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Thermo-mechanical load-transfer analysis of heating-cooling cycles in energy piles

Analyse thermomécanique du transfert de charge des cycles de chauffage-refroidissement dans les pieux énergétiques

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ABSTRACT: To exploit geothermal energy, a liquid is circulated through a closed loop that passes through the ground and the building. In order to approach large temperature differences, one needs to take this liquid medium deep in the ground and therefore existing foundation piles are usually used to accommodate the geothermal loop pipes. Such piles are called “energy piles” (a.k.a. heat exchanger piles).

Although this is a very promising technology for energy savings, concerns have been raised about the potential degradation of the bearing capacity due to temperature cycles and the resulting thermal expansion and contraction of the piles that might affect soil-structure interaction. To understand and predict the potential effects of thermal changes in the capacity and performance of piles, relevant practical numerical models have been developed, mainly using the finite element method and load-transfer analysis.

This paper presents a numerical study of the thermo-mechanical behaviour of energy piles due to successive heating-cooling cycles. A thermo-mechanical hysteretic load-transfer model is developed and calibrated on a well-documented field test, the Lambeth College test. Subsequently, parametric studies are performed aimed at investigating the effects of different modelling and physical testing parameters on the response of the pile. It is shown that load-transfer analysis is able to reproduce several aspects of the field-observed behaviour of energy piles.

RÉSUMÉ: Pour exploiter l’énergie géothermique, un liquide circule dans une boucle fermée qui traverse le sol et le bâtiment. Pour faire face à de grandes différences de température, il est nécessaire de prendre ce liquide dans les profondeurs moyenne du sol et par conséquent les pieux de fondation existants sont généralement utilisés pour loger les tuyaux de la boucle géothermique. Ces pieux sont appelées « pieux énergétiques» (p. Ex. Pieux d’échangeurs de chaleur).

Bien qu’il s’agisse d’une technologie très prometteuse en matière d’économie d’énergie, des préoccupations ont été soulevées concernant la dégradation potentielle de la capacité portante en raison des cycles de température, ainsi que la dilatation et la contraction thermique des pieux qu'ils pourraient affecter l’interaction sol-structure. Pour comprendre et prévoir les effets potentiels des changements thermiques sur la capacité et les performances des pieux, des modèles numériques pertinents ont été développés, principalement adoptant la méthode des éléments finis et l'analyse de transfert de charge.

Cet article présente une étude numérique du comportement thermo-mécanique des pieux énergétiques dû aux cycles de chauffage-refroidissement successifs. Un modèle de transfert de charge hystérétique thermo-mécanique est développé et calibré sur un test de terrain bien documenté, le test de Lambeth College. Par la suite, des études paramétriques sont effectuées dans le but d'enquêter sur les effets de différents paramètres de modélisation et de tests physiques sur la réponse du pieu. On montre que l'analyse de transfert de charge est capable de reproduire plusieurs aspects du comportement observé sur le terrain des pieux d'énergie.

Keywords: energy piles; thermo-mechanical analysis; load-transfer; finite elements; soil-structure interaction.

1 INTRODUCTION

The widespread exploitation of non-renewable energy has led to a significant greenhouse gases' emission in the atmosphere, thus contributing to global warming. Thermo-active geo-structures' adoption seems a well-promising solution for addressing the climate change issue, since these technologies exploit renewable energy. The first countries to have investigated the geothermal structures' potentialities have been Austria, Switzerland and Germany (Bouazza et al. 2011) and, recently, ground-source heat-pump systems are raising interests as a means for heating and cooling buildings (Amatya et al. 2012). In this sense, energy piles represent the most studied technology within the heat-exchanger geo-structure field (Gawecka et al. 2016). In particular, they are constituted of reinforced concrete containing U-shaped pipe loops, filled with a carrier fluid, allowing the extraction of the geothermal energy and the heat transfer to/from the building (Brandl 2006).

