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Numerical analysis of embedded retaining walls with coupled hydro-mechanical zero-thickness interface elements

L. Dong[1], W. Cui[1], D.M. Potts[2], L. Zdravković[2]

[1] *School of Transportation Science and Engineering, Beihang University, Beijing, China*
[2] *Department of Civil and Environmental Engineering, Imperial College London, London, UK*

ABSTRACT: Zero-thickness interface elements are often used in the finite element (FE) modelling of embedded retaining walls to represent more closely the mechanics of the wall-soil interface. When modelling the behaviour of such soil-structure interaction in an analysis where seepage and consolidation are allowed in the surrounding soil discretised by continuum elements, pore pressure variations at the wall-soil interface should also be adequately accounted for to ensure hydro-mechanical (HM) consistency. In this paper, a series of FE analyses of a retaining wall embedded in London clay has been performed in which the coupled hydro-mechanical (HM) behaviour of London clay is simulated. Different conditions at interface elements have been employed, undrained, drained and coupled, to assess their influence on the predicted results for design, such as wall movements, ground surface settlements and forces acting on the wall, demonstrating the significance of ensuring full compatibility between interface elements and the adjacent continuum elements. Parametric studies have also investigated the influence of the material properties of the coupled HM interfaces on the predicted wall behaviour.

Keywords: Finite element method; Interface element; HM coupling; retaining walls

1 INTRODUCTION

In a finite element (FE) analysis of embedded retaining walls, the wall can be discretised by either continuum elements or beam elements, while the surrounding soils are discretised by continuum elements. To model the interface behaviour between the wall and the soil, zero-thickness interface elements are often placed around the wall. The main advantages of using such a special element type are to allow relative movement between the wall and the soil and to enable the variation in constitutive behaviour of the soil-structure interface (Day and Potts, 1994).

If fully drained or undrained soil conditions are taken into account in the analysis, the same conditions can be prescribed at the interface to ensure the compatibility in pore fluid pressure change. In such analysis, as demonstrated by Day and Potts (1998), the adopted mechanical properties of the interface element (i.e. friction angle, stiffness and dilation angle) may have significantly different effects on the predicted behaviour of the retaining wall and the associated ground surface settlement. Special attentions should be paid to the stiffness matrix and stress gradients in the interface to avoid numerical instabilities (Day and Potts, 1994).

In current geotechnical design, however, the phenomenon of coupled consolidation is often required to be taken into account for modelling the real soil behaviour, as the variation of pore fluid pressures with time can significantly affect the evolution of the stress-strain behaviour of both the soil and the structure. Under such circumstances, the surrounding soils can be modelled by the coupled hydro-mechanical (HM) FE facilities, while the wall can be treated as being fully permeable, fully impermeable or with a finite permeability if continuum elements are used for discretisation (Potts and Zdravković, 1999, 2001). To further account for the behaviour of the soil-structure interface in such analysis, the formulation of the zero-thickness interface element needs to be extended to be capable of ensuring that the pore fluid pressure variation at each side of the interface is compatible with the adjacent continuum elements and/or beam elements.

A number of existing studies focused on the development (e.g. Ng and Small, 1997; Segura and Carol, 2008a; Cerfontaine et al., 2015; Cui et al., 2019) and application (e.g. Segura and Carol, 2008b; Cerfontaine et al., 2016) of coupled HM zero-thickness interface elements for the FE analysis in geotechnical engineering. However, to date the performance of these elements has not been shown in the modelling of embedded retaining walls, which is one of the most common problems involving interface behaviour in geotechnical engineering designs.

This paper aims to demonstrate the significance of using HM coupled interface elements in the modelling of soil-structure behaviour when seepage and consolidation are allowed in the adjacent soils, and to obtain a deeper insight into the effects of associated interface properties on the predicted results for design. A series of FE analyses of a retaining wall embedded in London clay was carried out with different interface conditions (i.e. undrained, drained and coupled). All FE analyses were carried out using the Imperial College Finite Element Program (ICFEP, Potts and Zdravković, 1999, 2001), which is developed specially for geotechnical engineering applications.

