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1 **Investigation on the undrained shear strength of loose sand**
2 **with added materials at various mean diameter ratios**

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19

20 **Abstract**

21 This study aims to investigate the undrained shear strength of sand with added materials at different
22 mean diameter ratios D_{50}/d_{50} (D_{50} and d_{50} respectively represent the mean diameter of host sand and
23 added materials) in loose state by performing a series of undrained triaxial tests. For this purpose, a
24 very low density index of sand matrix I_{Dmat} equaling to 0.00 was employed and the fines of C500 silt,
25 Illite and Speswhite with distinct mean grain sizes were respectively added into the host HN31 sand at
26 different contents F_c (5%, 10% and 15%). For comparison and further analysis, undrained triaxial tests
27 were also carried out on Sand-Sand mixtures, created by adding the HN31 sand into the host HN31
28 sand at F_c of 5%, 10%, 14% and 18.7%. Results showed that, for low value D_{50}/d_{50} (e.g., 1 for Sand-
29 Sand), the undrained shear strength S_u increased as F_c increased, which might be attributed to the
30 increase of occlusal friction, and in that case the sliding of host sand became difficult. By contrast, in
31 the case of higher value D_{50}/d_{50} (e.g., 58.33 for Sand-C500, 159.09 for Sand-Illite and 500 for Sand-
32 Speswhite), S_u decreased as F_c increased. This was explained by the fact that, with increase of F_c , the
33 quantity of added materials with small size increased, and in that case the sliding of host sand became
34 easy. A critical value of D_{50}/d_{50} (14.5) was thus deduced, which theoretically verified the negligible
35 effect of F_c on S_u . Furthermore, for sand mixed with fines (e.g., C500 silt, Illite and Speswhite clays),
36 F_c has an insignificant effect on the peak value of Δu_{max} , while the increase of D_{50}/d_{50} facilitated the
37 generation of excess pore water pressure after reaching about Δu_{max} . These findings provided useful
38 information for analyzing the mechanical behaviour of sand with added materials in geotechnical
39 practice.

40 **Keywords:** Loose sand; Added materials; Undrained shear strength; Mean diameter ratio

41

42 **1. Introduction**

43 There has been abundant evidence that the undrained shear strength of loose sandy soil is one of the
44 key issues when subjected to seismic loading. Due to the rapid build-up of excess pore water pressure,
45 loose sandy soil could undergo a transient loss of shear strength and behave as a liquid, resulting in
46 catastrophic consequences to geotechnical structures and foundations. A sound understanding of clean
47 sand has been already acquired since two destructive earthquakes occurred in United States and Japan
48 in 1964: the Alaska earthquake [1] and the Niigata earthquake [2]. However, a large number of in-situ
49 investigations proved that sand layer including low-percentage fines was also sensitive to liquefaction
50 when suffering from the earthquake [3–7]. With this concern, it is of cardinal importance to study the
51 undrained shear strength of sand with small-size inclusion (e.g., fines particles as added materials).

52 In the past three decades, numerous experimental investigations related to the effect of small-
53 size inclusion on the undrained shear strength of loose sand have been reported [8-31]. However, no
54 clear consensus was achieved and diverse views still exist as to whether the effect of fines is adverse
55 or beneficial to undrained shear strength. In the published literature, with increasing fines content, the
56 undrained shear strength of sand-fines mixtures could (i) increase [8–12]; (ii) decline [13–17]; (iii)
57 firstly decrease until a certain threshold and then increase [18–22]. Efforts were made to explain the
58 above seemingly at least controversial phenomena from the following aspects of added materials:
59 plasticity index I_p , grain shape and gradation characteristic [23–26]. It was found that: (i) there was no
60 specific monotonic relationship between plasticity index I_p and the shear strength, as a threshold
61 plasticity index I_{pth} existed [23]; (ii) the sand-silt mixtures with rounded silt tended to exhibit lower
62 shear strength, as compared with those contained angular silt [24]. However, a reversed trend was
63 reported by Monkul [25] that the angular nature of silt made the sandy samples more prone to lose
64 shear strength; (iii) the gradation characteristic had a significant effect on the overall soil response that
65 the liquefaction resistance of silty sand generally decreased with increasing coefficient of uniformity
66 [26]. Recently, the average size of silt grains relative to that of host sand has been repeatedly proved to
67 be an important factor influencing the mechanical behaviour of sand-fine mixtures [27–31].
68 Unfortunately, these studies [27–31] only focused on the silt being as the added materials, and the

69 corresponding size range (5-75 μm) was comparatively narrow in most instances, rendering the
70 complete conclusion to be drawn difficult. To the authors' knowledge, the effect of average size of
71 added materials on the mechanical response of sandy samples remains an open issue, and further
72 investigation is still needed.

73 In this study, emphasis was put on the effect of mean diameter of added materials on the
74 undrained shear strength of host sand matrix by performing a series of undrained triaxial compression
75 test. For this purpose, three types of fines (C500 silt, Illite and Speswhite clays) with distinct mean
76 diameters were respectively added into the host HN31 sand at different contents in a very loose state.
77 For comparison and further analysis, the triaxial tests were also carried out on Sand-Sand mixtures,
78 created by adding the HN31 sand into the host HN31 sand at different contents. Furthermore, the
79 hydraulic conductivity tests were also directly determined in the triaxial apparatus, allowing relatively
80 satisfactory interpretation of the evolution of excess pore water pressure. From a practical point of
81 view, the identified mechanism in this study is also helpful when further analyzing the case in cyclic
82 triaxial tests for liquefaction resistance.

83

84 2. Materials and method

85 2.1. Physical properties of test materials

86 The host sand used in this study is a poorly graded Hostun 31 sand (HN31), which is characterized by
87 uniform sub-angular grains and extensively used in many laboratories in Europe. The added materials
88 were respectively HN31 sand (the same material as the host HN31 sand), C500 (a non-plastic fine silt
89 powder), Illite Arvel (a plastic clay of Illite mineralogical content) and Speswhite (a plastic clay of
90 Kaolinite-mineralogical content). Their physical properties are listed in Table 1. The specific gravity
91 (G_s) of the test materials is found to be the same, which is 2.65. For Illite Arvel and Speswhite, the
92 plasticity indexes were determined using the French Standard NF P 94-051, which were respectively
93 equal to 34 and 30. It is worth noting that, for C500 silt, it was difficult to obtain a standard roller
94 determining its plasticity, being reasonably considered as NP (Non-Plastic) fines. The grain size
95 distribution curve of non-plastic materials (e.g., HN31, C500 in Fig. 1) was measured by laser
96 diffraction method. A particle could scatter the light at a certain angle and intensity while passing
97 through a laser beam, allowing the determination of its grain size [24]. The grain size distribution
98 curve of plastic fines (e.g., Illite) was determined by settlement analysis method (see Fig. 1) since it
99 was more accurate for finer cohesive soil ($< 80 \mu\text{m}$). The principle of the method is to measure the
100 sedimentation rate of soil particles in a static liquid that related to its equivalent particles diameter.
101 Note that the curve of Speswhite was measured and checked by Muhammed [32]. It appears that, for
102 host HN31 sand, the mean diameter is $350 \mu\text{m}$, while for C500, Illte Arvel and Speswhite, the mean
103 diameters are $6 \mu\text{m}$, $2.2 \mu\text{m}$ and $0.7 \mu\text{m}$, respectively. These four added materials (HN31 sand, C500
104 silt, Illte Arvel and Speswhite clays) into the host HN31 sand create the mean diameter ratio (D_{50}/d_{50})
105 of 1, 58.33, 159.09 and 500, respectively. It is worth noting that the D_{50} refers to the mean diameter of
106 the host HN31 sand, while the d_{50} refers to the mean diameter of the added materials.

107 In most of the existing references, the constant global void ratio was considered (see Fig. 2a):
108 the addition of small grains (e.g., fines) systematically reduced the quantity of host sand to keep the
109 global void ratio unchanged, allowing the preparation of tested samples with high content of added

110 materials (e.g., fines). The constant global void ratio considered also led to the reduction of quantity of
111 the host sand, and made the overall response of soil sample change from the dominance of host sand to
112 that of added materials (e.g., fines) [33,34]. However, the density index of sand matrix [35] I_{Dmat} was
113 considered in this study (see Fig. 2b). The clear benefit was that for each series of undrained triaxial
114 compression test, the quantity of sand matrix was kept exactly unchanged, irrespective of the varying
115 content of added materials (e.g., fines). In that case, the skeleton of the sample was greatly sand-
116 prevailing but polluted by a small amount of added materials (e.g., fines), which is accord with the real
117 field situation [5,36]. A low density index of sand matrix I_{Dmat} equaling to 0.00 was employed ($I_{Dmat} =$
118 $(e_{max}-e_{mat})/(e_{max}-e_{min})$, where e_{mat} is the void ratio of sand matrix, e_{max} and e_{min} represent the maximum
119 and minimum void ratios of clean sand [37], respectively). The content of added materials F_c was thus
120 defined as the dry mass of added materials divided by that of sand matrix (m_{add}/m_{sand}). Three low-
121 percentage of $F_c = 5\%$, 10% and 15% were selected for the added materials of C500, Illite and
122 Speswhite, as well as four low-percentage of $F_c = 5\%$, 10% , 14% and 18.7% for the added materials of
123 HN31 sand, as the skeleton of the sample was dominated by the sand-sand contacts in that case
124 ([9,38,39]). For comparison, the total content of added materials F_c^T (the ratio of the mass of added
125 materials to the total mass of composite sandy mixture) are also given in Table 2.

