pore water pressure dissipation to occur. This procedure allows to evaluate the influence of the loading rate on the mobilisation of local friction, in particular in terms of the influence of EPWP generation on the friction mobilization, the higher loading rate favouring the generation of higher EPWP. The hypothesis behind this is that for same loading rate, the loading curves should be very similar for successive loadings (past experience of the authors), the influence of stress history being therefore negligible with respect to the influence of loading rate.

Figure 12 presents the results obtained in terms of local friction and pore water pressure evolution versus probe displacement for the two successive loading tests.

In terms of local friction, the responses observed for the two loadings are very similar, with an initial rapid mobilization of friction followed by a clear plateau obtained after a displacement of about 600 μm . For the second loading, the initial response is more rigid and the plateau reached is a little higher than for the first loading (about 45 kPa reached for the second and 41 kPa reached for the first loading). These values of static friction are significantly higher than the friction values measured during installation of the probe at a penetration rate of 1 mm/s (17 kPa measured during the installation process). It must be noted that during the probe installation process, the excess pore water pressures developed are much higher than during the quasi static loading process and the volume of soil involved in excess PWP generation is also much larger during installation than during loading, which could explain the difference observed in terms of level of local friction reached between installation and loading.

In terms of pore pressure measurement, the responses observed are significantly different between both loadings. For the first loading (slow rate of 30 $\mu\text{m}/\text{min}$) there is an initial high rate increase of positive EPWP, followed by a decrease and a final stabilization. This shape accounts for the fact that the pore water pressure measured on the probe is a combination of an excess pore water pressure generation term and a dissipation term. At the beginning of the loading phase, the generation term is predominant, which explains the EPWP increase and after a certain time, the dissipation term becomes predominant, which explains the decrease. The pore water pressure response is completely different for the second loading with a continuously increasing pore water pressure, accounting for the faster loading rate and the

predominant effect, in this case, of EPWP generation, with respect to dissipation. This significant difference observed on pore water pressure generation however does not seem to affect the local friction mobilization, which appears to be controversial since due to higher PWP, the second frictional response should be “softer” than the first one. One possible explanation for this could be the fact that the friction mobilized at the interface does not depend only on the pore water pressure measured exactly at the interface but also on the PWP field within the interface zone which might result in normal effective normal stress values in this zone which are not so different in both cases.

Concerning EPWP generation, it is important to note here that for the two successive static loadings applied, the EPWP generated is positive, accounting for a normally consolidated state of the clay around the probe after installation (contractive response), which is consistent. It is also interesting to observe that the peak values of PWP reached in both static tests, 24 and 29 kPa respectively, are much smaller than the value reached upon the installation process (325 kPa).

Pore-water pressure evolution within the soil specimen during the two successive loading phases is presented in Figure 13 (mid-friction sleeve level, piezometers n° 1, 2, 3 and 5). It is interesting to note that it takes a certain amount of time before the pore pressure measured on the piezometers starts changing, due to the diffusion process. This also shows that the loaded zone of soil around the probe is much smaller than the zone involved during the installation process, which is normal (interface zone only involved during the loading process). These results are in good agreement with the results published by Potts and Martins (1982) and Randolph (2003).

The pore pressure responses on the different piezometers are qualitatively very similar for the two successive loadings with low values of maximum excess PWP reached.

3.5 Cyclic loading phase

Once the static loadings have been performed and full dissipation of excess PWP has been achieved, a displacement-controlled cyclic loading has been applied to the probe. Displacement-controlled cyclic loading allows to quantify, for a given displacement amplitude, the evolution of local friction at a given location corresponding to a given depth along a pile. Figure 14 shows the applied loading signal in terms of probe head displacement versus number of cycles (the probe is considered to be perfectly rigid and the probe head displacement is assumed to be the same as the local vertical displacement at the friction sleeve

level). For this test, 10^5 cycles have been applied at a frequency of 1 Hz and with a displacement amplitude ρ_c of $\pm 250 \mu\text{m}$ (alternated sinusoidal signal), which is a fairly high cyclic amplitude with respect to the probe size.

The results obtained during the cyclic loading test are shown in Figure 15 in terms of local friction and pore water pressure evolution versus number of cycles.

In terms of local friction, a rapid degradation of the local friction is observed from the very beginning of the test, which keeps going until cycle n° 40 (cyclic softening of the interface). The local friction then re-increase progressively on an average, up to the end the cyclic test (cyclic hardening of the interface) with a very slight relative re-decrease observed between cycles 300 and 2 000 and between cycles 20 000 and 50 000 (Figure 15-a). The cyclic strain-softening phenomenon is relatively classical and has already been described by different authors on both model tests and field tests (Chan and Hanna 1980; Poulos 1981b; Poulos 1982; Matlock et al. 1982). The subsequent cyclic strain-hardening phenomenon appears to be less classical and will be analysed below.

