Mechanism of Settlement Influence Zone due to Deep Excavation in Soft Clay

Mécanisme de la zone d'influence de tassement dû à une excavation profonde dans l'argile molle

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ABSTRACT: The objective of this study is to examine the mechanism of settlement induced by deep excavation through finite element analysis. The USC model was selected for this purpose through the calibration of different soil constitutive models. A series of parametric studies were then performed. It was found that in addition to the excavation depth, excavation width, the soft clay thickness and depth to the hard soil are also related to the settlement influence zone. A simple method derived from the basal heave failure mechanism is proposed to predict the settlement influence zone. One case history and one hypothetical excavation with the 80 m thick soft clay were used to verify the proposed method. For comparison, the existing empirical formulae were also used for prediction.

RÉSUMÉ : L'objectif de cette étude est d'examiner le mécanisme de tassement induit par une excavation profonde à travers l'analyse d'éléments finis. Le modèle USC a été choisi à cet effet par le calibrage de différents modèles de sol . Une série d'études paramétriques a ensuite été réalisée. Il a été constaté qu'en plus de la profondeur et de la largeur de l'excavation, l'épaisseur et la profondeur de l'argile molle sur le sol dur sont également liées à la zone d'influence du tassement. Une méthode simple dérivée du mécanisme de rupture parsoulèvement basal est proposée pour prédire la zone d'influence du tassement. Une étude de cas et un travail d'excavation hypothétique de l'argile molle sur 80 m d'épaisseur ont été utilisés pour vérifier la méthode proposée. A titre de comparaison, les formules empiriques existantes ont également été utilisées pour la prédiction.

KEYWORDS: Deep Excavation, Soft Clay, Settlement, Constitutive Model, Settlement Mechanism

1 INTRODUCTION

The finite element method and empirical methods are often used to predict the ground settlement induced by deep excavation. The finite element method usually gives better predictions for wall deflection than for ground settlements unless small strain characteristics of soil are taken into account. Ideally, empirical methods should be able to predict ground settlements well because they are mainly derived from field observations of case histories. However, most of them yield poor prediction in ground settlement because settlement mechanism is unclear, case histories adopted is limited, and the excavation depth is the only parameter used in formulas.

The objective of this paper is to investigate the mechanism of ground settlement induced by deep excavation under the plane strain condition through finite element analysis. The study focuses on the settlement under the normal excavation condition, that is, no dewatering induced settlement, no excessively long construction duration causing the occurrence of creep, and no serious construction defects. A suitable soil constitutive model was selected through calibration process. Then a series of parametric studies were performed and the settlement mechanism is proposed.

2 CALIBRATION OF SOIL CONSTITITIVE MODELS

Since "settlement influence zone" is not rigorously defined, the authors proposed the conception of the primary influence zone (*PIZ*) and the secondary influence zone (*SIZ*) on the basis of the principles of mechanics and regression analysis of excavation case histories (Hsieh and Ou 1998). The settlement curve is steep in the *PIZ* where buildings receive more influence and in the *SIZ* the slope of the curve is gentle and its influence on buildings is insignificant. Finite element analyses are used to capture the characteristics of *PIZ*.

Four soil constitutive models including the Hardening Soil (HS) model, Hardening Soil with Small Strain (HSS) model, ϕ=0 Mohr-Coulomb (MC) model, and Undrained Soft Clay (USC) model, were adopted. Of these, the HS and HSS model are the effective stress model and the ϕ=0 MC model and USC model are the total stress model. Both the HSS model and USC model take into account that the soil exhibits high stiffness at small strain.

Though the USC model is a total stress model, it considers the variation of undrained shear strength with principal stress rotation, variation of Young's modulus with the increase of stress level, high stiffness of soil at small strain, and rational way to determine the undrained shear strength (Hsieh and Ou 2011). Similar to Duncan and Chang's model, the tangent Young's modulus (E_t) in the primary loading is derived as

$$E_t = E_{ur}(1 - R_f SL)^2 \tag{1}$$

where R_f is the failure ratio, SL is the stress level, E_{ur} is the unloading/reloading Young's modulus.