Many authors have undertaken new research in order to give a further contribution to the understanding of energy piles' thermo-mechanical response: Laloui et al. 2006, Brandl 2006 and Bourne-Webb et al. 2009 represent the most important case studies in the context of the *in situ* experimental investigation.

On the other hand, there is much more published work about numerical simulations, by adopting non-linear load-transfer (t-z) Finite Element (FE) models (Knellwolf et al. 2011, Ouyang et al. 2011, Pelecanos et al. 2017), thermo-hydro-mechanical coupled FE analyses (Laloui et al. 2006, Di Donna and Laloui 2015, Gawecka et al. 2016), and off-the-shelf software (Yavari et al. 2014 and Saggu and Chakraborty 2015).

This paper presents the results of a parametric study developed for the Lambeth College energy pile case, by adopting an in-house FE code based on the non-linear load-transfer mechanism. The effects of material inhomogeneity, drained vs. undrained condition and soil stiffness on the thermo-mechanical response of the heat-exchanger pile have been studied.

2 NUMERICAL MODEL

It is well-known that the stability of an axially loaded pile relies on the resistance arising on the pile shaft, Q_s, and on the resistance at the pile tip, Q_b. The pile-soil interaction problem has been studied through a one-dimensional (1D) non-linear finite element load-transfer model. Within this static formulation, the pile's behaviour is described by an elastic constitutive law, whereas the soil's non-linear plasticity has been taken into account through the non-linear t-z model. Specifically, the deep foundation is discretised into a series of two-noded beam elements, with linear-elastic behaviour, whereas the surrounding soil behaviour at the pile's shaft and tip are respectively governed by non-linear springs (t-z)

applied at each beam element node and by an end-bearing spring (q-z) at the toe-node (Fig. 1).

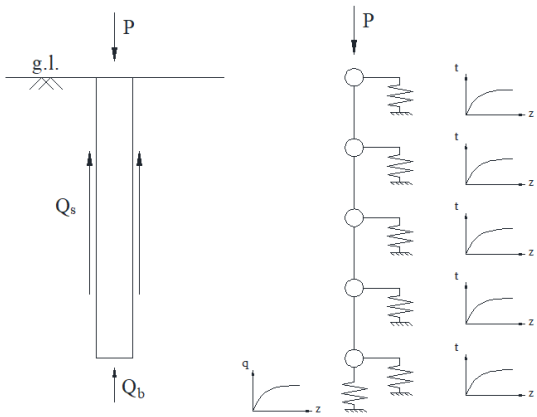

Figure 1. Discretisation of the axially-loaded pile into two-noded beam elements within the load-transfer FE model

The non-linear load-transfer curves follow the degradation and hardening hyperbolic model (DHHM) described in the following equation (Pelecanos et al. 2017):

$$t = \frac{k_m z}{\sqrt[d]{1+\left(\frac{k_m}{t_m}z\right)^{hd}}} \quad (1)$$

Where k_m is the maximum stiffness corresponding to a null displacement [force/length^3]; t_m is the maximum shear stress in the no hardening/softening case [force/length^2]; d is the dimensionless degradation parameter governing the degradation of subgrade modulus, k, with displacement z; h is the dimensionless hardening parameter that defines the model trend at large displacements (h=1 when no hardening/softening is considered).

The numerical model implements the computation of the ultimate shear stress (t_m) at the pile shaft (s) and at the pile tip (b), for cohesive and non-cohesive soils, as expressed by the following equations (Pelecanos et al. 2017):

$$t_{ms\ (cohesive)} = \alpha \cdot S_u \quad (2)$$

$$t_{ms\ (non-cohesive)} = K_0 \cdot \tan\delta \cdot \sigma'_{v0} \quad (3)$$

$$t_{mb\ (cohesive)} = N_c \cdot S_u \quad (4)$$

$$t_{mb\ (non-cohesive)} = N_q \cdot \sigma'_{v0} \quad (5)$$

Where α is the empirical shaft coefficient, usually considered 0.5 for London Clay (Tomlinson 1997); K_0 is the earth pressure coefficient at rest; δ is the friction angle at the pile-soil interface approximately 0.75 times the soil's friction angle (Stas and Kulhawy 1984); N_c and N_q are the bearing capacity coefficients, usually equal to 50 (Berezantzev et al. 1961 and Knappett and Craig 2012) and 9 (Kulhawy and Prakoso 1999) respectively.

