Article Type: Research
OPEN ACCESS

A Numerical Investigation on the Surface Settlement due to Tunneling in Structured Fine-grained Soils

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Received: 5 July 2024 / Accepted: 20 July 2024 / Published: 25 August 2024

Abstract: Soil behavior can be categorized into two types: on-site soil behavior and reconstituted soil behavior in the lab. The stronger the soil structure, the greater the difference between these two behaviors. This structural strength results in a higher porosity at the same stress levels and divides soil behavior into elastic and plastic parts. To model structured soil behavior, various constitutive models have been developed, with many based on the cam-clay and modified cam-clay models. However, these models differ in integration and lack accuracy under varying stress levels. In this study, a modified cam-clay model is used to simulate structured clays. The model incorporates unstructured soil parameters and introduces modifications to account for hardening-softening behavior using initial yield pressure and other structured soil parameters. Simulations of triaxial tests show that the proposed model provides accurate predictions of structured soil behavior. Since most existing models in commercial software focus on unstructured soils, there has been limited research on practical problems involving structured soils. This paper applies the proposed model to 3D finite element software to study settlement due to tunnel drilling, showing excellent accuracy compared to other analytical methods.

Keywords: Structured soil, Cam-clay model, Finite element method, Tunnel drilling.

I. Introduction

Numerical modeling of soil behavior has developed greatly with the development of computer technology and its availability. Common numerical methods such as finite element method, finite differentiation method, or more advanced methods such as discrete element method are frequently used in geotechnical analysis today. In numerical modeling of soil behavior, accurate estimation of elastoplastic behavior will play a major role in the accuracy of the obtained answers. The soil in nature can be structured for a variety of reasons (Pepper & Heinrich, 2005). Soil structure refers to the natural bonding forces between particles. For instance, cemented soils, carbonate soils, saline soils, etc. have been obtained over many years in a structure that will cause their behavior to differ from the reconstructed samples of the same soil in the laboratory (Punmia & Jain, 2005). Marl soil is another example of this structured soil (Anagnostopoulos et al., 1991). Extensive research has been done on the behavior of structured soils. For instance, Memarzadeh et al. (2018) studied soil with Shiraz structure. Figure 1 shows the difference between the consolidation behavior of soil with Shiraz structure and reconstituted samples without effects of soil structure. As shown in Figure 1, soils with a structure at the same stress level as unstructured (regenerated) soils will have a larger porosity ratio. Liu & Carter (2000) express the ideal constitutive difference due to soil structure as shown in Figure 2.

The importance of such a chart is that all the constitutive models proposed to estimate the behavior of structured soil using reconstructed soil parameters and use such ideal charts to calculate structured soil parameters during analysis (Liu & Carter, 2003). For instance, Nova & Lagioia (2022) modified the cam-clay constitutive model to predict the behavior of (structured) carbonate clay. In this constitutive model, two major differences are created compared to the original cam-clay constitutive model. The first difference, the presentation of a formulation for estimating structured soil parameters from reconstituted soil parameters (using idealized graphs as shown in Figure 2). The second difference, the modification of the hardening-softening part of the model. Integration of the cam-clay model can be associated with numerical problems. For this purpose, in numerical modeling, a modified cam-clay constitutive model is used (Carter & Liu, 2005). Liu & Carter (2000) improved the modified cam-clay constitutive model for structured clay modeling. This constitutive model also included the same two general modifications of the previous model. Suebsuk et al. (2010) proposed a generalized modified cam-clay model to simulate the behavior of structured clay. More advanced constitutive models have also been proposed to simulate the behavior of structured soils. One of these models is the bubble behavior model proposed by Rouainia & Wood (2000). This constitutive model, which has kinematic hardening, is suitable for researching and its coding is not as simple as the previously mentioned constitutive models (Rouainia & Wood, 2001). The modified SANIClay constitutive model, which has rotational hardening, can model the effects of soil structure (Dafalias et al., 2006). The use of constitutive models that can model the effects of soil structure to investigate the practical problems of geotechnical engineering, especially the problem of tunnel drilling and the resulting settlements has been done very rarely (Seidalinov & Taiebat, 2014).

