Abstract. Construction of underground structures may impose a hazardous effect on adjacent buildings due to ground movements caused by the construction of side support system and excavation. Most studies focus on predicting the soil movements due to excavation, however, limited research is performed to predict the effect of the construction process of the side support walls. Therefore, a 3D numerical model is developed to better understand the influence of installing diaphragm walls in sandy soils. A verification model has been used to validate the modeling of diaphragm wall construction sequence and its outputs using field data measurements of a selected case study. The diaphragm wall construction process is simulated using the WIM and WIP methods. This study has proved that WIM method is capable of simulating the construction stages and capturing the changes in soil stresses and displacements. Moreover, the results show that modeling diaphragm wall installation as a plane strain problem leads to the overestimation of the soil displacements. In addition, the effect of related parameters including; panel length, panel width, soil relative density, and moisture condition have been studied. The anticipated soil stresses and displacements during the construction process of diaphragm walls are presented. Finally, the effect of the selected modeling method (WIP or WIM) on the anticipated displacements during the following excavation stage is highlighted.

Keywords: construction stages, diaphragm wall, PLAXIS 3D, sand, WIP, WIM.

1. Introduction

A diaphragm wall is used as a part of side support systems for underground structures such as metro stations, basements, and underpasses. A diaphragm wall is a reinforced concrete structure constructed in-situ panel by panel. The wall can reach depths down to 50 m. The panel length ranges between 2.5 m and 7.0 m. The typical construction sequence of each panel includes three stages: (1) trench excavation under bentonite slurry support, (2), wet concrete injection, and (3) concrete hardening (Clear & Harrison, 1985).

The common practice is to design the side support system without considering the effect of the construction process of the walls. The construction sequence is assumed to have no effect on soil stresses or ground movements. However, previous studies indicate that diaphragm wall construction induces significant changes in stresses and movements of the surrounding soil (Burlon et al., 2013; Poh & Wong, 1998; Symons & Carder, 1993). Several case histories are reported in which the construction of a deep diaphragm wall resulted in the collapse of the constructed panels and settlement values up to 60 mm (Cowland & Thorley, 1984).

Numerical modeling is capable of simulating the diaphragm wall using two methods; WIP (wished in place) and WIM (wished in model). In the WIP method, the diaphragm wall is modeled as a plate element. The sequence of diaphragm wall installation is not considered and no changes in soil stresses or ground movements are assumed (De Moor & Stevenson, 1996). Conversely, in the WIM method, the model is developed to consider the three construction stages of each panel. The first stage involves removing soil elements constituting the ground to be excavated (the trench) and simultaneously applying the hydrostatic bentonite pressure along the sides and bottom of the trench. For the second stage, the hydrostatic wet concrete pressure should replace the hydrostatic bentonite pressure. Ling et al. (1994) found, based on field measurements, that the full hydrostatic wet concrete pressure is applied down till a certain depth (the critical depth) after which the wet concrete pressure increases with depth at a rate governed by the unit weight of the bentonite (bi-linear pressure). The critical depth \( h_{cr} \) is observed to be located approximately at one third of the wall depth. According to Ling et al. (1994), the wet concrete behaves as a heavy fluid because the solid particles (aggregates and cement grains) are suspended in the water. As the hydration process takes place, reduction in pore pressure is observed. The hydration mechanism results in the gradual transfer of the load from being a pore pressure into a solid weight. Consequently, the vertical stresses increase and the horizontal stresses acting on the trench sides are reduced. For the third stage, the trench is replaced by an elastic concrete material and the bi-linear...
In this paper, the effect of diaphragm wall installation in sandy soils is evaluated via a 3D numerical model. The paper consists of four parts; the first part validates the diaphragm wall numerical model and its outputs using a case study. The WIP and WIM methods are used to consider the effect of the construction process and the results of both methods are compared to field data measurements. The second part discusses the effect of the construction process on the ground movement and the soil stresses. The third part provides a parametric study to evaluate the effect of related parameters: panel length, panel width, soil relative density, and moisture condition. Finally, the effect of the selected modeling method (WIP or WIM) on the anticipated displacements during the subsequent excavation is highlighted.