126

127 2.2. Reconstitution method

128 As for reclaimed binary mixture, the major challenge in reconstitution step is the homogeneity of
129 composite soil mixture. First attempts had been made to apply moist tamping technique. However, a
130 non-uniform mixture was created through the observations of small aggregates since plastic fines (e.g.,
131 Speswhite) stuck together even with a small amount of water. To overcome this problem, dry tamping
132 technique was thus adopted. In this method, two materials were firstly mixed by shaking in an
133 enclosed container. Then, all mixtures were divided into 10 parts and each part was carefully
134 introduced into a split mould by a spoon (see Fig. 3a). The clear benefit of spooning was: (i) to reduce
135 the drop distance as much as possible to achieve such a loose state $I_{Dmat}=0.00$; (ii) to avoid initial
136 segregation between coarse and small-size grains during freefall. Finally, the sample with 10 cm in

137 diameter and 20 cm in height was created. During this process, the horizontality was checked by a
138 horizontal rule (see Fig. 3b).

139

140 2.3. Triaxial test program

141 The test program is summarized in Table 2. In test TM1, once the specimen had been formed, the top
142 triaxial cap was placed and sealed with two O-rings. A vacuum of about 20 kPa was applied to reduce
143 possible disturbances while removing the split mould, as well as to check the leakage of membrane.
144 After ensuring that the latex membrane was not defective, the confining pressure was increased to 20
145 kPa to maintain the sample before cancelling the inside vacuum and increased to 100 kPa for the
146 saturation stage. Carbon dioxide (CO₂) was then purged into the dry sample for about 15 mins under a
147 pressure of about 15 kPa to evacuate air bubbles, followed by injecting de-aired water to saturate the
148 sample. It is worth noting that the flux of de-aired water was minimized to avoid the possible
149 migration of added materials passing through the sample. Prior to applying back pressure, the falling
150 head permeability test was performed under a constant confining pressure of 100 kPa: an initial water
151 head (about 1 m height) was applied in the bottom drainage and the top base of the sample was kept at
152 atmospheric pressure. Based on the water head loss recorded at a fixed time, the hydraulic
153 conductivity k could be determined according to the variable water head method using Darcy's law.
154 Afterwards, the back pressure and confining pressure were increased alternatively in equal incremental
155 steps of 20 kPa until they were respectively equal to 200 kPa and 300 kPa. The Skempton's B value (B
156 $= \Delta u / \Delta \sigma_{\text{conf}}$, the variation of excess pore water pressure divided by that of confining pressure) was
157 checked, normally a value higher than 0.98 was achieved. This criterion was believed to be
158 sufficiently high for full saturation of sandy soils. Finally, the sample was sheared under undrained
159 condition at a constant rate of 0.5%/min. This rate was believed to be low enough to allow the
160 equilibrium of pore water pressure, which was also carefully checked by the measurements at the base
161 and top of the samples. The tests ended after the axial strain reached 20%. The similar procedures
162 were also applied to tests TM2-TM15. Note that the hydraulic conductivity test was only performed on
163 sand-fines samples TM5-15.

164 All the tests were carried out at a controlled ambient temperature 20 ± 1 °C.

165

166 3. Experimental results

167 3.1. Permeability tests

168 Fig. 4 presents the variation of hydraulic conductivity k with F_c for TM5-15 mixtures at different
169 contents before shearing. It is observed that, for Sand-C500 mixtures, the k decreased at a fast rate
170 when F_c was smaller than 5%. By contrast, in the case of F_c higher than 5%, the decline tendency of k
171 with F_c became small. While for Sand-Illite and Sand-Speswhite mixtures, the k almost linearly
172 decreased as F_c increased. Moreover, at the same F_c , the highest k values were identified for Sand-
173 Speswhite mixtures, the lowest ones were detected for Sand-C500 mixtures and the values in-between
174 for Sand-Illite mixtures. The results here suggest that even at the identical fines content, the clogs of
175 main pathway for water (e.g., the connectivity of inter-sand voids) for the three materials were not the
176 same: the severest extent was expected for the addition of C500 (NP fines), the fewest one was
177 expected for the addition of Speswhite ($I_p=30$), and the one in-between for Sand-Illite mixtures ($I_p=34$).
178 Considering that there is no clear monotonic relationship between hydraulic conductivity and plasticity
179 of added materials, the above phenomenon might be attributed to the fact that the main pathway for
180 water formed by host sand became increasingly clogged with the increase of added grains size ($d_{50-C500}$
181 $d_{50-Illite} > d_{50-Speswhite}$).

182

183 3.2. Triaxial tests

184 Fig. 5 shows the evolutions of deviator stress (q) with axial strain (ϵ_a) in the cases of two parallel
185 samples for Sand-C500 mixture, as well as for Sand-Speswhite mixture. It can be observed that for
186 Sand-C500 mixture, the two curves were almost the same, exhibiting a satisfactory repeatability of the
187 test. The similar phenomenon was also identified for Sand-Speswhite mixture. This confirms that the

188 protocol adopted in the experimental program is pertinent and repeatable with respect to mixture
189 homogeneity.

190 Fig. 6(a)-(b) presents the deviator stress/excess pore water pressure ($q/\Delta u$) against axial strain
191 (ϵ_a) in the cases of Sand-Sand mixtures ($F_c = 5\%$, 10% , 14% and 18.7%). It appears from Fig. 6 (a)
192 that: (i) for $F_c = 5\%$, the deviator stress q linearly increased with ϵ_a ($\epsilon_a < 0.5\%$) and then kept almost
193 constant till $\epsilon_a = 2\%$. Afterwards, a strain hardening tendency was noted. Finally, the q slightly
194 increased till the end of test; (ii) for $F_c = 10\%$, 14% and 18.7% , a dramatic increase in q was observed
195 at the beginning; then q began to decrease after reaching a peak deviator value (q_{max}). Finally, the
196 deviator stress reached the critical state, exhibiting a strong dilatant behavior; (iii) the F_c had a
197 significant effect on the q_{max} , and q_{max} increased as the F_c increased. The excess pore water pressure
198 characterized in Fig. 6(b) shows that: (i) a short-lived contractive behaviour occurred, followed by a
199 large dilatant phase range; (ii) cavitation was achieved in all cases and Δu decreased as the F_c
200 increased.

201 Fig. 6(c)-(d) describes the evolution of $q/\Delta u$ with ϵ_a in the cases of Sand-C500 mixtures at $F_c =$
202 5% , 10% and 15% . It can be noticed from Fig. 6(c) that q quickly increased in the range of ϵ_a smaller
203 than 0.5% , then slowly increased till the end of tests ($\epsilon_a = 20\%$). In addition, the F_c has an ignorable
204 effect on the q when ϵ_a was smaller than about 2% . By contrast, in the case of ϵ_a higher than 2% , the
205 dominant role of F_c was identified by the remarkable weakening in q . As for Δu in Fig. 6(d), it appears
206 that Δu quickly increased till ϵ_a equaling to 2% ; then decreased until the end of test ($\epsilon_a = 20\%$).
207 Moreover, F_c had an insignificant effect on the Δu when the ϵ_a was smaller than about 2% . On the
208 contrary, in the case of ϵ_a higher than 2% , F_c became prevailing, and Δu increased as F_c increased,
209 suggesting that the increase of F_c encouraged the generation of excess pore water pressure.
210 Concerning Sand-Illite and Sand-Speswhite mixtures, the similar variation tendency of deviator
211 stress/excess pore water pressure curves were observed that: with the increase of F_c , the deviator stress
212 decreased and the corresponding excess pore water pressure increased when ϵ_a was higher than about
213 2% , as shown in Fig. 6(e)-(h). This is consistent with the results reported by Jradi [10], who performed

214 the monotonic and cyclic triaxial tests on NE34 Sand-Speswhite mixtures, and found that the deviator
215 stress decreased as the plastic fines content increased.

216 Fig. 7(a)-(b) displays the evolution of $q/\Delta u$ for three mixtures at F_c equaling to 5%. The curves in
217 Fig. 7(a) show that for Sand-C500 mixture with low D_{50}/d_{50} (58.33), the gain in q with respect to ε_a
218 was steadily increasing. However, for Sand-Speswhite with higher D_{50}/d_{50} (500), different phenomena
219 were observed: the q almost linearly increased with ε_a ($\varepsilon_a < 0.5\%$), then slightly decreased till
220 ε_a equaling to 2%. Afterwards, the q increased until the end of test ($\varepsilon_a = 20\%$). In addition, D_{50}/d_{50} had
221 an insignificant effect on q when the ε_a was smaller than about 0.5%. On the contrary, in the case of ε_a
222 higher than 0.5%, D_{50}/d_{50} became prevailing, and the q decreased as the D_{50}/d_{50} increased, suggesting
223 that the increase of D_{50}/d_{50} decreased the shear strength. The results identified in Fig. 7(b) show that: (i)
224 Δu quickly increased with ε_a till ε_a equaling to 2%; then decreased with ε_a until the end of test (ε_a
225 =20%); (ii) the D_{50}/d_{50} had an insignificant effect on the Δu when the ε_a was smaller than about 0.5%.
226 By contrast, when ε_a was higher than 0.5%, the Δu increased as the D_{50}/d_{50} increased, suggesting that
227 the increase of D_{50}/d_{50} facilitated the generation of excess pore water pressure. The similar phenomena
228 were identified for Sand-C500, Sand-Illite and Sand-Speswhite mixtures at $F_c=10\%$ (Fig. 7c-d) and 15%
229 (Fig. 7e-f).

