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Simulation Of Deep Retaining Structures Using Finite Element Analysis

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Abstract:
Deep excavation is one of the more critical issues, various procedures or processes are utilized to reduce the impact of deep excavation on neighbouring structures related with the choose of the retaining system. The main purpose of this research is to assess the diaphragm wall behaviour for deep excavation of the Heliopolis Cairo metro tunnelling station and choose the more acceptable numerical modelling for simulation the case study depends on the field inclinometer data. The performance of deep excavations is influenced by both stability and deformation.Two-dimensional (2D) and three-dimensional (3D) analysis in the PLAXIS geotechnical program is used to simulate the case study problem.In this study, the sand clay soil layers are simulated by using two constitutive models: Mohr coulomb (MC) and hardening soil (HS). The final comparison results revealed that the 3D MC and 2D HS models have a high acceptable result with field measurements that have a significant path of lateral wall deformation.2D hardening soil is the more matching path for deformation comparation to the field measurement.3D Hardening soil model has the minimum deformation values by 8.74% less than the field inclinometer measured, while the maximum deformation is implemented by the 2D Mohr coulomb model is 11% greater than the field inclinometer readings.for both MC and HS models, the maximum value of deformation is near of the midpoint of the diaphragm wall height.Maximum surface settlement is applied by 2D hardening soil, while the minimum settlement is implemented using 3D MC. The plane strain ratio in this study, for both hardening soil and Mohr coulomb models in sand and clay soil is 0.7024 and 0.8165, respectively.
Keywords:*Deep excavation system, Simulation of diaphragm wall, Lateral deformation, Two-dimensional and three-dimensional numerical modeling, Hardening soil model, Mohr coulomb model, Plane strain ratio.*

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I. Introduction

Because of the rapid increase of urban development, an increasing number of design deep excavation projects Because of the intricacy of the interaction between the ground and the retaining structures for deep excavation, it might be difficult to forecast the behaviour of a retaining structure in detail and properly prior to the execution of the works. As a result, the role of the design engineer does not end with the design of the retaining structures. Rather, reviewing the performance of the retaining structure and comparing it to the design requirements and predictions, and taking the necessary actions to prevent the occurring of the critical limit state, such as large wall deformation causing damage to nearby structures or services. Peck's (1969) "Observational Method" is frequently used. [1]

Clough and O'Rourke (1990) established an empirical relationship for maximum lateral wall deformation with a parameter of safety toward basal heave, often known as system stiffness. Although the results are a valuable reference for estimating the extent of settlements/deflections, the majority of the known data came from excavations less than 15 meters deep with very flexible retaining walls. [2]

for high-rise buildings and tunnelling metro stations are being built and planned. Before the soil is excavated, diaphragm walls will be established. Diaphragm walls are frequently employed as both primary and permanent structural retaining in big deep excavations [3]. The majority of currently available empirical and semi-empirical excavation behaviour analysis approaches are based on projected excavations with flexible walls. As a result, investigating the impact of excavations on lateral wall deformation is important to deal with such a situation, it is vital to study the diaphragm wall behaviour during stages of excavation process [4], [5].However, ensuring the safety of neighbouring structures include an extensive cost, which is based on the lateral deformation behaviour of the diaphragm wall.As a result, choose the best the retaining system may reduce the overall project cost [6]-[8]. Peck (1969), Hsieh et al. (2003), Khoiri, Ou (2013), Koh and Chua

(2013), and others have conducted extensive research on the movement of supported walls and their surrounding grounds.

Surface settlement caused by excavation activities is a major cause of structural failure, and various studies have been performed to develop numerical and analytical approaches to estimate ground settlement [9]-[12]. Many researchers and engineers around the world have reported on observed deep excavation performance over the years (Peck, 1969, Mana and Clough, 1981, O'Rourke, 1981, Clough and Reed, 1984, Finno et al., 1989a, Finno and Nerby, 1989b; Clough and O'Rourke, 1990, Whittle et al., 1993, Ng, 1998, Ng, 1999, Ou et al., 1998, Ou et al.

