LARGE EXCAVATIONS IN URBAN PERIMETER: STUDY OF DISPLACEMENTS IN NEIGHBORING FOUNDATIONS

Allan Alexandre Honorio[1] and Vitor Pereira Faro[2]
[1] Postgraduate Program in Civil Engineering,
Federal University of Paraná, Paraná, Curitiba, Brazil
allan.honorio91@hotmail.com
[2] Postgraduate Program in Civil Engineering,
Federal University of Paraná, Paraná, Brazil
vitorpereirafaro@gmail.com

ABSTRACT

This article aims to numerically analyze the influence of excavations on the surface settlements of neighboring building foundations using the Plaxis 3D finite element software. This is a topic of great interest in urban areas with a high concentration of buildings. The need for increasingly deeper excavations has challenged geotechnical engineers to balance high horizontal pressures while minimizing soil displacements and avoiding damage to nearby structures. In this article, a detailed analysis of the main empirical methods available in the literature for predicting settlement profiles caused by excavations is presented, comparing them with results obtained through finite element software. The study results showed that the empirical method proposed by Hsieh and Ou (1998) provided the best approximation in predicting settlement profiles in the analyzed cases, both for maximum settlement and for the profile of surface settlements with respect to the distance from the excavation face. The method proposed by Ciria (2003), which indicates settlement values during the installation phase of the retaining structure before excavation, also demonstrated a good correlation with the data obtained from the software. Finally, the application of numerical modeling to the selected cases confirmed the observations highlighted by different authors, demonstrating its ability to satisfactorily reproduce horizontal wall displacements and provide an approximate prediction of surface settlements. These findings contribute to a better understanding of the effects of neighboring excavations on foundations and can assist geotechnical engineers in decision-making during the planning and execution of excavation projects in densely built urban areas.

KEYWORDS: *Excavation, Finite element analyses, Tieback Diaphragm Wall, Settlement*

I. INTRODUCTION

In urban areas with high population density, there is a significant need for expanded construction endeavors. Given the restricted surface space, the implementation of excavation techniques and peripheral retaining structures becomes essential to accommodate underground facilities like parking garages, tunnel shafts, and subway trenches. Nevertheless, guaranteeing the safety of excavations, particularly through the utilization of retaining structures, demands the development and execution of solutions that not only take into account the forces exerted on these structures but also evaluate their effects on the adjacent soil and pre-existing structures.

In response to the need for additional spaces, diverse techniques have surfaced to tackle these challenges. The main remedy involves the application of retaining structures, categorized into two types: rigid and flexible. Rigid structures, characterized by high stiffness and minimal bending deformation, contrast with flexible structures, as per the definition in Eurocode 7 [1]. Flexible structures are relatively thin and demonstrate substantial resistance to bending, with the wall's weight contributing insignificantly to its overall stability. These structures facilitate vertical excavations and offer peripheral support to the adjacent soils.

The effectiveness of retaining structures is intricately tied to variations in the soil's strength and deformation properties. As indicated by Milititsky [2], excavations in proximity to existing buildings are not consistently conducted with sufficient safety measures or well-designed plans, resulting in potential accidents. It is crucial to recognize that, due to inevitable displacements caused by excavation, incidents related to structural damage in nearby buildings are more commonplace than the overall failure of the excavated mass. Hence, during urban excavations, accurate prediction and anticipation of the magnitude and distribution of displacements become imperative to prevent structural failure and mitigate potential consequences for nearby structures.

As Massad [3] asserts, evaluating and foreseeing the functionality of the retaining system contributes to the project's safety by recognizing and managing potentially crucial elements, such as deformation control. In temporary excavation situations, the primary risk is concentrated during the construction phase, which may coincide with periods of intense and prolonged rainfall. Another potential hazard arises in developing countries, where certain deep excavations might remain exposed for an extended duration due to insufficient funding for project completion.

Ensuring the successful design and execution of excavation projects in urban settings necessitates a precise evaluation of settlements, along with forecasting and mitigating horizontal displacements. Through a comprehensive consideration of the effectiveness of retaining structures, vigilant monitoring of key factors, and proactive anticipation of potential risks, it becomes possible to guarantee the safety and success of such projects. This, in turn, contributes to the sustainable development of urban environments.

Empirical and semi-empirical approaches ([4]; [5]; [6]; [7]) are frequently utilized to estimate ground surface settlements resulting from deep excavations. However, their applicability across diverse ground conditions and their ability to predict subsurface soil movements are limited. To address these challenges, numerical methods, such as Finite Element Method (FEM) and Finite Difference Method (FDM), are commonly employed to model intricate soil–structure interaction scenarios ([8]; [9]; [10]). Enhancing the accuracy of predictions for excavation-induced ground movements can be achieved by incorporating a small-strain constitutive model ([11]; [12]; [13]; [14]; [15]) and accounting for the three-dimensional (3D) aspects of the model ([16]).

While numerical methods can address both geotechnical and structural aspects of deep excavations, the calibration of parameters required in advanced numerical models is intricate ([18]) and time-consuming. Numerous analytical methods are also available for estimating excavation-induced ground movements ([17], [18]; [19]; [20]; [21]; [22]).

II. MOVEMENTS ASSOCIATED WITH EXCAVATION

This segment of the article outlines the chronological development and principal contributions of the empirical methods employed. Anticipating settlements arising in the vicinity of deep excavations is a crucial consideration for projects situated in expansive urban centers. The execution of these excavations necessarily induces soil movement in conjunction with them or with the retaining structures, which occurs due to material loss, variation in initial stress conditions, or groundwater lowering, with eventual consolidation of saturated soils. Extra care must be taken, especially if there are old buildings or heritage-listed structures in the vicinity of the excavation, as these effects depend on existing foundations and the sensitivity to settlements of nearby structures. To regulate the execution of open excavations, the guidelines of [23] must be followed [24].

