



City Research Online

City St George's, University of London

Citation: Lalicata, L. M., Ritchie, E., Stallebrass, S. E. & McNamara, A. (2024). Novel experimental technique to model impression piles in centrifuge testing. *International Journal of Physical Modelling in Geotechnics*, 24(2), pp. 78-91. doi: 10.1680/jphmg.22.00065

This is the accepted version of the paper.

This version of the publication may differ from the final published version. To cite this item please consult the publisher's version.

Permanent repository link: <https://openaccess.city.ac.uk/id/eprint/32275/>

Link to published version: <https://doi.org/10.1680/jphmg.22.00065>

Copyright and Reuse: Copyright and Moral Rights remain with the author(s) and/or copyright holders. Copies of full items can be used for personal research or study, educational, or not-for-profit purposes without prior permission or charge, unless otherwise indicated, provided that the authors, title and full bibliographic details are credited, a hyperlink and/or URL is given for the original metadata page and the content is not changed in any way. For full details of reuse please refer to [City Research Online policy](#).



International Journal of Physical Modelling in Geotechnics

A novel experimental technique to model impression piles in centrifuge testing

PHMG-2022-065-R2 | Paper

Submitted on: 19-07-23

Submitted by: Leonardo Maria Lalicata, Eric Ritchie, Sarah Stallebrass, Andrew McNamara

Keywords: CENTRIFUGE MODELLING, CLAYS, IMPRESSION PILE, WORKING LOAD



1 Introduction

2 Deep foundations in centrifuge testing are typically modelled as solid metallic rods or tubes
3 (Rollins *et al.*, 2005; Rosquoët *et al.*, 2007; Ng *et al.*, 2014; Alnuaim *et al.*, 2015; Lalicata *et*
4 *al.*, 2019, 2020b; de Sanctis *et al.*, 2021; Li *et al.*, 2022, Shabanpour and Ghazavi, 2022),
5 the latter having open or closed ends according to testing needs. These are extensively
6 adopted owing to their cheap price, extensive availability on the market and ease of
7 machining and assembling on often complex centrifuge packages. Metal model piles can
8 reproduce the behaviour of prototype steel piles, such as monopiles for wind turbines, and
9 they are also suitable for investigating installation effects (Truong *et al.*, 2019; Maatouk *et*
10 *al.*, 2022; Li *et al.*, 2023). Recently, the use of rapid curing concrete piles in centrifuge tests
11 is receiving increasingly interest, especially to capture the true failure mechanisms (Knappett
12 *et al.*, 2011; Al-Dafae and Knappett, 2014; Goode and McCartney, 2015; Iovino *et al.*, 2023;
13 Ouzzine *et al.*, 2023). Although rapid, these concrete piles need approximately one week of
14 curing time in the mould before being installed in the soil models.

15 Therefore, such concrete model piles as well as metal model piles cannot be used, at least
16 in clay samples, to model piles with an uneven cross-section along their length such as
17 threaded/ribbed piles and impression piles (Hard & Carvalho, 2018; Gorasia and McNamara,
18 2016; Lalicata *et al.*, 2022, 2023). In those types of piles, the shaft capacity is enhanced by
19 modifying the profile of the shaft. Ribbed piles are created with a tool that cuts concentric
20 rings in the shaft, while in the impression pile the enhancement is provided by creating small
21 discrete impressions that lead to a nodular pile surface. On site, once the profiling is
22 concluded, the concrete is cast as in traditional bored piles. To reproduce such piles in
23 physical model testing, polyurethane resin can be used (Chandler and Martins, 1982;
24 McNamara *et al.*, 2009). Fast cast resin model piles have been extensively used at City,
25 University of London in the past 20 years for a variety of problems, including investigations
26 of settlement reduction associated with deep excavations (McNamara 2001, McNamara *et*

27 *al.*, 2009), and the investigation of innovative pile foundations such as the aforementioned
28 ribbed and impression piles (Taylor *et al.*, 2013; Gorasia and McNamara, 2016; Lalicata *et*
29 *al.*,2023).

30 As with all modelling techniques, the use of polyurethane resin has drawbacks. The curing
31 process seems to affect the soil-pile interface properties at small strain levels thus obscuring
32 the effect of the enhanced shaft geometry of, for instance, ribbed piles. Gorasia and
33 McNamara (2016) observed that ribbed piles with a ribbed length of 135mm (over 180mm
34 total pile length) needed a large head displacement (from 5 to 10% of the diameter) to
35 mobilise a significant increase in load compared to the plain pile, Figure 1. Moreover, for
36 resin plain model piles, Lalicata *et al.* (2022) measured quite high values of the adhesion
37 factor α , ~ 0.73 , thus potentially compromising a direct comparison with field performances.

38 To address the limitations linked to the curing effects of resin, an alternative method to
39 create model piles with profiled shafts has been developed. This is a model pile with a
40 special sliding mechanism that allows the model pile to be inserted into the bore with the
41 nodules retracted inside the pile and that, once the model pile is installed, subsequently
42 pushes the nodules against the soil creating the impressions. This has been achieved using
43 a model pile made of 3D printed rigid plastic components with a specially designed reverse
44 mandrel mechanism. A number of small-scale centrifuge tests are reported to evaluate the
45 performance of the plastic model piles and these are supported by interface direct shear
46 tests. The behaviour of the 3D printed plastic model piles has been assessed against cast
47 resin model piles by looking at the whole load-settlement curve, from small strain to ultimate
48 capacity.

49 1. The impression pile

50 A sketch of the impression pile is illustrated in Figure 2a. In the simplest configuration, four
51 nodules are impressed at a given cross section, spaced at 90° around the axis of the pile,
52 and nodules are aligned in the vertical direction. Following Lalicata *et al.* (2023) the portion
53 of the pile shaft where the impressions are created is called the active length L_a ; s is the
54 centre-to-centre vertical distance between two levels of nodules, n is the number of nodules
55 at a given cross section and z is the position of the centre of the impressed zone relative to
56 the soil surface. The geometry of the nodules is defined by the protruding length b , the width
57 in the horizontal plane l and the vertical height h . In the centrifuge tests the nodules had a
58 square cross section ($l = h$).

59 Lalicata *et al.* (2022, 2023) explored in greater detail the ultimate capacity of the impression
60 pile by centrifuge testing on cast resin model piles and the topic will not be discussed here.
61 For the purpose of this paper, it is recalled that the ultimate capacity of the impression pile
62 increases with the active length and the number of nodules at a given horizon. Lalicata *et al.*
63 (2023) demonstrated that the spacing s has little influence on the ultimate capacity of the
64 impression piles when below a threshold value (s_{cr}), because the failure surface around the
65 nodules bridges vertically, creating a vertical block of soil connecting adjacent nodules with
66 the failure surface on the outside of this block, Figure 2b. In this configuration, the
67 performance of the impression pile is maximised, i.e. there is a maximum increase in
68 capacity compared to the straight shafted pile.

69 2. Experimental technique

70 Both 3D printed and resin piles are cored, profiled and installed in the centrifuge model at
71 $1g$. For results to be repeatable it is key that the piles are vertical, of constant length into the
72 clay and the profiling of the impressions is precisely located and well defined. The piles were
73 16mm in diameter; embedded 180mm into the clay model.

74 The pile bores were cut using a hypodermic thin-walled tube with an external diameter of
75 16mm. The tube was mounted onto a brass handle to allow for easy cutting. The verticality
76 and position of the piles was achieved using a system of guides (Lalicata *et al.*, 2020b). To
77 minimise soil disturbance during boring, several precautions were taken: the internal part of
78 the tube was sprayed with a lubricant oil and the edge of the tube was sharpened. In
79 addition, the excavation of the bore was always undertaken in three steps of equal length.

80 2.1. Resin Model Pile

81 After profiling, the model piles were cast using a 'fast cast' polyurethane resin, Sika Biresin
82 G27 (McNamara, 2001; McNamara and Taylor, 2002). The pot life of the resin was
83 approximately 3 minutes. Aluminium powder was used as filler in an equal mass ratio with
84 the two components of the resin to ensure that the model pile was not buoyant. The mixture
85 was designed to have a good fluidity to fill the small impressions (1.5 x 1.5 x 3 mm). An
86 aluminium collar placed on the top of the model pile accommodated the loading and
87 measurement equipment.

88 The model impression piles were created from the plain pile bore. A custom designed tool
89 was inserted into the guide and used to profile the pile bore, Figure 3. Figure 4a shows some
90 of the exhumed model piles and demonstrates the success of this methodology. Several
91 uniaxial compression tests were undertaken to measure the mechanical properties of the
92 resin when set. The resin was found to have a Young's Modulus equal to 1.1 GPa and a
93 yield stress of 35 MPa.

94 2.2. 3D Printed Model Pile

95 Figure 4b shows the assembled model piles that were made from a rigid 3D printed plastic,
96 for simplicity referred to as plastic model piles in the following. The model pile is made from
97 a central core with eight guiding rails along which eight segmental plates can slide during the
98 installation process, Figure 5. Four plates have a curved but smooth external surface and

99 the other four house the nodules. During installation the plates are supported by a circular
100 base 10mm thick with 20mm long guides. On the central core, the rails where the plates with
101 nodules slide are slightly tapered to ensure that the nodules are fully retracted during
102 insertion of the pile in the smooth sided bore. Once the pile is fully inserted into the bore, the
103 core is manually pushed down, and the tapered profile of the rails makes the nodules
104 expand into the soil, Figure 5b and Figure 6. The core is specifically machined to allow for
105 easy handling and to accommodate the loading and measurement equipment. Several
106 uniaxial compression tests were undertaken to measure the mechanical properties of the
107 assembled plastic model pile. The plastic model pile was found to have a Young's Modulus
108 equal to 0.75 GPa which is lower than the Young's Modulus of the material (2.2 GPa),
109 declared by the manufacturer, the yield stress is equal to 55 MPa.

110 2.3. Soil

111 The Speswhite Kaolin clay used in the tests was prepared from slurry with an initial water
112 content of approximately 120%; which is twice the liquid limit. The slurry was created by
113 mixing dry powder and distilled water in an industrial ribbon blade mixer.

