Prediction of Ground Surface Settlements Caused by Deep Excavations in Sands

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ABSTRACT: The objective of this paper is to evaluate the applicability of a simple method for predicting movements, especially the ground surface settlements, caused by deep excavations in sands. A case history of deep excavation in thick layers of sand in Kaohsiung, Taiwan was adopted as a basis for numerical analyses. In order to improve the inconsistence in prediction of ground surface settlements induced by the deep excavation, the analysis using the simple constitutive model but with additional two factors, α and β is applied. The factor α defines the width of primary strain zone behind the retaining wall, and β indicates the difference of soil stiffness in two zones of the primary strain zone and small strain zone. It is concluded that changing α seems not to induce significant change, and values of β from 3 to 5 shall be taken once such approach intends to be adopted for predicting ground surface settlements caused by deep excavations in sands.

KEYWORDS: deep excavation; sand; numerical analysis; ground surface settlement.

1. INTRODUCTION

Deep excavations are often located very close to existing buildings in urban areas. As a result, they frequently cause unexpected movements, which can influence the safety of adjacent buildings. Movements of the retaining wall and surrounding ground caused by deep excavations have been studied by many researches, for example Peck (1969), Clough and O'Rourke (1990), Ou *et al.* (1993), Hsieh and Ou (1998), Hsieh *et al.* (2003), Ou (2006), Leung and Ng (2007), Kung *et al.* (2009), Hsiung (2009), Lim *et al.* (2010), Wang *et al.* (2010) and Ou and Hsieh (2011), Likitlersuang *et al.* (2013), Khoiri and Ou (2013), Ng *et al.* (2013), and Koh and Chua (2014). However, researches delivered regarding deep excavations in sands are comparatively limited.

Nowadays, finite element analyses have been commonly used to estimate the behavior of the ground caused by deep excavations. According to previous researches, such as Hsieh *et al.* (2003), Ou (2006), Brinkgreve *et al.* (2006), Kung *et al.* (2009), and Ou and Hsieh (2011), the finite element analysis that only uses a basic constitutive model of soil is difficult to yield an accurate prediction of ground surface settlements caused by deep excavations. A better prediction of the ground surface settlements can be obtained as an advanced constitutive model of soil that takes into account small strain characteristics of soil is adopted in the numerical analysis. However, input parameters of such advanced models of soil are often not available or have to be derived from complex test procedures in comparison with conventional tests.

This paper aims to evaluate the applicability of a simple method for predicting movements, especially the ground surface settlements, induced by deep excavations in sands. The main advantage of this method is that numerical analyses can obtain appropriate results of the ground settlements by only using a basic constitutive model of soil. A case history of deep excavation in thick layers of sand was adopted as a basis for the numerical analyses in this study. The commercial software PLAXIS 2D, version 9 (2009) was used as a numerical tool for two-dimensional finite element analyses in this paper. The results of this study can be helpful to engineers and researchers in using numerical analyses to estimate the ground settlements caused by deep excavations in sands.

2. A CASE HISTORY OF DEEP EXCAVATION

A case history of deep excavation in Kaohsiung, Taiwan was used as a basis for numerical analyses of this study. The excavation was adopted as the basement of a 15-floor building and located in the central area of Kaohsiung City, next to the O7 Station, which is on the Orange Line of Kaohsiung MRT system.

The shape of the excavation was rectangular with length of 70 m and width of 20 m. The pit was carried out using the bottom-up construction method and was retained by the diaphragm wall that is 0.9 m thick and 32 m deep. It was excavated to the maximum depth of 16.8 m with five stages of excavation. The retaining wall was propped by steel struts at four levels, and the horizontal spacing of struts was about 5.5 m on average. Figure 1 shows the cross section and ground condition of the excavation.

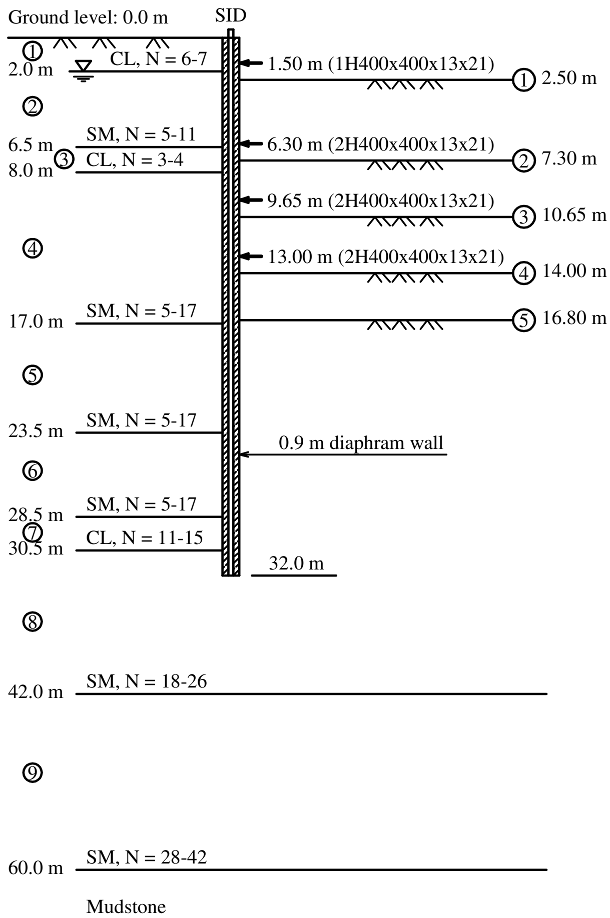

Figure 1 Cross section and ground condition of the excavation

According to the site investigation, the excavation was in the coastal plain of Kaohsiung City, Taiwan. As shown in Figure 1, because three clay layers (CL type) were very thin, their influence on the excavation behaviour is not significant. It can be thus concluded that the excavation is a typical case of deep excavations in sands.

