Cyclic thermomechanical response of fine-grained soil-concrete interface for energy piles applications

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Abstract

Understanding the behaviour of soil-structure interfaces is critical for addressing the analysis and design of energy geostructures. In this study, the interface failure mechanism of energy piles (where a shear band is detached from the surrounding soil that behaves under oedometric conditions) is experimentally analysed in laboratory for saturated conditions. The choice of material (clayey soil and concrete), temperature range, and stress level is based on conditions that are likely to be encountered in practice. Specifically, cyclic thermal tests under constant vertical effective stress in oedometric conditions as well as constant normal stiffness (CNS) interface direct shear tests (in which samples have been subjected to thermal cycles between 10 and 40 °C) are presented. From a practical perspective, the results show very low volumetric strain variations and negligible effects on shear strength. The volumetric aspects do not appear to have significant impact on the shear resistance of the interfaces against cyclic thermal loads. Fundamental insight on the effects of thermal cycles on the concrete-soil interface behaviour which are relevant to energy piles are presented. In addition, the
proposed interpretation procedure provides a basis for the standardisation of thermomechanical testing in geotechnical engineering.

**Keywords:** energy piles, soil-concrete interface, thermomechanical behaviour, laboratory testing, cyclic load.

### 1. Introduction

In geotechnical engineering, the study of non-isothermal soil behaviour dates back to the 1950s and 1960s. During that time, the focus was on understanding the effects of temperature on engineering properties, generating thermal pressurisation for saturated soils, and soil sampling. From the 1970s to the 1990s, several new geotechnical applications that required understanding of the behaviour of soils subjected to temperature changes (e.g., design in permafrost regions, nuclear waste storage, buried high voltage electrical cables) were introduced. Most of the available research and literature that started in those years and developed until more recent times focuses mainly on the design of repositories for radioactive waste disposal in deep geological media and the consequent development of advanced thermo-hydro-mechanical constitutive models to provide long-term nuclear waste management solutions (e.g., Laloui et al. 2008). Starting from the 2000s, there has been an interest in environmentally friendly technologies capable of addressing climate change challenges, such as energy geostructures and other thermoactive ground structures (McCartney et al. 2019; Laloui and Rotta Loria 2019). Understanding the behaviour of soil-structure interfaces is critical to address the analysis and design of energy geostructures, e.g., the analysis of
energy pile capacity subjected to cyclic thermal loads. As a result of their multifunctional roles, energy piles are exposed to daily and seasonal temperature variations during their lifetime. Therefore, possible effects of cyclic thermal load need to be investigated, e.g., thermal effects resulting in volumetric response of soil at the pile–soil interface at different temperatures, as well as mechanical effects (i.e., displacements at the interface) resulting from cyclic axial dilation and contraction of the concrete during heating and cooling. Both may affect the normal stress at the soil-pile interface which may lead to changes in the soil-pile shear resistance.

The analysis presented in this paper reproduces the interface failure mechanism of energy piles in the laboratory for saturated conditions. A schematic representation of the failure mechanism of the pile-soil interface which consists of large localised strains concentrated in a thin layer around the pile, i.e., the interface, is shown in Figure 1. Figure 1 also illustrates the modelling approach adopted in the laboratory to reproduce the failure mechanism where a shear band is detached from the surrounding soil that behaves under oedometric conditions. The presence of the surrounding soil is replicated by a spring stiffness which accounts for the effects of the volume changes (i.e., normal stress variations).

Therefore, the basic aspects that constitute the failure mechanism, i.e. the volumetric response of the soil, under oedometric conditions, and the shear response of the soil-concrete interface to cyclic thermal loads need to be addressed individually and combined to fully describe and better understand the failure mechanism at the pile-soil interface.
For this purpose, a critical overview of the current framework adopted for analysing phenomena related to the thermovolumetric behaviour of fine-grained soils and the thermomechanical behaviour of interfaces is first presented in the respective introductory sections. The study then focuses on the response of fine-grained soils and soil-concrete interfaces for energy piles applications where drained conditions are assumed upon thermal loading (Mimouni and Laloui 2015; Sutman 2016; Rotta Loria and Laloui 2017a) and temperature values are between 2 and 45 °C (Laloui and Rotta Loria 2019). The experimental programme aims to investigate the thermomechanical effects on the response of fine-grained soils and soil-concrete interfaces subjected to cyclic thermal loads. The choice of material, range of temperature, and applied stresses is based on the conditions that are likely to be encountered in practice. Specifically, cyclic thermal tests under constant vertical effective stress in oedometric conditions with temperatures ranging between 10 and 40 °C have been conducted to study the volumetric response of fine-grained soils. The focus on this type of material is based on their historically observed higher sensitivity to temperature variations compared to coarse-grained soils. The purpose is to discuss the influence of cyclic thermal loads on the material under consideration. In interface analysis, changes in volumetric response mean changes in boundary conditions. Moreover, constant normal stiffness (CNS) direct shear tests, in which the soil-concrete interface has been subjected to thermal cycles, are presented. The constant normal stiffness condition has been adopted to account for the effects of the volume changes that occur when the soil adjacent to the pile is sheared (Tabucanon et al. 1995). Spring stiffness is used to replicate the presence of the surrounding soil, which partially

