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Designing the nonlinear Moore-Coulomb model and constant barometric module in the stabilization of deep excavation

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Abstract

Determining the modulus of elasticity for soil is crucial in geotechnical engineering when conducting stress-deformation analyses. However, due to the difficulty involved in calculating this parameter, as well as the fact that the modulus of elasticity for soil is nonlinear in nature, there is often uncertainty surrounding its value. A study was therefore conducted to investigate these uncertainties and their impact on geotechnical analyses and plans. The study involved modeling and numerically analyzing a deep drilling guard structure using the anchoring method. To obtain the necessary information, two projects – depth and guard structure, which were both undertaken by Jahan Mall and Baran - in Mashhad were selected as case studies. In this study, the Jahan Mall project pit was analyzed using both two- and three-dimensional numerical models. Four different models were used: Moore-Coulomb (MC), hardening soil (HS), hardening soil with small strain stiffness (Cysoil HSS), and a newly developed nonlinear model based on the Moore-Coulomb model. To carry out the analysis, information obtained from barometric tests, standard penetration tests, and shear wave propagation was used. The results of the analysis were compared with each other and with the monitoring data. It was found that for the Moore-Coulomb behavior model, the pressure modulus (Ep) should be corrected to three to five times its original value in order to obtain accurate results. However, for the Cysoil HSS, HS models, and the newly developed model, there was no need to correct the pressurometric data or shear wave propagation. Additionally, it was determined that while no correction is necessary when using standard penetration numbers, an appropriate relationship should be used to convert them into the modulus of elasticity. Based on these findings, the Gud Baran project was analyzed, and it was concluded that the newly developed model and method for converting standard penetration numbers can be applied broadly and produce desirable results.

Keywords: soil type, stabilization methods, deep excavation
2020 MSC: 74B20

1 Introduction

When it comes to engineering problems, there are two main approaches: limit state and utilization state. The limit state approach involves analyzing the problem based on its stability under worst-case loading conditions (i.e., the bearing capacity of a foundation). In contrast, the utilization state approach involves examining the problem in terms of strains and shape changes that occur during use (i.e., foundation deposits). When conducting stress-deformation analysis, it is important to not only consider the physical and resistance characteristics of materials but

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also their formability characteristics. The most significant of these characteristics is the modulus of elasticity, which is notoriously difficult to determine. Due to this difficulty, there may be uncertainties in estimating the parameter, which can result in designs that are unsafe, perform poorly, or are costly. Other factors that complicate the estimation and consideration of the modulus of elasticity during analysis include the type of soil material, soil density, pasty properties, moisture percentage, stress path, stress history, stress level, strain level, and the heterogeneity of the soil mass. To assess the impact of uncertainty on geotechnical analysis and design, deep excavations were chosen as a case study due to their significance. Engineers typically use numerical modeling to estimate the variables associated with deep excavations and evaluate the safety of their stabilization plan. The accuracy of these models is highly dependent on selecting an appropriate behavior model for the soil and accurately estimating the modulus of elasticity and formability parameters for these models. Overestimating these parameters can increase the risk of failure while underestimating them can result in an uneconomical design for the stabilization structure.

Deep excavations have become a common feature in construction projects that require the design of an optimal and safe wall stabilization system. Engineers use different software to perform numerical modeling of the excavations and estimate variables, such as safety measures for the guard structure. The accuracy of these models is largely dependent on selecting the appropriate behavioral model to simulate soil behavior during excavation, as well as accurately estimating the soil elasticity modulus and associated parameters for these models. Overestimating these parameters can lead to the risk of the shaft breaking or collapsing while underestimating them can result in an uneconomical design for the stabilization structure. Therefore, it is crucial to identify the appropriate behavioral models for analyzing the excavation and developing a strategy for correctly estimating their formability parameters. Additionally, the lack of clear and simple methods for estimating these parameters for different behavioral models served as a motivating factor for conducting this research. This study involved creating a simple behavioral model using programming within the FLAC3D software. The parameters of this model can be easily estimated by analyzing the results of shear wave tests. Additionally, this behavioral model is accurate when estimating lateral wall variations. The simplicity, appropriate accuracy, and ease of parameter estimation through shear wave tests - which are often performed in construction projects and are inexpensive - are among the other significant aspects of this research.

This study references the deep excavations of the Jahan Mall and Baran projects, which took place in Mashhad City. The guard structures, geotechnical studies, instrumentation, and monitoring results for these projects are used as a basis for analysis. Methods for extracting formability parameters for different behavioral models based on tests performed during these projects were presented. To assess the accuracy of these methods, two- and three-dimensional analyses of the excavations were conducted and compared to the monitoring results. Two-dimensional analyses were carried out using the finite element method with PLAXIS software, while three-dimensional analyses were performed using the finite difference method with FLAC3D software.

2 Literature review

In 1990, Caliendo and colleagues [5] conducted a field study on the performance of an anchorage and guard candle maintenance system in a four-story underground parking structure located in Utah, USA. The soil profile in the area where the anchors were placed primarily consisted of soft clay. Field measurements included reading deflection gauges installed on several guard candles and strain gauges installed on the anchors. The researchers performed a two-dimensional finite element analysis using the SOILSTRUCT program, which was originally created by Clough and Duncan [6] and later developed by researchers such as Hansen [9] and Ebeling et al. [8]. They compared the results of the finite element analysis with the field measurements. One of the findings from this two-dimensional analysis indicated that the maximum possible change for the three original guard candles monitored by deflectometers was approximately 25 mm. The area where the maximum possible change occurred was in the last two-thirds of the wall and after the final planting level. The results of the finite element analysis were in good agreement with the measured values in the field, as the changes obtained from the analysis closely resembled the readings of the deflection gauges and followed the construction stages - specifically, soil removal, displacements towards the outside of the excavation, and then forward. When the anchors are removed, displacements will occur towards the inside of the cavity. Additionally, the maximum flexural anchor determined from the finite element analysis corresponded well with design anchors based on a beam with simple supports and bars.