The soil layers type plays an important role in the occurred diaphragm wall deflection of structures nearby during different stages of excavation [13]-[16]. The most important factors for predicting lateral deformation and ground settlement are soil characteristics, excavation geometry and retaining structure type [17]- [19]. An embedded diaphragm wall's behaviour is a complex soil-structure interaction problem that is commonly solved using the finite element technique (FEM) [20]- [23]. Goh, A. T. C. (1990) investigated the impacts of wall feature features, depth of soil layer, excavation breadth, and wall embedment on the stability of deep excavation in clay using the FEM technique.

One of the major examples of such collapse case histories is the collapsed 13-story skyscraper in Shanghai's Minhang District in 2009. According to Chai et al., the main cause of this failure was a nearby deeper excavation that stressed the falling structure's piles.To create a safe and cost-effective design related with deep excavation supported by diaphragm wall retaining system, it is critical to use an appropriate constitutive model that simulates thisissue in sand and clay soil layers [24]- [28]. Many constitutive models, such as the elastic-perfectly plastic model and the hyperbolic model, have been developed in recent decades to represent soil behaviour. It is necessary to have a solid understanding of the factors that influence or reduce deformations in a soil model in order to simulate excavation. Sensitivity assessments of soil and structural behaviours are vital, requiring additional sanity checks on all designs. As a result, Mohr coulomb and hardening soil constitutive models are utilized in this work to validate the field data [29]- [36].

The most commonly used constitutive models, which are utilized in geotechnical software such as PLAXIS, require different input values generated from different soil testing findings. PLAXIS contains constitutive models, the parameters of which can be obtained directly or indirectly by simple tests such as the SPT test [37],[38]. Because of its user-friendliness, it has become one of the most widely used geotechnical engineering software programs.

The "plane strain ratio" (PSR) was invented by Ou et al., and it can determine the maximum lateral wall deformation at every site based on the distance it is from the excavation's corner and the excavation geometry.Researchers defined the PSR as the ratio of determined 3D maximum deflection to 2D maximum deflection [39]- [41] (Finno et al 2006 and Ou et al 2006).

II. CASE STUDY

Project Description

Egypt is one of the most crowded cities in the world. Egypt's government decided a long time ago to build a Cairo Metro transportation network line. The Cairo Metro system has three lines: line one (main line), line two, line three, and a fourth line that is still under construction. The case study on the third line, which has 27 stations. We focused on the Heliopolis tunneling station, which is one of the largest stations in the Middle East and Africa. The station runs from Haroun to EL Nozha 1. The tunnel of station is constructed by Bored Machine (TBM) which has a circular shape with a diameter inside of 8.35 meters.Figure 1 depicts the station plan, which includes two crossing rectangular plots, phases (4A and 4C), with dimensions of approximately (225*23) m. The deep excavation measurements are 20.74 m wide by 225 m long.The dimensions of the deep excavation are 20.74 m wide, 225 m long, and approximately 28.5 m deep.The proposed case statical cross section system, seen in Fig. 2, includes of a rectangular diaphragm wall with a depth of 41m and a thickness of 1.20m. Sequence also built four levels of reinforced concrete slabs, the roof, ticket, intermediated, and raft slab, to support the diaphragm wall at various phases of excavation. Furthermore, as shown in Fig. 2, a temporary successive inclined strut is formed every 3m in the horizontal direction between the intermediated and raft slabs to provide wall stability at a deeper level.

