Thermo-hydro-mechanical coupling analysis of a thermo-active diaphragm wall

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Abstract

Thermo-active diaphragm walls that combine load bearing and ground source heat pump (GSHP) are considered to be one of the new technologies in geotechnical engineering. Despite the vast range of potential applications, current thermo-active diaphragm wall designs have very limited input from a geotechnical aspect. This paper investigates the wall-soil interaction behaviour of a thermo-active diaphragm wall by conducting a thermo-hydro-mechanical finite element analysis. The GSHP operates by circulating cold coolant into the thermo-active diaphragm wall during winter. Soil contraction and small changes in the earth pressures acting on the wall are observed. The strain reversal effect makes the soil stiffness increase when the wall moves in the unexcavated side direction, and hence gives different trends for long-term wall movements compared to the linear elastic model. The GSHP operation makes the wall move in a cyclic manner, and the seasonal variation is approximately 0.5-1 mm, caused by two factors: the thermal effects on the deformation of the diaphragm wall itself and the thermally induced volume change of the soil and pore water. In addition, it is found that the change in the bending moment of the wall due to the seasonal GSHP cycle is mainly caused by the thermal differential across the wall during the winter, because the seasonal changes in earth pressures acting on the diaphragm wall are very limited.

Keywords

Thermo-hydro-mechanical analysis; thermo-active diaphragm wall; finite element

1 Introduction

Ground Source Heat Pump (GSHP) technology can offer low carbon heating and cooling, and hot water provision. Their electrical consumption and maintenance requirements can be lower than those of conventional heating and cooling systems. The installation of the Ground Heat Exchanger (GHE) into the ground is the major component of the capital cost. In order to reduce installation costs and save underground space, GHE pipes are sometimes incorporated into various ground-embedded structures, such as tunnels, piles and diaphragm walls (e.g. Brandl, 2006; Adam and Markiewicz, 2009, Suckling and Smith, 2002; Amis et al., 2010; Waboso and Gilbey, 2007). They are called thermo-active tunnels/piles/walls, or collectively, thermo-active geostructures. When designing such geostructures, the principal constraint is that the thermal loads applied to the geostructures must not degrade their mechanical performance, i.e., their ability to support the load of the building or infrastructure (e.g. Bourne-Webb et al., 2009; 2013; Laloui and Donna, 2011; Amatya et al., 2012; GSHPA, 2012).

In thermo-active diaphragm walls, absorber tubes are installed inside the concrete by attaching them to the reinforcement cage as shown in Figure 1. There are two possible GSHP operating modes: (a) both heating and cooling and (b) heating only. If a thermo-active diaphragm wall is used for a basement, then the interior side of the diaphragm wall is insulated to ensure that the heat from the exchangers transfers into the soil rather than into the basement. By doing so, thermo-active diaphragm wall can cater for both heating and cooling demand of the aboveground structures. For underground railway stations, there is the possibility for extracting heat in winter time from both sides of the wall (station and soil) because stations often experience excessive heat generated by train operations, as shown in Figure 2. In summer, the GSHPs are not used and the excessive heat from the station would transfer into the soil. The heat stored during summer then can be used in winter for heating the aboveground structures. This type of thermo-active diaphragm wall will be discussed in details in subsequent sections.

Figure 1. Section plan drawing of geothermal heat exchangers embedded in diaphragm walls.

