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## Evolution of cyclic local friction along clay-pile interface for very large numbers of cycles: parametric study --Manuscript Draft--

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| <b>Abstract:</b>                                     | <p>In this paper, an experimental study is presented investigating the influence of cyclic displacement amplitude and effective consolidation stress on the evolution of mobilized local shaft friction along piles submitted to large number of cycles (up to <math>10^5</math> cycles). Two-way cyclic displacement-controlled tests were performed on an instrumented pile-probe installed and loaded in a calibration chamber. Tests were performed on reconstituted specimens of saturated clay to examine the shaft friction evolution in the soil-pile interface during cyclic loading. Displacement-controlled static tests were also performed before and after the cyclic loading in order to quantify the influence of cyclic parameters on post-cyclic static response. It was found that the amplitude of cyclic displacement and the initial state of stress have an influence on the evolution of local friction during cyclic loading. The degradation rate of local friction increased for larger cyclic displacement amplitudes whereas with increasing the effective consolidation stress, the degradation rate decreased. The application of displacement- controlled cycles resulted in a modification in the behavior of the interface. A significant peak of static friction followed by strain softening was observed during post cyclic static tests, which was not the case for pre-cyclic static tests. The peak value of friction obtained upon post-cyclic static loadings found to be more important for higher values of applied displacement amplitude during cyclic loading. Finally, a brief synthesis of the results is presented.</p> |

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# Evolution of cyclic local friction along clay-pile interface for very large numbers of cycles: parametric study

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**ABSTRACT:**

In this paper, an experimental study is presented investigating the influence of cyclic displacement amplitude and effective consolidation stress on the evolution of mobilized local shaft friction along piles submitted to large number of cycles (up to  $10^5$  cycles). Two-way cyclic displacement-controlled tests were performed on an instrumented pile-probe installed and loaded in a calibration chamber. Tests were performed on reconstituted specimens of saturated clay to examine the shaft friction evolution in the soil-pile interface during cyclic loading. Displacement-controlled static tests were also performed before and after the cyclic loading in order to quantify the influence of cyclic parameters on post-cyclic static response. It was found that the amplitude of cyclic displacement and the initial state of stress have an influence on the evolution of local friction during cyclic loading. The degradation rate of local friction increased for larger cyclic displacement amplitudes whereas with increasing the effective consolidation stress, the degradation rate decreased. The application of displacement- controlled cycles resulted in a modification in the behavior of the interface. A significant peak of static friction followed by strain softening was observed during post cyclic static tests, which was not the case for pre-cyclic static tests. The peak value of friction obtained upon post-cyclic static loadings found to be more important for higher values of applied displacement amplitude during cyclic loading. Finally, a brief synthesis of the results is presented.

**KEY WORDS:**

Pile behavior, local shaft friction, amplitude of cyclic loading, frequency, confining effective stress, saturated clay, physical modelling, instrumented pile-probe

## 1 – INTRODUCTION

1  
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3 During their lifetime, pile foundations are often subjected, in addition to permanent static loads,  
4 to variable transient loadings, which will sometimes modify their bearing capacity. These  
5 complex loadings, often identified under the generic term of “cyclic loadings”, can be  
6 characterized by various parameters like the amplitude of the cycles, their frequency, regularity,  
7 number of cycles, length of the sequences, etc. Understanding the behavior of piles submitted  
8 to such cyclic loadings constitutes an important aspect to be taken into account in the design of  
9 pile foundations. Over the past few decades, this subject has attracted considerable scientific  
10 interest and numerous research work has been published focusing on pile behavior under cyclic  
11 loading (Chan and Hanna [3], Lee and Poulos [9], Al-Douri and Poulos [1], Chin and Poulos  
12 [4], Le Kouby et al. [8], Lehane and White [11], Tsuha et al. [23], Poulos [19], Matlock et al.  
13 [12], Goulois et al. [6] or Procter and Khaffaf [21]).

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18 In most laboratory studies based on the use of physical modelling approach of pile-soil interface  
19 behavior, the authors usually applied displacement-controlled cyclic loading on their pile,  
20 which seems to be the appropriate way to study the evolution of local interface soil-pile friction  
21 (ex. Matlock et al. [12], Poulos [20], Procter and Khaffaf [21], Bekki *et al.* [2], Muhammed  
22 *et al.* [15]).

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25 Reported studies in the literature showed that the mechanical behavior of the soil–pile interface  
26 depends on various parameters like soil, interface and loading parameters. Poulos [19]  
27 conducted a number of small-scale laboratory tests on a model pile section (20 mm in diameter)  
28 in reconstituted saturated clay specimens (152 mm in diameter), up to maximum number of  
29 1000 cycles. According to this author, no degradation of skin friction occurs unless the half-  
30 amplitude of the cyclic displacement exceeds about 0.2% of the diameter. Above this value,  
31 increasing cyclic displacement amplitude leads to degradation and loss of skin friction. The  
32 amplitude of cyclic displacement required to initiate degradation may vary considerably from  
33 one test to another. Poulos [20] and Lee and Poulos [10] indicated that the critical amplitude of  
34 cyclic displacement, at which cyclic degradation is initiated, is related to the displacement  
35 required for full friction mobilization in a static load test.

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40 All laboratory test results published to date regarding the evolution of local soil-pile interface  
41 friction upon cyclic loadings for clays only address small to medium numbers of cycles, less  
42 than  $10^4$  cycles. To the authors’ knowledge, there are no published data showing the influence  
43 of cyclic parameters on the evolution of saturated clay-pile local friction for large numbers of  
44 cycles. However, in several cases, the foundation of structures can be exposed to cycles of  
45 loadings that can exceed  $10^4$  cycles (ex. offshore wind pile foundation). Investigating the  
46 evolution of local pile friction upon large number of cycles is therefore an important issue for  
47 better understanding the behavior of piles.

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52 Within this context, this paper presents the results of an experimental study aimed at better  
53 understanding the frictional behavior of a clay-pile interface under large number of cycles. The  
54 approach used is of the physical modelling type, using an instrumented prototype probe tested  
55 in a calibration chamber and loaded under various types of monotonic and cyclic loadings.

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60 Based on the results obtained, the influence of cyclic displacement amplitude as well as initial  
61 effective consolidation stress of the specimen, on the evolution of mobilized local shaft friction  
62 along the probe is investigated for large numbers of cycles (up to  $10^5$  cycles). Subsequently, the

1 effects of the cyclic sequences on the post-cyclic static response of the pile are presented and  
2 discussed.  
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## 6 **2 – TESTING SETUP, EXPERIMENTAL PROTOCOLS AND SOIL USED**

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9 The tests presented in this paper have been performed in the calibration chamber available in  
10 the geotechnical team of Navier laboratory at Ecole des Ponts ParisTech. The clay specimen is  
11 first reconstituted in a large size consolidometer and then transferred onto the calibration  
12 chamber for final consolidation. The corresponding detailed experimental protocols used for  
13 specimen preparation, application of stresses in the calibration chamber and for carrying out  
14 the loading tests with the pile-probe have been fully described by Muhammed [14],  
15 Muhammed *et al.* [15] and Muhammed *et al.* [16], who also report on the design of the  
16 instrumented pile-probe and the consolidometer. Therefore, a brief description only of the  
17 experimental setup and testing protocol is given in the following.  
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### 22 **2.1 - The calibration chamber testing setup**

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25 A 3D drawing of the calibration chamber test set-up is shown in Fig. 1. The chamber can hold  
26 soil specimens 700 mm high and 524 mm in diameter. Independent vertical and horizontal  
27 stresses may be applied to the specimens. This allows to apply isotropic or anisotropic initial  
28 states of stress or  $K_0$  conditions to the specimens, thus simulating the state of stress applied to  
29 a soil element at a given depth. The loading frame of the chamber is equipped with a long stroke  
30 classical 100 kN hydraulic jack, used for probe installation at constant displacement rate, and  
31 with a servohydraulic 100 kN actuator for carrying out precise displacement-controlled or  
32 force-controlled monotonic and cyclic loadings.  
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### 38 **2.2 - Large size consolidometer for reconstituting specimens of saturated fine-grained** 39 **soils**

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42 The consolidometer setup allows to reconstitute, starting from a slurry, saturated clay specimens  
43 524 mm in diameter and 600 to 800 mm high under  $K_0$  conditions (no lateral deformation is  
44 allowed during the consolidation process). The reservoir of the consolidometer is composed of  
45 two Plexiglas adjustable rigid halves equipped with top and bottom draining plates. A loading  
46 frame equipped with a double-action hydraulic jack, powered by a hydro-pneumatic pump,  
47 allows the application, during the consolidation process, of constant force increments on the  
48 top plate of the specimen. Fig. 2 shows a 3D drawing of the consolidometer set-up.  
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### 53 **2.3 - The instrumented pile-probe**

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56 The prototype probe used allows to make direct and independent measurements of tip resistance  
57 and local shaft friction representative of values occurring along a pile shaft. The probe has a  
58 cross section of 10 cm<sup>2</sup> (diameter of 36 mm), similar to standard penetrometers. The conical tip  
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1 of the probe is equipped with a 20 kN precision force transducer and the sleeve, 11 cm long  
2 (sleeve surface of 124.4 cm<sup>2</sup>), is equipped with a ±5 kN force transducer. The bottom part of  
3 the probe, including the friction sleeve, is threaded horizontally in order to provide a perfectly  
4 rough interface surface state with respect to friction mobilization. The average roughness of the  
5 rough part is  $R_a = 0.4 \mu\text{m} \pm 0.25$ . Fig. 3 shows a simplified cross section of the probe, together  
6 with different views.  
7

## 10 **2.4 - Experimental procedure**

### 13 *Specimen reconstitution*

15 The specimen preparation comprises two main steps: (1) the specimen is first pre-consolidated  
16 in the consolidometer, starting from a clay slurry, to a predefined value of vertical stress, under  
17  $K_0$  conditions; (2) the pre-consolidated specimen is then transferred and positioned on the  
18 bottom piston of the calibration chamber and consolidated to the final consolidation stress. The  
19 initial water content of the clay slurry has been fixed to about 1.7 times its liquid limit  
20 (Muhammed *et al.* [15]). The consolidation of the specimen in the consolidometer is achieved  
21 by progressively increasing the vertical load by increments (5, 15, 45, 125, 250, ...kPa) up to  
22 the maximum preselected value of the vertical load.  
23

24 Once the consolidation process has been completed, the pre-consolidated specimen is  
25 transported and carefully adjusted on the bottom piston of the calibration chamber. The  
26 consolidation reservoir is taken out and a rubber membrane is adjusted around the clay  
27 specimen. The circular confinement wall is then adjusted around the specimen. The final state  
28 of consolidation stress can then be slowly applied to the specimen, by increasing independently  
29 the vertical pressure within the piston chamber and the lateral confinement pressure. This allows  
30 to follow an anisotropic consolidation stress path as the state of stress initially applied in the  
31 consolidometer. Further details about the experimental procedure used can be found in  
32 Muhammed *et al.* [15].  
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### 41 *Loading protocol*

42 A given test is composed of different successive sequences listed below:

- 43 (i) Installation of the pile-probe at constant displacement rate (1 mm/s) with the long  
44 stroke hydraulic jack;
  - 45 (ii) Pre-cyclic static displacement-controlled loading tests up to failure;
  - 46 (iii) Cyclic displacement-controlled sequence;
  - 47 (iv) Post-cyclic static displacement-controlled loading tests up to failure.
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## 53 **2.5 - Tested soil**

54 The tests presented in this paper have been carried out on saturated specimens of kaolinite  
55 Speswhite. The physical properties of this reference clay, as determined by the authors, are  
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1 listed in table 1. The particle size distribution curve of the deflocculated Speswhite clay is  
2 shown in Fig. 4.  
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### 5 **3 – PRESENTATION OF A TYPICAL TEST**

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8 A typical test is presented in this section. For this test (Test 1), the initial consolidation of the  
9 specimen has been achieved in five steps in the consolidometer, corresponding to successive  
10 vertical stresses of 5, 15, 45, 125 and 250 kPa. Then, the pre-consolidated clay specimen has  
11 been placed in the calibration chamber and submitted to a final vertical effective stress  $\sigma'_{v0}$  of  
12 250 kPa and a final effective horizontal stress  $\sigma'_{h0}$  of 150 kPa corresponding to an estimated  
13 value of  $K_0$  equal to 0.59. The permeability and consolidation coefficient of the clay under this  
14 state of stress have been determined based on oedometer tests, giving  $k = 2.7 \cdot 10^{-9}$  m/s and  
15  $c_v = 3.0 \cdot 10^{-7}$  m<sup>2</sup>/s.  
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#### 21 **3.1 - Installation of the pile-probe**

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23 The probe is initially pushed into the clay specimen with a constant penetration rate of 1 mm/s  
24 down to a penetration depth of 460 mm. This depth corresponds to a position of the probe for  
25 which the friction sleeve is practically vertically centered in the clay specimen. Measurements  
26 of tip resistance, local skin friction and total load head versus probe penetration during the  
27 installation sequence are presented in Fig. 5. The tip resistance mobilization is fairly rapid and  
28 a plateau is reached after 180 mm of tip penetration. The mobilization of local friction starts  
29 when the sleeve friction gets into the soil specimen at about 250 mm of tip penetration after  
30 which it increases progressively and reaches an almost stabilized steady state value.  
31 This stabilization of tip resistance and local skin friction accounts for a good homogeneity of  
32 the clay within the reconstituted specimen, thus validating the specimen preparation protocol.  
33 As far as the total head load is concerned, which accounts for the global mobilization of both  
34 tip resistance and friction along the probe shaft, a rapid increase is first observed up to 50 mm  
35 of the penetration depth. This rapid increase corresponds to the rapid mobilization of tip  
36 resistance. Then, a second phase of increase with a lower rate is observed accounting for the  
37 progressive increase of the friction surface in the sample.  
38