Moreover, the non-linear analysis requires an incremental application of the mechanical load at the pile head, according to an equilibrium equation that is updated at each incremental load step:

$$\{P\} - \alpha_c EA\{\Delta T\} = \left([K_p] + [K_s]\right) \cdot \{z\} \quad (6)$$

Where K_p denotes the pile stiffness matrix; K_s is the soil stiffness matrix; z is the vector containing the vertical displacements' values; P is the vector containing the external applied loads, α_c and EA are the coefficient of thermal expansion and the axial rigidity of the pile respectively and ΔT is a vector containing temperature changes' values along the pile depth.

3 FIELD TEST: LAMBETH COLLEGE

The case study analysed in this paper is the Lambeth College pile load test presented in Bourne-Webb et al. 2009. As showed in Fig. 2, the test energy pile has been casted into a heterogeneous soil constituted of Made Ground in the first 1.5 m, Terrace Deposits for the consecutive 2.5 m and London Clay from 4 m

below ground level (b.g.l.). The water table is at 3 m b.g.l.

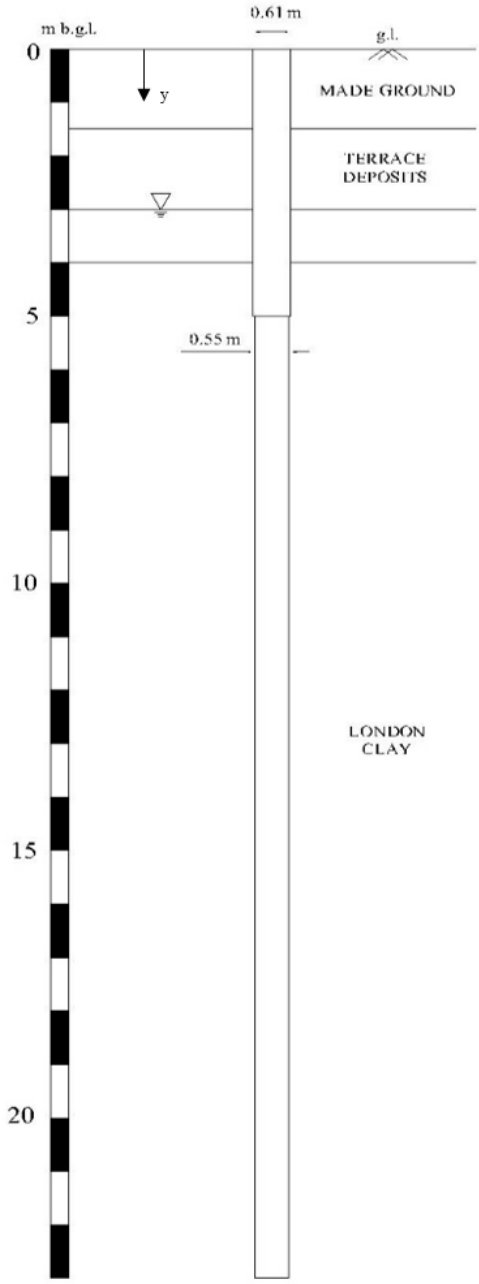

Figure 2. Sketch of the Lambeth College test energy pile and soil stratigraphy

The deep foundation presents a length of 23 m and a diameter 0.55 m wide, with an exception for its upper part, which has been encased into a 0.61 m diameter casing along the first 5 m b.g.l. Since the energy pile is made of reinforced concrete, the authors have considered a composite Young's modulus value, E_p, of 40 GPa and a composite thermal expansion coefficient value, α_c, of 8.5×10^{-6} m/m/°C.

