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# Local soil-pile friction evaluation in saturated clays under cyclic loading

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1 **ABSTRACT:**

2 The paper presents laboratory measurements of mobilized local friction along piles submitted  
3 to very large numbers of axial loading cycles. The experimental approach used is of the physical  
4 modelling type and consists in testing an instrumented prototype pile-probe installed and loaded  
5 in specimens of saturated clay reconstituted in a calibration chamber. The procedure developed  
6 for evaluating the local friction mobilized upon monotonic loading and further evolution during  
7 the application of displacement-controlled cycles, is described. After installation of the probe,  
8 a succession of monotonic and cyclic displacement-controlled loading phases, carried out on a  
9 reference kaolinite is presented and analysed. During the cyclic sequence, carried out up to  $10^5$   
10 cycles, an initial phase of friction degradation is observed, followed by a reinforcement phase,  
11 which keeps going until the end of the test. A coefficient of evolution is defined allowing to  
12 quantify, during application of the cycles, the evolution of mobilized friction in terms of  
13 degradation or reinforcement of friction. The evolution of the friction mobilized during the  
14 application of the cycles is interpreted in terms of the combination of excess pore water pressure  
15 generation and dissipation. A comparison is made between maximum static shear mobilized  
16 before the cycles and after the cycles, showing the influence of the cyclic sequence on this  
17 quantity. Elements are finally given on the repeatability of the test, showing a fairly good level  
18 of repeatability.

19

20 **KEY WORDS:**

21 Pile, local shaft friction, cyclic loading, saturated clay, physical modelling, calibration chamber,  
22 instrumented pile-probe, excess pore water pressure

23 **1 – INTRODUCTION**

24 The study of the local friction behaviour of pile-soil interface constitutes an important issue in  
25 geotechnical engineering, related to the improvement of the design of foundations of railway  
26 bridges and offshore structures with an emphasis on wind turbines and oil and gas platforms.  
27 In particular, in the case of piles submitted to cyclic axial loadings due to environmental or  
28 industrial actions, this frictional behaviour is particularly complex and further experimental and  
29 theoretical research is still needed.

30 As far as experimental research is concerned, physical modelling approach based on calibration  
31 chamber testing appears to be a good way to better understand, under well-controlled laboratory  
32 conditions, the local behaviour of the soil-pile interface under cyclic axial loading.

33 Since the beginning of the years 80', experimental research has been developed on this subject,  
34 very often in relation with the offshore oil production industry and results have been presented,  
35 of the physical modelling approach type, using various types of probes representing sections of  
36 model piles, tested in soil specimens under various boundary and confinement conditions. Both  
37 sands and clays have been studied, most of the time for small to medium numbers of cycles,  
38 say less than  $10^4$  cycles.

39 As far as sands are concerned, one may cite the studies published by Chan and Hanna (1980),  
40 Lee and Poulos (1990), Al-Douri and Poulos (1995), Chin and Poulos (1996), Le Kouby et al.  
41 (2004), Lehane and White (2004), Tsuha et al. (2012). In most studies, the authors find a  
42 significant degradation of the mobilized friction during application of the cycles.

43 As far as clays are concerned, more specifically related to the subject of this paper, one may  
44 cite the results published by Poulos (1981a), Matlock et al. (1982), Goulois et al. (1985) or  
45 Procter and Khaffaf (1987). In particular, Poulos (1981a) conducted a number of small-scale  
46 laboratory tests on a model pile section (20 mm in diameter) in reconstituted saturated clay  
47 specimens (152 mm in diameter), up to maximum number of 1000 cycles, showing that two-

48 way displacement-controlled cycles cause significant reduction in pile-shaft skin friction which  
49 may result in failure of the model pile section. In particular, this author noted the strong  
50 influence of the amplitude of the cyclic displacement on the importance of the friction  
51 degradation.

52 Matlock et al. (1982) have conducted axial cyclic displacement-controlled load tests on a model  
53 pile section (2.5 cm in diameter) inserted into a confined reconstituted soft clay in a calibration  
54 chamber (76.2 cm diameter specimens). These authors have pointed out a significant reduction  
55 in mobilized friction during the application of the cycles (maximum number of 300 cycles  
56 applied).

57 As far as the behaviour of piles under very large numbers of cycles is concerned (several  $10^4$  to  
58  $10^6$  cycles) corresponding to fatigue type of behaviour, it appears that very few if no  
59 publications are available. In the case of sands, Bekki et al. (2013) have presented results  
60 concerning the evolution of local friction along an instrumented pile-probe jacked into a sand  
61 specimen reconstituted in a calibration chamber, for large numbers of displacement-controlled  
62 cycles ( $10^5$  cycles). The results obtained show that after an initial degradation phase of the local  
63 skin friction (cyclic strain-softening), a reinforcement phase (cyclic strain-hardening) develops  
64 up to very high number of cycles ( $10^5$ ). However, for the case of fine-grained soils, typically  
65 saturated clays, no publication has been found on the subject in the literature.

66 Within this context, the objective of this paper is to present a complete laboratory prototype  
67 setup associated to a specific experimental procedure (physical modelling approach) which  
68 have been developed in order to study the local friction behaviour of soil-pile interface  
69 submitted to very large numbers of axial loading cycles, in the case of reconstituted saturated  
70 clay. After a description of the testing setup and of the experimental procedure developed, a  
71 typical test is presented and analysed, showing the interest of this prototype setup for better

72 understanding the mechanisms of local friction evolution along piles submitted to large to very  
73 large numbers of axial loading cycles.

74

## 75 **2 – TESTING SETUP**

### 76 **2.1 The calibration chamber testing setup**

77 The calibration chamber used in this research allows to reconstitute uniform soil specimens,  
78 524 mm in diameter and 700 mm high. Independent vertical and horizontal stresses may be  
79 applied to the specimens, allowing to generate isotropic or anisotropic initial states of stress,  $K_0$   
80 conditions, etc. The vertical stress is applied to the specimen through a large diameter bottom  
81 piston, the horizontal stress being applied through a water pressure confining the specimen  
82 contained in a neoprene membrane. Figure 1 presents a functional scheme of the calibration  
83 chamber showing the different parts of the setup. This calibration chamber is based on the same  
84 working principle as the ones described by Huang et al. (1988), Tumay and de Lima (1992) and  
85 Voyiadjis et al. (1993). The chamber itself is incorporated in a guiding and loading mechanical  
86 framework shown in figure 2. This framework allows the chamber to be moved and adjusted in  
87 translation and rotation on the rail track system, as shown in the figure. The four columns  
88 loading frame is equipped with two hydraulic jacks: a long stroke (1 m) 100 kN standard jack,  
89 allowing to perform displacement-controlled (0.1 to 100 mm/s adjustable displacement rate)  
90 push down operations, typically installation of probes, penetrometer testing, etc.; and a  
91 servocontrolled hydraulic actuator allowing to perform precise force or displacement-controlled  
92 loading operations, within the monotonic and cyclic ranges. Figure 3 presents a general view  
93 of the experimental setup.

94 This system has first been developed and used for sand testing, with development of a specific  
95 procedure for reconstitution of well-controlled sand specimens (dry and saturated) for carrying

96 out research on cyclic pressuremeter testing (Dupla and Canou 2003), on micropiles behaviour  
97 (Le Kouby et al. 2004) and on the behaviour of piles under cyclic loading (Bekki et al. 2013).

98 In order to study the behaviour of piles installed in saturated clay, complementary pieces of  
99 equipment as well as a specific experimental procedure have been recently developed and are  
100 presented in this paper. The main piece of equipment developed is a large size consolidometer  
101 described below.

102

### 103 **2.2 New consolidometer developed for testing saturated fine-grained soils**

104 The consolidometer developed is a rigid wall reservoir composed of two adjustable halves  
105 (Plexiglas tube reinforced with metallic annular parts), equipped with a top and bottom draining  
106 plates, which allows to reconstitute 524 mm diameter and 600 to 800 mm high specimens of  
107 saturated clay under  $K_0$  conditions (no lateral deformation during the consolidation process)  
108 starting from a soil slurry. This reservoir is equipped with a loading frame and a double-action  
109 hydraulic jack which allows to consolidate the specimen by application of constant force  
110 increments to the top plate (Figure 4(a) and Figure 4(b)). The hydraulic jack is powered by a  
111 hydro-pneumatic pump which allows to apply controlled pressure steps to the jack and therefore  
112 controlled force increments. The jack piston is equipped with a force transducer and a long  
113 stroke “wire” displacement transducer which allow to monitor force and displacement during  
114 the consolidation process. Vertical as well as radial drainage circuits are available and may or  
115 may not be both activated during the consolidation process.

116

### 117 **2.3 The instrumented pile-probe**

118 This prototype probe has been developed, within the framework of physical modelling research  
119 on piles, in order to make direct and independent measurements of tip resistance and local shaft  
120 friction representative of values occurring along a pile shaft. Figure 5 shows a simplified cross



121 section together with a view of the probe. The probe has a cross section of 10 cm<sup>2</sup>  
122 (diameter of 36 mm), similar to standard penetrometers. The conical tip is equipped with a  
123 20 kN precision force transducer which measures the tip resistance upon loading. A friction  
124 sleeve is located on the shaft, far enough from the tip (240 mm above the tip), in order to  
125 minimize interactions between tip resistance and sleeve friction. The sleeve is 11 cm long  
126 (sleeve surface of 124.4 cm<sup>2</sup>) equipped with a  $\pm$  5kN load sensor. The surface of the friction  
127 sleeve, as well as a significant part of the probe body, have been specially threaded in order to  
128 ensure a perfectly rough interface with respect to friction mobilization.

129

#### 130 **2.4 – Experimental procedure**

131 In order to prepare the initial clay slurry to be consolidated, dry clay powder is progressively  
132 added and thoroughly mixed with water, using a portable adjustable speed mixer, in order to  
133 prepare, in a large plastic container (200 l) a homogeneous clay slurry at a water content of  
134 about 1.5 times the liquid limit of the clay. This initial water content agrees fairly well with the  
135 recommendations suggested by Sheeran and Krizek (1971) as well as with the work of different  
136 authors (Anderson et al. 1991; Anderson et al. 2006; Cardoso and Nogueira 2013).

137 Once the preliminary mixing has been completed, the slurry is covered with a plastic film to  
138 prevent water from evaporating and left to soak for 48 hours to insure complete homogenization  
139 of the water content within the mixture. Finally, the slurry is mixed a second time for about 30  
140 minutes in order to obtain a better homogenization.

141 The slurry is then poured from the preparation reservoir into the consolidometer using a long  
142 tube which avoids air entrapment into the slurry during the process (Figure 6 (a)). A thin layer  
143 of silicon grease has first been applied on the inner wall of the consolidometer with addition of  
144 a thin plastic film in order to minimize parasite side friction during the consolidation process  
145 and ease the “unmolding” process” of the specimen after transportation onto the calibration

146 chamber base. The loading frame is then adjusted and fixed on top of the consolidometer  
147 reservoir. The consolidation process is then initiated by applying successive increasing load  
148 increments up to the maximum value selected (Figure 6 (b)). For each load increment applied,  
149 the consolidation process takes place and the next force increment is only applied when the  
150 major part of the primary consolidation has been obtained under the given load.

