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

Yi Rui¹, Mei Yin^{2√}

¹Dr Yi Rui

Centre for Smart Infrastructure & Construction, Department of Engineering,
University of Cambridge, UK.

²Dr. Mei Yin

Schofield Centre, Department of Engineering

University of Cambridge, UK.

myin0915@gmail.com

[√] Corresponding Author: Department of Engineering, Trumpington Street, Cambridge,
CB2 1PZ, UK.

myin0915@gmail.com

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1 **Abstract**

2 Thermo-active diaphragm walls that combine load bearing and ground source heat
3 pump (GSHP) are considered to be one of the new technologies in geotechnical
4 engineering. Despite the vast range of potential applications, current thermo-active
5 diaphragm wall designs have very limited input from a geotechnical aspect. This
6 paper investigates the wall-soil interaction behaviour of a thermo-active diaphragm
7 wall by conducting a thermo-hydro-mechanical finite element analysis. The GSHP
8 operates by circulating cold coolant into the thermo-active diaphragm wall during
9 winter. Soil contraction and small changes in the earth pressures acting on the wall are
10 observed. The strain reversal effect makes the soil stiffness increase when the wall
11 moves in the unexcavated side direction, and hence gives different trends for
12 long-term wall movements compared to the linear elastic model. The GSHP operation
13 makes the wall move in a cyclic manner, and the seasonal variation is approximately
14 0.5-1 mm, caused by two factors: the thermal effects on the deformation of the
15 diaphragm wall itself and the thermally induced volume change of the soil and pore
16 water. In addition, it is found that the change in the bending moment of the wall due
17 to the seasonal GSHP cycle is mainly caused by the thermal differential across the
18 wall during the winter, because the seasonal changes in earth pressures acting on the
19 diaphragm wall are very limited.

20 **Keywords**

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

22 **1 Introduction**

23 Ground Source Heat Pump (GSHP) technology can offer low carbon heating and
24 cooling, and hot water provision. Their electrical consumption and maintenance
25 requirements can be lower than those of conventional heating and cooling systems.

26 The installation of the Ground Heat Exchanger (GHE) into the ground is the major
27 component of the capital cost. In order to reduce installation costs and save
28 underground space, GHE pipes are sometimes incorporated into various
29 ground-embedded structures, such as tunnels, piles and diaphragm walls (e.g. Brandl,
30 2006; Adam and Markiewicz, 2009, Suckling and Smith, 2002; Amis et al., 2010;
31 Waboso and Gilbey, 2007). They are called thermo-active tunnels/piles/walls, or
32 collectively, thermo-active geostructures. When designing such geostructures, the
33 principal constraint is that the thermal loads applied to the geostructures must not
34 degrade their mechanical performance, i.e., their ability to support the load of the
35 building or infrastructure (e.g. Bourne-Webb et al., 2009; 2013; Laloui and Donna,
36 2011; Amatya et al., 2012; GSHPA, 2012).

37

38 In thermo-active diaphragm walls, absorber tubes are installed inside the concrete by
39 attaching them to the reinforcement cage as shown in Figure 1. There are two possible
40 GSHP operating modes: (a) both heating and cooling and (b) heating only. If a

41 thermo-active diaphragm wall is used for a basement, then the interior side of the
42 diaphragm wall is insulated to ensure that the heat from the exchangers transfers into
43 the soil rather than into the basement. By doing so, thermo-active diaphragm wall can
44 cater for both heating and cooling demand of the aboveground structures. For
45 underground railway stations, there is the possibility for extracting heat in winter time
46 from both sides of the wall (station and soil) because stations often experience
47 excessive heat generated by train operations, as shown in Figure 2. In summer, the
48 GSHPs are not used and the excessive heat from the station would transfer into the
49 soil. The heat stored during summer then can be used in winter for heating the
50 aboveground structures. This type of thermo-active diaphragm wall will be discussed
51 in details in subsequent sections.

