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Comparative Study of Anchored Wall Performance with Two Facing Designs

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ABSTRACT: The present study compared the performance of soldier pile and concrete bearing pad anchored wall facings. Using Abaqus finite element software, two case studies have been precisely represented for the facing designs and effects of the parameters of soil type, spacing of anchors and facings, surcharge and facing sizes were investigated. The analysis results indicate that the soldier pile method can efficiently reduce anchored wall deformation, especially at the wall crest. The horizontal deformation at the top of the anchored soldier pile wall was about half of the wall anchored with concrete bearing pads. Soil arching between the anchors in the horizontal direction was more effective in the soldier pile wall and the bending moment of the laggings in the soldier pile wall was considerably less than that of the anchorage with bearing pads.

Keywords: Anchor, Bearing Pad, Excavation, Facing Designs, Soldier Pile.

INTRODUCTION

The use of prestressed anchors is a suitable method for stabilization of deep urban excavations close to buildings which are sensitive to deformation. The facing of the anchored walls usually involves continuous elements such as sheet piles, secant concrete piles or discrete elements such as soldier piles with laggings. Concrete bearing pads are another alternative for anchored wall facings. In this method, the wall facing, which includes concrete bearing pads in place of anchors and a shotcrete layer between the bearing pads, is constructed in parallel to the excavation in a manner that is similar to the nailing method.

The main differences between the soldier pile and bearing pad methods are the rigidity of the soldier piles and their continuity along the height of the wall. Bearing pads are individual and discontinuous in the horizontal and vertical directions of the wall and therefore are less rigid than soldier piles. Another difference is related to the construction sequence. Soldier piles are constructed before excavation and bearing pads are constructed in parallel with the excavation; thus, the embedded depth of the soldier piles is below the final excavation

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level. This increases the rigidity of the wall, although there is no embedment depth for concrete bearing pads.

Arching effects can occur in both of these methods. The soil pressure is primarily tolerated by the soldier piles and bearing pads and the lagging elements do not bear a large percentage of the soil pressure. The arching effect in these two facing designs is expected to be different because of the difference in the rigidity of the elements of wall facing.

In some researches, scientists have studied anchored wall behavior using numerical simulation. The behavior of anchored soldier pile walls is mainly three-dimensional (3D); thus, 3D numerical models can accurately simulate the spacing between the soldier piles and the rigidity of the wall facing elements.

A leading method of 3D numerical analysis was developed by Briaud and Lim (1999). They studied the effect of design parameters for anchored walls using nonlinear analysis in Abaqus and investigated the design implications. Vermeer et al. (2001) assessed the impact of earth pressure on anchored soldier pile walls. They created 3D numerical models using Plaxis 3D software and found that soil pressure is mainly supported by the soldier piles and that the horizontal stresses between the piles are negligible. Hong et al. (2003) analyzed soldier-pile excavations using 2D and 3D numerical models and pointed out the shortcomings of 2D analysis when modeling a soldier pile wall. Mun and Oh (2016) presented a 3D numerical simulation of a real excavation project stabilized by the application of hybrid soldier piles, tiebacks and soil nails. There was accordance between the predicted hybrid wall deformations and soil nail forces with field measurements.

Fewer studies have focused on anchorage with concrete bearing pads. Baghaee and Dubakhshari (2011) compared the results predicted by Plaxis 2D for displacement of deep excavations stabilized by nailing and anchorage with concrete bearing pads. They concluded that the displacement values obtained from the anchorage method are smaller than for the nailing method. Iskandari (2013) studied the impact of wall and anchor properties on the behavior of anchorage with concrete bearing pads using a 2D numerical model. Talebi (2014) used Flac3D to study the effect of excavation depth and angle and length of anchors on the displacements of this system. Evaluating the behavior of the materials is also of considerable importance in analysis of deep excavation analyses (Ghanbari et al., 2013; Seyedan and Seyedi Hosseininia, 2015).

In the current study, case studies were selected for each excavation support system as a verification model to evaluate the agreement of measured results with that of models constructed using the finite element software Abaqus. Comparative analysis of the anchored wall performance using soldier piles and bearing pads was done by varying the parameters of c-phi soil type, surcharge, facing spacing and facing size. Abaqus was used for the 3D non-linear stress-strain analysis. The numerical models of the anchored walls were created in Abaqus to estimate the effects of these two facing designs on wall displacement and the internal forces of the facing elements.

VERIFICATION OF NUMERICAL MODEL

A real excavation supporting system was selected for each facing design to verify the numerical modeling in Abaqus 3D. These are discussed separately below.

Case Study of Soldier Pile Method

The selected case study for verification of numerical model is located in Karaj city, Iran. This excavation has a height of 16.2 m from the foundation of the adjacent building to the bottom of the excavation. Anchored wall section is demonstrated in Figure 2. The soldier piles have been placed 2.5 m apart and the section is an IPE 270 double profile. Figure 1 also shows other details including the total length and bond length of the anchors.

The anchors are R51L and R32S self-drilling rods with outer diameters of 51 and 32 mm and prestressing forces of 400 and 250 kN, respectively. A shotcrete wall with the thickness of 10 cm has been sprayed between the soldier piles and then reinforced with a layer of welded wire mesh. As it can be seen in Figure 1, the soldier piles are embedded 4 m below the final excavation level inside reinforced concrete piles of 80 cm in diameter. The neighboring building is 11 stories, which is considered to be a surcharge in the simulation. Each floor of the building weighs about 10 kN per unit area; totally 110 kPa is calculated for the building surcharge. A wall length of 6.25 m was used for modeling, as demonstrated in Figure 2, showing the location of piles at the wall facing and anchors.