In 1990, Mosher and Knowles [11] conducted a study on a temporary diaphragm wall made of concrete. The wall was about 15 meters tall and was anchored in place at Bonneville Dam, situated in the Columbia River. The purpose of building this wall was to prevent soil from collapsing into the excavation area where a new dam was being constructed and to protect the railway located near the wall. The temporary wall's intended function was to reduce soil settlement behind it, particularly in the area where the adjacent railway passes. The study aimed to achieve three primary objectives: The study conducted by Mosher and Knowles in [11] focused on a temporary concrete diaphragm wall that was built at Bonneville Dam, located along the Columbia River. The main purpose of the wall was to prevent soil from collapsing into the excavation area during the construction of a new dam and to safeguard the nearby railway. The study had three objectives: firstly, to validate the design processes used for constructing the wall; secondly, to predict how the wall would perform during the installation and removal of anchors; and thirdly, to interpret the results of the tools used to ensure that the wall's performance aligned with its design goals. To evaluate the behavior of the wall and soil, the researchers used the SOILSTRUCT computer program to conduct a two-dimensional finite element analysis. The results of this analysis helped to identify the behavior of the wall and soil and assess the design of the wall and the findings of the tools. The initial findings based on the finite element analysis indicated that the concrete diaphragm wall had satisfactory behavior when subjected to various loads, and the design was deemed desirable. However, in the preliminary analysis, conservative values were used for the soil hardness, which resulted in predicted displacements that were higher than the actual values. The final stage of the study involved conducting parametric studies to determine the soil hardness parameters that would match the wall's behavior with the instrument results. These studies revealed that the hyperbolic stiffness modulus of the soil was a significant parameter affecting the finite element analysis results. Increasing the modulus values for both initial loading and loading-reloading led to results that closely matched the observed behavior of the wall. Consequently, there was a close agreement between the shape changes and the bending anchors, and the analysis results confirmed the validity of the nonlinear behavior model and finite element analysis for this kind of problem.

Briaud and Lim [4] conducted a three-dimensional finite element analysis that was nonlinear in nature to assess the impact of various design decisions on the behavior of anchored walls in 1999. They used a modified hyperbolic model with hysteresis loading as the behavior model for the soil, which they calibrated for a real instrumented project. After calibration, they performed a parametric study to evaluate the effect of different factors on the wall's behavior, including the anchor location, anchor force, burial depth, and guard plug hardness. In 2001, Vermeer et al. [14] analyzed the effect of lateral soil pressure and buckling phenomena on a wall with guard piles anchored along with horizontal wood planks between the piles. This study involved simulating the behavior of a wall using numerical modeling with three-dimensional finite elements. To model the behavior of the soil, the researchers used the Moore-Coulomb model along with the HS hardening model. The findings indicated that there was substantial horizontal bending at the location of the guard plugs, and these plugs primarily restrained the lateral pressure of the soil. Moreover, the horizontal stresses were negligible at the distance of the plugs.

Bilgin and Erten conducted a study in 2009 [2] that involved using two-dimensional finite element numerical modeling to analyze the behavior of anchored shield walls. They evaluated how various factors, such as anchor location, shield hardness, anchor hardness, and the number of anchor rows, affected the wall's behavior under different soil conditions and pit wall heights. For the soil behavior model, they used the Moore-Coulomb model. The findings of the study revealed that while using multiple rows of anchors is the most effective way to minimize wall variability, employing shields with cross-sectional areas larger than the calculated values can also be beneficial in the design of structures. Additionally, based on the research results, recommendations were provided for the optimum design of such walls. Hsiung [10] conducted an analysis of a deep excavation site in the sandy soil of Kaohsiung, Taiwan, using the Moore-Coulomb model and FLAC software. The diaphragm wall and mutual restraints supported the pit wall. To determine the soil's modulus of elasticity, Hsiung used relation (2.1), which was proposed for determining the modulus of sand soil at the project site using the standard penetration number (N). He also used relation (2.2) to calculate the shear modulus (G) based on the shear wave speed (Vs) for small strains.

$$E = 2000N \qquad kN/m^2 \tag{2.1}$$

$$G = \rho V_s^2 \tag{2.2}$$

Figure 1 displays the lateral variation profiles obtained from the analysis using the two modules. The results obtained using the shear wave speed and standard penetration numbers are represented by the symbols "WV" and "SPT," respectively. As depicted in the figure, during the initial stage of excavation when small strains were generated, the lateral changes obtained using the moduli derived from the shear wave speed were consistent with the monitoring results. However, during the final stage of excavation when larger strains occurred, the analysis results based on the moduli calculated from the standard penetration numbers exhibited greater alignment with the monitoring outcomes.

According to Bilgin's research in 2012 [1], the conventional methods for determining lateral soil pressure coefficients in the design of anchored retaining walls do not consider stress concentration around the anchors or the behavior of these walls with a single row of anchors. Thus, to achieve a more realistic design of these walls, he used three-dimensional numerical modeling to analyze the behavior of shield walls with one row of anchors and determined new lateral pressure coefficients by taking into account the impact of stress concentration around the anchor location.

