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Yu Su, Yu-Jun Cui, Jean-Claude Dupla, Jean Canou. Effect of water content on permanent deformation of fine/coarse soil mixtures with varying coarse grain contents and subjected to multi-stage cyclic loading. *Acta Geotechnica*, 2022, 17 (8), pp.3259-3268. 10.1007/s11440-021-01445-w . hal-04155906

HAL Id: hal-04155906

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Submitted on 7 Jul 2023

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1 Effect of water content on permanent deformation of fine/coarse soil mixtures
2 with varying coarse grain contents and subjected to multi-stage cyclic loading

3

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22 **Abstract**

23 An interlayer soil in ancient rail tracks was identified as a mixture of ballast grains and
24 subgrade fines. As the permanent strain ε_1^p of such mixture was affected by water content,
25 cyclic triaxial tests were performed, under varying water contents of fines w_f and coarse grain
26 contents f_v . Comparison between present and previous studies showed the significant effect of
27 sample preparation method on ε_1^p . In present study, a constant fine dry density ρ_{d-f} was
28 maintained, leading to an unchanged suction of mixture whatever the f_v value. In this case,
29 only the reinforcement effect of f_v on ε_1^p was identified. By contrast, in previous studies, the
30 global dry density of mixture ρ_d was kept constant, resulting in a decrease of ρ_{d-f} with
31 increasing f_v and consequently a decrease of suction. In this case, when the negative effect of
32 decreasing suction prevailed on the positive reinforcement effect of increasing f_v , the ε_1^p
33 increased.

34 **Keywords:** interlayer soil; cyclic triaxial test; permanent deformation; water content; coarse
35 grain content; fabric/ structure of soils

36 INTRODUCTION

37 An interlayer was created in most conventional French rail tracks, mainly due to the
38 interpenetration of ballast grains and subgrade fine soils. This interlayer was maintained in the
39 railway substructure in the French program of rail track renewal considering its high dry
40 density (2.4 Mg/m^3 , Trinh [1]), Cui et al. [2]) and, hence, bearing capacity. As an important
41 component of rail track, the interlayer soil diffused static and dynamic stresses into the
42 substructure, avoiding excessive deformation. In this case, the deformation behavior of
43 interlayer soil appeared to be significant, especially for the long-term stability of rail track.
44 Field observation showed that the ballast grain content decreased over depth in the interlayer
45 soil (Trinh [1]). However, the interlayer could be approximately divided into two parts: the
46 upper part dominated by ballast grains and the lower part dominated by subgrade fine soil.
47 With the unstable groundwater and climate change (rainfall and evaporation), the water
48 content of interlayer soil can change over time, significantly affecting its permanent
49 deformation behavior. Thus, in order to ensure the good serviceability of the tracks, it is
50 essential to understand the effect of water content on the permanent deformation of interlayer
51 soil.

52 The effect of coarse grain content on permanent deformation of soil have been
53 investigated by several investigators. Song and Ooi [3] studied the deformation behaviour of
54 aggregates with varying fine soil contents, and found that increasing fine content gave rise to
55 an increase of permanent deformation of soil mixtures. Wang et al. [4] investigated the effect
56 of coarse grain content f_v (ratio of coarse grain volume to total volume) on permanent strain of
57 interlayer soil by cyclic triaxial tests. They identified a characteristic coarse grain content $f_{v\text{-cha}}$

58 in a narrow range from 25.8% to 27.8%: a large decrease of permanent strain with f_v was
59 observed at $f_v \leq f_{v\text{-cha}}$, but a slight decrease of that at $f_v \geq f_{v\text{-cha}}$. It is worth noting that these tests
60 were performed under a constant water content of fine soils fraction $w_{\text{opt-f}} = 13.7\%$, and the
61 effect of water content was not specifically addressed . Some studies evidenced the effect of
62 water content on permanent deformation of substructures: at saturation, an excess pore water
63 pressure accumulated under traffic loadings led to a decrease of effective stress and an
64 increase of permanent deformation. When the water content decreased, the permanent
65 deformation appeared to decrease, which was attributed to the contribution of suction (Gidel
66 et al. [5]; Werkmeister et al. [6]; Nie et al. [7]; Trinh et al. [8]; Jing [9]; Wan et al. [10]). Gu et
67 al. [11] studied the permanent deformation of unbound granular materials by suction-
68 controlled cyclic triaxial tests, and reported that an increase of suction led to a decrease of
69 accumulated permanent strain exponentially under varying deviator stress amplitudes. Duong
70 et al. [12] investigated the effect of water content on permanent deformation of the upper
71 interlayer soil with varying fine soil contents, and found that the effects of water content and
72 fine content on permanent deformation were strongly related. At saturated state, an increase of
73 fine content gave rise to an increase of permanent deformation, while at unsaturated state, an
74 opposite trend was observed, due to the contribution of suction developed in the fines. Jing [9]
75 studied the deformation behaviour of granular material with varying water contents and fine
76 contents, and also reported that increasing fine content led to an increase of permanent
77 deformation under saturated condition and a decrease of that under unsaturated condition.
78 Note that in most studies, the dry density of fine/coarse soil mixtures remained constant
79 during the sample preparation; thereby a variation of dry density of fine soil fraction was

80 induced with varying fine contents (or coarse grain contents). In that case, both suction and
81 coarse grain content varied, rendering the test results difficult to be analysed.

82 This study aims at investigating the effect of water content on the permanent deformation
83 of fine/coarse soil mixtures under various coarse grain contents. A series of cyclic triaxial tests
84 were performed for this purpose. Emphasis was put on keeping the fine dry density ρ_{d-f}
85 constant in all samples, allowing a constant suction of soil mixtures whatever the f_v value. A
86 multi-step loading procedure at various deviator stress amplitudes of 10, 15, 20, 25 and 30
87 kPa was applied, with a number of loading cycles at 90000 for each stress amplitude. Two
88 target water contents of fine soil (17.6% and 10.6%) and five coarse grain contents (0%, 10%,
89 20%, 35% and 45%) were considered.

90

91 MATERIALS AND SAMPLE PREPARATION

92 Considering the difficulty of obtaining intact interlayer soil from the field, the reconstituted
93 soils consisting of fine soil and coarse grains were fabricated in the laboratory. For re-
94 constituting the fine soil, nine different commercial soils (Table 1) are mixed with the pre-
95 determined proportions, to obtain a similar grain size distribution curve of fines from
96 ‘Senissiat site’ (Fig. 1). Note that in this study fines refer to the soil finer than ballast in the
97 field, which correspond to a mixture encompassing grains of clay to sand sizes. The liquid
98 limit and plasticity index of reconstituted fine soil were 32% and 20%, respectively,
99 classifying the soil as CL based on the universal soil classification system. Fig. 2 presents the
100 standard proctor compaction curve of fine soil, defining an optimum water content $w_{opt-f} =$
101 13.7% and a maximum dry density $\rho_{dmax-f} = 1.82 \text{ Mg/m}^3$.