151 During the application of each load increment, the applied force and top plate displacement are  
152 monitored, which allows to obtain the consolidation curve in terms of settlement versus time.

153 After the consolidation process has been completed, the loading frame is removed. The  
154 reservoir is transported and adjusted on the bottom piston of the calibration chamber with a  
155 travelling crane (Figure 6 (c)). The two parts of the consolidation reservoir are then untighten,  
156 separated and carefully removed, not to disturb the clay specimen (Figure 6 (d)). The top end  
157 plate is then adjusted on top of the specimen as well as the lateral confinement rubber membrane,  
158 using a specially designed PVC tube internally equipped with the membrane “stuck” against it  
159 and slowly lowered around the specimen (Figure 6 (e)).

160 The lateral confinement cell and the top cover of the calibration chamber are then adjusted and  
161 tightened. Then lateral and vertical chamber pressures are progressively increased in order to  
162 decrease the negative excess pore water pressures existing inside the specimen and finally  
163 retrieve the state of effective stress initially generated in the consolidometer, with no residual  
164 excess pore water pressure (Figure 6 (f)). This final state is obtained after full stabilization of  
165 the vertical piston of the calibration chamber base. The specimen of saturated clay is then ready  
166 for further use.

## 167 **3 – DESCRIPTION AND ANALYSIS OF A TYPICAL TEST**

### 168 **3.1 - Material used**

169 This typical test, called Test 1, has been carried out on a specimen of pure saturated kaolinite.  
170 The reference clay used, called Speswhite, is an industrial clay, that has been chosen in France  
171 as a reference clay for physical modelling purposes in geotechnics (in particular centrifuge and  
172 calibration chamber testing). Some of the geotechnical properties of this clay, as determined by  
173 the authors, are given in table 1. The grain size distribution curve of the deflocculated Speswhite  
174 clay has been determined using the sedimentometry method. It is shown in Figure 7, together  
175 with the curve given by the supplier.

176

### 177 **3.2 Clay specimen characteristics**

178 For this test, the clay specimen has been consolidated in the calibration chamber to a final  
179 vertical effective stress  $\sigma'_{v0}$  of 125 kPa and a final effective horizontal stress  $\sigma'_{h0}$  of 72 kPa  
180 corresponding to an estimated value of  $K_0$  equal to 0.58. In the consolidometer, the initial  
181 consolidation of the specimen has been achieved in four steps, corresponding to successive  
182 vertical stresses of 5, 15, 45 and 125 kPa. Figure 8 shows the corresponding settlement and  
183 vertical deformation versus time curves corresponding to the four successive load increments  
184 applied. The permeability and consolidation coefficient of the clay under this state of stress  
185 have been determined based on oedometer tests, giving  $k = 2.10^{-9}$  m/s and  $c_v = 2.10^{-7}$  m<sup>2</sup>/s

186

### 187 **3.3 Installation of the pile-probe**

188 After final consolidation of the specimen in the chamber, the pile-probe was first installed into  
189 the clay using the displacement-controlled long stroke hydraulic jack which allows to push the  
190 probe at a constant displacement rate (Figure 6 (g)). This installation process is representative  
191 of full displacement piles. The displacement rate used in this test was 1 mm/s. The probe was

192 pushed into the clay specimen until the friction sleeve was vertically centred within the  
193 specimen, corresponding to a penetration length of 460 mm of the tip into the specimen.  
194 Figure 9 presents the penetration curves obtained in terms of tip resistance, local friction and  
195 total force applied on top of the probe versus tip penetration depth during the installation  
196 process. It must be noted here that during the installation process as well as all subsequent  
197 loading operations, the boundary drainage of the specimen remains open.

198 The tip resistance increases rapidly during the initial phase of the penetration process  
199 (Figure 9(a)), down to a penetration depth of about 100 mm where a plateau is reached (about  
200 0.25 MPa), corresponding to steady state conditions, accounting for a fairly good uniformity of  
201 the clay specimen.

202 The local friction starts to be mobilized when the friction sleeve gets into the specimen  
203 (Figure 9(b)), for a penetration depth of about 250 mm of the tip. The friction then increases to  
204 progressively reach an almost stabilized value close to 10 kPa when the final tip penetration  
205 depth of 460 mm has been reached.

206 The total load measured on top of the probe is shown in figure 9(c). This curve accounts for the  
207 global mobilization of both tip resistance and friction along the probe shaft. The rapid initial  
208 mobilization of total load corresponds to the initial mobilization of the tip resistance. The  
209 following almost linear increase of the total load observed then corresponds to the progressive  
210 increase of friction surface of the probe getting into the clay specimen.

211

### 212 **3.4 Initial monotonic or « static » loading of the probe**

213 After the installation process has been completed, the probe is left alone without further loading  
214 for about 12 hours. This excess pore pressure dissipation time has been evaluated based on  
215 publications related to piezocone testing (Almeida and Parry, 1985; Kurup et al., 1994). It is  
216 also possible to use a theoretical solution proposed by Parez and Fauriel (1988), considering

217 the horizontal permeability of Speswhite ( $k_h=10^{-9}$  m/s (Feddemma and Bredeveld (2010)) to  
218 evaluate an approximate value of  $t_{50}$  ( $t_{50}$  equals to about 1500 seconds). Therefore, one may  
219 consider that 12 hours is largely enough to allow for practically complete dissipation of excess  
220 pore water pressures generated around the probe during the installation phase. Then, two  
221 successive displacement-controlled quasi-static monotonic compression loadings are applied to  
222 the probe up to failure (4mm total vertical displacement corresponding to about 11% of the  
223 probe diameter) with the servohydraulic actuator (Figure 6 (h)). The first loading is performed  
224 at a displacement rate of 30  $\mu\text{m}/\text{min}$ , the second one being performed at a rate of 300  $\mu\text{m}/\text{min}$ ,  
225 after waiting 2 hours after the first loading. Contrary to the installation phase, for subsequent  
226 static loadings, the pore water pressure variation is mainly restricted to the interface zone for  
227 which only a thin layer of soil around the probe is involved (only a few millimeters thickness)  
228 with high values of local hydraulic gradient. The dissipation time is then significantly lower  
229 (Potts and Martins (1982)). Therefore, a duration of 2 hours is estimated to be enough to  
230 practically obtain full dissipation of EPWP. After each displacement-controlled loading, force-  
231 controlled unloading is applied. This procedure allows to evaluate the effect of loading rate on  
232 the results obtained, in terms of the drainage conditions around the probe during loading.  
233 Figure 10 shows the results obtained for the typical test presented.  
234 In terms of tip resistance (Figure 10(a)), for the first static test (30  $\mu\text{m}/\text{min}$  displacement rate),  
235 after a rapid initial mobilization of tip resistance for small displacements, the tip resistance still  
236 progressively increases until an almost stable value is reached for the final 4mm vertical  
237 displacement. Concerning the second static test (300  $\mu\text{m}/\text{min}$  displacement rate), a stiffer initial  
238 mobilization of tip resistance is observed until a plateau is relatively rapidly reached. It is,  
239 though, interesting to observe that the final values of tip resistance reached in both tests, equal  
240 to about 0.45 MPa, are very close to each other. It is also interesting to note that this value is

241 higher than the value of 0.25 MPa reached upon the installation process, carried out at a much  
242 higher penetration rate (1 mm/s).

243 In terms of local friction mobilization (Figure 10(b)), the responses observed for the two  
244 loadings are very close to each other, with a very rapid mobilization of local friction, followed  
245 by a plateau equal to about 17 kPa obtained in both cases. The only difference concerns a peak  
246 followed by a slight strain softening, observed at a very small displacement for the second  
247 loading.

248 The results are also shown in terms of the total force measured on top of the probe during  
249 loading (Figure 10(c)), which accounts for the local measurements made in terms of tip  
250 resistance and local friction.

251 As a conclusion, it is interesting to note that the failure characteristics obtained in terms of local  
252 friction and tip resistance are very similar for the two pre-cyclic static tests and that the second  
253 loading is not significantly affected by the first one. Also, in terms of drainage conditions  
254 around the probe during loading, these conditions should be very similar for the two loadings  
255 (similar levels of excess pore water pressure reached) since the results are very close in terms  
256 of friction and tip resistance.

257

### 258 **3.5 Cyclic loading phase**

259 In order to investigate the influence of large numbers of loading cycles on the mobilization of  
260 local friction and tip resistance, cyclic displacement-controlled loading tests have been carried  
261 out on the probe after the initial static loadings. The main parameters involved in this type of  
262 cyclic tests are the displacement amplitude (alternated or non-alternated signal), the signal  
263 shape, the frequency of the signal and the number of cycles. In the test presented below,  
264  $10^5$  cycles have been applied with a cyclic displacement amplitude  $\rho_c$  equal to  $\pm 250 \mu\text{m}$   
265 (alternated signal) and a frequency of 1 Hz. The shape of the signal is sinusoidal. The local

266 displacement amplitude is known to be one of the key factors controlling the evolution of  
267 interface behaviour and the experimental setup developed allows to precisely investigate the  
268 influence of this parameter on the behaviours observed.

269 Figure 11 shows the corresponding results. Figure 11(a) shows the loading signal applied in  
270 terms of probe head displacement versus number of cycles (the probe is considered to be  
271 perfectly rigid and the probe head displacement is assumed to be the same as the local  
272 displacement at the friction sleeve level).

273 Figure 11(c) shows the probe response in terms of local friction mobilized versus number of  
274 cycles. A clear degradation process is observed from the very first cycle, which keeps going for  
275 about 50 cycles, corresponding to cyclic strain-softening of the probe-soil interface. Then, a  
276 progressive re-increase of mobilized friction is observed (cyclic strain-hardening) and keeps  
277 going, on an average, up to the end of the cyclic sequence (cycle n°  $10^5$ ) with a slight “re-  
278 decrease” observed between cycles 300 and 900. The degradation phenomenon, as observed in  
279 the initial part of the cyclic sequence, has already been observed by different authors on both  
280 model tests and field tests (Chan and Hanna 1980; Poulos 1981b; Poulos 1982; Matlock et al.  
281 1982), but the following cyclic strain-hardening phenomenon (friction reinforcement), has not,  
282 to our knowledge, been previously described for fine-grained soils. In the case of dry sand, a  
283 similar friction reinforcement has been described by Bekki et al. (2013) for large numbers of  
284 cycles ( $10^5$  cycles).

285 In terms of tip resistance (figure 11 (b)), a similar trend is observed as for the friction evolution,  
286 with a degradation up to about cycle n° 40, followed by a progressive reinforcement of the tip  
287 resistance mobilization until the end of the test. The total head load response, as measured on  
288 top of the probe, is shown in figure 11(d). The evolution observed for the total head load  
289 globally accounts for the evolutions of both local friction and tip resistance measured  
290 independently. Figure 12 shows typical loading cycles in terms of local friction versus local

291 displacement. Figure 12(a) shows cycles corresponding to the degradation phase. A substantial  
292 reduction is observed on the maximum and minimum friction mobilized on each cycle, which  
293 begins at cycle n° 1 and continues up to cycle n° 50. Upon push in phases, the maximum friction  
294 mobilized, which was 17 kPa on the first cycle, drops to 5 kPa on the 50<sup>th</sup> cycle. Upon pull out  
295 phases, the evolution is similar with a maximum value of about 17 kPa (absolute value) and a  
296 minimum value of about 6 kPa. Figure 12(b) shows typical cycles corresponding to the cyclic  
297 strain-hardening phase which develops from cycle n° 50 to the last cycle (end of the test). It  
298 may be noted that the maximum value of friction obtained, upon push in and pull out phases at  
299 the end of the test ( $\pm 12$  kPa) is still lower than the one reached on the first cycle.