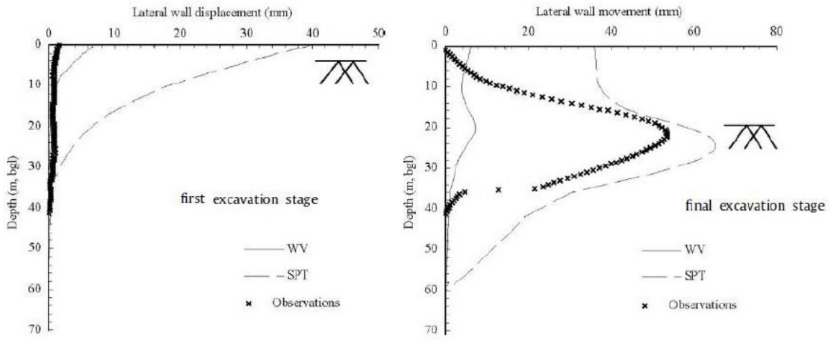

Figure 1: Comparing the lateral changes obtained from numerical analysis with monitoring results

These new coefficients could be utilized to enhance the design process for such walls. The behavioral model adopted for this research was the Moore-Coulomb model.

Costa and colleagues conducted a study in 2013 [7] on the behavior of a wall with anchored torsion piles. The soil profile at the site consisted of three layers of loose sand, dense sand, and sandy clay. The excavation depth was 15 meters, and the pit walls were supported by screw plugs with diameters of 25 and 35 centimeters and horizontal distances of 68 centimeters along with ten rows of anchors with a prestressing force of 350 kilonewtons. The primary objective of this study was to assess how the hardness of the retaining system and the depth of the buried wall affected the wall's shape change. To achieve this goal, a parametric study was conducted using the two-dimensional finite element method and the Moore-Coulomb behavior model to simulate changes in wall shape during construction stages. The results of the numerical modeling of shape change were compared with Good's monitoring outcomes. The analysis results revealed that an increase in the hardness of the retaining system led to a nonlinear decrease in the maximum possible change of the wall. Moreover, the buried depth of the wall played a secondary role in altering the wall shape.

Papagiannakis et al. [13] and Bin-Shafique et al. [3] conducted a study on how moisture affects the expansion pressures in expanding clays, as well as the impact of changes in moisture levels on the amount of these pressures and their effects on the flow. The wall was constructed in soil that is very sticky. To determine the expansion properties of the soil and simulate the behavior of an anchored wall exposed to significant lateral pressure caused by soil expansion, laboratory tests were performed, and a numerical model was created using the finite element method. Other factors affecting the wall's behavior, such as the soil moisture profile, overburden, the length of the anchors, and the hardness of the wall, were also evaluated through these numerical models. In 2017, Mun and Oh [12] discussed how the guard candle and anchoring and nailing dual stabilization system was applied in a project. The project they studied was a 39-story building that had five underground levels for parking. They used nailing in the upper parts of the underground parking pit to prevent damage to nearby facilities while anchoring was used in the lower parts of the wall for stabilization. To analyze this system's behavior, they used a three-dimensional finite element analysis with the HS model as their behavioral model. They determined the HS model's parameters using regression analysis based on numerical model variables and monitoring values correspondence. Their analysis results showed that the nails used in the upper part due to space limitations do not significantly reduce wall displacements, but the anchors used in the lower part have a significant effect in reducing adjacent structure settlements.

The majority of the analyses carried out in the studies that were reviewed to simulate pit behavior utilized finite element methods. The models used for behavior were predominantly the Moore-Coulomb and HS hardening models, with occasional use of the hyperbolic model. Soil formability parameters for case studies were typically obtained through regression analysis without a clear method being presented. Most of the studies examined were parametric studies, assessing the impact of various factors such as soil hardness, anchor number and location, anchor length and strength, lateral soil pressure, stabilization system stiffness, and hole geometry on hole behavior. Field studies were conducted primarily to investigate the factors that affect the displacement of pit walls, including wall hardness and pit geometry. The studies aim to quantitatively and qualitatively describe lateral wall displacements and sediment movements on the ground surface. As a result, experimental methods have been developed to estimate these factors and analyze the qualitative and quantitative effects of corners and stiffness in sections near the corners of the pit. Studies comparing two-dimensional and three-dimensional analyses of excavations suggest that the results of the two-dimensional analysis are generally more conservative than those obtained from the three-dimensional analysis, which better aligns with actual conditions. The magnitude of the three-dimensional effect is influenced by several factors, including the length-to-width ratio and length-to-depth ratio of the excavation, the type and hardness of the retention system, the distance to a hard layer beneath the excavation floor, and the safety factor against uplift.

3 Interpretation of the results obtained from the barometric test information

Before interpreting the results of the barometric test, it is necessary to first correct them. The correction process involves adjusting the raw test data by using calibration results in order to compensate for the loss of pressure caused by the reservoir's hardness and hydrostatic pressure from fluid column height. It is important to note that the volume increase resulting from the test was relatively small, so the volume correction was not taken into account for this project's tests. Once the results are corrected, the pressure values are plotted against the corresponding volumes on a diagram to facilitate interpretation. Figure 2 depicts the schematic diagram of the pre-excavated pressure meter test. In this figure, step 1 establishes a balance between the device assembly, reservoir, and ground up to point A. This allows the reservoir membrane to fully attach to the hole wall and reach the pressure value at the natural ground state before drilling, denoted by p_0. During stage 2 (up to point B) of the test, the behavior is predominantly elastic and follows an almost linear pattern. At this stage, it is possible to calculate the pressometric modulus or the coefficient of change of shape. The pressure recorded at point B is referred to as creep pressure (P_f). In stage 3, pasty behavior starts to set in, causing an increase in shape changes that eventually reach a significant amount. This leads to the limit pressure stage, where the changes in shape become extreme. At this point, the recorded pressure value is known as the limit pressure (P_l). However, in situations where the plotted curve lacks a clearly defined lateral line, conventionally, the limit pressure value is selected as the pressure that causes the hole volume to double.