102 For the coarse grains, based on a parallel gradation method applied by Wang et al. [13] ,
 103 Qi et al. [14] and Su et al. [15-17] , which was verified later by Qi et al. [18] , micro-ballast
 104 was adopted to replace the real ballast in Fig.1. A parameter of coarse grain content f_v (Wang
 105 et al. [13]), defined as the ratio of the volume of coarse grains V_c to the total volume of
 106 fine/coarse mixture V (Eq. (1)), was adopted in this study. All voids and water were assumed
 107 to be contained in the fine soil (Fig. 3). Thus, under a given f_v , the dry density of fine soil ρ_{d-f}
 108 and the water content of fine soil w_f , the masses of coarse grain m_{s-c} , fine grain m_{s-f} and the
 109 water content of fine soil m_{w-f} were calculated by Eqs. (2) – (4), respectively.

$$f_v = \frac{V_c}{V} = \frac{V_c}{V_c + V_f} = \frac{V_c}{V_c + V_{s-f} + V_{w-f} + V_{a-f}} \quad (1)$$

112 where V_f is the volume of fine soil; V_{s-f} , V_{w-f} and V_{a-f} are the volume of fine grains, water and
 113 air in the fine soil, respectively.

$$m_{s-c} = V_c \cdot G_{s-c} \cdot \rho_w = f_v \cdot V \cdot G_{s-c} \cdot \rho_w \quad (2)$$

$$m_{s-f} = \rho_{d-f} \cdot V_f = \rho_{d-f} \cdot V \cdot (1 - f_v) \quad (3)$$

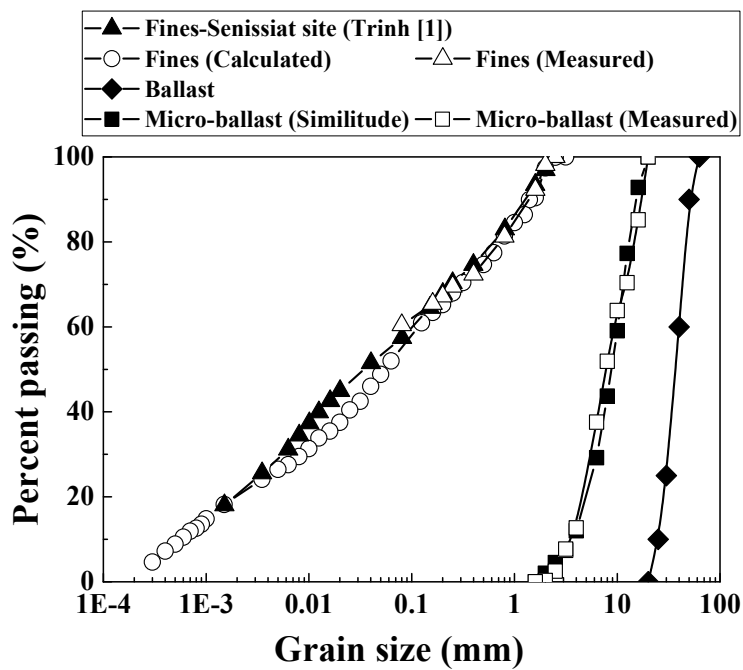
$$m_{w-f} = w_f \cdot m_{s-f} \quad (4)$$

119 where G_{s-c} is the specific gravity of coarse grains ($= 2.68 \text{ Mg/m}^3$); ρ_w is the water unit mass.

121 Table 1. Nine different commercial soils

Soil classification	Commercial Soil	Mass proportion (%)	The range of grain size (mm)
Sand	HN34	3.3	0.063 - 0.50
	HN31	3.3	0.16 - 0.63

	HN0.4-0.8	6.7	0.25 - 1
	HN0.6-1.6	6.7	0.32 - 2
	HN1-2.5	13.3	0.32 - 3.20
	C4	16.7	0.0009 - 0.50
	C10	20	0.0009 - 0.25
Clay	Speswhite	23.3	0.0003 - 0.01
	Bentonite	6.7	0.001 - 0.01



122 Fig. 1. Grain size distribution curves of fine soil and micro-ballast (after Wang et al. [4])

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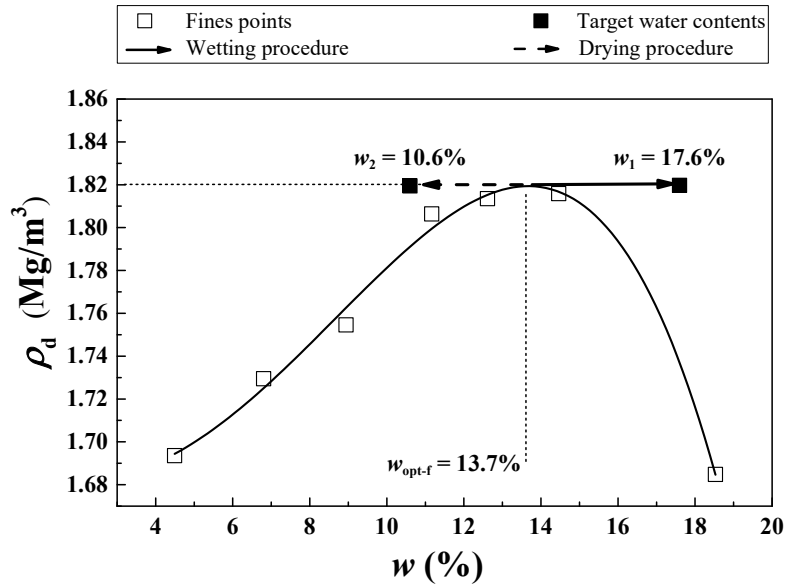


Fig. 2. Preparation of samples at two target water contents with respect to compaction curve of the fine soil

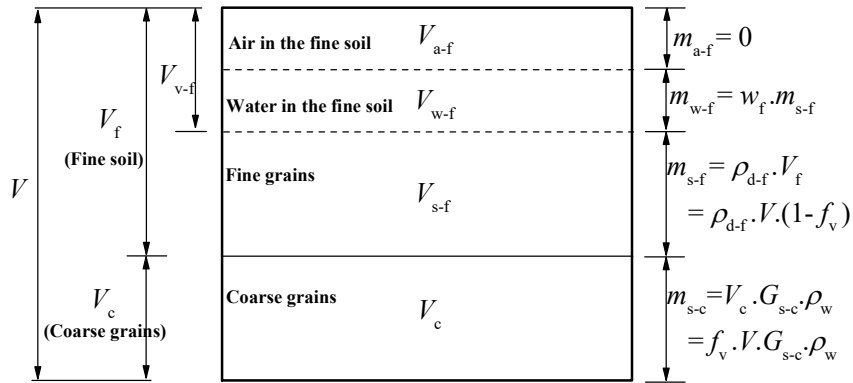


Fig. 3. Constitution of fine/coarse soil mixture

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In order to prepare a sample at a target f_v value and a target water content w_f , the fine soil was prepared at $w_{opt-f} = 13.7\%$, then stored in a container for 24h for the purpose of moisture homogenization. After that, the fine soil was mixed with coarse grains at the pre-determined mass to reach the target f_v value. The soil mixture was then dynamically compacted in three layers, with the equivalent amounts of fine soil and coarse grains for each layer, to attain a

