Effect of temperature on the mechanical properties of fine-grained soils - A review

Md Azhar, Somenath Mondal, Anh Minh A.M. Tang, Akhileshwar Singh

Effect of Temperature on the Mechanical Properties of Fine-Grained Soils - A Review

Md Azhar[1], Somenath Mondal[1]*, Anh Minh Tang[2] and Akhileshwar K. Singh[1]

[1]Department of Civil Engineering, National Institute of Technology Jamshedpur 831014, India.

[2]Laboratory Navier, École des Ponts Paris Tech, 77455 Marne-la-Vallée, France

Md Azhar: aazhar8603@gmail.com, Research Scholar

Somenath Mondal*: smondal.ce@nitjsr.ac.in, Assistant Professor

Anh Minh Tang: anh-minh.tang@enpc.fr, Research Director

Akhileshwar K. Singh: aksingh.ce@nitjsr.ac.in, Associate Professor

***Corresponding author**

Somenath Mondal

Assistant Professor, Department of Civil Engineering, National Institute of Technology Jamshedpur 831014, India.

Email: smondal.ce@nitjsr.ac.in

Running title:

Effect of Temperature on the Mechanical Properties of Fine-Grained Soil- A Review.

ABSTRACT: Contemporary geotechnical engineering practice encompasses design and construction of thermo-active geo-structures or structures which would encounter thermal cycles throughout its service life. Therefore, understanding the effect of temperature on soil mechanical properties becomes inevitable. Being a complex and coupled phenomenon it has been quite challenging for the researchers to study experimentally and theoretically the thermo-mechanical behaviour of fine-grained soil. Nonetheless, there are numerous studies which contribute significantly to gain insight about thermo-mechanical behaviour of soils. Incidentally, till date, on this arena of research, no comprehensive review, which provides a holistic development of the subject and its connection with the field application, is available. The prime objective of the present study is to critically appraise the state-of-the-art understanding on thermo-mechanical behaviour of fine-grained soils through a comprehensive literature review. Moreover, this paper also describes the concept of thermo-elasto-plastic strain and thermal creep behaviour of fine-grained soils.

Keywords: Fine-grained soils, Temperature-induced volumetric deformation, Mechanisms, Thermo-elasto-plastic behaviour.

1. Introduction

Understanding the thermo-mechanical behaviour of fine-grained soils has been indispensable in modern-day geotechnical engineering practices. This behaviour significantly influences the overall long-term behaviour of the fine-grained soil upon application of simultaneous temperature cycles and mechanical loading. In this context, there are numerous investigations conducted to identify the thermal response of diverse geo-materials. Among these, a major portion contribute to understand the effect of temperature on volumetric change (Campanella and Mitchell, 1968; Virdi and Keedwell, 1988) and strength parameters of soil (Cekerevac and Laloui, 2004; Yavari et al., 2016; Maghsoodi et al., 2020, 2021). In this regard, most of the investigations have been carried out for the projects related to the radioactive waste disposal facilities (Virdi and Keedwell, 1988; Rao and Singh, 1999; Krishnaiah and Singh, 2004a, 2004b; Naidu and Singh, 2004; Sreedeep et al., 2005; Ojovan et al., 2019), underground crude oil storage tanks (Padmakumar, 2013), oil-carrying pipelines (Brandon and Mitchell, 1989; Manthena and Singh, 2001; Abuel-Naga et al., 2008; Lee et al., 2010), air-conditioning ducts (Kadali et al., 2013), buried conduits and high voltage electrical cables (Abuel-Naga et al., 2008; De Moel et al., 2010; Vega and McCartney, 2014), geothermal energy piles (Di et al., 2000; Bozis et al., 2011), elevated temperature landfills (McCartney et al., 2019), furnaces, boiler units, forging units, brick kilns and rocket-launching pads (Kadali et al., 2013). These studies generally analysed the volumetric change of soil when temperature varies in the range between 0 and 100 °C.

This study mainly focuses on the thermo-mechanical behaviour of fine-grained soil, in which variation of temperature lies between 0 and 100 °C. Therefore, the phase change of pore water due to variation of temperature has not been induced. For this reason, several other phenomena of temperature-soil interaction such as soil water freezing/thawing or

evaporation, soil mineral alteration due to fire, phase change of soil due to volcanic eruption, *etc.* are not considered.

The thermo-mechanical behaviour of fine-grained soil has been analysed against application of varying temperature by many researchers in saturated soil (Campanella and Mitchell, 1968; Demars and Charles, 1982; Hueckel and Baldi, 1990; Burghignoli et al., 2000; Sultan et al., 2002; Cekerevac and Laloui, 2004; Abuel-Naga et al., 2008; Coccia and McCartney, 2012; Vega and McCartney, 2014) and unsaturated soil (Romero et al., 2005; Tang et al., 2008; Uchaipichat and Khalili, 2009; Alsherif and McCartney, 2015; Coccia and McCartney, 2016a, 2016b). Saturated soil will tend to change its volume in response to the thermal solicitation when it is heated. Hence this phenomenon is termed as thermal volumetric change. This process is defined as thermal consolidation if dissipation of pore water takes place. During undrained condition, the different coefficient of thermal expansions of soil pore water and solid particles is the reason behind the generation of thermally-induced excess pore water pressure of saturated specimen. On other hand, thermally-induced collapse (*i.e.* rearrangement of soil particles under thermo-mechanical load) of the soil skeleton and changes in viscosity of the pore water are the impact of temperature on the soils that causes thermally-induced excess pore water pressure (Campanella and Mitchell, 1968; Schiffmann, 1971). Further, Bai et al., (2021) discussed that the increase in temperature decrease the effective stress between soil particles and enhance the pore water pressure, which lead to particle rearrangement of soil. The effect of this particle rearrangement certainly enhance the growth in volumetric strain. Based on these mechanisms, it can be opined that thermal consolidation is similar to the primary consolidation of soils (Campanella and Mitchell, 1968; Shetty et al., 2019). In particular, the creep behaviour of the soil due to the mechanical loading or unloading prior to the thermal heating can affect the sign (contraction or expansion) and the magnitude of the change in volume of soils during heating in drained

condition (Coccia and McCartney, 2016a). If the secondary compression rate is positive (indicating decreasing volume) before heating, thermal volumetric shrinkage is observed for normal consolidated soils, whereas if the secondary compression rate is negative (indicating increasing volume) prior to heating, thermal volumetric expansion is observed over consolidated soil (Burghignoli et al., 2000; Coccia & McCartney, 2016a; Towhata et al., 1993). Fig. 1 shows the schematic diagram of secondary compression for NC and OC soil. In addition, on the basis of experimental investigations conducted by the earlier researchers, it can be stated that the thermal volumetric response of fine-grained soil may be indirectly regulated by the over-consolidation ratio (Abuel-Naga, Bergado, & Bouazza, 2007; Burghignoli et al., 2000; Campanella & Mitchell, 1968; Hoseinimighani & Szendefy, 2022; Vega & McCartney, 2014; Zhu & Qi, 2018).

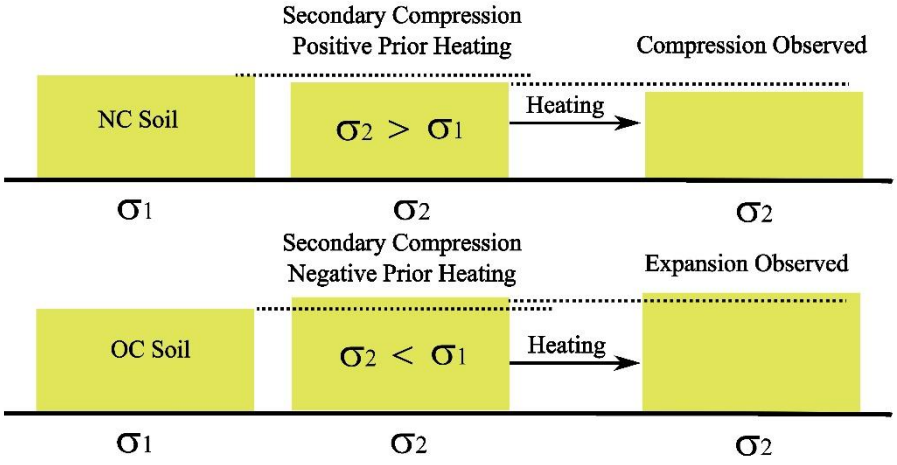


Fig. 1. Schematic diagram for secondary compression for NC and OC soil.

The application of thermal energy in soil can influence its properties and behaviour. Numerous researchers have recognized the effect of thermal energy on properties of fine-grained soil such as shear strength, permeability, pore water pressure, consolidation, creep index, mineralogical composition, *etc.* This effect is mainly attributed to volumetric change of soil against thermo-mechanical loading. In this context, there are a number of recent review articles available which have discussed the contemplating issues (Casarella et al.,

2021; Hoseinimighani & Szendefy, 2022; Loria & Coulibaly, 2021; Wang et al., 2020) on this topic. For example, Loria & Coulibaly, (2021) have discussed the thermally-induced volumetric change of geomaterials and its deriving mechanism. Moreover, Wang et al., (2020) have reported the influence on shear strength parameters of fine grained soil against the application of thermal energy. Furthermore, Casarella et al., (2021) have studied the effect of temperature on inter-particle forces of fine grained soil and its impact on macroscopic behaviour. Hoseinimighani & Szendefy, (2022) have discussed the proposed mechanism and special attention has been given to microstructural alteration, physio-chemical interaction and possible change at macro scale. Although there is extensive knowledge about the effect of temperature on the behaviour of fine-grained soil, there is still a dearth of review which collectively discusses and relates the microscopic, macroscopic, and theoretical approaches to address this topic. Therefore, this review paper on effect of temperature on fine-grained soil focuses on three main following aspects:

- First, the inter-particle forces are summarized for thermal volumetric change of fine-grained soil at microscopic level. In which diffused double layer (DDL) theory and particle orientation (e.g.- face to face, edge to face arrangement) are illustrated for a better understanding of volumetric change at microscopic level.
- Second, temperature-induced volumetric change and its deriving mechanism are reviewed separately for saturated and unsaturated soils at the macroscopic level. Subsequently, thermal volumetric change due to thermal cycle is reported at the macroscopic level. Moreover, effect of temperature on geotechnical properties of fine-grained soil is also discussed briefly.
- Third, additional approaches for modelling of temperature-induced volumetric change of fine-grained soil are presented.

The scope of this paper is to concentrate on the thermo-mechanical behaviour of fine-grained soil, where there is novelty and uncertainty due to recent development in thermal interaction with soil. The overview of this paper is presented in pictorial form in Fig. 2.

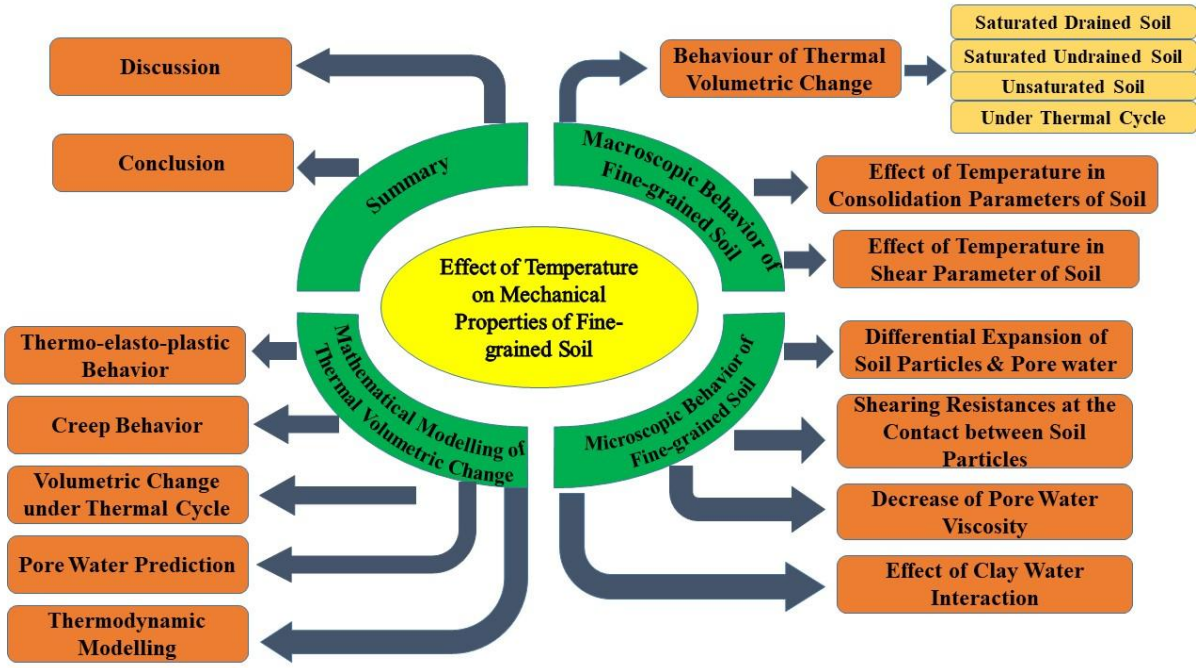


Fig. 2. Overview of thermally-induced mechanical behaviour of fine-grained soil.

2. Effect of Temperature on Microscopic Behaviour of Fine-Grained Soil

The microstructural constituents of soil are made up of alumino-silicate oxides in presence of water and variable amount of iron and alkali metals. Mineral layers, pore water and pore air are deposited in several form in soil. A Schematic diagram of soil structure is shown in fig. 3. Water molecules are held by the clay particles through hydrogen bonds due to clay's high affinity to water and cations. Therefore, the behaviour of fined grained particles is affected by the individual particle and water content in pores. Clay surface has negative charge, which adsorbs the positive charge from pore water. By this way, cations are distributed over the negatively charged surface, whether density is more near the surface and decreased with increase in distance. The cations created a positive charge layer and a double layer is formed with negative charge. These layers are termed as diffused double layer, which are electrostatically bound to each other. These electrostatic forces control the engineering

behaviour of fine grained soil at a large extent. Therefore, thermo-mechanical loading directly affects the formation of composition and volumetric change of soil by the conversion and dissipation of energy by phase transformation between liquid phase (i.e.- free water) to solid phase (i.e.- bound water) (Bai, Zhou, et al., 2021; Bai & Shi, 2017).

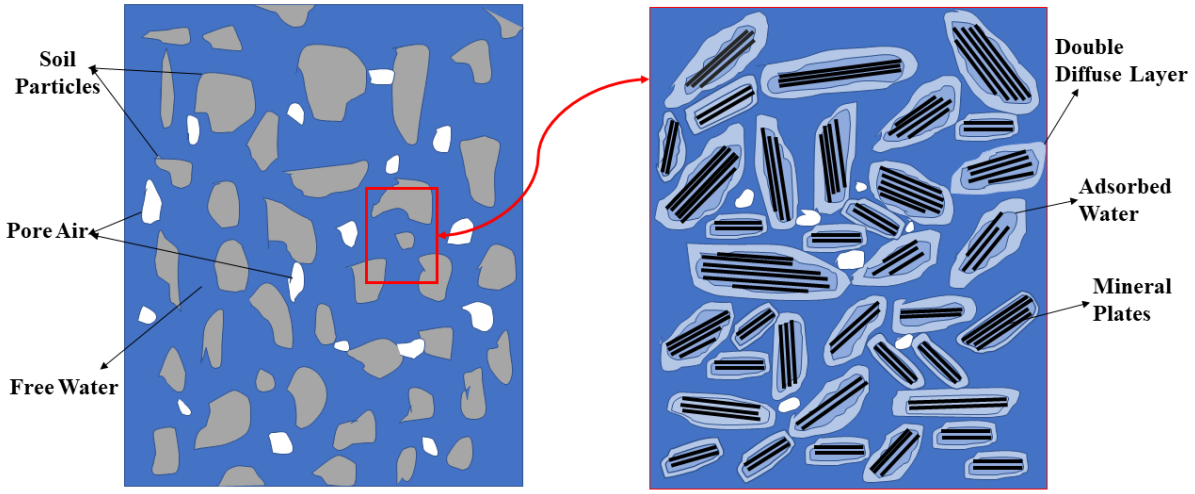


Fig. 3. Schematic diagram of mineralogical structure of soil.

The fundamental mechanisms behind the thermally-induced mechanical behaviour of saturated soils have been perceived in four ways (Paaswell, 1967; Campanella and Mitchell, 1968; Delage et al., 2000). Each type is described in the following sections.

2.1. Thermal expansion of soil particle and pore water

During heating, the discrete components such as soil skeleton and pore fluid of the soil material expand. The thermal expansion coefficient of water equals to 207×10^{-6} $^{o}C^{-1}$ at 20 °C and increases to 695×10^{-6} $^{o}C^{-1}$ at 90 °C (McKinstry, 1965). Those of some soil minerals are clay, quartz, calcite and felspar whose thermal coefficient of expansion lie between 1 and 3×10^{-5} $^{o}C^{-1}$ (Mohajerani et al., 2012). Depending on the mineralogy of the soil, water can expand 7 to 12 times more than solid particle.

2.2. Shear resistance at the contact between soil particles

The second mechanism, proposed by (Campanella & Mitchell, 1968; Tarantino et al., 2021), introduced the effect of temperature on the shear resistance of the contact between soil particles. The shear resistance of fine-grained soil is separated into two distinct types- micro-resistance and macro-resistance. Micro-resistance is developed due to inter-particle contacts of fine-grained soil. The adsorbed water surrounding the soil particles may alter the formation of inter-particle contacts. During consolidation process, water between soil particles can dissipate outside due to thermo-mechanical stress. Therefore, heating increases the number of contacts between particles and thus increases the micro-resistance of fine-grained soil. The increase in contacts between soil particles are described that the pore water pressure of soil mass is weaken due to increase in temperature, which causes dissipation of pore water due to decrease in viscosity of pore water. The formation of micro-resistance is observed for over consolidated (OC) and normal consolidated (NC) soils with increase in temperature. On the other hand, macro-resistance is a type of shear strength which is developed due to movement of soil particles over other in shear plane. Volumetric change has been observed in drained condition under thermo-mechanical loading, which generates the shear stress due to macro resistance between particles. Macro-resistance of fine-grained soil would either increase or decrease with heating. The macro-resistance increases if irreversible volumetric contraction occurs due to heating, while it decreases if reversible volumetric expansion occurs. On the other hand, in undrained condition volumetric change is not permitted. Thus, pore water pressure is developed within soil mass under thermo-mechanical loading in undrained state. In NC soil, macro-resistance causes positive pore pressure in heating. For OC soil, it causes negative pore pressure.