Increasing surface traffic and density of structures in urban areas have led to the expansion of the use of underground transportation systems such as the subway. Drilling tunnels in urban areas can cause excessive settlement and damage to adjacent buildings. Numerical study of tunnel drilling settlements allows the geotechnical engineer to obtain a preliminary estimate of the project hazards. It is obvious that, with the start of drilling and controls performed, doing back analysis by the numerical method will achieve a good safety factor (Moos et al., 2003). So far, many researchers have done in the field of numerical modeling of tunnel drilling. For instance, Choobbasti et al. (2008) simulated the tunnel drilling process using 2D surface strain modeling in Plaxis software. Katebi et al. (2020) have studied the surface settlement caused by drilling the subway tunnel of Line 2 of Tabriz city train using Plaxis software. Lakirouhani & Jolfaei (2018) have used Abaqus software to estimate the surface settlement due to tunnel drilling in 2D and 3D. Golshani et al. (2016) investigated the effect of tunnel drilling and the forces created in the tunnel cover using Abaqus commercial software and the Mohr-coulomb constitutive model. Mašın & Herle (2005) have used various constitutive models to model tunneling in London-structured clays and have shown that conventional Cam or Mohr-coulomb models are not able to properly model the behavior of structured soils and the use of hypoplastic models such as the SANIClay model which leads to better results (Taiebat et al., 2010). Using bubble behavior model and kinematic hardening, Gonzalez et al. (2012) have modeled the effects of tunnel drilling on London-structured clay in non-drained conditions. As can be seen, most of the existing research has been done either using commercial software that has ignored the effect of natural soil structure or has used advanced constitutive models in numerical modeling of tunnel settlement in structured soil, which their use requires more extensive knowledge e.g. plasticity, programming, and a larger range of experiments (to determine new parameters). Therefore, it is necessary to focus on simpler constitutive models that can consider the effects of soil structure (Taiebat et al., 2011).

In this paper, the behavior model of a modified Cam-clay is improved despite the effects of structure using the results of triaxial experiments and the definition of an experimental correction factor. Extracting the correction coefficient in such a way that the accuracy of the results of this model for calculating the volumetric strains of structured soil is significantly improved. So, the use of the modified cam-clay constitutive model with the structure for efficient simulation (in terms of computational cost) and with appropriate accuracy (increasing accuracy in calculating volumetric strains) is possible in Marl soils with strong structure.

The surface settlement of the structured clay resulting from 3D tunnel drilling has been investigated, after verifying and evaluating the performance of the proposed constitutive model. The unlimited soil environment is modeled by the scaled boundary finite element method and the soil of the near area is modeled by the common finite element method, to avoid the effect of boundary conditions on the accuracy of the obtained responses. Soil behavior modeling has a very limited structure in applied problems in technical literature. Also, the use of the finite element method in combination with the scaled boundary finite element method in the analysis of tunnel drilling problems and the resulting surface settlements has been evaluated for the first time in this paper. The contents ingredient of the article is as follows: First, a brief description of the finite element method and the scaled boundary finite element method is provided, then the proposed constitutive model for estimating the behavior of structured clay will be described. Verification of the proposed constitutive model with the results of the triaxial experiment (available in the technical literature) forms the next part of the article. Finally, the settlement problem caused by tunnel drilling in structured soil is modeled.

II. FEM and Scaled Boundary Finite Elements

The finite element method is a numerical process for analyzing continuous environments (Budhu & Wu, 1992). Classical analytical methods with many simplistic assumptions will not be very suitable for solving physical problems, since most of them have a variety of complexities. The finite element method transforms the partial equations governing issues using methods such as the weighted residuals method from the differential form to the integral form (Benz & Nordal, 2010). In this way, the strong form equations governing the issues become the weak form equations (Han & Gabr, 2002). The superiority of the weighted residual method in extracting and weakening the governing equations over methods such as the Rayleigh-Ritz method or the potential energy minimization method is that the weighted residual method does not need to know the governing issues (Bi, 2017; Daneshvar et al., 2020).

The inability of the finite element method to optimally model unlimited environments is one of the disadvantages. If the finite element method is used to directly model an infinite environment (which in the technical literature is interpreted as the extended mesh method), a very large range of issues will be required for modeling. Modeling such a large environment will increase the degree of freedom that governs the issue and will make the analysis time-consuming and computationally costly (Jin et al., 2019). Today, various methods are used to prevent this issue. The most common of these methods is the use of boundary discretization methods in combination with the finite element method. The boundary element method has been widely used in solving various engineering issues yet (Chen et al., 2014).