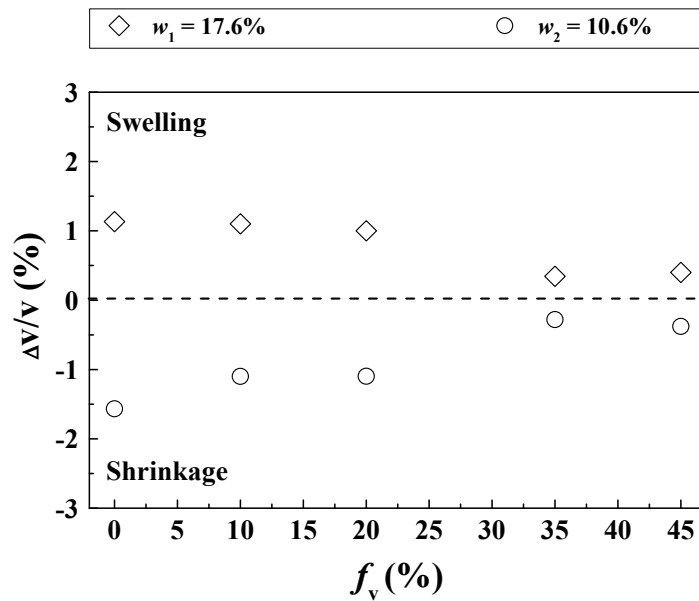
133 size of 100 mm diameter and 200 mm height. Note that the fine soil was kept at the maximum
134 dry density $\rho_{d_{\max-f}} = 1.82 \text{ Mg/m}^3$ for all samples with varying f_v values. With a higher f_v value,
135 more compaction energy was needed for the soil mixtures; thereby, a higher dry density ρ_d of
136 sample is obtained (Table 2). It is worth noting that when $f_v > f_{v\text{-cha}}$, the coarse grains
137 constitute the skeleton of soil mixture (Wang et al. [4]). For this fabric, two categories of fine
138 soil were identified by Su et al. [19] – dense fine soil between coarse grains and loose fine soil
139 surrounded by coarse grains. Accordingly, a relative high ρ_{d-f} and low ρ_{d-f} was obtained for
140 dense fines and loose fines, respectively, even though the global ρ_{d-f} of fine soil remained
141 constant (1.82 Mg/m^3).

142 After compacting a sample at a target f_v value, either a wetting or a drying process was
143 adopted for the sample to reach the target water contents w_f (Fig. 2): $w_1 = 17.6\%$ ($S_r = 100\%$)
144 on the wet side and $w_2 = 10.6\%$ ($S_r = 60\%$) on the dry side. The approach of wetting or drying
145 from $w_{\text{opt-f}} = 13.7\%$ to the target w_f value proposed by Su et al. [19] was applied: in the case of
146 drying, the sample was each time exposed to the air for 1 h in the laboratory, and then covered
147 with plastic film for at least 7 h equilibration. In the case of wetting, 10 g water was sprayed
148 on the sample each time, and then wrapped it with plastic film and conserved for the same
149 equilibration time of at least 7 h.

150 During the wetting and drying processes, the volume change of samples under different f_v
151 values was recorded (Fig. 4). It can be observed that at a given f_v value, the swelling of sample
152 upon wetting from $w_{\text{opt-f}} = 13.7\%$ to $w_1 = 17.6\%$ or shrinkage of that upon drying from $w_{\text{opt-f}} =$
153 13.7% to $w_2 = 10.6\%$ occurred. Moreover, the magnitude of swelling-shrinkage of sample
154 decreased with the increase of f_v , which was attributed to (i) a reduction of fine soil, which

155 was sensitive to water content change and (ii) part of total stress supported by the coarse grain
 156 skeleton at $f_v > f_{v\text{-cha}}$ (Wang et al. [4]). This response of fine/coarse soil mixture appeared to be
 157 dominated by the fine matrix for $f_v < 20\%$ but the coarse grain skeleton for $f_v > 35\%$ (Fig. 4).

158 The measured dry density ρ_d of sample after wetting or drying is shown in Table 2.



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Fig. 4. Variations of sample volume with f_v at two target water contents

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Table 2. Experimental program of cyclic triaxial tests

f_v (%)	Initial water content $w_{\text{opt-f}}$ (%)	Target w_f (%)	Target S_r (%)	Target $\rho_{d\text{max-f}}$ (Mg/m ³)	Target ρ_d (Mg/m ³)	Measured ρ_d (Mg/m ³)
0		17.6	100		1.82	1.80
		10.6	60			1.85
10		17.6	100		1.91	1.88
		10.6	60			1.93
20	13.7	17.6	100	1.82	1.99	1.97
		10.6	60			2.01
35		17.6	100		2.12	2.11
		10.6	60			2.13
45		17.6	100		2.21	2.20
		10.6	60			2.22

Note: f_v represents the volumetric ratio of coarse grains to fine/coarse soil mixtures. w_{opt-f} , w_f , S_r and ρ_{dmax-f} represent the optimum water content, water content, degree of saturation and maximum dry density of fine soil, respectively. ρ_d represents the dry density of soil mixtures sample. Measured ρ_d represents the dry density of soil mixtures sample after wetting or drying from compaction water content w_{opt-f} to target w_f .

162

163 CYCLIC TRIAXIAL TESTS

164 The cyclic triaxial apparatus used by Wang et al. [20] was adopted in this study, hosting a
165 sample with 100 mm diameter and 200 mm height. Using a 50 kN hydraulic actuator enabled
166 a force or displacement controlled mode to be applied in both monotonic and cyclic triaxial
167 tests. As for the cyclic loading, different signal shapes, amplitudes, frequencies and large
168 number of cycles (up to several millions) can be applied. A linear variable displacement
169 transducer (LVDT) was adopted to monitor the axial displacement, with a minimum capacity
170 of ± 0.1 mm. Considering the height (= 200mm) of the sample, the corresponding minimum
171 measurement capacity of axial strain was $\pm 0.05\%$. A force sensor installed at the bottom was
172 adopted to monitor the axial force.

173 A series of cyclic triaxial tests were performed on the samples at two target w_f values (w_1
174 = 17.6% and $w_2 = 10.6\%$) and five f_v values (0%, 10%, 20%, 35% and 45%) **with drainage**
175 **valves open to air**. A constant confining pressure $\sigma_3 = 30$ kPa was applied, corresponding to
176 the estimated average horizontal stress in the field by the consideration of train loadings, the
177 depth of interlayer soil and the Poisson's ratio (Duong et al. [21]). In the case of $w_1 = 17.6\%$
178 ($S_r = 100\%$), after applying the confining pressure $\sigma_3 = 30$ kPa, an overnight consolidation of

179 the sample was adopted, with both the top and bottom porous disks exposed to the air. This
180 allowed for the fully dissipation of generated pore water pressure. On the contrary, in the case
181 of $w_2 = 10.6\%$ ($S_r = 60\%$), after application of the confining pressure $\sigma_3 = 30$ kPa, the cyclic
182 loading was directly applied, because only air was expected to be expelled.

183 Fig. 5 shows a sine-shaped signal applied at a frequency of 1.78 Hz, corresponding to that
184 excited by two bogies at a train speed of 50 km/h. A multi-step loading procedure proposed by
185 Gidel et al. [5] , applied later by Wang et al. [4] was adopted, which can not only reduce the
186 number of tests but also avoid experimental dispersion due to the variability of sample. Fig. 6
187 depicts a multi-step loading procedure with various deviator stress amplitudes Δq of 10, 15, 20,
188 25 and 30 kPa, and a number of loading cycles $N = 90000$ for each Δq value. The deviator
189 stress amplitude Δq was defined as the difference of maximum deviator stress q_{\max} and
190 minimum deviator stress q_{\min} . These Δq values corresponded to the vertical stresses at varying
191 depths of interlayer soil in the field, as reported by Lamas-Lopez [22]. The number of loading
192 cycles $N = 90000$ was considered large enough for the stabilization of permanent strain under
193 a given Δq value, according to the number applied in previous studies (Gidel et al. [5]; Trinh
194 et al. [8]; Duong et al. [12]; Lamas-Lopez et al. [22]). Note that a constant cyclic stress ratio
195 $\Delta q / \Delta p = 3$ was adopted, which represented the typical stress path in the interlayer (Trinh et al.
196 [8]). During the tests, the deviator stress and axial strain were recorded.