39 During all sequences of a given test, results of tip resistance, local friction and total head load  
40 are systematically recovered. This provides fairly good confidence with respect to the measured  
41 local values (tip and friction) since at any stage of the test, the values obtained in terms of local  
42 friction and tip resistance can be validated by the total head load measurements. However, as  
43 mentioned in the objectives of the paper, the focus will mainly be on results concerning local  
44 friction in the following.  
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#### 53 **3.2 - Pre-cyclic static loading of the probe**

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55 The pre-cyclic static (monotonic) loading tests are carried out in order to obtain the failure  
56 characteristics in terms of local friction and tip resistance. Two successive displacement-  
57 controlled static loadings are performed after full dissipation of the excess pore water generated  
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1 during pile installation (the probe is left unloaded before static tests for at least 12 hours). The  
2 first loading is performed at a displacement rate of 30  $\mu\text{m}/\text{min}$  while the second one is  
3 performed at a rate of 300  $\mu\text{m}/\text{min}$  (Fig. 6). A two hours rest was permitted after the first  
4 loading.  
5

6 A very rapid mobilization of local friction, followed by a plateau, was obtained in both cases  
7 (about 34 kPa). Similar results were reported by Mochtar and Edil [13] for a model pile tested  
8 in a saturated kaolinitic clay with similar characteristics. It is interesting to note that the results  
9 obtained in terms of failure characteristics are very similar for both tests and that the second  
10 loading is not significantly affected by the first one. It is also worth mentioning that the local  
11 friction mobilized upon initial static tests is almost two times higher than the friction mobilized  
12 during installation of the probe. This can be explained by the fact that during the installation  
13 phase, very high values of excess pore water pressure (EPWP) can be generated due to full  
14 displacement process, which reduces significantly the normal effective stress level acting at the  
15 soil-probe interface, resulting in lower values of local friction.  
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### 21 **3.3 - Displacement-controlled cyclic loading**

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24 In order to study the local friction evolution during the application large number of cycles, it is  
25 necessary to run displacement-controlled cyclic loading sequences. The key parameters for this  
26 type of loading are the displacement amplitude (alternated or non-alternated signal), the signal  
27 shape, the frequency of the signal and the number of cycles applied. Fig. 7 presents the results  
28 corresponding to Test1, for which  $10^5$  cycles have been applied. For this typical test, the cyclic  
29 displacement amplitude  $\rho_c$  was chosen equal to  $\pm 250 \mu\text{m}$  (alternated signal) and the shape of  
30 the signal is sinusoidal (Fig. 7(a)). The cyclic displacement amplitude chosen gives a high value  
31 of  $\rho_c/\rho_{peak}$  ratio (ratio between cyclic displacement amplitude and displacement required to  
32 mobilize full friction in a static load test). The high value of this ratio allows to better highlight  
33 the effect of this parameter on the evolution of local shaft friction during cyclic loading. The  
34 test frequency chosen (1 Hz) is intended to study the case of offshore or onshore piles subjected  
35 to low speed road traffic or railway traffic (generally varying between 0.1 and 10 Hz).  
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42 A rapid degradation process is observed from the very first cycle, which keeps going for about  
43 40 cycles (Fig. 7(b)). This degradation phase corresponds to cyclic strain-softening of the  
44 probe-soil interface. Then, a progressive recovery of mobilized friction is observed (strain-  
45 hardening) up to the end of the cyclic sequence with a slight “re-decrease” observed between  
46 cycles 300 and 900.  
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50 A coefficient of friction evolution, called  $C_{e,fs}$  (Bekki et al. [2], Muhammed et al. [15]), is  
51 introduced to clearly visualize the evolution of local friction mobilization during the application  
52 of the cycles. The degradation phases corresponds to a decrease of the value of  $C_{e,fs}$  and the  
53 reinforcement phases, if there is any, corresponds to an increase of the value of this coefficient.  
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$$56 \quad C_{e,fs} = \frac{f_{s,\max(i)} - f_{s,\min(i)}}{f_{s,\max(1)} - f_{s,\min(1)}} \quad (1)$$

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1 where  $f_{s,\max(1)}$  and  $f_{s,\max(i)}$  are the maximum skin friction measured on first cycle and cycle  $i$   
2 respectively (push-in phases),  $f_{s,\min(1)}$  and  $f_{s,\min(i)}$  being the values of minimum skin friction  
3 measured on first cycle and on cycle  $i$  respectively (pull-out phases).  
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7 Fig. 8 shows the evolution of  $C_{efs}$  versus number of cycles. Most of the degradation occurs  
8 within the first 40 cycles. The minimum value reached is about 0.44 which is a low value  
9 corresponding to a significant degradation level due to the large cyclic displacement amplitude  
10 selected. After the degradation phase, the reinforcement phase toward the end of the cyclic  
11 sequence can be observed with a slight decrease between cycle n° 300 to cycle n° 900.  
12 Muhammed et al. [15] have explained the behavior observed during the application of the cycles  
13 in terms of the evolution of the effective normal stress acting on the probe shaft,  $\sigma'_n$ . The local  
14 friction mobilized being equal to  $f_s = \sigma'_n \text{tg } \delta_{mob}$ ,  $\delta_{mob}$  being the mobilized friction coefficient. It  
15 is well known that the evolution of effective normal stress depends on the evolution of excess  
16 pore water pressure (EPWP). Any variation of the EPWP results in a variation of effective  
17 normal stress.  
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24 Since the tested soil is a kaolinite, characterized by a rather low permeability, one can expect  
25 that the applied cyclic shear at 1 Hz frequency will result in the development of EPWP close to  
26 the soil-probe interface. A heterogeneous field of EPWP will be created within a small thickness  
27 annulus around the probe, resulting in high hydraulic gradients and initiation of EPWP  
28 dissipation. Since the interface shearing is not fully undrained, the problem is therefore coupled  
29 with combination of EPWP generation and dissipation. There is, indeed, a competition between  
30 the excess pore water pressure generation mechanism due to “undrained” cyclic deformation of  
31 the clay around the sleeve, and the EPWP dissipation, which starts taking place from the very  
32 beginning of the cyclic sequence due to the radial hydraulic gradient created by the excess pore  
33 pressure field. During the initial phase of the sequence (small numbers of cycles), the generation  
34 mechanism should be predominant with respect to dissipation, which should result in a  
35 relatively rapid increase of EPWP, a corresponding decrease in the normal effective stress and  
36 corresponding decrease of mobilized friction (cyclic strain-softening). Then, the dissipation  
37 process should become predominant, resulting in a progressive decrease of the EPWP, re-  
38 increase of  $\sigma'_n$  and corresponding re-increase of mobilized friction (cyclic strain-hardening).  
39 This hypothesis is in full agreement with the results published by Procteur and Khaffaf [21] and  
40 Muhammed *et al.* [15].  
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### 49 **3.4 - Final phase of static loading**

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51 The influence of the cyclic sequence on the post-cyclic static response of the interface has been  
52 investigated by performing two post-cyclic static loading tests. The two post-cyclic static tests  
53 were performed at the same displacement rate of 300  $\mu\text{m}/\text{min}$  up to failure (4 mm of vertical  
54 displacement). The first post-cyclic static test was performed directly after the end of the cyclic  
55 sequence. The second post-cyclic static test was performed after a resting period of two hours  
56 after the first test, allowing pore water pressure equilibrium of the soil around the probe. Fig. 9  
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1 shows the evolution of mobilized local friction versus vertical displacement for the two  
2 successive post-cyclic static tests. A significant difference may be observed between the two  
3 loadings tests. For the first test, a sharp peak of friction (of about 54 kPa) is obtained for a tip  
4 displacement of about 500  $\mu\text{m}$ , followed by a rapid strain softening and a stabilization at an  
5 ultimate value of about 25 kPa. For the second test, the response observed is qualitatively very  
6 similar to the response observed for the pre-cyclic static tests. No peak is observed and full  
7 mobilization of friction is obtained for a very small displacement (about 100  $\mu\text{m}$ ) followed by  
8 a constant friction plateau of about 23 kPa. It is worth noting that the ultimate value of friction  
9 obtained at large displacements is very similar for both loadings.

10  
11 In order to quantify the influence of the cycles on the post-cyclic static response, the degradation  
12 factor of skin friction,  $D_\tau$  as defined by Poulos [20], has been used:

$$13 \quad D_\tau = \frac{\text{property after cyclic loading}}{\text{property for initial static loading}} \quad (2)$$

14  
15 It is interesting to note that the value obtained for  $D_\tau$  factor ( $D_\tau=0.70$ ) is fairly close to the  
16 value obtained for the coefficient of evolution  $C_{e,fs}$  ( $C_{e,fs}=0.72$ ) at the end of the cyclic sequence  
17 (cycle n° 10<sup>5</sup>) accounting for a good consistency between the two coefficients.

## 18 19 20 21 22 23 24 25 26 27 28 29 30 31 **4 – INFLUENCE OF CYCLIC DISPLACEMENT AMPLITUDE AND INITIAL** 32 **STATE OF SPECIMEN ON LOCAL FRICTION EVOLUTION**

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34 In order to evaluate the influence of cyclic displacement amplitude and initial state of specimen  
35 in terms of initial state of stress on the mobilization of local friction, a series of five tests has  
36 been carried out. All specimens were normally consolidated and were saturated with no  
37 application of backpressure. Table 2 summarizes the main characteristics of the tests carried  
38 out.

### 39 40 41 42 43 **4.1 - Influence of the cyclic displacement amplitude**

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45 A very important parameter involved in the evolution of local friction upon cyclic tests is the  
46 cyclic displacement amplitude,  $\rho_c$ , which controls the evolution of interface properties. Real  
47 piles are generally head-loaded in a force-controlled way (either monotonic or cyclic loading).  
48 However, locally, along the shaft and under cyclic loading, this head loading results, at a given  
49 depth along the pile shaft, in a cyclic displacement with a given amplitude during a certain  
50 number of cycles, controlling the interface properties evolution. The cyclic displacement  
51 amplitude is not the same at all depths along the pile shaft (usually larger toward the pile head  
52 and smaller toward the pile toe) depending on the rigidity of the pile and the head loading  
53 amplitude. The calibration chamber setup, allowing to run displacement-controlled loadings, is  
54 therefore an appropriate tool to quantify the influence of this parameter on the evolution of local  
55 interface frictional properties.

1 Three tests, Test 1, Test 2 and Test 3 (table 1), have been performed under the same initial state  
2 of stress applied to the specimen. These tests were intended to quantify the influence of the  
3 cyclic displacement amplitude on the evolution of mobilized local friction upon cyclic loading  
4 and subsequent post-cyclic static friction. The vertical and horizontal stresses applied to the  
5 specimen, the number of cycles and the loading frequency were kept constant while the cyclic  
6 displacement amplitude was varied from low to high values. Before presenting the behavior  
7 observed upon cyclic sequences, Fig. 10 shows the repeatability of the measurements upon pre-  
8 cyclic static tests for the three tests, indicating a good consistency between the results (with a  
9 maximum variance of 10% at failure).  
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11 The influence of the cyclic displacement amplitude on the evolution of skin friction upon cyclic  
12 loading is shown in Fig. 11 in terms of the coefficient of evolution. Three cyclic displacement  
13 amplitudes, i.e.,  $\pm 100 \mu\text{m}$ ,  $\pm 250 \mu\text{m}$  and  $\pm 500 \mu\text{m}$ , have been applied for Test 2, Test 1 and  
14 Test 3, respectively. These values of cyclic displacement amplitude represent low to high ratios  
15 of  $\rho_c/\rho_{peak}$  (ratio between cyclic displacement amplitude, applied during the cyclic sequence,  
16 and displacement required to mobilize full friction in a static load test). This Figure clearly  
17 shows that the skin friction degradation rate increases with increasing cyclic displacement  
18 amplitude. This increase of the degradation rate could be explained in terms of EPWP  
19 generation around the probe. In fact, higher values of cyclic displacement amplitude induces  
20 relatively higher values of EPWP resulting in higher decrease in the effective normal stress and  
21 corresponding higher decrease of mobilized friction (cyclic strain-softening). For high to very  
22 high values of applied cyclic displacement amplitudes, after the degradation phase, a  
23 progressive phase of reinforcement can be observed. This reinforcement phase can be attributed  
24 to the EPWP dissipation. In fact, for high values of cyclic displacement amplitude, the induced  
25 high values of EPWP around the sleeve will first increase rapidly and then decrease after  
26 passing through a maximum value due to the dissipation phenomena, which becomes  
27 predominant rapidly.  
28