230 For further analyzing the effect of D_{50}/d_{50} , the results in terms of undrained shear strength S_u
231 ($q_{\max}/2$) versus F_c for Sand-Sand ($D_{50}/d_{50} = 1$), Sand-C500 ($D_{50}/d_{50} = 58.33$, NP fines), Sand-Illite
232 ($D_{50}/d_{50} = 159.09$, $I_{p, \text{Illite}}=34$) and Sand-Speswhite ($D_{50}/d_{50} = 500$, $I_{p, \text{Speswhite}}=30$) mixtures are
233 schematized in Fig. 8(a). The results in terms of excess pore water pressure ratio r_u ($\Delta u_{\max}/\sigma'_c$, the peak
234 value of excess pore water pressure divided by the effective confining pressure) versus D_{50}/d_{50} for
235 Sand-C500 ($D_{50}/d_{50} = 58.33$, NP fines), Sand-Illite ($D_{50}/d_{50} = 159.09$, $I_{p, \text{Illite}}=34$) and Sand-Speswhite
236 ($D_{50}/d_{50} = 500$, $I_{p, \text{Speswhite}}=30$) mixtures are shown in Fig. 8b. Note that Δu_{\max} is almost the same for
237 Sand-C500 (about 64 kPa, see Fig. 6d), Sand-Illite (about 76 kPa, see Fig. 6f), Sand-Speswhite (about
238 84 kPa, see Fig. 6h) mixtures at different F_c , while for Sand-Sand mixture, the excess pore water
239 pressure changed with F_c (see Fig. 6b). Therefore, the result of Sand-Sand mixture was not presented
240 in Fig. 8(b). It appears from Fig. 8(a) and Fig. 8(b) that: (i) for a given content of added materials, the

241 undrained shear strength decreased with the increase of D_{50}/d_{50} ; (ii) for Sand-Sand mixture of
242 $D_{50}/d_{50}=1$, S_u increased as F_c increased, while a reversed phenomenon was observed for higher D_{50}/d_{50} ,
243 and S_u decreased as F_c increased; (iii) r_u monotonically increased as D_{50}/d_{50} increased (see Fig. 8b),
244 indicating that the increase of D_{50}/d_{50} enabled the excess pore water pressure to be rapidly developed.
245 To further analyze, the slopes ($\Delta S_u/\Delta F_c$) were determined from the curves of S_u versus F_c , and the
246 values are listed in Table 3. It was found that $\Delta S_u/\Delta F_c$ monotonically decreased while D_{50}/d_{50} altering
247 from 1 to 500 (see Fig. 8a and Table 3). The results here suggest that the combined dimensionless
248 parameter (D_{50}/d_{50}) linking to relative grains size shows a good performance to describe the
249 mechanical behaviour since it is able to express the coupled influence of sand and added materials.

250 It is worth noting that attempts were also made to measure the grading curve for plastic fines
251 (e.g., Illite) for the determination of d_{10} . However, after applying the sedimentation analysis method
252 for Illite, the grain size distribution curve identified in Fig. 1 shows that it was extremely difficult to
253 precisely measure the grain size lower than $2 \mu\text{m}$, leading to the unreliable determination of d_{10} . This
254 also renders the further interpretation based on the parameters of D_{10}/d_{10} and C_u (d_{60}/d_{10}) difficult in
255 this study.

256

257 **4. Discussion**

258 4.1. Effect of mean diameter ratio

259 As for coarse sand with added materials at low-percentage F_c (dominance of host sand), the overall
260 skeleton of composite soil is constituted by coarse sand matrix [39]. The role of added materials
261 depends on the grain size with respect to that of the host sand. For grain size being close to that of the
262 host sand (see Fig. 9a), the addition of materials increased the occlusal friction represented by the
263 reciprocal constraint of adjacent sand on their relative movements. Consequently, the sliding of sand
264 became difficult, leading to the increase of undrained shear strength. By contrast, in the case of grain
265 size much smaller than that of the host sand (see Fig. 9b), small-size grains have two main positional
266 tendencies during the deposition process, as mentioned by Monkul [25]: (i) they could possibly be

267 accommodated into inter-sand voids space formed by host sand, and the small-size grains could be
268 viewed as inactive. In that case, they are free to move inside the inter-sand voids existed in such a
269 loose state ($I_{Dmat}=0.00$), and do not directly participate in global force at least in the small range of F_c
270 (dominance of host sand). This is consistent with the conclusion drawn by Finn [40] that small-size
271 grains completely accommodated within the voids produced by host sand made no contribution to the
272 mechanical response of composite soil; (ii) they could possibly be located between contacts points of
273 host sand by separating them, and the addition of smaller grains loosened the packing of host sand [41],
274 facilitating the relative sliding of host sand.

275 As mentioned previously, for Sand-Sand mixtures, the grain size of added materials was equal to
276 that of host sand, and in that case, the increase in F_c corresponded to the increase of occlusal friction
277 between sand-sand contacts, leading to the increase of S_u . This is also consistent with the results
278 obtained by Wang [39], who performed the monotonic triaxial tests on six kinds of soil samples with
279 different volumetric contents of coarse grains ($D_{50}/d_{50}<1$ of Sand-Ballast mixtures), and found that the
280 shear strength increased as the volumetric contents of coarse grains increased. Whereas, the mean
281 diameter of C500, Illite and Speswhite are much smaller than that of host sand, making the mean
282 diameter ratio D_{50}/d_{50} significantly greater than 1. In that case, the sliding effect became apparent,
283 resulting in the easy sliding of sand in such a loose state of $I_{Dmat} = 0.00$. This explained the decrease of
284 undrained shear strength S_u with F_c for Sand-C500, Sand-Illite and Sand-Speswhite mixtures observed
285 in Fig. 8(a). Meanwhile, for a given F_c , the increase of D_{50}/d_{50} corresponded to the decreasing diameter
286 of added materials, leading to the increasing quantity of added materials. And in that case, the sliding
287 of sand became relatively easier. This explained the decrease of undrained shear strength with the
288 increase of D_{50}/d_{50} , as shown in Fig. 8(a).

289 According to the above analysis, it could be logically deduced that there essentially exists a
290 critical value of D_{50}/d_{50} , at which F_c has a negligible effect on the S_u of mixtures. Following up this
291 logic, this critical value was deduced from the $\Delta S_u/\Delta F_c$ versus D_{50}/d_{50} (see Table 3), at which the
292 $\Delta S_u/\Delta F_c$ was equal to 0. A value of 14.5 was obtained from Fig. 10, which theoretically verified the
293 negligible effect of F_c on the S_u . In that case, the effect of occlusal friction was equal to that of sliding

294 of sand. Thereby, when D_{50}/d_{50} was smaller than 14.5, the addition of materials corresponded to the
295 global increase of occlusal friction, and the sliding of sand became difficult, leading to the increase of
296 shear strength. By contrast, in the case of D_{50}/d_{50} higher than 14.5, the sliding of sand became easy; as
297 a result, the shear strength decreased as F_c increased. It is worth noting that this critical value (14.5)
298 was determined from four points, more points are needed to clarify this aspect.

299 As regards the results of Δu versus ε_a , it is observed from Sand-Sand mixtures that: the peak
300 value of Δu decreased as the F_c increased (see Fig. 6b). This might be attributed to the fact that, with
301 the increase in F_c , the clog of main pathway for water increased. And in that case, the generation of
302 pore water pressure was inhibited during shearing. Following up this logic, the similar phenomenon
303 should be also observed for Sand-C500, Sand-Illite and Sand-Speswhite mixtures (the peak value of
304 Δu should decrease as the F_c increased), as the clog extent of main pathway for water increased,
305 characterized by the decrease in hydraulic conductivity k with F_c (see Fig. 4). However, the results
306 identified in Fig. 6(d), (f), (h) did not support this reasoning: F_c seemed to have an insignificant effect
307 on this peak value. This might be attributed to the following two factors: (i) although the hydraulic
308 conductivity k was determined prior to shearing, it still could be supposed to be representative in the
309 small range of ε_a where Δu_{\max} appeared. Therefore, with the increase of F_c , clog effect inhibiting the
310 generation of pore water pressure was confirmed by Fig. 4 that k decreased as F_c increased; (ii) the
311 addition of relatively small-size fines such as C500, Illite and Speswhite into host sand matrix
312 enhanced the contraction phenomenon, leading to the generation of more pore water pressure, as
313 mentioned previously. Thereby, the insignificant effect of F_c on the peak value of Δu_{\max} identified in
314 Fig. 6(d) for Sand-C500, in Fig. 6(f) for Sand-Illite and in Fig. 6(h) for Sand-Speswhite mixtures
315 combined the above two factors. As regards the increase of r_u with D_{50}/d_{50} (see Fig. 8b), it might be
316 attributed to the sliding effect that the contraction phenomenon was enhanced, as mentioned
317 previously. It might be also attributed to the less clog extent of main pathway for water, justified by
318 the increasing hydraulic conductivity with D_{50}/d_{50} (see Fig. 4).

