

GEOTHERMAL ENERGY PILE-SOIL INTERACTION UNDER MECHANICAL LOADING AND THERMAL CYCLES

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ABSTRACT: Harvesting shallow geothermal energy for heating and cooling building spaces in winter and summer is considered environmentally friendly and renewable. Recently, geothermal energy piles have been used as heat exchanger elements in ground source heat pump systems to exchange heat with the ground underneath buildings for heating and cooling purposes. However, imposing thermal cycles on such piles may result in possible adverse effects on their structural and geotechnical performance. A comprehensive understanding of the behavior of geothermal energy piles is therefore vital for the successful applications of such systems. This paper aims to investigate the interaction between the soil and geothermal energy pile subjected to a combination of mechanical loading and thermal cycles. Coupled thermo-hydro-mechanical (THM) finite element analyses were carried out on a hypothetical geothermal energy pile using climatic and geological conditions in Winnipeg, Manitoba, Canada. Numerical results were presented in this paper in terms of pile head displacements, strains, and stresses developed in the pile, as well as shaft friction and effective radial stresses along the pile-soil interface. The effects of heating and cooling on the ultimate geotechnical pile capacity were also presented. Based on the numerical results, it was found that the thermo-mechanical loads have considerable effects on the geothermal energy pile responses.

Keywords: Geothermal energy pile, Pile-soil interaction, Thermo-mechanical load, Thermal cycles, Coupled THM finite element modeling

1. INTRODUCTION

In recent years, geothermal energy piles or simply energy piles have been increasingly used as ground heat exchangers in geothermal heat pump or ground source heat pump (GSHP) systems to harness shallow geothermal energy for building heating and cooling purposes. The GSHP systems are considered renewable and environmentally friendly. The energy piles are structural piles with heat exchange pipes (loops), which can be a multiple U-loop configuration as illustrated in Fig.1 or a spiral coil configuration, attached inside the rebar cages before casting concrete. The heat exchange pipes are usually made of high-density polyethylene (HDPE) with diameter ranging from 19 to 32 mm and filled with the heat-carrying fluid [1]. Energy piles have two main functions; they serve as both structural elements to support the buildings and as ground heat exchangers to exchange heat between the buildings and the underlying ground. Using working structural piles as ground heat exchangers; however, has raised some concerns about possible adverse effects of thermal loads on the structural and geotechnical performance of the piles. Thermally loading the piles may lead to over-stressing, loss of load-carrying capacities of the piles, and excessive pile head vertical displacements.

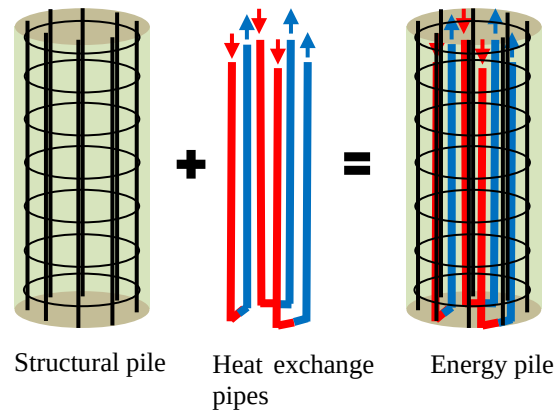


Fig.1 Geothermal energy pile with multiple U-loops

Energy piles were first used in the construction industry in Austria more than three decades ago [2]. Their use then has spread throughout Europe [3,4] and to North America in recent years [5,6]. Nonetheless, there is still limited understanding of their behavior under the combined effects of thermal and mechanical loads, especially the effects of temperature changes on geotechnical pile capacities. This paper aims to numerically investigate the behavior of an energy pile subjected to thermo-mechanical loads and under local climatic and geological conditions in Winnipeg, Manitoba, Canada.