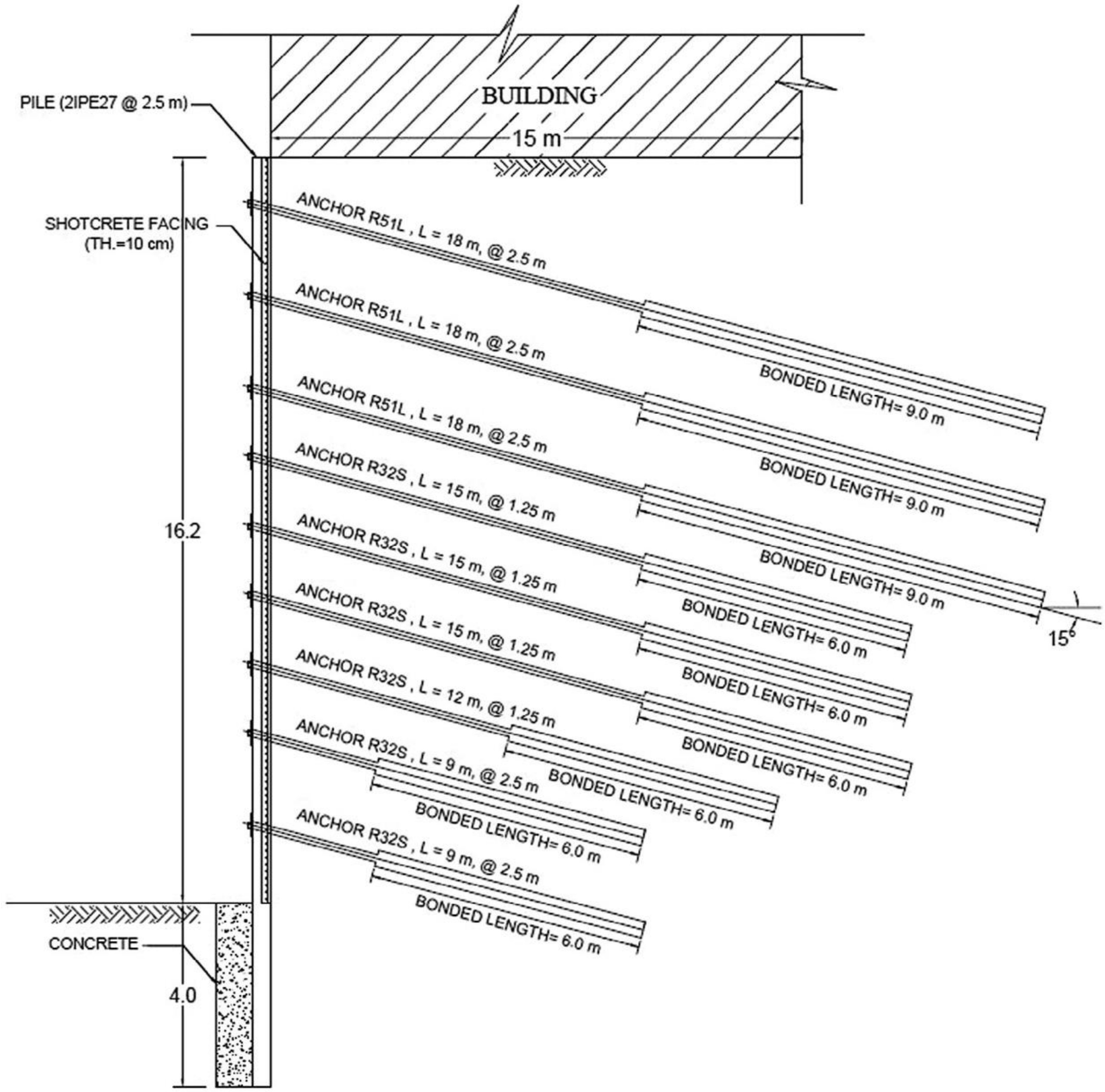

Fig. 1. Section of the case study wall for soldier pile method

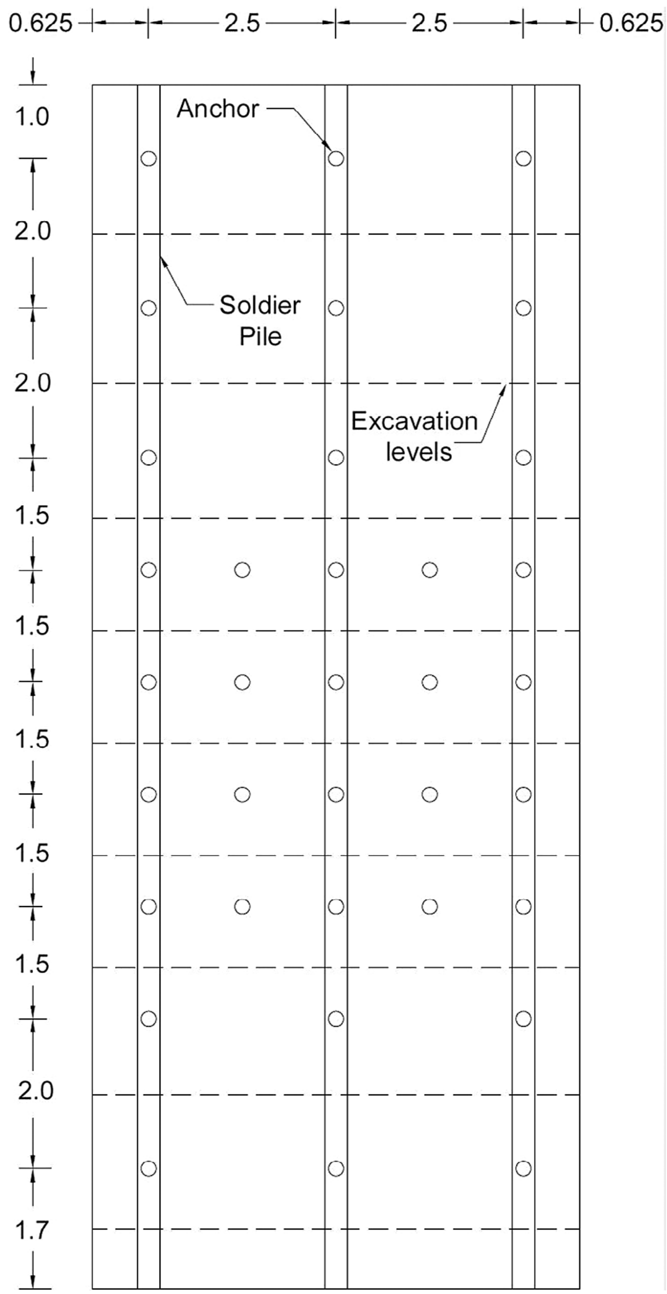

Fig. 2. Front view of the simulated length of the case study wall for soldier pile method (all dimensions are in m).

The anchors are R51L and R32S self-drilling rods with outer diameters of 51 and 32 mm and prestressing forces of 400 and 250 kN, respectively. A shotcrete wall with the thickness of 10 cm has been sprayed between the soldier piles and then reinforced with a layer of welded wire mesh. As it can be seen in Figure 1, the soldier piles are embedded 4 m below the final excavation level inside reinforced concrete piles of 80 cm in diameter. The neighboring building is 11 stories, which is considered to be a surcharge in the simulation. Each floor of the building weighs about 10 kN per unit area; totally 110 kPa is calculated for the building surcharge. A wall length of 6.25 m was used for modeling, as demonstrated in Figure 2, showing the location of piles at the wall facing and anchors.

Modeling of this excavation was done in

Abaqus 3D. In this software, the first step was the "initial phase" where the boundary conditions and interactions were applied. The second phase is where gravity load and surcharge is applied and all deformations were set to zero. In the third phase, the soldier piles were embedded in the soil, the soil excavated, the shotcrete sprayed on and the anchors set up and then prestressed. The last three steps were repeated until achieving the desired height. Figure 3 illustrates the excavation levels as horizontal dashed lines. The soil parameters were achieved using in situ shear and plate loading tests and laboratory experiments and are demonstrated in Table 1.

In the models simulated in Abaqus, the soil was modeled with 8-node 3D elements (C38DR), the shotcrete wall as a shell with 4-node 2D elements (S4R) having linear elastic behavior, the pile with elements of a 2-node beam (B3D2) and the bonded length of the anchor with elements of a 2-point truss (T3D2). The unbonded length of the anchors connected the beginning of the bonded part to the wall and only axial stiffness was considered. The equivalent modulus of the anchor and grout was used in the bonded length. In the unbonded length, in which the anchor exists without grout, the elastic modulus of the anchor was used.

Table 1. Geotechnical parameters of soil layers in the case study for soldier pile method

Parameter	Symbol	Unit	Value of soils: Upper layer (0-12) m	Value of soils: Lower layer > 12 m
Unit weight	γ_{sat}	kN/m^3	19	19
Elastic modulus	E	Mpa	60	60
Reference loading modulus	E_{50}^{ref}	Mpa	60	60
Reference Unloading	E_{ur}^{ref}	Mpa	180	180
Poisson's ratio	ν	-	0.3	0.3
Cohesion	c	kPa	10	20
Friction angle	φ	deg	40	38
Angle dilatancy	ψ	deg	10	8

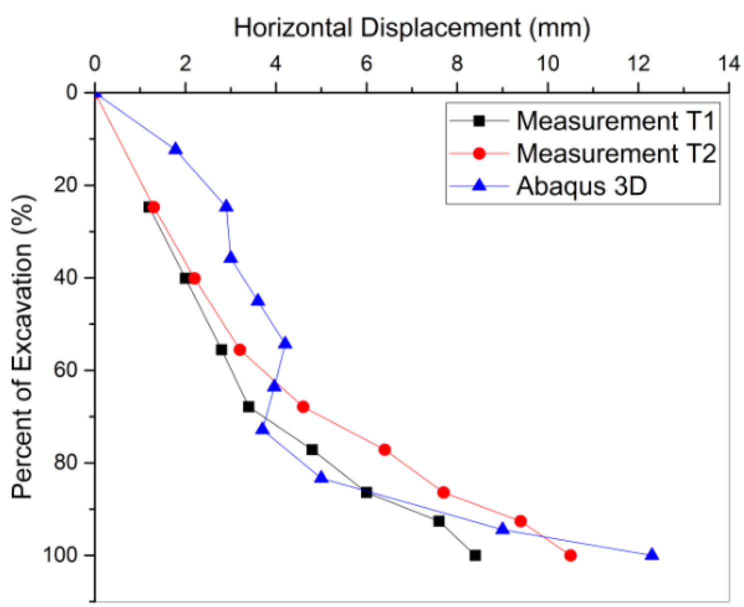

Fig. 3. Comparison of predicted and measured horizontal displacements of the case study wall for soldier pile method

Figure 3 compares the results from the numerical model for horizontal deformation of the wall crest and the values measured during excavation. This graph includes horizontal deformations versus the percentage of excavation completed. In this software, horizontal deformation of the wall crest was 12.3 mm. The final measured displacement was 8.4 and 10.5 mm at points T1 and T2, respectively (on the soldier piles). The model used for verification was a real project monitored by measurement of the displacement at the wall crest; therefore, there was inadequate formation to determine wall behavior and deformation along the excavation height. There were no large or abnormal deformation at the lower levels of the wall during excavation. The predicted deformations were very close to the real results. Note that displacements of the soldier piles and laggings were not equal and deflection of the laggings was usually greater than that of the soldier piles (Hong et al., 2003).