319

320 4.2. Effects of other indicators

321 It is worth noting that in addition to the grain size (represented by D_{50}/d_{50}), other factors such as
322 plasticity index I_p and grain shape may also affect the mechanical response [23–26,42]. General
323 speaking, the plasticity index I_p of added clayey fines (especially for the highly expansive clay) should
324 have a significant effect on the hydro-mechanical behaviour of sandy mixtures. However, the results
325 observed in this study suggest that I_p is not a good indicator for analyzing the effect of added materials
326 on undrained shear strength, at least for the three added materials with low I_p that were considered in
327 this study (C500, Illite and Speswhite) since the comparatively small difference in plasticity indexes
328 between Speswhite ($I_{p,Speswhite}=30$) and Illite ($I_{p,Illite}=34$) could not legitimately explain such a
329 significant discrepancy in mechanical responses (see Fig. 8a and Fig. 8b). This is also consistent with
330 the results obtained by Park [43], who performed the triaxial tests on loosely prepared sandy deposits
331 with the addition of fines materials of distinct plasticity indexes (e.g., from $I_p=8$ of silt to $I_p=377$ of
332 bentonite employed), and found that although liquefaction resistance of samples indeed decreased with
333 the increase of plasticity, however, the observed tendency was not obvious. As regards the effect of the
334 grain shape of added materials, it may depend on the grain size (represented by D_{50}/d_{50}): (i) for Sand-
335 Sand or Sand-Silt mixtures with low value of D_{50}/d_{50} , the angularity of the added materials might play
336 an important role because this physical parameter directly determines the occlusal level of overall soil
337 matrix [37]; (ii) on the contrary, for sand-fines mixtures especially with the addition of clayey
338 materials (e.g., Illite and Speswhite), one might expect an ignorable effect of shape characteristic since
339 the added materials are enough smaller than the host sand forming the overall soil matrix. Indeed,
340 Carraro [44] reported an increase/decrease of both peak and critical-state friction angle by performing
341 triaxial test on host sand with the addition of non-plastic silt/plastic kaolinite clay, and pointed out that
342 more angular non-plastic silt tended to increase undrained shear strength during shearing, whilst an
343 opposite phenomenon was observed for more rounded clay particles. This is also consistent with the
344 results obtained by Yang [24], who performed a series of triaxial tests on sand-silt mixture, and found
345 that the sand-silt mixtures with angular silt tended to exhibit higher shear strength, as compared with
346 that for sand-silt mixtures with the addition of rounded silt. And they attributed this phenomenon to

347 the fact that rounded fines could further facilitate the sliding in the loose state. However, a reversed
348 trend was reported by Monkul [25] that the angular nature of silt made the sandy samples more prone
349 to lose shear strength. Like the effect of plasticity index of the added materials on the undrained shear
350 strength (as mentioned previously), no common conclusion was achieved in terms of the effect of
351 grain shape. Further studies are thus needed to clarify these points. However, it is believed that the
352 mechanism identified in this study in the large range of D_{50}/d_{50} (from 1 to 500) would be still
353 applicable.

354

355 **5. Conclusions**

356 In this study, the effect of mean diameter ratio D_{50}/d_{50} on the undrained shear strength of a loose sand
357 matrix ($I_{Dmat}=0.00$) was investigated by respectively adding three fines (with different mean diameter
358 ratios D_{50}/d_{50}) at content of 5%, 10% and 15% (by mass), and by performing a series of triaxial
359 compression tests under undrained condition. For further comparison and analysis, the triaxial tests
360 were also performed on Sand-Sand mixtures at four contents of 5%, 10%, 14% and 18.7%. The results
361 obtained allow the following conclusions to be drawn.

362 The evolution of S_u and r_u proved that D_{50}/d_{50} could serve as a pertinent indicator to evaluate the
363 undrained shear strength of sand with added materials since it considered the coupled characteristics
364 between two materials. For low value D_{50}/d_{50} (e.g., 1 for Sand-Sand mixtures), S_u increased as F_c
365 increased, which might be attributed to the increase of occlusal friction, similar to the densification
366 effect. On the contrary, for higher value D_{50}/d_{50} (e.g., 58.33 for Sand-C500, 159.09 for Sand-Illite and
367 500 for Sand-Speswhite), the undrained shear strength S_u decreased as F_c increased. In fact, the
368 increase of D_{50}/d_{50} corresponded to decreasing diameter of added materials. In that case, the
369 corresponding quantity of added materials increased, leading to the easy sliding of sand. A critical
370 value of D_{50}/d_{50} (14.5) was thus deduced, which theoretically verified the negligible effect of F_c on S_u .
371 Moreover, for sand with fines (C500 silt, Illite and Speswhite) mixtures, the increase of F_c had
372 insignificant effect on the peak value of Δu_{max} , while the increase of D_{50}/d_{50} facilitated the generation
373 of excess pore water pressure after reaching about Δu_{max} .

374 From practical points of view, the results obtained in this study are very helpful for analyzing the
375 undrained shear strength in loose state, as well as for designing the geotechnical structures. Indeed, as
376 for quasi-binary mixture, when the D_{50}/d_{50} of mixtures is higher than the critical value, the shear
377 strength decreases as the F_c increases. Only when the D_{50}/d_{50} becomes lower than the critical value, the
378 undrained shear strength increases as the F_c increases, thus the resistance to liquefaction increases.

379

380 **Acknowledgments**

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383

384 **References**

- 385 [1] Wyss M, Brune J. The Alaska earthquake of 28 March 1964: A complex multiple rupture. Bull
386 Seismol Soc Am 1967;57:1017–23.
- 387 [2] Ishihara K, Koga Y. Case Studies of Liquefaction In the 1964 Niigata Earthquake. Soils Found
388 1981;21:35–52.
- 389 [3] Ohsaki Y. Effects of Sand Compaction on Liquefaction During The Tokachioki Earthquake.
390 Soils Found 1970;X. <https://doi.org/10.1061/ASCE1090-02412003129:112>.
- 391 [4] Shengcong F, Tatsuoka F. Soil Liquefaction During Haicheng and Tangshan Earthquake in
392 China; A Review. Soils Found 1984;24:22–9.
- 393 [5] Tokimatsu K, Yoshimi Y. Empirical Correlation of Soil Liquefaction Based on SPT N-Value
394 and Fines Content. Soils Found 1983;23.
- 395 [6] Tokimatsu K, Yoshimi Y. Field correlation of soil liquefaction with SPT and grain size. F.
396 Correlqtion Soil Liq. with SPT Grain Size, International Conferences on Recent Advances in
397 Geotechnicql Earthquake Engineering and Soil Dynamics; 1981, p. 203–8.
- 398 [7] Seed HB, ASCE F, Idriss IM, Arango I. Case Studies of Liquefaction In the 1964 Niigata
399 Earthquake. J Geotech Eng 1983;109:458–82.
- 400 [8] Amini F, Qi G. Liquefaction Testing of Stratified Silty Sands. J Geotech Geoenvironmental
401 Eng 2000;126:208–17.
- 402 [9] Benahmed N, Nguyen TK, Hicher PY, Nicolas M. An experimental investigation into the
403 effects of low plastic fines content on the behaviour of sand/silt mixtures. Eur J Environ Civ
404 Eng 2015;19:109–28. <https://doi.org/10.1080/19648189.2014.939304>.
- 405 [10] Jradi L. Study of the Influence of Fine Particles on the Properties of Liquefaction of Sands.
406 Ph.D Thesis University Paris-Est, 2018.
- 407 [11] Pitman T, Robertson P, Seg0 D. Influence of fines on the collapse of loose sands. Can Geotech

408 J 1994;31:728–39.

409 [12] Ni Q, Tan TS, Dasari GR, Hight DW. Contribution of fines to the compressive strength of
410 mixed soils. *Géotechnique* 2004;54:561–9. <https://doi.org/10.1680/geot.54.9.561.56936>.

411 [13] Belkhatir M, Arab A, Della N, Missoum H, Schanz T. Liquefaction resistance of Chlef river
412 silty sand: Effect of low plastic fines and other Parameters. *Acta Polytech Hungarica*
413 2010;7:119–37.

414 [14] Stamatopoulos CA. An experimental study of the liquefaction strength of silty sands in terms
415 of the state parameter. *Soil Dyn Earthq Eng* 2010;30:662–78.
416 <https://doi.org/10.1016/j.soildyn.2010.02.008>.

417 [15] Polito C. The Effects Of Non-Plastic and Plastic Fines On The Liquefaction Of Sandy Soils.
418 Virginia Polytechnic Institute, 1999.

419 [16] Bouferra R, Shahrour I. Influence of fines on the resistance to liquefaction of a clayey sand. *Gr*
420 *Improv* 2004;8:1–5. <https://doi.org/10.1680/grim.8.1.1.36366>.

421 [17] Ghahremani M, Ghalandarzadeh A. Effect of Plastic Fines on Cyclic Resistance of Sands. *Soil*
422 *Rock Behav Model* 2006:406–12.

423 [18] Lade P, Yamamuro J. Effects of nonplastic fines on static liquefaction of sands. *Can Geotech J*
424 1997;34:918–28. <https://doi.org/10.1139/t97-052>.

425 [19] Thevanayagam S, Shenthan T, Mohan S, Liang J. Undrained fragility of clean sands, silty
426 sands, and sandy silts. *J Geotech Geoenvironmental Eng* 2002;128:849–59.
427 <https://doi.org/10.1061/jggefck>.

428 [20] Abedi M, Yasrobi SS. Effects of plastic fines on the instability of sand. *Soil Dyn Earthq Eng*
429 2010;30:61–7. <https://doi.org/10.1016/j.soildyn.2009.09.001>.

430 [21] Yang SL, Sandven R, Grande L. Instability of sand-silt mixtures. *Soil Dyn Earthq Eng*
431 2006;26:183–90. <https://doi.org/10.1016/j.soildyn.2004.11.027>.

