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Experimental and Numerical Studies on Failure Behaviours of Sandstones Subject to Freeze-Thaw Cycles

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7 Abstract

8 The freeze-thaw induced damage of rock affects the durability and serviceability of geo-structures, 9 especially those constructed in the regions frequently impacted by climatic changes. A series of 10 laboratory tests including P-wave velocity tests, freeze-thaw tests, uniaxial compression strength (UCS) 11 tests and Brazilian tensile strength (BTS) tests are conducted to investigate the physical-mechanical 12 properties and failure behaviours of Tasmanian sandstones subjected to various freeze-thaw cycles. It 13 is observed that the P-wave velocity, BTS and UCS of the sandstone decrease as the number of freeze-14 thaw cycles increases, in which the decreasing rate from 0 to 20 freeze-thaw cycles was more 15 pronounced than that from 20 to 40 and 40 to 60 freeze-thaw cycles. Moreover, it is found that the main 16 failure mode of the sandstone changes from axial splitting to shearing along a single plane in the UCS 17 tests and from central smooth fractures to a central zigzag fracture in the BTS tests with the number of 18 freeze-thaw cycles increasing. Three-dimensional (3D) numerical modellings are then conducted using 19 a self-developed 3D hybrid finite-discrete element method (HFDEM) parallelized on the basis of 20 general-purpose graphic processing units (GPGPU) to further investigate the failure mechanisms of 21 Tasmanian sandstones subjected to various freeze-thaw cycles in the UCS and BTS tests. The 3D 22 numerical modellings agree very well with the experimental observations that the physical-mechanical 23 parameters of the sandstone degrade with the increasing number of the freeze-thaw cycles. Moreover, 24 the 3D numerical modellings reveal the deterioration and failure mechanisms of sandstones subjected 25 to various freeze-thaw cycles. For the sandstone specimens without subjecting to freeze-thaw cycles, 26 axial splitting is the main failure pattern while tensile and mixed-mode damages are the dominant failure 27 mechanism in the UCS tests. For the sandstone subjecting to various freeze-thaw cycles, the increasing 28 number of freeze-thaw cycles causes the macroscopic cracks to propagate, interact and coalesce in the 29 shear behaviour resulting in the final shear fracture pattern in the UCS test. The 3D numerical 30 modellings of the BTS test show that, although, for both the models with and without subjecting to 31 freezing and thawing cycles, a central fracture is the eventual failure pattern, the failure surface becomes 32 more zigzag as the number of freeze-thaw cycles increases.

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34 **1. Introduction**

35 Rock is one of the most common natural construction materials on Earth. Cold regions, including permafrost areas and seasonally freezing areas, constitute a large portion of Earth's 36 37 continent. More than 20% of the continental surface in the northern hemisphere are occupied 38 by permafrost, whereas more than half of the surface experiences seasonal freezing and 39 thawing [1]. In the southern hemisphere, Antarctica is known as the coldest continent on the 40 Earth. In most coastal regions of Antarctica, the mean annual temperature is around -12°C 41 while the lowest surface temperature ever recorded is -89°C at Russian Vostok station within 42 Australian Antarctic Territory [2]. Tasmania has a generally cooler and wetter climate than that 43 in other states of Australia. The temperature of Tasmania in winter can go as low as -14 °C in 44 the highlands such as Central Plateau although the coastal areas rarely go below freezing and the summer temperatures can reach up to 41°C. While increasing human economic activities 45 46 have led to the construction of more and more geotechnical engineering structures in cold regions, it is expected that, in the future, more infrastructures will be constructed in southern 47 hemisphere. 48

49 Any kind of natural rocks and geo-structures in these cold regions are not only subjected to 50 external loads but also experiencing severe environmental effects. Freeze and thaw induced 51 settlement is a common phenomenon observed in seasonal freezing areas and is the cause of 52 significant damage to the built infrastructure [3], [4]. As an example, an uplifting displacement 53 of 88 mm has been caused by the freeze and thaw cycles and has led to the deformations and 54 crackings of the supporting concrete structure of the centre columns of the Myllypuro Ice Rink facility in Finland [3]. Long-term monitoring of average global temperatures indicate an 55 56 ongoing change of the climate [5], which has resulted increasing temperatures in permafrost. 57 The increasing temperature leads to thawing of portions of the permafrost area and thickening 58 of the upper crust layer where the active freezing-thawing cycles occur, which then result in 59 considerable settlement of the ground surface damaging infrastructures. Therefore, 60 understanding the behaviour of rock upon the freezing and thawing cycles is essential to adapt the operation practices, design philosophies and develop suitable methods to minimize the 61 62 damages [6].

One of the earliest documented cases of investigating the behaviour of grounds subjected to the freezing and thawing cycles was the use of artificial freezing at Brunkeberg tunnel in Stockholm in 1876 [7]. As the industrial development was accelerating in northern Russia to develop railways and mining operations in the late nineteenth century, the study of permafrost as an applied science began to protect civil structures from freeze-thaw damages [1]. The deterioration of rock under frost-heave and freeze-thaw cycles has become an unavoidable concern in engineering such as road, railroad, bridge, tunnelling, pipeline, and building constructions [8]–[10].

71 The deterioration of rock under freeze-thaw actions has been investigated by many researchers through relating one or more physical-mechanical properties of rocks such as moisture content, 72 73 mineralogical composition, texture, rock strength, and pore microstructure to single factor of freeze-thaw actions such as temperature range, frequency and applied stress. However, the 74 75 freeze-thaw action and loading are both repetitive cycles and the joint action may worsen the damage of rock and lead to severe deterioration of the long-term mechanical properties of rock. 76 77 Tan et al. [10] found that the elastic modulus, compressive strength and cohesive strength of biotite granite decreased significantly with the number of freeze-thaw cycles increasing. A 78 79 statistical model was developed by Bayram [11] to estimate the UCS of limestones after subjecting to freeze-thaw cycles. Another statistical model was proposed to experimentally 80 investigate various rock index properties, including the dry density, ultrasonic velocity, point 81 load strength, and slake-durability test indexes, after freeze-thaw cycles [12]. Wang et al. [13] 82 83 investigated the physical properties of sandstone specimens subjected to freeze-thaw cycles, including the density, porosity, P-wave velocity, UCS, and deformation modulus. Various 84 85 researchers have extensively studied the mechanical behaviours of transversely isotropic rocks such as slate subjected to freeze-thaw cycles and developed some corresponding models [14]-86 87 [21]. Moreover, another study [22] conducted triaxial compression tests with various confining pressures of 5, 10 and 20 MPa to investigate the change of wave velocities, mechanical 88 89 properties and permeability characteristics of red sandstone. The specimens were divided into 90 four groups, and underwent 0, 4, 8 and 12 freezing and thawing cycles. It was found that the 91 increase of the numbers of freeze-thaw cycles decreased the UCS, elastic modulus, cohesion, angle of internal friction and P-wave velocity of the red sandstone. It was concluded that the 92 93 damage of rock caused by the freeze-thaw cycles led to the changes of microstructure and then 94 mechanical performance. Ultrasonic and mechanical tests including compression, tension and 95 shear, of red sandstones subjected to freeze-thaw cycles were carried out in another study, which concluded that the red sandstone performed worse as the numbers of freeze-thaw cycles 96 97 increased [23]. Generally the rock with a lower porosity has a higher strength so that a greater 98 suction force (hence, a lower temperature) is required for the low porosity rocks to crack [24]