Presently, numerical methods, including the finite element method, serve as valuable tools for designers. However, in many instances, professionals may lack the essential geotechnical parameters to construct an adequate model for the behavior of both the retaining wall and soil. Consequently, it is advisable to employ empirical methods, particularly during the preliminary design stages, for estimating settlements in neighboring areas. Empirical methods have evolved from monitoring horizontal displacements of retaining walls and settlements measured in the vicinity of a diverse array of projects.

The classical reference for displacement measurements near excavations is Peck [4], Figure 1. It consolidates extensive expertise gained from executive activities, providing valuable guidance for decision-making to ensure the proper execution of services.

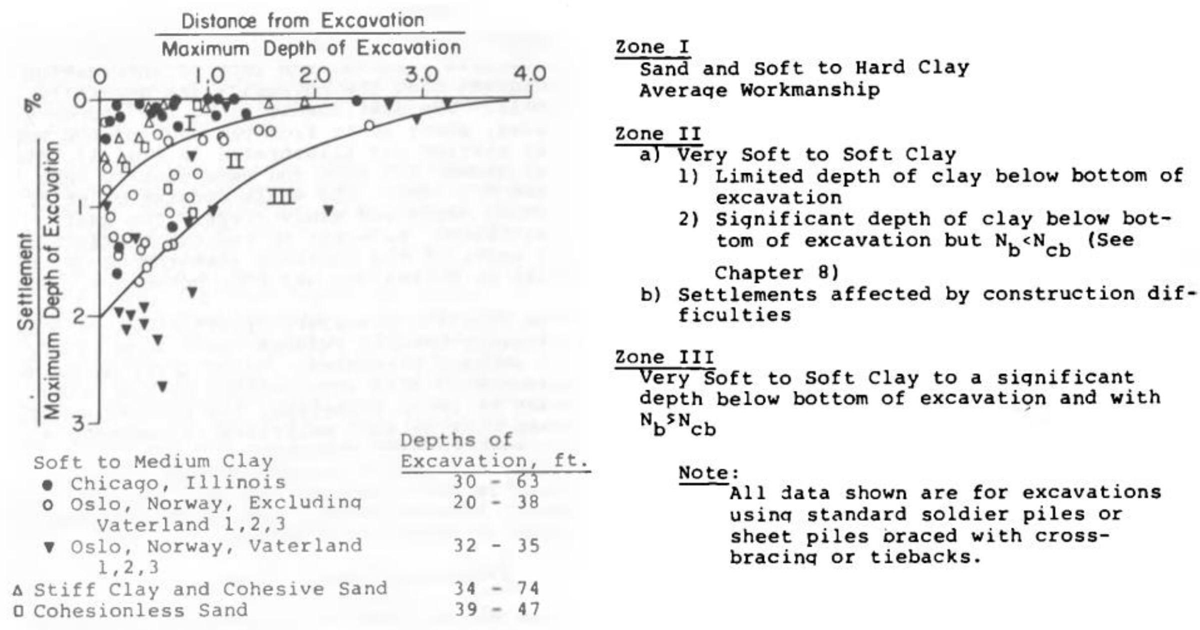

Figure 1. Summary of settlements adjacent to open cuts in various soils, as function of distance from edge of excavation (Peck, 1969)

Clough and O'Rourke [5] conducted a case study involving the monitoring of vertical and horizontal displacements for various materials. This study served as a preliminary estimation method for determining maximum values and displacement patterns. The surface displacements and distances from the wall are presented in relation to the maximum excavation depth (H), with the settlement distribution referencing the maximum settlement behind the wall. Figure 2 illustrates the outcomes of the case studies conducted in clay excavations.

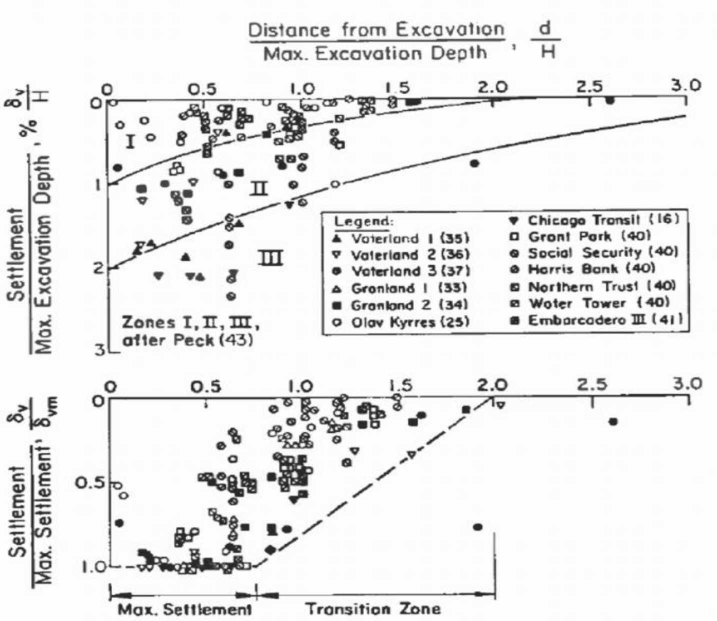

Figure 2. Measured settlements adjacent to excavations in soft to medium clay (Clough and O'Rourke, 1990)

Hsieh and Ou [6] provided a complementary study to the one presented by Ou, Hsieh, and Chiou [25] concerning two typical settlement profiles induced by excavations, termed concave and spandrel. They introduced a quantitative method to rationalize the distinct types of settlement profiles, namely concave and spandrel, triggered by excavations. Hsieh and Ou proposed that the area corresponding to the portion of wall displacement originating from deep movement, denoted as As, should be distinguished from the area corresponding to the displacement of the cantilever type, denoted as Ac. Based on the findings from the case studies conducted by Clough and O'Rourke, it can be inferred that the concave profile occurs when As ≥ 1.6 Ac. Finally, Hsieh and Ou's method simplifies and assumes linear behavior between the sections, presenting Figure 3 for the comprehensive prediction of the concave settlement profile.