Figure 2: Heating-only operating mode of thermo-active diaphragm wall

Compared to the number of research conducted for thermo-active piles, understanding of the thermo-mechanical performance of diaphragm walls with embedded heat exchangers is rather limited. Brandl (2006) described the application of geothermal infrastructure in Vienna. In this project, geothermal loops were installed in diaphragm walls, piles, base slabs and tunnel linings. Brandl (2006) predicted that the whole system could provide 81 kW heating, while the heating demand of the station was about 95 kW during peak hours. Another application of thermo-active diaphragm walls is in the Uniqa Tower, Vienna (Adam et al., 2009), which reached down to 35 m below the surface. The whole system produced a heating capacity of 420 kW and a cooling capacity of 240 kW. Amis et al. (2010) described the thermo-active diaphragm wall at Bulgari hotel in Knightsbridge, UK and discussed the potential effects of thermal changes during operation. It was found that the thermal resistance and thermal conductivity detected from TRT in the construction period changed when the station box was finished, due to removal of the soil. Amis (2011) reported a rise of 20% in thermal resistance as well as a 13% reduction in the thermal conductivity value. In addition, some other research were performed to investigate on the factors affecting the energy performance of thermo-active diaphragm walls (Xia et al., 2012; Donna et al. 2016). Stewart et al. (2014) used an analytical model to evaluate the effect of incorporating heat exchangers into a geosynthetic-reinforced retaining wall. It was assumed that the heat would improve the undrained shear strength and stiffness of the backfill soil. However, it was found that the heat also play an opposing role in the deformation response of the wall. Kürten et al. (2015) developed a semi-analytical calculation model to study the thermal performance of a thermo-active diaphragm wall. This new approach has been proven to be suitable for the design of plane energy geostructures through comparison with pure finite element simulations and laboratory results. Sterpi et al.(2017) investigated coupled thermo-mechanical behaviour of a thermo-active active diaphragm walls by finite element analysis and concluded that the thermally induced mechanical effects on internal axial forces and bending moments are not negligible. Another numerical analysis performed by Bourne-Webb et al. (2016) showed that thermally induced mechanical response of diaphragm wall are dominated by seasonal temperature changes.

The performance of a thermo-active diaphragm wall is different from a thermo-active pile because the pile is surrounded by soil, whereas only one side of the diaphragm wall is exposed to soil. The temperature difference between the unexcavated side and the excavated side of the wall induces bending stresses in the wall. The heating and cooling of the soil causes the soil to expand and contract. The expansion/contraction of soil induces displacement, and produces a bending moment in the wall. If hysteresis occurs between heating/cooling cycles, it may affect the structural performance of the diaphragm wall. Hence, it is proposed in this research that the design of a thermo-active diaphragm wall requires examination of the effect of (i) concrete expansion differential within the wall, (ii) variations in earth pressures acting on the wall, and (iii) soil contraction or expansion due to changes in the ground temperature after many years of GSHP operation.

In this paper, a case study on a thermo-active diaphragm wall installed at one new underground metro station in London is used to illustrate the engineering assessments conducted to address the above three issues. The heating only mode is considered in this study. A series of thermo-hydro-mechanical (THM) finite element analysis simulations were conducted for the assessment. First, the construction phase was modelled and the modelled displacement of each stage was compared to the actual measured lateral movement data from inclinometers. Second, the GSHP operation phase was modelled to investigate the short and long term responses of the diaphragm wall and soil due to (i) construction of the wall only, and (ii) construction of the thermo-active diaphragm wall and the operation of the GSHP. Both the linear elastic and the non-linear elastic model were used to show the effect of the nonlinear elasticity of soil on the performance of a thermo-active diaphragm wall.

2 Mechanics of THM coupled processes

Based on the theory of continuum mechanics, some assumptions have been adopted to develop the thermo-hydro-mechanical coupling model for soil:

(1) The soil is treated as a fully saturated porous medium. The voids of the soil are assumed filled with liquid water.

(2) Coexisting pore fluid components and solid components are assumed to be at the same temperature.

(3) Considering the poor permeability of soil, heat conduction is assumed the main mean of heat transfer in this problem.

In this study, the saturated soil is assumed as a mixed continuum of three independent overlapping phases, displacement of soil skeleton, pore fluid flow and heat transfer. The THM coupling model could be obtained according to principles of continuum mechanics.

(1) Mechanical equilibrium equations:

$$\nabla \cdot (\boldsymbol{\sigma} - \mathrm{p}\mathbf{I}) + \rho\mathbf{g} = \mathbf{0} \tag{1}$$

where p is pore pressure, $\boldsymbol{\sigma}$ is stress tensor, ρ is saturated density of soil, $\mathbf{g}$ is the gravity acceleration vector, $\mathbf{I}$ is the identity tensor. The component form of $\nabla \cdot \boldsymbol{\sigma}$ with the base vectors $\boldsymbol{e_i}$ can be written as

$$\nabla \cdot \boldsymbol{\sigma} = \frac{\partial \sigma_{ji}}{\partial x_j} \boldsymbol{e_i} \tag{2}$$

