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Review

Comprehensive overview of the macroscopic thermo-hydro-mechanical behavior of saturated cohesive soils

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Abstract. Understanding the effects of temperature on the hydro-mechanical behavior of geomaterials (i.e., soil and rock) has gained significance over the past three decades. This is due to new applications in which these materials are subjected to non-isothermal conditions. Examples of such applications include geothermal systems, nuclear waste disposal, and energy geo-structures. The analysis and design of such applications requires a thorough understanding of the macroscopic thermo-hydro-mechanical (THM) behavior of the geomaterials. Although various aspects of this behavior have been documented in the literature, a comprehensive overview of such behavior is lacking. This article presents such an overview of the macroscopically observed THM behavior of saturated cohesive soils.
Keywords: saturated cohesive soils, temperature, thermo-hydro-mechanical, energy geo-structures.

1. Introduction

The effects of temperature changes on the hydro-mechanical behavior of geomaterials (i.e., soil and rock) assumes importance in many applications. During the 1930's, perhaps the largest concern related to temperature effects on soils was the disturbance, in terms of variations of strength, and compressibility during the sampling, handling, or laboratory testing of specimens.

The subsequent incentive for improved understanding the thermal response of geomaterials was the necessity to solve a rather diverse set of problems such as 1) the improved stability of soil masses realized through their heating, 2) the interaction between soils and buried pipes transporting fluids or high-voltage cables at elevated temperatures, 3) the integrity of pavements and subgrades that are subjected to daily temperature variations, 4) the behavior of foundations (e.g., for furnaces) that are subjected to cyclic variations in temperature, 5) the drilling of deep wells, both offshore and on land, for hydrocarbon extraction, and 6) the effect of temperature variations on the behavior of clay liners, possibly in conjunction with geosynthetics, used in landfills.

In the 20th century, the major challenge that motivated an improved understanding of the thermo-hydro-mechanical (THM) behavior of soils was the disposal of radioactive waste in deep underground or offshore repositories, primarily consisting of low-permeability clay soils.

In the 21st century, an understanding of the aforementioned behavior was required in addressing two significant international socio-economic issues, namely the production and use of energy. During the last few decades, concerns over energy equity and security, excessive energy consumption rates, and issues related to climate change have intensified a worldwide effort to address such issues [1]. Increases in the price of fossil fuels have also accelerated the investigation of alternate energy sources. One such alternative, which is a renewable and possibly “clean” source of energy, is geothermal energy [2].

The basic premise underlying geothermal energy is to make use of thermal energy that is generated by the decay of radioactive elements within the ground or, more likely, by the storage of solar radiation. Several mechanical systems have been developed to facilitate the use of geothermal energy. These are commonly divided into two main categories based on their temperature range, namely 1) low thermal gradient zones known as Ground Source Heat Pumps [3], and 2) high thermal gradient zones known as Enhanced Geothermal Systems [4].

One of the newer advances in geothermal systems are the so-called energy geostructures. In such systems, structural elements, often associated with buildings, are used for heat transfer. Perhaps the most significant challenge associated with energy geostructures is the proper account for the effect of temperature on the response of the structure. Temperature changes typically induce variations in the settlement of the soil and structure, as well as in the forces acting within the structure. Consequently, supplementary considerations are required in the design of energy geostructures. The difference in response of the structure (typically thermo-elastic) and the soil (typically inelastic) also increases the complexity of the response. It follows that design of energy geostructures should be executed with consideration of the inelastic thermo-mechanical response of soils (e.g., the effect of temperature changes on consolidation and subsequent time dependent deformation of the soil), of the mobilization of side friction between structural elements and the soil that they are in contact with, and the additional loads due to degree of fixity at the bottom and top portions of the structure [5].

Since geothermal applications involve geomaterials (i.e., a soil or rock), understanding the behavior of such materials related to thermal loading is vital. Soils are heterogeneous particulate materials consisting of solid, liquid and gaseous phases. Consequently, they possess rather complex heat dissipation mechanisms such as conduction, convection, radiation, ion exchange, as well as vaporization and condensation. In unfrozen soils, heat transfer occurs by conduction and, to a lesser degree, by convection [5]. Thus, thermal properties of the soils related to phenomena such as heat capacity, thermal diffusivity and thermal conductivity are essential.

To further complicate the problem, other phenomena that can also influence the heat transfer. These include 1) the temperature dependence of the thermal parameters characterizing the geomaterial, 2) the effect of physical changes on the values of these parameters during mechanical and thermal loading, and 3) the different rates of heat transfer between the solid and liquid phases in a geomaterial [5].

The behavior of geomaterials is influenced by variables associated with the solid phase (e.g., mineralogy, size, shape, surface charge, composition, etc.), by the value of bulk (index) variables (e.g., void ratio, moisture content, relative density, etc.), and by the characteristics of the pore fluid. The relative importance of both micro- and macro-level variables is, however, dependent on the particular soil under consideration.

In saturated cohesive soils, the solid phase consists primarily of fine-grained constituents such as clays and silty clays. Clay particles are crystalline, plate-like, and have high specific surface (i.e., the surface area per unit mass) [6]. They have negative net surface charges and, due to breaks in their crystalline structure, a net positive charge at the plate edges [7]. Such edges may attract negative ions, dipole molecules, or may themselves be attracted to the negatively charged surface of another particle [6].

The properties of any cohesive soil are known to significantly be influenced by the presence of water [6], [8]. In a general cohesive soil, water is present in three forms, namely as 1) Ordinary ("free" or "bulk") liquid water, 2) Adsorbed water, and, particularly in low-porosity clays 3) An "organized" fluid. Macroscopically, the charged nature of clay particles thus influences such macroscopic measures of response as plasticity, shrinkage and swelling potential, permeability, compressibility, and strength parameters (e.g., the cohesion and friction angle) [6].