The experimental program followed in the investigative campaign reports a first axial mechanical loading up to 1800 kN, with a consecutive unloading, then a re-loading up to 1200 kN and a series of thermal cycles under maintained load (-6 °C during cooling mode and +56 °C during heating mode) and a final mechanical loading phase reaching 3600 kN before ending the pile test. During thetest, axial strains and thermal changes have been recorded by means of vibrating-wire strain gauges and Optical Fiber (OF) strain cables, and thermistors and OF temperature cables respectively.

4 PARAMETRIC STUDY

The non-linear load-transfer finite element model has been used for developing a parametric study, by taking into account the pile's characteristics and the problem geometry described in Sec. 3. Tab. 1 gives information about the soil's parameters considered within the numerical analysis, which have been collected from different papers (Ouyang et al. 2011; Amatya et al. 2012; Yavari et al. 2014; Pelecanos et al. 2017). Specifically, even if Pelecanos et al. 2017 does not analyse the Lambeth College case, its study concerns a pile almost 14.0 km far away (as the crow flies), where the stratigraphy can be compared with the Bourne-Webb et al. 2009 one.

The parametric analysis has been developed for three loading conditions: mechanical loading under 1200 kN and two thermal-mechanical loading phases by adopting temperature changes, ΔT, equal to -18.10 °C and 8.70 °C respectively for the cooling and heating mode.

Table 1. Soil parameters considered within the parametric study: γ, soil total unit weight; c', effective cohesion; ϕ', effective friction angle; S_u, undrained shear strength; E', drained Young's modulus; E_u, undrained Young's modulus; υ', drained Poisson's ratio; υ_u, undrained Poisson's ratio; y_{LC} is the depth value starting at top of the London Clay layer (4 m b.g.l.)

STRATIGRAPHY	γ [kN/m^3]	c' [kPa]	ϕ' [°]	S_u [kPa]	E' [MPa]	E_u [MPa]	υ' [-]	υ_u [-]
Made Ground	19	0	30	-	10	-	0.2	-
Terrace Deposits	20	0	35	-	10	-	0.2	-
London Clay	20	5	25	$65 + 8.2y_{LC}$	-	$600S_u$	-	0.5

Moreover, the code returns the results in terms of variability of mechanical axial strain, ε_{mech}, axial force, P, mobilised shaft shear stress, q_s, and vertical displacements with depth. These quantities are then computed:

$$\varepsilon_{mech} = \varepsilon_{real} - \varepsilon_{th} \tag{7}$$

$$P = E_p A \varepsilon_{mech} \tag{8}$$

$$q_s = \frac{dP}{\pi \cdot d_p \cdot dy} \tag{9}$$

$$z = z(y = 0) + \int_0^L \varepsilon_{real}\, dy \tag{10}$$

Where ε_{real} is the observed axial strain which the pile undergoes to; ε_{th} is the thermal strain due to the free expansion or contraction in the case of unrestrained body; E_p is the pile Young's modulus; A is the pile cross-sectional area; d_p is the pile diameter; y is the depth from the ground level; z(y=0) is the pile head displacement.

4.1 Material inhomogeneity

The first numerical investigation considers the influence of adopting Constant (London Clay) Soil Parameters (C-SP) and Variable ones (V-SP) with depth. When V-SP case is adopted, the soil strength and so its rigidity vary linearly with depth, affecting the slope and the magnitude of the curves, as showed in Fig. 3. Specifically, the mechanical axial strain's and axial force's trends exhibit a steeper slope until almost around 15 m b.g.l. (neutral point), then becoming gentle until the pile tip. Moreover, the use of variable soil parameters results in a more compressive pile's response, denoted by higher positive values for the mechanical, cooling and heating steps, with respect to the C-SP case. This returns smaller development of mobilised shaft shear stress and vertical displacements above the top part of LC, whereas it is possible to observe a reverse situation in the bottom part of it. Considering C-SP case for the clayey layer would bring to an underestimation of the vertical displacements in the top half of the LC layer, and, conversely, an overestimation of them in the bottom half of it.