2.3. Pore water viscosity

It is a well-known fact that viscosity of pore water decreases with increase in temperature (Paaswell, 1967). The energy of molecules of any substance is defined as the

function of temperature as explained by kinetic energy theory. As temperature increases kinetic energy of soil particles also increases. Therefore, the net attraction force between molecules decreases, which manifests the decrease in viscosity of soil pore water (Kuntiwattanakul et al., 1995; Towhata et al., 1993). A lower viscosity of pore water provides a higher hydraulic conductivity of soil mass. Variation in viscosity for free water under varying temperature was obtained from experimental results by (Hillel, 2013), which is written as follows (Eq. 1):

$$\mu(T) = -0.00046575 \ln(T) + 0.00239138. \quad (1)$$

Where, $\mu(T)$ is viscosity of water at temperature *'$T(^oC)$'*.

2.4. Electrostatic force between soil and pore water

Towhata et al., (1993) revealed that a negative charge on the surface of soil solids would create an electric field with a positive dipole charge of pore water molecule. This electric field binds neighbouring water molecules to soil grain due to strong electrostatic forces of attraction, and therefore restricts the motion of pore water molecules. It was further described that some kinetic energy is associated with each pore water molecule, which is electrostatically bound to the soil grain surface. When heating is applied, these bound water molecules may reach the energy threshold and are therefore dissipated from the soil mass. Temperature therefore appears to affect the clay-water interaction. This clay water interaction results water to be present in clay in three different forms (Hueckel, 1992; Stçepkowska, 1990): (1) Free or bulk water, which flows under hydraulic gradient; (2) Adsorbed water which forms due to enveloping action between molecules of soil (Hueckel, 1992; Iwata et al., 2020; Kemper et al., 1964), it cannot flow under gravitational effect; and (3 Hydroxyl or structural water, which is mainly a part of structural lattice (shape of mineralogical arrangement) of minerals and cannot leave the lattice below 350 ^oC. Clay water interaction can be classified in six different types of molecular interactions viz. water-water, water-

cation, water-clay, cation-cation, cation-clay and clay-clay (Casarella et al., 2021; Hueckel, 1992; Iwata et al., 2020; Skipper et al., 1991). Apart from these interaction, physiochemical internal forces is also developed at the soil particle contacts due to external thermo-mechanical loading (attraction and repulsion). Hydrate layer at particle surface has been developed due to equilibirium in physiochemical interaction. Change in thermo-mechanical loading, disturb the equilibirium balance, which change the thickness of hydrate layer. This change generates the additional pressure to keep the specimen in equilibirium, which is called as disjoining pressure. This disjoining pressure is mainly depend upon the rearrangement of soil particle resulting from physiochemical force and hydrate layer (Golchin et al., 2022a). Therefore, these potential interactions result in generation of attractive London forces or dispersion forces, short-range repulsion, and Coulomb force between charged particles, which can affect the volumetric change in soil lattice (Casarella et al., 2021). Change in temperature influences the intensity of inter-particle forces in soil lattice, inducing rearrangement of soil particles. As a result, particle surface and edge accumulate positive and negative charge correspondingly. Consequently, two different orientations (face-to-face and edge-to-face) are developed between soil lattices (Casarella et al., 2021; Yong, 1999). In face-to-face arrangement, negatively charged layer overlaps to adjacent parallel particle in such a way that a repulsive force generates in between. As a result, inter-particle distance gets adjusted by both external compressive force and internal repulsive force. Repulsive forces are directly proportional to temperature and inversely with distance between particles (Hoseinimighani & Szendefy, 2022). This is mainly due to development of the DLVO (Derjaguin, Landau, Vervey, and Overbeek) theory. The DLVO stated that the adsorption of mineral on suspended particle reduced the repulsive force between soil particles and suspended particle due to decrease in zeta potential (Bai, Nie, et al., 2021). Moreover, due to increase in temperature the inter-particle distance between particles of face-to-face

arrangement also increases for applied external compressive load which causes an expansion of clay. This expansion would be reversible in nature due to the reverse alteration of dielectric permittivity to its initial value upon cooling or removal of thermal load (Casarella et al., 2021; Ghahremannejad, 2003; Pedrotti & Tarantino, 2018). On the other hand, in case of edge-to-face arrangement, a contact configuration is developed by the Van der Waal's force of attraction between oppositely charged particles at edge and face respectively. However, due to increase in temperature, the net attractive force gets weakened. Additionally, the edge-to-face interaction between particles develop non reversible deformation if the contact is broken (Casarella et al., 2021).

The effect of temperature on diffuse double layer (DDL) remains inconclusive. DDL insures that soil particles have positive or negative charge on surface due to availability of charged group within, or attraction of charged layers from surrounding medium. Therefore, an electrical double layer is established between solid particles and pore water to confirm electro-neutrality within soil mass. Mitchell & Soga (2005) analysed the Gouy-Chapman equation and stated that DDL is unaffected by temperature variation. On the contrary, Morin & Silva, (1984) gathered the experimental results on high-porosity clay and found that the increase in temperature prone to compress the DDL and decrease the ionic concentration, which tends to breakdown of adsorbed water. Furthermore, below certain separation of mineral layers (less than 50 Å) and at low porosity, DDL theory fails to describe the behaviour of interaction forces (Baldi et al., 1988; Hueckel, 1992). This is mainly caused due to domination of solvation forces near the surface. This is due to the structural arrangement of pore water, which creates the potentials of short-range intermolecular pairs different from the potentials derived from any continuum theory. Solvation force is governed by the geometric constrain of structural packing imposed by solid particle surface, which is mainly caused due to the geometric constraints induced by the very thin separation between mineral layers. In

such system, repulsive hydration forces gets generated and the bond stress is usually higher than that estimated by DDL theory (Baldi et al., 1988; Hueckel, 1992; Ma & Hueckel, 1993).

3. Effect of Temperature on Macroscopic Behaviour of Fine-Grained Soil

3.1. Thermally-induced volume change behaviour of saturated soil under drained conditions

There are substantial number of studies available which elucidate the volumetric change of saturated soils due to temperature change under drained conditions (Abuel-Naga et al., 2007c; Baldi et al., 1988; Delage et al., 2000; Hueckel & Baldi, 1990; Sultan et al., 2002; Towhata et al., 1993). Baldi et al., (1988) noticed the effect of stress history on the expansion and contraction of soil specimen when heated beyond the room temperature. Fig. 4 presents volumetric strain versus temperature during heating from room temperature (close to 20 °C) at different magnitudes of over consolidation ratio (OCR) value. Data collected on various soils such as Potinda clay, Pasquasia clay, MC clay, Boom clay and Kaolin clay from different studies are presented in the same plot for comparison. The results in Fig. 4 reveal that Kaolin clay for high magnitude of OCR (i.e., 6, 12) expands up to a certain temperature, beyond which it contracts. Similar pattern can be identified for MC clay. Hence, this unique temperature is termed as transition temperature (Baldi et al., 1988). The transition temperature increased with increase in over consolidation ratio (Hoseinimighani & Szendefy, 2022). Moreover, thermal contraction is detected for lightly over consolidated sample from the beginning of heating such as Pontida clay for OCR = 1 and 1.4, Pasquasia clay for OCR = 1.4 and 2.5, and Boom clay for OCR = 2. However, only few soil specimens such as Pontida clay at OCR = 2.5, Pasquasia clay at OCR = 3.4 and MC clay at OCR = 3.4 show expansion on initial heating and contraction on further heating. Few discrepancies have also been observed in literature. Laloui & Cekerevac, (2003) have reported that transition temperature is not affected by over consolidated ratio, all OCR specimens are transiting almost at constant

temperature from expansion to contraction. Bai et al., (2019) have also reported the effect of OCR due to heating. It was reported that the soil showed contraction for OCR 1 and 4 and expansion for OCR 8. It has been reported from fig 4 that plastic volumetric strain is observed for OCR value less than 2. Moreover, elastic volumetric strain has been observed for OCR value greater than 2. As the OCR value increases the elastic volumetric strain also increases due to heating.

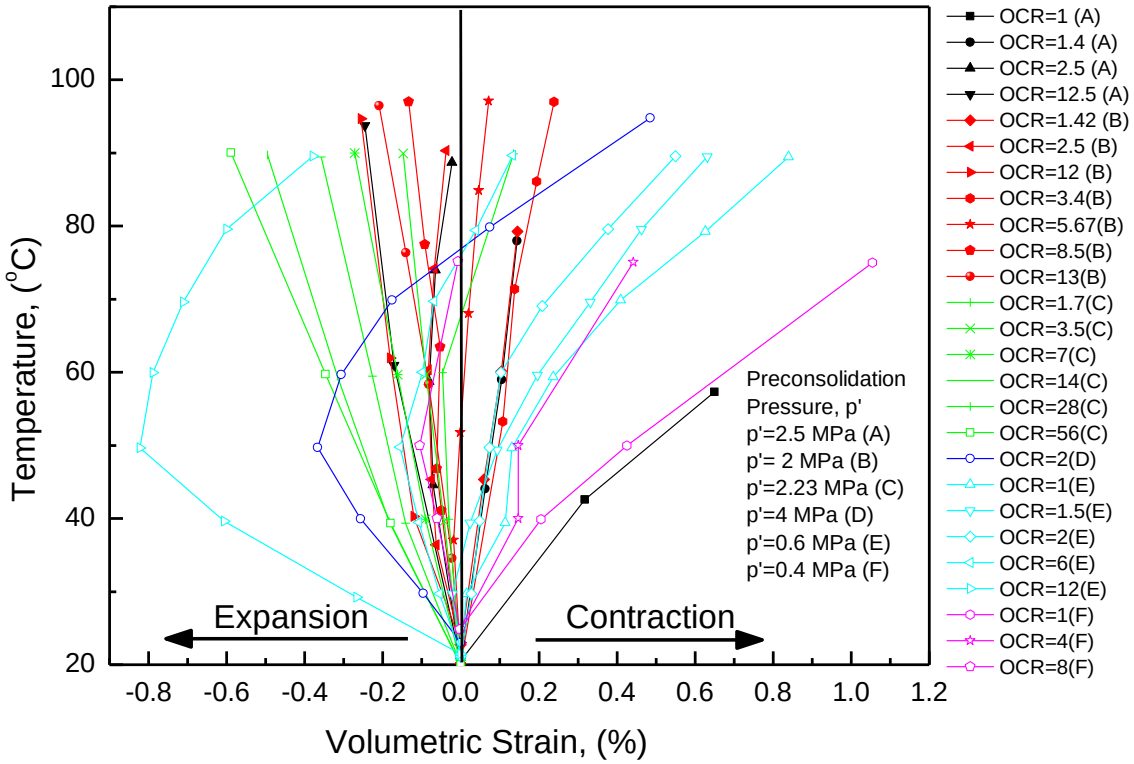


Fig. 4. Volumetric behaviour of saturated soils during heating: (A) Potinda clay (Baldi et al., 1988), (B) Potinda and Pasquasia clay (T. Hueckel & Baldi, 1990), (C) MC Clay (Towhata et al., 1993), (D) Boom Clay (Delage et al., 2000), (E) Kaolin clay (Cekerevac & Laloui, 2004) and (F) Saturated Clay (Bai et al., 2019b).

Subsequently, few studies have also performed heating under drained conditions with a heating/cooling path (Abuel-Naga, Bergado, Bouazza, et al., 2007; Sultan et al., 2002). Fig. 5 depicts the thermal volumetric change against a thermal cycle in drained state. It is opined that the thermal irreversible volumetric change of normally-consolidated specimens is independent of applied stress. Thermal volumetric change for over-consolidated soil is observed as reversible in nature. Higher the OCR value, large portion of reversible

volumetric strain is observed. Moreover, several researchers identified that soil specimen undergoes from expansion during heating to contraction during the subsequent cooling (Towhata et al., 1993; Sultan et al., 2002, Abuel-Naga et al., 2008; Coccia and McCartney, 2012).

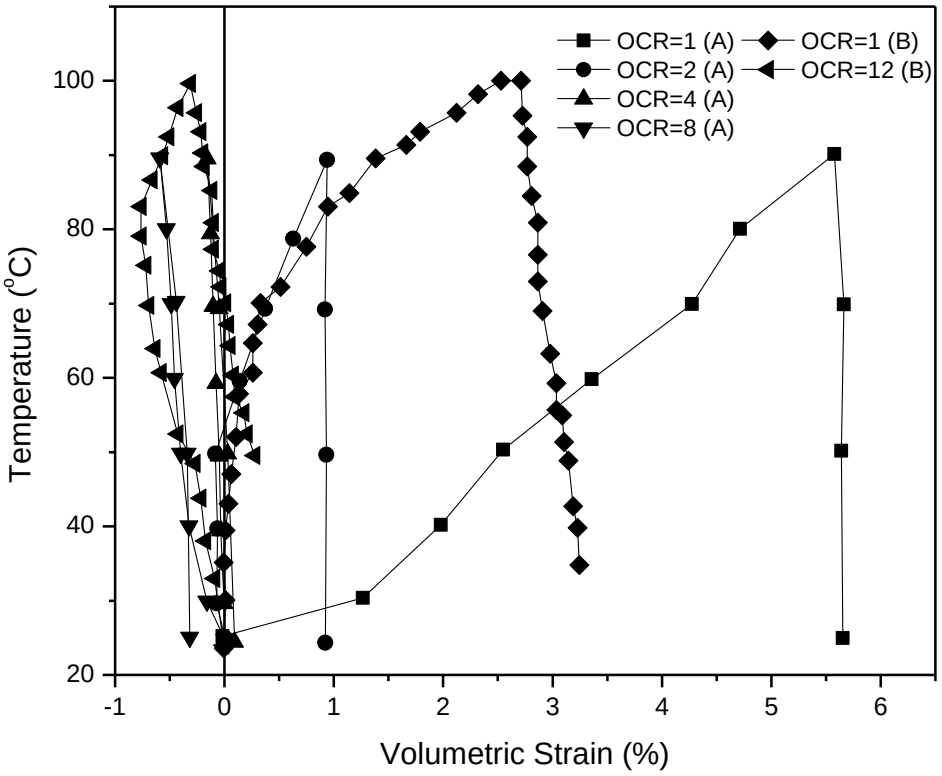


Fig. 5. Thermal volumetric change of saturated soil during thermal cycle; (A) Bangkok clay (Abuel-Naga, Bergado, Bouazza, et al., 2007), (B) Boom clay (Sultan et al., 2002).

The macroscopic thermally-induced volumetric change of fine-grained soil can be interpreted using knowledge on microscopic behaviour. In saturated soil mass, water particles and soil particles are bound by electrostatic force, which restrict the motion of pore water from soil. During heating, every particle gains some kinetic energy, which weaken this electrostatic bond between soil particles and pore water. With this in view, the viscosity of pore water is reduced and dissipation of water takes place due to thermo-mechanical loading. A major observation from the results is that a NC soil undergoes almost irreversible contraction during a heating/cooling, and reversible expansion has been observed for highly OC soil. Actually, under NC condition, a compressive rearrangement of soil particles take

place until a shear bond between soils particles are developed to withstand with stress at higher temperature. As at NC, the shear stress at the particles contacts is already close to the shear resistance, heating decreases the shear resistance at the particles contacts and induce irreversible collapses. Under OC condition, thermal heating cannot produce rearrangement of soil particles because it is more stable and exposed to lower mobilized force. Therefore, OC soil undergo into reversible thermo-elastic expansion up to certain temperature. However, heating beyond a certain temperature for OC soil weakens the shear resistance between soil particles, which lead to irreversible contraction of soil specimen.

3.2. Thermo-mechanical behaviour of saturated soils under undrained conditions

Numerous studies have been carried out against thermo-mechanical behaviour of fine-grained soil under undrained conditions (Abuel-Naga, Bergado, & Bouazza, 2007; Ghaaowd et al., 2015; T. Hueckel & Pellegrini, 1992; Mohajerani et al., 2012; Uchaipichat & Khalili, 2009). Thermally-induced pore water pressure increases with increase in temperature and this increase is less significant at a higher OCR value (Abuel-Naga, Bergado, & Bouazza, 2007; T. Hueckel & Pellegrini, 1992). (Abuel-Naga, Bergado, & Bouazza, 2007; Ghaaowd et al., 2015; Mohajerani et al., 2012; Uchaipichat & Khalili, 2009) have performed heating tests under undrained conditions on consolidated specimens at different OCR values. Fig. 6 illustrates the thermally-induced excess pore water pressure as a function of temperature. Abuel-Naga et al., (2007) have carried out undrained heating/cooling test on Bangkok clay and observed that thermally-induced pore water pressure for NC soil shows reversible trend, while OC soil shows irreversible trend with negative pore water pressure. Furthermore, Uchaipichat & Khalili, (2009) have conducted the undrained heating of saturated specimen with different effective stress on silty soil. Pore-water pressure of soil increases with increase in temperature as well as effective stresses. Moreover, Ghaaowd et al., (2015) have analyzed the pore water pressure for kaoline clay for OCR value equal to 1 and observed that pore

water pressure increases with increase in temperature. Subsequently, effective stress and shear resistance of adsorbed water decrease related to heating. Therefore, reversible thermally-induced pore water pressure can be observed for NC soil (Abuel-Naga, Bergado, & Bouazza, 2007). On the other hand, for OC soil Pusch et al., (1991) illustrated that thermally-induced pore water pressure in specimen would cause unstable stress within soil mass due to increase in kinetic energy of particles, which resulting rearrangement of soil particles to improve uniformity. Therefore, it can be opined that for OC soil thermally-induced pore water pressure becomes irreversible in nature due to rearrangement of particles within soil mass.

