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Experimental Analyses on the Multiphysical Phenomena Governing Energy Pile Behavior

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Abstract. Energy piles are exposed to temperature variations during their lifetime, due to their unique role combining structural support and geothermal heat exchange. Temperatures in the piles and in the surrounding soils fluctuate on a daily and seasonal basis which may cause axial displacements, additional axial stresses and changes in shaft resistance along energy piles. Furthermore, soils in the vicinity of the energy piles experience volumetric strains and changes in shear strength which may eventually have an impact on the structural behavior of energy piles. To understand the extent of temperature changes on energy piles, soils and soil-pile interfaces, various in-situ and laboratory tests have been performed. The goal of this paper is provide details regarding in-situ and laboratory tests performed on energy piles, as well as to compile an observational framework in understanding the mechanics of soils, structures and the interaction between them in consequence of thermal actions.

Keywords. Energy geostructures, energy piles, in-situ testing, laboratory testing, thermo-mechanical behavior, non-isothermal behavior

1. Introduction

In practice, energy piles can be employed only for space heating or cooling, depending on the demand of the region, or they can be used for both heating and cooling purposes. In any of the cases, as a result of their dual nature, energy piles and the surrounding soil are exposed to temperature changes during their life time.

Temperatures in the piles and in the surrounding soils fluctuate during the day in between operation and stoppage times as well as seasonally after episodes of heat injection during summer followed by heat extraction during winter. It is known that the seasonal temperature fluctuations can go up to ±15-20˚C of the in-situ temperature. Furthermore, energy piles are subjected to daily temperature variations of 4-8˚C, in between operation and stoppage times. These temperature changes may cause axial expansion and contraction of the energy piles which can alter resistance mobilization along their shaft. Moreover, the prevented portion of the axial elongation and contraction may cause thermally induced changes in the axial stresses.

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Due to heat exchange operations, the soil at the vicinity of energy piles is also exposed to thermal loads, which may affect soil properties such as preconsolidation pressure, shear strength and may cause excess pore water pressure generation and volumetric strains.

In this paper, details on in-situ and laboratory tests performed on energy piles, soils and soil-structure interfaces are provided along with breakthrough experimental results targeting fundamental information on mechanisms governing thermo-mechanical behavior of energy piles. First, full-scale in-situ tests investigating the thermo-mechanical behavior of energy piles are presented. Next, the response of soils subjected to monotonic and cyclic thermal loads is considered. Finally, results of laboratory tests evaluating the response of soil-concrete interfaces subject to thermal and shear cycles are presented, which is followed by discussions.

2. Full-Scale In-Situ Tests on Energy Piles

Given the great potential of energy piles on the path of less dependency on fossil fuels, various in-situ tests were performed on this topic [1-5]. The three pioneering full-scale in-situ tests on energy piles performed at Swiss Federal Institute of Technology in Lausanne (EPFL) and Virginia Tech, investigating (i) the response of a single energy pile to combinations of thermal and mechanical loads, (ii) the response of a group of closely spaced energy piles to thermo-mechanical loads and (iii) the response of a single energy pile to cyclic thermal loads are presented in this section. Compressive stresses and upward shaft resistance mobilization are considered positive, according to the adopted sign convention.

2.1. Single Energy Pile

The first in-situ test on a single energy pile [6], a pioneer in the area, was performed under a new 5-storey building, 100 m in length and 30 m in width, at EPFL campus. The building was supported by 97 bored piles, one of which was converted to an energy pile, having a diameter of 88 cm and a length of 25.8 m, with inclusion of heat exchanger tubes. A noteworthy number and type of sensors were placed along the length of the pile, which are shown in Figure 1: (i) vibrating-wire extensometers to measure vertical strain and temperature, (ii) fiber-optic extensometers, each 1 m long, to measure vertical strain, (iii) fiber-optic extensometers, each 2 m long, to measure radial strain at five depths, (iv) a load cell to measure the load at the toe of the pile and (v) extensometers at the head of the pile to measure the vertical strain in order to determine the load at the head of the pile.

To ascertain structural characteristics of the test pile, a pile integrity test was performed which showed the cross section of the pile to be considered constant. Furthermore, the Young's modulus of the concrete was evaluated with cross-hole ultrasonic transmission.

The soil profile at the field test site is shown in Figure 1, the properties of which were obtained from the boreholes, two static load tests, as well as triaxial tests on samples from layers A, B and C. The ground water table at the site is located at ground surface. Further information on soil stratigraphy and the test pile can be found in [6].

Being the first in-situ test on energy piles, the testing campaign was developed to reveal the fundamentals of an emerging field: (i) effect of temperature changes on the structural behavior of energy piles, (ii) extent of thermally induced axial stresses and mobilized shaft resistance and (iii) influence of end-restraining conditions on the thermo-mechanical response of energy piles.

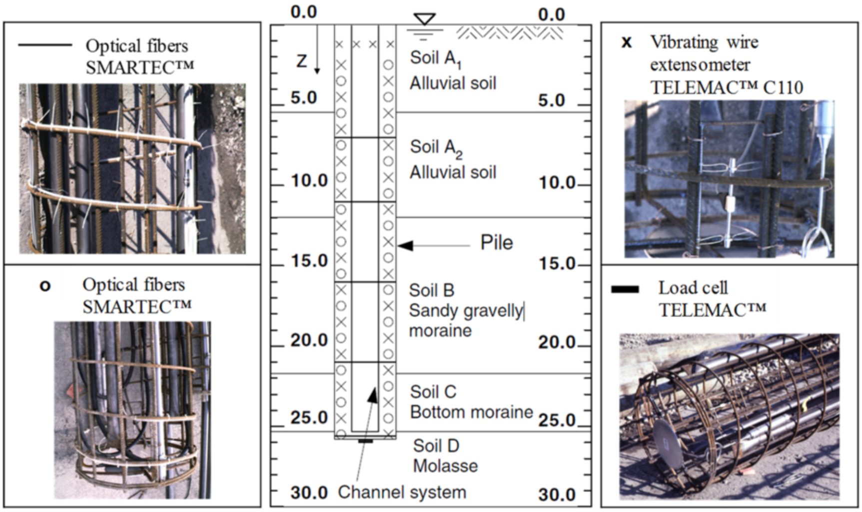

Figure 1. Soil profile at the site and sensors along the test pile (Test Site-1).