197

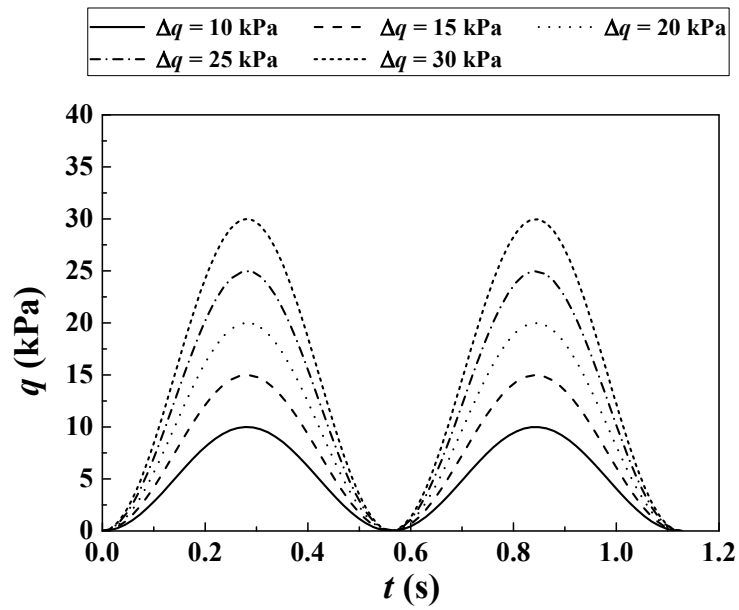


Fig. 5. Typical sine-shaped signals applied in cyclic triaxial tests

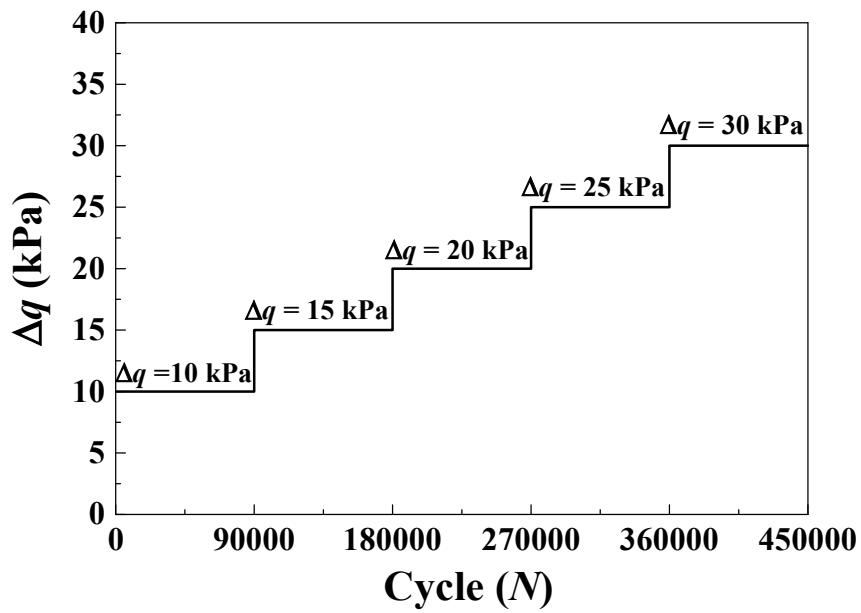


Fig. 6. Multi-step loading procedure with various stress amplitudes Δq

198

199 RESULTS AND DISCUSSIONS

200 *Effect of water content on the evolution of permanent strain with loading cycles*

201 Fig.7 depicts the evolution of axial strain ε_1 with loading cycles N at $f_v = 0\%$ and $w_1 = 17.6\%$
 202 under various Δq values ranging from 10 kPa to 30 kPa. It can be observed that under a given
 203 Δq value, the axial strain ε_1 increased significantly at the beginning of loading cycles, and
 204 then gradually stabilized. With the increase of Δq , the axial strain ε_1 increased significantly. In
 205 addition, the axial strain ε_1 could be separated into two parts: a plastic strain ε_1^p and a resilient
 206 strain ε_1^r . For the plastic strain ε_1^p , it increased with N and the increasing rate decreased with N
 207 under a constant Δq value. On the contrary, the resilient strain ε_1^r remained almost unchanged
 208 at a given Δq value and increased with increasing Δq .

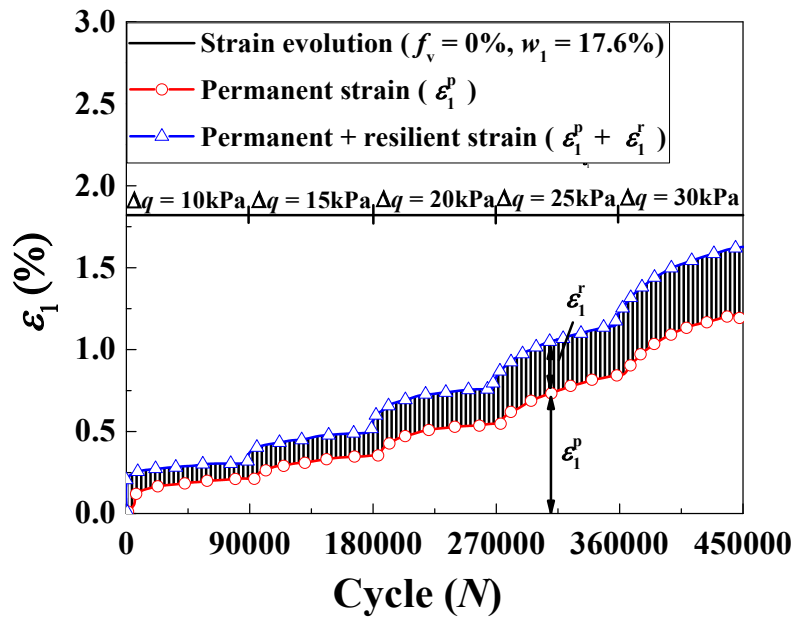


Fig. 7. Determination of permanent strain and resilient strain

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210 Fig. 8 presents the evolutions of ε_1^p with N under various Δq values for $f_v = 0\%$ and three
 211 water contents. At $w_1 = 17.6\%$, the permanent strain ε_1^p increased with N sharply at the initial
 212 loading cycles N , and then gradually reached stabilization under a specific Δq value. With the

213 increase of Δq , the permanent strain ε_1^p grew significantly. The similar observation can be
 214 made in the case of $w_{\text{opt-f}} = 13.7\%$ (obtained by Wang et al. [4]) and $w_2 = 10.6\%$. In addition, it
 215 can be observed that the decrease of water content from $w_1 = 17.6\%$ to $w_{\text{opt-f}} = 13.7\%$ or $w_2 =$
 216 10.6% led to a pronounced decrease of ε_1^p . This could be explained by the contribution of
 217 suction, as reported by Duong et al. [12-] and Jing [24].

