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1 **Experimental investigation and modeling of the mechanical behaviour of Yanji**
2 **swelling mudstone subjected to wetting-drying-freezing-thawing cycles**

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21 **Abstract:** Rocks and soils in seasonally frozen regions are subjected to cyclic
22 wetting-drying-freezing-thawing (WDFT) process. The influence mechanism of WDFT process
23 on their mechanical behaviour is not fully understood and the existing constitutive models are not
24 capable of capturing the strain-softening behaviour under WDFT cycles. In the present study, a
25 series of consolidated drained triaxial compression and mercury intrusion porosimetry tests were
26 performed on Yanji mudstone subjected to WDFT cycles. Results indicated that the WDFT cycles
27 could induce a large amount of cracks and large pores between aggregates and compress the
28 aggregates with a decrease of the intra-aggregate pores. After cyclic WDFT process, the specimens
29 exhibited more significant strain-softening and dilative behaviour. The elastic modulus, peak and
30 residual shear strengths of the mudstone decreased significantly with increasing WDFT cycles,
31 especially in the case of low confining pressures. These decreases were more pronounced in the first
32 four cycles and became negligible after 4 cycles. As the cyclic number increased, the cohesion of
33 specimens decreased remarkably owing to the presence of cracks and the destruction of bonds
34 between aggregates. By contrast, the effective friction angle, Poisson's ratio and dilatancy angle
35 increased with the WDFT cycles due to the more compact aggregates induced by the high suction
36 during drying process. A nonlinear elastic constitutive model was developed for describing the
37 strain-softening behaviour of Yanji mudstone subjected to WDFT cycles. The deviator stress-axial
38 strain relationship predicted by the proposed model agreed well with the measured one, indicating
39 the relevance of the predictive model.

40 **Keywords:** swelling mudstone; wetting-drying-freezing-thawing cycle; stress-strain behaviour;
41 microstructure; modelling

42 **1 Introduction**

43 Swelling rock and soils correspond to the deposits composed of large amounts of clay minerals (e.g.
44 montmorillonite and illite). These deposits are widely found in more than 60 countries such as
45 Australia, China, Sudan, Spain, USA, etc [1, 2]. Due to the special mineralogical compositions,
46 swelling rocks and soils are diverse in appearance and hydro-mechanical properties. They can
47 develop significant swell or swelling pressure after contact with water, and shrink upon desiccation.
48 Additionally, they might exhibit an acceptable strength in dry condition while soften significantly
49 upon the presence of water [1, 3]. These variable properties cause severe distress to the associated
50 geotechnical infrastructures, such as non-uniform settlement, pavement cracking, and slope failure
51 [4-7]. It is therefore essential to thoroughly evaluate the mechanical behaviour of swelling rocks
52 and soils to achieve a reliable design and operation of geotechnical infrastructures.

53 Over the past decades, many studies attempted to address the couplings between changes in
54 water content and the induced modifications in fabric and mechanical property for swelling rocks
55 and soils [8-12]. It was documented that the swelling behaviour of swelling rocks and soils could be
56 influenced by many factors, including moisture content, dry density and clay mineral content [13-15].
57 Generally, the final swelling pressure and swelling strain increased as the initial dry density and clay
58 mineral content increased. By contrast, as the initial water content increased, the final swelling
59 pressure or swelling strain decreased [16-18]. After swelling, the breakage of cementation between
60 particles would lead to a significant reduction of shear strength. Furthermore, a lower initial water
61 content could more significantly reduce the shear strength due to the larger swelling deformation
62 upon wetting [19-21].

63 Additionally, the mechanical properties of swelling rocks and soils can be possibly altered by
64 cyclic climate loadings [22-26]. In this regard, most common environmental factor is wetting-drying
65 (WD) cycles. Nowamooz and Masrouri [27, 28] indicated that the successive WD processes
66 induced a progressive accumulation of irreversible strains, and the loose and dense expansive soil
67 specimens achieved a unique final state after several cycles. Additionally, the desiccation and
68 shrinkage could facilitate the crack propagation, and the crack surface became more irregular and
69 coarse with repeated WD [29]. Such irreversible strain and cracks led to the destruction of bonds and
70 resulted in degradation of the hydro-mechanical behaviour of the swelling rocks and soils [23, 28,
71 30].

72 Recently, there is a rapid increase of geotechnical constructions in seasonally frozen regions
73 due to significant residential land and transportation network developments [31, 32]. In these
74 regions, the rocks and soils are subjected to not only seasonal WD cycles, but also frequent
75 freezing-thawing (FT) process [7, 33]. During freezing, the formation of ice lenses would modify
76 their structure on the micro and macro-scales and reduce their integrity [34, 35]. After FT cycles,
77 the large-pore volume increased and the surface crack propagated [32, 36]. These large pores and
78 cracks not only led to a larger hydraulic conductivity [37, 38] but also weakened the rocks and soils
79 in terms of loss in strength and stiffness [7, 39, 40].

80 In many seasonally frozen regions with monsoon climates, winters are cold with frequent and
81 heavy snowfall, and summers are hot with sufficient rainfall. It would be more relevant to take into
82 account the influences of both WD and FT processes. However, most of previous researches
83 focused on the mechanical behaviour of swelling rocks and soils subjected to separate WD or FT

84 process. In spite of some recent works by Zhao et al. [6] and Zhu et al. [41], the information about
85 the mechanical properties under the combined action of WD and FT cycles is quite scarce and the
86 influence mechanism of wetting-drying-freezing-thawing (WDFT) cycles has not been fully
87 understood. In addition, the swelling rocks and soils can possess a significant strain-softening
88 behaviour due to their relatively high fragility especially after cyclic WD process, and it becomes
89 essential to capture the strain-softening behaviour in the simulation of slope progressive failure
90 under WDFT cycles. However, the existing constitutive models are not capable of characterising
91 the strain-softening behaviour for swelling rocks and soils subjected to WDFT cycles.

92 In the present work, a series of consolidated drained (CD) triaxial compression tests under
93 different confining pressures were carried out on Yanji swelling mudstone specimens subjected to
94 various WDFT cycles, and the effect of WDFT on the Young's modulus, peak and residual shear
95 strengths, effective friction angle, effective cohesion, Poisson's ratio and dilatancy angle was
96 determined. In parallel, the microstructure evolution of swelling mudstone during WDFT cycles
97 was also investigated using MIP technique, enabling the deterioration mechanism of those
98 mechanical properties to be advanced. Additionally, a nonlinear elastic constitutive model
99 considering the WDFT effect was proposed to further simulate the triaxial stress-strain behaviour of
100 mudstone specimens. The predictive performance of the proposed model was verified using the test
101 results.