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prevents the volumetric response. Consequently, the normal stress acting on the interface may increase or decrease because of the dilative-contractive behaviour at the pile-soil interface (Fakharian and Evgin 1997). The purpose is to analyse the response of the interfaces between fine-grained soils and concrete to cyclic thermal and mechanical loads and the effects on shear strength. Finally, the combination of the volumetric and shear aspects aims to better represent and understand the fundamental failure mechanism at the soil-concrete interface.

2. Thermovolumetric behaviour of fine-grained soils

Although the first studies on the effects of temperature on the engineering behaviour of soils date back to the 50s and 60s (see the historical perspective provided by Laloui and Cekerevac 2003; McCartney et al. 2019), some of the mechanisms of thermal volume change remain to be understood. The lack of information about loading sequences is one of the main causes that prevent complete understanding. In many studies, details on the thermal and mechanical history of the material did not receive adequate attention (Hueckel et al. 2009). Particularly, the drainage conditions under which heating was performed were not discussed in depth and the time dependency on thermal consolidation was not considered (Coccia and McCartney 2016). Variations in procedures lead to varying outcomes and therefore, observations of studies targeting different applications are difficult to compare. Indeed, the results of each test depend primarily on how the test is performed (Laloui et al. 2014).

Based on both the pioneering work of Campanella and Mitchell (1968) and the critical analysis presented by Coccia and McCartney (2016), the current
framework identifies three possible mechanisms responsible for the thermal volume change of saturated soils. The first mechanism, thermal primary consolidation, is associated with a time-dependent volumetric contraction of the soil as the pore water pressure generated by the increase in temperature dissipates. Water in the pores of soil matrices has a thermal expansion coefficient approximately 7–12 times greater than that of the solid particles. This generates pore water pressure within the soil, if heated in undrained or drained conditions, for fine-grained soils. In the latter case, a temporary undrained condition is induced as the drainage is usually delayed because of the relatively low permeability of the soil. Therefore, the heating rate and soil permeability are the two predominant factors that govern and influence this thermal volume change mechanism. The second mechanism, thermal secondary compression, is associated with particle rearrangement. Current studies have sought to explain this phenomenon by assuming that physicochemical interactions govern the processes at the microscale. Campanella and Mitchell (1968) suggested that a decrease in the shear strength of inter-particle contacts occurs as temperature increases and that this phenomenon should end when new bonds are developed to carry the induced stresses. In addition, according to Laloui and Rotta Loria (2019), the physicochemical interactions contributing to this phenomenon appear to be (i) the degradation of the adsorbed water layer caused by an increase in temperature (Fleureau 1979; Pusch 1986), (ii) the variations in the rigidities of the mineral involved, which modify the contact force network (Kingery et al. 1976), and (iii) the modifications of the equilibrium between the Van der Waals attractive forces and the electrostatic repulsive forces (Laloui 2001). However, these theories lack
experimental observations; thus, they remain unproven. The third mechanism is related to the variation of water viscosity with temperature. An increase in temperature results in a decrease in pore water viscosity. This mechanism is considered to have both the ability to enhance the drainage and to accelerate the thermal secondary compression (Coccia and McCartney 2016).

Based on the described traditional framework, the literature links the thermally induced deformation of fine-grained soils mainly to one of these three mechanisms. Finn (1951), Plum and Esrig (1969), Towhata et al. (1993), Romero (1999), Abuel-Naga et al. (2007), and Vega et al. (2012) performed tests in which the heating phase was undrained or incompletely drained (leading to excess pore water pressure generation). Pore water pressure dissipation was allowed after the thermal equilibrium was established. In these cases, the thermal volume change was mainly associated with the effects of pore water dissipation. In the tests conducted by Campanella and Mitchell (1968), Baldi et al. (1991), Hueckel and Pellegrini (1996), and Cekerevac and Laloui (2004), the volumetric strain of the soils during the drained heating was calculated based on the volume of water expelled from the saturated samples. In all these cases, the observations were made when the temperature $T$ stabilised (i.e., $T=\text{const}$ over time). Therefore, the deformations observed after heating were attributed to the rearrangement of particles. Soil skeleton rearrangement resulted from allowing the samples to be maintained at a constant temperature after a change in temperature occurred; thus, it was more related to the long-lasting action of the heat on the clay. Furthermore, more recent studies have related thermal consolidation with thermal secondary compression (Di Donna and Laloui 2015; Shetty et al. 2019). Burghignoli et al.
(2000) considered the deformations observed after heating as an amplification of those related to creep deformation of the soil skeleton at a constant temperature. This concept of “thermally accelerated creep” was also revived by Coccia and McCartney (2016), where the volume changes were attributed to the variation in pore water viscosity with temperature. Therefore, it refers to the third mechanism originally proposed by Paaswell (1967).