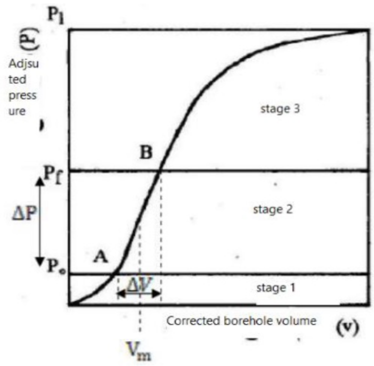

Figure 2: Schematic diagram of the barometric test

Figure 2 displays a range with a linear pattern, within which the constant pressure modulus of the soil can be computed using equation (3.1).

$$E_p = 2(1+v)(V_0+V_m)\frac{\Delta P}{\Delta V} \tag{3.1}$$

E_p: The volumetric strain at constant pressure

v: Poisson's ratio

V_0: Initial volume of the cylindrical sample under test (Note that the initial volume includes the volume change due to consolidation)

ΔV: Change in volume resulting from the volumetric strain caused by a ΔP increment of stress, following equation (2.2)

ΔP: Incremental change in stress causing the volumetric strain, following equation (2.2)

V_m: Average value of the incremental volume changes (ΔV) at the midpoint of the stress range considered

To calculate the elastic modulus (E_p) and shear modulus (G) based on experimental measurements, the following formulas are used:

$$E_p = 2(1+v)G \tag{3.2}$$

The barometric modulus can be determined by performing a loading-reloading cycle as illustrated in Figure 3. This module is referred to as the reloading module of the barometer.

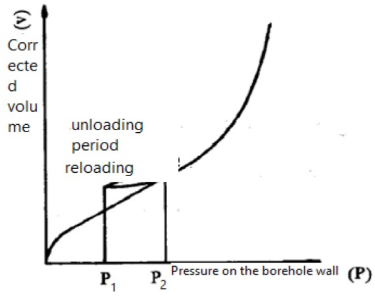

Figure 3: Diagram of the loading-reloading cycle in barometric test

The experiment involved conducting a barometric test at a maximum depth of 25 meters in boreholes BH30-3 and beyond 25 meters depth in boreholes BH90-30 and BH90-33. The outcomes of the barometric test are presented in Table 1, providing a summary of the results.

Table 1: Summary of barometric test results of the Jahan Mall project

Bore	Experiment depth (m)	Barometric module $\left(\frac{kg}{cm^2}\right)(E_p)$	Modulus of elasticity of reloading $\left(\frac{kg}{cm^2}\right)(E_p)$	Limit pressure $\left(\frac{kg}{cm^2}\right)(P)$	Coefficient of shear elasticity $\left(\frac{kg}{cm^2}\right)(G)$
BH30-3	5	253	–	21.6	101
	10	569	1361	45.9	227
	15	582	1610	38.1	232
	20	662	1722	51.3	265
	25	602	1179	64.1	240
BH90-3	25	745	1793	64.1	240
	31	160	1268	–	64
	37	251	–	–	100
	48	894	4843	60.5	358
	55	–	–	–	–
	61	595	911	60.8	238
	69	600	1323	–	240
	77	226	–	–	90
BH90-33	25	677	2099	54.9	271
	31	543	1330	53	217
	37	452	2260	66.2	181
	43	570	1921	69.8	228
	48	570	1921	69.8	228
	55	588	1497	62.6	235
	61	543	1409	58.3	217
	69	547	–	69.9	219
	71	525	1677	70.4	210

4 Numerical findings

This section presents the results of the numerical analysis of the built models, as explained in previous sections. The analyses differed in behavior model and modulus of elasticity value. They were done in both two and three dimensions using various behavior models, including Moore-Coulomb, Cysoil HSS HS hardening, and a non-linear model created in this research. The uncertainty in estimating the modulus of elasticity in the Jahan Mall and Baran projects was also investigated. Table 2 summarizes the analyses and their names.

4.1 2D and 3D modeling of the Jahan Mall excavation Project with the Moore-Coulomb model

In these models, the soil behavior is assumed to follow the Moore-Coulomb elastoplastic model. The soil elasticity moduli were determined based on the explanations in the previous chapter, using the results of standard pressure and penetration tests. The results of two and three-dimensional analyses using these moduli are presented in Table 3. The average modulus of elasticity in each analysis was calculated using the following relationship:

$$E_{avg} = \frac{\sum E_i h_i}{\sum h_i} \tag{4.1}$$

Table 2: Summary of performed numerical analyses

Project	**Modeling type**	**Behavioral module**	**Elasticity module extraction method**	**Analyses name**
Jahan Mall	3-D	Moore-Coulomb	From the results of the barometric test: $E = E_p, 3E_p, 5E_p$	$J3MC1E_p, J3MC3E_p, J3MC5E_p$
			From modified standard penetration numbers: $E = E_{SPT}$	$J3MC1E_{sm}$
			From non-modified standard penetration numbers: $E = E_{SPT}, 3E_{SPT}, 5E_{SPT}$	$J3MC1E_{snm}, J3MC3E_{snm}, J3MC5E_{snm}$
		Cysoil	From the results of the barometric test and based on the parameters of the model: HS	$J3CYE_p$
		Generated non-linear	From the results of the shear wave test	$J3DVV_s$
	2-D	Moore-Coulomb	The same as a 3-D model	Similar names by replacing 2 instead of 3
		HS	From the results of the barometric test	$J2HSE_p$
		HSS	From the barometric and shear wave tests results	$J2HSSE_pV_s$
Baran 4	3-D	Generated non-linear	From the shear wave test results	$B3DVV_s$
	2-D	Moore-Coulomb	From non-modified standard penetration numbers: $E = E_{SPT}, 3E_{SPT}$	$B2MC3E_{snm}, B2MC5E_{snm}$
		HS	From non-modified standard penetration numbers	$B2HSE_{snm}$
		HSS	From the barometric and shear wave tests results	$B2HSSE_{snm}V_s$

where E_i is the modulus and h_i is the height of the soil layers.