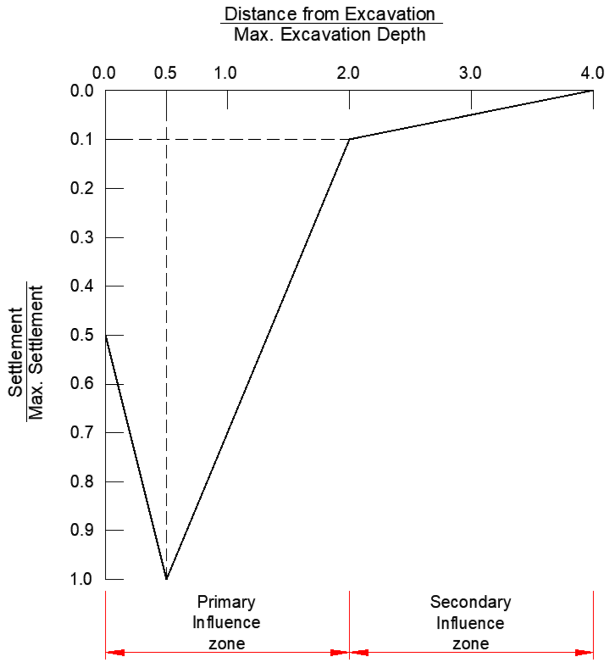

Figure 3. Shape of settlement trough for a deep inward wall deflection (adapted by Hsieh and Ou, 1998)

Hsieh and Ou emphasize that, typically, the maximum vertical surface displacement value, δ_{vm}, can be estimated based on the maximum horizontal displacement of the wall, δ_{hm}.

The approach introduced by Hsieh and Ou underwent validation through an extensive examination of various case studies, primarily focusing on clayey soils. However, the authors did not restrict the applicability of the method exclusively to clayey soils. The following procedure was outlined by Hsieh and Ou for predicting surface settlements induced by excavations:

1. Calculate the maximum horizontal displacement of the wall, denoted as δ_{hm}, by employing either empirical methods or numerical analysis.

2. Determine the equivalent depth of excavation, denoted as He, representing the depth at which the displacement equals δ_{hm}.

3. Estimate the maximum vertical surface displacement, denoted as δ_{vm}, by utilizing the relationship between δ_{hm} and δ_{vm} derived from the case studies.

4. Adjust the estimated δvm based on site-specific conditions, such as soil properties, groundwater level, and proximity to existing structures.

5. Validate the estimated settlements by comparing them with field measurements or data from similar projects.

By following this procedure, Hsieh and Ou provide a method for predicting surface settlements caused by excavations, taking into account the horizontal displacement of the wall as a key parameter.

In a more recent investigation, Ciria [26] presents results from displacement measurements solely during the construction stage, excluding excavation, of contiguous piled walls and diaphragm walls in clayey soils in London. The study discloses that the settlement-to-excavation depth ratio ranges from 0.04 to 0.06 for bored piles and 0.05 to 0.1 for diaphragm walls in stiff clays. Additionally, the study finds that the maximum distance of the settlement wedge is twice the excavation depth. These results imply that during the construction stage, without excavation, settlements induced by the installation of contiguous piled walls or diaphragm walls in clayey soils adhere to specific relationships with excavation depth. The mentioned settlement ratios offer insights into the anticipated settlement magnitude based on the construction method employed. Ciria's study contributes to comprehending the behavior of these soil-structure interactions and can provide valuable input for engineering design and construction planning in similar conditions.

III. PROBLEM DESCRIPTION AND MATERIAL PARAMETERS

The analysis of soil-structure interaction in the study was conducted through three-dimensional numerical modeling using Plaxis 3D, a software based on the finite element method (FEM). The analysis was divided into two main parts:

1. Individual behavior of settlement profile: The settlement profile was analyzed after each stage of the contiguous piled wall construction process. The numerical model simulated the behavior of the wall and soil, capturing the progressive settlement development. This allowed for a detailed understanding of the settlement distribution and its variation with each construction stage.

2. Comparison with empirical methods: The results obtained from the numerical model were compared with the predictions obtained through empirical methods. This comparison aimed to assess the accuracy and reliability of the empirical methods in predicting settlements induced by the construction of contiguous piled walls. By comparing the numerical results with the empirical predictions, the study evaluated the applicability and limitations of the empirical methods in practical engineering applications.

By utilizing three-dimensional numerical modeling, the study provided a comprehensive understanding of the soil-structure interaction and settlement behavior during the construction of contiguous piled walls. The comparison with empirical methods added insights into the adequacy of these methods and their potential limitations in capturing the complex behavior observed in the numerical analysis.

3.1. Project description

The analyses were conducted considering a hypothetical standard vertical cut excavation in the terrain with varying heights: 6.0 m, 8.0 m, and 12.0 m. These heights were chosen as they are common for the solution involving tieback anchored retaining walls. The cross-section of the excavation with strut levels and the final excavation depth is depicted in Figure 4.

In the analysis, the top of the excavated terrain is horizontal and consists of a single homogeneous and isotropic soil. The strength parameters of the soil were obtained through correlations with the Standard Penetration Test (SPT) N value of the soil. These correlations allowed for the determination of the soil's resistance parameters necessary for the numerical modeling.