2. Case Study and Model Verification

The selected case study is a diaphragm wall constructed to be a part of the basements of a multi-story building in Dokki, Giza, Egypt (El-Sayed & Abdel-Rahman, 2002; Abdel-Rahman & El-Sayed, 2009). The soil stratification at this site is shown in Fig. 1 as well as the results of the SPT tests with depth. The ground water table was monitored at a depth of 2.0 m below the ground surface.

As shown in Fig. 2, the excavation site is 24.6 m times 35.7 m and is surrounded by five existing buildings. Buildings A, B, and C are 12 to 14 stories high and are founded on piles with lengths ranging between 14.0 m and 16.0 m and are located at distances of 1.8 m, 3.15 m and 7.15 m away from the excavated site, respectively. Buildings D and E are 5 and 2 story buildings, respectively. Both buildings are founded on shallow foundations at 2 m depth and are located at a distance of 3.2 m from the excavation site.

The excavation depth was 10.8 m below the ground surface, therefore, a side support system was needed. This supporting system was composed of a diaphragm wall supported by two rows of anchors and struts. The diaphragm wall width \((w)\) is 0.6 m, depth \((D)\) is 21 m, and the panels’ lengths \((L)\) range between 2.70 m and 6.72 m. The total number of panels is 20.

An optical surveying program was adopted to monitor the settlement of adjacent buildings due to the construction of the side support system and excavation. As shown in Fig. 2, thirty one settlement points (SP-1 to SP-31) were fixed on selected columns at the location of adjacent buildings.

Figure 3 shows the measured total settlements due to construction of the diaphragm wall and excavation down till the designated depth. Despite being founded on piles, considerable settlement values were observed at Buildings A, B, and C. For these buildings, the total settlement ranged between 0.0 mm and 12.5 mm (Fig. 3a). The maximum value occurred at SP-19 (Building B). This point was located at a distance of 3.15 m from the diaphragm wall. None to negligible settlement values were detected at SP-1, SP-2, SP-28, SP-29, and SP-30 which were located at distances of 18 m to 40 m away from the corners of the site. Due to construction of the diaphragm walls, the measured settlements ranged between 0.0 mm and 8.6 mm which implied that at least 44 % of the total settlement values took place before excavation. The diaphragm wall is 21 m deep whilst the piles are 14 m to 16 m deep which may explain why most of the settlement took place during the execution of the diaphragm wall. For Buildings D and E (on shallow foundations), the total settlement ranged between 1.2 mm and 17.8 mm. The maximum value occurred at SP-23 (Building E). This point was located at a distance of 3.2 m from the diaphragm wall. Insignificant settlement value was detected at SP-24, which is located at a distance of about 14 m from the corner of the site. Due to construction of the diaphragm walls, the measured settlements ranged between 0.4 mm and 8.6 mm, which implied that 14 % to 50 % of the total settlement values took place (Fig. 3b).

These measurements confirm the importance of estimating the displacements induced during the construction of the diaphragm walls while studying the effect of installing a side support system and excavation. Therefore, verification models are constructed via the 3D finite element program (PLAXIS©). The soil mass is simulated as a continuum composed of 10-node tetrahedron volume elements. For all soil layers, the Hardening Soil Model (HSM) is applied. Table 1 presents the assigned parameters for each soil layer. For buildings on deep foundation, due to lack of data, a surcharge load of 150 kN/m² is simulated at a depth of 16 m below the ground surface. Buildings founded on shallow foundations are simulated as a 40 kN/m² surcharge load at 2 m below the ground surface as shown in Fig. 4.