29 After the cyclic sequence, two post cyclic static tests has been performed for each specimen at  
30 a displacement rate of  $300 \mu\text{m}/\text{min}$  (same loading procedure as for the typical test). The  
31 influence of the cyclic displacement amplitude on the evolution of skin friction upon post-cyclic  
32 static loading is presented in Fig. 12. Qualitatively, results are similar. For the three tests,  
33 a sharp peak of skin friction is obtained upon the first post-cyclic static loading followed by  
34 significant strain softening which keeps going with progressive stabilization for large  
35 displacements. However, quantitatively, a clear distinction of the post cyclic behavior can be  
36 observed for different cyclic displacement amplitudes tested. The test results show that the peak  
37 value of skin friction, obtained upon the first post-cyclic static test, increases with increasing  
38 the cyclic displacement amplitude applied during the cyclic loading. For the lowest cyclic  
39 displacement amplitude ( $\pm 100 \mu\text{m}$ ), the corresponding peak value is 43 kPa reached at 0.25mm  
40 of probe displacement. This value of local friction is fairly close the one mobilized upon pre-  
41 cyclic static tests while for the highest cyclic displacement amplitude ( $\pm 500 \mu\text{m}$ ), the  
42 corresponding peak value is 70 kPa reached after 0.95mm of probe displacement. It is  
43 interesting to note that the required displacement to reach the full mobilization of local friction  
44 (peak of local friction), upon post-cyclic static tests, also increases with increasing the cyclic  
45 displacement amplitude (Fig. 13).  
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1 One may attribute the variation in the values of the peak and required displacement to reach the  
2 peak, to the induced thickness of the modified zone around the probe after cyclic loading. The  
3 authors believe that higher cyclic displacement amplitude influences larger zone within the soil  
4 around the probe, thus resulting in a thicker modified interface zone. The soil within this zone,  
5 characterized as the modified zone of interface, undergoes shearing and variation of EPWP  
6 during the cyclic loading (coupled generation and dissipation phases). Recently,  
7 Muhammed *et al.* [17] showed that the soil within this zone presents a very different type of  
8 response upon post-cyclic static tests with respect to the one observed for the initial pre-cyclic  
9 static loadings. These authors have observed a significant decrease of the measured PWP  
10 (generation of negative excess pore water pressure EPWP) at the clay-probe interface upon  
11 post-cyclic static tests accounting for the modification of the soil-probe interface properties  
12 after cyclic loading. The authors have also indicated that this negative EPWP corresponds to a  
13 dilative behavior representative of an overconsolidated state of the clay at the soil-probe  
14 interface due to the cyclic loading. Thus, after the cyclic loading, the thicker overconsolidated  
15 modified zone around the probe presents higher values of friction and requires necessarily more  
16 displacement to be sheared upon post-cyclic static test, as shown on the 2D conceptual scheme  
17 presented in Fig. 14.

18 In terms of friction degradation at the end of each loading stage, i.e. end of cyclic and post-  
19 cyclic static loadings, results show close values between the coefficient of evolution,  $C_{e,fs}$ , at  
20 the end of cyclic loading and degradation factor of skin friction obtained from static tests,  $D_{\tau}$ ,  
21 which is consistent and gives more confidence to the results obtained.

#### 32 **4.2 - Influence of the initial state of effective stress applied to the specimen**

33 Another parameter that can be investigated in calibration chamber is initial state of effective  
34 stress applied to the specimen. This parameter is representative of a given depth within the soil  
35 below the ground surface: the higher the effective state of stress and the deeper the soil element.  
36 In calibration chamber, the effect of this parameter can be investigated by changing the values  
37 of horizontal and vertical confining pressure applied to the specimen. This allows to study the  
38 behavior of a pile segment at any depth.

39 Results of three tests, Test 1, Test 4 and Test 5 (Table 1), with three different initial states of  
40 stress, have been compared, corresponding to the state of a pile segment at three different depths  
41 below the ground surface.

42 The same loading procedure as the one described in the typical test was implemented here again  
43 for successive loading sequences: installation phase, pre-cyclic static phases, cyclic sequence  
44 and for post-cyclic static phases. The influence of the initial state of effective stress on the  
45 evolution of local friction versus probe penetration during the installation and subsequent pre-  
46 cyclic static tests is presented in Fig. 15. For the three stress levels tested, the mobilized skin  
47 friction and initial stiffness increase linearly with increasing the applied effective state of stress,  
48 which is consistent. It is important to notice that, upon initial static tests, the required  
49 displacement to mobilize full local friction increases with increasing the effective consolidation  
50 stress level.

1 The influence of the initial state of stress on the evolution of skin friction upon cyclic loading  
2 is shown in Fig. 16. The cyclic displacement amplitude, the number of cycles and the loading  
3 frequency were the same for the three tests. Again, the comparison is made in terms of  
4 coefficient of evolution. This figure shows that the rate of local friction degradation decreases  
5 with the increase in the initial effective stress. This can be explained in terms of the  $\rho_c/\rho_{peak}$   
6 ratio, which dominates the amount of degradation of the interface (Poulos [20]). The higher the  
7 value of  $\rho_c/\rho_{peak}$  ratio and the higher the degradation rate. For the three cyclic tests presented  
8 here, the  $\rho_{peak}$  value increases with increasing initial effective state of stress (visible in Fig. 15b)  
9 resulting in lower values of  $\rho_c/\rho_{peak}$  ratio and corresponding smaller rates of friction  
10 degradation.  
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12 As far as the post-cyclic static responses are concerned, results show that the mobilized local  
13 friction (maximum value  $f_{s,peak}$  and ultimate value  $f_{s,lim}$ ) increases with increasing initial  
14 effective state of stress. However, the required displacement to mobilize full local friction after  
15 the cyclic sequence decreases with increasing initial effective state of stress. This could be  
16 explained, again, in term of thickness of the modified zone created after cyclic loading. Since  
17 upon cyclic loading, higher initial effective state of stress allows lower degradation. It is thus  
18 believed that, thinner modified interface zone is formed for higher levels of applied effective  
19 stress after the cyclic loading. This means that the modified interface zone created for low  
20 consolidation stress level ( $\sigma'_{v0}= 125$  kPa -  $\sigma'_{h0}= 72$  kPa) is thicker than the ones formed for  
21 medium ( $\sigma'_{v0}= 250$  kPa -  $\sigma'_{h0}= 150$  kPa) and high ( $\sigma'_{v0}= 420$  kPa -  $\sigma'_{h0}= 252$  kPa) consolidation  
22 stress level, requiring necessarily more displacement to be sheared upon post-cyclic static test.  
23 It is also instructive to estimate the degradation factor of skin friction from Fig. 17. The  
24 specimen with the higher initial state of stress corresponds to higher  $D_\tau$  factor value  
25 corresponding to lower amount of degradation. The estimated values of  $D_\tau$  factor are 0.58, 0.70  
26 and 0.88 for ( $\sigma'_{v0}= 125$  kPa -  $\sigma'_{h0}=72$  kPa), ( $\sigma'_{v0}= 250$  kPa -  $\sigma'_{h0}=150$  kPa) and ( $\sigma'_{v0}= 420$  kPa  
27 -  $\sigma'_{h0}=252$  kPa) respectively. It is interesting to note that there is a reasonable agreement  
28 between the degradation factor values and the coefficient of evolution values at the end of cyclic  
29 loading.  
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## 43 **5 –SYNTHESIS OF THE RESULTS**

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45 To better analyze the effect of cyclic displacement amplitude and initial state of stress applied  
46 to the specimen, on the local interface friction, the results of the present work are synthesized  
47 in this section. Concerning the local friction mobilization upon displacement-controlled static  
48 tests, Fig. 18 shows the evolution of maximum and ultimate values reached in terms of local  
49 friction upon static tests. In the same figure, a comparison is made with the maximum value of  
50 friction mobilized upon installation phase. Results clearly show a quasi-linear relationship  
51 between the local friction and initial state of effective stress (Fig. 18a and 18b). The higher the  
52 initial state of effective stress and the higher the values of mobilized local friction. Fig. 18a and  
53 18b also emphasize that the mobilized local friction upon installation phase is much smaller  
54 than the one mobilized upon initial static loadings (nearly half). This aspect has been explained  
55 by the fact that during installation phase, high local values of excess pore water pressure is  
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1 generated due to the high increase of total stresses during the deep penetration process as the  
2 soil is forced outwards to accommodate the volume of the probe. This, decreases the effective  
3 normal stress acting on the pile shaft and by consequent, the mobilized friction reduces. During  
4 static loadings, the applied displacement rate is significantly smaller and the corresponding  
5 EPWP generation within the soil is limited to a thin layer around the probe. The corresponding  
6 mobilization is then higher. This interpretation is in reasonable agreement with the results  
7 published by Randolph [22], Gavin and Gallagher [5], O'Beirne [18] and Hosseini and  
8 Rayhani [7] and Muhammed *et al.* [17].

9  
10 As far as the effect of cyclic loading parameters on the evolution of local friction is concerned,  
11 in Fig. 19, the relationship between the coefficient of evolution and the degradation factor with  
12 the applied cyclic displacement amplitude and the initial consolidation level is synthetized.  
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14 This figure has an important practical implication as it demonstrates clearly the effect of both  
15 parameters on the degradation of local skin friction. Fig. 19a synthetizes the effect of cyclic  
16 displacement amplitude on the evolution of the degradation factor and of the coefficient of  
17 evolution. The figure shows clearly that the minimum value of  $C_{e,fs}$ , corresponding to maximum  
18 degradation, decreases with the increase in the applied cyclic displacement amplitude, which is  
19 consistent and has been explained in terms of EPWP generation around the probe. The values  
20 of  $C_{e,fs}$  at the end of the cyclic tests, i.e. cycle number  $10^5$ , are fairly close for the three cyclic  
21 displacement amplitudes after converging towards a completely drained regime with full  
22 dissipation of EPWP and corresponding re-increase of effective normal stress. The values of  
23  $C_{e,fs}$  at the end of the cyclic tests are confirmed by the values of degradation factor  $D_\tau$ .

24 Fig. 19b synthetizes the effect of initial state of effective stress on the evolution of the  
25 degradation factor and the coefficient of evolution. This figure shows clearly that the minimum  
26 value of  $C_{e,fs}$ , corresponding to maximum degradation, increases with the increase in effective  
27 horizontal stress. This aspect has been explained in terms of the thickness of the influenced  
28 zone created by the cyclic loading, which should be smaller for higher effective horizontal  
29 stress. The values of  $C_{e,fs}$  at the end of the cyclic tests, i.e. cycle number  $10^5$ , and  $D_\tau$  also increase  
30 with the increase of effective horizontal stress thus confirming the above interpretations made.  
31 Finally, Table 3 gives a brief synthesis of the results in terms of the coefficient of evolution and  
32 degradation factor. A reasonable degree of consistency in the results can be observed.  
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## 45 6 – CONCLUSIONS

46  
47 Evolution of local friction mobilisation during static and cyclic axial loadings at various initial  
48 state of stress and at different cyclic displacement amplitude was investigated. The experiment  
49 is based on the use of an instrumented pile-probe installed and loaded in specimens of saturated  
50 clay reconstituted in a calibration chamber. The following conclusions can be drawn:  
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- 53 • The local friction mobilized upon installation phase is much smaller than the one  
54 mobilized upon initial static loadings.
- 55 • The local friction and displacement required to mobilize full friction in a static test  
56 increase with increasing the initial state of effective stress applied to the specimen.  
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- The cyclic degradation of local friction is mainly controlled by the cyclic displacement amplitude applied. For the same initial state of the specimen, number of cycles and cyclic frequency, the skin friction degradation is higher for higher values of cyclic displacement amplitude.
- The cyclic displacement amplitude causing major friction degradation can be directly related to the displacement required to mobilize full local friction in a static test.
- For the same cyclic displacement amplitude, number of cycles and frequency, the skin friction degradation is smaller for higher values of initial consolidation pressure.

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## Evolution of cyclic local friction along clay-pile interface for very large numbers of cycles: parametric study