Case Study of Concrete Bearing Pad Method

To verify the numerical model for concrete bearing pads, an excavation in Tehran having a depth of 31.5 m which was stabilized with concrete block anchors was simulated and the results were compared with the data recorded during excavation. The monitoring data includes the horizontal displacement of top of the excavation walls measured using micro-geodetics by installing pillars around an excavation and attaching reflectors to the wall surface. At ground level around the pit, there were no specific structures or loadings.

Boreholes and test pits were drilled around the excavation and samples were obtained for laboratory testing. Several PLT and in situ direct shear tests were conducted to determine the subsurface layers and geotechnical parameters of the soil layers. The subsurface layers consisted of coarse-grained gravelly and sandy soil with a considerable fines content. Cementation in the soil texture increased its cohesion. The soil layers and geotechnical parameters of each layer are presented in Table 2.

Figure 4 shows the section of anchored wall. The anchors are multi-strand (4 and 6-strands), where each strand has a diameter of 15 mm, an area of 140 mm^2 and a design load of 156 kN. The dimensions and thickness of the concrete pad (Table 3) have been designed based on the prestressing forces of the anchors and the passive soil pressure by depth. The thickness of the shotcrete layer is 10 cm. The two upper rows are nails with rebar of 25 mm in diameter.

A 3D model of this wall was created in Abaqus. To accommodate the different horizontal spacing of the anchors and nails at various levels, 15 m of the wall was considered for analysis to allow the number of nails and anchors in the model to be proportional to the horizontal spacing.

Figure 5 compares the results of analysis for horizontal displacement according to the percentage of excavation completion and the measured values at reflectors T15 and T16. These reflectors were installed 2.5 m below the top of the wall and recording of displacement started from a depth of 6 m. As seen, the horizontal displacement obtained from 3D analysis is in relatively good agreement with the measured data. The final displacement of the wall was recorded to be about 6 cm and in the numerical model estimated to be about 8 cm. The difference between the results of modeling and the measurements can be attributed to uncertainty in the measurement of the geotechnical parameters of the soil layers.

NUMERICAL MODELS

Walls anchored with soldier piles and bearing pads were modeled and the horizontal displacements and shotcrete bending moments from these two methods were compared. Two types of c-phi soil with different cementation values and anchor spacing (2 and 3 m) were considered with and without surcharge. The soil and shotcrete wall characteristics are listed in Tables 4 and 5, respectively. Soil type 2 had lower cohesion than soil type 1. The other mechanical parameters of the soil were assumed to be constant in order to focus on the effects of soil cohesion and the difference rates of cementation in the soil samples. These mechanical parameters for soil were assumed to be similar to Tehran soil, which has high cementation value. Table 6 lists the appropriate size for the soldier pile and a 0.5 m thickness for the pads.

All of the elements were modeled in Abaqus according to specifications considered for case studies. The lateral sides of the model were tied in the horizontal directions and the base of the model was fixed in three directions as illustrated in Figure 6.

