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The mechanisms underlying long-term shaft resistance enhancement of energy pile
in clays

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Abstract:

Although there are several studies indicating that heating increases the long-term shaft resistance of energy piles, the mechanisms by which heating causes this increase have not been adequately evaluated yet. This article aims to perform comprehensive analysis and discussion to assess the important factors contributing to this increase by integrating the findings from three recently published papers studying the thermo-mechanical behavior of clay and clay-pile interface. In these three studies, reconstituted kaolin clay was used, and cyclic and monotonic heat ranging between 24° C and 34°C were applied to the clay and interface. The interface was sheared under two stiffness boundary conditions; Constant Normal Stiffness (CNS) and Constant Normal Load (CNL), where normal stresses varied between 100 kPa and 300 kPa. The analysis performed in this article reveals that the increase in strength of interface under CNL condition is primarily attributed to clay stiffening at interface. However, the increase in shaft resistance under CNS condition is primarily attributed to the heating-induced increase of effective lateral stress, although clay stiffening at interface also partially contributes to the total increase of shaft resistance.

Key Words: Energy Pile, Long-Term Shaft Resistance, Clay-Pile Interface, Heat Cycles

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1- Introduction

There have been several successful implementations of energy piles (foundations) reported in the literature (Brandl 2006; Moel et al. 2010). They have resulted in an important contribution to the annual saving of energy cost in building sector by reducing the use of fossil fuels commonly used to generate energy (Brandl 2006; Moel et al. 2010). However, frequent temperature fluctuations and imbalance between the heat extracted and injected impair the long-term thermal efficiency of energy pile system (Brandl 2006). The performance of energy pile as foundation element can also be influenced by temperature variations in the pile and surrounding soil. For instance, additional settlement and axial load can develop in the pile which may jeopardize serviceability limit state and ultimate limit states, respectively (Bourne-Webb et al. 2006). Several researchers, however, have reported that heating improves the ultimate resistance of pile, particularly the ultimate shaft resistance (Di Donna et al. 2016; Ghaaowd and McCartney 2018; Goode and McCartney 2015; Maghsoodi et al. 2019; McCartney and Rosenberg 2011; Murphy and McCartney 2014; Ng et al. 2015; Szymkiewicz 2015; Wang et al. 2016; Xiao et al. 2014; Yazdani et al. 2019b; Yazdani et al. 2019c).

The following briefly presents findings from available literature about the temperature influences on the soil strength and shaft resistance. Note that only research works examining the clay and clay-concrete interface response at low temperature (up to 50 °C) and low mechanical stresses (corresponding to relatively shallow depths, where an energy pile operates) are summarized. Other studies carried out at very high temperature and mechanical loads, such as those investigating the response of clay under thermo-mechanical conditions representing high level nuclear waste disposal, where temperature and mechanical stress can increase up to 120°C and 4 MPa,

respectively, were not included in this paper (For example: Bruyn and Thimus 1996; Burghignoli et al. 2011; and Cekerevac and Laloui 2004).

Using a modified direct shear test device capable of applying thermal cycles, Xiao et al. (2014) reported that silt demonstrates a thermal-stiffening response (under constant normal load (CNL) condition) due to heat application. On the other hand, Yavari et al. (2016) found that the effect of temperature (in the range of 5-40 °C) on strength of reconstituted clay and compacted dry sand (under CNL condition) is minor. Based on the results of direct shear tests performed under CNL condition, Maghsoodi et al. (2019) showed that drained cohesion of reconstituted NC clays (prepared at a water content of 63% which was slightly higher than its liquid limit (LL), 57%) increases with temperature, while the effects of temperature on drained friction angle was found to be minor. The non-zero value of drained cohesion (which is typically expected to be zero for saturated NC clays) reported in the above study is probably due to the fact that the clay specimens were sheared in unsaturated condition because of relatively low water content ($\approx$ liquid limit) used in preparing the specimens. Using a modified triaxial testing apparatus, Yazdani et al. (2019a) indicated that cyclic heating-cooling and elevated temperature increase peak drained shear strength of reconstituted normally consolidated (NC) clays, while the drained shear strength of reconstituted overconsolidated (OC) clays decreases with temperature.

Temperature controlled direct shear test device has also been used to assess thermo-mechanical behavior of soil-pile interface. For instance, Xiao et al. (2014) showed that compacted silty soil-concrete interface under CNL condition presents thermal stiffening (up to 15%). Under CNL condition, Di Donna et al. (2016) observed an increase in drained shear strength of clay-concrete interface at elevated temperatures (20°C, 50°C and 60°C), while a temperature-independent shearing behavior for sand-concrete interface was noted. Yavari et al. (2016) reported that shearing

response of both sand and clay-concrete interface (under CNL condition) is nearly independent of elevated temperature (5°C, 20°C, and 40°C). Maghsoodi et al. (2019) showed that NC clay-concrete interface sheared under both CNL and constant normal stiffness (CNS) conditions shows higher drained shear strength at elevated temperatures, possibly because of thermal consolidation of clay. Yazdani et al. (2019a) reported that NC clay-concrete interface (under CNL condition) shows thermal stiffening, while OC clay-concrete interface shows thermal softening.

Centrifuge modeling test able to apply thermal loading is another effective approach that has been used for characterizing base and shaft resistances at non-isothermal conditions. For instance, McCartney and Rosenberg (2011) showed that heat cycles can improve the total loading and shaft resistance of energy piles in compacted silt, possibly because of the greater lateral pressure generated on the pile surface during thermal loading. Under different restrained boundary conditions, Goode and McCartney (2015) showed that ultimate resistance of centrifuge-scale semi-floating energy piles in unsaturated silt increases at elevated temperatures, while the temperature effects on ultimate resistance are minor for the piles embedded in dry sand. They, however, observed that ultimate resistance of end-bearing piles presents temperature independency. Ng et al. (2015) reported that both shaft and base resistances of centrifuge model piles in saturated sand are improved at elevated temperatures. Significant temperature-induced increase in pull-out capacity of centrifuge model pile (in NC clay) was observed by Gharaeifard and McCartney (2018).

1-g laboratory pile test has also been employed to study the temperature effects on shearing behavior of soil-concrete interface. Wang et al. (2011) reported that ultimate shaft resistance (in silica sand) tends to insignificantly decrease because of temperature increase (up to 40°C). Wang et al. (2016) indicated that shaft resistance of energy piles in dry sand can be influenced by heat exchange operation as the contact pressure at interface increases and decreases, respectively,

during heating and cooling. Yazdani et al. (2019c) reported that NC clay-concrete interface significantly increases under both monotonic and cyclic heating.

Furthermore, the results of a series of borehole shear test conducted by Murphy and McCartney (2014) indicated that clay-concrete interface sheared at elevated temperature shows greater strength comparing to that sheared at isothermal condition. The results of full-scale tests on energy piles (in sand and silt) also shows that heating improves total ultimate resistance (Szymkiewicz et al. 2015; Wang et al 2014).

The results of the above-mentioned direct shear and small/full scale tests reveal that heating increases the soil-pile interface strength. Note that most of the studies reviewed above reported that the interface shearing (or pile loading) occurred after pore water pressure (PWP) dissipation was achieved at interface, which represents long-term shaft resistance of energy piles. This stiffening is more clear in soils with low hydraulic conductivity such as saturated/unsaturated clays and silts. It has been reported that the heating-induced interface stiffening is likely attributed to one or a combination of the following mechanisms: heating-induced increase in contact stress at interface, thermal stiffening of soils around the pile, and thermal consolidation or moisture migration away from the interface (or towards the pile surface during cooling) in the case of presence of fully and partially saturated soils around the pile. However, the existing literature lacks quantitative assessment of contribution of these mechanisms to the heating-induced interface stiffening.

