Evaluation of Performance of Diaphragm Walls by Wall Deflection Paths for Deep Excavations in Central Ha Noi

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ABSTRACT: The objective of this paper is to evaluate the performance of diaphragm walls by wall deflection paths for deep excavations in Central Ha Noi. The commercial software PLAXIS 2D was used as a numerical tool for 2D finite element analyses in this paper. A deep excavation in Central Ha Noi is adopted as a basis for numerical analyses in this study. A benchmark analysis was firstly conducted on the excavation to verify the validity of material models and their input parameters for predicting wall deflections; following that the reference envelopes of wall deflection paths were delivered for various conditions of deep excavations in Central Ha Noi. Considering the current prediction, up to 72 mm of the maximum lateral wall displacement for an excavation have a 21.9 m of excavation depth. However, some assumptions made have to be further confirmed by comparing data observed later on. Reference envelopes of excavations have been developed and discussed in various conditions of the excavation. It is first found that the maximum lateral wall displacement at the first stage of excavation is roughly inversely proportional to the Young's moduli of soils. In conclusion, changing wall thickness leads to the limited difference in reference envelope at shallow excavation stage, but it may not be true when the excavation goes deeper.

KEYWORDS: deep excavation, wall deflection, wall deflection path, reference envelope, PLAXIS 2D.

1. INTRODUCTION

In the process of design and construction of deep excavations, wall deflections and ground settlements must be carefully considered because their shape and magnitude influence the safety of neighbouring buildings, especially in urban areas. In a certain excavation, movements of ground and wall are also related to each other. Many empirical correlations between wall deflections and ground settlements have proposed in the literature such as the studies of Ou et al. (1993), Bowles (1996), Hsieh and Ou (1998), Ou (2006), Kung et al. (2007) and Wang et al. (2010). Thus, once the wall deflections are estimated, ground settlements will be calculated accordingly, and then the assessment of the safety of structures adjacent to the excavation can be made. Recent works related to ground behaviours of deep excavations and related simulation were also carried out by Tan and Wei (2012), Likitlersuang et al. (2013), Juang et al. (2013), Hsieh et al. (2013), Khoiri and Ou (2013), Finno et al. (2015), Orazalin et al. (2015) and Hsieh et al. (2015).

The concepts of wall deflection path and reference envelope of wall deflection paths were first proposed in the study of Moh and Hwang (2005), subsequently, they were further discussed in the studies of Hwang and Moh (2007 and 2008), Hwang et al. (2007a and 2007b), Hsiung and Hwang (2009b), Hwang et al. (2012), Hsiung et al. (2013) and Dao (2015). The additional concepts of backbone envelope and baseline wall deflection path were also proposed for evaluating the performance of diaphragm walls and factors affecting wall deflections for deep excavations on soft ground. However, these researches only discussed deep excavations in Taipei or Kaohsiung, Taiwan.

This study aims to evaluate the performance of diaphragm walls by wall deflection paths and factors affecting wall deflections for deep excavations in Central Ha Noi. The wall deflection paths and green reference envelopes of wall deflection paths were performed for various conditions of deep excavations in Ha Noi.

The commercial software PLAXIS 2D, version 9 (2009) was used as a numerical tool for two-dimensional finite element analyses in this study. This study is expected to be helpful for the process of design and construction of deep excavations that will be widely built in the near future in Central Ha Noi.

2. BENCHMARK ANALYSIS FOR PREDICTING WALL DEFLECTIONS

A deep excavation in Central Ha Noi, Vietnam is referenced as the basis for numerical analyses in this study; such large-scaled deep excavation has not yet been implemented in the city so far. The shape of the excavation is rectangular with 160.4 m in length and 22.7 m in width. The excavation is performed by the semi top-down construction method and is retained by the diaphragm wall that is 1.0 m thick and 34.0 m deep. It is excavated to the maximum depth of 21.9 m in five stages of the excavation. The retaining wall is propped by one level of concrete floor slab and three levels of steel struts. The horizontal spacing of the steel struts is about 3.5 m on average. Figure 1 below describes the cross section and ground condition of the excavation.

