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

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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  $\varepsilon_1^p$  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  $\varepsilon_1^p$ . In present study, a constant fine dry density  $\rho_{d-f}$  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  $\varepsilon_1^p$  was identified. By contrast, in previous studies, the  
30 global dry density of mixture  $\rho_d$  was kept constant, resulting in a decrease of  $\rho_{d-f}$  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  $\varepsilon_1^p$   
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 ( $2.4 \text{ Mg/m}^3$ , 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  $\rho_{d-f}$   
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  $\rho_{d-f}$   
 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.

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

111 (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.

$$m_{s-c} = V_c \cdot G_{s-c} \cdot \rho_w = f_v \cdot V \cdot G_{s-c} \cdot \rho_w$$

114 (2)

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

115 (3)

$$m_{w-f} = w_f \cdot m_{s-f}$$

116 (4)

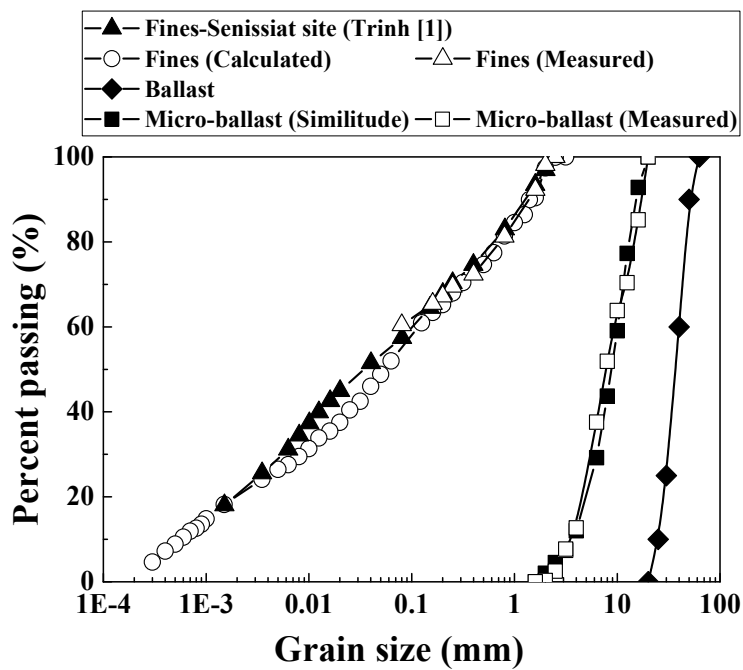
117 where  $G_{s-c}$  is the specific gravity of coarse grains ( $= 2.68 \text{ Mg/m}^3$ );  $\rho_w$  is the water unit mass.

120

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])

123

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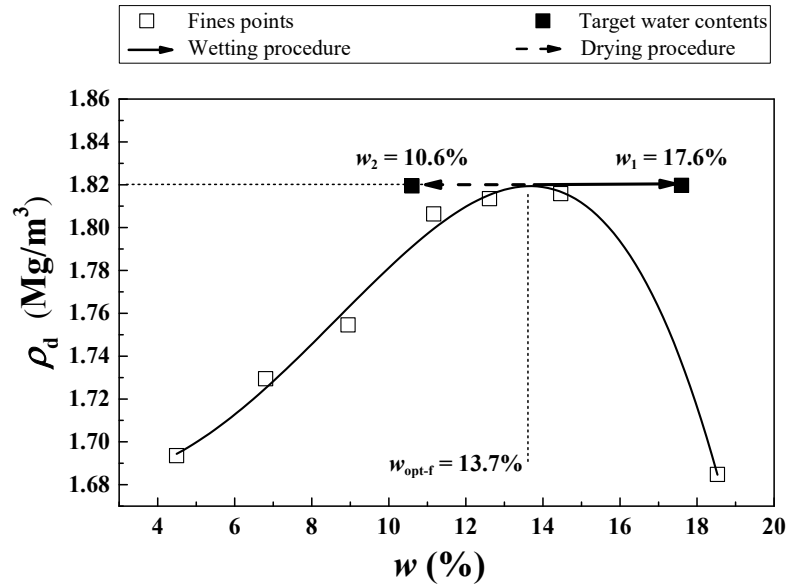


Fig. 2. Preparation of samples at two target water contents with respect to compaction curve of the fine soil

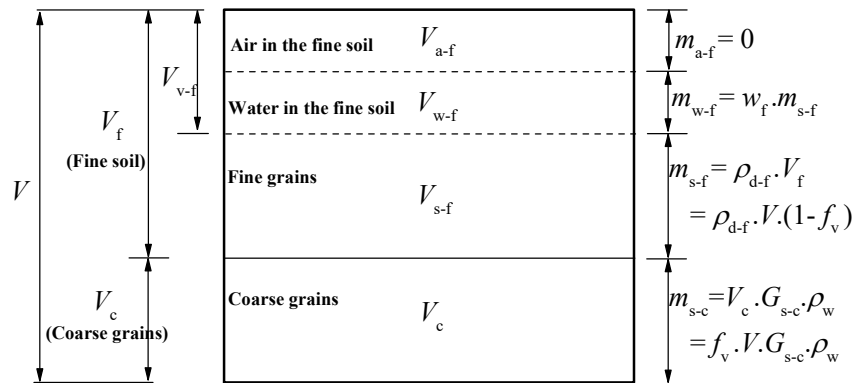


Fig. 3. Constitution of fine/coarse soil mixture

125

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127

128 In order to prepare a sample at a target  $f_v$  value and a target water content  $w_f$ , the fine soil

129 was prepared at  $w_{opt-f} = 13.7\%$ , then stored in a container for 24h for the purpose of moisture