This article aims to examine the relative contribution of the above mechanisms to the thermally increased long-term strength of clay-concrete interface (or long-term shaft resistance) frequently observed in the literature. To reach the aim of the article, a discussion will be created with reference to the results of a series of direct shear and laboratory-scale pile tests (Yazdani et al. 2019b;

Yazdani et al. 2019c), where heating-induced interface stiffening was also observed. Although the results of these two studies have recently been published, independently analyzing them fails to assess the relative contribution of each individual mechanism to the overall interface stiffening. Therefore, the outcomes of these two studies (Yazdani et al. 2019b; Yazdani et al. 2019c) and the results reported in another experimental study of thermo-mechanical behavior of clay (Yazdani et al. 2019a) were qualitatively and quantitatively synthesized and discussed using different analysis approach (than those used in each individual study) to meet the aim of the article.

This article is organized as follows. In the Section 2 the terminology and assumptions used along this paper are presented. Afterward, a qualitative characterization of thermo-mechanical behavior of interface is first presented in the form of a simple mathematical equation (the Section 3), which is followed by the Section 4, where the material and testing approach utilized in the studies by Yazdani et al. (2019a), Yazdani et al. (2019b), and Yazdani et al. (2019c) are shortly presented. In the Section 5, the findings of those studies are first briefly overviewed followed by more detailed and synergistic utilization of the results to interpret the heating-induced stiffening of interface. In the Section 6, the main conclusions are presented.

2- Assumption and Terminology

The key assumption underlying this article is that the drained shear strength parameters of clay measured in the consolidated drained (CD) triaxial tests (Yazdani et al. 2019a) describe the drained shear strength of clay in the vicinity of interface in the tests performed by Yazdani et al. (2019b) and Yazdani et al. (2019c). However, in the following, the conditions that may create violations of the underlying assumption will be briefly discussed to check the possible limitations associated with the analyses performed in this study. Different saturation condition and structure (or fabric)

of clay specimens in the above-mentioned tests (Yazdani et al. 2019a, Yazdani et al. 2019b, and Yazdani et al. 2019c) are considered to understand how the underlying assumption might be violated.

With respect to saturation condition, the clay specimens used by Yazdani et al. (2019a) were fully saturated due to back pressure saturation in triaxial cell, while those used in the other two test series (Yazdani et al. 2019b and Yazdani et al. 2019c) might have been unsaturated as back pressure saturation was not possible in the direct shear and small-scale pile tests. This, in turn, might create a violation of the assumption as unsaturated soils may show different shear strength because of capillary action. However, it is generally accepted that drained shear strength parameters at critical state (such as critical state friction angle) are almost unique at different saturation (Geiser et al. 2006; Khalili et al. 2004). On the other hand, it was reported by Khalili et al. (2004) that the peak drained shear strength parameters might change with degree of saturation, though they showed this change is insignificant. Therefore, with respect to saturation condition, the assumption is reasonably expected to be satisfied.

Regarding clay structure, all clay specimens used in the three experimental sets were initially reconstituted (under Ko-condition) meaning that they all initially had a dispersed structure (face to face arrangement). However, the specimens used in triaxial tests (Yazdani et al. 2019a) underwent a change of fabric adjustments due to isotropic pressure that probably caused a gradual transition from dispersed structure to flocculated structure (face to edge arrangement). On the other hand, one-dimensional pressure employed in the direct shear tests (Yazdani et al. 2019b) and the 1g pile models (Yazdani et al. 2019c) increased dispersion degree of the specimens used in these tests. Thus, it can be suggested that the clays specimens in the three sets of tests probably have different structures; while a flocculated structure (face to edge arrangement) is more likely to be

generated in triaxial cell (Yazdani et al. 2019a), dispersed structure (face to face arrangement) is expected at the interface (Yazdani et al. 2019b and Yazdani et al. 2019c).

Flocculated clays typically have larger void ratio and thus higher compressibility in comparison to the clays with dispersed structure (Mitchell 1993). On the other hand, flocculated structure typically causes higher strength than dispersed structure does (Mitchell 1993). Thus, it may be expected that the clay near the interface shows different (probably lower) drained shear strength compared to that sheared in the triaxial tests, although all specimens were initially reconstituted using the same clay, water, moisture content, and loading steps.

Throughout this paper, long-term shaft resistance refers to the interface shear strength at steady state, under which the heating-induced PWP is entirely dissipated and temperature changes stabilize at the soil-pile interface. Practically, this situation may occur after complete cessation of energy pile operation, where full recovery of temperature and PWP (complete dissipation of PWP) is almost achieved, as reported by Faizal et al. (2016). This situation may also occur in 24-hour continuous operation mode, where energy piles run for weeks or months without stopping. Based on their observation of thermal behavior of full-scale energy piles tested under 24-hour continuous operation mode, Faizal et al. (2016) reported that full stabilization of temperature and complete dissipation of excess PWP was achieved at interface few days after beginning the experiments and maintained till the end of operation.

Short-term shaft resistance, on the other hand, refers to the interface shear strength at transient state, under which excess PWP and temperature at interface keep changing during energy pile operation. Practically, the short-term shaft resistance occurs when energy piles operate under intermittent mode of operations, where they are operational only for few hours per day (with a resting time) as reported by Faizal et al. (2016).

3- Background

The ultimate unit shaft resistance q_s at isothermal condition is proportional to the drained friction angle available at clay-pile interface δ and the effective normal stress acting at interface σ'_h , which can mathematically be presented in the following equation:

$$q_s = \sigma'_h \tan(\delta) \quad (1)$$

δ is dependent on varying parameters including pile surface roughness, particle size/shape and strength/deformability characteristics of the surrounding clay (Lemos and Vaughan 2000). Due to low temperature variations typically occurring at an energy pile interface, the effect of thermal change of geometrical properties mentioned above (i.e. the roughness and particle size/shape) on δ would reasonably be minor. Therefore, the heating-induced change in clay strength and deformability characteristics is possibly responsible for causing any variation in interface friction angle during heat application (i.e. $\Delta\delta_{th}$). It can be described with temperature effects on clay microscopic structure and/or thermal consolidation occurring within the clay mass.

Shaft friction degradation may also occur due to the heating-induced cyclic displacement along interface, similar to that observed by several researchers for the piles cyclically loaded (Mortara et al. 2007; Tabucanon et al. 1995; Uesugi et al. 1989; Uesugi et al. 1990; Zhang and Zhang 2006). This is primarily attributed to the fact that soils (particularly sands) near pile surface tend to show contractive behavior under cyclic shearing, causing a progressive decrease in the effective normal stress acting at interface, thus reducing the interface frictional resistance. This degradation phenomenon has mainly been observed for the interface between sandy soils and different construction material (Di Donna et al. 2016, Mortara et al. 2007; Tabucanon et al. 1995; Uesugi et al. 1989; Uesugi et al. 1990; Zhang and Zhang 2006), while the response of clay-concrete interface under cyclic shearing has not been investigated well. Nevertheless, a progressive volume decrease

in clay is also expected due to cyclic shear that may cause a reduction in confining lateral pressure, an increment in PWP at interface, as well as shaft friction degradation. However, shaft degradation is less significant at clay-concrete interface since clays volumetric reduction toward cyclic shearing forces is generally lower than that in sandy soils. Furthermore, depending on the amount of contraction accumulated at the end of heat cycles, the “post-heat cycling” interface strength can be altered, that should be considered in analysis of long-term performance of energy piles.