The site investigation also reported that the groundwater level before excavation was about 2.0 m deep below the ground surface. The groundwater level inside the pit was lowered to a depth of 1.0 m below each excavation level before each stage of excavation to make a dry environment for excavation process.

The wall deflections and ground surface settlements were monitored by inclinometers and settlement observation sections during construction process of the excavation, respectively. Figure 2 below shows the wall deflections and ground surface settlements measured at the central section of long side of the excavation, in which the wall deflections were corrected to take into account the toe movements of inclinometers (see Hwang *et al.*, 2007, Hsiung and Hwang, 2009). It can be assumed that the movements of the wall and ground at the central section of long side of the excavation are in the plane strain condition because this section is 35.0 m far away from the excavation corners, and the ratio of the excavation width to length (B/L) is less than 0.3. According to studies of Ou *et al.* (1996), Wang (2012) and Yang (2013), the corner effect on a certain section decreased with increase of distance from the corner and with decrease of B/L. Wang (2012) and Yang (2013) also reported that the behaviour of a section that is 30.0 m or more far away from the corner is in the plane strain condition.

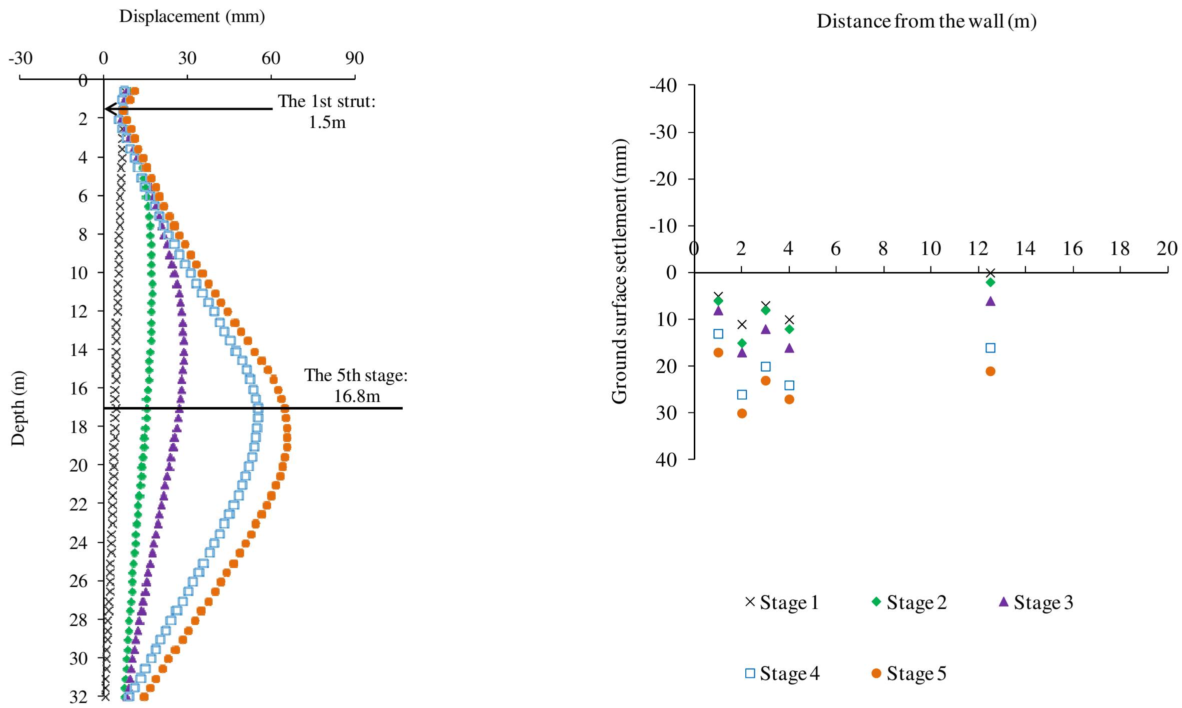

Figure 2 Wall deflections and ground surface settlements measured at the central section of long side of the excavation

As can be noted from Figure 2, the wall behaves as a cantilever at the first stage of excavation because the steel struts at the first level have not yet been installed and preloaded in this stage. The wall then displays the deep inward movements at subsequent stages of excavation. The maximum wall deflection at the final excavation stage is near the excavation level and equal to 0.39%H_e (H_e is the excavation depth). This value is thus consistent with the range of 0.2%H_e to 0.5%H_e found in the study of Ou *et al.* (1993).

The range of the monitoring settlement points behind the retaining wall was quite limited, i.e. 12.5 m, because there was a crowded traffic road next to the excavation, which causes the difficulties of settlement measurement. The maximum surface settlements (δ_{vm}) are about 21 mm to 30 mm at the final stage of excavation. The ratios of δ_{vm}/H_e are from 0.12% to 0.18%. Therefore, the ratio δ_{vm}/H_e in this study is similar to the previous study of Clough and O'Rourke (1990), in which the maximum surface settlement found was about 0.15%H_e on average for excavations in stiff clays and sands.