300

### 301 **3.6 Final phase of static loading**

302 After completion of the cyclic sequence, it is important to perform new static compression load  
303 tests in order to quantify the influence of the cycles on the static failure parameters, in terms of  
304 tip resistance and local friction. In a way similar to the two static load tests carried out before  
305 the cyclic sequence, two static compression tests have been performed after the cycles in order  
306 to evaluate the post-cyclic response of the model. For the test presented in this paper, the two  
307 final static tests were performed at the same displacement rate of 300  $\mu\text{m}/\text{min}$  up to failure (4  
308 mm vertical displacement). The first post-cyclic static test was performed directly after the end  
309 of the cyclic sequence while the second one was performed after a resting period of 2 hours  
310 after the first static test allowing for full drainage of the soil around the probe.

311 Figure 13 presents the results obtained in terms of mobilization of local friction and tip  
312 resistance versus vertical displacement.

313 In terms of local friction (Figure 13(b)), a significant difference may be observed between the  
314 two loadings. For the first post-cyclic loading, a sharp peak of friction (of about 23 kPa) is  
315 obtained for a low displacement (about 800  $\mu\text{m}$ ), followed by a significant strain softening



316 which keeps going with progressive stabilization for large displacements at an ultimate value  
317 of about 14 kPa. For the second post-cyclic loading, no peak is observed and full mobilization  
318 of friction is obtained for a very small displacement (about 100  $\mu\text{m}$ ) followed by a remarkably  
319 constant friction plateau of about 11 kPa, which is maintained up to large displacements. This  
320 second post-cyclic loading response is qualitatively very similar to the initial static response  
321 observed before the cyclic sequence. It is also interesting to note that the ultimate value of  
322 friction obtained at large displacements is very similar for both post-cyclic static loadings,  
323 accounting for a good consistency between the two successive loadings.

324 The comparison between pre-cyclic and post-cyclic static loading response shows that the  
325 cyclic sequence induces a significant “reinforcing” effect on the static friction that can be  
326 mobilized after cycling, which appears to be significantly higher than the friction mobilized  
327 before cycling. Also, it is interesting to note that the “memory” of the interface, due to the  
328 cycles may be easily erased by a first loading to failure. In terms of tip resistance, less  
329 differences are observed between the two successive loadings. The second loading appears  
330 more regular than the first one, with a progressive stabilization at about 0.35 MPa. The first  
331 loading appears less regular on its initial part with, however, a stabilization at about the same  
332 value as for the second loading (0.35 MPa), which is consistent.

333 For both local friction and tip resistance, the characteristic shape of the curves observed  
334 between 50 and 300 microns upon the first post-cyclic static loading, showing an upward  
335 concavity. This concave shape is typical of undrained shear with generation of negative excess  
336 pore water pressure, as usually observed in the triaxial apparatus upon shear. The interpretation  
337 of the differences observed between the pre-cyclic and post-cyclic static load tests in terms of  
338 mobilization of local friction is based on the following “well-known” principles: upon  
339 monotonic undrained shear test (triaxial test or interface shear test), normally consolidated clay  
340 exhibits positive excess pore water pressures (EPWP), due to contractive behavior whereas

341 overconsolidated clay exhibits negative EPWP due to dilative behavior (analogy with loose and  
342 dense sand behavior). When applied to our case, these principles result in the following  
343 interpretation: After installation and EPWP dissipation, the clay in the interface is in a normally  
344 consolidated state ( $\sigma'_n$  has increased in the process). Upon static undrained loading, positive  
345 EPWP will therefore certainly be generated, resulting in the loading curve presented in figure  
346 14 (2<sup>nd</sup> initial static test). This interpretation is in full agreement with the results published by  
347 Potts and Martins (1982). Then the cyclic sequence occurs, which results in an initial increase  
348 of EPWP followed by a progressive decrease due to EPWP dissipation (this aspect has been  
349 analysed in detail in part 4.2 of the paper). It is believed that after full dissipation, the cyclic  
350 shear should result in a strongly densified and overconsolidated state of the clay within the  
351 interface zone, which should therefore lead, upon subsequent undrained shear (1st post-cyclic  
352 static test) in the generation of negative EPWP. This generation of negative EPWP will strongly  
353 increase the local shear measured during loading (peak value of 23 kPa, before softening), due  
354 to an overall increase of  $\sigma'_n$ . This hypothesis, generation of negative EPWP, is in good  
355 agreement with the statement reported by Randolph (2003) and is confirmed by the very typical  
356 shape of the shear curve, showing an upward concavity, accounting for a strong strain-  
357 hardening, typical of undrained shear with development of negative EPWP in triaxial testing  
358 (Henkel (1956); Koutsoftas (1981); Hattab & Hicher (2004); Henni et al. (2012); Berre (2014)).  
359 The following rapid decrease of friction (post pick softening) is attributed to strain localization  
360 within the interface.

361 Figure 14 finally shows a comparison between the static tests carried out before and after the  
362 cyclic sequence, in terms of local friction, showing that the ultimate friction mobilized after the  
363 cyclic sequence remains lower, in terms of ultimate value, than the friction mobilized before  
364 the cyclic sequence (about 11 kPa instead of 17 kPa). This decrease in terms of ultimate value  
365 accounts for an overall weakening of the interface due to the cyclic sequence.

366 The second post-cyclic static test is consistent with the first one in the sense that the  
367 overconsolidation effect has been erased by the first post-cyclic loading and the shape of the  
368 shear curve looks very much like the pre-cyclic static test.

369

## 370 **4 – ANALYSIS OF THE CYCLIC SEQUENCE**

### 371 **4.1 – Coefficient of friction evolution**

372 In order to quantify the evolution of local friction mobilization during cyclic loading, a  
373 coefficient called coefficient of friction evolution  $C_{e,fs}$ , has been defined as follows :

$$374 \quad (1) \quad C_{e,fs} = \frac{f_{s,max(i)} - f_{s,min(i)}}{f_{s,max(1)} - f_{s,min(1)}}$$

375 where  $f_{s,max(1)}$  and  $f_{s,max(i)}$  are the maximum skin friction measured on first cycle and cycle  $i$

376 respectively (push-in phases),  $f_{s,min(1)}$  and  $f_{s,min(i)}$  being the values of minimum skin friction

377 measured on first cycle and on cycle  $i$  respectively (pull-out phases).

378 This coefficient allows to clearly visualize the evolution of local friction mobilization during  
379 the application of the cycles and to easily make a distinction between degradation phases,  
380 corresponding to a decrease of  $C_{e,fs}$  and reinforcement phases, corresponding to an increase of  
381 this coefficient, and to quantify the importance of the degradation or reinforcement with respect  
382 to the initial mobilization observed on the first cycle.

383 Figure 15 shows a plot of this coefficient versus number of cycles for the test presented above,  
384 which clearly accounts for the initial degradation phase observed up to cycle n° 50, followed  
385 by the reinforcement phase toward the end of the cyclic sequence (with a slight decrease  
386 between cycle 300 to cycle 900). The minimum value reached is about 0.32 which is a fairly  
387 low value corresponding to a significant degradation. It may also be noted that even if re-  
388 increasing, the maximum value of  $C_{e,fs}$  reached at the end of the test is about 0.68, still  
389 significantly lower than 1.

390

#### 391 **4.2 – Analysis of the friction behaviour observed during the cyclic sequence**

392 The analysis of the friction behaviour observed during the application of the cycles may be  
393 done in terms of the evolution of the effective normal stress acting on the probe shaft,  $\sigma'_n$ , the  
394 local friction mobilized being equal to  $f_s = \sigma'_n \operatorname{tg} \delta_{mob}$ ,  $\delta_{mob}$  being the mobilized friction  
395 coefficient. The evolution of  $\sigma'_n$  is directly related to the evolution of the excess pore water  
396 pressure  $\Delta u_s$  around the friction sleeve: when  $\Delta u_s$  increases,  $\sigma'_n$  decreases, resulting in a  
397 decrease of mobilized friction ; when  $\Delta u_s$  decreases,  $\sigma'_n$  increases, which results in an increase  
398 of mobilized friction.

399 It is believed that, in this low permeability clay, and by analogy with undrained cyclic triaxial  
400 testing on clays, cyclic shear at 1 Hz frequency will result in the development of excess pore  
401 water pressure (EPWP) close to the soil-probe interface. In fact, an heterogeneous field of  
402 EPWP will be created of small thickness, resulting in high hydraulic gradients and initiation of  
403 EPWP dissipation. The problem is therefore coupled with superposition of EPWP generation  
404 and dissipation.

405 There is, indeed, a competition between the excess pore water pressure generation mechanism  
406 due to “undrained” cyclic deformation of the clay around the sleeve, and the pore pressure  
407 dissipation phenomenon which starts taking place from the very beginning of the cyclic  
408 sequence due to the radial hydraulic gradient created by the excess pore pressure field. During  
409 the initial phase of the sequence (small numbers of cycles), the generation mechanism should  
410 be predominant with respect to dissipation, which should result in a relatively rapid increase of  
411 the resulting excess pore pressure, a corresponding decrease in the normal effective stress and  
412 corresponding decrease of mobilized friction (cyclic strain-softening). Then, the dissipation  
413 process should become predominant, resulting in a progressive decrease of the EPWP, re-  
414 increase of  $\sigma'_n$  and corresponding re-increase of mobilized friction (cyclic strain-hardening).

415 This decrease of excess pore water pressure should slow down with the number of cycles, which  
416 would explain the shape of the  $C_{e,fs}$  versus  $N$  curve for large numbers of cycles, when the  
417 conditions become practically fully drained. Figure 16 presents a conceptual scheme illustrating  
418 the above point. It appears that for the coupled case (generation coupled to dissipation of excess  
419 pore water pressure), the excess pore water pressure around the sleeve will first increase and  
420 then decrease after passing through a maximum value. This conceptual scheme is consistent  
421 with results presented by Procter & Khaffaf (1987) showing this type of evolution of the excess  
422 pore water pressure around a model pile cyclically loaded in a specimen of clay. The study  
423 presented by these authors is very interesting, with objectives close to ours, the main idea being  
424 to evaluate the influence of EPWP generation around a model pile element on the mobilization  
425 of friction. Even if the testing setup is slightly different from ours, the testing conditions are  
426 similar, with clay characteristics similar to ours. The loading conditions are also similar  
427 (displacement-controlled loading with similar range of displacement amplitudes and  
428 frequencies) and their results should, at least qualitatively, match ours. Figure 17 shows the  
429 evolution of cyclic EPWP close to their model pile element during cyclic loading ( $\Delta U_1$ ), clearly  
430 showing an initial increase followed by a decrease after a certain number of cycles (13 in this  
431 case). It is believed that the point corresponding to the maximum of excess pore water pressure  
432 should correspond to the minimum of mobilized friction and minimum value of  $C_{e,fs}$ . It will be  
433 very interesting in the future to try to make direct measurements of the pore water pressure  
434 around the probe in order to further check this hypothesis. For the test presented, the maximum  
435 excess pore water pressure would therefore be obtained at cycle n° 50. The slight re-decrease  
436 phase observed between cycles 300 and 900, before final reinforcement could be due to excess  
437 pore water pressure redistribution between the tip and the sleeve of the probe.