99 during the freeze-thaw tests. According to Matsuoka [24], high porosity rocks begin to crack 100 at 0 to -1 °C and terminate at about -5 °C in the freeze-thaw tests, while in medium porosity 101 rocks the cracking temperature is between -3 °C and -6 °C and in low porosity rocks the 102 cracking starts below -4 °C.

To summarize the findings above, although many researches have been conducted based on laboratory tests to investigate the effect of freeze-thaw cycles and have shown a decrease of the mechanical properties of the rocks with the freeze-thaw cycles increasing, there is no consensus on the effects of the specific number of freeze-thaw cycles, freezing temperature and combination of freeze-thaw cycles with loads on the deterioration of rock subjected to various number of freeze-thaw cycles.

109 In order to gain a further understanding of the effect of freeze-thaw cycles on the rock 110 deterioration and failure, some scholars have implemented numerical methods to analyse the 111 freeze-thaw induced damage besides the experimental methods. A simple linear stress-strain constitutive relationship was implemented into a two-dimensional (2D) plane strain finite 112 element model with a thermal-mechanical-flow coupling to simulate freeze-thaw experiments 113 [25]. An isogeometric analysis-based numerical model was applied to simulate the thermo-114 115 hydromechanically coupled processes in ground freezing [26]. Another research conducted numerical analyses to investigate the stability of a rock mass slope subjected to freeze-thaw 116 cycles [27], in which the freeze-thaw coefficient of the rock was firstly measured and the 117 generalized Hoek-Brown criterion was then employed to determine the parameters of slope 118 rock mass stablishing a numerical rock slope excavation model. According to another 119 120 numerical study, the volume expansion from the freeze-thaw cycles applied an stable force on 121 joints which resulted in the tensile stress concentration area emerging at the joint tip [28]. In another research [29], the variations of the modal parameters such as resonant frequencies, 122 123 damping ratios and mode shapes were analysed to assess the freeze-thaw resistance of limestone through a finite element model. 124

These numerical studies greatly increased our understanding of the effect of freeze-thaw cycles on the physical-mechanical behaviour of rocks. However, most of those numerical modellings are conducted in two-dimension (2D). Three-dimensional (3D) modelling is seldom conducted to investigate the effect of freeze-thaw cycles on the deterioration of the physical-mechanical parameters and failure progressive process although 3D modellings of rock failure process under mechanical loads using various numerical methods have been conducted in literatures

[30], [31], [32], [33]. Moreover, although complex thermo-mechanical-hydro coupling 131 modelling or even 3D modelling have been conducted using finite element methods (FEM) in 132 several literatures, the fracture behaviour of rocks is not modelled due to the limitation of 133 continuum mechanics - based FEM in dealing with rock fracture and fragmentation. Besides, 134 there is a lack of research on the effect of freeze-thaw cycles on the deterioration of rocks in 135 136 Australia or even southern Hemisphere. Therefore, further research is necessary to investigate the fracture mechanism of rock under freeze-thaw cycles using advanced numerical methods 137 138 which are capable of modelling rock fracture.

Correspondingly, the main aim of the present work is to experimentally investigate the physical-mechanical and failure behaviours of Tasmanian sandstones subjected to the freezing and thawing cycles and then numerically simulate them using a self-developed 3D hybrid finite-discrete element method (HFDEM), which is parallelized on the basis of general purpose graphic processing units (GPGPU) and capable of modelling rock fracture and fragmentation [34]–[36].

The remaining part of this paper is organized as follows; both the experimental testing method 145 and numerical method are firstly presented. The deterioration of the physical-mechanical 146 147 properties of the sandstone subjected to various freeze-thaw cycles are then quantified using the experimental testing method and discussed on the basis of damage mechanics, which 148 149 justifies that the freeze-thaw induced damage can be modelled by deteriorating the physicalmechanical parameters of rocks with the number of freeze-thaw cycle increasing. After that, a 150 151 GPGPU-parallelized HFDEM is implemented to model the damaging mechanism and failure 152 progressive process of the sandstone subjected to various freeze-thaw cycles in the uniaxial 153 compression strength (UCS) tests and Brazilian tensile strength (BTS) tests. Finally, the results from the experimental tests and numerical simulations are compared with each other to 154 155 conclude the deterioration and failure mechanism and the fracture patterns of the sandstones subjected to various freeze-thaw cycles. 156

157 2. Research Methods

158 **2.1. Experimental tests**

The target sandstone cores were obtained through diamond drilling with the drill size of HQTT
from the construction site of the Museum of Old and New Art (MONA 42.8125° S, 147.2615°
E) located on the Berriedale peninsula in Tasmania, Australia. The diamond drilling was
conducted perpendicularly to the bedding plane and the sandstone specimen was then cut

perpendicularly to the axis of the drill cores, i.e. parallelly to the bedding plane but the specimens around the bedding plane were dropped to avoid any obvious heterogeneities. Each sandstone specimen for the UCS test had a diameter of 60 mm and a height of 165 mm while that for the BTS test had a diameter of 60 mm and a height of 30 mm. Thus, the height to diameter ratios of the rock specimens were 2.75 and 0.5 for the UCS and BTS tests, respectively, and the sizes of the specimens met the requirements of ISRM for both UCS and BTS tests [37]. Before any UCS or BTS tests, P-wave velocities of all the rock specimens were measured using

a Pundit PL-200 testing equipment from Proceq, as shown in Fig. 1.

