Parametric Study of the Deep Excavation Performance of Underground Pumping Station Based on Numerical Method

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Abstract: Environmental responses to deep excavations are combined results of numerous factors. The effects of some factors are relatively straightforward and can be considered carefully during the design. On the other hand, more features impact excavation-induced performances indirectly, making their influences difficult to be clearly understood. Unfortunately, the complexity and non-repeatability of practical projects make it impossible to thoroughly understand these issues through realistic deep excavation projects. Therefore, parametric studies based on repeatable laboratory and numerical tests are desired to investigate these issues further. This work examines the influence of several key features on excavation-induced displacements through a series of 3D numerical tests. The study includes the choice of soil constitutive models, the modeling method of the soil–wall interface, and the influences of various key soil parameters. The comparison shows that the MCC model can yield a displacement field similar to the HSS model, while its soil movement is greatly improved compared to the MC model. Both the soil–wall interface properties and soil parameters impact the excavation-induced displacement to a large extent. In addition, the influence mechanisms of these parameters are analyzed, and practical suggestions are given. The findings of this paper are expected to provide practical references to the design and construction of future deep excavation projects.

Keywords: deep excavation; pipe galleries; deformation; elastic foundation; analytical solution

1. Introduction

The urbanization process has led to a depletion of internal land resources, prompting the placement of numerous municipal facilities, such as water supply pumping stations and substations, underground. Consequently, a significant number of deep excavation projects have been undertaken. The design, construction, and performance of deep excavations are closely interrelated, and it is not easy to carry out an excavation both efficiently and safely. From a design perspective, many factors (e.g., ground conditions, retaining schemes, construction methods, and construction costs) should be considered [1,2]. The existing design methods, which are generally based on observations of past case studies, are no longer capable of generating reliable predictions for new cases with increasingly complex construction conditions. Therefore, improved methods for predicting excavation-induced displacements should be developed to provide guidelines for designing the retaining systems.

Several approaches are generally used to conduct deformation analyses of deep excavations: simplified theoretical analyses (e.g., linear- and elastic-based close form approach, beam-spring approach, and limit equilibrium method), empirical/semi-empirical methods [3–7], laboratory tests [8–12], and numerical approaches [13–15]. Simplified theoretical methods are generally used to provide some basic understanding of the performance of deep excavations during the design, but they have many limitations because they are...
oversimplified [16]. Empirical and semi-empirical approaches are used to extrapolate the performance of deep excavations from the analyses of previously reported field data [6]. Since empirical approaches are based on numerous field measurements of different regions, they can generally provide results consistent with regional experiences [4]. Therefore, a large number of well-documented field measurements from different areas are required to improve the applicability of empirical/semi-empirical methods. Laboratory tests generally include centrifuge tests and large-scale model tests, through which the deformation law of deep excavations can be investigated. However, laboratory tests are both oversimplified and relatively prohibitive, making the approach rarely applied in practical engineering [8,10].

Numerical methods are the only available methods that can consider both geotechnical, structural, and constructional aspects of deep excavation problems [13,17]. Therefore, these approaches have the potential to be an effective tool for deep excavation analyses. With the developments in both hardware and software, numerical approaches are becoming more powerful and less consuming, which has resulted in considerable advances in deformation analyses of deep excavations. However, the complexity of numerical methods makes their results vulnerable to numerous factors, and neglecting some of these factors will reduce the accuracy of the model. On the other hand, it is unrealistic to consider all these factors in a single model [17,18]. Therefore, it is more practical to study the impact of these factors individually in parametric studies. However, studies focusing on the influence of several key individual factors are missing, making their influence mechanisms unclear. To address these issues, the current work is undertaken.

This work presents several parametric studies to examine the influence of several key features on excavation-induced displacements. To achieve this, a series of numerical tests based on the ABAQUS (2018) software are carried out. Considering that including too many features in a single numerical model is cumbersome for practical application in the design and analysis of deep excavations, a simplified model is generated to investigate the isolated influence of these features. The examined factors include the choice of constitutive soil models, the modeling method of the soil–wall interface, and the sensitivity of the calculated results to several key parameters of the soil. The findings of the current work are expected to have practical value in the design and construction of deep excavations.

2. Development of the Idealized Model

As mentioned in the last section, the study in this work is based on an idealized hypothetical deep excavation. In fact, investigations on some general characteristics of deep excavations based on hypothetical projects were not unusual in previous studies. For example, several researchers discussed the influence of different modeling techniques (e.g., different mesh types and element types for the soil and structural components) and structural features such as the operational stiffness of the diaphragm wall, horizontal supports, and vertical piles [15,18]. Based on an idealized excavation project, [19] evaluated the efficiency of isolation walls outside an excavation in modifying the excavation-induced displacement field. In order to have a comparison, a hypothetical deep excavation model of a similar size and construction procedure in the study of [15] is developed in this work. In addition, because the parametric studies in this paper aim to capture the general influence tendencies of several specific features, many aspects irrelevant to these features are simplified significantly. Detailed descriptions and simplifications of the applied model are described subsequently in this section.

