Effects of cyclic freezing and thawing on the shear behaviors of an expansive soil under a wide range of stress levels

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Abstract
The focus of this paper was directed towards investigating the influence of multiple freeze–thaw (FT) cycles on the mechanical behaviors of an expansive soil under a wide range of confining stresses ($\sigma_c$) from 0 to 300 kPa. Consolidated undrained (CU) shear tests with pore pressure measurement ($\sigma_c=10$–300 kPa) and unconfined compression (UC) tests ($\sigma_c=0$ kPa) were conducted on FT impact specimens to derive the shearing stress–strain relationships and the associated mechanical properties including (i) failure strength ($q_u$), elastic modulus ($E_u$), effective and apparent cohesion ($c'$ and $c$), and effective and apparent friction angle ($\phi'$ and $\phi$) obtained from CU tests and (ii) $q_u$ and reloading modulus ($E_{1\%}$) and stress ($S_{1\%}$) at 1% strain obtained from UC tests. Besides, the influence of FT cycles on the soil structure was investigated using mercury intrusion porosimetry (MIP) and scanning electron microscope (SEM) tests. Testing results show that FT cycles mainly influence the soil’s macropores with diameters between 5 and 250 μ. Cracks develop during FT cycles and result in slight swelling which contributes to an increase in the global volume of the soil specimens. There is a significant reduction in the investigated mechanical properties after FT cycles. They typically achieve equilibrium after about six cycles. The shearing stress–strain curves transit from strain-softening to strain-hardening as the confining stress increases. A simple empirical model is developed to well describe the strain-softening behavior of the stress–strain curves under low confining stresses.

Keywords Expansive soil · Freeze–thaw · Microstructure · Volumetric change · Shear behavior

Introduction
Expansive soils are typically referred to as problematic soils by geotechnical engineers due to their significant swelling and shrinkage characteristics upon variations in the water content (Nelson and Miller 1992; Zou et al. 2018). The complex variations in their volume and engineering properties usually result in damages to the associated geotechnical infrastructures, which include non-uniform settlement of structures, cracking of walls and pavements, and failure of expansive soil slope (Tripathy et al. 2002; Glade 2005; Ajdari et al. 2013; Zhang et al. 2015; Massat et al. 2016; Zou et al. 2020). Surficial expansive soils in cold regions are subjected to numerous freeze–thaw (FT) cycles annually (Lai et al. 2012; Kong et al. 2018). The growth of ice crystals, propagation of cracks, and the resultant weakening of the soil structure during freezing and thawing impose significant influences on the soil’s mechanical properties and usually result in remarkable volumetric changes (Ghazavi and Roustaei 2013; Aldaood et al. 2014, 2016; Zhang et al. 2015; Hotineanu et al. 2015; James et al. 2018). This imposes extra challenges to the safety and serviceability of geotechnical infrastructures in expansive soils in cold regions and may lead to serious damages such as massive landslides of water conveyance canals in northeastern China (Xu et al. 2016).
Current geotechnical and geological studies on expansive soils mainly focus on the effect of drying–wetting cycles on the structural, volumetric, and hydro-mechanical behaviors of expansive soils (Grant 1974; Albrecht and Benson 2001; Estabragh et al. 2016; Alonso et al. 2005; Cuisinier and Masrouri 2005; Pires et al. 2008; Nowamooz and Masrouri 2008; Ajdari et al. 2013). Current studies on the FT effects on soils are more focused on saline soil (Gray and Granger 1986), loess (Mahedi et al. 2019), dispersive soil (Han et al., 2019), silty soil (Jamshidi et al. 2014), and mine tailings (Proskin et al. 2010). The investigated soil properties include shear strength (Ahmadi et al. 2021; Ding et al. 2021), elastic moduli (Simonsen et al. 2002; Orakoglu et al. 2017; Zou et al. 2020), durability (Vahdani et al. 2020; Tiwari et al. 2021), and water-retention capacity (Ren and Vanapalli 2020; Ding et al. 2021). In general, it was found that the growth of ice crystals and the propagation of cracks weaken the integrity of soils and thus lead to a significant reduction in the mechanical properties as well as the water-retention capacity. Such influences on soils generally come into equilibrium after several FT cycles.

Similar studies that are devoted to investigating the FT effects on expansive soils are comparatively less reported. There is a lack of studies that comprehensively reveal the evolution in expansive soils’ microstructure during FT cycles, and compare and model the corresponding variations in their mechanical properties including the stress–strain behaviors, shear strength, and elastic modulus under both drained and undrained conditions.

In this study, the volumetric characteristics, the stress versus strain relationships, and shear strength behaviors of an expansive soil collected from Northeastern China under the effects of FT cycles were investigated. The investigated mechanical behaviors include the following: (i) unconfined compression strength and the unloading–reloading modulus; (ii) shear strength, elastic modulus, and strength parameters (i.e. cohesion and internal friction angle) that were obtained from consolidated undrained shear tests. Microstructural changes were derived from mercury intrusion porosimetry (MIP) and scanning electron microscope (SEM) tests. Besides, an empirical model is proposed to describe the strain-softening behavior of expansive soil specimens under low confining pressures that can serve as a useful numerical tool.