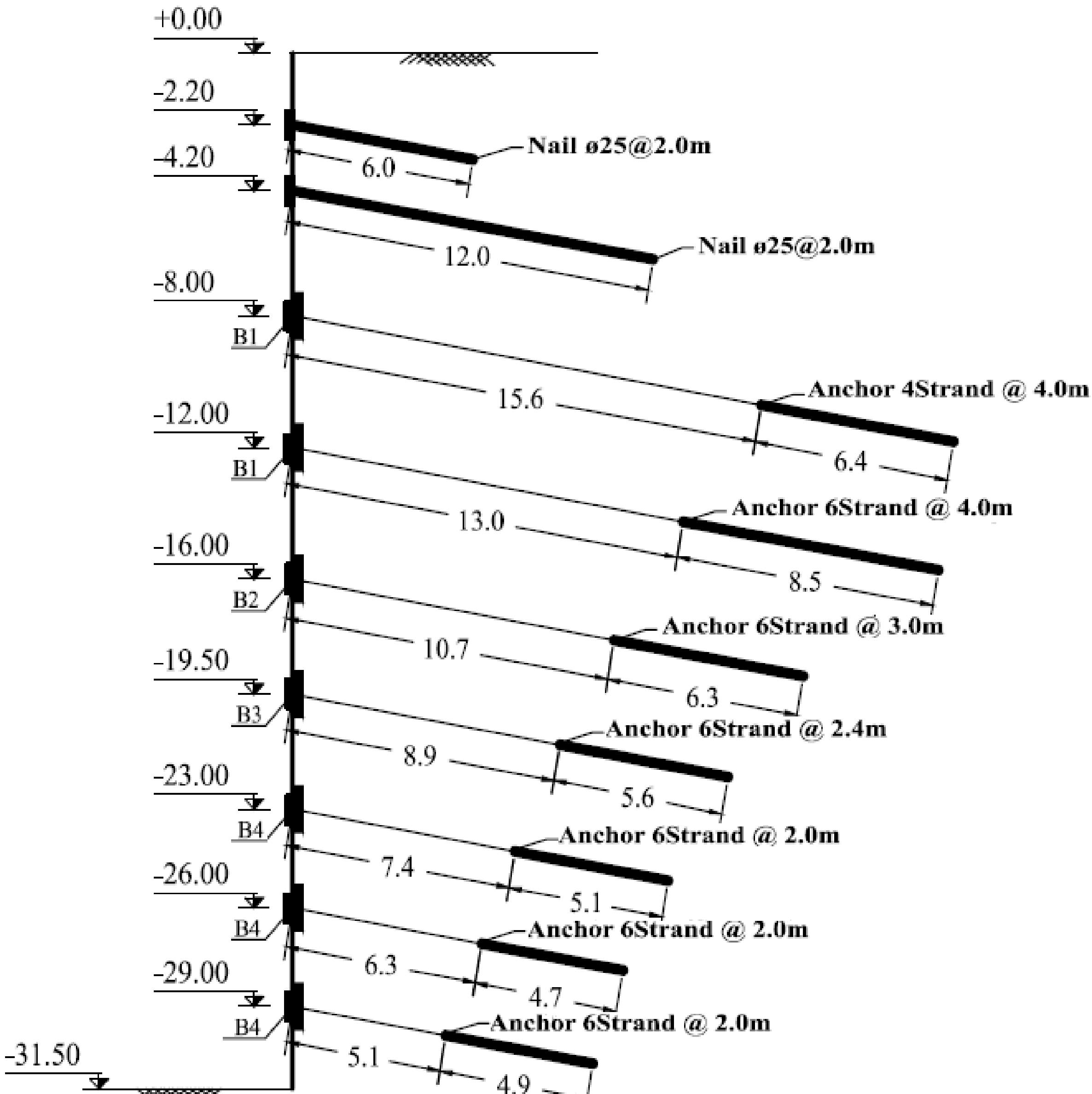

Fig. 4. Section of the case study wall for bearing pad method

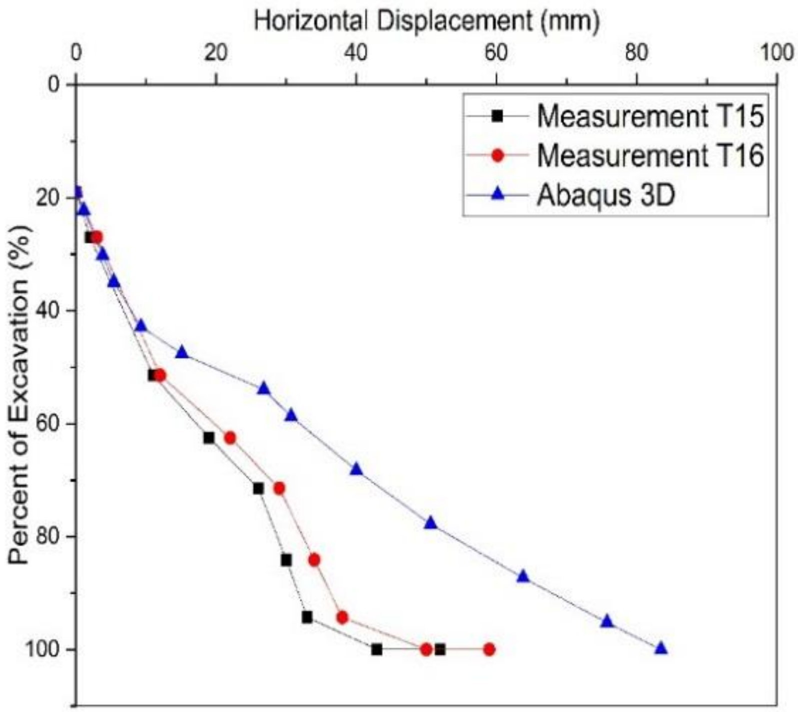

Fig. 5. Comparison of predicted and measured horizontal displacements of the case study wall for bearing pad method

Table 2. Geotechnical parameters of soil layers in the case study for bearing pad method

Soil layer	Depth (m)	γ (kN/m³)	φ	c (kPa)	E (MPa)
1	0-3	21	35	60	80
2	3-6	19.5	10	100	40
3	8-14	21	35	30	80
4	14-16	19.5	10	100	40
5	> 16	21	35	60	80

Table 3. Dimensions of the concrete pads for bearing pad method

Block type	Length (cm)	Width (cm)	Thickness (cm)
B1	110	110	40
B2	100	100	35
B3	90	90	35
B4	80	80	35

Table 4. Soil characteristics for parametric analysis

Soil type	γ (kN/m^3)	c (kPa)	ϕ (deg)	ψ (deg)	ν	E (MPa)
1	20	30	36	10	0.3	80
2	20	10	36	10	0.3	80

Table 5. Shotcrete wall characteristics for parametric analysis

Parameter	γ (kN/m^3)	E (GPa)	Thickness (cm)	ν
Value	25	21	15	0.2

Table 6. Soldier pile and bearing pad characteristics for parametric analysis

Element	Dimensions	γ (kN/m^3)	E (GPa)	ν	I (cm^4)
Pile	2IPE 300	78.5	210	0.3	1.67e4
Bearing pad	1×1×0.5 (m)	25	21	0.2	1.042e6

The overall geometry was created to conform to the two case studies, the characteristics of the materials were identified in the property module and all materials (soil, pile or bearing pad, shotcrete wall and anchors) were collected. For the soldier pile method, the piles were activated first. Next, the first level of soil was removed as the lagging between the soldier piles was activated and the anchors were prestressed. In the concrete bearing pad method, activation of the pads and shotcrete between the pads was done at the same time. After this, the appropriate mesh was selected for each part and the model was analyzed.

The mesh structure differed for each element in Abaqus. The soil was modeled using 3D structured hex mesh with high congestion near the supported wall. The shotcrete wall was modeled using quad-free mesh and its approximate element size was selected to be 0.25 m for both the soldier pile and concrete bearing pad methods. The anchors and piles were modeled as trusses and beams with element sizes of 1 and 0.5 m, respectively.

PARAMETRIC ANALYSIS

Limit equilibrium analysis was used to design the anchored walls. The FHWA anchorage manual (Sabatini et al., 1999) recommends this method to apply a uniformly disturbed load equivalent to the total anchor force on the wall face at the anchor angle. The manual shows that if a factor of safety of 1.3 for shear strength is used the limit equilibrium method gives results which are similar to the apparent earth pressure method in C-phi coarse-grained soil. Therefore, the uniformly distributed load on the wall face, which is equal to the anchor prestress force in the unit area of the wall, was determined in a way that all models featured the same factor of safety of 1.3.

The anchor prestress force in the unit area of the wall depends on the soil type and surcharge amount. Table 7 presents the prestress force in the unit area of the wall facing for two soil types and surcharges. These values were determined with a target safety factor of 1.3 and the anchor prestress force was calculated as:

$$T = t \times S_H \times S_V \quad (1)$$

where t: is the anchor prestress force in the unit area of the wall and S_H and S_V: are the horizontal and vertical spacing of the anchors, respectively.

The prestressing forces of all anchors in each model were held constant. The reinforcing elements were assumed to be multi-strands in which each strand had a diameter of 140 mm^2 and an allowable design load of 156 kN. The number of strands was determined according to the anchor prestress force and the allowable design load of the strand

The anchored wall model spaced the two anchor columns a value of S_H apart and $S_H/2$ from the boundaries. Horizontal and vertical spacing of the anchors were equal ($S_V = S_H$) and were spaced $S_V/2$ from the top and bottom of the wall. These details are demonstrated in Figures 7a and 7b. The embedment depth of the piles was assumed to be 25% of the excavation height. The wall height was assumed to be 24 m in all models. As shown in Figure 7, the number of anchors for the 2 and 3 m anchor spacing were 12 and 8, respectively.

The inclination of the anchors to the horizon was assumed to be 10°. The unbonded length of the anchors decreased linearly by depth and sufficiently extended behind the critical slip surface in accordance with FHWA recommendations. Figure 8 shown a section of the anchored wall for the non-surcharge, 3 m spacing model with soil type 1. The anchor specifications were shown precisely and the soldier pile and concrete bearing pad locations are displayed.

All assumptions were similar for both the soldier pile and bearing pad methods. The final 3D images constructed for this model are shown in Figures 9a and 9b for the concrete bearing pad and soldier pile systems, respectively. Figures 10 and 11 show the finite element mesh and horizontal deformation contours for the bearing pad and soldier pile methods, respectively.

RESULTS AND DISCUSSION

The results are presented in two sections for the horizontal deformation of the wall and bending moment of the shotcrete. The displacement was plotted along three lines in the wall facing as shown in Figure 7. Line 1 is located along the height of wall. Line 2 is a horizontal line passing through the row of anchors at a depth of 11 m for anchor spacing of 2 m and at a depth of 10.5 m for anchor spacing of 3 m. Line 3 is located at the middle of the wall height at a depth of 12 m for both models and passes through the shotcrete facing between the anchors.

Horizontal Deformation of Anchored Walls

Figure 12 plots the horizontal deformation of the anchored walls along line 1 for eight cases. For each case, the deformation of the soldier pile and bearing pad facings were compared. As shown, the deformation of the anchored wall with bearing pads was considerably larger than for that of the soldier pile, especially at the top. The horizontal displacement at the top of the soldier pile anchored wall was about half of that of the wall with concrete bearing pads. Increasing the anchor spacing and decreasing soil cohesion intensified the soldier pile anchored wall deformation sharply.