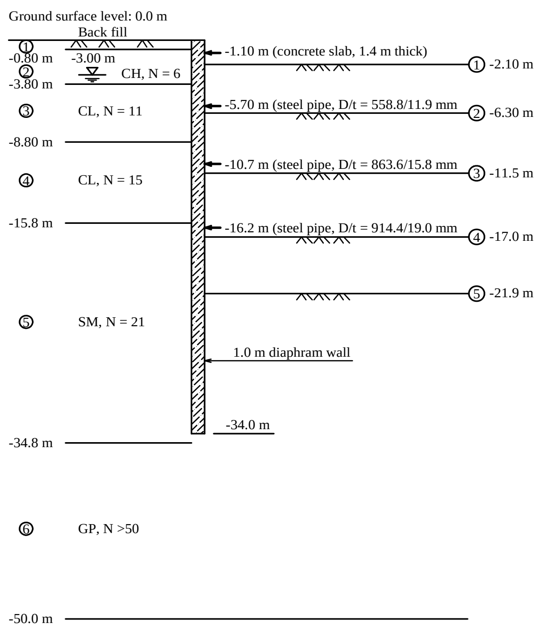


Figure 1 Cross section and ground condition of the excavation

From Figure 1, it clearly demonstrates that the clay layers are quite thick, and they are underlained by a thick sand layer in medium dense state. The depth of the clays is up to 15.8 m below the ground surface level, and their SPT-N values are in the range of 6 to 15. The sand layer is 19 m thick, and its SPT-N value is 21 on average. The gravel layer with SPT- N value greater than 50 is found below the sand layer. Several pumping tests were previously performed only in aquifer layers, which indicate that the permeability of ground is in a wide range of $2.2\text{x}10^{-6}$ m/sec to $1.9\text{x}10^{-3}$ m/sec. In an adverse manner, no other test data is available, therefore, the quality of soil samples as well as the soil parameters given can't be discussed herein.

The site investigation also reported that the groundwater level before the excavation is about 3.0 m deep below the ground surface level. Due to long-term pumping from deep groundwater aquifer layers in the area, current piezometric levels are lower than hydrostatic levels. The groundwater level inside the pit is lowered to a depth of 1.0 m below each excavation level before each stage of excavation to make a convenient space for the excavation process.

A two-dimensional finite element analysis, which is called "benchmark analysis", was conducted to simulate the excavation. The benchmark analysis was carried out to verify the validity of material models and their input parameters for predicting wall deflections caused by the deep excavation. The commercial software PLAXIS 2D, version 9 (2009), was used as a numerical tool for 2D finite element analyses in this study. PLAXIS 2D is a two-dimensional finite element program, which is developed at Deft University of Technology in the Netherlands and is made commercially available by PLAXIS Bv, Amsterdam, the Netherlands.

According to previous researchers, such as Hsieh et al. (2003), Ou (2006), Kung et al. (2007), Schweiger (2009), Khoiri and Ou (2013) and Dao (2015), the constitutive soil model adopted in numerical analysis has a limited influence on predicting the wall deflections induced by deep excavations. As a result, the linear elastic-perfectly plastic Mohr-Coulomb model (MC model) was adopted to simulate soils in the model of benchmark analysis. The MC model contains six input parameters, i.e. the internal friction angle, cohesion, Young's modulus, Poisson's ratio, dilatancy angle, and the coefficient of lateral earth pressure at rest. The sand layers (SM and GP types) were modelled by drained materials with effective strength and stiffness parameters, and the clay layers (CL type) were simulated by undrained materials with undrained strength and stiffness parameters.

For the sand layers, the effective friction angles (ϕ') were directly obtained from laboratory tests. Values of effective cohesion (c') were assumed to be zero, but to avoid the complication for the calculation of PLAXIS software, a very small value c' = 0.5 kPa was set for the sand layers. Drained Poisson's ratio (ν') was assumed to be 0.3 for the sands as suggested by PLAXIS 2D (2009), Khoiri and Ou (2013) and Dao (2015). As reported by Hsiung (2009), Hsiung and Hwang (2009a), Hwang et al. (2012) and Dao (2015), the effective Young's modulus (E') of the sand layers could be obtained by the following equation.

$$E' = 2000\,N\left(kPa\right) \tag{1}$$

in which N is blow counts in the Standard Penetration Tests (SPT). As proposed by Bolton (1986), the dilatancy angle of the sands could be computed as follows:

For sands with $\phi' \leq 30^0$:

$$\psi' = 0^0 \tag{2}$$

For sands with $\phi' > 30^0$:

$$\psi' = \phi' - 30^0 \tag{3}$$

The coefficient of lateral earth pressure at rest was determined by the following formula of Jaky (1944):

$$K_0 = 1 - \sin\phi' \tag{4}$$

Table 1 lists input parameters of the sand layers for the MC model used in the benchmark analysis. The value of γ_t means total unit weight of soils, and N indicates SPT- N value of soils.