130 homogenization. After that, the fine soil was mixed with coarse grains at the pre-determined

131 mass to reach the target  $f_v$  value. The soil mixture was then dynamically compacted in three

132 layers, with the equivalent amounts of fine soil and coarse grains for each layer, to attain a

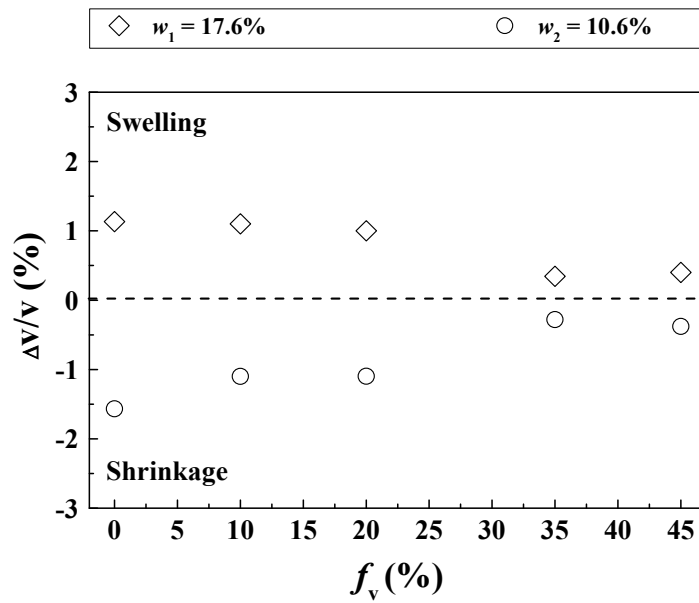
133 size of 100 mm diameter and 200 mm height. Note that the fine soil was kept at the maximum  
134 dry density  $\rho_{d\max-f} = 1.82 \text{ Mg/m}^3$  for all samples with varying  $f_v$  values. With a higher  $f_v$  value,  
135 more compaction energy was needed for the soil mixtures; thereby, a higher dry density  $\rho_d$  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  $\rho_{d-f}$  and low  $\rho_{d-f}$  was obtained for  
140 dense fines and loose fines, respectively, even though the global  $\rho_{d-f}$  of fine soil remained  
141 constant ( $1.82 \text{ Mg/m}^3$ ).

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  $\rho_d$  of sample after wetting or drying is shown in Table 2.



159

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

160

161

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 <sup>3</sup> )	Target $\rho_d$ (Mg/m <sup>3</sup> )	Measured $\rho_d$ (Mg/m <sup>3</sup> )
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  $\rho_{dmax-f}$  represent the optimum water content, water content, degree of saturation and maximum dry density of fine soil, respectively.  $\rho_d$  represents the dry density of soil mixtures sample. Measured  $\rho_d$  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  $\pm 0.1$ 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  $\Delta q$  of 10, 15, 20,  
188 25 and 30 kPa, and a number of loading cycles  $N = 90000$  for each  $\Delta q$  value. The deviator  
189 stress amplitude  $\Delta q$  was defined as the difference of maximum deviator stress  $q_{\max}$  and  
190 minimum deviator stress  $q_{\min}$ . These  $\Delta q$  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  $\Delta q$  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.

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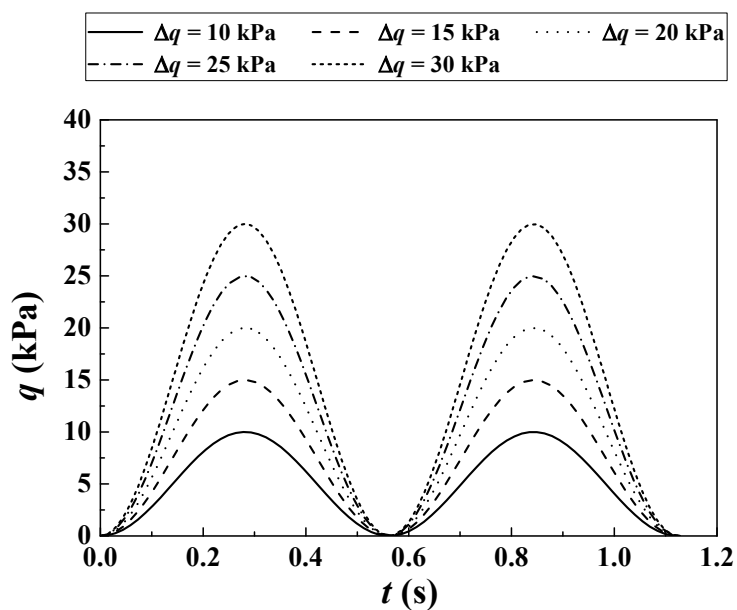