114 The slurry was carefully placed into the model container and manually agitated to expel the
115 main air bubbles. The inside faces of the model container had been previously coated with
116 water pump grease to minimise friction at the boundaries (Philipps, 1995). Beneath the slurry
117 there was a filter paper and a 3 mm porous plastic sheet, with an aluminium drainage plate
118 at the base. On top of the slurry, a second filter paper and porous plastic sheet were placed
119 and drainage was allowed through holes in a loading platen. Consolidation pressure was
120 applied in increments by means of a hydraulic press over a period of 9 days including 1 day
121 of swelling. Following Taylor *et al.* (2013) and Gorasia and McNamara (2016) the maximum
122 pressure was maintained for three days at which stage all measurable settlement had
123 ceased. The samples were compressed to a vertical stress of 500kPa that was then reduced
124 to 250kPa, producing a firm, but still workable, clay sample (McNamara *et al.*, 2009; Divall

125 and Goodey, 2015). On the day of the test, the sample was removed from the press and
126 prepared for the centrifuge test, as described below.

127 3. Methodology

128 The performance under axial loading of the novel plastic model impression piles is assessed
129 against that of cast resin model impression piles via centrifuge testing undertaken at 50g
130 using a homogeneous overconsolidated clay deposit. In each experiment, the model
131 impression piles were tested alongside a plain, straight shafted model pile of the same type
132 (i.e. plastic or cast resin) to provide a baseline response for comparison purposes. The
133 friction at the material-soil interface is further investigated by direct shear box testing for both
134 the resin and plastic.

135 3.1. Layout of the Centrifuge tests

136 The Geotechnical Engineering Research Group at City, University of London, makes use of
137 an Acutronic 661 beam centrifuge of 1.8m radius from centre to the swinging platform,
138 described in detail by Schofield and Taylor (1988). The package containing the model was
139 installed on the centrifuge once the piles had been bored, installed and the loading
140 apparatus assembled on the plane strain strongbox. The whole process; removing the
141 sample from the press to spin up, took approximately 2 hours. Once placed on the centrifuge
142 swing, the model was accelerated to 50g.

143 During flight, the water table was maintained at a depth of approximately 10 mm below
144 ground level by means of a constant head standpipe connected to the bottom of the model.
145 The top surface was sealed with a sprayed synthetic rubber coating to prevent clay drying
146 during the test; once dried this was only ~400 μm thick and it is known to not influence soil
147 settlements (Le and Taylor, 2018). The model was left at 50g for about 48 hours until
148 conditions of pore pressure equilibrium were established as identified by pore pressure

149 transducers embedded in the soil and one LVDT placed at the ground surface. The piles
150 were then loaded until failure at a displacement rate of 1mm/min.

151 For each test, the soil sample provided up to four testing sites within the rectangular
152 strongbox, Figure 7. The piles were positioned on the centreline of the model, 100 mm from
153 the sides, which was far enough to minimise boundary effects (Phillips, 1995). The piles
154 were spaced 110 mm or 165mm apart for the four or three pile configurations respectively
155 and loaded simultaneously by means of a very stiff beam connected to a lead screw and
156 motor. The apparatus was devised so that it was possible to obtain independent load and
157 settlement data for each pile. Three load cells, each of 5kN capacity, sandwiched between
158 two aluminium plates, measured the load on each pile ensuring that errors due to bending in
159 the load cell are eliminated, Figure 7. A ball bearing lying on top of a concave plate was
160 used to apply the centred and vertical load. A 2 mm thick aluminium plate, 175 mm long and
161 16 mm wide, was a tight fit to the pile collar and two LVDTs measure the plate displacement
162 at its extremes. The head displacement of the pile was given by the mean value of the LVDT
163 measurements. A 1 mm gap between the plate and the concave dish guaranteed that load
164 and displacement were independent readings. Additional details of the experimental
165 arrangement are given in Lalicata *et al.* (2020b).

166 *Soil characterisation*

167 Following testing on the centrifuge, the undrained shear strength was estimated from hand
168 vane tests and water content samples as in Lalicata *et al.* (2023). The soil strength
169 measurements for the tests given in Figure 8a are for the values derived from water
170 contents, via eq (1), where T12, T13 etc are the test names.

$$s_u = \frac{1}{2} M e^{\frac{(f-v)}{\lambda}} \quad (1)$$

171 where v is the specific volume the values of the parameters and their meaning are listed in
172 Table 1. Note that, under fully saturated conditions, water content w and specific volume v
173 are uniquely correlated: $v = 1 + wG_s$.

174 The measurements are largely consistent across all tests and in good agreement with the
175 previous experiments (Lalicata *et al.*, 2023). As might be expected, the undrained strength
176 increased slightly with depth. The data are generally inside the 10% error band with respect
177 to the best fit line:

$$s_{u,w} = 41.2 + 0.044z \quad (2)$$

178 where $s_{u,w}$ is expressed in kPa and z , in mm, is the soil depth at the model scale. The profile
179 of overconsolidation ratio (OCR), with depth for the tests is reported in Figure 8b.

180 3.2. Direct shear tests

181 Tests are performed on square samples (100x100mm) under a constant vertical stress of
182 50kPa, and a displacement rate of 1mm/min in a sealed box with no water, to avoid changes
183 in the water content and to ensure “undrained conditions”. Although this is not the standard
184 procedure for direct shear tests in clay, this methodology provides a reasonable
185 approximation to the loading conditions in the centrifuge tests. The results should, however,
186 be seen only as qualitative and treated with caution. The undrained shear strength is
187 calculated from the water content measurements taken before and after each test.

188 The soil samples are cut from a lump of clay one dimensionally consolidated in a CBR mould
189 to 500kPa and then unloaded to 250kPa, to give a similar loading history to that employed in
190 the centrifuge tests. Except in the tests where resin was cast against the clay, soil samples
191 were prepared and tested within 20 minutes, minimising any change in water content during
192 testing and no change in water content was observed.

193 Four types of tests have been investigated:

- 194 • clay-clay test: standard direct shear test for reference.
- 195 • resin-clay test: the clay occupies only the lower half of the shear box; the top of the
196 clay was carefully trimmed to the level of the shear plane. Once the shear box was
197 assembled, the resin was poured against the clay surface to form a uniform layer
198 4mm thick. The resin was left curing for one hour with the top exposed to the air in a
199 temperature-controlled room before the test began.
- 200 • pre-cast resin-clay test: the clay occupies only the lower half of the shear box and a
201 plate of pre-cast resin of the same size as the box was placed on top of the clay. No
202 curing time is required, and the test can readily commence once the model is
203 assembled.
- 204 • 3D printed plastic-clay test: as for the pre-cast resin test, but a 3D printed plastic
205 plate is used.

206 4. Results

207 The main features of the centrifuge tests analysed in this paper are listed in Table 2, results
208 refer to the model scale. Some of the results (i.e. the ultimate capacity) from tests T12, T13
209 and T14 have been already presented in Lalicata *et al.* (2023), while the others belong to a
210 new set of centrifuge tests. From the whole set of data, only the pile load test results with
211 reliable and direct measurements of settlement are used. All the impression piles have four
212 nodules per cross section ($n = 4$) and values of vertical spacing lower than critical ($s < s_{cr}$).

213 4.1. Load settlement curves

214 The load-settlement curves for all the seventeen pile load tests are presented in Figure 9a
215 and b for the resin and plastic piles respectively. Settlement is represented in normalised
216 form.

217 Plastic piles, regardless of whether they are impression or plain piles, show a lower ultimate
218 capacity indicating a lower value of the adhesion factor α , (see Table 2). Note that, α can be
219 directly calculated from the plain pile tests only. For the impression pile, α is 1 along the
220 vertical soil ribs connecting the nodules whilst elsewhere it is assumed equal to the value
221 calculated from the plain pile in the same test, see Figure 2b. The load settlement response
222 of the plain resin model piles is very different from that of the plastic model piles. Plain resin
223 model piles show a marked peak at a settlement of 7-10% of the diameter, after which the
224 load reduces to a stable ultimate value. The peak magnitude is highly variable being 1.15-
225 1.52 times the value of the ultimate load. Whereas, the plastic model piles show a ductile
226 behaviour, with a load that slowly increases after a significant reduction in gradient at
227 $w \sim 5\%d$. These differences are related to the different interface behaviour of the two
228 materials as manifest in the shear box results described below.

229 The impression model piles made from resin, exhibit a less marked peak than the
230 corresponding plain model piles because a proportion of the failure surface is now in the soil
231 and not at the soil pile interface. This proportion increases with an increase in active length.

232 The plastic model piles seem to have a better repeatability compared to the resin model
233 piles. This is expected since the plastic is an industrial material with precise properties while
234 the resin mixture is handmade for each model pile thus small differences in the preparation
235 may influence the results, especially at low settlement levels. However, it should be noted
236 that the larger number of pile tests available for the resin model piles and the natural
237 experimental variability of the soil models, may be partly responsible for those differences.

238 The measured values of ultimate capacity, and the corresponding increase in capacity for
239 the impression model piles are listed in Table 3 and Table 4 for resin and plastic model piles
240 respectively. For the resin model piles, the capacity Q_u is defined as the asymptotic value of
241 the curve after the peak where the shaft and the base resistance may be assumed fully

242 mobilised. The same method is not directly applicable to the plastic model piles as they
243 show a hardening behaviour, see Figure 9b. For those piles, Q_u has been defined with
244 different methods widely used in the literature: the settlement criterion (Kodsy *et al.*, 2022),
245 the double tangent and the hyperbolic method defined by Chin (Chin, 1970).

246 The settlement criterion assumes the pile capacity is attained at a given value of
247 displacement, typically equal to $10\%d$, regardless of the actual shape of the load-settlement
248 curve. Here, $w=25\%d$ is also used because, as suggested by Mandolini *et al.* (2005), bored
249 piles may need larger settlement values to fully mobilise their capacity. The double tangent
250 method defines the ultimate capacity as the intersection between the tangent at $w/d=0$ (initial
251 stiffness) and the tangent to the load-settlement curve at large displacements. It is worth
252 noting that with this method the ultimate capacity is attained at very small settlements ($w\sim 1$
253 $- 1.2\%d$). The Chin method interpolates the experimental curve with a hyperbolic function,
254 whose parameters are the initial stiffness and the capacity Q_u . As suggested by many
255 authors, the capacity is set equal to 90% of the capacity obtained from the best fit
256 parameters derived from the Chin interpolation.