The mechanical load on the test pile was induced by the self-weight of the building. To distinguish the sole influences of mechanical and thermal loads, a heating – passive cooling cycle was applied on the test pile at the end of the construction of each storey. In total, eight tests were performed on the test pile.

The results from the last test, following the completion of the building construction, are presented in Figure 2. The mechanical load, caused by the weight of the building, decreases with depth, leaving the toe resistance not being mobilized (Figure 2a).

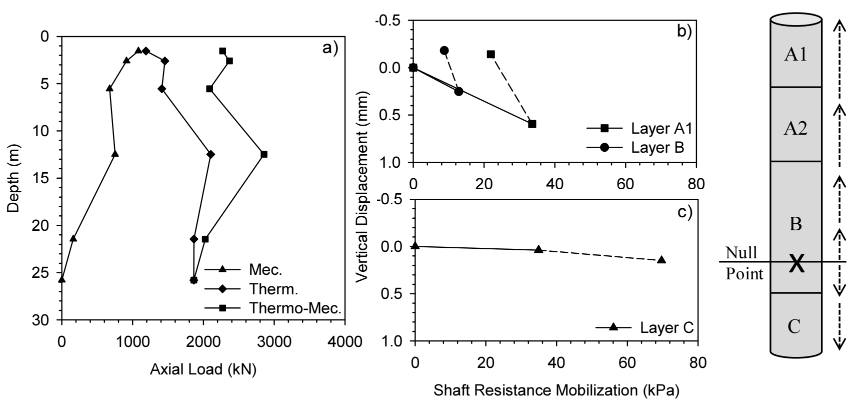

Figure 2. a) Mechanical, thermal and thermo-mechanical loads and b,c) shaft resistance mobilization along the test pile.

The subsequent heating episode (ΔT=13.4˚C) results in significant mobilization of the toe (2000 kN) as well as thermally induced axial load at the pile head (1000 kN) due to restraining effect of the building and the soil on the pile thermal expansion. Figure 2b and Figure 2c presents shaft resistance mobilization at various depths along the test pile, the solid and dashed lines representing the ones due to mechanical and thermal loads, respectively. The figure highlights thermally induced shaft resistance mobilization at layer C being in opposite direction with the one along layers A and B, as a consequence of the test pile expanding in opposite directions above and below the null point.

2.2. Group of Energy Piles

A second test site was constructed at EPFL campus, 200 m away from the single energy test pile, with the purpose of investigating the thermally-induced group effects and interactions characterizing closely spaced energy pile groups. For this purpose, four out of 20 piles supporting a water retention tank within Swiss Tech Convention Center (STCC) were converted to energy piles being 0.9 m and 28 m in diameter and length, respectively.

The test piles were bored and cast onsite and were equipped with four 24 m long polyethylene U-loops connected in series, leaving the top 4 m of the piles thermally inactive. The test piles were instrumented with (i) vibrating wire strain gages at every 2 m depth to monitor temperature and axial strain, (ii) vibrating wire strain gages at the head of the piles to measure thermal strains and stresses, (iii) pressure cell at the toe of the pile to monitor mobilized toe resistance and (iv) radial optical fibers to observe radial thermal strains (Figure 3).

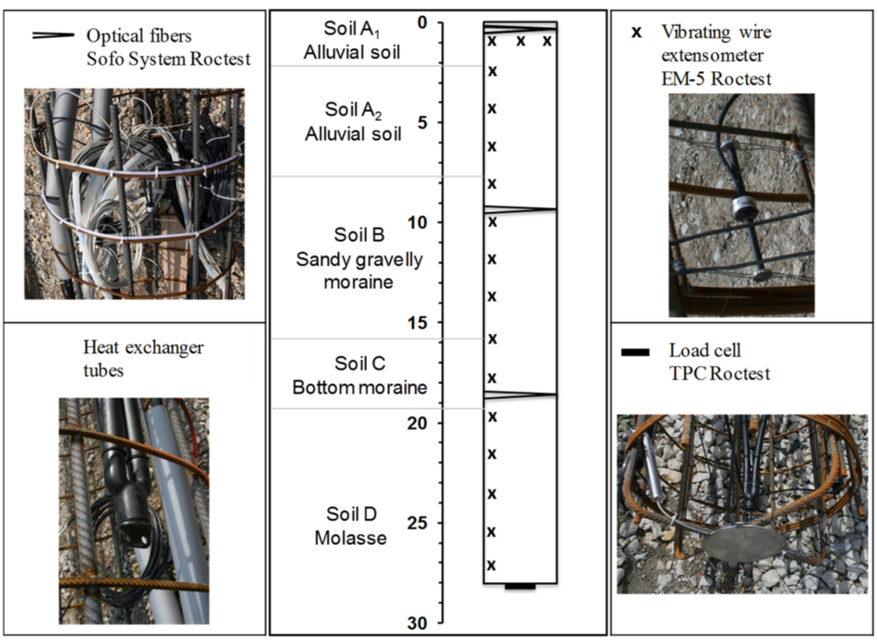

Figure 3. Soil profile at the site and sensors along the test piles.