By utilizing a standardized excavation section and incorporating geotechnical parameters obtained from correlations with Spt tests, the study aimed to simulate typical conditions encountered in the construction of tieback anchored retaining walls. This approach facilitated a more realistic representation of the soil-structure interaction and settlement behavior during the construction process.

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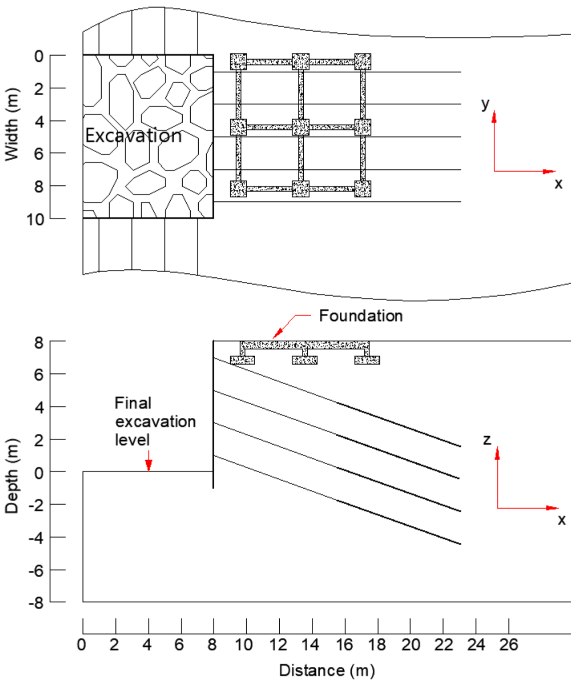

Figure 4. Cross section of excavation and strut levels

For each excavation, a retaining structure was designed with horizontal and vertical spacing of 2.0 meters between the ground anchors. The purpose of this design was to simulate excavation stages of the same height in the adopted models. The chosen retaining wall is made of reinforced concrete and is vertical. The construction process followed the sequence of constructing the wall panel and subsequently excavating before installing the ground anchor.

3.2. Material parameters

The soil strength parameters will be applied to the elasto-plastic constitutive model, defined by the Mohr-Coulomb failure criterion. For the analysis, a hypothetical young residual gneissic soil was chosen, which is common in the regions of Paraná, Santa Catarina, and São Paulo in Brazil. This soil is characterized by sandy silt, with an average NSPT value of 11. The soil's strength parameters will be determined through correlations based on this NSPT value.

The friction angle, denoted as "ϕ," was determined using the equation proposed by Kishida (1967), as cited in the book "Pile Foundation Analysis and Design" by [27]. The value of the cohesive intercept, denoted as "c," was determined based on the [28]. The elastic modulus, denoted as "E," was calculated using the equation presented by Tromfimenkov (1974, cited in [29]) specifically for sandy soils.

As a result of these calculations, Table 1 was generated, which presents the determined values of friction angle, cohesive intercept, and elastic modulus for the given hypothetical young residual gneissic soil:

Table 1. Input parameters (Honorio, 2022).

Parameter	Symbol	Value	Unit
Friction Angle	ϕ	30	º (degrees)
Cohesive intercept	c	25	kPa
Young's modulus	E	15	MPa
Poisson's ratio	ν	0.33	adm.
Unit weight	γ	19	kN/m³
Dilatancy angle	ψ	0	°

The Poisson's ratio (ν) of the soils was estimated based on the literature in [29]. In the elasto-plastic model, the requested dilation angle was considered zero for all materials. This approach, as suggested by [30], assumes that there is no plastic volumetric deformation, only plastic distortions.

For the modeling of the structural elements, all in concrete, the linear elastic constitutive model was assumed. The concrete Young's modulus was calculated according to the equation proposed in the Brazilian standard [31] as a function of the strength characteristics of the concrete subjected to simple compression.

The numerical modeling of the retaining structure was performed using the "Plate" element available in the software. The retaining wall was modeled with properties equivalent to a solid concrete section with a thickness of 30 cm.

The bulb of the anchor is represented by the "embedded pile" element. This embedded pile model consists of beam elements with a non-linear skin and tip interface, specifically developed in the software to describe the efficient interaction between the pile and the soil.

In the modeling, the free section is represented by a bar element, which does not mobilize any shear stress and is solely responsible for transmitting the forces from the retaining wall to the anchor.

The shallow foundation was a hypothetical square footing with a side length of 1.0 meter and a thickness of 60 cm. It was initially buried at an elevation of -1.5 meters and located at a distance of 100 cm from the retaining wall, adjacent to the anchor line. To provide a more realistic assessment, the footings' collars were connected to each other using reinforced concrete tie beams, forming a grid as shown in Figure 5. The spacing between the footings in the grid was set at 3.80 meters, chosen to ensure that no footing is located above an anchor line.

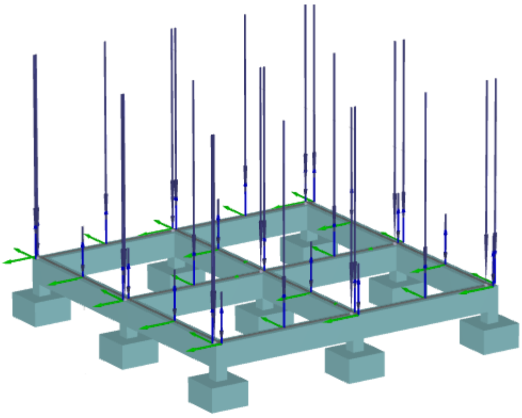

Figure 5. 3D model created by the software

Table 2 list the basic set of parameters used for diaphragm wall, tie rod, anchor bulb and foundation structure.

Table 2. Parameters for the structural elements (Honorio, 2022).