The E_{ur} should degrade with the increase of strain or stress level. The degraded Young's modulus is assumed to follow a hyperbolic function as

$$\frac{E_{ur}}{E_i} = 1 - \frac{SL - SL_i}{m + n(SL - SL_i)} \tag{2}$$

where m and n are the degradation parameters relative to the stress level, E_i is the Young's modulus at small strain, SL_i is the stress level corresponding to the threshold value of the small strain or the initial yield strain.

An elastic surface, ES, is defined to represent the small strain characteristics for the state of stress inside the elastic surface. Figure 1 shows the relationships of stress and strain and of elastic, yield and failure surfaces. Thus, A total of seven parameters are required for the USC model, i.e., s_{uc},(undrained shear strength from K_0-concolidated undrained compression test), E_i,(Young's modulus at small strain) ε_i,(threshold of small strain) R_f,(failure ratio) K_s,(ratio of the undrained shear strength from undrained shear strength from K_0-concolidated undrained compression test to that from undrained shear strength from K_0-concolidated undrained extension test) as well as m and n (degradation parameters).

The TNEC case history was used for calibration (Ou et al. 1998). Figure 2 shows the comparison of wall deflections and ground movements obtained from field observation and those from finite element analysis using different models. Except for the ϕ=0 MC model where E_u/s_u was assumed to be 400 according to the local experiences, other soil parameters such as undrained shear strength, E_{50}^{ref}, E_{ur}^{ref}, E_{oed}^{ref}, G_0^{ref} were determined from laboratory tests. $\gamma_{0.7}$ were calibrated to be 5×10^{-5}. Details of the soil parameter evaluation can be found in Lim et al. (2010). Though wall deflections can be predicted well for all models, only the USC model can yield ground settlements close to field observations (Figure 2). Moreover, a hypothetical excavation with an 80 m thick soft clay where its properties were assumed to be the same as the third soil layer of the TNEC case was used for further calibration. The USC model gives a more reasonable prediction in wall deflection and ground settlements than other three soil models (Figure 3). The USC model is thus adopted for parametric studies.

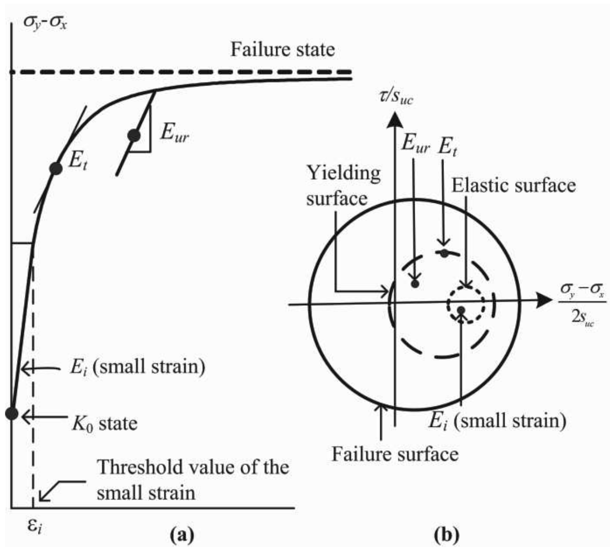

Figure 1. Concept of the USC model (a) Stress-strain behavior (b) Relationship of failure, yield and elastic surfaces.