For the three clay layers, which are modelled by the undrained total stress analysis, input parameters of undrained friction angle ϕ_u = 0, undrained cohesion $c_u = S_u$ (undrained shear strength), undrained Young's modulus E_u and undrained Poisson's ratio ν_u were used for the analysis. Undrained Poisson's ratio $\nu_u = 0.495$ ($\approx$ 0.5) was adopted to simulate the incompressible behaviour of water and to avoid numerical problems caused by the singularity of stiffness matrix. According to the previous studies of Bowles (1996), Lim et al. (2010), Likitlersuang et al. (2013), Khoiri and Ou (2013) and Dao (2015), undrained Young's modulus E_u of the clay layers can be calculated by the empirical equation as follows:

$$E_u = 500 S_u \tag{5}$$

Table 1 Input parameters of the sand layers for the MC model

Layer	Depth (m)	Soil type	γ_t (kN/m^3)	N	c' (kPa)	ϕ' (^{o})	ψ (^{o})	ν'	E' (kPa)	K_0
1	0.00-0.80	Back fill	19	-	0.5	30	0	0.3	20000	0.50
5	15.8-34.8	SM	20	21	0.5	34	4	0.3	42000	0.44
6	34.8-50.0	GP	21	> 50	0.5	40	10	0.3	100000	0.36

Table 2 shows input parameters of the clay layers for the MC model used in the benchmark analysis.

Table 2 Input parameters of the clay layers for the MC model

Layer	Depth (m)	Soil Type	γ_t (kN/m^3)	S_u (kPa)	E_u (kPa)	ν_u
2	0.80-3.80	CH	16	20	10000	0.495
3	3.80-8.80	CL	18.5	50	25000	0.495
4	8.80-15.8	CL	19	100	50000	0.495

The diaphragm wall was simulated by plate elements, and the struts were simulated by elements of fixed-end anchor. The linear elastic model was adopted to simulate both the diaphragm wall and steel struts. This model requires two input parameters, i.e. Young's modulus and Poisson's ratio. The Poisson's ratio was taken to be 0.2 for both the diaphragm wall and struts. The Young's modulus of the diaphragm wall and floor slab was calculated by the equation of ACI Committee 318 (1995) as follows:

$$E = 4700\sqrt{f_c'}\,(MPa) \tag{6}$$

in which $f_c'(MPa)$ is the standard compressive strength of the diaphragm wall and floor slab concrete. The Young's modulus of steel struts was taken as $2.1\text{x}10^5$ MPa. The stiffness of both the diaphragm wall and steel struts was reduced by 30% and 40% from their nominal values, respectively; consider the cracks in the diaphragm wall due to bending moments and repeated used and improper installation of steel struts as suggested by Ou (2006).

Tables 3 and 4 present input parameters of the diaphragm wall and struts used in the benchmark analysis. The weight of plate is obtained by multiplying the unit weight of plate by the thickness of plate. The observation showed that the unit weight of plate was subtracted by a value of soil unit weight because the wall is modelled as non-volume elements in PLAXIS program. The interface elements were also simulated to represent the friction between soil and the diaphragm wall. As proposed by PLAXIS 2D (2009), Khoiri and Ou (2013) and Dao (2015), the strength reduction factor of interface elements, R_{inter}, could be taken as 0.67 to simulate the disturbance of ground between the wall and soil. It is also noted that the input parameters of concrete floor slab were calculated for one width unit.

Table 3 Input parameters of diaphragm wall

Parameter	Name	Value	Unit
Compressive strength of concrete	f'_c	35	MPa
Young's modulus	E	2.78×10^7	kPa
Thickness	d	1	m
Axial stiffness x 70%	70%EA	1.95×10^7	kN/m
Flexural stiffness x 70%	70%EI	1.62×10^6	kNm²/m
Weight	w	5.5	kN/m/m
Poisson's ratio	ν	0.2	-

Table 4 Input parameters of struts

Strut level	Description	Section area (m²)	EA (kN)	60%EA (kN)
1	Concrete slab, 1.4 m thick, f'_c = 35 MPa	1.400	3.89×10^7	2.34×10^7
2	Steel pipe, D/t = 558.8/11.9 mm	0.020	4.29×10^6	2.58×10^6
3	Steel pipe, D/t = 863.6/15.8 mm	0.042	8.84×10^6	5.30×10^6
4	Steel pipe, D/t = 914.4/19.0 mm	0.053	1.12×10^7	6.73×10^6