The soil stratigraphy at the test site is similar to the one presented in Section 2.1, yet the information regarding the depth and thickness of the soil layers have been updated based on the boreholes at STCC area. Two piezometers, having the capability of temperature measurement as well, were installed along each of the two boreholes, to monitor heat propagation and excess pore water pressure generation in the soil between the piles. Furthermore, an additional thermistor was also placed in the boreholes for temperature monitoring. Detailed information on the soil stratigraphy as well as the instrumentation is presented by [7].

To investigate the thermally induced group effects on energy piles, the in-situ test involved application of heating and passive cooling cycles to a single energy pile, as well as to all four energy piles in the group [4]. This paper involves observations from the former case where EP1 corresponds to the operating pile while the other three surrounding piles (EP2, 3 and 4), 3-diameter away from EP1, act as non-operating piles.

Figure 4a shows the thermal interactions between the energy piles in the group due to the application of constant thermal power of 3 kW to EP1 for over 156 days. During the early stages of heating (i.e. 2 days), the temperature field of the non-operating energy pile, EP2, remained unchanged, while EP1 had an average temperature increase of 5˚C. In the subsequent stages of heating, heat diffusion resulted in an average temperature increase of 2˚C and 5˚C along EP2, while the uninsulated part of EP1 was characterized by an average temperature increase of 15˚C and 20˚C at 35 and 156 days, respectively.

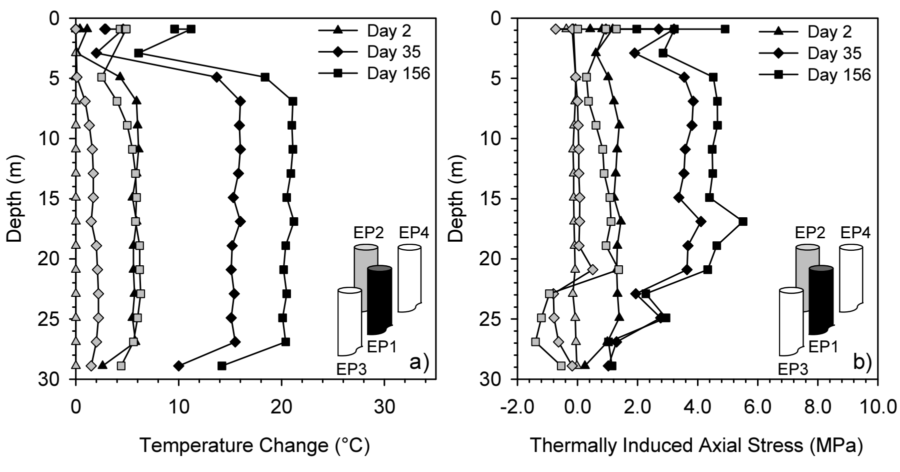

Figure 4. a) Temperature changes b) Thermally induced axial stresses along EP1 and EP2 due to geothermal operation of EP1.

Thermally induced axial stresses along the operating (EP1) and non-operating (EP2) energy piles are presented in Figure 4b.The gradual increase in the thermally induced compressive stresses along EP1 is observed in Figure 4b, which reached a maximum of 5500 kPa after 156 days of geothermal operation.

At the early stages of the field test (i.e. 2 days), temperature increase along EP1 induced axial stress changes also along the surrounding non-operating piles. The axial stresses along EP2 decreased by a magnitude of 250 kPa at the head, which were associated with the deformation field due to the heating of EP1 instead of a change in the thermal field of EP2. In other words, a change in the stress field of EP2 was observed even though its temperature remained constant which was due to the interplay between the responses of the piles, slab and soil to temperature changes.

At later stages of geothermal operation (i.e. 35 and 156 days), maximum axial stress increase of 1370 kPa and decrease of 1419 kPa were observed at the top and bottom portions of the non-operating energy piles, respectively. The decrease in axial stresses along the bottom portion of the piles has been associated with the thermally induced deformation of the molasse layer resulting in a pull-down effect on the group of energy piles [4]. This effect was less pronounced for the operational energy pile (EP1) since the increase in axial stresses due to active heating governed the stress field along EP1.

2.3. Single Energy Pile for Cyclic Thermal Loads

A full-scale in-situ test was performed on three energy piles in Houston, Texas to evaluate the influence of cyclic thermal loads as well as of different end-restraining conditions on the thermo-mechanical behavior of energy piles. In this paper the emphasis is put on the former case by presenting the data on a single pile subjected to five active heating-cooling cycles over a period of six weeks. Figure 5 shows the test pile profile with 45.7 cm in diameter and 15.24 m in length, together with the sensors placed along its length.

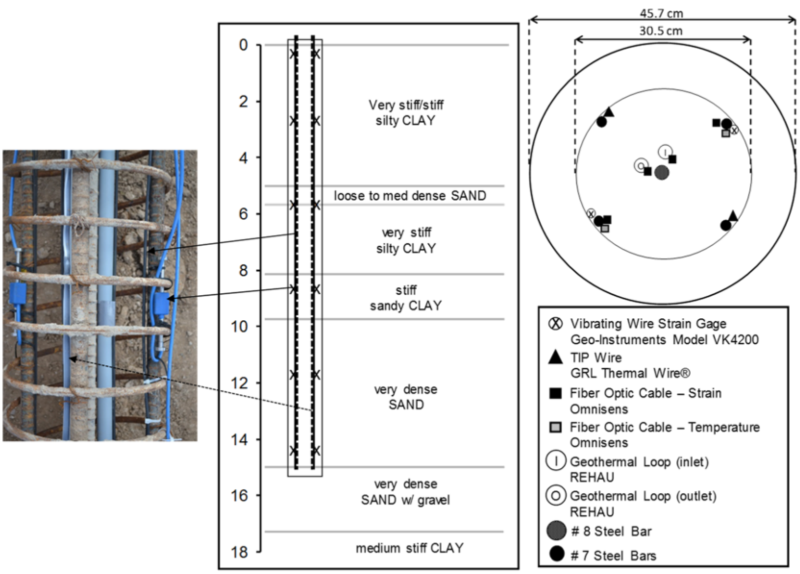

Figure 5. Soil profile at the site and sensors along the test pile.