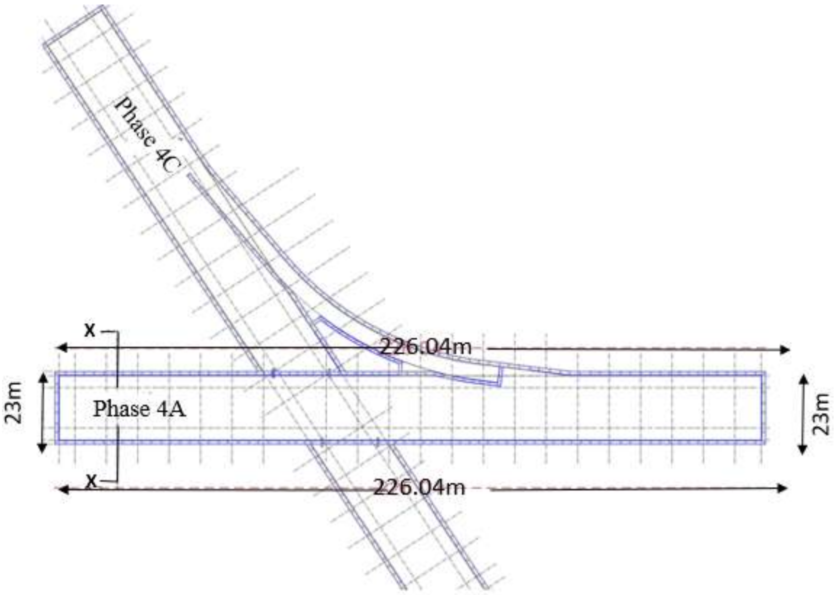

Figure 1 Layout of Heliopolis Underground Station

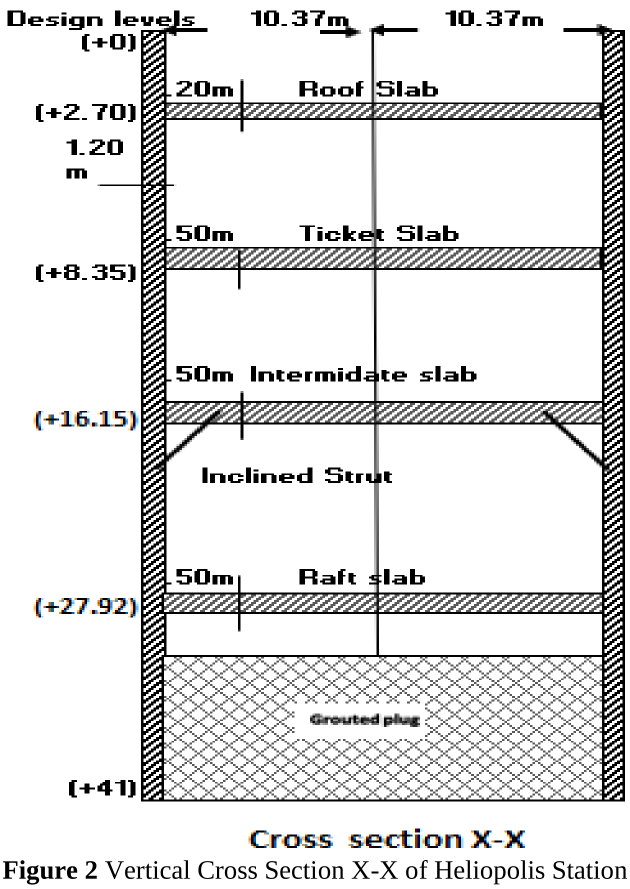

Figure 2 Vertical Cross Section X-X of Heliopolis Station

Soil Profile

The metro project's geotechnical specialists conducted a study of field soil boreholes. They investigated four boreholes BH 104D, BH11, B04, B24, B03and B25 as shown in Figure 3. This study is based on a borehole (B04) at a depth of 50 meters.