3. FINITE ELEMENT ANALYSES

Two finite element analyses were performed to evaluate their applicability for predicting movements, especially the ground surface settlements, induced by the excavation mentioned in Section 2 (i.e. a typical case of deep excavations in sands). The commercial software PLAXIS 2D, version 9 (2009), was selected as the numerical tool for the two-dimensional finite element analyses herein. PLAXIS 2D is a two-dimensional finite element program, which is developed at Delft University of Technology of the Netherlands and is made commercially available by PLAXIS Bv, Amsterdam, the Netherlands.

3.1 Common analysis (Analysis 1)

In this section, the finite element analysis that only uses a basic constitutive model of soil, i.e. the linear elastic-perfectly plastic Mohr-Coulomb model (MC model), was carried out to simulate the excavation. This analysis is named to be "common analysis" or "Analysis 1". Figure 3 shows the finite element model of Analysis 1. Only a half of the excavation was modelled because of its symmetrical geometry. The base of the finite element model was placed at the top of mudstone layer, i.e. at a depth of 60 m below the ground surface. The distance from the lateral boundary of the model to the retaining wall was taken to be 120 m, which is approximately seven times excavation depth as suggested by Khoiri and Ou (2013) for deep excavations in sands. The horizontal movement was restrained for the lateral boundaries, and both vertical and horizontal movements were restrained for the bottom boundary of the model.

The MC model is a basic constitutive model of soil, and it represents a first-order approximation of soil behaviour. This model assumes the stress-strain relation to be linear elastic-perfectly plastic, and its failure criterion is Mohr-Coulomb's failure criterion. The slope of the linear elastic phase of stress-strain curve is defined as

Young's modulus, and the perfectly plastic phase is obtained when the stress states reach the Mohr-Coulomb's failure criterion. This model is often used for preliminary analyses of the considered problem. The computations with the MC model are relatively fast because only constant average stiffness of each soil layer is computed, and the behaviour of stress-dependent stiffness of soil is not considered. The MC model involves six input parameters, i.e. the internal friction angle, cohesion, Young's modulus, Poisson's ratio, dilatancy angle, and coefficient of lateral earth pressure at rest.

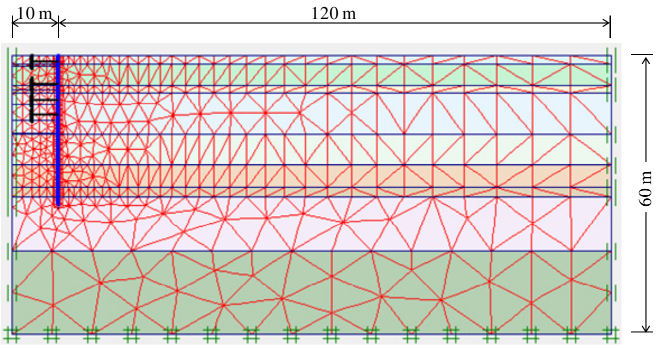

Figure 3 Finite element model of Analysis 1

The sand layers (SM type) were assumed to be drained materials with effective strength parameters, and the clay layers (CL type) were assumed as undrained materials with total strength parameters. For the sand layers, the effective friction angle (ϕ') was directly obtained from laboratory tests. Values of effective cohesion (c') were assumed to be zero, but to avoid complication for calculation of PLAXIS software, a very small value c' = 0.5 kPa was set for sand layers. The drained Poisson's ratio (ν') was assumed to be 0.3 for sands as suggested by PLAXIS 2D (2009), Khoiri and Ou (2013). As reported by Hsiung (2009) for a deep excavation in sands, the effective Young's modulus (E') of sand layers could be obtained by the following equation.

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

in which N is blow counts in Standard Penetration Tests (SPT). As proposed by Bolton (1986), the dilatancy angle of 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 the input parameters of sand layers for the MC model used in Analysis 1.

Table 1 Input parameters of sand layers for the MC model

Layer	Depth (m)	Soil type	γ_t (kN/m^3)	N value	ϕ' (°)	c' (kPa)	E' (kPa)	ν'	ψ (°)	K_0
2	2.0-6.5	SM	20.9	5-11	32	0.5	16000	0.3	2	0.47
4	8.0-17.0	SM	20.6	5-17	32	0.5	22000	0.3	2	0.47
5	17.0-23.5	SM	18.6	5-17	32	0.5	22000	0.3	2	0.47
6	23.5-28.5	SM	19.6	5-17	33	0.5	22000	0.3	3	0.46
8	30.5-42.0	SM	19.6	18-26	34	0.5	44000	0.3	4	0.44
9	42.0-60.0	SM	19.9	28-42	34	0.5	70000	0.3	4	0.44

For the three clay layers, which are modelled by the total stress undrained 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 analysis. Undrained Poisson's ratio $\nu_u = 0.495$ (≈ 0.5) was adopted to simulate the incompressible behavior of water and to avoid numerical problems caused by an extremely low compressibility of water (i.e. singularity of the stiffness matrix). According to previous researches of Bowles (1996), Lim *et al.* (2010), Likitlersuang *et al.* (2013), Khoiri and Ou (2013), undrained Young's modulus E_u can be calculated by the empirical equation as follows:

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

Table 2 shows the input parameters of clay layers for the MC model used in Analysis 1.

