

The Deflection and Bending Moment of Existing Piles Influenced by Trenching Diaphragm Wall Panel(s)

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Abstract. Diaphragm walls are normally used for the deep excavations and underground construction inside the cities. The trenching process of diaphragm walls causes deformation in the ground surface. The construction of the diaphragm walls near deep foundations may not be avoided specially in the crowded cities. This research focuses on the deflection and bending moment of piled foundations near slurry trench. Since the trench is considered as a three dimensional problem, a three-dimensional numerical analysis was used in this research. It was conducted using a commercial analysis software known as FLAC 3D which is based on the finite different analysis method. Verification was made for the numerical analysis method using two case histories, one in Hong-Kong and another one in Giza, Egypt. The results from the numerical analysis were in quite a good contrast with the field data results. This means the trenching process could be numerically modeled with the proposed method and provides good results. The verified numerical analysis method was used to conduct a numerical parametric study that discusses the effect of a single or double panel(s) on the nearby piled foundation. The parametric study showed that the pile deflection and bending moment are affected by the panel length, pile diameter, pile location from the trench, groundwater level and the existence of a weak soil layer that could cause reduction in slurry pressure. The panel thickness did not show any influence on the pile bending moment or deflection. The decrease of the panel length and controlling the slurry level will reduce the influence of trenching on nearby piles. Lowering the groundwater level before trenching could probably reduce the influence on the pile as well.

Keywords: Diaphragm wall · Pile · Trenching · Deflection · Bending moment · Numerical analysis

1 Introduction

The soil deformation caused by the diaphragm wall trenching process was monitored by many researchers. Generally, they used the settlement points to measure the surface soil settlement and inclinometers to measure the horizontal soil displacement. Deep soil settlements could also have been measured in some cases. Monitoring during trenching

process was made for either tested panel or multiple panels. DiBiagio and Myrvoll (1972), Tsai et al. (2000) and Ng et al. (1999) had intensely monitored tested panel for research purposes, while Karlsrud (1983), Cowland and Thorley (1985), Hamza et al. (1999), Poh et al. (2001) and L'Amante et al. (2012) monitored the diaphragm wall installation of case histories in real projects. The monitoring results were varied according to many parameters such as panel dimensions, groundwater level and soil properties.

Trench panels were simulated using three-dimensional numerical analysis by Ng and Yan (1998), Gourvenec and Powrie (1999), Grandas-Tavera (2012) and Comodromos et al. (2013). These researchers compared the results from the numerical analysis with that from the field data. Their intention was to find out the ability of the trenching numerical analysis in modeling the trenching process. They found out that three-dimensional numerical analysis could be used in modeling the trenching problem.

Generally, the trenching process causes settlement and horizontal displacement for the ground surface which could probably affect the nearby existing deep foundations. However, a very limited research has been made regarding such effect. Davies and Henkel (1982), Abdel-Rahman and El-Sayed (2009) and Korff (2013) monitored the trenching process near existing piled foundation but they were not able to monitor the existing piles. Choy et al. (2007) studied the effect of the trenching process on a single pile using the centrifuge model test through conducting a limited parametric study. His study did not take into consideration the effect on pile group and the possible existence of groundwater.

In this research, the three-dimensional numerical analysis was used to simulate the trenching process of diaphragm walls adjacent to piled foundation for two different case histories. The results from the numerical analysis were compared to the field data results. Such simulation method was used to conduct a parametric study which discusses the different parameters that affect the existing piled foundation near diaphragm wall panel(s).

2 Numerical Analysis of Case Histories and Verification

The numerical modeling for different geotechnical engineering problems is considered to be an acceptable tool. However, the modeling method of the different types of such problems should be verified. The trenching process of slurry trench walls required a three-dimensional simulation with a special attention to the stages and simulation assumptions. This section describes the numerical modeling of two case histories using FLAC 3D. The results from modeling were compared to those from the field.

2.1 Case History 1 (Underground Station Near the Court of Justice in Hong Kong)

A diaphragm wall system was used as a part of the Charter underground metro station. This station was very close to the court of justice which was constructed in a timber piled foundation. The piles cross section area was equal to 0.0254 m^2 and it extended

to a level of 14 m beneath the ground surface. The building load was distributed over the beams that connect the piles. The details of the project are described by Davies and Henkel (1982). The numerical modeling and verification related to the diaphragm wall panels construction adjacent to the building are described in the following subsections.

2.1.1 Modeling and Construction Stages

The construction stages of the diaphragm wall panels were modeled as was described in Fig. 1. According to Stroud and Sweeney (1977) the soil was found to be consisted of five layers. The soil layers' depths and their properties are presented in Table 1. The soil was modeled using strain hardening softening soil model which is defined in FLAC 3D by conducting a relation between mobilized friction angle and plastic shear strain which can be calculated according to Byrne (2003) as:

$$\zeta_p = \frac{P_{ref}}{\beta G_{ref}^e} \times \frac{\sin \phi}{R_f} \left(\left(1 - \frac{\sin \phi_m}{\sin \phi} R_f \right)^{-1} - 1 \right) \quad (1)$$

where

- P_{ref} is the reference pressure,
- β is calibration factor,
- ϕ is the ultimate friction angle,
- ϕ_m is the mobilized friction angle,
- R_f is the failure ratio.