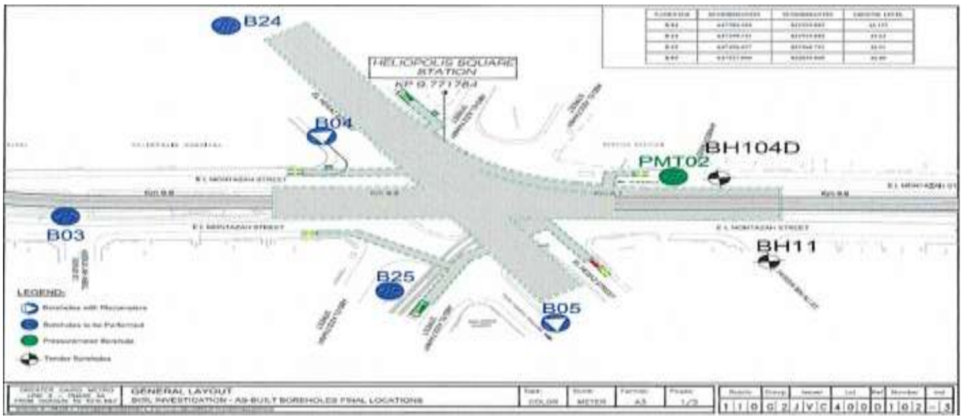

Figure 3 Locations of Borehole at Heliopolis Underground Station

The characteristics of BH no.4 with standard penetration test, which was employed in this case study, are depicted in Figure 4. According to the results of in-situ and laboratory tests, the subsurface water level is +28.30 m below the earth surface. The following is a definition of soil stratification: Layer (1), a two-meter-thick layer, this layer consists of asphalt, sand, gritty gravel, and sand with small gravel.Following the first bottom layer, the thickness of the layer 2 is about 9.8m. This layer was thick to extremely dense, fine to coarse sand and gravel, with traces of silt, calcareous, and brown. Layer (3) changes the thickness of the following layer by 9.20m. This stratum is stiff, silty, calcareous, and gravelly brown. Individually, the liquid limit, plasticity index, and water content are 54%, 21%, and 22%. The thickness of the succeeding layer is 29.5m in layer (4). This layer is composed of sand, silt, calcium carbonate, and gravel.It was described all over deep excavation in sand.

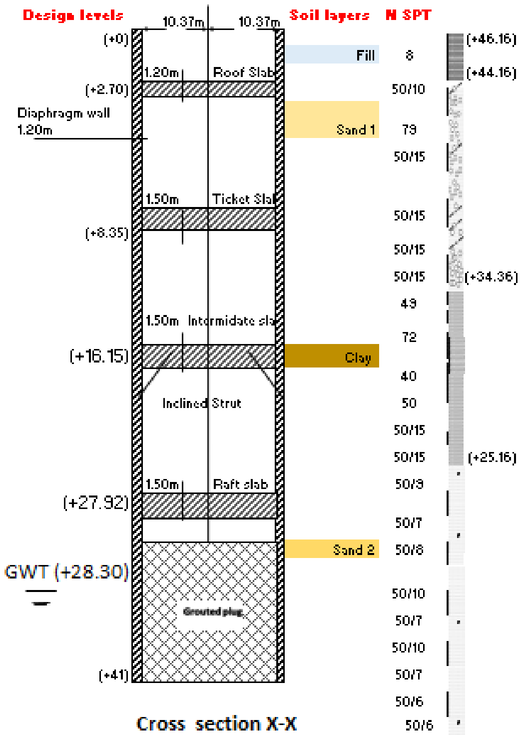

Figure 4 Soil Lithology at Borehole No.4

Construction Stages

Figure 5 displays the top-down construction method utilized in the station's construction [42]. Install a diaphragm wall around the station first, as shown in Fig. 5.a. Second, as shown in Fig. 5.b, soil injection is employed to form a non-porous pluge beneath the raft. Third, excavate beneath the roof slab to build a concrete roof slab, as shown in Fig. 5.c. Fourth, excavate beneath the ticket slab to build the concrete ticket slab, as shown in Fig 5.d. Fifth, excavate beneath the intermediate slab to build a concrete intermediate slab, as shown in Fig. 5.e.Six excavation under-steel inclined strut supports, as shown in Fig. 5.f, are required to form the inclined steel struts. Seventh, as shown in Fig. 5.g, dig beneath the raft slab to build a concrete raft slab. Remove the inclined struts, as shown in Figure 5.h.