257 Irrespective of the method used to define the ultimate capacity, the plastic model piles show,
258 in general, a higher increase in ultimate capacity ($\sim 40\%$) compared to the resin model piles
259 ($\sim 20\%$) with similar L_a values (0.75 and 0.78 respectively). This result is consistent with the
260 smaller values of adhesion factor measured for the plastic model piles, as decreasing the
261 value of α , increases the importance of the soil-soil shearing created by the nodules.

262 The increase in capacity of the plastic impression model piles reduces for increasing w/d
263 values because the progressive mobilisation of the base resistance acts at both the
264 numerator and the denominator. The increase in ultimate capacity is highest when
265 calculated using the double tangent method, but the scatter in these values is also highest,
266 varying from 46 to 68%. Consistent increases in capacity of around 37% are obtained using

267 the Chin interpolation or a settlement criterion of $w=25\%d$ and they appear to provide the
268 best representation of the full mobilisation of the capacity of the model pile.

269 4.2. Small strain behaviour

270 To examine the performance of the model pile at small displacements, secant stiffness is
271 plotted against normalised settlement in a semi-log plane in Figure 10.

272 The range of interest ($w/d=0.3\%$ - 2%) is highlighted in the figure. Measurements of
273 displacement below $w/d=0.3\%$, $w=0.048\text{mm}$, cannot be determined accurately, as the sum
274 of the accuracy of the micrometre used for the calibration, $\pm 0.01\text{mm}$, and that of the LVDTs
275 itself, $\pm 0.02\text{mm}$, gives an overall accuracy of $\pm 0.03\text{mm}$ which is 60% of the displacement at
276 $w/d=0.3\%$. Above $w/d=2\%$ the piles are at 45-88% of ultimate capacity and consequently not
277 all the piles could be considered to be under working load conditions.

278 Data from tests on resin model piles, Figure 10a, are highly scattered at $w/d<1\%$ making it
279 difficult to discern a clear difference between the response of the plain or impression model
280 piles. It appears that the high interface strength of the resin, before the peak capacity is
281 reached, obscures the effect of the nodules on working load behaviour.

282 Conversely, the effect of the impressions on the working load behaviour of the plastic model
283 piles is clear, Figure 10b. In this case, the impression model piles show a significant
284 increase in secant stiffness at every load level, for instance, equal to $\sim 90\%$ and $\sim 73\%$ at
285 $w/d=0.3\%$ and 0.5% , respectively. For increased clarity, the ratio between the load taken by
286 the impression pile and the load taken by the plain pile at the corresponding displacement in
287 each test is shown in Figure 11. The coloured areas represent the experimental range of the
288 data. Plastic model piles show a smooth decrease in load ratio with settlement from 1.9 at
289 $w/d=0.3\%$ to 1.4 at capacity ($w/d=25\%$ in the plot). Whereas resin model piles show a large
290 variability at small displacements, the load ratio ranging from 0.6 to 1.2, i.e. lower than unity

291 in some cases. After $w/d=10\%$, there is an increase in the load ratio because the plain piles
292 have reached ultimate capacity and the increase in capacity ranges between 1.2 and 1.5 as
293 already discussed in the previous section.

294 The average values of the secant stiffness at selected settlement levels are listed in Table 5.
295 To allow a direct comparison between the resin and plastic impression model piles, only
296 similar piles are included in this table (i.e. all the impression model piles with $L_a/L \sim 0.75$, see
297 Table 2). The lower scatter in the data measured for the plastic model piles confirms the
298 greater consistency of the plastic-soil interface and the lower variability of the 3D printed
299 plastic compared to the cast in place resin. From both Table 5 and Figure 10 it is clear that
300 the resin model piles have a stiffer response compared to plastic model piles at any
301 displacement level.

302 The last two columns in Table 5 list the secant stiffness ratio (equivalent to the load ratio) for
303 the resin and plastic model piles for the plain and the impression model piles respectively.
304 This ratio increases for increasing values of w/d , and it is larger for the plain pile case, for
305 any w/d value. The plain pile resin to plastic ratio is representative of the material properties
306 only. It is interesting to note that, even at very small displacement values ($w/d=0.5\%$), this
307 ratio is larger than the stiffness ratio of the two materials
308 ($E_{resin}/E_{plastic}=1.1\text{GPa}/0.75\text{GPa}=1.46$). The rationale can be found in the different shape of
309 the load-settlement curves (see Figure 9) that is linked to the shear stress mobilisation along
310 the shaft. For the impression model piles the ratio is lower (it ranges from 1.14 to 1.85)
311 because the presence of the nodules creates a slip plane away from the shaft and,
312 consequently, the differences in the interface behaviour are masked even at small strains.

313 4.3. Direct shear tests

314 The results of the direct shear tests are depicted in Figure 12, where the shear stress has
315 been normalised by the undrained shear strength, evaluated from water content

316 measurements via eq. (1). The tests using clay only with no interface are fairly consistent,
317 with a peak that is approximately equal to $0.9s_u$. The results from tests where there was an
318 interface with precast resin and plastic show τ/s_u values that become constant after a small
319 peak at displacements of between 0.5 and 1.5mm, indicating that failure has occurred. The
320 ultimate τ/s_u values are ~ 0.50 and ~ 0.33 for the precast resin and the plastic respectively,
321 Table 6.

322 In contrast, the cast resin test results are significantly steeper at the beginning than the other
323 curves and reach a large peak stress (of 2 and 3.25 times s_u); in the softening that follows,
324 τ/s_u reduces to ~ 0.57 that is close to the value measured in the precast resin tests.
325 Although the two tests carried out on the cast in place resin look very different, it should be
326 noted that they share the same value of τ/s_u at large displacements.

327 It is thought that the reason for this behaviour is the curing process of the resin against the
328 clay. The curing phase is an exothermic chemical process (McNamara & Taylor, 2009), thus
329 a thin clay layer around the pile is likely to experience some heating and possibly some
330 chemical bonding with the resin, affecting the properties of the soil-resin interface. In the
331 direct shear tests, the peak is significantly larger than would be consistent with the pile
332 response observed in the centrifuge tests (see Figure 9a) because, in the latter, pore
333 pressures are allowed to come into equilibrium at 50g for 48 hours and this will mitigate most
334 of the effects the exothermic reaction has on water contents near the pile. Conversely, in the
335 shear box tests, the samples were sheared just after the end of the curing process.

336 5. Discussion

337 Previous research on ribbed and impression model piles (Figure 1 and Figure 9a) seemed to
338 suggest that those model piles needed to experience a relatively large head settlement
339 before being able to mobilise the enhanced shaft resistance provided by the profiling of the

340 model pile shaft. However, the results presented have indicated that this conclusion was a
341 function of the material used to cast the model pile. Changing the material, but not the
342 geometry of the impression model piles, a different behaviour has been observed (Figure
343 11). These new data suggest that in the tests on cast *in situ* resin model piles, the effect of
344 the nodules on the working load behaviour of the impression resin model pile is masked by
345 the properties of the soil-resin interface. All the resin plain model piles show a peak and a
346 softening. Often, the peak load achieved by the plain model piles is close to the capacity of
347 the impression model piles in the same test. A similar strain-softening behaviour observed in
348 the plain resin model piles has been found in model tests on epoxy resin (Araldite) piles
349 (Chandler and Martins, 1982) but is seldom observed in full scale tests on traditional bored
350 concrete piles, probably because other technological factors play a major role in this case
351 and the curing process will be different between the resin and the concrete.

352 However, with a printed plastic model pile, the soil-plastic interface is stable and relatively
353 smooth. This means that the maximum allowable shear stress on the interface will be always
354 lower than that of the soil and consequently, when the failure surface is moved away from
355 the pile due to the presence of the nodules the shaft resistance of the model pile will
356 necessarily increase, resulting in a soil-pile secant stiffness (or load) which is always higher
357 for the impression model pile at a given displacement.

358 On site, there is limited evidence of the concrete curing effects on the skin friction of piles
359 (Mascarucci & Mandolini, 2013) and a detailed study of the curing effect of the resin is
360 outside the scope of this study. However, the direct shear tests carried out confirm the
361 centrifuge test findings. The peak-softening behaviour of the resin only occurs when the
362 resin is cast against the soil affecting the soil close to the shaft interface. When the resin is
363 not cast *in situ* the peak resistance reduces to values close to the ultimate resistance in line
364 with the observations of other authors (Yavari *et al.*, 2016; Parchment & Shepley, 2018). The
365 ultimate resistance of the “cast in place” resin is similar to that of the “precast” resin,

366 suggesting that particle rearrangement at the interface upon shearing finally overcomes the
367 curing effects.

368 Conclusions

369 A novel experimental technique to model impression piles has been presented in this paper.
370 The novel model piles (both impression and plain) are made of 3D printed rigid plastic
371 components assembled during the installation process.

372 The behaviour of these plastic model piles has been investigated by means of geotechnical
373 centrifuge testing on overconsolidated clay samples. In each test, the impression model piles
374 were tested together with one plain model pile against which the impression model piles
375 could be assessed. Installation effects are neglected as the model piles were installed at 1g
376 rather than in the high-g environment. This is clearly not representative of the prototype
377 scale installation, but may be considered as an ideal wished-in-place installation of the pile.
378 Results from five model pile load tests have been compared with resin model piles of similar
379 geometry and tested in similar conditions. The tests on the plastic model piles showed an
380 excellent repeatability across the two centrifuge tests on both the two plain model piles and
381 the three impression model piles.

382 The method of installation of the plastic model piles reduced experimental variability and
383 eliminated the consequences of the curing process on the cast resin model piles. This
384 helped to highlight the beneficial effects of the impressions on the whole load-settlement
385 curve, from small strain to capacity.