Two models have been studied to determine the effect of the diaphragm wall simulation technique. The dia-
The phragm wall is simulated using either WIP method or WIM method. In the WIP method, the diaphragm wall is simulated as a plate element. The plate element is formed based on Mindlin’s plate theory to simulate a thin two-dimensional structure with flexural rigidity and a normal stiffness. Herein, the plate is modeled as an elastic isotropic concrete material and its properties are: unit weight ($\gamma_c = 24\, \text{kN}/\text{m}^3$), Young’s modulus ($E = 2.6 \times 10^7\, \text{kN}/\text{m}^2$) and Poisson’s ratio ($\nu = 0.20$). The model considers the sequence of construction in three stages. At the first stage, the soil initial stresses are generated using the $K_p$ procedure. At
the second stage, the surcharge loads of adjacent buildings are activated. At the third stage, the diaphragm wall (plate element) is activated.

In the WIM method, the diaphragm wall panels are simulated as volume elements. The model is developed to consider the construction stages of each panel. First, the soil initial stresses are generated using the $K_o$ procedure. Second, the surcharge loads of adjacent buildings are activated. Third, the excavation under slurry support is simulated by deactivating soil elements inside the trench. Simultaneously, the hydrostatic bentonite pressure with a unit weight ($\gamma_{b}$) of $10.4 \text{ kN/m}^3$ is applied along the trench sides and bottom (Fig. 5a). Fourth, wet concrete is poured into the trench replacing the bentonite slurry. Thus, the bentonite hydrostatic pressure is replaced by bi-linear pressure (Fig 5b). A full concrete pressure with a unit weight ($\gamma_{c}$) of $24 \text{ kN/m}^3$ is applied down to a critical depth ($h_{c}$) below which the pressure increases along depth with the bentonite pressure. Fifth, concrete hardens, hence, the bi-linear pressure is removed and the volume elements inside the trench are activated as elastic isotropic concrete volumes with unit weight ($\gamma_{c}$) equal to $24 \text{ kN/m}^3$, $E = 2.6 \times 10^7 \text{ kN/m}^2$ and Poisson’s ratio $\nu = 0.20$ as shown in Fig. 5c. For each panel, the last three stages (third to fifth) are repeated according to the construction schedule executed on site.

Figure 6 depicts that the WIP method underestimates the settlement values at the selected points by 63 % to 85 %. The calculated settlements range between 0.4 mm and 1.6 mm for buildings A, B, and C and between 0.4 mm and 1.2 mm for buildings D and E. On the other hand, the WIM method presents a better prediction. The calculated settlements range between 0.7 mm and 10.6 mm for buildings A, B, and C and between 2.1 mm and 10.1 mm for buildings D and E. Figure 7 presents the horizontal displacement contours using both the WIP and WIM methods. It can be noted that negligible values are acquired using the WIP method, however, higher values are attained using the WIM method. Furthermore, the WIP method shows an almost uniform distribution of displacements all over the site. The WIM method shows a more realistic trend where higher displacement values occur along the diaphragm panels.

### Table 1: Soil properties for the case study (Hardening Soil Model).

| Layer          | Fill | Silty sand | Fine sand | Graded sand |
|----------------|------|------------|-----------|-------------|
| Thickness (m)  | 2    | 3          | 6         | 14          |
| Saturated unit weight, $\gamma$ (kN/m$^3$) | 17   | 18         | 19        | 20          |
| Secant stiffness in standard drained triaxial test, $E_{sc}$ (MPa) | 6    | 16         | 25        | 25          |
| Tangent stiffness for primary oedometer loading, $E_{to}$ (MPa) | 6    | 16         | 25        | 25          |
| Unloading/reloading stiffness, $E_{ur}$ (MPa) | 18   | 48         | 75        | 75          |
| Power for stress-level dependency of stiffness, $m$ | 0.5  | 0.5        | 0.5       | 0.5         |
| Effective cohesion, $c'$ (kPa) | 0.01 | 0.01       | 0.01      | 0.01        |
| Effective angle of shearing resistance, $\phi$ (deg) | 29   | 31         | 33.5      | 36          |
| Dilatancy angle, $\psi$ (deg) | 0    | 1          | 3.5       | 6           |
| Poisson’s ratio, $v$ | 0.2  | 0.3        | 0.3       | 0.3         |
| Earth pressure coefficient at rest, $K_o$ | 0.515 | 0.485      | 0.448     | 0.412       |

Figure 4 - PLAXIS three dimensional model configuration for case study.