σ'_h can also vary due to the heating-induced excess PWP, which is strongly dependent on the rate of heating/cooling as well as clay hydraulic conductivity (Fuentes et al. 2016). The induced excess PWP is due to differential expansion between clay particles and pore water (Campanella and Mitchell 1968). If drainage is allowed (even at a slow rate), the excess PWP generated during heating will dissipate with time, leading to a thermal consolidation in clay. Conversely, when the interface is cooled down to the initial temperature (or lower temperature), depending on rate of temperature change development a negative PWP (suction) may occur in clay followed by restoration if there is enough time. The earlier case can lead to a reduction in σ'_h and shaft resistance, while the latter can enhance both.

The above complex interaction between the mechanical stresses-PWP-mechanical properties available at interface, ultimate unit shaft resistance and heating-cooling cycles can be expressed in the following mathematical form;

$$q_s = (\sigma'_{h0} + \Delta\sigma'_{h-th}) \tan(\delta + \Delta\delta_{th}) \quad (2a)$$

$$\Delta\sigma'_{h-th} = \Delta\sigma_{h-th} - \Delta u_{th} \quad (2b)$$

Where, σ'_{h0} is effective lateral stress before heat application, and $\Delta\sigma'_{h-th}$ and $\Delta\delta_{th}$ are thermally induced changes in effective lateral stress and interface friction angle, respectively, with a sign,

either positive or negative. $\Delta\sigma_{h-th}$ and Δu_{th} are thermally induced changes in total lateral stress and PWP, respectively.

In general, any experimental study aiming to assess the shaft resistance of an energy pile in saturated clays requires to tackle the questions arising from the above discussion, which are: how heat exchange between pile and clay can change strength parameters of clay, distribution of effective lateral stresses and PWP at the interface.

4- Material, testing apparatus, and approach

The modified triaxial testing apparatus used by Yazdani et al. (2019a) is shown in Figure A (appendix). The test setup was able to test soils at non-isothermal conditions. Reconstituted specimens of Kaolin clay, prepared (using deaired water) at moisture content two times the liquid limit (LL) of clay, were used in this study. Figure 1 and Table 1 show grain-size distribution of the clay and its basic physical properties, respectively. The specimen temperature, measured by a thermocouple attached to its side, was controlled by circulating a heat carrier fluid through a hollow metal coil within the triaxial cell wall. Four main successive steps were involved in building this experimental test: application of *back-pressure saturation*, *isotropic pressure (at ambient temperature $\approx 24\text{ }^{\circ}\text{C}$)*, *drained specimen temperature variation* and *deviatoric stress (drained shearing)*. NC clay specimens were sheared at isotropic pressure of 150 kPa, 200 kPa, and 300 kPa, while isotropic pressure of 100 kPa was used to shear OC clay specimens with over consolidation ratios (OCR) of 2, 5, and 9. The clay specimens were subjected to non-cyclic heating (i.e. 24-34 $^{\circ}\text{C}$) and cyclic heating-cooling (i.e. 24-34-24 $^{\circ}\text{C}$) under drained condition (for more details on the test approach see Yazdani et al. 2019a).

Figure 1- Particle size distribution of Kaolin clay (After Yazdani et al. 2019a).

Table 1- Physical properties of kaolin clay (After Yazdani et al. (2019a))

Parameter	Value
D ₅₀	0.0026 mm
Plastic Limit (PL)	25 %
Liquid Limit (LL)	45 %
Plastic Index (PI)	20 %
Poisson's Ratio (ν)	0.25
G _s	2.62

The modified direct shear test utilized by Yazdani et al. (2019b) is shown in Figure B (appendix). To reproduce heating-induced cyclic shear between energy pile and soil, one end of the concrete plate was free to thermally expand/contract, while the other end was affixed to the wall. Similar clay and specimen preparation method was also used in this study (i.e. reconstituted kaolin clay at moisture content of 2 LL). The interface temperature was controlled by circulating heat carrier fluid through a W-shaped copper tube embedded in the concrete plate. The testing program consisted of three main steps: *normal stresses application (at ambient temperature ≈ 24 °C)*, *drained interface temperature variation* and *(drained) interface shearing under CNL condition*.

The NC clay-concrete interface was sheared at normal stresses of 150 kPa, 225 kPa and 300 kPa, while the OC clay-concrete interface (with OCR = 2 and 5) was sheared at normal stress of 150 kPa. Similar non-cyclic heating (24-34 °C) and cyclic heating-cooling (24-34-24 °C) were employed in this study (for more details on the test approach see Yazdani et al. 2019b).

The test setup used by Yazdani et al. (2019c) is shown in Figure C (appendix). The model concrete pile and surrounding clay were placed in a cylindrical container with an inner diameter and a height of 170 mm and 300 mm, respectively. Similar clay and specimen preparation method were also used in this study (i.e. reconstituted kaolin clay at moisture content of 2 LL). The clay was reconstituted around the pile, fixed at its base, under a vertical pressure of 100 kPa. Therefore, the clay in this test was NC clay. As shown in Figure C, the model pile exclusively resisted the applied axial loads through the shaft resistance. The experiment was conducted in three successive steps including application of *overburden pressure* ($= 100 \text{ kPa}$, at ambient temperature $\approx 24 \text{ }^{\circ}\text{C}$), *interface temperature variations* and *pile loading* (at constant strain rate of 0.25 mm/min). Temperature change conditions similarly included non-cyclic heating (24-34 °C) and a number of heat cycles (24-34-24 °C). The temperature of the clay-pile interface was controlled by circulating a heat carrier fluid inside the pile through two holes made at the top and bottom of the pile, as shown in Figure C. The pile loading occurred after all temperature-induced PWP at interface was dissipated.

5- Analysis and discussion of the laboratory test results

In this section the results of the study conducted by Yazdani et al. (2019a) are shortly presented first. The results allow for observing the heating-induced changes in shear strength of cl, and thus the subsequent effects on the friction angle at interface (δ), as discussed previously in the Section

3. Afterward, the results from the second and third test series (Yazdani et al. 2019b; Yazdani et al. 2019c) are briefly reviewed and integrated with the first set of results (Yazdani et al. 2019a) followed by detailed analysis and discussion to provide more complete understanding of interface response at non-isothermal condition.

5-1-Temperature controlled CD triaxial test results (Yazdani et al. 2019a)

Table 2 summarizes the results of the tests performed by Yazdani et al. (2019a), including the initial confining pressure (σ_0), thermal loading path, peak deviatoric stress ($\Delta\sigma_d$), and the estimated effective normal stress (σ'_n) and shear stress (τ_f) at failure plan (corresponding to the peak deviatoric stress). σ'_n and τ_f were determined by drawing the line crossing the origin and tangent to the effective stress Mohr's circle plotted for each individual effective confining pressure (σ_0) and its corresponding peak deviatoric stress ($\Delta\sigma_d$).