Table 2 Input parameters of clay layers for the MC model

Layer	Depth (m)	Soil Type	γ_t (kN/m^3)	S_u (kPa)	E_u (kPa)	ν_u
1	0.0-2.0	CL	19.3	28	14000	0.495
3	6.5-8.0	CL	19.7	21	10500	0.495
7	28.0-30.5	CL	18.6	84	42000	0.495

The diaphragm wall was simulated by plate elements, and the steel 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 steel struts. The Young's modulus of the diaphragm wall 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 concrete. The Young's modulus of steel struts was taken to be 2.1x10^5 MPa. The stiffness of both the diaphragm wall and steel struts was reduced by 30% and 40% from their nominal values, respectively, to consider the cracks in the diaphragm wall due to bending moments and to consider the repeated uses and improper installation of steel struts as suggested by Ou (2006). Tables 3 and 4 present input parameters of the diaphragm wall and steel struts used in the common analysis. The weight of plate is obtained by multiplying the unit weight of plate by the thickness of plate. It is noted that the unit weight of plate was subtracted 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), 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.

Table 3 Input parameters of diaphragm wall

Parameter	Name	Value	Unit
Compressive strength of concrete	f'_c	28	MPa
Young's modulus	E	$24.8x10^6$	kPa
Thickness	d	0.9	m
Axial stiffness x 70%	70%EA	$15.66x10^6$	kN/m
Flexural stiffness x 70%	70%EI	$1.057x10^6$	kNm^2/m
Weight	w	4.95	kN/m/m
Poisson's ratio	ν	0.2	

Table 4 Input parameters of steel struts

Strut level	Strut level	Preload (kN)	Section area (m^2)	EA (kN)	60%EA (kN)
1	1H400x400x13x21	900	0.0219	$4.59x10^6$	$2.75x10^6$
2	2H400x400x13x21	2000	0.0437	$9.18x10^6$	$5.50x10^6$
3	2H400x400x13x21	2800	0.0437	$9.18x10^6$	$5.50x10^6$
4	2H400x400x13x21	2800	0.0437	$9.18x10^6$	$5.50x10^6$

3.2 Analysis with additional parameters (Analysis 2)

According to previous researches, such as Hsieh *et al.* (2003), Ou (2006), Brinkgreve *et al.* (2006), Kung *et al.* (2009), Ou and Hsieh (2011), the finite element analysis that only uses a basic constitutive model of soil, as the common analysis mentioned in Section 3.1, is often difficult to give a proper prediction of ground surface settlements induced by deep excavations. With using such analyses, in general, the settlements in the primary influence zone (PIZ) are underestimated while the settlements in the secondary influence zone (SIZ) are overestimated. The concepts of PIZ and SIZ were discussed in the previous studies of Hsieh and Ou (1998), Hsieh *et al.* (2003), Ou (2006), Wang *et al.* (2010), Ou (2011), Lin *et al.* (2011), Likitlersuang *et al.* (2013). The main reason of this restriction is that the basic constitutive models of soil cannot take into account the small strain characteristics of soil. The most important characteristic of soil at small and very small strain levels is that soil stiffness at these strain levels is much greater than that at conventional engineering strain levels. To achieve good predictions of both the wall deflections and ground surface settlements, an advanced constitutive model of soil, which can consider the small strain characteristics of soil, needs to be adopted in the numerical analyses of deep excavations. However, input parameters of the advanced models are often not available or can only be derived from complex test procedures in comparison with conventional tests.

To avoid the complexity in using the advanced models, Ou (2006) first proposed a simple method to estimate the ground surface settlements caused by deep excavations in clays. This method only uses a basic constitutive model of soil, but it can consider the small strain characteristics of soil. With this method, the ground of the excavation was divided into two zones of the primary strain zone (PSZ) and the small strain zone (SSZ) as shown in Figure 4. In Figure 4, the PSZ is the marked area, and the rest is the SSZ. Furthermore, Young's modulus of each soil layer in the small strain zone was increased by three times compared to that in the primary strain zone, respectively.

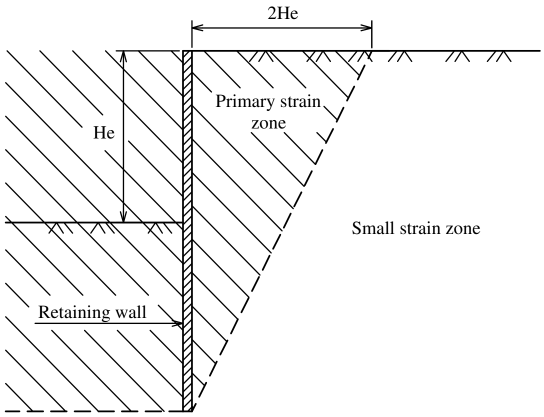

Figure 4 Two strain zones of an excavation in clays according to the simple method of Ou (2006)

Considering interpretation from both field observations and numerical analyses, Ou (2006) further indicated that strains of soil in the SSZ are from 0.1% to 0.01% at the final stage of excavation, which are in small strain levels. Output of soil strain induced by the excavation from Analysis 1 are thus further examined, and it was found that by varying the width of PSZ behind the wall from $0.5H_e$ to $4.0H_e$, the SSZ may be covered reasonably. Further, as indicated by Benz (2007), the stiffness of sands at small strain levels can be from 2 to 5 times that at conventional strain levels.