As far as the excess PWP evolution on the piezo-probe pore pressure transducer is concerned (Figure 15-b), a progressive increase of average excess pore water pressure is first observed during the application of the cycles, which keeps going until cycle n° 130 (with an almost constant PWP amplitude of 3 to 5 kPa). The maximum value of the generated excess PWP reached during this phase is about 126 kPa which is a very high value as compared to the maximum value obtained upon static loading. The EPWP then decreases progressively until the end of the test. Like for the monotonic loading (see Fig. 12-b), the PWP evolution is a combination of excess PWP generation and dissipation. During the initial phase of the test, the generation phenomenon is predominant (high generation rate of excess pore water pressure), which results in a global increase of the PWP. After cycle n° 130, the dissipation phenomenon becomes predominant, which results in a global decrease of the PWP until the end of the test. It is interesting to note that for very large number of cycles (10^5 cycles) a full dissipation of the excess PWP is finally observed (completely drained regime, amplitude of PWP equals to 1 kPa).

One would normally expect that the minimum of friction measured should correspond to the maximum of EPWP measured. The time delay between the two points is of approximately 90 seconds corresponding to 90 cycles. This small discrepancy could be due to the fact that the PWP is not measured exactly at the same point than the local friction and therefore the maximum of PWP actually acting on the friction sleeve could then not reach its maximum at the same time as the PWP measured on the filter (the maximum would occur before).

Figure 16 shows some typical cycles corresponding to both strain-softening (Fig. 16-a) and strain-hardening (Fig. 16-b) phases of the test. A substantial friction degradation is rapidly observed upon the strain-softening phase (Fig. 16-a) corresponding to a significant decrease of maximum and minimum values of friction mobilized on each cycle, which begins at cycle n° 1 and continues up to cycle n° 40. Upon push in phases, the maximum friction mobilized drops rapidly from 62 kPa to 24 kPa on the 40th cycle. Upon pull out phases, the evolution is similar (almost symmetrical cycles) with a rapid decrease from 41 kPa to 19 kPa. Figure 16-b shows typical cycles corresponding to the cyclic strain-hardening phase, which develops starting from cycle n° 40. It may be noted that for cycles n°40 to n°200, the rate of friction re-increase is significantly lower than the rate of friction decrease during the softening phase.

Pore-water pressure distribution within the clay specimen

Pore water pressure (PWP) evolution within the clay specimen upon the cyclic loading sequence is presented in Figure 17 for the mid-sleeve friction level (piezometers n° 1, 2, 3 and 5). Pore water pressures remain constant on all piezometers during the first 30 cycles. Then, PWP start increasing on piezometer n° 3 (u_{r3}) which is the nearest to the soil-probe interface. At this stage, the EPWP generated at the interface has diffused into the soil specimen and has reached the first piezometer. Then, the PWP starts rising successively on piezometers n° 2 and 1 due to the consolidation process. After reaching a maximum value, the PWP starts decreasing on all piezometers with a time delay between them, accounting for the predominant effect of the dissipation phenomenon for large numbers of cycles. The maximum value of PWP reached is the higher as the piezometer are closer to the probe interface, which is consistent. The evolutions obtained on piezometers n° 2 and 5 are very close to each other, which accounts for a good axisymmetrical distribution of PWP around the probe.

Figure 18 presents PWP isochrones showing the evolution of the PWP field around the probe at different times during the cyclic loading sequence. The evolution of the isochrones is fairly consistent. From cycle n°1 to cycle n°130, the PWP rises at the soil probe interface (piezo-probe measurement), accounting for the predominant influence of EPWP generation with respect to the dissipation process. It may be noted from the isochrones that the hydraulic gradient within the interface zone keeps increasing during this phase. Then, the PWP starts decreasing at the interface because the dissipation process becomes predominant. The hydraulic gradient starts decreasing and the PWP starts increasing at a certain distance from the probe (normalized radial distance above 5 to 6) corresponding to pore water redistribution.

For large numbers of cycles, a completely drained state is reached, for which EPWP have practically disappeared.

As far as the PWP evolution at tip apex level is concerned, similar trends have been observed, which are not reported here.