2 PROBLEM DESCRIPTION AND MODELLING PROCEDURES

2.1 Numerical model

A set of plane strain analyses of an existing case study of an embedded retaining wall with a single prop (Grammatikopoulou et al., 2008) was performed in this study. In the analysed case, a 10m deep excavation was considered in which the 14.6m deep concrete wall with a thickness of 0.6m was supported by a single prop placed 3m from the top of the wall. The ground behind the wall consisted of 2.5m of superficial deposits (made ground and Terrace gravel) overlying London clay which was assumed to have a depth of over 60m.The FE mesh used for all analyses presented in this paper is shown in Figure 1. Both the soil and the wall were discretised by 8-noded quadrilateral continuum elements, while 6-noded zero-thickness interface elements were specified between the soil and the backside of the wall. A modified Newton–Raphson non-linear solver with an error-controlled sub-stepping stress point algorithm was employed for all analyses.

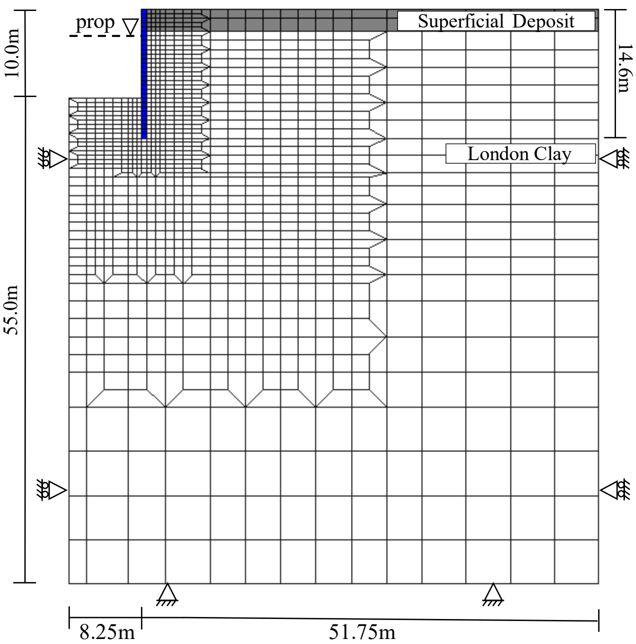

Figure 1. Finite element mesh with stratigraphy

2.2 Material properties

In all analyses, the London clay deposit was characterised by a nonlinear elasto-perfectly plastic model with a Mohr-Coulomb yield surface and a non-associated flow rule (Jardine et al., 1986), which has been extensively used for modelling geotechnical problems involving London Clay. The layer of superficial deposits was modelled by a linear elasto-perfectly plastic Mohr-Coulomb model with a Young's Modulus of $E' = 1.0 \times 10^4$kPa and a Poisson's ratio of $\mu = 0.2$. The adopted mechanical soil properties are summarised in Tables 1 and 2, the values of which are the same as those suggested by Grammatikopoulou et al. (2008). A linear elastic model with $E' = 7.0 \times 10^6$kPa and $\mu = 0.2$ was adopted for modelling the wall to take into account an out-of-plane pile spacing of 750mm, while the temporary prop was simulated as a spring with a stiffness of 7.5×10^4kN/m/m. An elasto-plastic Mohr-Coulomb model was used for modelling the interface, in which the friction angles of the interface elements were assumed to be the same as those in the adjacent soils, while both the dilation angle and the cohesion of the interface were set as zero for simplicity. The normal and shear stiffness of the zero-thickness interface elements were specified as $K_s = K_n = 1.0 \times 10^5 \mathrm{kN/m^3}$.