Nevertheless, attributing the volumetric response to a single mechanism is not sufficient for describing this phenomenon completely. Gaps and contradictions persist in each of them. Logically, all mechanisms are considered to contribute to the observed global response; additionally, the prevalence of one mechanism over another depends on the load history to which the sample is subjected. Moreover, the time factor, both in the heating rate adopted and in the interpretation of the consolidation curve, plays a decisive role.

One of the main challenges at the macroscopic scale (i.e., when soil is idealised as a superimposed continua) is the relative role of primary consolidation and secondary compression in the volumetric response of fine-grained soils (Rotta Loria and Coulibaly 2020). Special attention has been devoted to this issue in the interpretation phase of the experimental results of this study. In addition, by focusing on energy piles, the current knowledge is mainly based on lessons learned from research on other applications (mainly nuclear waste storage) and there are therefore some differences in the stress paths to which the sample should be exposed, such as the cyclic variations (heating and cooling) and temperature ranges investigated. Recently, Di Donna and Laloui (2015) made efforts in this direction; however, there are still few experimental results to date. All these
aspects, as well as the time dependency, require further understanding and have therefore been addressed in this study.

2.1 Experimental investigation in oedometric conditions

This section aims to present several experimental results that are significant for the application of energy piles, focusing on the engineering aspects of the thermomechanical behaviour of fine-grained soils.

Thermovolumetric behaviour is studied through oedometric tests. Temperature variations are cyclic and cooling phenomena are involved; they occur in drained conditions. Moreover, an interpretation procedure based on mechanical analogy is proposed, in which the time dependency of the thermal consolidation is considered. The following sections provide details on the apparatus, sample preparation, and experimental procedures adopted in the tests.

2.1.1 Materials and method

Materials

A clayey soil with a mineralogical composition of 77% illite, 10% kaolinite, 12% calcite, and traces of feldspar and quartz was used in the experiments. At laboratory conditions (Temperature, $T = 20 \degree C$), the clayey soil had a liquid limit of 56%, plastic limit of 32%, plasticity index of 24%, and specific gravity of 2.65. The particle size distribution is shown in Figure 2.

The clay was maintained in the laboratory in powder form with a hygroscopic water content of approximately 5%.

Sample preparation

A hydraulic press (Wykeham Farrance Eng. Ltd.) was used to prepare the samples via static uniaxial compaction. The press was equipped with a load cell
(BLH U3G1, maximum vertical force 22 kN, accuracy 1 N) and a linear variable differential transformer (LVDT) (HBM W 10 TK, accuracy 1 μm). The air-dried powder under laboratory conditions was mixed with the required amount of water to achieve an initial water content of 36%. The mixture was stored in airtight containers for at least three days for moisture equilibration. For the compaction, the soil mass was confined in a rigid mould, and a variable static force was gradually applied through the movement of a piston in the strain-controlled press, until the desired dry density of 1.10 Mg/m$^3$ was reached. The setup was designed to keep the water content constant during the compaction. This corresponds to a process of compressing a partially saturated sample under undrained conditions. The compaction velocity was controlled through the motor of the machine, at a piston travel rate of 0.5 mm/min. The maximum vertical compaction stress was 82 kPa. After compaction, the samples were trimmed down in the corresponding oedometric ring. A height of 45 mm (corresponding to three oedometric samples, being the height of the oedometric ring 15 mm) was chosen to allow two samples to be prepared simultaneously. Parallel tests were conducted to ensure repeatability of the results. The remaining material was used for water content control. At this stage, size and weight measurements of the samples were recorded to determine the initial conditions.