Table 7. Calculated anchor prestress force in the unit area of wall facing (kPa)

		Soil type	
		1	**2**
Surcharge (kPa)	0	65	85
	50	85	105

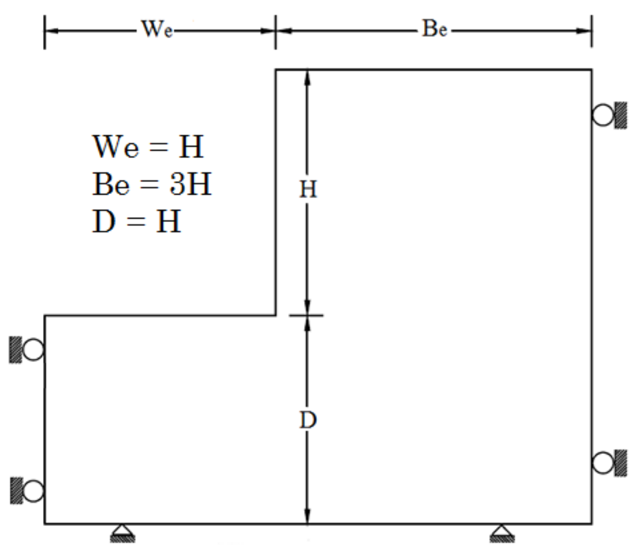

Fig. 6. Boundary conditions and model dimensions

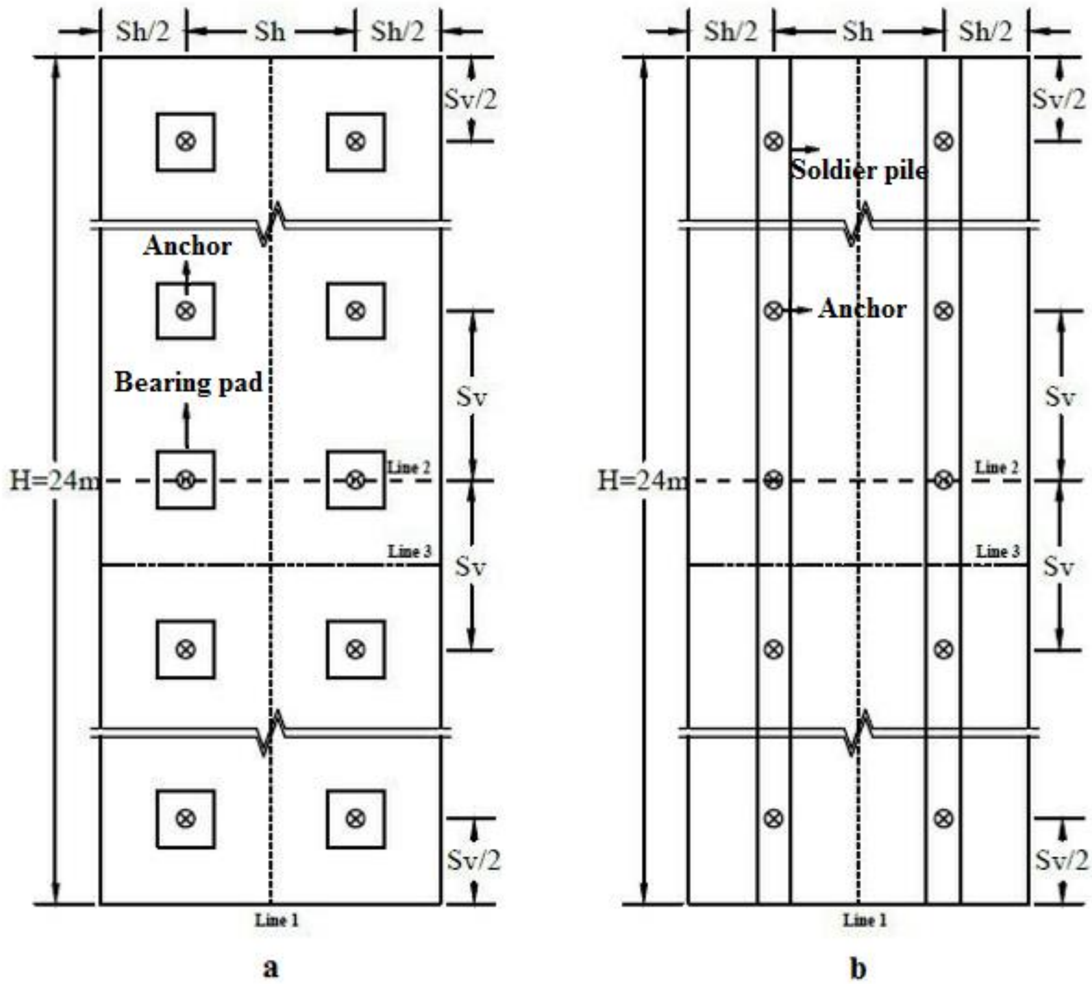

Fig. 7. Schematic of anchored wall: a) bearing pad, b) soldier pile

Figures 12c, 12d, 12g and 12h show that the surcharge increased the horizontal displacement at the crest of the wall in the bearing pad method to about twice that of the soldier pile method. Furthermore, the surcharge caused the maximum horizontal displacement in the bearing pad method at the crest of the wall, while the surcharge-induced deformation for the soldier pile method is nearly uniform.

Figures 12e to 13h show that the maximum horizontal displacement in soil type 2 was about twice that of soil type 1. This result is similar to the results of Hong et al. (2003). For soil type 1, the difference between the deformations at the lower depths was minimal. However, as the soil cohesion decreased, the deformation for the bearing pad method increased considerably at lower depths. For soil type 2, the difference in horizontal displacement between these two facing designs was greater than for soil type 1.

Horizontal displacement along lines 2 and 3 are plotted in Figure 13 to show the wall deformation on the horizontal plane. As shown, for the soldier pile method, horizontal displacement at the site of the soldier piles was somewhat less than for the shotcrete. However, relatively uniform deformation occurred using the bearing pad facing, which probably relates to the flexibility of the facing elements in this method. An increase in the anchor spacing caused the deformation in the soldier pile method to become more uniform, as shown in Figures 13c and 13d.

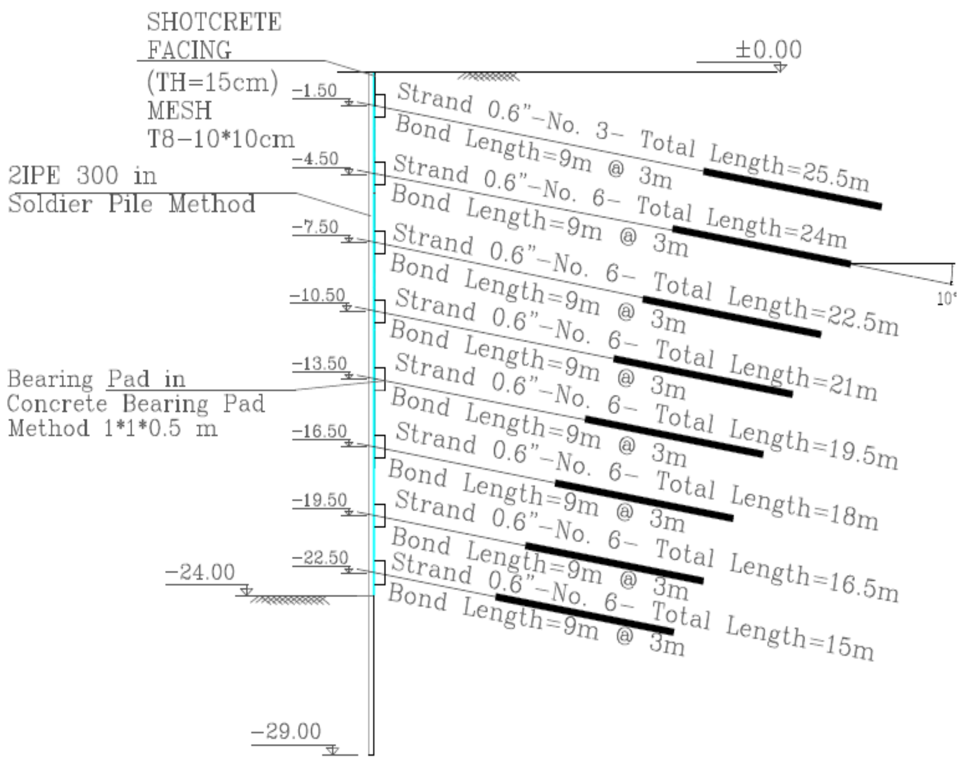

Fig. 8. Anchoring system for both excavation supporting methods for one of the parametric analysis models