The pile was equipped with a considerable number and type of sensors including (i) a pair of vibrating wire strain gages (VWSG) placed at 6-depths along the pile to measure strain and temperature, (ii) thermal integrity profiler wires along the full-length to measure temperature, (iii) fiber optic cables to measure strain and temperature, (iv) two linear variable differential transformers at the head of the pile to measure head displacement, and (v) two thermistors at the inlet and outlet of the geothermal loop entering and exciting the pile. To apply thermal loads, a single loop polyethylene (PEX) heat exchanger tube was placed at the center along the full-length of the pile. Finally, a borehole was drilled 0.5 m away from the pile through which three piezometers and three thermistors were placed, at 4.3, 7 and 8.8 m, to monitor the temperature evolution and excess pore water pressure generation within the soil.

The soil stratigraphy (shown in Figure 5) was obtained from four boreholes at the test site, along with standard penetration and pocket penetrometer tests. Furthermore, thermal oedometer and soil characterization tests were performed on soil samples collected from the boreholes in the Laboratory of Soil Mechanics at EPFL. The soil stratigraphy consists of very stiff and stiff clay layers until 9.8 m depth which is followed by a very dense sand layer until 17.4 m depth. Further information on the field test setup and soil profile is presented in [8].

Five heating-cooling cycles with a maximum circulating fluid temperature of 43˚C and minimum of 8˚C were applied on the test pile, the measurements of which were taken by the two thermistors placed at the inlet and outlet of the heat exchanger tube (Figure 6). Unlike the former two tests, active cooling cycles, bringing the test pile below the in-situ temperature, were part of the experimental campaign. The thermal cycles were imposed without a mechanical load application at the head of the test pile.

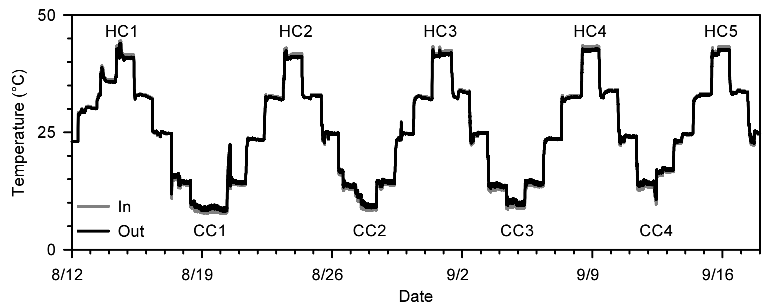

Figure 6. Circulating fluid temperature in the heat exchanger tubes embedded in the test pile.

As indicated in Section 2.1, heating of an energy pile causes an increase in compressive axial stresses as well as mobilization of downward and upward shaft resistance at the top and bottom portions of the pile, respectively, which is inverse for the case of cooling. In the case of cyclic temperature change, the pile is expected to alter between the two cases, having variations of axial stresses and mobilized shaft resistance in cyclic nature. The response of the test pile to cyclic thermal loads presented in Figure 7 confirms this expected behavior.

The thermally induced axial stresses (Figure 7a) and mobilized shaft resistance (Figure 7b) along the test pile associated with the applied thermal cycles were calculated employing the strain and temperature readings from VWSGs. The changes in axial stress, mobilized shaft resistance and temperature data are presented with respect to the in-situ state of the test pile.

Figure 7a shows the thermally induced axial stresses at the null point of the test pile, which have a transition between compressive (temperature increase) and tensile (temperature decrease) in nature, for each peak heating and cooling episode. The null point of the pile appears to be consistently at 8.7 m depth, which would be expected to change while transitioning between heating and cooling episodes, since the end-restraining conditions do not remain the same. However, although the null point depth may have changed, the change was not drastic to be detected by the VWSGs which have almost 3 m vertical distance between each other along the test pile. Apart from that, while the thermally induced axial stresses per unit temperature change during heating are slightly higher than the ones during cooling, they remain almost unchanged in between thermal cycles, suggesting that an accumulation of thermal stresses did not occur during the field test.

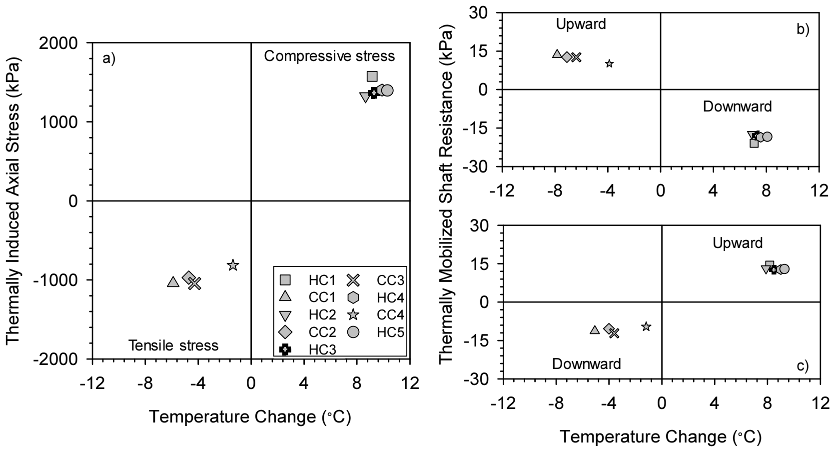

Figure 7. a) Thermally induced axial stress at null point; Mobilized shaft resistance at b) clay layer, c) sand layer due to thermal cycles.