- 432 [22] Xenaki VC, Athanasopoulos GA. Liquefaction resistance of sand-silt mixtures: An
433 experimental investigation of the effect of fines. *Soil Dyn Earthq Eng* 2003;23:1–12.
434 [https://doi.org/10.1016/S0267-7261\(02\)00210-5](https://doi.org/10.1016/S0267-7261(02)00210-5).
- 435 [23] Papadopoulou AI, Tika TM. The effect of fines plasticity on monotonic undrained shear
436 strength and liquefaction resistance of sands. *Soil Dyn Earthq Eng* 2016;88:191–206.
437 <https://doi.org/10.1016/j.soildyn.2016.04.015>.
- 438 [24] Yang J, Wei LM. Collapse of loose sand with the addition of fines: The role of particle shape.
439 *Geotechnique* 2012;62:1111–25. <https://doi.org/10.1680/geot.11.P.062>.
- 440 [25] Monkul MM, Etminan E, Şenol A. Coupled influence of content, gradation and shape
441 characteristics of silts on static liquefaction of loose silty sands. *Soil Dyn Earthq Eng*
442 2017;101:12–26. <https://doi.org/10.1016/j.soildyn.2017.06.023>.
- 443 [26] Monkul MM, Etminan E, Şenol A. Influence of coefficient of uniformity and base sand
444 gradation on static liquefaction of loose sands with silt. *Soil Dyn Earthq Eng* 2016;89:185–97.
445 <https://doi.org/10.1016/j.soildyn.2016.08.001>.
- 446 [27] Doygun O, Brandes HG, Roy TT. Effect of Gradation and Non-plastic Fines on Monotonic and
447 Cyclic Simple Shear Strength of Silica Sand. *Geotech Geol Eng* 2019;37:3221–40.
448 <https://doi.org/10.1007/s10706-019-00838-9>.
- 449 [28] Belkhatir M, Arab A, Schanz T, Missoum H, Della N. Laboratory study on the liquefaction
450 resistance of sand-silt mixtures: Effect of grading characteristics. *Granul Matter* 2011;13:599–
451 609. <https://doi.org/10.1007/s10035-011-0269-0>.
- 452 [29] Monkul MM, Yamamuro JA. Influence of silt size and content on liquefaction behavior of
453 sands. *Can Geotech J* 2011;48:931–42. <https://doi.org/10.1139/t11-001>.
- 454 [30] Cherif Taiba A, Belkhatir M, Kadri A, Mahmoudi Y, Schanz T. Insight into the Effect of
455 Granulometric Characteristics on the Static Liquefaction Susceptibility of Silty Sand Soils.
456 *Geotech Geol Eng* 2016;34:367–82. <https://doi.org/10.1007/s10706-015-9951-z>.

- 457 [31] Huang Y, Zhao L. The effects of small particles on soil seismic liquefaction resistance: current
458 findings and future challenges. *Nat Hazards* 2018;92:567–79. [https://doi.org/10.1007/s11069-](https://doi.org/10.1007/s11069-018-3212-4)
459 018-3212-4.
- 460 [32] Muhammed R. Etude en chambre d'étalonnage du frottement sol-pieu sous grands nombre de
461 cycles. Application au calcul des fondations profonds dans les sols fins saturé. Ph.D Thesis
462 Université Pierre et Marie Curie, 2015.
- 463 [33] Yang S, Lacasse S, Sandven R. Determination of the transitional fines content of mixtures of
464 sand and non-plastic fines. *Geotech Test J* 2006;29:102–7. <https://doi.org/10.1520/gtj14010>.
- 465 [34] Zhu Z, Dupla J, Canou J, Foerster E. Experimental study of liquefaction resistance : effect of
466 non-plastic silt content on sand matrix. *Eur J Environ Civ Eng* 2020;0:1–19.
467 <https://doi.org/10.1080/19648189.2020.1765198>.
- 468 [35] El Dine BS, Dupla JC, Frank R, Canou J, Kazan Y. Mechanical characterization of matrix
469 coarse-grained soils with a large-sized triaxial device. *Can Geotech J* 2010;47:425–38.
470 <https://doi.org/10.1139/T09-113>.
- 471 [36] Ishihara K, Kawase Y, Nakajima M. Liquefaction characteristics of sand deposits at an oil tank
472 site during the 1978 Miyagiken-Oki earthquake. *Soils Found* 1980;20:97–111.
- 473 [37] Benahmed N. Comportement mécanique d'un sable sous cisaillement monotone et cyclique:
474 application aux phénomènes de liquéfaction et de mobilité cyclique. Ph.D Thesis Ecole
475 Nationale Des Ponts et Chaussées, 2001.
- 476 [38] Cubrinovski M, Ishihara K. Maximum and Mimimum Void Ratio Characteristics of Sands.
477 *Soils Found* 2002;42:65–78.
- 478 [39] Wang HL, Cui YJ, Lamas-Lopez F, Calon N, Saussine G, Dupla JC, et al. Investigation on the
479 mechanical behavior of track-bed materials at various contents of coarse grains. *Constr Build*
480 *Mater* 2018;164:228–37. <https://doi.org/10.1016/j.conbuildmat.2017.12.209>.
- 481 [40] Finn WD, Ledbetter RH, Wu G. Liquefaction in silty soils: design and analysis. *Liq. silty soils*

- 482 Des. Anal. Gr. Fail. under Seism. Cond., 1994, p. 51–76.
- 483 [41] Liu ZR, Ye WM, Zhang Z, Wang Q, Chen YG, Cui YJ. Particle size ratio and distribution
484 effects on packing behaviour of crushed GMZ bentonite pellets. *Powder Technol* 2019;351:92–
485 101. <https://doi.org/10.1016/j.powtec.2019.03.038>.
- 486 [42] Eseller-Bayat EE, Monkul MM, Akin Ö, Yenigun S. The Coupled Influence of Relative
487 Density, CSR, Plasticity and Content of Fines on Cyclic Liquefaction Resistance of Sands. *J*
488 *Earthq Eng* 2019;23:909–29. <https://doi.org/10.1080/13632469.2017.1342297>.
- 489 [43] Park SS, Kim YS. Liquefaction resistance of sands containing plastic fines with different
490 plasticity. *J Geotech Geoenvironmental Eng* 2013;139:825–30.
491 [https://doi.org/10.1061/\(ASCE\)GT.1943-5606.0000806](https://doi.org/10.1061/(ASCE)GT.1943-5606.0000806).
- 492 [44] Carraro J, Prezzi M, Salgado R. Shear Strength and Stiffness of Sands Containing Plastic or
493 Nonplastic Fines. *J Geotech Geoenvironmental Eng* 2009;135(9):1169–78.
494 [https://doi.org/10.1061/\(ASCE\)1090-0241\(2009\)135](https://doi.org/10.1061/(ASCE)1090-0241(2009)135).
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496 **Table and Figure**

497

Table 1 Physical properties of test materials

Material	Mineralogical Content	D_{50} or d_{50} (μm)	e_{\min}	e_{\max}	G_s	I_p (-)
HN31	Sand	350	0.656	1		NP
C500	Silt	6	-	-		NP
Illite Arvel	Illite	2.2	-	-	2.65	34
Speswhite	Kaolinite	0.7	-	-		30

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Table 2 Undrained triaxial test programs

Test	Added materials	$F_c^{(1)}$ (%)	$F_c^{T(2)}$ (%)	D_{50}/d_{50}	e_{mat}	$I_{D\text{mat}}$	σ'_c (kPa)
TM1		5	4.8				
TM2	HN31 (NP)	10	9.1	1			
TM3		14	12.3				
TM4		18.7	15.8				
TM5		5	4.8				
TM6	C500 (NP)	10	9.1	58.33			
TM7		10	9.1				
TM8		15	13.0			1.00	0.00
TM9	Illite Arvel ($I_p=34$)	5	4.8	159.09			
TM10		10	9.1				
TM11		15	13.0				
TM12		5	4.8				
TM13	Speswhite ($I_p=30$)	10	9.1	500			
TM14		10	9.1				
TM15		15	13.0				

Note

1. $F_c = (m_{\text{add}} / m_{\text{sand}}) \times 100\%$
2. $F_c^T = (m_{\text{add}} / (m_{\text{add}} + m_{\text{sand}})) \times 100\%$

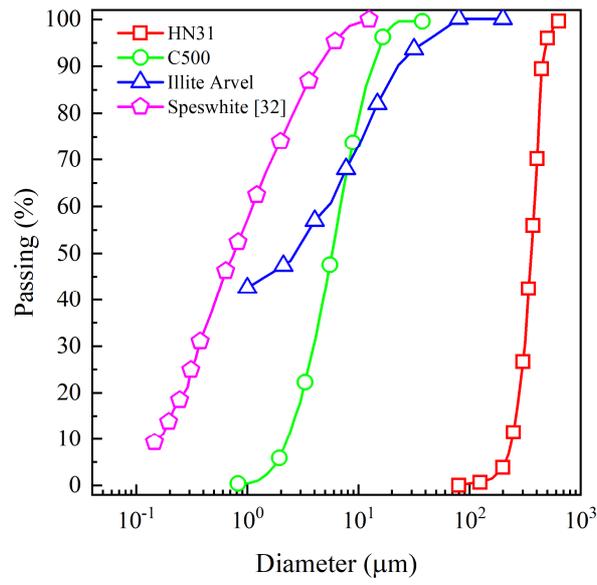
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Table 3 Summary of undrained triaxial test results