It light of the above discussion, it is evident that the macroscopic thermo-hydro-mechanical (THM) behavior of saturated cohesive soils should be interpreted in terms of the microscopic and physicochemical aspects associated with such geomaterials. This behavior is complicated by the discontinuous and heterogeneous nature of the microstructure (i.e., the geometric arrangement of pore spaces, particles, and particle groups, along with the interparticle forces present in the geomaterial), as well as by sundry physicochemical aspects. A rudimentary discussion of such aspects, as they apply to the THM behavior of saturated cohesive soils, was presented by Kaliakin et al. [9]; this was superseded by the updated and more in-depth treatment presented by Mashayekhi [10]. More thorough treatments of the microscopic and physicochemical aspects are given in books by Scott [11], Mitchell [6] and Mitchell and Soga [12].

When mathematically modeling the THM behavior of saturated cohesive soils, it is not yet possible to account for microscopic and physicochemical aspects in a manner that is both practical and robust, especially when simulating actual boundary value problems. Instead, in such models these aspects must be accounted for indirectly. This approach is largely based on the macroscopic THM behavior of saturated cohesive soils that is observed in laboratory tests. This article presents a comprehensive overview of such behavior. This overview updates and expands the earlier work of Kaliakin et al. [9] and Mashayekhi [10].

2. Methods

The subject matter related to the THM behavior of saturated cohesive soils has been available in the literature since the 1960's. As such, some of the older references were available only in hardcopy form. More current references were accessed from electronic databases such as Web of Science.

In searching through the available literature, only subject matter pertaining to saturated cohesive soils was considered. Herein, "cohesive soil" is restricted to clays and silty clays. Other fine-grained soils such as silts and clayey silts were not included in the literature reviewed for this article. This was done for reasons of brevity and because relatively little information is available regarding the THM behavior of such materials.

The search of pertinent subject matter included master theses and doctoral dissertations. Unfortunately, not all academic institutions provide easy access to such documents. Consequently, the theses and dissertations cited in this article do not constitute a complete collection such references.

A topic associated with the THM behavior of saturated cohesive soils is the variation, with temperature and possibly pressure, of the parameters used to characterize the thermal properties geomaterials. Such parameters include the density of the pore fluid and solid phase, the isotropic or anisotropic coefficients of thermal expansion for these two phases, the viscosity of the pore fluid, the hydraulic and thermal conductivities, and the heat capacity and specific heat of the solid and pore fluid phases. For brevity, the discussion of the aforementioned topic has been omitted from the present review article. A rather thorough overview of this topic has, however, been given by Mashayekhi in Chapter 4 of his dissertation [10].

3. Effect of Temperature on General Material Characteristics

Historically, the effect of temperature changes on the macroscopic behavior of cohesive soils was evaluated using standard soil mechanics laboratory experiments. To begin the present overview of the THM behavior of saturated cohesive soils, consider the effect of temperature on general material characteristics.

3.1 Atterberg Limits

The plasticity of cohesive soils is largely characterized by the values of the liquid limit (w_L) and plastic limit (w_P). As noted by by Mashayekhi [10], "the effect of temperature on the Atterberg limits has not been extensively investigated. This may, in part, be due to questions that have been raised regarding the reliability of the Atterberg limit tests". Consider the determination of w_P. In this test, an exchange of heat typically takes place between the soil and the hand of the individual performing the test [13], [14]. In addition, it is a well-known fact that the determination of w_P is more subjective and more prone to error than the determination of w_L [15], [16]. It is timely to note that measurements of w_L using the standard Casagrande device are more subjective and tend to be less repeatable than those obtained when using a cone penetrometer [17], [18].

The first experimental investigation of the effect that changes in temperature have on the Atterberg limits was that performed by Youssef et al. [19], who conclude that temperature increases lead to a reduction in the values of w_L and w_P. These findings were explained by the observation that, since temperature increases reduce the moisture content (w) of a soil, index properties dependent on w will likewise decrease under such conditions.

Similar reductions in w_L at elevated temperatures were reported for the three major clay mineral types (kaolinite, illite, and monmorillonite) by Laguros [13]. In this study, the results obtained for w_P were, however, rather inconsistent. Reductions in w_L and w_P with increases in temperature were also observed by Ctori [14] and by Wohlbier and Henning [20].

After reviewing the aforementioned results of Youssef et al. [19] and Laguros [13], Mitchell [21] concluded that the reductions in w_L with increases in temperature to be consistent with the reductions in strength observed at elevated temperatures (to be discussed in sub-section 3.3), especially since w_L is considered to be an indirect measure of shear strength of cohesive soils.

The results of more recent investigations seem to indicate that the *mineralogy* of the cohesive soil plays a significant role in determining the effect of temperature changes on the Atterberg limits. This observation stems from the rather contradictory results [22], [23], [24], [25] which indicated that such limits for kaolinite were essentially unaffected by changes in temperature. However, for kaolinite-bentonite mixtures, Reifer [23] found that w_L increased with increases in temperature. Such increases intensified as the percentage of bentonite was increased. The value of w_p was, however, essentially unaffected by these increases in temperature.

A somewhat more systematic study of the effects of mineralogy on the temperature dependence of w_L was carried out by Jefferson and Rogers [26]. In this study, it was determined that although w_L decreased with temperature for kaolinite, it increased for specimens that contained percentages of bentonite. Jefferson and Rogers [26] attributed these variations in w_L to the specific surface of the particular clay mineral. Thus, for clays with relatively high specific surface (e.g., bentonite), w_L will increase with increased temperature; in clays with low specific surface (e.g., kaolinite), it will slightly decrease for these same temperature increases.

3.2 Elastic Response

Although it is widely accepted that soils exhibit elastic response only at very low strains [27], [28] their elastic response has been rather extensively studied in the past. Of particular interest to the present development is elastic response under non-isothermal conditions.