Figure 2 below presents the finite element model of the benchmark analysis. Only a half of the excavation was modelled due to its symmetrical geometry. The base of the model was placed at a depth of 50 m below the ground surface level, i.e. approximately 15 m deep into the GP layer that is assumed to have very small deformations caused by the excavation. The distance from the lateral boundary of the model to the retaining wall was assumed to be 120 m, which is approximately five times excavation depth. This value was considered because according to many studies, such as Peck (1968), Clough and O'Rourke (1990), Ou et al. (1993), Hsieh and Ou (1998), Ou (2006), Kung et al. (2007), Wang et al. (2010), Ou and Hsieh (2011) and Dao (2015), ground settlements were found to be zero for the positions there are slightly more than four times the excavation depth. The horizontal movement was restrained for the lateral boundaries, and both vertical and horizontal movements were restrained for the bottom boundary of the model.

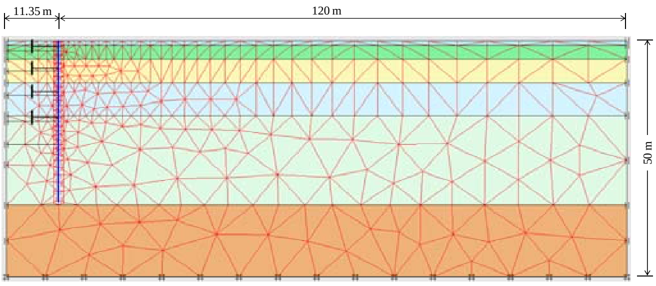


Figure 2 Finite element model of the benchmark analysis

Figure 3 shows the wall deflections predicted from the benchmark analysis for all stages of excavation. As shown in from Figure 3, the wall behaves as a cantilever at the first stage of excavation. It is because the concrete floor slab at the first strut level has not yet been constructed in this stage. The wall then displays deep inward movements at subsequent stages of excavation. Up to 72 mm of lateral wall displacement was predicted, which is equal to $0.33\%H_e$ (H_e is the excavation depth). It is also found that the occurrence of maximum wall displacements were almost at the excavation levels.

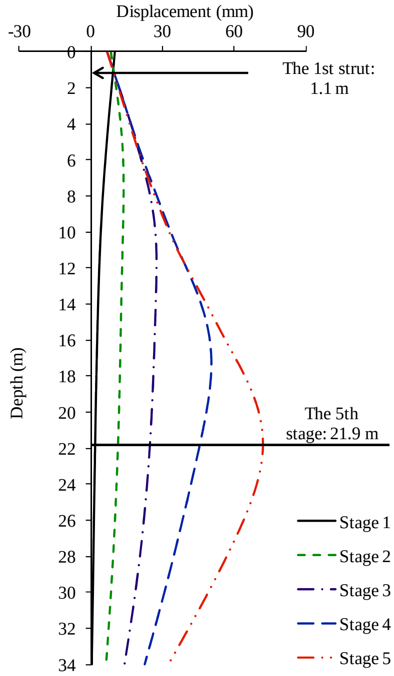


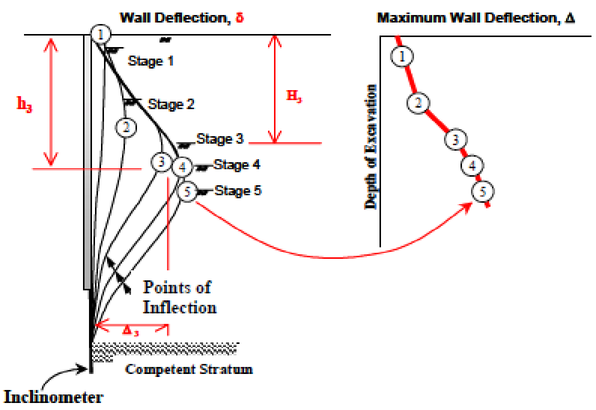
Figure 3 Wall deflections predicted from the benchmark analysis

At the later stages of excavation (Stages 4 and 5), movements of lower depths of the wall were quite large, especially at the wall toe. The main reason which relating to the fact that the MC model uses only one single Young's modulus, which does not distinguish between loading and unloading stiffness of soil, and also it does not include the high stiffness at small strain levels. These features of the MC model cause the over-prediction of excavation bottom heave because due to the high unloading stiffness and high stiffness at small strain levels of ground. The large heave of excavation bottom then causes the larger displacements of the wall toe as mentioned above.