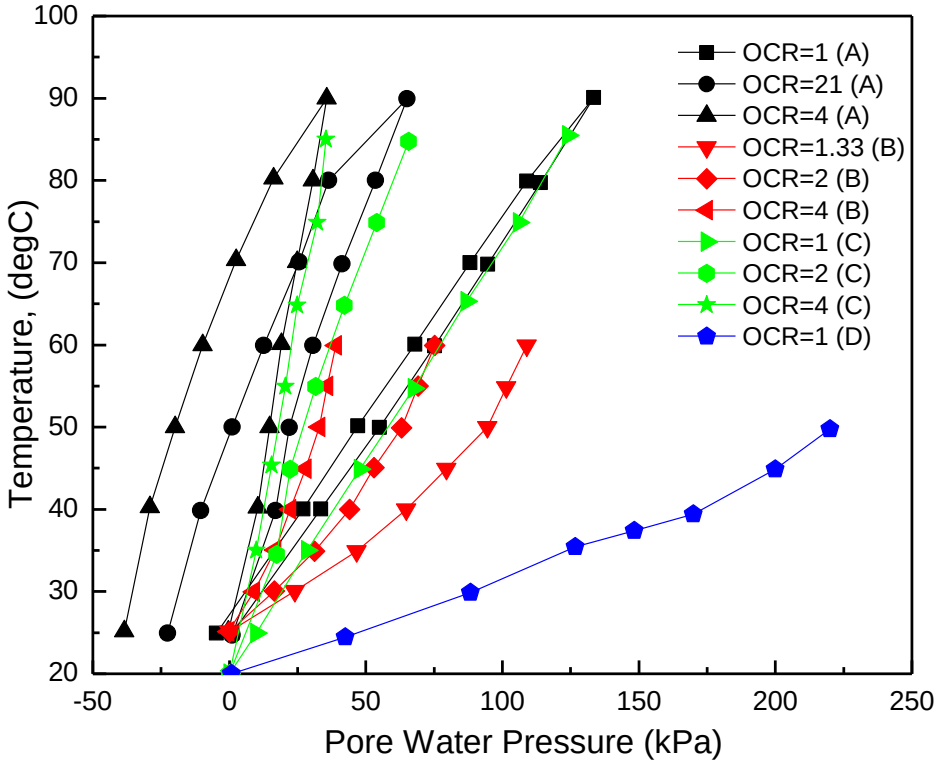


Fig. 6. Thermal effect on pore water pressure of soil; (A) Bangkok Clay (Abuel-Naga, Bergado, & Bouazza, 2007), (B) Silt (Uchaipichat & Khalili, 2009), (C) Bangkok Clay (Ghaaowd et al., 2015), (D) Kaoline Clay (Ghaaowd et al., 2015).

3.3. Volume change behaviour of unsaturated soil

Several researchers investigated the volumetric behaviour of unsaturated fine-grained soil under thermo-mechanical loadings (Romero et al., 2005; Salager et al., 2008; Tang et al., 2008; Uchaipichat and Khalili, 2009; Alsherif and McCartney, 2015; Coccia and McCartney,

375 2016a, 2016b;). It can be perceived from the literature that unsaturated soil results are also
376 similar to that of saturated soil (Salager et al., 2008; Uchaipichat and Khalili, 2009). Fig. 7
377 illustrates the result of volumetric strain during heating of unsaturated samples at different
378 suctions and net stresses. Fig. 7 states that the thermal dilation of the sample increases with
379 increase in matric suction and decreases with increase in net stresses (Uchaipichat and
380 Khalili, 2009). Tang et al., (2008) performed a thermo-mechanical experimental investigation
381 on compacted bentonite specimen with different suctions, in a temperature-controlled
382 isotropic cell. Subsequently, it was shown that the expansion of the soils was induced due to
383 heating at low stress (i.e., high OCR) and high suction. Conversely, contraction was reported
384 while heating at high stress (i.e., low OCR) and low suction (Tang et al., 2008). Moreover,
385 Uchaipichat and Khalili, (2009) conducted thermo-mechanical tests on different silt samples
386 of varying initial suction (i.e., 100 and 300 kPa) at different confining pressures (i.e., 50 and
387 150 kPa). It can be observed that upon heating, the specimen with lower confining stress
388 experienced expansion and for higher confining stress, contraction was observed
389 (Uchaipichat and Khalili, 2009). This study stated that pre-consolidation stress of sample
390 increases with increase in suction magnitude. Hence, as suction value increases, the OCR
391 value also increases. Therefore, thermal collapse is insignificant for over consolidated sample
392 and increases with decrease in over consolidation ratio (Uchaipichat and Khalili, 2009).
393 Furthermore, Salager et al., (2008) conducted an experimental study to explore the
394 mechanical behaviour at different temperature and suction values of Sion Silt. The results
395 demonstrated that temperature has not a significant effect on the compressibility of soil. On
396 the other hand, compressibility of soil is decreased with increase in suction level. However,
397 the apparent pre-consolidation stress is affected by the applied temperature and suction.
398 Similarly, Coccia and McCartney, (2016a) have performed the thermo-mechanical tests at
399 three different stages: i.e. 1) Matric suction level; 2) Isotropic compression under constant

rate of strain; and 3) drained heating. Through this study, efforts have been made by (Alsherif & McCartney, 2015; Coccia & McCartney, 2016a) to investigate the influence of secondary consolidation on thermal volume change response of bonny silt and the role of stress history and loading pattern on thermal volumetric response of the soils. Decrease in viscosity of pore water in soil mass reduces the interfacial tension between pore water and soil particles due to heating (Romero et al., 2005). This reduction in interfacial tension may also accelerate the rate of secondary creep due to the reduction in the applied load to the soil transfers to the grain-to-grain contacts (Coccia & McCartney, 2016a, 2016b).

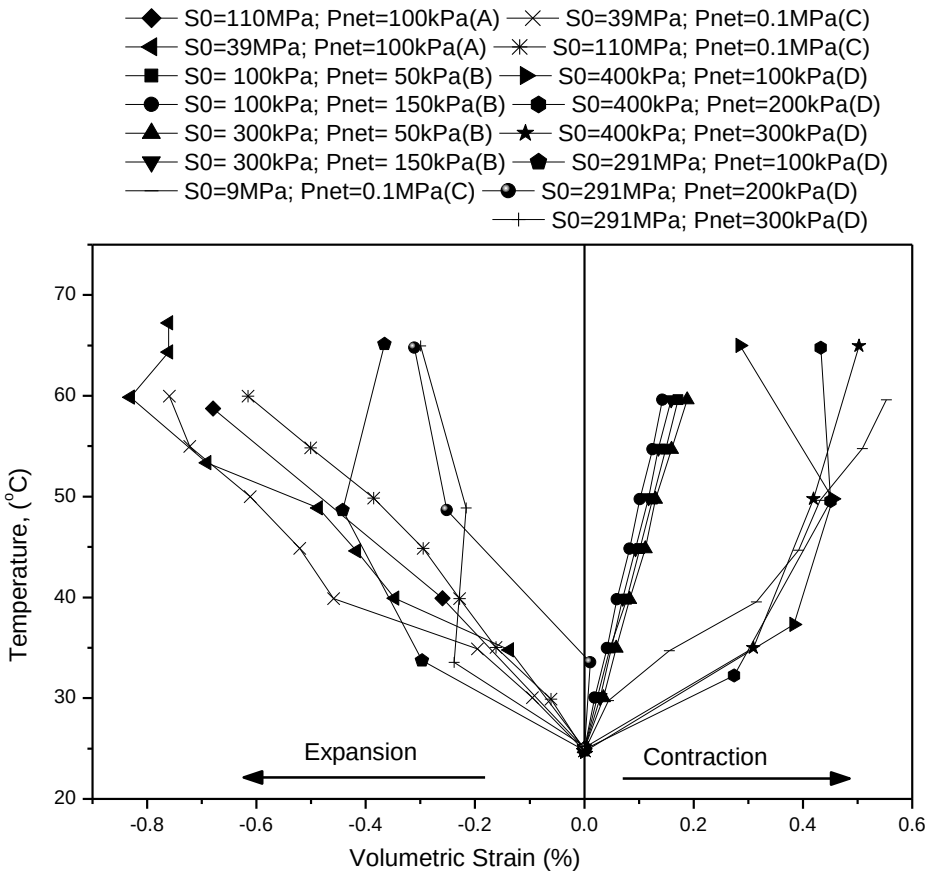


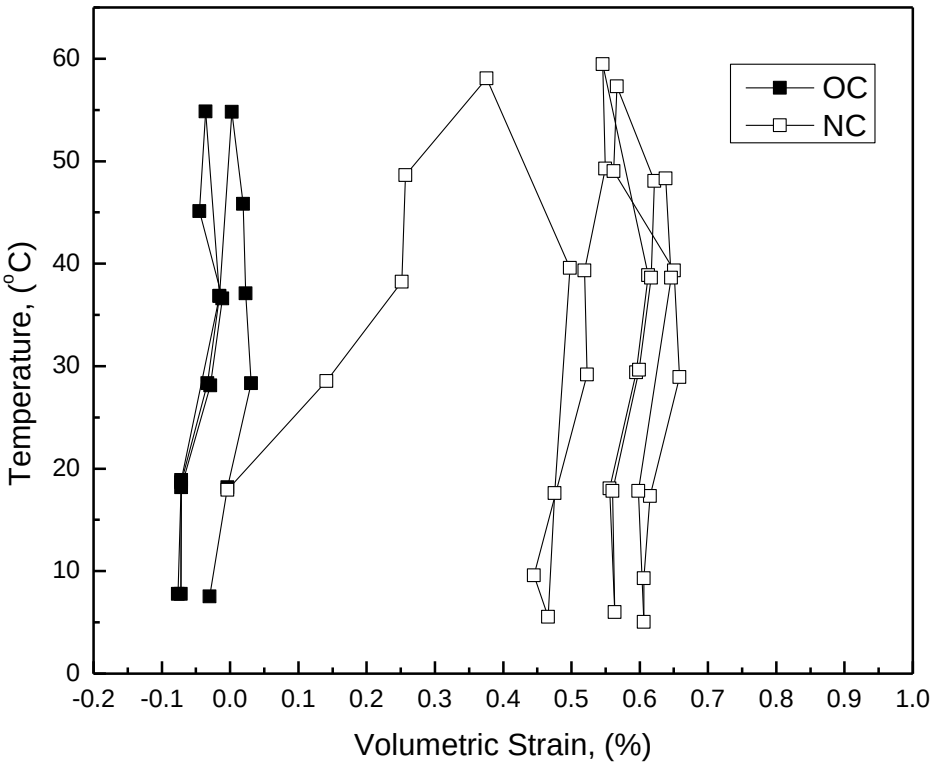
Fig. 7. Volumetric behaviour of unsaturated soils during heating: (A) Bentonite (Tang et al., 2008), (B) Silt (Uchaipichat & Khalili, 2009), (C) Bentonite (Y.-J. Cui et al., 2011) and (D) Silt (Alsherif & McCartney, 2015).

3.4.Volumetric change under thermal cycles

A cyclic thermal loading on a saturated fine-grained soils for NC and OC soil

condition causes thermo-elasto-plastic response and thermo-elastic response, respectively during heating (Di Donna and Laloui, 2015). Several researchers stated that for approximately 10 cycles of heating and cooling, an additional irreversible contraction is generated for NC and slightly OC soil before approaching to reversible behaviour which is expansive for heating and compressive for cooling (Bai & Shi, 2017; Di Donna & Laloui, 2015; Loria & Coulibaly, 2021; Mu et al., 2019; Zhang, 2017). Initially, expansion was observed due to the expansion of solid grains and pore water, then the pore water started draining with dissipation of pore pressure which led to change in volumetric strain (Bai & Shi, 2017). Although, accumulation behaviour of volumetric strain was developed on sequential temperature cycles (refer fig. 8). The magnitude of compression, which was developed on soil under thermal cycles, increased with increase in clay content (Loria & Coulibaly, 2021; Mu et al., 2019). As a result, the friction force at the inter-particle contacts decreases with increasing temperature., the friction force at the inter-particle contacts decreases with increasing temperature, which causes partial collapse of the soil structure. After this collapse, inter-particle equilibrium is again reached, when the number of additional contacts created between the particles is sufficient to transfer the imposed thermal stress. Bond slippage is more likely when the contact position is close to a failure condition, which is the case for NC materials. This is mainly driven by the particle rearrangement, which associated due to expansion of soil grains against thermo-mechanical loading (Di Donna & Laloui, 2015). Therefore, gradual collapse of soil structure which leads to continuous drop in void ratio until further stress bond develops between soil particles at higher temperature. This phenomenon is totally irreversible, hence expressed as thermal secondary compression. After successive temperature cycles, there is no sign of volume change under secondary compression because the specimen has already been recomposed with earlier temperature cycles (Campanella and Mitchell, 1968). Under OC conditions, the rearrangement of particles

439 is more limited, and the thermo-elastic expansion of the components predominates. When
440 considering the cyclic thermal load on the NC soil, the configuration becomes more stable
441 with each cycle depending on the amount of accumulated plastic strain, and the available
442 space for additional failure decreases.



443

444 **Fig. 8.** Volumetric change of soil under thermal cycles (Q. J. Ma et al., 2017).

445 **3.5.Thermal hardening**

446 Apart from that, the effect of temperature cycles on soil compressibility was analysed
447 in various studies. In this regard, Shetty et al., (2019) performed a series of experiments on
448 fine grained soil, which was exposed to sequential thermo-mechanical stress. This signifies
449 that, the specimen first experienced mechanical stress and subsequently temperature cycle
450 was applied. Fig. 9 shows the relation between volumetric strain and effective stress. A
451 change in volumetric strain was noticed when specimen was imposed to temperature cycle.
452 The cumulative temperature-induced volumetric strain has been defined as the sum of
453 volumetric strains for each temperature cycle. Moreover, the change in slope caused by

imposition of temperature cycle designates that the compressibility of specimen along the sequential thermo-mechanical path decreases on exposure to temperature cycle. This would basically escalate the stiffness *(E)* and hardness *(H)* of soil solids due to application of temperature cycle (Shetty et al., 2019). Similarly, (Kadali et al., 2013) have also used nano-indentation technique to investigate the effect of temperature on stiffness *(E)* and hardness *(H)* of several fined grained soils. The change in stiffness and hardness of soil specimen due to temperature cycle has been reported by various research studies (Plum and Esrig, 1969; Cui et al., 2000; Abuel-Naga et al., 2007c). Furthermore, it has been stated that exposure to a temperature cycle, at a particular effective stress, has no impact on the volumetric strain due to temperature experienced by the specimen on subsequent thermal cycle at a higher effective stress. Similarly, Ma et al., (2017) have perceived that decrease in volumetric strain of the specimen is observed when it is exposed to repeated thermal cycles at a specific stress.

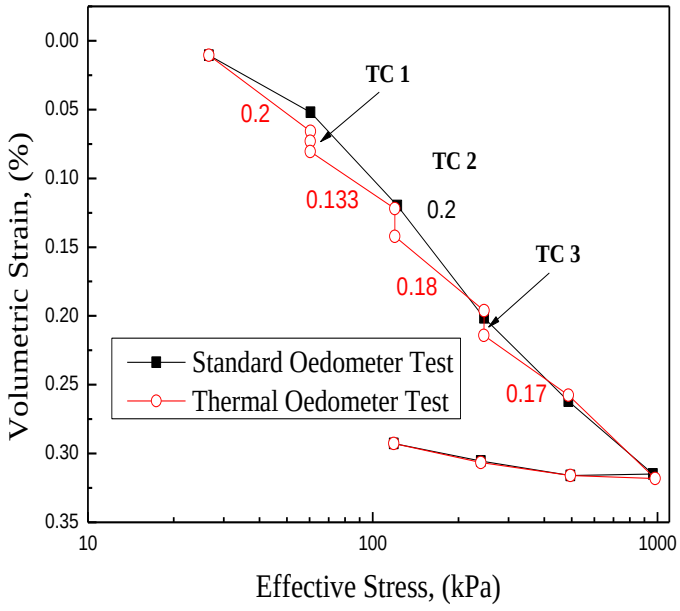


Fig. 9. Effect of TCs on Volumetric Strain with Effective Stress (Shetty et al., 2019)

Additionally, numerous researchers (Salager et al., 2008; Tang et al., 2008; Uchaipichat and Khalili, 2009) examined the effects of suction, effective stress, and OCR on the unsaturated soil at different temperatures. Unreported in the literature as of yet is the

study on unsaturated fine-grained soil under cyclic temperature. Additionally, it has been suggested that fluctuation in pore water pressure is seen at various effective stresses when the saturated specimen is exposed to a temperature cycle (Bai & Shi, 2017). Due to the expansion of the pore water and solid grains during rapid heating, the pore water pressure increases until it reaches its maximum value. Again pore water drainage causes pore water pressure to decrease to zero. Due to water absorption, pore water pressure decreases to a negative value during cooling before returning to zero (Bai & Shi, 2017). Due to the increase in effective stress, the amplitude of pore pressure therefore decreases over successive heat cycles. Additionally, it is said that the saturated specimen experience identical volumetric strain under variable effective stress during consecutive temperature cycles (Shetty et al., 2019). The similar trend has been depicted in Fig. 10(a) (application of TC under varyingσ') and Fig. 10(b) (application of TC under constantσ'). However, in Fig. 10(a), the soil matrix is loaded to varying effective stress, which increases after imposition of each TC. This results in same degree of the structural rearrangement of the sample due to thermal cycle under the varying σ'. Therefore, the volumetric change remains constant under each TC (i.e.- $\Delta V_{T1}, \Delta V_{T2}\ and\ \Delta V_{T3}$), and thus the volumetric strain achieved in the subsequent TC remains unchanged (i.e.- $\Delta\varepsilon_{T1} \approx \Delta\varepsilon_{T2}$) (Q. J. Ma et al., 2017; Shetty et al., 2019). There are several studies that applied a repetitive TC on specimen on constant σ', which reduce the extent of structural rearrangement in each successive TC, hence the magnitude of volumetric change decreases with each consecutive TC (i.e.- $\Delta V_{T1} > \Delta V_{T2} > \Delta V_{T3}$) (Campanella and Mitchell, 1968; Ma et al., 2017). It would result in lower generation of pore pressure due to thermal stress and the same is responsible to decrease in volumetric strain (refer Fig. 10(b)).

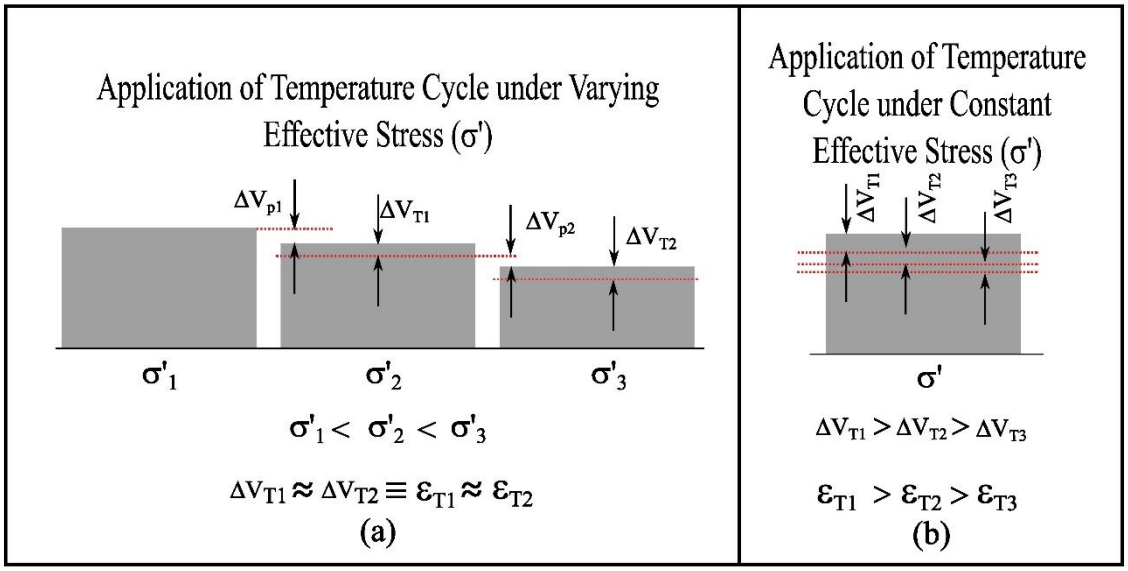


Fig. 10. Effect of temperature cycle on soil compressibility (a) under varying effective stress and (b) under constant effective stress.