**Experimental investigations**

**Materials**

An expansive soil collected from Qiqihar, Heilongjiang, China was used for this research. Qiqihar is a seasonally frozen region. Its average temperature ranges from 23.1 °C in summer to −18.6 °C in winter. The expansive soil was excavated from approximately 1 m under the ground surface, and the in-situ natural gravimetric water content ($w_n$) and dry density ($\rho_{dn}$) of the sampled soil was 26.3% and 1540 kg/m$^3$, respectively. Figure 1 shows the grain size distribution of the expansive soil determined from sieve analysis per ASTM D6913 protocol (ASTM 2017a, b) and hydrometer tests per ASTM D7928 protocol (ASTM 2017a, b). The physical index properties of the tested soil are summarized in Table 1.

![Fig. 1 Grain-size distribution of the tested expansive soil](image)

**Table 1 Basic index properties of the tested expansive soil**

| Properties                              | Values |
|-----------------------------------------|--------|
| Specific gravity, $G_s$                 | 2.68   |
| Liquid limit, $w_L$ (%)                 | 43     |
| Plastic limit, $w_P$ (%)                | 22     |
| Plasticity index, $I_p$                 | 21     |
| Optimum moisture content, $w_{opt}$ (%) | 21     |
| Maximum dry density, $\rho_d$ (kg/m$^3$)| 1660   |
| Free swelling ratio (%)                 | 67     |
| pH value                                | 8.2    |

| USCS          | CL    |

XRF tests were conducted to determine the chemical components of the soil. The major chemical components and their contents by mass are SiO$_2$, 60.48%; Al$_2$O$_3$, 18.53%; Fe$_2$O$_3$, 6.63%; K$_2$O, 3.05%; CaO, 4%. XRD tests were conducted to reveal the mineral components of the soil. The major minerals are Quartz, Illite, Albite, and Calcite. The soil has a free swell ratio of 67%. According to GB 50112-2013 (Ministry of Housing and Urban-Rural Development 2013), this soil is classified as moderately expansive soil.
Specimen preparation and application of FT cycles

Collected soil samples were air-dried, pulverized with a rubber mallet, and then passed through a 2-mm sieve. To simulate the field condition, the air-dried soil was wetted with water such that the natural water content of 26.3% is achieved. The moist soil was statically compacted into cylindrical specimens (38 mm in diameter and 76 mm in height) at the natural dry density of 1540 kg/m³. Compacted specimens were sealed in layers of Saran wrap and stored in plastic containers for at least 72 h at room temperature of 25 ± 1 °C to ensure that water was evenly distributed within the specimens.

Specimens were subjected to freeze–thaw cycles in a closed-system condition because there is limited moisture migration during freezing and thawing processes due to the low coefficient permeability of expansive soils, especially under unsaturated and frozen conditions (Yarbasi et al. 2007; Jean Marie and Samson 2000). A temperature chamber with a precision of ± 0.05 °C was used to apply FT cycles. During FT cycles, all specimens remain wrapped in Saran sheets and stored in plastic containers which were first frozen at −20 °C for 12 h and then allowed to thaw at 20 °C for 12 h. The temperature range was chosen following the local annual temperature variation (i.e. −18.6–23.1 °C). A 12-h period is considered adequate for achieving thermal equilibrium in the small-size specimens used in this study (Lu et al. 2019; Ding et al. 2020). Such freezing and thawing procedure was repeated until the designated number of FT cycles (i.e. \( N_{FT} = 0, 1, 4, 6, 10 \)) was reached. The number of FT cycles was selected because the variation in mechanical properties of expansive soils is most significant during the first 1–2 cycles and commonly level off after 4–6 FT cycles. Such behaviors were demonstrated by previous research (Zeng et al. 2018; Lu et al. 2019).

Measurement of volumetric change

To study the volumetric behavior of expansive soil specimens during FT cycles, the volume measurements were performed on specimens after each freezing and thawing process. An electronic Vernier caliper with an accuracy of 0.005 mm was used to directly measure the diameter and height of the specimens. Preliminary studies have shown that the volumetric strain (\( \varepsilon_v \)) during FT cycles was found to be uniform in the homogenous soil specimens which is consistent with the assumption of the uniform distribution of water phase in soil mass and is widely used by various researchers (Zeng et al. 2018; Lu et al. 2019). Thus, diameter (i.e. \( d_1, d_2, \) and \( d_3 \)) and height (i.e. \( h_1, h_2, \) and \( h_3 \)) measurements were taken at three different cross-sections that are evenly distributed on the surface of the specimens. The average values of diameter and height measurements were used to calculate the global volume of the specimens and, Therefore, determine the global void ratio (\( e \)) or global volumetric strain (\( \varepsilon_v \)) versus the number of cycles (\( N_{FT} \)) relationships.