3. REFERENCE ENVELOPES OF WALL DEFLECTION PATHS

Figure 4a indicates the normal wall deflection induced by propped excavation and the maximum wall deflection at each stage together with corresponding excavation depth, so called "wall deflection path" (Hwang et al., 2007a), as shown in Figure 4b. According to further studies of Hwang and Moh (2007 and 2008), Hwang et al. (2007a and 2007b), Hsiung and Hwang (2009b), Hwang et al. (2012), and Dao (2015), wall deflection paths of various ground conditions, surcharges and retaining system are developed. It is again suggested by Hwang et al. (2007a) that this relationship can become more obvious once a log-log scale is adopted and named reference envelope of wall deflection paths was thus defined in such log-log scale plot.

From conclusions made by Hwang et al. (2007a), the reference envelope can be determined by the maximum wall deflection at the excavation depth of 4 m, i.e. Δ4 and the maximum wall deflection at the excavation depth of 100 m, i.e. Δ100 (refer to Figure 4). The depth of 4 m is chosen because the first digs are usually within 4 m, and the depth of 100 m is chosen for convenience because Microsoft Excel only plots full log-cycles. The value remains the same from 1 m to 4 m of excavation depth as in general the maximum wall deflection keeps the same for excavation depth less than 4 m. In addition, the extension of reference envelope to the depth of 100 m amplifies the differences in reference envelopes among various cases and makes it easier to study the effects of various factors that affects the performance of walls. Based on the concepts of wall deflection path and reference envelope, the evaluation of performance of a diaphragm wall can be conducted by comparing its wall deflection path with the corresponding reference envelope, as illustrated in Figure 5.



(a) Ideal deflection profile (b) Wall deflection path

Figure 4 Ideal wall deflection profile and concept of wall deflection path (Hwang et al. 2007a)

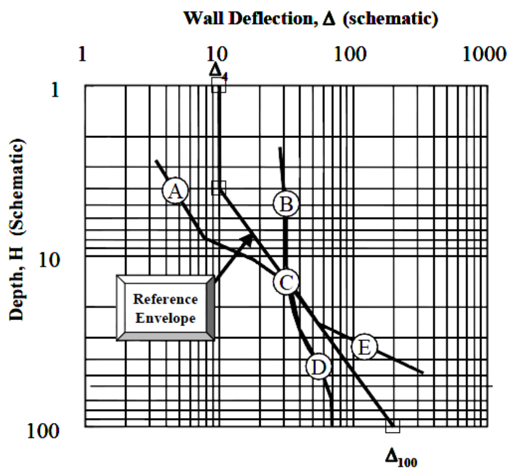


Figure 5 Evaluation of performance of diaphragm walls by wall deflection paths (Hwang and Moh, 2007)

Path A: The presence of basements, retaining walls, foundation piles and so on in the vicinity of the excavation is likely to reduce wall deflections in the early stages of excavation.

Path B: On the contrary, surcharge loads, such as embankments and buildings in the vicinity of excavation, if any, will increase wall deflections in the early stages of excavation.

Path D: As the excavation exceeds a certain depth, the performance of wall is affected by the stability of the wall toe. For wall with sufficient lengths beyond the formation level (excavation bottom) and/or with their toes properly embedded in competent stratum, wall deflections will increase with diminishing rates (in a log-log scale), and their wall deflection paths are thus expected to bend downward. Ground treatments below the formation level will have similar effects.

Path E: Inversely, if the wall deflection path for a certain wall becomes flatter than the reference envelope, it is most likely that the wall toe has become unstable. Soft strut system and poor workmanship will have similar effects.

Furthermore, the previous studies also pointed out that the reference envelope has two basic characteristics as follows:

1) Wall deflection paths tend to converge to a narrow band and more linear when excavation depths are in the range of 10 m to 20 m. Thus, the reference envelope should be established by using the data points of excavation depths in the range of 10 m to 20 m.
2) The value Δ4 is insensitive to wall stiffness, or the values of Δ4 are the same for various wall thicknesses. The value Δ100 is insensitive to ground conditions, or the values of Δ100 are the same for various ground conditions.

In this study, the reference envelope of an excavation is based on green field condition. The green field condition of an excavation is the condition that there are not the presence of adjacent structures, such as piles, basements and walls, conduits, utilities, tunnels, metro stations, buildings and embankment, in the vicinity of the excavation.

