



HAL
open science

Geosynthetic reinforcement of pile-supported embankments

Romain Girout, Matthieu Blanc, Luc Thorel, Daniel Dias

► **To cite this version:**

Romain Girout, Matthieu Blanc, Luc Thorel, Daniel Dias. Geosynthetic reinforcement of pile-supported embankments. *Geosynthetics International*, 2018, 25 (1), pp.37-49. 10.1680/jgein.17.00032 . hal-01915220

HAL Id: hal-01915220

<https://hal.science/hal-01915220>

Submitted on 8 Jun 2021

HAL is a multi-disciplinary open access archive for the deposit and dissemination of scientific research documents, whether they are published or not. The documents may come from teaching and research institutions in France or abroad, or from public or private research centers.

L'archive ouverte pluridisciplinaire **HAL**, est destinée au dépôt et à la diffusion de documents scientifiques de niveau recherche, publiés ou non, émanant des établissements d'enseignement et de recherche français ou étrangers, des laboratoires publics ou privés.

1 **Geosynthetic reinforcement of pile-supported embankments**

2
3 Author(s): Romain Girout^{1*}, Matthieu Blanc², Luc Thorel³ and Daniel Dias⁴

4 ¹ Post-doctoral Researcher, Department of Civil and Environmental Engineering, COPPE,
5 Federal University of Rio de Janeiro, Ilha do Fundão, RJ 21945-970, Brazil.

6 E-mail: rgirout@gmail.com (corresponding author)

7 ² Researcher, IFSTTAR, Department GERS, Laboratory Geomaterials and Models in
8 Geotechnics, Route de Bouaye, Point 86, CS4, 44344 Bouguenais Cedex, France.

9 E-mail: matthieu.blanc@ifsttar.fr

10 ³ Senior Researcher, IFSTTAR, Department GERS, Laboratory Geomaterials and Models in
11 Geotechnics, Route de Bouaye, Point 86, CS4, 44344 Bouguenais Cedex, France.

12 E-mail: luc.thorel@ifsttar.fr

13 ⁴ Professor, Grenoble Alpes University, 3SR Laboratory, 3SR Lab, UMR 5521, BP53, 38041
14 Grenoble Cedex 09, France,

15 E-mail: daniel.dias@3sr-grenoble.fr

16 •

17 **ABSTRACT**

18 Rigid piles are used to reinforce soft soil base and increase embankment stability. This
19 technique is improved by placing one or more geosynthetic reinforcement (GR) layers inside
20 or at the base of the embankment. A series of 33 small scale models has been tested using a
21 geotechnical centrifuge. Soft soil settlement is imposed by the downward displacement of a
22 tray. First, a series of models is prepared to examine how the load transmitted to the pile
23 network increases with the embankment thickness. For a same configuration, two identical
24 models are prepared to test successively two different types of GR (Geosynthetic
25 Reinforcement). Another approach consists in studying how the external surcharges applied
26 on the embankment affect load transfer. The results show that, in comparison with the piled
27 embankment, the load transfer is increased in the case of the Geosynthetic-Reinforced Pile-
28 Supported Embankment (GRPSE) due to the membrane effect. The membrane effect is higher
29 when the GR is stiff and its vertical distance from the pile is reduced. Numerical modelling
30 reveals that, when adding a GR layer, the second GR has an effect only if punching is
31 sufficient. However, the benefits of it could not be highlighted here.

32

33 **Key-words:** Geosynthetics, piles, embankment, arching, membrane effect

34

35 1. INTRODUCTION

36 One technique for the safe building of embankments on soft to very soft soils consists in
37 driving a network of rigid piles into the soft soil until they reach harder surfaces. Part of the
38 load is transferred within the embankment towards the piles through shearing and arching
39 effects.

40 In order to enhance the efficiency of this composite foundation, a Geosynthetic
41 Reinforcement (GR) can be added (Fig. 1a). The deformation of the GR acts as a horizontal
42 reinforcement through membrane effect (Villard and Giraud, 1998). The presence of the GR
43 within the embankments also allows for the reduction of the post-construction final
44 settlements. To this day, Geosynthetic-Reinforced Pile-Supported Embankment (GRPSE) is a
45 technique, which is studied using field or full-scale experimental cases (Le Hello, 2007;
46 Briançon and Simon, 2012; Briançon and Dias, 2015 ; Nunez et al., 2013 ; Xing et al., 2014 ;
47 Chen et al., 2016), 1×g small-scale models at a scale of 1/N (Jenck et al., 2005, Jenck et al.,
48 2007, Eskişar et al., 2012, Van Eekelen et al. (2012a, b, 2015)) and centrifuge modelling at a
49 scale 1/N and a g-level equals to N (Ellis and Aslam, 2009; Blanc et al., 2013; Girout et al.,
50 2014). These physical modelling tests have been often supplemented by numerical analyses
51 (Le Hello and Villard, 2009; Girout et al., 2014 or Han-Jiang et al., 2014; Zhuang and Ellis,
52 2016; Zhuang and Wang, 2015; 2016) .

53 Some standard and design guidelines like EBGEO (2011) have been examined by Blanc et al.
54 (2014), for which the vertical displacement of a GR strip between two piles is compared with
55 the physical modelling results. Girout et al. (2016) have carried out this comparison in the case
56 of unreinforced piled embankment. Van Eekelen et al. (2015) have validated their concentric
57 arching model through a comparison between seven field tests and four small scale models. A
58 recent analytical computation (not compared in the present study) proposed by King et al.
59 (2017) demonstrates the need for some models coupling arching and stress-deformation to
60 describe embankment behaviour reliably.

61 Different in-situ studies have been conducted to carry out a comparison between piled-
62 embankment and GRPSE, including a multi-layer GRPSE (Bhasi and Rajagopal; 2014;
63 Briançon and Simon, 2017) to examine load transfer toward the piles, settlement and GR
64 deformation.

65 Okyay et al. (2014) and Fagundes et al. (2015) have conducted their tests using the Mobile
66 Tray (MT) presented in Fig. 2. This device principle consists in simulating, through the tray
67 displacement, the settlement due to the compressibility of the soft soil instead of modelling

68 soft soil. Using the same MT device, Blanc et al. (2013) have carried out tests on thin granular
69 embankments, with a surcharge applied on the surface in order to examine GR improvement
70 (Fig. 1b). As a conclusion of this study, it appears that a pre-tension applied to the GR does
71 not increase significantly load transfer toward the piles. The same observation is made for the
72 tests conducted with two layers of GR instead of one. This research has also led to the
73 conclusion that the stiffer the GR is, the lower the final deformation of the GR. The
74 embankment thickness never exceeds more than 1.8m. Only one secant stiffness is tested for
75 the geosynthetic reinforcement. The hydraulic load is kept constant and the geosynthetic
76 reinforcement is placed below the embankment. Girout et al. (2016) and Fagundes et al.
77 (2015) have used the MT device only for unreinforced piled embankment studies (e.g.,
78 without GR) and performed many tests to investigate embankment behaviour by varying the
79 characteristics of the piled-embankment (thickness H , spacing s between the piles axis, and
80 pile diameter d). The measurements obtained from the load sensors, placed inside the piles at
81 the network center and used to obtain the mean load F_m , have been analyzed using the load
82 transfer efficiency equation (Hewlett and Randolph, 1988) as:

$$E_F = \frac{F_m}{(\rho_d \cdot g \cdot N \cdot H + q_0) \cdot s^2} \quad (1)$$

83 where F_m is the average load measured per pile unit, $\rho_d \cdot g \cdot N \cdot H$ is the vertical stress due to
84 the embankment thickness with ρ_d the embankment density, g the earth acceleration and N the
85 scale of model, q_0 is the additional homogeneous stress resulting from the hydraulic load
86 applied on the embankment surface with a membrane that separates the water from the soil.
87 The nine load sensors are placed on the inside of the top of the piles. For a piled embankment,
88 this measurement corresponds to the arching effect (called A in the literature). For a GRPSE,
89 it corresponds to the sum $A + B$, where B is the load transferred by membrane effect.
90 Considering the difficulty of calibrating total stress cells dealing with scaling law issues in
91 small-scale model samples, it was not possible to install load cells inside the embankment
92 above the geosynthetic reinforcement to measure B separately. For this reason, the results are
93 analyzed considering the efficiency E_F .

94 The aim of this paper is to present the findings of the experimental investigation carried out to
95 examine the GR layer (s) reinforcement of a piled-embankment using the MT device. The
96 spacing and the pile diameter are kept constant and are equal to 2.0 m and 0.5m, respectively.
97 First, some tests are conducted to examine the load transfer as a function of the embankment
98 thickness for unreinforced and reinforced piled-embankments. These investigations are

99 carried out by considering the geosynthetic reinforcement with different secant stiffness
100 values (4.0 MN/m and 16.8 MN/m). Then, the impact on the arching effect of the distance
101 between the piled embankment basal and the horizontal GR is studied through a series of tests
102 conducted on a 7.2-m thick embankment by increasing the GR elevation within the
103 embankment. Finally, the results of the tests carried out with the same initial load on the
104 mobile tray, though applied differently (by using water tank or embankment thickness), are
105 compared in terms of efficiency.
106

107 2. PHYSICAL MODELLING IN GEOTECHNICS

108 By increasing artificially the standard earth gravity, g , by a factor N , centrifuge modelling
109 makes it possible to study a $1/N$ small scale model using the same stress and deformation levels
110 than a full scale prototype (Garnier et al., 2007). In this study, N is equal to 20. Table 1 presents
111 the different scaling factors linking both model and corresponding prototype. The scale factors
112 for the geosynthetic secant stiffness and the length here are equal to $1/N$.

113 3. EXPERIMENTAL DEVICE

114 3.1.1. *Description*

115 The Mobile Tray method, used by Blanc et al. (2013) for the investigation of GR insertions,
116 consists in imposing some displacements $\Delta\omega$ to a perforated steel tray above a rigid pile
117 network to simulate soft soil settlement (Fig. 2a). A model embankment is installed on the tray
118 using a varying number of steel rings depending on the chosen thicknesses (Fig. 2b). The
119 embankment sand material is poured manually until the desired total height is reached and the
120 density is $\rho_d = 1.62 \text{ kg/m}^3$. The pile network and the granular embankment main characteristics
121 (spacing and thickness) can be modified. Depending on the tests, one or two GR layers are
122 optionally inserted beneath the model embankment on the MT surface. A surcharge can also be
123 applied on the embankment top surface using a tank filled with water. The tank bottom is made
124 of a soft membrane resting on the top of the embankment (Blanc et al., 2013).

125 3.1.2. *Materials*

126 The embankment is made of a mix of five Hostun sand fractions used to model the full scale
127 gravel (Girout et al., 2014). In order to avoid scale effects, the mix is cut at 1 mm (the pile
128 diameter considered here being equal to 25mm). The characteristics of the Hostun sand mix
129 are summarized in Table 2.