2.1. Geometry of the Model

The idealized excavation (40 × 40 m in plane and 12 m in depth) is the simplification of a typical square, three-level excavation. The enclosure structures are diaphragm walls (1 m in thickness and 30 m deep) supported by three levels of floor slabs (0.15 m in thickness) with beams (0.4 m × 0.6 m in section) and a base slab (1 m thick). The horizontal structures are supported by erect column piles with diameters of 0.8 m and lengths of 30 m. The
vertical distance between adjacent horizontal structures is 4 m, and the horizontal distance between adjacent beams (adjacent erect column piles) is 8 m. Besides four erect column piles firmly connected with the columns, 25 engineering piles are also included in the quarter-model (as shown in Figure 1). These engineering piles are assumed to be uniformly distributed below the base slab and duplicate other properties from the erect column piles.

![Geometric features and mesh of the model.](image1)

**Figure 1.** Geometric features and mesh of the model.

The model comprises 50,424 nodes and 45,397 elements, in which the soil mass is modeled with 41,070 linear hexahedral elements of type C3D8I (where the “I” refers to the incompatible modes method), the diaphragm wall is modeled with 2016 linear hexahedral elements of type C3D8R (where the “R” refers to the reduced integration method), the slabs are modeled with 1600 linear quadrilateral elements of type S4R (where the “R” also refers to the reduced integration method), and the beams and piles are modeled with 711 linear line elements of type B31. The bottom boundary of the model is fixed; two of the vertical boundaries are symmetrical, and the other two are rollers. The roller boundary makes sure that the horizontal movements of the model are restrained while the meshes are free in the vertical plane. The piles are assumed to be embedded into the soil without interface properties. The assumption is made according to the conclusion that the impact of the soil–pile interface on excavation performance is minimal [18]. Different from the soil–pile interaction, the soil–wall contact properties influence the excavation behavior more dramatically. Unfortunately, a systematic study on the influence of the soil–wall contact properties is still missing. Therefore, several parametric studies regarding this issue will also be included in the current work.

### 2.2. Construction Sequence

Without any doubt, construction procedures impact the excavation-induced performance significantly. Consequently, numerical models aiming at replicating the performance of realistic projects should capture the construction procedures in practice as much as possible. However, the studies conducted in this paper are based on a simplified hypothetical project, which does not aim to discuss the influence of different construction procedures or different modeling methods. Therefore, the simulation of the construction process can be simplified significantly. The main construction sequence of the excavation in this article is summarized in Table 1.
Table 1. Modeled construction sequence.

<table>
<thead>
<tr>
<th>Steps</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Geostatic</td>
</tr>
<tr>
<td>2</td>
<td>Installation of the diaphragm wall, piles, and the beams and floor slab at the ground level;</td>
</tr>
<tr>
<td>3</td>
<td>(Exc-1) Excavation to 4 m below ground surface (BGS);</td>
</tr>
<tr>
<td>4</td>
<td>Installation of the beams and floor slab at the second level;</td>
</tr>
<tr>
<td>5</td>
<td>(Exc-2) Excavation to 8 m BGS;</td>
</tr>
<tr>
<td>6</td>
<td>Installation of the beams and floor slab at the third level;</td>
</tr>
<tr>
<td>7</td>
<td>(Exc-3) Excavation to 12 m BGS;</td>
</tr>
<tr>
<td>8</td>
<td>Installation of the base slab.</td>
</tr>
</tbody>
</table>

2.3. Constitutive Models and Input Parameters

Since a number of input parameters of the soil and the soil–wall interface are variables in the subsequent parametric studies, the current section only presents the input parameters applied in the basic analysis (i.e., denoted as the BA model subsequently), as described in Section 2.1.

The floor slab, beams, and piles in the model are reinforced concrete materials and are assumed to behave linearly and elastically for simplicity. The density of these reinforced concrete components is assumed to be 2500 kg/m$^3$, the Poisson’s ratio is assumed to be 0.2, and the nominal Young’s modulus is 30 GPa. To consider the effect of imperfection in the concrete as well as the workmanship, the applied Young’s modulus for those reinforced concrete components in the calculations is reduced by 20% of the nominal value.

The diaphragm wall is represented by the cross-anisotropic material introduced by a previous study to consider the joints between different wall panels [19], and the input parameters are described in Table 2.

Table 2. Input parameters of the diaphragm wall.