3.6.Thermal dependent creep behaviour

Soft clays manifest time-dependent behaviour due to their viscosity. Prolonged deformation of these clays after dispersing excess water pressure from the pores, is called as creep deformation. Creep behaviour of clay is vigorously associated with bond elimination due to application of mechanical stress (Yin et al., 2017; Zhu et al., 2017) and change in temperature (Houston et al., 1985; Fox and Edil, 1996; Laloui et al., 2008; Coccia and McCartney, 2016a; Li et al., 2018; Zhu and Qi, 2018; Jarad et al., 2019). Houston et al. (1985) have shown that the rate of volumetric creep increases at elevated temperature. increase in volumetric creep rate with upraising the temperature has also been established. Likewise, Fox and Edil (1996) have also described the increment of creep coefficient under induced temperature. Based on the experimental results, thermal creep coefficient varies linearly with the temperature (Plum and Esrig, 1969). Furthermore, Jarad et al., (2019) have performed experiments to determine the creep coefficient for two specimen of clay soils. The results have indicated that the creep index increased with heating. Besides, it has been mentioned that the clay of lower dry density has a higher creep index. At a higher

temperature, the decrease in the apparent viscosity of the contacts between the particles and pore water would increase creep. Other reasons can be due to the weak particle bonding or inter particle shear strength at high temperatures, and therefore the strain increases with increasing temperature (Jarad et al., 2019). Subsequently, the variation in creep rate of soil due to heating is evaluated due to change in bond between particles or due to irreversible internal deformation in orientation of soil (Li et al., 2018).

3.7. Coefficient of consolidation

In some studies, the consequences of temperature change on the coefficient of consolidation *(C_v)* were illustrated and it was stated that this coefficient depends on temperature and vertical effective stress (Delage et al., 2000; Favero et al., 2016; Jarad et al., 2019; Ogawa et al., 2020). After synthesis of the obtained results, the coefficient of consolidation is higher at a higher temperature (Fig. 11). Moreover, Terzaghi, (1923) developed a relation among the volume compressibility *(m_v)*, permeability *(k_s)* and the coefficient of consolidation *(C_v)* in the theory of one dimensional consolidation, which is extracted in Eq. 2:

$$C_v = \frac{k_s}{m_v \gamma_w}. \quad (2)$$

Where, γ_w is unit weight of water. It has been mentioned earlier that the permeability increases with increase in temperature. This phenomenon occurs due to the decrease in the viscosity of the pore water with increase in temperature, which can be calculated from eq. 1. Thereby, it leads to increasing the dissipation of the excess pore water pressure and, therefore, the consolidation process also expedites (Towhata et al., 1993; Di Donna and Laloui, 2015). Hence, coefficient of consolidation is also higher for a higher temperature. In addition, the influence on the coefficient of volume compressibility (m_v), which basically reflects the compressibility of specimen, was evaluated for different temperature and consolidation pressure (Bag and Rabbani, 2017; Jarad et al., 2019). It has been further

explored that at a constant consolidation pressure, the volume compressibility shows insignificant change with respect to temperature. Since, the volume compressibility decreases with increase in consolidation pressure, it can be opined that the volume compressibility strongly depends upon imposed consolidation pressure (Bag and Rabbani, 2017).

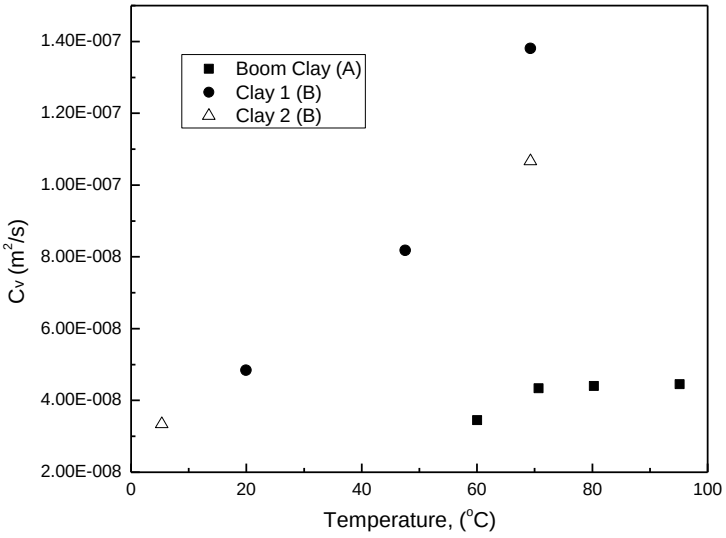


Fig. 11. Variation of Coefficient of Consolidation with Temperature (A) Delage et al., (2000) and (B) Jarad et al., (2019).

Alteration in permeability of fine grained soil as an influence of temperature has been illustrated by several researchers (Cho et al., 1999; Ye et al., 2012; Mon et al., 2014; Jarad et al., 2019). Abuel-Naga et al., 2006 have conducted the permeability test on soft Bangkok clay under elevated temperature. The result indicated that the increase in soil temperature also increased the permeability of soil. This could be due to alteration in viscosity and unit weight of pore water. Permeability of soil can be evaluated by the eq. 3, which is a relation between viscosity and unit weight of water, and it can also be measured from direct permeability test at different temperature.

$$\frac{K(T)}{K(T_o)} = \frac{\mu(T_o)\gamma_w(T)}{\mu(T)\gamma_w(T_o)} \quad (3)$$

$K(T)$ and $K(T_o)$ is defined as the permeability at temperature, T and room temperature T_o, respectively. $\mu(T)$ and $\mu(T_o)$ is defined as viscosity of pore water at temperature T and room temperature T_o, respectively, which can be calculated from eq. 1. $\gamma_w(T)$ and $\gamma_w(T_o)$ is

555 defined as unit weight of pore water at temperature T, and room temperature T_o, respectively.
556 Likewise, Delage et al., (2000) have also adopted eq. 3 to study the effect of temperature on
557 permeability. Besides, Jarad et al., (2019) have also conducted tests at different temperature
558 to explore the variation in intrinsic permeability of soil. Based on these literatures, it can be
559 established that permeability is proportional to the temperature due to decrease in viscosity of
560 pore water at elevated temperature. But the intrinsic permeability, which is a function of
561 porosity, is independent of temperature. It has also been conveyed that the dissipation of pore
562 water pressure is higher at a higher temperature (Delage et al., 2000; Jarad et al., 2019).

563 **3.8.Shear strength parameters**

564 Despite of having several extensive studies, impact of temperature on shear strength
565 of soil remains a contentious matter, which needs to be discussed clearly due to diverse
566 response obtained from several studies (Abuel-Naga, Bergado, & Lim, 2007; Campanella &
567 Mitchell, 1968; Cekerevac & Laloui, 2004; Maghsoodi et al., 2020, 2021; Wang et al., 2020;
568 Yavari et al., 2016). An increase in shear stress upon heating in drained and undrained
569 conditions has been found for Bangkok clay irrespective of OCR value (Abuel-Naga et al.,
570 2007b). (Campanella & Mitchell, 1968; T. Hueckel & Baldi, 1990; Kuntiwattanakul et al.,
571 1995; Maghsoodi et al., 2020) have also reported an increase in shear strength with increase
572 in temperature. While, (Burghignoli et al., 2000; Cekerevac & Laloui, 2004; Graham et al.,
573 2001; Towhata et al., 1993; Yavari et al., 2016) have observed that shear strength remains
574 unchanged upon heating. On the other hand, several researches reported decrease in shear
575 strength upon heating (Ghahremannejad, 2003; T. Hueckel & Borsetto, 1990; Lingnau et al.,
576 1996). For NC soil, a compressive deformation has been reported during heating, which is
577 irreversible in nature. This is mainly caused by rearrangement of soil particles, which
578 improve the contact bond between particles thus shear strength should be increased.
579 Similarly, couple of studies Abuel-Naga et al., (2007b) have reported increase in shear

strength of soil with drained heating condition for Bangkok clay and Kaolin clay, respectively. Moreover, Kuntiwattanakul et al., (1995) have performed undrained shear strength tests under heating condition and increase in shear strength is reported. Nonetheless, few researchers claimed decrease in shear strength under undrained heating (Laguros, 1969; Sherif and Burrous, 1969; Hueckel and Borsetto, 1990; Ghahremannejad, 2003). Baldi et al., (1988) explained that this thermo-elasto-plastic stress could lead to microstructural changes of soils, which create the contact between particles and creates irreversible stress for NC soil. For OC soil, Kuntiwattanakul et al., (1995) have explained that shear strength of soil increases with increase in temperature when micro-resistance dominates. Micro resistance arises due to bonding between inter-particle contact of solid-solid. In micro-resistance, contact between soil particles increases due to increase in temperature. Moreover, shear strength of soil decreases with increase in temperature for OC soil when macro-resistance dominates. Macro resistance arises due to the movement of clay particles over other in shear plane. Macro-resistance would decrease the shear strength when temperature rise induces a volumetric expansion.

Subsequently, the shear strength parameters with elevated temperature has been evaluated by several researchers (Hueckel and Baldi, 1990; Robinet et al., 1996; Burghignoli et al., 2000; Graham et al., 2001; Ghahremannejad, 2003; Yavari et al., 2016; Maghsoodi et al., 2020). It can be noted that the effect of temperature on friction angle is insignificant, and no clear trend has been reported. Similarly, cohesion of clayey soil is increased for kaolin with increase in temperature (Maghsoodi et al., 2020) however reverse result is also reported by Yavari et al., (2016). The effect of temperature on shear strength parameter is summarized in Fig. 12. In drained heating condition of soil, contraction of sample occurred, subsequently shear strength increased due to thermal hardening phenomena. Due to this, thermally induced irreversible contraction and increase in shear strength observed for NC soil. The reason for

this thermo-plastic stress may lie in micro-structural changes because the tendency of soils to group together is simulated which increases mineral-to-mineral interaction and generates irreversible stress. Therefore, it can be opined that the influence on the shear strength of soil with respect to different temperatures is dependent upon the thermal loading history, loading path adopted, mineralogical composition and behaviour of soil. Therefore, further investigations are requisite to understand clearly the effect of temperature on shear strength of the soil.

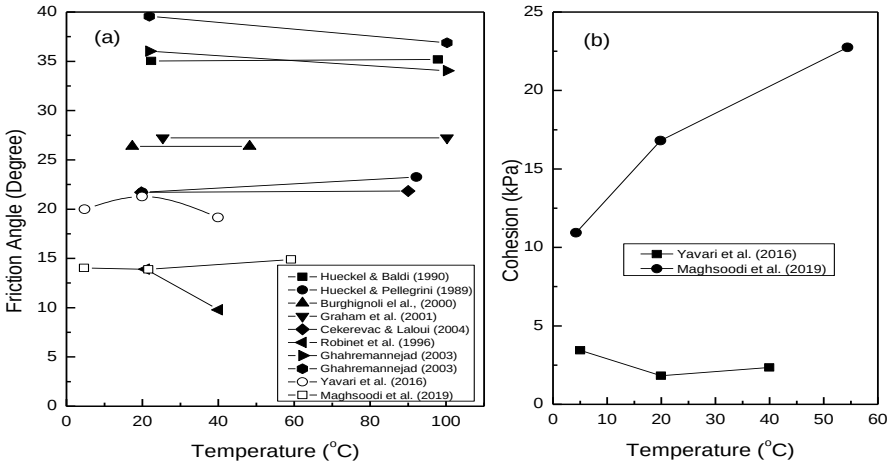


Fig. 12. Effect of Temperature on (a) Friction Angle and (b) Cohesion of Soil.

4. Mathematical Modelling of Thermally-Induced Volumetric Change

This section unveils the studies regarding the modelling approach of thermally-induced volumetric change of fine-grained soil. Several researchers have adopted the mathematical model to define the thermal volumetric change with respect to different behaviour such as – thermo-elasto-plastic behaviour, creep behaviour and thermal stabilisation under temperature cycle. Therefore, these modelling aspects are briefly discussed in the following sections.

4.1.Elastic strains

Thermo-elasto-plastic volumetric change can take place due to change in temperature and mean effective stress. The first thermo-elastic model was described for saturated soil by

thermal softening mechanism (Hueckel and Borsetto, 1990). There are several consecutive models available for thermo-elastic behaviour of fine grained soil (Abuel-Naga et al., 2007c; Cui et al., 2000; Hong et al., 2013; Laloui & François, 2009; Yao & Zhou, 2013; Zhu et al., 2021). Several researchers (Y. J. Cui et al., 2000; Hong et al., 2013, 2016; Laloui & François, 2009; Zhu et al., 2021) have proposed a model for elastic volumetric strain, which is mentioned in eq. 4.

$$d\varepsilon_v^e = \alpha dT + \frac{dp'}{K} \quad (4)$$

Similarly, (Abuel-Naga, Bergado, Bouazza, et al., 2007) have also proposed a model regarding elastic volumetric strain considering the effect of temperature and stress, both, which is written in eq. 5.

$$d\varepsilon_v^e = \frac{\alpha\prime}{T} dT + \frac{dp'}{K} \quad (5)$$

Where, α and α' is termed as volumetric thermal expansion coefficient of soil for eq. 4 and eq. 5, respectively. *T* and *p'* are temperature and effective stress, respectively. *K* is defined as bulk modulus which is independent of temperature. It is noteworthy to notice that (Abuel-Naga, Bergado, Bouazza, et al., 2007) have developed logarithmic equation versus temperature for elastic volumetric strain (refer Eq. 5). On this note, α' is a dimensionless quantity, on contrary α' should have dimension inverse of temperature. Although, α could be a function of temperature and stress (Yao & Zhou, 2013). However, it is essentially considered as constant for simplicity (Cui et al., 2000; Hong et al., 2013; Hong et al., 2016; Yao & Zhou, 2013; Zhu et al., 2021). Still, (Abuel-Naga, Bergado, Bouazza, et al., 2007) have developed an equation for α', which varies logarithmically with mean effective stress as follow (Eq. 6).

$$\frac{d\alpha\prime}{\alpha\prime} = b\frac{dp\prime}{p\prime} \quad (6)$$

Where, b is soil parameter as a function of microscopic and mineralogical configuration of clay. Moreover, (Laloui & François, 2009) have also developed an equation of α, which varies with temperature and OCR, both (refer Eq. 7)

$$\alpha = [\alpha_o + \zeta(T - T_o)]\frac{p'_{c0}}{p'} \quad (7)$$

Where, α_o is coefficient of thermal expansion at T_o; ζ is defined as slope of variation α with T. p'_{c0} is pre-consolidation pressure at T_o. The bulk modulus is considered same as Cam-clay model by (Abuel-Naga, Bergado, Bouazza, et al., 2007; Y. J. Cui et al., 2000; Hong et al., 2013; Yao & Zhou, 2013; Zhu et al., 2021), is mentioned in eq. 8.

$$K = \frac{(1+e)}{k}p' \quad (8)$$

e and k is initial void ratio and elastic unloading index, respectively. Moreover, (Laloui & François, 2009) have developed a hypo-elastic modulus for K as follows (Eq. 9).

$$K = K_{ref}\left(\frac{p'}{p'_{ref}}\right)^{n^e} \quad (9)$$

Where, K_{ref} is equal to reference bulk modulus at reference pressure p'_{ref}. n^e is soil parameter.

4.2.Yield surfaces

Existing thermal constitutive models can predict thermo-elasto-plastic behaviour by adopting one or more thermal yield surfaces to define the boundary between the thermo-mechanical elastic and plastic domains. It is more common to avail one yield surface to observe the influence of effective stress as well as temperature changes (François & Laloui, 2008; T. Hueckel & Baldi, 1990; Laloui & François, 2009; Yao & Zhou, 2013). Likewise, some studies have employed two yield surfaces to independently record actual temperature and stress effects on soil (Abuel-Naga, Bergado, Bouazza, et al., 2007; Y. J. Cui et al., 2000; Hong et al., 2016; Zhu et al., 2021). Yield locus, which is developed to enhance the volumetric change due to effect of OCR, is called as loading yield, LY. Locus of loading

yield has been developed by several researchers. Thermo-mechanical yield limits for several consecutive models are represented in fig. 13.