130 As the secant stiffness for the small scale GR has to be N times lower than the prototype, a
131 woven polypropylene geosynthetic has been selected to simulate prototype geogrids. The GR
132 is characterized by the stiffness in the machine direction (i.e., the direction in paper and board
133 that coincides with the longitudinal direction of the web) and the stiffness in the cross
134 machine direction (direction perpendicular to the machine direction). The characteristic values
135 (T_{max} and ε_{max} , which are the maximum tensile load and deformation, respectively) are
136 presented in Table 3 for PP25 and PP60 materials. As an illustration, PP25 basal GR results
137 are displayed in Fig. 3. The test begins when the MT is placed into the swinging basket of the

138 centrifuge. The entire process is described in Blanc et al. (2013) as regards the tests including
139 a surcharge, for which the water tank is filled until the expected stress level is reached. The
140 tests conducted without surcharge are quicker because the tank does not need to be filled with
141 water. Then, the test procedure only consists in prescribing a displacement $\Delta\omega$ to the tray up
142 to 30mm (i.e., a soft soil displacement equal to 600 mm on prototype scale).

143 3.1.3. *Experimental campaign*

144 In situ, the pile network is designed by the ratio of the pile area $\pi.d^2/4$ to the mesh area s^2 (for
145 a square pattern), called the area replacement ratio, α . The tests are conducted for one single
146 area replacement ratio: $\alpha = 4.91 \%$ and four thicknesses: $H = 1.8$ m, 3.2 m, 5.0 m and 7.2 m on
147 prototype scale. In order to establish the relationship between H , s and d , the authors use the
148 parameter H_{arch} , defined as half the length of the diagonal between the pile edges as described
149 by:

$$H_{arch} = (s\sqrt{2} - d) / 2 \quad (2)$$

150 The tests and their characteristics are presented in Table 4. The tests written in bold (17 tests)
151 have already been published in Fagundes et al. (2017), Girout et al. (2014) or Girout et al.
152 (2016). Initially developed during the French national project A.S.I.Ri (IREX, 2012), the MT
153 device is used to investigate load transfer towards rigid inclusions either within embankments
154 toward the rigid piles, into granular mattresses below a slab or into lime and cement treated
155 soils (Okyay, 2010; Okyay and Dias, 2010). To this a GR material can be added (Blanc et al.,
156 2014). The influence of the different types of surcharge loads on the stress applied on the mobile
157 tray surface is investigated. As illustrated in Fig. 4, different configurations can be used to
158 obtain the same initial stress applied on the tray. Considering $\rho_d = 1.62 \text{ kg/m}^3$, the initial stress
159 applied on the GR is equal to the sum of the hydraulic column and the granular column, i.e.,
160 $80\text{kPa} + 1.62 \cdot 9.81 \cdot 1.8 = 109 \text{ kPa}$, to which the weight of the soft membrane (4 kPa) must be
161 added (Okyay et al., 2010). We then obtain 113kPa. As a comparison, the stress induced by a
162 7.2-m high embankment (T10 and T17 test models) is 114kPa.

163 **4. TESTS ANALYSIS**

164 The test series is conducted to examine how a GR insertion influences piled embankment
165 behaviour for different thicknesses H and surcharges q_0 . With this aim in view, different types
166 of models are prepared: with basal GR (i.e., at the soft soil/embankment interface) and
167 without GR. Two different GR stiffness values are tested, characterized by their secant
168 stiffness J_a : first, $J_a = 4.0$ MN/m and then, $J_a = 16.8$ MN/m.

169 The efficiency E_F is plotted as a function of $\Delta\omega$, of H (Fig. 5, Fig. 7a, Fig. 8a, b and Fig. 9)
170 and of h_{gx}/H (Fig. 7b), where h_{gx} is the elevation of the GR within the embankment. In order
171 to address the influence of the thickness, the efficiency is compared as a function of H for
172 different $\Delta\omega$ (0.02, 0.08 and 0.56m) in Fig. 5b. We consider both non-reinforced and
173 geosynthetic-reinforced pile-supported embankments. The displacement values are chosen in
174 order to compare efficiency values ranging from very small displacements (just before the
175 maximum efficiency is reached), to maximum displacement values imposed to the tray. When
176 the efficiency reaches its maximum value, the particular value of the tray displacement is
177 plotted as a function of the embankment thickness (Fig. 5b). For the reinforced case, this
178 corresponds to the vertical displacement δ_g of the GR at the center of the mesh.

179 **4.1. Unreinforced piled embankments**

180 The load transfer efficiency versus embankment thickness is first investigated for four different
181 thickness values ($H = 1.8$ m, 3.2m, 5.0m and 7.2m, respectively). First, no surcharge is
182 considered (i.e., $q_0 = 0$ kPa).

183 Considering the curves $E_F - \Delta\omega$ in Fig. 5a for the unreinforced piled embankment model
184 (tests T1-4-7 and 10, No GR), the efficiency increases with the tray displacement. All the
185 curves are asymptotic save for the case $H = 1.8$ m. The maximum value reached by the
186 efficiency E_F clearly increases with the thickness. However, for large tray displacements ($\Delta\omega$
187 = 0.56m), the efficiency for a thin embankment ($H = 1.8$ m) decreases up to its initial value.
188 This behaviour is similar to that of the stress deviator for a dense sample in triaxial tests. In
189 other words, the decrease of the efficiency only is due to the high displacement induced by the
190 tray. As regards in-situ embankments, settlement is lower and results from the stress applied
191 on its surface. The difference in results between the test in the case $H = 1.8$ m and the other
192 tests is due only to the fact that it is not possible to impose sufficient displacement.

193 Fig. 5b presents the comparison for the same tray displacement levels: low (0.02m), medium
194 and high (0.56m). The curves with the circular marker (case No GR) show that when the

195 thickness is low ($H = 1.8\text{m}$), the efficiency is less dependent upon soft soil settlement
196 because, contrary to thick embankments ($H = 7.2\text{m}$), the arching effect is low.
197 Fig. 5 (case No GR), on the other hand, displays the tray displacement values when the
198 efficiency reaches its maximum (i.e., $E_F = E_{Fmax}$). The curves increase linearly up to an
199 embankment thickness equal to 5 m and then plateau for the higher thicknesses studied.
200 Considering that the efficiency reaches a maximum value (approximately 65% for $H = 7.2\text{m}$
201 and $\alpha = 4.91\%$) and that it increases with the tray displacement, it is consistent that the
202 settlement itself reaches a maximum value. In other words, above a certain thickness (related
203 to the area replacement ratio), the efficiency does not increase any more. Because of a lower
204 total embankment weight, the thinner the embankment, the smaller the displacement required
205 to reach the maximum efficiency.
206 It can, therefore, be concluded that the load transfer efficiency increases with the embankment
207 thickness by arching effect. In the modelling study presented here, it reaches 65% for $H =$
208 7.2m (with $\alpha = 4.91\%$). The arching effect occurs with the prescribed displacement (i.e., soft
209 soil displacement) whereas settlement increases with embankment thickness.
210

4.2. Geosynthetic-Reinforced Pile-Supported Embankments

The aim of this section is to investigate how the GR decreases the stress applied on soft soil with relation to embankment thickness and GR stiffness. The same models as those already used are then reinforced with a basal horizontal GR. We first examine how the secant stiffness of the GR layer affects load transfer. Two different GR stiffness values are tested (4.0 MN/m and 16.8 MN/m, respectively). According to Fig. 5a, in which the efficiency versus the tray displacement is plotted for the 4.0 MN/m stiff GR, the efficiency increases because of the membrane effect when the soft soil settlement is high enough. For very small displacements ($\Delta\omega < 0.05\text{m}$), unreinforced and 4.0-MN/m stiff GR reinforced embankments are very close (Fig. 5b), even equal for thick embankments. This may be accounted for by the fact that the GR must be subjected to stress to “act” and because the stress applied on the soft soil in the case of thick embankments is higher (i.e., in case of lower efficiency). The lower the tray, the higher the GR is in tension (Fig. 3). For larger displacements ($\Delta\omega > 0.23\text{m}$), the efficiency reaches 100% whatever the thicknesses considered.

In the present case, it is possible to lower the tray until the GR loses its contact with the tray ($\delta_g = \Delta\omega$ for $E_F = 100\%$). This point is equivalent to the conservative case used in standards to design GR, for which the soft soil strength is not taken into account to calculate the stability of this composite foundation.

Fig. 6 displays a comparison of the tray displacements, for which the efficiency is highest ($\Delta\omega_{max} = \Delta\omega$ red on the y-axis to the left when unreinforced piled embankments are considered) with the case, for which the GR displacement is at its maximum (case GR, corresponding to the point where E_F reaches its maximum value). Fig. 6 shows that δ_g (GR deflection, red on the y-axis to the right) increases with the stress applied on the tray (i.e., the embankment thickness) whatever the thickness studied. The load transferred inside the embankment by shearing effect increases with the embankment thickness. The difference between this particular value $\Delta\omega$ and the deflection value δ_g increases with the thickness until full arching effect occurs. Regarding thin non-reinforced embankments, the GR supports fewer loads and its maximum deformation is nearly reached when the maximum efficiency is obtained. Fig. 6 presents the results for both GR stiffness values (4.0 MN/m and stiffer 16.8 MN/m). The slope of the trend line for $J_a = 4.0$ MN/m is approximately twice that for $J_a = 16.8$ MN/m. This can be accounted for by the fact that the GR is tensioned according to its stiffness and to the load applied on its surface. As a result, the deflection of the geosynthetic

243 depends mainly on the load applied on the geosynthetic, therefore on the thickness H and
244 consequently also on its stiffness.

245 The interesting finding of this work is the fact that, with a GRPSE, the membrane effect
246 permits the improvement of the total load transfer toward the piles. The stiffer the
247 geosynthetic is, the lower the deflection.

248 **4.3.Elevation of geosynthetic layers within the embankment**

249 Commonly, piled-embankments are reinforced using a geosynthetic layer placed close to the
250 embankment/soft soil interface. As recommended by the EBGE design guide (2011) in order
251 to prevent the stress concentration at the pile cap corner from damaging the GR, a thin granular
252 layer can be laid between GR and soft soil, as was also implemented on an experimental in situ
253 embankment (Brianchon and Simon, 2017). The aim of this section is to examine how the load
254 transfer efficiency depends on the thickness of the interface layer. The influence of the GR
255 height h_{gx} within the embankment is studied using a 7.2-m high embankment and a pile network
256 spacing of $s = 2.0\text{m}$.