Within the Mohr-Coulomb framework, the soil is modelled as an isotropic elastic-perfectly plastic material. The elastic behaviour is modelled assuming isotropic elasticity, of which the stress tensor is defined from the elastic strain tensor as:

$$\mathrm{d}\boldsymbol{\sigma}' = \mathbf{D}^{\mathrm{e}}: \mathrm{d}\boldsymbol{\varepsilon}^{\mathrm{e}} + \mathbf{D}^{\mathrm{Te}}\mathrm{dT} \tag{3}$$

$\boldsymbol{\varepsilon}^{\mathbf{e}}$ is elatic strain tensor, T is temperature, $\mathbf{D}^{\mathrm{e}}$ is the fourth-order elastic material tensor, $\mathbf{D}^{\mathrm{Te}}$ is the second-order thermo-elastic material tensor.

$\mathbf{D}^{\mathrm{e}}$ can be written as:

$$\mathrm{D}^{\mathrm{e}}_{\mathrm{ijkl}} = \lambda\delta_{\mathrm{ij}}\delta_{\mathrm{kl}} + 2\mathrm{G}\delta_{\mathrm{ik}}\delta_{\mathrm{jl}} + 2\mathrm{G}\delta_{\mathrm{il}}\delta_{\mathrm{jk}} \tag{4}$$

where $\lambda = \frac{2\mathrm{G}\nu'}{1-\nu}$, ν' is the poisson ratio and G is the shear modulus. In addition, the components of thermo-elastic tensor $\mathbf{D}^{\mathrm{Te}}$ can be written as:

$$\mathrm{D}^{\mathrm{Te}}_{\mathrm{ij}} = \mathrm{D}^{\mathrm{e}}_{\mathrm{ijkl}}\alpha_{\mathrm{T}}\delta_{\mathrm{kl}} \tag{5}$$

where α_{T} is the thermal expansion coefficient of soil skeleton.

The double contraction of $\mathbf{D}^{\mathbf{e}}$ with $\mathrm{d}\boldsymbol{\varepsilon}^{\mathbf{e}}$ can be written in the component form as,

$$\mathbf{D}^{e}:\mathrm{d}\boldsymbol{\varepsilon}^{e}=\mathrm{D}^{e}_{ijkl}\mathrm{d}\varepsilon^{e}_{kl} \tag{6}$$

It is well recognized that the non-linear stiffness degradation of soil is important in simulating the movements of geotechnical structures. Hence, both the linear and nonlinear elastic models were implemented and the study aims to identify the effect of small-strain stiffness degradation and strain reversal on the performance of the thermo-active diaphragm wall. Atkinson et al. (1990) concluded that the magnitude of strain stiffness depends on the angle of the rotation of the stress path. This assumption may play a crucial role in thermo-active diaphragm wall, due to the thermally induced cyclic strain change in the soil between winter and summer. Hence, the following hyperbolic non-linear model by Pyke (1979) was adopted for the non-linear model:

$$\tau=\tau_{ref}+\frac{G_{max}(\gamma-\gamma_{ref})}{1+\frac{a}{n}|\gamma-\gamma_{ref}|} \tag{7}$$

where τ is the current shear stress, G_{max} is the value of the horizontal tangent on the stress-strain curve at small strain, a is a constant, n depends on the loading/unloading conditions, γ is the current shear strain, and τ_{ref} and γ_{ref} are the reference shear stress and strain which are set to the values of the last strain reversal.

The model behaves elastically until the onset of yielding which is determined by the Mohr-Coulomb yield criterion. The thermo-elastic soil model is used in this study, which indicates that soil yielding is independent of temperature change. This assumption applies for the soil with high over consolidation ratio (Hueckel and Baldi, 1990; Boudali et al., 1994; Laloui et al., 2003), like London Clay and Lambeth Group.

