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Bucket Foundation Model Testing under Tensile Axial Loading (re-submission of cgj-2015-0497)

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Abstract

The present study focusses on the bucket foundation behaviour under long-term cyclic loading. The paper analyses model testing results of a bucket foundation model exposed to cyclic tensile loading. The model dimensions are 1 m in diameter and 0.5 m in skirt length. It is installed in dense water-saturated sand. Slow monotonic loading tests and cyclic tensile loading tests are performed (up to 40,000 load cycles) including tests with mean cyclic load in tension which is unique in this sense. High quality data is documented for load, displacement and pore pressure response. Conclusions are drawn regarding static and cyclic loading stiffness and displacement development during the long-term cyclic loading. Four cyclic loading tests induced partially drained soil conditions and showed that pore pressure can accumulate during the long-term loading. Post-cyclic monotonic tensile loading tests showed up to 25% reduction in capacity. The research results supply valuable information for the design of an upwind bucket foundation under a jacket structure.

Keywords: bucket foundation, tensile loading, axial loading, cyclic loading, long-term loading, 1g testing, dense sand

75 Introduction

76 Bucket foundations have been used for decades as a suitable support for various offshore
77 structures in the oil and gas industry. Tjelta (2015) thoroughly explains the development of
78 suction foundation technology including historical overview and main features. Currently,
79 interest is growing for building extra-large, but still relatively light and very slender wind turbines
80 in deep waters. Compared to an oil platform, a large wind turbine has a small self-weight. To
81 stand in deep water, a wind turbine can be supported on a multi-foundation system. Thus, the
82 large horizontal loading coming from wind and waves would be transferred to dominating axial
83 loads on each foundation. An optimal foundation design solution for extra-large wind turbines
84 should be found, and it is not a straight forward process for two reasons: firstly, the foundations
85 should be cost-effective; and secondly, they should be able to resist tensile loading. In some
86 cases, a long-term cyclic tensile loading would be unavoidable unless the spacing between
87 each foundation is increased or an additional dead-load is placed.

88
89 Until today, a significant number of bucket foundation studies were performed at several
90 research institutes and universities, among the best known are Oxford University, Norwegian
91 Geotechnical Institute and Aalborg University. At Aalborg University, monopod bucket
92 foundations were analysed in detail by Feld et al. (1999 and 2000), Larsen et al. (2013), Foglia
93 and Ibsen (2013) and Barari et al. (2015).

94
95 An extensive study at Oxford University focussed on axially loaded suction caisson foundations.
96 It included tests with caisson foundation models in different soils in a pressure chamber (Kelly et
97 al. 2003) as well as a large-scale field testing (Kelly et al. 2006a). Most of the times, the
98 performed cyclic loading tests had mean compressive loading and a small part of the loading
99 amplitude was tensile; the highest number of cycles was 1000. It was claimed that, to have a
100 safe design, the highest tensile loading acting on a bucket foundation should be limited to the
101 drained friction resistance (Kelly et al. 2003, 2004, 2006a and 2006b). Thus, the tensile capacity
102 would consist of friction on the outer skirt, self-weight of the structure and the lower of the soil
103 plug weight and the inner skirt friction. Byrne and Houlsby (2006) stated that the tension on the
104 upwind foundation should be generally avoided. However, limited amount of model testing data

105 is publicly available focussing on the cyclic tensile loading on bucket foundations, especially
106 with high cycle number and various mean tensile loads.

107

108 A medium-scale bucket foundation model, corresponding to 1:10 prototype size, was used for
109 the testing of load, displacement and pore pressure response in a 1g testing rig. Cyclic loading
110 program consisted of up to 40,000 constant load amplitude cycles followed by a final monotonic
111 pull-out quantifying the cyclic degradation of the frictional resistance. Furthermore, dimensional
112 analysis on stiffness and displacement was applied according to Kelly et al. (2006a). Through
113 the physical model testing, this paper aims at visualizing the bucket foundation behaviour under
114 long-term cyclic loading conditions. The influence of the axial tensile load on the displacement
115 and cyclic degradation is studied.

116

117 **Equipment and testing**

118 Quasi-static and permanent loads, such as mean cyclic loads, induce drained conditions in soil.
119 Under drained conditions, the tensile capacity of the bucket foundation corresponds to the dead
120 load of the structure and the frictional resistance generated in the soil-structure interface.
121 Furthermore, it is a lower bound solution for the tensile capacity. This was taken as a starting
122 point in the present model testing program which focussed on the drained response of the
123 bucket foundation model examined by a specially designed laboratory test set-up.