438

439

### 440 **4.3 - Repeatability of the cyclic sequence**

441 In order to evaluate the repeatability of the test and, in particular, the repeatability of the cyclic  
442 sequence, two more tests (Test 2 and Test 3) with same testing conditions and parameters as  
443 the ones implemented for Test 1 have been carried out. Figure 18 shows a comparison of the  
444 results obtained in terms of friction evolution coefficient. It may be observed that the trends  
445 obtained are qualitatively and quantitatively fairly similar for the three tests, accounting for a  
446 good level of repeatability of the test in terms of friction evolution. It is interesting to note that  
447 the relative decrease of  $C_{e,fs}$  observable between about cycle n° 300 and cycle n°900 is obtained  
448 in the three tests. This relative decrease has been interpreted above in terms of excess pore water  
449 pressure redistribution between the tip and the sleeve of the probe but this will need further  
450 validation.

451 It must also be noted here that the repeatability of the specimen preparation has also been  
452 checked by measuring the water content of the reconstituted specimens at different points on a  
453 given specimen (9 points) and by comparing the values obtained for different specimens (test  
454 1, test 2 and test 3 in table 2). For those three specimens, an average value of 49.6% was found  
455 for the water content with a standard deviation of  $\pm 0.9$  %, which accounts for a fairly good  
456 repeatability of reconstitution of the specimens of saturated clay.

457

## 458 **5 – CONCLUSIONS**

459 In this paper, based on an original testing setup associated to a specific experimental procedure,  
460 corresponding to a physical model type of approach, the response of a pile-probe, in terms of  
461 local friction and tip resistance evolution, submitted to large numbers of cycles in a saturated  
462 clay (fatigue type of problems), have been presented.

463 A series of monotonic and cyclic displacement-controlled loading phases, carried out on a  
464 reference kaolinite, called Speswhite, have been presented, with description of the successive

465 phases of the tests and corresponding results. This type of tests allow, in particular, to quantify  
466 the evolution of local friction mobilization during the cyclic sequence as a function of the cyclic  
467 displacement amplitude and to quantify the effect of the cyclic sequence on the maximum static  
468 friction that can be mobilized at failure after the cycles, with respect to the initial value (before  
469 the cycles).

470 As far as the cyclic loading sequence is concerned, the test presented shows, after an initial  
471 phase of local friction degradation (cyclic strain-softening), a phase of friction reinforcement  
472 (cyclic strain-hardening) which keeps going up to very large numbers of cycles. The coefficient  
473 of friction evolution  $C_{e,fs}$  introduced allows to quantify the evolution of friction mobilization  
474 during the cycles, showing that even if this coefficient re-increase after an initial phase of  
475 decrease, its value remains below one (maximum value reached of about 0.70 for large numbers  
476 of cycles). As far as the static maximum friction is concerned, the application of the cycles  
477 results, with respect to the value obtained before the cycles, in a significant increase of static  
478 friction (peak value), however followed by rapid strain-softening.

479 Also, repeatability tests have been carried out, accounting for a good level of repeatability of  
480 the results obtained, thus validating the testing setup and the experimental procedure developed.  
481 Finally, this testing setup associated with the experimental procedure developed should help to  
482 better understand the behaviour of piles submitted to large numbers of axial loading cycles and  
483 therefore help to improve the design of such piles.

484

485

## 486 **ACKNOWLEDGEMENTS**

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489 submitted to cyclic axial and lateral loads.

490

491 **REFERENCES**

- 492 Al-Douri, R. H. and Poulos, H.G., 1995. Predicted and Observed Cyclic Performance of Piles  
493 in calcareous sand. *Journal of Geotechnical Engineering ASCE* 121 (1): 1-16.
- 494 Almeida, M. S, S. and Parry, R. H. G., 1985. "Small Cone Penetrometer Tests and Piezocone  
495 Tests in Laboratory Consolidated Clays," *Geotechnical Testing Journal, GTJODJ*, Vol. 8,  
496 No. 1, March, pp. 14-24.
- 497 Anderson, W. F., Nguyen, D. H. and Adrian, F. L. 2006. A New Analysis Of Data From  
498 Statnamic Tests On Piles In Clay. In *Proceedings of the 10th International Conference On*  
499 *Piling And Deep Foundations*, Amsterdam, The Netherlands, pp. 618-626 .
- 500 Anderson, W. F., Pyrah, I. C. and Fryer, S. J., 1991. A Clay Calibration Chamber for Testing  
501 Field Devices. *Geotechnical Testing Journal* 14(4): 440-450.
- 502 Bekki, H., Canou, J., Tali, B., Dupla, J.-C. and Bouafia, A., 2013. Evolution of Local Friction  
503 Along a Model Pile Shaft in a Calibration Chamber for a Large Number of Loading Cycles.  
504 *Comptes Rendus - Mecanique* 341(6): 499-507.
- 505 Berre, T., 2014. Effect of sample disturbance on triaxial and oedometer behaviour of a stiff and  
506 heavily overconsolidated clay. *Canadian Geotechnical Journal*, 51:896-910,  
507 <https://doi.org/10.1139/cgj-2013-0077>
- 508 Cardoso, R. and Nogueira, J. S., 2013. An Experimental Study on the Consolidation of Soft  
509 Clayey Soils Using Electrochemical Methods. In *Proceedings of the 18th International*  
510 *Conference on Soil Mechanics and Geotechnical Engineering*. Paris, France, pp. 891-894.



- 511 Chan, S.F. and Hanna, T. H., 1980. Repeated Loading on Single Piles in Sand. Journal of  
512 Geotechnical Engineering 106(2): 171-188.
- 513 Chin, J. T. and Poulos, H. G., 1996. Tests on Model Jacked Piles in Calcareous Sand.  
514 Geotechnical Testing Journal 19(2): 164 180.
- 515 Dupla, J.-C., Canou, J., 2003. Cyclic pressuremeter loading and liquefaction properties of  
516 sands. Soils and Foundations 43(2): 17-31.
- 517 Feddema, A., Breedevelt, J. and Van Tol, A. F., 2010. Lateral loading of pile foundations due  
518 to embankment construction. Numerical Methods in Geotechnical Engineering: 631–636
- 519 Goulois, A., Whitman, R. V. and Høeg, K., 1985. Effects of Sustained Shear Stresses on the  
520 Cyclic Degradation of Clay. Strength Testing of Marine Sediments: Laboratory and In-  
521 situ Measurements, ASTM STP 883, R. C. Chaney and K. R. Demars, Eds., American  
522 Society for testing and Materials, Philadelphia, pp. 336-351.
- 523 Hattab, M. and Hicher, P. Y., 2004. Dilating behavior of overconsolidated clay. Soils and  
524 Foundations, Japanese Geotechnical Society, 2004, 44 (4), pp.27-40.  
525 <10.3208/sandf.44.4\_27>.
- 526 Henkel, D. J., 1956. The effect of overconsolidation on the behaviour of clays during shear.  
527 Geotechnique, 6, 139-150
- 528 Henni, A. D., Arab, A., Khelafi, H., 2012. Laboratory study of overconsolidated ratio of the  
529 undrained soil behavior. *2nd International Seminar INVACO – Innovation & Valorization*  
530 *in Civil Engineering & Construction Materials*

- 531 Huang, A.-B., Holtz, R. D. and Chameau, J.-L., 1988. A Calibration Chamber for Cohesive  
532 Soils. *Geotechnical Testing Journal* 11(1): 30-35.
- 533 Koutsoftas, D. C., 1981. Undrained Shear Behavior of a Marine Clay. *Laboratory Shear*  
534 *Strength of Soil*. ASTM STP 740. R. N. Yong and F. C. Townsend, Eds., American  
535 Society for Testing and Materials, 1981, pp. 254-276.
- 536 Kurup, P. U., Voyiadjis, G. Z. and Tumay, M .T., 1994. "Calibration Chamber Studies of  
537 Piezocone Test in Cohesive Soils" *Journal of Geotechnical Engineering*, Vol. 120, No. 1,  
538 January, pp. 81-107.
- 539 Le Kouby, A., Canou, J. and Dupla, J.-C., 2004. Behaviour of Model Piles Subjected to Cyclic  
540 Axial Loading. In *Proceedings of the International Conference on Cyclic Behaviour of*  
541 *Soils and Liquefaction Phenomena*, Bochum, Germany, 31 March - 2 April 2004, pp.  
542 159-166.
- 543 Lee, C. Y. and Poulos, H. G., 1990. Experimental Investigations of Axial Capacity of Model  
544 Grouted Piles in Marine Calcareous Sediments. Research Report No. R618, University of  
545 Sydney, School of Civil and Mining Engineering.
- 546 Lehane, B. M. and White, D. J., 2004. Friction Fatigue on Displacement Piles in Sand.  
547 *Géotechnique* 54(10): 645-658.
- 548 Matlock, H. , Bogard, D. and Cheang, L. 1982. A Laboratory Study of Axially Loaded Piles  
549 and Pile Groups Including Pore Pressure Measurements. In *Proceeding of The Third*  
550 *International Conference on the Behavior of Offshore Structure (BOSS)*, Vol. 1, pp. 105-  
551 121.

552 Parez, I. and Fauriel, R., 1988. Le piézocône. Améliorations apportées à la reconnaissance de  
553 sols. *Revue Française de Géotechnique* 33: 13-27.

554 Potts, D. M. and Martins, J. P., 1982. The Shaft Resistance of Axially Loaded Piles in Clay.  
555 *Geotechnique*, Vol. 32, No. 4, pp. 369-386. Poulos, H. G., 1981a. Some Aspects of Skin  
556 Friction of Piles in Clay Under Cyclic Loading. *Geotechnical Engineering*, Vol. 12, No.  
557 1, pp. 1-17.

558 Poulos, H. G., 1981b. Cyclic Axial Response of Single Pile. *Journal of Geotechnical*  
559 *Engineering Division* 107(1): 41-58.

560 Poulos, H. G., 1982. Influence of Cyclic Loading on Axial Pile Response. R413 Monograph,  
561 University of Sydney, Sydney, New South Wales 2006 Australia, 36 p.

562 Procter, D. C. and Khaffaf, J. H., 1987. Cyclic axial displacement tests on model piles in clay.  
563 *Géotechnique* 37(4): 505-509.

564 Randolph, M. F., 2003. Science and empiricism in pile foundation design. *Geotechnique* 53,  
565 No. 10, 847-875

566 Sheeran, D. E. and Krizek, R. J., 1971. Preparation of Homogeneous Soil Samples by Slurry  
567 Consolidation. *Journal of Materials* 6(2): 356-373.