The elastic tangent shear modulus is calculated from the following equation:

$$G_{ref}^e = \frac{E_{ur}^{ref}}{2(1 + \nu_{ur})} \quad (2)$$

where

- E_{ur}^{ref} is the required strain to mobilize the limit friction angle,
- ν_{ur} is the undrained Poisson's ratio.

The relation between the plastic shear strain and mobilized friction angle based on the previous equations for the soil layers are presented in Fig. 2.

The trenching process of each panel was made by replacing the soil elements at the panel location with a hydrostatic pressure equivalent to the slurry pressure. The concreting process of the panel is made then by reactivation of the zones and changing its properties to concrete properties. Figure 3 shows the trenching process of a panel in different situations. The pile and beams connecting the piles were modeled using the beam and pile elements, respectively. These elements are described and discussed by Itasca (2012). The mesh model contains 157800 zones and its shape and dimension are presented in Fig. 4. The relative normal and shear displacement between the pile and

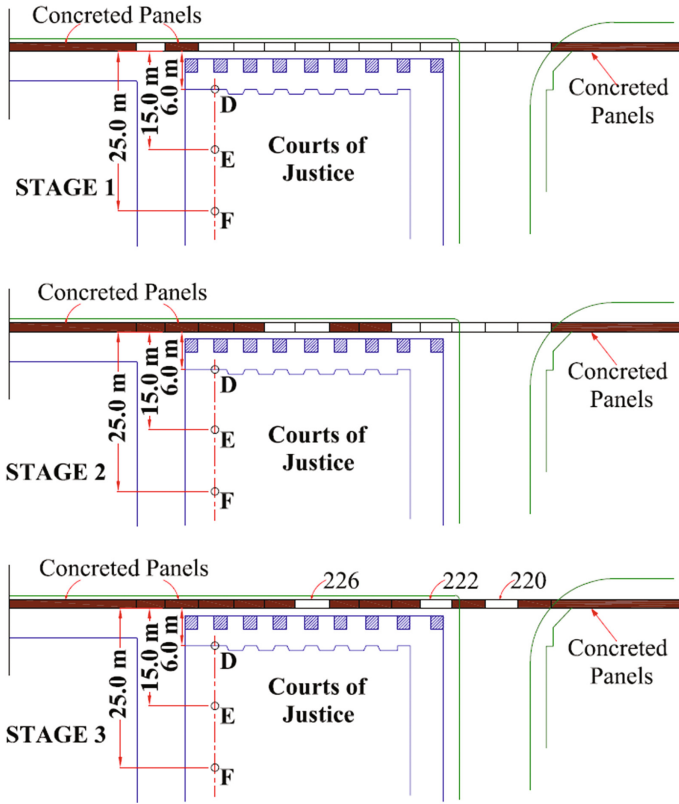


Fig. 1. Courts of justice construction stages and points of monitoring

Table 1. Soil properties (Courts of Justice, Hong Kong)

Soil layer	Bottom level (m)	SPT	γ (kN/m ³)	c'/cu (kN/m ²)	ϕ' (°)	E (MPa)
Fill	-1.0	5-30	17.0	0	27	10
Marine deposits	-2.6	5-30	17.0	5/35	15	10
Highly weathered Decomposed granite	-11.0	10-20	17.62	0	30	40
Decomposed granite	-31.75	>40	20.0	0	36	85
Granite	-	-	22.2	0	40	100

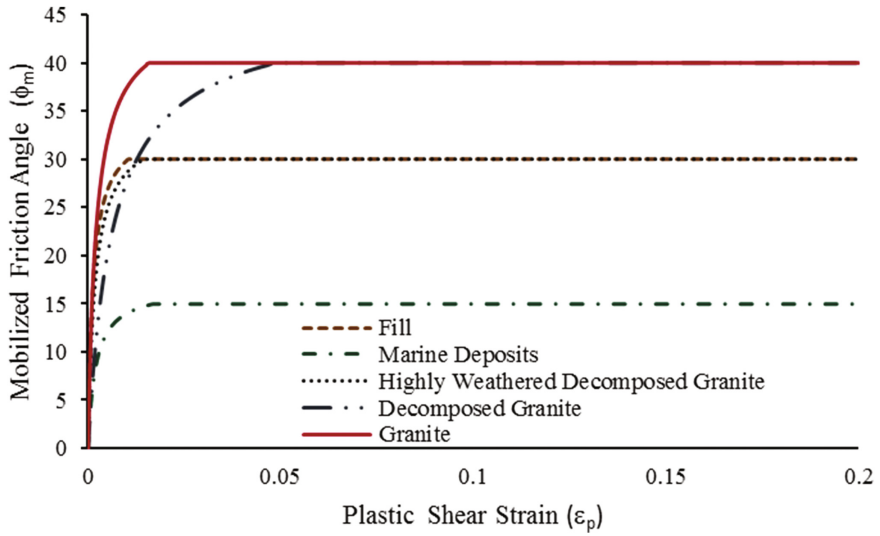


Fig. 2. Relation between mobilized friction angle and plastic shear strain for different soil layers