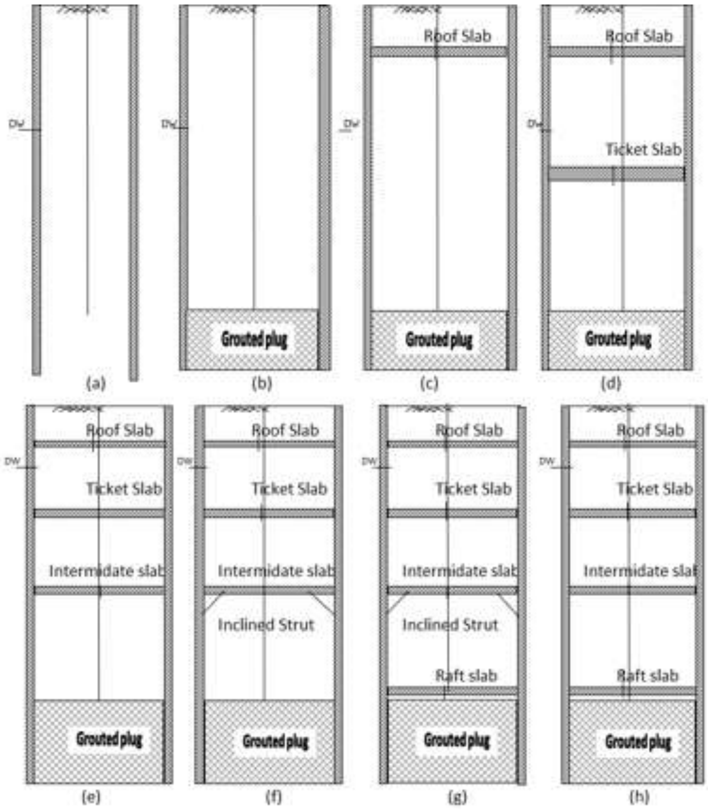

Figure 5 Stages of Deep Excavation Station Construction System

Field Measurement

Inclinometer devices are placed in the panels throughout the station to monitor the field actual deformation during top-down diaphragm wall performance, as shown in Fig. 6. It is critical to emphasize that the study is concerned with inclinometer (1) readings collected after the completion of station construction. The remaining four tests reveal that the readings are consistent, and the largest lateral displacement of the wall was 16.58mm, as shown in Fig. 7.

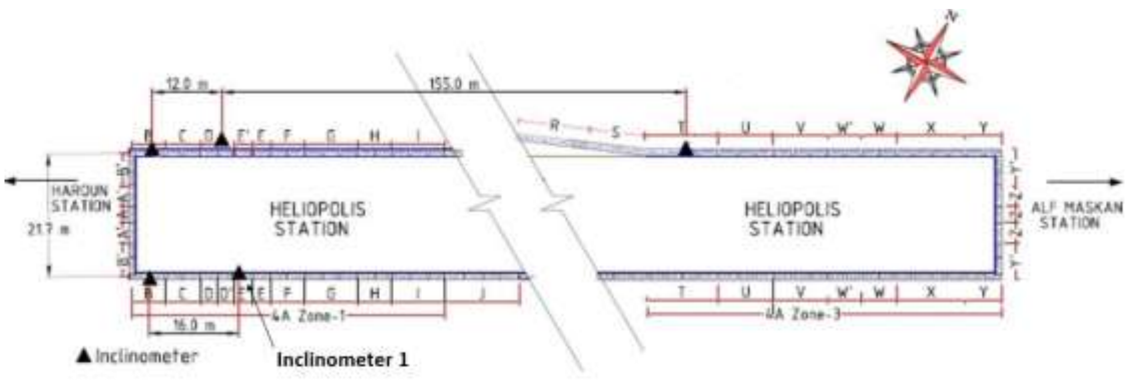

Figure 6 Inclinometer Places of Station phase (4A)

III. Analysis of numerical model

(2D and 3D) FE models are created using (MC and HS) two constitutive models to investigate the impact of the Heliopolis station's deep excavation problem on the lateral deformation of the wall and surface settlement results [43]. As a result, the lateral deformation findings of both 2D and 3D numerical models are compared to field data. The analytical stages of both the 2D and 3D models show that the first step identifies the soil's initial pressures (K0) process prior to the deployment of the support system. In the second step, the volume of the diaphragm wall is changed from dirt to concrete. To reduce numerical instability, stiff interface elements were employed to connect the wall and soil mesh elements at this phase, and the wall's self-weight was also considered. As seen in Figure 5, deformation occurs as a result of station excavation phases. The characteristics of structural components are clarified in Table 1.

Table 1 Structure elements Parameters

Thickness (m)	**Identification of slab**	**Axial force (EA)** (kN/m)	**Flexural stiffness (EI)** (kNm²/m)	**weight (w)** (kN/m/m)	**Poisson's ratio (n)** (-)
1.20	D-Wall Continuous	2.39E+07	2.87E+06	30	0.20
1.20	Roof Continuous	2.80E+07	3.36E+06	0	0.20
1.50	Ticket & Intermediate Continuous	3.50E+07	6.56E+06	0	0.20
1.50	Raft Continuous	3.50E+07	6.56E+06	37.50	0.20

Adapted Constitutive Models and Soil Parameters

Soils display non-linear behavior when subjected to changes in strain or stress. Soil stiffness depends on the stress law, the stress and the strain level. The MC is a totally plastic essential and linearly elastic material. Based on Hooke's isotropic elasticity, the model may be used as a first evaluation for soil behavior. Schanz (1998) describes the HS model as a comprehensive model for simulating the behavior of various soils ranging from soft to hard. When subjected to main deviatoric load, soil stiffness decreases and irreversible plastics stresses occur.The HS advances the hyperbolic model by use plasticity theory instead of elasticity, including soil dilatancy, and establishing a yield cap. The HS model depicts soil material by considering three different values of stress moduli (Eoed, E50, and Eur). Furthermore, the stress-strain relationship for initial loading is quite nonlinear, and the unloaded impact may be modeled. The major fundamental characteristics for the two models associated by soil layers are shown in Table 2.