The first experimental study of the effect of temperature changes on the response of clays, consisting of drained stress relaxation tests performed on saturated, undisturbed specimens, was performed by Murayama and Shibata [29]. Based on the results of this study, an analytical formulation in which the solid phase of the soil was idealized as being *viscoelastic* was proposed. Subsequent work by Murayama [30] extended this formulation by basing it on statistical mechanics (i.e., the rate process theory).

As summarized by Mashayekhi [10], Murayama and Shibata [29] conjectured that "the elastic resistance of clay particles is due, in part, to an imbalance of attractive and repulsive physicochemical forces, as well as to the viscous resistance due to adsorbed water between particles. Thus, although the elasticity of the skeleton is likely due to bending of the thin plate particles, it is also attributed to physicochemical interparticle forces. According to the Gouy-Chapman theory, temperature increases reduce the thickness of the diffuse double layer, thus also reducing the electric repulsion between clay particles" [31]. As a result, it follows that the elastic response of the clay tested will be a function of temperature changes.

Another important observation attributed to Murayama and Shibata [29] is the apparent existence of a "threshold" axial strain value (on the order of 1.0%), below which the elastic moduli will be unaffected by changes in temperature. Once this value is exceeded, temperature increases will cause the elastic moduli to *decrease* in magnitude. Murayama and Shibata [29] attributed this behavior to "fracture" of the solid phase once the aforementioned "threshold" axial strain value was exceeded.

With an eye towards using the mathematical theory of elastoplasticity to simulate the behavior of cohesive soils, investigations have been made into the effect of temperature changes on the size of the elastic domain in such simulations [32], [33], [34]. Based on these investigations, there appears to be general consensus that the size of this domain *decreases* with increases in temperature.

3.3 Shear Strength

One of the most fundamental material characteristics associated with geomaterials is their shear strength. The variation of shear strength of saturated cohesive soils, as a function of temperature, has been the subject of a number of studies [31], [35], [36], [37], [38], [39], [40], [41], [42], [43], [44], [45]. However, as noted by Mashayekhi [10], a general consensus regarding the effect of temperature variations on the shear strength of saturated cohesive soils has not been reached.

In their study of compacted cohesive soils, Hogentogler and Willis [35] found that increases in temperature *decreased* the strength of such materials. Lambe [31], on the other hand, postulated that an increase in temperature should result in an *increase* in shear strength. Citing the earlier findings of other investigators [35], [36], [37], Leonards [38] postulated that increases in temperature should cause a *reduction* in the shear strength of clays, thus contradicting Lambe's [31] hypothesis. From undrained axisymmetric triaxial tests performed on specimens of San Francisco Bay Mud, Mitchell [40] found that higher temperatures produced *lower* shear strength and higher excess pore pressure generation. Duncan and Campanella [42] acquired experimental data for soils first consolidated and then sheared under undrained axisymmetric triaxial conditions. Their findings also indicated that an increase in temperature causes a *reduction* in strength. Sherif and Burrous [44] investigated the undrained strength of kaolin subjected to undrained thermal loading and also found substantial *reductions* in strength. Figure 1 summarizes some of their findings.

Noble and Demirel [43] sheared specimens of a highly plastic alluvial clay at temperatures that were lower than the temperature at which they were consolidated. It was observed that the higher the consolidation temperature, the greater the shear strength at any given temperature. This observation, which agrees with the results of Laguros [13], is attributed to the greater decrease in volume (and thus void ratio) at higher consolidation temperatures. However, for a given consolidation temperature, the strength decreases in a regular manner with increasing test temperature. Leroueil and Marques [46] concluded that the undrained shear strength of saturated cohesive soils decreases by approximately 10% per 12^{o}C change in temperature.

In the case of undrained shearing that follows heating under *drained* conditions, the shear strength of soil tends to *increase*. This is explained by the fact that thermal compression of the soil during drained heating reduces the soil's void ratio and thus increases its shear strength.

By contrast, following undrained heating, the undrained shear strength tends to decrease. Hueckel and Baldi [34] attribute this phenomenon to the decrease in effective stress due to the increase in the generation of excess pore pressure during the process of undrained heating.

3.4 Anisotropy

The results of many experiments indicate that the mechanical properties of many natural and remolded soils are associated with certain preferred directions in space [47], [48]. Mathematically, such soils are thus classified as being *anisotropic* [49].

The anisotropy of cohesive soils is commonly attributed to preferred orientations of particles or clusters of particles, and possibly to the development of residual microstructural stresses [49]. Such preferred orientations result in both elastic and inelastic anisotropy. According to Dafalias [50], the consideration of elastic anisotropy is important for small strain levels. For larger strains, however, inelasticity is predominant, and elastic anisotropy can thus be neglected.