386 Plastic plain model piles showed a monotonically increasing load settlement curve while
387 resin plain model piles had a marked strain softening behaviour. Consequently, the
388 impression plastic model piles outperformed the plain plastic model piles at any pile
389 displacement. These results suggest that the displacement needed to mobilise the enhanced

390 shaft capacity of the impression piles is small and of the same order of magnitude of that
391 needed to mobilise the skin friction in ordinary bored piles, making the impression pile
392 solution attractive. Such behaviour was not observed in the resin model piles due to the high
393 shear resistance of the interface induced by the curing process of the resin. In those model
394 piles, the improved performance of the impression model piles was clear only at ultimate
395 capacity. Qualitatively, the significance of the type of pile-soil interface has been confirmed
396 by rapid direct shear interface tests, which demonstrated that the curing process of the resin
397 significantly affected the peak resistance of the interface. At large displacements, the particle
398 rearrangement at the interface during shearing finally overcomes the curing effects and the
399 ultimate resistance of the cast *in situ* resin is similar to the precast resin.

400 To date, the set of data available is not sufficient to quantify the impact of different
401 impression geometries on the working load behaviour of the impression piles, but further
402 tests are planned to enlarge the experimental dataset.

403 Acknowledgements

404 This research project would not have been developed without funding from Innovate UK and
405 the vision of Keltbray Piling. They are gratefully acknowledged. The authors also want to
406 thank Mr Keith Pamment from City, University of London for his support in the development
407 of the 3D printed impression pile.

408 *CRedit authorship contribution statement*

409 **Leonardo Maria Lalicata:** Conceptualisation, Methodology, Validation, Formal analysis,
410 Investigation, Data curation, Writing – original draft, Visualisation, Project administration.
411 **Eric Ritchie:** Investigation. **Sarah Elizabeth Stallebrass:** Conceptualisation, Methodology,
412 Writing – Review & Editing, Supervision, Project administration, Funding acquisition.
413 **Andrew McNamara:** Conceptualisation, Methodology, Writing – Review & Editing,
414 Supervision, Project administration, Funding acquisition.

415 References

- 416 Al-Defae, A. H., & Knappett, J. A. (2014). Centrifuge modeling of the seismic performance of
417 pile-reinforced slopes. *Journal of Geotechnical and Geoenvironmental Engineering*,
418 140(6), 04014014.
- 419 Alnuaim, A. M., El Naggar, M. H., & El Naggar, H. (2015). Performance of micropiled raft in
420 clay subjected to vertical concentrated load: Centrifuge modelling. *Canadian*
421 *Geotechnical Journal*, 52(12), 2017–2029. <https://doi.org/10.1139/cgj-2014-0448>
- 422 Chandler, R. J., & Martins, J. P. (1982). An experimental study of skin friction around piles in
423 clay. *Géotechnique*, 32(2), 119–132. <https://doi.org/10.1680/geot.1982.32.2.119>
- 424 Chin, F.V. (1970). Estimation of the ultimate load of piles not carried to failure. *Proceedings*
425 *of the 2nd Southeast Asian Conference on Soil Engineering*, Singapore, pp. 81–90.
- 426 de Sanctis, L., Di Laora, R., Garala, T. K., Madabhushi, S. P. G., Viggiani, G. M. B., &
427 Fargnoli, P. (2021). Centrifuge modelling of the behaviour of pile groups under vertical
428 eccentric load. *Soils and Foundations*, 61(2), 465–479.
429 <https://doi.org/10.1016/j.sandf.2021.01.006>
- 430 Divall, S., & Goodey, R. J. (2015). Twin-tunnelling-induced ground movements in clay.
431 *Proceedings of the Institution of Civil Engineers - Geotechnical Engineering*, 168(3),
432 247–256. <https://doi.org/10.1680/genq.14.00054>
- 433 Goode, J. C. III, & McCartney, J. S. (2015). Centrifuge modeling of end-restraint effects in
434 energy foundations. *J. Geotech. Geoenviron. Eng.*, 141(8),
435 [https://doi.org/10.1061/\(ASCE\)GT.1943-5606.0001333](https://doi.org/10.1061/(ASCE)GT.1943-5606.0001333)
- 436 Gorasia, R. J., & McNamara, A. (2016). High-capacity ribbed pile foundations. *Proceedings*
437 *of the Institution of Civil Engineers - Geotechnical Engineering*, 169(3), 264–275.
438 <https://doi.org/10.1680/jgeen.15.00073>
- 439 Hard, D. A. & Carvalho, A. T. (2018). Threaded rotary bored piles at Paddington new yard.
440 *Proceedings of the 2018 DFI-EFFC international conference on deep foundations and*
441 *ground improvement*, Rome, Italy.
- 442 Iovino, M., Iodice, C., Alagha, A., & Viggiani, G. M. (2023). Centrifuge Experiments Dealing
443 with Monotonic and Cyclic Loads on Pile Foundations in Sand. In: Ferrari, A., Rosone,
444 M., Ziccarelli, M., Gottardi, G. (eds) *Geotechnical Engineering in the Digital and*
445 *Technological Innovation Era. CNRIG 2023*. Springer Series in Geomechanics and
446 Geoenvironment. Springer, Cham. https://doi.org/10.1007/978-3-031-34761-0_81
- 447 Knappett, J. A., Reid, C., Kinmond, S., & O'Reilly, K. (2011). Small-scale modeling of
448 reinforced concrete structural elements for use in a geotechnical centrifuge. *Journal of*
449 *Structural Engineering*, 137(11), 1263-1271.
- 450 Kodsý, A., Iskander, M. G., & Sampurnanand Pandey, A. (2022). Universal Criterion for
451 Interpreting Capacity from Load Tests on Piles. *Transportation Research Record: Journal*
452 *of the Transportation Research Board*, 036119812210846.
453 <https://doi.org/10.1177/03611981221084686>
- 454 Lalicata, L. M., Desideri, A., Casini, F., & Thorel, L. (2019). Experimental observation on
455 laterally loaded pile in unsaturated silty soil. *Canadian Geotechnical Journal*, 56(11),
456 1545–1556. <https://doi.org/10.1139/cgj-2018-0322>
- 457 Lalicata, L. M., McNamara, A. M. & Stallebrass, S. E. (2020a). Experimental technique for
458 creating enhanced capacity piles in a centrifuge environment. *In Proceedings of the*

- 459 *European conference on physical modelling in geotechnics ECPMG2020*, pp. 49–55.
460 Luleå, Sweden: Luleå University of Technology.
- 461 Lalicata, L. M., Rotisciani, G. M., Desideri, A., Casini, F., & Thorel, L. (2020b). Physical
462 modelling of piles under lateral loading in unsaturated soils. *In 4th European Conference*
463 *on Unsaturated Soils, E-UNSAT 2020*. Cardoso R., Jommi C., Romero E. (Eds). E3S
464 Web of Conferences 195(01021). <https://doi.org/10.1051/e3sconf/202019501021>
- 465 Lalicata, L. M., Stallebrass, S. E., & McNamara, A. (2023). An experimental study into the
466 ultimate capacity of an 'impression' pile in clay. *Géotechnique*, 73(5), 455-466.
467 <https://doi.org/10.1680/jgeot.21.00168>
- 468 Lalicata, L. M., Stallebrass, S. E., McNamara, A., & Panchal, J. P. (2022). Design method for
469 the 'impression pile'. *Proceedings of the Institution of Civil Engineers - Geotechnical*
470 *Engineering*, 175(1), 75–85. <https://doi.org/10.1680/jgeen.21.00033>
- 471 Le, B. T., & Taylor, R. N. (2018). Response of clay soil to three-dimensional tunnelling
472 simulation in centrifuge models. *Soils and Foundations*, 58(4), 808–818.
473 <https://doi.org/10.1016/j.sandf.2018.03.008>
- 474 Li, Q., Askarinejad, A., & Gavin, K. (2022). Lateral response of rigid monopiles subjected to
475 cyclic loading: Centrifuge modelling. *Proceedings of the Institution of Civil Engineers -*
476 *Geotechnical Engineering*, 175(4), 426-438. <https://doi.org/10.1680/jgeen.20.00088>
- 477 Li, Z.-S., Blanc, M., & Thorel, L. (2023). Effects of embedding depth and load eccentricity on
478 lateral response of offshore monopiles in dense sand: A centrifuge study. *Géotechnique*,
479 73(9), 811-825. <https://doi.org/10.1680/jgeot.21.00200>
- 480 Maatouk, S., Blanc, M., & Thorel, L. (2022). Impact driving of monopiles in centrifuge: Effect
481 on the lateral response in sand. *International Journal of Physical Modelling in*
482 *Geotechnics*, 22(6), 318-331. <https://doi.org/10.1680/jphmq.21.00035>
- 483 Mandolini, A., Russo, G. & Viggiani, C. (2005). Piled foundations: experimental
484 investigations, analysis and design, State-of-the-art report. *In Proceedings of the 16th*
485 *international conference on soil mechanics and geotechnical engineering: geotechnology*
486 *in harmony with the global environment*, vol. 1, pp. 177–213. Rotterdam, the
487 Netherlands: Millpress.
- 488 Mascarucci, Y., Mandolini, A., & Miliziano, S. (2013). Effects of residual stresses on shaft
489 friction of bored cast in situ piles in sand. *Journal of Geo-Engineering Sciences*, 1(1), 37–
490 51. <https://doi.org/10.3233/JGS-13009>
- 491 McNamara, A. M. & Taylor, R. N. (2002). Use of heave reducing piles to control ground
492 movements around excavations. *In Physical modelling in geotechnics: proceedings of*
493 *international conference ICPGM'02* (eds P. Guo, R. Phillips and R. Popescu), pp. 847–
494 852. Rotterdam, the Netherlands: Balkema.
- 495 McNamara, A. M. (2001). Influence of heave reducing piles on ground movements around
496 excavations. PhD thesis, City University, London, UK.
- 497 McNamara, A. M., Goodey, R. J. & Taylor, R. N. (2009). Apparatus for centrifuge modelling
498 of top down basement construction with heave reducing piles. *Int. J. Phys. Modelling*
499 *Geotech*, 9(1), 1–14, <https://doi.org/10.1680/ijpmg.2009.9.1.01>.
- 500 Ng, C. W. W., Shi, C., Gunawan, A., & Laloui, L. (2014). Centrifuge modelling of energy piles
501 subjected to heating and cooling cycles in clay. *Géotechnique Letters*, 4(4), 310–316.
502 <https://doi.org/10.1680/geolett.14.00063>
- 503 Ouzzine, B., de Sauvage, J., Madabhushi, G., Viggiani, G., & Reiffsteck, P. (2023).
504 Centrifuge modelling of an energy pile group with ground water flow. *International*