Table 3: The modulus of elasticity of the soil layers in the analyzes performed with the Moore-Coulomb model

Elasticity module $(MP_a)(E)$							**Depth (m)**
$J2MC5E_{snm}$ and $J3MC5E_{snm}$ models	$J2MC3E_{snm}$ and $J3MC3E_{snm}$ models	$J2MC1E_{snm}$ and $J3MC1E_{snm}$ models	$J2MC1E_{sm}$ and $J3MC1E_{sm}$ models	$J2MC5E_p$ and $J3MC5E_p$ models	$J2MC3E_p$ and $J3MC3E_p$ models	$J2MC1E_p$ and $J3MC1E_p$ models	
85	51	17	6	75	45	15	0-2
116.65	70	23.33	10.8	126.5	75.9	25.3	2-9
265.5	129.3	53.1	28.7	302	181.2	60.4	9-30
326.65	196	65.33	8.5	171	102.6	34.2	30-44
266.65	160	53.33	7.5	262	157.2	25.4	44-50
254.71	152.83	50.94	17.09	226.87	136.12	45.37	mean

4.2 Modules obtained from the barometric test

This section presents the results of analyses performed using the moduli obtained from the barometric test. In these analyses, the soil's modulus of elasticity is assumed to be equal to the pressure modulus (E_p) (models $J2MC1E_p$ and $J3MC1E_p$). Table 3 presents the parameters of the Moore-Coulomb model for this analysis mode. However, when using the barometric modulus as the soil's modulus of elasticity, it is important to consider the difference in soil tension path between seeding and the barometric test. In the pressure metric test, the pressuremeter device's container exerts pressure on the cylinder wall to calculate the pressure metric modulus, and the soil is in the loading path. However, during excavation, the soil stress path is in the form of loading and unloading. Therefore, it is necessary to use the soil's loading-reloading modulus of elasticity (Eur), which is typically considered to be three times the soil's modulus of elasticity in the loading state. In another analysis, the soil's modulus of elasticity in the Moore-Coulomb model is assumed to be three times the pressure modulus (models $J2MC3E_p$ and $J3MC3E_p$).

As mentioned in the barometric test modeling section with the Moore-Coulomb model, using the moduli obtained from the barometric test results in greater changes than what occurs in the test. In other words, the values of the A meter are used to determine the soil's modulus of elasticity under loading conditions. Comparing the variables of models that used three times the barometric modulus with recorded variables shows that there is a need to consider higher values for the modulus of elasticity. Therefore, in other analyses (models $J2MC5E_p$ and $J3MC5E_p$), Panj soil's modulus of elasticity is assumed to be equal to the barometric modulus.

Figure 4 presents the profiles of lateral wall displacement with growth progress for the $J2MC1E_p$ model. Figure 5 presents the same for the $J3MC1E_p$ model for the side section of the section (in the middle of the two guard candles).

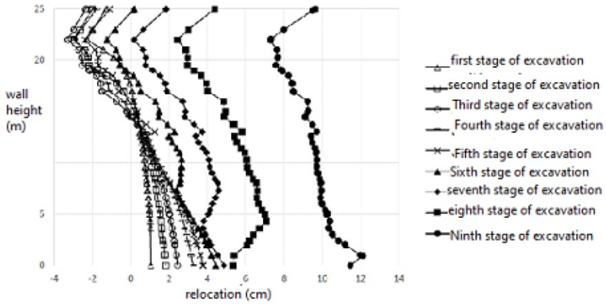

Figure 4: Profiles of the lateral wall displacement with the growth progress for the $J2MC1E_p$ model

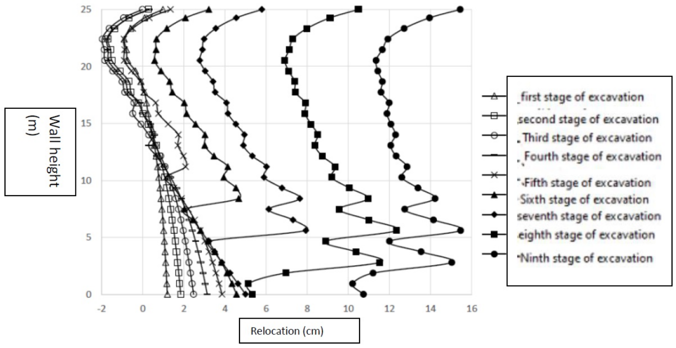

Figure 5: Profiles of the lateral wall displacement with the growth progress for the $J3MC1E_p$ model

In the first five stages of excavation, which reached a depth of 13 meters, the changes in both two and three-dimensional models were similar and small compared to later stages. However, from the sixth stage onwards, a large volume of soil was released from the elastic state and the increase in plastic points in the model caused significant growth of variables. At this stage, a difference in visualization between the results of two and three-dimensional analysis also occurred, which increased with fertilization progress and reached up to 60 percent.