After that, the rock specimens were divided into 4 sets with 6 specimens in each set (three for 171 172 UCS and another there for BTS). One set of specimens were directly subjected to the UCS and BTS tests. This set corresponded to the reference case of zero freeze-thaw cycle. For other 3 173 174 sets of specimens, before either UCS or BTS tests were conducted, the specimens were firstly immersed in the pure water for two weeks and then put into a climatic cabinet for freeze-thaw 175 176 tests. A climatic controlled cabinet with an environmental chamber controller (Fig. 2. i)) was employed for the freeze-thaw tests, during which the conditions of the climatic cabinet, i.e. 177 temperatures and moistures were set following those in previous researches [22], [38]-[40] and 178 corresponding to the climatic conditions in Tasmania. As can be seen from Fig. 2. ii), one cycle 179 180 consisted of 9 hours of freezing time (which included 1 hour for reducing the temperature from +20 to -20 °C and 8 hours at the constant of -20 °C) and 9 hours of thawing time (which 181 182 included 1 hour for increasing the temperature from -20 to +20 °C and 8 hours at the constant of +20 °C), which were efficient for the heat transfer and uniform temperature distribution 183 184 during each phase according to previous researches [22], [38]–[40]. During the switch between the freeze-thaw cycles, the temperature varied between +20 °C to -20 °C by the changing rate 185 of 0.67 °C/min. The humidity in the climatic cabinet was kept 100%, which made sure that the 186 187 specimens would not lose any moistures although they were not sealed. The number of freezethaw cycles for three sets of the rock specimens was set as 20, 40 and 60. 188

After the freeze-thaw tests, the P-wave velocities of cylindrical sandstone specimens along their axial directions were measured again under the room temperature +20 °C. Finally, the UCS and BTS tests were conducted for all the specimens in each set using a Matest high stability compression testing apparatus with an automatic servo-controlled system (Fig. 3) by following the ISRM standards. The testing system includes a hydraulic loading system with the capacity of 2000 kN, an electronic measurement system and a data acquisition and processing UTMII software which records all testing data in a USB disc drive.

196 **2.2. Numerical methods and models**

197 2.2.1 Introduction to the numerical method of GPGPU-parallelized HFDEM

GPGPU-parallelized HFDEM is to be implemented to model the damaging mechanism and 198 199 failure progressive process of the sandstones subjected to various freeze-thaw cycles in the UCS and BTS tests. It is a hybrid finite-discrete element method (HFDEM) developed by the 200 201 authors [41] on the basis of the open source Y library [42] and further parallelized by the authors on the basis of general-purpose graphic processing units (GPGPU) [36]. The GPGPU-202 203 parallelized HFDEM has passed a series of fundamental rock mechanics tests [41], [43] and 204 provides a powerful numerical tool to investigate the fracture and fragmentation of rocks under various static and dynamic loading conditions. Compared with the numerical methods 205 206 reviewed in the introduction section, GPGPU-parallelized HFDEM can naturally model the damage and cracking behaviour of rocks induced by the freeze-thaw cycles and the fracture 207 208 and fragmentation of rocks in the UCS and BTS tests through its damaging and fracturing 209 algorithm, i.e. transition from continuum to discontinuum, which is explained in next paragraph 210 since a complete introduction of GPGPU-parallelized HFDEM can be found in the authors' 211 former publications [36], [41].

In GPGPU-parallelized HFDEM, rocks are modelled in 3D and discretized into an assembly 212 of tetrahedral finite elements in the same way as that in the traditional FEM. Different from the 213 traditional FEM, initially zero-thickness cohesive elements are inserted between any adjacent 214 215 finite elements, which may damage and then completely break to allow open and/or slide to 216 occur depending on local loading conditions there. The damage of the cohesive elements follows the concept of a smeared crack [35], [44], as shown in Fig. 4. Once the Mohr-Coulomb 217 strength criterion with tension cut-off is satisfied, the cohesive element damages in either 218 219 tensile or shear modes, i.e. mode-I or mode-II, following the tensile or shear softening curves shown in Fig. 4. i) and ii), respectively. The corresponding mode-I and mode-II damage 220 variables are defined as $D_I = (o - o_p)/o_t$ and $D_{II} = (s - s_p)/s_t$, respectively, where o and s 221 are the opening and sliding displacements, respectively, o_p and s_p are the elastic limits of the 222 223 opening and sliding displacements, and o_t and s_t are the ultimate limits of the opening and sliding displacements. The mixed-mode damage variable is defined as $D_{Mixed} =$ 224 $min\left(1,\sqrt{D_I^2+D_{II}^2}\right).$ 225

227 2.2.2 Numerical models

The modelled cylindrical specimens have a size of 60 mm in diameter and 165 mm in height 228 in the UCS test (Fig. 5. i)) and a size of 60 mm in diameter and 30 mm in height for the BTS 229 230 test (Fig. 5. ii)). For the UCS test, two loading plates with each having a size of 70 mm in diameter and 10 mm in height were placed at the top and bottom of the specimens and moved 231 towards each other with a constant vertical loading rate of 5 cm/s to apply a uniaxial load on 232 the cylindrical specimens through Coulomb frictional contacts. The cylindrical specimen was 233 234 completely free except the contacts. The cylindrical model was discretised into 97,162 tetrahedral elements, 19,511 nodes and 143,639 initially zero-thickness cohesive elements. For 235 236 the BTS test, two square loading plates with each having a length of 60 mm and a width of 30 mm were placed at the top and bottom of the specimens and moved towards each with a 237 238 constant vertical loading rate of 5 cm/s, too. The BTS model was discretised into 64,513 tetrahedral elements, 12,307 nodes and 125,222 initially zero-thickness cohesive elements. The 239 240 input parameters assigned to the models are obtained following an iterative calibration procedure until the stress-strain curve and the failure pattern from the simulations closely 241 matched those obtained from the laboratory tests. Four sets of separate calibrations were 242 performed for both control specimens and those subjected to 20, 40 and 60 freezing and 243 244 thawing cycles. The input parameters obtained during the calibration procedures generate 245 acceptable strength and elastic properties as well as fracture patterns, which, however, are not 246 a unique set [45]. Although the loading velocity of the simulation is higher than that in the laboratory experiment, it has been demonstrated that a quasi-static loading condition is ensured 247 through the artificial critical damping scheme with viscous damping coefficient [36], [46]. A 248 mass scaling factor of 10 is further applied to reduce the computing time of the 3D FDEM 249 250 simulations, which won't affect the obtained simulation results notably [36], [46].