3.6 Final phase of static loading

Once the cyclic sequence has been completed, two post-cyclic static tests are performed in order to evaluate the influence of the cyclic sequence on the post-cyclic static frictional response of the interface. Both tests have been performed at a constant rate of displacement of 300 $\mu\text{m}/\text{min}$ up to failure (corresponding to 4 mm of probe displacement). The first static test has been carried out directly after the cyclic sequence, the second test being carried out after two hours rest after the first static test.

The corresponding results are presented in Figure 19. In terms of skin friction (Figure 19-a), a rapid mobilisation is observed upon the first post-cyclic static test, followed by a remarkably constant plateau of about 80 kPa, obtained after a displacement of about 400 μm . This plateau value of 80 kPa is significantly higher than the one observed upon the initial static test (40 kPa). This result accounts for a significant “reinforcing” effect, induced by the cyclic sequence, on the static friction. This reinforcement effect induced by the cyclic sequence, on the post-cyclic static response was also observed by Muhammed (2015) while testing a friction-probe installed in a saturated clay. During the second post-cyclic static loading, the mobilized friction (about 68 kPa) is slightly lower than the maximum value reached upon the first post-cyclic test.

In terms of pore water pressure (PWP) evolution (Figure 19-b), results show a very different type of response with respect to the one observed for the first monotonic loadings (Fig. 11), with a significant decrease of the measured PWP (generation of negative excess pore water pressure EPWP) at the clay-probe interface upon both post-cyclic static tests. This negative EPWP accounts for the modification of the soil-probe interface properties induced by the cyclic sequence. This negative EPWP corresponds to a dilative behaviour of the clay at the soil-probe interface, accounting for an overconsolidated state of the clay induced by the cyclic shear, as compared to the normally consolidated state observed before cycling (see section 3.4).

Pore-water pressure distribution within the clay specimen

PWP evolution within the clay specimen during both final monotonic loadings is shown in Figure 20. The PWP remains practically constant on all piezometers installed at the mid-friction sleeve level during both tests, showing that the slight negative excess PWP generated at the soil-probe interface does not have any significant influence on the PWP evolution at some distance within the clay specimen.

3.7 Analysis of the cyclic sequence

Different authors (Tali 2011; Bekki et al. 2013; Muhammed 2015) have defined and used a coefficient of friction evolution $C_{e,fs}$, as defined below, in order to easily quantify the evolution of local friction during cyclic loading and to make an easy distinction between degradation phases, corresponding to a decrease of $C_{e,fs}$ and reinforcement phases, corresponding to an increase of this coefficient, with respect to the initial mobilization of friction observed on the first cycle.

$$(1) \quad C_{e,fs} = \frac{f_{s,\max(i)} - f_{s,\min(i)}}{f_{s,\max(1)} - f_{s,\min(1)}}$$

where $f_{s,\max(1)}$ and $f_{s,\max(i)}$ are the maximum skin friction measured on first cycle and cycle i respectively (push-in phases), $f_{s,\min(1)}$ and $f_{s,\min(i)}$ being the values of minimum skin friction measured on the first cycle and on cycle i respectively (pull-out phases).

In the present work, the coefficient of evolution has also been adopted in order to further analyse and quantify the evolution of local friction behaviour during the cyclic sequence.

Figure 21 shows the coefficient of friction evolution versus number of cycles for the test presented. A clear degradation is observed from the very first cycles to cycle n° 50. Then a progressive re-increase of friction mobilization is observed on an average until the end of the test, with small (degradation) fluctuations. The minimum value of $C_{e,fs}$ is equal to 0.40 corresponding to a significant strain-softening of the interface. Then this coefficient increases again progressively to reach a final value of about 0.60, still accounting for a weakening of the interface with respect to the initial state.

The interpretation of this evolution may be related to the evolution of the PWP around the piezo-probe during loading, which has also been plotted in figure 21. The local friction at the pile-soil interface is controlled by the evolution of the effective normal stress acting on the probe shaft, σ'_n . The mobilized local friction at any time can be written as below:

$$(2) \quad f_s = \sigma'_n \operatorname{tg} \delta_{mob},$$

where σ'_n is the effective normal stress acting at the soil-probe interface and δ_{mob} is the mobilized friction coefficient, which varies between 0 and the interface friction angle δ . The progressive increase of excess PWP observed during the initial phase of the test results in a corresponding progressive decrease of σ'_n , which would induce the strain-softening observed on the mobilized friction. The decrease in excess PWP then observed would result in a re-increase of σ'_n , which would induce a progressive strain-hardening of the interface. The minimum value of $C_{e,fs}$ occurs at cycle n° 50 whereas the maximum value of excess PWP occurs at cycle n° 130 when one would expect the minimum value of $C_{e,fs}$ to correspond with the maximum of excess PWP. This difference of 80 cycles, corresponding to a short time duration of 80 seconds, could simply result from the fact that the measurement of PWP is done below the friction sleeve, the real average PWP acting on the friction sleeve being possibly slightly different from the one measured on the piezo-probe filter.