MIP and SEM tests

To track the evolution of the microstructure of specimens upon freeze–thaw cycles, the mercury intrusion porosimetry (MIP) and scanning electron microscopy (SEM) tests were performed on untreated specimens and specimens subjected to FT cycles. MIP tests determine quantitatively the distribution and size of soil’s pores while SEM tests capture the cross-section morphology of the microstructure. Approximately 2 g of mass trimmed from specimens were used for MIP and SEM tests. They were freeze-dried in liquid nitrogen before MIP tests using a PoreMaster 33 porosimeter and SEM tests using an FEI Quanta 200 SEM testing system.

Determination of mechanical properties

A GDS static triaxial testing system shown in Fig. 2 (GDS Instruments Ltd., Hampshire, U.K.) was used for
performing consolidated undrained (CU) triaxial compression and unconfined compression (UC) tests to determine the mechanical properties of expansive soil subjected to FT cycles. UC tests were performed on saturated and unsaturated specimens while CU tests were only performed on saturated specimens. Unsaturated specimens refer to specimens that were directly sheared after FT cycles while saturated specimens refer to specimens that were vacuum saturated under constant volume conditions for a period of 24 h and then sheared.

In CU tests, saturated specimens were transferred to the triaxial chamber where they were further subjected to back-pressure saturated until the pore-water pressure parameter, $B$ exceeds 0.90 (Kamruzzaman et al. 2009). Afterward, the specimens were consolidated under seven levels of confining pressures ($\sigma_v$) of 10, 20, 30, 50, 100, 200, 300 kPa. The consolidation process was assumed to have been completed when the specimens' volumetric changes leveled off and their excess pore water pressure has dissipated. Finally, the specimens were sheared under undrained conditions at an axial strain rate of 0.066%/min until the axial strain reached 20%. During consolidation and shearing, the variation of the excess pore-water pressure was monitored to determine the evolution of soils’ microstructure

The UC tests were performed following the method suggested by Han and Vanapalli (2017). This method involves an unloading–reloading loop at 1% axial strain ($\varepsilon_a$) to determine the axial stress at $\varepsilon_a = 1\%$ before the unloading process (i.e. $S_{u1%}$) and the reloading modulus at $\varepsilon_a = 1\%$ (i.e. $E_{1%}$). The unconfined compression strength ($q_u$) was determined after the specimens were loaded to failure. A loading and unloading rate of 1 mm/min was adopted following the ASTM D2166-13 (2013) protocol. The $S_{u1%}$ and $E_{1%}$ are elastic properties and $q_u$ indicates the unconfined compressive strength of the specimens (Lee et al. 1997; Lu and Kaya 2013). Detailed discussions of the use of $E_{1%}$, $S_{u1%}$, and $q_u$ and testing procedures of the revised UC tests are available in Han and Vanapalli (2017). Figure 3 shows the stress–strain relationship during the revised UC tests and the determination of the $E_{1%}$, $S_{u1%}$, and $q_u$.

**Results and discussions**

**Volumetric characteristics**

The volumetric strain ($\varepsilon_v$) of specimens during FT cycles is defined by Eq. (1) is follows:

$$\varepsilon_v = \left( V_N - V_0 \right) / V_0 \times 100\%,$$

where $V_0$ is the initial volume of the untreated specimen, $V_N$ is the volume of specimens after experiencing $N$ FT cycles. Positive $\varepsilon_v$ indicates swelling and negative $\varepsilon_v$ means shrinkage.

The evolution of $\varepsilon_v$ during FT cycles is shown in Fig. 4. The volume change of specimens with $N_{FT}$ shows that there is (i) an increase in the $\varepsilon_v$ (i.e. expansion) during freezing and (ii) a decrease in the $\varepsilon_v$ (i.e. shrinkage) during thawing. Variation in the $\varepsilon_v$ during the initial several FT cycles is significant but its scale reduces with further $N_{FT}$ increases. The volumetric deformation becomes elastic after about 4 FT cycles, meaning that the non-recoverable $\varepsilon_v$ becomes almost negligible. This is consistent with observations of similar studies (Viklander 1998; Zeng et al. 2018). FT cycles result in a slight expansion of $\varepsilon_v$ ($\varepsilon_v = 1.2\%$) in the specimens’ volume after 10 FT cycles. However, Kamei et al. (2012) reported reductions in the global volume of a soft clay modified with recycled Bassanite in the process of freeze–thaw and attributed it to the high absorption rate as the freezing–ice melted during thawing.

**Evolution of soils’ microstructure**

The MIP test results of specimens subjected to different FT cycles ($N_{FT} = 0, 1, 4, 6, 10$) are plotted in Fig. 5. Related SEM images are shown in Fig. 6. MIP results are presented in the forms of (i) cumulative intrusion (CI) curves (Fig. 5a) and (ii) pore size distribution (PSD) curves (Fig. 5b). CI curves are the relationship between the intruded mercury void ratio, $e_{MIP}$ ($e_{MIP} = V_m/V_s$ where $V_m$ is the volume of intruded mercury and $V_s$ is the volume of the solids), and the pore diameter, $d$. The PSD curves are the derivatives of CI curves.