52

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

55

56

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

58

59 Compared to the number of research conducted for thermo-active piles, understanding
60 of the thermo-mechanical performance of diaphragm walls with embedded heat
61 exchangers is rather limited. Brandl (2006) described the application of geothermal
62 infrastructure in Vienna. In this project, geothermal loops were installed in diaphragm

63 walls, piles, base slabs and tunnel linings. Brandl (2006) predicted that the whole
64 system could provide 81 kW heating, while the heating demand of the station was
65 about 95 kW during peak hours. Another application of thermo-active diaphragm
66 walls is in the Uniqa Tower, Vienna (Adam et al., 2009), which reached down to 35 m
67 below the surface. The whole system produced a heating capacity of 420 kW and a
68 cooling capacity of 240 kW. Amis et al. (2010) described the thermo-active
69 diaphragm wall at Bulgari hotel in Knightsbridge, UK and discussed the potential
70 effects of thermal changes during operation. It was found that the thermal resistance
71 and thermal conductivity detected from TRT in the construction period changed when
72 the station box was finished, due to removal of the soil. Amis (2011) reported a rise of
73 20% in thermal resistance as well as a 13% reduction in the thermal conductivity
74 value. In addition, some other research were performed to investigate on the factors
75 affecting the energy performance of thermo-active diaphragm walls (Xia et al., 2012;
76 Donna et al. 2016). Stewart et al. (2014) used an analytical model to evaluate the
77 effect of incorporating heat exchangers into a geosynthetic-reinforced retaining wall.
78 It was assumed that the heat would improve the undrained shear strength and stiffness
79 of the backfill soil. However, it was found that the heat also play an opposing role in
80 the deformation response of the wall. Kürten et al. (2015) developed a semi-analytical
81 calculation model to study the thermal performance of a thermo-active diaphragm
82 wall. This new approach has been proven to be suitable for the design of plane energy
83 geostructures through comparison with pure finite element simulations and laboratory
84 results. Sterpi et al.(2017) investigated coupled thermo-mechanical behaviour of a

85 thermo-active active diaphragm walls by finite element analysis and concluded that
86 the thermally induced mechanical effects on internal axial forces and bending
87 moments are not negligible. Another numerical analysis performed by Bourne-Webb
88 et al. (2016) showed that thermally induced mechanical response of diaphragm wall
89 are dominated by seasonal temperature changes.

90

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

103

104 In this paper, a case study on a thermo-active diaphragm wall installed at one new
105 underground metro station in London is used to illustrate the engineering assessments
106 conducted to address the above three issues. The heating only mode is considered in

107 this study. A series of thermo-hydro-mechanical (THM) finite element analysis
108 simulations were conducted for the assessment. First, the construction phase was
109 modelled and the modelled displacement of each stage was compared to the actual
110 measured lateral movement data from inclinometers. Second, the GSHP operation
111 phase was modelled to investigate the short and long term responses of the diaphragm
112 wall and soil due to (i) construction of the wall only, and (ii) construction of the
113 thermo-active diaphragm wall and the operation of the GSHP. Both the linear elastic
114 and the non-linear elastic model were used to show the effect of the nonlinear
115 elasticity of soil on the performance of a thermo-active diaphragm wall.

116

117 **2 Mechanics of THM coupled processes**

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

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

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

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

126

127 In this study, the saturated soil is assumed as a mixed continuum of three independent

128 overlapping phases, displacement of soil skeleton, pore fluid flow and heat transfer.

129 The THM coupling model could be obtained according to principles of continuum
130 mechanics.

131 (1) Mechanical equilibrium equations:

$$132 \quad \nabla \cdot (\boldsymbol{\sigma} - p\mathbf{I}) + \rho\mathbf{g} = \mathbf{0} \quad (1)$$

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

$$136 \quad \nabla \cdot \boldsymbol{\sigma} = \frac{\partial \sigma_{ji}}{\partial x_j} \mathbf{e}_i \quad (2)$$

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

$$140 \quad d\boldsymbol{\sigma}' = \mathbf{D}^e : d\boldsymbol{\varepsilon}^e + \mathbf{D}^{Te} dT \quad (3)$$

141 $\boldsymbol{\varepsilon}^e$ is elastic strain tensor, T is temperature, \mathbf{D}^e is the fourth-order elastic material
142 tensor, \mathbf{D}^{Te} is the second-order thermo-elastic material tensor.