- 505 *Journal of Physical Modelling in Geotechnics*, 1-25.
506 <https://doi.org/10.1680/jphmq.22.00041>
- 507 Parchment, J., & Shepley, P. (2018). The influence of temperature on shear strength at a
508 soil-structure interface. In A. McNamara, S. Divall, R. Goodey, N. Taylor, S. Stallebrass,
509 & J. Panchal (Eds.), *Physical Modelling in Geotechnics* (1st ed., pp. 149–154). CRC
510 Press. <https://doi.org/10.1201/9780429438660-15>
- 511 Phillips, R. (1995). Centrifuge modelling: practical considerations. In *Geotechnical centrifuge*
512 *technology* (ed. R. N. Taylor), Ch. 3, pp. 34–59. Glasgow, UK: Blackie Academic and
513 Professional. <https://doi.org/10.1201/9781482269321-3>
- 514 Rollins, K. M., Lane, J. D., & Gerber, T. M. (2005). Measured and Computed Lateral
515 Response of a Pile Group in Sand. *Journal of Geotechnical and Geoenvironmental*
516 *Engineering*, 131(1), 103–114. [https://doi.org/10.1061/\(ASCE\)1090-0241\(2005\)131:1\(103\)](https://doi.org/10.1061/(ASCE)1090-0241(2005)131:1(103))
- 518 Rosquoët, F., Thorel, L., Garnier, J., & Canepa, Y. (2007). Lateral Cyclic Loading of Sand-
519 Installed Piles. *Soils and Foundations*, 47(5), 821–832.
520 <https://doi.org/10.3208/sandf.47.821>
- 521 Schofield, A. N. & Taylor, R. N. (1988). Development of standard geotechnical centrifuge
522 operations. In *Centrifuge 88: proceedings of the international conference on geotechnical*
523 *centrifuge modelling* (ed. J.-F. Corté), pp. 29–32. Rotterdam, the Netherlands: Millpress.
- 524 Shabanpour, A., & Ghazavi, M. (2022). Centrifuge tests on axially loaded tapered piles with
525 different cross-sections under compressive and tensile loading. *Canadian Geotechnical*
526 *Journal*, 59(7): 1205-1214. <https://doi.org/10.1139/cgj-2020-0732>
- 527 Taylor, R. N., Rose, A. V., & Gorasia, R. J. (2013). Pile and Pile Group Capacity: Some
528 Findings from Centrifuge Tests. *Int. J. Geotech. Engng* 5(2), 5-15.
- 529 Truong, P., Lehane, B. M., Zania, V., & Klinkvort, R. T. (2019). Empirical approach based on
530 centrifuge testing for cyclic deformations of laterally loaded piles in sand. *Géotechnique*,
531 69(2), 133–145. <https://doi.org/10.1680/jgeot.17.P.203>
- 532 Wood, D. M. (2004). *Geotechnical modelling*. New York, NY, USA: Spon Press.
- 533 Yavari, N., Tang, A. M., Pereira, J. M., & Hassen, G. (2016). Effect of temperature on the
534 shear strength of soils and the soil–structure interface. *Canadian Geotechnical Journal*,
535 53(7), 1186-1194. <https://doi.org/10.1139/cgj-2015-0355>
- 536 Zhao, R., Leung, A. K., Vitali, D., Knappett, J. A., & Zhou, Z. (2020). Small-scale modeling of
537 thermomechanical behavior of reinforced concrete energy piles in soil. *Journal of*
538 *Geotechnical and Geoenvironmental Engineering*, 146(4), 04020011.
539 [https://doi.org/10.1061/\(ASCE\)GT.1943-5606.0002225](https://doi.org/10.1061/(ASCE)GT.1943-5606.0002225)
- 540

541 List of Figures

- 542 **Figure 1: Normalised load for the ribbed piles (Modified from Gorasia and McNamara, 2016).**
- 543 **Figure 2: a) sketch of the impression pile; b) failure mechanism for values of spacing lower than**
544 **critical (modified from Lalicata *et al.*, 2022).**
- 545 **Figure 3: Nodule's creation from the plain pile bore (after Lalicata *et al.*, 2021).**
- 546 **Figure 4: a) exhumed resin piles; b) assembled 3d printed piles.**
- 547 **Figure 5: a) plastic model pile components; b) construction of the plastic model impression pile.**
- 548 **Figure 6: Installation of the 3D printed plastic model pile.**
- 549 **Figure 7: Sketch of centrifuge test (modified from Lalicata *et al.*, 2021).**
- 550 **Figure 8: a) shear strength distribution from water content; b) OCR profile with depth.**
- 551 **Figure 9: Load settlement curves: a) Resin model piles; b) Plastic model piles.**
- 552 **Figure 10: secant stiffness: a) Resin model piles; b) Plastic model piles.**
- 553 **Figure 11: Load ratio for resin and plastic model piles.**
- 554 **Figure 12: Normalised direct shear tests results.**

555 List of Tables

556 **Table 1. Mechanical parameters of the Speswhite Kaolin clay.**

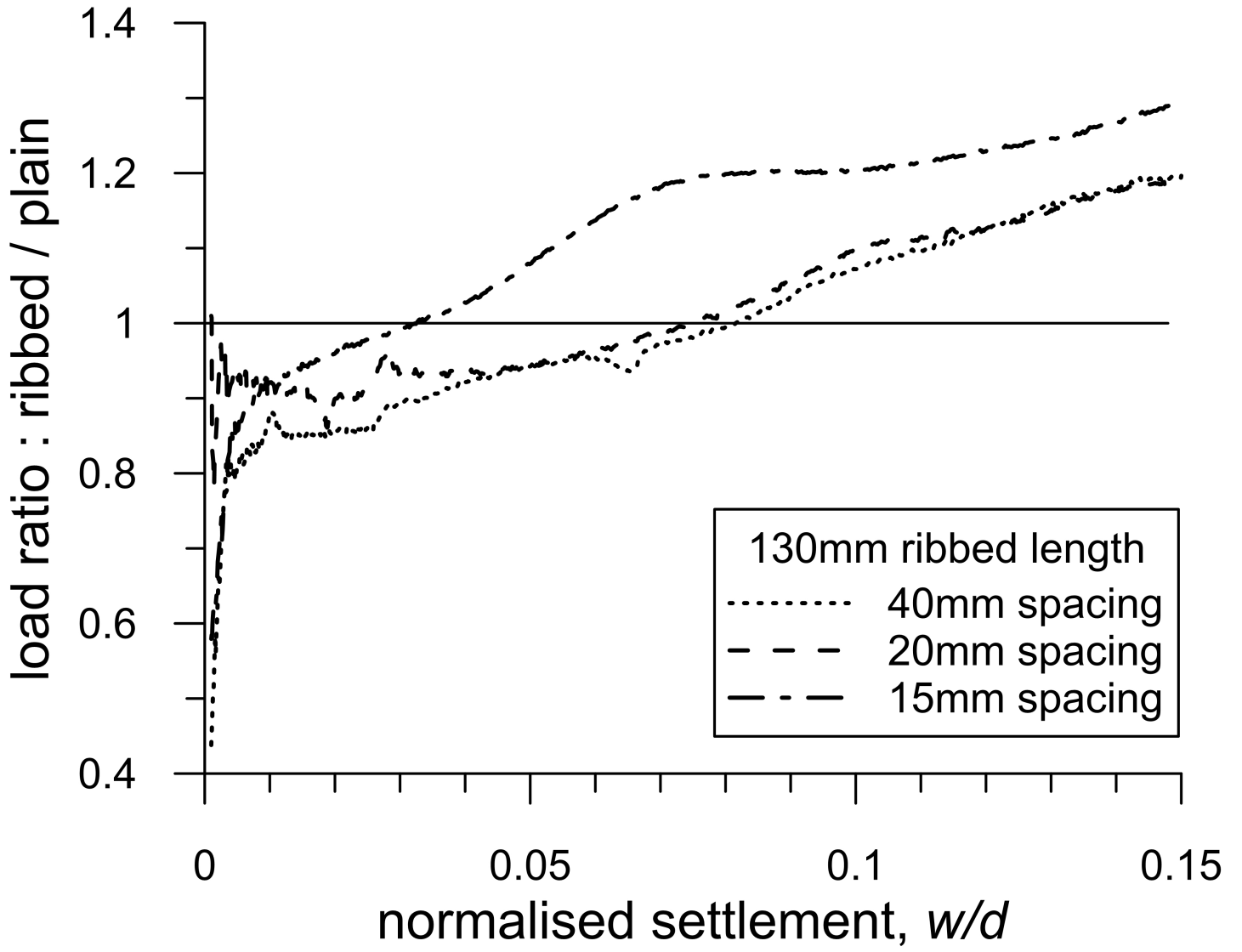
557 **Table 2. Summary of the centrifuge tests.**