Fig. 5. Typical sine-shaped signals applied in cyclic triaxial tests

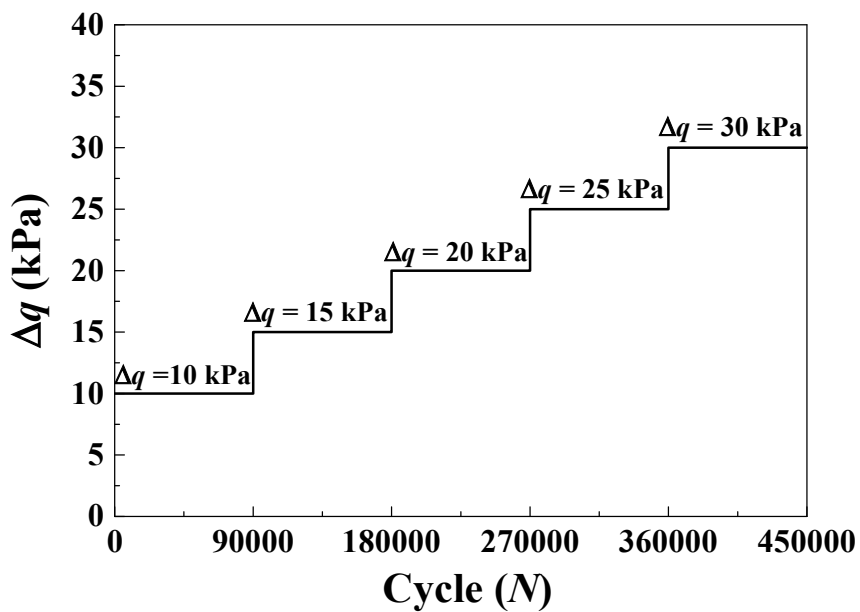


Fig. 6. Multi-step loading procedure with various stress amplitudes  $\Delta q$

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  $\varepsilon_1$  with loading cycles  $N$  at  $f_v = 0\%$  and  $w_1 = 17.6\%$   
 202 under various  $\Delta q$  values ranging from 10 kPa to 30 kPa. It can be observed that under a given  
 203  $\Delta q$  value, the axial strain  $\varepsilon_1$  increased significantly at the beginning of loading cycles, and  
 204 then gradually stabilized. With the increase of  $\Delta q$ , the axial strain  $\varepsilon_1$  increased significantly. In  
 205 addition, the axial strain  $\varepsilon_1$  could be separated into two parts: a plastic strain  $\varepsilon_1^p$  and a resilient  
 206 strain  $\varepsilon_1^r$ . For the plastic strain  $\varepsilon_1^p$ , it increased with  $N$  and the increasing rate decreased with  $N$   
 207 under a constant  $\Delta q$  value. On the contrary, the resilient strain  $\varepsilon_1^r$  remained almost unchanged  
 208 at a given  $\Delta q$  value and increased with increasing  $\Delta q$ .

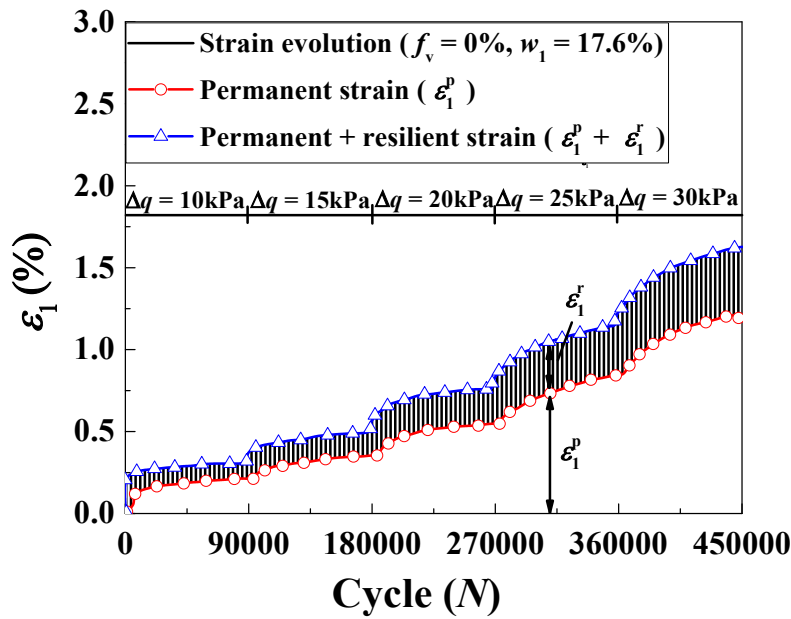


Fig. 7. Determination of permanent strain and resilient strain

209  
 210 Fig. 8 presents the evolutions of  $\varepsilon_1^p$  with  $N$  under various  $\Delta q$  values for  $f_v = 0\%$  and three  
 211 water contents. At  $w_1 = 17.6\%$ , the permanent strain  $\varepsilon_1^p$  increased with  $N$  sharply at the initial  
 212 loading cycles  $N$ , and then gradually reached stabilization under a specific  $\Delta q$  value. With the