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

$$144 \quad D_{ijkl}^e = \lambda \delta_{ij} \delta_{kl} + 2G \delta_{ik} \delta_{jl} + 2G \delta_{il} \delta_{jk} \quad (4)$$

145 where $\lambda = \frac{2G\nu'}{1-\nu'}$, ν' is the poisson ratio and G is the shear modulus. In addition, the

146 components of thermo-elastic tensor \mathbf{D}^{Te} can be written as:

$$147 \quad D_{ij}^{Te} = D_{ijkl}^e \alpha_T \delta_{kl} \quad (5)$$

148 where α_T is the thermal expansion coefficient of soil skeleton.

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

150
$$\mathbf{D}^e: d\boldsymbol{\varepsilon}^e = D_{ijkl}^e d\varepsilon_{kl}^e \quad (6)$$

151

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

161
$$\tau = \tau_{\text{ref}} + \frac{G_{\text{max}}(\gamma - \gamma_{\text{ref}})}{1 + \frac{a}{n}|\gamma - \gamma_{\text{ref}}|} \quad (7)$$

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

166

167 The model behaves elastically until the onset of yielding which is determined by the
168 Mohr-Coulomb yield criterion. The thermo-elastic soil model is used in this study,
169 which indicates that soil yielding is independent of temperature change. This
170 assumption applies for the soil with high over consolidation ratio (Hueckel and Baldi,
171 1990; Boudali et al., 1994; Laloui et al., 2003), like London Clay and Lambeth Group.

172

173 (2) The transient saturated groundwater flow:

174
$$\nabla \cdot \left(-\frac{k}{r_w} (\nabla p - \rho_w \mathbf{g}) \right) + \operatorname{div} \dot{\mathbf{u}} + \frac{n}{K_w} \dot{p} - n \alpha_{T_w} \dot{T} = 0 \quad (8)$$

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

$$\operatorname{div} \dot{\mathbf{u}} = \sum_{\alpha=1}^3 \frac{\partial \dot{u}_i}{x_i}$$

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

180
$$-\nabla \cdot (\lambda \nabla T) + c_{sw} \dot{T} = 0 \quad (9)$$

181 where λ is heat conductivity of saturated soil, c_{sw} is the volumetric heat capacity of
182 saturated soil.

183

184 **3 Thermo-active diaphragm wall**

185 This paper investigated on the thermo-active diaphragm installed at a new
186 underground metro station in London, which has already been constructed, and will
187 be used to provide 200kW-1MW of energy for station space and water heating only.

188 The station box was constructed using the ‘bottom up’ method. The station box is
189 16 m wide and 29 m deep, as shown in Figure 3. It has four temporary props and six
190 slabs, where one slab is a direct replacement of a temporary prop. A one metre thick
191 diaphragm wall (up to 41 m depth) was first installed by excavating a trench to the

192 required depth. The absorber pipes were attached to the reinforcement cage and
193 lowered into the trench. After that, concrete was poured in to cast the diaphragm wall.
194 The soil inside the diaphragm box was excavated 29 m deep, and temporary props
195 were added to support the excavation. Slabs were then cast from the bottom and work
196 proceeded upwards, replacing the props with slabs to form five levels station box.

197

198 **Figure 3. Geometry of Dean Street Station Box**

199

200 **4 Finite element model**

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

208

209

210

211

212

(a)

213

(b)

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

214

215

216

217 Only horizontal displacement is restricted in the left-hand-side (LHS) and

218 right-hand-side (RHS) boundary. The top soil boundary is free, allowing possible

219 settlements induced by the operation of the thermo-active diaphragm wall. The lateral

220 pressure ratio k_0 of the diaphragm wall is set as 1, considering the wall installation

221 effect. In addition, the water table is kept constant at zero pressure at the soil surface

222 for simplicity. The pore pressure distribution is hydrostatic for all elements initially,

223 and the RHS of the mesh is kept hydrostatic throughout. Drainage is allowed at the

224 bottom and the RHS boundaries. The circulating flow in the pipe was not simulated.