102 **2 Materials and Experimental methods**

103 *2.1 Materials*

104 The study area is located at the Yanji section of Jilin-Hunchun railway line, where a great number of
105 mudstone cut slopes failed after rainfalls (Fig. 1a). Subsequently, many excavation works and
106 reinforcement measures, such as anti-slide piles, arch grille beams and retaining walls, were
107 conducted after the failure of cutting slopes [19]. The mudstone samples were extracted from a
108 typical K275 landslide (Fig. 1b). Table 1 summarises the geotechnical properties and mineralogical
109 composition of the mudstone. The specific gravity, liquid limit, plastic limit and shrinkage limit of
110 the mudstone are 2.72, 41.4%, 15.8%, and 13.9%, respectively. The mudstone has a natural dry
111 density of 1.67 Mg/m^3 , a natural water content of 20.0% and a liquidity index of 0.16. Its free
112 swelling ratio and volume shrinkage ratio are 54% and 14%, respectively, defining a low swell
113 potential [42]. For the measurement of shrinkage characteristics, a specimen with natural water
114 content and dry density was prepared and then dried until the specimen mass decreased to a constant
115 value. The shrinkage limit was determined at the inflection point where the increasing rate of
116 shrinkage strain changed significantly with the decreasing water content. From the X-ray diffraction
117 (XRD) analysis (Fig. 2), the dominant clay mineral of the mudstone is montmorillonite (23.0%), with
118 others being illite, feldspar, quartz and dolomite. The maximum dry density and the optimum water
119 content of the mudstone were also determined by standard Proctor compaction test, equal to 1.72
120 Mg/m^3 and 20.6%, respectively.

121 As shown in Fig. 1a, the mudstone was subjected to extensive crushing and backfilling works
122 during the landslide remediation and the original structure of the mudstone was greatly disturbed by
123 the artificial activities [43]. To represent field conditions, the tested mudstone was first air dried and
124 then crushed to less than 2 mm. The grain size distribution of the crushed mudstone is shown in Fig. 3.

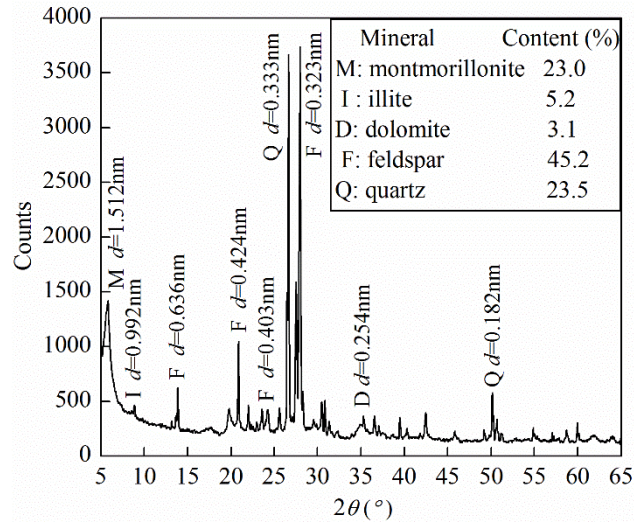
125 The grains larger than 0.075 mm and those smaller than 0.075 μm were measured by the wet-sieving
 126 and hydrometer methods, respectively. The mean grain diameter (D_{50}) of crushed mudstone is 0.068
 127 mm and the clay-size fraction is 7.4%.



128
 129 **Fig. 1** Sample site and extracted sample: (a) the mudstone slope at the K275 section of Jilin-Hunchun
 130 railway line, and (b) the swelling mudstone in a 4.0 m deep trench
 131

132 **Table 1** Characteristics of the studied mudstone

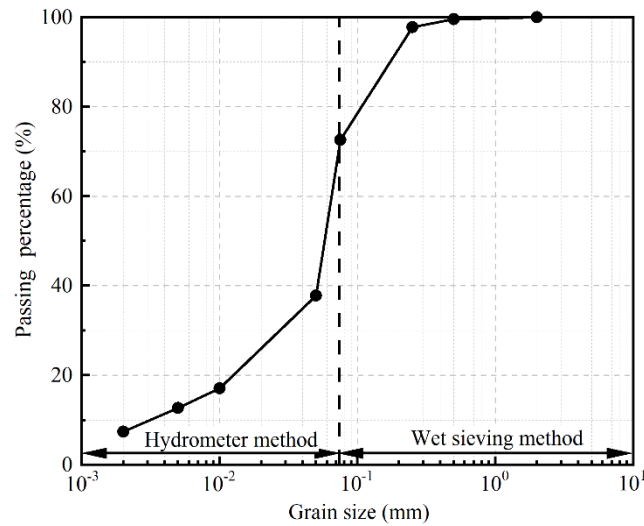
Property	Value	
Water content (%)	20.0	
Dry density (Mg/m^3)	1.67	
Specific gravity	2.72	
Void ratio	0.63	
Liquid limit (%)	41.4	
Plasticity index	15.8	
Free swelling ratio (%)	54	
Volume shrinkage ratio (%)	14.0	
Contraction coefficient	0.49	
Shrinkage limit (%)	13.9	
Maximum dry density (Mg/m^3)	1.72	
Optimum water content (%)	20.6	
Mineral composition (%)	Montmorillonite	23.0
	Illite	5.2
	Dolomite	3.1
	Feldspar	45.2
	Quartz	23.5



133

134

Fig. 2 X-ray diffraction pattern of the tested mudstone



135

136

137

Fig. 3 Grain size distribution of the tested mudstone

138 *2.2 Specimen preparation*

139 In the field, the mudstone was crushed and compacted without adding more water in September 2013
 140 because the natural water content was close to its optimum water content (20.6%). To simulate field
 141 conditions, the crushed mudstone was wetted by spraying deionised water to achieve a natural water
 142 content of 20.0%. The wet mudstone was mellowed in plastic bags for 48 hours for water
 143 homogenization. After that, the wet mudstone was subjected to static compaction to reach a dry
 144 density of 1.67 Mg/m³, which corresponded to a compaction degree of 97%. The diameter and

145 height of obtained specimens are 61.8 and 125 mm, respectively. In this work, 64 specimens were
146 prepared in total, with 4 specimens for each cycle.

147 In the study area, the monthly and daily mean temperatures are in the ranges of -13.2-21.7 °C
148 and -18.2-29.3 °C while the relative humidity is 53-79%. To investigate the influence of climate
149 condition on the mechanical behaviour, some of the prepared specimens were subjected to cyclic
150 WDFT treatment using a temperature and humidity control box (Fig. 4). The detailed operating
151 procedures of WDFT cycles are shown in Fig. 5. The specimens were first dried at a prescribed
152 temperature (25 °C) and a prescribed relative humidity (75%) in a temperature and humidity control
153 box until the shrinkage limit of 13.9% was reached (Fig. 4b). Then, they were vacuum saturated
154 with deionised water and then subjected to drying at a prescribed temperature (25 °C) and a
155 prescribed relative humidity (75%) in the control box until the water content achieved the initial one
156 (20.0%). Afterwards, freezing-thawing cycles were applied to the specimens. They were wrapped
157 with plastic film and transferred to the control box of -15 °C for 12 h freezing (Fig. 4c).
158 Subsequently, the specimens were stored at 15 °C for 12 h thawing. To account for the number of
159 WDFT cycles (n), the specimens were subjected to 1, 2, 4, 6, and 8 cycles, respectively.