(2) The transient saturated groundwater flow:

$$\nabla \cdot \left(-\frac{\mathrm{k}}{\mathrm{r_w}}(\nabla \mathrm{p} - \rho_w \mathbf{g})\right) + \div \dot{\mathbf{u}} + \frac{\mathrm{n}}{\mathrm{K_w}}\dot{\mathrm{p}} - \mathrm{n}\alpha_{\mathrm{Tw}}\dot{\mathrm{T}} = 0 \tag{8}$$

Where $\mathrm{r_w}$ is the unit weight of water, k is the permeability coefficient, ρ_w is the density of water, $\dot{\mathbf{u}}$ is the time derivative of displacement vector of soil skeleton, n is the porosity of soil, $\mathrm{K_w}$ is the bulk modulus of water, α_{Tw} is the thermal expansion coefficient of water, $\div \dot{\mathbf{u}}$ is the trace of the gradient of $\dot{\mathbf{u}}$, which can be written as,

$$\div \dot{\mathbf{u}} = \sum_{\alpha=1}^{3} \frac{\partial \dot{\mathrm{u}}_{\boldsymbol{i}}}{x_i}$$

(3) The governing equation of the heat transport process:

$$-\nabla \cdot (\lambda \nabla \mathrm{T}) + \mathrm{c_{sw}}\dot{\mathrm{T}} = 0 \tag{9}$$

where λ is heat conductivity of saturated soil, $\mathrm{c_{sw}}$ is the volumetric heat capacity of saturated soil.

3 Thermo-active diaphragm wall

This paper investigated on the thermo-active diaphragm installed at a new underground metro station in London, which has already been constructed, and will be used to provide 200kW-1MW of energy for station space and water heating only. The station box was constructed using the ‘bottom up’ method. The station box is 16 m wide and 29 m deep, as shown in Figure 3. It has four temporary props and six slabs, where one slab is a direct replacement of a temporary prop. A one metre thick diaphragm wall (up to 41 m depth) was first installed by excavating a trench to the required depth. The absorber pipes were attached to the reinforcement cage and lowered into the trench. After that, concrete was poured in to cast the diaphragm wall. The soil inside the diaphragm box was excavated 29 m deep, and temporary props were added to support the excavation. Slabs were then cast from the bottom and work proceeded upwards, replacing the props with slabs to form five levels station box.

Figure 3. Geometry of Dean Street Station Box

4 Finite element model

The finite element simulation of the excavation stage and the GSHP operation stage was conducted using an in-house THM finite element code developed at the University of Cambridge (Rui, 2014). The diaphragm wall is assumed to be long enough to ensure that the mechanically or thermally induced movement satisfy plain strain condition. Hence, the whole station box is simplified into a 2D model, and only half of the box is modelled as shown in Figure 4. In the FE model, the soil is 76 m deep and extends for 120m laterally from centre of station.

(a)

(b)

Figure 4. Finite element model: (a) Geometry; (b) Meshing

Only horizontal displacement is restricted in the left-hand-side (LHS) and right-hand-side (RHS) boundary. The top soil boundary is free, allowing possible settlements induced by the operation of the thermo-active diaphragm wall. The lateral pressure ratio k_0 of the diaphragm wall is set as 1, considering the wall installation effect. In addition, the water table is kept constant at zero pressure at the soil surface for simplicity. The pore pressure distribution is hydrostatic for all elements initially, and the RHS of the mesh is kept hydrostatic throughout. Drainage is allowed at the bottom and the RHS boundaries. The circulating flow in the pipe was not simulated. Instead, the temperature of pipe circuit is set as variable values to simulate the 20 years GSHP operation stage. Underground metro stations often have excessive heat generated by train operations (Rees, 2016). Hence, the temperatures of the station box were kept at 18˚C and the far-field soil were kept at 12˚C respectively for the whole 20 years. The initial soil temperature was 12˚C. The temperature in the pipes varied between winter and summer cycles; the pipe temperature was set to be 2˚C and 18˚C for winter and summer, respectively. The temperature boundary conditions applied are summarised in Figure 5.

Figure 5. Temperature boundary conditions of the thermo-active diaphragm wall

The scope of this case study involves the analysis of the seasonal operation effects on the structural performance of the wall and the surrounding soil. The analysis was split into two phases; (a) the Construction Phase (Hydro-Mechanical Response), to calibrate the governing model parameters for soil behaviour using displacement data obtained during the construction of the wall, and (b) the Operation Phase (Thermo-Hydro-Mechanical Response), to analyse the THM interactions between the soil and the diaphragm wall during heating and cooling cycles for 20 years in order to assess the structural response of the wall to GSHP operations. Two types of soil model were used: (i) linear model, the linear elastic-perfectly plastic Mohr Coloumb model, and (ii) non-linear model, the non-linear elastic-perfectly plastic Mohr Coloumb model.