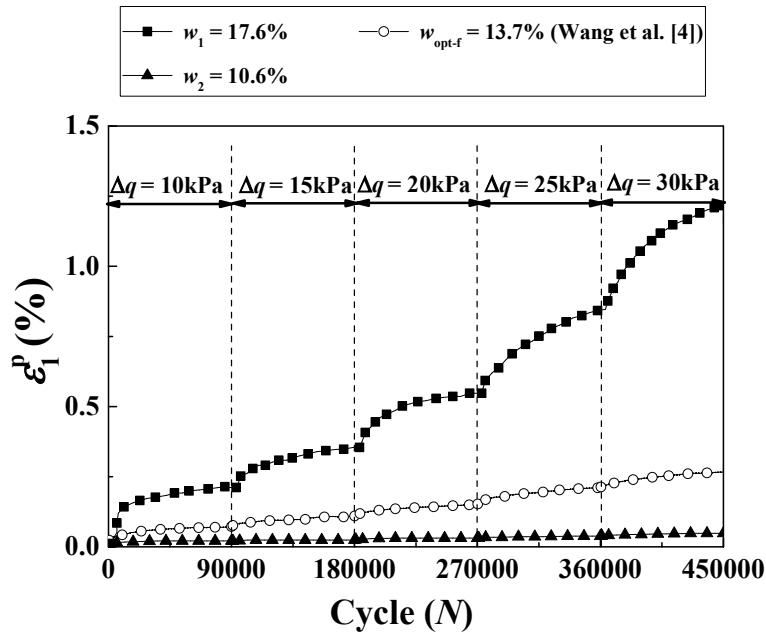
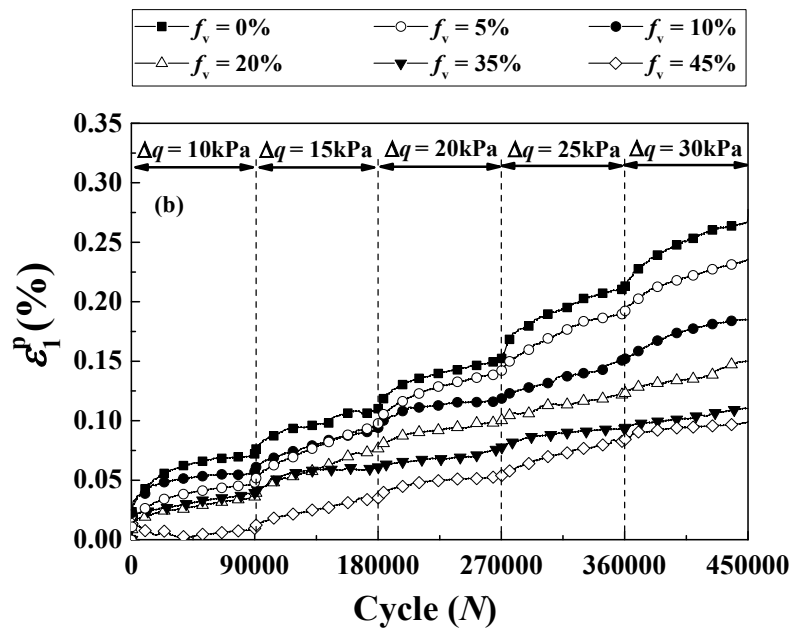
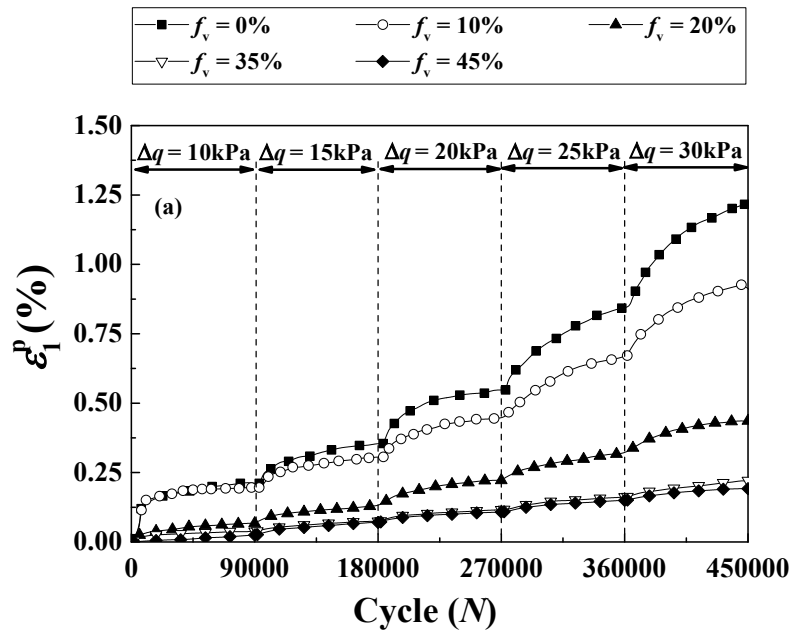


Fig. 8. Evolutions of permanent strain with number of cycles at $f_v = 0\%$ and different Δq values for three different water contents

218

219 Fig. 9 shows the effect of water content on the evolutions of ε_1^p with N under various f_v
 220 values. In the case of $w_1 = 17.6\%$ (Fig. 9 (a)), under a given f_v value, the increasing trend of ε_1^p
 221 versus N decreased with the increase of N at a constant Δq value. However, ε_1^p increased
 222 significantly with the increase of Δq . In addition, it can be found that with the increase of f_v ,
 223 the permanent strain ε_1^p decreased. This was attributed to the reinforcement effect of coarse

224 grains in the soil mixtures. Moreover, different decreasing trends of ε_1^p with f_v was evidenced
225 at $f_v \leq 20\%$ and $f_v \geq 35\%$: a pronounced decrease of ε_1^p with f_v at $f_v \leq 20\%$ and a slight decrease
226 of that at $f_v \geq 35\%$. This could be explained by the transition of two soil fabrics: a fine soil
227 dominated fabric at $f_v \leq 20\%$ and a coarse grain dominated fabric at $f_v \geq 35\%$. **Note that for the**
228 **coarse grain skeleton fabric ($f_v > 35$), two categories of fine soil were identified by Su et al. [19]**
229 **– dense fines (with ρ_{d-f} higher than 1.82 Mg/m^3) and loose fines (with ρ_{d-f} lower than 1.82**
230 **Mg/m^3). In spite of this inhomogeneous distribution of fine soil in the mixture, the suction of**
231 **mixture was found to be mainly controlled by the global dry density of fines $\rho_{d-f} = 1.82 \text{ Mg/m}^3$**
232 **(Su et al. [25]).** The similar observation was made at $w_{\text{opt-f}} = 13.7\%$ (Fig. 9 (b)), which was
233 obtained by Wang et al. [4]. At $w_2 = 10.6\%$ (Fig. 9 (c)), the ε_1^p at $f_v = 0\%$ was very small, with
234 a maximum value around 0.05% identified. The ε_1^p at $f_v = 10\%$, 20% , 35% and 45% was
235 expected to be smaller than that at $f_v = 0\%$. Considering the minimum measurement capacity
236 (0.05%) of axial strain by the adopted LVDT, inaccuracy measurements of ε_1^p at $f_v = 10\%$,
237 20% , 35% and 45% were generated. Therefore, these results are not presented in Fig. 9 (c).



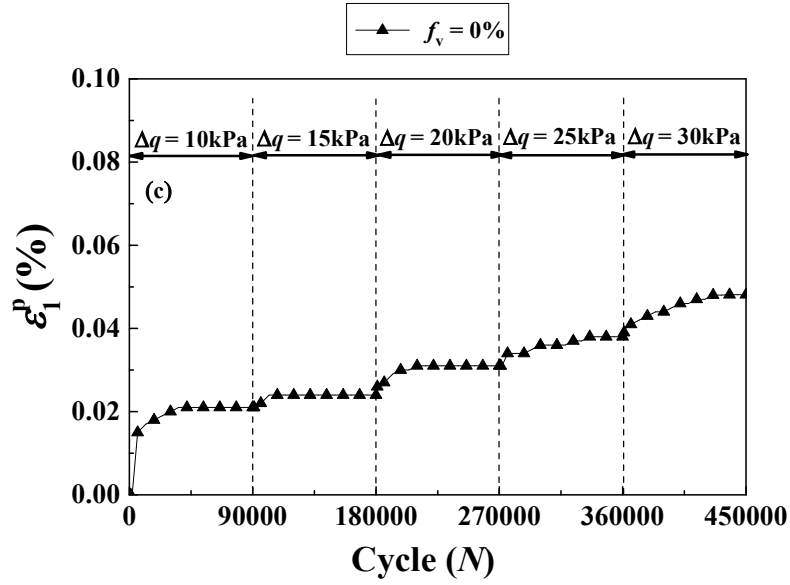


Fig. 9. Evolutions of permanent strain with number of cycles at different f_v and Δq values for various water contents: (a) $w_1 = 17.6\%$; (b) $w_{\text{opt-f}} = 13.7\%$ (after Wang et al. [4]); (c) $w_2 = 10.6\%$