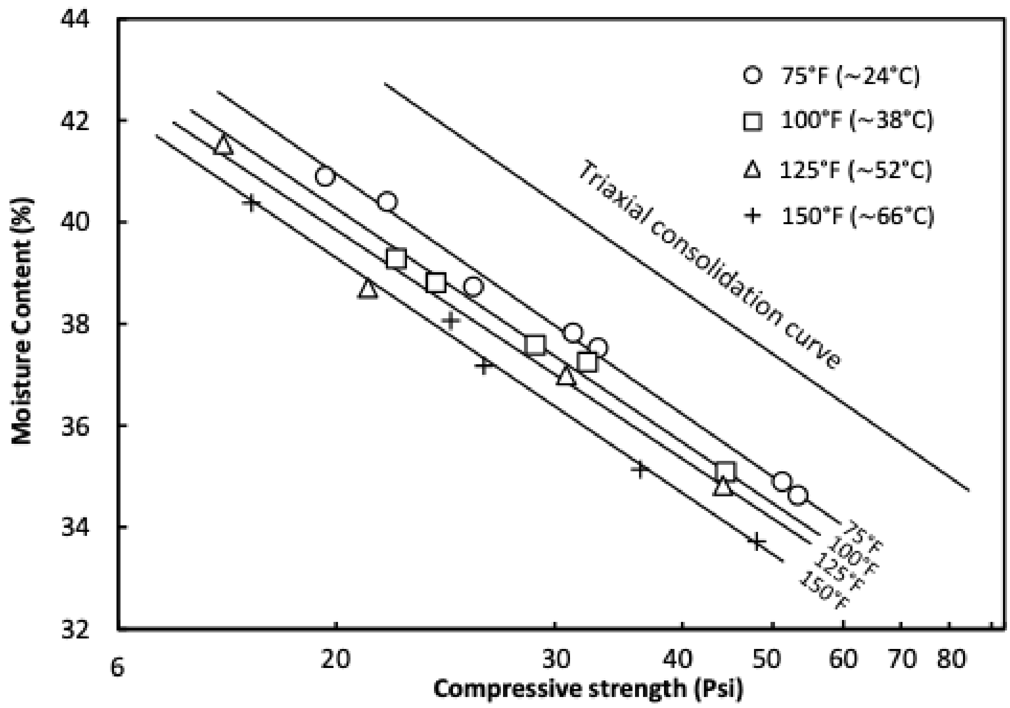

Figure 1 – Effect of temperature on the undrained shear strength of kaolinite in unconfined compression (after Sherif and Burrous [44])

Very few experimental studies have been performed to investigate the effect that changes in temperature have on the behavior of anisotropically consolidated cohesive soils. In such studies, the specimens are assumed to be transversely isotropic [49].

One important contribution in this area is the critical review by Hueckel and Pellegrini [51] of experimental data for two deep ocean clays [52], [53] that exhibited pronounced anisotropy. Hueckel and Pellegrini [51] noted that when these clays are isotropically compressed under isothermal conditions, they exhibit greater compression in the "vertical" direction (i.e., the direction perpendicular to the plane of isotropy) than in the "horizontal" direction (i.e., the direction parallel to the plane of isotropy). They attributed such behavior to the past loading history for these clays, which consisted largely of one-dimensional straining in the "vertical" direction, followed by unloading. Due to such a loading history, the clay particles and particle clusters tend to align themselves at right angles to the "vertical" direction, thus giving rise to a primarily horizontal orientation [54]. Upon subsequent laboratory re-loading, these clays tend to deform more easily in the "vertical" direction. When both of these clays were subjected to isotropic heating, the irreversible strain in the "horizontal" direction was larger than in the "vertical" direction. Although Hueckel and Pellegrini [51] could not definitively explain such results, their conjecture was that under isotropic conditions, lateral strains tend to be larger than for anisotropic (K_0) conditions.

Somewhat limited experimental evidence seems to indicate that the largest degree of anisotropy in cohesive soils is present during elastic deformation [55]. Then, under isotropic inelastic re-loading, it essentially disappears. Such behavior is consistent with the aforementioned hypothesis of Dafalias [50].

The effect of temperature changes on the behavior of overconsolidated anisotropic cohesive soils has received precious little attention in past studies. The only work in this area appears to be that reported by Virdi and Keedwell [56]. In this study, overconsolidated kaolin specimens were subjected to transient temperature distributions. The higher deviatoric stresses that resulted were attributed by Virdi and Keedwell [56] to stress-induced anisotropy, which tended to make the specimens stiffer in the axial ("vertical") direction.

In a more recent study, Russell Coccia and McCartney [57] employed a true triaxial apparatus to study the effect of anisotropy on thermally induced volume changes of a saturated silt tested under drained conditions. They concluded that such volume changes are not significantly influenced by stress-induced anisotropy.

4. Effect of Temperature on Behavior Under Drained Conditions

In drained tests, the drainage valves in the test apparatus remain open. The stresses are applied very slowly so that essentially no excess pore pressure is developed in the specimen.

The earliest investigation of the effect of temperature on the drained response of saturated cohesive soils appears to be that of Gray [58]. Based on the results of this, as well as subsequent investigations, there is general consensus that at constant levels of stress under drained conditions, normally consolidated (NC) and lightly overconsolidated (OC) cohesive soil specimens compress when heated. Temperature decreases typically cause such specimens to swell; pore fluid is adsorbed by such specimens [42], [59]. The aforementioned compression in heated NC specimens appears to be independent of the material state but to be dependent on the mineralogy and porosity of the specimen [52].

The behavior of saturated NC specimens subjected to cycles of heating and cooling under drained conditions at constant stress states has also been investigated. During such thermal cycling, the behavior appears to be inelastic; that is, some part of the axial strain is irreversible [42], [60], [61]. The associated permanent decrease in void ration during such thermal cycling appears to be independent of the effective confining stress [25], [43], [60], [62], [63], [64], [65]. Specimens subjected to such temperature cycles behave as if they were overconsolidated [25], [33], [34], [42], [63], [66].

The magnitude of the volume change under drained conditions has been reported to reduce with increases in the overconsolidation ratio (OCR) [34], [53], [59], [60], [61], [63], [67]. The rate of this reduction seems to be a function of the stress level (typically represented in terms of the OCR [60]). In the case of cohesive soils of low plasticity, such reductions tend to be rather pronounced. For cohesive soils with high plasticity, the rate of such reduction appears to be less significant. For low porosity clays, volume contraction becomes dilative for high OCRs [25], [61].

When NC and OC cohesive soil specimens are cooled under drained conditions, the associated volumetric strains are generally independent of the stress state, thus implying elastic response. The volumetric strains generated during such cooling do not necessarily have the same sense. For example, in the case of illitic clays [21], [62], [63], and for Pontida silty clay [61], the volumetric strains are rather pronounced and dilative (and thus negative). In the case of soft Bangkok clay [65] and Boom clay [61], the volumetric strains generated during cooling are quite small as compared to the strains produced during heating. By contrast, when Spanish clay [53] is cooled, the volumetric strain generated tends to be compressive (and thus positive). As noted by Mashayekhi [10], "the aforementioned variations in the sign of the recoverable volumetric strain have, in some cases, necessitated the use of rather elaborate functional forms for the coefficients of thermal expansion for cohesive soils".