Table 1. Model parameters for Mohr-Coulomb yield and plastic potential

Soil layer	γ_s (kN/m^3)	c'	φ'(°)	ψ'(°)
Superficial deposits	19.0	0.0	30.0	0.0
London Clay	20.0	0.0	23.0	11.5

Table 2. Elastic parameters of the non-linear model for London Clay

Parameter	Value	Parameter	Value
A	1400.0	B	1270.0
C(%)	1.0×10^{-4}	β	1.335
γ	0.617	$E_{d\,min}$(%)	8.66×10^{-4}
$E_{d\,max}$(%)	0.693	G_{min}(kPa)	2666.7
R	686.0	S	633.0
T(%)	1.0×10^{-3}	δ	2.069
η	0.42	$\varepsilon_{v\,min}$(%)	5.0×10^{-3}
$\varepsilon_{v\,max}$(%)	0.15	K_{min}(kPa)	5000.0

In the analyses where the coupled consolidation behaviour of London clay was taken into account, a non-linear permeability model was employed for the soil, in which the permeability was assumed to be related to the mean effective stress, p', expressed as:

$$k = k_0 \cdot e^{-Dp'} \tag{1}$$

where k_0 is permeability at zero mean effective stress (m/s) and D is a model parameter (m^2/kN). In this paper, $k_0 = 1 \times 10^{-9}$m/s and $D = 0.007\mathrm{m^2/kN}$ were used. Both the superficial deposits and their adjacent interfaces were assumed to be fully drained in all analyses, while the permeability condition of the interface elements adjacent to the London clay was varied in this study.

2.3 Initial stress conditions

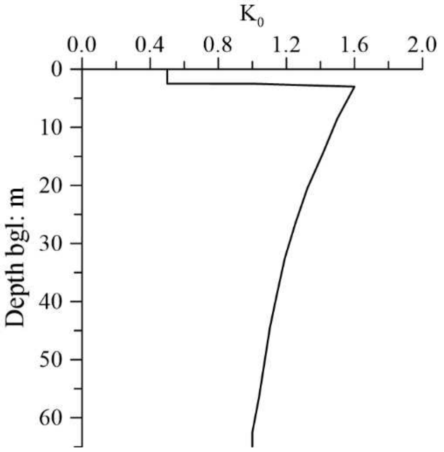

Figure 2. K_0 profile adopted in this study

The initial vertical and horizontal stresses were calculated with the unit weights listed in Table 1 and the K_0 profile shown in Figure 2. The K_0 profile was estimated based on the data from Mayne and Kulhawy (1982), in which a constant value of $K_0 = 0.5$ was specified for the superficial deposits, while K_0 was considered to vary nonlinearly with depth from $K_0 = 1.6$ at the top of London Clay to $K_0 = 1.0$ at the bottom. The initial pore water pressure distribution, prior to excavation, was assumed to be hydrostatic with depth in the ground with the water table at the top of the London clay.

2.4 Boundary conditions and modelling procedure

As shown in Figure 1, both the vertical and horizontal displacements at the bottom of the mesh were restrained, and a zero horizontal displacement boundary condition was prescribed at both lateral sides of the mesh. In the coupled HM analyses, the lateral boundaries of the London clay layer, as well as the excavated surface, were assumed to be impermeable, while a no change in pore fluid pressure boundary condition was prescribed at the top and bottom boundaries of the London clay layer.

The 10m deep excavation was simulated by excavating sequential rows of elements over a total of 25 increments and the prop was placed when the excavated surface reached 3.5m below ground level (bgl). In the coupled HM analyses, a total of 2 days was adopted for the whole excavation process, which was considered to be close to an undrained condition for the London clay. The conventional θ-method was employed for time integration with a value of $\theta = 0.8$. It should be noted that the process of wall installation was not modelled in the analysis (i.e. it was "wished in place" at the beginning of the analysis).