213 increase of  $\Delta q$ , the permanent strain  $\varepsilon_1^p$  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  $\varepsilon_1^p$ . This could be explained by the contribution of  
 217 suction, as reported by Duong et al. [12-] and Jing [24].

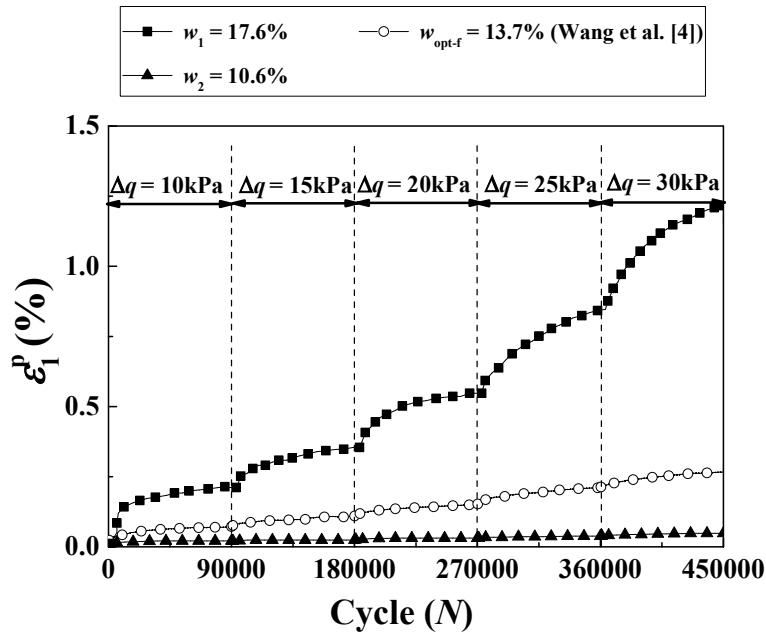


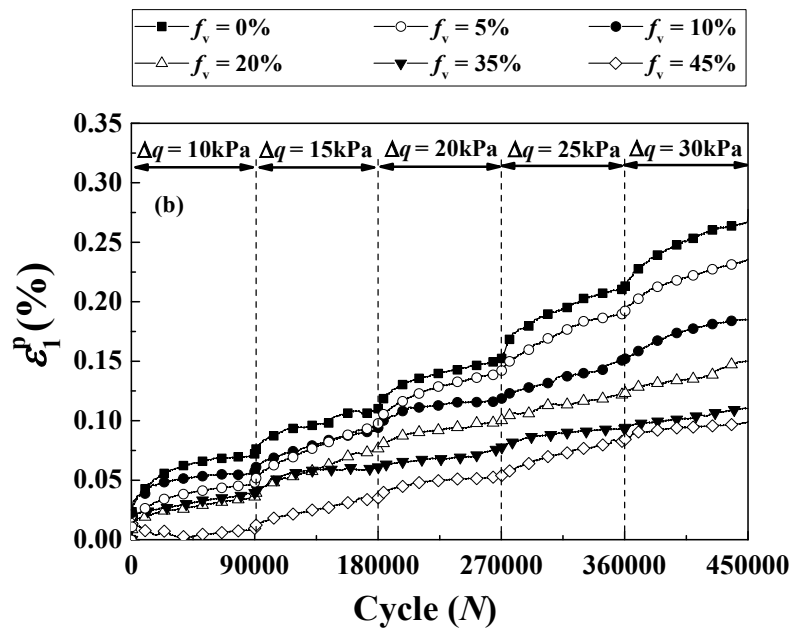
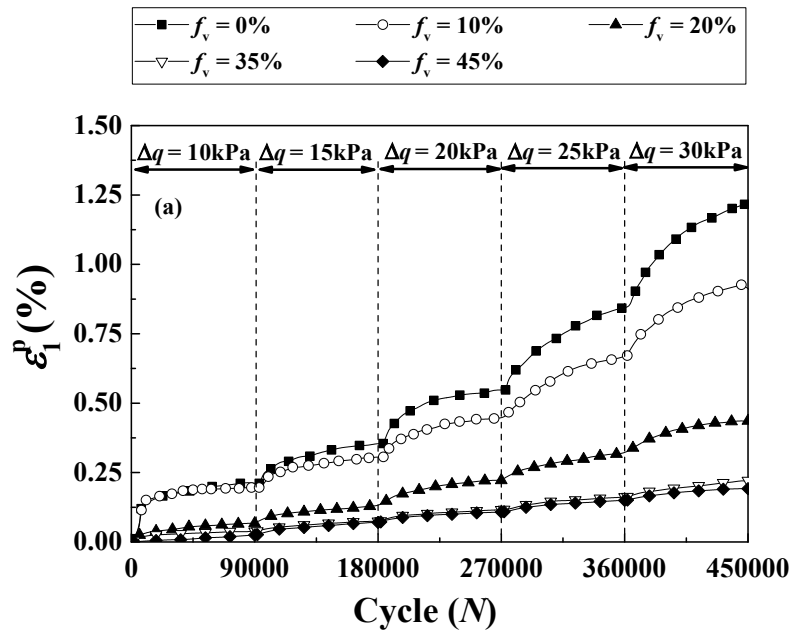
Fig. 8. Evolutions of permanent strain with number of cycles at  $f_v = 0\%$  and different  $\Delta q$  values for three different water contents