568 Tsuha, C. H. C., Foray, P. Y., Jardine, R. J., Yang, Z. X., Silva, M. and Rimoy, S., 2012.  
569 Behaviour of Displacement Piles in Sand Under Cyclic Axial Loading. *Soils Foundations*  
570 52(3): 393-410.

571 Tumay M. T. and De Lima, D. C., 1992. Calibration and Implementation of Miniature Electric  
572 Cone Penetrometer and Development, Fabrication and Verification of the LSU In-Situ

573 Testing Calibration Chamber (LSU/CALCHAS). FHWA Research Report No.GE-92/08,  
574 Louisiana Transportation Research Center, Baton Rouge, LA.

575 Voyiadjis, G. Z., Kurup, P. U. and Tumay, M. T., 1993. Preparation of Large-Size Cohesive  
576 Specimens for Calibration Chamber Testing. Geotechnical Testing Journal 16(3): 339-  
577 349.

# Local soil-pile friction evaluation in saturated clays under cyclic loading

## FIGURES

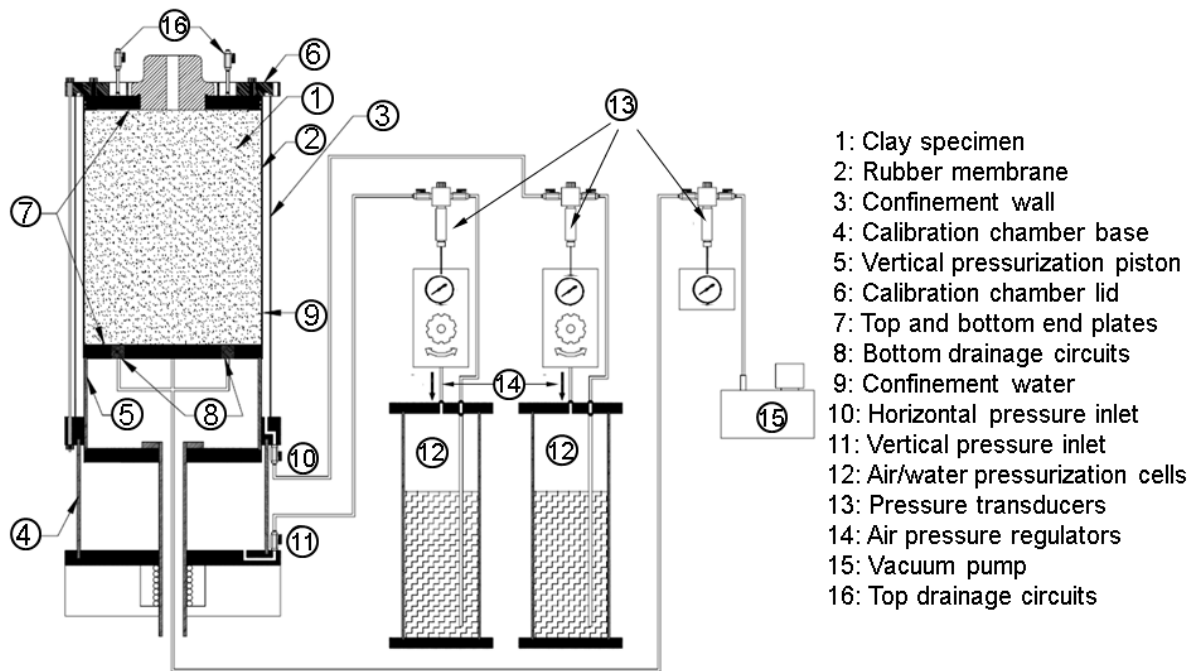


Fig. 1. Functional scheme of the calibration chamber setup

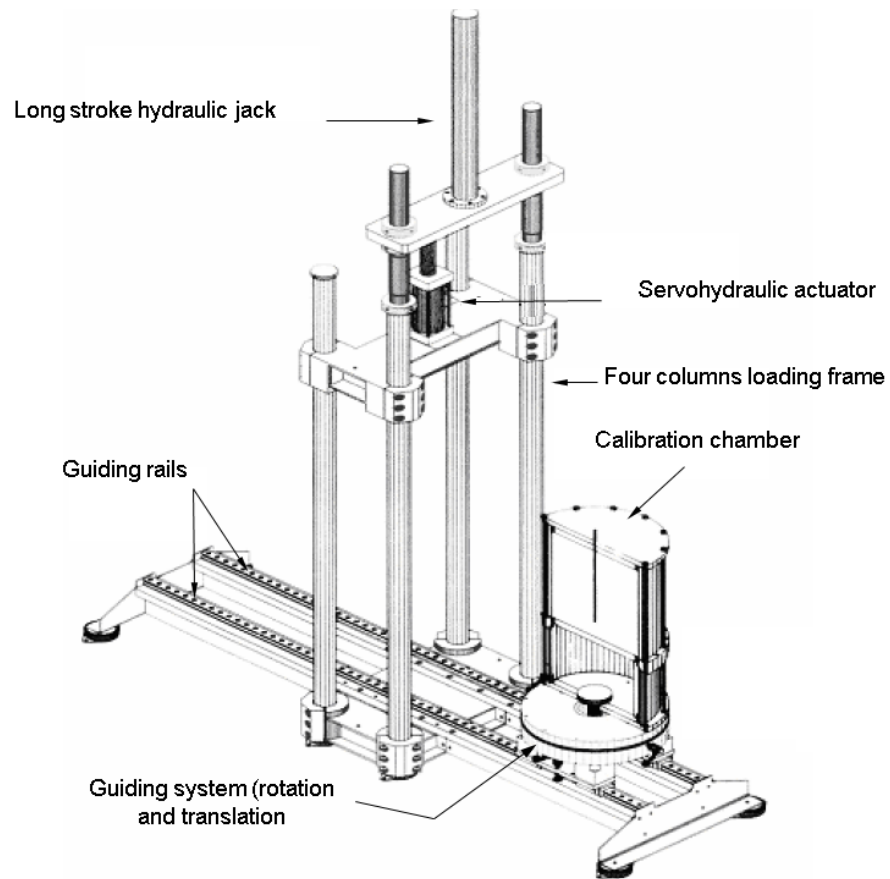
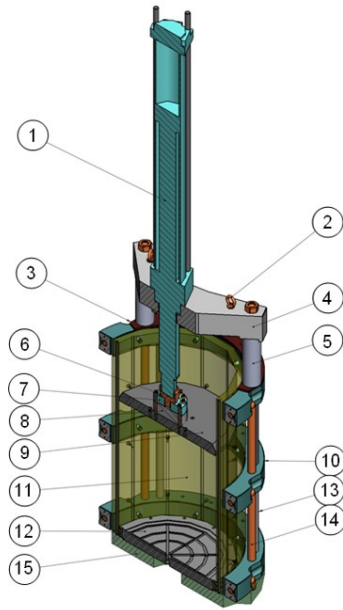


Fig. 2. Functional scheme of the complete setup, including guiding and loading framework

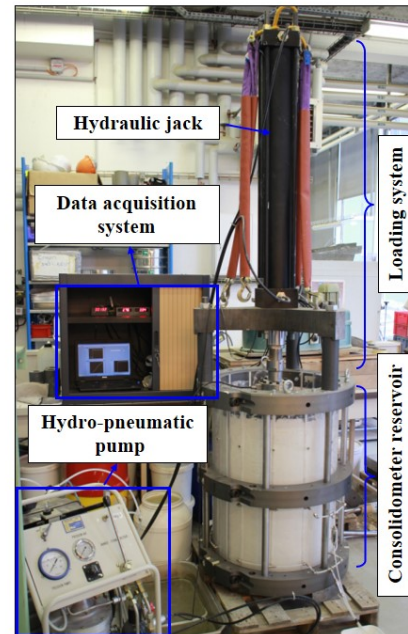


Fig. 3. Overall view of the experimental setup including the control and data acquisition booth

- 1 : Hydraulic jack
- 2 : Lifting rings
- 3 : Positioner
- 4 : Loading frame
- 5 : Loading columns
- 6 : Fitting piece
- 7 : Force transducer
- 8 : Protection piece
- 9 : Top loading and draining plate
- 10 : Metallic reinforcements
- 11 : Plexiglas tube with radial outward drainage
- 12 : Bottom draining plate
- 13 : Intermediate rods
- 14 : Metallic rods
- 15 : Base frame



(a)



(b)

Fig. 4. Consolidometer set up and ancillary equipment: (a) 3-D drawing showing one half of the consolidometer, (b) Overall view of the consolidometer set up and ancillary equipment



- 1: Accelerometer
- 2: Force transducer
- 3: Electrical cables
- 4: Pile-probe body
- 5: Instrumented sleeve friction