Table 2- CD triaxial test results [Data from Yazdani et al. (2019a)]

Test #	OCR	σ_0 (kPa)	Thermal loading path (experienced by clay)	$\Delta\sigma_d$ (kPa)	σ'_n (kPa)	τ_f (kPa)
1	1	150	24 °C	222.3	213.8	100.6
2	1	150	(24 – 34 °C)	241.6	216.9	108.1
3	1	150	(24 – 34 – 24 °C) - 10 cycles	251.3	218.4	111.8
4	1	200	24 °C	306.71	286.8	138.2
5	1	200	(24 – 34 °C)	303.44	286.3	136.9
6	1	200	(24 – 34 – 24 °C) - 10 cycles	318	288.6	142.6
7	1	300	24 °C	449.56	428.5	203.1
8	1	300	(24 – 34 °C)	455.75	429.5	205.5
9	1	300	(24 – 34 – 24 °C) - 10 cycles	481.63	433.6	215.6
10	2	100	24 °C	169.7	145.9	75.4
11	2	100	(24 – 34 °C)	163.63	145	73.1
12	2	100	(24 – 34 – 24 °C) - 10 cycles	162.89	144.9	72.8
13	5	100	24 °C	186.56	148.3	81.7
14	5	100	(24 – 34 °C)	170.65	146	75.7
15	5	100	(24 – 34 – 24 °C) - 10 cycles	175	146.7	77.4
16	9	100	24 °C	198.53	149.8	86.1
17	9	100	(24 – 34 °C)	190.8	148.8	83.3
18	9	100	(24 – 34 – 24 °C) - 10 cycles	171.5	146.2	76.1

σ_0 : Initial confining pressure, $\Delta\sigma_d$: Peak deviatoric stress, σ'_n and τ_f : Estimated normal and shear stress on the failure plan

304 Table 2 indicates that the thermo-mechanical response of clay is critically OCR-dependent: while
305 the NC clay specimens demonstrate a thermal stiffening behavior, the peak deviatoric stress of OC
306 clays decrease with temperature. Similar observations were also reported by other researchers
307 (Baldi et al. 1993; Baldi et al. 1991; Hueckel and Baldi 1990; Tanaka et al. 1997). Two possible
308 contributing factors were considered by Yazdani et al. (2019a) to interpret the thermo-mechanical
309 behavior of clay observed in Table 2: heating-induced change of contact stress and inter-particle
310 forces, and heating-induced volumetric strain of clay. In their study, however, they showed that
311 the volumetric strain due to applying temperature variations is always less than 0.25% for tested
312 specimens, according to which they concluded that this small volumetric strain does not
313 importantly contribute to the overall change in the shear strength (up to 14 % in OCR case, as
314 shown in Table 2). Therefore, it was suggested by them that the clay shearing response to drained
315 heating is controlled by the heating-induced redistribution of contact (surface) stress and physico-
316 chemical forces¹ between clay particles, which are likely caused by thermal rearrangement of clay
317 particles during thermal consolidation, as also observed by Darbari et al. (2017). However, the
318 changes of these two forces (due to drained heating) must be equal and opposite in sign so the
319 overall changes of effective stress² between clay particles remain zero and the clay structure can
320 carry the same effective stress. Similarly, under larger temperature variations (up to 65 °C), Trani
321 et al. (2010) experimentally showed that the thermally induced increase of shear strength is
322 attributed to thermal volume change or/and temperature-induced redistribution of contact stress
323 and physico-chemical forces (due to restructuring of clay particles). However, unlike the findings

¹ The physico-chemical forces between two clay particles is a combination of repulsive interaction due to negatively charged clay surfaces and attractive interaction related to Van der Waals forces (Mitchell 1993).

² In the soil mechanics literature, effective stress in clay is generally defined as the sum of contact stress and physico-chemical forces between clay particles (Mitchell 1993; Hueckel 1992).

of the study by Yazdani et al. (2019a), Trani et al. (2010) indicated that thermal volume change contribution to the increase of strength is not ignorable for the clays they tested, probably because of larger temperature change applied.

5-2-Temperature-controlled direct shear test results (Yazdani et al. 2019b)

A summary of the results reported by Yazdani et al. (2019b) is presented in Table 3. The normal stress σ'_n , peak shear stress τ_{peak} , residual shear stress (i.e. the shear stress at shear displacement of about 10 mm) τ_{res} , and thermal loading path are presented.

Table 3- Direct shear test results [Data from Yazdani et al. (2019b)].

Test #	OCR	σ'_n (kPa)	Thermal loading (experienced by interface)	τ_{peak} (kPa)	τ_{res} (kPa)
1	1	150	24 °C	54.13	41.52
2	1	150	(24 – 34 °C)	54.1	40
3	1	150	(24-34-24 °C)-10 cycles	54.7	42.8
4	1	150	(24-34-24 °C)-20 cycles	56.6	-
5	1	150	(24-34-24 °C)-40 cycles	55.74	-
6	1	225	24 °C	67.92	53.6
7	1	225	(24 – 34 °C)	72.19	54.13
8	1	225	(24-34-24 °C)-10 cycles	79.38	-
9	1	300	24 °C	82.53	78.26
10	1	300	(24 – 34 °C)	90.3	76
11	1	300	(24-34-24 °C)-10 cycles	91.06	81.75
12	2	150	24 °C	54.8	49.63
13	2	150	(24 – 34 °C)	51.2	45.85
14	2	150	(24-34-24 °C)-10 cycles	49.3	41.47
15	5	150	24 °C	62.25	50.46
16	5	150	(24 – 34 °C)	60.6	-
17	5	150	(24-34-24 °C)-10 cycles	57.47	42.2

OCR: Overconsolidation ratio of clay, σ'_n : Normal stress, τ_{peak} : Peak shear strength at interface, and τ_{res} : Residual shear strength at interface

Table 3 indicates that heat stiffening and heat softening of NC clay-concrete and OC clay-concrete interface, respectively, similar to the NC and OC clay responses to heating presented in Table 2. The following analysis is made to determine the contributing factors to the interface thermo-mechanical response observed under CNL condition (Table 3). In this analysis, it is assumed that

the clay at the interface is fully saturated, as previously stated in the introducing the underlying assumption. In the end of this section, however, a short alternative analysis will be performed to evaluate the interface response to temperature variation in unsaturated state.

The analysis consists of integration the output from Yazdani et al. (2019a) (Table 2) and Yazdani et al. (2019b) (Table 3) and analyzing them using Equation 2. It should be mentioned that Equations (1) and (2) defining the ultimate unit shaft resistance of pile (q_s) is also applicable to the unit interface frictional resistance (τ_s) developing in direct shear test since σ'_h and δ can interchangeably illustrate the effective normal stress (σ'_n) and friction angle between the concrete plate and clay in direct shear test. The observed interface response to heating is largely explained therefore, by possible heating-induced changes in effective normal stress σ'_n , and interface friction angle δ .

Since the tests were conducted under CNL condition, any possible changes in the total normal stress during both temperature variation and shearing steps are expected to be zero, which mathematically means $\Delta\sigma_{h-th} = 0$. Similarly, the heating-induced PWP change is most likely to be zero ($\Delta u_{th} = 0$), as the interface temperature variation was applied under drained condition. Note that the shearing also occurred under drained condition (at a rate of 0.005%/min). This is in turn concluded that the heating-induced change in effective normal stress throughout a test is zero ($\Delta\sigma'_{h-th} = 0$). Therefore, according to Equation (2), it is suggested that the heating-induced change in interface strength under CNL condition is linked to $\Delta\delta_{th}$, which is directly proportional to the heating-induced change of internal friction angle of soil ($\Delta\phi_{th}$). It can be better explained as follows:

The frictional resistance developing between clay and concrete slightly before the peak strength, is primarily offered by a thin layer of clay (near the interface), which undergoes a uniform shear