257 Fig. 7a shows the efficiency for one GR layer at the following altitudes: 0m (like in the models
258 presented in Fig. 6), 0.2, 0.4, 0.8, 1.2 and 1.8m, respectively. The case model without GR is
259 also displayed here for comparison purposes. Both the reference model (i.e., $h_{gx} = 0\text{m}$) and the
260 one with a small altitude ($h_{gx} = 0.2\text{m}$) obtain an efficiency $E_F = 100\%$. However, the efficiency
261 with $h_{gx} = 0.2\text{m}$ is lower than in the reference case until the tray displacement $\Delta\omega$ reaches
262 0.28m. In both cases, the GR has been subjected to irreversible plastic strain (as shown in Fig.
263 3). In the next tests conducted with increasing geosynthetic altitudes ($h_{gx} = 0.4, 0.8, 1.2$ and
264 1.8m), the efficiency is lower than in the reference case ($h_{gx} = 0\text{m}$). Compared with the case
265 without GR, the efficiency is higher for small displacements. For settlement higher than 0.35m,
266 the efficiency is even lower for h_{gx} equal or higher than 0.8m. The reason for that is that the GR
267 “breaks” the arching effect occurring within the embankment. The consequence is that the soil
268 below the GR transmits a load directly to the tray, the value of which is much higher than the
269 stress transferred towards piles. Because of this the efficiency can be even lower than in the
270 case without GR. For $h_{gx} = 0.4\text{m}$, the efficiency increases again for a displacement equal to
271 0.45m. This is due to the fact that, as soon as the embankment has been sufficiently punched
272 by the pile (i.e., $\Delta\omega$ equal to the geosynthetic level), the tension force in the GR increases and
273 the membrane effect increases. Moreover, the arching effect is produced again above the GR.
274 For $h_{gx} \geq 0.4\text{m}$, the GR strain does not reached plastic yield.

275 The load transferred towards the piles depends on the stiffness and position of the GR within
276 the embankment. A stiff and basal GR produces higher transfer for low soft soil settlement
277 because of maximum membrane and arching effects. In other words, the GR has no effect in
278 terms of load transfer if it is placed too high within the embankment. In this case, indeed, load
279 transfer is even lower than the value achieved for the same embankment without GR.
280

281 **4.4.Surcharge applied on the embankment**

282 4.4.1. *Water tank*

283 Until now only load transfers due to the embankment weight itself have been considered. In
284 this section, we examine how an external surcharge applied here using a water tank can
285 modify the load transfer efficiency.

286 Using an additional device, it is possible to apply a surcharge at the top of the embankment to
287 investigate how the load transfer can be affected by a homogeneous and static surcharge (Fig.
288 8). Two different embankment thicknesses ($H = 1.8$ m and $H = 3.2$ m) and two surcharge
289 values (40 and 80 kPa) are tested.

290 Fig. 8 (a) displays the efficiency versus the tray displacement for the case where H is equal to
291 1.8m. Without GR, the surcharge (40kPa for T20 & 80kPa for T23) improves the load transfer
292 efficiency in comparison with the case where no surcharge is applied (T1). In fact, applying
293 an increased average pressure on the embankment top surface increases effective stress
294 significantly. As a result the arching effect, which is stress dependent, increases. It means that
295 the load transferred towards the piles increases when a hydraulic load is applied on the
296 embankment top. The case of a 3.2-m thick embankment where the intensity of q_0 has less
297 influence than for the case $H=1.8$ m is presented in Fig. 8 (b). The efficiency is increased only
298 for the case $q_0 = 80$ kPa and for displacements larger than 0.2m. For a thicker embankment,
299 the hydraulic stress causes a relatively lower increase in effective stress. Therefore, the same
300 hydraulic load has less influence on the efficiency for $H = 3.2$ m than for $H = 1.8$ m. With a
301 geosynthetic reinforcement, the efficiency is very similar for all cases (no surcharge, $q_0 =$
302 40kPa or 80kPa). The decreasing efficiency observed for test T27 and T30 is due to the
303 heterogeneity of the load sensors and provides an underestimated mean load inside the piles.
304 Only the tests without surcharge and with $H = 3.2$ m reveal a small difference. If we consider
305 that, for a GRPSE, the efficiency is due to both arching and membrane effects, applying a
306 stress does not modify the behaviour of the GR, and thus the efficiency value.

307 The interesting finding of this section is that the water tank by applying a homogeneous stress
308 increases arching significantly, particularly in the case of unreinforced piled embankments.
309 This increase is particularly noticeable in case of thin embankments because they have
310 relatively low stress levels due to their weight. In GRPSE, however, the membrane effect
311 almost entirely conceals the benefits of external loading.

312

313 4.4.2. *Nature of the surcharge*

314 Different loads can be applied on an embankment of constant thickness (here 1.8m). The aim
315 of this section is to compare the embankment behaviour for different model configurations
316 (i.e., different models corresponding to a hydraulic surcharge or a granular additional
317 thickness).

318 A thin embankment model is then considered (i.e., $H/H_{arch} = 1.4$ with $H = 1.8\text{m}$) and three
319 different configurations are adopted (Fig. 4):

- 320 (i) Thick embankments (7.2m) without surcharge,
- 321 (ii) Thin embankments (1.8m) supporting the weight of an additional 5.4m one separated
322 by a PP25 GR,
- 323 (iii) Thin embankments (1.8m) with some hydraulic surcharges applied via a soft
324 membrane.

325 All the configurations are tested for piled embankments without (T10, T17 and T20) and with
326 (T11, T19 and T21) geosynthetic basal reinforcement (PP25: $J_a = 4.0 \text{ MN/m}$). For instance,
327 T17 corresponds to an embankment with no basal GR and a PP60 geosynthetic layer ($J_a =$
328 16.8MN/m) placed at $h_{gx} = 1.8\text{m}$. T19 corresponds to a 7.2m thick embankment, within which
329 two PP25 geosynthetic layers ($J_a = 4 \text{ MN/m}$) are placed (one at basal level and the second at
330 $h_{gx} = 1.8\text{m}$) as schematically shown in Fig. 4b.

331 For the configurations without basal reinforcement (Fig. 9a), the load transfer increases with
332 the tray displacement and reaches approximately the same value (around 65%). We observe
333 that the surcharge applied using the water tank (test T20) produces a uniform pressure
334 whereas some arching effects appear within the granular thickness above the PP60
335 geosynthetic (test T17). For the T17 model, the GR is located at a height of 1.8m and presents
336 some higher efficiencies due to the membrane effect. The difference observed between the
337 T17 and T20 curves can be explained by the fact that the total embankment thickness is larger
338 resulting in some additional arching effect. This is also the case when the water surcharge
339 produces an increasing effective stress at the embankment top surface in the case of test T20.
340 As regards arching effect within a thick embankment, the granular thickness is more relevant
341 than the effective stress at its base.

342 The curves $E_F - \Delta\omega$ of the tests conducted with the basal GR are plotted in Fig. 9(b). The
343 efficiencies are very similar save for the case with a single embankment (T11), for which they
344 are lower. For the two other cases, the 1.8-m thick layer between the basal GR and the
345 membrane/second GR is stressed and may generate some higher load transfer efficiencies.

346 The difference observed between curve T11 and the other curves can be accounted for by test
347 variability.

348 We can then conclude that the load transfer efficiency is slightly higher for thicker
349 embankments because of arching/shearing effects occurring within the whole thickness and
350 not only at the bottom of the embankment (case without GR). As for GRPSE, on the other
351 hand, the impact of the surcharge load type on efficiency is negligible.

352

353 **5. NUMERICAL MODELLING**

354 Physical modelling is combined to numerical calculation to obtain the stress and deformation
355 distributions inside the embankment and on the GR.

356 **5.1. Definition of the numerical model**

357 A previous numerical model has been developed by Girout et al. (2014) using the finite element
358 software Plaxis for a two-dimensional (2D) model in an axisymmetrical unit-cell approach
359 (corresponding to one pile of a network situated far from the embankment slope). This study
360 has also demonstrated that a three-dimensional (3D) model is not required in this case because
361 no better fitting with the experimental results is obtained.

362 5.1.1. Geometry

363 The equivalent radius R_{eq} of the unit-cell is given by:

$$R_{eq} = \sqrt{S^2/\pi} \quad (3)$$

364 The mesh is built automatically with 15-nodes elements. Each model contains three
365 distinctive parts as shown in Fig. 10:

- 366 (i) the embankment, on which the surcharge load q_0 is applied,
- 367 (ii) the pile,
- 368 (iii) the tray, at the bottom of which the downward vertical displacement $\Delta\omega$ is applied.

369 Pile/tray, GR/tray and pile/embankment interactions are modelled using some interfaces. The
370 interface on top of the GR is associated to a material whose characteristics are equal to that of
371 the embankment. The other interface characteristics are found in Girout et al. (2014). The
372 geosynthetic reinforcement is modeled as a beam working under tension only. The beam mean
373 secant stiffness is chosen to model the PP25 GR ($J_a^{(p)} = 4000\text{kN/m}$).

374 5.1.2. Constitutive models

375 In order to model the embankment behaviour, a hypoplastic model is chosen. This hypoplastic
376 model, indeed, takes the change in the embankment void ratio during the tray downward
377 displacement into account, as showed by Girout et al. (2014). The pile and the tray are modelled
378 considering an elastic law with an infinite stiffness in contrast to the embankment material
379 stiffness.

380 5.1.3. Process

381 Numerical modelling follows the same process as that for centrifuge testing. The first step
382 consists in numerically applying macro-gravity by increasing earth standard gravity 20 times

383 ($N = 20$). Then, the surcharge load is applied. Finally, the tray downward displacement is
384 initiated with a 1-mm step until the GR comes loose from the tray. Additional information is
385 found in Girout et al. (2014).

386 **5.2.Results**

387 This study focuses on stress and displacement/deformation distributions within embankments.
388 It is also interesting in pointing up GR displacements and tensile loads, which is impossible
389 with the centrifuge at the scale used ($N = 20$). The authors have chosen here to discuss the
390 results for the GRPSE case. Two reinforcement layers are considered (one basal and the second
391 at $h_{gx} = 0.2\text{m}$) within a thin embankment ($H = 1.0\text{m}$), on which a hydraulic surcharge ($q_0 =$
392 80kPa) is applied and with the same spacing that the centrifuged tests ($s = 2.0\text{m}$). This
393 configuration is compared with a previous case used as the reference case by Girout et al. (2014)
394 in order to investigate how the second GR behaves within the embankment. These
395 configurations have also been experimentally tested using the geotechnical centrifuge and are
396 listed in Table 4 as TA (one GR) and TB (two GR).

397 Fig. 11 presents the load transfer efficiency for both configurations. The numerical and
398 experimental efficiencies obtained are quite similar, or even lower for the case with two GR.
399 This trend is also observed with the numerical calculation. As observed previously by the
400 authors, the total displacement requires for the GR to come loose from the tray (i.e., $\Delta\omega = \delta_g$)
401 is numerically underestimated.