**Equipment**

The clayey soil was tested in specially designed temperature-controlled oedometric cells (Figure 3). The experimental setup was developed by Di Donna and Laloui (2015); it consisted of various oedometric cells equipped with a hydraulic temperature control system. Each cell was heated using a spiral tube.
positioned around the sample, through which water was circulated at the desired temperature. The hydraulic circuit was connected to a thermal bath, which had the dual function of maintaining the temperature and introducing water into the system using a pump. This system allowed for heating and cooling cycles to be performed. The delay and the difference in temperature between the thermal variation in the heating system and the real temperature change within the sample were assessed during the calibration process. For this purpose, the relationship between the imposed temperature and the temperature inside the cell was carefully determined using a series of thermocouples (Thermocoax type K, Chromel-Alumel, 2AB 35 DIN, resolution 0.01 °C). Thermal losses were minimised by using a polystyrene box as an insulating system around the cells. In addition, the equipment was carefully calibrated to assess the deformability of the cells under thermal and mechanical loads. The vertical displacement of the sample was measured using an LVDT (HBM W 10 TK, accuracy 1 μm), which had to account for the deformation of the device. To correct the measured vertical displacement, a steel sample with a known linear thermal expansion coefficient and Young’s modulus was used during the calibration tests. The oedometric conditions were ensured using an invar oedometric ring which guarantee the minimisation of thermal radial deformation. During the calibration tests, the same loading paths and configuration adopted subsequently in the real tests were applied. Finally, saturation conditions were maintained using a water supply system that compensated for water evaporation.

Experimental programme
The experimental campaign aimed to characterise the cyclic thermovolumetric response of clayey soils by applying stress paths representative of the real conditions of the soil in situ when an energy pile foundation is in operation. It consisted of saturated oedometric tests, in which thermal cyclic loads were applied under a constant mechanical load. Temperature and vertical stress were the two variables controlled during the tests; the imposed values of these variables were defined based on the operating ranges of the energy piles.

The experimental work included three types of tests involving heating-cooling cycles at different constant vertical stresses and different overconsolidation ratios (OCRs). For each type of test, two samples were tested in parallel to ensure repeatability of the results. Standard oedometric tests were performed to characterise the material. Table 1 provides a summary of the variables that distinguish the tests. Figure 4 shows the thermomechanical paths of the tests.

Regarding the mechanical load, the consolidation was performed following the ASTM D2435/2435M-11 standard. The saturation phase was performed by flushing the sample without back pressure saturation. Although according to the ASTM D2435/2435M-11 standard, inundation of the test sample is not always a guarantee of achieving complete saturation, the final degree of saturation was computed to be approximately 90% or higher in all the tests. Therefore, a quasi-saturated state was achieved in all the tests. During the initial saturation phase, the sample was allowed to swell. Nonetheless, the initial void ratio was preserved, considering the low swelling potential of the material. Table 2 shows the initial conditions of the samples after the saturation phase. After saturation, the tests were conducted in two distinct stages. In the first stage, a conventional
mechanical consolidation was performed where load increments of constant total axial stress were applied to the sample until a target vertical stress was reached. In the second stage, drained heating-cooling cycles were performed. Five (i.e., 15 days) and two (i.e., 6 days) complete heating-cooling cycles were conducted for Test 1, and Test 2 and 3, respectively. The temperatures investigated were in the T=10 °C range. Attention was paid to heating and cooling rates to avoid the generation of significant excess pore water pressure at a rate of 2.0 °C/h. Observations from previous in-situ tests (Mimouni and Laloui 2015; Sutman 2016; Rotta Loria and Laloui 2017a) highlighted that the temperature changes induced in the soil occurred in drained conditions.

2.1.2 Test results

One of the challenges of testing under non-isothermal conditions is the lack of standards and interpretation procedures, making it difficult to compare results among different studies.

In this section, the proposed procedure is based on the analogy of the consolidation process caused by pressure changes and thermal variations (Campanella and Mitchell 1968). In the context of energy piles, this approach is convenient because it allows analysing the effects of mechanical and thermal loads for the design of a foundation, according to the classic scheme of geotechnical analysis.

In one-dimensional incremental mechanical loading tests, the experimental data represent a consolidation, partly resulting from the dissipation of pore water pressure (generated by the applied load) and partly from creep phenomena. For this analysis, only the consolidation corresponding to the dissipation of pore water
pressure (i.e., primary consolidation) is of interest. This is defined in time-
deformation curves typically using the Log Time Method, where it is identified as
the intersection of two straight lines; the first made tangentially to the inflexion
point and the second coinciding with the final part of the experimental curve. An
alternative procedure is to use the Square Root of Time Method.