Table 1 Essential material parameters used for numerical modeling of the Winnipeg energy pile

Parameter	Winnipeg clay	Silt till	Dolomitic limestone	Concrete pile
Material constitutive model	HSSM	HSSM	LEM	LEM
Young's modulus, E' (kPa)	-	-	11×10^6	40×10^6
Poisson's ratio, ν' (-)	-	-	0.2	0.15
Reference secant stiffness, E_{50}^{Ref} (kPa)	21×10^3	110×10^3	-	-
Reference tangent stiffness, E_{eod}^{Ref} (kPa)	21×10^3	110×10^3	-	-
Reference un/reloading stiffness, E_{ur}^{Ref} (kPa)	63×10^3	330×10^3	-	-
Un/reloading Poisson's ratio, ν_{ur} (-)	0.2	0.2	-	-
Exponential power, m (-)	1	0.5	-	-
Failure ratio, R_f (-)	0.9	0.9	-	-
Shear stiffness at very small strain, G_o^{Ref} (kPa)	84×10^3	440×10^3	-	-
Threshold shear strain,	0.0002	0.0002	-	-
Cohesion, c' (kPa)	3	3	-	-
Friction angle, ϕ' (°)	23	40	-	-
Dilatancy angle, ψ (°)	0	0	-	-
Interface strength reduction factor, R_{inter} (-)	1	1	1	-
Hydraulic conductivity, k_x, k_y (m/day)	6.70×10^{-5}	0.028	8.64	-
Specific heat capacity, c_s (kJ/t/°C)	830	720	1300	800
Thermal conductivity, λ_s (kW/m/°C)	1.2×10^{-3}	1.5×10^{-3}	2.3×10^{-3}	1.8×10^{-3}
Soil density, ρ_s (t/m ³)	1.83	2.34	2.45	2.55
Linear thermal expansion coefficient, α_{sL} (1/°C)	5×10^{-6}	5×10^{-6}	5×10^{-6}	10×10^{-6}

Note: HSSM = Hardening Soil with Small strain stiffness Model; LEM = Linear Elastic Model

2. NUMERICAL MODEL DISCRIPTION

Energy pile-soil interaction is a complex thermo-hydro-mechanical (THM) coupling problem. The geotechnical finite element software called PLAXIS 2D-2018 with an add-on Thermal Module, which has the capability to solve transient THM coupling problems, was selected as the modeling tool. Note that in this paper the negative sign (-) is used for compression and positive sign (+) is used for tension. Currently, there are no actual energy piles installed in Winnipeg – one of the coldest regions in the world; therefore, a generic energy pile was hypothesized based on local ground conditions and pile foundation engineering practice. The pile was assumed to have a diameter (D) of 0.8 m and a length (L) of 20 m. It was installed through the Winnipeg lacustrine clay with a pile toe embedded 5 m into the underlying very dense silt till layer.

2.1 Material Characteristics

For numerical modeling, the soil profile was simplified based on [7] and consists of three main layers. The first layer is the Winnipeg lacustrine clay, from 0 to 15 m below ground level (bgl). This layer is underlain by the silt till, from 15 to 21 m bgl. Below this silt till is the dolomitic limestone bedrock, extending to a great depth. The groundwater table is typically at 3 m bgl. For material constitutive models, the concrete energy

pile was modeled as a non-porous (solid) elastic material using a linear elastic model (LEM). A hardening soil with small strain stiffness model (HSSM) was used for the clay and the silt till. The LEM was also used for the dolomitic limestone bedrock. Table 1 gives a summary of essential material parameters used for numerical modeling.

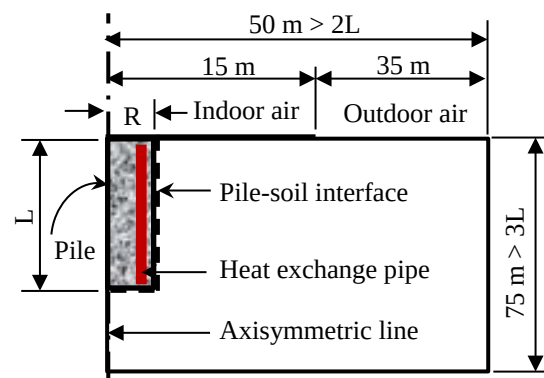


Fig.2 Axisymmetric model geometry setup (not to scale)

2.2 Model Setup and Boundary Conditions

The energy pile was assumed to be located right underneath the center of the building with a width of 30 m and without any basement. With these assumptions, the axisymmetric finite element model was used ($30 \text{ m}/2 = 15 \text{ m}$). The model

domain was set at a distance of 50 m ($> 2L$) and 75 m ($> 3L$) for the side and bottom boundaries; respectively, as illustrated in Fig.2. These distances were considered to minimize the potential effects of the assumed boundary conditions from an engineering point of view. The model domain was divided into zones for discretization in which very fine mesh sizes were used for the pile body, along the pile-soil interface, and around the pile toe in order to ensure that there were enough elements to capture the appropriate pile behavior. Coarser mesh sizes were used for the zones further away from the pile.