<table>
<thead>
<tr>
<th>$E_1$</th>
<th>$E_2$</th>
<th>$G_{12}$</th>
<th>$G_{13}$</th>
<th>$G_{23}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>30 GPa</td>
<td>3 GPa</td>
<td>1.5 GPa</td>
<td>12.5 GPa</td>
<td>1.5 GPa</td>
</tr>
</tbody>
</table>

For the convenience of carrying out parametric studies, a single soil layer is applied to the models. The soil adopted in the basic analysis is a typical soil (i.e., silty clay) in Shanghai (specified in Table 3) reported in a substation project [19]. In the basic analysis, the MCC model is applied, and the detailed input parameters are presented in Table 3.

Table 3. Input parameters of the MCC model.

<table>
<thead>
<tr>
<th>$\gamma_0$ (kN/m$^3$)</th>
<th>$\nu$</th>
<th>$K_0$</th>
<th>$\epsilon_0$</th>
<th>Modified Cam-Clay Parameters</th>
</tr>
</thead>
<tbody>
<tr>
<td>18.6</td>
<td>0.3</td>
<td>0.49</td>
<td>0.877</td>
<td>$M$</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>1.327</td>
</tr>
</tbody>
</table>

The isotropic Coulomb friction model with a shear stress limit is applied to represent the soil–wall interface properties in the basic analysis. Its detailed parameters (i.e., $\mu = 0.3$, $\tau_{max} = 25$ kPa) are decided by referring to a case study in Shanghai reported by Cui [13]. More possible methods to consider the soil–wall interface, as well as the influence of $\mu$ and $\tau_{max}$, will be further investigated through several parametric studies in the next section.

3. Parametric Investigations

3.1. Strategy of Analyses

A comparison study of different constitutive soil models (e.g., the MCC model, the MC model, and the HSS model) is first presented in Section 3.2. Additionally, three groups
of parametric studies, which discuss the methods of considering soil–wall contact, the impact of the value of $\mu$, and the influence of the value of $\tau_{\text{max}}$, respectively, are conducted. The results and analyses of these parametric studies are presented in Section 3.3. Finally, a series of parametric studies discussing the impact of different soil features based on the MCC model are carried out, and the results are illustrated in Section 3.4. To be noted, for models without special notes in this work, the applied input parameters are the same as the basic analysis.

3.2. Comparison of Different Soil Models

The MC model (i.e., the Mohr–Coulomb model), the MCC model (i.e., the Modified Cam-Clay model), and the HSS model (i.e., the Hardening Soil–small model) are three commonly applied soil models in deep excavation analyses. However, straight comparisons of them are lacking from previous studies. Based on the hypothetical excavation project, the performances of the three models are compared in this section. To be noted, the comparison is made based on a hypothetical project, which means there is no field data to calibrate the results of these models. Therefore, the study in this section only aims to find the difference rather than provide an evaluation of different models. Despite the lack of field data, a previous study by Dong [18] has calculated a model sharing the same geometry as the current research, the results of which can be used as references. The input parameters of the MC model are shown in Table 4, and the input parameters of the HSS model are presented in Table 5.

Table 4. Input parameters of the MC model.

<table>
<thead>
<tr>
<th>$\gamma$ (kN/m$^3$)</th>
<th>$E$ (MPa)</th>
<th>$\nu$</th>
<th>$c'$ (kPa)</th>
<th>$q'$ (°)</th>
</tr>
</thead>
<tbody>
<tr>
<td>18.6</td>
<td>44.65</td>
<td>0.3</td>
<td>18.6</td>
<td>32.9</td>
</tr>
</tbody>
</table>

Table 5. Input parameters of the HSS model.

<table>
<thead>
<tr>
<th>$\gamma_r$ (kN/m$^3$)</th>
<th>$v_{ur}$</th>
<th>$c'$ (kPa)</th>
<th>$q'$ (°)</th>
<th>$E^{\text{ref}}$ (MPa)</th>
<th>$E^{\text{ref}}$ (MPa)</th>
<th>$E^{\text{ref}}$ (MPa)</th>
<th>$C_0$ (MPa)</th>
<th>$\gamma_0.7$</th>
<th>$e_0$</th>
</tr>
</thead>
<tbody>
<tr>
<td>18.6</td>
<td>0.2</td>
<td>18.6</td>
<td>32.9</td>
<td>7.003</td>
<td>4.902</td>
<td>21.008</td>
<td>77.02</td>
<td>$1 \times 10^{-4}$</td>
<td>0.877</td>
</tr>
</tbody>
</table>

The calculated diaphragm wall deflections, ground surface settlements, and ground surface horizontal movements at the middle section are presented and compared in Figure 2. In this figure, the detailed description of the legend is presented in Table 6.

Table 6. Description of curves in Figure 2.