2.5 Analysis types

In order to demonstrate the performance and significance of the coupled HM zero-thickness interface element in the modelling of embedded retaining walls when seepage and consolidation are allowed in the adjacent soils, different types of analysis were performed in this study, which are summarised as follows:

- CN analysis: a HM coupled analysis without interface elements, in which full friction at the interface is assumed and the interaction between the pore fluid flow and mechanical deformation in the London clay is taken into account. This analysis type was adopted here as a benchmark.
- UU analysis: uncoupled analysis in which both London clay and the adjacent interface elements were assumed to be undrained. In this analysis, the relative movement between the wall and the soil was allowed. This conventional analysis type was adopted here as another benchmark.
- CU analysis: analysis with coupled HM continuum elements for London clay and fully undrained interface elements for the adjacent interface. In this analysis, the interface elements are purely mechanical and the excess pore fluid pressure at the interface was not allowed to dissipate.
- CD analysis: analysis with coupled HM continuum elements for London clay and fully drained interface elements for the adjacent interface. In this analysis, the interface elements are purely mechanical and the pore fluid pressure at the interface was assumed to remain constant.
- CC analysis: analysis with the coupled HM behaviours of both London clay and its adjacent interface being taken into account. A permeability of $k_i = 1.0 \times 10^{-9} \mathrm{m}^2/\mathrm{s}$ was adopted here for the coupled HM zero-thickness interface, which was considered to result in a similar permeability condition compared to that of the London clay. It is noted that the units for the permeability of the interface elements (m^2/s) differs to that of the soil (m/s) due to the assumption of the zero-thickness.

3 RESULTS AND DISCUSSION

3.1 Interface stress

Figures 3 demonstrates the predicted distributions of normal effective stresses in the interface behind the wall after excavation. In analysis CC, the shape of the normal effective stress distribution exhibited an almost linear variation with depth, although a considerable drop was observed at the boundary between the superficial deposits and the London Clay mainly due to the difference in their initial values of K_0. The analyses UU and CU predicted a similar shape of effective stress variation with noticeable but negligible difference compared to that from analysis CC.

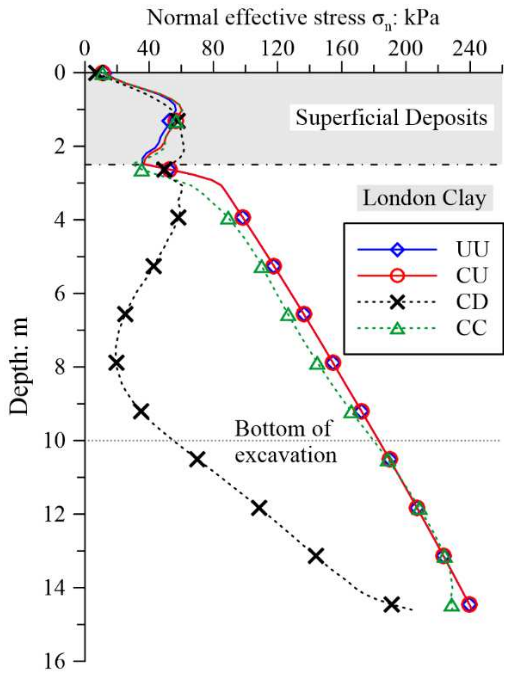

Figure 3. Distribution of predicted horizontal effective stress behind the wall after excavation.

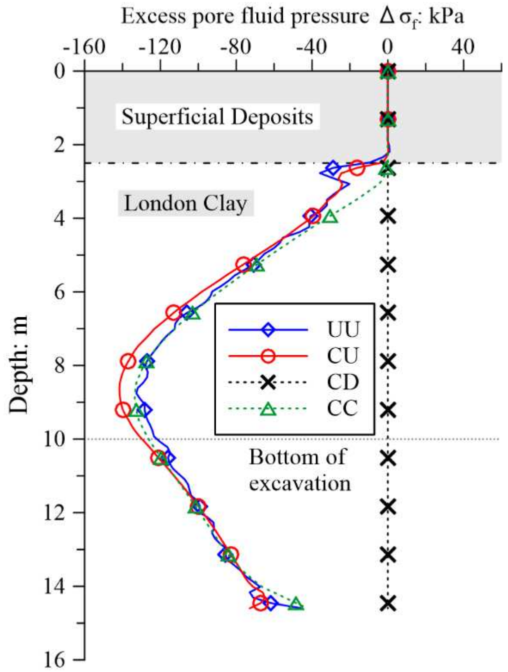

Figure 4. Distribution of predicted excess pore fluid pressure in London clay behind the wall after excavation.