Figure 5 - Construction stages of diaphragm wall.

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addition, concentration of displacements is noticed around the panels especially at the right and bottom sides of the site. By reviewing the panels’ lengths and construction schedule, it is found that these higher values occurred where longer successive panels were executed. The model constructed using the WIP method also shows that the construction of diaphragm walls generates insignificant values of straining actions.

Based on the above results, the WIM method is adopted for the parametric study. The current study focuses on investigating the effect of construction of the diaphragm wall on the generated soil stresses and displacements.

3. Results and Discussion

In order to investigate the effect of diaphragm wall construction on ground movement and soil stresses, a 3D model of a diaphragm wall with a depth ($D$) of 17 m is simulated (Fig. 8a and b). The diaphragm wall is composed of three panels as presented in Fig. 8c. Panel 1 is a primary panel, panels 2 and 3 are secondary panels. Each panel is simulated in stages according to the construction sequence mentioned in the previous section, with a total number of nine stages. Construction of panel 1 is simulated in stages 1 to 3, followed by panel 2 (stages 4 to 6) and panel 3 (stages 7 to 9).

The effect of parameters related to diaphragm wall geometry including panel length ($L = 3$ m, 6 m, and infinity) and panel width ($w = 0.6$ m, 0.9 m, 1.2 m) have been stud-
ied. In addition, the effect of soil relative density is investigated. The diaphragm wall is constructed in loose, medium dense, and dense sand with properties shown in Table 2. Next, a comparison between constructing the diaphragm wall in dry sand vs. saturated sand is conducted.

The model dimensions (80 m times 180 m) are selected such that the model borders have no influence neither on induced settlements, nor on stresses. The bottom of the geometry is fixed and the upper boundaries are fully free to move. For the sides, the displacements normal to the boundary are fixed and the tangential displacements are kept free.

The construction process of the diaphragm wall panels has a major impact on the stress distribution behind the wall. For a diaphragm wall with \( w = 0.6 \text{ m} \) and \( L = 3.0 \text{ m} \) constructed in dry medium dense sand, Fig. 9 depicts the change of horizontal stresses behind the wall along the center of panel 1. In order to understand the trend in which the stresses change, three additional lines are plotted: initial horizontal stress, bentonite pressure, and concrete pressure. For medium dense sand:

Initial horizontal stress = \( \gamma K_o Z \)

\[= 19 \times 0.426 Z = 8.09 \text{ ZkN/m}^3/\text{m} \] (1)

Bentonite pressure = \( \gamma_b Z \)

\[= 10.4 \text{ ZkN/m}^3/\text{m} \] (2)

Concrete pressure = \( \gamma_{wet} Z \)

\[= 23 \text{ ZkN/m}^3/\text{m} \] (3)

where \( g \) is the soil unit weight, \( K_o \) is the earth pressure coefficient at rest, \( Z \) is the depth below ground surface, \( \gamma_b \) is the bentonite unit weight, and \( \gamma_{wet} \) is the wet concrete unit weight. The current study is performed in sandy soils, hence, the initial horizontal stress is lower than the subsequent applied pressures (bentonite and concrete). Thus, during the construction of panel 1, the horizontal stresses generally increase (stages 1 to 3, Fig. 9a). This trend is the opposite of the results observed by Ng & Yan (1999). In clayey soils, the initial horizontal stress is larger than the subsequent applied pressures (bentonite and concrete). Therefore, horizontal stresses are reduced during the construction of the panel.