<table>
<thead>
<tr>
<th>Name</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>MCC model</td>
<td>the result of the numerical model with a soil model described in Table 3;</td>
</tr>
<tr>
<td>HSS model</td>
<td>the result of the numerical model with a soil model described in Table 5;</td>
</tr>
<tr>
<td>MC model</td>
<td>the result of the numerical model with a soil model described in Table 4;</td>
</tr>
<tr>
<td>Dong’s study</td>
<td>the result of a published work reported by Dong [18].</td>
</tr>
</tbody>
</table>

In general, all three models yield displacement patterns similar to Dong’s results [18]. The calculated maximum wall deflections of the MCC model, MC model, and HSS model are 13.3 mm, 14.1 mm, and 11.8 mm, respectively. In general, the differences in the wall deflections calculated from different models are insignificant; all of them (i.e., the MCC model, MC model, and the HSS model) seem to be applicable if the wall deflection is the main concern of a deep excavation.
By further examination, the comparison suggests that the HSS model yields the smallest wall deflection. However, it is not safe to say the reason for the small wall deflections of the HSS model is its consideration of the small strain stiffness of soil. The reason is that the determination of the parameters of the HSS model relies very much on experience correlations and an inverse analysis, which makes its parameters more flexible and favorable. In contrast, methods for determining the MCC parameters are relatively unified, and they all can be obtained from conventional laboratory tests. This advantage makes the MCC model a more appropriate choice for design purposes.

In terms of the ground settlements behind the wall, it is apparent that the MC model underestimates the settlement significantly. This is because the MC model produces unrealistically large upward movements of deep soils [20], which has offset the settlement at the ground surface. The MCC model and the HSS model yield similar maximum ground settlements, and both of them are close to Dong’s results [18]. However, considering the larger maximum wall deflection of the MCC model and the close relationship between the maximum wall deflection and the maximum ground settlement, it is safe to say that the MCC model would yield smaller maximum ground surface settlements if the wall deflection were the same as the HSS model. Meanwhile, the ground settlements far from the wall calculated from the MCC model are more evident than the HSS model and Dong’s result. This tendency should arise from the MCC model’s inability to account for the small strain behavior of the soil. In fact, a similar tendency was also reported from previous studies; for example, Kung [21] concluded that soil models without considering small strain stiffness tend to underestimate surface settlements near the walls while overestimating settlements far from the walls. The horizontal ground movements show the same tendency with the settlements: the MC model underestimates the deformation significantly, and the MCC model and the HSS model yield a similar displacement profile.

Because of the lack of field data, the current study can only provide a preliminary comparison between different soil models. Assuming that Dong’s results [18] are reliable, it can be said that both the MCC model (also denoted as the BA model subsequently) and the HSS model yield acceptable displacement fields, while the MC model underestimates the ground movements significantly. Considering that it is more convenient to determine the input parameters of the MCC model, the subsequent parametric studies in this work will be carried out based on the BA model.

Figure 2. Calculated displacement fields of models with different soil constitutive models.
3.3. Soil–Wall Interface Properties

3.3.1. Methods to Consider Soil–Wall Interaction

Several methods for considering soil–wall interactions (e.g., tie constraints, embedded approach, and surface-to-surface contact with different tangential behaviors) have been applied in previous studies. However, most of these studies generally assumed the methods used were reliable without detailed descriptions. To improve the understanding of this issue, the first parametric study in this section investigates the difference between different methods in considering the soil–wall contact. The analysis strategy and the corresponding descriptions of different cases are described in Table 7.

Table 7. Strategy of analyses on different methods of soil–wall contact.

<table>
<thead>
<tr>
<th>Case</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>Tie</td>
<td>Use Tie constraints to connect the diaphragm wall and the soil</td>
</tr>
<tr>
<td>Embedded</td>
<td>The nominal diaphragm wall is embedded in the soil without interface properties</td>
</tr>
<tr>
<td>Embedded (modified)</td>
<td>The modified diaphragm wall is embedded in the soil without interface properties</td>
</tr>
<tr>
<td>BC (basic analysis)</td>
<td>Surface-to-surface contact with the tangential behavior represented by the basic Coulomb friction model (( \mu = 0.3 )).</td>
</tr>
<tr>
<td>BA (basic analysis)</td>
<td>Surface-to-surface contact with the tangential behavior represented by the Coulomb friction model with a shear stress limit (( \mu = 0.3, \tau_{\max} = 25 \text{ kPa} )).</td>
</tr>
<tr>
<td>Frictionless</td>
<td>Surface-to-surface contact with frictionless tangential behavior</td>
</tr>
<tr>
<td>Rough</td>
<td>Surface-to-surface contact with rough tangential behavior</td>
</tr>
</tbody>
</table>

Notice that there are two “Embedded” scenarios (i.e., the “Embedded” and the “Embedded (modified)”) in Table 7. This is because when applying the “Embedded region” approach, the soil at the location of the diaphragm wall cannot be removed. Therefore, both the weight and stiffness of the soil accumulate in the diaphragm wall’s properties during the subsequent simulation processes, which may impact the calculated results. To quantify this impact, the “Embedded (modified)” case, in which the diaphragm wall applies a modified unit weight (i.e., by reducing the unit weight of the soil from the nominal concrete value), is further included in this study.