Table 2 Soil Data Sets Parameters

	Soil layers	**Fill**	**Sand 1**	**Clay**	**Sand 2**	**non-Porous**
	Depth (m)	2	9.80	9.20	27	
Parameters	Type	Drained	Drained	Drained	Drained	Non-porous
Unit weight (g $_{unsat}$)	(kN/m³)	18	19	19	21	19
Unit weight (g$_{sat}$)	(kN/m³)	18	19	19	21	19
void ratio (e init)	(-)	0.50	0.50	0.50	0.50	0.50
Triaxial loading stiffness (E_{50})	(kN/m²)	15000	75000	40500	180000	180000
Poisson's ratio (n)	(-)	0.30	0.30	0.35	0.30	0.30
Oedometer loading stiffness (E_{oed})	(kN/m²)	15000	75000	40500	180000	69230
Triaxial unloading stiffness (Eur)	(kN/m²)	45000	225000	121500	540000	-
Cohesion (C)	(kN/m²)	1	1	15	1	1
Shear modulus (G)	(kN/m²)	20190	101000	65000	76920	69230
Friction angle (f)	(°)	27	37	29	42	42
Dilatancy angle (y)	(°)	0	7	0	12	12
interface reduction factor (Rinter)	(-)	0.67	0.67	0.67	0.67	0.67

2D Numerical Modeling

2D FEM is extremely useful when the analysis is primarily focused on horizontal surfaces, making it suitable for many common geotechnical engineering problems. The soil layers are represented in the 2D FEM by high order 15-noded 2D mesh components. In addition, the diaphragm walls and concrete slabs were modeled as plate elements. Based on the results of prior sensitivity tests, a geometry with dimensions of 115 m width and 96 m depth was chosen, as illustrated in Figure 8. The wall distance from the boundary is about 47.13m.

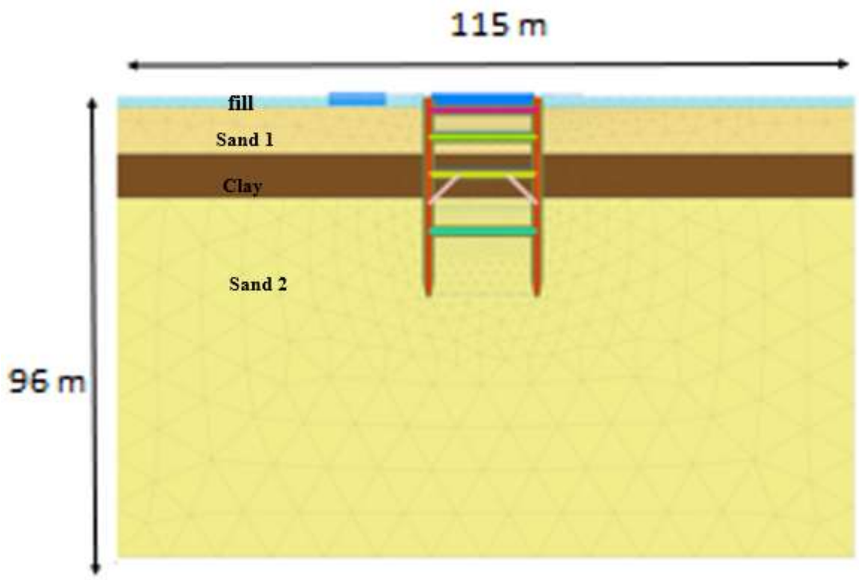

Figure 8 Geometry of 2D plain strain FE Model

3D Numerical Modeling

The geometry of the soil structure is represented by a 3D model. offering a more accurate portrayal of field condition. The soil layers in the 3D FM model are represented by high order 10-noded mesh elements with the same mesh dimensions as in the 2D model [43], [44], as illustrated in Figure 9. The mesh density within the excavation had an important impact on the precision of the analysis, but the mesh density outside the excavation had little effect.