238

239 *Estimation of permanent strain*

240 Considering a multi-step loading procedure applied in this study, the loading history would
 241 significantly affect the evolution of permanent strain ε_1^p with N under various water contents
 242 and coarse grain contents. Thereby, the estimation approach proposed by Gidel et al. [5] was
 243 adopted to eliminate such effect of loading history on permanent strain. As shown in Fig. 10,
 244 the permanent strain ε_1^p evolved with N at two successive loading levels: loading level M and
 245 loading level M+1. To eliminate the effect of loading level M on permanent strain at loading
 246 level M+1, the increment of permanent strain $\delta\varepsilon_1^{p(M+1)}$ at loading level M+1 was transferred
 247 to start at $\varepsilon_1^p = 0$ and $N = 0$, which was the starting point of permanent strain at loading level
 248 M. As shown in Eq. (1), the estimated permanent strain $\varepsilon_1^{p(M+1)}$ at loading level M+1 without

249 the influence of loading history can be determined:

$$250 \quad \varepsilon_1^{p(M+1)} = \varepsilon_1^{p(M)} + \delta\varepsilon_1^{p(M+1)} \quad (5)$$

251 where $\varepsilon_1^{p(M+1)}$ represents the estimated permanent strain at loading level M+1, $\varepsilon_1^{p(M)}$
252 represents measured permanent strain at loading level M, $\delta\varepsilon_1^{p(M+1)}$ represents the increment
253 of permanent strain at loading level M+1. Note that the estimated $\varepsilon_1^{p(M+1)}$ at $N = 90000$
254 coincides with the measured $\varepsilon_1^{p(M+1)}$ at $N = 180000$. In addition, the slope θ of estimated
255 $\varepsilon_1^{p(M+1)}$ with N after $N = 90000$ was kept the same as that in the last cycle of
256 measured $\varepsilon_1^{p(M+1)}$, which enabled a linear increase of estimated $\varepsilon_1^{p(M+1)}$ with N after $N =$
257 90000 in Fig. 10.

258 Fig. 11 presents the evolution of the estimated ε_1^p with N at $f_v = 0\%$ and $w_1 = 17.6\%$ under
259 various Δq values. With the increase of Δq , the estimated ε_1^p appears to grow. In addition, it
260 can be found that the estimated ε_1^p was larger than the measure ε_1^p under the same Δq and N
261 values (except $\Delta q = 10$ kPa). For instance, at $\Delta q = 30$ kPa and $N = 450000$, the estimated $\varepsilon_1^p =$
262 1.75% was much larger than the measured $\varepsilon_1^p = 1.20\%$. This could be attributed to the effect of
263 loading history. Different from the estimated ε_1^p under a constant $\Delta q = 30$ kPa, the measured
264 ε_1^p experienced a series of lower stress amplitudes $\Delta q = 10, 15, 20$ and 25 kPa prior to $\Delta q = 30$
265 kPa, which resulted in a smaller value. Considering that a relative good accuracy for the
266 estimated ε_1^p could be obtained at the first loading stage (e.g. $N = 0 \sim 90000$ in this study), as
267 reported by Gidel et al. [5] and confirmed later by Lamas-Lopez [22] and Wang et al. [4], the
268 estimated end-stage permanent strains ε_1^p (at $N = 90000$) under various Δq values were
269 selected for further analysis.

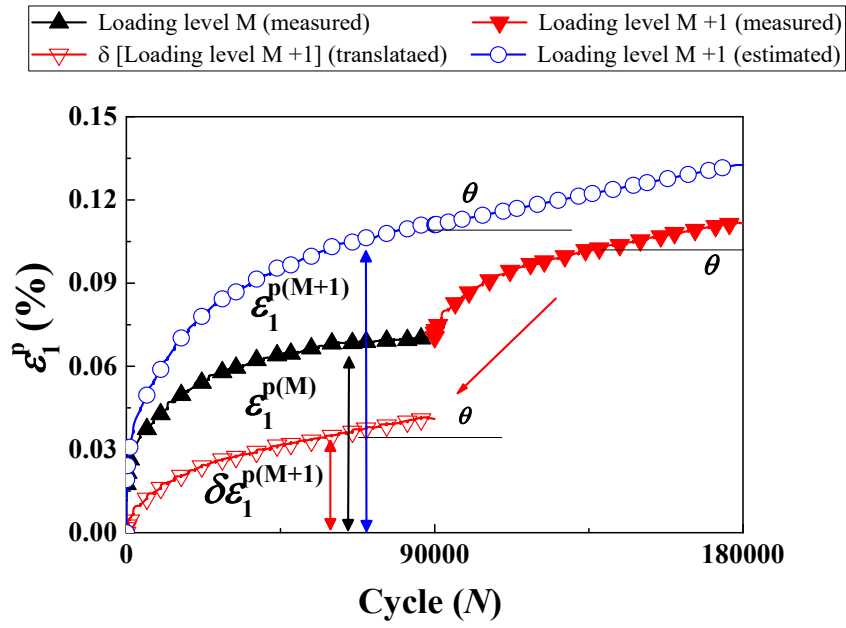


Fig. 10. Estimation method of ε_1^p proposed by Gidel et al. [5]

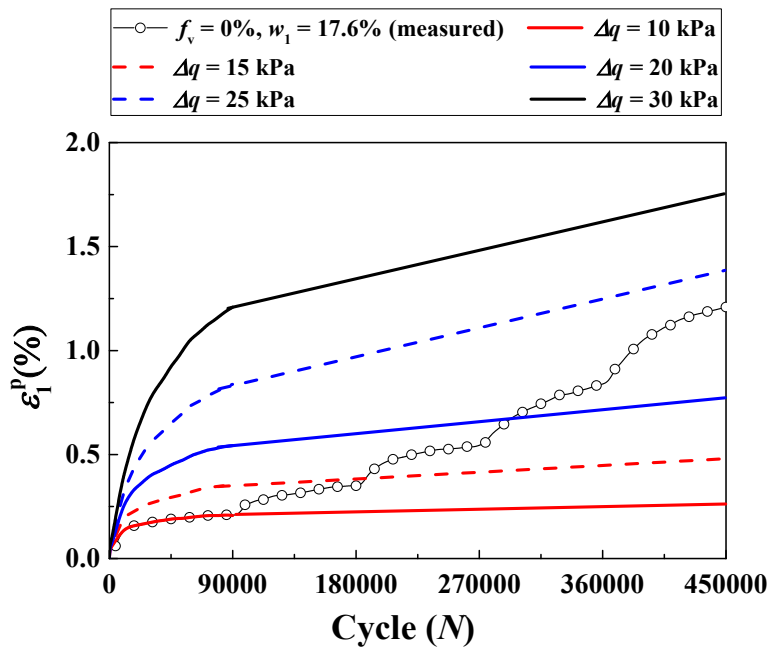


Fig. 11. Evolutions of estimated permanent strain with number of cycles at various Δq values for $f_v = 0\%$ and $w_1 = 17.6\%$

271 *Effect of water content on the variations of estimated end-stage ε_1^p with Δq*

272 Fig. 12 shows the variations of estimated end-stage ε_1^p with Δq under different f_v values for
273 two water contents. In the case of $w_1 = 17.6\%$ ($S_r = 100\%$, Fig. 12 (a)), the estimated end-stage
274 ε_1^p increased with increasing Δq for various f_v values. Under a given Δq , the ε_1^p decreased with
275 the increase of f_v , evidencing the reinforcement effect of coarse grains. The similar
276 observation was obtained at $w_{\text{opt-f}} = 13.7\%$ ($S_r = 78\%$) (Fig. 12 (b)). In addition, the decrease
277 of water content from $w_1 = 17.6\%$ to $w_{\text{opt-f}} = 13.7\%$ led to a decline of the estimated end-
278 stage ε_1^p , owing to the contribution of suction.