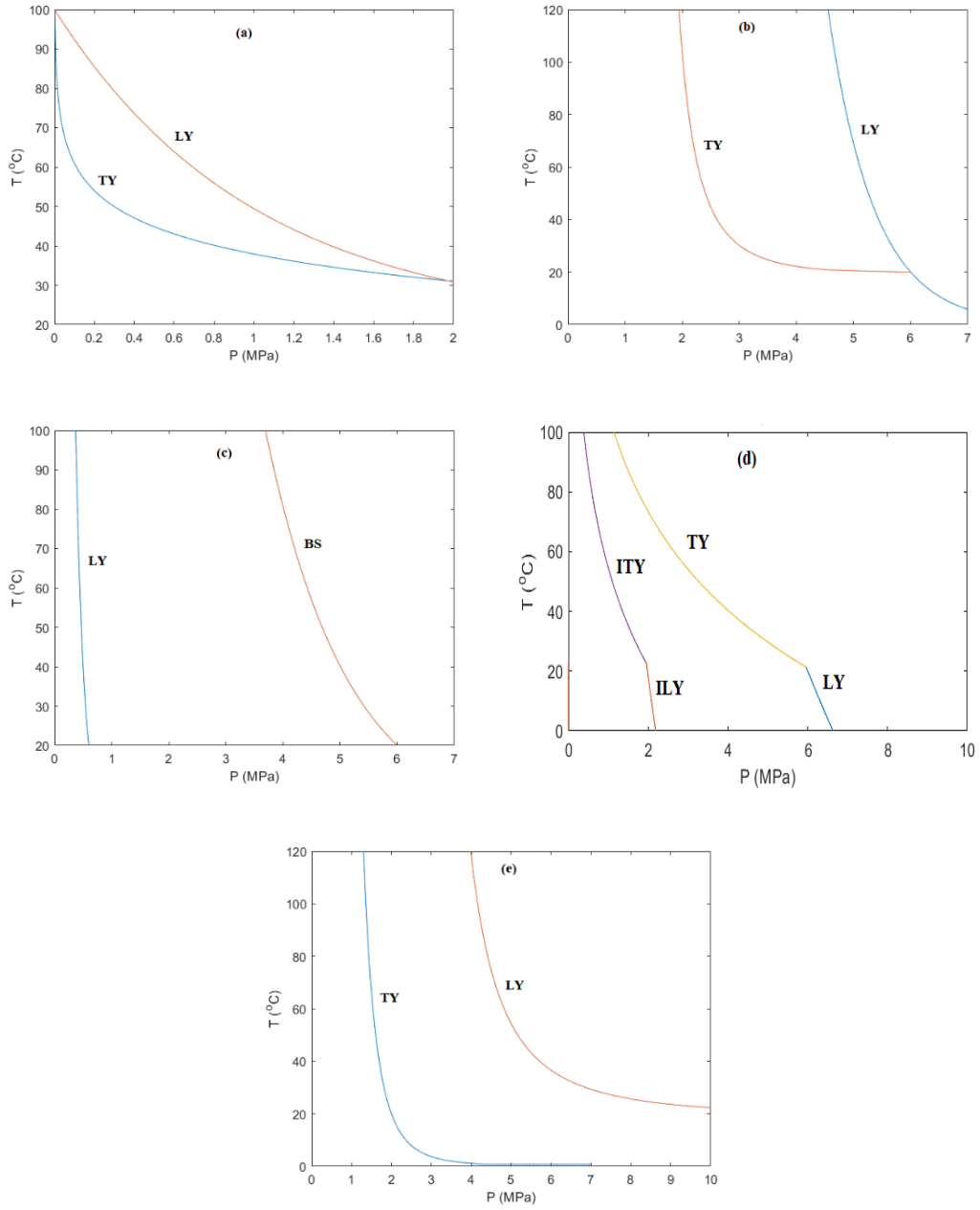


Fig. 13. Thermo-mechanical yield limits (LY and TY) for a) (Y. J. Cui et al., 2000), b) (Abuel-Naga, Bergado, Bouazza, et al., 2007), c) (Laloui & François, 2009), d) (Hong et al., 2016) and e) (Zhu et al., 2021)

Cui et al., (2000) have developed an equation for LY (refer fig. 13(a)), in which decrease in pre-consolidation pressure has been observed with increasing temperature (refer Eq. 10)

$$p' = p'_0 \, exp(-\beta_o \Delta T) \quad (10)$$

Where, p_0' is pre-consolidation pressure at T_o and intersection of LY curve at p'-axis. β_0 is curvature of LY. Moreover, (Abuel-Naga, Bergado, Bouazza, et al., 2007) have also defined the LY curve in different concept. Pre-consolidation pressure is not affected during heating, however increase during cooling phase (refer fig. 13(b)). Mathematical equation is written in eq. 11.

$$\frac{p_c(T)}{p_c(T_o)} = 1 - \gamma^{LY}\left[log\left(\frac{T}{T_o}\right)\right] \quad (11)$$

γ^{LY} is defined as model parameter and dependent on type of soil. It is zero in heating phase and non-zero for cooling state. Laloui & François, (2009) have developed a bounding surface (BS) concept along with LY curve, as shown in fig. 13(c). LY curve is defined as to deal with both normally consolidated and over-consolidated states. The bounding surface (BS) can be used for the conventional thermo-mechanical state. The equation for LY and BS is mentioned in eq. 12 and eq. 13, respectively.

$$\frac{p_c(T)}{r_{iso}p_c(T_o)} = 1 - r_T\left[log\left(\frac{T}{T_o}\right)\right] \quad (12)$$

$$\frac{p_c(T)}{p_c(T_o)} = 1 - r_T\left[log\left(\frac{T}{T_o}\right)\right] \quad (13)$$

Where, r_T is defined as curvature of yield surface. $r_{iso}$is defined as the ratio of mean effective stress, $p_c(T_o)$ at loading surface to mean effective stress at bounding surface. r_{iso} is evaluated between 0 to 1, if its value is unity then BS and LS coincide to each other. Furthermore, Yao & Zhou, (2013) have adopted similar LY curve as Cui et al., (2000) to define the thermo-elasto-plastic behaviour of soil. The equation of LY curve is adopted by (Yao & Zhou, 2013), which is written in eq. 14.

$$p' = p_0' \, exp(-\beta \, \Delta T) \quad (14)$$

$$\beta = \upsilon(\lambda_T - K_T)/(\lambda - K) \quad (15)$$

Where, υ is defined as specific volume. β is written in eq. 15. λ and λ_T is referred as thermo-elasto-plastic unloading index and thermo-elasto-plastic compression index, respectively. K

and K_T is thermo-elastic unloading index and thermo-elastic compression index, respectively. Additionally, (Cheng et al., 2020; Hong et al., 2016) have adopted the two yield surface approach for smooth transition from elastic to plastic state, as shown in fig. 13(d). Conventional yield behaviour and inner yield behaviour has been introduced for LY and ILY respectively by (Cheng et al., 2020; Hong et al., 2016), expression for LY is written in p-q-T space in eq. 16, respectively.

$$f_{LY} \equiv q^2 + \frac{M_f^2}{1-k_f}\left(\frac{p'}{p'_{cT}}\right)^{2/k_f} p'^2_{cT} - \frac{M_f^2 p'^2}{1-k_f} = 0 \quad (16)$$

With, $p'_{cT} = p'_{co} exp[-\alpha_o(T - T_o)]$ (16a)

Where, M_f and k_f are parameter to specify the shape of LY at given temperature. p'_{cT} and p'_{co} are defined as pre-consolidation pressure at temperature T and $T_{o,}$ respectively. α_o controls the curvature of LY. ILY is described as homologous to LY with respect to origin in p-q-T space, the definition of ILY is defined in eq. 17.

$$f_{ILY} \equiv q^2 + \frac{M_f^2}{1-k_f}\left(\frac{p'}{r^{LY}p'_{cT}}\right)^{2/k_f} (r^{LY}p'_{cT})^2 - \frac{M_f^2 p'^2}{1-k_f} = 0 \quad (17)$$

Where, r^{LY} is defined as a ratio of size of yield surface and corresponding inner yield surface. Its value is always smaller than or equal to 1. Furthermore, (Zhu et al., 2021) have developed a logarithmic equation for mechanical yield in *p*-*q* space (refer fig. 13(e)). Expression of LY curve is written in eq. 18.

$$ln\left(\frac{p_c(T)}{p_c(T_o)}\right) = \theta ln\left(\frac{T_o}{T}\right) \quad (18)$$

Where, θ is soil parameter which determine the thermal dependence of preconsolidation pressure. According to the LY yield surface, an instantaneous contraction would turn out for normal consolidated specimen. But LY surface could not clearly define for the specimen with higher OCR value, which undergo to expansion upon heating. Further, utilization of LY surface may over predict the transition temperature for light to medium OCR soils.

To accommodate these drawbacks, a new plastic mechanism has been proposed, which is based on a new locus yield, is called TY. This yield curve is related to the yield curve LY, with both curves forming the boundary of the elastic region in the plane T-p'. This mechanism allows the generation of thermoplastic deformations for over consolidated soils exposed to heat under load. Expansion-contraction behaviour is also defined with TY surface. It is evaluated that the sample exhibits entirely elastic deformation upon increasing the temperature and effective stress below TY surface. Hence, increase in temperature beyond the previous temperature imposed on soil develops the plastic strain. Cui et al., (2000) have defined the locus of TY curve, as shown in fig. 13(a). An expression is also developed to define the locus of TY surface, which is as follows (Eq. 19):

$$T_{yield} = (T_c - T_0)\, exp(-\beta p') + T_{0.} \quad (19)$$

Where, T_c is reference temperature at $p' = 0$. β is soil parameter to define curvature of the curve. Similar consecutive model has been mentioned by (Abuel-Naga, Bergado, Bouazza, et al., 2007) and (Zhu et al., 2021) in fig. 13(b) and fig. 13(e), respectively, to evaluate the TY curve as a nonlinear function with modelling parameters in eq. 20.

$$\frac{p_c(T_o)}{p_c(T)} = \gamma^{TY} \sqrt{ln\left(\frac{T}{T_o}\right)} + 1 \quad (20)$$

Where, γ^{TY} is defined as shape parameter. Furthermore, the expression of TY curve, which is developed by Cheng et al., (2020) and Peng Yun Hong et al., (2016), shows in fig. 13(d). The mathematical equation is written in eq. 21.

$$f_{TY} \equiv T - T_c + \frac{1}{\beta} ln\left(\frac{p'}{p'_{ref}}\right) = 0 \quad (21)$$

Where, p'_{ref} is taken as reference pressure and equal to atmospheric pressure. T_c and T are the yield temperature corresponding to p'_{ref} and p'. β is defined as parameter to control the TY curve. Similar to ILY, ITY has been developed as homologous to TY curve, which is mentioned in eq. 22.

$$f_{ITY} \equiv T - r^{TY}T_c + \frac{1}{\beta} ln\left(\frac{p'}{p'_{ref}}\right) = 0 \quad (22)$$

Where, r^{TY} is defined as a ratio of size of yield surface and corresponding inner yield surface. Its value is always smaller than or equal to 1.

4.3.Plastic strains

A mathematical expression has been developed to predict the plastic strain with change in temperature (Y. J. Cui et al., 2000), is as follows (Eq. 23):

$$d\varepsilon_{vT}^{P} = \alpha_P \ [exp(\alpha_P \Delta T) - a] \ dT. \quad (23)$$

Where, 'a' is the shape parameter and usually near about unit. α_P is slope of thermal volumetric curve, which can be dependent on OCR. Hence, (Y. J. Cui et al., 2000) have developed a separate equation for normal consolidation (i.e.- LY) and over consolidated (i.e.- TY) soil in Eq. 24 and Eq. 24a, respectively.

$$\alpha_p = -\frac{\alpha}{1-a} \quad (24)$$

$$\alpha_p = \frac{\alpha}{exp\left(\frac{1}{c_2} ln\left(\frac{1}{c_1}\frac{p'}{p'_o}\right)\right) - a} \quad (24a)$$

Where c_1 and c_2 are defined as parameters which can be evaluated from transition expansion and contraction of total volumetric change due to temperature change. Moreover, (Abuel-Naga, Bergado, Bouazza, et al., 2007) have also developed a model for thermomechanical plastic strain due to mechanical and thermal loading, which is defined as follows (refer eq. 25)

$$d\varepsilon_v^p = \frac{1}{1+e_o}\left\{\lambda_T \frac{dp_c}{p_c} + \omega_T \frac{dp_T}{p_T}\right\} \quad (25)$$

Where λ_T and ω_T are defined as plastic hardening modulus of LY and TY respectively. p_c and p_T are considered as mechanical and thermal pre-consolidation pressure respectively.

Furthermore, (Zhu et al., 2021) have described the plastic volumetric strain due to mechanical and thermal loading. Plastic volumetric strain is mentioned below for TY and LY, in eq. 26 and eq. 27, respectively.

$$d\varepsilon_{vTY}^{p} = \omega(T - T_o)\left[\frac{p}{p_{co}}\right]^2 dT \tag{26}$$

$$d\varepsilon_{vLY}^{p} = d\lambda_m \frac{\partial g^{LY}}{\partial p} \tag{27}$$

Where, $d\lambda_m$ is described as a plastic multiplier, a positive value. g^{LY} is defined as a plastic potential function as per LY. The expression of g^{LY} is written below in eq. 28.

$$g^{LY} = kln\left[1 + (k-1)\left(\frac{\eta}{M}\right)^2\right] + 2(k-1)ln\left(\frac{p}{C}\right) \tag{28}$$

Where, η is a stress ratio (=q/p). M is defined as slope of critical state line at q-p scale. C indicates the plastic potential surface.

4.4.Hardening law

If the stress is increased at constant temperature, or temperature is increased at constant stress, or both stress and temperature are increased, the yield surface would gradually approach towards the plastic zone. Further, as the yield loci move towards the right, a hardening tendency develops which expands towards the elastic zone (Y. J. Cui et al., 2000). The movement of LY curve has not any alteration of TY curve. Contrariwise, influence of TY on the LY curve should be considered for the modelling of over consolidation effect (Y. J. Cui et al., 2000; Towhata et al., 1993). When TY curve is passed by the thermo-mechanical path, hardening phenomena has been developed and governed by β as a hardening parameter for TY curve. The following hardening law is defined for TY by (Y. J. Cui et al., 2000) in eq. 29.

$$d\beta = \frac{-\exp(\beta p')}{p'(T-T_o)\alpha_p[\exp(\alpha_p \Delta T) - a]} d\varepsilon_{vT}^{p} - \frac{\beta}{\alpha_1} d\varepsilon_{vTp}^{p} \tag{29}$$

Where, $d\varepsilon_{vT}^{p}$ and $d\varepsilon_{vTp}^{p}$ are defined as the thermal and mechanical plastic strains, respectively, when loading path touches TY curve. α_1 is defined as plastic modulus. Hardening law for LY curve has also been defined by (Y. J. Cui et al., 2000) in eq. 30, where p_o' is considered as hardening parameter.

$$\frac{dp_o'}{p_o'} = \frac{\alpha_o}{\alpha_p[\exp(\alpha_p\Delta T)-a]} d\varepsilon_{vpT}^{p} + \frac{v_o}{\lambda-k}\left[d\varepsilon_{vp}^{p} + d\varepsilon_{vTp}^{p}\right] + \left\{\frac{\alpha_o}{\alpha_p[\exp(\alpha_p\Delta T)-a]} + k\left(\frac{v_o}{\lambda-k}\right)\right\} d\varepsilon_{vT}^{p} \quad (30)$$

Where, ε_{vpT}^{p} and $d\varepsilon_{vp}^{p}$ are referred as the thermal and mechanical plastic strain, respectively, when loading path touches LY curve. *k* is defined as parameter that developed due to coupling effect of TY and LY. The value of k depends on the magnitude of over consolidation ratio. If k=0, no thermal hardening has been observed for NC sample. α_o is a curvature of LY curve. Moreover, (Abuel-Naga, Bergado, Bouazza, et al., 2007) have defined a theoretical mechanism to evaluate the plastic strain along thermal loading path. Along heating, the hardening effective stress on TY decreased with increase in temperature according to locus of TY curve. No hardening law is developed for TY. However, hardening law for LY is developed by (Abuel-Naga, Bergado, Bouazza, et al., 2007), which is controlled by hardening parameter, p_{co}' (refer eq. 31).

$$\frac{dp_{co}'}{p_{co}'} = \frac{v_o}{\lambda-k-\omega_T} d\varepsilon_{vpT}^{p} \quad (31)$$

(Laloui & François, 2009) have adopted a hardening law for TY as a function of hardening parameter, r_{iso}. r_{iso} is governed by the consistency condition. Following expression is considered for the loading condition, which is a hyperbolic function of plastic volumetric strain (refer eq. 32).

$$dr_{iso} = \frac{(1-r_{iso})^2}{c} d\varepsilon_{v}^{p} \quad (32)$$

For unloading condition, r_{iso} decreased to follow the decrease in mean effective stress, *p'*. At reloading condition last change in direction of mean effective stress on unloading- loading, r_{iso} is adjusted to define the elastic nuclei, r_{iso}^{e}, which be defined in eq. 33.

$$r_{iso} = r_{iso}^{e} + \frac{p'_{cyc}}{p'_o} \quad (33)$$

p'_{cyc} is defined as mean effective stress at last change of direction of unloading-loading. The hardening law for LY is defined as a relationship between volumetric plastic strain, ε_v^p and pre-consolidation pressure, p'_o (refer eq. 34)

$$dp' = \beta p'_o(d\varepsilon_v^p) \quad (34)$$

Where β can be classified as plastic compressibility modulus. Furthermore, (Yao & Zhou, 2013) have also described the hardening law, which is associated with modified cam clay model and consider the effect of stress history. Its hardening is written in eq. 35.

$$\overline{p_x} = \overline{p_{xo}} \int \frac{1+e}{\lambda-k} d\varepsilon_v^p \quad (35)$$

Where, $\overline{p_x}$ is defined as the cross point between yield surface and p-axis. $\overline{p_{xo}}$ is defined as isotropic pre-consolidation pressure. Furthermore, (Cheng et al., 2020; Hong et al., 2016) have developed the hardening law with parameters p'_{co} and r^{LY} for LY and ILY respectively. Experimental evidence from (Y. J. Cui et al., 2000) shows that hardening of LY develops when TY is activated. By the activation of LY, no hardening has been produced for TY. It is unidirectional coupling of TY with LY. Therefore, plastic strain increment influenced with plastic mechanism TY will also develop an increment in p'_{co}. The plastic hardening law for p'_{co} is described in eq. 36.

$$dp'_{co} = \frac{v_o}{\lambda-k} p'_{co}\left(d\varepsilon_{vLY}^p + d\varepsilon_{vTY}^p\right) \quad (36)$$

Where, $d\varepsilon_{vLY}^p$ and $d\varepsilon_{vTY}^p$ is referred as plastic strain for LY and TY, respectively. To describe the hardening law along thermomechanical loading as ILY is activated. The expression of r^{LY} is written in eq. 37.

$$dr^{LY} = \frac{v_o}{\lambda-k} s_{LY}(1 - r^{LY})\left(d\varepsilon_{vLY}^p + d\varepsilon_{vTY}^p + A_d d\varepsilon_{sLY}^p\right) \quad (37)$$

Where, s_{LY} is a material constant which control the rate of approaching ILY toward LY. A_d is a parameter that control the shear plastic strain, $d\varepsilon_{sLY}^p$ to soil hardening. Similarly, (Cheng et

al., 2020; Hong et al., 2016) have developed the hardening law for TY and ITY, which is defined in eq. 38 and eq. 39 respectively.

$$dT_c = \frac{v_o}{(\lambda-k)\alpha_o} f(OCR)\, d\varepsilon_{vTY}^{p} \tag{38}$$

$$dr^{TY} = \frac{v_o}{(\lambda-k)\alpha_o} f(OCR) s_{TY}\,(1 - r^{TY}) d\varepsilon_{vTY}^{p} \tag{39}$$

$$f(OCR) = 1 + s_{LY}(\frac{1}{r^{LY}} - 1) \tag{39a}$$

Where, s_{TY} is defined as material constant that control the rate of approaching ITY to TY. Similar to r^{LY}, contribution to r^{TY} can be used to predict the smooth elastoplastic transition when ITY is activated. Recently, (Zhu et al., 2021) have described the hardening law by coupling mechanical and thermal plastic mechanism, which is appeared in LY and TY function both as a hardening parameter. The mechanical and thermal volumetric strain move the LY and TY limit simultaneously; the hardening law is written as follow (refer eq. 40).

$$dp_c = p_{co} \frac{1+e}{\lambda-k} (d\varepsilon_{vLY}^{p} + d\varepsilon_{vTY}^{p}) \tag{40}$$

Several thermo-mechanical models are discussed to evaluate the elastic and plastic volumetric strain due to thermal and mechanical loading. (Abuel-Naga, Bergado, Bouazza, et al., 2007; Cheng et al., 2020; Y. J. Cui et al., 2000; Hong et al., 2016; Zhu et al., 2021) have introduced the two-yield surface to describe the thermo-mechanical plastic strain. Meanwhile, (Laloui & François, 2009; Yao & Zhou, 2013) have adopted single yield surface mechanism. For better understanding, TY and LY is coupled to determine the plastic strain through the change in size of yield surface. It can be demonstrated that many thermo-mechanical features such as thermal volumetric strain of soil at different OCR and thermally induced strength of soil are well described with constitutive model.