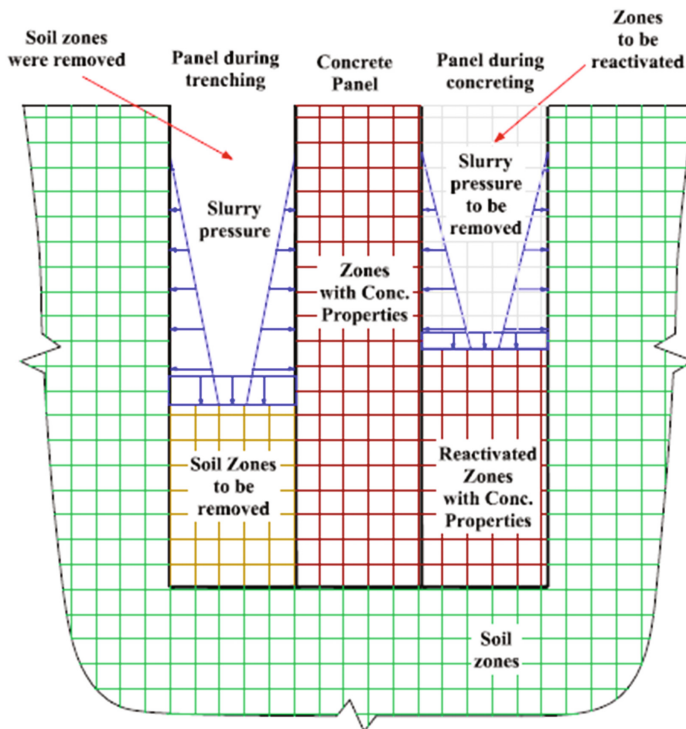


Fig. 3. Trench modeling process

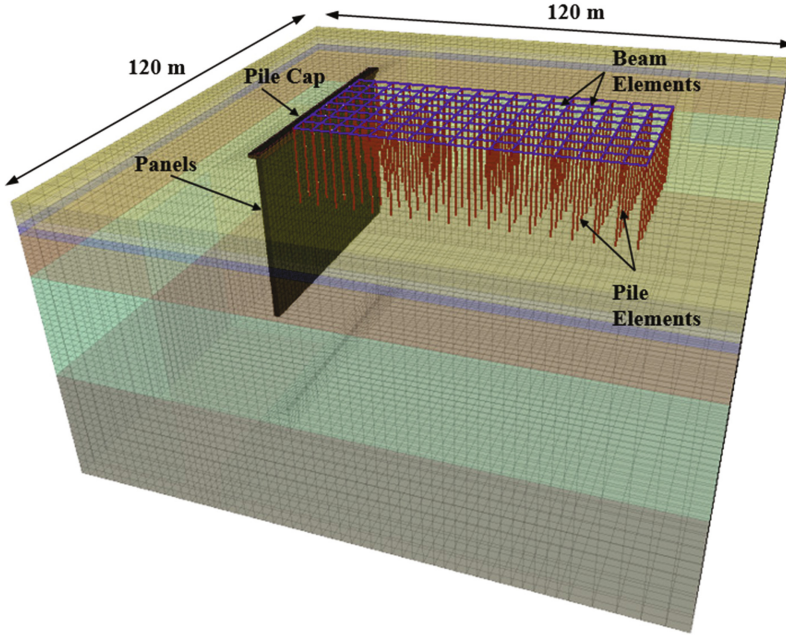


Fig. 4. Mesh Geometry of trench near court of justice

the soil are defined by the normal stiffness K_n and shear stiffness K_s , respectively. They are considered to be equal and can be calculated from the following empirical equation:

$$K_n = K_s = 10 \max \left[K + \frac{4G}{\Delta z_{\min}} \right] \quad (3)$$

where

k is the soil bulk modulus

G is the soil shear modulus

Δz_{\min} is the minimum distance in the vertical direction of the mesh

2.1.2 Results and Comparison

The settlement results of the numerical analysis compared to field data for points D and E at different stages are presented in Fig. 5. The results showed that the settlement values increase with stages and decrease with the distance from the trench.

The values of settlement from the numerical analysis nearest to the trench (point D) were in a very good agreement with those from the field data. This good agreement was not found in case of comparing the results at point E. Generally, the comparison showed that the numerical analysis provides reliable results that can present the reality.