4.1 Compressibility Characteristics

The compressibility of soils is typically studied using oedometer and axisymmetric triaxial devices under drained conditions. Several experimental investigations have examined the effect that temperature cycling has on volume change characteristics in such tests that are carried out constant levels of confining stress. These investigations used a standard laboratory oedometer [25], [33], [63], [66], [68], [69], [70], a standard triaxial cell with isotropic confining pressure [42], [53], [67], and a special triaxial test cell [59], [60]. The tests performed using such devices tests were carried out at varying (constant) stress levels and temperatures.

The results of such tests indicate that, when subjected to temperature increases, NC and lightly OC specimens consolidate; temperature decreases lead to the swelling of such specimens. As a result, the soil behaves as if it were overconsolidated [34], [42], [63], [66]. Consequently, the heating of such specimens at constant mean normal effective stress produces a quasi-preconsolidation behavior similar to that associated with long term consolidation [25], [33], [42], [63].

For NC specimens, the magnitude of irreversible volume change generated by heating, while material specific, was shown to be essentially *independent* of stress level. In particular, when subjected to temperature increases, illitic clays [62], [63] and smectites [43] exhibited a clear reduction of void ratio. For clays possessing high volume fractions of silt this was not, however, the case, as such soils did not undergo appreciable void ratio reductions when heated [63].

By contrast, in OC cohesive soils the thermally induced volume changes are typically independent of stress level. They are, however, a function of the stress history, which is typically quantified by the OCR.

The effect of temperature cycling on the volume changes in OC cohesive soils has also been investigated [34], [53], [60], [61], [63], [67]. As summarized by Mashayekhi [10], "the consensus appears to be that compared to normally consolidated specimens, thermally induced volumetric strains are relatively small, depend on the OCR [34], [63], [71], and are largely reversible". In support of this summary is the observation that the amount of compression exhibited under drained conditions appears to decrease with increasing OCR. In the case of clays with low porosity, the behavior of OC specimens was actually found to be *thermoelastic* [34], [53], [61]. Furthermore, when heated, highly OC specimens of such clays dilated. This is partly explained by the fact that when such clays are heated, the pore fluid expands to a greater degree as compared to the solid phase. Since the permeability of clays is relatively low, this fluid cannot quickly drain from the pores, resulting in dilative behavior.

4.2 Compression and Swell/Recompression Indices

The compressibility of soils is typically characterized by the values of compression index C_c and the swell/recompression index C_r. Based on the available experimental findings, there does not appear to be consensus regarding the effect of temperature increases on the values of C_c and C_r (or on their critical state counterparts λ and κ, respectively). The following findings are presented in support of this conclusion.

4.2.1 The Compression Index

Initially, most of the experimental studies were performed at relatively low stress levels [13], [63], [66], [69], [72]. The results of these studies indicated that the value of C_c changed with temperature. Such changes were dependent on the stress level [72], the soil type, and on temperature [63], [66], and pressure [63].

At higher confining pressures, the value of C_c was found to be practically temperature independent [32], [42], [60], [62], [67], [68], [73], [74], [75]. Increases in temperature tended to displace the virgin compression line in void ratio versus logarithmic of stress space to lower void ratios; the slope of the line, which is equal to C_c, remained essentially the same. These findings were supported by the results of a subsequent analytical study that was based on the Gouy-Chapman theory [76].

The effect of stress history on the thermally induced variation in C_c was also studied by Habibaghi [69]. Based on the result of this experimental program, it was concluded that for NC clays at stresses greater than 300 kPa, the C_c values were temperature independent. However, for OC specimens, such values were dependent on the temperature. Habibaghi [69] noted, however, that this dependency seemed to diminish with reductions in the OCR. Consequently, the effect of temperature on the value of C_c appears to be a function of the stress history.

4.2.2 The Swell/Recompression Index

A few researchers [13], [34], [42] have reported that the value of C_r was essentially temperature independent. Other findings [32], [73], [77], however, showed that C_r was an increasing function of temperature.

This assessment of the temperature dependence of C_r appears to depend on whether the *swell* or the *recompression* portion of the response was considered in a given experimental study. This conjecture is largely due to the findings of Campanella [62], who attributed the observed variation in C_r to hysteresis effects (which is related to changes in a soil's compressibility), and to differences in unloading-reloading response. In support of this conclusion, is the fact that the tests of Eriksson [32] were performed on the recompression portion of the void ratio versus logarithm of stress curve. By contrast, the test results of Laguros [13] were, however, obtained from the swell portion.

The apparent discrepancies associated with the dependence of C_r on temperature were investigated by Abuel-Naga et al. [67], who performed tests involving both the swell and recompression phases of the response. In tests that made use of the recompression line, C_r was found to be temperature dependent, while in those involving the swelling line, C_r was found to be temperature independent. These results led Abuel-Naga et al. [67] to conclude that, since the recompression portion of the response involves irreversible deformations, the proper assessment of the temperature dependence of C_r must make use of the swell portion.

In an attempt to explain the elastic response associated with the swell portion, Mashayekhi [10] notes that "from a micromechanical viewpoint, temperature increases will tend to reduce interparticle bond strength and thus increase the elastic deformations. However, under drained conditions, the same temperature increase will cause a reduction in void ratio, thus compensating for the reduction in bond strength. The overall response may thus be largely temperature independent".