124

125 The laboratory test facility was launched in 2013 and presented for the first time by Vaitkunaite
126 et al. (2014). The test set-up consisted of a large sand container, an installation, a loading frame
127 equipped with two hydraulic cylinders and an automatic load regulation system. The tests
128 presented in this article were performed with bucket foundation model of 1 m in diameter D , 0.5
129 m in skirt length d and 3 mm skirt thickness t , which corresponds to prototype foundation with
130 d/D ratio 0.5. The bucket is made of steel and has a self-weight of 204 kg including the
131 connection flange between the model and the load application point, see Fig. 1.

132

133

Sand

The rigid sand container had an inner diameter of 2.5 m. It was filled with 0.3 m of gravel used for drainage and 1.2 m of Aalborg University sand No. 1 (Fig. 1). The latter is quartz sand which contains a small part of biotite and feldspar. The small sand particles are sub-angular while the larger particles are rounded. The main sand properties were as follows: specific grain density 2.64 g/cm^3 , lowest void ratio e_{min} 0.549, highest void ratio e_{max} 0.858, 50%-quantile 0.14 mm, uniformity coefficient 1.78. Borup and Hedegaard (1993) and Ibsen and Boedker (1994) thoroughly reported the sand properties. Fig. 3 shows particle size distribution performed by sieve analysis. The sand was saturated and compacted to a planned density level before every test. Aalborg University sand No. 1 has a hydraulic conductivity of $8 \times 10^{-5} \text{ m/s}$ when prepared to relative density $D_R = 80\%$. Sjelmo (2012) performed a permeability analysis of Aalborg University sand No. 1 through a falling head apparatus.

Testing procedure and data sampling

Vaitkunaite et al. (2014) described testing procedure that was applied for this program. To begin with, the sand was loosened with an upward gradient. Afterwards, the sand volume was compacted by a rod vibrator. Later on, at least four small-scale laboratory cone penetration tests (CPT) were performed to inspect soil conditions. Larsen (2008) described the laboratory CPT device and the methodology for the CPT interpretation. Ibsen et al. (2009) estimated D_R from the laboratory cone resistance. Tests presented in this article were performed in dense sand with $D_R \approx 80\%$, see Table 1.

Mechanical push installation of bucket foundation models was used by Kelly et al. (2003), Kelly et al. (2006a) and Foglia (2015). Kelly et al. 2006a compared installation by suction and pushing and documented that the soil disturbance due to suction installation was more visible in small-scale model tests, probably, because of local soil disturbances which do not increase in proportion to caisson diameter. It is expected that suction installation loosens soil in the inner interface zone and reduces the strength properties of soil-structure interface in a short-term. However, the properties are probably regained after some time due to small cyclic wave loading which is typical in summer weather conditions. Nielsen (2016) performed laboratory tests with

164 bucket foundation models. He showed that a bucket foundation model gained 1.6 times larger
165 bearing capacity when pre-loaded with small load cycles. This study dealt with the foundation
166 model installed by mechanical pushing, because the focus was operational loading discharging
167 the installation impact.

168

169 Installation started with positioning the bucket right on the soil surface level, zeroing the signal
170 of the load cell and penetrating the bucket skirt down to the soil with 0.2 mm/s velocity. Two
171 large valves were kept open during the installation to let the air flow out. The installation ended
172 when approximately 70 kN force was reached. Right after that, the model was completely
173 unloaded. The installation required about 50 kN force from the hydraulic cylinder. An elastic
174 compressive preload of 20 kN ensured that the bucket was fully installed and kept the
175 procedure repetitive (Fig. 4). In most of the cases, during the installation, the water table was
176 about 3 cm above sand surface. During installation procedure, it was noticed that the load cell
177 reacted to the lid contact with the water table. In a few tests, where the water table was at the
178 same level of the sand surface, only one change in the response was noticed corresponding to
179 the level of the soil surface. This means that the installation rate was slightly higher than would
180 be required for completely drained soil response. However, the installation velocity was limited
181 by the equipment and could not be lower than 0.2 mm/s. Thus, bucket model was left to rest in
182 the sand at least 2 hours allowing pore pressure dissipation. Load and position signals were
183 measured during the installation process with sampling frequency of 1 Hz.

184

185 The testing program of the monotonic tensile loading and cyclic loading tests is presented in
186 details in the following section. Load, displacement and pore pressure responses were
187 measured during the tests. Load cell had a capacity of 250 kN and was positioned right above
188 the foundation model. Position of the hydraulic cylinder piston was measured with a
189 displacement transducer. Additionally, two displacement transducers were placed on the lid of
190 the foundation model; they could measure displacements of up to 125 mm. To analyse the
191 drainage conditions, pore pressure response was measured at different levels on the model
192 skirt. Pore pressure transducers were placed above the bucket model and connected to
193 measuring positions via narrow water-filled pipes. As the cyclic tests typically took up to five

194 days, the atmospheric pressure changes were also measured and separated from the pore
 195 pressure measurements. Fig. 2 shows all the mentioned transducers. Data sampling rate
 196 depended on the type of loading, i.e. 1 Hz frequency was chosen for monotonic loading tests
 197 and 2 Hz frequency for cyclic loading tests.