Regarding displacement boundary conditions, free displacements were allowed at the top boundary. Both vertical and horizontal displacements were restrained at the bottom. Only vertical displacements were allowed at the left- and right-hand side boundaries. For hydraulic boundary conditions, drainage was allowed at the top and right-hand side boundaries. A closed flow boundary was assigned along the axisymmetric line and the bottom boundary. For thermal boundary conditions, heat flow was closed at the left- and right-hand sides. At the top boundary, the indoor air temperature was set at a constant value of 20°C , corresponding to the commonly controlled air temperature inside the building all year round. The concrete slab (slab-on-ground) was not placed in the model. However, its effect was represented using the thermal boundary (convective boundary condition) with the assumed overall thermal transmittance value (U-value) of $0.2 \times 10^{-3} \text{ kW/m}^2/^{\circ}\text{C}$ which was assumed based on the field observation [8]. The outdoor air temperature corresponding to seasonal air temperature variation was used as a boundary condition outside the building. This was represented by the convective boundary condition with assumed overall thermal transmittance value (U-value) of the ground surface of $15 \times 10^{-3} \text{ kW/m}^2/^{\circ}\text{C}$. This U-value was used because it provided the simulated frost depth of about 2.0 m below the ground surface, which was close to the frost depth of -1.8 m reported in Winnipeg [7]. The authors are aware that the frost depth may vary from place to place, depending on the local conditions such as the ground surface cover materials and vegetation. The observed seasonal variation of the mean daily air temperature data for 30 years (from 1981 to 2010) at the Winnipeg Richardson International Airport; as shown in Fig.3, were taken from the Environment and Climate Change Canada (ECCC) [9]. A constant ground temperature of 7°C was assigned at the bottom boundary. The initial ground temperature for the entire model domain was also set at 7°C which was approximately the undisturbed average ground temperature in the Winnipeg area [10,11].

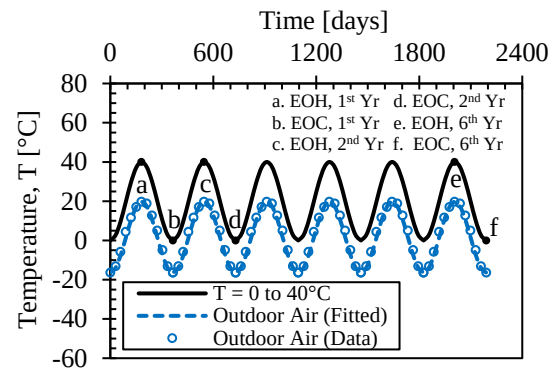


Fig.3 Outdoor air temperature variation and thermal cycles applied to the pile starting from the first of January

A mechanical load of -1300 kN (compression) – a working load of the pile, corresponding to the axial stress of -2586 kPa on the pile head, was applied in a drained manner and maintained on the pile head. The energy pile was also subjected to thermal cycles due to heating and cooling by the circulating heat-carrying fluid in the heat exchange pipe installed inside the pile. In this study, a change in temperature of the fluid in the pipe inside the pile as a function of time from 0 to 40°C was used, as shown in Fig.3. The convective boundary condition (a line-based internal thermal boundary which implies a circular shell in the axisymmetric model) at 70 mm from the pile shaft was used to represent the heat exchange pipe. It was assumed that the changing pattern of the temperature in the pile with time followed the rising and falling pattern of the seasonal air temperature. This assumption was made based on the field observation by other researchers [2] on the operational energy geostructures. In the simulations, the energy pile was subjected to six heating-cooling cycles, corresponding to six years of heating and cooling of the building. Here, the term at the end of heating (EOH) means the pile was heated to the maximum temperature (the peak). Likewise, the term at the end of cooling (EOC) means the pile was cooled to the minimum temperature (the trough) in a particular year. Note that the THM computation was very time-consuming. Due to the time constraint; therefore, in this paper only six-year of heating-cooling cycles were considered. This was relatively a short duration in comparison with the design life of the energy pile (typically 50 years or more).

3. RESULTS AND DISCUSSIONS

3.1 Temperature Distributions

Figure 4 shows the temperature profiles at the mid-depth of the pile from the pile center to 15 m

sideways in a horizontal direction at the EOH and EOC periods in the 1st, 2nd, and 6th year. These particular times were deliberately chosen in order to determine how the numerical results change for different simulation times, i.e., at the end of first year, after two years, and then after six years of heating and cooling periods. Large changes in the ground temperatures only occurred within a distance of about 2 m ($\approx 2.5D$) from the pile center.