### Figures

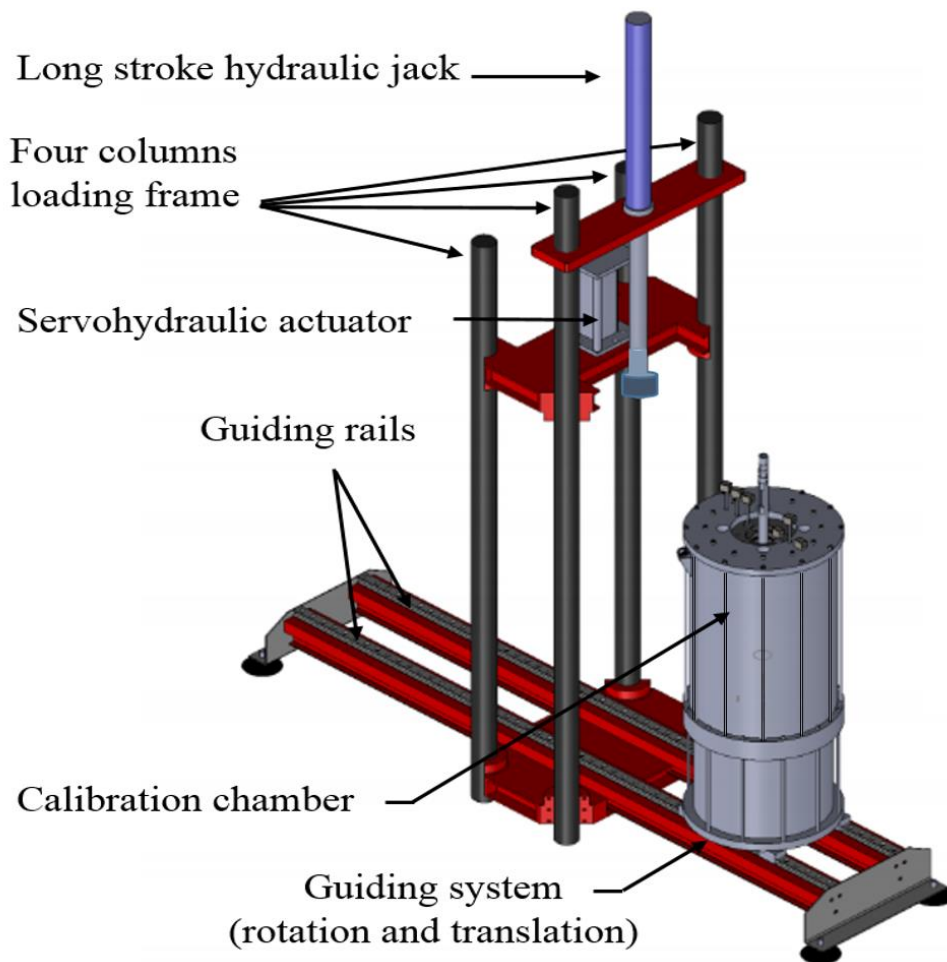


Fig. 1. General 3D view of the experimental setup

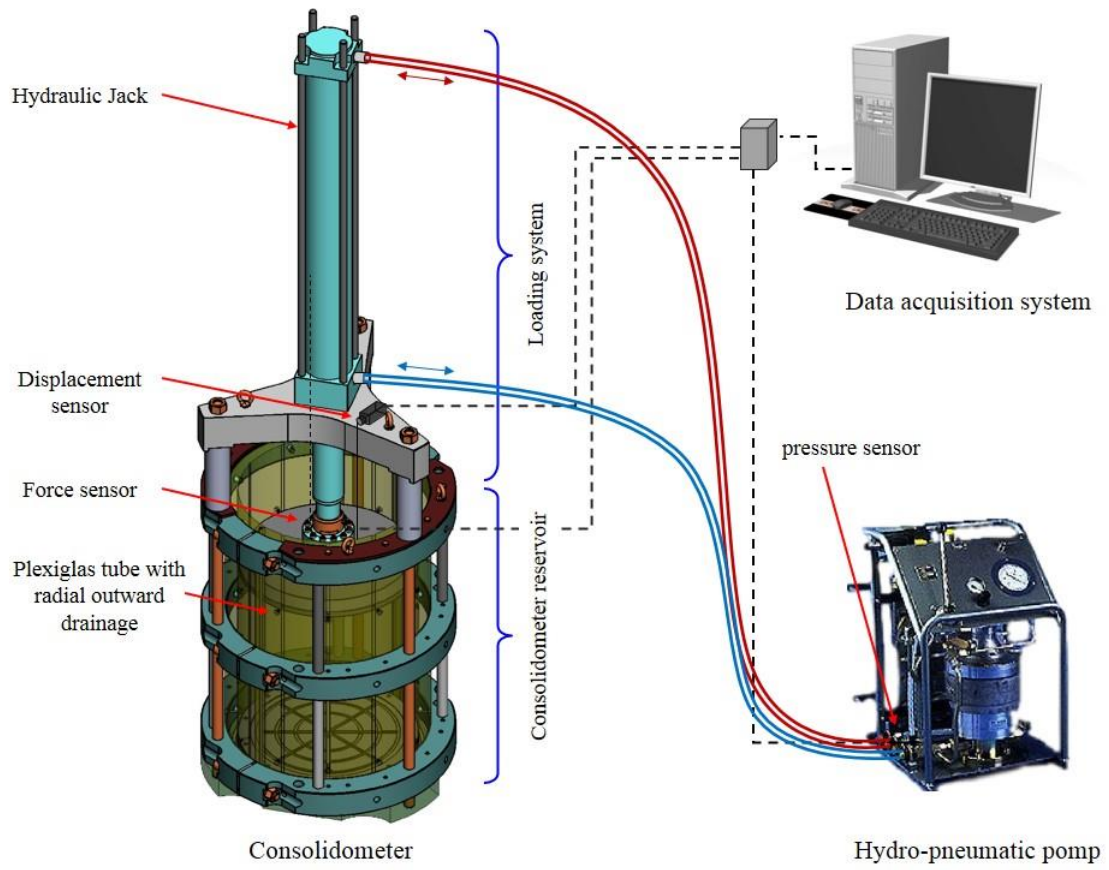


Fig. 2. General view of the consolidometer set up and ancillary equipment

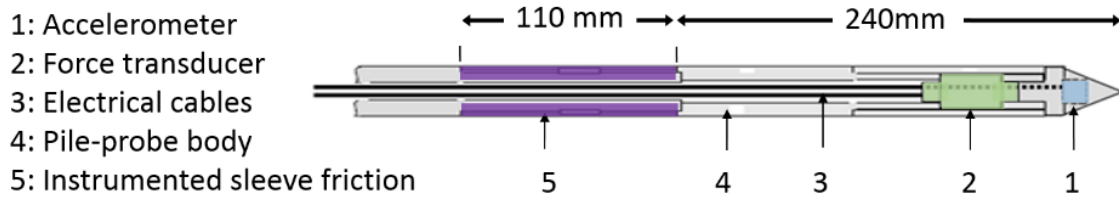


Fig. 3. Simplified cross-section and view of the instrumented pile-probe

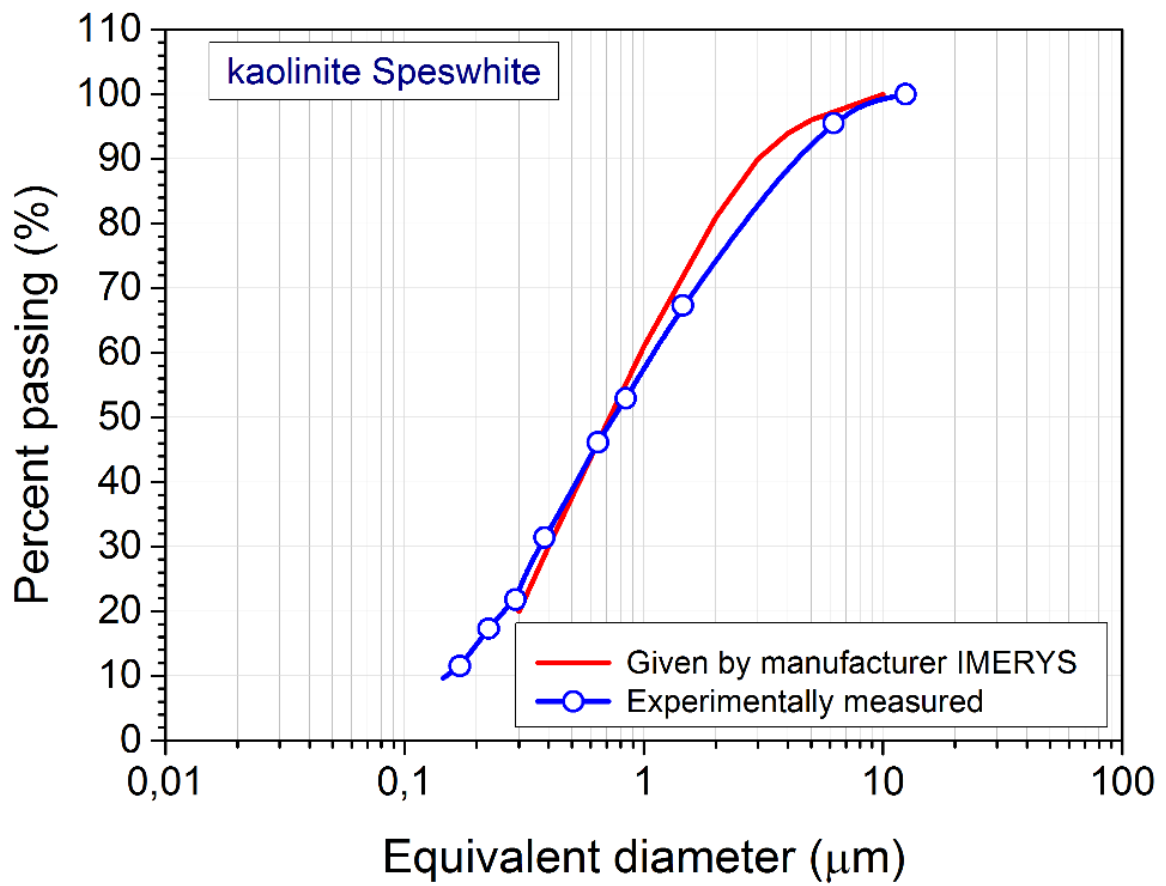


Fig. 4. Particle size distribution of Speswhite

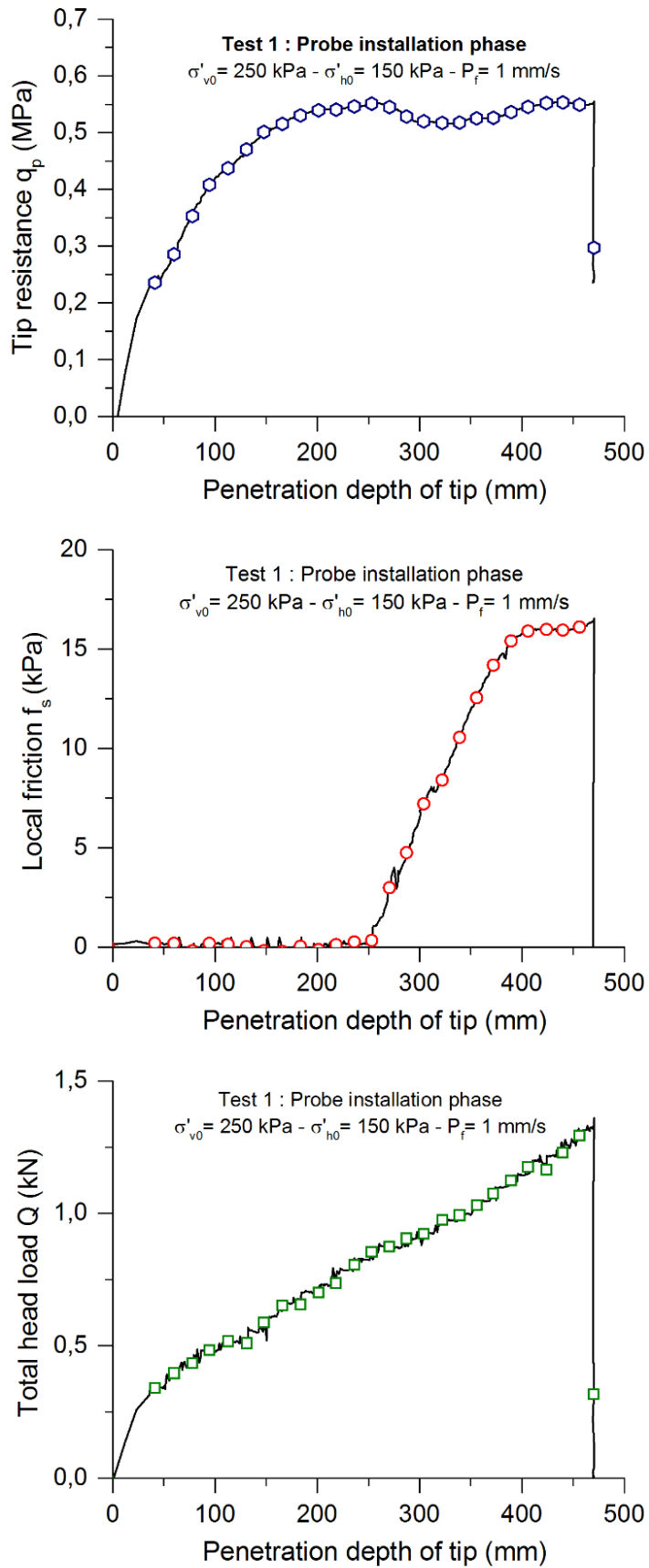


Fig. 5. Installation phase: (a) tip resistance versus penetration depth; (b) local friction versus penetration depth; (c) total load applied versus penetration depth



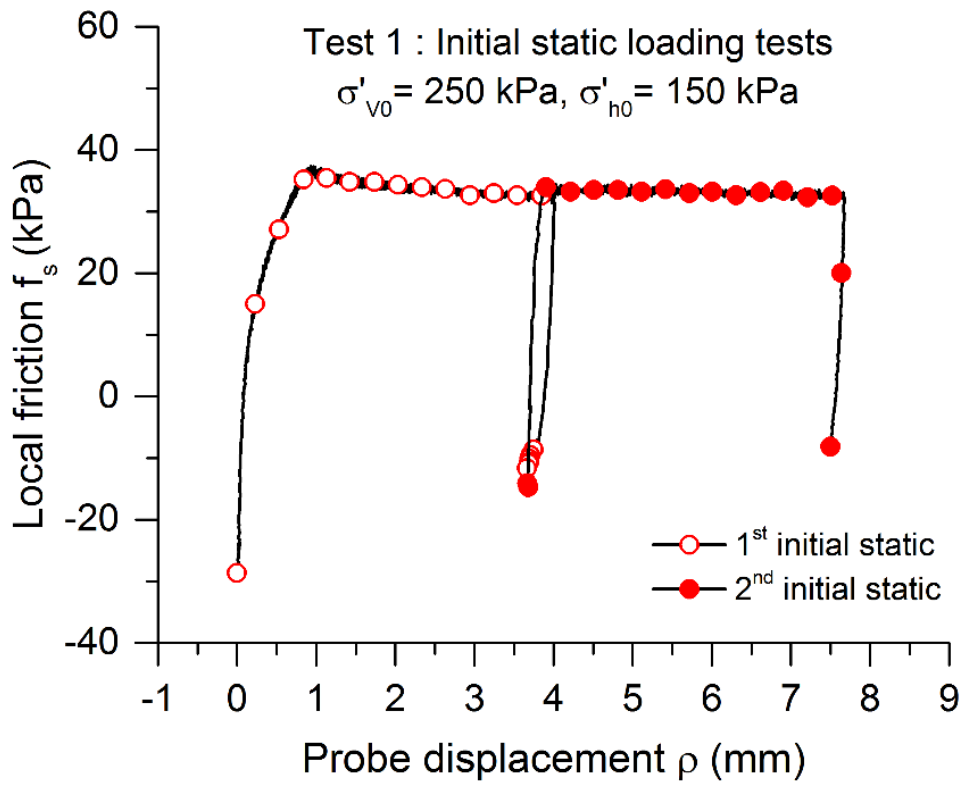


Fig. 6. Initial static loading tests: (a) tip resistance versus vertical displacement; (b) local friction versus vertical displacement; (c) total load applied versus vertical displacement