Thermally mobilized shaft resistance due to thermal cycles imposed along the test pile are presented in Figure 7b and Figure 7c, for clay and sand layers, respectively. The two layers, being above and below the null point, show shaft resistance mobilization in opposite directions, which would be expected considering axial expansion and contraction of the test pile away from the null point, with temperature increase and decrease. Furthermore, observing the data points corresponding to each peak heating and cooling cycle, the thermal shaft resistance mobilization per unit temperature change can be considered almost unchanged, suggesting that a degradation of pile shaft resistance have not occurred. Yet, this observation cannot be generalized for energy pile applications considering the fact that the induced temperature changes on the test pile ($\Delta T \approx \pm 10$°C) were lower than what would be expected from an actual geothermal operation ($\Delta T \approx \pm 15$-20°C).

3. Thermo-mechanical Behavior of Soils and Soil-Concrete Interfaces

Geothermal operation of energy piles induces temperature change not only along the piles but also within the soil around them due to heat exchange phenomenon. Hence, investigating the behavior of soils and soil-concrete interfaces in non-isothermal conditions is crucial to administer a thorough knowledge on the response of energy piles to thermal and structural loads. As a consequence, three laboratory testing devices were designed and constructed at the Laboratory of Soil Mechanics at EPFL which are (i) thermal triaxial testing apparatus to evaluate the effects of temperature change on the stress-strain behavior of soils, (ii) thermal oedometer to investigate the volumetric response of soils to cyclic temperature changes and (iii) thermal direct shear device to examine the effect of cyclic thermal and mechanical loads on the soil-concrete interface.

The design of these devices are presented in this section along with a summary of the results from their employment.

3.1. Thermal Triaxial Tests

The effect of temperature change on the stress-strain behavior of clays was investigated by employing a designed, temperature-controlled triaxial testing device (Figure 8). For this purpose, an isothermal triaxial cell was modified to apply temperature changes to the soil sample with the inclusion of (i) an electric heater, which was placed in a thermal bath to serve water as circulating fluid, (ii) circulation pump, (iii) metal tube placed spirally around the sample through which heat carrier water was circulated, (iv) the insulation and (v) temperature controlling unit.

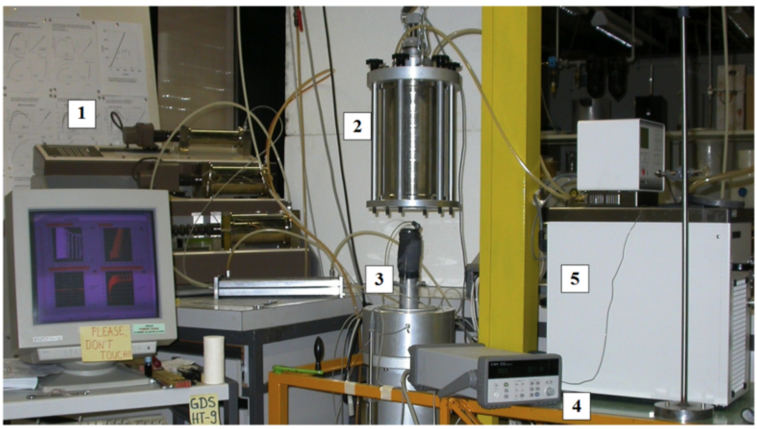

Figure 8. Thermal triaxial testing apparatus (1: GDS Controller, 2: Triaxial Cell, 3: Thermocouple, 4: Multiplexer, 5: Heating bath).

Five thermocouples were included in the setup to measure the temperature of the confining fluid (one for data acquisition and one for sending feedback to the heating bath), heating bath, room temperature and the cooling container. Finally, the standard Perspex cell was replaced with a stainless steel one to withstand high temperature and pressure, as well as corrosion. Further information on the design and calibration of the thermal triaxial apparatus can be found in [9].

The experimental campaign on Kaolin samples included (i) isotropic consolidation, which was followed by an unloading for over consolidated samples in some cases, (ii) heating from 22˚C to 90˚C with a rate of 10˚C/3h to ensure drained conditions and finally (iii) drained shearing. To quantify the influence of temperature on shear strength, the tests performed with the same mechanical path but with different temperature history are compared in Figure 9 for normally consolidated (NC) samples as well as overconsolidated (OC) samples with an over consolidation ratio (OCR) of 2 and 6. The results demonstrate that samples tested at high temperature (T=90˚C) show higher shear strength compared to the ones tested at ambient temperature (T=22˚C). However, at large strains, the shear stresses obtained for the samples at high temperature tend to the same critical state as the samples tested at ambient temperature. Therefore, the stress path at critical state constitutes a single envelop, irrespective of the test temperature. Figure 9b presents the volumetric response of the samples sheared at different temperatures, from which no clear trend was attained.