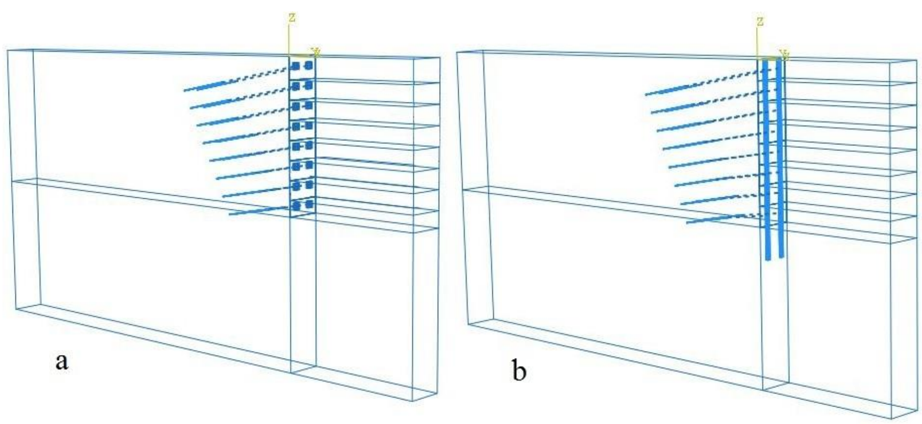

Fig. 9. Final 3D constructed whole model in Abaqus: a) bearing pad, b) soldier pile

Bending Moment of Shotcrete

The soil pressure produces the bending moment and internal forces on the wall facing. The distribution of soil pressure on the horizontal plane along the wall height and width is affected by the rigidity of wall facing elements caused by arching. The stiffness of the soldier piles and bearing pads is much greater than that of the lagging elements (which is shotcrete in this case). A great portion of the soil pressure is attracted by the soldier piles and bearing pads and just negligible portion of pressure is exerted on the shotcrete layer, as reported by Vermeer et al. (2001).

The bending moment induced in the shotcrete is plotted along line 1 in Figure 14. Figure 14a shows that the maximum bending moment was 7 kN.m/m for bearing pads and 5 kN.m/m for soldier piles. This suggests an increase in the arching effect in both cases. However, its effect in the soldier pile method was more pronounced than in the bearing pad method because of the difference in the stiffness of the facing elements.

The spacing of anchors had a great impact on the bending moment of the laggings. As shown in Figure 14b, the bending moments increased significantly to maximum values of 28 and 15 kN.m/m for the soldier piles and bearing pads, respectively. The maximum bending moment occurred at a height of 22.5 m, which was the site of the last anchor.

Rashidi and Shahir (2017) reported that the surcharge considerably increases the soldier pile bending moment; however, its impact on the shotcrete bending moment is not significant. Comparison of Figures 14a and 14b with Figures 14c and 14d confirms this for the both soldier pile and bearing pad methods. It appears that the greatest share of the soil pressure due to surcharge was carried by the soldier piles and bearing pads. This behavior is similar to the arching effect in the vertical direction.

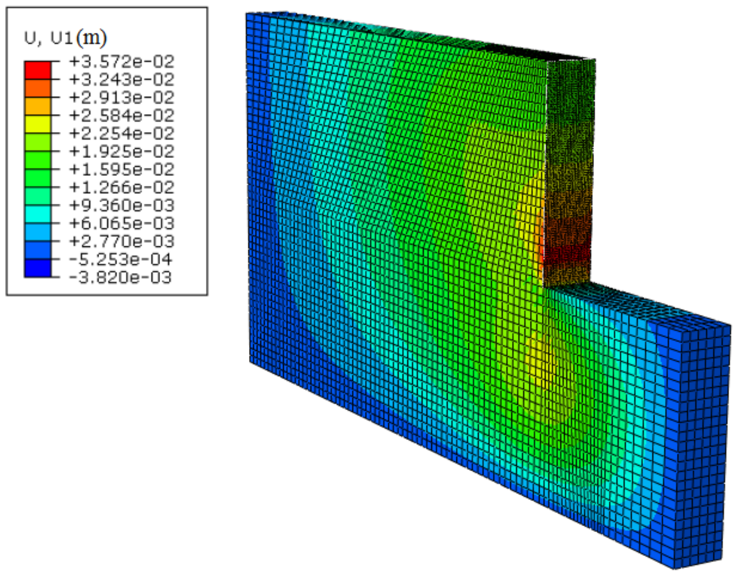

Fig. 10. Contours of horizontal displacement in bearing pad method

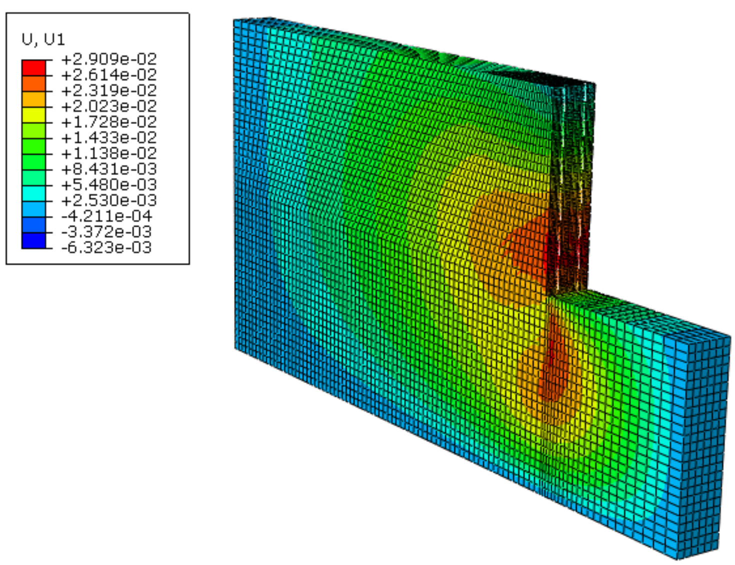

Fig. 11. Contours of horizontal displacement in soldier pile method

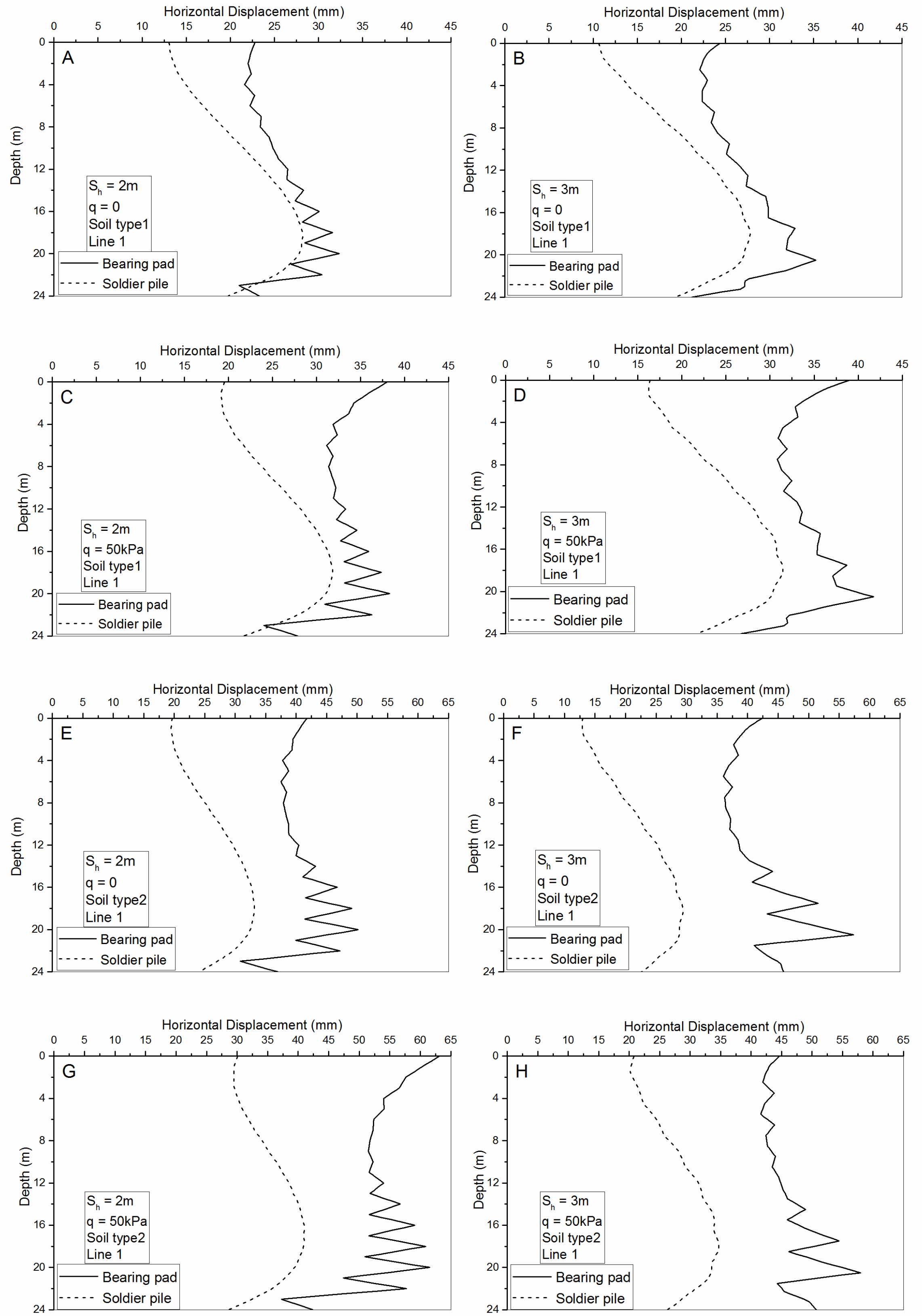

Fig. 12. Comparison of horizontal displacement along the wall height for two facing designs