Test	Added materials	F_c (%)	q_{max} (kPa)	S_u (kPa)	$\Delta S_v/\Delta F_c$ (kPa)	D_{50}/d_{50} (-)	Δu_{max} (kPa)	r_u (-)
TM1		5	928	464			70	0.7
TM2	HN31 (NP)	10	1080	540	1374	1	44	0.44
TM3		14	1125	562.5			35	0.35
TM4		18.7	1324	662			34	0.34
TM5	C500 (NP)	5	652	326				
TM6		10	576	288	-670	58.33	64	0.64
TM8		15	518	259				
TM9	Illite ($I_p=34$)	5	546	273				
TM10		10	480	240	-870	159.09	76	0.76
TM11		15	372	186				
TM12	Speswhite ($I_p=30$)	5	276	138				
TM13		10	180	90	-920	500	84	0.84
TM15		15	92	46				



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Figure 1 Grain size distribution curves of test materials

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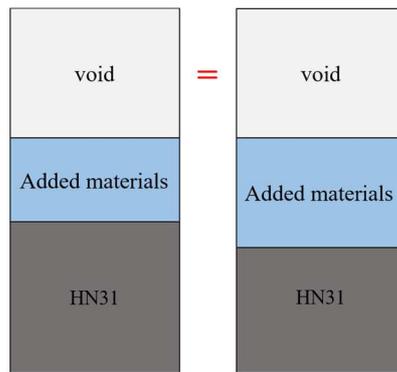
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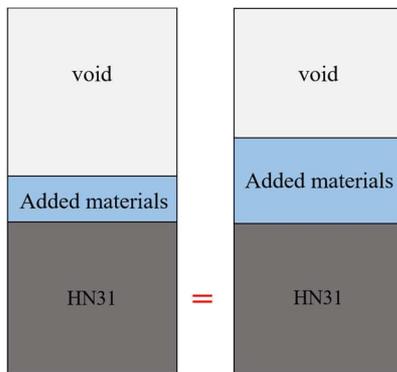
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(a)



(b)



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Figure 2 (a) Constant global void ratio; (b) constant density index of sand matrix (after Zhu [34])

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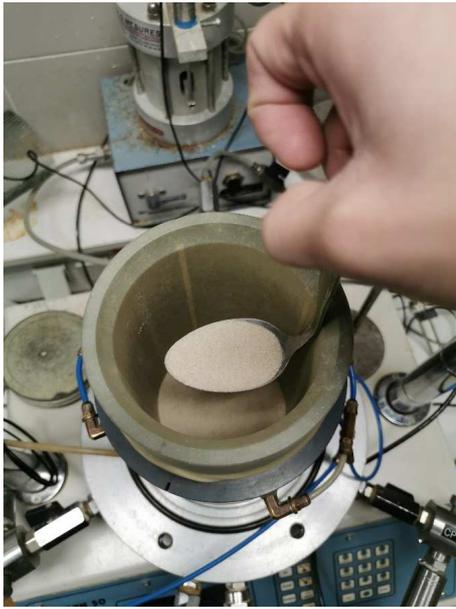
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(a)



(b)



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Figure 3 Dry tamping method (a) spooning the mixture into a split mould; (b) horizontality of each layer after slight compaction

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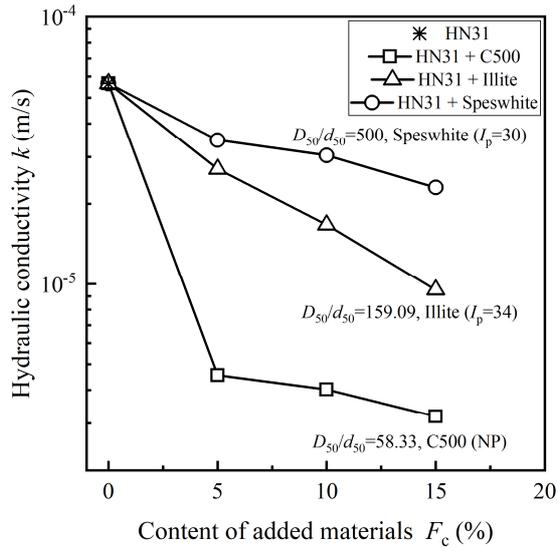
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Figure 4 Effect of content of added materials F_c on the hydraulic conductivity k

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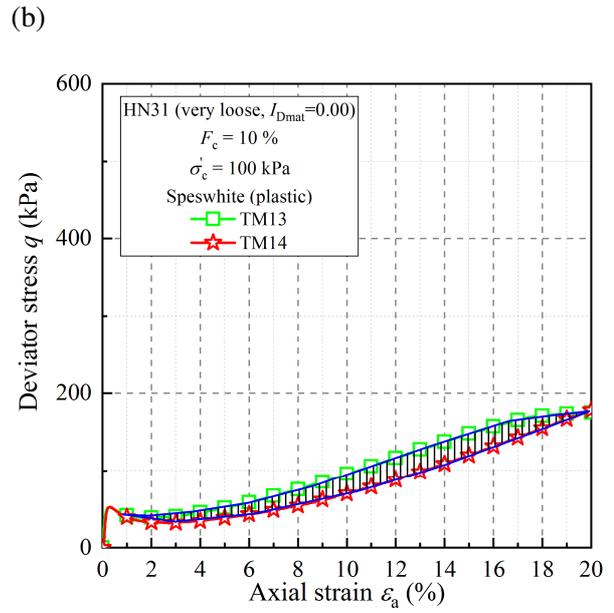
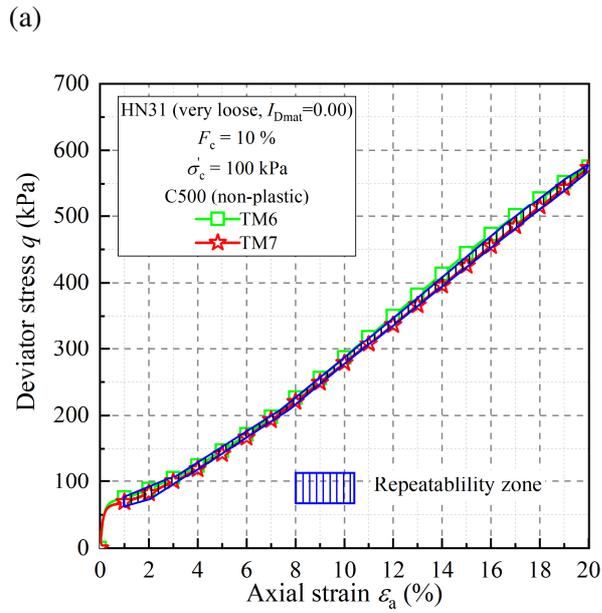
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559 Figure 5 Repeatability tests under a confining pressure of 100 kPa in terms of stress-strain for (a) Sand-C500 mixture; (b)
 560 Sand-Speswhite mixture