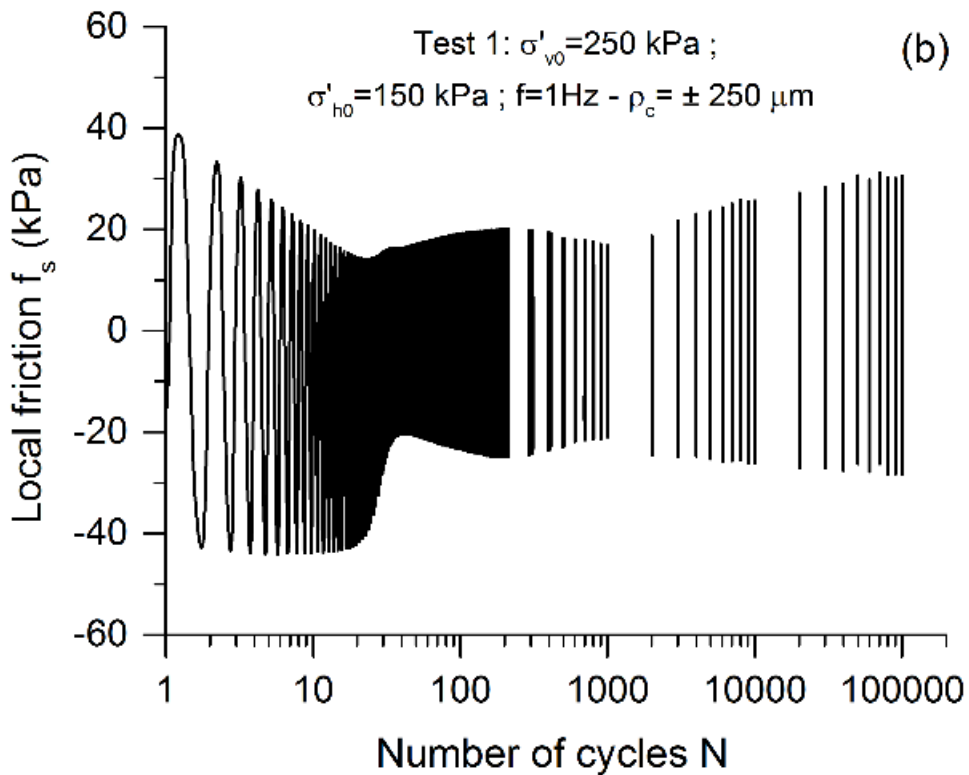
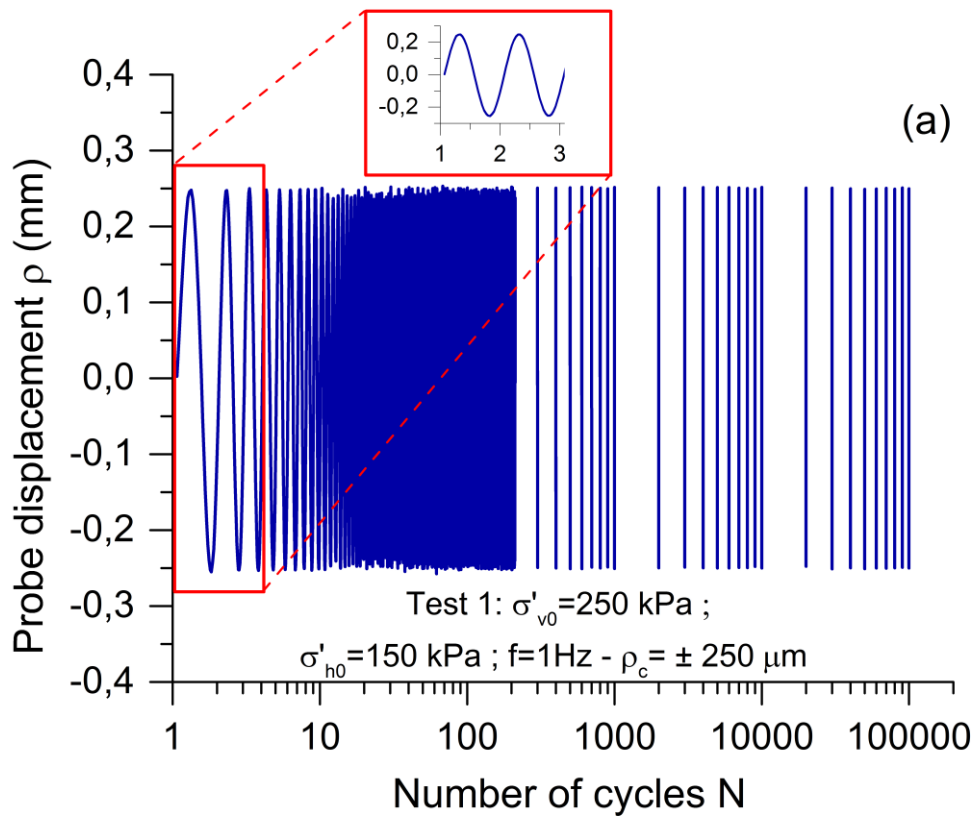


Fig. 7. Cyclic loading phase: (a) displacement-controlled loading signal versus number of cycles; (b) local friction versus number of cycles

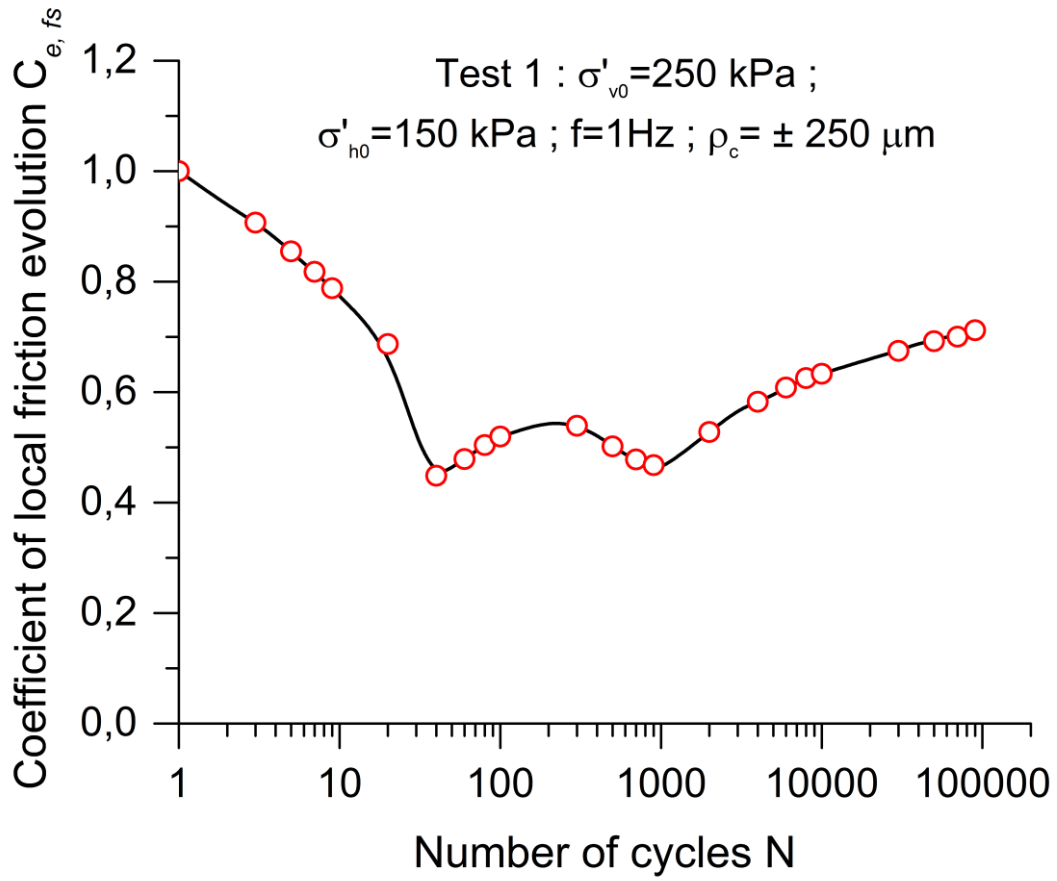


Fig. 8. Cyclic sequence: Coefficient of evolution ( $C_{e,fs}$ ) versus number of cycles

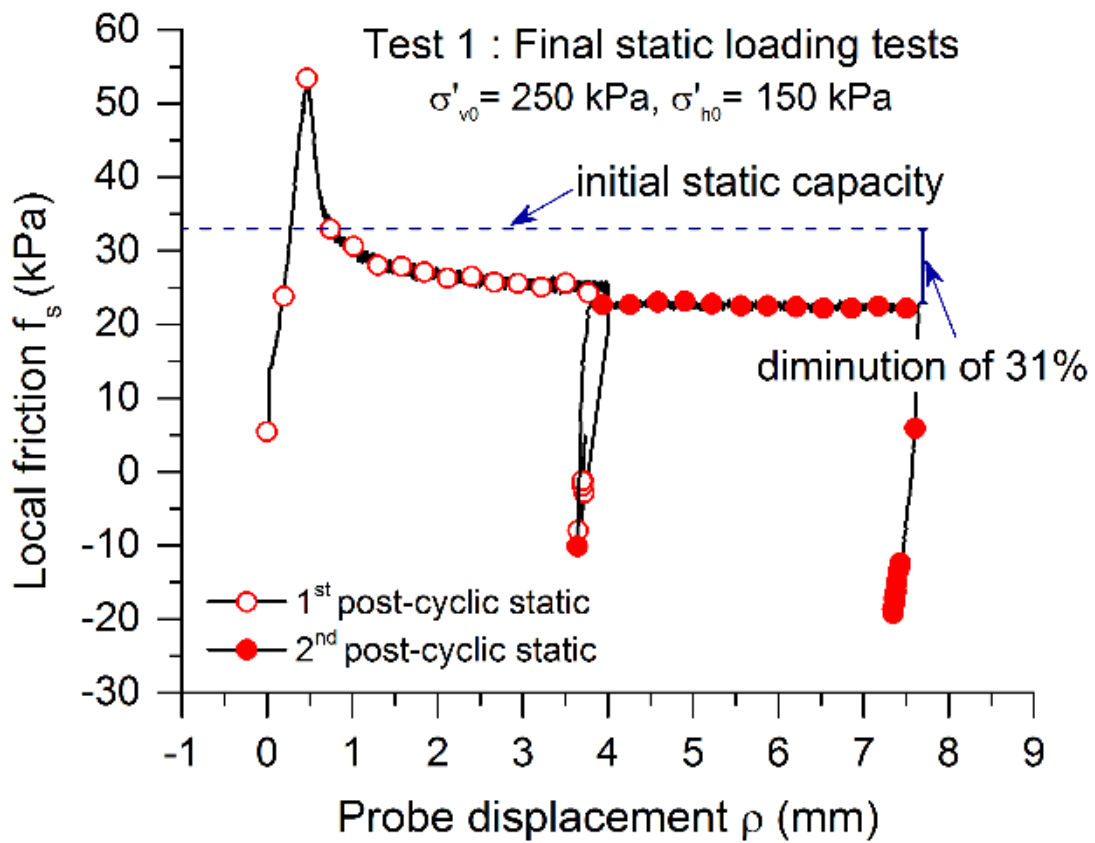


Fig. 9. Final static loading phase: (a) tip resistance versus vertical displacement; (b) local friction versus vertical displacement