In analysis CD, however, the predicted stress distribution was significantly different compared to those from other analyses which resulted in a maximum difference of around 135kPa. This was thought to be caused by the inconsistency in the pore fluid pressure variation between the interface and the soil. In analyses UU, CU and CC, as the excavation was completed rapidly, both the soil and the interface exhibited an approximately undrained behaviour. Under this condition, with the wall moving away from the soils, tensile excess pore fluid pressures were generated in the interface adjacent to the London clay with a maximum value of approximately -150kPa at around 8m bgl, as shown in Figure 4. In analysis CD, the interface was assumed to be fully drained indicating that the related interface pore fluid pressure remained constant (also see Figure 4), while the excess pore fluid pressure could generate in the adjacent continuum element representing London clay.

3.2 Ground settlement and wall behaviour

Figure 5 shows the predicted ground surface settlement troughs behind the wall after excavation. The maximum surface settlements in analyses CN, UU, CU and CC were 20.8mm, 19.7mm, 20.8mm and 20.8mm respectively, while that in analysis CD was 24.2mm which was much higher than the others. A similar feature can be seen in the predicted horizontal wall movement, see Figure 6, with the maximum displacements in analyses CN, UU, CU, CC, and CD being 45.4mm, 43.0mm, 45.4mm, 45.4mm and 49.2mm respectively.

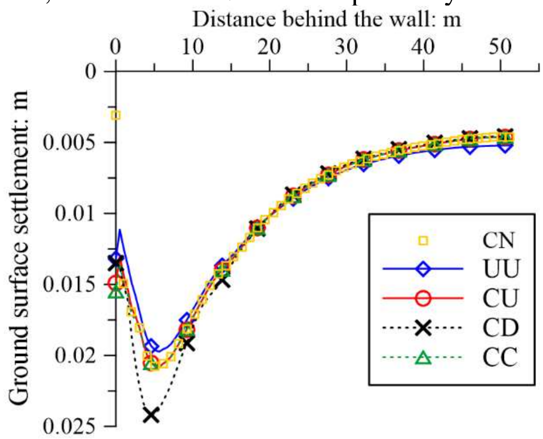

Figure 5. Predicted ground surface settlement behind the wall after excavation.

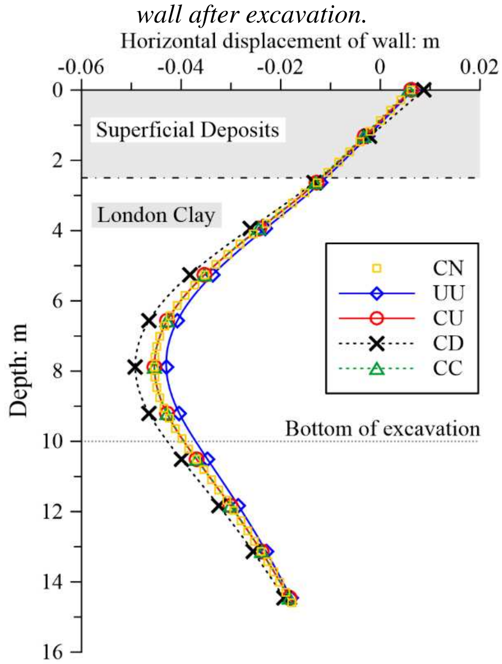

Figure 6. Predicted horizontal displacement of the wall after excavation.

Figures 7 and 8 present the distribution of bending moments and axial forces in the wall after excavation respectively. A similar variation of bending moments with depth was observed in all analyses with an almost identical maximum positive value at the position where the prop was placed. The maximum negative bending moment in analysis CD was only 5% larger than those in the other analyses, indicating that the influence of the interface permeability conditions on the predicted bending moment is insignificant.