361 deformation during interface shearing (Goh and Donald 1984; Tsubakihara and Kishida 1993). It
 362 is followed by a relative displacement (sliding) at interface and a gradual reduction in the
 363 mobilized frictional resistance as shearing continues (Goh and Donald 1984; Tsubakihara and
 364 Kishida 1993). Accordingly, the strength/stiffness of clay in the vicinity of interface makes an
 365 important contribution to the total frictional resistance offered by the interface, that thus suggests Δ
 366 δ_{th} and the overall change in frictional resistance (under CNL condition) are linked to $\Delta\phi_{th}$.
 367 The above qualitative explanation can be further expanded by examining the data presented in
 368 Figure 2, where a comparison has been made between the peak friction coefficient estimated for
 369 the interface ($\tan\delta_p$) and clay (in the triaxial tests) ($\tan\phi_p$) under different temperature variations.
 370 For NC clay presented in Figure 2, $\tan\phi_p$ was estimated by plotting the best-fit tangent to the three
 371 Mohr's circles at $\sigma_0 = 150$ kPa, 200 kPa and 300 kPa for each thermal loading path. Similarly, for
 372 NC clay-concrete interface, $\tan\delta_p$ is the slope of best fit line through the peak strength points
 373 measured at $\sigma'_n = 150$ kPa, 225 kPa, and 300 kPa. However, since the OC clays and OC clay-
 374 concrete interface were only tested at $\sigma_0 = 100$ kPa and $\sigma'_n = 150$ kPa, respectively, one Mohr's
 375 circle was available for each thermal loading path. As a result, for OC case, $\tan\phi_p$ and $\tan\delta_p$ were
 376 directly determined by dividing τ_f (Table 2) and τ_{peak} (Table 3) by σ'_n (Tables 2 and 3),
 377 respectively (i.e. $\tan\phi_p = \frac{\tau_f}{\sigma'_n}$, data from Table 2, and $\tan\delta_p = \frac{\tau_{peak}}{\sigma'_n}$, data from Table 3).

Figure 2- Peak friction coefficient versus thermal loading path (Test1: 24 °C, Test2: (24-34 °C), and Test3: (24-34-24 °C)-10 cycles) (Data from Yazdani et al. (2019a) and Yazdani et al. (2019b)).

From Figure 2, it can be seen that temperature effects on $\tan\delta_p$ is qualitatively similar to those on $\tan\phi_p$; while heating produces higher friction coefficient for the NC clay and NC clay-concrete interface, a heating-induced decrease can be noted for the OC clay and OC clay-concrete interface. For example, looking at Table 4 summarizing the data in Figure 2, $\tan\phi_p$ ($\tan\delta_p$) for the NC case rises from 0.47 (0.3), at ambient temperature, up to 0.48 (0.325) and 0.5 (0.33) because of non-cyclic heating and cyclic heating-cooling, respectively. As for the OC case, for instance OCR=2, $\tan\phi_p$ ($\tan\delta_p$) drops from 0.52 (0.37), at ambient temperature, down to 0.5 (0.34) and 0.5 (0.33) because of non-cyclic heating and cyclic heating-cooling, respectively.

387 However, there is to some extent quantitative disagreement between $\Delta\delta_{th}$ and $\Delta\phi_{th}$. In fact, the
 388 effect of temperature on the interface friction angle (under CNL condition) is greater than that on
 389 the internal friction angle (i.e. $|\Delta\delta_{th}| > |\Delta\phi_{th}|$), particularly for the NC and lightly OC clays. It
 390 may be attributed to the different structures developing in the clay specimens in the triaxial tests
 391 (Yazdani et al. 2019a) and the direct shear tests (Yazdani et al. 2019b), as already explained in the
 392 Section 2. While a flocculated arrangement of particles is expected to be generated under isotropic
 393 pressure (in the triaxial tests), parallel arrangement is more likely under one-dimensional pressure
 394 (in direct shear tests). Furthermore, the clay specimens in the triaxial tests and those near the
 395 interface are sheared under different boundary conditions that may result in different friction
 396 angles. However, the highest heating-induced change in δ and ϕ is approximately 11% and 7%,
 397 respectively, which are relatively close to each other. This in turn indicates that, in the range of
 398 temperature applied by Yazdani et al. (2019a) and Yazdani et al. (2019b), the heating-induced
 399 change in δ under CNL condition is not significant and mainly related to the heating-induced
 400 change in ϕ of the clay at interface.

401

Table 4- Summary of data presented in Figure 2 (Data from Yazdani et al. (2019a) and Yazdani et al. (2019b)).

Clay stress history	OCR=1 (NC)			OCR=2			OCR=5		
	$\tan\delta_p$	$\tan\phi_p$	$\frac{\tan\delta_p}{\tan\phi_p}$	$\tan\delta_p$	$\tan\phi_p$	$\frac{\tan\delta_p}{\tan\phi_p}$	$\tan\delta_p$	$\tan\phi_p$	$\frac{\tan\delta_p}{\tan\phi_p}$
24 °C	0.3	0.47	0.64	0.37	0.52	0.71	0.415	0.55	0.75
(24-34 °C)	0.32 (8.3%)	0.48 (2.1%)	0.67	0.34 (-8.1 %)	0.5 (-3.85%)	0.68	0.4 (-3.6%)	0.52 (-5.4%)	0.77
(24-34-24)-10 cycles	0.33 (10%)	0.5 (6.4%)	0.66	0.33 (-10.8%)	0.5 (-3.85%)	0.66	0.38 (-8.43%)	0.52 (-5.4%)	0.72

402

403 The variation of interface friction coefficient at residual condition (at large displacement), $\tan\delta_r$,
 404 is also plotted versus thermal loading path in Figure 3. Note that $\tan\delta_r$ for NC clay-concrete

interface and OC clay-concrete interface was determined using the data shown in Table 3 (i.e. σ'_n and τ_{res}), in a similar way previously described for $\tan\delta_p$ determination. The figure does not show the residual friction coefficient for the clay specimens (in the triaxial tests), as it was not attained in the range of applied displacement. Therefore, it is not possible to make general suggestion similar to that made for the peak shear strength. However, it can be noted that the residual friction coefficient also shows similar OCR-dependent response to heating; while heat stiffening of NC clay-concrete interface is observed at large displacements, the residual strength of OC clay-concrete interface presents heat softening.

Figure 3- Residual friction coefficient versus thermal loading path (Test1: 24 °C, Test2: (24-34 °C), and Test3: (24-34-24 °C)-10 cycles) (Data from Yazdani et al. (2019b))

Another contributing factor to the thermo-mechanical response of interface under CNL condition could be clay stiffening due to cumulative clay contraction developed during the heating-induced cyclic shear at interface. This phenomenon has been observed for the sand-concrete interface cyclically sheared under CNL condition, where the “post-cyclic” strength increases because of sand densification during cyclic shearing (Mortara et al. 2007; Tabucanon et al. 1995). To assess the contribution of this phenomenon to the increase in interface strength, the volume change of clay (above the concrete surface) during 10 cycles of heating-cooling is shown in Figure 4 (contraction is positive here). Note that the presented volume change corresponds to the end of each heating-cooling cycle when the interface was cooled back down to 24 °C. The data shown in Figure 4 was carefully calibrated based on thermal expansion/contraction of concrete plate measured in a series of calibration tests performed prior to the actual tests (Yazdani et al. 2019b). It can be seen that the overall change in clay volume (during 10 heating-cooling cycles) is insignificant (the greatest change is 0.74% for the NC clay-concrete interface at $\sigma_n = 150$ kPa). Therefore, clay stiffening due to cumulative contraction and its contribution to the long-term stiffening of interface is minor. This phenomenon is actually expected due to very small amplitude of linear thermal expansion of concrete plate (i.e. tangential displacement along the interface), which is estimated to be 0.0225 mm (considering $\alpha_{\text{concrete}} = 1.5 \times 10^{-5} \frac{1}{^\circ\text{C}}$, $L_{\text{concrete}} = 150$ mm, and $\Delta T = 10$ °C (Data from Yazadani et al. 2019b)).

Figure 4- Heating-induced volumetric strain of clay above the interface (during 10 heat cycles)
(Data from Yazdani et al. (2019b)).