225 Instead, the temperature of pipe circuit is set as variable values to simulate the 20

226 years GSHP operation stage. Underground metro stations often have excessive heat

227 generated by train operations (Rees, 2016). Hence, the temperatures of the station box

228 were kept at 18°C and the far-field soil were kept at 12°C respectively for the whole

229 20 years. The initial soil temperature was 12°C. The temperature in the pipes varied

230 between winter and summer cycles; the pipe temperature was set to be 2°C and 18°C

231 for winter and summer, respectively. The temperature boundary conditions applied are

232 summarised in Figure 5.

233

234

235

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

237

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

249

250

(a)

(b)

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

253

254 The model parameters for the linear model were selected based on Crossrail's design
255 guideline, as shown in Table 1. For non-linear model, Parameter G_{\max} and a for
256 London Clay were determined by matching the shear modulus with triaxial test data
257 by Gasparre (2005), as shown in Figure 6(a). Parameter G_{\max} and a for Lambeth
258 Group were determined by matching the shear modulus, which is the converted
259 undrained young's modulus by Hight et al. (2004), as shown in Figure 6(b). Parameter

260 a for Terrace Gravel and Chalk came from the tests by Liao et al. (2013) and
261 Heymann (1998). Model calibration was performed by varying the
262 thermo-mechanical model parameters so that the numerical results matched the field
263 test data, such as the wall displacement. All mechanical parameter values for the soil
264 model are listed in Table 1. Thermal parameters are the same for both linear and
265 nonlinear analysis, as shown in Table 2.

268

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

270

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

272

273 **5 Construction phase and long-term behaviour without** 274 **GSHP operation**

275 *Short-term behaviour*

276 The construction processes were simulated prior to simulating the GSHP operation in
277 the wall. The model performance was checked first with the actual monitoring data
278 during excavation to verify whether an appropriate set of stiffness in the soil stratum
279 had been selected for the simulation. Figure 7 shows the lateral displacement profile
280 of the wall at different stages of excavation. In general, the wall displacements
281 computed by the non-linear model are smaller than those using the linear model, and
282 compare well with the inclinometer data. The difference between the two model cases
283 decreased with increase in excavation depth. During the final excavation step, the

284 difference was rather small because the amount of shearing increased enough at the
285 end of excavation that the stiffness values of the two models became relatively similar.
286 The profile and magnitude of displacements were similar between the computed
287 results and the measured data, providing some confidence in using the model.

288

289 (a) Excavation level (117.2 m) (b) Excavation level (111.9 m)
290 (c) Excavation level (109.7) (d) Excavation level (96.8)

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

293

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

299

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

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

313

314 (a) (b)

315 (b)

316 (c) (d)

317

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

321

322 *Long-term behaviour*

323 After the station box construction, the soil was allowed to consolidate so that the
324 long-term “drained” conditions could be achieved. Figure 9 shows the horizontal
325 incremental displacement profiles over the long term (20 years) when there is no
326 GSHP operation. In both models, the magnitudes of the computed long-term
327 movements are much smaller than the short-term movements (20 mm horizontal
328 movement in the short-term versus 1 mm horizontal movements in the long-term).

329

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

344

345

346

(a)

(b)

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

349

350 Figure 10 shows the changes in pore pressure with time. The pore pressures slowly

351 converge to the initial hydrostatic distribution with time. If there is no GSHP
352 operation, the excess pore pressures developed during the excavation stage dissipates
353 to a large extent within the first 10 years.