402 Fig. 12 shows the distribution of four parameters within the embankment at their final state
403 (i.e., $\Delta\omega$ higher than δ_g): the vertical stress σ_{zz} (Fig. 12a), the vertical displacement U_{zz} (Fig.
404 12b), the shear stress σ_{rz} (Fig. 12c) and the void ratio e (Fig. 12d). Before the beginning of the
405 tray displacement, the initial vertical stress σ_{ss} is the sum of the embankment column weight
406 plus the hydraulic load, i.e., 95kPa . After the downward displacement, three distinctive areas
407 for the σ_{zz} distribution are noticeable (Fig. 12a): 1) above the pile head (σ_{zz} higher than 600 kPa),
408 2) above the center between the piles (σ_{zz} lower than 50 kPa) and 3) a transitional area (σ_{zz} value
409 decreasing in intensity but increasing in volume). The σ_{zz} isovalues of the two GR case are
410 similar to the single GR case presented by Girout et al. (2014). Higher stresses are applied on
411 the pile and part of the GR. The installation of the GR increases pile head loading by membrane
412 effect. The embankment column in the half-space between both piles is under-stressed. During
413 the downward displacement, the granular material layer between both GR is compressed,
414 especially above the pile (which is much stiffer than the GR). U_{zz} (Fig. 12b) makes it possible

415 to locate the second GR layer location with a small lateral displacement of the iso-value at h_{gx}
416 = 0.2m. σ_{rz} (Fig. 12c) shows a similar trend than in the single basal GR case, except that some
417 higher values are obtained above the pile edge because of the stress concentration below the
418 top GR layer. The void ratio distribution (Fig. 12d) also reveals a fragmented shear band
419 (characterized by lower void ratio) because of the presence of the second GR layer.
420 Nevertheless, the other bands occurring above the embankment suggest that arching is well
421 developed.

422 The tensile load T and the vertical displacement U_{zzg} in the GR are displayed in Fig. 13.
423 Considering TB (the 2 GR case), T reaches its maximum value at the pile edge (as shown in
424 Fig. 13a) and decreases until the equivalent radius, R_{eq} . In Fig. 13b U_{zz} is nil above the pile and
425 increases until reaching a maximum value δ_g at $r = R_{eq}$. The basal GR layer shows that the
426 behaviour is very similar to TA (single basal GR). The only difference is a peak at 220kN/m
427 (instead of 230kN/m in the single basal GR case). The second GR ($h_{gx} = 0.2m$) has a tensile
428 load equal to 65kN/m, which is maximum at the pile center. This value is much lower than the
429 value obtained in the basal GR case. The same observation is made *in situ* as, for example, in
430 Briançon and Simon (2012). The tensile load is reduced when the spacing r exceeds 0.25m (i.e.,
431 the pile radius, R_{eq}) reaching its minimum value at the center.

432 The vertical displacement of the basal GR (Fig. 13 b) is no different from the single GR
433 case except above the pile. Above the pile, indeed, the second GR descends vertically some
434 0.02m because of the densification of the granular material. Above the tray, the second GR
435 behaves like the basal GR. The second GR also introduces a membrane effect, whose impact
436 on the tests, however, is negligible.

437 **6. CONCLUSIONS**

438 The present study has been conducted to examine the behaviour of a piled embankment system
439 implemented with and without basal geosynthetic reinforcement. 33 tests have been performed
440 using a geotechnical centrifuge to determine the influence of the embankment thickness, the
441 geosynthetic altitude within the embankment, the secant stiffness of the geosynthetic layers and
442 the surcharge applied on the surface.

443 Different findings are obtained:

444 (i) The load transferred towards the piles increases with the embankment thickness, in the
445 same way as the load applied on the soft soil. For a 7.2- m high embankment and an
446 area replacement ratio of 4.91%, the maximum load transfer efficiency is equal to 65%.
447 Some arching effects appear within the embankment, the extent of which depends on
448 the prescribed tray displacement.

449 (ii) The presence of geosynthetic reinforcements enhances the efficiency of load transfer
450 towards the piles. The membrane effect is more noticeable as the prescribed tray
451 displacement increases. For the area replacement ratio studied here ($\alpha = 4.91 \%$) and
452 because of the prescribed settlement of the soft soil, the efficiency reaches 100%. The
453 stiffer the geosynthetic, the lower the deflection.

454 (iii) The part of the load transferred towards the piles depends on the stiffness and position
455 of the GR within the embankment. A stiff and basal GR generates higher transfers for
456 low soft soil settlements because of maximum membrane and arching effects. Two
457 geosynthetic reinforcement layers are used for the same model geometry. The total
458 vertical displacement of the geosynthetic decreases when its secant stiffness increases.
459 The membrane effect is at the maximum when the geosynthetic layer is located close to
460 the pile top. In order to reduce plastic strain within the geosynthetic, a thin granular
461 layer can be laid under the geosynthetic, which is no thicker than 0.2m. According to
462 the test results, the efficiency is similar for very high soft soil settlement with the
463 geosynthetic layer no thicker than 0.2m.

464 (iv) Considering the results available (embankment thicknesses of 1.8m and 3.2m,
465 respectively, area replacement ratio of 4.91%, surcharge of 40 and 80 kPa), applying a
466 temporary surcharge results in some quite different load transfer efficiencies for thinner
467 embankments. The main finding here is that the water tank exerts a homogeneous stress.
468 Consequently, arching, which is stress dependent, is significantly increased and
469 improves the efficiency. Improvement is limited in the case of thick embankments

470 because of a relatively lower increase in the effective stress. GRPSE is largely
471 unaffected because of an additional load transfer at the embankment bottom due to the
472 membrane effect.

473 (v) Considering the same average stress applied at the bottom of pile-supported
474 embankments, the key feature as regards load transfer is arching, which can be improved
475 by applying an external surcharge. As for GRPSE, arching and external surcharge have
476 similar effects.

477 (vi) Numerical modelling demonstrates that the addition of a second GR within
478 embankments does not enhance GR deflection reduction. Although the presence of a
479 second GR layer breaks the arching mechanisms at the bottom, it, however, does not
480 stop these mechanisms in the embankment areas situated above this GR and even
481 enhances load transfer through its membrane effect.

482 **NOTATION**

483 Basic SI units are shown in parentheses.

- 484 A Load transferred towards the pile by arching effect (N)
- 485 B Load transferred towards the pile by membrane effect (N)
- 486 C_U Uniformity coefficient
- 487 C_C Coefficient of gradation
- 488 d Pile diameter (m)
- 489 d_{50} Diameter of 50% passing
- 490 E_F Load transfer efficiency defined in Equation (1) (%)
- 491 e Void ratio of the embankment material
- 492 F_m Mean load transferred toward the rigid pile network (N)
- 493 g Standard earth acceleration ($m.s^{-2}$)
- 494 H Embankment thickness (m)
- 495 H_{arch} Radius of the arching effect for the analytical calculation defined in Equation (2) (m)
- 496 h_{gx} Altitude of the geosynthetic layer inside the embankment (m)
- 497 J_a Average Secant stiffness for the geosynthetic reinforcement (N/m)
- 498 q_0 Surcharge applied via the water tank (Pa)
- 499 N Scale factor
- 500 R_{eq} Equivalent radius in numerical modelling (m) defined in Equation (3)
- 501 r Radial spacing from pile axis in axisymmetrical configuration (m)
- 502 s Spacing between pile-axis (m)
- 503 $T_{(max)}$ (Maximum) tensile load of the geosynthetic (N/m)
- 504 U_{zz} Numerically obtained vertical displacement in the embankment (m)
- 505 U_{zzg} Numerically obtained vertical displacement in the geogrid (m)
- 506 z Height within embankment (m)
- 507 $X^{(m)/(p)}$ Parameter X given at model/prototype scale
- 508 α Area replacement ratio of the pile network (%)
- 509 ε_{gmax} Geosynthetic deformation at T_{max} (%)
- 510 $\Delta\omega_{(max)}$ (Maximum) Tray displacement in its center (m)
- 511 δ_g Deflection of the geosynthetic layer (m)
- 512 ϕ_p Friction angle of the Hostun sand mix ($^\circ$)
- 513 ρ_d Embankment density (kg/m^3)
- 514 ρ_s Volumic mass of solids (kg/m^3)

515 σ_{zz} Vertical stress (Pa)
516 σ_{rz} Shear stress (Pa)
517 γ_d Embankment unit weight (N/m³)
518

519 **ABREVIATION**

520 GR Geosynthetic Reinforcement

521 GRPSE Geosynthetic-Reinforced Pile-Supported Embankment

522 MT Mobile Tray

523 PP25 Polypropylene Woven Geotextile with $T = 25$ kN/m

524 PP60 Polypropylene Woven Geotextile with 62 kN/m $< T < 66$ kN/m

525

- 527 Bhasi, A. & Rajagopal, K. (2014). Geosynthetic-Reinforced Piled Embankments: Comparison
528 of Numerical and Analytical Methods. *International Journal of Geomechanics*, 15(5):1-
529 12.
- 530 Blanc, M., Rault, G., Thorel, L. & Almeida, M.S.S. (2013). Centrifuge investigation of load
531 transfer mechanisms in a granular mattress above a rigid inclusions network. *Geotextiles
532 and Geomembranes*, 36 (0), 92–105.
- 533 Blanc, M., Thorel, L., Girout, R. & Almeida, M. (2014). Geosynthetic reinforcement of a
534 granular load transfer platform above rigid inclusions: comparison between centrifuge
535 testing and analytical modelling. *Geosynthetics International*, 21 (1), 37–52.
- 536 Briançon, L. & Simon, B. (2012). Performance of Pile-Supported Embankment over Soft Soil:
537 Full-Scale Experiment. *Journal of Geotechnical and Geoenvironmental Engineering* 138
538 (4), 551–561.
- 539 Briançon L. & Dias D. (2015). Monitoring and numerical investigation of rigid inclusions
540 reinforced industrial building, *Canadian Geotechnical Journal*. 52 (10), 1592-1604
- 541 Briançon, L. & B. Simon. (2017) “Pile-Supported Embankment over Soft Soil for a High-Speed
542 Line.” *Geosynthetics International*, 1–13.
- 543 Chen, R. P., Wang, Y. W. , Ye, X. W., Bian, X. C. Dong, X. P. (2016). Tensile Force of
544 Geogrids Embedded in Pile-Supported Reinforced Embankment: A Full-Scale
545 Experimental Study. *Geotextiles and Geomembranes*, 44 (2): 157–169.
- 546 EBGEO (2011). Recommendations for Design and Analysis of Earth Structures using
547 Geosynthetic Reinforcements - *EBGEO, Deutsche Gesellschaft fr Geotechnik e V. /*
548 *German Geotechnical*. John Wiley & Sons. Berlin, Germany, 2011, 361p
- 549 Ellis, E. & Aslam, R., (2009). Arching in piled embankments: comparison of centrifuge tests
550 and predictive methods - part 2 of 2. *Ground Engineering*, 42 (0):34–38.
- 551 Eskişar, T., Otani, J. & Hironaka, J. (2012). Visualization of soil arching on reinforced
552 embankment with rigid pile foundation using X-ray CT. *Geotextiles and Geomembranes*,
553 32: 44–54.
- 554 Fagundes, D., Almeida, M.S.S., Girout, R., Blanc, M. &Thorel, L. (2015). Behaviour of piled
555 embankment without reinforcement’’. *Proceedings of the Institution of Civil Engineers -*
556 *Geotechnical Engineering*, 168(6): 1-12.