The calculated results of different models described in Table 7 are presented in Figure 3. According to the figure, the excavation-induced displacement field is sensitive to different modeling methods of soil–wall contact. Typically, the “Frictionless” model yields particularly large displacements from all aspects, indicating the significant impact of the friction at the soil–wall contact. On the other hand, all the models except the “Embedded” case yield the same displacement patterns, while the magnitudes vary with different modeling methods. The discrepancy of the “Embedded” model arises from the great accumulated weight of the diaphragm wall, which causes dramatic settlements. When the weight is modified (i.e., “Embedded (modified)”)), the calculated ground displacement profiles become nearly identical with the “Rough” and “Tie” models. At the same time, despite the unrealistic ground movements of the “Embedded” case, the lateral wall deflection is similar to the “Rough” and “Tie” models (only with a difference of around 1 mm). This tendency suggests that the weight of the diaphragm wall has a significant impact on the surrounding soils while influencing the lateral deformations of the retaining structures negligibly.
Figure 3. Deformations of models with different soil–wall contact types.

The similar vertical ground movements near the wall (i.e., where the x coordinate is 0) between the “BC” model and the “Tie”, “Rough”, and “Embedded (modified)” models suggest that the relative sliding between the wall and the soil outside the pit of the “BC” model is minimal. However, the lateral wall deflection of the “BC” scenario is larger than the “Tie”, “Rough”, and “Embedded (modified)” cases significantly. This discrepancy might be because of different normal behaviors of the contact. In the “Tie”, “Rough”, and “Embedded (modified)” scenarios, the separation of the surfaces once they are closed is prohibited, while the “Hard contact” applied to represent the normal behaviors in the “BA” and “BC” models allows separations after contact. However, the contact problems are so complicated that more possibilities explaining the differences are likely. Overall, even if the relative sliding between the soil and the wall does not occur, the approaches of Tie constraints, Embedded region, and Rough tangential property should be applied with caution because they generally yield conservative lateral wall deflections.

3.3.2. Influence of the Shear Stress Limit, $\tau_{\text{max}}$

The difference between the “BC” and “BA” models, although very limited, is observed in Figure 3, illustrating the influence of the shear stress limit, $\tau_{\text{max}}$, set in the “BA” model. In order to make clear this influence, a further parametric study concerning the soil–wall contact was conducted. According to a previous study [14], the value range of ultimate side friction of cast-in-place piles is 15 kPa−100 kPa. In the current parametric study, a domain of 5 kPa−100 kPa for the values of $\tau_{\text{max}}$ is considered. To be noted, the coefficient of friction, $\mu$, is kept to be 0.3 for all the models in this study. The results of this study, as well as the results of the “Frictionless” and “Rough” models described in Table 7, are compared in Figure 4.

As expected, the results from models with different values of $\tau_{\text{max}}$ are bounded by the “Frictionless” model and the “Rough” model (in which no shear stress limit is defined), and the displacements (in all aspects) generally increase with a decreasing $\tau_{\text{max}}$ value. At the same time, Figure 4 shows that the “$\tau_{\text{max}} = 50$ kPa”, “$\tau_{\text{max}} = 100$ kPa”, and “BC” (i.e., $\tau_{\text{max}} = \infty$) models produce identical results, indicating that the vertical shear stress between the diaphragm wall and the soil in this study is smaller than 50 kPa. Furthermore, the difference between the “$\tau_{\text{max}} = 15$ kPa” and “$\tau_{\text{max}} = 50$ kPa” (i.e., the possible range for Shanghai soft soils) models is limited (e.g., about 2 mm of the lateral wall deflections, around 3 mm of horizontal ground movements, and approximately 3 mm of ground
settlements), which indicates that the excavation-induced displacements are insensitive to $T_{\text{max}}$ values.

![Figure 4. Deformations of models with different shear stress limit.](image)

3.3.3. Influence of the Coefficient of Friction, $\mu$

No specific scope of the $\mu$ values of soil–wall contact has been reported previously; practitioners in Shanghai generally apply a scope of 0.25–0.75 by referring to the friction coefficient between the cushion cap bottom and foundation soils regulated by the Technical Code for Building Pile Foundations [22]. This parametric study, fixing the shear stress limit at $\tau_{\text{max}} = 25\,\text{kPa}$, considers a $\mu$ scope of 0.005–1.0. In addition, the results from the “Frictionless” and “Rough” cases are also included for comparison. The calculated displacements are presented in Figure 5.