354

355

356

(a)

(b)

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

359

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

366

367

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

370

371 **6 GSHP operation stages**

372 *Soil behaviour*

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

381

382

383 **Figure 12. Contours of temperature changes with operation of GSHP**

384

385 Figure 13 shows the pore pressure profiles at the unexcavated side of the wall for the
386 two model cases. If the GSHP system were to be operated, the pore pressures would
387 fluctuate seasonally. The pore pressure in the soil at both sides of the wall increases
388 and decreases in summer and winter, respectively. When the soil is heated, both the
389 soil skeleton and the pore fluid expand. However, due to the difference in the thermal
390 expansion coefficients as well as the low water permeability, negative excess pore
391 pressure is generated in winter when the soil is cooled. The profiles in the early years

392 have bigger differences between the two seasons because the excess pore pressure
393 from the construction phase is also added. It is shown that GSHP operation of the wall
394 extends the time needed for the excess pore pressure to dissipate.

395

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

402

403 (a) (b)

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

406

407 Figure 14 shows the total horizontal stress profiles along the unexcavated side of the
408 wall. At the unexcavated side, the total horizontal stress gradually increases with time
409 due to swelling. The general trend is similar to the no GSHP operation case. With the
410 GSHP operation, the total horizontal stress increases and decreases in the summer and
411 winter cycles, respectively. However, the changes in total stress between the cycles
412 are rather small (up to 25 kPa) compared to pore pressure as shown in Figure 13,
413 which indicates the seasonal changes of effective stress are large and opposite to those

414 of pore pressure. The small change in total stress suggests that the effect of
415 thermally-induced earth pressure on diaphragm wall behaviour, like bending moment
416 and axial force, is likely to be small. But the large cyclic change in effective stress
417 may cause more soil yielding and affect long-term stability of diaphragm wall, which
418 needs to be considered in the design.

419

420 (a)

(b)

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

423

424 *Diaphragm wall behaviour*

425 Figure 15 shows the horizontal displacement profiles during the GSHP operation
426 computed by the two models. Although the magnitude is small, the long-term
427 movement of the wall toward the excavated side first and then toward the
428 unexcavated side due to the long term swelling on the unexcavated side and excavated
429 side. The GSHP operation of the wall makes the wall move in a cyclic manner; the
430 seasonal variation is approximately 0.5 mm 1.0mm. Seasonal changes in relative
431 displacement when the GSHP is operating are mainly affected by two factors: the
432 thermal effects on the deformation of the diaphragm wall itself and the thermally
433 induced volume change of the soil and pore water. In winter, the pipe temperature is
434 cold and the station side of the wall is hot, so this creates thermal strain variations in
435 the wall, causing the wall to bend toward the excavated side. In summer, the

436 temperatures of both faces of the wall are similar, bringing the wall back toward the
437 unexcavated side. However, the seasonal changes in relative wall displacement are
438 also affected by the thermally induced volume change of the soil and pore water. In
439 summer, the soil on the unexcavated side expands, pushing the wall toward the
440 excavated side. In winter, the soil on the unexcavated side contracts, pulling the wall
441 back toward the unexcavated side. As shown in Figure 15(a), the seasonal change in
442 wall movement shows positive in winter and negative in summer in the same
443 direction as the influence of soil expansion/contraction, which indicates that the latter
444 factor plays a more important role in the seasonal movement of diaphragm wall.
445 When linear elastic model is used, after 20 years the wall moved about 1 mm towards
446 the unexcavated side, while when nonlinear model is used, it moved 1 mm towards
447 the excavated side (see Figure 15(a) versus Figure 15 (b)). This large difference is
448 caused by the strain reversal effect. When the non-linear soil model implemented, a
449 very large soil movement towards the excavated side occurred during excavation
450 stage, which would make the soil stiffness increase sharply when the wall moves
451 toward the unexcavated side. Hence, the ground can move normally towards
452 excavated side in summer but pushes back much less during winter and long-term
453 consolidation.