A similar interpretation procedure for the thermal load is shown in Figure 5. A
normally consolidated (NC) soil sample is first heated in drained conditions (i.e.,
2 °C/h for 10 increments). At each temperature increment, the sample initially
dilates owing to the thermal expansion of the soil constituents. Drainage is usually
delayed because of the relatively low permeability of the clay and the impedance
effects of the ceramic disc (Campanella and Mitchell 1968; Romero 1999). After
ten increments (i.e., for approximately every 20-degree increase in temperature),
the excess pore water pressure was allowed to completely dissipate, keeping the
temperature constant for a prolonged time. Figure 5a shows the intersection of the
two straight lines, identified based on a procedure similar to that used in the
mechanical case, corresponding to the end of the primary consolidation. Evidence
shows that, given a drained thermal increase, the primary consolidation is
minimised as attention was placed to avoid creating excessive pore pressures in
the sample; additionally, most of the thermal consolidation is associated with the
secondary compression. This result agrees with the observations made by
Campanella and Mitchell (1968), Burghignoli et al. (2000), Shetty et al. (2019),
according to which the deformations are mainly attributed to secondary
compression. Soil skeleton rearrangement results from allowing the sample to
remain at constant temperature after a temperature change. Therefore, it is related
to the long-duration effects of heat on clay. Thermal primary consolidation becomes more important for the low permeability and high heating rate because of the higher pore pressure generation. Figure 5b shows the same interpretation procedure adopted for the 5th cycle. Notably, the primary consolidation and secondary compression can be recognised as in the first cycle. In this case, the secondary compression is almost negligible as particle rearrangement already occurred in the first cycles and a stable configuration of the solid particles is progressively reached. Figure 6 shows the same features of Figure 5, but for an over consolidated (OC) clay. In OC conditions, the available space for inducing additional collapse is reduced compared to NC conditions. Therefore, the particle rearrangement is limited. In this case, as expected, the secondary compression did not occur in the first cycle due to the soil structure being more stable. During the cooling phase, the constituents contract leading to a stable configuration of the solid particles; thus, as a result of this phase, there is no rearrangement of the particles for neither NC nor OC clays. The main observation is that cyclic loads contribute more to primary consolidation as temperature changes are continuous over time. The effect of pore pressure generation and dissipation on the volumetric behaviour depends on factors such as the heating-cooling rate and permeability of the soil. Secondary compression, however, is attributed to the presence of an elevated temperature (in the case of heating) for extended periods. This can be recognised as an indirect effect of cyclic loads. Different ground temperatures than the initial temperature result from unbalanced heating and cooling cycles. Indeed, cycle imbalance can lead to an average ground temperature that deviates from the initial one in the
Therefore, the long-lasting effects must be considered in the long-term design. In this regard, the work of Campanella and Mitchell (1968), Baldi et al. (1991), Hueckel and Pellegrini (1996), and Cekerevac and Laloui (2004), in which observations of volumetric strain with temperatures referred to a condition in which the temperature stabilised and remained constant for an extended period, are useful. In addition, temperature increases may influence time-dependent effects such as creep (Leroueil and Soares Marques 1996; Mitchell and Soga 2005; Laloui et al. 2008).

The results of this study focus primarily on the effects of cyclic thermal loads. Therefore, the points shown in Figure 8 identify the deformations of the sample resulting from the thermal load at the end of the primary consolidation, as commonly reported in the oedometric curves interpreted according to the method of Casagrande or Taylor for mechanical loads. The standard oedometric results, which refer to Test 0, are shown as an example in Figure 7. Notably, although the loading process is to be considered drained in the case of fine-grained soils, drainage is usually delayed because of the relatively low permeability (Campanella and Mitchell 1968). Therefore, it is possible to distinguish the primary consolidation (in this case reduced to a minimum given the drained loading process) and the secondary compression (as explained in the previous paragraph concerning Figures 5 and 6). Based on the results in Figure 8, which refer only to the primary thermal consolidation, very low residual strain variations are observed within the temperature range analysed. During temperature cycles (i.e., temperature variations) the soil components expand when heated and contract when cooled leading to very small residual strain. The phenomenon is
observed both in samples under NC (Figure 8a) and OC conditions (Figure 8c). In the case of higher confinement (i.e., 1000 kPa), the behaviour is also similar for the temperature ranges analysed (Figure 8b). However, a greater tendency to contract at the end of the primary consolidation is observed, which is attributed to the increase in pore pressure owing to lower porosity. Comparison between mechanically induced axial strain (Figure 7) and thermally induced axial strain (Figure 8), associated with the operation of the energy piles, highlight that the latter is significantly small (2-3 order of magnitudes smaller than mechanical ones). Although residual thermal strain variations have been observed, these appear to be small and negligible from a practical perspective. Therefore, the assumption of a reversible behaviour may be justified in analysis approaches for energy piles.

Multiple tests were conducted in parallel under the same conditions to ensure the repeatability of the phenomena.

The interpretation presented may serve as a basis for a future standardisation process of thermomechanical tests in geotechnical engineering. The contributions of these experimental results add knowledge on the thermovolumetric behaviour of fine-grained soils and provide an answer on how the soil behaves with a focus on cyclic loads. Usually, this is sufficient for resolving geotechnical issues. More fundamental questions related to the description of microscale processes (i.e., physicochemical processes that occur in the soil at the particle level) that influence macroscopic behaviour are not addressed in this study.

Based on these results, it is possible to make some considerations for the analysis and design of energy piles and highlight how neglecting the temperature
sensitivity of the volumetric behaviour of the soil may be justified, e.g., using simplified analysis approaches (Rotta Loria and Laloui 2016, 2017b; Sutman et al. 2019; Ravera et al. 2020a, 2020b). In addition, considering these results, when analysing the soil-concrete interface failure mechanism, reversible volumetric deformations imply that the boundary conditions of the interface are not affected by the temperature cycles.