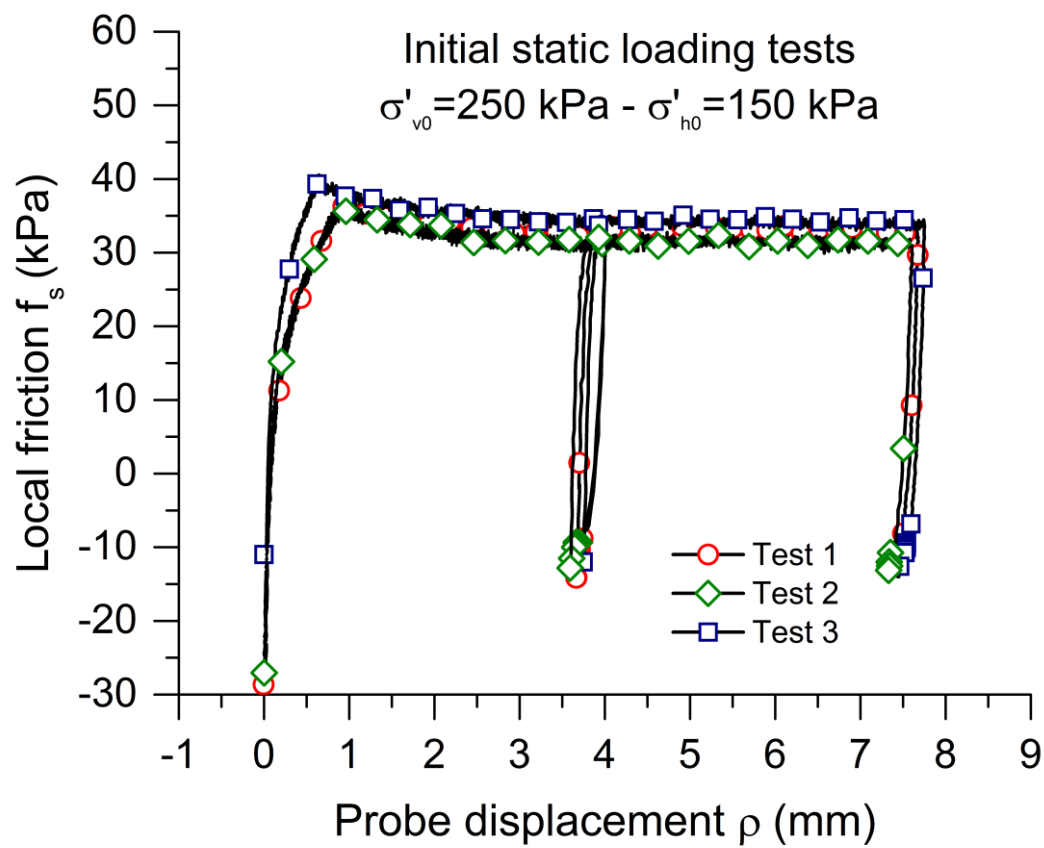


Fig. 10. Repeatability of pre-cyclic static tests

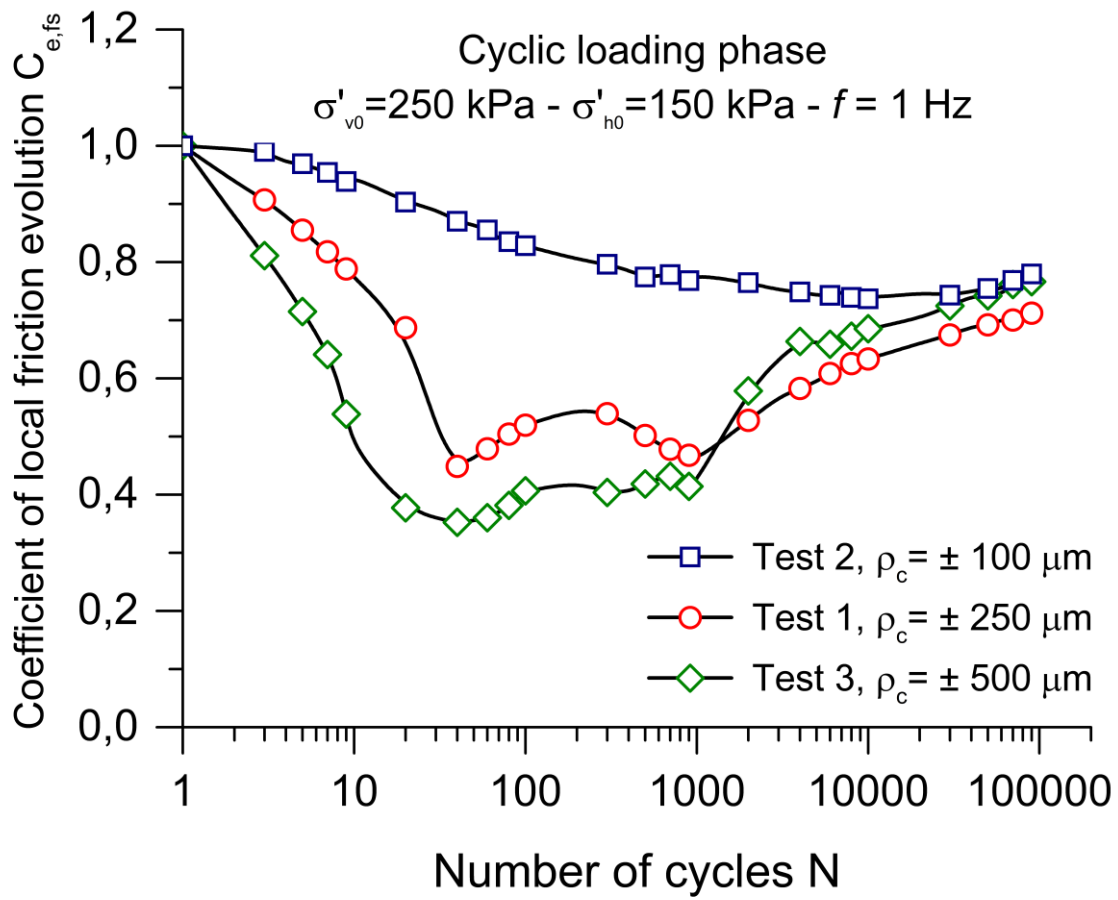


Fig. 11. Influence of the cyclic displacement amplitude on the evolution of local friction upon cyclic loading

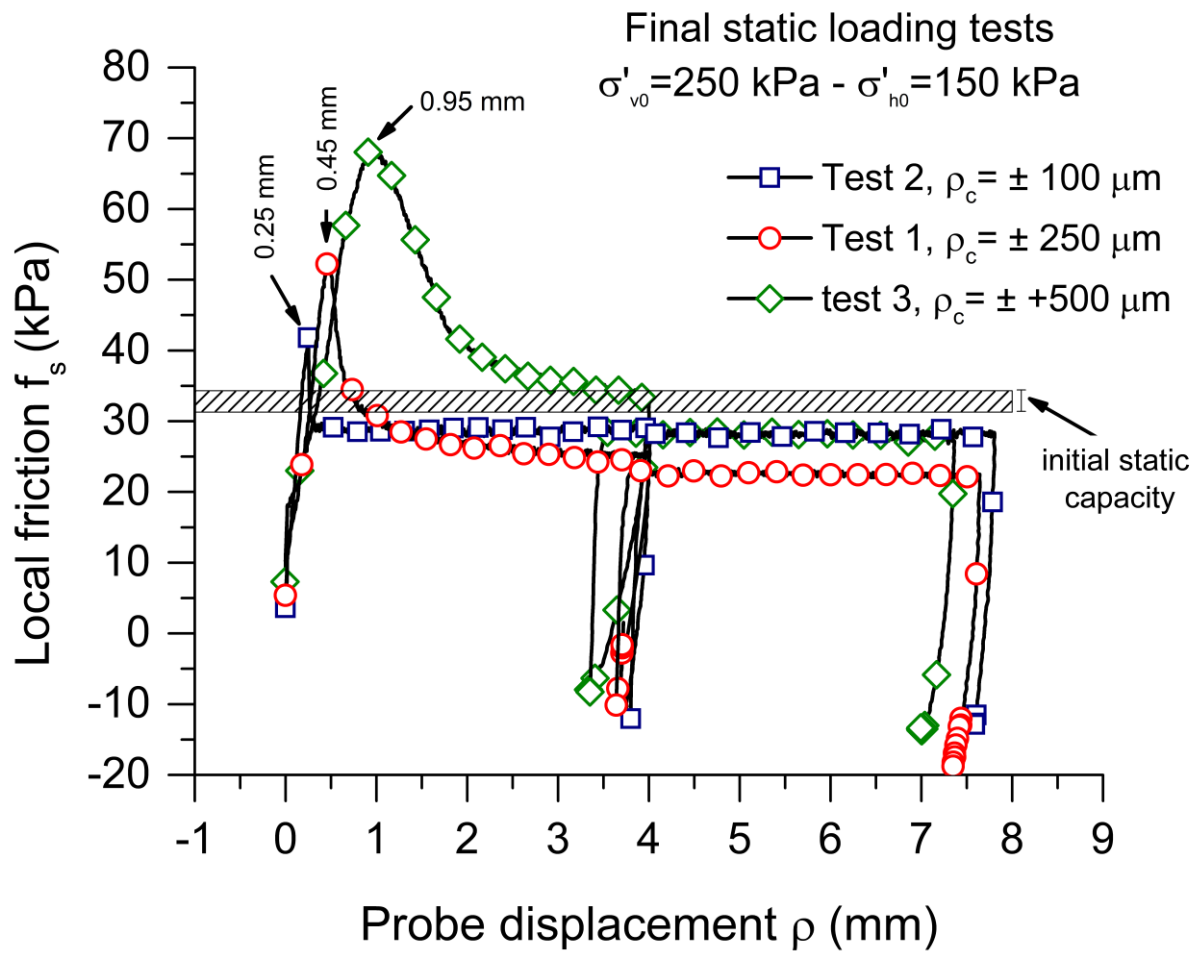


Fig. 12. Influence of cyclic displacement amplitude on the evolution of local friction upon post-cyclic static loadings

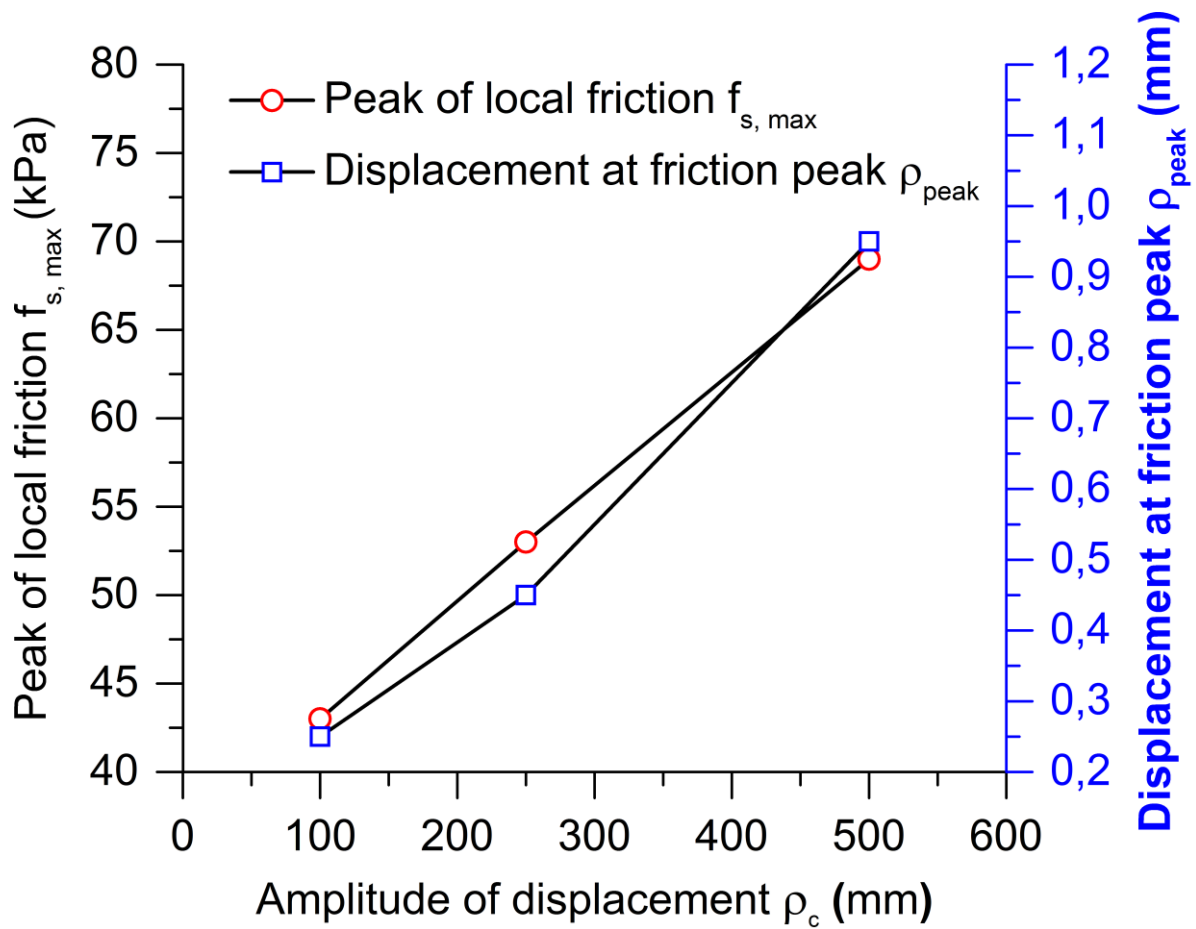


Fig. 13. Influence of cyclic displacement amplitude on the peak of local friction and the necessary displacement to reach the peak upon post-cyclic static loading