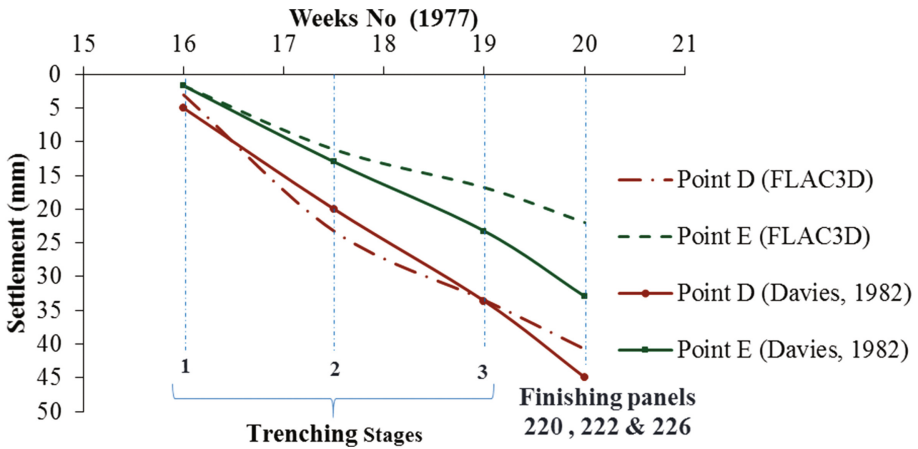


Fig. 5. Settlement at different stages

2.2 Case History 2 (Basement Near High-Rise Buildings in Giza, Egypt)

The underground construction of a basement in a crowded area in Giza, Egypt was done using diaphragm wall technique. The construction area was surrounded by several buildings as described by Abdel-Rahman and El-Sayed (2009). The soil was mainly sand and it was simulated using the same soil model that was used in the first case history. The soil properties are shown in Table 2. The panels construction stages are presented in Fig. 6 and they were modelled as in the field. Each panel was modeled as previously described and as shown in Fig. 3.

Table 2. Soil properties (Basement near piled foundation, Giza)

Soil layer	Bottom level (m)	SPT	γ_b (kN/m ³)	ϕ' (°)	E_{oed} (MPa)	E_{ur} (MPa)
Fill	-2.0	-	17.0	28	16.0	48.0
Silty sand	-5.0	12	18.0	30	17.0	49.0
Medium sand	-11.0	20	19.0	33.5	36.0	108.0
Dense sand	-25.0	42	20.0	36	42.0	124.0

The piles and the grade beams connecting the pile caps were modelled as described in the first case history. The pile caps were modeled using the shell elements and they carry the building load. The mesh used to simulate this case history is presented in Fig. 7.

The results from the numerical analysis compared to those from the field are presented in Fig. 8. The comparison was made for the three sections. The values of settlement were measured during trenching of the last panel (i.e. stage P-20-B) and it shows a decrease with distance from the trench. There was a slight difference regarding the settlement shape between the field and the numerical analysis. The settlement values from the numerical analysis adjacent to the trench and at a distance of 19 m