4.5.Heating under undrained conditions

The method for prediction of excess pore pressure due to undrained heating response has been described in numerous literatures (Abuel-Naga, Bergado, & Bouazza, 2007;

Burghignoli et al., 2000; Campanella & Mitchell, 1968; Ghaaowd et al., 2015; Houston et al., 1985; Mohajerani et al., 2012; Salager et al., 2008; Uchaipichat & Khalili, 2009). These studies have indicated that the initial void ratio, effective stress, stress history and change in temperature can affect the magnitude of excess pore pressure. (Mohajerani et al., 2012) observed that in undrained thermo-mechanical study, pore pressure increased in porous material in following manner (Eq. 41).

$$\Delta u = B\Delta\sigma + \Lambda\Delta T \tag{41}$$

Where, B and Λ are described as Skempton coefficient and thermal pressurization coefficient. In this context, Ghaaowd et al., (2015) have illustrated while performing heating test without drainage that, the water pressure in the pores increased linearly with temperature. Moreover, over-consolidation ratio alters the pore pressure in a way that as OCR value increases while pore water pressure decreases for a particular temperature (Abuel-Naga et al., 2007a; Uchaipichat and Khalili, 2009; Ghaaowd et al., 2015). A theoretical formula has also been established to estimate the pore pressure for saturated soil without drainage during heating cycles by utilizing the thermo-elastic and linear elastic concept in Eq. 42 (Campanella and Mitchell, 1968). The theoretical formula is written as follows:

$$\Delta u = \frac{\eta\Delta T\,(\alpha_s - \alpha_w) + \alpha_{st}\,\Delta T}{m_v}. \tag{42}$$

Where, α_{st} is coefficient of structural volume change. α_s *and* α_w are the coefficient of thermal expansion for soil solids and pore water, respectively. The value of α_s and α_w were reported by researchers such as equal to 0.000035/^{o}C and 0.00017/^{o}C (Campanella and Mitchell, 1968; Burghignoli et al., 2000), whereas the reported value of α_{st} is -0.00005/^{o}C (Mitchell and Soga, 2005). Further, Campanella and Mitchell, (1968) did not consider the effect of plasticity index, PI, so the effect of average PI was consider to plot the curve for α_{st}. Although, it has been conveyed with some scatter that the value of α_{st} decreases non-linearly with increase in

plasticity index of specimen (Mitchell and Soga, 2005). Additionally, an relationship between α_{st} and PI has been derived by Ghaaowd et al., (2015) and the same is expressed in Eq. 43:

$$\alpha_{st} = 1\times10^{-4}e^{-0.014PI}. \quad (43)$$

Nonetheless, the above investigation has recommended that extensive studies should be carried out on different soil type, so that a better relationship between α_{st} and *PI* can be obtained. It is accepted that such development of the Eq. 43 will facilitate quick and accurate determination of Δu without resorting to detailed test reports. Furthermore, it has been stated that the pore water pressure of saturated soil is effected by the stress history (OCR) at different temperature (Abuel-Naga et al., 2007a) . It has been reported that the pore water pressure decreases with increase in OCR. Similar trend has been observed by (Ghaaowd et al., 2015). Subsequently, thermally induced pore water pressure is observed as reversible for NC soil and it is irreversible for OC soil, that also generates negative pore pressure while cooling to room temperature (Hueckel and Pellegrini, 1992; Abuel-Naga et al., 2007a). However, thermal induced pore water pressure for unsaturated soil has not been explored significantly.

4.6.Creep behaviour

Several researchers have proposed the equation to bridge the creep coefficient and temperature (Fox & Edil, 1996; Houston et al., 1985; Zhu & Qi, 2018). Regarding this, Zhu and Qi, (2018) have adopted an elastic visco-plastic model, in which temperature dependent creep against one dimensional loading is described and the results were compared vis-à-vis with the results obtained from experimental investigation. For visco-plastic strain (ε_v^{vP}), 1-D model proposed by Kutter & Sathialingam, (1992), which is based on creep coefficient, ψ_T (refer eq. 44).

$$\varepsilon_v^{vP} = \frac{\psi_T}{(1+e_o)t_r}\left(\frac{\sigma_v}{\sigma_v^r}\right)^{\lambda-k/\psi_T} \quad (44)$$

Where, e_o is initial void ratio. t_r is defined as a reference time and equal to time taken in each load increment. σ_v is effective vertical stress. σ_v^r is reference stress corresponding to load increment time, t_r. $\lambda \, and \, k$ is referred as slope of compression line and recompression line respectively. It has been reported that the secondary thermal consolidation initiates taking place after completion of dissipation of pore water, and the induced settlement occurs due to the effect of time and temperature. Thus, coefficient of thermal creep is defined as the slope of thermal secondary consolidation curve, given in void ratio and time in log scale (e vs log (t)) (Zhu and Qi, 2018). Moreover, linear function is derived between thermal creep coefficient and temperature (Houston et al., 1985), which is written (eq. 45),

$$\psi_T = \text{A} \times \text{T}. \tag{45}$$

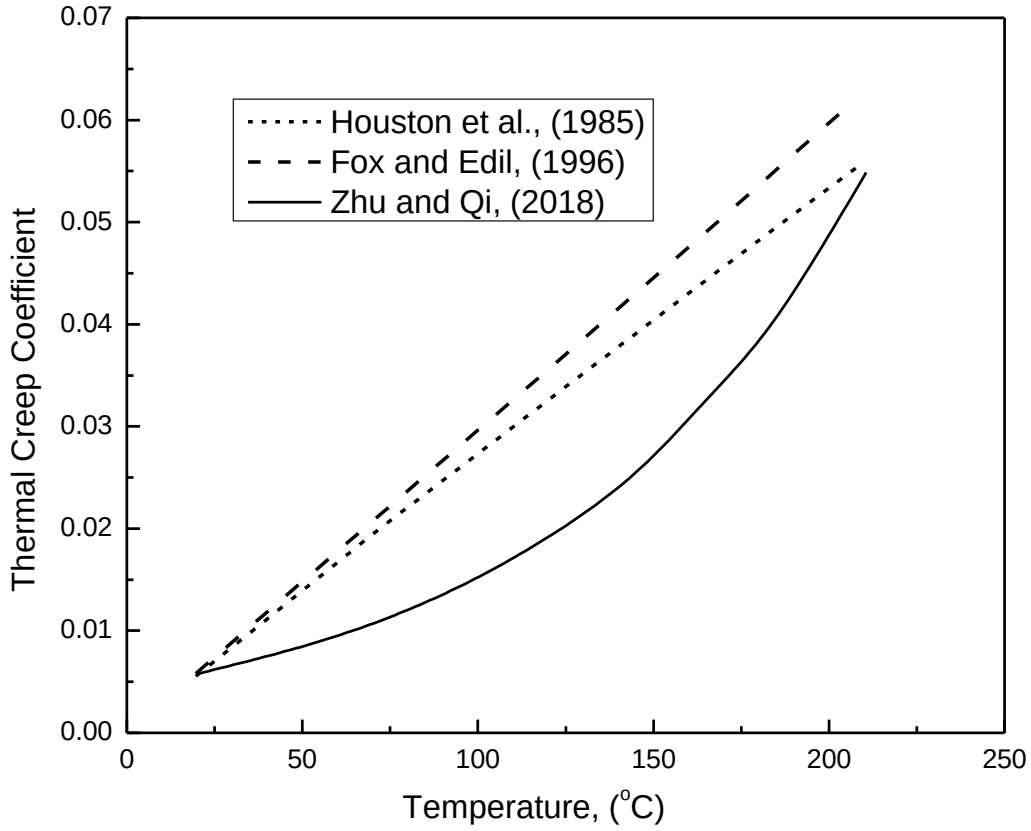
Where, ψ_T is thermal creep coefficient under specific temperature 'T' and A is thermal parameter. The value of A has been considered as 0.0105 (Houston et al., 1985) and it was stated that 'ψ_T' linearly increases with temperature. Hence, ψ_T will be zero and negative when the corresponding temperature is zero and negative respectively. On the contrary, another equation has been represented based on the experimental study for development of 'ψ_T' with change in temperature (Fox and Edil, 1996), which is mentioned below (Eq. 46):

$$\psi_T = \psi_{Tr} \exp [\text{B} (\text{T}-\text{T}_\text{r})]. \tag{46}$$

Where, ψ_{Tr} is reference thermal creep coefficient for reference temperature 'T_r' and B is thermal parameter, whose value is equal to 0.25 ± 0.02/ ^{o}C (Fox and Edil, 1996). However, at a higher temperature, the value of ψ_T crosses beyond the normal range as per eq. 46. Additionally, (Zhu and Qi, 2018) have also noticed a nonlinear increment in thermal creep coefficient with increase in temperature, which can be written as (Eq. 47):

$$\psi_T = \psi_{Tr} \left(\frac{T}{T_r}\right)^{C_T}. \tag{47}$$

940 Where, C_T is thermal related parameter. Based on the experimental results the value
941 of C_T was considered as 0.95 (Zhu and Qi, 2018). It is worthwhile to mention that eq. 47
942 resolves the limitation associated with Eq. 45 and eq. 46 in terms of minimum and maximum
943 value of thermal creep coefficient. Hence, fig. 14 developed by comparison among eq. 45, eq.
944 46 and eq. 47, which describing the relation between thermal creep coefficient with
945 temperature. However, the effect of temperature on creep behaviour of soil was illustrated
946 experimentally (Jarad et al., 2019; Zhu et al., 2017; Zhu & Qi, 2018) but needs a clear
947 representation. Moreover, a 1-D direct modelling of thermal creep is developed by (Zhu &
948 Qi, 2018), which is elastic-visco-plastic (EVP) model. This model is developed to assume
949 that the loading condition is one dimensional and limited to soft clay. Hence, a particular
950 model which replicates the real time loading pattern for soils needs to be developed.

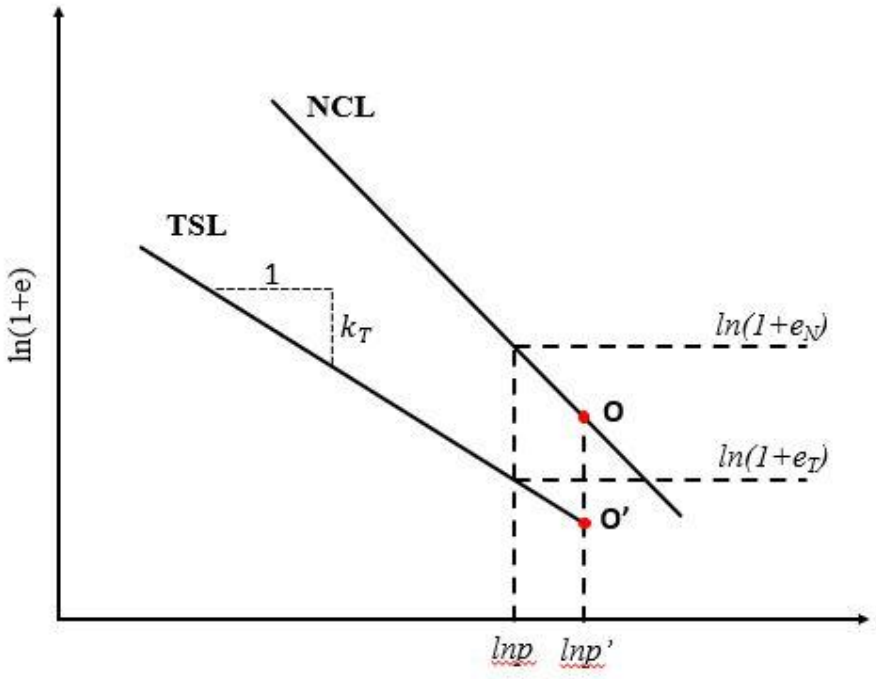


951

952 **Fig. 14.** Relationship of thermal creep coefficient with temperature from eq. (42), (43) and
953 (44).

954 **4.7.Modelling of volumetric change under temperature cycle**

955 As obtained from experimental results, a thermal stabilisation line has been proposed,
956 which controls the stabilized soil against the cyclic temperature loading (Campanella &
957 Mitchell, 1968; Di Donna & Laloui, 2015; Vega & McCartney, 2014). As shown in fig. 15,
958 the normal compression line (NCL) and the thermal stabilization line (TSL) at elevated
959 temperature is assumed to be a straight line in *lnv - lnp'*. Where, the point O refers to the state
960 of NC soil for first heating cycle. Similarly, point O' represents the stabilized soil after
961 several temperature cycle with given temperature increase. The distance between OO'
962 represents the irreversible volumetric contraction of NC soil with respect to first cyclic
963 heating. If point O and O' coincides then it is said that no accumulation of irreversible
964 volumetric compression occurs. Therefore, it can be opined that as temperature changes, the
965 TSL and NCL shifts together, which also reflect from experimental results by Campanella &
966 Mitchell, (1968). During heating, if soil state lies above TSL then irreversible volumetric
967 compression develops, otherwise the response is elastic in nature. While cooling, reversible
968 volumetric change develops (Ma et al., 2017).



969

970 **Fig. 15.** Proposed Model Line for TSL (Q. J. Ma et al., 2017).

971 Hence, it is very difficult to quantify the effect of temperature on TSL experimentally. As
972 stated by Ma et al., (2017), the accumulation and stabilization of irreversible volumetric
973 compression along temperature cycles can be well defined by thermal shakedown. According
974 to concept of thermal shakedown, the soil sample reaches the stabilized state after certain
975 temperature cycle. For mechanical loading, volumetric change of specimen is equivalent to
976 change in void ratio. However, during thermal loading volume change of specimen is due to
977 deformation of soil void as well as soil particles, which is reversible in nature during cooling.
978 Therefore, NCL and TSL return to its initial position during cooling for first thermal cycle.
979 (Ma et al., 2017; Mašín & Khalili, 2012). TSL and NCL moves down with increase in
980 temperature. TSL shifts more as compared to NCL. Furthermore, after several temperature
981 cycle, soil state reaches the TSL with elevated temperature and vertical distance becomes
982 zero. So there is no irreversible amount of compression if the temperature does not exceed the
983 maximum value of the previous heating. Mašín & Khalili, (2012) have developed an
984 expression for temperature dependent NCL, which is written following (Eq. 48):

985 $$\ln(1+e) = N(T) - \lambda(T) * \ln(\sigma'/\sigma_r) \quad (48)$$

986 Where, e is void ratio; σ' is mean effective stress; σ_r is reference stress (1kPa). λ(T) and
987 N(T) are defined as temperature dependent slope and intercept of NCL, respectively, which is
988 expressed in eq. 49 and eq. 49a:

989 $$\lambda(T) = \lambda(T_r) + l_T * \ln\left(\frac{T}{T_r}\right) \quad (49)$$

990 $$N(T) = N(T_r) + n_T * ln\left(\frac{T}{T_r}\right) \quad (49a)$$

991 Where, l_T and n_T are the modelling parameters which controls the change of slope and shifts
992 of NCL, respectively. T_r is the reference temperature. From the experimental results, it can be
993 concluded that λ is independent of temperature (Campanella & Mitchell, 1968). Therefore, l_T

should be taken as zero. Similarly, an expression has been developed to introduce TSL with respect to temperature, which is written below in eq. 50 (Ma et al., 2017):

$$\ln(1+e) = \ln(1+e') - k_T * \ln(p/p') \quad (50)$$

Where, p' is termed as pre-consolidation pressure. Therefore, Ma et al., (2017) have developed another equation to calculate $\ln(1+e')$, which in mentioned in eq. 51:

$$\ln(1+e') = \ln(1+e_o) + c_T * n_T * \ln(T/T_r) \quad (51)$$

Where e_o & e' is defined as void ratio for point O & O', respectively. Here, it is assumed that if temperature is less than reference temperature then e' is equal to e_o. Moreover, c_T is used to determine the accumulated volumetric compression after the first thermal cycle. If there is no accumulation, then c_T is equal to zero. k_T is slope parameter which is used to simulate the irreversible volumetric contraction of soil. As mentioned, k_T is inversely proportional to irreversible volumetric contraction. If k_T decreases, simulated irreversible volumetric contraction increases for soil. Di Donna & Laloui, (2015) have developed a bounding surface to reproduce the response of soil under thermal cyclic loading. This model can able to show the behaviour of soil beyond the thermo-elastic and thermo-plastic behaviour depending upon the soil stress state and accumulation phenomena. Therefore, isotropic yield mechanism, r_{iso} is developed to re-initialisation of locus at which plasticity starts during next heating phase (in Eq. 52).

$$r_{iso} = r_{iso}^{cyc} + \frac{p'}{p_c'} \quad (52)$$

Where, r_{iso}^{cyc} is defined as material parameter that depends to accumulate plastic deformation during thermal cycle. When heating restarts, r_{iso} is constant till the re-initialised yield surface is achieved. Therefore, during re-heating plasticity again started at temperature T^* that define as a following equation 53.

$$p' - p_o' e^{\beta \varepsilon_v^p} \left[1 - \gamma_T . ln\left(\frac{T^*}{T_o}\right)\right] \left(r_{iso}^{cyc} + \frac{p'}{p_o' e^{\beta \varepsilon_v^p}}\right) \quad (53)$$

Furthermore, (Zhou et al., 2017) have purposed a two boundary surface to capture the effect of thermal cycle. The original bounding surface was modified with two bounding surface (i.e. - a bounding surface and a memory surface). The bounding surface, F is expressed in the following expression (refer eq. 54):

$$F = \left(\frac{q}{Mp'}\right)^n + \frac{\ln\left({}^{p'}/_{p_o}\right)}{lnr} \quad (54)$$

Where, n and r is the soil parameter, which controls the shape of bounding surface. M is the value of critical state stress ratio. Moreover, the memory surface, F^* can be expressed as a following equation 55:

$$F^* = \left(\frac{q}{Mp'}\right)^n + \frac{\ln\left({}^{p'}/_{p_l}\right)}{lnr} \quad (55)$$

Where, p_l can be described as the size of the loading surface. Pre-consolidation pressure, which is a function of plastic volumetric strain, and maximal stress state, respectively, control the boundary surface and memory surface. The double surface idea effectively captures thermal cyclic effects because the two surfaces have separate hardening laws. The consistency of measured and estimated soil volume changes during temperature cycles is demonstrated. Furthermore, Golchin et al., (2022b) have established a kinematic rule to measure the progressive volumetric strain of soil under thermal cycles. The inner yield surface, which is controlled by a kinematic rule, begins to translate when plastic strain increments are developed. The kinematic rule was activated during heating phase and deactivated during cooling phase. Therefore, the plastic volumetric strain increments produced and were defined in the eq. 56 (Golchin et al., 2022b).

$$\varepsilon_v^p = \left(\frac{1+e_o}{\lambda-k}\right)^{-1}\{\mu\langle T'\rangle + \mu\langle T - T_o\rangle\} \quad (56)$$

Where, ε_v^p is defined as plastic volumetric strain increment. λ and k are defined as elastoplastic compression index and elastic compression index, respectively. T, T' and T_o represent the absolute temperature, temperature increment and initial temperature,

correspondingly. μ shows the coefficient of thermal softening of inner yield surface. Further, $\langle\ \rangle$ is defined as Macaulay bracket, which operates as $\langle x\rangle = (x + |x|)/2$. This model leads to the prediction of plastic strains in highly over-consolidated states as well as during the heating phase of cyclic thermal loads. As the soil becomes over-consolidated and the model has control over the accumulated plastic strains brought on by thermal loads, the magnitude of thermally-induced plastic strains decreases on subsequent thermal cycles.