160 2.3 Experimental methods

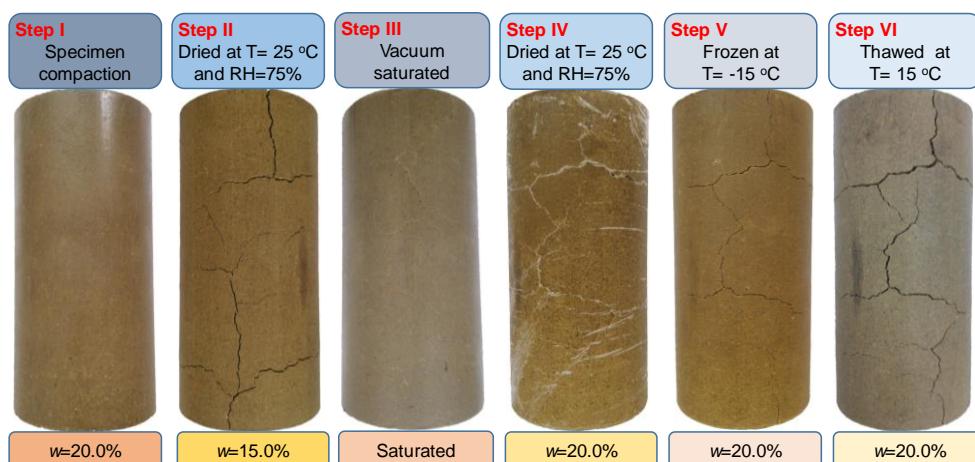
161 The mechanical behaviour of specimens after various WDFT cycles was studied by CD triaxial tests
162 using a conventional strain-controlled triaxial compression apparatus. The specimens were first
163 vacuum saturated for more than 24 h. Then, a back pressure of 100 kPa and a cell pressure of 110
164 kPa were applied on the specimens to dissolve air bubbles for another 24 h. After that, the
165 Skempton's B value was determined, larger than 0.98 for all tests. The cell pressure was increased

166 to the designed values of 150, 200, 300, and 400 kPa (corresponding to effective confining
 167 pressures of 50, 100, 200 and 300 kPa) for subsequent consolidation. Note that the adopted
 168 maximum confining pressure was determined according to the mudstone burial depth [19]. After
 169 that, according to ASTM D7181-20 [44], a shear rate as low as 0.015 mm/min was adopted for all
 170 tests, allowing a full dissipation of pore water pressure. During subsequent shearing, axial force and
 171 volume change were recorded. The tests ended when the axial strain reached 20%.



172
 173
 174

Fig. 4 WDFT process in the temperature and humidity control box: (a) test apparatus, (b) drying process, and (c) freezing-thawing (FT) process



175
 176
 177

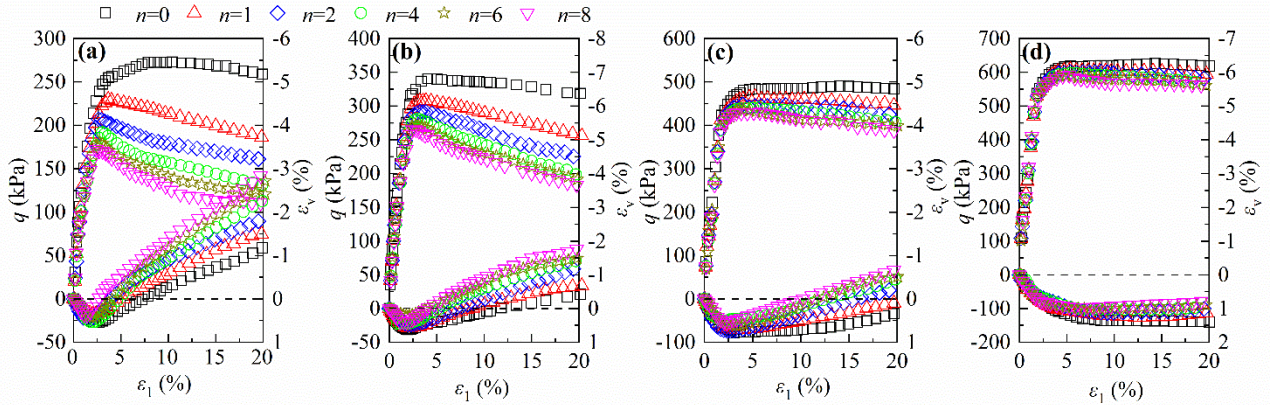
Fig. 5 Operating procedures of cyclic WDFT for Yanji swelling mudstone

178 To clarify the microstructure evolution of swelling mudstone with WDFT cycles, the pore size
179 distributions of the as-compacted specimen ($n=0$) and those after various WDFT cycles ($n=1, 2, 4, 6$
180 and 8) were determined using an Autopore IV 9500 mercury intrusion porosimeter produced by Mike
181 Co. The specimens were first cut into small cubes and rapidly frozen. After that, they were dried
182 through sublimation in a vacuumed chamber for more than 24 h. During MIP test, the freeze-dried
183 cube was placed into a low pressure chamber, and then a high pressure chamber. The working
184 pressure increased from 3.6 to 200 MPa, allowing the pores with a diameter of 350 μm -6 nm to be
185 detected.

186 **3 Experimental results**

187 *3.1 Stress-strain behaviour*

188 The variations of deviator stress (q) with axial strain (ε_1) for the as-compacted specimens ($n=0$) and
189 those subjected to various WDFT cycles ($n=1, 2, 4, 6$ and 8) under different confining pressures (σ_3)
190 are displayed in Fig. 6. For the specimens before WDFT cycles, the deviator stress increased rapidly
191 at the beginning with the increase of axial strain, and then tended to stabilization, showing
192 strain-hardening or slightly strain-softening behaviour. By contrast, for the specimens after WDFT
193 cycles, the deviator stress increased, reached a peak and finally decreased, exhibiting remarkable
194 strain-softening behaviour. Obviously, the deviator stress-axial strain curves of specimens subjected
195 more WDFT cycles lied below those for less WDFT cycles. In addition, the lower the confining
196 pressure, the more pronounced the influence of WDFT cycles on the stress-strain behaviour of
197 specimens.



198
 199 **Fig. 6** Deviator stress and volumetric strain versus axial strain curves for the specimens before and
 200 after various WDFT cycles: (a) $\sigma_3=50$ kPa, (b) $\sigma_3=100$ kPa, (c) $\sigma_3=200$ kPa, and (d) $\sigma_3=300$ kPa

201 *3.2 Volumetric strain*

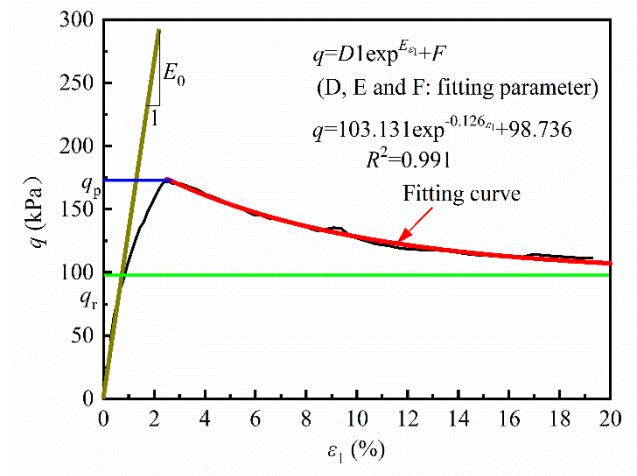
202 **Fig. 6** also depicts the variations of volumetric strain (ε_v) with axial strain (ε_1) under various
 203 confining pressures before and after different WDFT cycles. For the specimens under low confining
 204 pressures, the volume change behaviour was characterized by contraction and subsequent dilatancy.
 205 Furthermore, the higher the cyclic number, the more significant the dilatancy behaviour. Nevertheless,
 206 the dilatancy was reduced with increasing confining pressure. As the confining pressure increased
 207 from 50 to 300 kPa, the contraction increased and the dilatancy decreased. In the case of a confining
 208 pressure of 300 kPa, pure contraction behaviour was identified especially for the specimens without
 209 application of WDFT cycles.