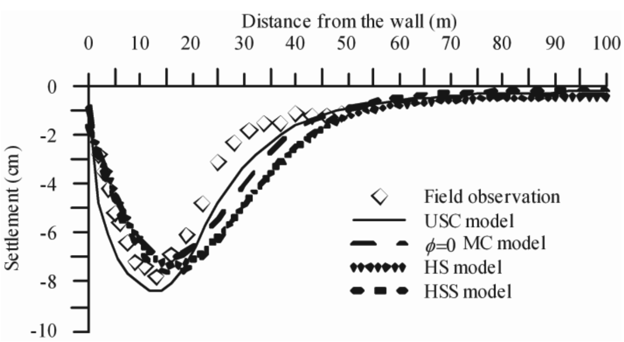

Figure 2. Comparison of settlements from field observation with those from analyses for TNEC

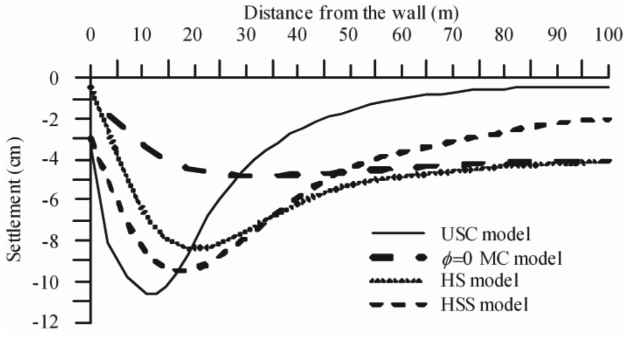

Figure 3. Comparison of settlements from various soil models for a hypothetical excavation with 80 m thick soft clay

3 PARAMETRIC STUDIES AND MECHANISM OF SETTLEMENT

A wide range of assumed excavation cases including excavation depth of 9 to 20 m, excavation width of 20 to 60 m, normalized undrained shear strength (CK_0UC) of 0.28 to 0.34, depth to hard rock of 25 to 50 m was analyzed using the USC model. A typical parametric result, variation of movements with the excavation width, is shown in Figure 4, indicating that the *PIZ* changes with the excavation width. The excavation depth, excavation depth, thickness of soil clay and depth to hard rock are all affecting the *PIZ*. Based on the parametric results, we have found the following relationship

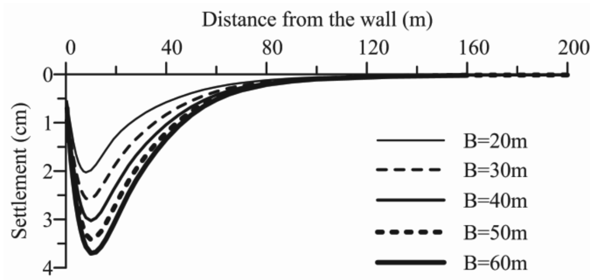

Figure 4. Variation of ground settlement with excavation width (B) for s_{uc}/σ'_v=0.3.

When the rock-like soil is very deep, i.e., H_g is very large

If $\sqrt{B^2+H_e^2} \le 2H_e$, $PIZ \approx 2H_e$

If $\sqrt{B^2+H_e^2} > 2H_e$, $PIZ \approx \sqrt{B^2+H_e^2}$

When the rock-like soil is of the limited depth, i.e., H_g is relatively small

If $\sqrt{B^2+H_e^2} \le 2H_e$,

$2H_e \le H_g$, $PIZ \approx 2H_e$; $2H_e > H_g$, $PIZ \approx H_g$

If $\sqrt{B^2+H_e^2} > 2H_e$,

$\sqrt{B^2+H_e^2} \le H_g$, $PIZ \approx \sqrt{B^2+H_e^2}$;

$\sqrt{B^2+H_e^2} > H_g$, $PIZ \approx H_g$.