557 Fagundes D., Almeida M., Thorel L., Blanc M. (2017) Load transfer mechanism and
558 deformation of reinforced piled embankments. *Geotextiles and Geomembranes* 45 (2):1-
559 10,

560 Garnier, J., Gaudin, C., Springman, S.M., Culligan, P.J., Goodings, D.J., Konig, D., Kutter,
561 B.L., Phillips, R., Randolph, M.F. & Thorel, L. (2007). Catalogue of scaling laws and
562 similitude questions in geotechnical centrifuge modelling. *International Journal of*
563 *Physical Modelling in Geotechnics*, 17(3): 1–24.

564 Girout, R., Blanc, M., Thorel, L, Fagundes, D. F. & Almeida, M. (2016). Arching and
565 deformation in a piled embankment: centrifuge tests compared to analytical calculations.
566 *Journal of Geotechnical and Geoenvironmental Engineering*, 142 (12):1-10.

567 Girout, R., (2014). Prise en compte des géosynthétiques dans les mécanismes de transfert de
568 charge d'un matelas granulaire renforcés par inclusions rigides: Modélisation physique et
569 numérique. Thèse de doctorat, Ecole Centrale de Nantes, 300p (in French)

570 Girout, R., Blanc, M., Dias, D. & Thorel, L. (2014). Numerical analysis of a geosynthetic-
571 reinforced piled load transfer platform – Validation on centrifuge test. *Geotextiles and*
572 *Geomembranes* 42 (5): 525–539.

573 Han-Jiang, L., Zhen, J.-J., Zhang, J., Zhang, R.-J. & Cui, L. (2014). DEM analysis of soil -
574 arching within geogrid -reinforced and unreinforced pile-supported embankments.
575 *Computer and Geotechnics*, 61(0): 13–23.

576 Hewlett, W. & Randolph, M.A. (1988). Analysis of piled embankments. *Ground Engineering*
577 (21): 12–18.

578 IREX (2012). Recommendations for the Design Construction and Control of Rigid Inclusion
579 Ground Improvements. ASIRi, Presses Des Ponts. Paris. 384p.

580 Jenck O., Dias, D. & Kastner R. (2005). Soft Ground improvement by vertical rigid piles-Two-
581 dimensional physical modelling and comparison with current design methods. *Soils and*
582 *Foundation*, 45 (6): 15-31.

583 Jenck O., Dias D. & Kastner R. (2007). Two-dimensional physical and numerical modelling of
584 a pile supported earth platform over soft soil, *Journal of Geotechnical and*
585 *Geoenvironmental Engineering*, ASCE, 133 (3) :295-305.

586 King, D. J., Bouazza, A., Gniel, J.R., Rowe, K. R. & Bui, H. H.. (2017). Serviceability Design
587 for Geosynthetic Reinforced Column Supported Embankments. *Geotextiles and*
588 *Geomembranes*:1-19.

- 589 Le Hello, B. (2007). Renforcement par géosynthétiques des remblais sur inclusions rigides,
590 étude expérimentale en vraie grandeur et analyse numérique. Thèse de doctorat,
591 l'université Grenoble I, 234p (in French).
- 592 Le Hello, B. & Villard, P. (2009). Embankments reinforced by piles and geosynthetics –
593 Numerical and experimental studies with the transfer of load on the soil embankment.
594 *Engineering Geology*, 106: 78 – 91.
- 595 Nunez, M., Briançon, L. & Dias, D. (2013). Analyses of a pile-supported embankment over
596 soft clay: Full-scale experiment, analytical and numerical approaches, *Engineering*
597 *Geology*.153: 53-67
- 598 Okyay, U., Dias, D., Thorel, L. & Rault, G. (2014). Centrifuge Modelling of a Pile-Supported
599 Granular Earth-Platform. *Journal of Geotechnical and Geoenvironmental Engineering*,
600 140 (2): 1-12.
- 601 Okyay, U. (2010). Etude expérimentale et numérique des transferts de charge dans un massif
602 renforcé par inclusions rigides. Applications à des cas de chargements statiques et
603 dynamiques. Thèse de Docteur ès Sciences, Institut National des Sciences Appliquées,
604 Univ. of Lyon, Lyon, France 402 p. (in French)
- 605 Rault, G., Thorel, L., Néel, A., Buttigieg, S., Derkx, F., Six, G., & Okyay, U. (2010). Mobile
606 Tray for Simulation of 3D Load Transfer in Pile-Supported Earth Platforms. Laue,
607 Swards, (Editor) *Physical modelling in Geotechnics*. Vol 2. Proceedings of a symposium
608 held in Zurich, Switzerland, 28th June- 1st July 2010, pp 261-266.
- 609 Van Eekelen, S.J.M., Bezuijen, A. & van Tol, A.F. (2015). Validation of analytical models for
610 the design of basal reinforced piled embankments. *Geotextiles and Geomembranes*, 43 (1):
611 56-81
- 612 Van Eekelen, S. J. M., Bezuijen, A., Lodder, H. J. & van Tol., A. F. (2012a). Model
613 Experiments on Piled Embankments. Part I. *Geotextiles and Geomembranes* 32: 69–81.
- 614 Van Eekelen, S. J. M., Bezuijen, A., Lodder, H. J. & van Tol., A. F. (2012b). Model
615 Experiments on Piled Embankments. Part II. *Geotextiles and Geomembranes* 32: 82–94.
- 616 Villard, P. & Giraud, H. (1998). Three-Dimensional Modelling of the Behaviour of Geotextile
617 Sheets as Membranes. *Textile Research Journal* 68, 11: 797–806.
- 618 Xing, H., Zhang, Z., Liu, H. & Wei, H. (2014). Large-scale tests of pile-supported earth
619 platform with and without geogrid. *Geotextiles and Geomembranes*, 42 (6): 586–598.

- 620 Zhuang, Y. & Wang, K.Y. (2015). Three-Dimensional Behaviour of Biaxial Geogrid in a Piled
621 Embankment: Numerical Investigation. *Canadian Geotechnical Journal* 52 (10):1629-
622 1635.
- 623 Zhuang, Y. & Wang, K.Y. (2016). Finite-Element Analysis on the Effect of Subsoil in
624 Reinforced Piled Embankments and Comparison with Theoretical Method Predictions.
625 *International Journal of Geomechanics*, 16 (5): 1–15.
- 626 Zhuang, Y., & Ellis, E. A. (2016). Finite-Element Analysis of a Piled Embankment with
627 Reinforcement and Subsoil. *Géotechnique* 66 (7): 596–601.
- 628
- 629

630	Table 1 Scaling factors	30
631	Table 2 Characteristics of the mix Hostun sand ($\gamma_d = 15.8 \text{ kN/m}^3$ and $\phi_p = 38^\circ$)	31
632	Table 3 Properties of the geosynthetic reinforcement on prototype scale	32
633	Table 4 List of the experimental test campaign ($s = 2.0 \text{ m}$, $d = 0.5 \text{ m}$, $H_{arch} = 1.2\text{m}$ (as defined in	
634	Equation (2), $\alpha = 4.91 \%$, $\rho_d = 1.62\text{kg/m}^3$)	33
635		
636	Fig. 1 (a) General configuration of a piled embankment reinforced with one/two geosynthetic	
637	layers (b) Mobilisation of the geosynthetic strength during soft soil settlement	34
638	Fig. 2 Mobile Tray Device (a) Surface of the tray and square pile network with a pile diameter	
639	equal to 25 mm (b) Mobile Tray with model embankment in the centrifuge swinging basket	35
640	Fig. 3 Geosynthetic layer after testing: localization of the geosynthetic deformations ($h_{gx} = 0\text{m}$	
641	and $H^{(p)} = 7.2\text{m}$)	36
642	Fig. 4 Geometry of the models in the swinging basket of the centrifuge (a) Single embankment	
643	(b) Embankment reinforced at $z = H - h_{gx}$ and (c) (Dry) embankment loaded by means of	
644	hydraulic surcharge	37
645	Fig. 5 Efficiency of the load transfer (for $H = 1.8 \text{ m}$, $H = 3.2 \text{ m}$, $H = 5.0\text{m}$ and $H = 7.2 \text{ m}$): (a)	
646	with and without geosynthetic ($J_a = 4\text{MN/m}$) (b) Variations of the efficiency versus H for	
647	different $\Delta\omega$	38
648	Fig. 6 Tray displacement for the maximum efficiency reached (non-reinforced case) and	
649	deflection δ_g of both geosynthetic layers ($J_a = 4.0 \text{ MN/m}$ and 16.8 MN/m) versus thickness H ...	39
650	Fig. 7 Influence of the GR for different elevations (a) Efficiency versus tray displacement (b)	
651	Efficiency versus adimensionalized geosynthetic elevation	40
652	Fig. 8 Influence of the surcharge ($q_\theta = 0$; 40 and 80 kPa): testing with and without geosynthetic	
653	(a) $H = 1.8 \text{ m}$ (b) $H = 3.2 \text{ m}$	41
654	Fig. 9 Comparison of the different types of surcharge applied above the geosynthetic: (a)	
655	Without basal reinforcement and (b) With basal reinforcement ($h_{gx} = 0 \text{ m}$)	42
656	Fig. 10 Definition of the numerical model	43

657	Fig. 11 Load transfer efficiency for GRPSE with one and two GR layers (numerical and physical	
658	modelling)	44
659	Fig. 12 Finite element modelling of a GRPSE with two GR ($h_{gx} = 0$ and 0.2m with $J_a = 4.0\text{MN/m}$),	
660	$s = 2.0\text{m}$, $H = 1.0\text{m}$ and $q_0 = 80\text{ kPa}$.....	45
661	Fig. 13 Distributions for the single basal GR layer case and for the two GR case: (a) tensile load	
662	T and (b) vertical displacement U_{zz}	46
663		
664		

665 **TABLES**

666

Table 1 Scaling factors

	Prototype	Small-scale model
Force (N)	1	$1/N^2$
Stress (Pa)	1	1
Deformation (%)	1	1
Length (m)	1	$1/N$
Secant stiffness (N/m)	1	$1/N$