	Element	Diameter / Thickness (cm)	γ (kN/m³)	Constitutive model	E (Gpa)	ν (adm)
Diaphragm wall	Plate	30	25.0	Linear elastic	23.8	0.2
Anchor bulb	Embedded pile	10	24.0	Linear elastic	23.8	0.2
Free stretch tie rod	Node to node	10	78.5	Linear elastic	207.0	0.3
Foundation	Solid	-	25.0	Linear elastic	23.8	0.2

For residential or commercial buildings, columns can bear very high loads. In the context of the article, a fixed load value was used for the foundation footing, representing a column load of 100 metric tons, which is commonly seen in reinforced concrete structures. Additionally, a linear load of

400 kgf/m was applied to the beams to account for the masonry load of a wall with a height of 2.80 meters. These load values are typical in the design of residential or commercial structures.

3.3. Characteristics of the mesh resulting from finite elements

The numerical model developed is present in Figure 6. The finite element mesh is formed by tetrahedral elements of 10 nodes and has a medium degree of refinement. The number of soil elements is 83,610, and the number of nodes is 139,303.

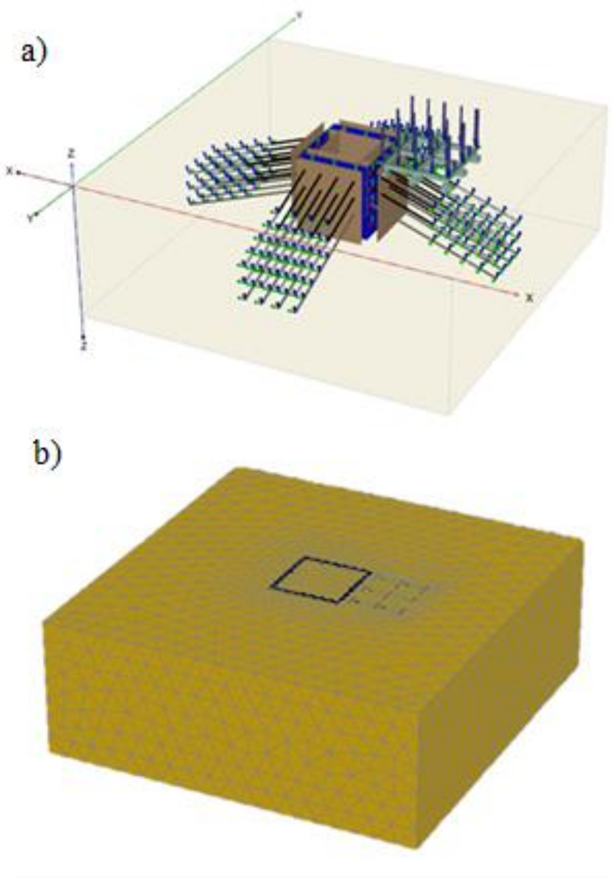

Figure 6. (a) Numerical modeling, (b) Finite element mesh

IV. Analysis and Results

This section encompasses the results obtained from the application of the methods and the use of numerical simulation, presenting the analysis outcomes for settlements.

4.1. Empirical methods

In this chapter, the proposed construction cases were analyzed based on empirical methods developed by Peck, Clough and O'Rourke, Hsieh and Ou and Ciria. The analyses were performed for each stage of the excavation. The majority of the methods rely on estimating the horizontal displacements of the wall to subsequently predict the vertical displacements of the surface. Considering the availability of information regarding the horizontal displacements of the wall obtained from the software for each stage of the studied cases, the following applications take these displacements as a starting point for predicting settlements. The predictions by the different methods will then be compared to the settlements obtained from the initial model developed by the software.

4.1.1. Peck's Method (1969)

The method proposed by Peck suggests that an estimation of settlements, as well as their distribution with excavation distance, of significant practical interest, can be made using Figure 1. In this figure, where settlements and distances are indicated in a dimensionless manner, based on the excavation depth, the author characterized three distinct zones. From Peck's graph, estimates of the settlement influence zone were determined for each excavation height, specifically for the final excavation level. These values are plotted in Figure 7 for each excavation case.