251 It must be emphasized that the input parameters related to fracturing (i.e. tensile strength, cohesion, internal friction angle and fracture energies) in FDEM should be considered as 252 253 microscale parameters and are different from the general macroscale mechanical parameters as 254 obtained in the UCS and BTS tests. Meanwhile, the macroscopic input parameters including 255 elastic modulus and Poisson's ratio, can be directly obtained from experimental tests. In quasi-256 static condition, gravity is neglected since the density has no physical meaning in this case. For 257 model calibration purposes, it is convenient to classify the input parameters into the following categories: (1) penalty terms and numerical control parameters namely contact penalty, and 258 259 cohesive penalty, (2) macroscopic parameters, including the elastic modulus (E), and Poisson's

260 ratio (v), and (3) microscopic parameters including microscopic tensile strength, microscopic cohesion and microscopic internal friction angle. For (1), we follow the procedure for the 261 determination of these values established in [36], [47]. For macroscopic physical parameters 262 in (2) appropriate values can be directly established according to those from direct 263 measurements. One of the most critical steps in modelling with FDEM is the calibration of 264 265 input microscale parameters in (3). In general, the calibration process is a tedious trial and error process against the physical testing results. Table 1 lists the input parameters of the numerical 266 267 modelling for the control specimen, which is defined as the specimen without subjecting to any 268 freezing and thawing cycles and the specimens subjected to 60 freezing and thawing cycles which selected among three specimens with its UCS and elastic modulus closest to the average. 269

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3. Experimental and numerical results and analyses

272 3.1. Deterioration of physical-mechanical properties of sandstones subjected to various 273 freeze-thaw cycles

Table 2 summarizes the variations of the P-wave velocity, elastic modulus, UCS and BTS of 274 275 the sandstones subjected to 0, 20, 40 and 60 freeze-thaw cycles. Fig. 6. i) illustrates the change 276 of the P-wave velocity of the sandstone with the increasing number of freeze-thaw cycles. In 277 terms of the average value, the P-wave velocity reduced by 9%, 23% and 24% after 20, 40 and 278 60 freeze-thaw cycles, respectively. The reason might be that the increased freeze-thaw cycles resulted in increased micro cracks, which led to the reduction of P-wave velocity. Moreover, 279 the size of pores become larger with the freeze-thaw cycle increasing, which resulted in longer 280 time for waves to transmit through the specimens due to longer path, or an increase in 281 discontinuities leading the decrease of the P-wave velocity. As shown by the trend line in Fig. 282 283 6. i) there was a linear correlation between the P-wave velocity and the number of the freeze-284 thaw cycles.

As can be seen from Fig. 6. ii), the average value of BTS of sandstones decreases from 5.35 MPa to 5.24 MPa, 5.10 MPa and 5.06 MPa with the number of the freeze-thaw cycles increasing from 0 to 20, 40 and 60, respectively. In other words, the tensile strengths of the sandstones subjected to 20, 40 and 60 freeze-thaw cycles decrease by 2.1%, 4.4% and 5.5%, respectively. The sandstone specimens subjected to the increased number of freeze-thaw cycles have more microcracks [48], which decrease the ability of the sandstone for saving the strain energy and thus reduce BTS. The correlation between BTS and the number of freeze-thaw cycles is also linear, as shown in Fig. 5, and the correlation coefficient is larger than that in thecase of P-wave velocity.

The effect of the number of the freeze-thaw cycles on average value of UCS of the sandstone is depicted in Fig. 6. iii), which shows the average UCS of the sandstones changes from 37.84 MPa to 35.18 MPa, 32.98 MPa and 29.15 MPa as the number of the freeze-thaw cycles increases from 0 to 20, 40 and 60, respectively. Thus, the UCS of the sandstone decreases overall as the number of the freeze-thaw cycles increases. There are some deviations in some of results, which can be considered by the cause of inhomogeneity inside the specimens [49].

300 Since there are various microcracks induced by the freeze-thaw cycles in the sandstone 301 specimens, these pre-existing microcracks with respect to the axial loading direction first close 302 during the UCS test when the applied compressive stress reaches to the crack-closure stress. 303 Further compressions induce local tensile stresses, which may exceed the local tensile strength at the tips of the pre-existing flaws and initiate the new cracks on those tips. Thereafter, these 304 cracks propagate in a manner of wing cracks, parallel to the applied maximum principal stress 305 306 [50] resulting in axial splitting. The increasing number of freeze-thaw cycles induces more freezing loads to be applied on the microcracks and pores in the sandstone. The freezing load 307 then further increases the number and size of the microcracks and pores, which result in new 308 309 microcracks generated by freeze-thaw cycles facilitating the crack propagation to occur during UCS discussed above. That is why, the UCS of sandstone reduces with the freeze-thaw cycles 310 increasing. The correlation in Fig. 6. iii) is still approximately linear although the correlation 311 312 coefficient is smaller.

Fig. 6. iv) shows the relationship between the elastic modulus of sandstones and the number of freeze-thaw cycles. It can be seen from Fig. 6. iv) that, when the number of freeze-thaw cycles is increased to 20, 40 and 50, the average elastic modulus reduces by 10.4%, 15.6%, and 16.9%, respectively, compared with that of the sandstone without subjecting to the freeze-thaw cycles. The correlation between the elastic modulus and the number of freeze-thaw cycles is also linear, as shown in Fig. 6. iv), and the correlation coefficient is larger than that in the case of P-wave velocity and UCS.

The mechanisms of the deterioration of the physical-mechanical properties of sandstones may be that the increased freeze-thaw cycles resulted in increased micro cracks. Jiang et al. [51] investigated the deterioration mechanisms using scanning electron microscope (SEM) and then concluded that the freeze-thaw cycles increased the width of microcracks resulting in

microscopic structure changes. Niu et al. [52] pointed out these microscopic structure changes 324 of the rocks were due to the coupling effects of water and temperature variation after 325 conducting experimental studies on crack coalescence behaviour of double unparallel fissure-326 contained sandstone specimens subjected to freeze-thaw cycles under uniaxial compression. 327 328 The effect of water includes the dissolution and stress effects while temperature variation 329 damages rocks by means of different temperature gradients and phase transitions of water 330 during the freeze-thaw process [52], [53]. Moreover, Zhou et al. [54] and Niu et al. [52] investigated the microstructure around the crack tips in sandstones with and without subjecting 331 332 to freeze-thaw cycles by SEM and concluded the nature of the failure of the flawed rock mass under freeze-thaw cycles lies in the fatigue damage of rocks. They further elaborated the stress 333 334 fields around the tips of pre-existing fissures periodically redistributed due to the repeated frostheaving actions in response to various freeze-thaw cycles, which led to the fatigue damage 335 around these tips of the fissures and contributed to the failure evolution of the flawed rock mass. 336 To better characterize the deterioration of the physical-mechanical properties of the Tasmanian 337 sandstones subjected to the freeze-thaw cycles, a damage variable, D, is defined on the basis 338 339 of the average uniaxial compressive strength as Eq. 1:

$$340 D = \frac{\overline{\sigma}_c - \overline{\sigma}'_c}{\overline{\sigma}_c} Eq. 1$$

where $\bar{\sigma}_c$ is the average uniaxial compressive strength of the sandstone specimens without the freeze-thaw cycles and $\bar{\sigma}'_c$ is that of the specimens subjected to the freeze-thaw cycles. The damage variables obtained using Eq. 1 are summarized in Table 1 for the sandstones subjected to 0, 20, 40 and 60 freeze-thaw cycles, which clearly shows that the damage variable increases as the number of the freeze-thaw cycles increases.