This section illustrates how the finite element analysis that uses the simple method was carried out to model the excavation. This analysis uses the same constitutive model of soil as Analysis 1, i.e. the MC model. All input parameters of soil in the primary strain zone, input parameters of structures and boundary conditions were taken completely the same as those in the common analysis, respectively. This analysis is named "analysis with additional parameters" or "Analysis 2". It is evident that two additional parameters, which influence results of Analysis 2, are the width of the primary strain zone behind the retaining wall and Young's modulus of soils in the small strain zone. A series of parametric studies was carried out to evaluate the effects of the two parameters on predicting movements, especially the ground surface settlements, caused by the deep excavation. Based on statements of the width of PSZ and small strain stiffness of sands above, in these parametric studies, the ratio of the width of primary strain zone behind the retaining wall to the excavation depth is defined as a parameter "α" and is varied in values of 0.5, 1.0, 1.5, 2.0, 2.5, 3.0, 3.5 and 4.0. Ratio of Young's modulus of each soil layer in the small strain zone to that in the primary strain zone, respectively, is defined as a parameter "β" and is varied from 1.5, 2.0, 2.5, 3.0, 3.5, 4.0, 4.5 to 5.0. Figure 5 presents the finite element model of Analysis 2.

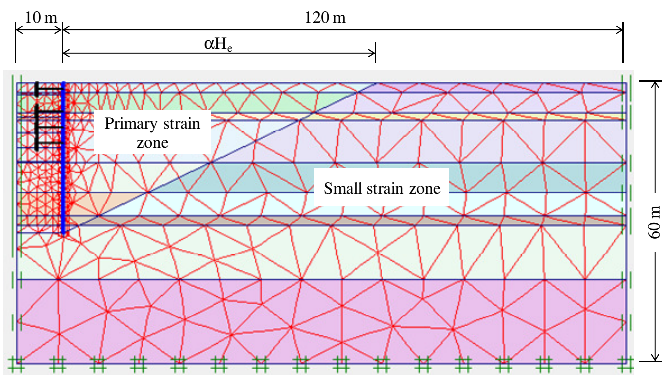

Figure 5 Finite element model of Analysis 2

4. RESULTS AND DISCUSSIONS

4.1 Wall deflections

Figure 6 shows the comparisons of the wall deflections obtained from the field measurements, Analysis 1 and Analysis 2 at various stages of excavation. In this figure, only selected data of the two stages of excavation, i.e. Stage 1 and Stage 5, are presented due to very large amount of analytical data produced. It is noted that "A1" means "Analysis 1", and "A2 (x, y)" indicates "Analysis 2 with value pair of (α=x, β=y)" in Figure 6.

It is clearly seen from Figure 6 that the wall deflections obtained from Analysis 2 are closer to the field measurements rather than those obtained from Analysis 1 at most of stages of excavation. It is also observable that the wall deflections of Analysis 2 are much influenced by the parameter of β but less influenced by the parameter of α. The lateral wall displacements obtained from Analysis 2 with values of $\beta \geq 3.0$ are slightly larger than the field measurements at the first stage of excavation (Stage 1) and are very close to the field measurements at the final stage of excavation (Stage 5), respectively.

For the final stage of excavation, the lateral wall movements of Analysis 1 are consistent with the field measurements at the upper wall parts but are much larger than those at the lower wall parts, especially at the wall toe. The main reason can be related to the fact that the MC model adopted in Analysis 1 uses only a single Young's modulus, which does not distinguish between loading and unloading stiffness of soil. This feature of the MC model causes an over-prediction of the excavation bottom heave because the higher unloading stiffness of ground below the excavation level is not considered. This over-prediction of the excavation bottom heave then causes larger wall deflections at the lower wall parts as mentioned above. By using the simple method in Analysis 2 and values of $\beta \geq 3.0$, the large movements of the wall toe are not found in the results of Analysis 2. It is because higher stiffness of soil below the excavation level was computed through the parameter β of the simple method.

For the first stage of excavation, the wall deflections of Analysis 1 are all larger than the field measurements, respectively. It is because the MC model does not take into account the small strain characteristics of soil that involve higher stiffness of soil at small strain levels in the first stage of excavation. It is thus concluded that the effective Young's modulus of sands calculated by Eq. (1), which is adopted for the MC model in Analysis 1, is underestimated at the earlier stages of excavation but is reasonably estimated at the final or critical stage of excavation.