Figure 5b shows that untreated specimens ($N_{FT} = 0$) present a bimodal PSD curve with two dominant peaks: the
peak of micro-pores at about 1.3 μm and the peak of macro-pores at about 30 μm. The 5 μm is the delimiting boundary that separates the macro-pores and micro-pores in PSD curves of the tested soil. This is because 5 μm is the turning point of PSD and CI curves and marks the beginning of the main population of micro-pores. This agrees with the criterion proposed by Burton et al. (2015). These observations are consistent with previous studies that the as-compact ed soil usually shows bimodal PSD curves (Monroy et al. 2010; Burton et al. 2015). Macropores with diameters of 30 μm also can be observed in SEM images (see Fig. 6).

For a better description of the microstructure’s evolution during FT histories, the CI and PSD curves were separated into three zones (i.e. zones A, B, and C) by two boundaries which are defined at 0.1 μm and 5 μm (see Fig. 5). 0.1 μm is chosen because all PSD curves converge at the left of this threshold suggesting that these pores with diameters smaller than 0.1 μm are less affected during FT cycles.

In order to quantitatively evaluate the variation of microstructures, the void ratio in zones A, B, and C (denoted as \( \varepsilon_{\text{MIP,A}} \), \( \varepsilon_{\text{MIP,B}} \), \( \varepsilon_{\text{MIP,C}} \)) are calculated by

\[
\varepsilon_{\text{MIP,A}} = \varepsilon_{\text{MIP,0.01}} - \varepsilon_{\text{MIP,0.1}},
\]

\[
\varepsilon_{\text{MIP,B}} = \varepsilon_{\text{MIP,0.1}} - \varepsilon_{\text{MIP,5}},
\]

\[
\varepsilon_{\text{MIP,C}} = \varepsilon_{\text{MIP,5}}
\]

where \( \varepsilon_{\text{MIP,N}} \) is the \( \varepsilon_{\text{MIP}} \) value at \( N \)μm. The \( \varepsilon_{\text{MIP,0.01}} \) is the total void ratio intruded by mercury in the MIP tests (\( \varepsilon_{\text{MIP,0.01}} = \varepsilon_{\text{MIP,A}} + \varepsilon_{\text{MIP,B}} + \varepsilon_{\text{MIP,C}} \)). Values of \( \varepsilon_{\text{MIP,A}}, \varepsilon_{\text{MIP,B}}, \varepsilon_{\text{MIP,C}}, \) and \( \varepsilon_{\text{MIP,0.01}} \) are summarized in Table 2. The following observations can be made from Figs. 5 and 6 and Table 2 as follows:

(i) In Zone A, the FT cycles have a negligible effect on the microstructure as the shape of CI and PSD curves
remain unchanged and $e_{\text{MIP,A}}$ is approximately the same after different FT cycles.

(ii) In Zone B, $e_{\text{MIP,B}}$ decreases, which suggests that micro-pores shrank during FT cycles. This is associated with the irreversible strain that takes place during the growth of ice crystals and thawing. The peak of the micro-pores in the PSD curve shifts to the right, indicating a slight increase in the diameter of the dominant micro-pores. The frequency (i.e. $-\delta e_{\text{MIP}}/\delta \log d$) at the peak of the PSD curve, also decreases after FT cycles.

(iii) In Zone C, the $e_{\text{MIP,C}}$ generally increases after 10 FT cycles although there is a slight decrease in the $e_{\text{MIP,C}}$ during the first FT cycle. The peak of compaction-induced macro-pores in the PSD curve of the initial untreated specimen disappears after a few FT cycles. This behavior also suggests that the compaction-induced macro-pores are unstable and prone to collapse upon FT processes. Meanwhile, a new plateau in the range of 5–20 μm develops which remains stable during FT cycles.

(iv) The evolution of microstructures during FT cycles observed in MIP results is consistent with the SEM observations. As shown in Fig. 6, the variation of pore size and the development of cracks induced by FT cycles can be easily observed. Cracks induced by FT cycles segregate the soil particles and ultimately lead to a fragmented soil structure. FT cycles induce cracks in the soil structure even the soil is chemically stabilized prior to FT cycles. Jamshidi et al. (2016) reported that fine cracks were formed in cement-treated soils during FT processes.