Underneath the wall, the horizontal stress values fall below their initial state \((K_o \text{ condition})\) due to the restraint provided by the underlying soil. This trend extends to a distance of about 5 m \((0.3D)\) underneath the wall bottom, which complies with the results of Conti et al. (2012).

During the construction of panel 1, the horizontal stresses increase during the stage of trench excavation under bentonite support (stage 1, Fig. 9a). The injection of wet concrete (stage 2, Fig. 9a) leads to a further increase in the horizontal stresses, particularly in the upper third of the retaining wall. The horizontal stresses follow the bi-linear pressure applied during stage 2. However, negligible change is detected during concrete hardening (stage 3, Fig. 9a). The same trend is found when the construction advances from stage 5 to 6 and from stage 8 to 9, as shown in Fig 9b. The construction of panels 2 (stages 4 to 6) and panel 3 (stages 7 to 9) causes a drastic decrease in horizontal stresses behind panel 1 (Fig. 9b). At stage 9, the stresses are less than the initial stresses. This could be further examined using Fig. 10, which shows the total horizontal stress at a distance of 0.1 m behind the panels at a depth of 8.5 m below the ground surface. A horizontal section is plotted with distance measured from the edge of panel 2 normalized by the length of panel \((\gamma/L)\). It is found that the construction of

![Figure 9 - Horizontal stress distribution with depth across the center of panel 1: (a) construction of panel 1 and (b) construction of panels 2 and 3 (medium dense dry sand, \( w = 0.6 \text{ m}, L = 3.0 \text{ m}, D = 17 \text{ m} \).]
a certain panel causes a maximum increase in the horizontal stresses behind the center of this panel, then, stresses decrease gradually toward the edge of this panel and adjacent panels. This trend is attributed to the lateral stress transfer which is also observed by Conti et al. (2012).

Figure 11 inspects the effect of diaphragm wall installation on the soil movement behind the wall along the center of panel 1. During the stages of bentonite and wet concrete injections, the soil horizontal stresses increase, consequently, the soil horizontal displacements increase (Fig. 11a). However, no further horizontal displacements are experienced behind this panel once concrete hardens. The construction of the adjacent panels does not cause any additional displacements to the studied panel. In addition, the maximum horizontal displacement values occur approximately at half the panel depth (0.5D) below the ground surface, which matches the results obtained by Conti et al. (2012). Moreover, the effect of diaphragm wall installation almost diminishes at 5 m below the wall toe, i.e., approximately one third of the wall depth (0.3D), which is verified by Ng & Yan (1998).

The soil horizontal displacements are accompanied by settlements near the diaphragm wall. As shown in Fig. 11b, settlement values occur directly behind the wall and become marginal after a distance of 17 m behind the wall which is equivalent to 1D, which matches the results of Powrie & Kantartzzi (1996) and Ng & Yan (1998). The settlements increase as the wall installation proceeds. Behind a given panel, about 75 % of the total expected settlement is developed by the end of construction of this panel while the remaining 25 % occurs during the construction of the two adjacent secondary panels.

3.1. Effect of diaphragm wall geometry

Figure 12 presents the effect of the panel length (L) and width (w) on the soil horizontal stresses and displacements. Panels with lengths of 3 m, 6 m and infinity and widths of 0.6 m, 0.9 m, and 1.2 m are selected. A panel length of infinity is attained via plane strain condition, which can be done using a 2D model (Fig. 13). The same procedure adopted for 3D modeling of the construction sequence of the diaphragm wall is used. In engineering practice, 2D modeling is usually adopted because 3D modeling is more complicated and time consuming.