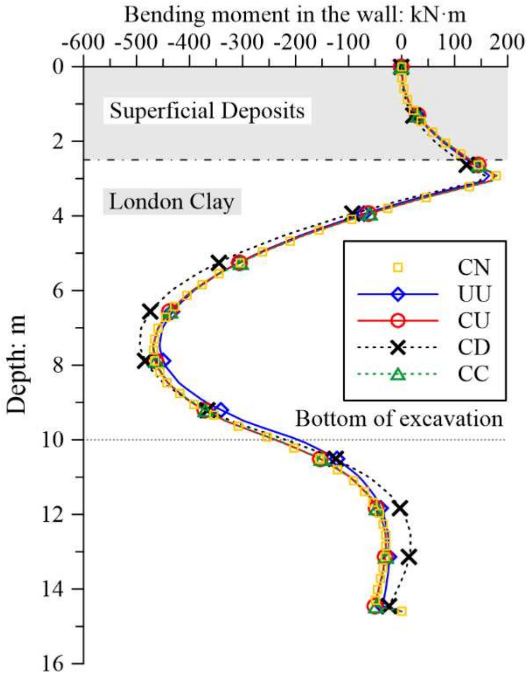

Figure 7. Predicted bending moment in the wall after excavation.

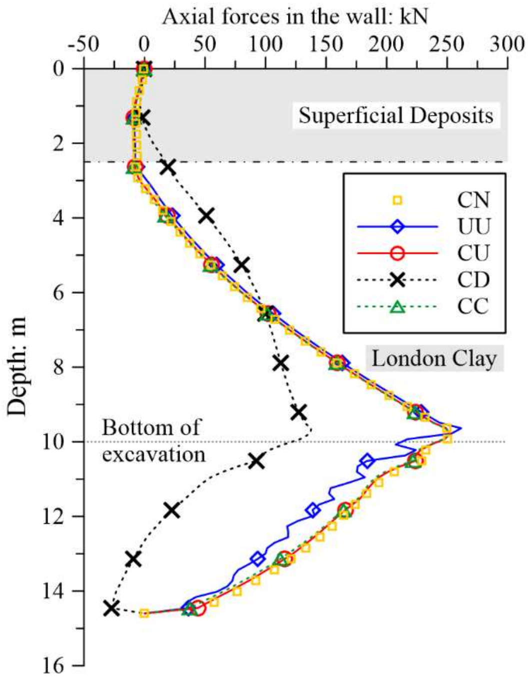

Figure 8. Predicted axial forces in the wall after excavation.

In contrast, analysis CD significantly underestimated the axial forces in the wall, with the maximum value being 50% lower compared to those in the other analyses. As shown in Figures 3 and 4, this was induced by the inconsistency in the pore fluid pressure variation between London clay and the adjacent interface, leading to erroneous effective normal stresses acting on the wall which in turn lead to lower shear stresses acting on the wall.

4 EFFECT OF INTERFACE PERMEABILITY

A permeability of $k_i = 1.0 \times 10^{-9} \mathrm{m}^2/\mathrm{s}$ was adopted for the coupled HM zero-thickness interface in the analysis shown above, which was found to result in an approximately undrained condition for the interface. Parametric studies were carried out in which analysis CC was repeated with different permeability values of $k_i = 1.0 \times 10^{-20}\ \mathrm{m}^2/\mathrm{s}$, $1.0 \times 10^{-7}\ \mathrm{m}^2/\mathrm{s}$ and $1.0 \times 10^{-5}\ \mathrm{m}^2/\mathrm{s}$, marked as analyses A1, A2 and A3 respectively.