3. Thermomechanical behaviour of the fine-grained soil-concrete interface

An effective stress approach for both coarse- and fine-grained soils is applied today in the evaluation of pile capacity. Therefore, two aspects need to be considered as a result of the effects of thermal loads: (i) the possible variation of the pile-soil interface angle of shear strength, which may be approximated as the soil angle of the shear strength under constant volume conditions and (ii) the variation of the normal effective stress acting on the pile shaft, which is associated with volume changes. Most of the research to date has focused on the first aspect. Hueckel and Pellegrini (1989), Hueckel and Baldi (1990), Robinet et al. (1997), Burghignoli et al. (2000), Graham et al. (2001), Cekerevac and Laloui (2004), Ghahremannejad (2003), Yavari et al. (2016), Li et al. (2019), and Maghsoodi et al. (2020) performed studies on clay to assess the angle of shear strength under non-isothermal conditions for different thermal paths. Few studies have been conducted on the thermal effects on the interface shear strength. Recently, Di Donna et al. (2016), Yavari et al. (2016), Li et al. (2019) Yazdani et al. (2019) and Maghsoodi et al. (2020) have performed clay-structure interface tests. Despite some potential variations that turned out to be negligible, all these studies
concluded that, from a practical perspective, the angle of shear strength appears to be essentially independent of temperature. When addressing aspect (ii), Di Donna et al. (2016) and Maghsoodi et al. (2020) performed CNS tests at different temperatures where changes in normal stress refer to mechanical shearing. To date, however, no study provides information on the variations of normal stress, referring to variations due solely to thermal cycles which better represents the actual conditions at the pile-soil interface during the operation of the energy foundation.

### 3.1 Experimental investigation with direct shear tests

The experimental study presented in this paper to investigate the cyclic thermomechanical response of fine-grained soil-concrete interfaces is based on direct shear tests. This test has been recognised as the most representative of the failure mechanism occurring at the pile-soil interface, as it examines both the effects of interface volume changes and the characteristic of surface discontinuity (Boulon and Foray 1986; Boulon 1989; Boulon and Nova 1990; Boulon et al. 1995; Ghionna and Mortara 2002; Pra-ai and Boulon 2017). The objective is to provide results for the typical temperature ranges of energy piles where both CNS conditions and cyclic thermal loads are applied simultaneously. These aspects represent the subject matter of this study, as these loading conditions are the most representative of the operation of energy piles and, to date, no results are available in these conditions.

#### 3.1.1 Materials and method

**Materials and sample preparation**
The soil used was the same as that used in the oedometric tests. The samples were prepared with the same technique adopted for the oedometric samples, as described above. After compaction, the samples were trimmed down in the direct shear frame with minimal disturbance and were placed in the testing configuration.

To perform the interface direct shear tests, a concrete sample was designed and used as the structure. The concrete was prepared in the laboratory in special moulds following the BS EN 206-1:2000 standard; the specifications are shown in Table 3. The roughness of the concrete surface was measured with a Bruker 3D optical microscope (Figure 9). Owing to the size limit of the images, it was not possible to scan the entire surface of the samples. Therefore, special attention was placed on the production of a concrete sample with surface homogeneity. A 12 \times 12 \text{mm} area was subsequently identified as being representative of the entire surface. Image processing involved the analysis of profiles, every 0.4 mm, parallel to the shear direction (x-direction in Figure 9). To determine the roughness of the interface, each profile was divided into a fixed length \( L = 0.5 \text{mm} \), and the maximum roughness \( R_{\text{max}} \) was measured at each \( L \). An average \( R_{\text{max}} \) of 96 \( \mu \text{m} \) was obtained, which is within the usual range for concrete surfaces, according to Yoshimi and Kishida (1981).

**Equipment**

The experimental device adopted in this work was a modified version of the direct shear device produced by GDS instruments described in Di Donna et al. (2016). In the previous version, three main modifications were made to adapt the apparatus to the application of heating episodes at the concrete-soil interface: (i)
the lower part of the shear box, accommodating the concrete sample and heating
system, was redesigned to have a constant contact area between the two materials
and a larger space for installing the thermal system; (ii) the possibility of
imposing CNS conditions was implemented in the GDSLAB software; and (iii) a
heating system composed of an electrical resistance, electrical power supplier,
insulation system, and thermocouples was introduced.