### Unconfined compressive strength

Figure 7 shows the stress–strain curves (i.e. axial stress, $\sigma_1$ versus axial strain, $\varepsilon_1$) obtained from UC tests on saturated and unsaturated specimens with different FT cycles ($N_{\text{FT}} = 0, 1, 4, 6, 10$). It is noted that specimens generally failed at $\varepsilon_1 = 7\%$. The $\sigma_1$–$\varepsilon_1$ relationships show strain-softerning characteristics. With increasing FT cycles, the pre-peak stress–strain curves tend to become flat, and axial strain at the peak strength (denoted as $\varepsilon_p$) increases from 2% of the untreated specimen to 3% after 10 FT cycles, for both saturated and unsaturated specimens. This phenomenon is
consistent with the observations of previous studies (Ghazavi and Roustaie 2010; Jafari and Esna-Ashari 2012; Kamei et al. 2012) that the axial strain at the peak strength of reinforced soil specimens also increases with increasing FT cycles during unconfined compression tests. Such behavior is associated with the cracks generated during FT cycles that provide additional space for the deformation of the soil skeleton during shearing.

The pre-peak stress–strain curves tend to become flat. The stress–strain curves of specimens with different $N_{FT}$ seem to have an identical residual strength of approximately 15 and 30 kPa for saturated and unsaturated specimens, respectively. Comparing Fig. 7a and b, it can be observed that the saturated specimens have lower residual strength and much lower peak strength than the unsaturated specimens, despite the consistent trends in stress–strain curves.

The evolution of $q_u$, $E_{1\%}$, and $S_{u1\%}$ values with $N_{FT}$ obtained from UC tests is shown in Fig. 8. A power function (Eq. 2) is used to describe the variation of $q_u$, $E_{1\%}$, and $S_{u1\%}$ with $N_{FT}$.

$$\frac{\Omega_N}{\Omega_0} = \frac{1 + \alpha e^{-\beta N}}{1 + \alpha},$$

where $\Omega_0$ collectively represents the $q_u$, $E_{1\%}$, and $S_{u1\%}$ of untreated specimens, $\Omega_N$ is the $q_u$, $E_{1\%}$, and $S_{u1\%}$ after $N$ cycles of treatment, $\alpha$ and $\beta$ are model parameters, $e = 2.7182$. The fitting curves are shown in Fig. 8 and the values of fitting parameters (i.e. $\alpha$ and $\beta$) are summarized in Table 3. The variation in the $q_u$, $E_{1\%}$, and $S_{u1\%}$ with $N_{FT}$ is well described by the power function with $R^2 > 0.9$. The following observation can be derived from Fig. 8 and Table 3:

(i) The $q_u$, $E_{1\%}$, and $S_{u1\%}$ decrease with increasing $N_{FT}$, especially during the initial several FT cycles. This is followed by a slower decrease with further FT cycles and reach equilibrium for unsaturated specimens at
approximately \(N_{FT} = 6\) and saturated specimens at \(N_{FT} = 4\), respectively.

(ii) The values of \(q_u\), \(E_{1\%}\), and \(S_{u1\%}\) obtained from saturated specimens are much lower than the ones from unsaturated specimens at the same \(N_{FT}\). For example, the \(q_u\), \(E_{1\%}\), and \(S_{u1\%}\) of saturated specimens decrease by 49.1%, 54.0%, and 71.4% from the initial values of 55 kPa, 8.45 MPa, and 49 kPa for untreated specimens to 28 kPa, 3.89 MPa, and 14 kPa after 10 FT cycles, respectively. For unsaturated specimens, the \(q_u\), \(E_{1\%}\), and \(S_{u1\%}\) decrease only by 46.0%, 45.2%, and 56.7% from 113 kPa, 15.65 MPa and 97 kPa to 61 kPa, 8.57 MPa, and 42 kPa, respectively. Thus, the decrease in the mechanical properties is more significant during FT cycles when the moisture content is higher.

### Shear strength and elastic modulus during consolidated undrained shearing

CU tests were conducted on specimens under seven confining stress levels (\(\sigma_c = 10, 20, 30, 50, 100, 200, 300\) kPa) and with different \(N_{FT}\) (\(N_{FT} = 0, 1, 4, 6, 10\)) for comprehensive evaluation of the influence of FT cycles on the mechanical properties of expansive soil.

The stress–strain relationships (i.e. deviator stress \(q\) versus axial strain \(\varepsilon_1\); \(q = \sigma_1 - \sigma_3\)) obtained from consolidated undrained triaxial shear tests under various confining pressures of the specimens subjected to FT cycles shown in Fig. 9. The untreated specimens and the specimens subjected to FT cycles exhibit strain-softening behavior (see Fig. 9) under low confining pressures (i.e. 10, 20, and 30 kPa). However, the strain-softening characteristics become less apparent with increasing confining pressure. The stress–strain curves start to show a strain-stabilization behavior when the confining pressure exceeds 50 kPa and show a strain-hardening behavior at a confining pressure of 100 and 300 kPa (see Fig. 9). The shear strength (i.e. \(q_u\)) is equal to the peak deviator stress for the specimens exhibiting strain-softening characteristics. The shear strength is estimated at the axial strain of 15% for the specimens exhibiting strain stabilization or hardening characteristics (Zhang et al. 2015; Güneyli and Rüşen 2015). Typical results of the failure strength due to FT cycles under low and high confining pressures are given in Fig. 10.