210 *3.3 Elastic modulus and Shear strength*

211 As schematically illustrated in **Fig. 7**, the elastic modulus, peak shear strength and residual shear
 212 strength can be determined from the deviator stress-axial strain curve. The elastic modulus represents
 213 the tangent modulus at the beginning of shearing and it is defined as the ratio of deviator stress to the
 214 axial strain in the range of 0%–1% [45]. The peak shear strength refers to the maximum deviator
 215 stress, while the residual shear strength is the asymptote of the deviator stress. Considering the

216 evolution law of deviator stress, the asymptote was determined by fitting the softening phase of
217 deviator stress-axial strain curve using a logarithm function. As an example, the fitting results of the
218 specimen 8 WDFT cycles and under a confining pressure of 50 kPa is presented in the Fig. 7. The
219 deviator stress asymptotic value corresponds to the constant term of logarithm function (98.736 kPa).
220 The variations of calculated elastic modulus, and peak and residual shear strengths against the
221 number of WDFT cycles for the specimens under various confining pressures are summarized in Figs.
222 8 and 9. As expected, larger elastic modulus and peak shear strengths were identified for the
223 specimens under a larger confining pressure, regardless of the WDFT cycles. From Fig. 8, it appeared
224 that the elastic modulus declined with the increasing WDFT cycles for the specimens under a given
225 confining pressure. Additionally, the decreasing rate reduced with the increase of WDFT cycles and it
226 approached zero after the four cycles. A comparison of the specimens under various confining
227 pressures showed that the decrease of shear strength was more pronounced for the specimens under a
228 confining pressure of 50 kPa. For the specimens under a given confining pressure, both the peak and
229 residual shear strengths decreased significantly at the four cycles and then tended to stabilise (Fig. 9),
230 which was consistent with the observed WDFT effect on the elastic modulus. On the whole, the lower
231 the confining pressure, the more significant the decreases of peak and residual shear strengths. The
232 variation of the initial elastic modulus with peak shear strength for all the specimens is presented in
233 Fig. 10. The elastic modulus increased with the peak shear strength. The relationship between the
234 elastic modulus and peak shear strength can be well described using the following equation with a
235 squared correlation coefficient R^2 as high as 0.969:

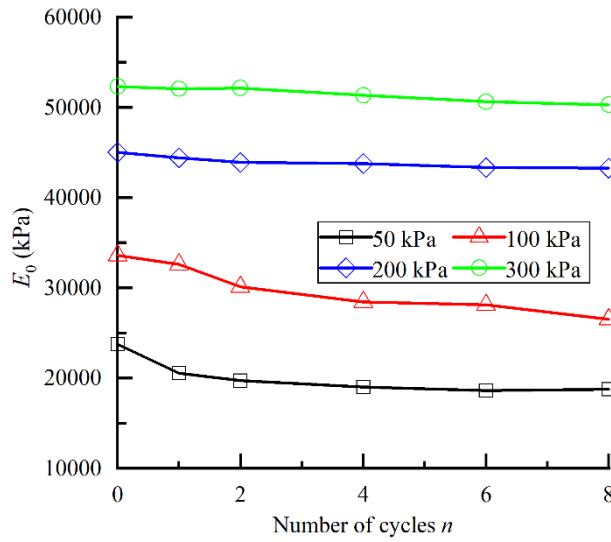
236
$$E_0=5547.603+79.081q_p \quad (1)$$



237

238

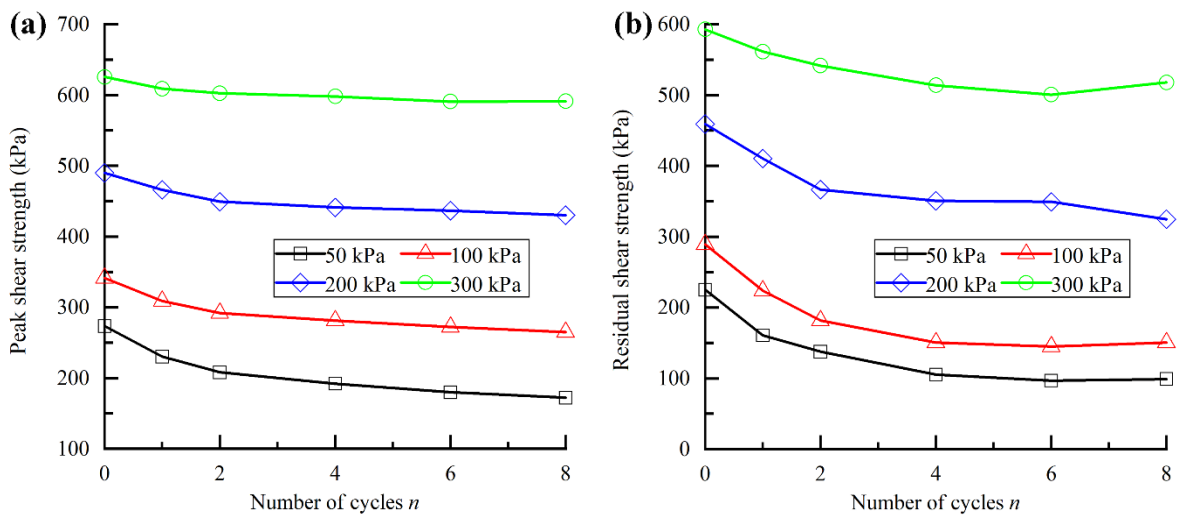
Fig. 7 Determination of elastic modulus E_0 , peak shear strength q_p and residual shear strength q_r



239

240

Fig. 8 Variations of elastic modulus with the number of WDFT cycles

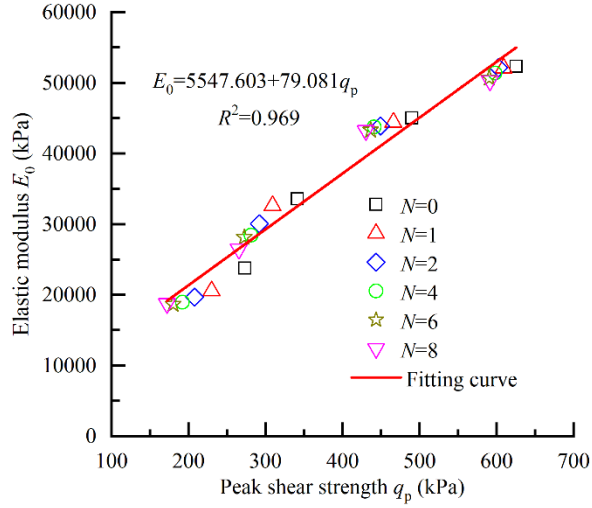


241

242

243

Fig. 9 Variations of peak and residual shear strengths with the number of WDFT cycles: (a) peak shear strength, and (b) residual shear strength



244

245

Fig. 10 Relationship between of elastic modulus and peak shear strength

246

247

To further analyse the influence of WDFT cycles on the mechanical behaviour, the peak and

248

residual shear strengths are replotted in the plane of deviator stress (q) and mean effective stress (p')

249

in Fig. 11. As the cyclic number increased, the state lines at peak and critical states shifted down.