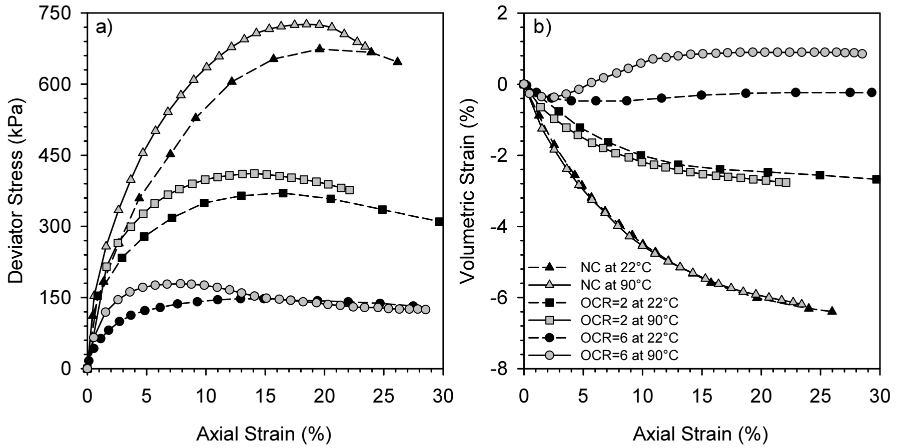

Figure 9. Drained triaxial tests at ambient (22˚C) and elevated (90˚C) temperatures

3.2. Thermal Oedometric Tests

The response of soils to monotonic and cyclic temperature change was investigated with the use of four oedometric devices (Figure 10). In order to adopt conventional oedometers to allow temperature control, spiral tubes were placed around the soil samples through which heat carrier water was provided with the use of a thermostat and a pump. The modified device allowed circulating water temperature of 5-60˚C which covers the temperature range of interest in relation to geothermal operation of energy piles. Four K-type thermocouples were placed in each oedometer cell to monitor the temperature throughout the tests. In addition to the temperature control and acquisition equipment, the cells were insulated with a polystyrene box to prevent thermal losses. Finally, water suppliers were included in the testing equipment to overcome the water evaporation during heating episodes.

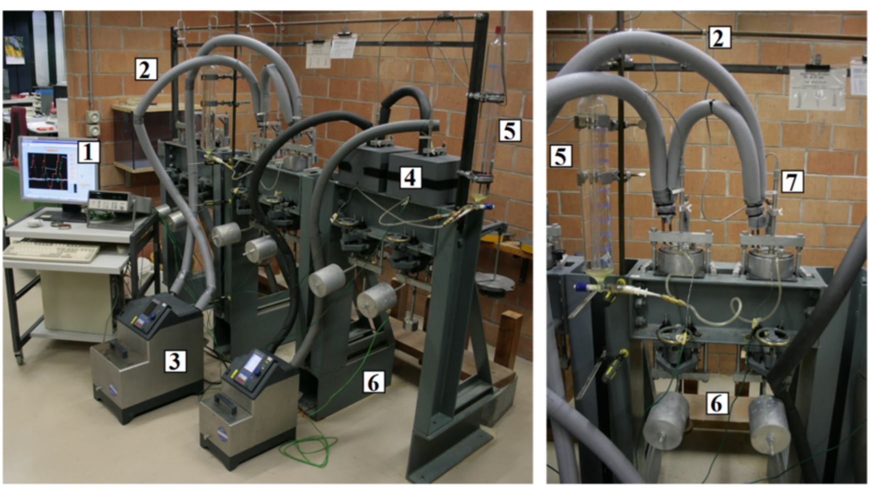

Figure 10. Thermal oedometer device (1: Acquisition system, 2: Tubes with circulating water, 3: Heater, 4: Insulation, 5: Water supplier, 6: Thermocouple, 7: LVDT).

Four soil samples collected from Geneva, Switzerland, were employed in the laboratory testing campaign. The samples were classified as silty-clay according to USGS classification and they were naturally in normally consolidated conditions. Further information on the soils samples, as well as the thermal oedometers is provided by [10].

The experimental program was designed to investigate two phenomena: (i) characterization of soil behavior in non-isothermal conditions, (ii) evaluation of soil response to thermal loads in cyclic nature. For the former aspect, oedometric tests at various constant temperatures (i.e. 20, 40, 60˚C) were performed, while for the latter one thermal cycles under constant vertical effective stress were applied on soil samples to which the emphasis is put in the present paper. Within the second testing campaign, the samples were loaded to a vertical stress at normal consolidation conditions in ambient temperature (20˚C), following which thermal cycles with a minimum temperature of 5˚C and maximum of 60˚C were imposed. Heating rate of 2˚C/h and a cooling rate of 5˚C/h were employed during the tests to prevent excess pore water pressure generation. Moreover, the same thermal cycles were applied to the soil samples under highly OC conditions, with an OCR of 16.

The results regarding the volumetric strain are presented in Figure 11. Thermo-elastic response corresponding to thermal expansion and compression of soil skeleton was observed for the clay sample in OC conditions. On the other hand, the NC sample showed irreversible volumetric contraction related to the thermo-plastic collapse. It is also observed that the sample accumulated irreversible contraction during the initial cycles, which becomes increasingly stabilized during the subsequent ones.

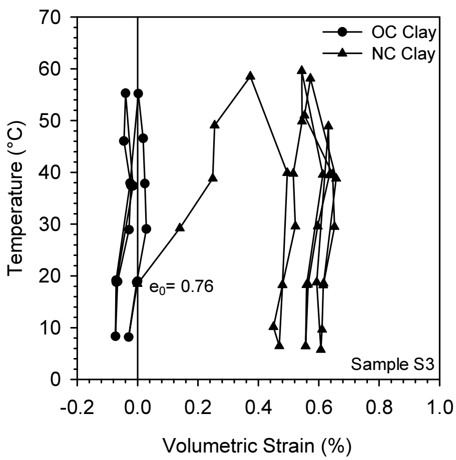

Figure 11. Volumetric response of NC and OC samples to thermal cycles.