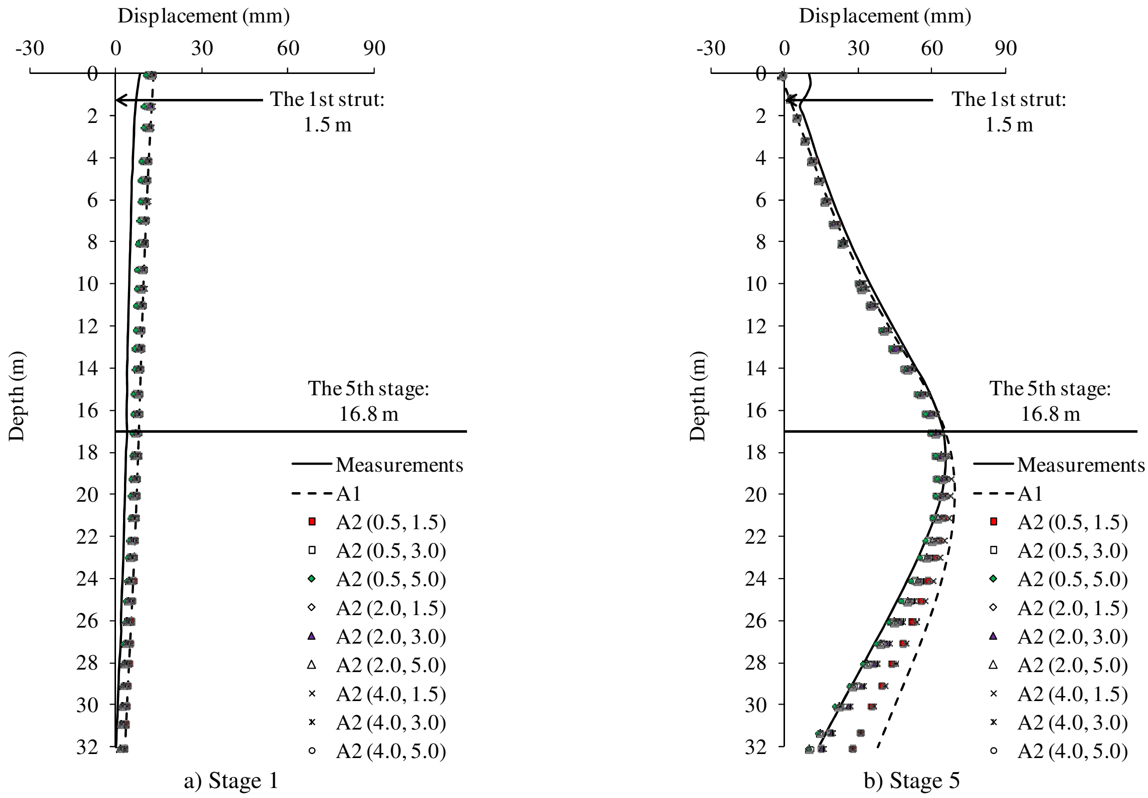

Figure 6 Comparisons of the wall deflections obtained from the field measurements, Analysis 1 and Analysis 2

4.2 Ground surface settlements

Figure 7 presents the comparisons of the ground surface settlements obtained from the field measurements, Analysis 1 and Analysis 2 at various stages of excavation. Similar to the wall deflections, only selected data of Stage 1 and Stage 5 of excavation are shown in Figure 7. It is also noted that "A1" means "Analysis 1", and "A2 (x, y)" indicates "Analysis 2 with value pair of (α=x, β=y)" in Figure 7.

From Figure 7, in general, it is seen that the shape and magnitude of ground surface settlements obtained from Analysis 1 and Analysis 2 are very different at most of stages of excavation, especially at the final stage of excavation. For the first stage of excavation, the settlements of Analysis 2 are all smaller than those of Analysis 1. For the final stage of excavation, the settlements in the PSZ of Analysis 2 are greater than those of Analysis 1, but the settlements in the SSZ of Analysis 2 are smaller than those of Analysis 1. In addition, a heave area of ground next to the wall is also seen in results of Analysis 1 in the final of excavation, which is not consistent with the field observations and very unrealistic. By using the simple method, the heave area is not found in results of Analysis 2 with values of $\beta \geq 3.0$.

According to the empirical methods for excavations in sands of Peck (1969), Bowles (1996), Clough and O'Rourke (1990), the ground surface settlements behind the wall can be taken as zero for positions that are far away from the wall a distance more than about $2H_e$. It can be thus concluded that Analysis 1 underestimates the settlements in the PSZ and overestimates the settlements in the SSZ whereas Analysis 2 are more reasonable in the prediction of ground surface settlements. The main reason for better prediction of ground surface settlements of Analysis 2 may be related to the fact that the higher stiffness of soils in the SSZ is calculated through the parameter β of the simple method. This higher stiffness decreases the vertical and horizontal movements of soils in the SSZ and heave of the excavation bottom, which then decreases the settlements in the SSZ but increases the settlements in the PSZ. In the contrary, because Analysis 1 does not take into account the higher stiffness of soils in the SSZ, the settlements in the SSZ and heave of the excavation bottom are overestimated in Analysis 1. The over-prediction of the excavation bottom heave then pushes the wall and surrounding ground up. Thus, the unrealistic heave of ground surface settlements next to the wall is seen in results of Analysis 1.

By varying the parameters of α and β, different profiles of the ground surface settlement can be obtained from Analysis 2. Therefore, the best value pair of (α, β) can be found by comparing the ground surface settlements predicted with those of the field measurements and previously empirical methods. Unfortunately, the field measurements of the ground surface settlement of the excavation are very limited, i.e. there is a narrow range of settlement observation. Thus, it is not highly feasible to find the best value pair of (α, β) used in Analysis 2 for the excavation. However, it is presented in this paper that values of $\beta \geq 3.0$ can have more reasonably predicted results, and previous literature shows that value of β can be up to 5. It is thus suggested that values of β from 3 to 5 shall be taken once such approach intends to be adopted for predicting ground surface settlements caused by deep excavations in sands.

4.3 Results of parametric studies

For evaluating the overall influence of the parameters of α and β adopted in Analysis 2 on the wall deflections and ground surface settlements induced by deep excavations in sands, Figures 8, 9, 10 and 11 show the results obtained from Analysis 2 with various values of α and β. In these figures, δ_{hm1}, δ_{ht1}, δ_{vm1} and δ_{vb1} are the values of the maximum wall deflection, wall deflection at the wall toe, maximum ground surface settlement, and ground surface settlement at the model boundary, respectively, obtained from Analysis 1 at the final stage of excavation. The δ_{hm2}, δ_{ht2}, δ_{vm2} and δ_{vb2} are the same terms of Analysis 2.