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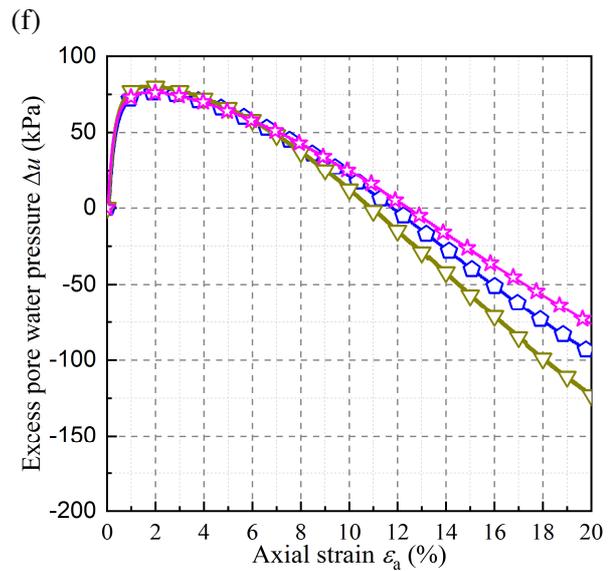
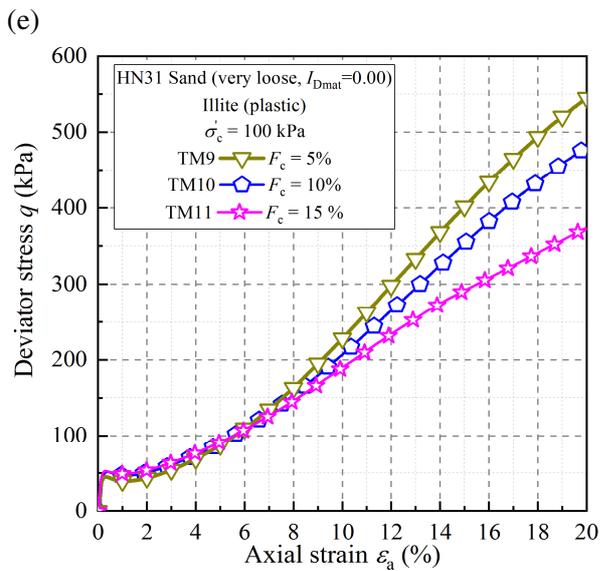
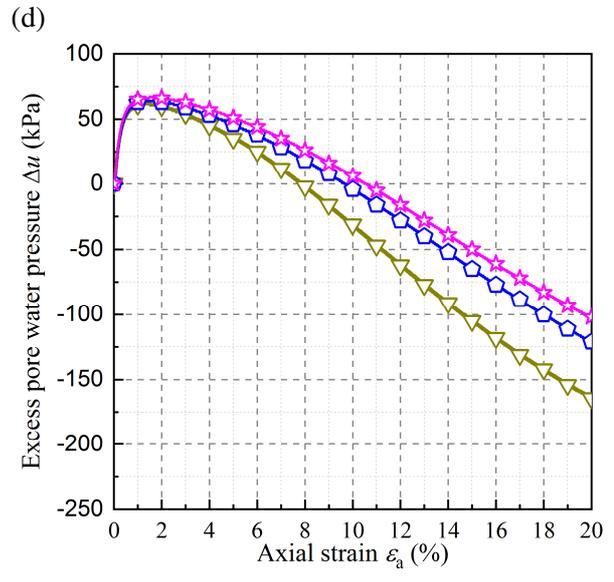
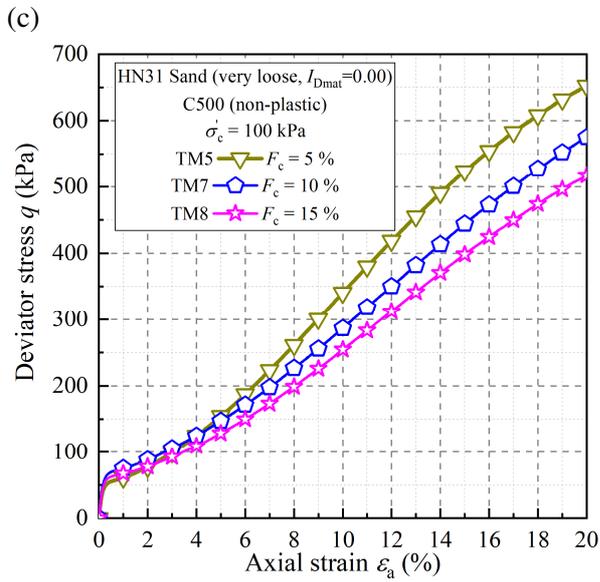
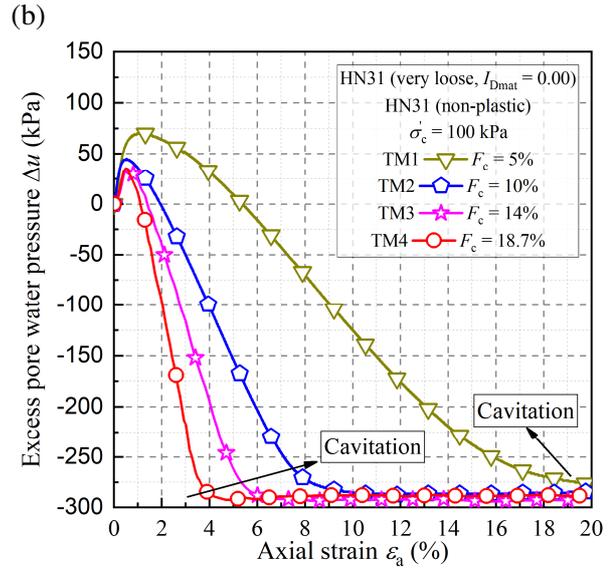
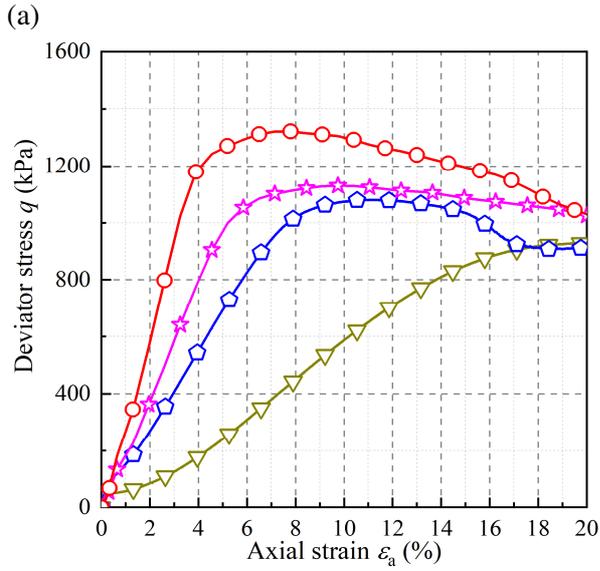
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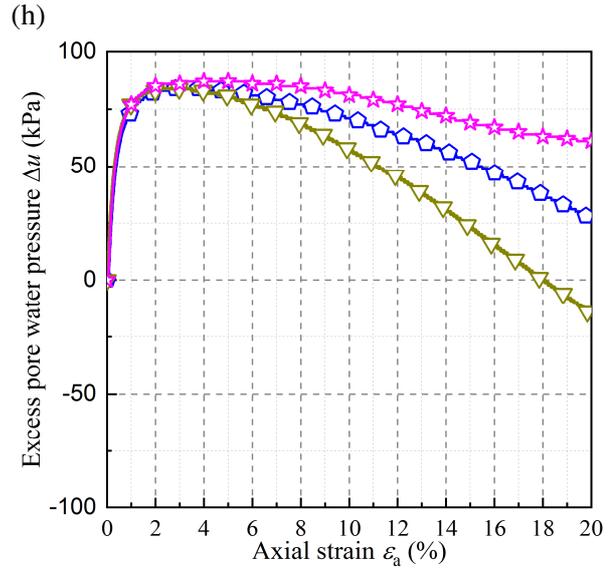
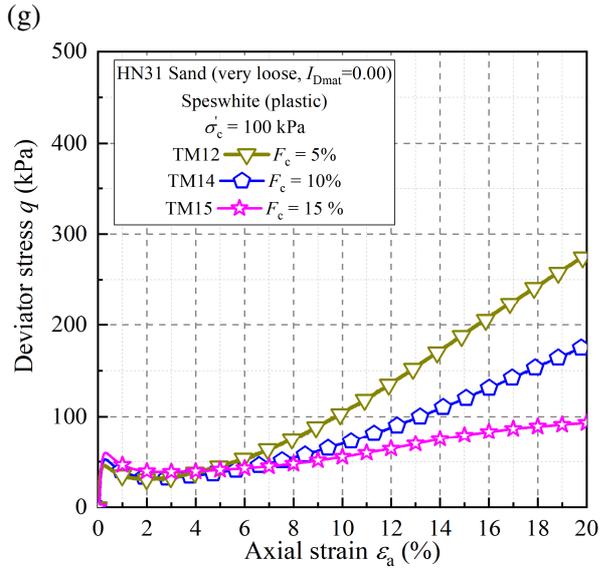


Figure 6 Effect of content of added materials on (a), (c), (e), (g) stress-strain curves; (b), (d), (f), (h) excess pore water pressure-strain curves