346 3.2. Three-dimensional modelling of the failure process of the sandstones without 347 subjecting to freeze-thaw cycles

Fig. 7. i) and ii) shows the modelled rock failure progressive process in terms of the distributions of the minor principal (i.e. compressive) stress and the damage variable [i.e., D, D=1 means macroscopic (broken) CE6 while D=0 corresponds to intact], respectively, at different loading stages (points A-F in the stress-strain curve in Fig. 7. iii) in the FDEM simulation of the UCS test for specimens without subjecting to freezing and thawing cycles. Fig. 7. i) A shows the stress in the specimen at the stage before the onset of nonlinearity in the stress-strain curve. The growth of unstable microscopic cracks commences as the loading displacement continues and extends until the peak stress of the stress-strain curve is reached (point B in Fig. 7. iii)). Afterwards, macroscopic cracks form due to the propagation and coalescence of the microscopic cracks coalescence, which results in the loss of bearing capacity of the sandstones. As can be seen in Fig. 7. i) C-E, the stress begins to decrease with increasing strain (point C-E in Fig. 7. iii)). Finally, the formed macroscopic cracks propagate further, resulting in the complete failure of sandstones (point F in Fig. 7. iii)).

Fig. 7. ii) shows the distributions of all damage (including pure mode-I, pure mode-II and 361 362 mixed-mode) (A-F-1) and pure mode-II damage (F-2) at different loading stages. As can be 363 seen from the comparison between Fig. 7. ii) A-F-1 and F-1, the tensile and mixed-mode 364 damage is the dominant failure mechanism at both pre-peak stage (A-B in Fig. 7. iii)) and postpeak stage (C-F-1 in Fig. 7. iii)). The modelled tensile and mixed-mode failure mechanism is 365 366 consistent with the multiple axial splitting failures observed in the experiments considering the limitation of FDEM in modelling pure mode I failure mechanism and usually treating it as 367 368 mixed-mode failure mechanism for the numerical simulations with unstructured meshes [29]. After that, at final stage of strain softening (F-1 in Fig. 7. iii)), the formed tensile and mixed-369 370 mode cracks propagate and coalescence forming the axial splitting failure pattern.

Fig. 8. i) and ii) illustrates the modelled rock failure progressive process in terms of the 371 distributions of the horizontal stresses and the damage variable D at different loading stages 372 (points A-D in the stress-strain curve in Fig. 8. iii)) in the FDEM simulation of the BTS test. It 373 374 can be seen from Fig. 8. i) A that uniform tensile stress fields are formed around the central 375 line of the disc and there is no failure at the stage before the peak stress (points A in Fig. 8. 376 iii)). Once the indirect tensile strength of the rock is reached (points B in Fig. 8. iii)), tensile 377 macroscopic cracks initiate around the central diametrical line of the rock disc. After that, the macroscopic cracks start to propagate along the central diametrical line, as shown in Fig. 8. i) 378 379 C. Till this stage, the tensile damage is the dominant failure mechanism, as indicated by the mixed-mode damage in Fig. 8. ii) A-C. Finally, the further propagation and coalescence of the 380 381 formed macroscopic cracks split the rock disc into two halves (Fig. 8. ii) D), during which there 382 are some shear damages around the two loading locations. Thus, the modelled failure 383 mechanism and fracture pattern are consistent with those observed in the experiments. 384 However, the numerical modelling shows a single failure surface along the central diametrical 385 line of the disc while the experiment indicates multiple failure surfaces. Double checking of the final fracture pattern in Fig. 8. ii) D reveals that there are multiple unmerged macro fractures 386 387 around the main fracture surface, which are similar to those observed in the BTS experiment.

388 3.3 Three-dimensional modelling of the failure process of the sandstones subject to 389 various freeze-thaw cycles

390 Fig. 9. i) and ii) illustrates the modelled rock failure progressive process in terms of the 391 distributions of the principal stress and the damage variable at different loading stages (points 392 A-F in the stress-strain curve in Fig. 9. iii) in the FDEM simulation of the UCS test of the 393 sandstone subjected to 60 freezing and thawing cycles. Fig. 9. i) A shows the stress distribution in the specimen at the pre-peak stress stage and the stress is lower than that at the same stage 394 395 in the control model (Fig. 7. i) A). At the peak stress (point B in Fig. 9. iii)), which is smaller than that of the control model (Fig. 7. i) B), the microscopic cracks propagate form a 396 macrocrack (Fig. 9. ii) B. As can be seen from Fig 9. i) C-E, the stress begins to decrease with 397 increasing strain (point C-E in Fig. 9. iii)). At the final stage (Fig. 9. i) F), the formed 398 399 macroscopic cracks propagate further, resulting in the complete failure of sandstones (point F 400 in Fig. 9. iii)).

Fig. 9. ii) A-F-1 and F-2 show the distributions of all damage and the pure mode-II damage 401 402 only, respectively, at different loading stages. It should be noted that all damage includes the pure mode-I, pure mode-II and mixed-mode damages. At the peak stress (point B in Fig. 9. iii)), 403 404 the dominant failure mechanism is the tensile and mixed-mode damages, which is also more 405 manifest than that in the previous modelling of the control specimen at the peak stress. Similar 406 to the modelling of the control model, the macro cracks form after the peak stress and the mixed-mode damage continues to be the dominant failure mechanism (Fig. 9. ii) C-F-1). 407 408 Finally, during the late strain-softening stage, the tensile and mixed-mode cracks propagate, 409 interact and coalesce in the mode-II damage mechanism resulting in the final shear failure 410 pattern (Fig. 9. ii) F-2).