The above results are summarized below:

When $\sqrt{B^2+H_e^2} > 2H_e$ (wide), $PIZ = \min(\sqrt{B^2+H_e^2}, H_g)$ (3)

When $\sqrt{B^2+H_e^2} \le 2H_e$ (narrow), $PIZ = \min(2H_e, H_g)$ (4)

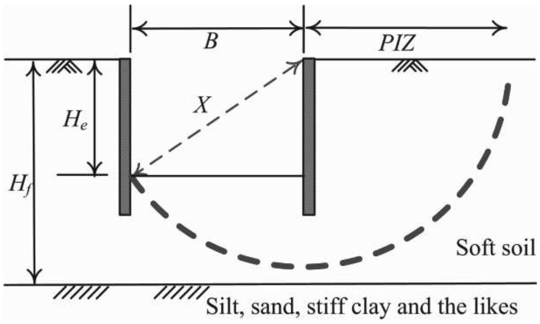

Figure 5. Basal heave failure mode and PIZ.

Comparing Eq. 3 and Figure 5 may show that the *PIZ* matches the failure zone or potential failure zone. The *PIZ* also matches the strain contours from the analysis of the TNEC excavation at stage 7 (H_e=11.8 m) and that of the plastic-points when the strength is reduced to induce basal heave (Figure 6). This is because the strain in the *PIZ* should be very large, which in turn induces a relatively large settlement. Therefore, for excavation in soft clay, the *PIZ* is assumed to be the potential basal heave zone but limited by the non-soft clay, such as silt, sand etc (Figure 5). For simplification, $(B^2+H^2)^{1/2}$ in Eq. 3 is replaced with the excavation width, *B*. Eq. 3 is thus rewritten as

$$PIZ_1 = \min(H_f, B) \tag{5}$$

where H_f is the thickness of the soft clay.

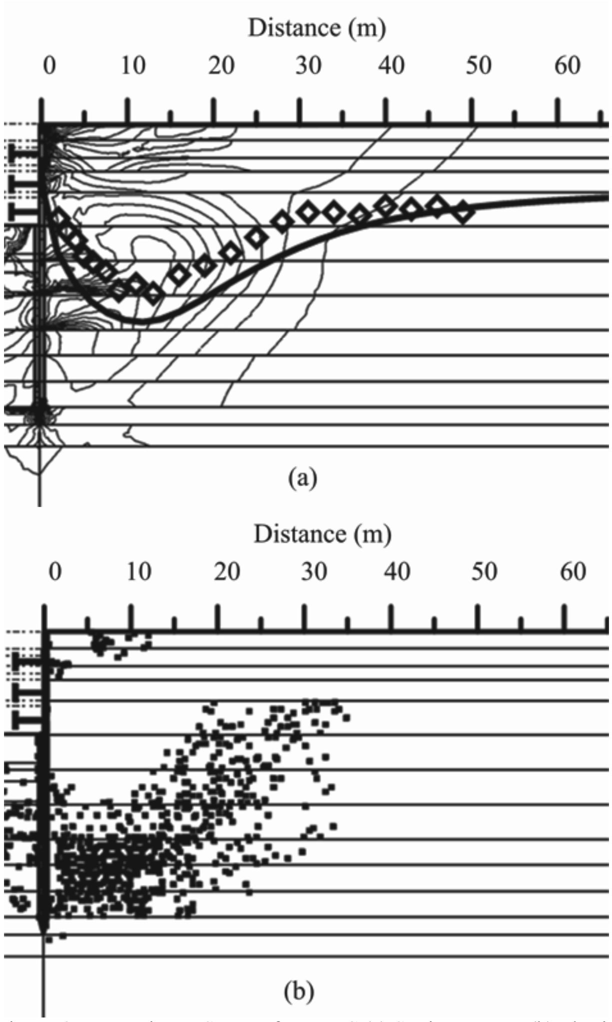

Figure 6. Excavation at Stage 7 for TNEC (a) Strain contours (b) Plastic points when the strength is reduced to cause basal heave.