198

199 MOOG modular test controller was used to control and monitor various loading regimes; for
 200 example, tension, compression and periodic loading with various force or displacement
 201 amplitudes and frequencies. The system was able to work continuously for several days, which
 202 was important for the high cycle number loading tests.

203

204 **Loading program**

205 Before describing the loading program, it is worth mentioning that tensile loading, upward
 206 displacement and suction pore pressure are marked with a negative sign or drawn on the
 207 negative axis.

208

209 For the presentation of the load cases, two parameters were used to characterise the cyclic
 210 axial loading, i.e. ξ_A and ξ_m . Ratio ξ_A (eq. 1) describes the cyclic loading amplitude F_{cyc}
 211 normalized with reference monotonic tensile load F_{TR} . ξ_A takes a value from 0 to 1, where F_{cyc}
 212 equal to F_{TR} results in $\xi_A=1$ and the smaller F_{cyc} results in $\xi_A<1$. The second parameter (eq. 2)
 213 expresses the ratio between the mean cyclic load F_{mean} and the reference monotonic tensile
 214 capacity F_{TR} . ξ_m takes a value from -1 to 1, where mean tensile cyclic load equal to F_{TR} results in
 215 $\xi_m=1$ while mean compressive cyclic load equal to $|F_{TR}|$ results in $\xi_m=-1$. A perfect two-way
 216 cyclic loading with $F_{mean}=0$ results in $\xi_m=0$. Fig. 5 visualizes the mentioned loading parameters.

$$217 \quad (1) \quad \xi_A = -\frac{F_{cyc}}{F_{TR}}$$

$$218 \quad (2) \quad \xi_m = -\frac{F_{mean}}{F_{TR}}$$

219 In the first set of tests, the monotonic tensile capacity was measured. The tests were position
 220 controlled with maximum vertical pull-out velocity of 0.002 mm/s which ensured that the soil
 221 response was drained.

222

223 The second set of tests focussed on drained behaviour of bucket foundation subjected to cyclic
 224 axial loading. Sinusoidal cyclic loading with constant amplitude and mean load was applied on
 225 the model. Loading frequency f was 0.1 Hz, except in one test presented in this paper where the
 226 frequency was 0.05 Hz. Each time, the testing program consisted of more than 20,000 load
 227 controlled cycles followed by a position controlled monotonic pull-out F_{Pc} .

228

229 **Test results**

230 All monotonic tensile loading tests showed very similar behaviour and similar tensile peak
 231 resistance. Pore pressures did not exceed 1.3 kPa during the loading sequence (Fig. 6), which
 232 was within the measuring accuracy of the transducers. Two different displacement rates were
 233 applied in the four monotonic tests: two tests with 0.001 mm/s and two with 0.002 mm/s. No
 234 influence on the pore pressure transducers was seen for the different displacement rate tests.
 235 The displacement rate was extremely low for water saturated sand and it took 1.4-2.8 hours to
 236 lift the model only 10 mm. Thus the response was assumed to be drained.

237

238 It was noticed that the peak tensile load F_T was reached within the first 6 mm upward
 239 displacement ($0.006D$), which was followed by a continuous decrease in load resistance due to
 240 a decrease in the soil-structure interface area and softening behaviour of the soil. The reference
 241 monotonic tensile resistance F_{TR} was chosen as the average of F_T .

242

243 This paper presents five cyclic loading tests with mean tensile load, three tests with mean
 244 compressive load and one with zero mean load. Fig. 7 shows a cyclic loading test followed by a
 245 post-cyclic monotonic pull-out. The bucket foundation model was allowed to move up to -65 mm
 246 upwards ($0.065D$). In Table 1, two tests are marked as "pulled"; it refers to vertical upward
 247 displacement of -65 mm.

248

249 If no significant displacement rate and pore pressure change were seen, then it was expected
 250 that there was no cycle that could exceed the drained tensile capacity. However, during four
 251 cyclic loading tests, i.e. A0.3m-0.4:1, A0.3m-0.4:2, A0.7m-0.4:1 and A0.7m-0.4:2, pore pressure
 252 build up during the loading sequence was noticed. Loading rate analysis showed that the pore
 253 pressure build up was present in the tests where the loading rate was higher than 0.77 kN/s and
 254 mean tensile load was applied in the loading sequence (see Table 1). Pore water pressure
 255 during cyclic loading is addressed in section *Pore pressure vs. cycles*.