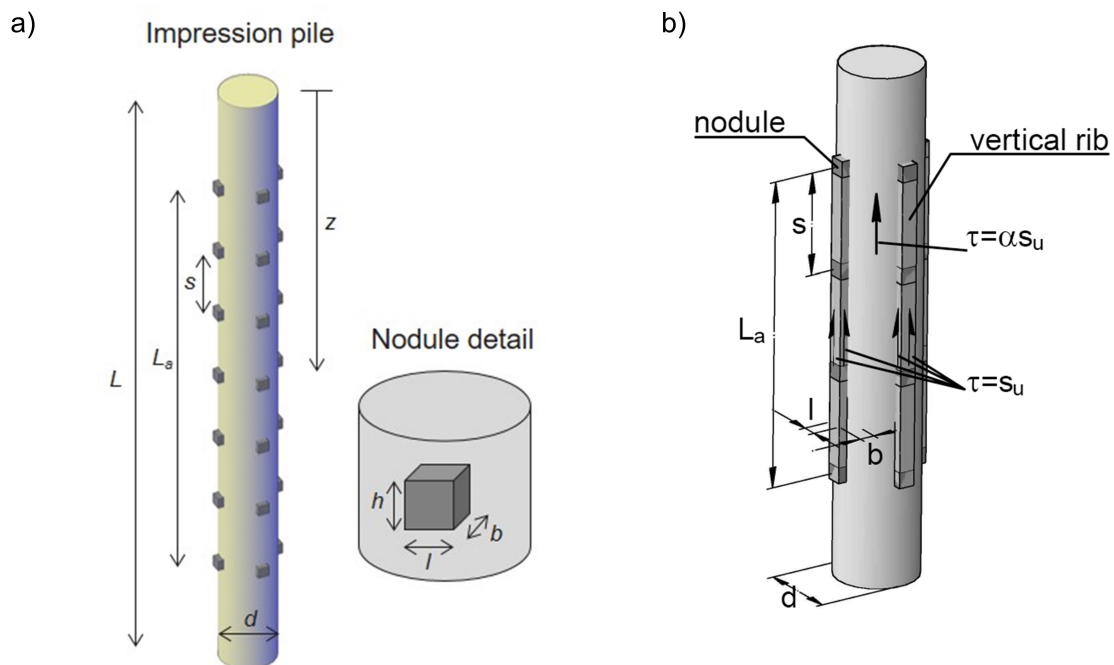
558 **Table 3. Capacity and increase in capacity of the resin piles.**

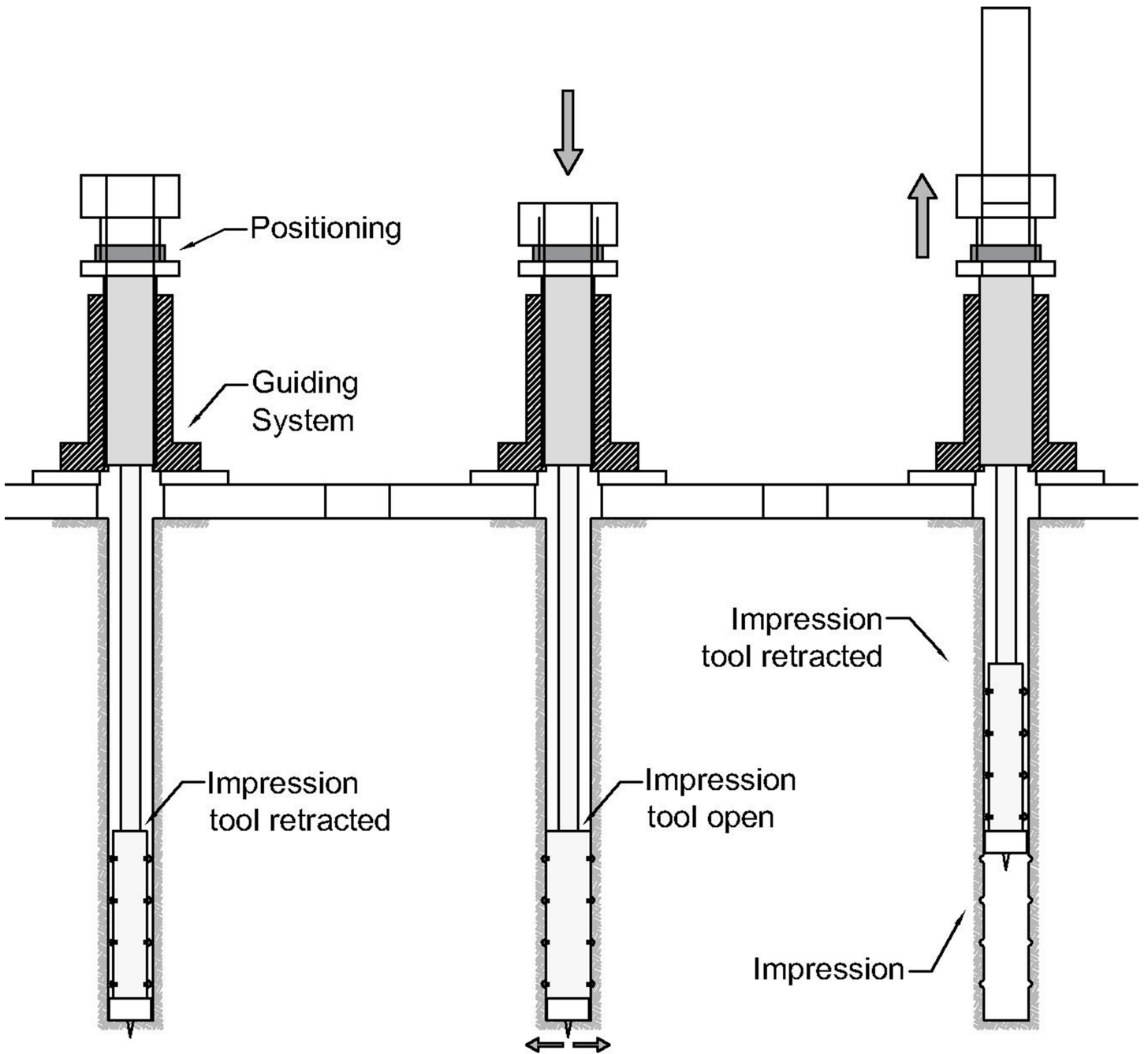
559 **Table 4. Capacity and increase in capacity of the plastic piles.**