The relationship in Eq. 4 indicates that the *PIZ* matches the active zone based on two times the excavation depth. This is because when excavation begins, the wall moves toward the excavation zone and the active zone also occurs behind the wall. Based on the stability analysis, the embedment depth of the wall is usually equal to the excavation depth. The *PIZ* is coincident with the active failure zone but limited by the rock-like soil. The above equation can be rewritten as

$$PIZ_2 = \min(2H_e, H_g) \tag{6}$$

where H_g is the depth of rock-like soil.

Both PIZ_1 and PIZ_2 are the failure zone or potential failure zones. Therefore, the *PIZ* is the maximum of the potential failure zones. The method for predicting concave and spandrel types of ground settlement by Hsieh and Ou (1998) is then modified, in which the *PIZ* derived in this study replaces the $2H_e$, as shown in Figure 7. Details of the derivation can be found in Ou and Hsieh (2011).

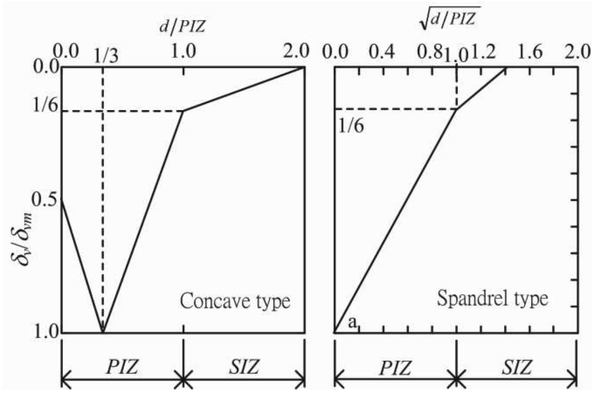

Figure 7. The proposed method for predicting the ground surface settlement.

4 VERIFICATION

The TNEC case history and the ground settlement obtained from finite element analysis of the hypothetical excavation with the 80 thick are used for verification. In the TNEC case history, at stage 5, $2H_e$=17.2 m. If the cobble-gravel soil is regarded as a rock-like soil, H_g =46 m. Concerning the active failure zone, PIZ_2=17.2 m. With the depth of the bottom of the soft clay (H_f) being 37.5 m, for the potential basal heave failure mode, PIZ_1= 37.5 m. Thus, the *PIZ* is 37.5 m. At stage 7, $2H_e$=23.6 m, H_g =46 m, PIZ_2=23.6 m; *B*=40 m, H_f =37.5 m, PIZ_1=37.5 m. Thus, the *PIZ* is 37.5 m. Similarly, the *PIZ* at the final stage ($2H_e$=39.4 m), is inferred to be 39.4 m. Figure 8 show the comparison between the proposed method (Ou and Hsieh 2011), Hsieh and Ou (1998) and Clough and O'Rourke (1990). The proposed method satisfactorily conforms to the field measurements, while those from other two methods are not.

In the hypothetical excavation with the 80 m thick soft clay, the excavation depths at stages 5, 7 and final are also 8.6, 11.8 and 19.7 m, respectively. The excavation width=40m. The hard soil is located at 80 m. Using the method similar to those in the TNEC case, the estimated *PIZ* for stages 5, 7 and final are all equal to 40m. Figure 9 shows the comparison of settlement obtained from the USC analysis with those from the three methods. The proposed method is able to give a more reasonable prediction in the settlement of PIZ than the other two methods.