(a) (b)

Figure 6. Test data for stiffness degradation and hyperbolic match: (a) London Clay; (b) Lambeth Group（after Schwamb 2014）

The model parameters for the linear model were selected based on Crossrail's design guideline, as shown in Table 1. For non-linear model, Parameter G_{max} and a for London Clay were determined by matching the shear modulus with triaxial test data by Gasparre (2005), as shown in Figure 6(a). Parameter G_{max} and a for Lambeth Group were determined by matching the shear modulus, which is the converted undrained young's modulus by Hight et al. (2004), as shown in Figure 6(b). Parameter a for Terrace Gravel and Chalk came from the tests by Liao et al. (2013) and Heymann (1998). Model calibration was performed by varying the thermo-mechanical model parameters so that the numerical results matched the field test data, such as the wall displacement. All mechanical parameter values for the soil model are listed in Table 1. Thermal parameters are the same for both linear and nonlinear analysis, as shown in Table 2.

Table 1. Mechanical Properties used in the linear and non-linear model

Table 2. Thermal Properties used in both linear and non-linear analysis

5 Construction phase and long-term behaviour without GSHP operation

Short-term behaviour

The construction processes were simulated prior to simulating the GSHP operation in the wall. The model performance was checked first with the actual monitoring data during excavation to verify whether an appropriate set of stiffness in the soil stratum had been selected for the simulation. Figure 7 shows the lateral displacement profile of the wall at different stages of excavation. In general, the wall displacements computed by the non-linear model are smaller than those using the linear model, and compare well with the inclinometer data. The difference between the two model cases decreased with increase in excavation depth. During the final excavation step, the difference was rather small because the amount of shearing increased enough at the end of excavation that the stiffness values of the two models became relatively similar. The profile and magnitude of displacements were similar between the computed results and the measured data, providing some confidence in using the model.

(a) Excavation level (117.2 m) (b) Excavation level (111.9 m)

(c) Excavation level (109. 7) (d) Excavation level (96.8)

Figure 7. Comparison of the relative horizontal displacements between the linear elastic model and the non-linear elastic model

Figure 8 shows the profiles of horizontal total stress and pore pressure variations before and after excavation. During the excavation, the horizontal total stresses on both sides reduced due to ground movement towards the excavated side. The largest change happens at the position between fourth prop and base slab, where the final excavation took place.

The pore pressure profiles in Figure 8 show that, after the excavation, the soil on the unexcavated side develops a large negative excess pore pressure along the wall on the unexcavated side. The London Clay and Lambeth Group has low permeability, so the excess pore pressure cannot dissipate much during the excavation and the soil remains in undrained conditions. At the excavated side, the soil at the bottom of the excavation also exhibits negative excess pore pressure compared to the original pore pressures.

As the drainage in horizontal direction under the base slab is not allowed, only upwards pore water movement happens slowly due to the low permeability coefficient of London Clay and Lambeth Group. On the unexcavated side, the drainage can occur in both horizontal and vertical direction. The negative pore pressure zone is close to Terrace Gravel, which has very high permeability coefficient. Hence, the swelling on the excavated side is much slower than that of the unexcavated side. This results in interesting wall movements with time, which are discussed next.

(a) (b)

(b)

(c) (d)

Figure 8. Total stress and pore pressure: (a) Excavated side with linear elastic model; (b) Unexcavated side with linear elastic model; (c) Excavated side with non-linear elastic model; (d) Unexcavated side with non-linear elastic model

Long-term behaviour

After the station box construction, the soil was allowed to consolidate so that the long-term “drained” conditions could be achieved. Figure 9 shows the horizontal incremental displacement profiles over the long term (20 years) when there is no GSHP operation. In both models, the magnitudes of the computed long-term movements are much smaller than the short-term movements (20 mm horizontal movement in the short-term versus l mm horizontal movements in the long-term).