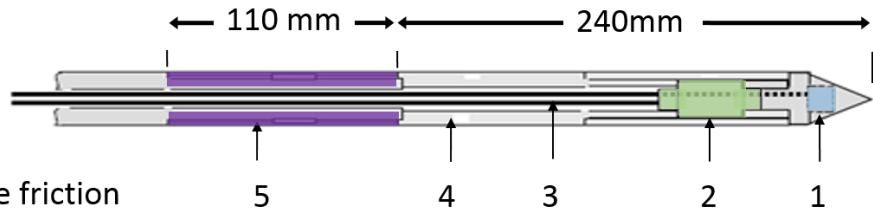


Fig. 5. Simplified cross-section and view of the instrumented pile-probe

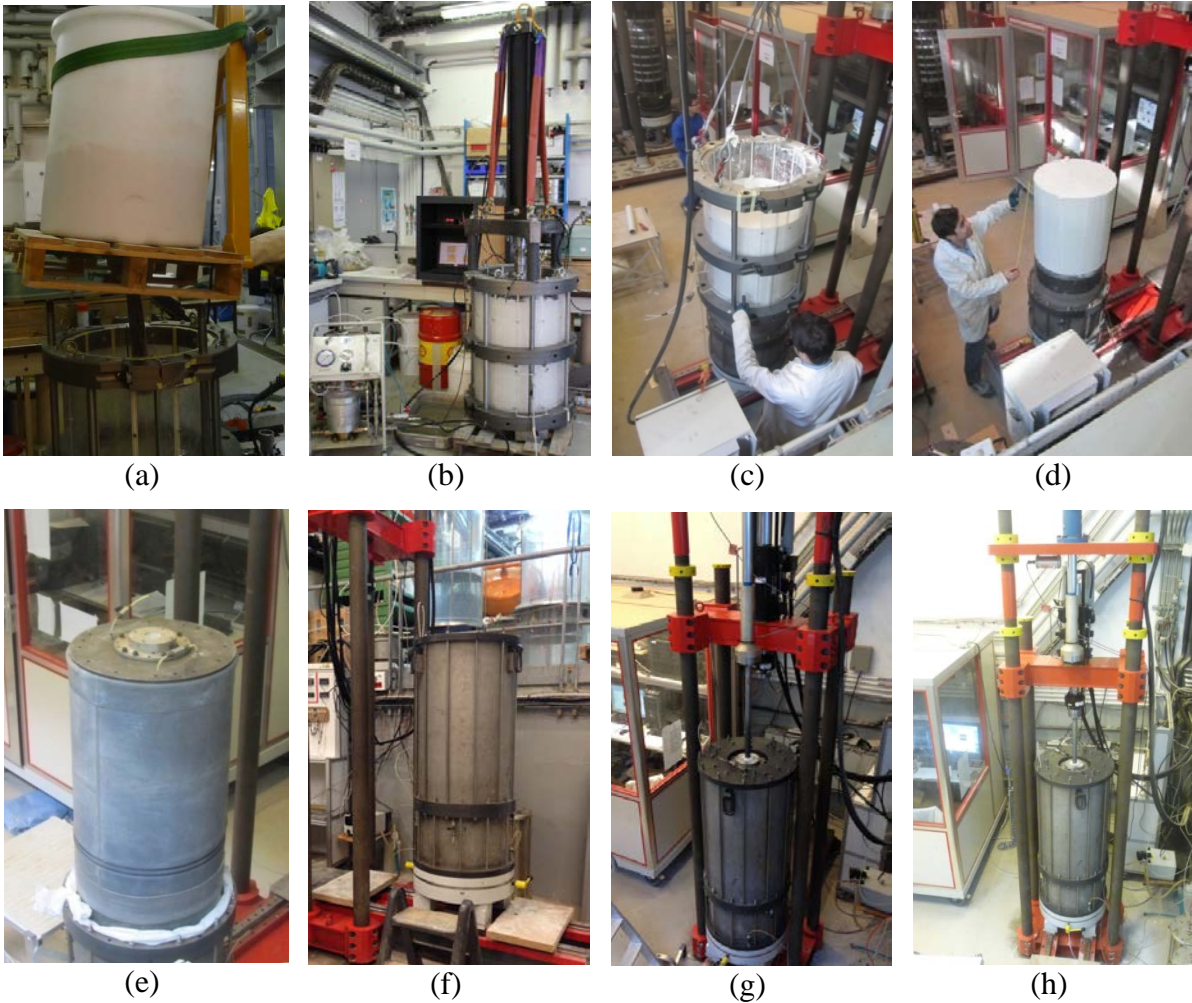


Fig. 6. Experimental procedure : (a) pouring the kaolinite slurry into the consolidometer, (b) application of increasing load increments (consolidation process), (c) adjustment of the clay specimen on the piston of the calibration chamber, (d) removing the consolidometer reservoir, (e) adjustment on top of the specimen the lateral confinement rubber membrane, (f) application of the initial state of stress to the specimen, (g) installation of the pile-probe, (h) ongoing test (after installation of the pile-probe)

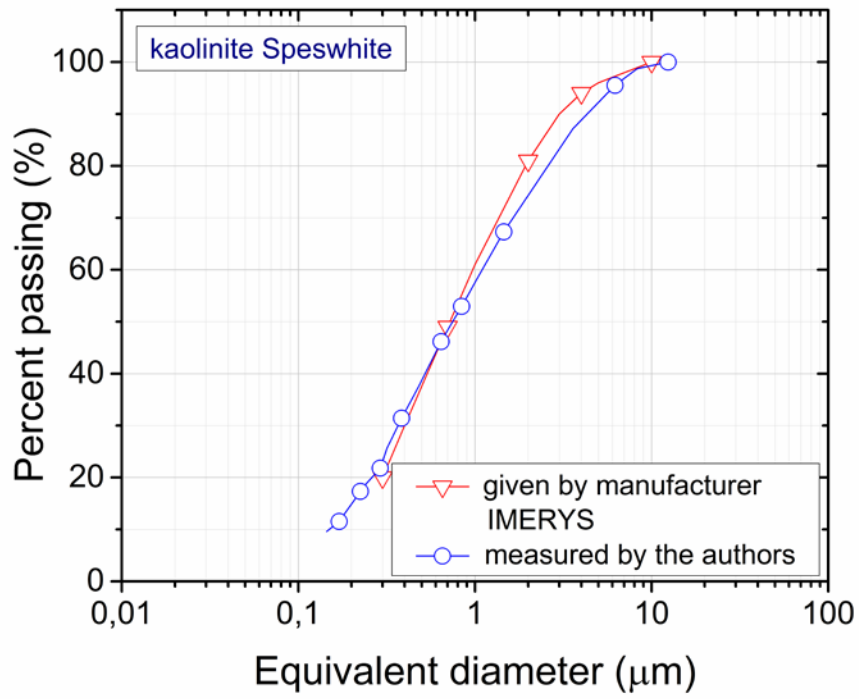


Fig. 7. Grain size distribution curves of Speswhite kaolinite

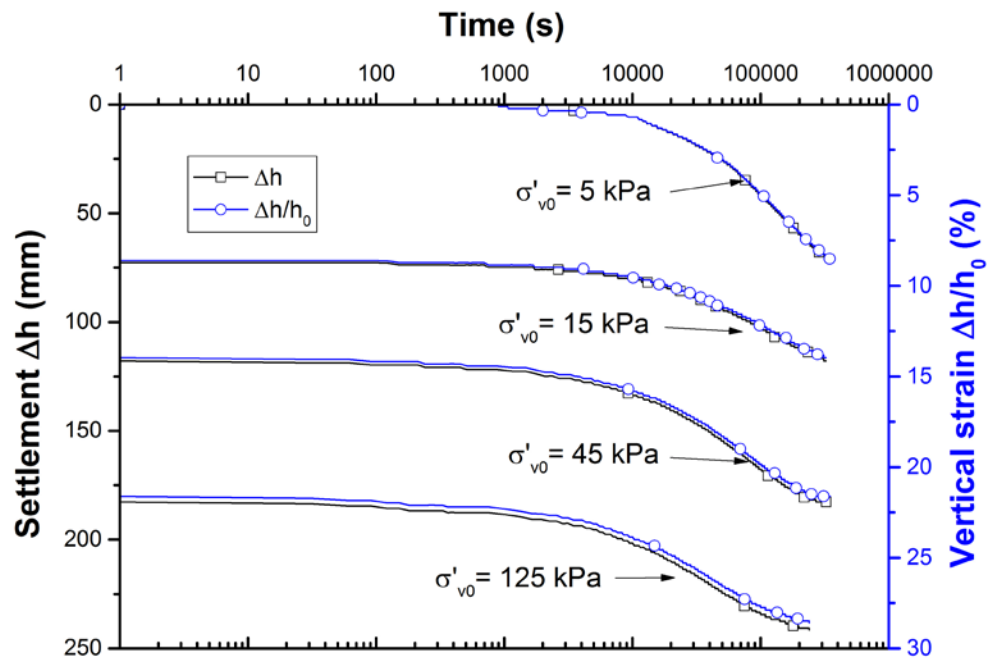
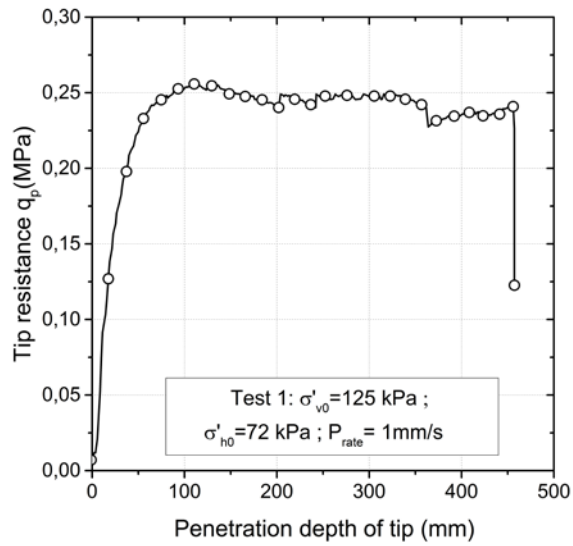
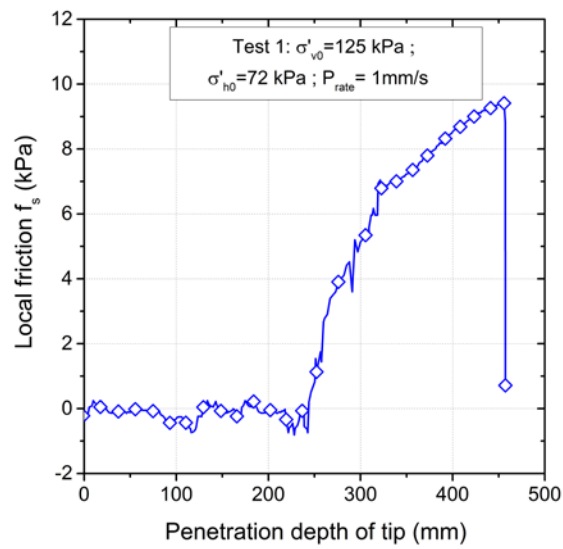


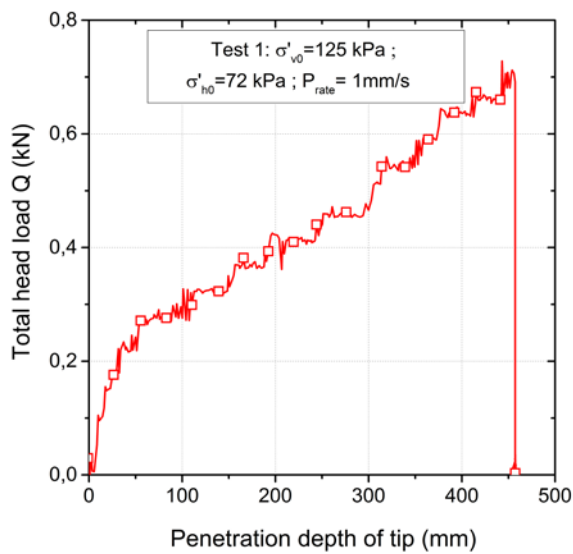
Fig. 8. Settlement and vertical strain versus time consolidation curves



(a)

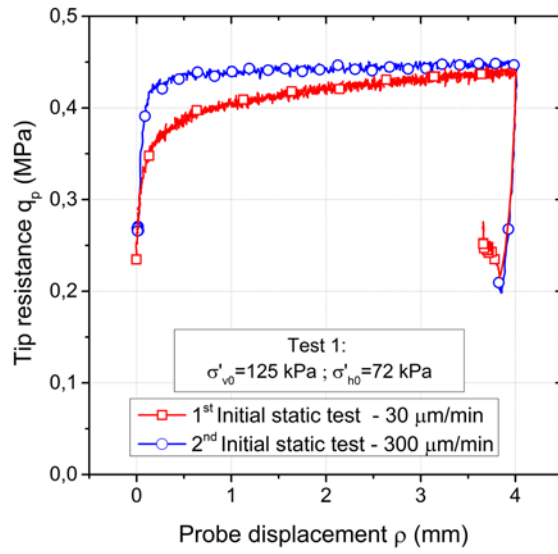


(b)