250

According to the state lines, the effective friction angle and effective cohesion at critical and peak

251

states can be determined using the following equations:

252

$$\sin\phi'_p = \frac{3M_p}{6+M_p} \quad (2)$$

253

$$c'_p = \frac{S_p(3-\sin\phi'_p)}{6\cos\phi'_p} \quad (3)$$

254

$$\sin\phi'_r = \frac{3M_r}{6+M_r} \quad (4)$$

255

$$c'_r = \frac{S_r(3-\sin\phi'_r)}{6\cos\phi'_r} \quad (5)$$

256

where M_p and M_r are the slopes of the peak and critical state lines, respectively, S_p and S_r are the

257

intercepts of the peak and critical state lines, ϕ'_p and ϕ'_r are the effective friction angles at the peak

258

and critical states, and c'_p and c'_r refer to the effective cohesions at the peak and critical states. Fig.

259

12 displays the variations of the effective friction angles and effective cohesions at peak and critical

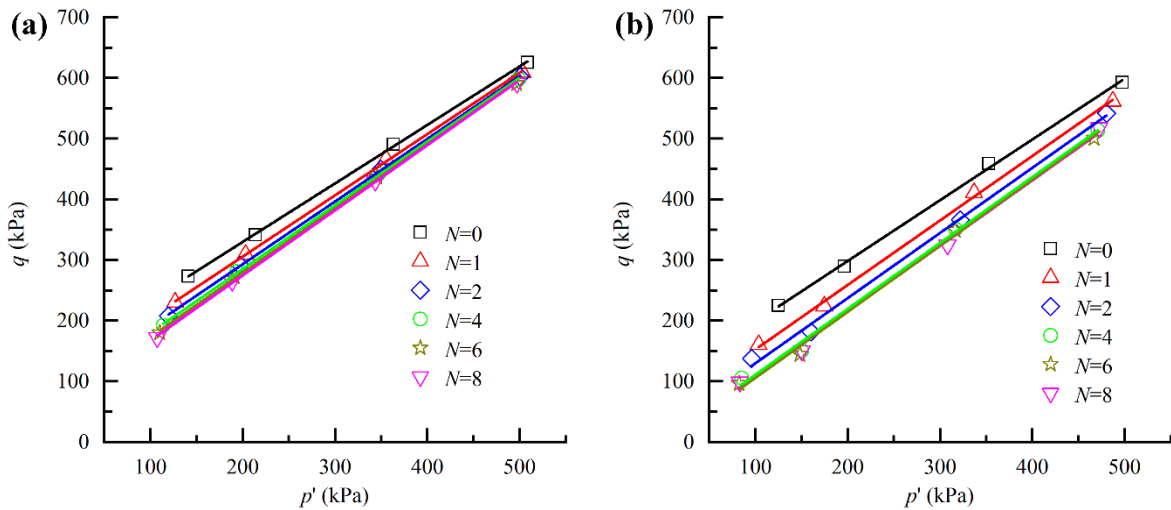
260 states with the number of WDFT cycles. It could be observed that for all the specimens, the
 261 effective cohesion at peak state was significantly larger than the effective cohesion at critical state
 262 while the effective friction angle at peak state was slightly lower than that at critical state. As the
 263 cyclic number increased, the effective cohesions at both peak and critical states decreased
 264 significantly while the effective friction angles slightly. The relationships between the effective
 265 cohesion, effective friction angle and the number of WDFT cycles could be described using Eqs.
 266 (6-9):

267
$$c'_p = 29.785 + 34.684e^{-0.535n} \quad (6)$$

268
$$c'_r = 46.733e^{-0.754n} \quad (7)$$

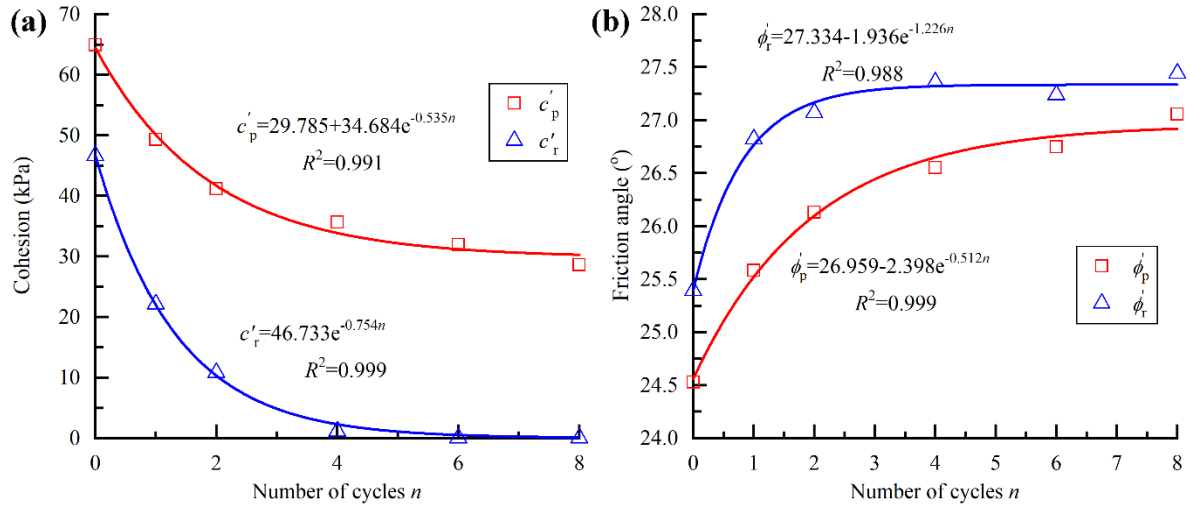
269
$$\phi'_p = 26.959 - 2.398e^{-0.512n} \quad (8)$$

270
$$\phi'_r = 27.334 - 1.936e^{-1.226n} \quad (9)$$



271
 272 **Fig. 11** State lines in the plane of deviator stress and mean effective stress: (a) peak state, and (b)
 273 critical state

274



275

276 **Fig. 12** Relationship between of peak and residual shear strength parameters, and the number of
 277 WDFT cycles: (a) effective cohesion, and (b) effective friction angle
 278