218

219 Fig. 9 shows the effect of water content on the evolutions of  $\varepsilon_1^p$  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  $\varepsilon_1^p$   
 221 versus  $N$  decreased with the increase of  $N$  at a constant  $\Delta q$  value. However,  $\varepsilon_1^p$  increased  
 222 significantly with the increase of  $\Delta q$ . In addition, it can be found that with the increase of  $f_v$ ,  
 223 the permanent strain  $\varepsilon_1^p$  decreased. This was attributed to the reinforcement effect of coarse



224 grains in the soil mixtures. Moreover, different decreasing trends of  $\varepsilon_1^p$  with  $f_v$  was evidenced  
225 at  $f_v \leq 20\%$  and  $f_v \geq 35\%$ : a pronounced decrease of  $\varepsilon_1^p$  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  $\rho_{d-f}$  higher than  $1.82 \text{ Mg/m}^3$ ) and loose fines (with  $\rho_{d-f}$  lower than  $1.82$**   
230  **$\text{Mg/m}^3$ ). 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  $\varepsilon_1^p$  at  $f_v = 0\%$  was very small, with**  
234 **a maximum value around  $0.05\%$  identified. The  $\varepsilon_1^p$  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  $\varepsilon_1^p$  at  $f_v = 10\%$ ,**  
237  **$20\%$ ,  $35\%$  and  $45\%$  were generated. Therefore, these results are not presented in Fig. 9 (c).**



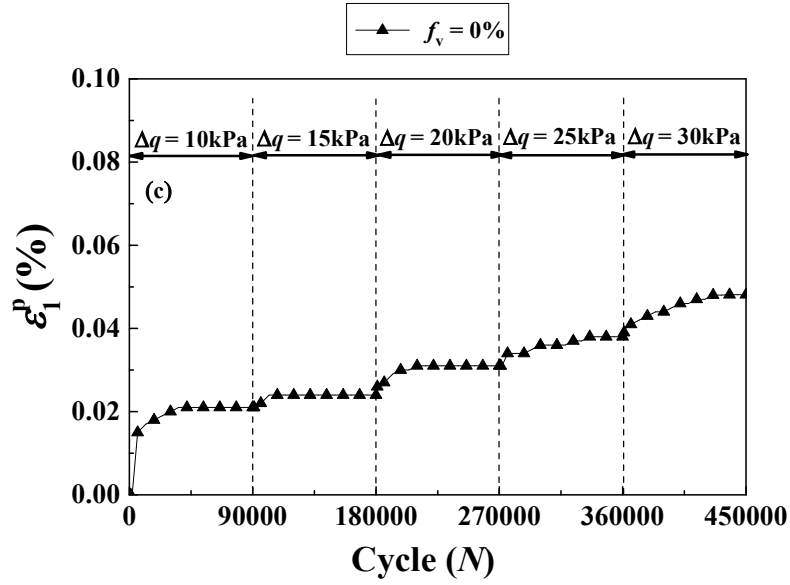


Fig. 9. Evolutions of permanent strain with number of cycles at different  $f_v$  and  $\Delta q$  values for various water contents: (a)  $w_1 = 17.6\%$ ; (b)  $w_{\text{opt-f}} = 13.7\%$  (after Wang et al. [4]); (c)  $w_2 = 10.6\%$

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  $\varepsilon_1^p$  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  $\varepsilon_1^p$  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  $\theta$  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  $\varepsilon_1^p$  with  $N$  at  $f_v = 0\%$  and  $w_1 = 17.6\%$  under  
259 various  $\Delta q$  values. With the increase of  $\Delta q$ , the estimated  $\varepsilon_1^p$  appears to grow. In addition, it  
260 can be found that the estimated  $\varepsilon_1^p$  was larger than the measure  $\varepsilon_1^p$  under the same  $\Delta q$  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  $\varepsilon_1^p$  under a constant  $\Delta q = 30$  kPa, the measured  
264  $\varepsilon_1^p$  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  $\varepsilon_1^p$  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  $\varepsilon_1^p$  (at  $N = 90000$ ) under various  $\Delta q$  values were  
269 selected for further analysis.