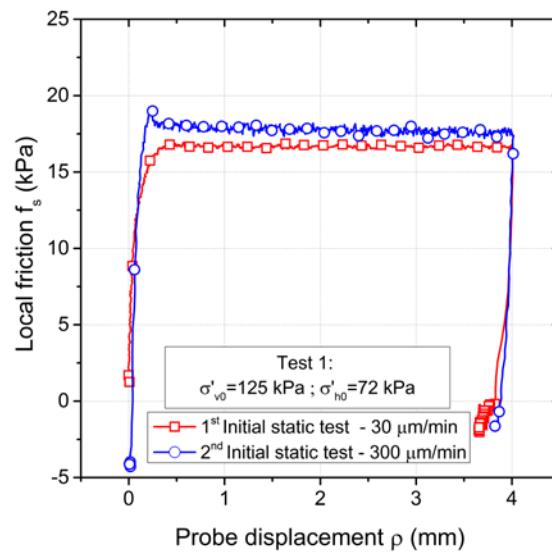


(c)

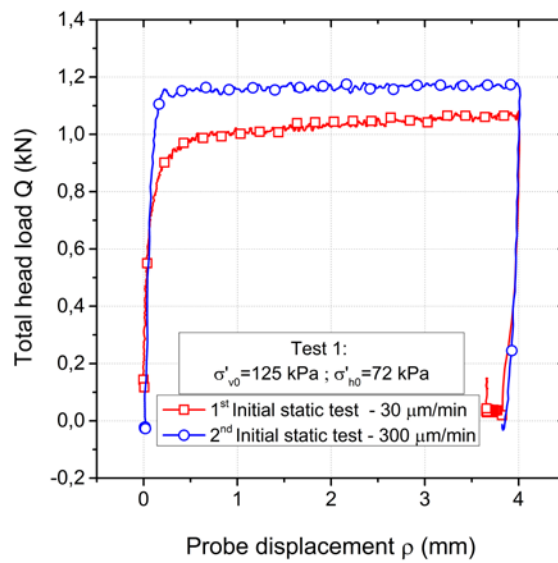
Fig. 9. Installation phase: (a) tip resistance versus penetration depth, (b) local friction versus penetration depth, (c) total load applied versus penetration depth



(a)

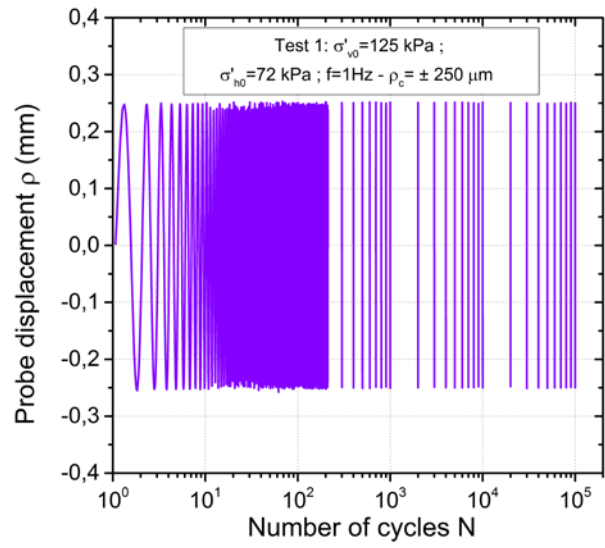


(b)

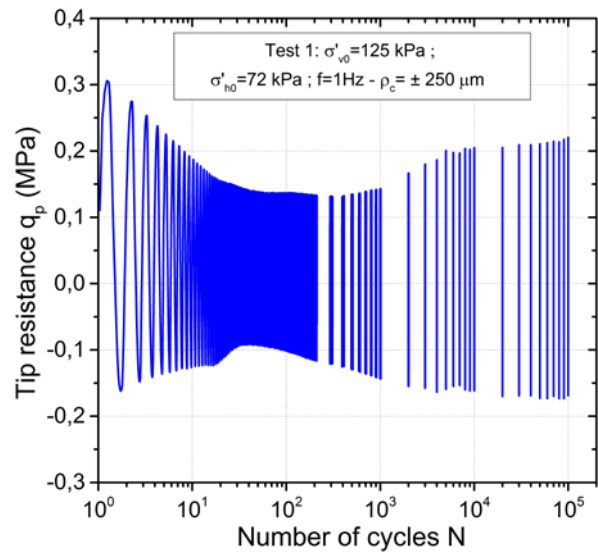


(c)

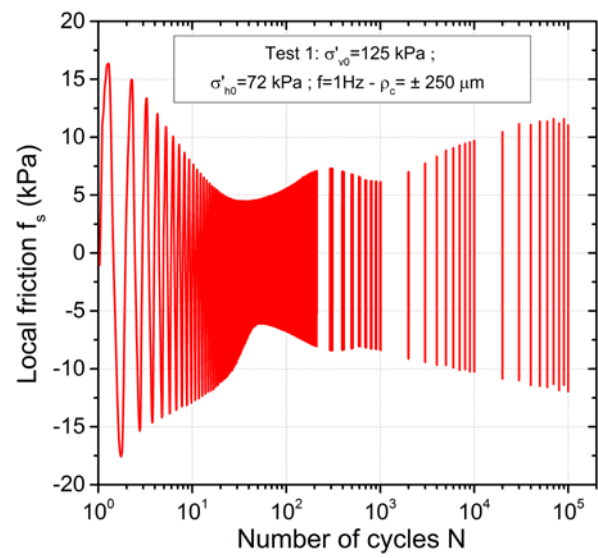
Fig. 10. Initial static loading tests: (a) tip resistance versus vertical displacement, (b) local friction versus vertical displacement, (c) total load applied versus vertical displacement



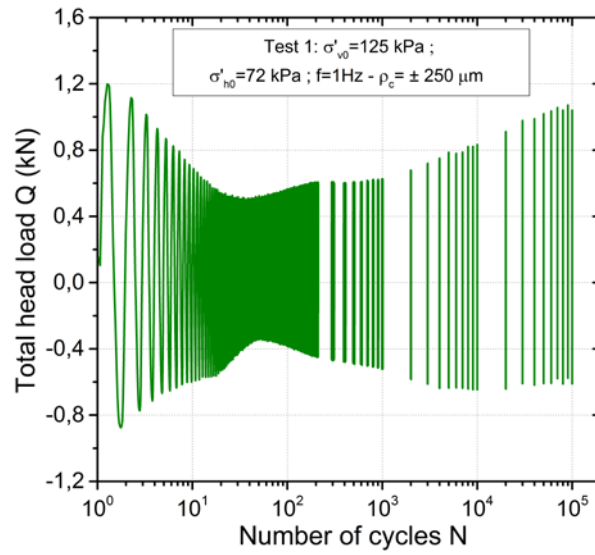
(a)



(b)



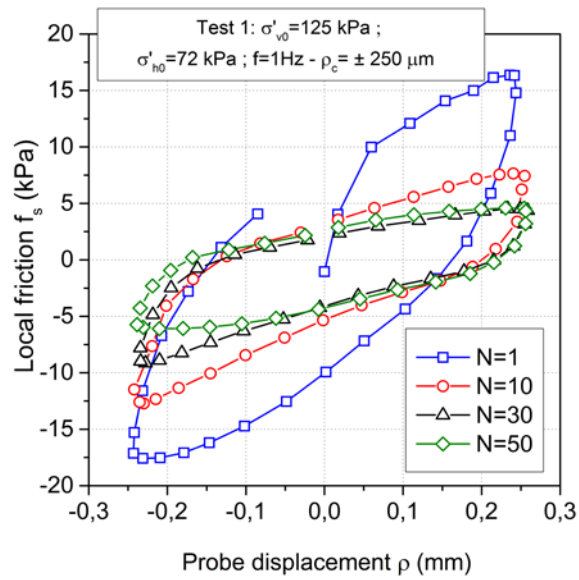
(c)



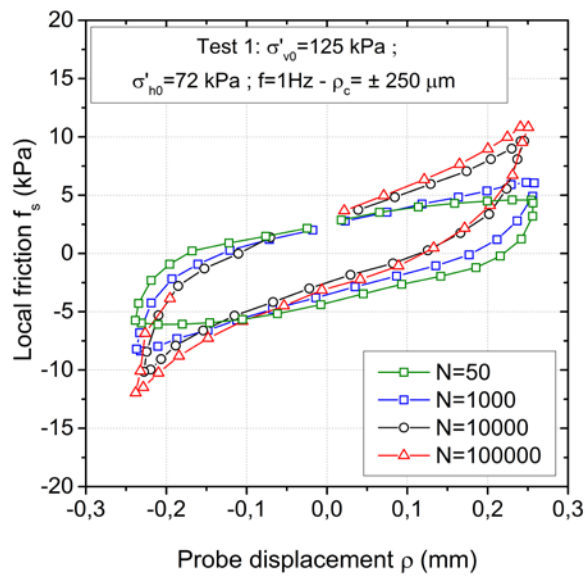
(d)

Fig. 11. Cyclic loading phase: (a) displacement-controlled loading signal versus number of cycles, (b) tip resistance versus number of cycles, (c) local friction versus number of cycles, (d) total load applied versus number of cycles



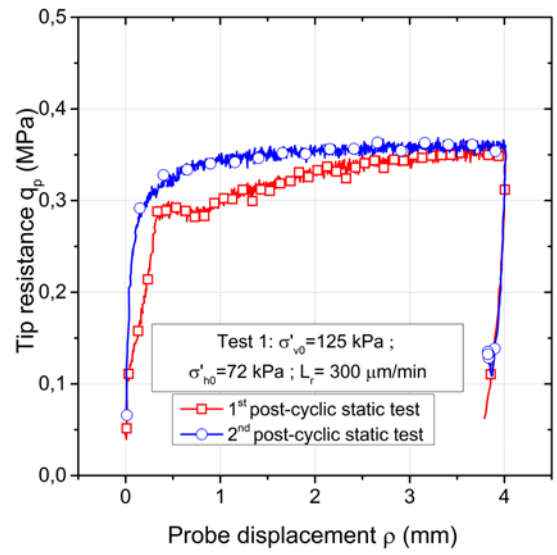


(a)

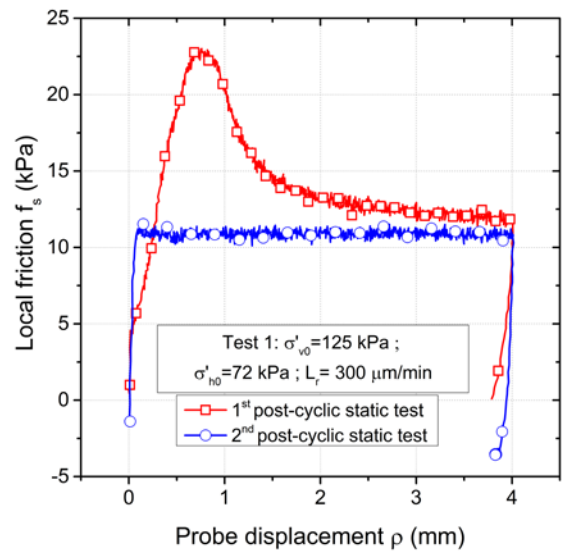


(b)

Fig. 12. Representation of selected cycles of friction mobilization for the typical test presented: (a) corresponding to the cyclic degradation phase, (b) corresponding to the cyclic reinforcement phase



(a)



(b)