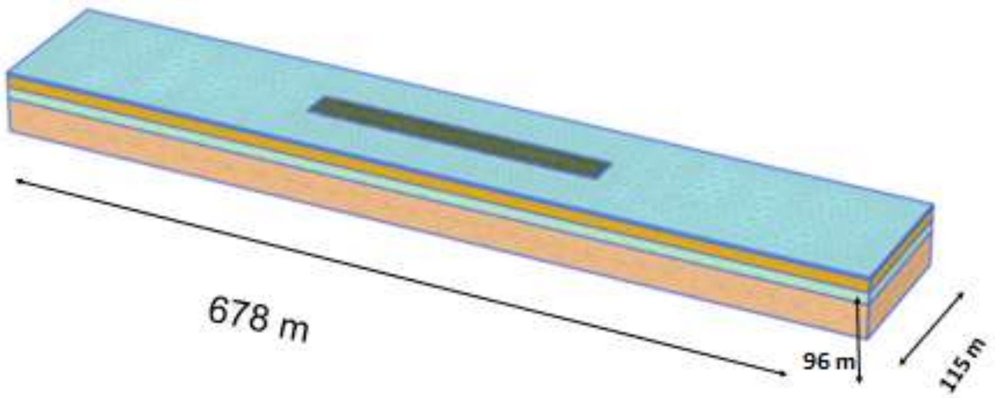

Figure 9 Dimension and boundary of 3D FE Model

IV. Discussion of the results

For the MC numerical models (2D and 3D), Figure 10a depicts the relationship between wall depth and lateral deformation. In both the (2D and 3D) MC, the most significant lateral deformation value occurs at the diaphragm's mid height. The largest 2D MC deformation measured is 18.52mm, which is 11% larger than the field measurement. The 3D MC model, on the other hand, forecasts a minimum deformation of 15.12 mm with a field of less than 8.74%. Fig. 9b also depicts the link between depth and (2D and 3D) HS lateral deformation. The maximum deformation result in both the (2D and 3D) HS is nearly the same as the (2D and 3D) MC value around the diaphragm's mid height.The 3D HS provides lateral wall deformation that is 12.442mm less than field measurements by 24.90%. In addition, the 2D HS deformation curve is more suited for field measurement. PSR as calculated by equation 1 is.8165 for MC and.7024 for HS.