The modelled failure progressive processes of the sandstone subjected to 60 freeze-thaw cycles 411 412 in the BTS test are illustrated in Fig. 10. i), ii) and iii) in terms of the distributions of the 413 horizontal stresses, the evolution of the damage variable D, and the completed stress-strain 414 curve, respectively. Before the peak stress (point A in Fig. 10. iii), almost uniform tensile 415 stresses around the central diametral line of the disc are observed again. At the peak load, the 416 macroscopic cracks are initiated in the mixed-mode along the central diametrical line of the 417 rock disc (Fig. 10. i) B). Compared with the control model, this model presents severe mixedmode damages at this stage due to the low physical-mechanical parameters for the rock 418 subjected to 60 freeze-thaw cycles. Afterward, the macroscopic cracks propagate along the 419

420 central diametrical line (Fig. 10. i) C) splitting the rock disc into two halves (Fig. 10. i) D). The

- 421 comparison between the failure mechanisms shown in Fig. 10. ii) C (mixed-mode I-II damage
- 422 only) and D (all damages) indicates that the mixed-mode damage is still dominant in the BTS
- 423 modelling of the sandstone subjecting to various freezing and thawing cycles.

424 4 Discussions

425 4.1 Comparisons between the experimental and numerical stress-strain curves of the 426 sandstones subject to various freeze-thaw cycles

427 Fig. 11. i-iv) compares the axial stress versus axial strain curves obtained from both 428 experimental tests and numerical simulations of sandstones subjected to various freeze-thaw cycles in the UCS tests, in which, the crack initiation stress (dark blue colour marks) and peak 429 430 stress (red colour marks) from both the laboratory tests and numerical simulations are marked and their values presented in Table 3. It can be seen from Fig. 11. that the experimental and 431 432 numerical stress-strain curves agree well with each other in the terms of the linear elastic stage, crack initiation stress, nonlinear crack growth stage, peak strength, post-peak unstable crack 433 434 growth stage and brittle failure behaviour although the numerical simulations fail to model the 435 initial compaction stage. All stress-strain curves from the laboratory tests show a nonlinear 436 downward concave shape in the initial compaction stage, which are omitted in Fig. 11. for the convenience of the comparisons between the laboratory tests and numerical simulations. The 437 initial compaction stage is resultant from the closure of initial pores and fissures within the 438 rocks subjected to various freeze-thaw cycles. The numerical simulations consider those initial 439 pores and fissures caused by various freeze-thaw cycles through deteriorating the physical-440 mechanical parameters of the rocks. Thus, it is reasonable that the numerical simulation is 441 442 unable to simulate the initial compaction stage. It is obvious from Fig. 11. and Table 3. that the 443 linear elastic stage and crack initiation stress from the laboratory tests and numerical 444 simulations are comparable, which also show that they decrease as the number of freeze-thaw 445 cycles increases. The numerical simulations are able to capture the nonlinear crack growth stages, too, although the exact forms of the nonlinear crack growth stages are not the same as 446 447 those from the laboratory tests. The peak strength and post-peak unstable crack growth stage of the stress-strain curves from the numerical simulation have an acceptable conformity with 448 449 those from the experimental tests, too. However, the post-peak stress curves from the 450 experimental tests become flatten with the number of freeze-thaw cycles increasing, i.e. 451 somewhat ductile crack growth in the early phase of the post-peak unstable crack growth stage, 452 which has not been captured by the numerical simulations. After that, a sudden loss of bearing 453 capacities does happen for all the stress-strain curves obtained from the laboratory tests, which454 is well captured by all numerical simulations.

455 4.2 Comparison between the experimental and numerical failure patterns of the 456 sandstones subject to various freeze-thaw cycles

Fig. 12. compares the final failure patterns of of the sandstone specimens subjected to various 457 458 freeze-thaw cycles in the UCS tests obtained from both laboratory experiments and numerical 459 simulations. It is found that all specimens without undergoing freeze-thaw cycles failed in axial 460 splitting during the laboratory UCS tests (Fig. 12. i)A). For the specimens subjected to 20 461 freeze-thaw cycles, one of the specimens still failed in axial splitting model while the other two 462 specimens failed in shearing along a single plane during the laboratory UCS tests (Fig. 12. i) 463 B). The failure mode of all the specimens subjecting to 40 freeze-thaw cycles changed to the 464 shear failure although shearing along a single plane was only observed for one specimen and 465 the other two specimens failed in double shear (Fig. 12. i) C). Moreover, all the specimens subjected to 60 freeze-thaw cycles failed in the shearing mode along a single plane (Fig. 12. i) 466 D). Thus, the analysis of the failure mode shows that it changes from the axial splitting to 467 various shear failure modes in the laboratory UCS tests of sandstones subjecting to with an 468 469 increasing number of freeze-thaw cycles. Moreover, it is noted that the shearing along a single 470 plane is the most common failure mode of the specimens in the laboratory UCS tests after subjecting to the freeze-thaw cycles. As it is clear from Fig. 12. ii) A, the failure pattern of the 471 control specimen from the numerical simulations was axial splitting failure, which agrees with 472 the experimental observation (Fig. 12. i) A). With the number of freeze-thaw cycles increasing, 473 474 the failure pattern changed to the shear failure pattern, as shown in Fig. 12. ii) B, C and D 475 which agrees with the experimental observation, too (Fig. 12. i) B, C and D).

Fig. 13. compares the final failure patterns of of the specimens subjected to various freeze-476 477 thaw cycles in the BTS tests obtained from both laboratory experiments and numerical 478 simulations. It can be seen from Fig. 13. A that the failure pattern of the specimens subjecting 479 to zero freeze-thaw cycles from both laboratory experiments and numerical simulations was 480 central multiple splitting fractures. The failure pattern of central multiple splitting fractures is 481 the dominant failure mode in the BTS test of sandstones with a relatively higher tensile strength since the central multiple splitting fractures are developed in order to release the stored high 482 strain energy. The main failure pattern changed to a central fracture for most of the specimens 483

treated by various freeze-thaw cycles Fig. 13. B-D, which can be explained by the increasing
number of microcracks in sandstones caused by various numbers of freeze-thaw cycles.