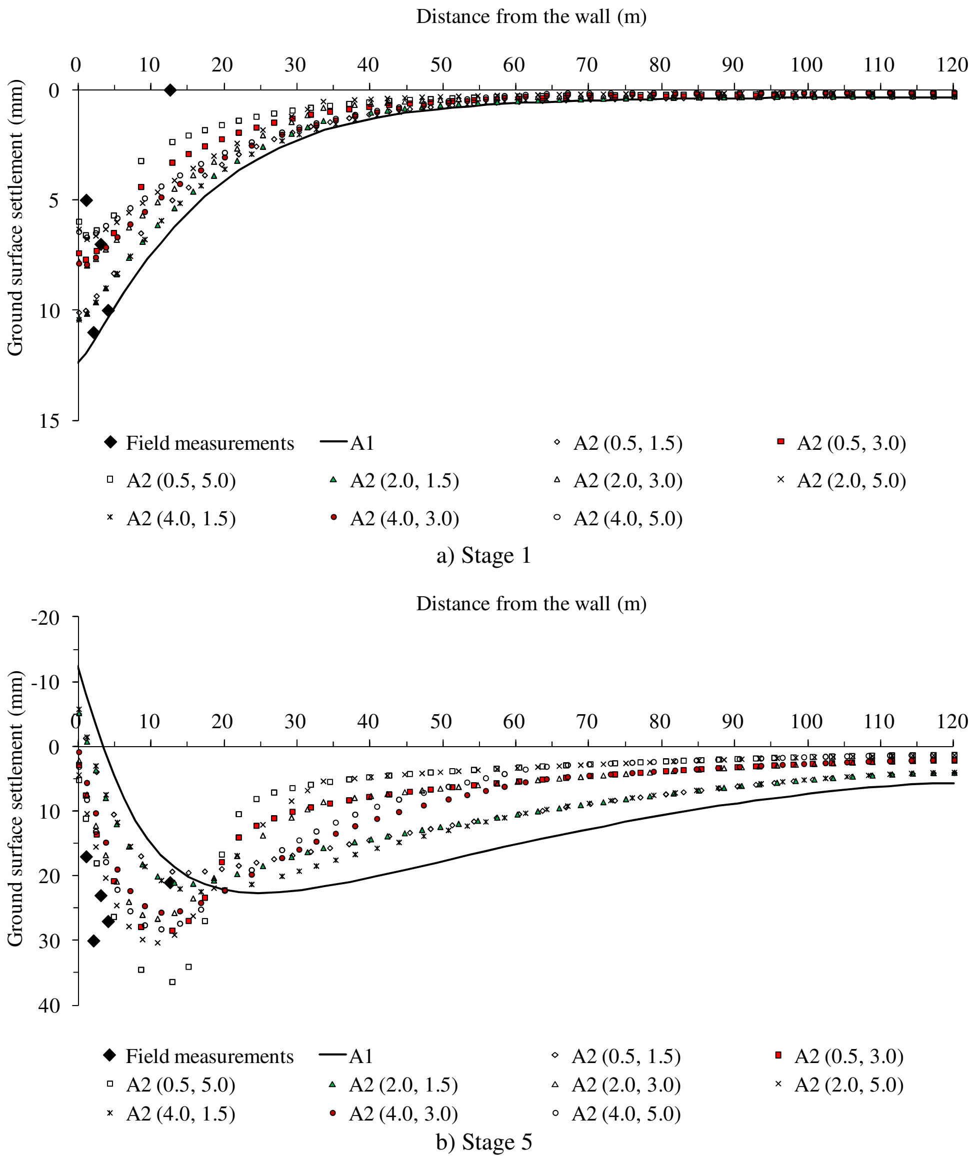

Figure 7 Comparisons of the ground surface settlements obtained from the field measurements, Analysis 1 and Analysis 2

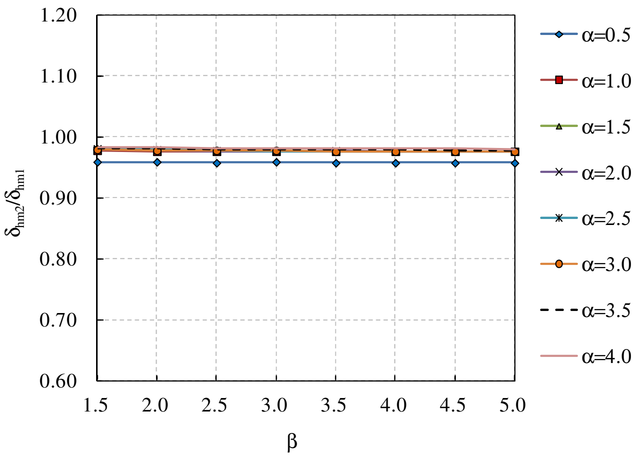

Figure 8 Relationship between $\delta_{hm2}/\delta_{hm1}$ and β with various values of α

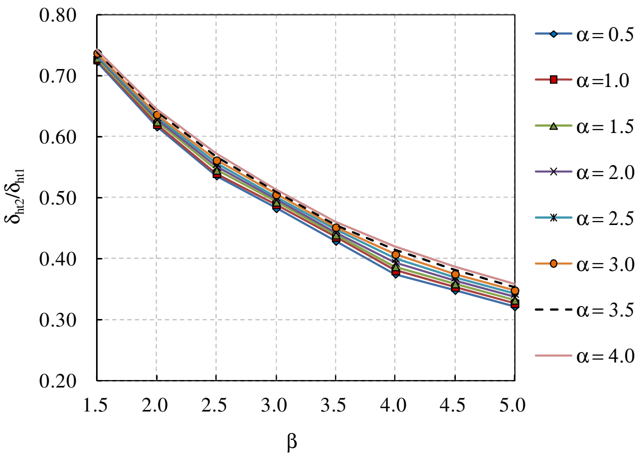

Figure 9 Relationship between $\delta_{ht2}/\delta_{ht1}$ and β with various values of α