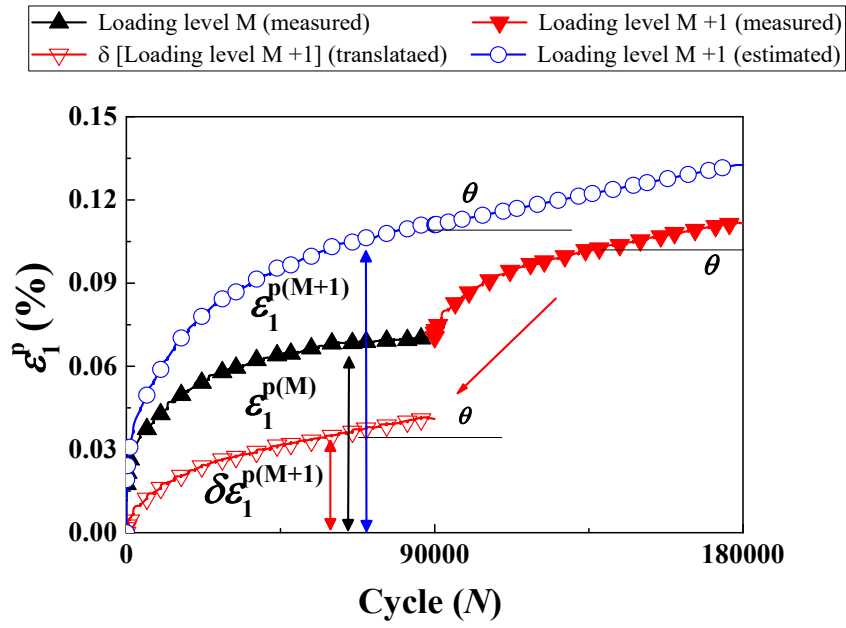


Fig. 10. Estimation method of  $\varepsilon_1^p$  proposed by Gidel et al. [5]

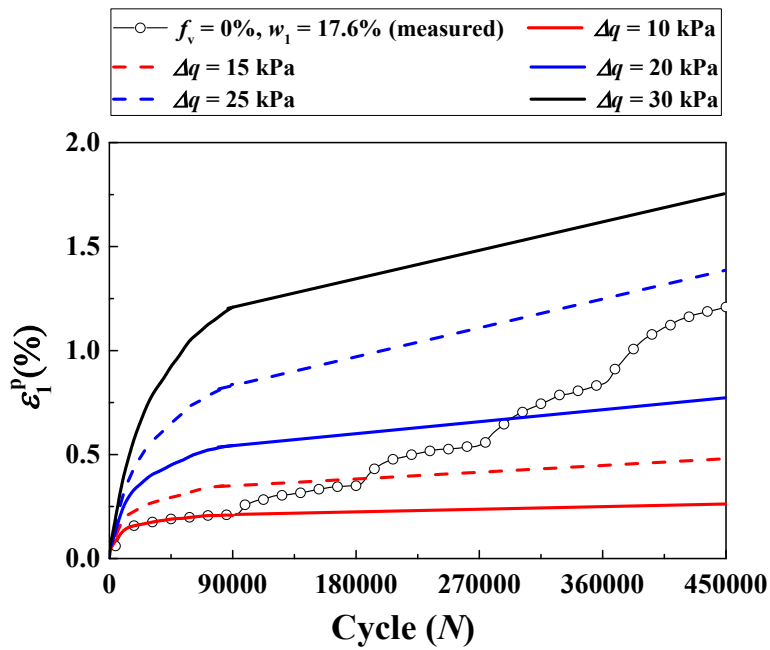


Fig. 11. Evolutions of estimated permanent strain with number of cycles at various  $\Delta q$  values for  $f_v = 0\%$  and  $w_1 = 17.6\%$

271 *Effect of water content on the variations of estimated end-stage  $\varepsilon_1^p$  with  $\Delta q$*

272 Fig. 12 shows the variations of estimated end-stage  $\varepsilon_1^p$  with  $\Delta q$  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  $\varepsilon_1^p$  increased with increasing  $\Delta q$  for various  $f_v$  values. Under a given  $\Delta q$ , the  $\varepsilon_1^p$  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  $\varepsilon_1^p$ , owing to the contribution of suction.

279 **The study of Dong et al. [12] was carried out at a constant dry density of mixture  $\rho_d$  for**  
280 **varying  $f_v$  values (50.3%, 55.5% and 61.4%), and therefore the fines fraction density  $\rho_{d-f}$**   
281 **decreased as  $f_v$  increased (Table 3).** Fig. 13 shows that the estimated end-stage  $\varepsilon_1^p$  increased  
282 with  $\Delta q$  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  $\varepsilon_1^p$  under a constant  $\Delta q$ , 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  $\varepsilon_1^p$   
286 under a constant  $\Delta q$ , 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  $\varepsilon_1^p$  (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  $\rho_{d-f}$  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  $\rho_{d-f}$ , 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  $\varepsilon_1^p$  (Fig. 12 (b)). Conversely, in the study of Duong et al. [12] , the  $\rho_{d-f}$  of  
302 fine soil declines from 1.33  $\text{Mg/m}^3$  to 1.17 and 0.94  $\text{Mg/m}^3$  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  $\varepsilon_1^p$  increased with increasing  $f_v$ . (Fig. 13  
306 (b) – (c)).