486 The change of the failure patterns from the axial splitting failure to shear failure in the UCS 487 tests and those from the central multiple splitting failures to the central single fracture in the BTS tests can be explained in terms of the pre-existing cracks, the local fatigue-damage zones 488 489 around these pre-existing cracks caused by the freezing and thawing cycles [51, 53], and their propagations during the uniaxial loading of the sandstones subjected to various freeze-thaw 490 cycles. Under uniaxial loading in the UCS test, the pre-existing cracks become free to 491 propagate when they are aligned parallel to the maximum principal stress (mostly tensile). In 492 493 other words, the specimen fails in axial splitting mode when the microstructure of a specimen 494 cannot prevent the propagation of these cracks [55], which is the case for the sandstone without 495 subjecting to the freeze-thaw cycles. For the sandstone subjected to the freeze-thaw cycles, 496 local fatigue-damaged zones form around the pre-existing cracks according to [51, 53], which 497 are further motivated during the loading process. When the compressive loads are applied during the UCS tests, the pre-existing cracks propagate through these local fatigue damaged 498 499 zones. Since these local fatigue damaged zones are not necessary to be parallel to the maximum principal stress, the coalescences of the pre-existing cracks take place in adjacent or in close 500 501 proximity to the tips of the suitably oriented microcracks and releases the strain energy in the form of shear failure, which is why macroscopic shear failure patterns are observed during the 502 503 UCS tests of the sandstones subjected to more freeze-thaw cycles. Similarly, during the BTS tests of the sandstones without subject to the freeze-thaw cycles, the pre-existing cracks aligned 504 505 along the loading diametral line are forced to initiate and propagate resulting in smooth central splitting fracture patterns. For the sandstone samples subjected to various freeze-thaw cycles, 506 507 the freeze-thaw cycles induce the local fatigue-damaged zones around the tips of the pre-508 existing cracks, which cause the pre-existing crack to initiate, propagate and coalesce through 509 these local fatigue-damage zones resulting in more zigzag central splitting fracture patterns.

However, the 3D numerical simulations model the random pre-existing defects in sandstones through non-uniform meshes and model the freeze-thaw induced damages through deteriorating physical-mechanical properties of the sandstones subjected to various freezethaw cycles. Thus, the 3D numerical simulations fail to capture the local fatigue-damaged zones around the tips of the pre-existing defects due to the freeze-thaw cycles although the 3D numerical simulations well capture the deterioration mechanism and failure process of the 516 sandstones including the transition of the failure patterns of the sandstones with the number of 517 freeze-thaw cycles increasing.

518 **5. Conclusions**

A series of laboratory tests were firstly conducted to investigate the physical-mechanical behaviours of Tasmanian sandstones subjected to different freezing and thawing cycles, which includes P-wave velocity, UCS, BTS and freeze-thaw tests. A self-developed 3D GPGPUparallelized FDEM is then employed to model the failure processes of these sandstones with the focus on clarifying the damage mechanisms of the sandstones subjected to various freezethaw cycles. The following specific conclusions were obtained:

- The laboratory tests showed that P-wave velocity, UCS and BTS of Tasmanian sandstones reduced with the number of freeze-thaw cycles increasing. Their reduction rates from 0 to 20 freeze-thaw cycles were higher than those from 20 to 40 cycles and from 40 to 60 cycles.
- The main failure pattern of the sandstone changed from axial splitting to shearing in the
 UCS tests and from smooth central fractures to a zigzag central fracture in the BTS tests
 with the number of freeze-thaw cycles increasing.
- The GPGPU-parallelized 3D modelling reproduces the failure processes of Tasmanian sandstones subjected to various freeze-thaw cycles in the UCS and BTS tests, which validates the experimental observations that the physical-mechanical parameters of the sandstone degrade with the increasing number of freeze-thaw cycles.
- The 3D modelling clarifies the deterioration and failure mechanisms of sandstones 536 subjected to various freeze-thaw cycles. For the sandstone without subjecting to freeze-537 thaw cycles, axial splitting is the main failure pattern in the UCS test since tensile and 538 539 mixed-mode damages are dominant failure mechanisms observed in the corresponding 3D modelling. For the sandstones subjecting to various freeze-thaw cycles, the 540 541 increasing number of freeze-thaw cycles causes the macroscopic cracks initiated and nucleated by the tensile and mixed-mode damages to propagate, interact and coalesce 542 543 in the mode-II failure mechanisms resulting in the final shear fracture pattern in the 544 UCS test. For both the control model and that subjected to freezing and thawing cycles 545 in the BTS test, a central splitting fracture is the dominant failure pattern, which, however, is accompanied by zigzag fractures in the BTS test of the specimens subject 546 547 to various freeze-thaw cycles.

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551

552 Data availability

553 The raw/processed data required to reproduce these findings are available from the 554 corresponding author upon request.

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- 708
- 709 List of Tables
- Table 1. Physical-mechanical properties of Tasmanian sandstones subject to various freeze-thaw (F-T)
 cycles
- Table 2. Input parameters used in the FDEM simulations of the behaviour of the control specimens and
 the specimens subjected to 60 freezing and thawing cycles in the UCS tests
- Table 3. Comparisons between the experimental and numerical stress and elastic modulus
- 715
- 716
- 717 List of Figures
- Fig. 1. Pundit PL-200 equipment for measuring P-wave velocities of the cylindrical rock specimens
- 719 Fig. 2. Climatic controlled cabinet for the freeze-thaw tests
- Fig. 3. Servo-controlled high stability compression testing apparatus for the UCS and BTS tests

- Fig. 4. Constitutive behaviour of cohesive elements under the tension and shear & sliding conditions: i) under tension conditions, and ii) under shear and sliding conditions (Modified after [36])
- Fig. 5. Fig. 5. 3D numerical models for simulating the behaviour of the sandstones in the UCS and BTS tests: i) UCS and ii) BTS
- Fig. 6. Deterioration of the physical-mechanical parameters of the sandstones subjected to various freeze-thaw cycles i) P-Wave velocity, ii) Brazilian tensile strength, iii) uniaxial compressive strength, and iv) elastic modulus
- Fig. 7. 3D modelling of the failure process of the sandstones without subjecting to any freeze-thaw cycles in the UCS test: i) Distribution of minor principal (most compressive) stresses in selected loading steps, ii) Distribution of the damage variable in selected loading steps, and iii) Stress-strain curveFig. 8. Final failure patterns observed in the UCS tests of the sandstones subjected to various freeze-thaw cycles
- Fig. 8. 3D modelling of the failure process of the sandstones without subjecting to any freeze-thaw cycles in the BTS test: i) Distribution of horizontal stresses, ii) Distribution of the damage variable, and iii) Stress-strain curve
- Fig. 9. 3D modelling of the UCS test of the sandstone subjected to 60 freeze-thaw cycles: i) Distribution of minor principal (most compressive) stresses in selected loading steps, ii) Distribution of the damage variable in selected loading steps, and iii) Stress-strain curve
- Fig. 10. 3D model of the BTS test for the sandstone subjected to 60 freezing and thawing cycles: i) Distribution of horizontal stresses, ii) Distribution of the damage variable, and iii) Stress-strain curve
- Fig. 11. Comparisons of the stress-Strain curves from both experimental tests and numerical simulations of the UCS tests of the sandstones subjected to various freezing and thawing cycles: i) 0 freeze-thaw cycle, ii) 20 freeze-thaw cycles, iii) 40 freeze-thaw cycles, and iv) 60 freeze-thaw cycles
- Fig. 12. Comparisons between the failure patterns of the sandstones subjected to 0 (A), 20 (B), 40 (C) and 60 (D) freeze-thaw cycles in the UCS tests obtained from i) laboratory experiments and ii) 3D numerical simulations
- Fig. 13. Comparisons between the failure patterns of the sandstones subjected to 0 (A), 20 (B), 40 (C) and 60 (D) freeze-thaw cycles in the BTS tests obtained from i) laboratory experiments and ii) 3D numerical simulations