667

668

669

670

Table 2 Characteristics of the mix Hostun sand ($\gamma_d = 15.8 \text{ kN/m}^3$ and $\phi_p = 38^\circ$)

d_{50} (mm)	C_U	C_C	$\rho_{d \text{ min}}$ (g/cm ³)	$\rho_{d \text{ max}}$ (g/cm ³)	ρ_s (g/cm ³)
0.32	3.52	0.88	1.40	1.73	2.65

671

672

673

674

Table 3 Properties of the geosynthetic reinforcement on prototype scale

Geosynthetic	PP25		PP60	
	Cross	Machine	Cross	Machine
T_{max} (kN/m)	478	490	1320	1240
J_a (MN/m)	4.76	2.96	16.8	16.8
ε_{gmax} (%) for $T = T_{max}$	12.9	19.3	9	9

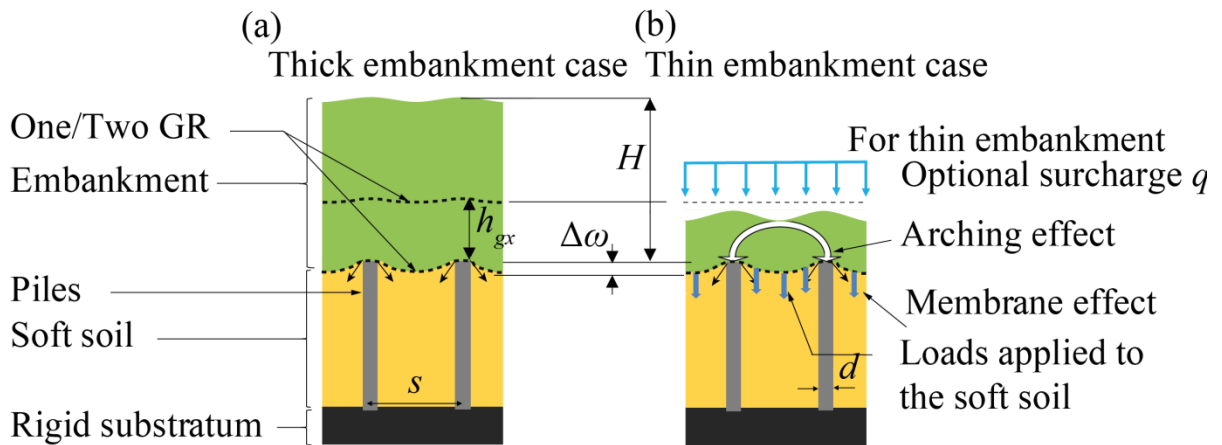
675

676 **Table 4 List of the experimental test campaign ($s = 2.0$ m, $d = 0.5$ m, $H_{arch} = 1.2$ m (as defined in**
 677 **Equation (2), $\alpha = 4.91$ %, $\rho_d = 1.62$ kg/m³)**

Tests (TX)	H (m)	H/H_{arch}	J_a (MN/m)	h_{gx} (m)	q_0 (kPa)
TA/TB	1.0	0.78	4	0/0;0.2	80
T1/T2/T3	1.8	1.5	-/4/16.8	0	0
T4/T5/T6	3.2	2.7	-/4/16.8	0	0
T7/T8/T9	5	4.3	-/4/16.8	0	0
T10/T11/T12	7.2	6.2	-/4/16.8	0	0
T13/T14/T15/T16/T17	7.2	6.2	16.8	0.2/0.4/0.8/1.2/1.8	0
T18 /T19 ^(2 PP25 GR)	7.2	6.2	4	1.8 / 0;1.8	0
T20/T21/T22	1.8	1.5	-/4/16.8	0	80
T23/T24/T25	1.8	1.5	-/4/16.8	0	40
T26/T27/T28	3.2	2.75	-/4/16.8	0	40
T29/T30/T31	3.2	2.75	-/4/16.8	0	80

678

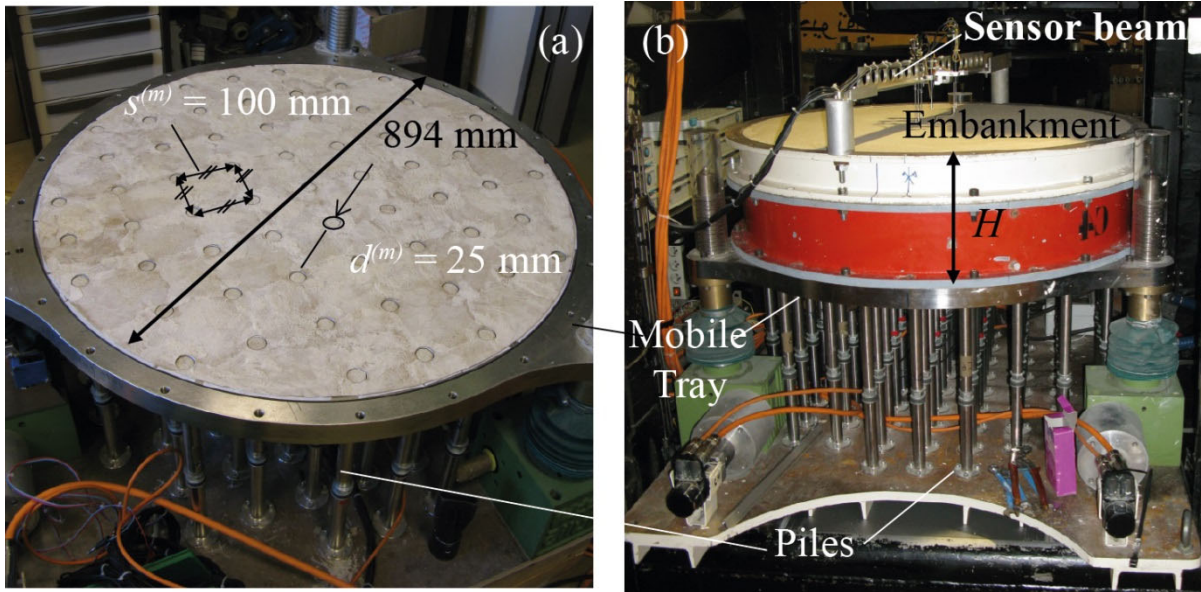
679



681

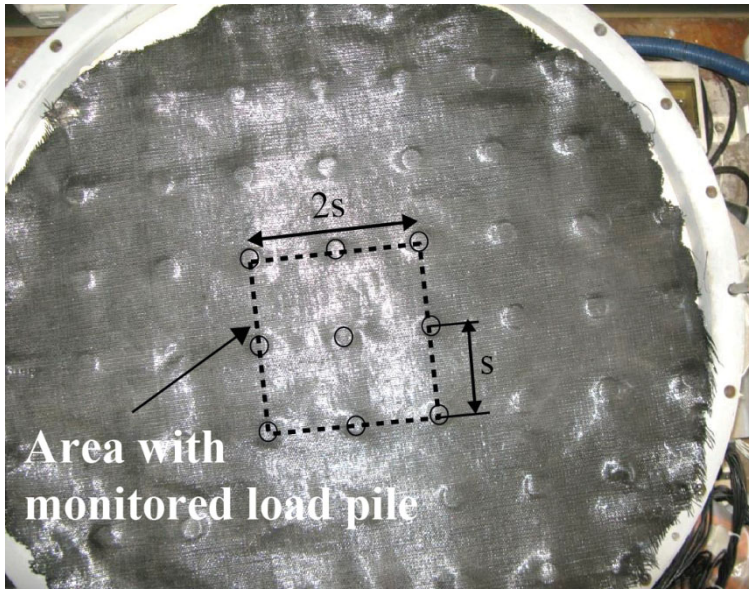
682 **Fig. 1 (a) General configuration of a piled embankment reinforced with one/two geosynthetic**
 683 **layers (b) Mobilisation of the geosynthetic strength during soft soil settlement**

684

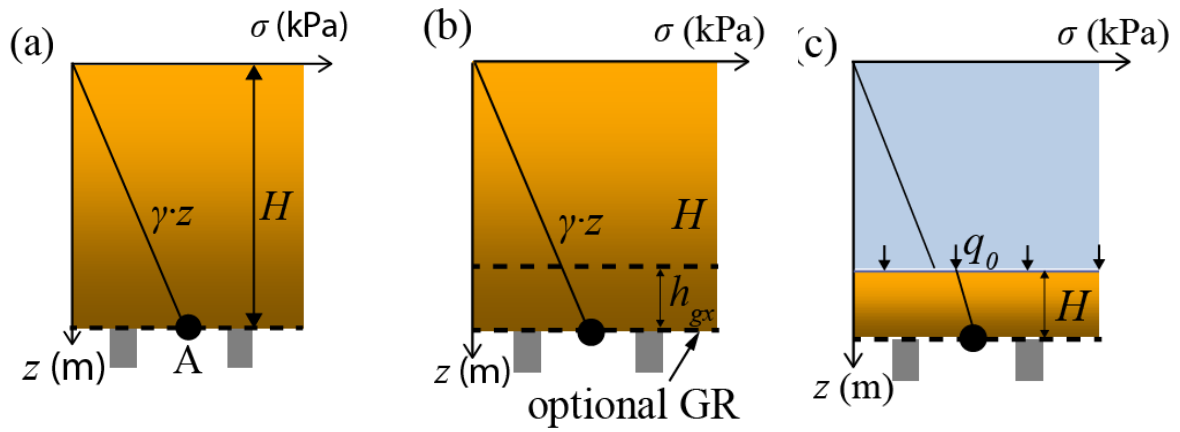


685 **Fig. 2 Mobile Tray Device (a) Surface of the tray and square pile network with a pile diameter**
 686 **equal to 25 mm (b) Mobile Tray with model embankment in the centrifuge swinging basket**