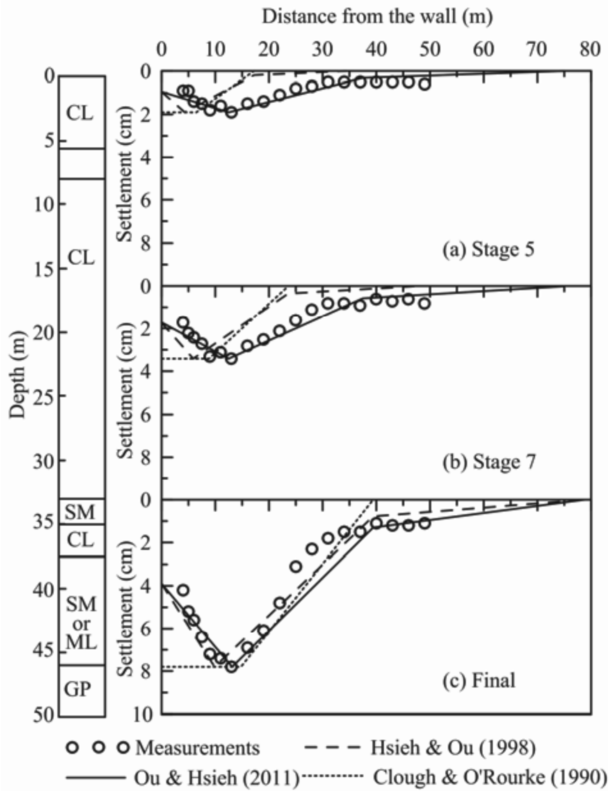

Figure 8. Verification of the proposed method for TNEC excavation.

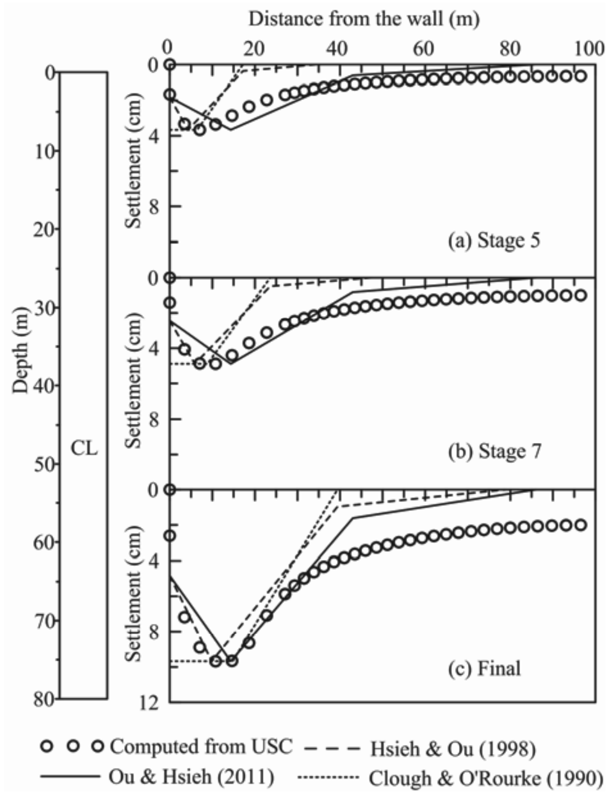

Figure 9. Verification of the proposed method for the hypothetical excavation with the 80 m thick soft clay.

5 CONCLUSION

The objective of this paper is to investigate the mechanism of ground settlement induced by deep excavation under the plane strain condition through finite element analysis. The study focuses on the settlement under the normal excavation condition, that is, no dewatering induced settlement, no excessively long construction duration causing the occurrence of creep, and no serious construction defects. The USC model was selected to perform parametric studies to find the dominating factors affecting settlement influence zone based on the calibration of a well-documented case history and a hypothetical excavation with 80 m thick soft clay using various soil models. It is found that the primary influence zone is mainly the active failure zone or the potential failure zone due to basal heave. A method is then proposed to estimate the primary influence zone from the relevant parameters such as two times excavation depth, excavation width, depth to rock-like soil layer and depth of the bottom of the soft clay. Case studies reveals that the proposed method improves the prediction of settlement for excavations whose twice the excavation depth are very different than excavation width, depth to rock-like soil layer and depth of the bottom of the soft clay. The methods of Clough and O'Rourke and Hiseh and Ou only yield moderately good prediction results for the settlement at the final stage for most of the cases and largely poor predictions at the intermediate stages, which can be treated as single case histories because the excavation depth is the only parameter used in the formula.

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