The irreversible contractive behavior of NC clays can be mainly attributed to the particle rearrangement, which overcomes the thermo-elastic expansion of the grains and the water. For the OC clays, the particle rearrangement is limited which results in the thermo-elastic expansion of the soil constituents (water and grains) to govern. As the NC samples are continuously exposed to thermal cycles, the configuration becomes increasingly stable at each cycle, with decreasing possibility of additional collapse, suggesting a transition from an NC to OC condition [10].

To further investigate this phenomenon, the response of a clay sample mechanically loaded to a normally consolidated state and exposed to thermal cycles is presented in Figure 12. The sample was first mechanically loaded to 125 kPa (from point A to B), which was followed by four thermal cycles (between points B and C) and finally by mechanical loading up to 2000 kPa (from point C to E). It is observed in Figure 12a that during the final mechanical loading phase, the material first showed an elastic response from point C to D, which was followed by plastic response joining the normal consolidation line. Details on the volumetric response of the sample to cyclic thermal loads is shown in Figure 12b.

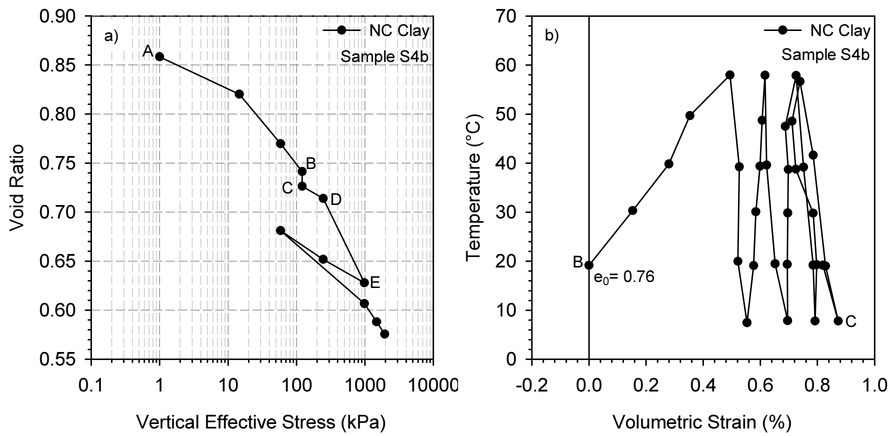

Figure 12. Effect of thermal cycles a) on the oedometric curve, b) on volumetric strain

3.3. Thermal Interface Shear Tests

The effects of cyclic temperature change on the behavior of energy piles can be considered in two main groups being; mechanical and thermal effects. The mechanical effects are the ones caused solely by the cyclic axial displacement of an energy pile due to elongation during temperature increase and contraction during temperature decrease. On the other hand, the thermal effects are due to the changes in soil behavior and soil-pile interaction caused as a consequence of temperature changes in the surrounding soil.

The second aspect, related to the response of soil-concrete interfaces to thermal variations, was investigated employing a modified direct shear device (Figure 13). Several modifications were realized on a conventional direct shear device to investigate soil-concrete interfaces in non-isothermal conditions: (i) a heating system was introduced which was composed of an electric resistance heater tissue, an electrical power supplier, an insulation and a thermocouple, (ii) the lower part of the shear box was redesigned to have a larger space with the purpose of accommodating the electrical heater as well as to have a constant contact area between the soil (60 mm x 60 mm x 10 mm) and concrete (60 mm x 105 mm x 16 mm) during shear. Regarding the placement of the samples, the heating tissue was placed first on the lower part of the shear box, which was protected by a metal plate to minimize its compression causing false normal displacement during the test. Next, the concrete sample was placed on the metal plate. Finally, the upper part of the shear box containing the soil sample was placed. Further information on the modified direct shear device is provided in [11].

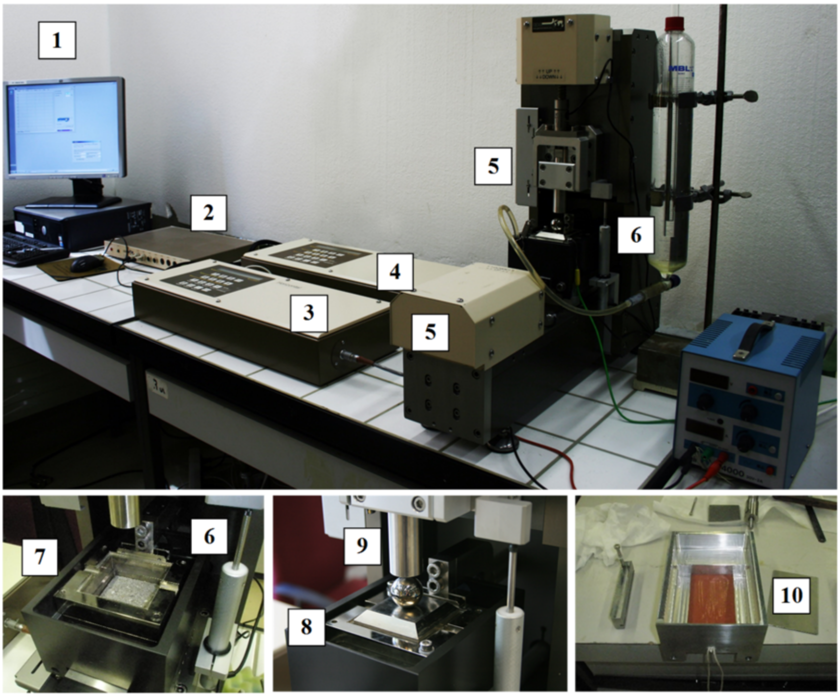

Figure 13. Thermal interface shear testing apparatus (1: GDSLAB software control, 2: LVDT Acquisition, 3: Normal actuator, 4: Horizontal actuator, 5: Load cells, 6: LVDT, 7: Specimen placement, 8: Top cap, 9: Axial piston, 10: Heating system).