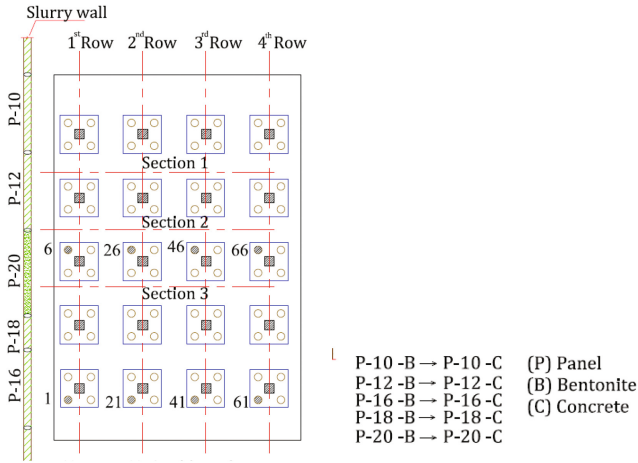


Fig. 6. The construction stages of the panels adjacent to the studied building

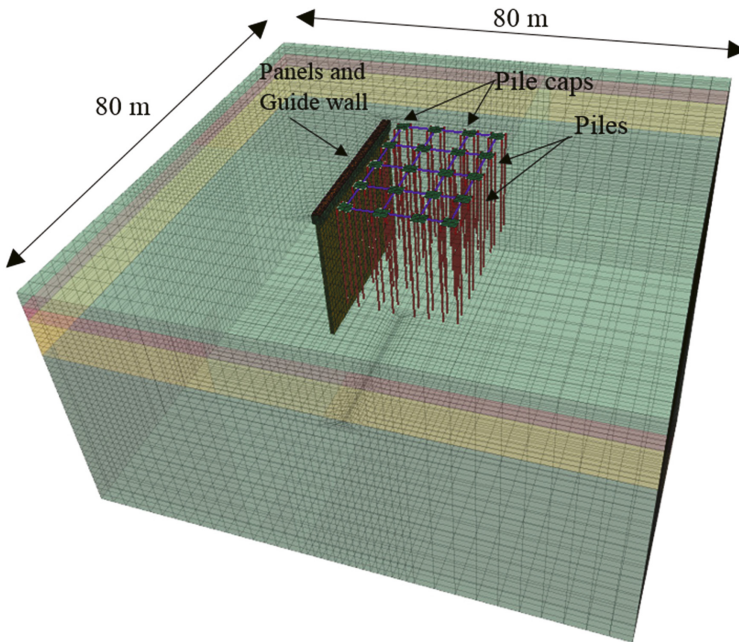


Fig. 7. Mesh geometry of the trench near the multi-story building

from the trench are identical with those from the field, but the settlement values from the numerical analysis were slightly higher than those from the field at a distance of 5 m from the trench. Generally, the output from the numerical analysis is in a good contrast with the field results.

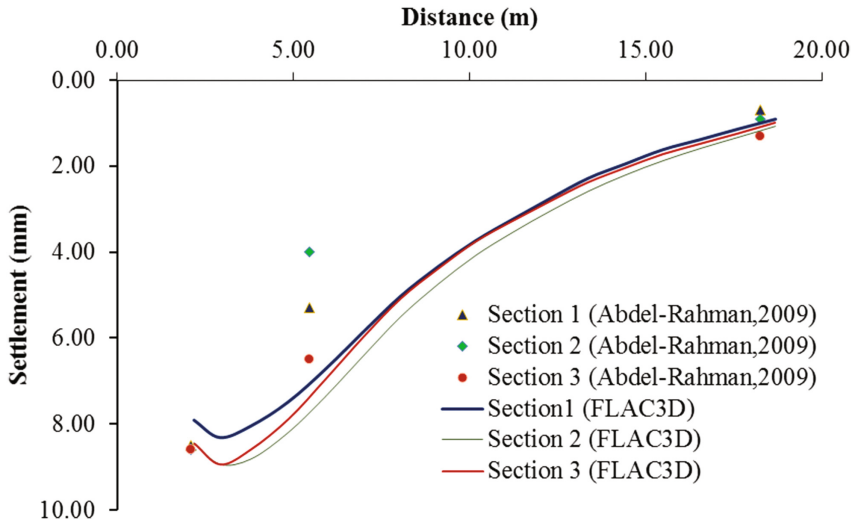


Fig. 8. Settlement at the last stage for different sections

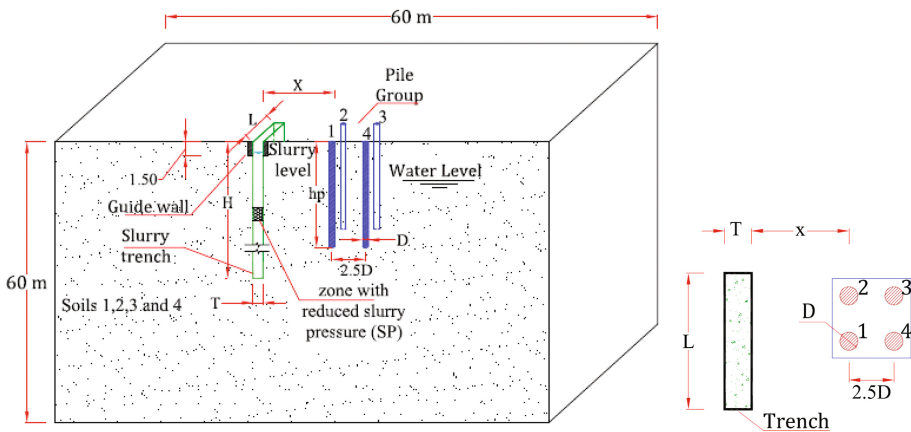


Fig. 9. Model group (MG1) – single panel

3 Numerical Parametric Study

The numerical simulation and comparison for the presented case histories showed that the method of simulating the trench numerically is acceptable and provides reliable results. This method was used to conduct a numerical parametric study includes several parameters. Two main models were used for the parametric study. The first model simulated the effect of the single panel on the pile group as presented in Fig. 9, while the second model simulated the effect of the double panels on similar pile group as shown in Fig. 10.