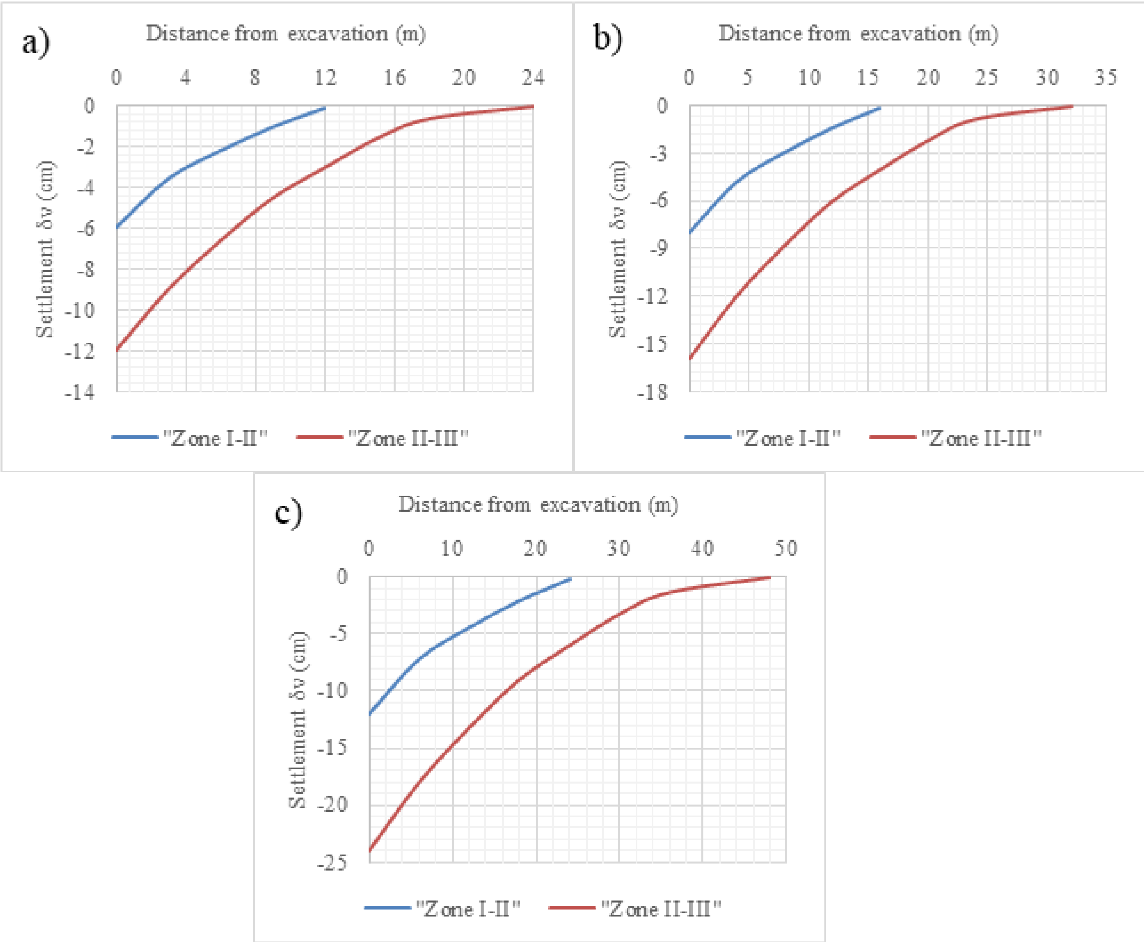

Figure 7. Estimation of settlement by Peck's method: a) 6 meters depth excavation b) 8 meters depth excavation c) 12 meters depth excavation

4.1.2. Clough e O'Rourke Method (1990)

Clough and O'Rourke use a triangular profile to predict settlements due to excavations in sands or stiff to very stiff clays. In this method, the maximum settlement (δvm) should be obtained from the graphs provided by the method. Therefore, predictions of settlement profiles were developed for each excavation stage. Similar to Peck, Clough and O'Rourke assume that the maximum settlement occurs near the face of the excavation. The results are presented in Figure 8, where it can be observed that for the case of a 6-meter high retaining wall, the maximum predicted settlement value by the method is -1.8 cm, and for the heights of 8 and 12 meters, the values are -2.4 cm and -3.6 cm, respectively.

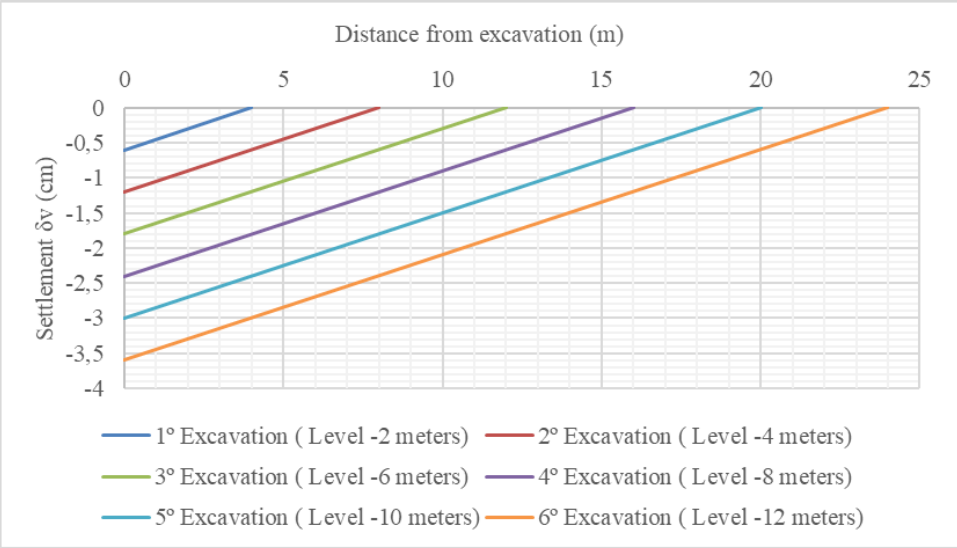

Figure 8. Settlement proposed by Clough e O´Rourke's method.

4.1.3. Hsieh e Ou Method (1998)

According to Hsieh and Ou (1998), in order to draw the settlement profile, it is necessary to initially predict the maximum horizontal deformation of the retaining wall (δhm) using the Finite Element Method or beam-based methods on an elastic base. In this case, the results obtained from the models developed in the software were considered for δhm and are summarized in Table 3.

Table 3. Maximum lateral wall deformation for each step (Honorio, 2022).

Stage	Lateral wall deformation δ_{hm} (mm)		
	Excavation 6 Meters	Excavation 8 Meters	Excavation 12 Meters
1º Excavation (N=-2m)	3.98	3.78	3.57
2º Excavation (N=-4m)	4.85	4.51	4.08
3º Excavation (N=-6m)	6.12	5.45	4.87
4º Excavation (N=-8m)	-	6.63	5.98
5º Excavation (N=-10m)	-	-	7.31
6º Excavation (N=-12m)	-	-	8.67

The next step in the method proposed by Hsieh and Ou is to determine the expected settlement profile. This involves comparing the areas of horizontal deformation caused by the first stage of cantilever excavation (Ac1) and the "cantilever" horizontal deformation area of the last stage of excavation (Ac2). Since in all situations the final displacement is greater than 1.6 times the displacement of the first excavation, according to Hsieh and Ou, we can conclude that the expected settlement profile should be concave for all excavations, as As ≥ 1.6Ac. However, the spandrel type profile will also be plotted for comparison. The next step in the method is to estimate the maximum surface settlement (δvm), which is a function of the maximum horizontal deformation of the wall (δhm). According to Hsieh and Ou, the value of the maximum settlement is between 0.5δhm, 0.75δhm, and δhm.