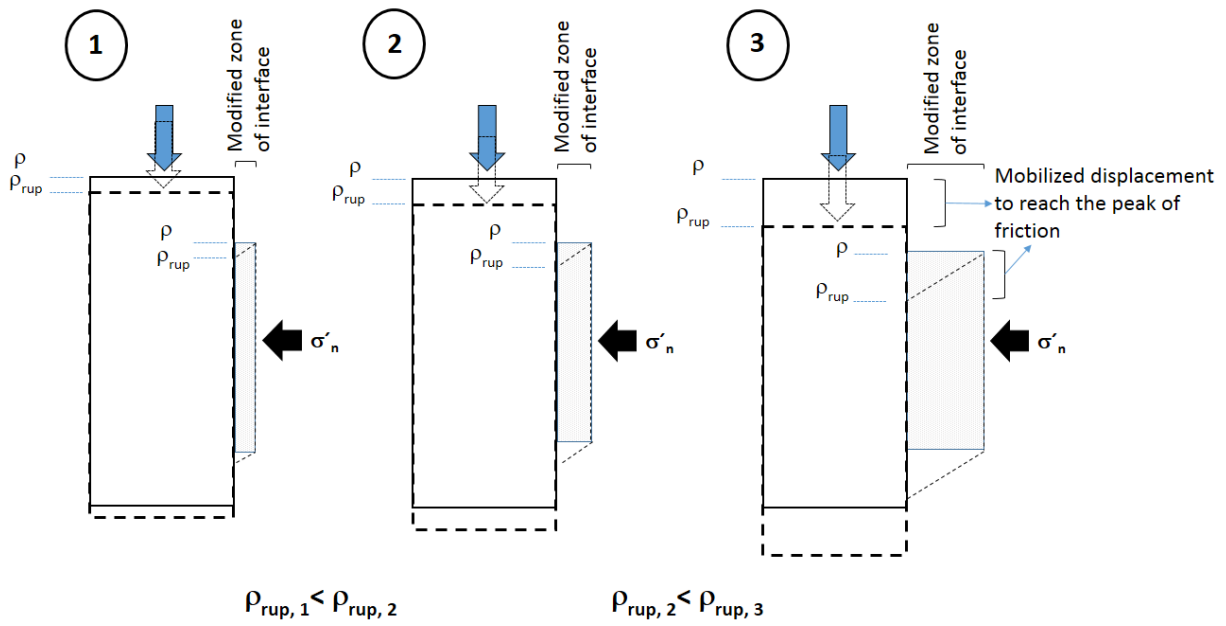


Fig. 14. 2D conceptual scheme of the influence of cyclic displacement amplitude on the modified zone of interface

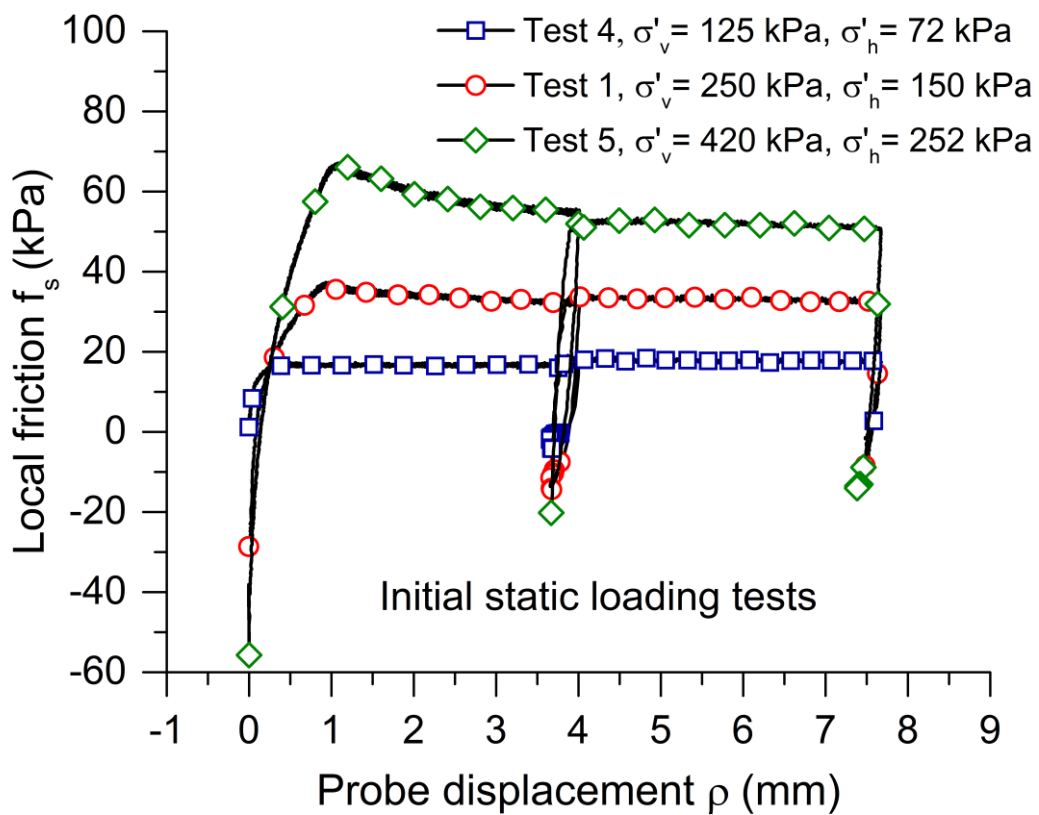
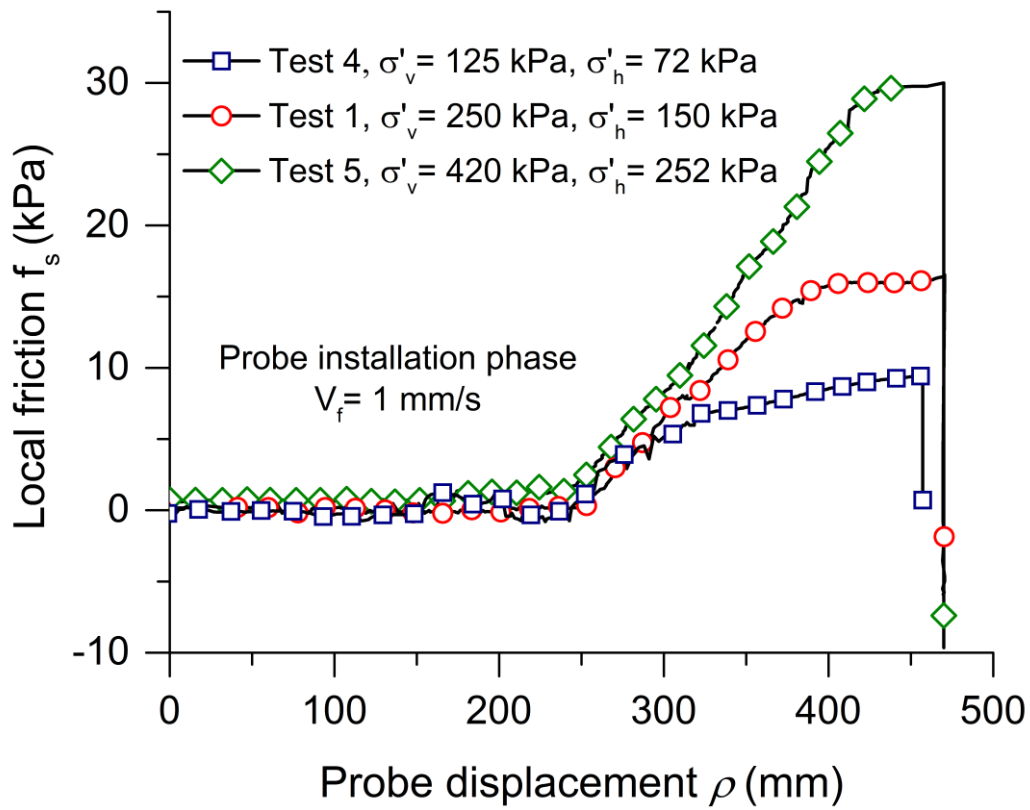


Fig. 15. Influence of initial effective consolidation stress level on the evolution of local friction upon installation and pre-cyclic static loading phases

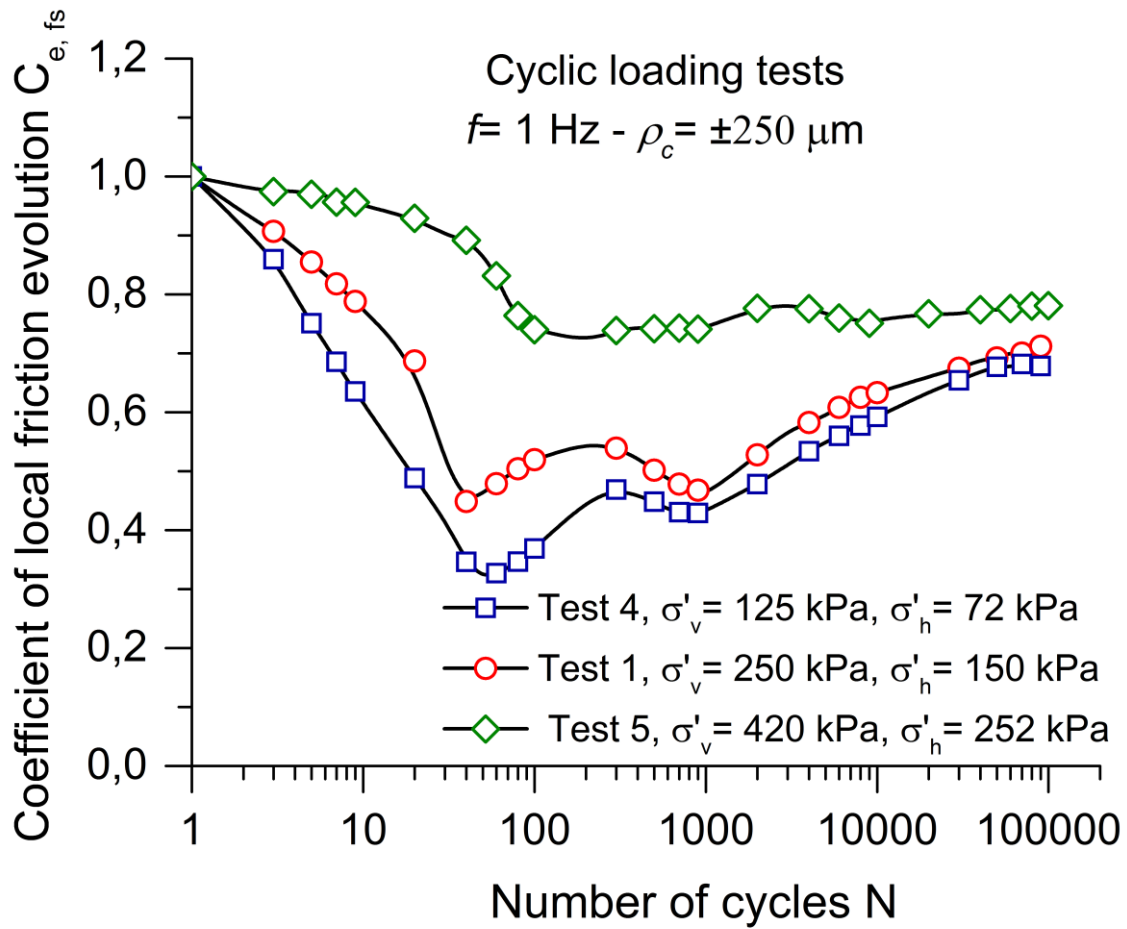


Fig. 16. Influence of initial effective consolidation stress level on the evolution of local friction upon cyclic loading

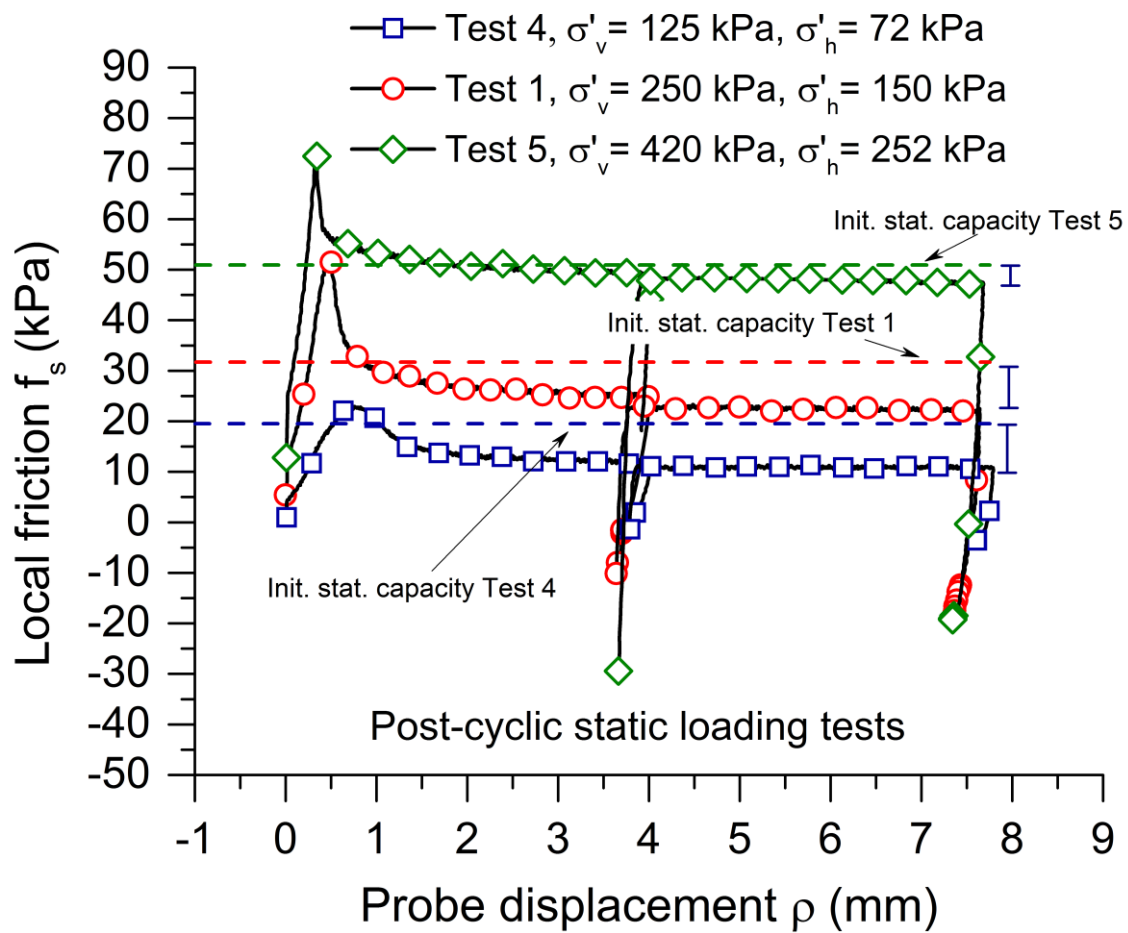


Fig. 17. Influence of initial effective consolidation stress level on the evolution of local friction upon post-cyclic static loading phases