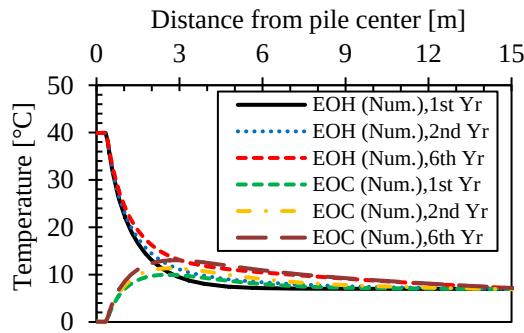


Fig.4 Temperature profiles at the mid-depth of the pile from the pile center to 15 m away at the EOH and EOC in the 1st, 2nd, and 6th year

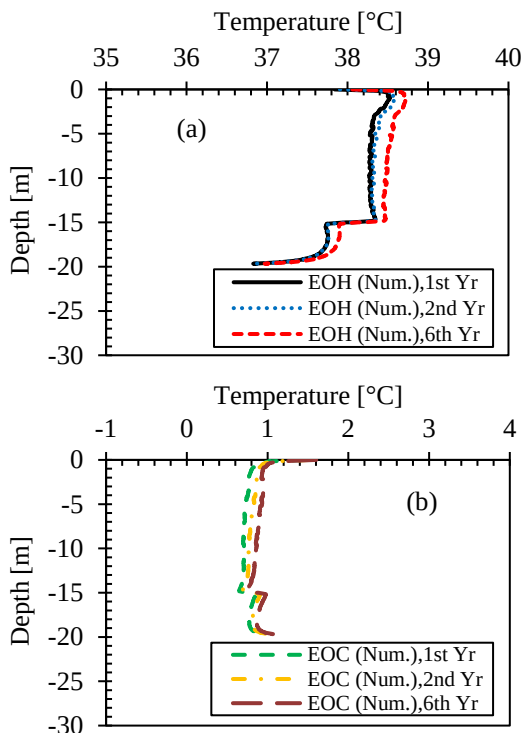


Fig.5 Temperature profiles along the pile-soil interface (a) at the EOH and (b) at the EOC in the 1st, 2nd, and 6th year

Temperature profiles along the pile-soil interface are shown in Fig.5. Note that there were slight changes (increased from 38.5 to 38.7°C and

from 7.9 to 8.9°C) in the maximum temperature values at the EOH and EOC from the first to sixth year, respectively. Nonetheless, the ground temperatures appeared to increase slightly with time (overall ground warming over time). This was due to the influence of heat loss through the ground floor slab that accumulated in the ground underneath the building with time and also due to the imbalance in thermal loads during heating and cooling periods in relation to the initial ground temperature.

3.2 Excess Porewater Distributions

Thermally-induced excess porewater pressures (EPWPs) could occur in the clay layer having significantly low permeability. There were no EPWPs induced in the till layer located below -15 m because of its relatively high permeability. Heating induced negative EPWPs and cooling produced positive EPWPs. As shown in Fig.6, the maximum negative values of EPWPs at the EOH were -9.1, -8.8, and -7.5 kPa in the first, second, and sixth year, respectively. The reduction in EPWPs maybe due in part to consolidation. Another reason maybe due to the lesser temperature difference between the pile and surrounding ground at the EOH because of overall ground warming. In other words, the temperature of the ground surrounding the pile was getting higher with a longer simulation time, as can be seen in the temperature profiles along the pile-soil interface shown in Fig.5 earlier.

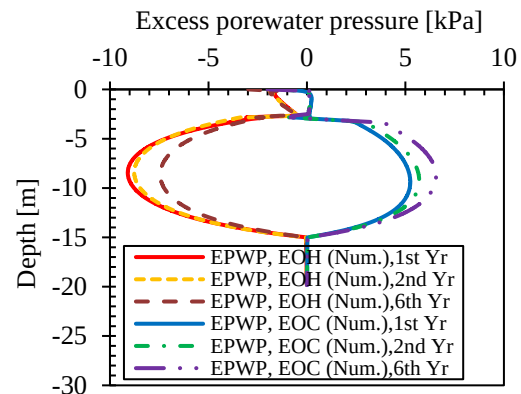


Fig.6 Thermally-induced excess porewater pressures (EPWPs) along the pile-soil interface at the EOH and EOC in the 1st, 2nd, and 6th year

The opposite occurred during cooling, as shown in Fig.6, in which the negative EPWPs were generated with the maximum values of 5.2, 5.7, and 6.6 kPa at the EOC in the first, second, and sixth year, respectively. The negative EPWPs

somewhat increased with time from the first to the sixth year, resulting from the larger temperature difference between the pile and surrounding ground at the EOC due to overall ground warming.