560 **Table 5. Values of secant stiffness at selected settlement levels.**

561 **Table 6. Interface direct shear tests results.**





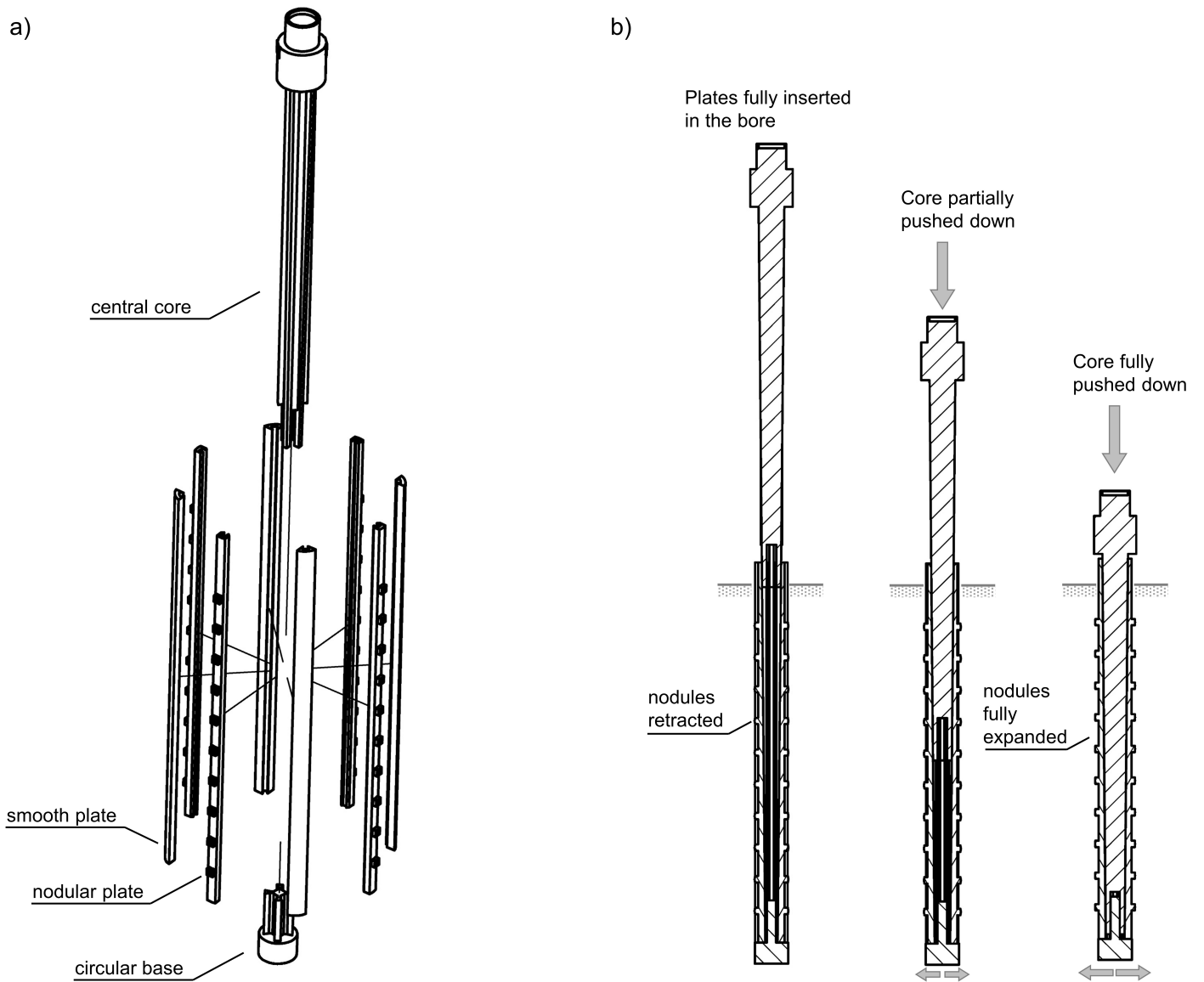


a) Resin Piles

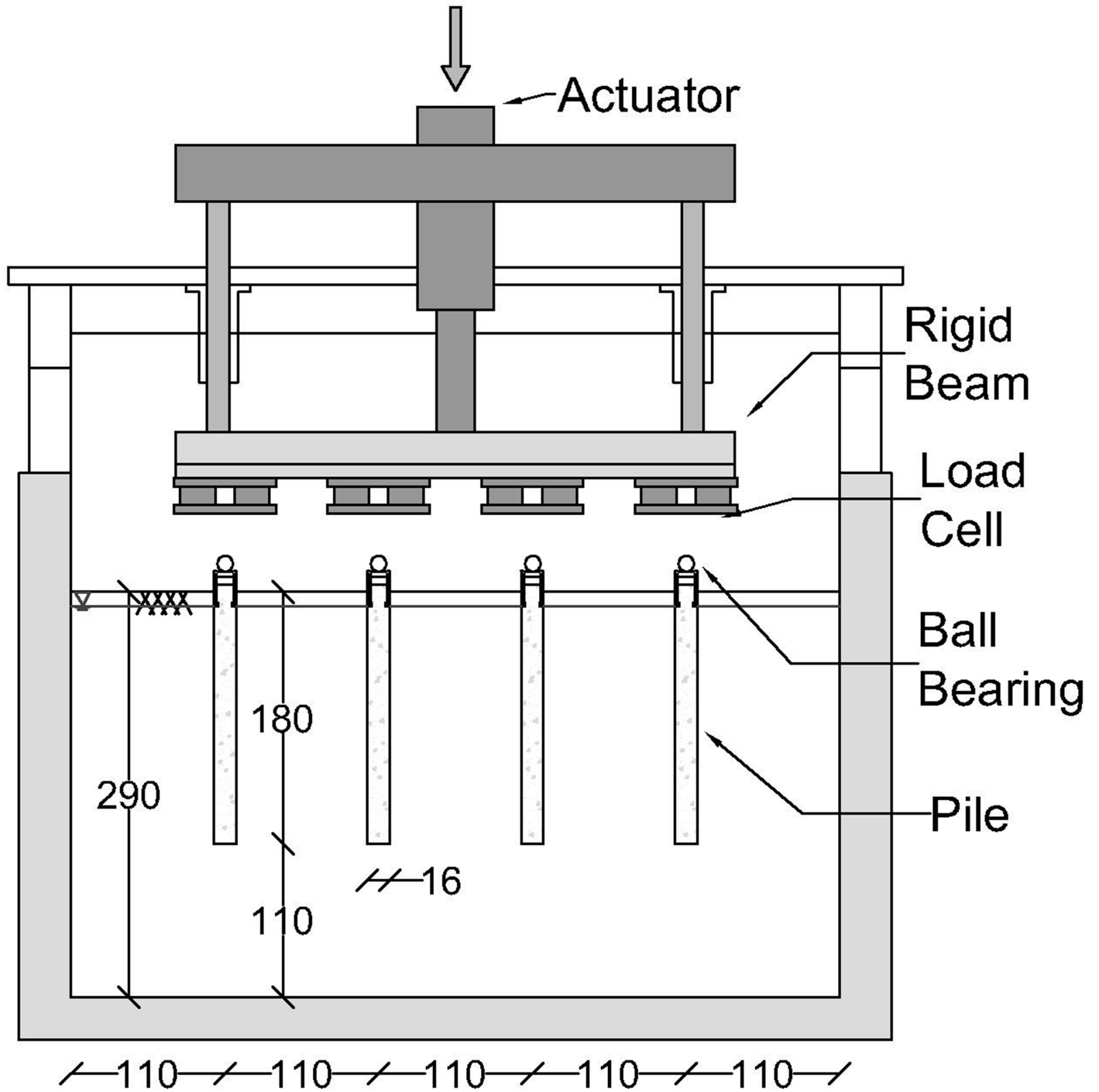


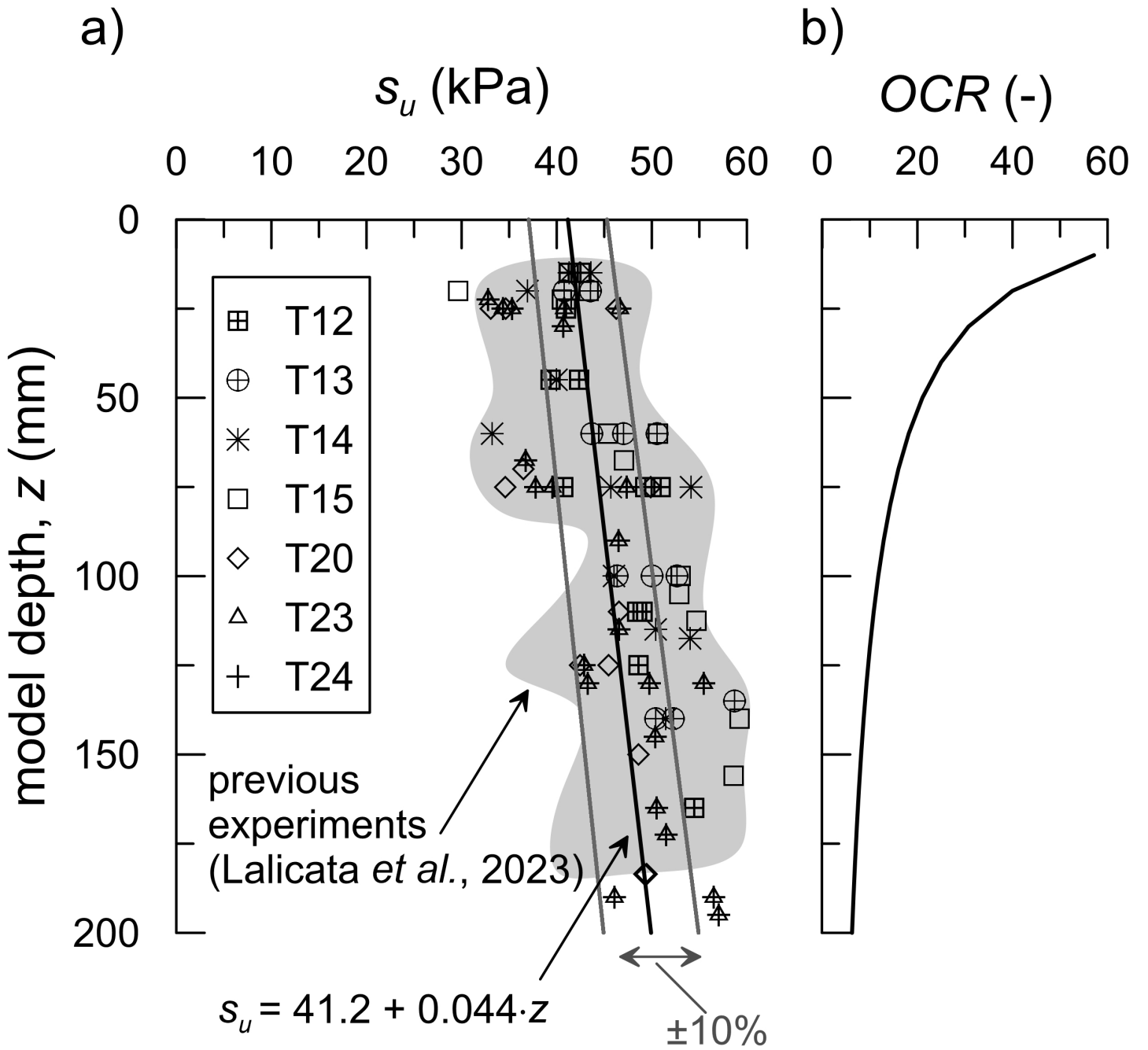
b) 3D Printed Piles

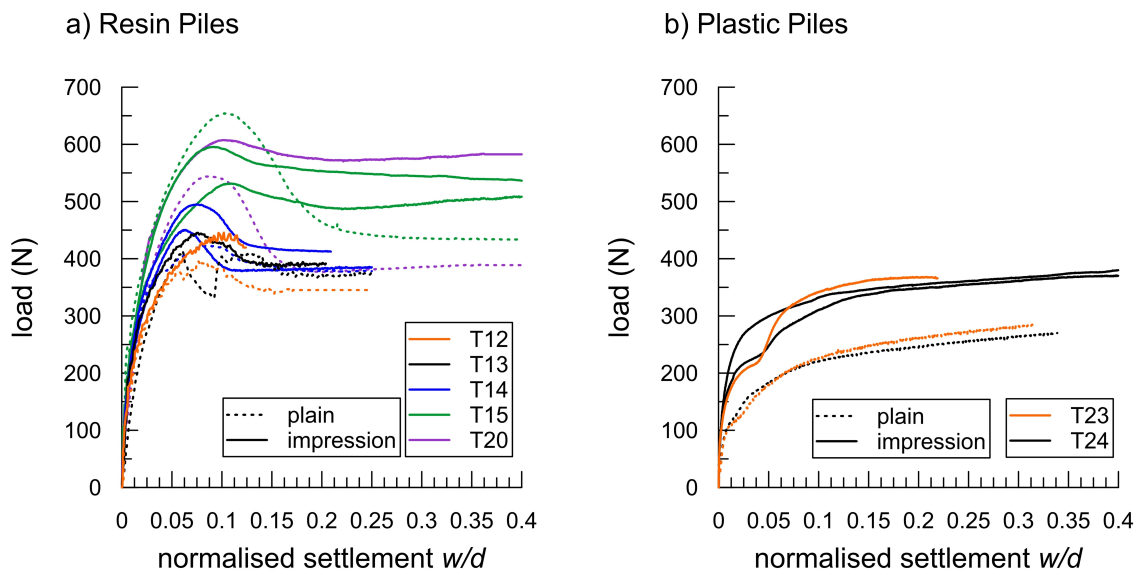


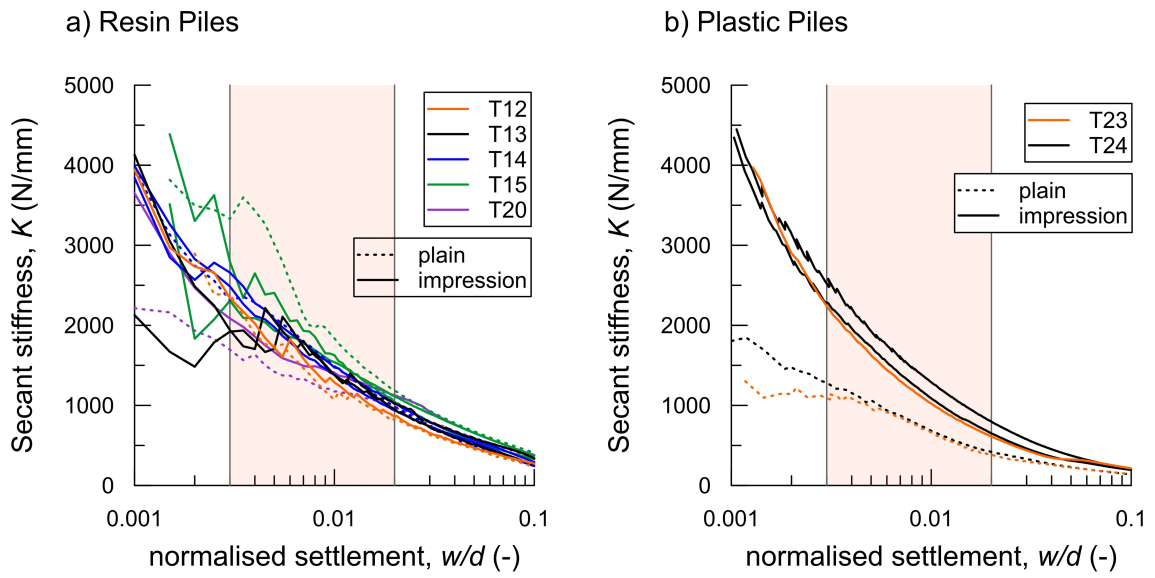


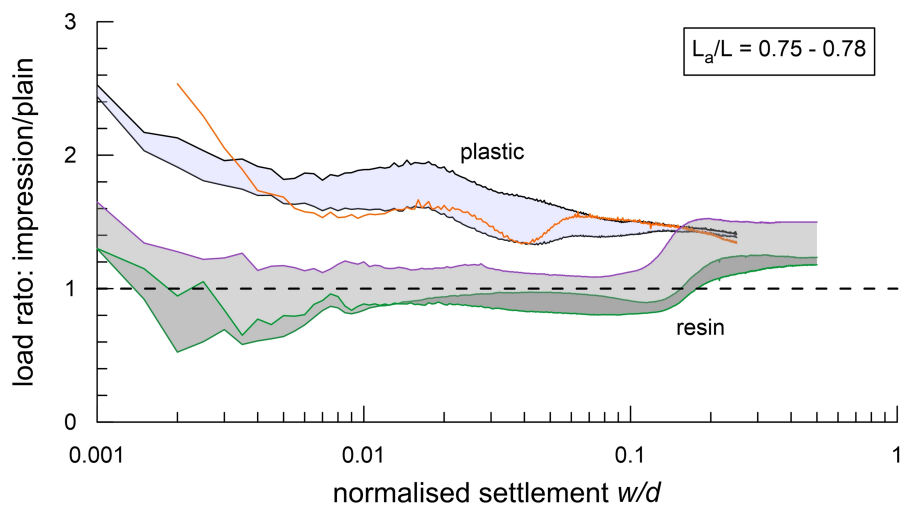












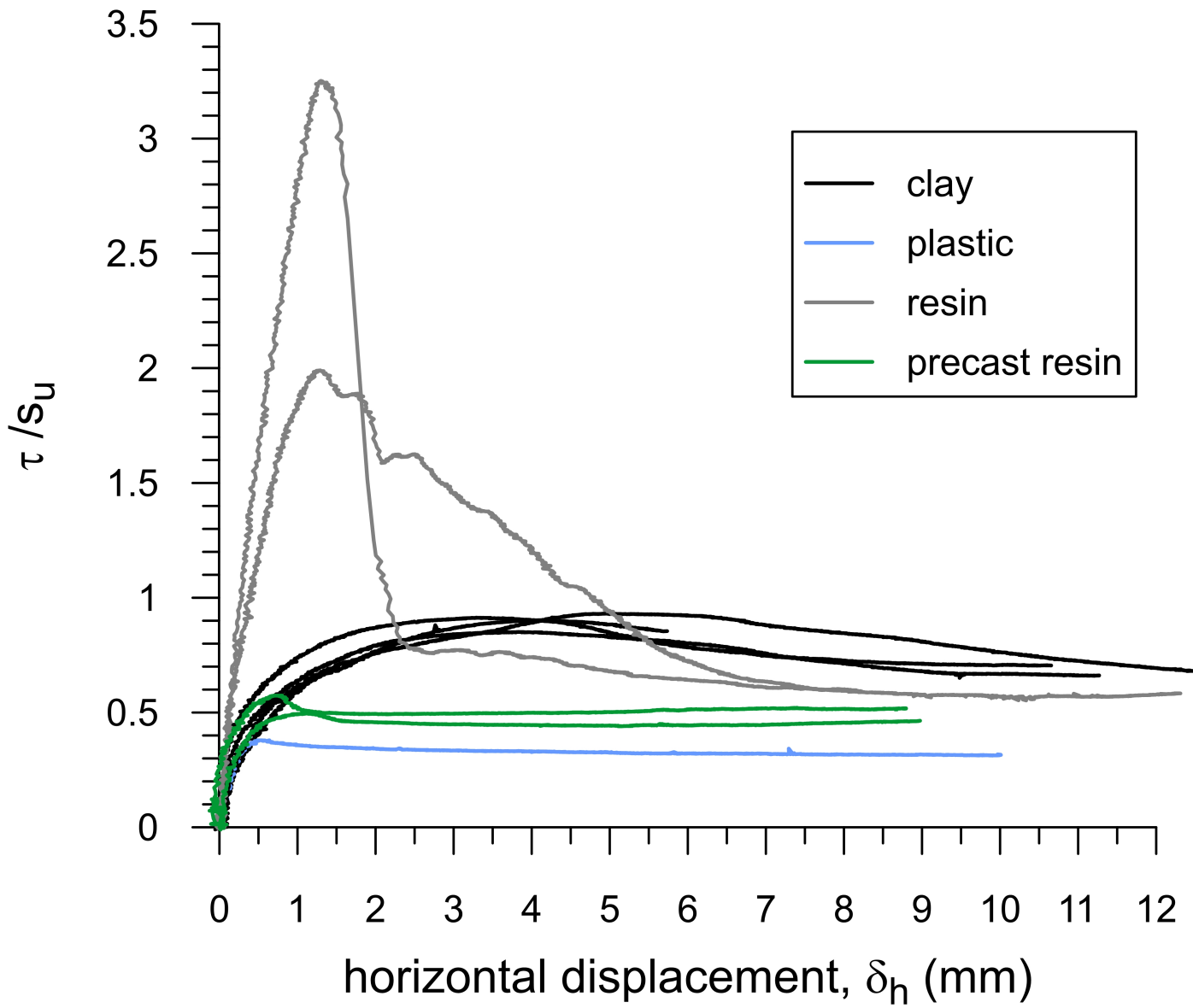


Table 1. Mechanical parameters of the Speswhite Kaolin clay.

Parameter	Value
Slope of the critical state line in the q - p' space, M	0.89
Specific volume on the critical state line at $p'=1\text{kPa}$, Γ	3.04
Slope of the normal compression line in the v - $\ln p'$ space, λ	0.18
Specific gravity, G_s	2.61
Unit weight of soil, γ (kN/m^3)	17.5
Coefficient of vertical consolidation, c_v (mm^2/s)	0.45

Table 2. Summary of the centrifuge tests.

Test ID	Material	Pile Type	Normalised Active Length, L_a/L	Dead Weight, W (N)	Adhesion factor, α
T12	resin	plain		46	0.66
		impression	0.67	46	
T13	resin	plain		46	0.74
		impression	0.29	46	
T14	resin	plain		46	0.74
		impression	0.29	46	
		impression	0.29	46	
T15	resin	plain		46	0.88
		impression	0.78	46	
		impression	0.78	46	
T20	resin	plain		46	0.77
		impression	0.78	46	
T23	plastic	plain		43	0.47
		impression	0.75	43	
T24	plastic	plain		43	0.41
		impression	0.75	43	
		impression	0.75	43	

Table 3: Capacity and increase in capacity of the resin piles.

Resin	La/L	Graphical method	
		capacity, Qu (N)	Increase in capacity (-)
plain_T12	-	342	-
plain_T13	-	375	-
plain_T14	-	375	-