![Figure 5. Deformations of models with different coefficient of friction.](image)

In general, the displacements decrease with the increasing $\mu$ values, while all the results are bounded by the “Frictionless” and “Rough” models. The identical results between the “$\mu = 0.6$” and “$\mu = 1.0$” models demonstrate that when $\mu$ is larger than 0.6, the
interface property will be dominated by the shear stress limit. Considering the close results between the \( \mu = 0.3 \) and \( \mu = 0.6 \) models, the maximum value of \( \mu \) should be somewhere slightly larger than 0.3. Similar to the shear stress limit, the difference between different models is insignificant, which suggests that calculated displacements are not sensitive to the coefficient ratio of friction, either.

3.3.4. General Evaluation

In general, the parametric studies in this section suggest that the soil–wall interface properties can impact the excavation-induced displacement fields. Typically, the modeling methods to consider the contact affect the calculated results dramatically. Even under situations without relative sliding between the wall and the soils, Tie constraints, Embedded region, and Rough tangential properties are not recommended because they generally produce over-conservative lateral wall deflections. When the soil–wall contact is simulated by surface-to-surface contact with a tangential behavior represented by the Coulomb friction model with a shear stress limit, both the coefficient ratio of friction and the shear stress limit, if not values that are apparently unreliable, influence the displacements negligibly. Therefore, the \( \mu \) and \( \tau_{\text{max}} \) values can generally be determined according to experiences.

3.4. Soil Parameters

Studies in this work generally involve the MCC model and the HSS model. Numerous studies concerning the input parameters of the HSS model have been reported [23–25]. In contrast, the influence of the input parameters of the MCC model on the excavation-induced displacement has never been investigated. In order to fill this gap in knowledge, as well as to improve the understanding of the application of the MCC model in deep excavation problems, this section presents a series of parametric studies based on the MCC model.

The discussed items include Poisson’s ratio, \( \nu \), a void ratio, \( e_0 \), the coefficient of lateral earth pressure at rest, \( K_0 \), a frictional constant, \( M \), an isotropic logarithmic compression index, \( \lambda \), and the swelling index, \( \kappa \). To be noted, there will be only one variable, which is the corresponding discussed parameter, in each parametric study, while the possible correlation between the discussed parameter and other unchanged ones (e.g., \( \lambda \) and \( \kappa \); \( \nu \), \( M \), and \( K_0 \)) is not considered. The parameter bound of each group of the input parameters can generally cover most of the possible values of Shanghai soft soils, while specific values within the bounds applied in the parametric studies are only to describe an increasing or decreasing order but do not necessarily correspond to particular soils.

3.4.1. Influence of the Poisson’s Ratio, \( \nu \)

Figure 6 presents the calculated results of models with different Poisson’s ratios, \( \nu \), which range from 0.2 to 0.4. According to the figure, the changing Poisson’s ratio impacts the excavation performance significantly. Both deformations and the influence weight of \( \nu \) increase with an increasing value in \( \nu \). For example, the maximum wall deflection increased by 19%, 23%, 28%, and 37%, respectively, for each 0.5 increment of the \( \nu \) value from 0.2 to 0.4. When the value of \( \nu \) changed from 0.2 to 0.4, the maximum wall deflection, the maximum ground surface settlement, and the maximum horizontal movement of the ground surface increased by 98%, 98%, and 151%, respectively.

The influence of the Poisson’s ratio can be explained by the expression of Young’s modulus in the MCC model:

\[
E = 3K(1-2\nu)
\]

where \( K \) is the bulk modulus, which can be expressed by

\[
K = \frac{(1+e_0)p'}{\kappa}
\]
where $e_0$ is the void ratio, $p'$ is the mean effective stress, and $\kappa$ is the unloading–reloading line slope. Substituting Equation (1) into Equation (2), we can obtain

$$E = \frac{3(1 - 2\nu)(1 + e_0)p'}{\kappa} \quad (3)$$

Therefore, when other variables in the expression remain unchanged, a larger Poisson’s ratio reflects a weaker stiffness of the soil, and thus larger deformations occur under the excavation-induced unloading.

Unfortunately, despite the great impact of Poisson’s ratio on the calculated performance of deep excavations, corresponding values for in situ soils can only be determined empirically or computed through back analyses.

3.4.2. Influence of the Void Ratio, $e_0$

According to the Technical Code for Building File Foundations [22], the void ratio of Shanghai soft soils is generally between 0.6 and 1.6. To cover this range, five $e_0$ values (i.e., 0.5, 0.9, 1.2, 1.6, and 2.0) are compared in Figure 7.

3.4.3. Influence of the Coefficient of Lateral Earth Pressure at Rest, $K_0$

The impact of the coefficient of lateral earth pressure at rest on deep performance reflects the influence of the magnitude of the initial horizontal stresses in the ground. The sensitivity of the excavation-induced deformations to different values of $K_0$ is investigated in Figure 8.