3.3 Pile Head Vertical Displacements

Figure 7 shows the simulated vertical pile head displacements (settlements and uplifts) for six-year heating and cooling cycles. The mechanical load (M) caused a pile head settlement of -1.89 mm (-0.24% D). The thermal cycles caused the pile head displacements to move downwards gradually (a phenomenon of ratcheting settlements). This was because of the cyclic accumulated elastoplastic strains in the surrounding soils. The dissipations of excess porewater pressures in the clay may also contribute to the continuing settlements. At the EOH in the first to sixth year, the uplifts of the pile head of 2.78 mm and 0.52 mm were obtained, a drop of 81.3%. On the other hand, the settlements at the EOC in the first to the sixth year were -4.18 mm and -5.91 mm (-0.74% D), increasing by 41.4%. This downward pile head displacement trend suggests that a long-term settlement may exceed the specified tolerable limit and may cause problems to the superstructure. Therefore, it is important to consider the long-term settlement for the design life of the energy pile. Even though only six-year heating-cooling cycles were considered in this paper due to time constraint, a longer simulation time (50 years or more) should be considered for future research to better capture the long-term settlement phenomenon of the energy pile.

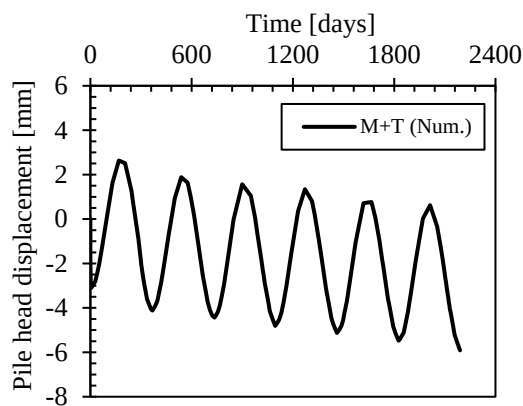


Fig.7 Pile head vertical displacements (uplifts and settlements) during six thermal cycles

3.4 Pile Axial Strains

The mechanical load (M) produced contractive strains (negative) in the entire pile as shown in

Fig.8. In contrast, heating induced expansive strains (positive) in the pile as shown in Fig.9(a). The strains at the EOH in the sixth year were generally higher than in the 1st year. Similar to the M load, cooling induced contractive strains in the pile as shown in Fig.9(b). As seen in the graph, lower contractive strains were observed at the EOC for the longer simulated times.