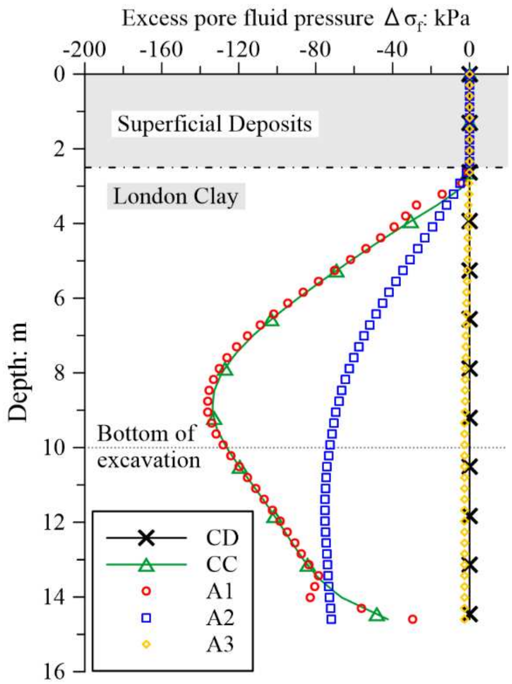

Figure 9. Predicted excess pore fluid pressure in London clay behind the wall after excavation with various interface permeability conditions.

Figure 9 compares the predicted excess pore fluid pressure distributions in the London clay along the wall with different interface permeabilities. It is noted that an almost identical excess pore fluid pressure distribution was obtained from analyses CC and A1, verifying that the interface permeability adopted in analysis CC represents an almost undrained condition. Further increasing the interface permeability accelerates the dissipation of excess pore fluid pressure via the interface, significantly altering the shape of the distribution. When the interface permeability became sufficiently large (i.e. analysis A3), a hydro static pore fluid pressure distribution was obtained, indicating that a fully drained condition was assigned to the interface. As shown in Figure 10, similar features could be observed from the comparison in the distribution of the predicted axial forces in the wall. Analyses CD and A3 produced similar results, while the results from analysis A2 were within the two extreme interface permeability conditions.

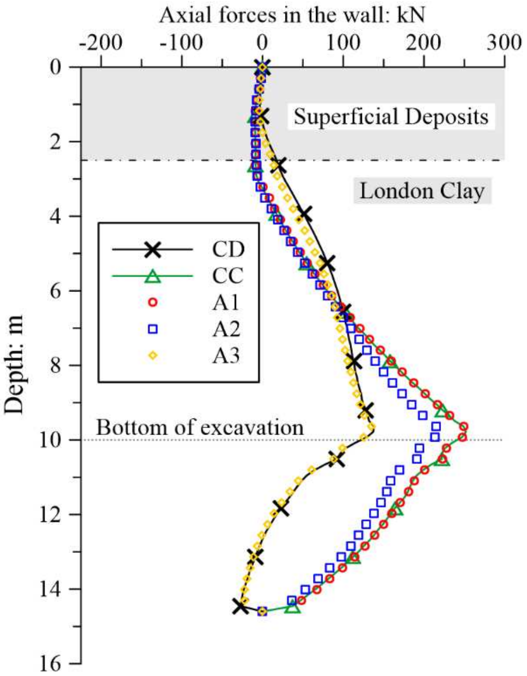

Figure 10. Predicted axial forces in the wall after excavation with various interface permeability conditions

5 CONCLUSIONS

This paper presents a numerical study on the use of coupled hydro-mechanical (HM) zero-thickness interface elements in the modelling of embedded retaining walls. The main findings are summarised as follows:

(1) When the soil-structure behaviour is taken into account in a FE analysis of embedded retaining walls where seepage and consolidation are allowed in the surrounding soils, coupled HM interface elements are required to ensure the consistency in pore fluid pressure variation at the interface.

(2) Full compatibility, in respect of permeability conditions, between zero-thickness interface elements and the adjacent continuum elements for discretising the surrounding soils is necessary to avoid erroneous numerical solutions.

(3) As the units of the interface permeability differ from that of the soil, special attention should be paid to the chosen value which may significantly affect the predicted results for design.

6 ACKNOWLEDGEMENTS

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