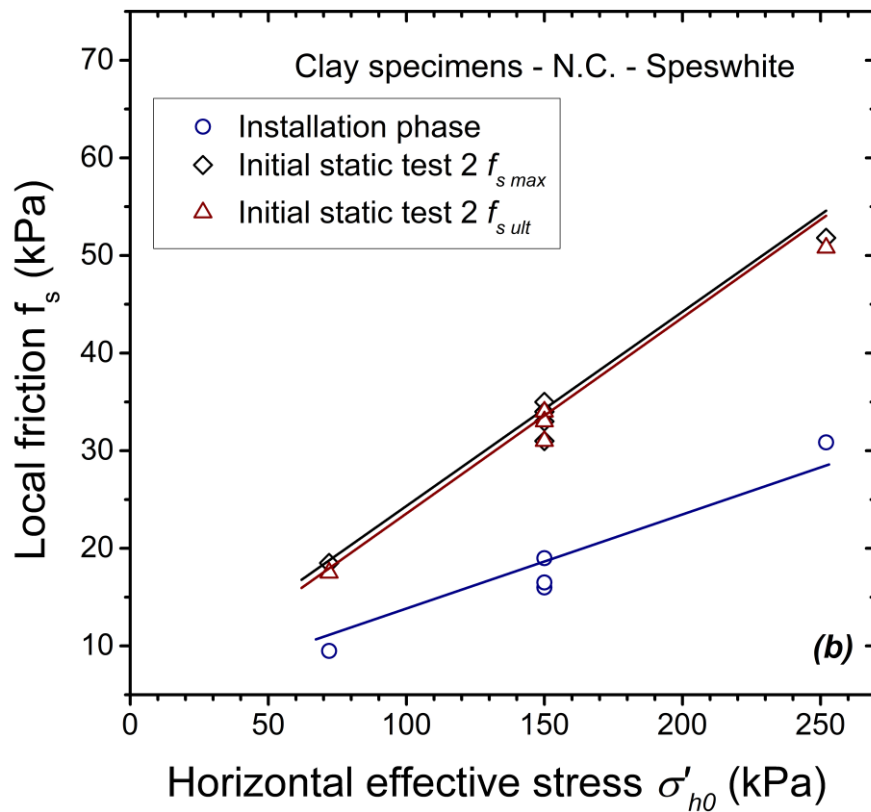
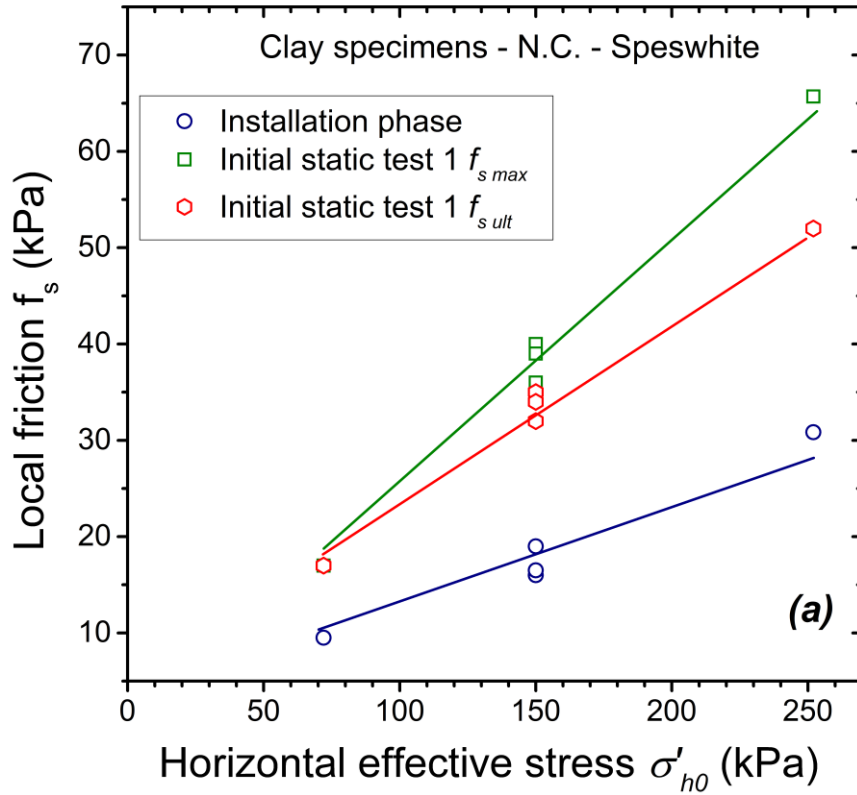


Figure 18. Synthesis of maximum and ultimate values of local friction upon: (a) 1<sup>st</sup> pre cyclic static tests and (b) 2<sup>nd</sup> pre cyclic static tests compared to the maximum mobilized friction upon installation phase

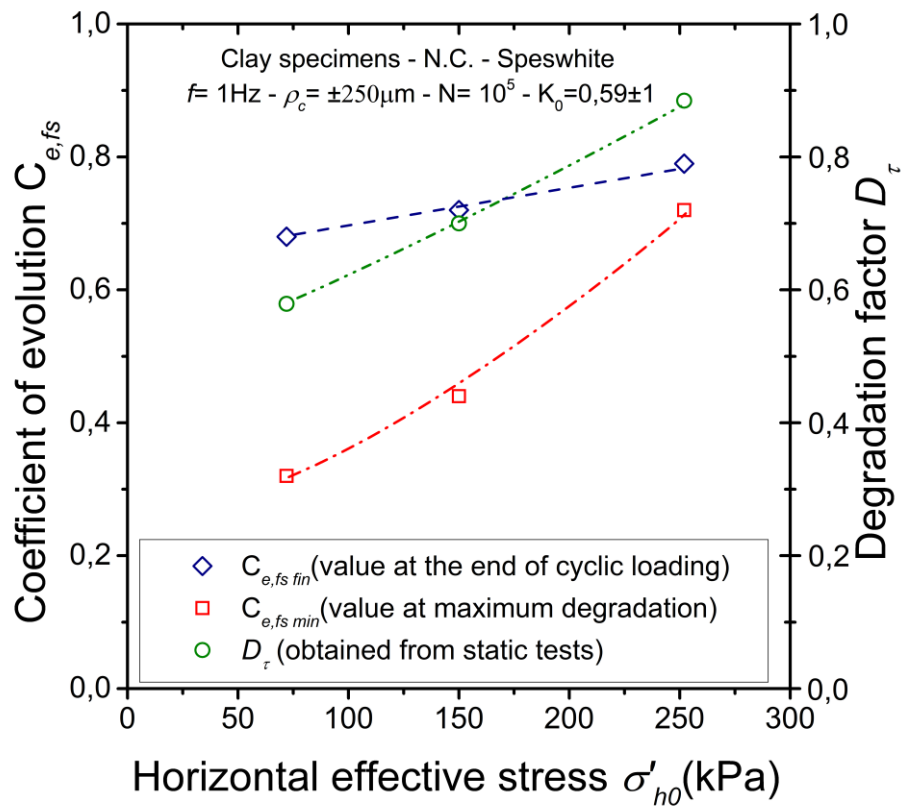
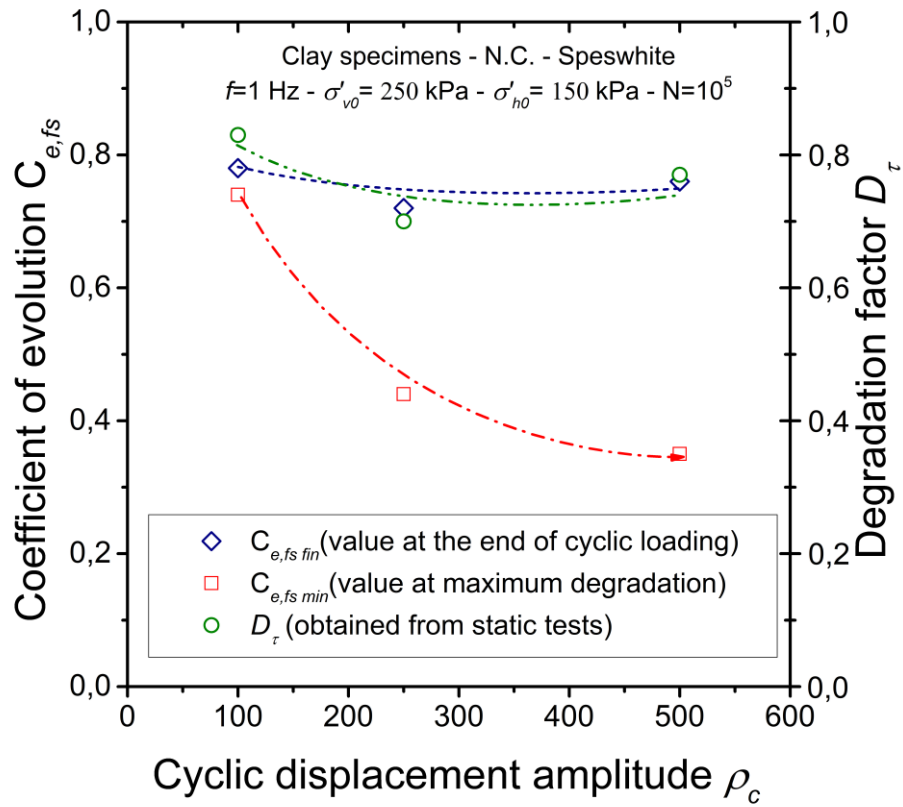


Figure 19. Synthesis of the effect of cyclic displacement amplitude and effective horizontal stress on coefficient of evolution  $C_{e,fs}$  and degradation factor  $D_\tau$

## Evolution of cyclic local friction along clay-pile interface for very large numbers of cycles: parametric study

### Tables

Table 1. Physical properties of Speswhite kaolinite

| Mineralogy | Liquid limit (%) | Plastic limit (%) | Plasticity index (%) | Specific gravity (g/cm <sup>3</sup> ) | Percentage finer than 10 $\mu$ m (%) |
|------------|------------------|-------------------|----------------------|---------------------------------------|--------------------------------------|
| kaolinite  | 58               | 28                | 30                   | 2,64                                  | 98                                   |

Table 2. Main characteristics of tests performed

| Test identification | $\sigma'_{v0}$ (kpa) | $\sigma'_{h0}$ (kpa) | Frequency $f$ (Hz) | Cyclic amplitude $\rho_c$ ( $\mu$ m) | Number of cycles | Observations            |
|---------------------|----------------------|----------------------|--------------------|--------------------------------------|------------------|-------------------------|
| Test 1              | 250                  | 150                  | 1                  | $\pm 250$                            | 100 000          | Reference test          |
| Test 2              | 250                  | 150                  | 1                  | $\pm 100$                            | 100 000          | Effect of amplitude     |
| Test 3              | 250                  | 150                  | 1                  | $\pm 500$                            | 100 000          | Effect of amplitude     |
| Test 4              | 125                  | 72                   | 1                  | $\pm 250$                            | 100 000          | Effect of initial state |
| Test 5              | 420                  | 252                  | 1                  | $\pm 250$                            | 100 000          | Effect of initial state |

Table 3. Synthesis of the results in term of coefficient of evolution and the degradation factor

| Test identification | $\sigma'_{v0}$ (kPa) | $\sigma'_{h0}$ (kPa) | Frequency $f$ (Hz) | Cyclic amplitude $\rho_c$ ( $\mu$ m) | Number of cycles N | $C_{efs}$           |                           | $D_\tau$ |
|---------------------|----------------------|----------------------|--------------------|--------------------------------------|--------------------|---------------------|---------------------------|----------|
|                     |                      |                      |                    |                                      |                    | maximum degradation | at the end of cyclic test |          |
| Test 1              | 250                  | 150                  | 1                  | $\pm 250$                            | 100 000            | 0,44                | 0,72                      | 0,70     |
| Test 2              | 250                  | 150                  | 1                  | $\pm 100$                            | 100 000            | 0,74                | 0,78                      | 0,83     |
| Test 3              | 250                  | 150                  | 1                  | $\pm 500$                            | 100 000            | 0,35                | 0,76                      | 0,77     |
| Test 4              | 125                  | 72                   | 1                  | $\pm 250$                            | 100000             | 0,32                | 0,68                      | 0,58     |
| Test 5              | 420                  | 252                  | 1                  | $\pm 250$                            | 100000             | 0,72                | 0,79                      | 0,88     |