The \(q_u\) of all specimens decreases with increasing \(N_{FT}\) and reaches equilibrium after approximately six cycles regardless of the confining pressure. A more significant reduction, however, was observed under low confining stress (i.e. \(\sigma_c = 10, 20,\) and 30 kPa). The elastic modulus (i.e. \(E_u\)) is an important parameter that reflects the ability of a soil to resist deformation (Lee et al. 1995; Han and Vanapalli 2017). The elastic modulus can be determined using Eq. (3) from stress–strain curves as follows:

\[
E_u = \frac{q_{1\%} - q_0}{\varepsilon_{1\%} - \varepsilon_0},
\]

### Table 3

| Table 3 | Values of parameters in Eq. (2) for unconfined compression tests |
|---------|---------------------------------------------------------------|
| Parameter types | As-compacted specimens | Saturated specimens |
| \(\alpha\) | 0.815 | 0.482 |
| \(\beta\) | 0.893 | 0.721 |
| \(q_u\) | 1.185 | 1.400 |
| \(E_{1\%}\) | 1.788 | 0.450 |
| \(S_{u1\%}\) | 1.551 | 2.456 |
| \(\varepsilon_{1\%}\) | 0.642 | 0.660 |

Fig. 9  Stress–strain relationships of specimens after 0, 1, 4, 6, and 10 FT cycles from CU tests

\(\alpha\) Springer
where $q_{1\%}$ is the deviator stress corresponding to an axial strain of 1% (i.e. $\varepsilon_{1\%}$) and $q_0$ is the initial deviator stress corresponding to initial axial strain $\varepsilon_0$. The results of $E_u$ for specimens subjected to FT cycles under various low and high confining pressures are given in Fig. 11. A significant reduction in the $E_u$ with an increase in the $N_{FT}$ can be observed. Such behavior is due to the modified microstructures introduced by freeze–thaw cycles, which greatly reduce the ability of specimens to resist elastic deformation. These observations are consistent with the variations of $q_{1\%}$ that almost no appreciable decrease in the $E_u$ was observed after $N_{FT} = 6$.

The effective and apparent cohesion ($c'$ and $c$, kPa) and the effective and apparent cohesion internal friction angle ($\phi'$ and $\phi$) under each confining pressure was separately determined from the Mohr–Coulomb model (Eqs. 4a, 4b) to evaluate their evolutions with $N_{FT}$.

\begin{align*}
\sigma_1 &= \frac{2c \cos \phi}{1 - \sin \phi} + 1 + \sin \phi \sigma_3, \quad (4a) \\
\sigma_1' &= \frac{2c' \cos \phi'}{1 - \sin \phi'} + 1 + \sin \phi' \sigma_3'. \quad (4b)
\end{align*}

The evolution of determined $c'$ and $c$, $\phi'$ and $\phi$ with different $N_{FT}$ are summarized in Figs. 12 and 13, respectively.

Because of the particle rearrangement and compaction effects during the consolidation and shearing processes,
the effective and apparent cohesion (i.e. \(c'\) and \(c\)) and internal friction angle (i.e. \(\phi'\) and \(\phi\)) exhibit a non-linear characteristic as the confining pressure increases. A lower cohesion and higher internal friction angle were observed for lower confining pressures, these results are consistent with the previous studies (Chen and Liu 1990; Jiang et al. 2003). An obvious reduction was also observed in the shear strength parameters \(c'\) and \(\phi'\) with the increasing \(N_{FT}\). Such a behavior can be attributed to the influence of FT cycles that introduce cracks to soil structure (see Fig. 6), which reduce the integrity of soil particles and result in a reduction of their effective contacts. Ishikawa and Miura (2011) also reported similar observations on coarse-grained soils that the degree of particle breakage increases with increasing FT cycles, which results in a decrease in the soil’s cohesion and friction angle.

The power function (Eq. 2) was also used to describe the variation of \(q_u, E_u, c'\) and \(\phi'\) and \(c\) and \(\phi\) with different \(N_{FT}\) under consolidated undrained shearing. The fitting curves are shown in Figs. 10, 11, 12, and 13, and the fitting parameters (i.e. \(\alpha\) and \(\beta\)) are given in Table 4.

Modelling stress–strain behaviors

The strain stabilization and hardening behavior of specimens under high confining pressures (i.e. \(\sigma_c=50, 100, 200, 300\) kPa) can be described by the Duncan-Chang model (i.e. Equation (5)) and shown in Fig. 9.

\[
q = \frac{\varepsilon_1}{a + b\varepsilon_1},
\]

where \(a\) and \(b\) are model parameters. \(1/a\) is the slope of the initial curve and equals the initial undrained elastic modulus, \(E_u\), \(1/b\) is the ultimate deviator stress and indicates the undrained shear strength, \(q_u\). Fitting parameters and \(R^2\) are summarized in Table 5.