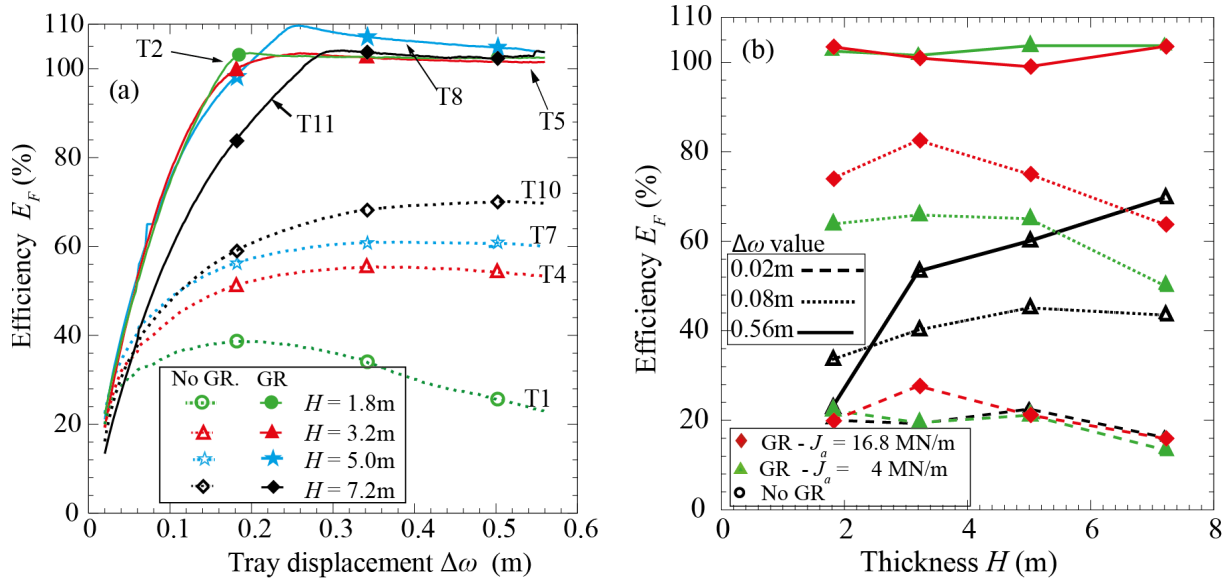
687
 688



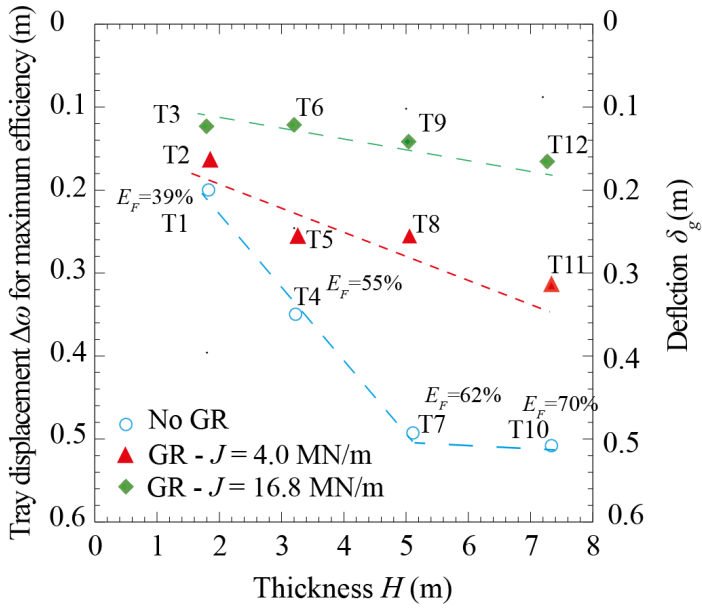
689 Fig. 3 Geosynthetic layer after testing: localization of the geosynthetic deformations ($h_{gx} = 0\text{m}$ and
690 $H^{(p)} = 7.2\text{m}$)
691



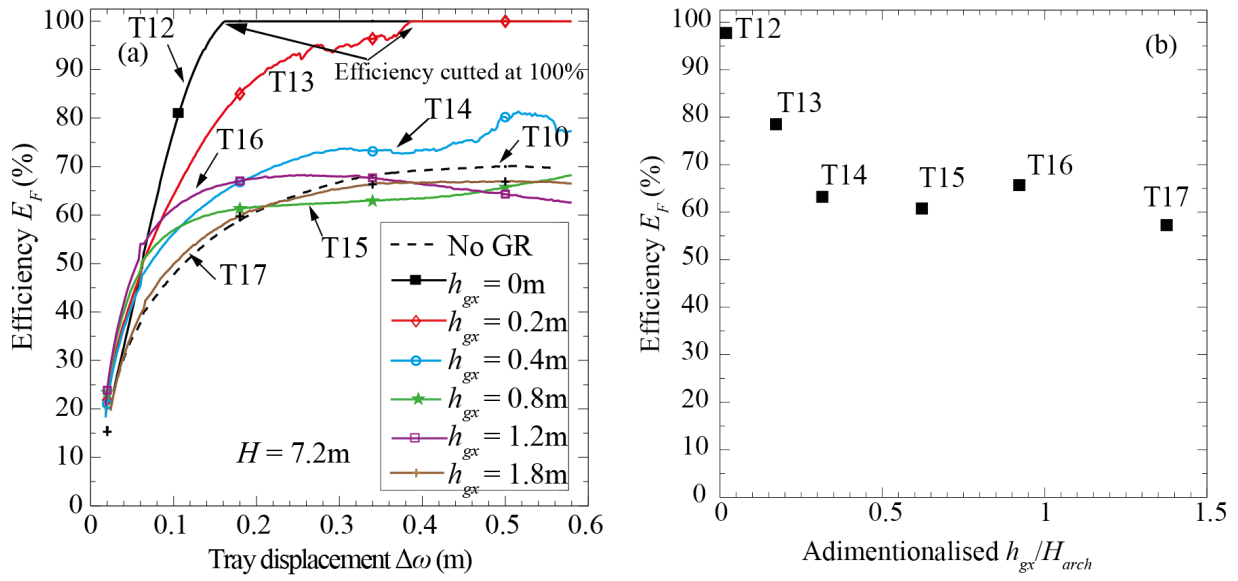
692 **Fig. 4 Geometry of the models in the swinging basket of the centrifuge (a) Single embankment**
 693 **(b) Embankment reinforced at $z = H - h_{gx}$ and (c) (Dry) embankment loaded by means of**
 694 **hydraulic surcharge**
 695



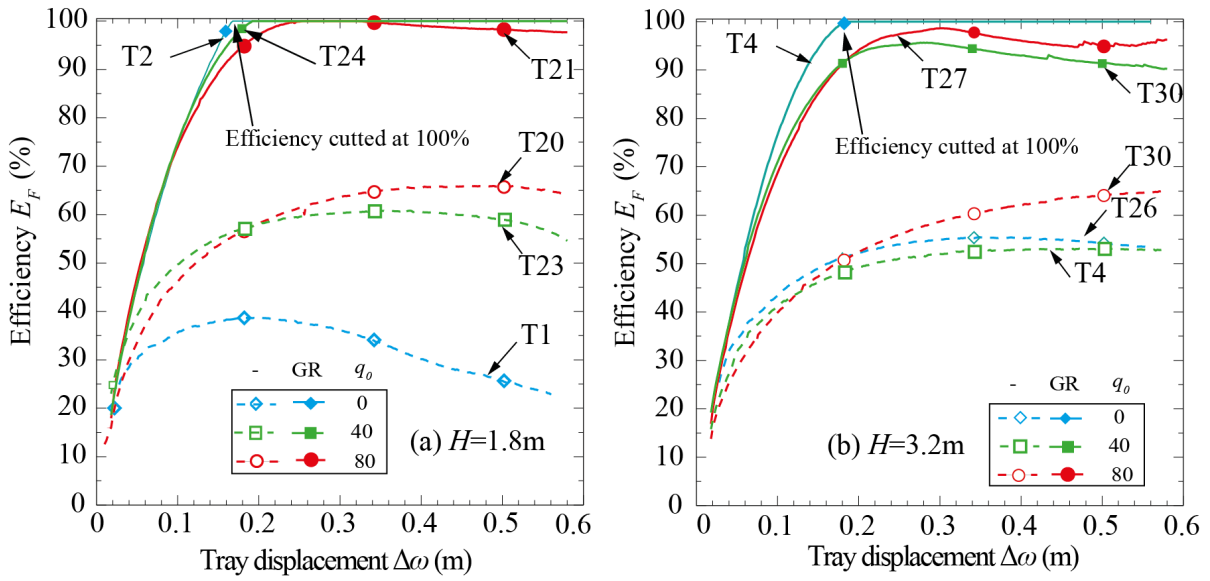
696 **Fig. 5 Efficiency of the load transfer (for $H = 1.8\text{ m}$, $H = 3.2\text{ m}$, $H = 5.0\text{ m}$ and $H = 7.2\text{ m}$): (a)**
 697 **with and without geosynthetic ($J_a = 4\text{ MN/m}$) (b) Variations of the efficiency versus H for**
 698 **different $\Delta\omega$**
 699



700 **Fig. 6 Tray displacement for the maximum efficiency reached (non-reinforced case) and**
 701 **deflection δ_g of both geosynthetic layers ($J_a = 4.0$ MN/m and 16.8 MN/m) versus thickness H**
 702

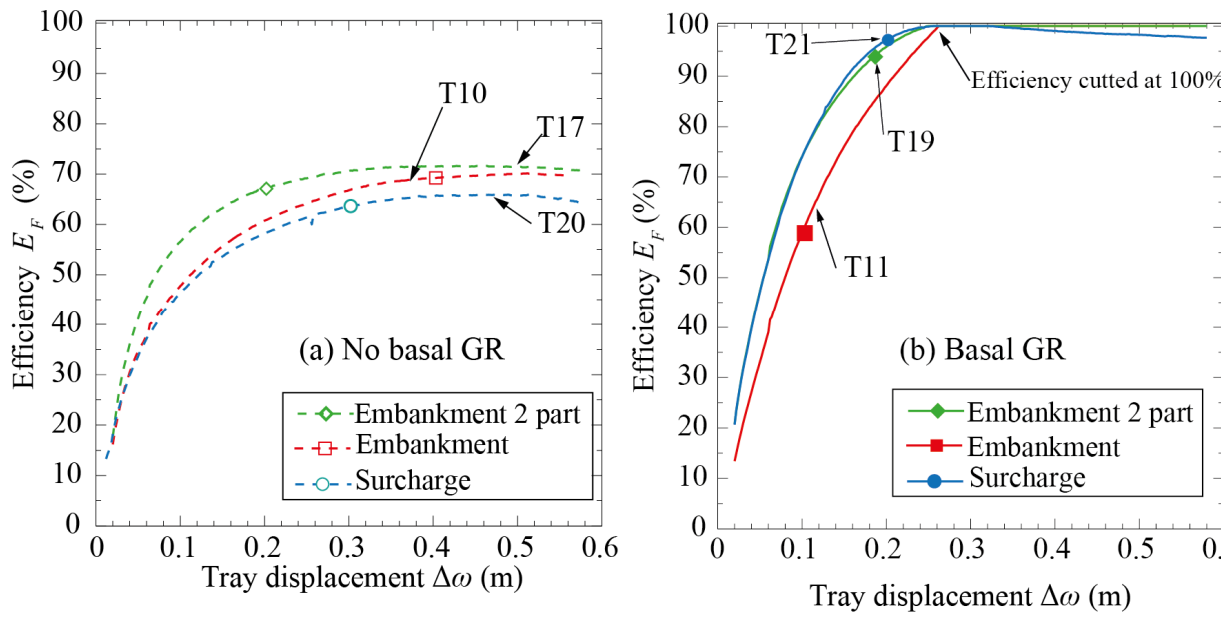


703 **Fig. 7 Influence of the GR for different elevations (a) Efficiency versus tray displacement (b)**
 704 **Efficiency versus adimensionalized geosynthetic elevation**
 705