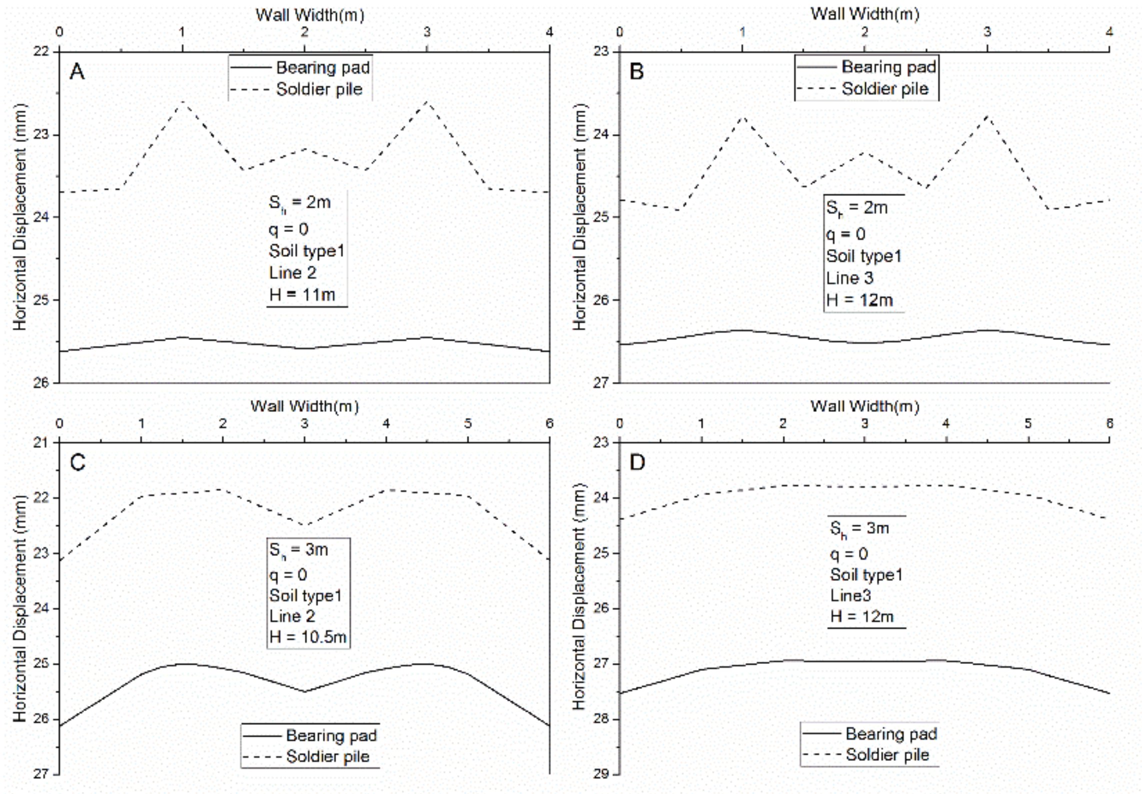

Fig. 13. Comparison of horizontal displacement along lines 2 and 3 for two facing designs

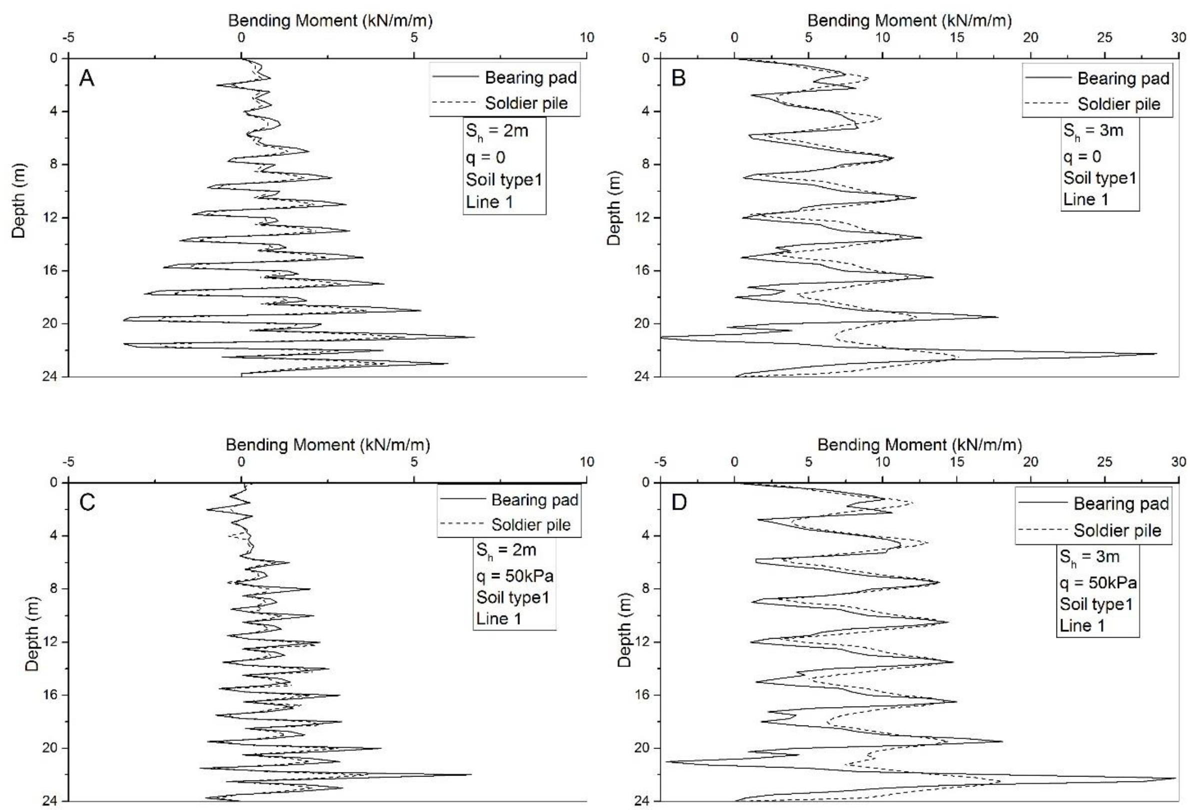

Fig. 14. Comparison of bending moment of shotcrete layer along wall height for two facing designs

The bending moment is plotted along lines 2 and 3 in Figure 15. Figure 15a shows two jumps in the bending moment at the sites of the anchors. These values are considerably more than the bending moment of the shotcrete. In line 3, the number of jumps is much less than for line 2 and the maximum bending moment for the soldier piles and bearing pads are nearly the same as in Figure 15b. An increase in anchor spacing to 3 m increased the bending moment in the anchor location up to three times that of an anchor spacing of 2 m, as shown in Figure 15c.

