

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

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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 wetting-drying-freezing-thawing (WDFT) process. The influence mechanism of WDFT process 22 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 residual shear strengths of the mudstone decreased significantly with increasing WDFT cycles, 30 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 strain-softening behaviour of Yanji mudstone subjected to WDFT cycles. The deviator stress-axial 37 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**

Swelling rock and soils correspond to the deposits composed of large amounts of clay minerals (e.g. 43 44 montmorillonite and illite). These deposits are widely found in more than 60 countries such as Australia, China, Sudan, Spain, USA, etc [1, 2]. Due to the special mineralogical compositions, 45 swelling rocks and soils are diverse in appearance and hydro-mechanical properties. They can 46 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 [4-7]. It is therefore essential to thoroughly evaluate the mechanical behaviour of swelling rocks 51 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 cyclic climate loadings [22-26]. In this regard, most common environmental factor is wetting-drying 64 65 (WD) cycles. Nowamooz and Masrouri [27, 28] indicated that the successive WD processes induced a progressive accumulation of irreversible strains, and the loose and dense expansive soil 66 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].

In many seasonally frozen regions with monsoon climates, winters are cold with frequent and heavy snowfall, and summers are hot with sufficient rainfall. It would be more relevant to take into account the influences of both WD and FT processes. However, most of previous researches 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 the mechanical properties under the combined action of WD and FT cycles is quite scarce and the 85 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 different confining pressures were carried out on Yanji swelling mudstone specimens subjected to 93 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.

As shown in Fig. 1a, the mudstone was subjected to extensive crushing and backfilling works during the landslide remediation and the original structure of the mudstone was greatly disturbed by the artificial activities [43]. To represent field conditions, the tested mudstone was first air dried and 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%.

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

132

Table 1 Characteristics of the studied mudstone

Property				
Water content (%)				
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				
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

136

Fig. 3 Grain size distribution of the tested mudstone

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138 2.2 Specimen preparation

In the field, the mudstone was crushed and compacted without adding more water in September 2013 because the natural water content was close to its optimum water content (20.6%). To simulate field conditions, the crushed mudstone was wetted by spraying deionised water to achieve a natural water content of 20.0%. The wet mudstone was mellowed in plastic bags for 48 hours for water homogenization. After that, the wet mudstone was subjected to static compaction to reach a dry density of 1.67 Mg/m³, which corresponded to a compaction degree of 97%. The diameter and height of obtained specimens are 61.8 and 125 mm, respectively. In this work, 64 specimens wereprepared 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 $^{\circ}$ 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 $^{\circ}$ 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, the they were vacuum saturated with deionised water and then subjected to drying at a prescribed temperature (25 $^{\circ}$ C) and a 154 155 prescribed relative humidity (75%) in the control box until the water content achieved the initial one (20.0%). Afterwards, freezing-thawing cycles were applied to the specimens. They were wrapped 156 157 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 to the designed values of 150, 200, 300, and 400 kPa (corresponding to effective confining pressures of 50, 100, 200 and 300 kPa) for subsequent consolidation. Note that the adopted maximum confining pressure was determined according to the mudstone burial depth [19]. After that, according to ASTM D7181-20 [44], a shear rate as low as 0.015 mm/min was adopted for all tests, allowing a full dissipation of pore water pressure. During subsequent shearing, axial force and volume change were recorded. The tests ended when the axial strain reached 20%.