256

257 Some of the tests are referred as critical loading tests or tests where a critical load is reached.
 258 The critical load in this loading program is the drained tensile resistance F_{TR} . The drained soil
 259 response is normally assumed for long-term loading conditions. As the main subject of this
 260 testing program is drained soil response, the loads reaching F_{TR} are referred as critical.

261

262 ***Displacement vs. cycles***

263 All the tests had a tendency to either keep the bucket in the initial position or develop upward
 264 displacement. During tests with $\xi_m=0.3$ and $\xi_A \leq 0.7$, the foundation model stayed in the initial
 265 position (A0.4m0.3, A0.7m0.3:1, A0.7m0.3:2). Very small upward displacements ($<0.006D$)
 266 developed in tests with $\xi_m \in [-0.4, 0]$ and $\xi_A \leq 0.3$ (A0.2m0, A0.3m-0.4:1 and A0.3m-0.4:2). Fig. 8
 267 clearly shows that when critical loading was exceeded - the tensile load reached F_{TR} - the
 268 bucket eventually was pulled out (A0.7m-0.4:1, A0.7m-0.4:2). However, the foundation model
 269 had to be subjected to more than 1,000 cycles of such critical loading to be lifted up to $0.05D$.

270

271 Tests A0.7m-0.4:1 and A0.7m-0.4:2 had identical loading conditions, but slightly different
 272 behaviour, the first test required 8 times more cycles to lift the bucket model to -60 mm (Fig. 8).
 273 Obviously, the displacement rate was changeable in the tests A0.7m-0.4:2 while in A0.7m-
 274 0.4:1 was rather constant. Furthermore, the inner pore pressure in test A0.7m-0.4:2 decreased
 275 constantly (Fig. 13) whereas in the test A0.7m-0.4:1 it was around 0 kPa during the first 5,000
 276 cycles (Fig. 14). This indicated that the accumulation of negative pore pressure resulted in
 277 higher displacement. As the loading rate was identical, it should be expected that slightly lower
 278 soil permeability (thus, higher D_R) could result in the lower negative pore pressure of test

279 A0.7m-0.4:2 compared to test A0.7m-0.4:1. Unfortunately, the density ratio was not estimated
280 for the test A0.7m-0.4:2 due to technical issues.

281

282 Kelly et al. 2006b analysed incremental and cumulative cyclic displacement during one test with
283 mean compressive load (referred as Test15) and showed that the incremental displacement
284 decreased within the first 200 cycles and, afterwards, kept constant with a small positive value
285 pushing the bucket model down to the soil at a constant rate. Positive incremental displacement
286 indicates a safe foundation design range, because the bucket cannot be lifted due to the loading
287 conditions.

288

289 During the present testing campaign it was seen that in most of the examined cases the
290 incremental displacement $w_{i,d}$ became constant within the first 200 loading cycles. Fig. 9 shows
291 accumulated and incremental cyclic displacement during test A0.7m0.3:2. The incremental
292 displacement changed within the first 200 cycles and continued almost constant with a very
293 small positive increment of 10^{-6} mm until the end of the test. Within the first 200 cycles, the
294 model reached the uppermost position and, afterwards, it was slowly pushed downwards to the
295 initial position (0 mm). The accumulated displacement during this long-term cyclic loading test
296 was very small ($<10^{-3}D$). Similar behaviour was seen in tests A0.7m0.3:1 and A0.4m0.3.

297

298 However, all tests with $\xi_m \leq 0$ resulted in negative incremental displacements, where average $w_{i,d}$
299 was in the range from -10^{-7} mm to -10^{-2} mm depending on loading conditions (see Table 2).
300 Furthermore, under the critical tensile loading, the displacement increment varied during the
301 loading sequence, as seen in Fig. 10. Overall, the negative incremental displacements are
302 small, but they would eventually lead to an inadmissible accumulated displacement of the
303 foundation.

304

305 ***Post-cyclic load***

306 Cyclic degradation is a very important factor that should be evaluated when designing offshore
307 foundations. In this testing program, the cyclic degradation was evaluated comparing the virgin
308 loading response and the post-cyclic loading response. The measured monotonic load

309 decreased continuously due to a decrease in the soil-structure interface area and softening
310 behaviour. Thus, the corresponding shaft friction was calculated dividing the measured load by
311 the area of the shaft in contact to soil (Fig. 11). The peak post-cyclic shaft friction was very
312 similar to or lower than the virgin shaft friction; in the performed tests, up to 25% difference in
313 the peak capacity. The peak resistance was reached within the upward displacement of -10
314 mm, which corresponded to $0.01D$.