Although the two models give small long-term wall movements, the trends are different, as shown in Figure 9. In the first year, the wall moves toward the excavated side because of the relatively quick swelling of the soil on the unexcavated side. In the following years, the wall gradually moves toward the unexcavated side when the linear model is used. This is because of the delayed swelling of the soil on the excavated side, pushing the wall back. When the non-linear model is used, the wall continues to move toward the excavated side, indicating that the magnitude of soil swelling on the excavated side is smaller than that on the unexcavated side. The soil on the unexcavated side strains in the same direction as in the excavation stage, and hence the soil stiffness is small. On the other hand, the soil on the excavation during swelling strain the opposite direction to the excavation stage, and hence the soil is stiff due to the strain reversal effect. Therefore, the magnitude of swelling on the excavated side is smaller than that on the unexcavated side, and hence the wall tends to move toward the excavated side.

(a) (b)

Figure 9. Relative horizontal displacement of the diaphragm wall without operation of the GSHP: (a) Linear elastic model; (b) Non-linear elastic model

Figure 10 shows the changes in pore pressure with time. The pore pressures slowly converge to the initial hydrostatic distribution with time. If there is no GSHP operation, the excess pore pressures developed during the excavation stage dissipates to a large extent within the first 10 years.

(a) (b)

Figure 10. Long-term pore pressure change on the unexcavated side: (a) Linear elastic model; (b) Non-linear elastic model

Figure 11 shows profiles of the total horizontal stress after construction. A large increase in horizontal total stress is observed in the first year. The maximum change is about 70kPa for linear model and 80 kPa for nonlinear model, which occurs between the fourth and base slab. The greatest decrease in pore pressure during construction is found here due to the largest excavation taking place here, as shown in Figure 8. Hence the largest swelling of soil happens in this zone.

Figure 11. Long-term total stress change on the unexcavated side: (a) Linear elastic model; (b) Non-linear elastic model

6 GSHP operation stages

Soil behaviour

Figure 12 shows the temperature contours during the operation of the GSHP after different years of the GHSP operation. The soil temperature near the wall drops/rises during the winter/summer cycle. The effect of the changes in wall temperature on the soil reduces with increasing distance away from the wall. The soil temperature below the base slab is also influenced by the change in coolant temperature and a distinctive thermal gradient is observed at this location. The contours also show that there is a cumulative effect of cooling (slight blue in the soil) in the soil slightly away from the wall after 20 years of operation.

Figure 12. Contours of temperature changes with operation of GSHP

Figure 13 shows the pore pressure profiles at the unexcavated side of the wall for the two model cases. If the GSHP system were to be operated, the pore pressures would fluctuate seasonally. The pore pressure in the soil at both sides of the wall increases and decreases in summer and winter, respectively. When the soil is heated, both the soil skeleton and the pore fluid expand. However, due to the difference in the thermal expansion coefficients as well as the low water permeability, negative excess pore pressure is generated in winter when the soil is cooled. The profiles in the early years have bigger differences between the two seasons because the excess pore pressure from the construction phase is also added. It is shown that GSHP operation of the wall extends the time needed for the excess pore pressure to dissipate.

The trends and magnitude of pore pressure changes predicted by the two models are similar. For example, at +95 m on the unexcavated side, the maximum difference in pore pressure between summer and winter cycles after 20 years is 50 kPa for the linear model and 80 kPa for the non-linear model. The thermal contraction/expansion of the pore fluid governs the excess pore pressure development, which indicates that elastic modulus of the soil has a small influence on the pore pressure.

(a) **(b)**

Figure 13. Comparison of pore pressure on the unexcavated side with operation of GSHP: (a) Linear elastic model; (b) Non-linear elastic model

Figure 14 shows the total horizontal stress profiles along the unexcavated side of the wall. At the unexcavated side, the total horizontal stress gradually increases with time due to swelling. The general trend is similar to the no GSHP operation case. With the GSHP operation, the total horizontal stress increases and decreases in the summer and winter cycles, respectively. However, the changes in total stress between the cycles are rather small (up to 25 kPa) compared to pore pressure as shown in Figure 13, which indicates the seasonal changes of effective stress are large and opposite to those of pore pressure. The small change in total stress suggests that the effect of thermally-induced earth pressure on diaphragm wall behaviour, like bending moment and axial force, is likely to be small. But the large cyclic change in effective stress may cause more soil yielding and affect long-term stability of diaphragm wall, which needs to be considered in the design.