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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

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

178 To clarity 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.





Fig. 6 Deviator stress and volumetric strain versus axial strain curves for the specimens before and 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 3.2 Volumetric strain

Fig. 6 also depicts the variations of volumetric strain (ε_v) with axial strain (ε_1) under various 202 203 confining pressures before and after different WDFT cycles. For the specimens under low confining pressures, the volume change behaviour was characterized by contraction and subsequent dilatancy. 204 205 Furthermore, the higher the cyclic number, the more significant the dilatancy behaviour. Nevertheless, the dilatancy was reduced with increasing confining pressure. As the confining pressure increased 206 207 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 application of WDFT cycles. 209

210 3.3 Elastic modulus and Shear strength

As schematically illustrated in Fig. 7, the elastic modulus, peak shear strength and residual shear strength can be determined from the deviator stress-axial strain curve. The elastic modulus represents the tangent modulus at the beginning of shearing and it is defined as the ratio of deviator stress to the axial strain in the range of 0%-1% [45]. The peak shear strength refers to the maximum deviator 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 confining pressure. Additionally, the decreasing rate reduced with the increase of WDFT cycles and it 225 226 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 the confining pressure, the more significant the decreases of peak and residual shear strengths. The 231 232 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 elastic modulus and peak shear strength can be well described using the following equation with a 234 squared correlation coefficient R^2 as high as 0.969: 235

$$E_0 = 5547.603 + 79.081q_{\rm p} \tag{1}$$





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





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





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



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

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To further analyse the influence of WDFT cycles on the mechanical behaviour, the peak and residual shear strengths are replotted in the plane of deviator stress (q) and mean effective stress (p') in Fig. 11. As the cyclic number increased, the state lines at peak and critical states shifted down. According to the state lines, the effective friction angle and effective cohesion at critical and peak states can be determined using the following equations:

$$\sin\phi_{\rm p}^{\prime} = \frac{3M_p}{6+M_p} \tag{2}$$

253
$$c'_{\rm p} = \frac{S_{\rm p}(3-\sin\varphi_{\rm p})}{6\cos\varphi_{\rm p}}$$
(3)

$$\sin\phi'_{\rm r} = \frac{3M_r}{6+M_r} \tag{4}$$

255
$$c_{\rm r}' = \frac{S_{\rm r}(3-\sin\varphi_{\rm r})}{6\cos\varphi_{\rm r}}$$
(5)

where M_p and M_r are the slopes of the peak and critical state lines, respectively, S_p and S_r are the intercepts of the peak and critical state lines, ϕ'_p and ϕ'_r are the effective friction angles at the peak and critical states, and c'_p and c'_r refer to the effective cohesions at the peak and critical states. Fig. 12 displays the variations of the effective friction angles and effective cohesions at peak and critical states with the number of WDFT cycles. It could be observed that for all the specimens, the effective cohesion at peak state was significantly larger than the effective cohesion at critical state while the effective friction angle at peak state was slightly lower than that at critical state. As the cyclic number increased, the effective cohesions at both peak and critical states decreased significantly while the effective friction angles slightly. The relationships between the effective cohesion, effective friction angle and the number of WDFT cycles could be described using Eqs. (6-9):

267 $c_{\rm p}^{\prime} = 29.785 + 34.684 e^{-0.535n}$ (6)

268
$$c'_{\rm r}$$
=46.733e^{-0.754n} (7)

269
$$\phi'_{\rm p} = 26.959 - 2.398 {\rm e}^{-0.512n}$$
 (8)

$$\phi'_{\rm r} = 27.334 - 1.936 e^{-1.226n} \tag{9}$$





270

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



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

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279 *3.4 Poisson's ratio and dilatancy angle*

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

$$v = \frac{1 - k_c}{2} \tag{10}$$

$$\sin\psi = \frac{k_{\rm D}}{-2+k_{\rm D}} \tag{11}$$

Fig. 14a displays the variation of Poisson's ratio with WDFT cycles under various confining pressures. For the specimens under various confining pressures, the Poisson's ratio increased with the increasing cyclic number, albeit at a small increasing rate. After given WDFT cycles, the Poisson's 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 more contractive behaviour after given WDFT cycles. Fig. 14b presents the variations of dilatancy 293 294 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 dialatancy angle were not 299 noticeably influenced by the WDFT cycles. Additionally, the dialatancy 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.