279 *3.4 Poisson's ratio and dilatancy angle*

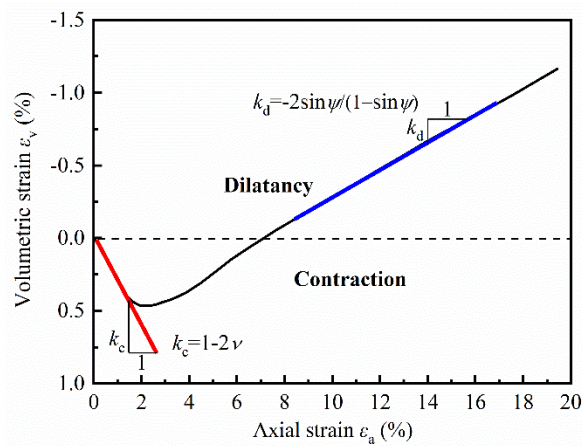
280 To further analyse the volume changes during shearing, the Poisson's ratio (ν) and dilatancy angle (ψ)
 281 were determined according to the volumetric strain-axial strain curves. As an example, Fig. 13
 282 illustrates the deamination of Poisson's ratio and dilatancy angle. The slopes of volumetric
 283 strain-axial strain curves in the contractive and dilatancy phases are designated as k_c and k_D ,
 284 respectively. These two parameters Poisson's ratio and dilatancy angle can be derived using Eqs. (10)
 285 and (11), respectively [46]:

286
$$\nu = \frac{1 - k_c}{2} \quad (10)$$

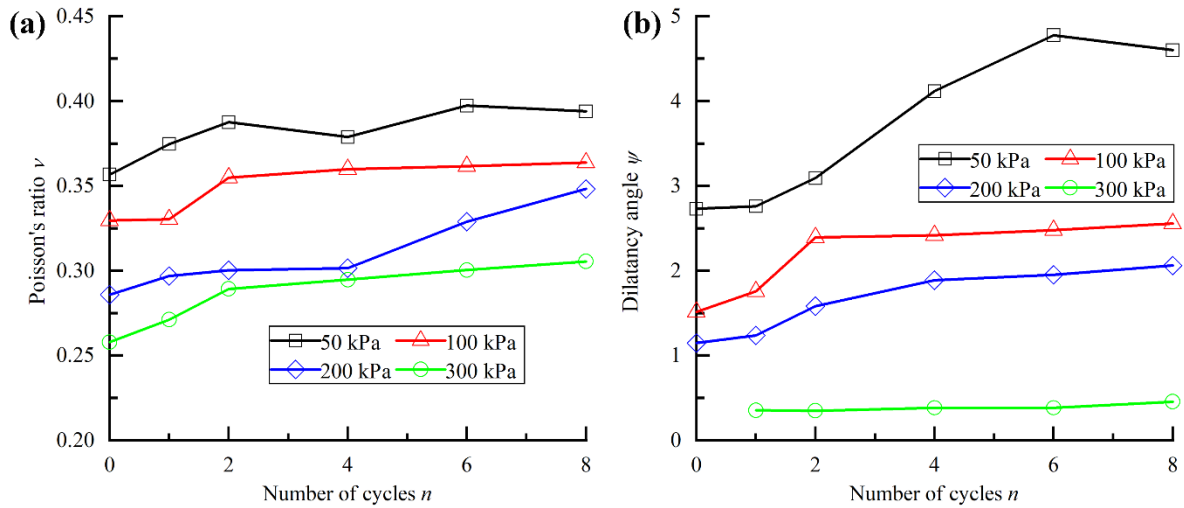
287
$$\sin \psi = \frac{k_D}{-2 + k_D} \quad (11)$$

288 Fig. 14a displays the variation of Poisson's ratio with WDFT cycles under various confining
 289 pressures. For the specimens under various confining pressures, the Poisson's ratio increased with the
 290 increasing cyclic number, albeit at a small increasing rate. After given WDFT cycles, the Poisson's
 291 ratio value decreased as the confining pressure increased, which was consistent with the volumetric

292 strain-axial strain relationships (Fig. 6). As the confining pressure increased, the specimens presented
 293 more contractive behaviour after given WDFT cycles. Fig. 14b presents the variations of dilatancy
 294 angle with the number of WDFT cycles under four various confining pressures. Note that the
 295 specimens with pure contractive behaviour are not included. For the specimens under given confining
 296 pressures lower than 200 kPa, the dilatancy angle increased with the increase of WDFT cycles,
 297 suggesting that the specimens after more WDFT cycles exhibited more pronounced dilative
 298 behaviour. For the specimens under a confining pressure of 300 kPa, the dilatancy angle were not
 299 noticeably influenced by the WDFT cycles. Additionally, the dilatancy angles of the specimens
 300 under a lower confining pressure was obviously larger than those of the specimens under a higher
 301 confining pressure, indicating the inhibition effect of confining pressure on the dilatant behaviour.



302
 303 **Fig. 13** Determined Poisson's ratio and dilatancy angle
 304

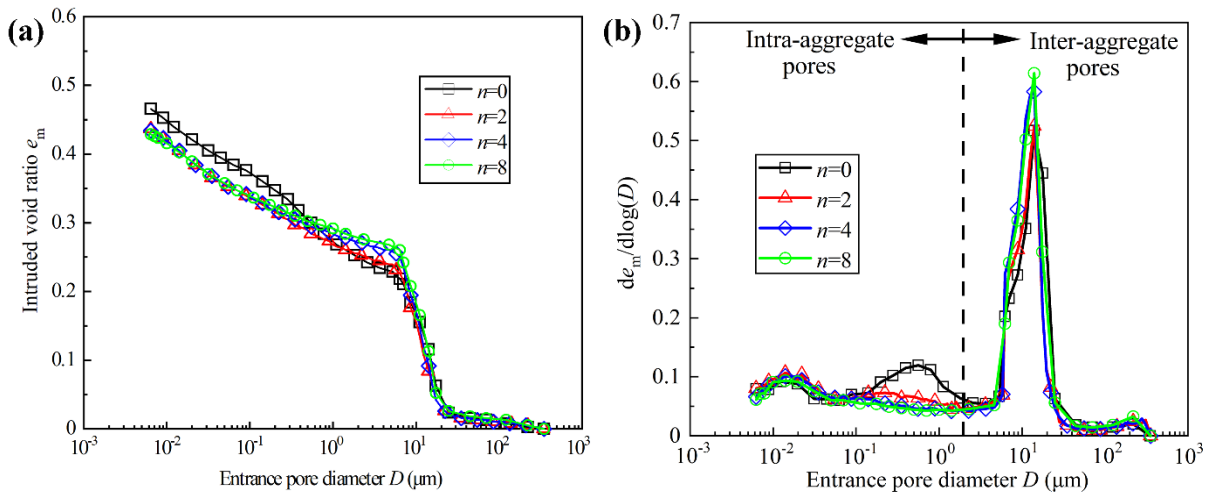


305
 306 **Fig. 14** Variations of Poisson's ratio and dilatancy angle with the number of WDFT cycles: (a)
 307 Poisson's ratio, and (b) dilatancy angle

308 *3.5 Microstructure analysis and deterioration mechanism of swelling mudstone*

309 Fig. 15 depicts the cumulative mercury intrusion and density function curves of the as-compacted
 310 specimen and those subjected to various WDFT cycles. From the cumulative curves, three fast rises
 311 for the as-compacted specimen but two fast rises for the specimens subjected to WDFT cycles could
 312 be observed (Fig. 15a). The total intruded void ratio decreased after WDFT cycles because of slight
 313 volume shrinkage [33]. From the density function curves it was observed that the as-compacted
 314 specimen exhibited a trimodal porosity with a large pore population with a mean pore diameter of 11
 315 μm, a medium pore population with a mean diameter of 0.55 μm and a small pore population with a
 316 mean diameter of 15 nm. According to the studies of Thom et al. [47] and Wang et al. [48], 2 μm was
 317 used to separate the intra-aggregate and inter-aggregate pores. The calculated void ratios of
 318 intra-aggregate and inter-aggregate pores for the specimens before and after various WDFT cycles
 319 are summarised in Table 2. It appeared that the WDFT cycles ~~increased~~decreased the intra-aggregate
 320 pore volume and ~~decreased~~increased the inter-aggregate pore volume. This increase of
 321 inter-aggregate pores could be ascribed to the combined effects of the increasing suction during