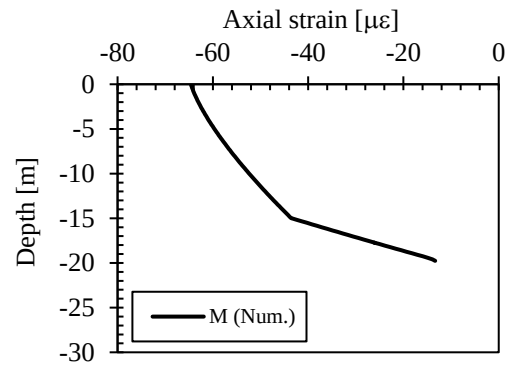


Fig.8 Axial strain profiles due to the mechanical load (M) only

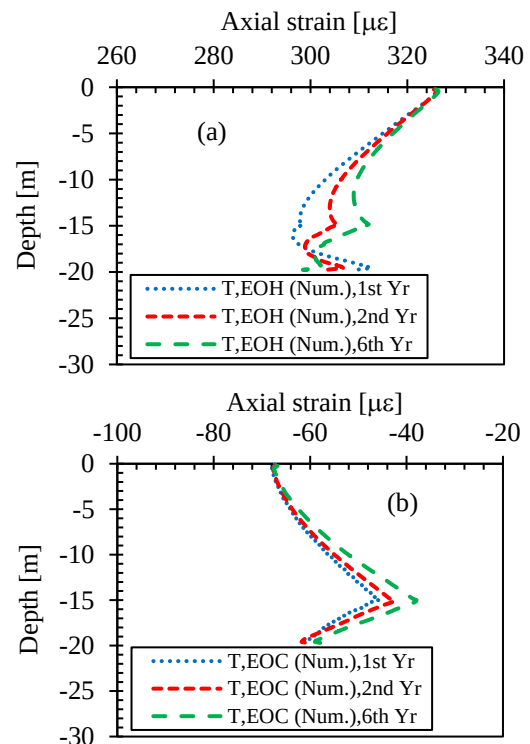


Fig.9 Axial strain profiles (a) thermal load (T) at the EOH and (b) at the EOC in the 1st, 2nd, and 6th year

3.5 Pile Axial Stresses

The M load of -1300 kN on the pile head induced compressive stresses in the pile, which

were plotted in both Fig. 10 and Fig.11 for reference. In these figures, the mechanically-induced stresses transferred mostly into the silt till layer, located below -15 m, beneath the Winnipeg lacustrine clay. Heating induced axial compressive stresses in the pile, which is similar to the effect of the mechanical load. The maximum thermally-induced compressive stresses, as shown in Fig.10(a), were -1391 kPa at the EOH in the first year. This value reduced to -1302 kPa in the sixth year. As shown in Fig.10(b), at the EOH, the axial compressive stresses increased in the pile as a result of thermo-mechanical loads. This was because heating resulted in additional compressive stresses in the pile.

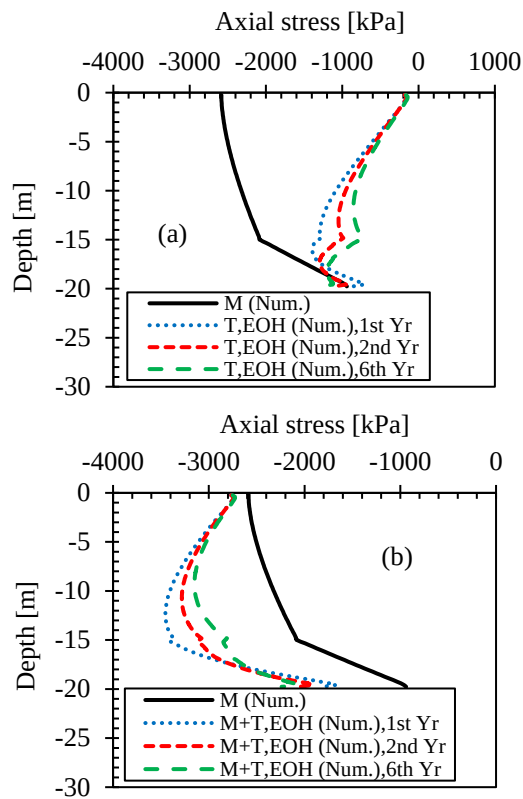


Fig.10 Axial stress profiles (a) thermal (T) and (b) thermo-mechanical (M+T) at the EOH in the 1st, 2nd, and 6th year

Cooling generated axial tensile stresses in the pile in contrast to the effect of the mechanical load as shown in Fig.11(a). The maximum thermally-induced tensile stresses located at about -15 m were 982 kPa and 1290 kPa at the EOC in the first and sixth year, accordingly. This shows an increase in the maximum thermally-induced tensile stress of 31.4% from the first to sixth year. As shown in Fig.11(b), there were large reductions of axial compressive stresses in the pile for the combined

effects of mechanical load and thermal cooling.

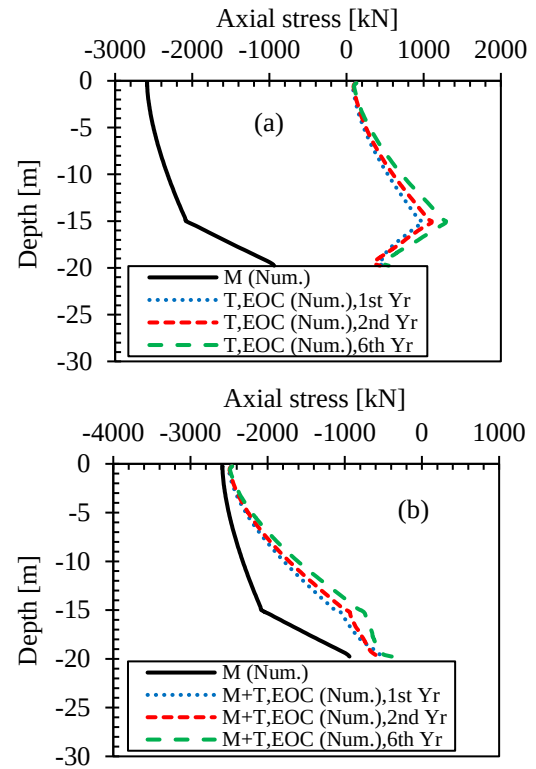


Fig.11 Axial stress profiles (a) thermal (T) and (b) thermo-mechanical (M+T) at the EOC in the 1st, 2nd, and 6th year