As a final step, the settlement values for various distances from the wall are calculated according to Figure 3. For this calculation, only the lowest value among the maximum settlement alternatives (δvm= 0.5δhm) was used, as the alternatives are multiples. Figure 9 show the profiles plotted for the maximum values for all stages of excavation.

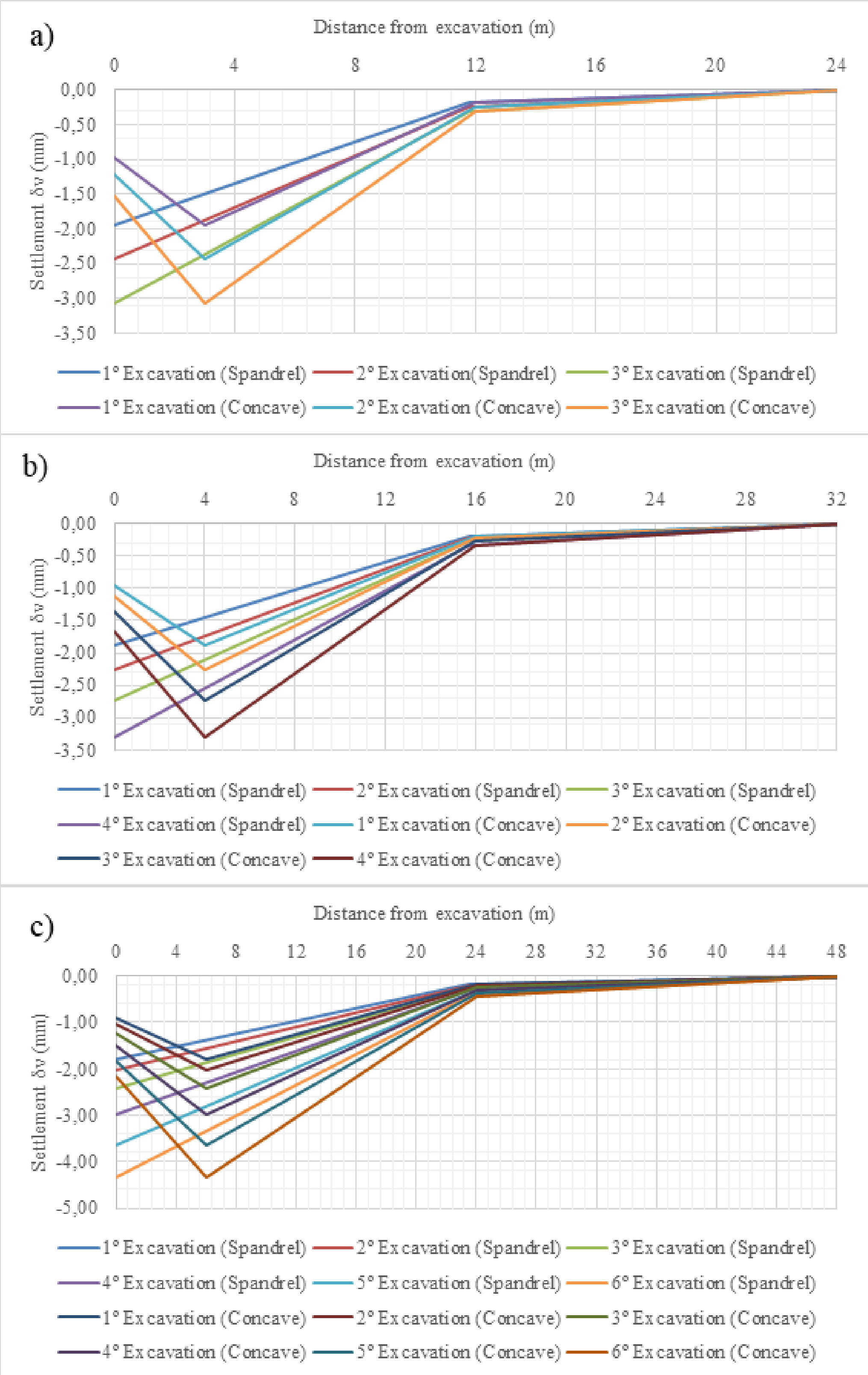

Figure 9. Estimation of settlement: a) 6 meters b) 8 meters c) 12 meters

4.1.4. Comparison of numerical analysis with empirical methods

This section presents the comparison of settlement results obtained from computational modeling with the results obtained from empirical methods.

Figure 10 illustrate the theoretical settlement profiles as well as those developed by mathematical modeling for walls of 6, 8, and 12 meters, respectively. It can be observed that the settlement profile proposed by Hsieh and Ou closely approximated the actual values. On the other hand, the values proposed by the Peck method were more conservative.