4.3 Preconsolidation Stress

The effective preconsolidation stress (σ_p') is thought to be a "yield limit", that under both isotropic and anisotropic stress states, separates elastic response from inelastic response [46]. Based on the results of oedometer and constant rate of strain axisymmetric triaxial tests, it has been ascertained that, at a given value of void ratio, increases in temperature cause σ_p' to decrease [32], [33], [34], [65], [74], [77], [78], [79], [80]. The change in σ_p' appears to be greater for cohesive soils with increased clay content, as well as for those characterized by larger values of w_L [33]. Figure 2 summarizes some experimental results related to the effect of temperature on σ_p'.

The reduction in σ_p' appears to depend on the stress ratio, $\eta = q/p'$, where q is the deviator stress and p' is the mean normal effective stress. The higher the stress ratio, the smaller the reduction of σ_p' [67].

5. Effect of Temperature on Behavior Under Undrained Conditions

In undrained tests, the drainage valves in the test apparatus are closed following consolidation. The stresses subsequently applied to a specimen generate excess pore pressures, which are recorded.

For NC and lightly OC saturated cohesive soils tested under undrained conditions at a constant level of total stress, temperature increases, even minor ones, lead to the development of excess pore pressure and thus to a decrease in effective stress [30], [45], [56], [81], [82]. Decreases in temperature cause the excess pore pressure to decrease.

The aforementioned changes in excess pore pressure are typically attributed to sundry physicochemical effects, and to the difference in thermal expansion characteristics associated with the pore fluid and solid phases. The *magnitude* of excess pore pressure change is thought to be a function of the 1) path followed in thermal loading, 2) temperature increments used in a given test, 3) compressibility of the soil being tested and, 4) thermal expansion properties of the pore fluid and solid phase [45].

Investigations of thermal cycling indicate that the resulting pore pressure changes are irreversible [81], [83], [84], [85]; the pore pressure response is characterized by hysteresis loops that may [81] or may not [63], [83], [84], [85], [86] be closed.

Campanella and Mitchell [42] hypothesized that continued temperature cycling and the dissipation of "secondary compression tendencies at high temperatures" would result in excess pore pressure-axial strain hysteresis loops that were *closed*. In support of this conjecture was the lack of residual excess pore pressure for specimens of illite tested by Campanella and Mitchell [42]. Before being subjected to undrained conditions, these specimens were subjected to temperature changes under drained conditions.

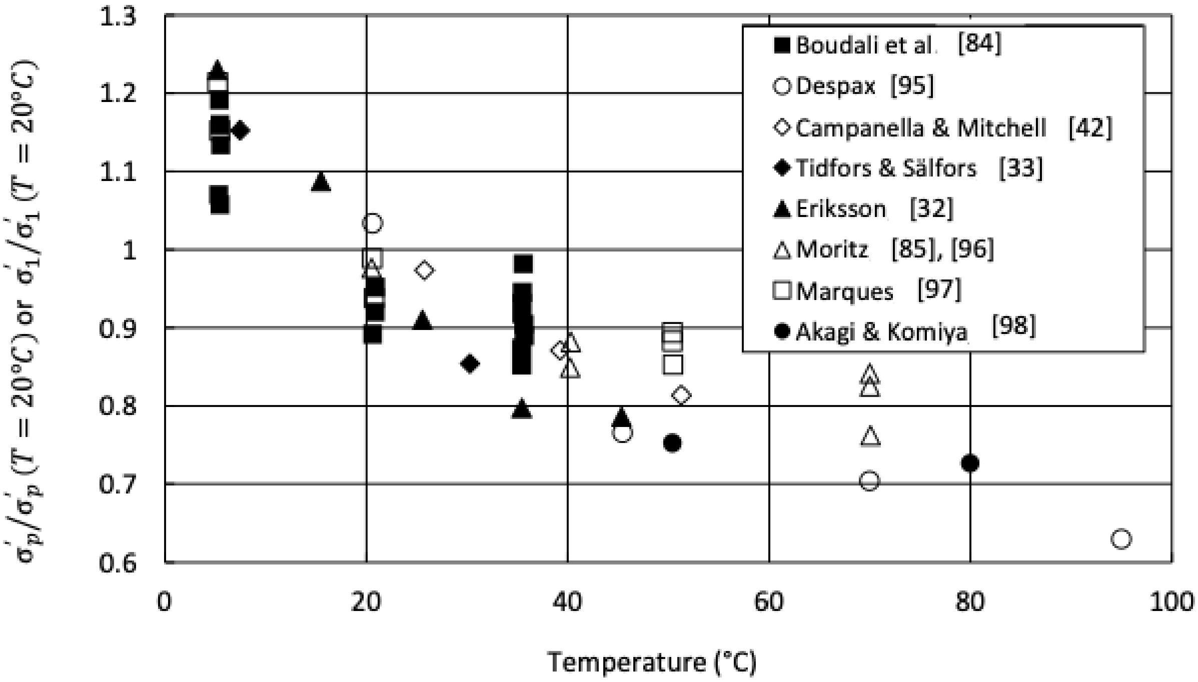

Figure 2. Effect of temperature on the normalized preconsolidation stress (after [46]).

Two important observations related to temperature dependent response of saturated cohesive soils under undrained conditions were made by Plum and Esrig [63]. First, they noted that hysteresis loops would only be expected to occur following several thermal loading-unloading cycles or after variations in temperature. Secondly, they noted that a closed hysteresis loop would be developed only when the soil is somewhat OC. This second observation leads credence to earlier results presented by Mitchell and Campanella [81] that produced closed hysteresis loops, for this study involved two OC cohesive soils.