279 **The study of Dong et al. [12] was carried out at a constant dry density of mixture ρ_d for**
280 **varying f_v values (50.3%, 55.5% and 61.4%), and therefore the fines fraction density ρ_{d-f}**
281 **decreased as f_v increased (Table 3).** Fig. 13 shows that the estimated end-stage ε_1^p increased
282 with Δq for the three f_v values and three w values. Under saturated conditions ($w = 12\%$, Fig.
283 13 (a)), an increase of f_v led to a decrease of ε_1^p under a constant Δq , which was consistent with
284 that observed in Fig. 12 (a). On the contrary, under unsaturated conditions ($w = 6\%$ and $w =$
285 4% , Figs. 12 (b) - (c)), an increase of f_v resulted in an increase of the estimated end-stage ε_1^p
286 under a constant Δq , which was contradictory with the observation in Fig. 12 (b). This could
287 be explained by the fact that the permanent strain behavior of soil mixtures was affected by
288 both the reinforcement effect of coarse grains and the effect of suction in fines. Under
289 saturated conditions, without the effect of suction, the reinforcement effect of f_v played a
290 dominant role in the permanent strain behavior of mixtures. In this case, an increase of f_v
291 induced a decrease of ε_1^p (Fig. 12 (a) and Fig. 13 (a)). By contrast, under unsaturated
292 conditions, both the reinforcement effect of coarse grains and the effect of suction in fines

293 affected the permanent strain behavior. Note that the suction of fines fraction was strongly
294 related to its ρ_{d-f} under a constant water content, as evidenced by Romero et al. [26] and Gao
295 and Sun [27] . In the present study, the fine soil was controlled at $\rho_{dmax-f} = 1.82 \text{ Mg/m}^3$ (Table
296 2), leading to an unchanged suction (= 739 kPa in Wang et al. [13]) at $w_{opt-f} = 13.7\%$ under
297 varying f_v values. This was supported by the findings of Su et al. [25] on the same fine/coarse
298 soil mixture, who reported that the soil-water retention curve was only affected by the dry
299 density of fine soil ρ_{d-f} , while independent of coarse grain content f_v . In this case, the
300 reinforcement effect of f_v on the permanent strain behavior was clearly identified: an increase
301 of f_v led to a smaller ε_1^p (Fig. 12 (b)). Conversely, in the study of Duong et al. [12] , the ρ_{d-f} of
302 fine soil declines from 1.33 Mg/m^3 to 1.17 and 0.94 Mg/m^3 with the increasing f_v values from
303 50.3% to 55.5% and 61.4% (Table 3), which would result in a decrease of suction within the
304 fine fraction. In this case, the negative effect of decreasing suction prevailed on the positive
305 reinforcement effect of increasing f_v . As a result, the ε_1^p increased with increasing f_v . (Fig. 13
306 (b) – (c)).

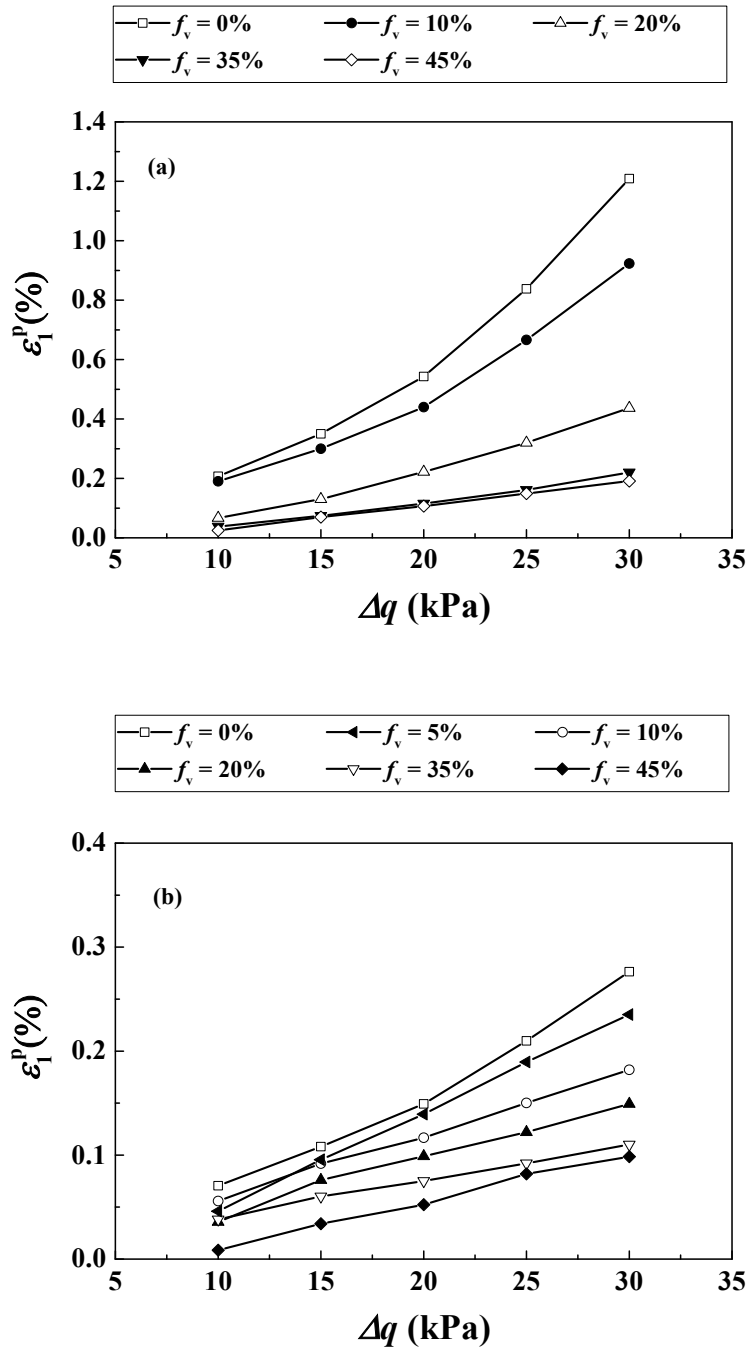
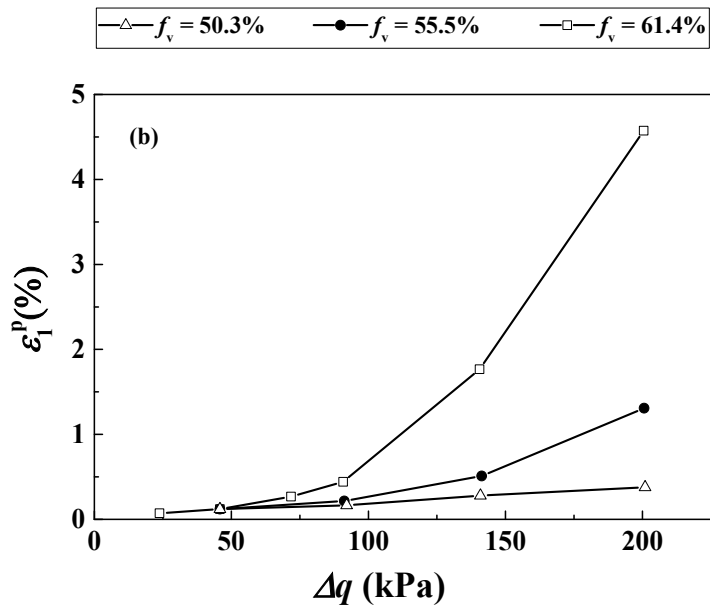
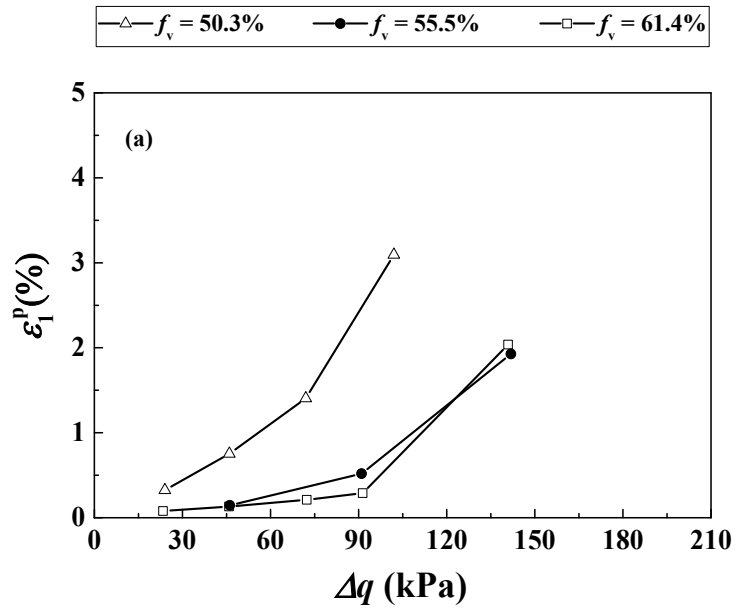