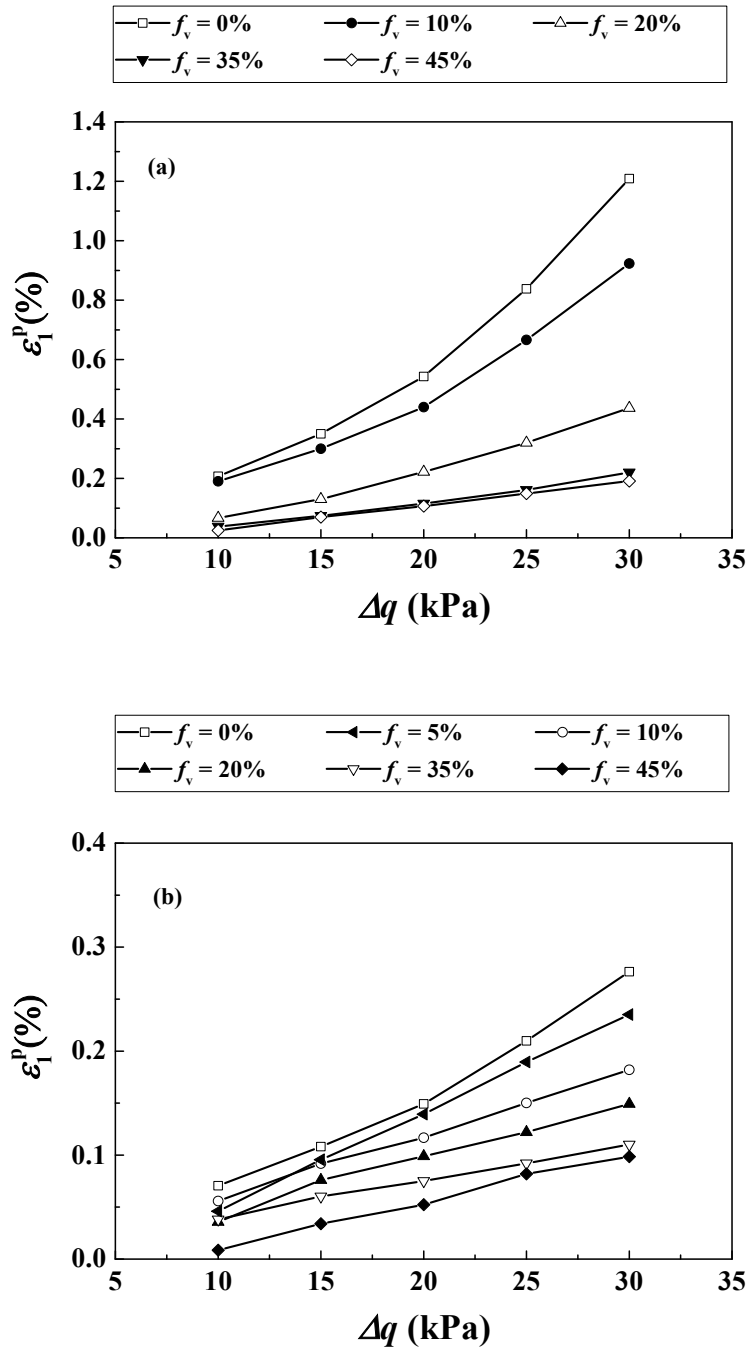
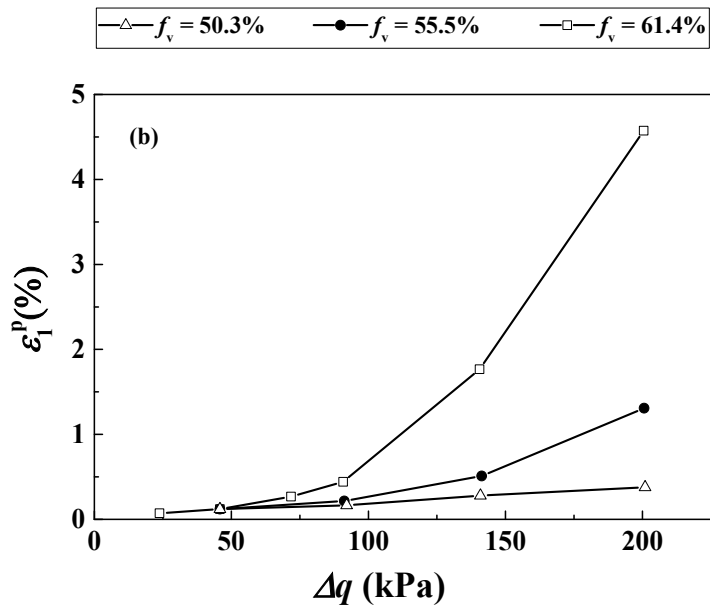
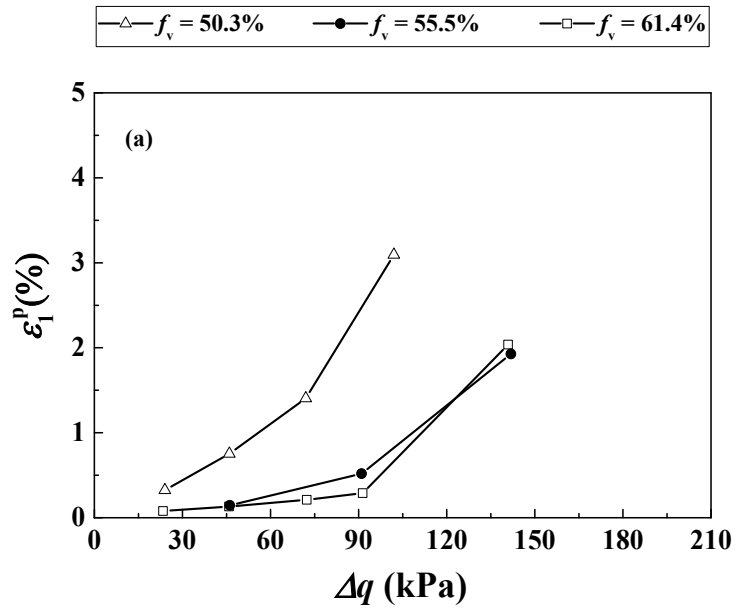