6. Time Dependent Behavior

The effect of temperature changes on the time dependent behavior of saturated cohesive soils is somewhat complicated by the fact that it is rather difficult to isolate temperature effects from time effects [10]. When discussing the macroscopically observed time-related behavior of saturated cohesive soils, this subject is commonly divided into the following categories: a) constant stress creep, b) constant strain relaxation, and c) strain-rate effects. Such a division is admittedly somewhat artificial. Indeed, it is the feeling of many researchers that the same micromechanical mechanisms may be responsible for all three of these phenomena [9], [87]. Nevertheless, it successfully separates the different types of time dependent behavior for specific loading conditions, and it aligns with the manner in which this subject has been presented in past literature. Consequently, it facilitates the presentation of subject matter in this section.

6.1 Constant Stress Creep

Of the aforementioned categories related to time dependent behavior, the creep of saturated cohesive soils has been studied more comprehensively than relaxation or strain-rate effects. Constant stress creep is defined as the time dependent development of shear- and/or volumetric deformations under a constant state of total stress. Creep occurs under both drained (constant effective stress), as well as undrained (constant volume) conditions. It is used to depict a number of field applications in which a constant load is applied to a soil deposit (e.g., from an embankment built on the ground surface), and causes either undrained or drained creep in the supporting soil.

In what appears to be one of the earliest systematic investigations of the effect of temperature on the time dependent behavior of cohesive soils, Murayama [30] performed creep tests, under drained conditions, on NC specimens of an alluvial marine clay. Based on the results of these tests, at a given temperature, the creep strain rate was determined to be linear with the logarithm of time. Although increases in temperature caused the strain rate to increase, it remained approximately linear with the logarithm of time.

A similar series of drained creep tests was performed by Campanella [62], albeit on specimens of illite. For a given (constant) level of total stress, and under isothermal conditions, only relatively small volume changes were measured during creep. If, however, a specimen's temperature was increased after consolidation, the volume changes measured during creep increased rather appreciably. Campanella [62] also found that, at a given time during creep, specimens consolidated at higher temperatures exhibited higher strain rates. Unlike the findings of Murayama [30], Campanella [62] noted that his experimental results were best represented by a linear relationship between the logarithm of strain rate versus the logarithm of time.

The subsequent drained creep tests performed by Virdi and Keedwell [56] were noteworthy because they not only maintained a constant temperature during the tests but also imposed a transient temperature state consisting of two full cycles. Based on the results of these tests, Virdi and Keedwell [56] noted that temperature increases reduced the volume of their specimens. Temperature decreases had the reverse effect. In all of the drained creep tests performed, the effect of temperature changes on the volumetric and axial strain rates was more pronounced at higher deviator stress levels. Virdi and Keedwell [56] also noted that, in response to temperature variations, the volume of pore fluid expelled from, or absorbed into a specimen appeared to be proportional to the moisture content.

The topic of secondary compression (drained volumetric creep [87]) is very often included in a discussion of consolidation. The effect of temperature increases on secondary compression has thus been the focus in several investigations [25], [42], [63], [88], [89]. The results of these investigations have not, however, been in general agreement. For example, the conclusion reached by Plum and Esrig [63] was that if sufficient time is allowed for thermally induced volume changes to occur, the rate of secondary compression will only slightly be affected by temperature increases. This conclusion, however, contradicted the findings of other investigators [25], [42], [88], [89]. As noted by Mashayekhi [10], "it is possible, however, that in the latter investigations insufficient time was allowed for thermally induced volume changes to fully manifest themselves".

In their study of thermal effects on secondary compression, Towhata et al. [25] increased the temperature at different times during the secondary phase of consolidation. The found that such actions accelerated the amount of volume change generated during this phase.

Burghignoli et al. [75] also performed thermal consolidation tests on NC specimens. They found that in heated specimens the variation in void ratio during secondary compression was relatively large. Burghignoli et al. [75] concluded that during the secondary phase of consolidation of NC clays, creep deformations were dominant. When such specimens were cooled, they did not exhibit any appreciable secondary compression. In their tests on OC specimens, Burghignoli et al. [75] reported essentially *identical* behavior during both heating and cooling phases. Such results underscore the importance of stress history when studying temperature and time dependent processes.

It is timely to note that in discussing the somewhat controversial subject temperature effect on secondary compression, there appears to be consensus that the overconsolidation of cohesive soils due to cooling reduces the rate of secondary compression [42], [63], [75].

Relatively few experimental studies have investigated the effect of temperature on creep of cohesive soils under undrained conditions. Before subjecting them to undrained creep, Houston et al. [45] consolidated specimens at constant temperatures under drained conditions. They found that, with increasing consolidation temperatures, undrained creep strain rates decreased. Houston et al. [45]

attributed such behavior to the greater degree of densification and higher stiffness that is obtained during consolidation at elevated temperatures. During undrained creep, specimens tested at higher temperatures were more likely to fail under undrained conditions.

Similar to the aforementioned drained creep tests, Virdi and Keedwell [56] also performed undrained creep tests in which a constant temperature was maintained and during which a transient temperature state consisting of two full cycles was imposed. In these tests, four different (constant) values of deviator stress (30, 50, 70, and 90% of the failure stress under isothermal conditions) were maintained. Virdi and Keedwell [56] found that the magnitude of excess pore pressures generated in a specimen increased with temperature; when cooled, such pore pressures decreased. During temperature increases, the axial strain in the specimens increased. For specimens subjected to thermal cycling, such increases decreased with each cycle. When the temperature of a specimen was decreased, the axial strain did not decrease appreciably; this was particularly true at the higher deviator stress levels.

6.2 Constant Strain Relaxation

Constant strain relaxation tests attempt to duplicate the behavior of loaded soil masses whose dimensions in-situ remain essentially unchanged. Prime examples of such scenarios are problems involving soil-structure interaction in which the presence of the structure prevents excessive deformation of the soil mass, or in the case of objects penetrating into soil that are held stationary for some period of time (e.g., a cone penetration test held stationary while load and pore pressure changes are measured). In such problems, the reduction (relaxation) of stresses in the soil mass is of primary interest.