3.1. Reference envelopes for various ground stiffnesses

To investigate how ground stiffness affects the reference envelope, numerical analyses were carried out for various sets of ground stiffness as follows:

Set A: Young's moduli of soils were obtained by using Eqs. (1 & 5). This set was adopted in the benchmark analysis mentioned Section 2 above.

Set B: Young's moduli of soils were a half of those in Set A.

Set C: Young's moduli of soils were twice those in Set A.

In these analyses, the finite element model, other input parameters of soils and structures are completely the same as those in the benchmark analysis, respectively. The reference envelopes obtained are shown in Figure 6.

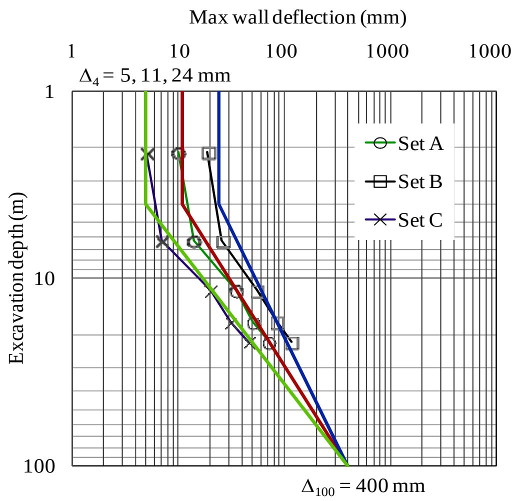


Figure 6 Reference envelopes for various ground stiffnesses

From Figure 6, values of Δ4 are 11, 24 and 5 mm for Sets A, B and C are shown, respectively. They are roughly inversely proportional to the Young's moduli of soils. The main reason could be relating to the fact that soils in the first excavation stage are essentially linear elastic materials because their strains are very small in this stage. Inversely, at this depth of excavation, soil is in the plasticity so values of Δ100 are insensitive to the ground stiffness. The values of Δ100 are the same for the three sets, i.e. 400 mm; this means that soil stiffness can govern the wall displacements at the shallow stage, but once excavation goes deeper, soil becomes plastic, which elastic modulus of soil may not control the wall displacements any more. It becomes evident again, that the reference envelopes are mainly established on the data of excavation depths in the range of 10 m to 20 m.

3.2. Reference envelopes for various wall stiffness

To investigate how wall stiffness or thickness affects the reference envelope, numerical analyses were performed for various wall thicknesses of 0.6 m, 1.0 m and 1.5 m. Figure 7 presents the reference envelopes of various wall thicknesses. Figure 7 presents that the values of Δ100 are 200, 400 and 800 mm for wall thicknesses of 0.6, 1.0 and 1.5 m, respectively. On the contrary, values Δ4 are the same for all of those wall thicknesses, i.e. 11 mm. It implies that the value Δ4 is not sensitive to the wall thickness. It is thus, verifies that thicker wall can reduce the wall deflections occurring at the deeper stages but not for the shallow stage.

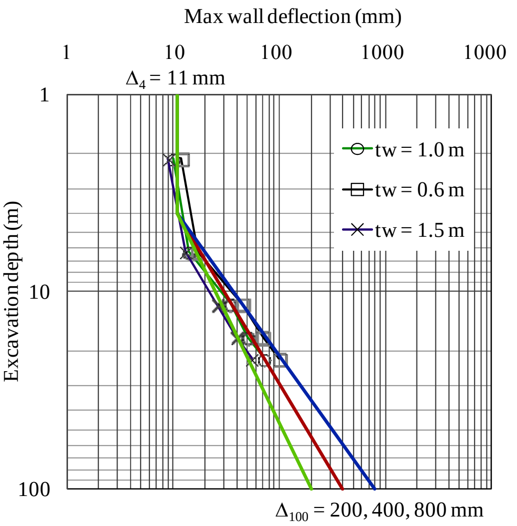


Figure 7 Reference envelopes for various wall stiffness

3.3. Reference envelopes for various excavation widths

To investigate how excavation width affects the reference envelope, numerical analyses were performed for various excavation widths of 10 m, 22.7 m and 40 m. Figure 8 below shows the reference envelopes of various excavation widths. It is seen from Figure 8 that values of Δ4 are 6, 11 and 15 mm for excavation widths of 10, 22.7 and 40 m, respectively. On the contrary, values Δ100 are the same for all of those excavation widths, i.e. 400 mm. It can be thus concluded that the effect of excavation width on reference envelope is similar to that results of soil stiffness.