Also, the slight fluctuations observed for large numbers of cycles could result from PWP redistribution along the friction sleeve.

4 – COMPARISONS BETWEEN PIEZO-PROBE AND FRICTION-PROBE RESULTS

In order to further validate the measurements made, the results obtained with the piezo-probe have been compared to results obtained with the friction-probe described in paragraph 2.3.

The same testing condition (σ'_{v0} equal to 250 kPa and σ'_{h0} equal to 150 kPa) and loading protocol have been implemented for both types of tests (installation, initial static tests, cyclic sequence, final post-cyclic static tests). After probe installation and subsequent initial static tests, 10^5 displacement-controlled cycles have been performed at a displacement amplitude ρ_c of $\pm 250 \mu\text{m}$ and a frequency of 1 Hz. Figure 22 shows the results corresponding to the cyclic loading phase in terms of the coefficient of evolution versus number of cycles for both probes. Similar trends are found in terms of the evolution of the coefficient of friction versus number of cycles, with an initial degradation phase followed by a phase of progressive reinforcement, on an average, until the very end of the test. The minimum values of $C_{e,fs}$, corresponding to the maximum degradation, were reached after about 40 cycles for both probes with a $C_{e,fs}$ values of 0.42 and 0.45 respectively for the piezocone and friction-probe, which are fairly close to each other, thus validating both experiments.

5– CONCLUSIONS

In this paper, a physical modelling approach has been presented, aimed at studying the evolution of local friction along pile shafts embedded in saturated clay during static and cyclic axial loadings for large numbers of cycles. The experiment is based on the use of instrumented probes installed and loaded in specimens of saturated clay reconstituted in a calibration chamber. After installation of the probes, simulating a full displacement pile, the testing protocol involves successive phases of static and cyclic axial loadings allowing to quantify the influence of cyclic loading on the static frictional capacity of the pile shaft. Also, the evolution of local friction mobilization may be quantified during the cyclic sequence, showing, for the typical test presented, an initial degradation of mobilized friction (cyclic strain-softening) followed by a friction reinforcement (cyclic strain-hardening) which keeps going on up to very large numbers of cycles (10^5 cycles).

The evolution of pore water pressures (PWP) is measured on the probe itself and within the soil specimen during installation and loading, providing very interesting information which may help to better understand the mechanisms involved in the soil-interface shearing process. In particular, these measurements allow to obtain a good picture of the PWP field within the soil specimen during probe installation.

For the typical test presented, very high excess pore water pressures (EPWP) are measured on the probe and within the soil specimen during the installation phase, followed by a dissipation phase which is also recorded. Upon cyclic displacement-controlled axial loading, a significant accumulation of EPWP is first measured, accounting for a strong cyclic contractancy and a predominant effect of EPWP generation in the early cycles (nearly undrained response), which is then followed by a decrease of EPWP accounting for the subsequent predominant effect of PWP dissipation.

The comparison between response observed during the pre-cyclic static loadings and the post-cyclic static loadings allows to quantify the effect of the cyclic sequence on the static response up to failure, showing that the cyclic sequence induces a strain-hardening of the interface, with a stronger dilative response after the cyclic sequence, as compared to the weaker contractive response observed before the cyclic sequence.

Finally, the typical test presented has been repeated, showing a consistency of the experiment and of the testing protocol and it has been compared to tests results obtained with another prototype probe, without pore pressure measurement, but showing a good consistency with

the results obtained with the piezo-probe, in terms of local friction evolution during cyclic loading.

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Table 1 - Physical properties of Speswhite kaolinite

Property	Value
Mineralogy	kaolinite
Liquid limit (%)	58
Plastic limit (%)	28
Plasticity index (%)	30
Specific gravity (t/m^3)	2,64
Percent finer than $10\ \mu m$ (%)	98
Permeability coefficient k^*	$2 \cdot 10^{-9}$ m/s
Consolidation coefficient c_v^*	$3 \cdot 10^{-7}$ m^2/s

* Measured based on odometer test ($\phi 70$ mm) at a vertical effective stress σ'_{v0} equal to 250 kPa.

Table 2 – Cam-clay parameters for Speswhite kaolinite

λ	κ	M	ν	e_N at $p'=1$
0,18	0,03	0,81	0,20	2,25