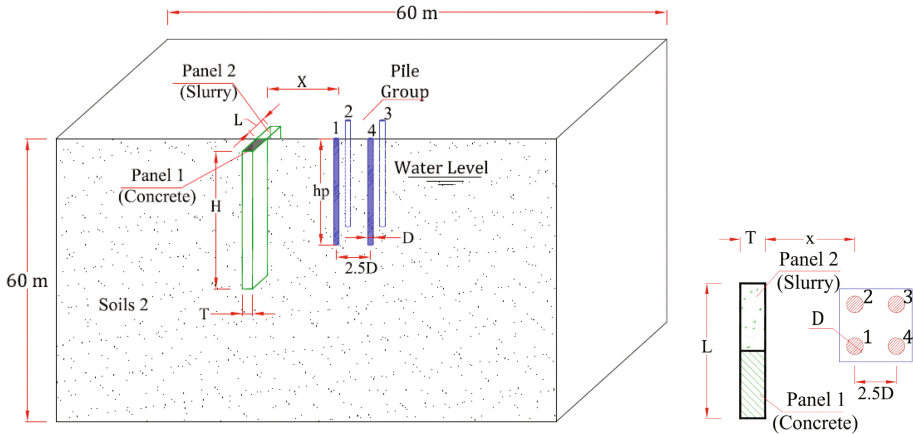


Fig. 10. Model group (MG2) – double panel

3.1 Studied Parameters

The soil used in this study was sand with a friction angle $\phi = 32^\circ$, a bulk density $\gamma_b = 18 \text{ kN/m}^3$ and shear modulus $G = 9.6 \text{ MPa}$. The pile depth was 12 m and its diameter (D) was 0.8 m in all cases, but in case of studying the effect of the pile diameter and its properties were chosen as variables according to Table 3. The panel depth was chosen to be 30 m because below this depth the pile with a 12 m depth will almost not be affected (Mohamed 2015). The trench length (L) was either 3 m or 6 m while its thickness (T) was 0.6 m and 1.2 m. The slurry and groundwater levels were chosen to be 0.5 m and 2 m below the ground surface, respectively. Five other different values of groundwater levels were studied separately. The effect of loss in the slurry pressure due to the existence of a weak soil layer at a certain depth of the trench was simulated by reducing the slurry pressure (SP) at some depths. The pile group was located at a distance (x) from the trench. This distance was considered to be 3.5 m in case of studying the other parameters.

3.2 Results from the Parametric Study

The effect of each parameter on piles deflection and bending moment is shown in Figs. 11, 12, 13, 14, 15 and 16, such an effect was generally presented for the first pile within the group. The effect of change of panel length on the pile behavior was very high but the effect of change in panel thickness shows was very low as shown in Fig. 11.

The location of the pile within the group and its distance from the trench controls the values and shape of the pile deflection and bending moment as shown in Figs. 12 and 13. The piles nearest to the trench within the group (i.e. piles 1 and 2) their tip deflect more than their top, because they move with the soil while the rear piles (i.e. piles 3 and 4) deflect from their top higher because they are affected by the drag force

Table 3. Piles properties used in the parametric study

Pile diameter	Pile				Interface			
	Area (m ²)	Inertia (m ⁴)	Perimeter (m)	E (kN/m ²)	k _s	k _n	φ _s	φ _n
					(kN/m ³)		(degree)	
0.2	0.0314	0.00015	0.6283	2.5 × 10 ⁷	Equation 3		2/3φ	
0.4	0.125	0.0025	1.25					
0.6	0.283	0.0120	1.88					
0.8	0.502	0.0402	2.51					
1.0	0.785	0.0981	3.14					
1.2	1.131	0.203	3.77					