Reviewing the technical literature shows that the common method of boundary elements has two important objections. Firstly, this method requires a fundamental answer for each issue. Getting this fundamental answer to a variety of issues is costly and time-consuming. The second objection of the boundary element method is the fullness of the coefficient matrices and the increase in time required for the matrix analysis of the solution process (Jin et al., 2019). In the last two decades, Wolf & Song (2000) have proposed a new method that incorporates the advantages of the finite element and boundary element methods. This method, today called scaled boundary finite elements, extracts coefficient matrices with a formulation similar to the finite element method (Birk & Behnke, 2012). This means that coefficient matrices are extracted in the scaled boundary finite element method using virtual working principles or weighted residues (Chen et al., 2015). This relatively new method does not require a fundamental response, unlike the boundary component method. The advantages of the scaled boundary finite element method can be expressed as follows (Wolf and Schanz 2004):

- Formulation similar to the finite element method,
- Discretize only the domain boundary and thus reduce one dimension of the issue,
- No need for a fundamental answer,
- Unlimited environment modeling capability.

So far, various researchers have scaled numerical issues using the scaled boundary finite element method or by combining this method with other numerical methods, issues of water leakage in soil (Efendiev et al. 2006), soil-structure interaction (Genes and Kocak 2005), crack propagation (Ooi et al. 2012) have been successfully modeled by this method. The details of the formulation of this method have been mentioned in previously published papers by various researchers and validations related to the scaled boundary method. This section briefly reviews the basic relationships of the scaled boundary method. The formulation of the scaled boundary method is performed by mapping the Cartesian coordinate system to the scaled boundary coordinate system. So, each point in the new coordinate system can be defined as follows:

$$x(\xi) = x_0 + \xi\, x(\eta) \quad (1)$$

$$y(\xi) = y_0 + \xi\, y(\eta) \quad (2)$$

Where, ξ is the radial coordinate and η is the circumferential coordinate. Also, x_0 and y_0 show the coordinates of the center of scale in the discrete boundary system. The center of the scale should be selected so that the entire domain boundary is visible from that point. In static mode, the scaled boundary method has three matrix coefficients E_1, E_2, and E_0. These matrices of coefficients are defined as follows:

$$[E^0] = \int_{-1}^{+1} [B^1]^T [D][B^1] \,|\,J\,|\, d\eta \quad (3)$$

$$[E^1] = \int_{-1}^{+1} [B^2]^T [D][B^1] \,|\,J\,|\, d\eta \quad (4)$$

$$[E^2] = \int_{-1}^{+1} [B^2]^T [D][B^2] \,|\,J\,|\, d\eta \quad (5)$$

The mapping performed from the Cartesian coordinate system to the scaled boundary coordinate system and the use of virtual work principles weakens the partial differential equation governing the issue to a second-order ordinary differential equation of the Cauchy-Euler type. Eq. 6 represents the resulting differential equation.

$$[E^0]\xi^2 \{u_h(\xi)\}_{\xi\xi} + \left([E^0] + [E^1]^T - [E^1]\right) \left(\{u_h(\xi)\}_{\xi} - [E^2]\{u_h(\xi)\}\right) \quad (6)$$

Eq. 6 is known as the scaled boundary equation in displacement. Solving this differential equation, various methods can be used and have been evaluated in the existing technical literature.

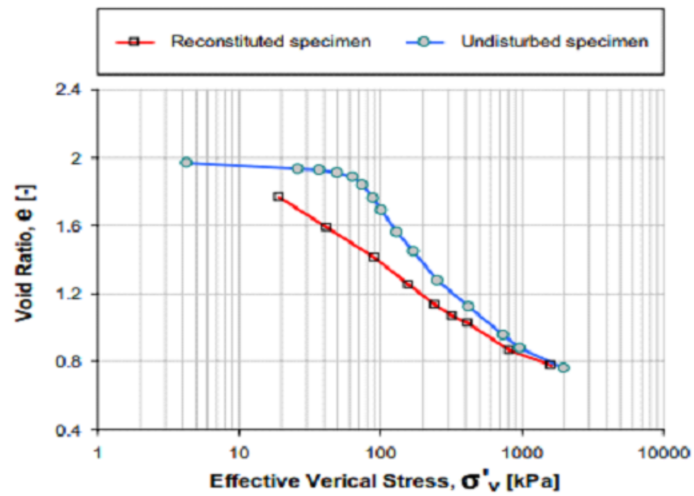

Fig. 1 Chart of changes in the ratio of the porosity to effective stress for soil with Shiraz structure and reconstituted sample in the laboratory (Memarzadeh et al., 2018)