4.2 Undrained-Drained

The Undrained (U) and Drained (D) cases are characterised by different constitutive laws' formulations. The D-case shows development of strains, and so axial force, in the made ground and the terrace deposits layers (Fig. 4), accentuating the mechanical + cooling effect. Furthermore, the neutral point, again roughly around 15 m b.g.l., results left-shifted in the z(y) chart, where vertical displacements are more contained with respect to the U-case, due to a further mobilised shaft shear stress.

4.3 Soil stiffness

In this last case, the effect of doubling and halving the soil stiffness has been studied by considering the undrained case with constant S_u with depth. As the soil stiffness has doubled, the vertical displacements are more contained (Fig. 5), whereas halving the soil stiffness results in an increase of vertical displacements (Fig. 6).

Moreover, mechanical axial strains and axial force values are not strongly affected by this numerical process, differently from displacements.

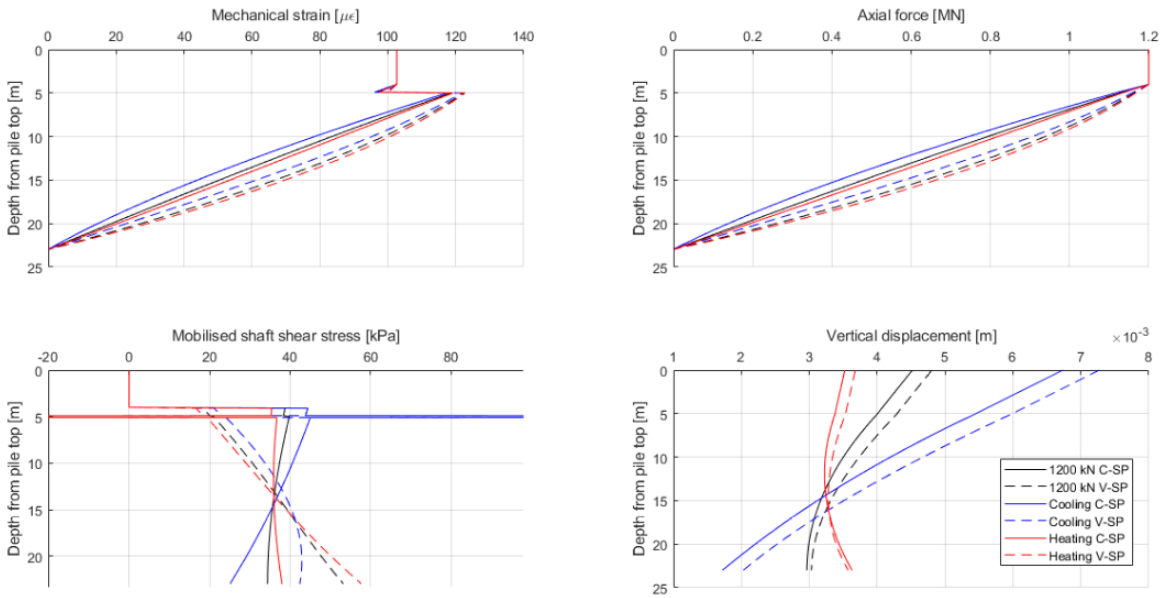

Figure 3. Comparison between Constant Soil Parameters (continuous lines) and Variable Soil Parameters (dashed lines) with depth for mechanical loading case (black), cooling phase (blue) and heating phase (red)