(a) (b)

Figure 14. Horizontal total stress on the unexcavated side with operation of GSHP: (a) Linear elastic model; (b) Non-linear elastic model

Diaphragm wall behaviour

Figure 15 shows the horizontal displacement profiles during the GSHP operation computed by the two models. Although the magnitude is small, the long-term movement of the wall toward the excavated side first and then toward the unexcavated side due to the long term swelling on the unexcavated side and excavated side. The GSHP operation of the wall makes the wall move in a cyclic manner; the seasonal variation is approximately 0.5 mm 1.0mm. Seasonal changes in relative displacement when the GSHP is operating are mainly affected by two factors: the thermal effects on the deformation of the diaphragm wall itself and the thermally induced volume change of the soil and pore water. In winter, the pipe temperature is cold and the station side of the wall is hot, so this creates thermal strain variations in the wall, causing the wall to bend toward the excavated side. In summer, the temperatures of both faces of the wall are similar, bringing the wall back toward the unexcavated side. However, the seasonal changes in relative wall displacement are also affected by the thermally induced volume change of the soil and pore water. In summer, the soil on the unexcavated side expands, pushing the wall toward the excavated side. In winter, the soil on the unexcavated side contracts, pulling the wall back toward the unexcavated side. As shown in Figure 15(a), the seasonal change in wall movement shows positive in winter and negative in summer in the same direction as the influence of soil expansion/contraction, which indicates that the latter factor plays a more important role in the seasonal movement of diaphragm wall. When linear elastic model is used, after 20 years the wall moved about 1 mm towards the unexcavated side, while when nonlinear model is used, it moved 1 mm towards the excavated side (see Figure 15(a) versus Figure 15 (b)). This large difference is caused by the strain reversal effect. When the non-linear soil model implemented, a very large soil movement towards the excavated side occurred during excavation stage, which would make the soil stiffness increase sharply when the wall moves toward the unexcavated side. Hence, the ground can move normally towards excavated side in summer but pushes back much less during winter and long-term consolidation.

(a) (b)

Figure 15. Relative horizontal displacement of the diaphragm wall with operation of GSHP: (a) Linear elastic model; (b) Linear non-elastic model

Fig 16 shows the bending moment profiles between winter and summer for (a) no GSHP case and (b) with GSHP case. With the GSHP operating, there is a clear difference in the bending moment between winter and summer cycles. As described earlier, the pipe circuit is installed near the unexcavated side. During winter, the coolant is 10 ˚C below the far-field soil temperature of 12 ˚C, so this side of the wall contracts. The excavated side temperature is always maintained at 18 ˚C. This temperature gradient in the wall causes contraction in the unexcavated side of the wall, inducing an additional bending moment in the wall. However, in summer, the temperature across the wall is more or less uniform, which brings back the bending moment close to the initial condition. The large cyclic bending moment variation is in accordance with the small wall movement (Figure 15) due to the mechanical constraints from soil and slabs. When the diaphragm wall is subject to the cooling/heating load, the wall tends to contract/expand, but is resisted by the slabs and surrounding soil. Therefore large thermally induced stress and bending moment would occur.

(a) (b)

Figure 16. Bending moment of the diaphragm wall with linear elastic model: (a) Without operation of GSHP; (b) With operation of GSHP

(a) (b)

Figure 17. Bending moment of the diaphragm wall with non-linear elastic model: (a) Without operation of GSHP; (b) With operation of GSHP

As shown in Figure 17, when implementing the small-strain stiffness degradation, there are no large differences to the bending moment. This is primarily because the bending moment of the diaphragm wall is governed by the differential thermal expansion of concrete due to the temperature differential across the wall. To further illustrate the key impact factor on the bending moment of the diaphragm wall, an analysis of the thermo-active diaphragm wall with different thermal expansion coefficient values of concrete was conducted. Fig 18 shows the bending moment distributions in the wall using nonlinear elastic model. With 4 times thermal expansion coefficient, the maximum seasonal change in bending moment is about 2000kNm. But when the thermal expansion coefficient is zero, the seasonal change in bending moment is purely due to the expansion/contraction of the soil, which is quite small. Therefore, the thermal expansion of the diaphragm wall has great influence on the structural performance during the GSHP operation phase. In winter, the temperature difference is the largest, resulting in large increases to curvature and therefore moment. The thermal expansion coefficient of concrete can be controlled by the type of binding material (e.g. cement) and aggregates used when it is cast, which can be part of the design consideration of the thermo-active diaphragm wall.