According to the above analysis and discussion, the mechanisms contributing to the thermo-mechanical response of interface under CNL condition (studied by Yazdani et al. (2019b), Maghsoodi et al. (2019) and Xiao et al. (2014)) can be summarized as follows:

(1) Heat stiffening of NC clay is the key contributing factor to the heat stiffening of NC clay-concrete interface. Likewise, the softening of OC clay-concrete interface appears to be mainly attributed to heat softening of OC clay.

(2) In the range of temperature applied by the above authors, the contribution of both cumulative clay contraction (due to heating-induced cyclic shear at interface) and clay thermal consolidation is found to be blunted.

5-3-Temperature controlled 1g lab-scale pile test results (Yazdani et al. (2019c))

Figure 5 illustrates the (compressive) load-settlement response of the pile after it experienced different thermal loading path. Note that the temperature variations presented in the figure are those measured at the pile surface. Temperature within the surrounding clay at distances of one and two pile diameters was also reported by Yazdani et al. (2019c), where the highest temperature change experienced by the clay was $\approx 9^\circ\text{C}$ and $\approx 7^\circ\text{C}$, respectively (it was 10°C at the pile surface) (Yazdani et al. 2019c). In other words, temperature variation recorded within the clay at these two distances (during one heat cycle) was $(24-33-24^\circ\text{C})$ and $(24-31-24^\circ\text{C})$, respectively, while it was $(24-34-24^\circ\text{C})$ at the pile surface, as mentioned in the Section 4.

It is evident that heating has significant effects on the shaft resistance of the pile. The pile loaded after it was subjected to non-cyclic heating $(24-34^\circ\text{C})$ showed an increase of 100% in the shaft resistance relative to the pile sheared at isothermal condition, while the piles subjected to cyclic heating-cooling showed almost similar increase of 50%. Similar heating-induced increase in shaft resistance (up to 95%) was also reported by other researchers using small-scale and centrifuge-scale tests (for example: Ghaaowd and McCartney 2018; Goode and McCartney 2015; Murphy and McCartney 2014).

An analysis, by integrating the data from Yazdani et al. (2019c) (Figure 5) with the data from Yazdani et al. (2019a) (Table 2) and Yazdani et al. (2019b) (Table 3), is made to interpret the increased shaft resistance observed in Figure 5. Since the clay-pile interface in this study was sheared under CNS condition, this analysis can be applied to characterize the shaft resistance improvement observed in other studied carried out under CNS condition (e.g. Ghaaowd and McCartney 2018; Goode and McCartney 2015; McCartney and Rosenberg 2011; Wang et al. 2016; Yazdani et al. 2019c; Szymkiewicz et al. 2015; Wang et al. 2014).

Figure 5- (a) load-settlement curves for the model pile under different thermal loading (After Yazdani et al. (2019c); (b) corresponding maximum pile load (Test1: 24 °C, Test2: (24-34 °C), and Test3: (24-34-24 °C)-10 cycles, Test4: (24-34-24 °C)-20 cycles and Test5: (24-34-24 °C)-40 cycles).

Before discussing the data in Figure 5, it is worthwhile to quantitatively compare the outcomes of the direct shear tests performed under CNL condition (Yazdani et al. 2019b) and 1g small-scale

tests conducted under CNS condition (Yazdani et al. 2019c) at both isothermal and non-isothermal states, as both testing approaches aim at characterizing the frictional resistance of interface. Assuming that the effective normal stress acting round the pile shaft σ'_h is 58 kPa (considering $\sigma'_h = K_0 \sigma'_v$, in which $K_0 = 1 - \sin \phi'_{\text{clay}}$, $\phi'_{\text{clay}} = 25^\circ$ (Table 4) and $\sigma'_v = 100$ kPa), the peak interface friction coefficient $\tan \delta_p$ at isothermal condition is estimated to be 0.16 (= “pile axial load” ÷ “effective lateral load on the pile surface”), which is almost 50% lower than that (0.3) measured using the direct shear test (Table 4). It is apparently because the pile surface is smoother than the concrete surface used in the direct shear tests, as shown in Figure 6.

Figure 6- Roughness profile of interface (laser photography): (a) pile surface; (b) concrete plate (After Yazdani et al. (2019b)).

According to the method of Arithmetic mean deviation (Equation (3)), which is commonly used for characterizing the clay-solid interface roughness (Goh et al. 1984; Hammoud et al. 2006), average roughness is estimated to be 4.48 μm and 28.8 μm for the pile surface and the concrete plate (used in direct shear test), respectively.

$$R = \frac{1}{n} \sum_{i=1}^n |y_i| \quad (3)$$

Where, R is the surface relative roughness, n is the number of sampling points and y is the height of asperities at the sampling points. 366 sampling points were used to calculate the interface roughness for each profile shown in Figure 6.

At non-isothermal condition, however, the “ $\tan\delta$ - based” comparison of interface resistance may not be applicable since the lateral stress is unlikely to remain constant ($= 57$ kPa) during heat application. Instead, the peak and residual shear strength of interface (i.e. τ_{peak} and τ_{res}) and their corresponding ratios at non-isothermal and isothermal conditions (i.e. $\tau_{\text{peak}-T} / \tau_{\text{peak}-T_0}$ and $\tau_{\text{res}-T} / \tau_{\text{res}-T_0}$) are compared in Figure 7. T and T_0 denote the non-isothermal (i.e. non-cyclic heating and cyclic heating-cooling) and isothermal (i.e. ambient temperature) conditions, respectively.

From Figure 7, at any thermal state, the peak and residual shear stress mobilized at the pile interface is less than that observed in the direct shear tests, which is mainly related to smoother surface of the pile. Furthermore, less lateral pressure generated at the clay-pile interface ($\approx$ a fraction of the overburden pressure, 100 kPa) is the other important reason for the observed lower interface resistance. Note that interface shearing in the direct shear tests took place under 150 kPa, 225 kPa and 300 kPa.

With respect to peak interface shear strength ratio ($\tau_{\text{peak}-T} / \tau_{\text{peak}-T_0}$), however, Figure 7a indicates that smaller values were measured by the direct shear test (under CNL condition). In fact, the peak interface shear strength ratio estimated from the direct shear tests is upper-bonded by 1.2, while that calculated from the 1g small-scale pile test results is upper-bonded by 2. Different stiffness boundary conditions available at the interface (i.e. CNL or CNS condition) are possibly responsible for this behavior that will be explained with more details in the following. Regarding

510 the residual interface shear strength ratio ($\tau_{\text{res}} - T / \tau_{\text{res}} - T_0$), no clear trend can be seen towards the
 511 temperature variations. Nevertheless, the residual shear strength is affected less significantly as
 512 opposed to the peak shear resistance.

Figure 7- Quantitative comparison between interface shear strength obtained by the direct shear and the small-scale pile tests: (a) peak interface shear strength; (b) residual interface shear strength. (Test1: 24 °C, Test2: (24-34 °C), and Test3: (24-34-24 °C)-10 cycles) (Data from Yazdani et al. (2019b) and Yazdani et al. (2019c)).

Now the heat stiffening of shaft resistance observed in Figure 5 is analyzed and discussed. Let us rewrite the ultimate unit shaft resistance presented in Equation (2a) in a way that helps examine the results of small-scale pile test:

$$\frac{q_s - T}{q_s - T_0} = \left(\frac{\sigma'_h - T}{\sigma'_h - T_0} \right) \left(\frac{\tan \delta_T}{\tan \delta_{T_0}} \right) \quad (4)$$

The equation shows the ratio of the ultimate unit shaft resistance developing at the pile-clay interface, where T and T_0 denote the non-isothermal and isothermal conditions, respectively.