plain_T15	-	432	-
plain_T20	-	389	-
impression_T13	0.29	390	1.04
impression_T14	0.29	410	1.09
impression_T14	0.29	390	1.04
impression_T15	0.78	531	1.23
impression_T15	0.78	510	1.18
impression_T20	0.78	583	1.5

Table 4: Capacity and increase in capacity of the plastic piles.

Plastic Pile	La/L	Methods to estimate the pile's capacity							
		settlement				Double tangent		Chin	
		w/d = 10%		w/d = 25%		w/d ~ 1 – 1.2%		0.9*Chin	
		capacity, Qu (N)	Increase in capacity (-)	capacity, Qu (N)	Increase in capacity (-)	capacity, Qu (N)	Increase in capacity (-)	capacity, Qu (N)	Increase in capacity (-)
plain_T23	-	226.3	-	272	-	201.9	-	270.3	-
plain_T24	-	220.7	-	256.5	-	204.3	-	246.2	-
impression_T23	0.75	341.7	1.51	365.7	1.34	339.2	1.68	363	1.34
impression_T24	0.75	332.4	1.51	361.1	1.41	321.2	1.57	337.3	1.37
impression_T24	0.75	310.3	1.41	354.7	1.38	298.7	1.46	345.4	1.4

Table 5: Values of secant stiffness at selected settlement levels.

w/d	Resin Plain	Resin Impression	Plastic Plain	Plastic Impression	Ratio: Resin/Plastic Plain	Ratio: Resin/Plastic Impression
(-)	(N/mm)	(N/mm)	(N/mm)	(N/mm)	(-)	(-)
0.5%	1840 ±702	2002 ±306	1012 ±47	1762 ±124	1.82	1.14
1%	1296 ±332	1525 ±90	672 ±10	1133 ±113	1.93	1.35
2%	946 ±150	1095 ±35	396 ±21	686 ±80	2.39	1.60
5%	538 ±84	622 ±48	226 ±2	336 ±28	2.38	1.85

Table 6: Interface direct shear tests results.

Material	Shear stress s_u (kPa)	Peak stress, τ_p/s_u	Ultimate stress τ_u/s_u
clay	52.4	0.9	-
clay	57.2	0.93	0.68
clay	45.1	0.85	0.66
clay	53.8	0.91	0.7
resin	49.3	2.0	0.57
resin	55.1	3.25	0.57
precast resin	54.7	0.57	0.52
precast resin	48.9	0.50	0.47
plastic	42.7	0.38	0.33