Fig. 12. Variations of estimated end-stage permanent strain with  $\Delta q$  at different  $f_v$  values for various water contents: (a)  $w_1 = 17.6\%$ ; (b)  $w_{opt-f} = 13.7\%$





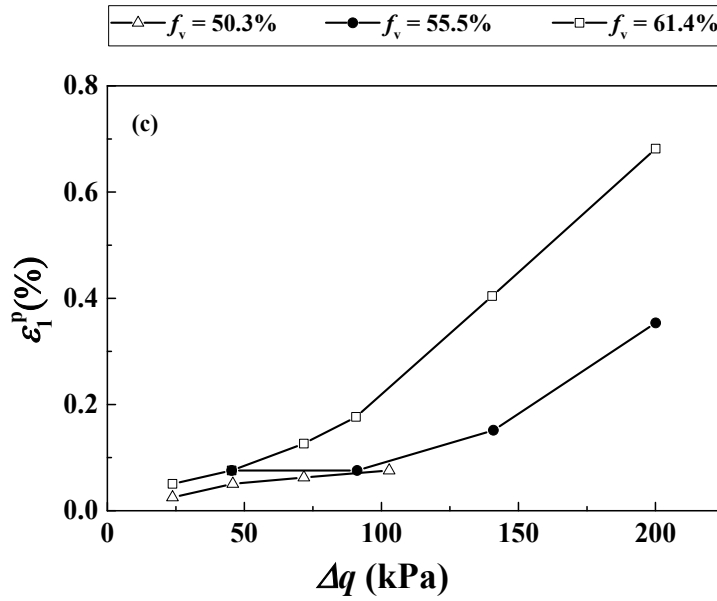


Fig. 13. Variations of estimated end-stage permanent strain with  $\Delta q$  at different  $f_v$  values for various water contents: (a)  $w = 12\%$ ; (b)  $w = 6\%$ ; (3)  $w = 4\%$  (after Duong et al. [12])

307

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

$f_v$ (%)	$w$ (%)			$S_r$ (%)			$\rho_d$ (Mg/m <sup>3</sup> )	$\rho_{d-f}$ (Mg/m <sup>3</sup> )
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  $\varepsilon_1^p$  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  $\Delta q$  of 10, 15,

313 20, 25 and 30 kPa was applied, with a number of loading cycles  $N = 90000$  for each  $\Delta q$  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  $\varepsilon_1^p$  proposed by Gidel et al. [5] was adopted to eliminate the effect of loading history on  $\varepsilon_1^p$ .  
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  $\varepsilon_1^p$ , due to the  
321 contribution of suction. An increase of  $f_v$  gave rise to a decrease of  $\varepsilon_1^p$ , 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  $\rho_{d-f}$  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  $\varepsilon_1^p$ . 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  $\rho_{d-f}$  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  $\varepsilon_1^p$  increased with  
331 the increase of  $f_v$ .

332

333

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427

#### 428 NOTATIONS

$\varepsilon_1$	axial strain
$\varepsilon_1^r$	resilient strain
$\varepsilon_1^p$	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
$\rho_d$	dry density of sample
$\rho_{d-f}$	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
$\Delta q$	deviator stress amplitude
$\Delta p$	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
$\sigma_3$	confining pressure
$\theta$	increasing slope of permanent strain with loading cycles at the end of loading cycles for a given stress amplitude

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