(a) (b)

(c)

Figure 18. Bending moment of the diaphragm wall with variations in the thermal expansion coefficient of concrete: (a) 0 times; (b) 1 times; (c) 4 times

7 Conclusions

The wall-soil interaction behaviour of a thermo-active diaphragm wall at Crossrail's Tottenham Court Road Station was investigated by conducting a thermo-hydro-mechanical finite element analysis using both linear elastic model and nonlinear elastic model. The study in this paper proposes that GSHP operation of a thermo-active diaphragm wall requires examination of the effect of (i) the concrete expansion differential within the wall, (ii) variations in earth pressures acting on the wall, and (iii) soil contraction or expansion due to changes in the ground temperature after many years of GSHP operation.

During construction stage, the diaphragm wall moves towards the excavated side. A smaller displacement is observed when using the non-linear elastic model than linear elastic model at the beginning of excavation. Due to the application of stiffness degradation of nonlinear model, this difference disappears during the final excavation step. In addition, the horizontal total stresses and pore pressure on both sides reduced due to ground movement towards the excavated side. The largest change happens at the installation of the base slab, where the final excavation took place. The soil then swells with time so that the wall is then pushed toward the excavated side by the swelling.

With no operation of the GSHP, the excess pore pressures developed during the excavation stage return to almost hydrostatic after 10 years. The wall gradually moves back toward the unexcavated side due to swelling of the soil surrounding the wall. It is found that the relative displacement in the non-linear model is much smaller than that in the linear model, caused by the sharp increase of soil stiffness due to the strain reversal effect.

During the GSHP operation cold coolant is circulated in the absorber pipes in winter. In summer, the ground temperature recovers by heat flux from the station box and the heat is stored for the following winter cycle. The long-term trend of wall movement is similar to that with no operation of the GSHP. But the GSHP operation makes the wall move in a cyclic manner; the seasonal variation is approximately 0.5-1 mm. This is caused by two factors: the thermal effects on the deformation of the diaphragm wall itself and the thermally induced volume change of the soil and pore water. Additionally, the soil contracts and small changes in earth pressures acting on the wall are observed. The pore pressures would fluctuate when the GSHP system operated due to low permeability of the soil as well as differences in the thermal expansions of the soil and water. The maximum seasonal change in pore pressure after 20 years remains 50 kPa for the linear model and 80 kPa for the non-linear model. This seasonal fluctuation in pore pressure extends the time needed for the excess pore pressure to dissipate. With the GSHP operation, the total horizontal stress increases and decreases in the summer and winter cycles, respectively. However, the seasonal changes in total stress between the cycles are rather small (up to 25 kPa) compared to pore pressure, which indicates the seasonal changes of effective stress are opposite to those of pore pressure.

In this particular case study, it is found that the thermal expansion of the wall concrete has a significant effect on the wall performance. The thermo-mechanical performance of the soil does not affect the structural performance much during the GSHP operation phase. That is, the change in bending strain is mainly governed by the temperature differential across the wall. This is when one side of the wall is exposed to the warm station temperature and the other half of the wall at the unexcavated side is cooled by the coolant in the buried pipe.

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List of notation

G_{max}	value of the horizontal tangent of the stress-strain curve at small strain
τ_{ref}	reference shear stress
γ_{ref}	reference shear strain
τ	current shear stress
γ	current shear strain

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List of tables

List of figures

GSHP: (a) Linear elastic model; (b) Non-linear elastic model

Figure 14. Horizontal total stress on the unexcavated side with operation of GSHP: (a) Linear elastic model; (b) Non-linear elastic model

Figure 15. Relative horizontal displacement of the diaphragm wall with operation of GSHP: (a) Linear elastic model; (b) Linear non-elastic model

Figure 16. Bending moment of the diaphragm wall with linear elastic model: (a) Without operation of GSHP; (b) With operation of GSHP

Figure 17. Bending moment of the diaphragm wall with non-linear elastic model: (a) Without operation of GSHP; (b) With operation of GSHP

Figure 18. Bending moment of the diaphragm wall with variations in the thermal expansion coefficient of concrete: (a) 0 times; (b) 1 times; (c) 4 times