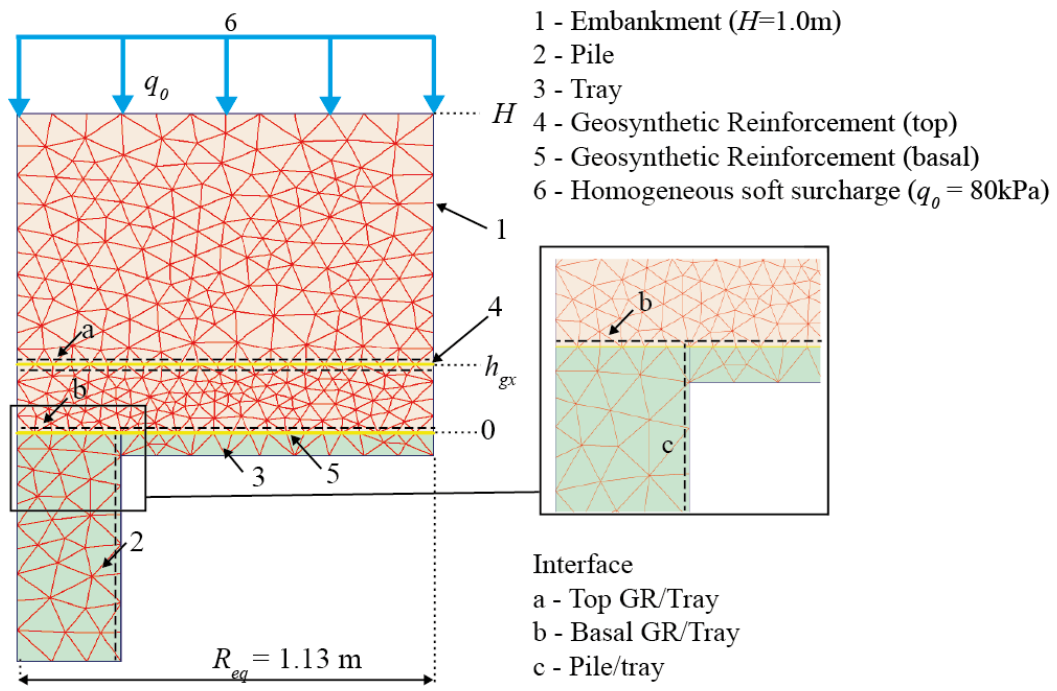


706 **Fig. 8 Influence of the surcharge ($q_0 = 0; 40$ and 80 kPa): testing with and without geosynthetic**
 707 **(a) $H = 1.8$ m (b) $H = 3.2$ m**

708
 709



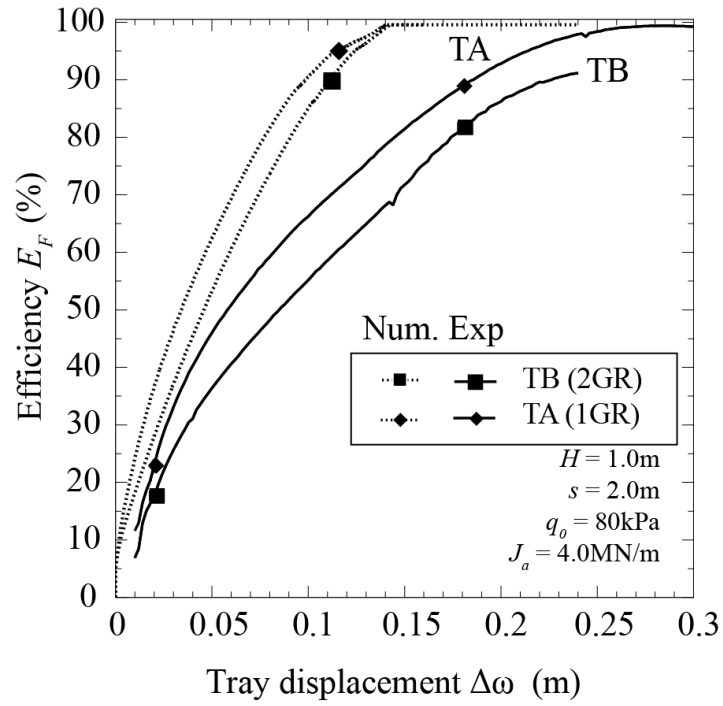
710 **Fig. 9 Comparison of the different types of surcharge applied above the geosynthetic: (a)**
 711 **Without basal reinforcement and (b) With basal reinforcement ($h_{gx} = 0$ m)**
 712



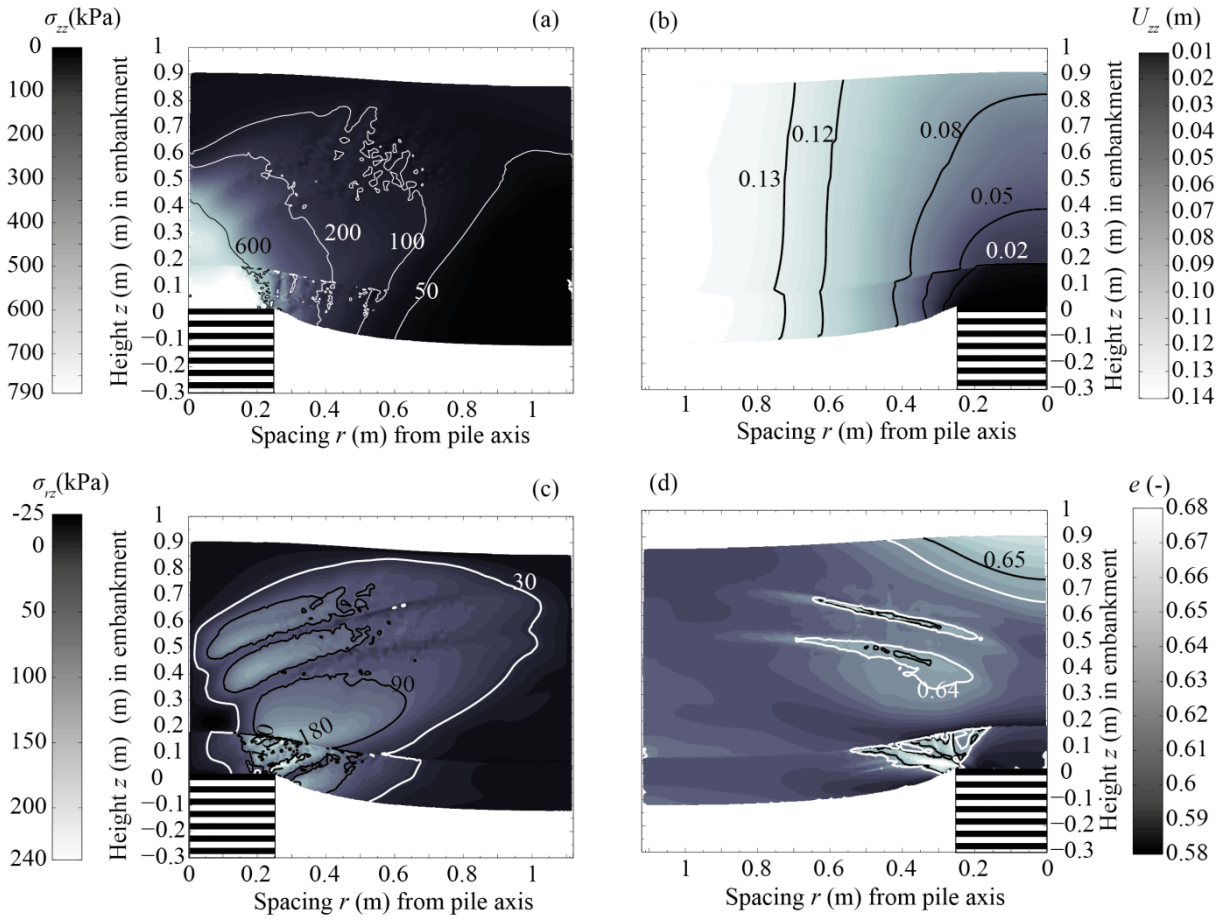
713

Fig. 10 Definition of the numerical model

714



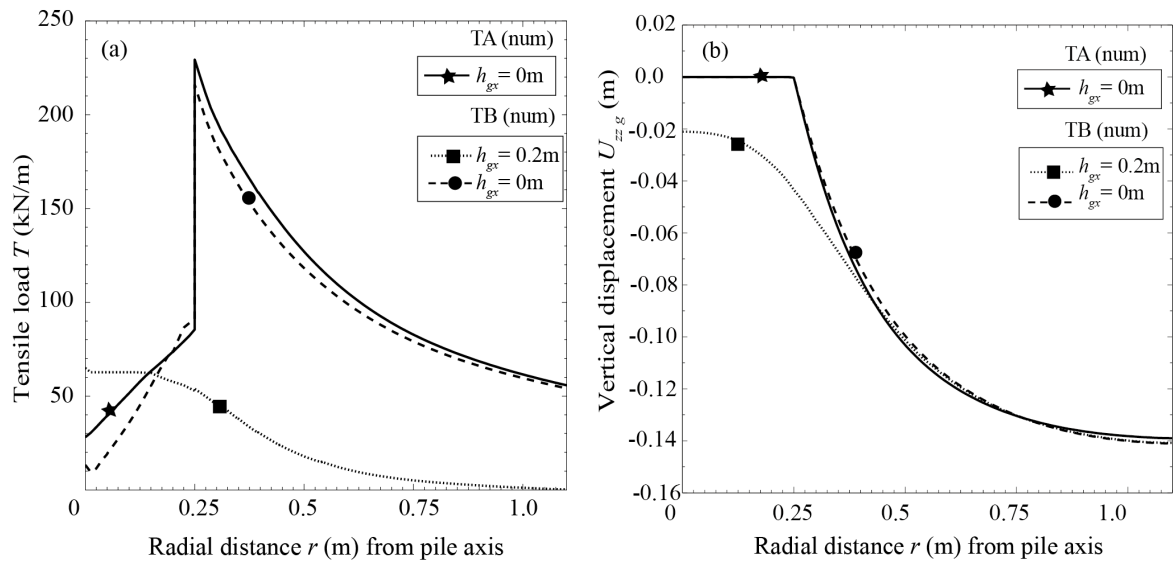
716 **Fig. 11 Load transfer efficiency for GRPSE with one and two GR layers (numerical and physical**
 717 **modelling)**



719 **Fig. 12** Finite element modelling of a GRPSE with two GR ($h_{gx} = 0$ and 0.2m with $J_a = 4.0\text{MN/m}$),
 720 $s = 2.0\text{m}$, $H = 1.0\text{m}$ and $q_0 = 80\text{ kPa}$

721

722



723 **Fig. 13 Distributions for the single basal GR layer case and for the two GR case: (a) tensile load**

724 **T and (b) vertical displacement $U_{zz,g}$**