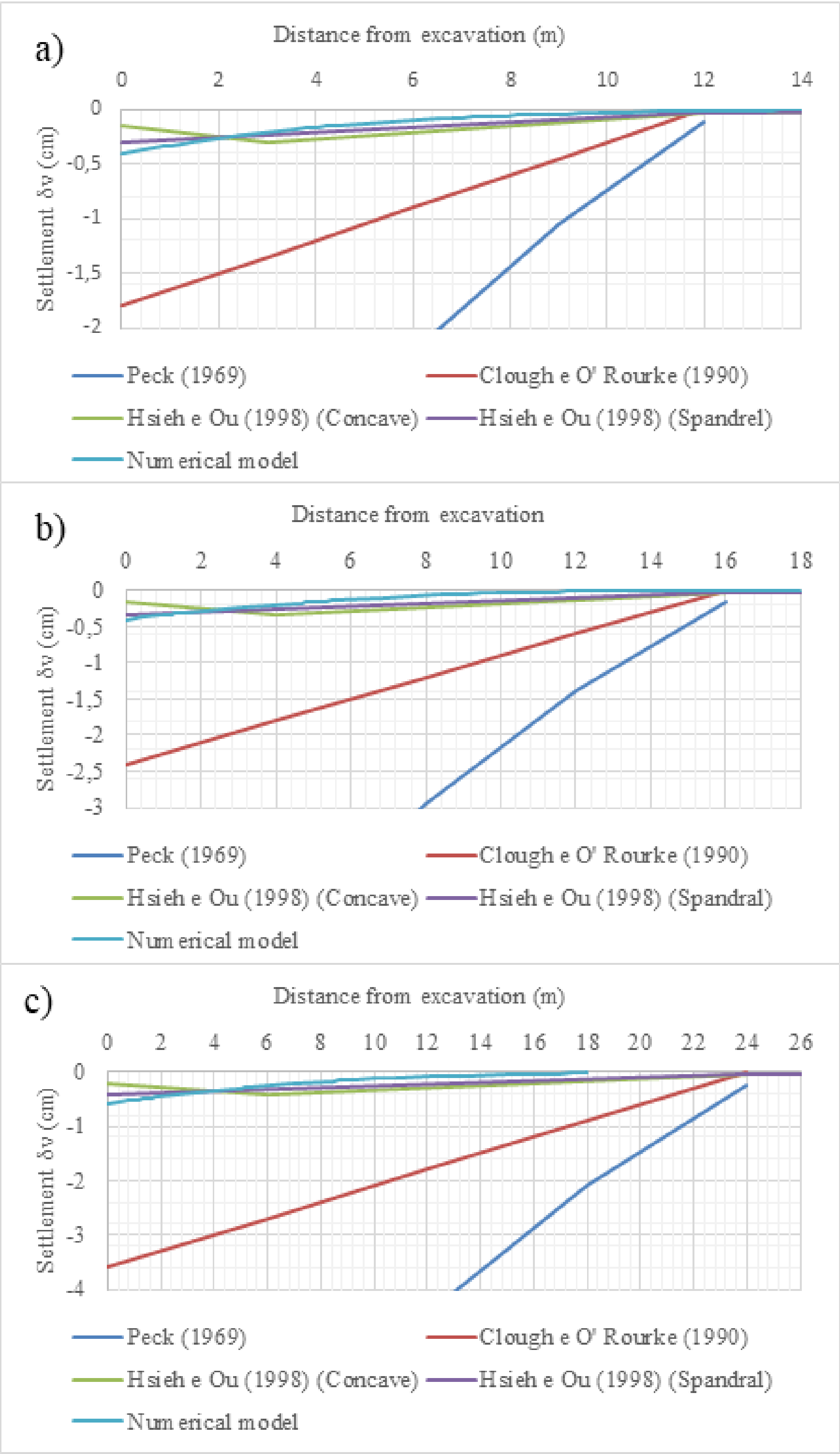

Figure 10. Comparison of theoretical settlement and numerical modeling: a) 6 meters b) 8 meters c) 12 meters

The settlement profile proposed by Peck deviates the most from the profile obtained from the numerical model, not only presenting a highly conservative maximum settlement value near the wall but also showing significantly higher values than those measured for the majority of the basin. The extent of the settlement region predicted by Peck (1969) is also larger than that obtained from the software. The settlement profile proposed by Clough and O'Rourke also does not closely match the measured values. The settlement profile proposed by Hsieh and Ou with $\delta vm = 0.5\delta hm$ provides the closest approximation to the results obtained from the numerical model. For the 12-meter wall, $\delta vm = 0.75\delta hm$ also provides a good approximation near the face of the wall, although with noticeable differences in this stage. The profile shape, resembling a spandrel, is quite similar to that of the developed model, which can be justified by the fact that the area As is close to the value of 1.6 Ac.

By employing the approach proposed by Ciria, which exclusively accounts for soil mass movement adjacent to the wall during the construction stage without excavation, maximum settlement values of 3.0, 4.0, and 6.0 mm would be obtained for each excavation. In comparison, the numerical model yielded maximum settlement values of 3.9, 4.1, and 5.8 mm. It can be observed that the settlement values obtained from the Ciria method are slightly lower than those obtained from the numerical model. This difference may be attributed to simplifications and assumptions made in the empirical method, as well as the inherent uncertainties in predicting ground movements. Nevertheless, in general, the Ciria method offers a reasonable approximation of the settlement behavior observed in the numerical model.

V. CONCLUSIONS

In summary, this investigation undertook a comparative analysis of settlement outcomes derived from both empirical methodologies and numerical modeling in the context of a retaining structure. The assessment involved the application of methods advocated by Peck, Clough and O'Rourke, and Hsieh and Ou to estimate settlements at various phases of the excavation process.

The findings indicated that the approach presented by Hsieh and Ou yielded a close approximation to the settlement values derived from numerical modeling. Conversely, the methodologies suggested by Peck and Clough and O'Rourke exhibited more conservative values, deviating significantly from the results obtained through numerical modeling. Additionally, the method proposed by Ciria, which exclusively considers the installation stage of containment without excavation, also demonstrated a favorable approximation to the numerical model results.

Hence, the conclusion can be drawn that numerical modeling, under the condition of using suitable parameters, has the potential to yield more precise and realistic results in contrast to empirical methods. Nevertheless, it remains crucial to take into account the inherent limitations and simplifications associated with each method. A comprehensive and meticulous analysis is essential when interpreting the results to ensure a thorough understanding of the implications and potential variations.

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