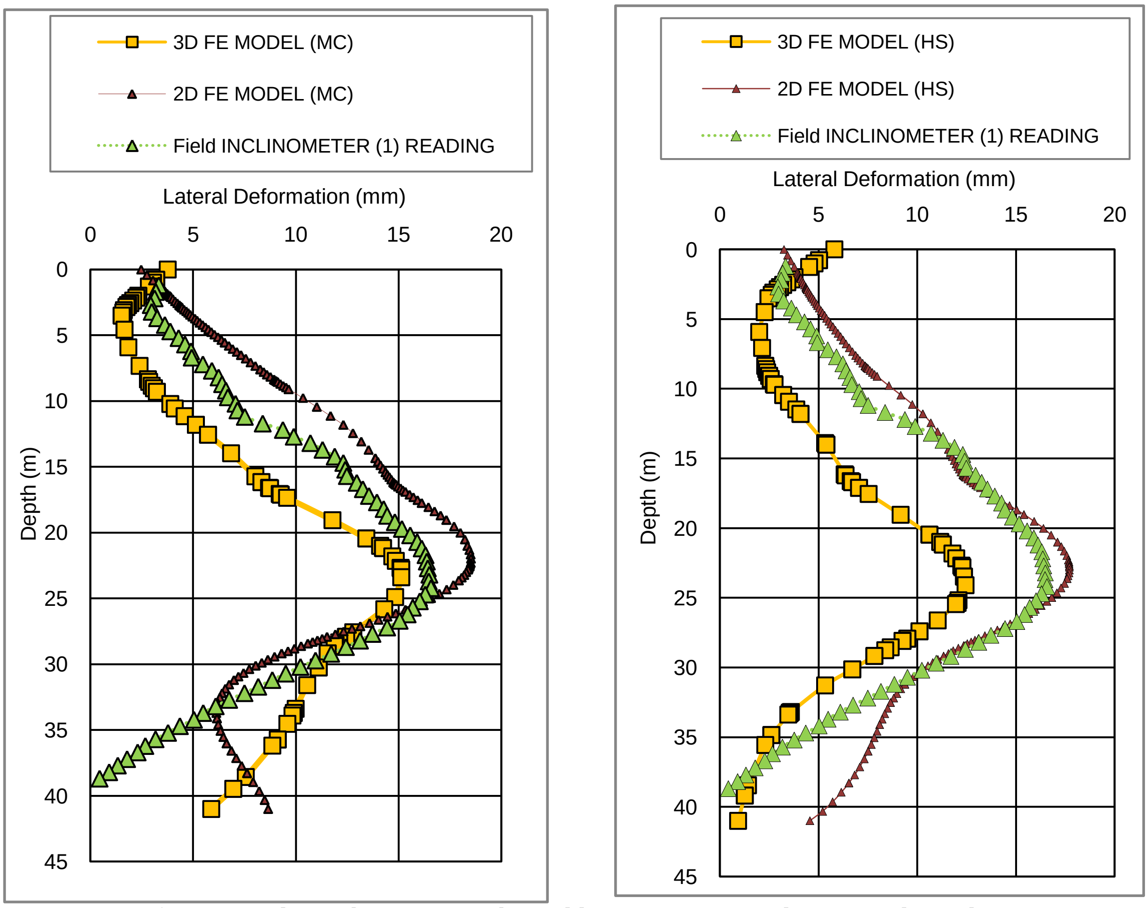

Figure 10 Relationship Between the Field measurements and Numerical Results.
(a) (2D and 3D) MC models. (b) (2D and 3D) HS models

Figure 11a shows the relationship of ground surface settlements on the left side of the diaphragm wall to the results of both (2D and 3D) MC models. The greatest settlement of 19.43 mm is obtained by 2D MC at a distance of (10-15) m. 3D MC, on the other hand, obtained the greatest settlement of 16.31 mm at a distance of (20-30) m. The results of ground settlement using (2D and 3D) HS are shown in Fig. 10b. The findings reveal that the greatest settlement of 30.12mm is achieved using the 2D HS model at a distance of (20-30) m. Figure 12 compares the expected vertical soil settlement determined using Peak's (1969) graphic [45] to the findings of the numerical models. As previously stated, Figure 13 demonstrates a satisfactory result for numerical models and Clough and O'Rourke's (1990) [46]. In contrast, Figure 14 depicts the relationship between maximum numerical settlement and depth of excavation on Clough and O'Rourke's, where a high degree of agreement is presented.

V. Conclusions

There is a brief conclusion in this paper for modeling of the deep excavation supported by diaphragm wall construction utilizing (2D and 3D) FE numerical models with (MC and HS) constitutive models in dense sand and clay soil layers.

1- A good recognized agreement is seen when comparing numerical modeling results (2D and 3D) with field measurements for the identical soil and structural component characteristics.
2- In both (2D and 3D) models and field measurements, the highest value of lateral deformation is close to the mid height of the diaphragm wall at an average depth of (20-25) m.
3- The lateral deformation by 3D (MC and HS) and 2D (MC and HS) have a appropriated path with Field measurement.
4- The lateral deformation curve produced by the 2D hardening soil model corresponds better to field measurements of deformation in sand and clay soil layers deep excavation.
5- The minimum wall lateral displacement estimated by the 3D HS model is approximately 8.74% smaller than the values recorded by the field inclinometer. On the other hand, the largest lateral deformation observed by the 2D MC model is 11% more than the field inclinometer observations.

6- The maximum surface settlement is seen for the 2D MC, 2D HS, and 3D HS models at distances averaging between (10-20) m, and the maximum value of surface settlement is obtained by using the 2d HS model, while the smallest settlement is produced by using the 3D MC.
7- Clough and O'Rourke's reached a good agreement on maximum numerical settlement, with maximum surface settlements averaging approximately 0.15% of excavation depth.
8- In sand and clay deep excavation, the PSR value for MC is.8165, while the value for HS is.7024. According to (Lee et al., 1998), it is an anticipated value.

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