454

455

(a)

(b)

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

458

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

474

475

476

477

(a)

(b)

478 **Figure 16. Bending moment of the diaphragm wall with linear elastic model: (a)**

479

Without operation of GSHP; (b) With operation of GSHP

480

481

(a)

(b)

482 **Figure 17. Bending moment of the diaphragm wall with non-linear elastic model:**

483

(a) Without operation of GSHP; (b) With operation of GSHP

484

485 As shown in Figure 17, when implementing the small-strain stiffness degradation,

486 there are no large differences to the bending moment. This is primarily because the

487 bending moment of the diaphragm wall is governed by the differential thermal

488 expansion of concrete due to the temperature differential across the wall. To further

489 illustrate the key impact factor on the bending moment of the diaphragm wall, an

490 analysis of the thermo-active diaphragm wall with different thermal expansion

491 coefficient values of concrete was conducted. Fig 18 shows the bending moment

492 distributions in the wall using nonlinear elastic model. With 4 times thermal

493 expansion coefficient, the maximum seasonal change in bending moment is about

494 2000kNm. But when the thermal expansion coefficient is zero, the seasonal change in

495 bending moment is purely due to the expansion/contraction of the soil, which is quite

496 small. Therefore, the thermal expansion of the diaphragm wall has great influence on

497 the structural performance during the GSHP operation phase. In winter, the

498 temperature difference is the largest, resulting in large increases to curvature and

499 therefore moment. The thermal expansion coefficient of concrete can be controlled by

500 the type of binding material (e.g. cement) and aggregates used when it is cast, which

501 can be part of the design consideration of the thermo-active diaphragm wall.

502

503

(a)

(b)

504

(c)

505 **Figure 18. Bending moment of the diaphragm wall with variations in the thermal**

506 **expansion coefficient of concrete: (a) 0 times; (b) 1 times; (c) 4 times**

507

508 **7 Conclusions**

509 The wall-soil interaction behaviour of a thermo-active diaphragm wall at Crossrail's

510 Tottenham Court Road Station was investigated by conducting a

511 thermo-hydro-mechanical finite element analysis using both linear elastic model and

512 nonlinear elastic model. The study in this paper proposes that GSHP operation of a

513 thermo-active diaphragm wall requires examination of the effect of (i) the concrete

514 expansion differential within the wall, (ii) variations in earth pressures acting on the

515 wall, and (iii) soil contraction or expansion due to changes in the ground temperature

516 after many years of GSHP operation.

517

518 During construction stage, the diaphragm wall moves towards the excavated side. A

519 smaller displacement is observed when using the non-linear elastic model than linear

520 elastic model at the beginning of excavation. Due to the application of stiffness

521 degradation of nonlinear model, this difference disappears during the final excavation

522 step. In addition, the horizontal total stresses and pore pressure on both sides reduced

523 due to ground movement towards the excavated side. The largest change happens at the
524 installation of the base slab, where the final excavation took place. The soil then swells
525 with time so that the wall is then pushed toward the excavated side by the swelling.

526

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

533

534 During the GSHP operation cold coolant is circulated in the absorber pipes in winter. In
535 summer, the ground temperature recovers by heat flux from the station box and the heat
536 is stored for the following winter cycle. The long-term trend of wall movement is
537 similar to that with no operation of the GSHP. But the GSHP operation makes the wall
538 move in a cyclic manner; the seasonal variation is approximately 0.5-1 mm. This is
539 caused by two factors: the thermal effects on the deformation of the diaphragm wall
540 itself and the thermally induced volume change of the soil and pore water. Additionally,
541 the soil contracts and small changes in earth pressures acting on the wall are observed.
542 The pore pressures would fluctuate when the GSHP system operated due to low
543 permeability of the soil as well as differences in the thermal expansions of the soil and
544 water. The maximum seasonal change in pore pressure after 20 years remains 50 kPa

545 for the linear model and 80 kPa for the non-linear model. This seasonal fluctuation in
546 pore pressure extends the time needed for the excess pore pressure to dissipate. With
547 the GSHP operation, the total horizontal stress increases and decreases in the summer
548 and winter cycles, respectively. However, the seasonal changes in total stress between
549 the cycles are rather small (up to 25 kPa) compared to pore pressure, which indicates
550 the seasonal changes of effective stress are opposite to those of pore pressure.

551

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

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587 **List of notation**

588 G_{\max} value of the horizontal tangent of the stress-strain curve at small strain

589 τ_{ref} reference shear stress

590 γ_{ref} reference shear strain

591 τ current shear stress

592 γ current shear strain

593

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