For the present study, further modifications were applied to the direct shear
device to allow the application of cooling episodes to reproduce the in-situ energy
pile-soil interaction on a laboratory scale. First, the lower part of the shear box
was reproduced by opening holes on its wall, through which circulation tubes
were introduced inside. Additionally, a thermal bath was attached to the
circulation tubes to ensure that the circulating water had the target temperatures
and to enable the heating and cooling episodes. For this purpose, the relationship
between the applied temperature, the temperature in the water, and the
temperature inside the cell was carefully determined using a series of
thermocouples (Thermocoax type K, Chromel-Alumel, 2AB 35 DIN, resolution
0.01 °C). During the calibration, thermocouples were inserted into the cell that
was in contact with a sample specifically dedicated to this stage. Four of these
thermocouples were permanently placed in the water container around the cell to
monitor temperature evolution throughout the testing campaign. The shear box
was insulated using polystyrene to minimise heat loss and temperature variations
of the sensors. Horizontal and vertical displacements were measured using two
LVDTs (accuracy 1 μm). Two load cells (maximum vertical force 5 kN, accuracy
0.001 kN) measured the horizontal and vertical loads applied to the sample.
During the calibration tests, the same loading paths were applied to steel samples using the same configuration that was applied subsequently in the real tests. Additionally, a water supply system was installed to ensure saturation conditions. The direct shear device, with the new built-in changes, was capable of applying subsequent and multiple heating-cooling cycles to the samples and could thus simulate the actual soil conditions in the vicinity of the energy piles. The modified direct shear device (Figure 10) was used to investigate various conditions related to the field of energy piles.

**Experimental program**

Preliminary tests were performed under constant normal load (CNL) and CNS conditions for soil-soil and soil-concrete samples under isothermal conditions. However, the main objective of this experimental campaign was to perform tests under non-isothermal conditions in which the samples are subjected to cyclic thermal loads. The goal is to simulate the effects of the operation of the energy piles on the ultimate shear response. In general, the load paths applied were characterised by the following steps: (i) Saturation. (ii) Consolidation under constant mechanical load. In all cases, the samples consisted of clayey soil under NC conditions. (iii) Three thermal cycles (i.e., 5 days) were performed using a heating rate of 2.0 °C/h and a temperature range of $T = 10$ $\div 40$ °C. Therefore, at this stage, the thermal stress path was analogous to that in the oedometric tests; the only difference was that in this case, to simulate the conditions of volume change at the pile-soil interface more accurately, the third phase was executed under CNS conditions. (iv) Monotonic shearing was performed under CNS conditions at ambient temperature, with an
applied strain rate of 0.003 mm/min after the application of cyclic thermal loads.

Specifically, for tests conducted under isothermal conditions, the third phase described above was not performed, and the fourth phase was executed under both CNL and CNS conditions. Conventional tests were performed following the specifications of the ASTM D3080/D3080M-11 standard. Table 4 summarises the main variables of all the tests performed. Table 5 summarises the initial conditions of the samples. Additionally, Figure 11 shows the stress paths.

3.1.2 Test results

Preliminary tests were performed as reference cases for comparison to better clarify the role of interface, and CNS conditions with respect to the role of temperature for the soil used in these experiments. Therefore, Figure 12 represents a characterisation of the behaviour of the soil and the soil-interface under isothermal conditions. Soil-interface tests in CNS conditions were used as a reference for the test where thermal cycles were applied. The results in Figure 12 refers to samples that are NC after the consolidation phase and, consequently, the material do not show any peak. Accordingly, the material contracts during the shear tests. Similar results were obtained by Di Donna et al. (2016) when comparing the soil-soil (Test 4) and soil-concrete (Test 6) test profiles under CNL conditions, where soil failure was assumed to occur. The comparison of the soil-concrete test profiles under CNL and CNS conditions (i.e., Tests 6 and 7, respectively) is similar to the behaviour of a loose sand/smooth interface observed, for instance, by Porcino et al. (2003). A reduction in maximum shear stress can be observed with an increase in the stiffness. Simultaneously, the increase in normal stiffness causes a reduction in the current normal stress. The
above effects can be correlated with the contractive behaviour exhibited by the material. A contraction yields to a decrease of the current normal stress, corresponding to a decrease of the shear stress, as a consequence of the elastic constraint provided by the soil surrounding the interface.

Figure 13 shows four clay-concrete interface tests performed under CNS conditions. Tests 7 and 9 refer to a standard CNS direct shear test in which a purely mechanical stress path is applied. For these tests, the third step described earlier was not performed. The results of the shearing phase, following the cyclic thermal loads, are represented by Tests 8 and 10. Notably, the application of thermal loads has a negligible effect on shear stress in the shearing phase. The variations in the shear strength angles are minimal and are not crucial in the definition of the failure envelope, which remains practically unaffected (Figure 14). Some differences in the axial displacement are attributed to the effect of time because, in Tests 8 and 10, shearing occurs after the thermal cycles (i.e., 7 days), and minor importance is attributed to the cyclic thermal consolidation. Volume changes when applying thermal loads under CNS conditions are in the order of magnitude of those measured during the oedometric tests (Figure 15). The non-linear behaviour of the device at this test stage makes precise quantification difficult. However, with almost no effect, it is possible to assume that the volume change is not significant for the shear resistance under the investigated conditions (i.e., there are no significant changes in the effective normal stress during the thermal cycle as shown in Figure 15).
Additionally, the failure mechanism of the soil-structure interface within the framework of elastoplasticity is discussed by Ravera and Lalouï (2020), and a constitutive model applicable to both conventional and energy piles is described.