3.6 Mobilized Shaft Friction

Figure 12 shows the mobilized shaft friction (side shear stresses) along the pile-soil interface at the EOH and EOC in the first, second, and sixth year. It was observed that the M load induced positive skin friction along the whole pile length. Larger values were mobilized in the silt till layer below -15 m depth than in the overlying clay layer. This is because the till has much higher stiffness and strength. At the EOH, as shown in Fig.12(a), the shaft friction in the upper two-thirds of the pile reduced and resulted in small negative values. In the lower one-third; the shaft friction increased considerably. Conversely, the shaft friction at the EOC increased in the upper two-thirds portion, but it reduced in the lower one-third part where the pile was installed through the silt till layer as shown in Fig.12(b). Some irregularities in shaft friction in the silt till were observed as shown in the graphs. This may be due to sudden change in the soil stiffness from a very low value in the clay to a very high value in the silt till and also high permeability of the silt till which somehow caused oscillations in the THM computations.

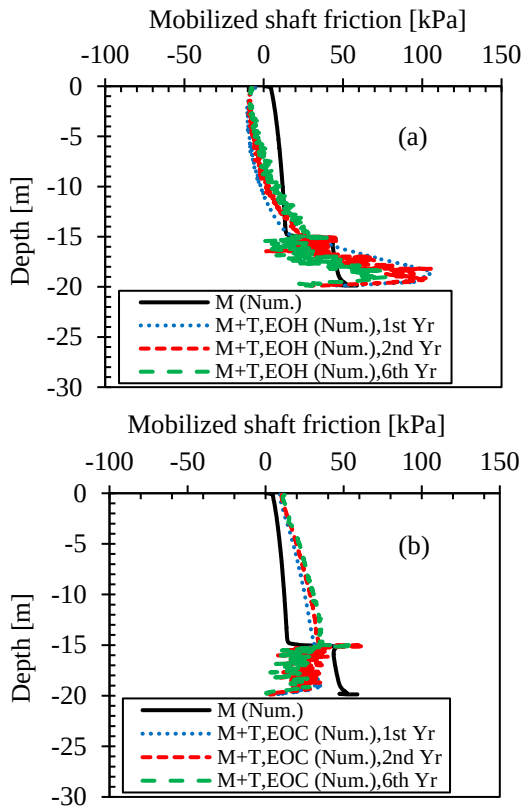


Fig.12 Mobilized shaft friction profiles along the pile-soil interface (a) at the EOH and (b) at the EOC in the 1st, 2nd, and 6th year

3.7 Mobilized Effective Radial Stresses

Mobilized effective radial stress profiles along the pile-soil interface at the end of heating (EOH) and at the end of cooling (EOC) are shown in Fig. 13. In Fig.13(a), heating generally caused the effective radial stresses along the pile-soil interface to increase in the entire pile length. There were marked rises in the silt till layer, and again, some irregularities in the effective radial stresses were noticed which may be due to the same reasons explained earlier in the mobilized shaft friction section. Conversely, cooling did the opposite to the heating in which the effective radial stresses reduced in the lower two-thirds of the pile in the silt till layer but slightly increased in the upper two-thirds as shown in Fig.13(b). The stress increase of the latter was due to the stress redistributions to somewhat compensate the reduction in effective radial stresses along the lower part of the pile and effective stresses at the pile toe at the EOC. The change in effective radial stresses are mainly due to relative thermal expansions and contractions of the pile and the surrounding soils during heating and cooling, respectively.