769 Table 1. Physical-mechanical properties of Tasmanian sandstones subject to various freeze-thaw (F-

T) cycles							
Specimen	Number of F-T Cycles	P-wave before F-T (m/s)	P-wave after F-T	BTS (MPa)	UCS (MPa)	Elastic Modulus (GPa)	Average damage variable
<u> </u>	0	2448	(11/5)	5 53	30.93	45	vuriuoie
S0-2	0	2843	_	5.87	38.93	7.9	0
S0-3		2815	-	5.17	36.76	7.7	
S20-1	20	2192	1697	5.97	27.70	5.6	
S20-2		2816	2646	4.82	39.59	8.2	0.07
S20-3		2758	2539	4.91	38.26	7.1	
S40-1	40	2659	2297	4.67	34.06	6.7	
S40-2		2821	2443	5.57	42.16	9.9	0.13
S40-3		2820	2492	5.11	31.89	6.3	
S60-1	60	2753	1262	5.10	22.90	4.9	
S60-2		2845	2389	5.23	30.40	6	0.23
S60-3		2618	2323	4.83	34.16	7.3	



Parameter	Unit	Control	60 freeze-thaw cycles
Density	kg/m ³	2200	2200
Young's modulus	GPa	7.7	6.1
Poisson's ratio	-	0.25	0.25
Microscopic tensile strength	MPa	5.35	5.05
Microscopic cohesion	MPa	10	7.7
Microscopic internal friction angle	0	25.5	25.5
Microscopic mode I fracture energy	J/m ²	80	50
Microscopic mode II fracture energy	J/m ²	300	180
Normal contact penalty	GPa	77	61
Cohesive normal penalty	GPa	770	610
Cohesive tangential penalty	GPa	770	610

Table 2. Input parameters used in the FDEM simulations of the behaviour of the control specimens and
 the specimens subjected to 60 freezing and thawing cycles in the UCS tests

			Crack initiation stress (MPa)	Peak strength (MPa)	Elastic modulus (GPa)
	Control	Experimental	36.88	38.93	7.9
		Numerical	37.07	38.50	7.5
	20 Freeze-Thaw	Experimental	34.72	38.26	7.1
	40 Freeze-Thaw	Experimental	29.69	31.89	6.3
	cycles	Numerical	28.70	31.69	6.2
	60 Freeze-Thaw	Experimental	28.03	30.40	6.0
	cycles	Numerical	27.42	29.38	6.1
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818 Table 3. Comparisons between the experimental and numerical stress and elastic modulus



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846	Fig. 1. Pundit PL-200 equipment for measuring P-wave velocities of the cylindrical rock specimens
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ii). Generalized temperature versus time curve of one freeze-thaw cycle





881 Fig. 3. Servo-controlled high stability compression testing apparatus for the UCS and BTS tests







Fig. 6. Deterioration of the physical-mechanical parameters of the sandstones subjected to various
freeze-thaw cycles i) P-Wave velocity, ii) Brazilian tensile strength, iii) uniaxial compressive strength,
and iv) elastic modulus



i) Distribution of minor principal (compressive) stresses

(A) (B) (C) (D) (E) (F-1) (F-2)

ii) Distribution of the damage variable (A-E and F-1: all damage including pure mode-I, pure mode-II and mixed-mode while F-2: pure mode-II damage only)



iii) Stress-strain curve

Fig. 7. 3D modelling of the failure process of the sandstones without subjecting to any freeze-thaw
cycles in the UCS test: i) Distribution of minor principal (most compressive) stresses in selected loading
steps, ii) Distribution of the damage variable in selected loading steps, and iii) Stress-strain curve

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i) Distribution of horizontal stresses



ii) Distribution of the damage variable



iii) Stress-strain curve

Fig. 8. 3D modelling of the failure process of the sandstones without subjecting to any freeze-thaw
cycles in the BTS test: i) Distribution of horizontal stresses, ii) Distribution of the damage variable,
and iii) Stress-strain curve

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ii) Distribution of the damage variable (A-E and F-1: all damage including pure mode-I, pure mode-II and mixed-mode while F-2: pure mode-II damage only)





- 980 Distribution of minor principal (most compressive) stresses in selected loading steps, ii) Distribution
- 981 of the damage variable in selected loading steps, and iii) Stress-strain curve



i) Distribution of horizontal stresses



iii) Stress-strain curve

Fig. 10. 3D model of the BTS test for the sandstone subjected to 60 freezing and thawing cycles: i)
Distribution of horizontal stresses, ii) Distribution of the damage variable, and iii) Stress-strain curve



Fig. 11. Comparisons of the stress-Strain curves from both experimental tests and numerical simulations
of the UCS tests of the sandstones subjected to various freezing and thawing cycles: i) 0 freeze-thaw
cycle, ii) 20 freeze-thaw cycles, iii) 40 freeze-thaw cycles, and iv) 60 freeze-thaw cycles



ii) Numerical simulations

Fig. 12. Comparisons between the failure patterns of the sandstones subjected to 0 (A), 20 (B), 40 (C)
and 60 (D) freeze-thaw cycles in the UCS tests obtained from i) laboratory experiments and ii) 3D
numerical simulations

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Fig. 13. Comparisons between the failure patterns of the sandstones subjected to 0 (A), 20 (B), 40 (C) 1032 1033 and 60 (D) freeze-thaw cycles in the BTS tests obtained from i) laboratory experiments and ii) 3D numerical simulations