4.8.Thermodynamics Model

In this section, granular thermodynamics model is used to build a new approach towards THM mechanism. This model takes into account influencing elements such temperature, saturation, the solid skeleton's microstructure, and phase transitions between the liquid and solid/gas phases. The proposed conceptual model has universal physical significance because it is supported by common experimental findings. In recent years, emphasis has been placed on nonlinear thermodynamic constitutive models developed through the so-called deformation work that closely adhere to the principle of energy conservation. The product of each phase stress and the accompanying generalised strain is exactly what makes up the deformation work. Changes in water content, soil deformation, and microstructures all have an impact on one another. Solid, liquid, and gas phases all undergo mutual transformation, therefore the deformation work brought on by phase transitions must also be taken into account (Bai, Zhou, et al., 2021; Liu & Lai, 2020). Several researchers have been implemented the thermodynamic momentum conservation law to understand the thermo-mechanical behaviour of fine grained soil (Bai, Zhou, et al., 2021; Zhang & Cheng, 2017). Particle temperature is the thermodynamic observable variable entropy of particles, which is comparable to the relationship between thermodynamic temperature and entropy at the macroscopic level. In terms of energy dissipation, this description is consistent with classical thermodynamics, which uses entropy or temperature to denote the random motion of

materials. Thus, the rearrangement of particles can reflect the plastic deformation of fine grained soil, which is released at this kinetic energy, can be transformed into molecular motion, accompanied by the production of entropy. Based on this concept, Zhang & Cheng, (2017) have developed a model to evaluate the mean effective stress according to thermodynamic law in drained condition (refer eq. 57).

$$p' = 1.5B(\varepsilon_v^e + c)^{0.5}[0.4(\varepsilon_v^e)^2 + \zeta(\varepsilon_d^e)^2] + 0.8B(\varepsilon_v^e + c)^{1.5}\varepsilon_v^e + K_e\beta_T(T - T_r) \quad (57)$$

Where, ε_v^e and ε_d^e is volumetric elastic strain and deviatoric elastic strain, respectively. B and ζ is defined as the material parameter. K_e is defined as secant elastic modulus of soil. β_T is thermo-elastic volumetric coefficient of expansion of soil. T_r is reference temperature in kelvin. c is a parameter for allowable maximum volumetric tensile strain (i.e.- $\varepsilon_v^e \leq c$). Similarly, Bai et al., (2019a) have also developed a model based on conservation of energy for mean effective stress, which is written in eq. 58:

$$p' = B[(\varepsilon_v^e + \beta_T\Delta T)^{0.5} + 0.5\zeta(\varepsilon_v^e + \beta_T\Delta T + c)^{-0.5}(\varepsilon_s^e)^2] + sS_r \quad (58)$$

Where, s is defined as the metric suction. S_r is representing the rate of dissipative flow. ε_s^e is defined as second elastic strain.

5. Discussion

This section highlights the issues, which arises due to application of the thermal loading in fine-grained soil. Generally, the major issue is associated with the volumetric change due to the application of temperature. In this context, there are numerous independent literatures have been reported in this paper to mention the thermo-mechanical behavior of fine-grained soil. Nonetheless, there are still many studies whose observations and discussions are conflicting to each other against thermal volumetric change, swelling pressure and shear strength of soil. There are mainly two major states (macroscopic and microscopic), which has been considered to understand the thermally induced volumetric change of fine-grained soil. A considerable number of studies are available against the volumetric change of

soil at macroscopic scale. However, there is few consecutive models available to describe the direct method to identify the creep behavior, but still need to be clearly describe the creep behavior of soil and coupled it with several parameters of soil. The available literatures also need to describe the magnitude of primary and secondary consolidation of thermally induced volumetric change. Moreover, there are separate experimental investigations for over consolidation ratios and secondary compression index. Hence, these studies should be coupled with each other for better understanding, which would also help to develop a constitutive model regarding thermal volumetric change due to OCR and secondary compression index. Furthermore, the speculation of thermally induced volumetric change at microscopic scale are very limited. At microscopic level, it is very difficult to disintegrate the bond stress between particles, degradation of adsorbed water and development of intermolecular forces. The experimental investigation is very difficult to achieve for such type of problems. Hence, there is an opportunity to push the knowledge boundaries and develop a numerical model to illustrate these problems. Additionally, soil is inorganic in nature and several inorganic minerals are present in it. Therefore, experimental investigation remains elusive to evaluate the differential expansion of minerals upon heating. Still, there is an opportunity to illustrate the thermally induced volumetric change with respect to the differential thermal expansion of minerals of soil. Apart from that, there are still few parameters such as swelling pressure due to concentration of available cations, generation of pore water pressure with unsaturated soil and shear strength parameters of soil due to application of temperature which needs to be discussed clearly. For the fine-grained soil, there is no standard procedure for rate of heating, which can affect the thermal volumetric change. On the other hand, there are still few problems (i.e., - volume change due to cyclic thermal load, pore water pressure and shear strength parameter) which is unanswered in literatures available for the unsaturated soil. Further investigation and constitutive models

should be conducted for the better understanding of some contrasting phenomena related to couple thermal and mechanical stress regarding microscopic behavior and mechanism.

6. Concluding Remark

In this article, a comprehensive review of the available literature on temperature induced volumetric deformation of saturated and unsaturated, fine-grained soils has been analysed thoroughly. Several parameters such as volumetric change, measurement of swelling pressure, change in consolidation parameters and thermal elasto-plastic behaviour of fine-grained soil specimen have been evaluated, which can get effected by the alteration in temperature. Moreover, the mechanisms behind the thermal volumetric deformation behaviour of the soils have also been explained in terms of alteration of the abovementioned parameters. Among these parameters, thermo-elasto-plastic analysis has gained immense popularity among the researchers. Additionally, it has been explained that the response of the soils in terms of the thermal volume change behaviour has substantial influence on the initial over-consolidation ratio. The following major conclusions are yielded from the available literature regarding thermal volumetric change of fined grained soil:

- The basic mechanism of the thermal volumetric change has been studied from literature and which are 1) difference in thermal coefficient of soil solid and pore water, 2) weak shearing bond between soil particles, 3) decrease in viscosity of pore water at high temperature and 4) clay water interaction against thermal loading at microscopic level.
- The thermal volumetric change behaviour typically depends on the effective stress applied on the soil sample prior to heating. Matric suction and temperature cycle can also alter the thermal volumetric change behaviour. Besides, it has been elucidated that the thermal drained heating response of soil in terms expansion and contraction occur for high and low OCR values, correspondingly.

- The volumetric change of the soil mass due to thermal energy is independent of thermal history i.e., imposed as temperature cycle and experienced by specimen at different effective stress, nonetheless it can aid to enhance the stiffness and hardness of soils.
- Nonetheless, some parameters such as intrinsic permeability and coefficient of volume compressibility have no repercussion of temperature. These parameters are dependent of stress magnitude instead.
- Shear strength of soil under elevated temperature remains a contradictory matter; more laboratory investigation is required to clarify the thermal effect of soil on shear strength.
- The thermal loading has significant influence on creep index of specimen. Creep Coefficient of soil is proportional to temperature and increase with increase in temperature.
- Several parameters such as permeability, Creep index, pore water pressure and coefficient of consolidation is directly proportional to temperature. Whereas viscosity of pore water is inversely proportional to induced temperature.
- Moreover, the thermo-mechanical model including elastic and plastic strain is studied to understand the volumetric change of soil from various available literature. Especially, some researchers have described an alternative approach to understand the volumetric change due to thermal secondary compression and developed the relevant mathematical expressions. The experimental results have been utilized to validate the mathematical models.
- Furthermore, The Yield surface has also been defined to plot a distinct boundary between thermo-elastic region and thermo-elasto-plastic region from temperature versus pre-consolidation pressure plot. Different yield surfaces such as loading yield and thermal yield surface have been defined to measure elastic and plastic volumetric change.

- Mathematical modelling regarding creep behaviour and volumetric change under temperature cycle has also been reported.
- Thermodynamic modelling has also been discussed to evaluate the preconsolidation pressure of fine grained soil using energy conservation and phase transformation.

The temperature induced volumetric deformation analysis of fine-grained soil reviewed in this study is reported mainly through laboratory experiments. It can be seen that laboratory experiments are usually carried out in a controlled environment condition, which in many cases could not be possible to replicate in a field scenario. Consequently, real-time thermal volumetric deformation analysis can be affected by harsh environmental conditions and climate change, which may or may not be possible on a laboratory scale. However, it should be appreciated that large scale studies and their interpretation are carried out in ongoing research and can assist in decision making by the radioactive waste management authority, in developing holistic solutions for researchers and in identifying unforeseen problems events to be found in the future.

CRediT authorship contribution statement

Md Azhar: Conceptualisation, Methodology, Formal analysis, Investigation, Writing- original draft, Writing- review and editing, Visualisation. **Somenath Mondal:** Conceptualisation, Methodology, Writing- review and editing, Visualisation, Supervision**.** **Anh Minh Tang:** Conceptualisation, Methodology, Writing- review and editing, Visualisation, Supervision. **Akhileshwar Kumar Singh:** Supervision.

Declaration of competing Interest

On behalf of all the authors of the submission, I declare that there are no conflicts of interests involved in this study.

Data availability

All data and models generated or used during the study appear in the submitted article.

Acknowledgment

The authors sincerely acknowledge the financial support provided by the Science and Engineering Research Board (SERB), Government of India in the form of Start-up Research Grant (SRG/2020/000167) at the Department of Civil Engineering, NIT Jamshedpur. In addition, the authors also wish to thank the department of civil engineering, NIT Jamshedpur for its immense help and guidance during the course of the study.

References

Abuel-Naga, H. M., Bergado, D. T., & Bouazza, A. (2007). Thermally induced volume change and excess pore water pressure of soft Bangkok clay. *Engineering Geology*, *89*(1–2), 144–154.

Abuel-Naga, H. M., Bergado, D. T., & Bouazza, A. (2008). Thermal conductivity evolution of saturated clay under consolidation process. *International Journal of Geomechanics*, *8*(2), 114–122.

Abuel-Naga, H. M., Bergado, D. T., Bouazza, A., & Ramana, G. V. (2007). Volume change behaviour of saturated clays under drained heating conditions: experimental results and constitutive modeling. *Canadian Geotechnical Journal*, *44*(8), 942–956.

Abuel-Naga, H. M., Bergado, D. T., & Lim, B. F. (2007). Effect of temperature on shear strength and yielding behavior of soft Bangkok clay. *Soils and Foundations*, *47*(3), 423–436.

Abuel-Naga, H. M., Bergado, D. T., Ramana, G. V, Grino, L., Rujivipat, P., & Thet, Y. (2006). Experimental evaluation of engineering behavior of soft Bangkok clay under elevated temperature. *Journal of Geotechnical and Geoenvironmental Engineering*, *132*(7), 902–910.

Alsherif, N. A., & McCartney, J. S. (2015). Nonisothermal behavior of compacted silt at low degrees of saturation. *Géotechnique*, *65*(9), 703–716.

Bag, R., & Rabbani, A. (2017). Effect of temperature on swelling pressure and compressibility characteristics of soil. *Applied Clay Science*, *136*, 1–7.

Bai, B., Nie, Q., Zhang, Y., Wang, X., & Hu, W. (2021). Cotransport of heavy metals and SiO2 particles at different temperatures by seepage. *Journal of Hydrology*, *597*, 125771. https://doi.org/10.1016/j.jhydrol.2020.125771

Bai, B., & Shi, X. (2017). Experimental study on the consolidation of saturated silty clay subjected to cyclic thermal loading. *Geomechanics and Engineering*, *12*(4), 707–721.

Bai, B., Yang, G., Li, T., & Yang, G. (2019a). A thermodynamic constitutive model with temperature effect based on particle rearrangement for geomaterials. *Mechanics of Materials*, *139*, 103180. https://doi.org/10.1016/j.mechmat.2019.103180

Bai, B., Yang, G., Li, T., & Yang, G. (2019b). A thermodynamic constitutive model with temperature effect based on particle rearrangement for geomaterials. *Mechanics of Materials*, *139*, 103180. https://doi.org/10.1016/j.mechmat.2019.103180

Bai, B., Zhou, R., Cai, G., Hu, W., & Yang, G. (2021). Coupled thermo-hydro-mechanical mechanism in view of the soil particle rearrangement of granular thermodynamics. *Computers and Geotechnics*, *137*, 104272. https://doi.org/10.1016/j.compgeo.2021.104272

Baldi, G., Hueckel, T., Peano, A., & Pellegrini, R. (1991). *Developments in modelling of thermohydro-geomechanical behaviour of Boom clay and clay-based buffer materials (Volume 1)*. https://inis.iaea.org/search/search.aspx?orig_q=RN:23030425

Baldi, G., Hueckel, T., & Pellegrini, R. (1988). Thermal volume changes of the mineral–water system in low-porosity clay soils. *Canadian Geotechnical Journal*, *25*(4), 807–825.

1239 Barry-Macaulay, D., Bouazza, A., & Singh, R. M. (2011). Study of thermal properties of a
1240 basaltic clay. In *Geo-frontiers 2011: Advances in Geotechnical Engineering* (pp. 480–
1241 487).
1242 Bozis, D., Papakostas, K., & Kyriakis, N. (2011). On the evaluation of design parameters
1243 effects on the heat transfer efficiency of energy piles. *Energy and Buildings*, *43*(4),
1244 1020–1029.
1245 Brandon, T. L., & Mitchell, J. K. (1989). Factors influencing thermal resistivity of sands.
1246 *Journal of Geotechnical Engineering*, *115*(12), 1683–1698.
1247 Burghignoli, A., Desideri, A., & Miliziano, S. (2000). A laboratory study on the
1248 thermomechanical behaviour of clayey soils. *Canadian Geotechnical Journal*, *37*(4),
1249 764–780.
1250 Campanella, R. G., & Mitchell, J. K. (1968). Influence of temperature variations on soil
1251 behavior. *Journal of the Soil Mechanics and Foundations Division*, *94*(3), 709–734.
1252 Casarella, A., Pedrotti, M., Tarantino, A., & Di Donna, A. (2021). A critical review of the
1253 effect of temperature on clay inter-particle forces and its effect on macroscopic thermal
1254 behaviour of clay. *International Conference of the International Association for*
1255 *Computer Methods and Advances in Geomechanics*, 608–615.
1256 Cekerevac, C., & Laloui, L. (2004). Experimental study of thermal effects on the mechanical
1257 behaviour of a clay. *International Journal for Numerical and Analytical Methods in*
1258 *Geomechanics*, *28*(3), 209–228.
1259 Cheng, W., Hong, P.-Y., Pereira, J.-M., Cui, Y.-J., Tang, A. M., & Chen, R.-P. (2020).
1260 Thermo-elasto-plastic modeling of saturated clays under undrained conditions.
1261 *Computers and Geotechnics*, *125*, 103688.
1262 Cho, W. J., Lee, J. O., & Chun, K. S. (1999). The temperature effects on hydraulic
1263 conductivity of compacted bentonite. *Applied Clay Science*, *14*(1–3), 47–58.

Coccia, C. J. R., & McCartney, J. S. (2012). A thermo-hydro-mechanical true triaxial cell for evaluation of the impact of anisotropy on thermally induced volume changes in soils. *Geotechnical Testing Journal*, *35*(2), 227–237.

Coccia, C. J. R., & McCartney, J. S. (2016a). Thermal volume change of poorly draining soils I: Critical assessment of volume change mechanisms. *Computers and Geotechnics*, *80*, 26–40.

Coccia, C. J. R., & McCartney, J. S. (2016b). Thermal volume change of poorly draining soils II: model development and experimental validation. *Computers and Geotechnics*, *80*, 16–25.

Cui, Y.-J., Tang, A.-M., Qian, L.-X., Ye, W.-M., & Chen, B. (2011). Thermal-mechanical behavior of compacted GMZ Bentonite. *Soils and Foundations*, *51*(6), 1065–1074.

Cui, Y. J., Sultan, N., & Delage, P. (2000). A thermomechanical model for saturated clays. *Canadian Geotechnical Journal*, *37*(3), 607–620.