Fig. 13. Final static loading phase: (a) tip resistance versus vertical displacement, (b) local friction versus vertical displacement

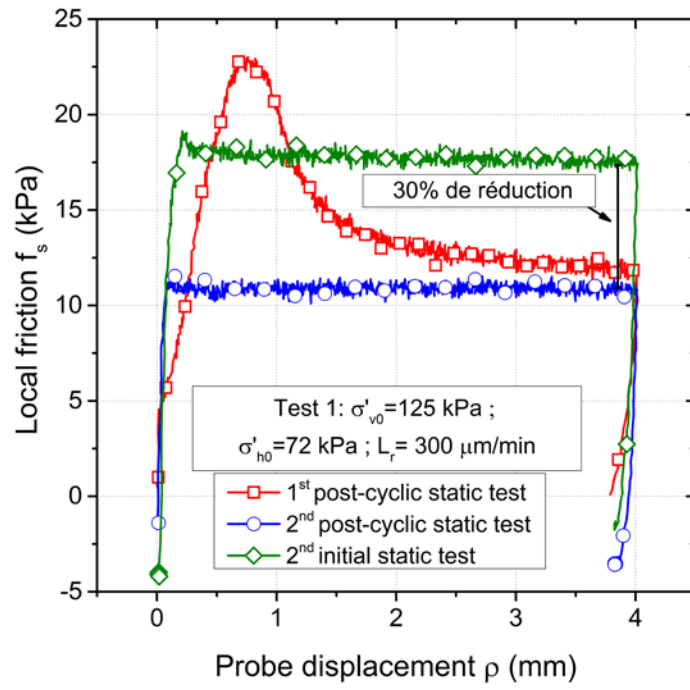


Fig. 14. Comparison between the initial and final static load tests in terms of local friction

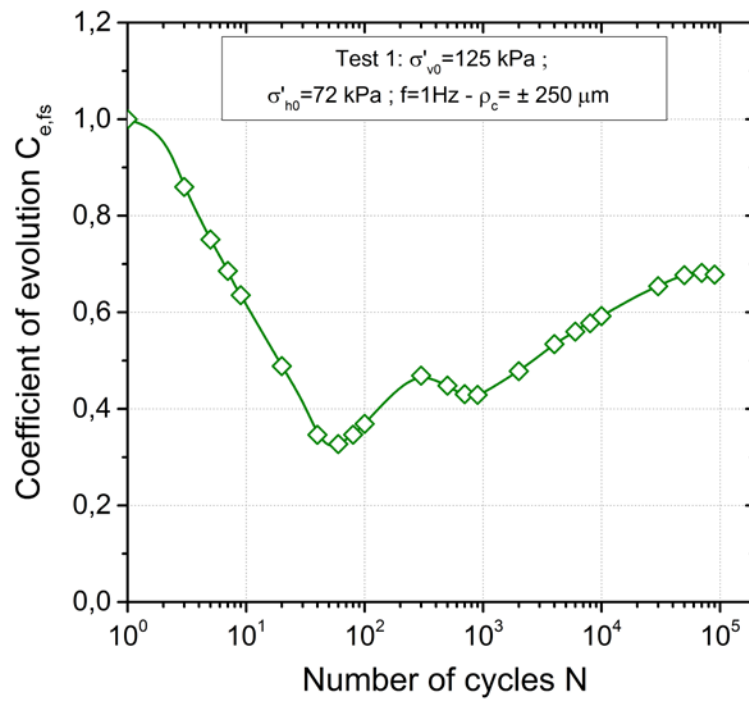


Fig. 15. Coefficient of evolution  $C_{e,fs}$  versus number of cycles for the test presented

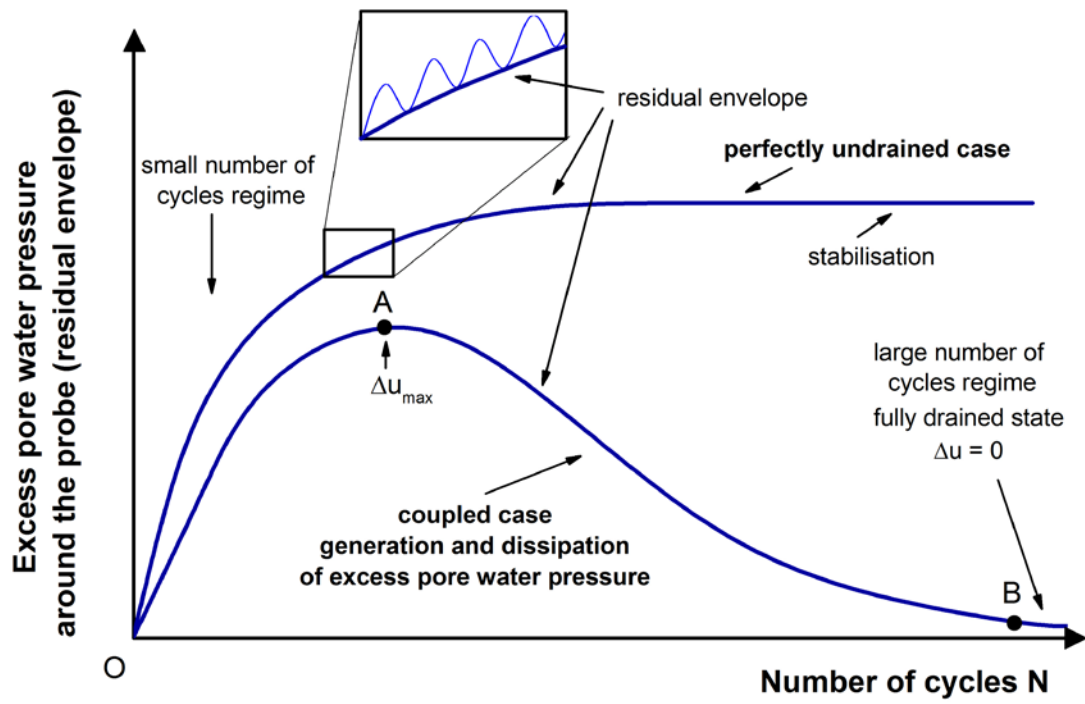


Fig. 16. Conceptual scheme of excess pore water pressure generation and dissipation around the pile-probe sleeve friction

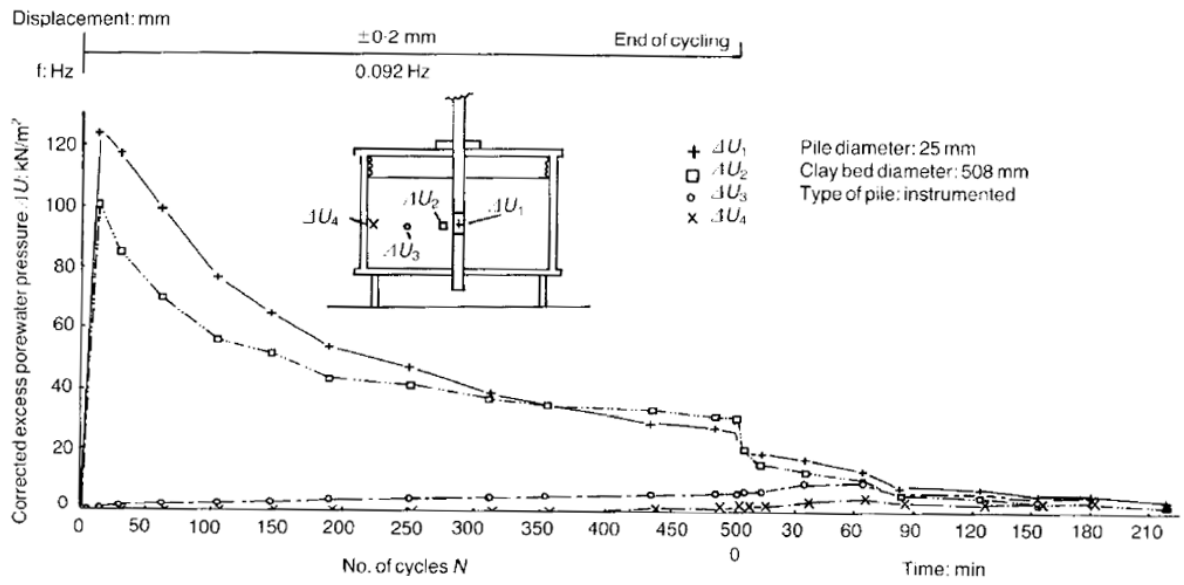


Fig. 17. Excess pore water pressure evolution during a displacement controlled cyclic test on a model pile element (Procter and Khaffaf, 1987)

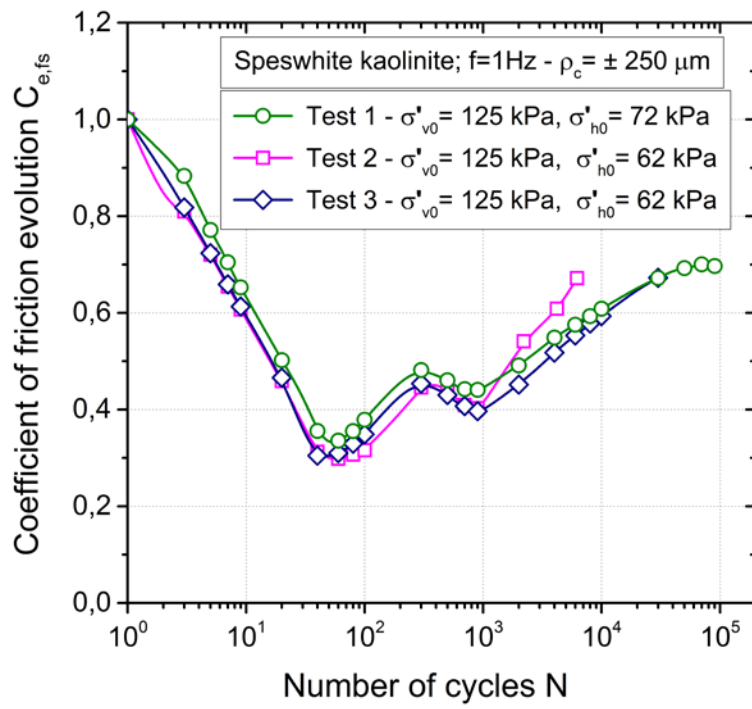


Fig. 18. Evaluation of test repeatability in terms of  $C_{e,fs}$  coefficient versus number of cycles

## Local soil-pile friction evaluation in saturated clays under cyclic loading

### TABLES

Mineralogy	Liquid Limit (%)	Plastic limit (%)	Plasticity index (%)	Soil particle density (t/m <sup>3</sup> )	Percent. finer than 10 μm (%)
kaolinite	58	28	30	2,64	98

Table 1. Physical properties of Speswhite kaolinite

Test identification	$\sigma'_{v0}$ (kPa)	$\sigma'_{h0}$ (kPa)	Frequency $f$ (Hz)	$\rho_c$ (μm)	Number of cycles	Observations
Test 1	125	72	1	±250	100 000	Reference test
Test 2	125	62	1	±250	7 000	Repeatability test
Test 3	125	62	1	±250	100 000	Repeatability test

Table 2. Main characteristics of tests performed