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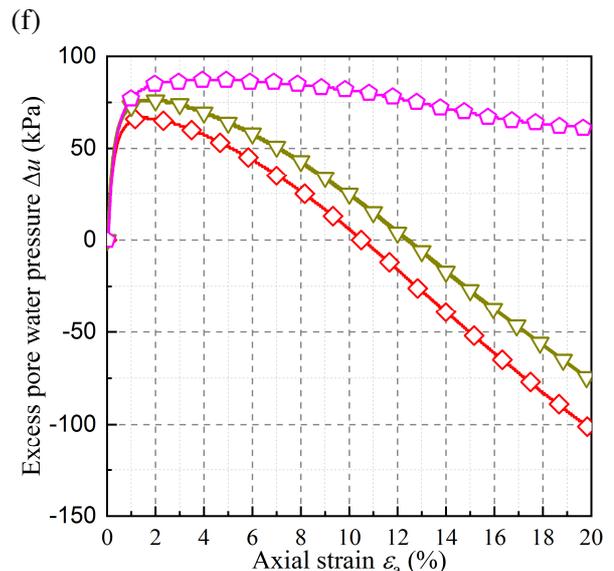
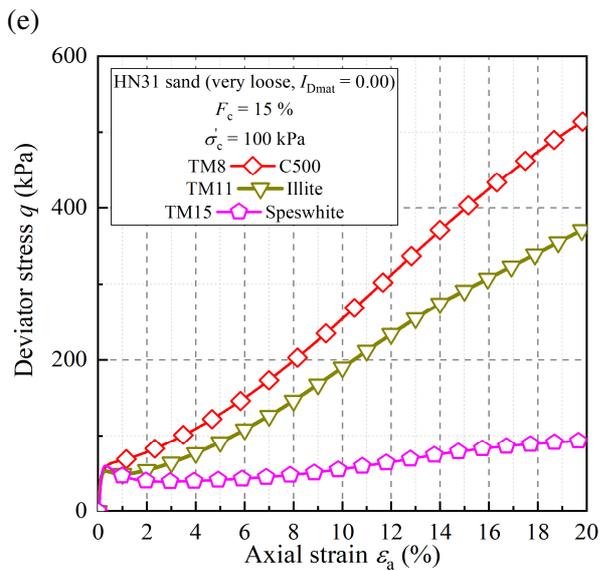
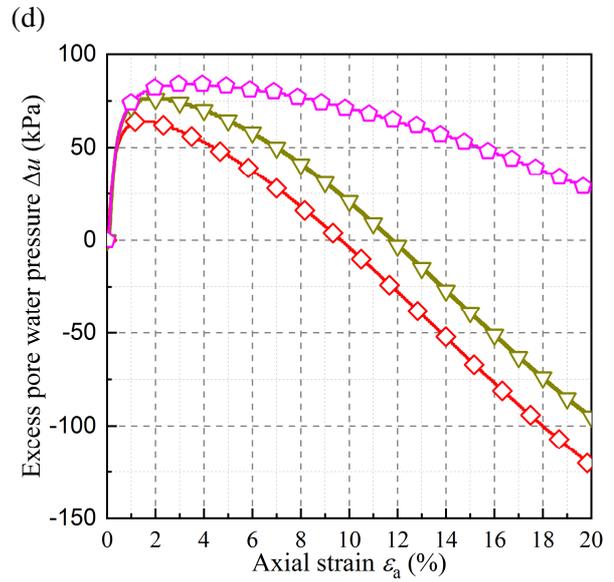
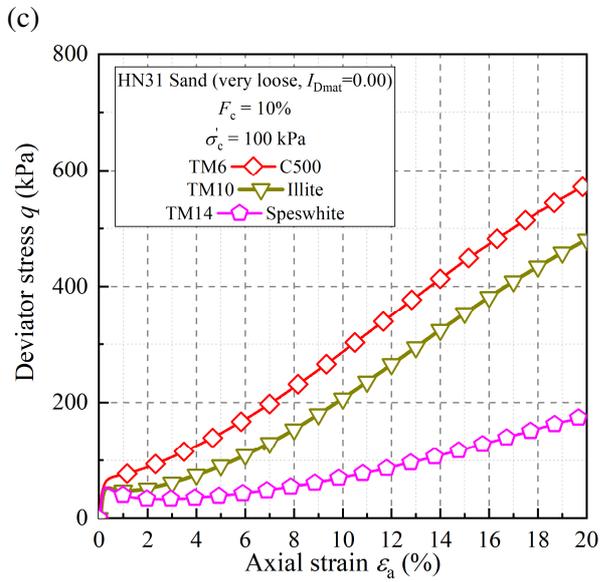
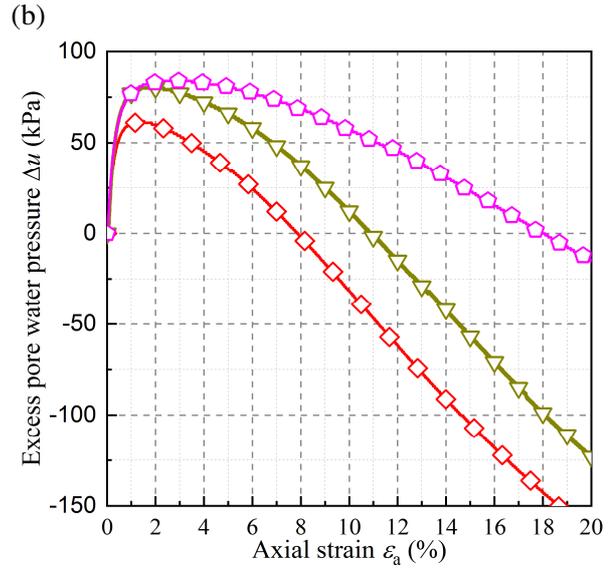
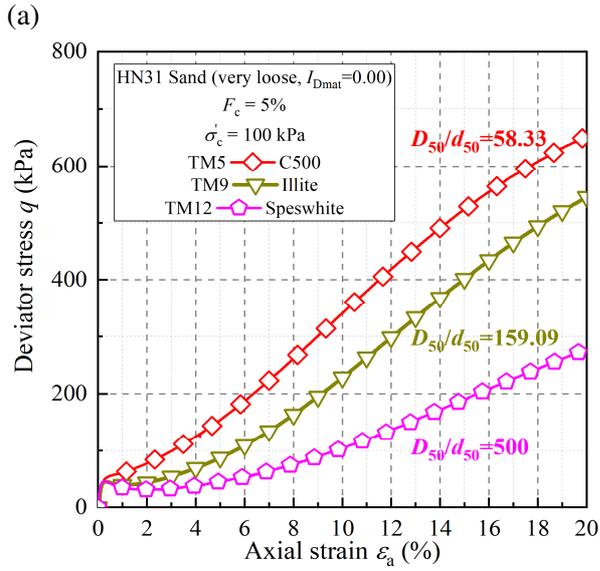
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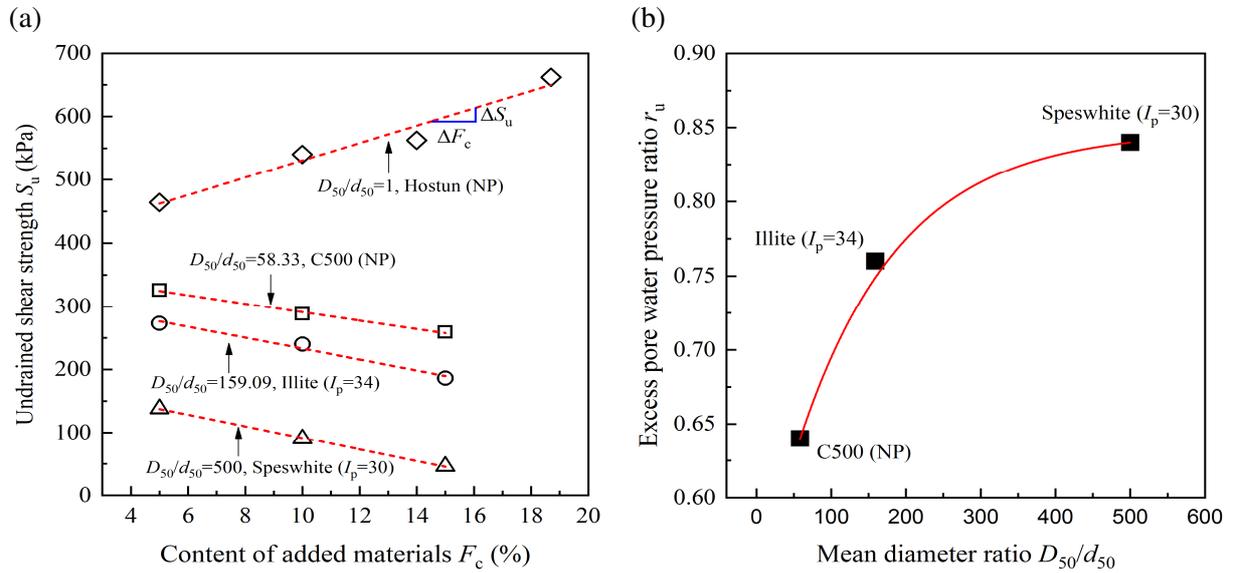
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589 Figure 7 Effect of mean diameter ratio on (a), (c), (e) stress-strain curves; (b), (d), (f) excess pore water pressure-strain curves
 590 for mixtures at different F_c

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592 Figure 8 (a) Undrained shear strength versus content of added materials F_c ; (b) excess pore water pressure ratio r_u versus
 593 mean diameter ratio D_{50}/d_{50}

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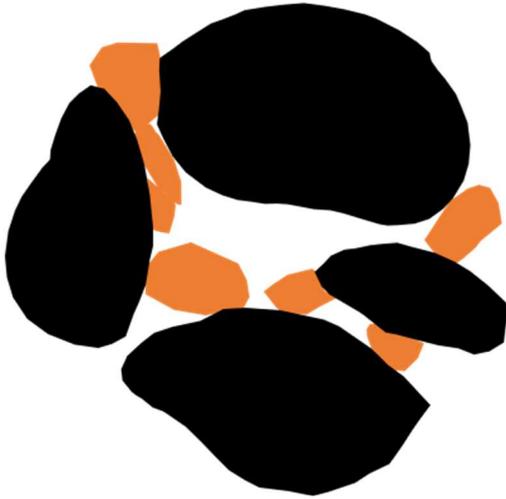
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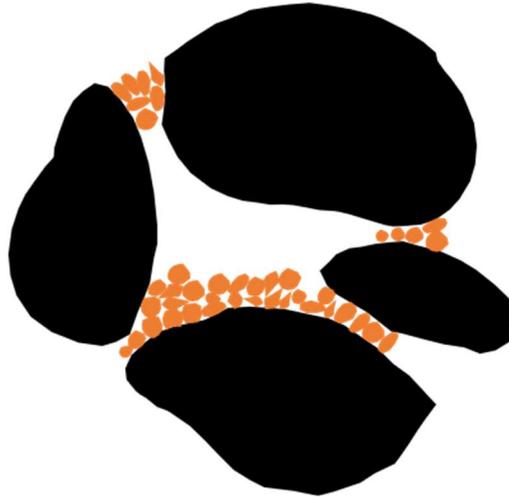
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(a)



(b)



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Figure 9 Schematic diagram of loose host sand containing added materials with (a) large-size grains; (b) small-size grains

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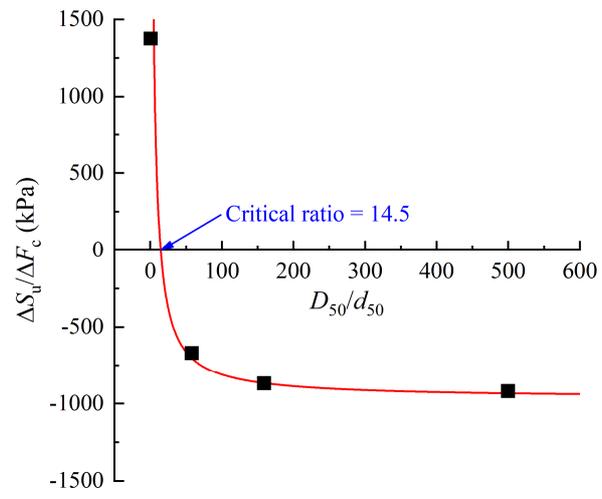
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Figure 10 Variation of $\Delta S_u / \Delta F_c$ with D_{50} / d_{50}

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