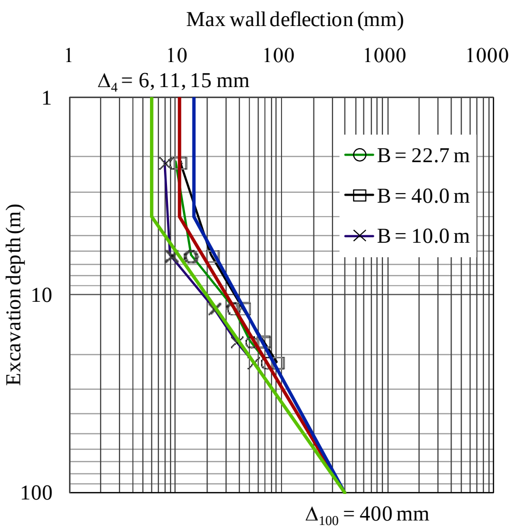


Figure 8 Reference envelopes for various excavation widths

3.4. Reference envelopes for various preloads of struts

To investigate how preload of struts affects the reference envelope, numerical analyses were carried out for various sets of preloads of struts as follows:

Set A: Steel struts are not preloaded. This set is the same as the benchmark analysis mentioned Section 2 above.

Set D: Steel struts are preloaded to about 25% of their yield loads, respectively. The yield strength of steel is 250 MPa. Thus, preloads per steel strut are 1250, 2600 and 3300 kN for Level 2, 3 and 4 of struts, respectively.

Set E: Preloads of steel struts are twice those of Set D.

The reference envelopes for various preloads of struts obtained are shown in Figure 9. It is seen from Figure 9 that values of Δ100 are 400, 200 and 150 mm for Sets A, D and E, respectively. On the contrary, values Δ4 are the same for all of the three sets, i.e. 11 mm. Since there is no steel strut installation in the shallow excavation stage, adding the extra preloads is not eligible to reduce the wall deflection in this stage, but impact could become significant once steel struts are installed in this stage.

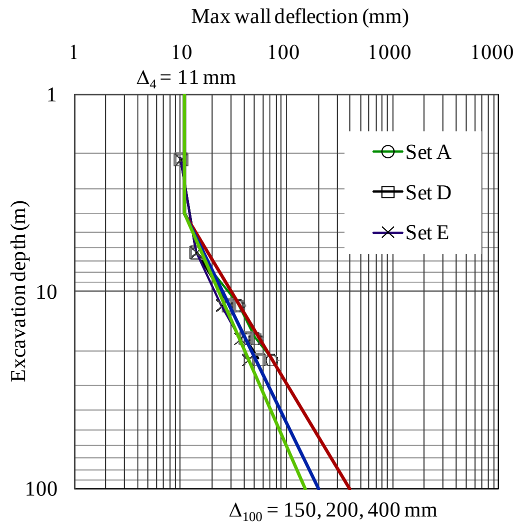


Figure 9 Reference envelopes of various preloads of struts

4. DISCUSSIONS

Firstly, "Class A" prediction is presented in this study as this prediction is delivered before the excavation. Therefore, it is valuable to collect reliable observed data later on for confirming the outcome from the prediction.

As previously indicated, limited soil tests were done to interpret soil parameters used for analyses in this study. Therefore, discussions can be carried out for the reliability of test results as well as used parameters of soils. Terzaghi et al. (1996) has addressed that the quality of soil sample can be categorized by the sample quality designation (SQD). As presented in Table 5, SQD is defined by volumetric change, in which Level A is the best, Level E is the worst, and Level B shall be considered as an acceptable sample for the delivery of further high-quality laboratory test.

Table 5 SQD level of sampling

SQD Level	Volumetric change (%)
A	<1
B	1-2
C	2-4
D	4-8
E	>8

It was found that most soil samples could reach Level C or worse of SQD from a project in one of emerging countries in Southeast Asia. However, SQD could be much improved once adequate sampler used, such as Mazier Triple Tube Sampler with proper site supervision. Further, as shown in Figure 10, the tape of thin tube sampler shall be rounded, but a tube with twisted tape is still used for having samples. This shall affect the quality of soil sample.

Figure 10 Thin tube sampler with twisted tape

Important properties of soil, such as effective friction angle, undrained shear strength, elastic modulus, K_0 and even permeability etc., can affect the predicted behaviour of ground, but unfortunately detailed and more advanced tests of these properties can't be found in related literatures. Furthermore, by using the example at top of Layer 4 in the excavation mentioned above, the ratio of Su to consolidation pressure is equal to approximately 1.0, which it seems to be much higher than the same ratio concluded by Terzaghi et al. (1996) for normal consolidated or slightly over-consolidated clay. In the study of Orazalin et al. (2015), the relationship between shear modulus of soil and its effective stress is interpreted as one of inputs. It is thus suggested that high-quality in-situ tests, such as CPT or pressure-meter tests and lab tests, shall be carried out in the future since these tests could provide the direct feedback of soil parameters used for geotechnical design and analysis.