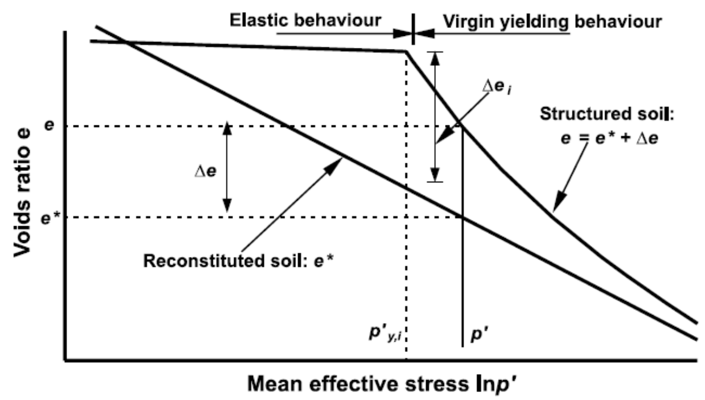

Fig. 2 Ideal isotropic compaction behavior for structured soils compared to unstructured reconstituted soils (Memarzadeh et al., 2018)

III. Modified Cam-Clay Constitutive Model for Structured Soil

At the first ISSMFE conference, Terzaghi stated that the strength of pre-strengthened clays follows the Mohr rupture hypothesis rather than the failure criterion (Terzaghi, 1931). Schofield and Wroth (1968) find this theory of treachery to be wrong and even contrary to what is stated in Coulomb's articles. According to Roscoe et al. (1958) experimental studies unstructured soil is a non-viscous material, so, peak strength is only due to interlocking and inter-friction between particles and not due to adhesion. So that, they considered the use of the Mohr-Coulomb rupture criterion and consequently the Mohr-Coulomb constitutive model in the study of geotechnical issues (Griffiths, 1990; Robert, 2017). For this reason, these researchers began to try to present a new constitutive model based on the concept of the critical state. Roscoe & Poorooshasb (1963) introduced a cam-clay constitutive model. Roscoe & Burland (1968) introduced a modified cam-clay constitutive model. One of the main disadvantages of the cam-clay and the modified cam-clay constitutive models was their formulation based on triaxial tests (and other conventional tests) on reconstructed soil samples that could not show the effect of soil structure (Matsuoka et al., 1999). This constitutive model flaw was not a hidden flaw in the side of view of their developers. Rather, because they focused on understanding the true soil mechanical behavior and saw the structure in the soil as a side effect (not an inherent feature), they deliberately removed the effect of the structure from the experiments. But in the face of the actual soil in the site (not just for theoretical study), the effect of soil structure cannot be ignored.

The modified cam-clay model is a hardening elastoplastic model based on critical limit state theory. The elastic part of the Cam-clay model or the modified cam-clay model is linear. Although the amount of elasticity modulus varies from pattern to pattern, but in each stress or strain pattern, the amount of elastic modulus is assumed to be constant (the same is true of other constitutive models, such as the University of British Columbia sand model). The assumption of logarithmic dependence between the average effective stress and the porosity ratio is considered in deriving the relationships of the modified Cam-clay model. It is largely able to simulate the behavior of clays, however, this assumption is not always true, and sometimes a different power ratio is seen between the average effective stress and the porosity ratio in experiments. The yield function can be expressed as Eq. 7 in the modified Cam-clay constitutive model.

$$f = q^2 + p^2 + M^2 p(p + p_c) = 0 \tag{7}$$

Where q is the deviatoric stress, p is the average stress, pc is the amount of pre-strengthened stress and M is the ratio of the deviation stress to the critical average stress. In the numerical solution of an issue with a hardening elastoplastic constitutive model such as the modified cam-clay model, if an implicit solution is chosen, it is usually necessary to calculate the D^{pl} plastic stress-strain interface matrix. This matrix is expressed for the modified cam-clay model as follows.

$$D^{pl} = \frac{De\dfrac{df}{ds}\cdot\dfrac{df^T}{ds}D^e}{H + \dfrac{df^T}{ds}D^e\dfrac{df}{ds}} \tag{8}$$

Where f is the yield function, D^e is the elastic strain-stress interface matrix, and s is the stress vector. In Eq. 8, for the modified cam-clay model, the value of H, which controls the hardening or softening of the material behavior, is obtained by multiplying the yield function derivative by the plastic strain by the stress yield derivative. As mentioned before, various constitutive models have been proposed to consider the effect of soil structure. The structured modified Cam-clay model proposed by Liu & Carter (2000) leads to many errors in modeling clay-based marl soil. Figure 3 shows a sample of the results of a triaxial test (volumetric strain versus deviatoric strain) and numerical modeling results using the constitutive model presented by Liu and Carter on Marl Corinth soil (Carter et al., 2014; Liu et al., 2021). As can be seen in this figure, the error of the constitutive model in calculating the volumetric strain is not acceptable.