Figure 12a shows that using longer panels leads to higher horizontal stresses behind panel 1, hence, larger val-
Horizontal displacements are expected. In Fig. 12b, the maximum horizontal displacements behind panel 1 are normalized to the maximum values obtained from the plane strain condition \( \frac{U_{\text{max}}}{U_{\text{max}}(X)} \). The results are plotted vs. the panel depth to length ratio \( \frac{D}{L} \). As the panel length increases (\( \frac{D}{L} \) reduces), the horizontal displacements increase gradually until reaching a maximum value at \( \frac{D}{L} = 0 \) (plane strain condition). The results presented herein show that modeling diaphragm wall installation as a plane strain problem leads to the over-prediction of the soil displacements and stresses during installation by 230% to 400%. This approach leads to unrealistic values of ground movement because it does not account for the arching effect (Ng & Yan, 1998). On the other hand, panel width has no effect on soil horizontal displacements during the construction of the diaphragm wall as shown in Fig. 12b.

**3.2. Effect of soil relative density and moisture condition**

The effect of soil relative density is investigated using sand with three different relative densities presenting loose, medium dense, and dense soil as proposed in Table 2. The angle of shearing resistance \( \phi \) is introduced in Fig. 14 as an indication of the soil relative density. As the soil becomes denser, the maximum soil displacements decrease. This is attributed to the fact that the increase in soil relative density is associated with an increase in soil stiffness \( E \) as shown in Table 2. The rate of decrease in displacement values declines as the relative density increases. At the end of construction of panel 3 (stage 9), the maximum vertical displacement (settlement) decreases from 22 mm for loose sand to 10 mm for medium dense sand (54.5% reduction), and to 5.5 mm for dense sand (31.3% reduction).

The diaphragm wall construction sequence is also simulated in saturated sand in order to inspect the effect of moisture on the performance of the wall. First, horizontal stresses are investigated as shown in Fig. 15a. The horizontal stresses decrease during bentonite injection (Stage 1), then increase again during the concrete injection and hardening (stages 2 & 3). This trend contradicts the results obtained from the dry sand case (Fig. 9a). This is attributed to the fact that the initial stresses, in saturated sand, are larger than bentonite pressure and lower than the wet concrete bi-linear pressure. For medium dense sand:

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**Figure 12** - Effect of panel dimensions on soil: (a) horizontal stresses, (b) horizontal displacements at stage 9 (at the center of panel 1 - medium dense dry sand, \( D = 17 \) m).

**Figure 13** - 2D Model (plane strain condition).
Initial horizontal stress = \( \gamma_{sul} K_c Z + \gamma_w Z \) = 9 \times 0.426 Z + 10 Z = 13.8 Z kN/m\(^3\)/m \( (4) \)

Bentonite pressure = \( \gamma_b Z \) = 10.4 Z kN/m\(^3\)/m \( (5) \)

Concrete pressure = \( \gamma_{wet} Z \) = 23.0 Z kN/m\(^3\)/m \( (6) \)

where \( \gamma_{sul} \) is the soil submerged unit weight, \( K_c \) is the earth pressure coefficient at rest, \( Z \) is the depth below ground surface, \( \gamma_w \) is the water unit weight, \( \gamma_b \) is the bentonite unit weight, and \( \gamma_{wet} \) is the wet concrete unit weight. This reduction in stresses during bentonite injection (Stage 1) causes the trench side to move in the reverse direction. Figure 15b shows the horizontal displacements along one side of the trench during stage 1 for dry sand and saturated sand. The positive values of the horizontal displacement indicate that the trench cross section increases (bulging). On the other hand, the negative values of the horizontal displacement indicate that the trench cross section decreases (necking). The maximum necking (saturated sand) or bulging (dry sand) occurs at a depth of 14.25 m (0.84 \( D \)). At stage 2 (Fig. 15c), the horizontal stresses increase again; accordingly, the direction of the horizontal displacement is reversed again and becomes positive for saturated sand. Meanwhile, further increase in the horizontal displacement is noticed at the same stage for dry sand. It is also noted that the depth at which the maximum horizontal displacement occurs is shifted upward. The maximum bulging occurs at depth of 4.5 m (0.26\( D \)) for saturated sand and at depth of 7.7 m (0.45\( D \)) for dry sand. Figure 15d shows the maximum horizontal displacement \( (U_{max}) \) and settlement \( (U_{set}) \) during construction (stages 1 to 9) for dry and saturated sand. Once the concrete hardens (stage 3), the horizontal displacements become constant and no further change is expected due to the following construction stages. On the other hand, the settlement values continue to increase after stage 3 at a lower rate.