Extending the concepts of the Duncan-Chang model, an empirical model Eq. (6) was proposed to describe the strain-softening behavior of specimens subjected to different FT cycles which are observed from CU tests under low \(\sigma_c\) and UC tests.

\[
q = \frac{\varepsilon_1}{a + b\varepsilon_1 - \lambda\ln\left(\frac{2.718 + (10^{10\varepsilon_1}/\kappa)^{-n}}{m}\right)^m},
\]

where \(\lambda, \kappa, n, m\), are model parameters. \(\lambda\) is related to the ratio of residual strength to peak strength, \(\kappa, n, m\), are the model parameters related to the axial strain at peak deviator stress, the slope of descending portion of the stress–strain curves, and the axial strain at residual deviator stress.

The fitting curves are shown in Figs. 7 and 9 and fitting parameters are summarized in Table 6. There are good agreements between the measurements and the predictions. The residual axial strain and corresponding residual deviator stress and the subsequent horizontal section of stress–strain curves were well described.

Conclusions

In this paper, the influence of FT cycles on the microstructure and mechanical behaviors of an expansive soil was investigated from experimental studies. The following observations and conclusions were derived:

(i) The compaction-induced macro-structures are altered in the range of 5–250 \(\mu\) due to the influence of FT cycles. Besides, the micro-pores with a radius less
than 5 μ transfer to macro-pores during FT cycles which contribute to the formation of cracks in soil structure. The volumetric behaviors become elastic after approximately four cycles during FT treatments.

(ii) The $q_u$, $E_1\%$, and $S_{u1\%}$ obtained from unconfined compression tests decrease significantly with increasing $N_{FT}$, especially during the initial two FT cycles, and reach equilibrium after four and six FT cycles for saturated and unsaturated tests, respectively.

(iii) The values of $q_u$, $E_1\%$, and $S_{u1\%}$ for saturated specimens are lower than the ones of unsaturated specimens at the same $N_{FT}$. Reduction in the $q_u$, $E_1\%$, and $S_{u1\%}$ at $N_{FT} = 10$ are more significant for saturated specimens (up to 71.4%) than unsaturated specimens (up to 56.7%).

(iv) The $q_u$, $E_{u}$, $c$, and $\phi$ obtained from consolidated undrained triaxial tests reduce due to the influence of FT cycles; however, these are notably constant after 6 FT cycles.

A simple empirical model is developed to describe the strain-softening behavior of stress–strain curves obtained from CU and UC shearing under low-stress levels of specimens undergoing 0, 1, 4, 6, and 10 FT cycles. The performance of this model is verified by the test results. The model presented in this paper could be a useful tool in the numerical analysis of the strain-softening behaviors that can be derived from the stress–strain curves.

### Table 4
Values of parameters in Eq. (2) for consolidated undrained tests

| Parameter types | $\sigma_c$ (kPa) | $a$ | $\beta$ | $R^2$ | $\sigma_c$ (kPa) | $a$ | $\beta$ | $R^2$ | $\sigma_c$ (kPa) | $a$ | $\beta$ | $R^2$ |
|----------------|-----------------|-----|---------|-------|-----------------|-----|---------|-------|-----------------|-----|---------|-------|
| $q_u$          | 10              | 0.282 | 0.150  | 0.968 | 10              | 0.083 | 1.575   | 0.712 | 10              | 0.161 | 0.082  | 0.961 |
|               | 20              | 0.278 | 0.260  | 0.968 | 20              | 0.079 | 1.338   | 0.858 | 20              | 0.080 | 0.186  | 0.975 |
|               | 30              | 0.435 | 0.187  | 0.915 | 30              | 0.309 | 0.200   | 0.964 | 30              | 0.204 | 0.147  | 0.938 |
|               | 50              | 0.391 | 0.466  | 0.983 | 50              | 0.457 | 0.225   | 0.982 | 50              | 0.269 | 0.176  | 0.987 |
|               | 100             | 0.418 | 0.429  | 0.995 | 100             | 0.562 | 0.221   | 0.970 | 100             | 0.416 | 0.182  | 0.957 |
|               | 200             | 0.329 | 0.574  | 0.967 | 200             | 0.798 | 0.205   | 0.999 | 200             | 0.536 | 0.319  | 0.998 |
|               | 300             | 0.248 | 0.465  | 0.924 | 300             | 0.699 | 0.274   | 0.962 | 300             | 0.620 | 0.303  | 0.994 |
| $E_u$          | 10              | 0.301 | 0.327  | 0.980 | 10              | 0.421 | 0.110   | 0.979 | 10              | 0.329 | 0.332  | 0.988 |
|               | 20              | 0.408 | 0.269  | 0.985 | 20              | 0.425 | 0.116   | 0.967 | 20              | 0.318 | 0.313  | 0.993 |
|               | 30              | 0.565 | 0.304  | 0.998 | 30              | 0.333 | 0.131   | 0.828 | 30              | 0.283 | 0.323  | 0.978 |
|               | 50              | 1.152 | 0.377  | 0.998 | 50              | 0.436 | 0.072   | 0.984 | 50              | 0.254 | 0.310  | 0.987 |
|               | 100             | 0.776 | 0.399  | 0.999 | 100             | 0.173 | 0.200   | 0.975 | 100             | 0.184 | 0.232  | 0.957 |
|               | 200             | 0.672 | 0.237  | 0.997 | 200             | 0.134 | 0.158   | 0.973 | 200             | 0.130 | 0.163  | 0.948 |
|               | 300             | 0.655 | 0.348  | 0.957 | 300             | 0.174 | 0.079   | 0.933 | 300             | 0.067 | 0.290  | 0.989 |