315

316 ***Pore pressure vs. cycles***

317 Shear and dilation occur when dense sands are subjected to external loads. Negative pore
318 pressure occurs if water is unable to flow into the pores when the soil dilates. Dilation is
319 especially high in low soil stresses (shallow depth). As mentioned earlier, the negative pore
320 pressure is limited by the cavitation.

321

322 Until now, no tensile high cycle number tests were performed measuring pore pressure
323 behaviour at different depths of the bucket model skirt. In this testing campaign, it was noticed
324 that pore pressure depends not only on loading rate, but can also change/develop during long-
325 term cyclic loading as seen in Fig. 12 (test A0.3m-0.4:2). It should be noted that any influence
326 from the atmospheric pressure change during the tests was removed from data, so it shows
327 only the pore pressure state in the soil contact to the foundation model, see also Fig. 2 for the
328 pore pressure measuring positions.

329

330 No pore pressure development (thus, fully drained response) was seen during full loading
331 program in tests A0.2m-0.4, A0.7m0.3:1, A0.7m0.3:2, A0.4m0.3 and A0.2m0. Small suction
332 accumulation during cyclic loading was visible in tests A0.3m-0.4:1 and A0.3m-0.4:2 that were
333 subjected to a higher loading rate of 0.77 kN/s and mean tensile load (Table 1). The highest
334 measured pore pressure reached -8.5 kPa in test A0.7m-0.4:2 which was small compared to the
335 cavitation limit of -100 kPa (Fig. 12 and Fig. 13). However, the generated suction pressure of -
336 8.5 kPa under the bucket lid resulted in 6.7 kN resistance to tensile load which was larger than
337 F_{TR} . As seen in Fig. 13, negative pore pressure was continuously generated through all the
338 loading sequence. Due to measuring accuracy of the pore pressure transducers (± 2 kPa) it is

hard to say the exact time when drained behaviour changed to partially drained and the tensile resistance was induced by negative pore pressure additionally to the interface friction. Moreover, the pore pressure transducers measured different suction in different levels on the bucket skirt, i.e. the inner part of the bucket was subjected to more suction than the outer indicating partial drainage conditions, see Figs. 2 and 13. As mentioned in section *Displacement vs. cycles*, the accumulation of the negative pore pressures influenced the displacements. Vaitkunaite et al. 2016 performed experimental analysis with a bucket foundation model in a pressure tank. The analysis confirmed that pore suction level is closely related to upward displacement rate. More analysis of this matter was out of the focus of the present research.

348

349 ***Stiffness during monotonic and cyclic loading***

Monotonic loading stiffness during the first 10% of load, $k_{10\%}$ was about 11 MN/m, see Fig. 15 for the definition of stiffness. Peak loading stiffness k_{peak} , from the beginning to the peak of the load, was 1.1 MN/m. Post-cyclic tensile load resulted in rather high stiffness. Here $k_{10\%}$ was very high, i.e. from 46 MN/m to 3300 MN/m, while the peak loading stiffness k_{peak} was from 1.0 to 7.3 MN/m.

355

It was noticed that cyclic loading and unloading stiffness were significantly higher than the monotonic loading stiffness and it had similar values to post-cyclic pull-out values of $k_{10\%}$. Moreover, the stiffness was rather constant during the loading sequence except tests A0.3m-0.4:2 and A0.7m-0.4:2, where the stiffness dropped at some point. Fig. 16 shows that test A0.7m0.3:2 unloading stiffness k_{cyc} was about 230 MN/m (mean). On the contrary, k_{cyc} was quite low (only 24 MN/m) in test A0.7m-0.4:2 where large tensile loading dominated. Clear stiffness degradation during cyclic loading was visible only when critical tensile cyclic loading was applied and no degradation in other loading cases.

364

Kelly et al. 2006a performed cyclic loading tests with mean compressive load and showed that unloading cyclic stiffness had no degradation during cyclic loading, i.e. it was constant or increasing during the loading sequence. Such tendency is confirmed with the present tests, except when critical tensile cyclic loading was applied (tests A0.7m-0.4:1 and A0.7m-0.4:2).

369

370 ***Dimensional analysis***

371 Kelly et al. 2006a provided equations for comparison of laboratory and field tests which
372 indicated that the method used for data normalization was very successful. Indeed, the study
373 showed good comparability of stiffness in any of the cycles. However, accumulation of
374 deformation during the cyclic loading (90 cycles) had smaller similarity when comparing
375 laboratory and field test data. Analysis of vertically loaded caissons showed that disturbance
376 due to suction installation was less important for field caisson. It is worth mentioning that the
377 dimensional analysis was applied to test data with foundation models having 0.15, 0.2 and 1.5
378 m diameter while the present study analyses a foundation model with 1 m in diameter.
379 Moreover, the loading conditions were quite different compared to the present study, since
380 herein more attention is given to the tensile loading. Model testing data provides valuable
381 knowledge for the full-scale foundation design. Therefore, the application of the theoretical
382 background is of interest.