As noted by Mashayekhi [10], “data concerning the effect of temperature on constant strain stress relaxation is quite scarce”. Indeed, the only extensive experimental study of this subject was that performed by and Murayama and Shibata [29]. In this study, stress relaxation tests were performed in axisymmetric triaxial compression under undrained conditions. Based on the results of such tests, Murayama and Shibata [29] found that, during relaxation, the deviator stress decreased linearly with the logarithm of time and approached a non-zero limiting value. The results of these tests also showed that three initials as well as the fully relaxed levels of deviator stress decrease with increases in temperature.

6.3 Strain-Rate Effects

As in the case of constant stress creep, there have been two primary bodies of work performed for this subject. First are *undrained* shear tests using axisymmetric triaxial, direct simple shear, or other devices in which a shear stress can be applied to a specimen. Secondly are *drained* one-dimensional consolidation tests using either calculated strain rates in conventional, incrementally loaded consolidation tests or in the constant rate of strain consolidation test. Unfortunately, to date no systematic studies of the effect that temperature changes have on the behavior of saturated cohesive soils subjected to different rates of loading have been performed. In such studies, the THM behavior of saturated cohesive soils is a function of not only stress, strain, and temperature, but also of the strain-rate.

7. Discussion

This article presented a comprehensive overview of the macroscopically observed thermo-hydro-mechanical behavior of saturated cohesive soils. Several findings related this behavior are noteworthy. These are summarized below.

Based on the results of several experimental studies, it appears that clay *minerology* assumes importance in determining the temperature dependence of the liquid limit (w_L). Due to the subjective nature of the plastic limit test, no definitive conclusions have been reached regarding its temperature dependence.

The effect of temperature on w_L is explained by the fact that mineralogy is intimately related to the specific surface associated with a particular clay mineral [6], [7]. In kaolinites, which have the lowest specific surface among clay minerals, the adsorbed water layer is relatively thin and the interparticle bonding would thus be realized mostly through solid bonds. When heated, kaolinite particles lose some of the adsorbed water and the interparticle forces are influenced by sundry physicochemical effects that cause some of the bonds to break. This results in, at most, a slight reduction in w_L. In smectites, which possess the largest specific surface among clay minerals, the adsorbed water layers are relatively thick. Since experimental results indicate that the heating of smectite-rich soils causes w_L to increase, this implies that heating has caused an increase in the thickness of the adsorbed water layer. This, however, contradicts the observation [6] that changes in temperature have only a negligible on the thickness of the diffuse double layer. Clearly, additional research regarding the temperature dependence of w_L is warranted.

The thermal response of saturated cohesive soils is affected by the soil type and, in the case of clays, by the minerology of the soil. Consequently, for heating under drained conditions, higher volume change will be observed for soils with higher plasticity indices. For heating under undrained conditions, higher excess pore pressures will be observed in such soils.

Based on the results of a somewhat limited number of experimental studies, temperature increases reduce the magnitude of elastic moduli. There is also general consensus that the extent of the elastic domain for a saturated cohesive soil decreases with increases in temperature.

A general consensus has not been reached regarding the effect of temperature on the shear strength of saturated cohesive soils. The strength of such soils can increase or decrease with changes in temperature. In certain cases, however, the shear strength appears to be unaffected by temperature changes. It is the feeling of some researchers [90] that this apparent confusion is attributed to the lack of consideration of the thermal and mechanical history of the soil prior to failure. This notwithstanding, clearly, this subject requires additional experimental investigation.

The effect that temperature increases have on the internal friction angle is rather inconclusive. For NC clays, temperature changes have little effect on the effective friction angle, at least for values lower than about 50°C [46]. Depending on the specific conditions maintained during a test, increases in temperature can cause the magnitude of the friction angle at critical state either to slightly increase or to decrease.

Anisotropy does not appear to be induced in saturated cohesive soils by temperature increases, though pertinent experimental results are relatively scarce. Increases in temperature can reduce the degree of anisotropy in such soils.

In normally consolidated saturated cohesive soils subjected to temperature increases under drained conditions, the resulting reduction in void ratio appears to be independent of the stress state. The magnitude of this reduction depends on the predominant clay mineral present in the soil, as well as its moisture content.

Based on the available experimental results, there does not appear to be consensus regarding the effect of temperature increases on the values of the compression index C_c and on the swell/re-compression index C_r. Except for very low confining stresses, the value of C_c appears to be temperature independent. If it is measured from the re-compression portion of the void ratio versus logarithm of stress plot, the value of C_r will likewise be essentially independent of temperature.

The effective preconsolidation stress (σ_p') is affected by changes in temperature. Increases in temperature cause to σ_p' to decrease. The rate of this decrease appears to a function of the material characteristics of the saturated cohesive soil.

Based on the results of several experimental studies, the excess pore pressures generated under undrained conditions by temperature increases depend on the magnitude of the applied temperature increment, on the stress state in the soil, and on the magnitudes of the thermal expansion coefficients for the fluid and solid phases.

During undrained heating, positive excess pore pressures are generated in normally consolidated specimens. The magnitude of such pore pressures decreases with increasing degree of overconsolidation.

When specimens are cooled under undrained conditions, the excess pore pressures decrease. In normally consolidated specimens, temperature cycles produce irreversible changes in excess pore pressure. In overconsolidated specimens, such pore pressures appear to be reversible.

Experimental investigations of temperature cycling under undrained conditions seem to indicate that the resulting changes in excess pore pressure are irreversible, thus implying that effective stresses will likewise be affected. During such thermal cycling, hysteresis loops shall be formed, though necessarily during the first cycle. In addition, the amount of residual excess pore pressure that is generated seems to decrease with the number of cycles, for the soil is becoming increasingly overconsolidated.

In constant stress creep tests performed on normally consolidated and on lightly overconsolidated cohesive soils, increases in temperature result in increased axial strain rates and increased excess pore pressures.

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