The bending moment was also evaluated for different geometries and physical characteristics of the facing designs. Figure 16a shows the bending moments for the bearing pads having sizes of 75, 100 and 125 cm. The bending moments at line 1 for the larger pads (1.25×1.25×0.5 m) was less than for the smallest pads (0.75×0.75×0.5 m) because the larger facing element carried a greater share of the soil pressure. The shotcreted surface carried a smaller share of the soil pressure and less pressure and bending moment occurred in line 1. This trend also can be observed in Figure 16b. Soldier piles with 3 profiles (2IPE 240, 2IPE 300 and 2IPE 360) were selected and the bending moment induced in line 1 in soldier pile with 2IPE 240 was 20% greater than that of 2IPE 360. Comparison of the bending moment for the bearing pads and soldier piles shows that using the latter method, even for the small size (2IPE 240) resulted in more stability than the concrete bearing pad method with large dimensions (1.25×1.25×0.5 m).

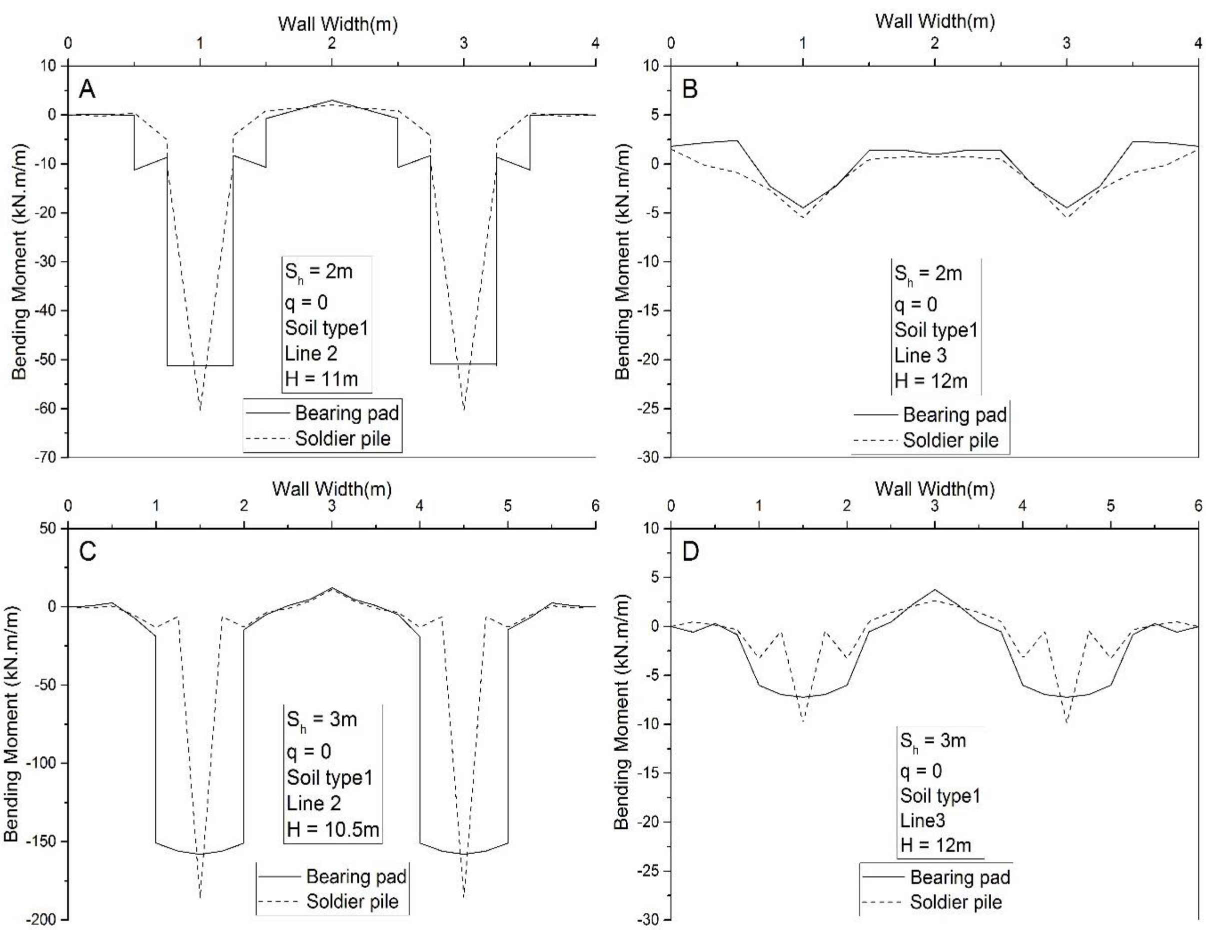

Fig. 15. Comparison of bending moment of shotcrete layer along lines 2 and 3 for two facing designs

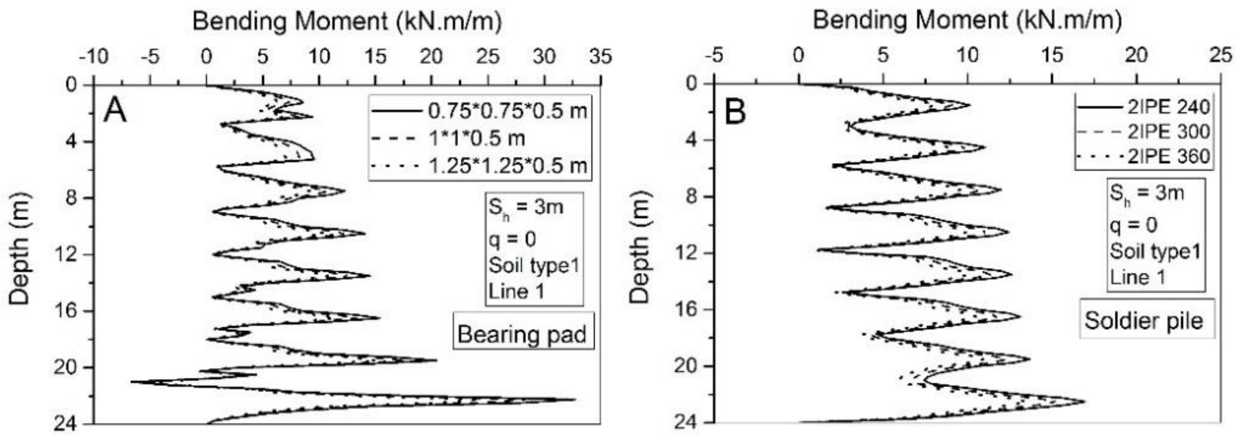

Fig. 16. Comparison of bending moment with different facing sizes along line 1 for two facing designs

CONCLUSIONS

The performance of a wall anchored either with soldier pile or concrete bearing pad facing designs were investigated using 3D finite element analysis. Based on results obtained in this study the main conclusions derived in relation to considered facing designs/geometries are as follows:

1. Horizontal displacement in the anchored soldier pile method was considerably less than in the bearing pad method, especially at the top of the wall. The horizontal displacement at the crest of the anchored soldier pile wall was about half of that of the wall with concrete bearing pads because of the embedment depth, rigidity and continuity of the soldier piles.

2. The presence of surcharge at top of the anchored wall increased horizontal displacement in the bearing pad method over that of the soldier pile method. An increase in the surcharge significantly increased the deflection at the wall crest in the bearing pad method and maximum horizontal displacement occurred at the wall crest. However, the surcharge-induced deformation along the soldier pile is nearly uniform. The difference between the deflections in these two systems was greater for c-phi soil with low cohesion than for soil with more cohesion. Therefore, the concrete bearing pad method is an appropriate alternative for c-phi soil with considerable cohesion where no sensitive structures are located at the top of the excavation pit.

3. In both methods, most of the soil pressure was attracted by the soldier piles or bearing pads because of the arching effect and only a small amount of pressure was exerted on the shotcrete layer. However, the effect of arching in the anchored soldier pile wall was considerably greater than for the bearing pad method and the bending moment of the shotcrete layer for the bearing pad method was 1.5 to 2 times greater than that of the soldier pile method, especially at the lower depths of the wall. Generally, arching and differences in the analysis methods indicate that deep excavation using soldier piles will sharply reduce displacement because it attracts more pressure exerted behind the wall due to its continuity and rigidity. The concrete bearing pad method is an appropriate approach when there is no considerable surcharge in the vicinity of the anchored wall.

It should be noted that these results are limited to the conditions presented in this study and their application to different conditions requires further investigation.

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