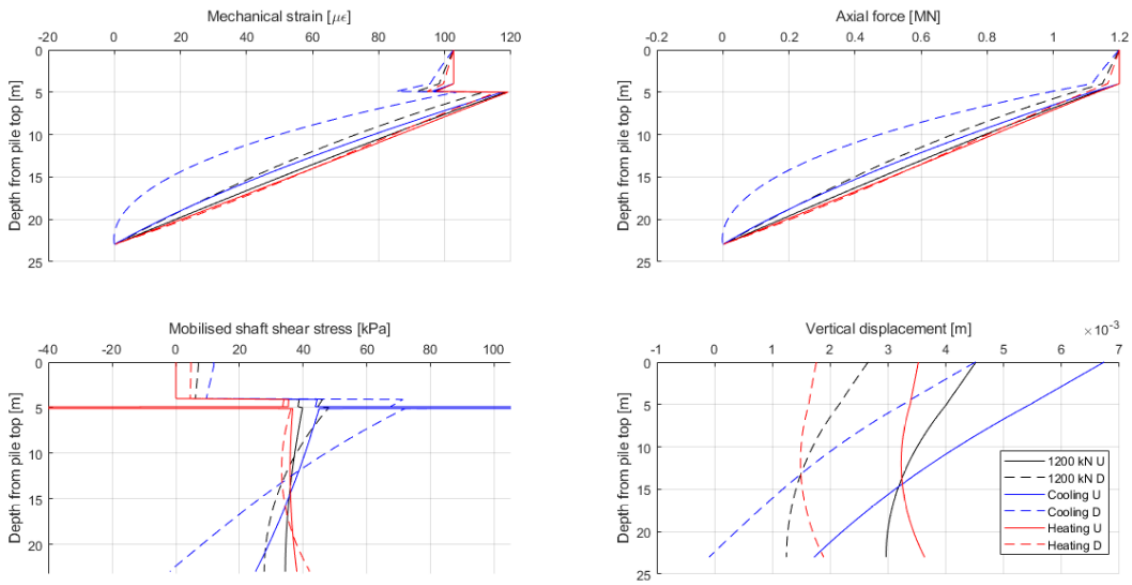

Figure 4. Comparison between Undrained (continuous line) and Drained (dashed lines) conditions for mechanical loading case (black), cooling phase (blue) and heating phase (red)

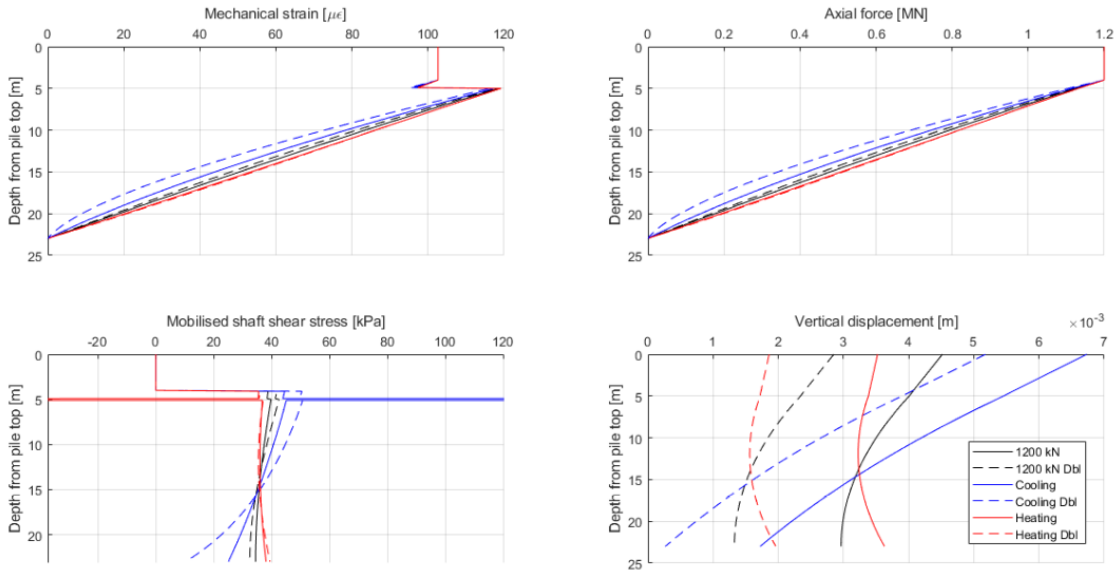

Figure 5. Comparison between Doubled soil stiffness (dashed lines) and unmodified soil stiffness (continuous lines) for mechanical loading case (black), cooling phase (blue) and heating phase (red)