The experimental campaign included tests on sand-concrete and clay-concrete interfaces at isothermal and non-isothermal conditions, under constant normal load (CNL) and constant normal stiffness (CNS). Furthermore, the effect of surface roughness on soil-concrete interaction was investigated by employing concrete samples with different degrees of roughness. CNS test results aiming attention at sand- and clay-concrete interface at different temperatures are discussed in this paper.

The CNS tests were initiated with a consolidation phase, which was followed by drained heating for the cases at high temperature. For clay-concrete interfaces, the heating was performed by increasing the temperature at a rate of 2 °C/h to prevent excess pore water pressure generation. Following the drained heating, once the desired temperature and associated deformation were stabilized, the shearing phase was applied under CNS conditions where constant normal stiffness of 500 or 1000 kPa/mm for the sand–concrete samples and 200 kPa/mm for the clay–concrete samples was imposed.

The results of CNS tests performed at ambient and elevated temperature are presented in Figure 14a and Figure 14b for sand-concrete and clay-concrete interfaces, respectively. It is observed from Figure 14a that the sand-concrete interface did not show a temperature-dependent behavior, which would be expected since sandy soils are known to be indifferent to temperature change. On the other hand, the response of clay-concrete interface showed a temperature-dependent behavior during cyclic CNS tests. It is observed from Figure 14b that the interface friction angle decreased slightly at high temperatures while the adhesion at clay-concrete interface increased. This behavior was attributed to the thermal consolidation of the clay which caused an increase of contact surface between clay and concrete surfaces.

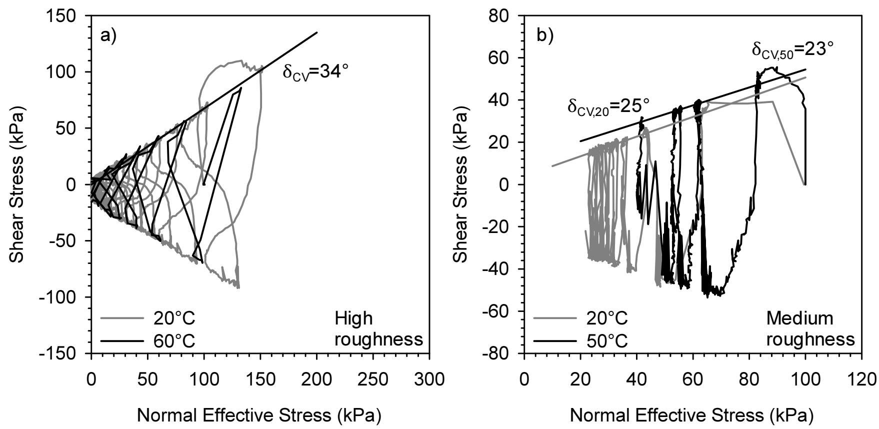

Figure 14. CNS tests on a) Sand-concrete interface (high roughness), b) Clay-concrete interface (medium roughness).

4. Summary and Discussions

Full-scale in-situ tests on energy piles, as well as laboratory test on soils and soil-concrete interfaces provided, without a doubt, insights of paramount importance on the multiphysical phenomena governing the behavior of energy piles.

The first part of this paper summarized outcomes of three full-scale in-situ tests that had diverse objectives but served the unique goal of conceiving the thermo-mechanical behavior of energy piles. The first in-situ test on a single energy pile, performed more than two decades ago at EPFL, revealed that geothermal operations cause changes in the stress state of an energy pile, the extent of which is highly governed by the end-restraining conditions. Furthermore, the null point, the depth at which no thermally induced displacement occurs, is associated with the maximum change in axial stresses and the direction of shaft resistance mobilization. The in-situ test performed on a group of closely-spaced, partially operating energy piles uncovered the interactions among active and non-operating energy piles: (i) thermally induced mechanical interactions which are governed by the changes in deformation field, due to the interplay between the pile-slab-soil responses, and (ii) thermal interactions which occur at later stages of the geothermal operation, due to the heat propagation from the active to the non-operating energy pile. The last in-situ test investigated the response of energy piles to cyclic thermal loads where cooling cycles below the in-situ temperature were applied. Concerning the outcomes of multiple heating-cooling cycles applied on the test pile, continuous thermal cycles did not cause (i) an accumulation of thermally induced axial stresses or (ii) degradation of mobilized shaft resistance with increased number of cycles. However, this outcome should not be generalized, considering the limited temperature change applied during the field test.

Design and development of non-isothermal laboratory testing devices (thermal triaxial cell, thermal oedometer and thermal direct shear device), as well as the outcomes of their employment to investigate thermo-mechanical behavior of soils and soil-concrete interfaces were presented at the second part of the paper. Since sandy soils are known to be characterized by elastic response to temperature changes, laboratory testing campaigns mainly focused on revealing the response of clayey soils to thermal loads.

The influence of temperature change on the stress-strain behavior of clayey soils was evaluated with the use of temperature-controlled triaxial testing device. The results of the test showed an increase in shear strength with temperature increase, which tend to the same critical state at large strains. Regarding the volumetric response of clayey soils, samples in OC conditions showed reversible dilative behavior while the NC ones accumulated irreversible contraction, especially during the initial cycles, which was mainly attributed to particle rearrangement. Finally, the results of soil-concrete interface shear test in non-isothermal conditions showed that sand-concrete interface was not affected by temperature changes. Contrarily, for the clay concrete interfaces, the interface friction angle decreased slightly at high temperature while the adhesion between clay and concrete increased which was attributed to the thermal consolidation of clay.

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