In the profiles related to the three-dimensional model, severe fluctuations in the variable profile can be seen in the lower half of the wall where concrete pads and anchors are used instead of steel plugs to stabilize the wall. This is due to the occurrence of plastic areas between the pads and the lack of compensation for these developments by applying force to the anchors. Figure 6 presents lateral wall displacement profiles for two-dimensional models, while Figure 7 presents the same for the side and middle sections (place of guard candle) of three-dimensional models. Ground surface settlement profiles for two-dimensional models are presented in Figure 8 and for three-dimensional models in Figure 9.

According to the lateral change profiles presented for two and three-dimensional models (Figures 7 and 8), it is observed that the lateral changes related to $J3MC1E_p$ and $J2MC1E_p$ models are much larger than those monitored for a long time. This is due to the difference in soil stress path between tillage and the persimeter test, and the fact that the persimeter modulus is lower than the soil's real loading modulus. By increasing the soil's modulus of elasticity to three and five times the barometric modulus in two and three-dimensional analyses, a suitable match between these models' lateral variables and monitoring results has been created. The appropriate method for estimating the lateral wall positions is to use three to five times the pressure modulus as the soil's modulus of elasticity in both two- and three-dimensional models. The $J3MC1E_p$ three-dimensional model has a greater lateral variation compared to the symmetrical two-dimensional model, but the other three-dimensional models have a similar or even lower variation. When $E = E_p$, it is challenging to compare payment results due to the high variability, and the analysis does not match reality. The maximum lateral change occurs at the top of the wall in three-dimensional models and at the foot of the wall in two-dimensional models. Consequently, it can be concluded that the general pattern of profiles obtained from three-dimensional models is more comparable. The plastic behavior of soil is due to the fact that the reduction of lateral variables is not directly proportional to the increase in modulus of elasticity, as observed in two- and three-dimensional analyses. The reduction of variables in the plastic state with increasing soil hardness is less

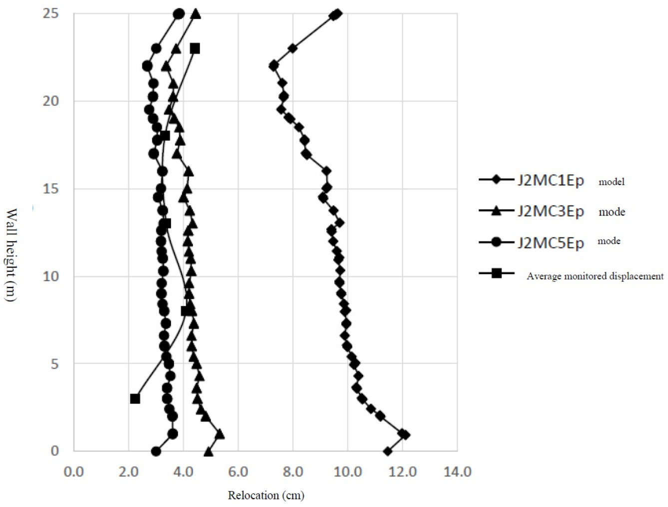

Figure 6: Wall lateral displacement profiles for $J2MC1E_p, J2MC3E_p, and J2MC5E_p$ models

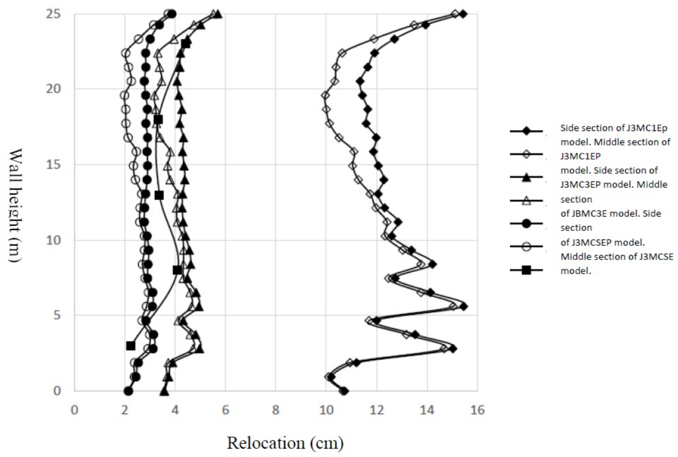

Figure 7: Wall lateral displacement profiles for models $J2MC1E_p, J2MC3E_p and J2MC5E_p$

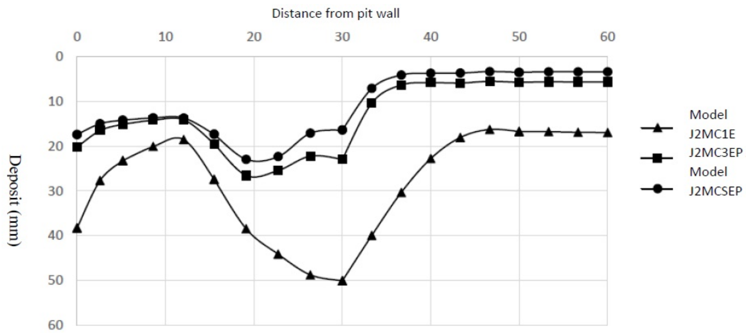

Figure 8: Ground surface settlement profiles for $J2MC1E_p, J2MC3E_p and J2MC5E_p$ models

than in the elastic state. Additionally, Figure 6 shows that at a height of 12 meters above the wall, the lateral profiles of the middle section and side section are dissimilar, while at a depth of 13 meters below the wall, they are nearly identical because of the presence of a watchman's candle in the middle section of the section 12 meters above the wall. As a means of comparison with other analyses, the impact of the watch candle on the variables of the middle section in three-dimensional analyses of lateral section variables has been considered. In Figures 8 and 9, it is evident that the $J3MC1E_p$ and $J2MC1E_p$ models have resulted in ground-level settlements that are disproportionately large and irrational. Nevertheless, the application of two- and three-dimensional analyses with moduli increased to three and