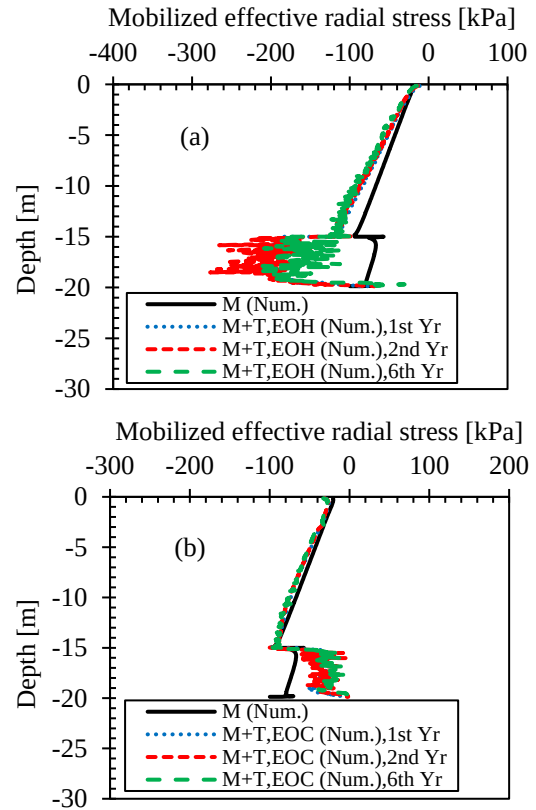


Fig.13 Mobilized effective radial stress profiles along the pile-soil interface (a) at the EOH and (b) at the EOC in the 1st, 2nd, and 6th year

3.8 Thermal Effects on Geotechnical Pile Capacities

As shown in Fig.14, the geotechnical pile capacity generally increased upon heating. At the EOH, the ultimate pile capacity, determined using the 10% D failure criterion [12], increased by 12%. On the other hand, cooling caused the ultimate pile capacity to reduce. The reduction of 9% was observed at the EOC. The reason for the increase in ultimate geotechnical capacity of the pile during heating was mainly due to the increase in effective radial stresses along the pile-soil interface. When the pile was heated, it would expand vertically and radially. The radial expansion, in this case, could not fully mobilize because of the restraining effect of the surrounding soils. Consequently, higher radial stresses developed along the pile-soil interface, leading to an increase in the ultimate geotechnical capacity of the pile. The opposite reason seemed to be valid when the pile was cooled. In addition, because of the contraction (moving upwards) of the pile toe during cooling, there was a reduction of the end-bearing stress which could also contribute to the decrease in the

ultimate geotechnical capacity of the pile at the end of cooling.

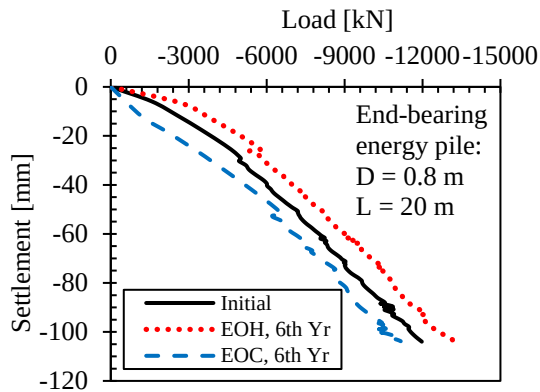


Fig.14 Load-settlement curves simulated at the initial stage, at the EOH and EOC in the 6th year

4. CONCLUSIONS

Temperature changes in the pile had significant effects on its displacements, strains, and stresses of the pile and surrounding soils. In general, heating caused the pile to expand. These expansions resulted in expansive strains in the pile. If the pile cannot expand freely due to the shaft friction and end restraints, thermal compressive stresses would be generated in addition to the mechanically-induced stresses. For the thermal and mechanical combined effects, the compressive loads in the pile would be higher during heating. Cooling caused reverse effects in which the pile contracted. Consequently, compressive strains were generated and led to the tensile stresses build up in the pile. The compressive stresses in the pile were less for the combined thermo-mechanical effects. In terms of pile head displacements, heating produced uplifts of the pile head while cooling caused settlements. The settlements kept on increasing with increasing thermal cycles (ratcheting settlements).

The heating-cooling cycles of the pile also affected the surrounding ground temperatures to some distances which were dependent on the simulation times. Excess porewater pressures were generated in the clay with low permeability. The mobilized shaft friction and the effective radial stresses along the pile-soil interface were also affected by the temperature changes to some extent.

Geotechnical pile capacities generally increased during heating but reduced during cooling. For the energy pile in Winnipeg, the ultimate pile capacity increased at the end of the heating by about 12% but decreased at the end of cooling by about 9%.

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