322 drying and the formation of ice lenses during freezing process. While the specimens were dried, the
 323 aggregates were pulled apart by the increasing matric suction, and cracks appeared once the pulling
 324 stress is larger than the tensile strength. In the subsequent freezing process, the water would freeze
 325 with an increasing volume, and push away the adjacent aggregates. This facilitated to the propagation
 326 of cracks and enlargement of large pores [49]. The visual view of the specimens before and after
 327 various WDFT cycles is presented in Fig. 16. After the first cycle, several cracks initiated on the
 328 surface of specimen. With the further WDFT cycles, more and more cracks appeared and the
 329 pre-existing cracks became wider and longer. These cracks destructed the links between the
 330 aggregates and reduced the cohesion of aggregates. Thereby, a decrease of effective cohesion was
 331 observed in Fig. 12a. Meanwhile, the increasing suction during drying could induce void collapse
 332 inside aggregates, resulting in the decrease of the intra-aggregate pores (Table 2). This collapse was
 333 irreversible during the following wetting process [30, 50]. Thus, after WDFT cycles, more compact
 334 and rigid aggregates could be expected, giving rise to a larger internal friction (Fig. 12b).
 335 Additionally, the developed contacts between compact aggregates would favourite more dilative
 336 behaviour (Fig. 14b).



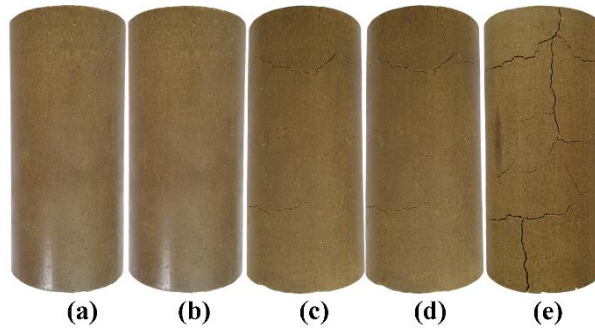
337

338 **Fig. 15** MIP results of specimens before and after different WDFT cycles: (a) cumulative intruded
 339 void ratio curves, and (b) density function curves

340 **Table 2** Intra-aggregate and inter-aggregate void ratios of the mudstone after various WDFT cycles

Cyclic number	$n=0$	$n=2$	$n=4$	$n=8$
Inter-aggregate void ratio	0.125	0.250	0.256	0.272
Intra-aggregate void ratio	0.340	0.218	0.174	0.158

341



342

343 **Fig. 16** The photos of specimens before and after different WDFT cycles: (a) $n=0$ (before WDFT
 344 cycles), (a) $n=1$, (c) $n=2$, (d) $n=4$, and (e) $n=8$

345

346 **4 Modelling of mechanical behaviour subjected to WDFT cycles**

347 In geotechnical engineering, constitutive model is very important for mechanical prediction.

348 Stress-strain relation is a measure of material stiffness and shear strength under different

349 circumstances and a key input parameter for mechanistic-empirical geotechnical engineering design

350 method. As discussed above, the specimens exhibited remarkable strain-softening behaviour after

351 WDFT cycles in this study. Although the Modified Cam Clay (MCC) model [51] has been widely

352 used in describing many important features of the mechanical behaviour of clay, it is limited to the

353 softening simulation of the soils in a heavily overconsolidated state. For most clayey deposits, MCC

354 often underestimates the softening degree. As for other MCC-based models, such as the enforced

355 strain-softening model [52] and Structured Cam Clay (SCC) model [53], the model parameters are

356 increasingly complex and the models are not easily realizable. In this work, to capture the

357 strain-softening behaviour, a three-parameter nonlinear elastic constitutive model was proposed:

358
$$q = \frac{\varepsilon_1(a+c\varepsilon_1)}{(a+b\varepsilon_1)^2} \quad (12)$$

359 where a , b , and c are the parameters related to specimen property. The first-order derivative $dq/d\varepsilon_1$
 360 can be derived as follows:

361
 362
$$\frac{dq}{d\varepsilon_1} = \frac{(a+2c\varepsilon_1)(a+b\varepsilon_1)-2b\varepsilon_1(a+c\varepsilon_1)}{(a+b\varepsilon_1)^3} \quad (13)$$

363 The slope of the tangent line at the initial stage is the elastic modulus E_0 :

364
$$E_0 = \left. \frac{dq}{d\varepsilon_1} \right|_{\varepsilon_1=0} = \frac{1}{a} \quad (14)$$

365 Then, the relationship between the elastic modulus and parameter a can be written as:

366
$$a = \frac{1}{E_0} \quad (15)$$

367 To find the critical point of $\varepsilon_{1-\max}$, setting $dq/d\varepsilon_1$ equal to zero yields:

368
$$\varepsilon_{1-\max} = \frac{a}{b-2c} \quad (16)$$

369 By substituting $\varepsilon_{1-\max}$ into Eq. (12), the peak deviator stress can be derived:

370
$$q_p = \frac{1}{4(b-c)} \quad (17)$$

371 When the axial strain reaches to a very large value (tend to infinity), the deviator stress remains at a
 372 residual value (q_r). The value of residual deviator stress can be calculated using the equation below:

373
$$q_r = \lim_{\varepsilon_1 \rightarrow \infty} \frac{\varepsilon_1(a+c\varepsilon_1)}{(a+b\varepsilon_1)^2} = \frac{c}{b^2} \quad (18)$$

374 By combining Eqs. (17) and (18), parameters b and c can be calculated using Eqs. (19) and (20),
 375 respectively:

376
$$b = \frac{1 - \sqrt{1 - q_r/q_p}}{2q_r} \quad (19)$$

377
$$c = \frac{2 - q_r/q_p - 2\sqrt{1 - q_r/q_p}}{4q_r} \quad (20)$$

378 According to Eq. (15), the model parameter a represents the elastic modulus. The higher the value of
 379 a , the larger the elastic modulus. Based on Eqs. (17-18), the difference between parameters b and c
 380 reflects the peak shear strength while the ratio of c to b^2 is equal to the residual shear strength. The

381 higher difference between b and c and ratio of c to b^2 mean a lower peak shear strength and a larger
382 residual shear strength, respectively. As remarked earlier, Eqs. (6-9) describe the relationships
383 between the effective cohesion, effective friction angle and the number of WDFT cycles. Thus, the
384 shear strength parameters at peak and critical states for the specimens after various WDFT cycles can
385 be obtained using Eqs. (6-9). Correspondingly, the shear strengths at peak and critical states for the
386 specimens under various confining pressures can be calculated based on the relationship between the
387 axial stress σ_1 and confining pressure σ_3 described by the Mohr-Coulomb criterion:

$$388 \quad \sigma_1 = \frac{2c \cdot \cos\phi}{1 - \sin\phi} + \frac{1 - \sin\phi}{1 + \sin\phi} \sigma_3 \quad (21)$$

389 where ϕ and c are the effective friction angle and effective cohesion at peak or critical state,
390 respectively. According to the calculated peak and residual shear strengths at various confining
391 pressures, the values of parameters a and b can be obtained for the specimens after different WDFT
392 cycles using Eqs. (19) and (20). Based on the calculated peak shear strength, the elastic modulus
393 can be obtained. The obtained elastic modulus is employed to calculate parameter a according to Eq.
394 (15). The obtained values of parameters a , b , and c are also summarised in Table 3.