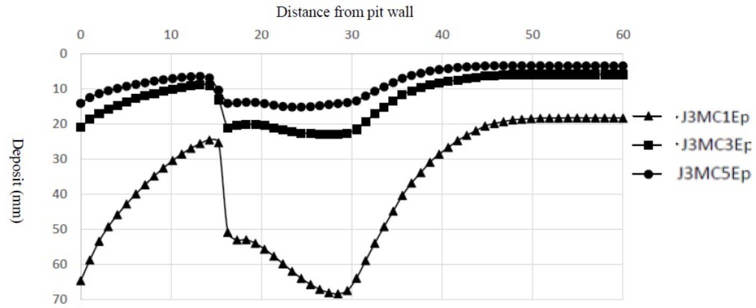

Figure 9: Ground surface settlement profiles for models $J2MC1E_p, J2MC3E_p, andJ2MC5E_p$

five times the barometric modulus has markedly reduced the settlements produced in these models. In both two- and three-dimensional models, the ground level exhibited a sudden increase and reached its maximum value at a distance of 15 meters from the settlement wall. This is attributed to the collision of the soil mass surface with the ground surface, leading to the plasticization of the soil in that area, as well as a shift in an overhead amount from 10 to 80 kilopascals due to the presence of 8-story buildings. Settlements at locations distant from the wall have stabilized at a constant value. Table 4 presents values for the axial forces generated in the non-locking section of anchors in two-dimensional models, while Table 5 provides these values for three-dimensional models. Furthermore, while the sittings of the $J3MC1E$ model are greater than their corresponding two-dimensional model, the opposite is true for other three-dimensional models, where their sittings are less than their corresponding two-dimensional models.

Table 4: Axial forces of anchors in $J2MC1E_p, J2MC3E_p$ and $J2MC5E_p$ models

Anchor No	**Axial force** (kN/m)		
	$J2MC1E_p$ model	$J2MC3E_p$ model	$J2MC5E_p$ model
1	313.6	304.6	301.4
2	312.1	304.2	301.1
3	320.7	306.1	302.2
4	318.3	306.3	303.3
5	329.9	311.8	317.7
6	337.1	319.7	341.4
7	366.5	343.6	439.6
8	474	428.5	307.5

Table 5: Axial forces of anchors in $J2MC1E_p, J2MC3E_p$ and $J2MC5E_p$ models

Anchor No	**Axial force** (kN/m)		
	$J3MC1E_p$ model	$J3MC3E_p$ model	$J3MC5E_p$ model
1	906.6	893.7	882.9
2	879.3	877.7	876.1
3	914.1	889.5	881.3
4	937.4	902.9	890.3
5	936.5	920.5	901.9
6	938.3	9.9	902.9
7	939.3	939.3	934.6
8	938.9	939.2	929.9

It is observed that by comparing the reason for the shift in the anchor forces to their initial applied values (300 kilonewtons/meter for 2D models and 900 kilonewtons for 3D models) and the change in earth mass shape following anchor implementation, the strength of the anchors appears to be higher in 2D models than 3D models due to more significant variations. However, differences in software, problem-solving methodology, and local errors could also contribute to this disparity, particularly in the 7th and 8th-row anchors. The study found that considering the modulus of elasticity in soil reduces changes in shape and decreases anchor values. Lower rows had stronger anchors, particularly in 2D models, but overall, the strength of one row was 1 to 4 less than the others. The analysis using the Moore-Coulomb model indicates a suboptimal design for wall anchorage. Additionally, modeling $J3MC3E_p$ as a 6-meter section with guard candles and anchor columns did not affect the analysis results.

5 Conclusions and suggestions

In summary, the presented results indicate that the numerical analysis conducted with HS, and HSS Cysoil models, as well as the nonlinear behavior model created, yielded very similar outcomes. The two- and three-dimensional analyses in nonlinear behavior showed almost identical results, considering the similarity of these behavior models. Additionally, the side variables obtained from the analysis using these models agreed well with the monitoring results. It is possible to obtain an accurate estimation of the side variables in two- and three-dimensional analyses by utilizing three to five times the pressure modulus and the modulus derived from the uncorrected standard penetration numbers from relations (3.2) and (4.1) as the soil's elasticity modulus in the Moore-Coulomb model compared to monitored values. The present study provides a comprehensive comparison of the analysis results conducted for the Jahan Mall project. The maximum and average column charts illustrating the lateral displacement in the models are presented in Figure 10 and Figure 11, respectively. Similarly, the maximum and average charts of the settlements occurring at the ground level are depicted in Figure 12 and Figure 13, respectively. The analyses and interpretations of these variables have been discussed in previous sections.

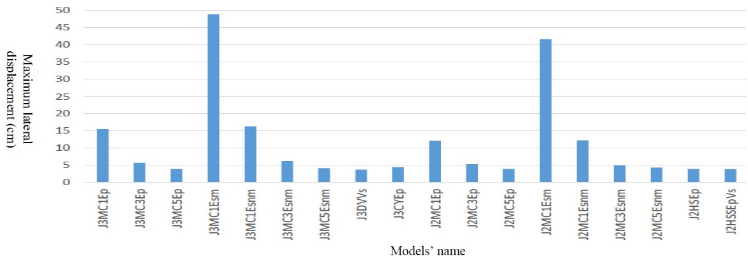

Figure 10: The diagram of the maximum possible lateral change in the analysis of the Jahan Mall project