Ou (2006) indicated that the ratio of the maximum lateral wall displacement to the excavation depth shall be in the range of 0.2% to 0.5%. The prediction made in this study is still in the said range.

Reference envelopes can be recognized as a handy tool once prediction of the maximum lateral wall displacement is needed Taking the final excavation levels similar to this study (approximately 20 m), it can be understood that the maximum wall displacement can reduce 2/3 times of original value once the elastic moduli of soils are increased by 50%. On the contrary, the same value can be increased up to approximately 67% when the elastic moduli of soils are decreased by 50%. However, the soil could easily become plastic once its stiffness is reduced, and then the related displacements are increased a lot, though same percentage of soil stiffness is increased and decreased.

At the similar strain level of soil induced by deep excavation, Yong (2015) suggested that effective elastic modulus of soil can be assumed to be 4 times of SPT- N value with the unit of MPa. It is assumed to be 2 times of SPT- N value for effective elastic modulus of soil in this study. As a result, the actual stiffness of soil can be under-estimated, and the predicted lateral wall displacements are thus anticipated to be smaller than real values. However, this prediction has to be confirmed by later observations.

No matter what the soil stiffness is, all three reference envelopes moves to one certain point when the excavation goes to very deep, and this may be due to the fact that all soils become failure or almost failure at that time, and thus the wall displacements become the same.

In Figure 7, impact on reference envelopes from wall thickness is examined. It shows that wall thickness has very limited influence for displacements occurred at shallow excavation stage. It is because soils may be still in elastic, and thus the magnitude of soil displacements are fully controlled by elastic modulus of soil, nothing related to wall thickness. However, stress and strain level of soils continue to increase, and soil may become yielding at the end once the excavation goes deeper, and this gives more and more difference in maximum lateral wall displacement once different dimension walls are adopted.

Although changing excavation width may have similar impact with changing soil stiffness on reference envelope, it may not be consistent with engineering practice as in general, necessary kingposts are usually installed once excavation width is so large in order to reduce so called "unsupported span". However, such influence from the installation of kingposts is not included in this study and may have to be delivered later on.

Finally, the influence from the pre-stress of strut is evaluated too. No strut is installed until the excavation depth reaches 6.3 m. Hence, no significant impact is seen in reference envelope for shallow excavation. It is seen that the preload can limit the generation of lateral wall displacement at deeper excavation, but it still needs further observations for verification.

5. CONCLUSIONS

The following conclusions can be drawn:

(1) Due to limitations in both soil tests and monitoring data, only a before-event prediction of ground behaviour of the deep excavation associated with a future case in Central Ha Noi is carried out in this study. Observations during the construction stage have to be collected to confirm some uncertainties and assumptions in this study. The ooutcomes from the current prediction shows that up to 72 mm of maximum lateral wall displacement can be reached once the excavation reaches 21.9 m.

(2) Reference envelope of deep excavation can be used as a helpful tool to evaluate lateral wall displacement at a certain excavation level in a certain area with similar retaining structures and strutting. By using two-dimensional analysis, initial green reference envelopes for deep excavations in Central Ha Noi have been developed in this study.

(3) Limited soil tests were done to interpret soil parameters used for analyses. Therefore, discussions can thus be carried out for the reliability of test results as well as used parameters of soils. It is thus suggested that high-quality in-situ and lab tests shall be carried out in the future since these could provide the direct feedback of soil parameters used for geotechnical design and analysis.

(4) Influence from change of soil stiffness is explored, and it is found that the impact on lateral wall displacement from reducing soil stiffness tends to be greater than increasing soil stiffness. It is suspected that soil can become plastic easier once its stiffness is reduced, and then the related displacements are increased a lot.

(5) Changing wall thickness has very limited influence for displacements occurred at shallow excavation stage. It is because soils may be still in elastic, and thus the magnitude of soil displacements are fully controlled by elastic modulus of soil, nothing related to wall thickness. However, stress and strain level of soils continue to increase, and soil may become yielding at the end once the excavation goes deeper, and this gives more and more difference in maximum lateral wall displacement once different dimension walls are adopted

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