Fig. 13 Determined Poisson's ratio and dilatancy angle



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

308 3.5 Microstructure analysis and deterioration mechanism of swelling mudstone

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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 be observed (Fig. 15a). The total intruded void ratio decreased after WDFT cycles because of slight 312 313 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 μ m, a medium pore population with a mean diameter of 0.55 μ m 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 819 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 aggregates were pulled apart by the increasing matric suction, and cracks appeared once the pulling 323 324 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).



20

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-a	aggregate and in	nter-aggregate	void ratios	of the r	nudstone	after var	ious	WDFT	cycles
-----	-----------------	------------------	----------------	-------------	----------	----------	-----------	------	------	--------

Cyclic number	<i>n</i> =0	<i>n</i> =2	n=4	<i>n</i> =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



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

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346 4 Modelling of mechanical behaviour subjected to WDFT cycles

In geotechnical engineering, constitutive model is very important for mechanical prediction. 347 Stress-strain relation is a measure of material stiffness and shear strength under different 348 349 circumstances and a key input parameter for mechanistic-empirical geotechnical engineering design 350 method. As discussed above, the specimens exhibited remarkable strain-softening bahaviour after WDFT cycles in this study. Although the Modified Cam Clay (MCC) model [51] has been widely 351 352 used in describing many important features of the mechanical behaviour of clay, it is limited to the softening simulation of the soils in a heavily overconsolidated state. For most clayey deposits, MCC 353 354 often underestimates the softening degree. As for other MCC-based models, such as the enforced strain-softening model [52] and Structured Cam Clay (SCC) model [53], the model parameters are 355 356 increasingly complex and the models are not easily realizable. In this work, to capture the strain-softening behaviour, a three-parameter nonlinear elastic constitutive model was proposed: 357

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

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

361

368

370

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

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

364
$$E_0 = \frac{\mathrm{d}q}{\mathrm{d}\varepsilon_1}\Big|_{\varepsilon_1 = 0} = \frac{1}{a} \tag{14}$$

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

$$a = \frac{1}{E_0} \tag{15}$$

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

$$\varepsilon_{1-\max} = \frac{a}{b-2c} \tag{16}$$

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

$$q_{\rm p} = \frac{1}{4(b-c)} \tag{17}$$

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

373 $q_{\rm r} = \lim_{\varepsilon_1 \to \infty} \frac{\varepsilon_1(a + c\varepsilon_1)}{(a + b\varepsilon_1)^2} = \frac{c}{b^2}$ (18)

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

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

377
$$c = \frac{2 - q_{\rm r}/q_{\rm p} - 2\sqrt{1 - q_{\rm r}/q_{\rm p}}}{4q_{\rm r}}$$
(20)

According to Eq. (15), the model parameter *a* represents the elastic modulus. The higher the value of *a*, the larger the elastic modulus. Based on Eqs. (17-18), the difference between parameters *b* and *c* reflects the peak shear strength while the ratio of *c* to b^2 is equal to the residual shear strength. The higher difference between *b* and *c* and ratio of *c* to b^2 mean a lower peak shear strength and a larger residual shear strength, respectively. As remarked earlier, Eqs. (6-9) describe the relationships between the effective cohesion, effective friction angle and the number of WDFT cycles. Thus, the shear strength parameters at peak and critical states for the specimens after various WDFT cycles can be obtained using Eqs. (6-9). Correspondingly, the shear strengths at peak and critical states for the specimens under various confining pressures can be calculated based on the relationship between the axial stress σ_1 and confining pressure σ_3 described by the Mohr-Coulomb criterion:

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

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

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

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

Table 3 The parameter values in the proposed model



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

407 **5 Conclusions**

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

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

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

438

439 **Declaration of Competing Interest**

440 The authors declare that they have no known competing financial interests or personal relationships

that could have appeared to influence the work reported in this paper.

442

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447 Data availability

448 Data will be made available on request.

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