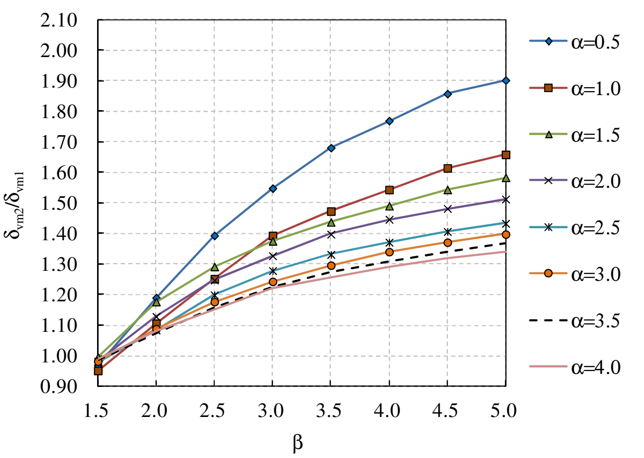

Figure 10 Relationship between $\delta_{vm2}/\delta_{vm1}$ and β with various values of α

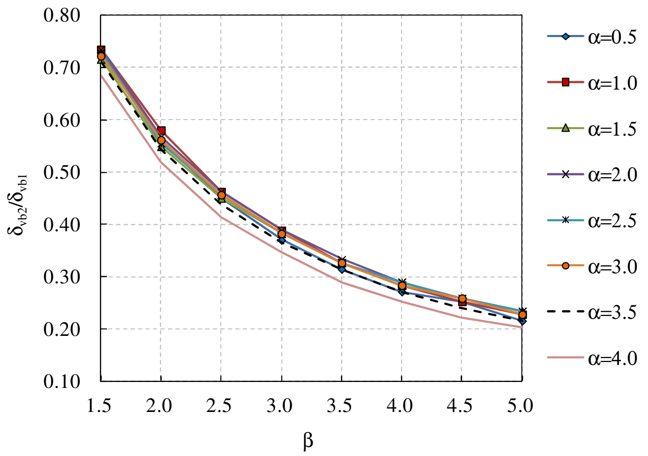

Figure 11 Relationship between $\delta_{vb2}/\delta_{vb1}$ and β with various values of α

From Figures 8, 9, 10 and 11 above, several conclusions can be drawn as follows:

(1) Ratio $\delta_{hm2}/\delta_{hm1}$ varies very little with the variations of both α and β. It is in the range of 0.96 to 0.98. Therefore, it can be concluded that the use of the simple method does not make a significant change in the aspect of the maximum wall deflection prediction.

(2) Ratio $\delta_{ht2}/\delta_{ht1}$ varies in the range of 0.32 to 0.74 and decreases gradually with the increase of β at every value of α. It varies slightly with the variation of α at every value of β. It is thus implied that higher β can significantly decrease the movements of the wall toe, but the change of α can only lead to a minor influence on said movements.

(3) Ratio $\delta_{vm2}/\delta_{vm1}$ increases gradually with the increase of β at every value of α, but the increasing rate of $\delta_{vm2}/\delta_{vm1}$ decreases gradually with the increase of α. It can be thus concluded that the predicted maximum settlement becomes larger when the analysis uses smaller α and larger β.

(4) Ratio $\delta_{vb2}/\delta_{vb1}$ varies in the range of 0.20 to 0.73 and decreases gradually with the increase of β at every value of α. It varies very little with variation of α at every value of β. It is thus implied that higher β can significantly decrease the settlements in the SSZ, but the change of α can only lead to a minor influence on said settlements.

5. CONCLUSIONS

The following are conclusions drawn from this study:

(1) Analysis 1, which does not use the simple method, cannot well estimate the wall deflections at every stage of excavation. Analysis 1 with drained Young's modulus of sands computed by Eq. (1) reasonably estimates the wall deflections at the final or critical stage of excavation but overestimates the wall deflections at the earlier stages of excavation. Moreover, Analysis 1 still gives an over-prediction of the lateral wall displacements at the lower wall parts, especially at wall toe, at the final stage of excavation.

(2) Analysis 2, which uses two additionally simple factors α and β, can well estimate the wall deflections at every stage of excavation with values of β more than or equal to 3.0, no matter which value of α is used in the analysis.

(3) Analysis 1 underestimates the ground surface settlements in the PSZ and overestimates the settlements in the SSZ whereas Analysis 2 results in a more reasonable prediction of the ground surface settlements in both the PSZ and SSZ. The best value pair of (α, β) adopted in Analysis 2 can be found by comparing the settlements calculated with those of the field measurements and empirical methods. It is thus suggested that values of β from 3 to 5 shall be taken once such approach intends to be adopted for predicting ground surface settlements caused by deep excavations in sands.

(4) At the final stage of excavation, the influences of the parameters of α and β of the simple method on the wall deflections and ground surface settlements computed from Analysis 2 can be summarized as follows:
   - (i) Using the simple method in Analysis 2 produces an insignificant change in the aspect of the maximum wall deflection prediction.
   - (ii) Higher values of β can significantly decrease the movements of the wall toe, but the change of α can only lead to a minor influence on said movements.
   - (iii) The predicted maximum settlement becomes larger when values of smaller α and larger β are used in the analysis.
   - (iv) Higher values of β can significantly decrease the settlements in the SSZ, but the change of α can only lead to a minor influence on said settlements.

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