### Table 5
Values of parameters in Eq. (5) for describing the strain-stabilization and hardening behavior of specimens in CU tests

| Number of cycles | Confining pressures (kPa) | $a \times 10^{-5}$ | $b$ | $R^2$ |
|------------------|---------------------------|-------------------|-----|-------|
| $N_{FT} = 0$     | 50                        | 4.301             | 0.007 | 0.997 |
|                  | 100                       | 2.299             | 0.005 | 0.999 |
|                  | 200                       | 2.687             | 0.003 | 0.999 |
|                  | 300                       | 1.634             | 0.002 | 0.999 |
| $N_{FT} = 1$     | 50                        | 5.555             | 0.008 | 0.998 |
|                  | 100                       | 3.740             | 0.005 | 0.999 |
|                  | 200                       | 2.759             | 0.004 | 0.999 |
|                  | 300                       | 2.118             | 0.003 | 0.999 |
| $N_{FT} = 4$     | 50                        | 10.068            | 0.009 | 0.998 |
|                  | 100                       | 4.713             | 0.006 | 0.999 |
|                  | 200                       | 3.521             | 0.004 | 0.999 |
|                  | 300                       | 2.723             | 0.003 | 0.999 |
| $N_{FT} = 6$     | 50                        | 12.112            | 0.009 | 0.997 |
|                  | 100                       | 5.045             | 0.006 | 0.999 |
|                  | 200                       | 4.356             | 0.004 | 0.999 |
|                  | 300                       | 3.142             | 0.003 | 0.999 |
| $N_{FT} = 10$    | 50                        | 13.973            | 0.009 | 0.997 |
|                  | 100                       | 5.722             | 0.007 | 0.997 |
|                  | 200                       | 4.896             | 0.004 | 0.999 |
|                  | 300                       | 3.556             | 0.003 | 0.998 |
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Declarations

Conflict of interest 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.

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Table 6 Values of parameters in Eq. (6) for describing the strain-softening behavior of specimens in CU tests

| Number of cycles | Confining pressures (kPa) | a ($10^{-4}$) | b | λ | κ | n | m | R² |
|------------------|---------------------------|--------------|---|---|---|---|---|----|
| N<sub>FT</sub> = 0 | 10 | 2.292 | 0.013 | 0.848 | 14.329 | −0.2680 | −0.243 | 0.998 |
|                  | 20 | −4.428 | −0.039 | −3.402 | 20.940 | −2.475 | −0.120 | 0.999 |
|                  | 30 | −65.481 | −0.008 | −0.897 | 27.968 | −2.343 | −0.084 | 0.999 |
| N<sub>FT</sub> = 1 | 10 | −15.600 | −0.106 | −6.462 | 16.804 | −5.135 | −0.155 | 0.997 |
|                  | 20 | −7.052 | −0.053 | −4.221 | 26.131 | −2.511 | −0.159 | 0.999 |
|                  | 30 | −1.991 | −0.024 | −2.282 | 25.552 | −4.147 | −0.086 | 0.999 |
| N<sub>FT</sub> = 4 | 10 | −2.535 | −0.016 | −0.909 | 21.310 | −5.456 | −0.168 | 0.999 |
|                  | 20 | −3.525 | −0.064 | −2.189 | 26.189 | −3.321 | −0.202 | 0.998 |
|                  | 30 | −7.779 | −0.048 | −4.378 | 67.053 | −2.880 | −0.102 | 0.999 |
| N<sub>FT</sub> = 6 | 10 | −12.2 | −0.069 | −3.668 | 25.852 | −5.760 | −0.170 | 0.998 |
|                  | 20 | −4.485 | −0.052 | −4.406 | 28.075 | −3.072 | −0.216 | 0.998 |
|                  | 30 | 7.669 | 0.041 | 3.425 | 33.599 | −2.016 | −0.170 | 0.999 |
| N<sub>FT</sub> = 10 | 10 | −9.439 | −0.045 | −2.468 | 16.164 | −4.247 | −0.218 | 0.997 |
|                  | 20 | −6.987 | −0.043 | −2.953 | 26.852 | −3.641 | −0.155 | 0.999 |
|                  | 30 | 1.746 | 0.011 | 0.898 | 26.508 | −5.847 | −0.080 | 0.999 |

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