Fig. 12. Variations of estimated end-stage permanent strain with Δq at different f_v values for various water contents: (a) $w_1 = 17.6\%$; (b) $w_{opt-f} = 13.7\%$



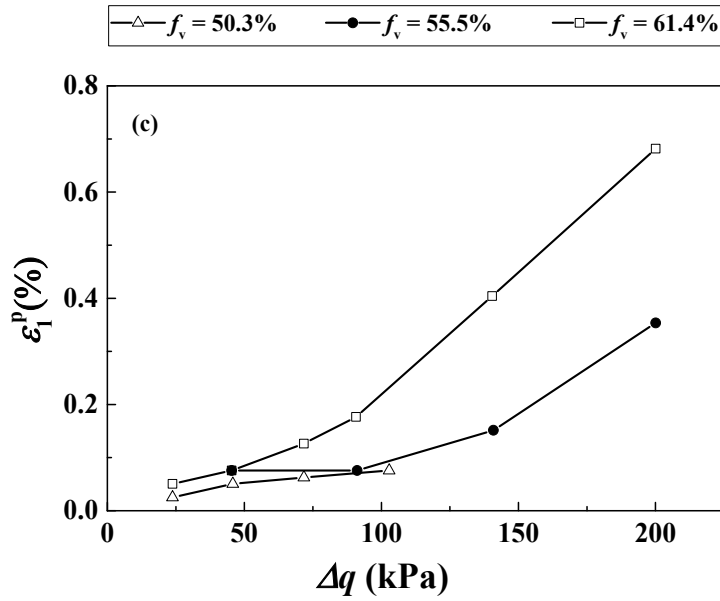


Fig. 13. Variations of estimated end-stage permanent strain with Δq at different f_v values for various water contents: (a) $w = 12\%$; (b) $w = 6\%$; (3) $w = 4\%$ (after Duong et al. [12])

307

Table 3. Soil properties of Duong et al. [12]

f_v (%)	w (%)			S_r (%)			ρ_d (Mg/m ³)	ρ_{d-f} (Mg/m ³)
50.3	4	6	12	32	49	100	2.01	1.33
55.5	4	6	12	32	49	100		1.17
61.4	4	6	12	32	49	100		0.94

Note: w represents the water content of soil mixtures.

308

309 CONCLUSIONS

310 To investigate the effect of water content on permanent strain ε_1^p of fine/coarse soil mixtures
 311 under varying coarse grain contents, a series of cyclic triaxial tests were performed. For the
 312 cyclic triaxial test, a multi-step loading procedure under various stress amplitudes Δq of 10, 15,

313 20, 25 and 30 kPa was applied, with a number of loading cycles $N = 90000$ for each Δq value.
314 Two target fines water contents w_f ($w_1 = 17.6\%$ and $w_2 = 10.6\%$) and five coarse grain
315 contents f_v (0%, 10%, 20%, 35% and 45%) were considered. The estimation approach of
316 ε_1^p proposed by Gidel et al. [5] was adopted to eliminate the effect of loading history on ε_1^p .
317 Through the comparison of the present study and the study of Duong et al. [12], the effects of
318 water content and coarse grain content on the permanent strain behavior of fine/coarse soil
319 mixtures were clarified. The following conclusions to be drawn:

320 A decrease of water content led to a decrease of permanent strain ε_1^p , due to the
321 contribution of suction. An increase of f_v gave rise to a decrease of ε_1^p , owing to the
322 reinforcement effect of f_v . The comparison of present study and the study of Duong et al. [12]
323 indicated a significant effect of sample preparation approach on the permanent strain behavior
324 of soil mixtures. In the present study, a constant ρ_{d-f} of fine soil fraction was maintained,
325 leading to an unchanged suction of soil mixtures with varying f_v values. In this case, the
326 reinforcement effect of f_v on the permanent strain behavior was clearly identified: an increase
327 of f_v led to a decrease of ε_1^p . Conversely, in the study of Duong et al. [12], with a global dry
328 density kept constant, an increase of f_v led to a decrease of the dry density of fines ρ_{d-f} and
329 consequently a decrease of suction. In that case, when the negative effect of decreasing
330 suction prevailed on the positive reinforcement effect of increasing f_v , the ε_1^p increased with
331 the increase of f_v .

332

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334 ACKNOWLEDGEMENTS

335 This work was supported by the China Scholarship Council (CSC) and Ecole des Ponts
336 ParisTech.

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427

428 NOTATIONS

ε_1	axial strain
ε_1^r	resilient strain
ε_1^p	permanent strain
$\varepsilon_1^{p(M)}$	measured permanent strain at loading level M
$\varepsilon_1^{p(M+1)}$	estimated permanent strain at loading level M+1
$\delta\varepsilon_1^{p(M+1)}$	increment of permanent strain at loading level M+1
f_v	volumetric coarse grain content
$f_{v\text{-cha}}$	characteristic volumetric coarse grain content
N	number of loading cycles
ρ_d	dry density of sample
ρ_{d-f}	dry density of fine soil
$\rho_{d\text{max-f}}$	maximum dry density of fine soil
q	deviator stress

q_{\max}	maximum deviator stress
q_{\min}	minimum deviator stress
Δq	deviator stress amplitude
Δp	mean stress amplitude
S_r	degree of saturation
$w_{\text{opt-f}}$	optimum water content of fine soil
w_f	water content of fine soil
σ_3	confining pressure
θ	increasing slope of permanent strain with loading cycles at the end of loading cycles for a given stress amplitude

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