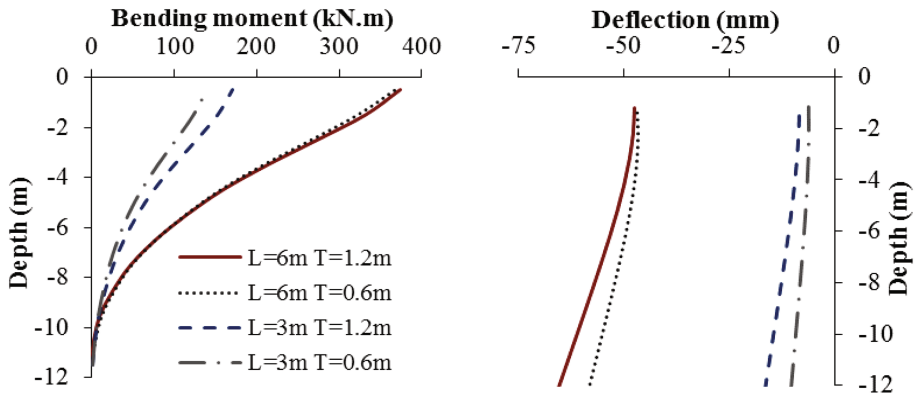


Fig. 11. Panels with different lengths and thickness

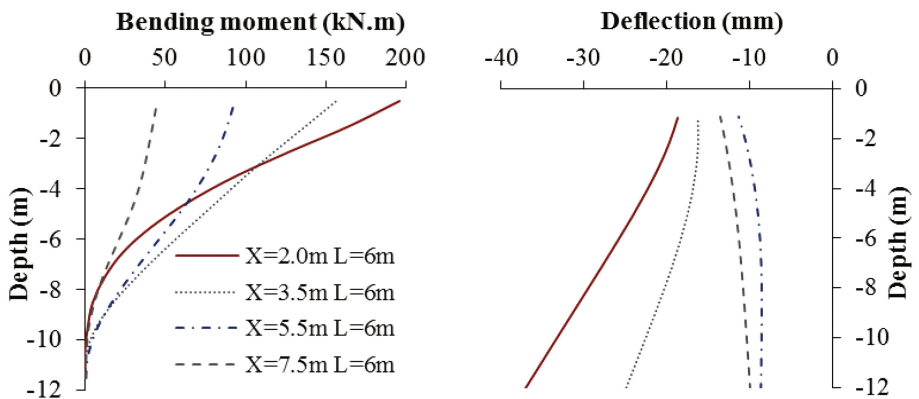


Fig. 12. Piles at different distances (x) from the trench

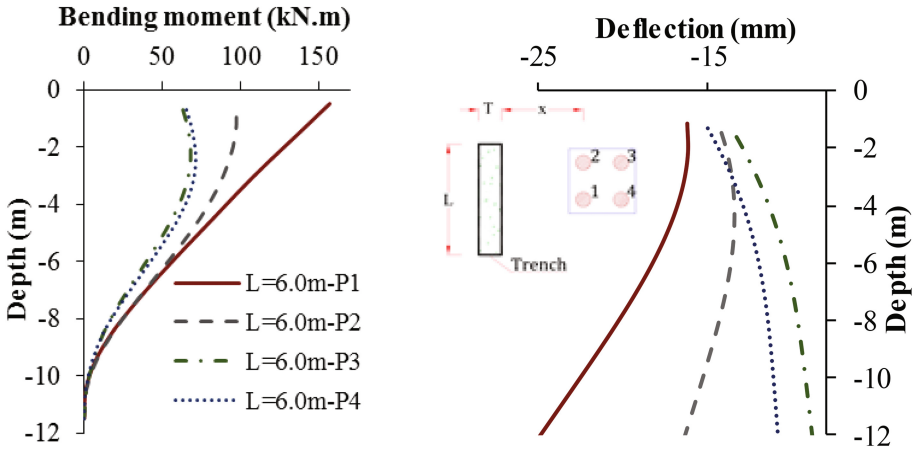


Fig. 13. Piles at different positions within the group

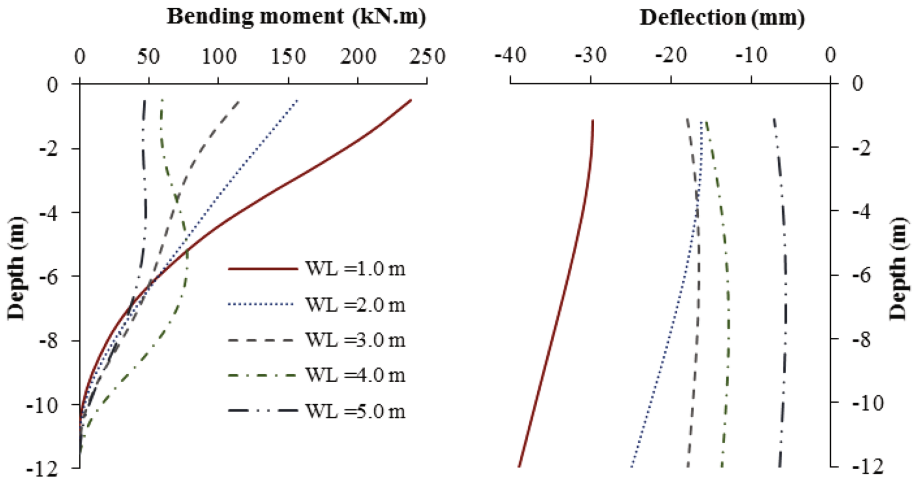


Fig. 14. Different values of ground water level

of the front pile. Generally, the deflection and bending moment values decrease with the distance from the trench.

The effect of different ground water levels on the first pile within a pile group is presented in Fig. 14. The deeper the ground water level the lower the deflection and bending moment. A noticeable difference was found when the ground water table was 1.0 m below the ground surface. The effect of the slurry pressure reduction was higher than the effect of change in the ground water level as shown in Fig. 15. The pile was greatly affected when the slurry pressure was reduced at depths between 11.5 m and 12.5 m (i.e. the location of the pile tip).