383

384 Fig. 17 shows the first, middle and last cycles of the corresponding loading sequence in the
385 normalized space. It is clearly seen that test with mean compressive loading (A0.7m0.3:2)
386 presented significantly stiffer behaviour compared to test with mean tensile loading
387 (A0.7m-0.4:2). Softer behaviour of an axially loaded bucket foundation model in tension was
388 also noticed by Kelly et al. 2006a. Actual unloading stiffness is plotted in Fig. 18 which shows
389 that the unloading stiffness was approximately 6.5 times higher in test A0.7m0.3:2. It is
390 impossible to compare directly Kelly et al. 2006a results to the present data, because the load
391 amplitudes herein are much smaller. The softer behaviour of tensile loading tests with negative
392 F_{mean} resulted in progressive uplift of the foundation mode is as seen in Fig. 19.

393

394 **Conclusions**

395 The study drew attention to the cyclic tensile loading on a bucket foundation model. The paper
396 presented results from four monotonic and nine cyclic loading model tests where several tests
397 had a mean tensile load which has not been publically documented until now. The medium
398 scale bucket foundation model, corresponding to 1:10 prototype size, was subjected to large

399 number of load cycles with various mean loads. Continuous measurements of load,
400 displacement and pore pressures visualized the actual tensile loading behaviour. Based on the
401 performed laboratory tests, the following findings can be drawn:

- 402 • Displacement during long-term cyclic loading: All the tests had a tendency to either
403 keep the bucket in the initial position or develop upward displacement. During tests with
404 ξ_m from -0.4 to 0.3 and $\xi_A \leq 0.3$, the accumulated displacement was smaller than $0.006D$.
405 Tests with $\xi_m = -0.4$ and $\xi_A = 0.7$ resulted in gradual pull-out. However, the foundation
406 model had to be subjected to more than 1,000 cycles of such critical loading to be lifted
407 up to $0.05D$.
- 408 • A constant negative incremental displacement developed in the tests with $\xi_m \leq 0$ which
409 indicated that the bucket foundation would be eventually moved to an inadmissible
410 upward displacement.
- 411 • Pore pressure during long-term cyclic loading: Negative pore pressures developed
412 during four tests indicating partially drained behaviour. Consequently, it was noticed that
413 pore pressure can continuously accumulate during long-term cyclic loading.
- 414 • Cyclic loading stiffness during long-term cyclic loading: During tests where the mean
415 compressive loading dominated, the cyclic unloading stiffness was higher compared to
416 tests where mean tensile loading dominated. Generally, cyclic stiffness was much
417 higher than monotonic loading stiffness. Clear stiffness degradation during cyclic
418 loading was visible only when critical tensile cyclic loading was applied and no
419 degradation in other loading cases. Unloading stiffness was slightly higher compared to
420 loading stiffness.
- 421 • Cyclic degradation: The peak post-cyclic shaft friction was very similar to or lower than
422 the virgin shaft friction; in the performed tests, up to 25% difference in the peak
423 capacity. The peak tensile resistance was reached within upward displacement of 1% of
424 bucket diameter ($0.01D$).
- 425 • Dimensional analysis by Kelly et al. 2006a was applied and visualized well the different
426 trends in the tests for load, displacement and cyclic stiffness development. The

dimensional analysis can be validated only performing similar loading program on bucket foundation models of different size.

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498

499 **List of symbols**

500	D	is the bucket foundation diameter
501	D_R	is the sand relative density
502	d	is the bucket foundation skirt length
503	e_{min}	is the minimum void ratio
504	e_{max}	is the maximum void ratio
505	F_{cyc}	is the cyclic loading amplitude
506	F_{mean}	is the mean cyclic load
507	F_{Pc}	is the post-cyclic monotonic tensile capacity
508	F_T	is the monotonic peak tensile capacity
509	F_{TR}	is the reference monotonic tensile capacity
510	f	is the loading frequency
511	g	is the acceleration of gravity
512	$k_{10\%}$	is the tensile loading stiffness taken at 10% of the tensile loading
513	k_{cyc}	is the cyclic unloading stiffness
514	k_{peak}	is the peak tensile loading stiffness
515	N	is the number of cycles
516	t	is the bucket foundation skirt wall thickness
517	$w_{i,d}$	is the incremental displacement
518	γ'	is the effective soil unit weight
519	ξ_A	is the ratio describing cyclic loading amplitude normalized with reference monotonic
520		tensile load
521	ξ_m	is the ratio describing mean cyclic load normalized with reference monotonic tensile
522		load

Tables (2)

Table 1. Testing program.