Unlike the direct shear test, the ratios of effective lateral stress $\left(\frac{\sigma'_h - T}{\sigma'_h - T_0} \right)$ and friction coefficient

$\left(\frac{\tan \delta_T}{\tan \delta_{T_0}} \right)$ were not directly measured in the lab-scale pile tests (Yazdani et al. (2019c)). However,

$\left(\frac{\tan \delta_T}{\tan \delta_{T_0}} \right)$ may be approximated by that previously estimated for NC clay-concrete interface in direct

shear test (Table 4). As previously explained, the ratio of friction coefficient at the interface $\left(\frac{\tan \delta_T}{\tan \delta_{T_0}} \right)$

is expected to be mainly controlled by the ratio of internal friction coefficient of the clay near the

interface (i.e. $\frac{\tan \phi_T}{\tan \phi_{T_0}}$), whereby similar ratio would be expected to be observed at the interface in

both sets of tests (i.e. the direct shear and the 1g lab-scale pile tests). Note that according to the

main assumption underlying this article, the clay near the pile surface and the concrete plate shows

equal drained shear strength properties. Furthermore, it is widely accepted that the interface

friction coefficient is independent of stiffness boundary condition (e.g., Di Donna et al. 2016;

Mortara et al. 2007; Tabucanon et al. 1995). Therefore, $\left(\frac{\tan \delta_T}{\tan \delta_{T_0}} \right)$ is equal to that estimated using the

direct shear test, as shown in Table 4 (i.e. 1.083 and 1.1 for non-cyclic heating and 10 heating

cycles, respectively).

Figure 8a shows $\left(\frac{q_s - T}{q_s - T_0}\right)$ along with $\left(\frac{\tan \delta_T}{\tan \delta_{T_0}}\right)$. Using the data in Figure 8a and Equation 4, the corresponding $\left(\frac{\sigma'_h - T}{\sigma'_h - T_0}\right)$ at the pile interface can be calculated, as shown in Figure 8b. According to Table 5 that summarizes the results in Figure 8, the increased effective lateral stress is the primary reason for the improvement of pile shaft resistance reported by Yazdani et al. 2019c.

Figure 8- Heating-induced variation of (a) the ultimate unit shaft resistance and friction coefficient ratios; (b) effective lateral stress ratios with thermal loading path (Test1: 24 °C, Test2: (24-34 °C), and Test3: (24-34-24 °C)-10 cycles) (Data from Yazdani et al. (2019b) and Yazdani et al. (2019c)).

Table 5- Summary of the pile test results presented in Figure 8.

	$\frac{q_s - T}{q_s - T_0}$	$\frac{\tan \delta_T}{\tan \delta_{T_0}}$	$\frac{\sigma'_h - T}{\sigma'_h - T_0}$
24 °C	1	1	1
(24-34 °C)	1.96	1.083	1.81
(24-34-24)-10 cycles	1.49	1.1	1.35

Note: $\frac{\tan \delta_T}{\tan \delta_{T_0}}$ is estimated from the direct shear test results (Yazdani et al. (2019b)).

The following explanation may be offered for the heating-induced increase of effective lateral stress observed in Figure 8a and Table 5. After subjecting the clay-pile interface to higher

545 temperature (heating), the clay in the vicinity of the pile surface undergoes thermal consolidation
546 primarily because of greater coefficient of thermal expansion of the pore water than that of clay
547 particles (Campanella and Mitchell 1968). As a consequence, a time-dependent downward clay
548 deformation (i.e. vertical consolidation) occurs along the pile surface, which also results in time-
549 dependent lateral clay movements (or radial expansion) in the vicinity of the pile surface due to
550 particles rearrangement (similar to Poisson effect phenomenon, in which materials tend to expand
551 in the direction normal to the compression direction, except that consolidation deformations are
552 not elastic). However, although the induced radial expansion of clay can occur away from the pile
553 surface, it is not allowed to take place toward the pile surface as it is constrained by the pile thus
554 resulting in an increase in the radial stress acting on the pile surface.

555 According to the above explanation, upon the completion of thermal loading, the pile loaded at 34
556 °C (test 2) is expected to experience an effective lateral stress larger than σ'_{h-T_0} leading to larger
557 shaft resistance. Note that in test 2, the pile interface was loaded after its temperature reached 34
558 °C from the room temperature (i.e. (24-34 °C)). With respect to the tests performed under cyclic
559 heating-cooling, however, the cooling phase of each cycle (back to 24 °C) reverses the trend
560 explained during the heating phase. In other words, the thermal consolidation of clay during
561 heating phase is followed by lateral contraction during cooling phase inducing a reduction in the
562 horizontal stress ($\Delta\sigma'_{h-th} < 0$). This in turn causes a cyclic variation in the sign of $\Delta\sigma'_{h-th}$ during
563 heating and cooling phases of each cycle. However, the cumulative $\Delta\sigma'_{h-th}$ is expected to be
564 positive at the end of each cycle, due to the plastic deformation occurring during heating phase.
565 Therefore, for Test 3, where the model pile was loaded after applying 10 cycles of heating-cooling,
566 the effective lateral stress would be greater than σ'_{h-T_0} , but smaller than that in Test 2 due to
567 contraction occurring in each cooling phase.

The other possible mechanism by which the heating-induced increase of lateral stress may be explained is thermal radial expansion of pile. In general, thermal radial expansion of energy piles is partially or fully constrained by the surrounding soil that thus increases the contact pressures at interface during heat application. However, the following simple analysis shows that contribution of thermal radial expansion of the model pile is minor and can reasonably be ignored:

Under perfectly restrained condition at the interface, thermal radial expansion of pile is zero and the consequent increase of contact stress is approximated to be $E_{\text{clay}}\alpha_{\text{pile}}\Delta T$, where E_{clay} , α_{pile} , and ΔT are elasticity modulus of the clay, linear thermal expansion coefficient of the pile, and temperature change at the interface, respectively. Accordingly, considering $E_{\text{clay}} = 18 \text{ MPa}^1$, $\alpha_{\text{pile}} = 1.5 \times 10^{-5} \text{ } ^\circ\text{C}^{-1}$, and $\Delta T = 10 \text{ } ^\circ\text{C}$, the contact pressure increase is estimated to be 2.7 kPa, which is equal to 4.7% of the initial lateral stress at isothermal condition σ'_{h0} , previously approximated to be 58 kPa. Note that 4.7% is much smaller than the total increase of effective lateral stress, 81%, as shown in Table 5. However, the condition under which the model pile behaves lies between free thermal expansion condition and perfectly restrained condition, meaning that the increase of contact pressure should be less than 2.7 kPa estimated above. Moreover, using two-dimensional (2D) Finite Element (FE) analysis, Olgun et al. (2014) showed that for most soils, with modulus of elasticity lower than 100 MPa, piles freely expand into the surrounding soil during heating (i.e. contact pressure increase is zero).

Thermally induced excess PWP in the clay, at one pile diameter from the pile surface (as shown in Figure C), was also reported by Yazdani et al. (2019c). PWP was reported to rise up to 3.5 kPa during heating phase of the first thermal cycle and during non-cyclic heating, while a PWP drop

¹ Ranjan and Rao (2006) reported that $E_{\text{clay}} = (750 - 1200)S_u$ for NC clays, where S_u is undrained shear strength. In this study it was assumed that $E_{\text{clay}} = (1000)S_u$, and S_u was measured to be 18 kPa by conducting unconfined compression tests.

of about -9 kPa during the cooling phase (cooling back to 24 °C) of the first cycle was observed by them. In the following thermal cycles, the induced PWP reached +8 kPa and -8 kPa during the heating and cooling phases, respectively. Note that pile loading was occurred after all induced PWP was fully dissipated. In the analysis and discussion of thermal interface stiffening performed in this article, however, this information was not considered since all the PWP reported was measured within the clay at one pile diameter away from the pile surface, rather than at the pile interface. Note that throughout this article the stress state and PWP at interface are essentially considered for analysis.