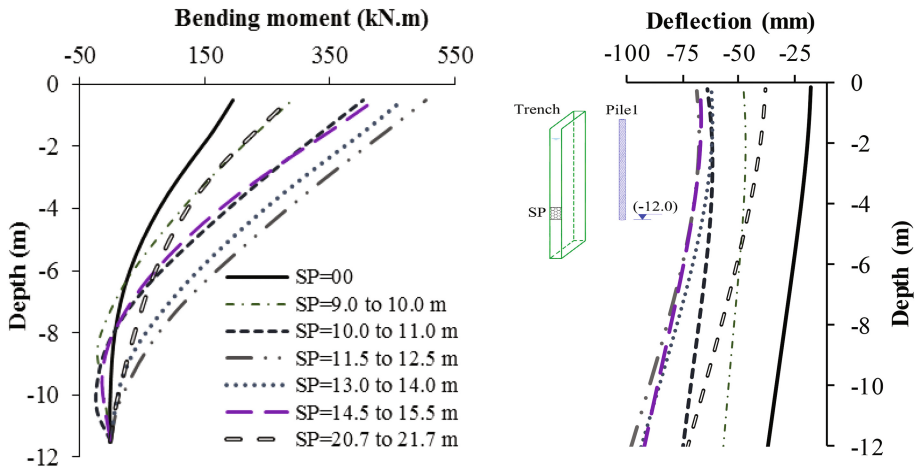


Fig. 15. Piles at different positions within the group

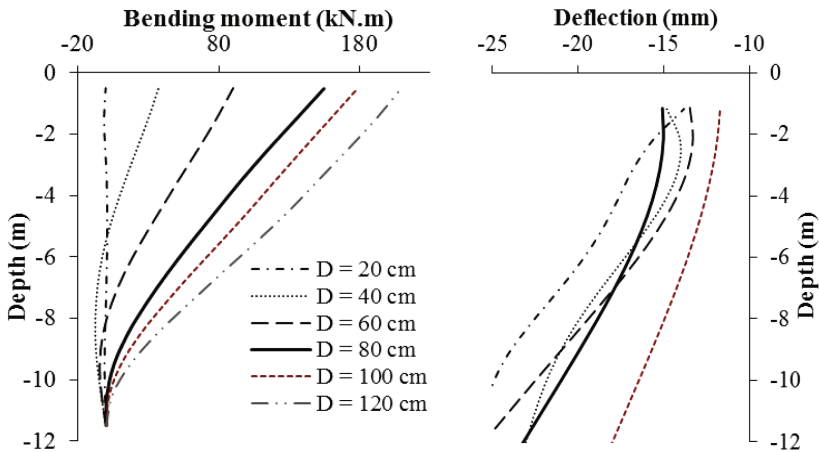


Fig. 16. Piles with different diameters

The large diameter pile is associated with low values of deflection but high values of bending moment as shown in Fig. 16. The large diameter pile provides a high stiffness and hence a high resistance to the soil movement. The ratio of change of bending moment values associated with the decrease in deflection is almost constant but the difference in deflection values was not constant. The deflection at pile diameters of 40, 60 and 80 cm was almost the same but it was slightly higher at a pile diameter of 20 cm and relatively low for a pile diameter of 100 cm.

The effect of panel construction stages on the first and third piles within the group is shown in Fig. 17. The values of the pile deflection and bending moment due to trenching the panel in two stages were lower than that due to trenching the panel in one stage.

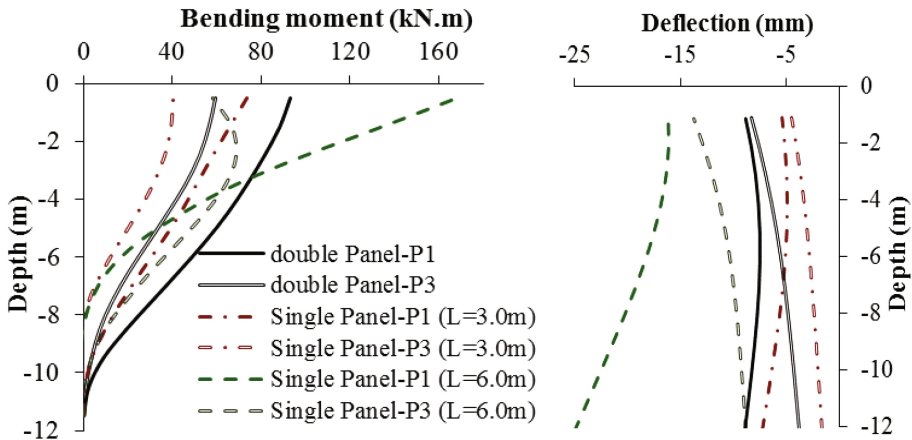


Fig. 17. Panel construction stages

4 Conclusions

The three-dimensional numerical simulation of the diaphragm wall trenching process was verified using two different case histories. The settlement results from the numerical analysis and field data were in a good agreement. However, some differences between both results were found due to the random nature of the field data results, while the numerical analysis depends on mathematical relations and provides systematic results.

The pile deflection and bending moment due to trenching were presented through a parametric study. The different values of panel thickness and pile diameter did not show a noticeable difference in the pile deflection, but the different values of pile diameter causes a noticeable change in pile bending moment. On the other hand, the change in panel length is associated with a great change in pile deflection and bending moment.

The change in values of groundwater level and pile group location causes a noticeable effect on the pile behavior, while the reduction of slurry pressure at some levels causes a great effect on the pile deflection and bending moment.

The piles within the group behave differently according to their position within the pile group. Generally, the piles nearest to the trench show higher deflection than those far from the trench and they are affected by the soil movement while the rear piles are affected by the drag force from the front piles.

The effect of trenching on piles could be reduced by conducting the panel in two stages, using shorter panel lengths and controlling the slurry pressure. It also could be reduced by lowering the groundwater table in the area before trenching.

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