4. Conclusion

The cyclic thermomechanical behaviour of the soil-concrete interface was investigated by performing a testing programme including cyclic thermal tests under constant vertical effective stress in oedometric conditions, and CNS interface direct shear tests, where the samples have been subjected to thermal cycles. The main objective was to experimentally analyse the soil-concrete interface failure mechanism of energy piles. The shear behaviour of the soil-pile interface is characterised by large localised strains concentrated in a thin layer around the pile, i.e., the interface zone, surrounded by soil behaving in oedometric conditions. The tests conducted allowed the volumetric and shear aspects that characterise the failure mechanism to be analysed first separately and then in combination.

The framework proposed here provides answers on how the soil-concrete interface responds to thermal and mechanical loads, which is useful for solving geotechnical problems. This study extrapolates information of interest (e.g., consolidation behaviour and failure envelopes) to an engineering application such as energy piles.

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The results of the thermovolumetric behaviour study show very low residual strain variations towards cyclic thermal variations. This means that the boundary conditions of the interface zone are not altered by the temperature cycle. In addition, this section presents an interpretation procedure based on the mechanical analogy with the advantage of analysing the effects of mechanical and thermal loads for an energy pile design according to the classical scheme of geotechnical analysis. The presented procedure can provide a basis for the development of new testing standards. Presently, the lack of standards and interpretation procedures is one of the major issues that hinder the extension of this technology from academia to industry. Therefore, the development of testing methods and equipment designed for engineering purposes can facilitate the spread of this technology.

The main interest in analysing the pile-soil interface behaviour is associated with the fact that the greater the volume variation under cyclic loads, the greater the probability of shear resistance degradation (i.e., reduction of normal stress). For this purpose, a direct shear test device was modified to enable the application of heating-cooling cycles under CNS conditions. The results of this study show that the probability of pile shear resistance degradation, when subjected to a thermal cyclic load is minimal and that changes in the failure envelope are negligible. With these insignificant effects, volume change does not appear to be decisive in affecting shear resistance (i.e., there are no significant variations in effective normal stress during a thermal cycle).

It is acknowledged that the results are valid for the sequence of the applied heating and cooling cycles in this study. The relevance of these observations in
the event that the cooling cycle precedes the one of heating requires further investigation.

Based on these results, it is possible to make some considerations for the analysis and design of energy piles. From a practical perspective, the results show an almost reversible volumetric behaviour and negligible effects on shear strength of the soil-concrete interface towards cyclic thermal loads. Thus, neglecting temperature sensitivity may be justified, as when simplified analysis methods are used in the preliminary design stages.

Acknowledgements

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References


Pusch, R. 1986. Permanent crystal lattice contraction, a primary mechanism in thermally induced alteration of Na Bentonite. In MRS Online Proceedings Library, vol 84.


### Table 1. Thermal cycle oedometer: test variables

<table>
<thead>
<tr>
<th>Test N°</th>
<th>Label</th>
<th>Sample N°</th>
<th>OCR</th>
<th>Confining pressure [kPa]</th>
<th>Rate of heating [°C/h]</th>
<th>T range [°C]</th>
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### Table 2. Thermal cycle oedometer: initial conditions of the samples

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### Table 3. Concrete mix design

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### Table 4. Direct shear tests: test variables

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**Figure 3.** Temperature-controlled oedometric cell: (a) global view of the system (1: software control and datalogger thermocouples, 2: thermal bath, 3: thermocouples, 4: insulation, 5: thermal hydraulic system, and 6: water supply system). (b) schematic representation of the temperature-controlled oedometric cell

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Figure 9. Determination of concrete roughness by optical microscope: (a) image of the concrete surface acquired by a Bruker 3D optical microscope, (b) examples of roughness profiles for two specific sections, and (c) photograph of the concrete surface.

Figure 10. Modified direct shear device: (a) global view of the system (1: software control, 2: datalogger thermocouples, 3: insulation, 4: thermal bath, and 5: water supply system), (b) and (c) details of the modified shear box, (d) and (e) schematic representation of the shear box.

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Figure 14. Clay-concrete interface failure envelopes for testing under CNS conditions: (black) being subjected to thermal cycles, (grey) not subjected to thermal cycles.

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