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**Figure 6.** Deformations of models with different $\nu$.

**Figure 7.** Deformations of models with different $e_0$.

**Figure 8.** Deformations of models with different $K_0$. 

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In general, all aspects of deformations decreased with the increasing void ratio. When the input value of $e_0$ reduced from 2.0 to 0.5, the maximum wall deflections, the maximum ground surface settlements, and the maximum horizontal ground surface movements increased by 43%, 31%, and 33%, respectively. According to Equation (2), the bulk modulus, $K$, of soils is proportional to the void ratio, $e_0$. Therefore, soils with a larger void ratio accordingly have a larger bulk modulus and, thus, a smaller rebound during unloading. In addition, the results also indicate that, compared with the ground surface movements, the lateral wall deflection was more sensitive to the value of $e_0$.

### 3.4.3. Influence of the Coefficient of Lateral Earth Pressure at Rest, $K_0$

The impact of the coefficient of lateral earth pressure at rest on deep performance reflects the influence of the magnitude of the initial horizontal stresses in the ground. The sensitivity of the excavation-induced deformations to different values of $K_0$ is investigated in Figure 8.

![Figure 8. Deformations of models with different $K_0$.](image)

The figure shows that the differences of the wall deflections, the vertical and the horizontal ground movements between the “$K_0 = 0.5$” model and the “$K_0 = 1$” model were around 12%, 50%, and 23%, respectively. The diaphragm wall deflections increased negligibly during $K_0$ value’s increase from 0.5 to 1. Theoretically, larger values of $K_0$ mean larger lateral earth pressure on the back of the diaphragm wall and, thus, a larger lateral stress relief when the soil inside the excavation is removed. However, the influence of the $K_0$ is insignificant. Furthermore, the limited differences of the lateral wall deflections with different $K_0$ values are generally above the bottom of the excavation, while the lower part of the wall inserted into the soil remains stable.

However, the obtained results might only represent the current idealized excavation because the tendency is not observed universally. In fact, the general influence of the value of $K_0$ is much more complicated. Several researchers have previously investigated the impact of the coefficient of lateral earth pressure at rest on excavation performances, but no consistent conclusion has been drawn. For example, Potts [26] carried out a series of 2D numerical analyses and found that the wall displacements were very much dependent on the values of $K_0$, in which a larger value of $K_0$ led to much larger wall deflections. A series of 3D numerical analyses conducted by Dong [18] indicated that changes in the values of the coefficient of lateral earth pressure at rest had an insignificant influence on diaphragm wall movements. At the same time, Xu [27], through his parametric studies,
summarized that the diaphragm wall deflections decrease along with the increasing $K_0$ values. Although the influence tendency of $K_0$ in this study is minimal, it is opposite to Xu’s results [27]. These contradictory conclusions might arise from various factors (such as different modeling methods, excavation scales, and constitutive models), which need more targeted investigations. Unmistakably, the impact of $K_0$ on excavation-induced displacements is still ambiguous, and further research is desired.

Unlike the lateral wall deflections, the ground surface movements seem to be less complicated. The lateral movements generally increased, while the vertical movements generally decreased with an increasing value of $K_0$. Nevertheless, considering that the excavation-induced ground movements are closely related to lateral wall deflections, the influence of $K_0$ on ground movements should be further investigated.

3.4.4. Influence of the Slope, $\lambda$, of the Normal Consolidation Line in the $e - \ln p$ Plane

The values of $\lambda$ reported from the several realistic projects in Shanghai range from 0.07 to 0.17, reflecting a general scope of the parameter for Shanghai soft soils [13,14]. This parametric study expands this scope to 0.03–0.4, and the calculated results are compared in Figure 9.

![Figure 9. Deformations of models with different $\lambda$.](image)

The figure shows that $\lambda$ values have a very limited impact on the lateral wall deflections (i.e., the maximum wall deflections only increased by 6.5% when the $\lambda$ value increased from 0.03 to 0.4). This is because $\lambda$ mainly reflects soil characteristics under loading conditions, but deep excavations are basically unloading processes.

Different from the lateral wall deflections, the ground outside the pit is very sensitive to the variation in the value of $\lambda$; i.e., when the $\lambda$ value differs from 0.03 to 0.4, the maximum settlements and the maximum horizontal movements of the ground surface increased by 445% and 42%, respectively. This tendency indicates the existence of loading conditions outside the pit during the excavation. In fact, the loading conditions arise from the upward movement of the deep soil caused by the excavation process.

3.4.5. Influence of the Slope, $\kappa$, of the Unloading–Reloading Line in the $e - \ln p$ Plane

The influence of the values of $\kappa$ is described in Figure 10. The parameter is the slope of the unloading–reloading lines in the $e - \ln p$ plane and mainly reflects the unloading behavior of the soils. During the excavation, the soil in front of the wall was in the unloading condition. Consequently, the wall deflections were directly related to the swelling index.
Moreover, according to Equation (2), the bulk modulus is in inverse proportion to the value of $\kappa$. Therefore, as shown in the figure, the maximum wall deflections increased by as much as 115% when the $\kappa$ value increased from 0.005 to 0.025.