395 According to the obtained parameters, the deviator stress-axial strain relationship for
396 specimens before and after various confining pressures can be predicted using Eq. (12). The
397 proposed model presents an effective description of the strain-softening behaviour of the
398 as-compacted specimens and those subjected to WDFT. The deviator stress-axial strain curves were
399 then compared with the experimental results in Fig. 17. A satisfactory agreement was obtained
400 between the estimated and the measured mechanical behaviour, showing the good performance of

401 the proposed model. The proposed model could well take the detrimental effect of WDFT cycles
402 into account on the mechanical behaviour of the swelling mudstones in seasonally frozen regions.

Table 3 The parameter values in the proposed model

n	σ_3 (kPa)	a ($\times 10^{-5}$)	b ($\times 10^{-3}$)	c ($\times 10^{-4}$)
0	50	3.698	1.291	3.711
	100	3.061	1.070	3.408
	200	2.277	0.807	2.918
	300	1.813	0.655	2.569
1	50	4.149	1.344	2.780
	100	3.324	1.082	2.760
	200	2.378	0.793	2.512
	300	1.851	0.635	2.274
2	50	4.478	1.414	2.358
	100	3.504	1.108	2.478
	200	2.441	0.788	2.298
	300	1.873	0.620	2.071
4	50	4.840	1.518	2.103
	100	3.691	1.151	2.331
	200	2.503	0.791	2.161
	300	1.893	0.610	1.917
6	50	4.979	1.567	2.076
	100	3.760	1.171	2.317
	200	2.524	0.793	2.123
	300	1.899	0.606	1.862
8	50	5.028	1.589	2.105
	100	3.783	1.182	2.349
	200	2.531	0.797	2.145
	300	1.901	0.608	1.876

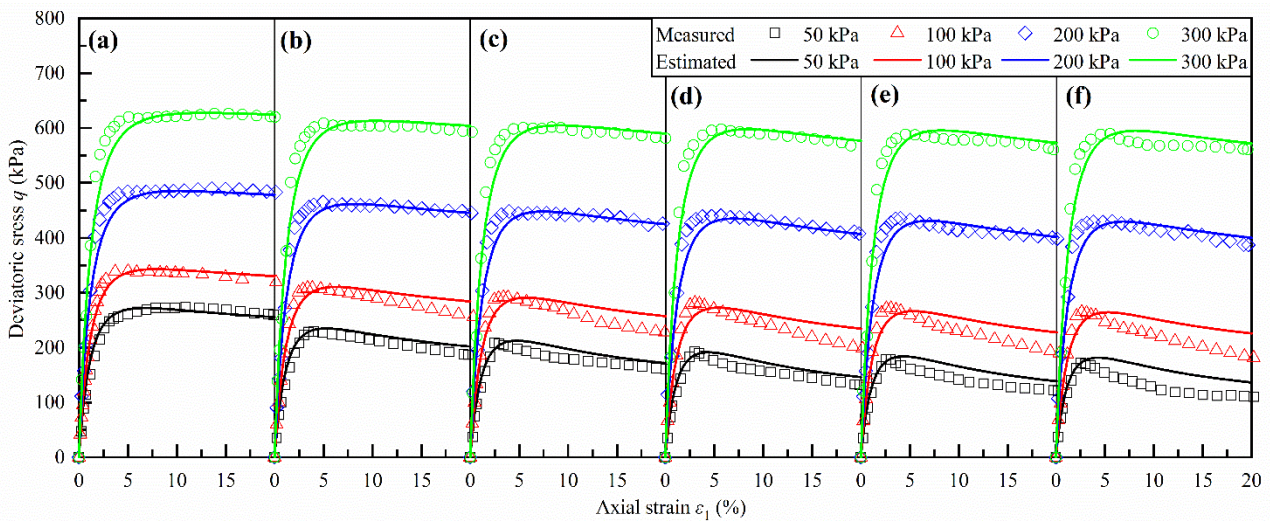


Fig. 17 Predicted stress-strain curves of specimens before and after various WDFT cycles: (a) $n=0$ (before WDFT cycles), (a) $n=1$, (c) $n=2$, (d) $n=4$, (e) $n=6$, and (f) $n=8$

407 **5 Conclusions**

408 In this study, the effect of WDFT cycles on the mechanical behaviour of Yanji mudstone and the
409 involved mechanism were investigated by carrying out a series of CD triaxial tests and
410 microstructure investigation. A nonlinear elastic constitutive model was then proposed to predict the
411 mechanical behaviour the mudstone subjected to WDFT cycles. According to the obtained results,
412 the following conclusions are drawn:

413 (1) From the microstructure analysis, drying and freezing processes could induce cracks on the
414 specimen surface and destroyed the links between aggregates, resulting in an increase of the
415 inter-aggregate pore volume. Meanwhile, the increasing suction during drying would lead to the
416 void collapse inside aggregates and the contraction of aggregates. As a result, the aggregates
417 became more rigid and compact with a decrease of the intra-aggregate pore volume after WDFT
418 cycles.

419 (2) From CD triaxial tests, it was observed that the specimens exhibited more significant
420 strain-softening and dilative behaviour after WDFT cycles. The elastic modulus, and peak and
421 residual shear strengths decreased with the increase of WDFT cycles and the decreasing rate tended
422 to stabilization after four cycles. The lower the confining pressure, the more significant the
423 influence of WDFT cycles. As the number of WDFT cycles increased, the effective cohesion at
424 both the peak and critical states decreased due to the deconstruction of aggregates while the
425 effective friction angle at both the peak and critical states increased slightly owing to the more
426 compact aggregates. Moreover, the Poisson's ratio and dilatancy angle increased because of the
427 more rigid contacts between aggregates after WDFT cycles.

428 (3) According to strain-softening behaviour, a nonlinear elastic constitutive model considering the
429 detrimental effect of WDFT cycles was proposed to predict the mechanical behaviour of swelling
430 mudstone in seasonally frozen regions. A satisfactory agreement between the predicted and
431 measured stress-strain behaviour was identified, indicating the good prediction performance of the
432 proposed method.

433

434 **CRedit authorship contribution statement**

435 Zhixiong Zeng: Conceptualization, Investigation, Formal analysis, Writing - original draft; Ling-Wei
436 Kong: Methodology, Writing - review & editing; Yu-Jun Cui: Methodology, Writing- review &
437 editing

438

439 **Declaration of Competing Interest**

440 The authors declare that they have no known competing financial interests or personal relationships
441 that could have appeared to influence the work reported in this paper.

442

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446

447 **Data availability**

448 Data will be made available on request.

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