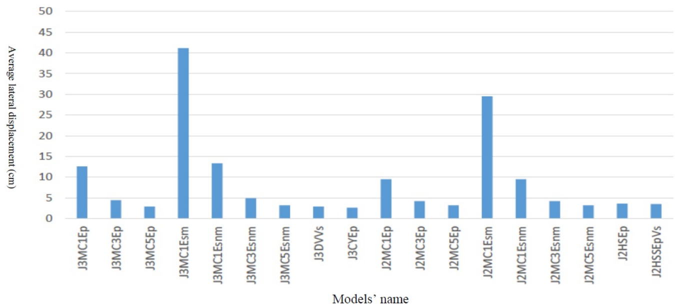

Figure 11: The average diagram of the lateral variation in the analysis of the depth of the Jahan Mall project

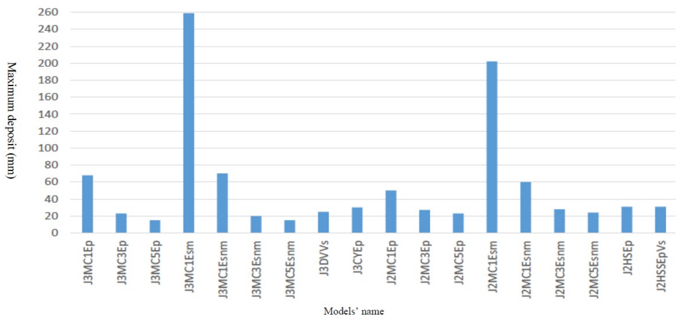

Figure 12: The diagram of the maximum ground-level settlement in the analysis of the Jahan Mall project

This research investigates the impact of the modulus of elasticity of soil materials on the analysis and design of guard structures for deep excavations, which is a critical and commonly encountered issue in civil engineering. The study addresses the challenges associated with measuring this parameter and its uncertainties and explores how it can be effectively incorporated into modeling and analysis. To address the difficulties in measuring the modulus of elasticity of soil materials, a new method was devised using programming. This method, based on the Moore-Coulomb

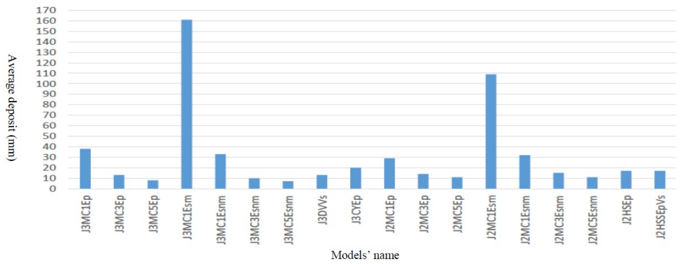

Figure 13: Average ground surface deposit in the analysis of the Jahan Mall project

model, considers changes in the modulus of elasticity with stress and strain levels. Two case studies were conducted to validate the method's accuracy and effectiveness. Future research directions include investigating similar studies in saturated lands or taking into account the hardness of surrounding structures in the analysis.

References

[1] Ö. Bilgin, *Lateral earth pressure coefficients for anchored sheet pile walls*, Int. J. Geomech. **12** (2012), no. 5, 584–595.

[2] Ö. Bilgin and M.B. Erten, *Analysis of anchored sheet pile wall deformations*, Contemporary Topics in Ground Modification, Problem Soils, and Geo-Support, 2009.

[3] S. Bin-Shafique, J. Huang, M. Ahmed, S. Rezaeimalek and A.T. Papagiannakis, *Tieback retaining wall in high plasticity expansive soil*, J. Perform. Const. Facil. **31** (2017), no. 2, D4016006.

[4] J.L. Briaud and Y. Lim, *Tieback walls in sand: numerical simulation and design implications*, J. Geotech. Geoenviron. Eng. **125** (1999), no. 2, 101–110.

[5] J.A. Caliendo, L.R. Anderson and W.J. Gordon, *A field study of a tieback excavation with a finite element analysis*, Design and performance of earth retaining structures, American Society of Civil Engineers, 1990.

[6] G.W. Clough and J.M. Duncan, *Finite element analyses of Port Allen and Old River locks*, J. Soil Mech. Found Div. **97** (1969).

[7] Y.D.J. Costa, J. Gurgel, C.M.L. Costa and O.F. Santos, *Deflection analysis of anchored retaining wall in dune sand*, Appl. Mech. Mater. **405** (2013), 101–105.

[8] R.M. Ebeling, J.F. Peters and G.W. Clough, *User's guide for the incremental construction soil-structure interaction program SOILSTRUCT*, Technical Report ITL90-6, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS, 1992.

[9] L.A. Hansen, *Prediction of the behavior of braced excavation in anisotropic clay*, Ph.D. Thesis, Stanford University, 1981.

[10] B.-C.B. Hsiung, *A case study on the behaviour of a deep excavation in sand*, Comput. Geotech. **36** (2009), no. 4, 665–675.

[11] R.L. Mosher and V.R. Knowles, *Finite element study of tieback wall for Bonneville navigation lock*, Army Engineer Waterways Experiment Station Vicksburg MS Information Technology Lab, 1990.

[12] B. Mun and J. Oh, *Hybrid soil nail, tieback, and soldier pile wall–case history and numerical simulation*, Int. J. Geotech. Eng. **11** (2017), no. 1, 1–9.

[13] A.T. Papagiannakis, S. Bin-Shafique and R.L. Lytton, *Retaining structures in expansive clays*, Geotech. Geological Eng. **32** (2014), 1405–1414.

[14] P.A. Vermeer, A. Punlor and N. Ruse, *Arching effects behind a soldier pile wall*, Comput. Geotech. **28** (2001), no. 6-7, 379–396.