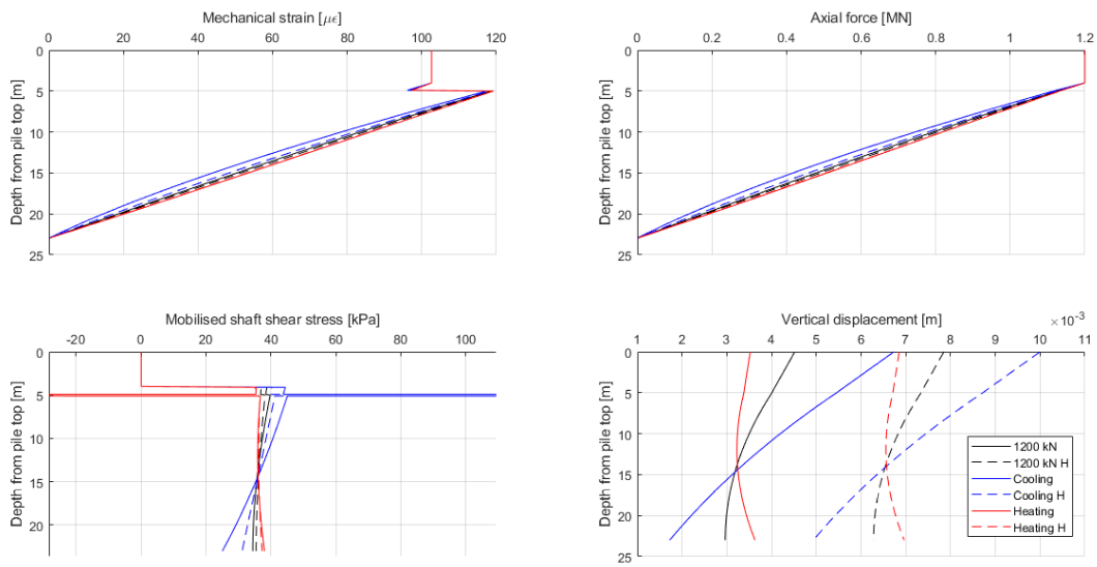

Figure 6. Comparison between Halved soil stiffness (dashed lines) and unmodified soil stiffness (continuous lines) for mechanical loading case (black), cooling phase (blue) and heating phase (red)

5 CONCLUSIONS

A parametric analysis has been performed considering the Lambeth College pile test case study, by adopting a 1D load-transfer FE code. Despite its simplicity, the numerical method shows a strong potential in examining several aspects of the energy pile‘s behaviour. As observed, the code not only is able to reproduce the effects derived from different thermo-mechanical loading conditions, but it is also able

to simulate the thermo-active geostructure response for various soil features.

Specifically, adopting non-variable soil parameters results in an underestimation of the vertical displacements above the top half of the London Clay layer, and, conversely, an overestimation of them in the bottom half. The adoption of different constitutive laws also plays an important role on the numerical pile behaviour, whereby the drained case presents contained values of the vertical displacements with respect the undrained case. Moreover, increasing the value of the soil stiffness mobilises less displacements, whereas decreasing the soil stiffness values results in an increase of them. Therefore, vertical displacements are strongly affected by the soil parameters‘ choice and by the constitutive law adopted. This leads to highlight the sensitivity of the numerical analysis‘s final results in relation to the initial setting up of the problem.

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