Test ID	Type	D_R (%)	γ' (kN/m ³)	F_T / F_{Pc} (kN)	$F_{mean} \pm F_{cyc}, f$ (kN, Hz)	LR (kN/s)	ξ_m	ξ_A	N ($\times 10^3$)
St.1	St	80	9.3	-5.7 / -	-	0.02 [†]	-	-	-
St.2	St	80	9.5	-6.3 / -	-	0.02 [†]	-	-	-
St.3	St	84	9.6	-5.3 / -	-	0.02 [†]	-	-	-
St.4	St	85	9.6	-5.9 / -	-	0.02 [†]	-	-	-
A0.2m-0.4	Cyc+Pc	78	9.3	- / -5.3	-2.1 $\pm$ 1.0, 0.1	0.41	-0.4	0.2	40.0
A0.3m-0.4:1	Cyc+Pc	77	9.2	- / -5.7	-2.1 $\pm$ 1.9, 0.1	0.77	-0.4	0.3	38.2
A0.7m-0.4:1	Cyc	79	9.3	pulled	-2.1 $\pm$ 3.9, 0.1	1.54	-0.4	0.7	8.1
A0.7m0.3:1*	Cyc	85	9.6	-	1.8 $\pm$ 3.9, 0.1	1.54	0.3	0.7	28.3
A0.4m0.3	Cyc+Pc	78	9.3	- / -5.0	1.9 $\pm$ 2.3, 0.05	0.46	0.3	0.4	19.6
A0.3m-0.4:2	Cyc+Pc	-	9.1	- / -4.7	-2.1 $\pm$ 1.9, 0.1	0.77	-0.4	0.3	39.7
A0.7m-0.4:2	Cyc	-	9.1	pulled	-2.1 $\pm$ 3.9, 0.1	1.54	-0.4	0.7	1.3
A0.7m0.3:2	Cyc+Pc	81	9.4	- / -4.9	1.8 $\pm$ 3.9, 0.1	1.54	0.3	0.7	40.0
A0.2m0	Cyc+Pc	81	9.4	- / -4.9	0 $\pm$ 1.00, 0.1	0.40	0	0.2	40.0

Note: St – monotonic tensile test, Cyc – cyclic, Pc – Post-cyclic, D_R – soil density ratio, γ' – soil unit weight, $F_{T/Pc}$ – monotonic tensile peak load, F_{mean} – mean cyclic load, F_{cyc} – cyclic amplitude, f – load frequency, LR – load rate, N – cycle number.

*Test stopped after cycle No. 28,263; loading program was repeated in test A0.7m0.3:2.

[†]Maximum load rate.

Table 2. Displacement analysis.

Test ID	ξ_m	ξ_d	N	Comment for displacement
A0.7m0.3:1	0.3	0.7	160	Developed constant $w_{i,d}$ of 10^{-6} mm
A0.7m0.3:2	0.3	0.7	200	Developed constant $w_{i,d}$ of 10^{-6} mm
A0.4m0.3	0.3	0.4	1100	Developed constant $w_{i,d}$ of 10^{-6} mm
A0.2m0	0	0.2	40	Developed stable $w_{i,d}$ of -10^{-7} mm
A0.3m-0.4:2	-0.4	0.3	188	Developed stable $w_{i,d}$ of -10^{-4} mm
A0.7m-0.4:1	-0.4	0.7	-	Large displ. during all cycles
A0.7m-0.4:2	-0.4	0.7	-	Average $w_{i,d}$ of $-5 \cdot 10^{-2}$ mm

Note: D_R – soil density ratio, $w_{i,d}$ – incremental displacement, N – cycle number to reach a constant $w_{i,d}$.

List of Figures (19)

Fig. 1. Test set-up plan (Vaitkunaite 2015).

Fig. 2. Bucket model used for the testing: pressure transducer (1), valves (2), displacement transducers (3), positions for the pore pressure measurements (PP1-PP7); dimensions in mm.

Fig. 3. Sieve analysis of Aalborg University Sand No. 1. performed in 2012.

Fig. 4. Model installation with preload and elastic unload, test A0.7m0.3:1.

Fig. 5. The definition of loading parameters.

Fig. 6. Monotonic tensile loading tests and pore pressure development under the bucket model lid vs. upward displacement (tests St.1, St.2, St.3 and St.4).

Fig. 7. Full loading program for test A0.2m0: Load and pore pressure vs. upward displacement.