### 4. Effect of the Modeling Technique on the Displacements During Subsequent Excavation

The effect of the modeling technique is investigated for a diaphragm wall with \( w = 0.6 \) m and \( L = 3.0 \) m constructed in dry medium dense sand. Two models are developed using WIP and WIM methods. For each model, the soil in front of the diaphragm wall is excavated to a depth of 8 m. Due to excavation, the WIP method overestimates the horizontal displacement of the diaphragm wall by around 50 % (Fig. 16a). The maximum horizontal displacement is about 26 mm using the WIM method and 40 mm using the WIP method. On the other hand, the estimated settlement values behind the wall using the WIP method are higher by 14 %. Nevertheless, the WIP method underestimates the settlement values due to diaphragm wall construction by 87 %; after excavation the effect of using this simplification has less impact on the results (Fig. 16b).

### 5. Conclusions

In this study, the finite element numerical model succeeded in simulating the complicated construction se-
The model output has been validated through a comparison with the field measurements of the settlement values recorded during execution of the side support system and excavation of a site in Dokki, Egypt. The diaphragm wall construction process can be simulated using the WIM and WIP methods. The WIP method is the conventional finite element method. The effect of diaphragm wall construction stages is not considered, therefore, no changes in soil stresses or movements are anticipated. On the other hand, this study has proved that the WIM method is capable of simulating the construction stages and capturing the changes in soil stresses and displacements. The construction sequence for each panel is simulated through three stages representing: (1) excavation under slurry support, (2) wet concrete injection, and (3) concrete hardening. The construction sequence is found to have a major impact on the stress distribution behind the wall. In dry sandy soils, an increase in horizontal stresses.

**Figure 15** - Effect of moisture condition on: (a) horizontal stresses (b) horizontal displacements at stage 1 (c) horizontal displacements at stage 2 and (d) max displacements for stages 1 to 9 (at the center of panel 1 - medium dense Sand, $w = 0.6$ m, $L = 3.0$ m, $D = 17$ m).
behind the primary panel is expected during the bentonite
and wet concrete injection stages. However, a drastic de-
cline in horizontal stresses is noticed during the construc-
tion of secondary panels. The installation of the diaphragm
wall causes an increase in soil movement. The maximum
horizontal displacement values occur approximately at
$0.5D$ below the ground surface. The horizontal displace-
ment along the primary panel take place only during the
bentonite and wet concrete injection stages. The construc-
tion of secondary panels does not cause any additional hori-
zontal displacement. However, behind the primary panel,
about 75% of the total expected vertical displacements are
developed by the end of construction of this panel, while
the remaining 25% occur during the construction of the
secondary panels.

In engineering practice, 2D modeling is used as a con-
ventional tool to determine the expected displacements.
However, the results presented herein show how that modeling
diaphragm wall installation as a plane strain problem leads
to the overestimation of the displacements. Subsequently,
over-designed side support systems are provided. Simu-
lating the actual panel length using a 3D model leads to
lower and more realistic values of stresses and displace-
ments. In addition, an increase in soil relative density leads
to a pronounced decrease in the induced displacement.
Moreover, in saturated sandy soils, initial horizontal
stresses are larger than bentonite pressure, therefore, neck-
ing of the trench section occurs. Finally, adopting different
techniques of modeling the construction sequence of dia-
phragm wall has a major impact on the estimated displace-
ment during the following excavation stage.

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