De Moel, M., Bach, P. M., Bouazza, A., Singh, R. M., & Sun, J. O. (2010). Technological advances and applications of geothermal energy pile foundations and their feasibility in Australia. *Renewable and Sustainable Energy Reviews*, *14*(9), 2683–2696.

Delage, P., Sultan, N., & Cui, Y. J. (2000). On the thermal consolidation of Boom clay. *Canadian Geotechnical Journal*, *37*(2), 343–354.

Demars, K. R., & Charles, R. D. (1982). Soil volume changes induced by temperature cycling. *Canadian Geotechnical Journal*, *19*(2), 188–194.

Di Donna, A., & Laloui, L. (2015). Response of soil subjected to thermal cyclic loading: experimental and constitutive study. *Engineering Geology*, *190*, 65–76.

Di, P., Chang, D. P. Y., & Dwyer, H. A. (2000). Heat and mass transfer during microwave steam treatment of contaminated soils. *Journal of Environmental Engineering*, *126*(12), 1108–1115.

ESRIG, M. I. (1969). Some temperature effects on soil compressibility and pore water pressure. *Special Report*, *103*, 231.

Favero, V., Ferrari, A., & Laloui, L. (2016). Thermo-mechanical volume change behaviour of Opalinus Clay. *International Journal of Rock Mechanics and Mining Sciences*, *90*, 15–25.

Fox, P. J., & Edil, T. B. (1996). Effects of stress and temperature on secondary compression of peat. *Canadian Geotechnical Journal*, *33*(3), 405–415.

François, B., & Laloui, L. (2008). ACMEG- TS: A constitutive model for unsaturated soils under non- isothermal conditions. *International Journal for Numerical and Analytical Methods in Geomechanics*, *32*(16), 1955–1988.

Gangadhara Rao, M., & Singh, D. N. (1999). A generalized relationship to estimate thermal resistivity of soils. *Canadian Geotechnical Journal*, *36*(4), 767–773.

Ghaaowd, I., Takai, A., Katsumi, T., & McCartney, J. S. (2015). Pore water pressure prediction for undrained heating of soils. *Environmental Geotechnics*, *4*(2), 70–78.

Ghahremannejad, B. (2003). *Thermo-mechanical behaviour of two reconstituted clays*.

Golchin, A., Vardon, P. J., & Hicks, M. A. (2022a). A thermo-mechanical constitutive model for fine-grained soils based on thermodynamics. *International Journal of Engineering Science*, *174*, 103579.

Golchin, A., Vardon, P. J., & Hicks, M. A. (2022b). A thermodynamically consistent two surface/bubble thermo-mechanical model considering thermal and mechanical cyclic behaviour of fine-grained soils. *International Journal of Solids and Structures*, *254–255*, 111847. https://doi.org/10.1016/j.ijsolstr.2022.111847

Graham, J., Tanaka, N., Crilly, T., & Alfaro, M. (2001). Modified Cam-Clay modelling of temperature effects in clays. *Canadian Geotechnical Journal*, *38*(3), 608–621.

Hillel, D. (2013). *Fundamentals of soil physics*. Academic press.

1314 Hong, P. Y., Pereira, J.-M., Cui, Y.-J., & Tang, A. M. (2016). A two- surface
1315 thermomechanical model for saturated clays. *International Journal for Numerical and*
1316 *Analytical Methods in Geomechanics*, *40*(7), 1059–1080.
1317 Hong, P. Y., Pereira, J.-M., Tang, A. M., & Cui, Y.-J. (2013). On some advanced
1318 thermo- mechanical models for saturated clays. *International Journal for Numerical*
1319 *and Analytical Methods in Geomechanics*, *37*(17), 2952–2971.
1320 Hoseinimighani, H., & Szendefy, J. (2022). A Review on Mechanisms of Thermally Induced
1321 Volume Changes in Fine Soil. *Minerals*, *12*(5), 572.
1322 Houston, S. L., Houston, W. N., & Williams, N. D. (1985). Thermo-mechanical behavior of
1323 seafloor sediments. *Journal of Geotechnical Engineering*, *111*(11), 1249–1263.
1324 Hueckel, T. A. (1992). Water–mineral interaction in hygromechanics of clays exposed to
1325 environmental loads: a mixture-theory approach. *Canadian Geotechnical Journal*, *29*(6),
1326 1071–1086.
1327 Hueckel, T., & Baldi, G. (1990). Thermoplasticity of saturated clays: experimental
1328 constitutive study. *Journal of Geotechnical Engineering*, *116*(12), 1778–1796.
1329 Hueckel, T., & Borsetto, M. (1990). Thermoplasticity of saturated soils and shales:
1330 constitutive equations. *Journal of Geotechnical Engineering*, *116*(12), 1765–1777.
1331 Hueckel, T., & Pellegrini, R. (1992). Effective stress and water pressure in saturated clays
1332 during heating–cooling cycles. *Canadian Geotechnical Journal*, *29*(6), 1095–1102.
1333 Iwata, S., Tabuchi, T., & Warkentin, B. P. (2020). *Soil-water interactions: Mechanisms and*
1334 *applications*. CRC Press.
1335 Jarad, N., Cuisinier, O., & Masrouri, F. (2019). Effect of temperature and strain rate on the
1336 consolidation behaviour of compacted clayey soils. *European Journal of Environmental*
1337 *and Civil Engineering*, *23*(7), 789–806.
1338 Kadali, S., Lekshmi, S., Sharma, S., & Singh, D. N. (2013). Investigations to establish the

influence of the thermal energy field on soil properties. *Acta Geotechnica Slovenica*, *10*(2), 59–76.

Kemper, W. D., Maasland, D. E. L., & Porter, L. K. (1964). Mobility of water adjacent to mineral surfaces. *Soil Science Society of America Journal*, *28*(2), 164–167.

Krishnaiah, S., & Singh, D. N. (2004a). A device for determination of thermal properties of soil. *Journal of Testing and Evaluation*, *32*(2), 114–119.

Krishnaiah, S., & Singh, D. N. (2004b). Centrifuge modelling of heat migration in soils. *International Journal of Physical Modelling in Geotechnics*, *4*(3), 39–47.

Kuntiwattanakul, P., Towhata, I., Ohishi, K., & Seko, I. (1995). Temperature effects on undrained shear characteristics of clay. *Soils and Foundations*, *35*(1), 147–162.

Kutter, B. L., & Sathialingam, N. (1992). Elastic-viscoplastic modelling of the rate-dependent behaviour of clays. *Géotechnique*, *42*(3), 427–441.

Laguros, J. G. (1969). Effect of temperature on some engineering properties of clay soils. *Highway Research Board Special Report*, *103*.

Laloui, L., & Cekerevac, C. (2003). Thermo-plasticity of clays: an isotropic yield mechanism. *Computers and Geotechnics*, *30*(8), 649–660.

Laloui, L., & François, B. (2009). ACMEG-T: soil thermoplasticity model. *Journal of Engineering Mechanics*, *135*(9), 932–944.

Laloui, L., Leroueil, S., & Chalindar, S. (2008). Modelling the combined effect of strain rate and temperature on one-dimensional compression of soils. *Canadian Geotechnical Journal*, *45*(12), 1765–1777.

Lee, J., Kim, J.-T., Chung, I.-M., & Kim, N. W. (2010). Analytical solution for heat and moisture diffusion in layered materials. *Canadian Geotechnical Journal*, *47*(6), 595–608.

Li, Y., Dijkstra, J., & Karstunen, M. (2018). Thermomechanical creep in sensitive clays.

Journal of Geotechnical and Geoenvironmental Engineering, *144*(11), 4018085.

Lingnau, B. E., Graham, J., Yarechewski, D., Tanaka, N., & Gray, M. N. (1996). Effects of temperature on strength and compressibility of sand-bentonite buffer. *Engineering Geology*, *41*(1–4), 103–115.

Liu, E., & Lai, Y. (2020). Thermo-poromechanics-based viscoplastic damage constitutive model for saturated frozen soil. *International Journal of Plasticity*, *128*, 102683. https://doi.org/10.1016/j.ijplas.2020.102683

Loria, A. F. R., & Coulibaly, J. B. (2021). Thermally induced deformation of soils: A critical overview of phenomena, challenges and opportunities. *Geomechanics for Energy and the Environment*, *25*, 100193.

Ma, C., & Hueckel, T. (1993). Thermomechanical effects on adsorbed water in clays around a heat source. *International Journal for Numerical and Analytical Methods in Geomechanics*, *17*(3), 175–196.

Ma, Q. J., Ng, C. W. W., Mašín, D., & Zhou, C. (2017). An approach for modelling volume change of fine-grained soil subjected to thermal cycles. *Canadian Geotechnical Journal*, *54*(6), 896–901.

Maghsoodi, S., Cuisinier, O., & Masrouri, F. (2020). Thermal effects on mechanical behaviour of soil–structure interface. *Canadian Geotechnical Journal*, *57*(1), 32–47.

Maghsoodi, S., Cuisinier, O., & Masrouri, F. (2021). Non-isothermal soil-structure interface model based on critical state theory. *Acta Geotechnica*, 1–21.

Manthena, K. C., & Singh, D. N. (2001). Measuring soil thermal resistivity in a geotechnical centrifuge. *International Journal of Physical Modelling in Geotechnics*, *1*(4), 29–34.

Mašín, D., & Khalili, N. (2012). A thermo- mechanical model for variably saturated soils based on hypoplasticity. *International Journal for Numerical and Analytical Methods in Geomechanics*, *36*(12), 1461–1485.

McCartney, J. S., Jafari, N. H., Hueckel, T., Sánchez, M., & Vahedifard, F. (2019). Emerging thermal issues in geotechnical engineering. In *Geotechnical fundamentals for addressing new world challenges* (pp. 275–317). Springer.

McKinstry, H. A. (1965). Thermal expansion of clay minerals. *American Mineralogist: Journal of Earth and Planetary Materials*, *50*(1–2), 212–222.

Mitchell, J. K., & Soga, K. (2005). *Fundamentals of soil behavior* (Vol. 3). John Wiley & Sons New York.

Mohajerani, M., Delage, P., Sulem, J., Monfared, M., Tang, A. M., & Gatmiri, B. (2012). A laboratory investigation of thermally induced pore pressures in the Callovo-Oxfordian claystone. *International Journal of Rock Mechanics and Mining Sciences*, *52*, 112–121.

Mon, E. E., Hamamoto, S., Kawamoto, K., Komatsu, T., & Møldrup, P. (2014). Temperature effects on geotechnical properties of kaolin clay: simultaneous measurements of consolidation characteristics, shear stiffness, and permeability using a modified oedometer. *GSTF Journal of Geological Sciences (JGS)*, *1*(1).

Morin, R., & Silva, A. J. (1984). The effects of high pressure and high temperature on some physical properties of ocean sediments. *Journal of Geophysical Research: Solid Earth*, *89*(B1), 511–526.

Mu, Q., Ng, C. W., Zhou, C., & Zhou, G. G. (2019). Effects of clay content on the volumetric behavior of loess under heating-cooling cycles. *Journal of Zhejiang University-SCIENCE A*, *20*(12), 979–990.

Naidu, A. D., & Singh, D. N. (2004). Field probe for measuring thermal resistivity of soils. *Journal of Geotechnical and Geoenvironmental Engineering*, *130*(2), 213–216.

Ogawa, A., Takai, A., Shimizu, T., & Katsumi, T. (2020). Effects of temperature on consolidation and consistency of clayey soils. *E3S Web of Conferences*, *205*, 9010.

Ojovan, M. I., Lee, W. E., & Kalmykov, S. N. (2019). *An introduction to nuclear waste*

immobilisation. Elsevier.

Paaswell, R. E. (1967). Temperature effects on clay soil consolidation. *Journal of the Soil Mechanics and Foundations Division*, *93*(3), 9–22.

Padmakumar, G. P. (2013). *Laboratory investigations on heat migration through soil mass*. Doctoral thesis, Indian Institute of Technology Bombay, Bombay, India.

Pedrotti, M., & Tarantino, A. (2018). An experimental investigation into the micromechanics of non-active clays. *Géotechnique*, *68*(8), 666–683.

Pusch, R., Karnland, O., & Hökmark, H. (1991). The nature of expanding clays as exemplified by the multifaced smectite mineral montmorillonite. *Workshop on Stress Partitioning in Engineering Clay Barriers*.

Robinet, J.-C., Rahbaoui, A., Plas, F., & Lebon, P. (1996). A constitutive thermomechanical model for saturated clays. *Engineering Geology*, *41*(1–4), 145–169.

Romero, E., Villar, M. V, & Lloret, A. (2005). Thermo-hydro-mechanical behaviour of two heavily overconsolidated clays. *Engineering Geology*, *81*(3), 255–268.

Salager, S., Francois, B., El Youssoufi, M. S., Laloui, L., & Saix, C. (2008). Experimental investigations of temperature and suction effects on compressibility and pre-consolidation pressure of a sandy silt. *Soils and Foundations*, *48*(4), 453–466.

Schiffmann, R. L. (1971). A thermoelastic theory of consolidation. *Environmental and Geophysical Heat Transfer*.

Sherif, M. A., & Burrous, C. M. (1969). Temperature effects on the unconfined shear strength of saturated, cohesive soil. *Effects of Temperature and Heat on Engineering Behavior of Soils, Special Report, 103*, 267–272.

Shetty, R., Singh, D. N., & Ferrari, A. (2019). Volume change characteristics of fine-grained soils due to sequential thermo-mechanical stresses. *Engineering Geology*, *253*, 47–54.

Skipper, N. T., Refson, K., & McConnell, J. D. C. (1991). Computer simulation of interlayer

water in 2: 1 clays. *The Journal of Chemical Physics*, *94*(11), 7434–7445.

Sreedeep, S., Reshma, A. C., & Singh, D. N. (2005). Generalized relationship for determining soil electrical resistivity from its thermal resistivity. *Experimental Thermal and Fluid Science*, *29*(2), 217–226.

Stçepkowska, E. T. (1990). Aspects of the clay/electrolyte/water system with special reference to the geotechnical properties of clays. *Engineering Geology*, *28*(3–4), 249–267.

Sultan, N., Delage, P., & Cui, Y. J. (2002). Temperature effects on the volume change behaviour of Boom clay. *Engineering Geology*, *64*(2–3), 135–145.

Tang, A.-M., Cui, Y.-J., & Barnel, N. (2008). Thermo-mechanical behaviour of a compacted swelling clay. *Géotechnique*, *58*(1), 45–54.

Tarantino, A., Casarella, A., Pedrotti, M., Donna, A. Di, Pagano, A., Carvalho Faria Lima Lopes, B. de, & Magnanimo, V. (2021). Clay Micromechanics: an analysis of elementary mechanisms of clay particle interactions to gain insight into compression behaviour of clay. *International Conference of the International Association for Computer Methods and Advances in Geomechanics*, 183–201.

Towhata, I., Kuntiwattanaku, P., Seko, I., & Ohishi, K. (1993). Volume change of clays induced by heating as observed in consolidation tests. *Soils and Foundations*, *33*(4), 170–183.

Uchaipichat, A., & Khalili, N. (2009). Experimental investigation of thermo-hydro-mechanical behaviour of an unsaturated silt. *Géotechnique*, *59*(4), 339–353.

Vega, A., & McCartney, J. S. (2014). Cyclic heating effects on thermal volume change of silt. *Environmental Geotechnics*, *2*(5), 257–268.

Virdi, S. P. S., & Keedwell, M. J. (1988). Some observed effects of temperature variation on soil behaviour. *International Conference on Rheology and Soil Mechanics*, 336–354.

von Terzaghi, K. (1923). Die Berechnung der Durchassigkeitsziffer des Tones aus dem Verlauf der hydrodynamischen Spannungs. erscheinungen. *Sitzungsber. Akad. Wiss. Math. Naturwiss. Kl. Abt. 2A*, *132*, 105–124.

Wang, S., Vienna, L. S., Wu, W., Vienna, L. S., Zhang, D., & Kim, J. (2020). *Thermal-mechanical Behaviour of Saturated Soil : A Review Study SOIL MECHANICS AND*. *May*.

Yao, Y.-P., & Zhou, A. N. (2013). Non-isothermal unified hardening model: a thermo-elasto-plastic model for clays. *Géotechnique*, *63*(15), 1328–1345.

Yavari, N., Tang, A. M., Pereira, J.-M., & Hassen, G. (2016). Effect of temperature on the shear strength of soils and the soil–structure interface. *Canadian Geotechnical Journal*, *53*(7), 1186–1194.

Ye, W.-M., Wan, M., Chen, B., Chen, Y.-G., Cui, Y.-J., & Wang, J. (2012). Temperature effects on the unsaturated permeability of the densely compacted GMZ01 bentonite under confined conditions. *Engineering Geology*, *126*, 1–7.

Yin, Z.-Y., Zhu, Q.-Y., & Zhang, D.-M. (2017). Comparison of two creep degradation modeling approaches for soft structured soils. *Acta Geotechnica*, *12*(6), 1395–1413.

Yong, R. N. (1999). Overview of modeling of clay microstructure and interactions for prediction of waste isolation barrier performance. *Engineering Geology*, *54*(1–2), 83–91.

Zhang, Z. (2017). A thermodynamics-based theory for thethermo-poro-mechanical modeling of saturated clay. *International Journal of Plasticity*, *92*, 164–185.

Zhang, Z., & Cheng, X. (2017). A fully coupled THM model based on a non- equilibrium thermodynamic approach and its application. *International Journal for Numerical and Analytical Methods in Geomechanics*, *41*(4), 527–554.

Zhou, C., Fong, K. Y., & Ng, C. W. W. (2017). A new bounding surface model for thermal cyclic behaviour. *International Journal for Numerical and Analytical Methods in*

Geomechanics, *41*(16), 1656–1666.

Zhu, Q.-Y., & Qi, P. (2018). Numerical Modeling of Thermal-Dependent Creep Behavior of Soft Clays under One-Dimensional Condition. *Advances in Civil Engineering*, *2018*.

Zhu, Q.-Y., Yin, Z.-Y., Zhang, D.-M., & Huang, H.-W. (2017). Numerical modeling of creep degradation of natural soft clays under one-dimensional condition. *KSCE Journal of Civil Engineering*, *21*(5), 1668–1678.

Zhu, Q.-Y., Zhuang, P.-Z., Yin, Z.-Y., & Yu, H.-S. (2021). State parameter–based thermomechanical constitutive model for saturated fine-grained soils. *Canadian Geotechnical Journal*, *58*(7), 1045–1058.