Fig. 8. Accumulated displacement vs. number of cycles (tests A0.3m-0.4:1, A0.3m-0.4:2, A0.7m-0.4:1, and A0.7m-0.4:2).

Fig. 9. Incremental and accumulated displacement vs. number of cycles, test A0.7m0.3:2.

Fig. 10. Incremental and accumulated displacement vs. number of cycles, test A0.7m-0.4:2.

Fig. 11. Virgin and post-cyclic monotonic shaft friction vs. upward displacement.

Fig. 12. Pore pressure development during cyclic loading (tests A0.2m-0.4, A0.3m-0.4:2 and A0.7m-0.4:2).

Fig. 13. Accumulated displacement and pore pressure vs cyclic loading in test A0.7m-0.4:2.

Fig. 14. Accumulated displacement and pore pressure vs cyclic loading in test A0.7m-0.4:1.

Fig. 15. Cyclic unloading stiffness (a), Monotonic loading stiffness (b).

Fig. 16. Unloading stiffness vs cycles for tests A0.7m-0.4:2 and A0.7m0.3:2.

Fig. 17. The normalized cycles of tests A0.7m0.3:2 (black) and A0.7m-0.4:2 (grey).

Fig. 18. Normalized unloading stiffness vs. cycle number during the first 200 cycles.

Fig. 19. Normalized accumulative displacements vs. cycle number during the first 200 cycles.

Fig. 1. Test set-up plan (Vaitkunaite 2015).
Fig. 1
148x133mm (300 x 300 DPI)

Fig. 2. Bucket model used for the testing: pressure transducer (1), valves (2), displacement transducers (3), positions for the pore pressure measurements (PP1-PP7); dimensions in mm.

Fig. 2
118x129mm (300 x 300 DPI)

Fig. 3. Sieve analysis of Aalborg University Sand No. 1. performed in 2012.
Fig. 3
53x34mm (300 x 300 DPI)

Fig. 4. Model installation with preload and elastic unload, test A0.7m0.3:1.

Fig. 4

55x35mm (300 x 300 DPI)

Fig. 5. The definition of loading parameters.
Fig. 5
38x23mm (300 x 300 DPI)

Fig. 6. Monotonic tensile loading tests and pore pressure development under the bucket model lid vs. upward displacement (tests St.1, St.2, St.3 and St.4).

Fig. 6

55x35mm (300 x 300 DPI)

Fig. 7. Full loading program for test A0.2m0: Load and pore pressure vs. upward displacement.
Fig. 7
55x35mm (300 x 300 DPI)

Fig. 8. Accumulated displacement vs. number of cycles (tests A0.3m-0.4:1, A0.3m-0.4:2, A0.7m-0.4:1, and A0.7m-0.4:2).

Fig. 8

59x41mm (300 x 300 DPI)

Fig. 9. Incremental and accumulated displacement vs. number of cycles, test A0.7m0.3:2.
Fig. 9
70x57mm (300 x 300 DPI)

Fig. 10. Incremental and accumulated displacement vs. number of cycles, test A0.7m-0.4:2.

Fig. 10

70x57mm (300 x 300 DPI)

Fig. 11. Virgin and post-cyclic monotonic shaft friction vs. upward displacement.
Fig. 11
55x35mm (300 x 300 DPI)

Fig. 12. Pore pressure development during cyclic loading (tests A0.2m-0.4, A0.3m-0.4:2 and A0.7m-0.4:2).

Fig. 12

55x35mm (300 x 300 DPI)

Fig. 13. Accumulated displacement and pore pressure vs cyclic loading in test A0.7m-0.4:2.
Fig. 13
79x74mm (300 x 300 DPI)

Fig. 14. Accumulated displacement and pore pressure vs cyclic loading in test A0.7m-0.4:1.

Fig. 14

79x74mm (300 x 300 DPI)

Fig. 15. Cyclic unloading stiffness (a), Monotonic loading stiffness (b).
Fig. 15
126x73mm (300 x 300 DPI)

Fig. 16. Unloading stiffness vs cycles for tests A0.7m-0.4:2 and A0.7m0.3:2.

Fig. 16

55x35mm (300 x 300 DPI)

Fig. 17. The normalized cycles of tests A0.7m0.3:2 (black) and A0.7m-0.4:2 (grey).
Fig. 17
55x35mm (300 x 300 DPI)

Fig. 18. Normalized unloading stiffness vs. cycle number during the first 200 cycles.

Fig. 18

55x35mm (300 x 300 DPI)

Fig. 19. Normalized accumulative displacements vs. cycle number during the first 200 cycles.

Fig. 19
59x41mm (300 x 300 DPI)