The results discussed above can essentially be used for exploring the long-term shaft resistance as in the studies by Yazdani et al. (2019b) and Yazdani et al. (2019c) interface shearing occurred after completion of thermal loading, where full stabilization of temperature and PWP at the interface was attained. Therefore, the mechanisms contributing to the increased long-term shaft resistance under CNS condition, observed by Ghaaowd and McCartney (2018), Goode and McCartney (2015), McCartney and Rosenberg (2011), Wang et al (2016), and Yazdani et al. (2019c), can be summarized as follows:

(1) The heat stiffening of shaft resistance is primarily attributed to the heating-induced increase of effective lateral stress at pile surface.

(2) The shaft resistance improvement may also be partially attributed to heat stiffening of soil near pile surface.

In terms of the short-term predictions of shaft resistance, the changes in effective lateral stress and interface friction angle should also be considered. In fact, at short-term period, when heat exchange process continuously takes place at the interface, Δu_{th} is expected to grow at a fast rate (particularly in the beginning of heating periods), as shown by Fuentes et al. (2016), while the degree of thermal

consolidation is expected to be low. Correspondingly, the overall change in σ'_h tends to be negative (i.e. $\Delta\sigma'_{h-th} < 0$) that can potentially reduce the shaft resistance during heat application. Note that in the above statement, it was assumed that the thermally induced change of total lateral stress is zero ($\Delta\sigma_{h-th} = 0$). Under similar assumption, Fuentes et al. (2016) reported that the loss of short-term shaft resistance is significant when an energy pile is surrounded by a clay with low hydraulic conductivity ($k < 1\text{E-}11$ m/s) and high compressibility ($k_s > 20000$ MPa).

5-4- Thermal stiffening of unsaturated clay-concrete interface

As mentioned earlier, in the above analysis and discussion of thermal stiffening of interface, under both CNL and CNS conditions, it was assumed that the clay in the vicinity of interface is fully saturated, whereas the clay is likely to be unsaturated as back-pressure saturation was not used in the pile chamber and the direct shear tests. Nevertheless, a similar analysis considering an unsaturated state at the interface can be performed to interpret the thermal stiffening of shaft resistance observed above. In fact, the effect of presence of an unsaturated clay on the thermo-mechanical response of interface can be accounted for via applying the effective stress principle to unsaturated soils.

Bishop effective stress framework (Bishop 1959), which is commonly used to express effective stress in unsaturated soils can alternatively be used to define σ'_{h0} and $\Delta\sigma'_{h-th}$ in equation 2:

$$\sigma'_{h0} = (\sigma_{h0} - u_a) + \chi(u_a - u) \quad (5a)$$

$$\Delta\sigma'_{h-th} = (\Delta\sigma_{h-th} - \Delta u_{a-th}) + \chi\Delta(u_a - u)_{th} \quad (5b)$$

Where, σ'_{h0} , σ_{h0} , u_a , u and χ are effective lateral stress, total lateral stress, pore air pressure, pore water pressure and effective stress parameter, respectively. In the above equation, $(u_a - u)$ is

known as matric suction and χ is a function of degree of saturation. $\Delta\sigma'_{h-th}$, $\Delta\sigma_{h-th}$, Δu_{a-th} , and $\Delta(u_a - u)_{th}$ are thermally induced change in effective lateral stress, total lateral stress, pore air pressure and matric suction, respectively.

Accordingly, the following alternative explanation can be given for the thermal stiffening of interface observed in the pile test (Yazdani et al. 2019c): the temperature rise at the interface causes moisture migration away from the pile surface, which is associated with gradual increase in matric suction (and effective stress at the interface), and time-dependent downward and lateral deformation (radial expansion) within the clay around the pile. Similar to the saturated state, the clay particles are not allowed to move freely toward the pile surface, which results in an increase in the effective lateral stress (i.e. $\Delta\sigma'_{h-th} > 0$) and thus the shaft resistance.

For the interface stiffening observed under CNL condition (Yazdani et al. 2019b), however, since total stress at the interface was constant and heating occurred under drained condition, effective stress at the interface remained constant (i.e. $\Delta\sigma'_{h-th} = 0$). Therefore, similar to the saturated state, the interface shear strength improvement in unsaturated state is exclusively attributed to $\Delta\delta_{th}$ (i.e. $\Delta\delta_{th} > 0$ for NC clay-concrete interface and $\Delta\delta_{th} < 0$ for OC clay-concrete interface, as shown earlier). Similarly, $\Delta\delta_{th}$ is probably related to heating-induced redistribution of contact stress and physico-chemical forces between clay particles, although the changes of these two forces must be equal and opposite in sign so the overall changes of effective stress between clay particles remain zero.

6- Conclusion

This study aims to quantitatively/qualitatively determine the contributing factors to the increased interface strength of energy piles reported in the existing literature. The results of three separate

experimental studies of thermo-mechanical behavior of clay and clay-concrete interface were integrated and analyzed. Based on this analysis the following conclusions can be made:

- (1) The heating-induced improvement of shaft resistance can generally be explained based on the heating-induced increase of effective lateral stress $\Delta\sigma'_{h-th}$, and interface friction angle $\Delta\delta_{th}$.
- (2) The enhanced interface strength observed in the tests performed under constant normal load (CNL) condition, like those employing CNL direct shear tests, is primarily related to the clay stiffening (or $\Delta\delta_{th}$).
- (3) The thermally enhanced shaft resistance presented by the interface sheared under CNS condition, such as when a small/full scale pile test is conducted, is primarily attributed to $\Delta\sigma'_{h-th}$. $\Delta\delta_{th}$ also makes a partial contribution to the thermal stiffening of shaft. For instance, according to the work by Yazdani et al. (2019c), for the pile loaded at elevated temperature (34 °C), the heating-induced rise in shaft resistance, effective lateral stress and interface friction coefficient were estimated to be 96%, 81% and 8%, respectively. While, they were 49%, 35% and 10%, respectively, for the piles loaded after applying heating-cooling cycles.
- (4) Under assumption that there is no change in the total lateral stress due to heat application, shaft resistance of energy piles in clay at the short-term period possibly decreases due to insufficient time for drainage of the generated excess PWP. Further research, however, is needed.
- (5) Both short/long term shaft resistance will be better understood by future laboratory works using more realistic boundary condition and direct measurement of the effective lateral stress and PWP at the interface.

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Appendix:

Figure A- Temperature controlled triaxial apparatus (Yazdani et al. 2019a).

Figure B-(a) Schematic view of modified direct shear apparatus (b) Layout of the heating/cooling tubing in the concrete plate (Yazdani et al. 2019b).

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PPT: Pore Pressure Transducer

TC: Thermocouple

Figure C- Schematic view of the small-scale pile setup ((Yazdani et. al. 2019c).

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Table 1- Physical properties of kaolin clay (After Yazdani et al. (2019b))

Parameter	Value
D ₅₀	0.0026 mm
Plastic Limit (PL)	25 %
Liquid Limit (LL)	45 %
Plastic Index (PI)	20 %
Poisson's Ratio (ν)	0.25
G _s	2.62

Note: Please use this table. The reference in the manuscript (Yazdani et al. (2019a)) is wrong. The correct reference is (Yazdani et al. (2019b))

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