![Figure 10. Deformations of models with different $\kappa$.](image)

Theoretically, the great increase in the lateral wall deflections brings great volume loss to the ground behind the wall and hence should increase the ground settlements. Yet, as shown in Figure 10, the ground settlements decrease with the increasing value in $\kappa$. This indicates that the deep soil is more sensitive to the value of $\kappa$. The larger $\kappa$ causes more significant upward movements of the deep soil, which outweighs the additional volume loss caused by the increased lateral wall deflections, and thus, decreasing settlements are produced.

3.4.6. Influence of the Slope, $M$, of the Critical State Line in the $p - q$ Plane

The MCC model does not involve a cohesion parameter, and $M$ is derived from the effective friction angle through Equation (4):

$$M = \frac{6\sin \varphi}{3 - \sin \varphi}$$

In addition, $M$ is the only parameter reflecting the shear strength of MCC soils. The results of models with different $M$ values are presented in Figure 11. In general, the diaphragm wall deflections decrease with the increase in the $M$ value, and the deformations are very sensitive to $M$ with small values. This tendency is because $M$ affects the shape of the plastic surface, and a small value of $M$ means the soil can enter the plastic zone easily. When the value of $M$ was larger than one, the diaphragm wall was no longer sensitive to the change of $M$, indicating the soil was in the elastic behavior.

The ground settlements increased slightly when the value of $M$ differed from 0.3 to 0.5. When the value of $M$ was larger than 0.5, however, the settlements decreased with the increasing value in $M$. Different from the diaphragm wall, a further increase in the value of $M$ when it is larger than one can still reduce the ground settlements, which should arise from the additional upward movement of the deep soil. Meanwhile, the lateral ground movement seems to be insensitive to the change in the value of $M$. 
Figure 11. Deformations of models with different $M$.

4. Conclusions

This work first developed an idealized square basement excavation. Based on the development of the hypothetical project, the procedure of a finite element analysis of deep excavations was introduced. Thereafter, the results of several calculations with different soil models were compared. The comparison with other models proves the reliability of the MCC model in deep excavation problems. Finally, a series of parametric studies based on the idealized excavation was carried out to investigate the influence of several critical aspects (e.g., soil–wall contact properties and several soil properties based on the MCC model). On the basis of the parametric studies, the following conclusions are drawn.

1. The soil–wall interface properties can impact the excavation-induced displacements from all aspects. Typically, the modeling methods to consider the contact affect the calculated results dramatically, in which the model assuming the tangential behavior of the soil–wall contact as frictionless yields the most significant displacements. On the contrary, the “Tie”, “Embedded (modified)”, and “Rough” models, which generally assume no relative sliding between the wall and where the soil occurs, produce the smallest displacements. At the same time, the results of the two models using the Coulomb friction model to consider the tangential behavior of the soil–wall interface lie somewhere in between. Although it is the same without relative sliding between the wall and the soil, Tie constraints, Embedded region, and Rough tangential property yield much smaller wall deflections than the BC model because the former three generally do not allow separation between the soil and the wall. Therefore, the approaches of “Tie constraints”, “Embedded region”, and “Rough tangential property” to represent the soil–wall contact are not recommended because they may produce over-conservative lateral wall deflections.

2. When the soil–wall contact is simulated by surface-to-surface contact with a tangential behavior represented by the Coulomb friction model with a shear stress limit, both the coefficient ratio of friction and the shear stress limit, if not values that are apparently unreliable, influence the displacements negligibly.

3. The discussion of the soil parameters of the MCC models shows that different parameters influence the displacements to various degrees. The lateral wall deflections are sensitive to Poisson’s ratio, $v$, the void ratio, $e_0$, the frictional constant, $M$, and the swelling index, $\kappa$. The ground settlements behind the wall are sensitive to Poisson’s ratio, $v$, the coefficient of lateral earth pressure at rest, $K_0$, the frictional constant, $M$, the isotropic logarithmic compression index, $\lambda$, and the swelling index, $\kappa$. The
lateral ground movements are sensitive to Poisson’s ratio, \( \nu \), the isotropic logarithmic compression index, \( \lambda \), and the swelling index, \( \kappa \). Typically, the parametric study on the lateral earth pressure at rest highlights the ambiguity of its impact, and further examinations are needed to make clear the issue.

This study investigated the influencing mechanisms of different factors in deep excavation analyses based on FEM and highlighted the importance of several key parameters. The findings and conclusions drawn in this study provide valuable references to the design and construction of deep excavations.

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