From past to present, one of the ways to modify the existing constitutive models has been to add an experimental corrective part to the constitutive model. For instance, the Finn-Martin or Finn-Berne constitutive models proposed to investigate the liquefaction phenomenon using the Mohr-Coulomb constitutive model in calculating the constant volumetric strain (and corresponding water pressure of cavities) from an experimental formulation they use that help the constitutive model (Soriano Martínez 2015). In this paper, due to the inefficiency of the hardening or softening part of the modified Cam-clay model in modeling the structured soil, a correction factor for the hardening or softening part of the plastic stress-strain interface matrix is presented. Eq. 9 shows the modified state plastic stress-strain interface matrix.

$$D^{pl} = \frac{De\dfrac{df}{ds}\cdot\dfrac{df^T}{ds}D^e}{\alpha H + \dfrac{df^T}{ds}D^e\dfrac{df}{ds}} \tag{9}$$

Where f is the yield function, D^e is the elastic strain-stress interface matrix, s is the stress vector, and alpha is the experimental correction coefficient of the hardening or softening part. According to Figure 2, the behavior of structured soil in addition to non-structured soil parameters should be a function of a series of parameters related to soil structure. Generally, it can be shown that the value of H must be expressed as a function such as Eq. 10.

$$H = \left(s, \varepsilon^p, f, p_{y,i}, p_c, M, \lambda, \omega, \Delta\right) \tag{10}$$

In this regard, ω is a structured soil hardening parameter and can be calculated similarly to what is stated in reference (Kavvadas & Amorosi, 2000). Other parameters are defined as shown in Figure 2. Compared to the value of H for the modified cam-clay model (equation (11)), it can be seen that the non-interference of the parameters $p_{(y,i)}$, p_c, M, λ, ω, Δe causes a large computational error.

$$H = F\left(s, \varepsilon^p, f\right) \tag{11}$$

So, a correction factor including the above parameters in the value of H and reduce the model error is necessary to make a necessary correction. To obtain this correction factor, constitutive model calibration and experimental results were performed. Finally, the alpha correction parameter is defined as follows:

$$\alpha = -\left(\frac{p_{y,i}}{p_c}\right)^2\left(\frac{1}{M}\right)^{-1}\left[(M^2 - \lambda^2)(1 - \omega\Delta e)\right] \quad for\ \ p < p_{y,i} \tag{12}$$

$$\alpha = -\left(\frac{p_{y,i}}{p_c}\right)^2\left(\frac{1}{M}\right)^{+1}\left[(M^2 - \lambda^2)(1 - \omega\Delta e)\right] \quad for\ \ p > p_{y,i} \tag{13}$$

In Eqs. 12 and 13, M is the slope of the critical boundary line in the space q-p', λ is the slope of the consolidation line of the reconstructed soil sample under loading conditions, and ω is a parametrically structured soil that can be obtained experimentally. In these relationships, the parameters defined by Liu & Carter (2003) have been used. The other parameters in Eq. 2 are shown in Figure 2. To validate the modified formulation and the effect of the experimental coefficient to improve the soil hardening or softening behavior, the results of numerical modeling and triaxial experiments on Marl Corinth soil with the assumption of confining stress of 4000 kPa are presented in Figure 4. The characteristics of the mechanical properties of Marl Corinth soil are presented in Table 1.

Numerical modeling was performed for the confinement stress of 1500 kPa and the results were compared with experimental results. Figure 5 shows a good agreement between the numerical and experimental results in this case. Numerical modeling was performed for the confinement stress of 903 kPa and the results were compared with experimental results. Figure 6 shows an improvement in the accuracy of the constitutive model results compared to that presented by Liu & Carter (2000).

IV. Numerical Verifications

For surface settlement numerical modeling due to tunnel drilling in structured clay, a structure with mechanical properties of Marl Corinth soil is considered. As shown in the previous session, the modified constitutive model presented in this paper has been validated for the results of the Marl soil triaxial test. Figure 7 shows a face of the geometric characteristics, the location of the tunnel, the dimensions of the tunnel, and the cross-section of the mesh used for this issue. The scaled boundary finite element method has been used to eliminate the effect of fundamental boundary conditions on the recorded response.

Table 1 Marl Corinth soil mechanical properties

$P_{y,i}$ Kpa	ω	b	ν	e_{IC}	κ	λ	M
3800	4.9	0.4	0.25	0.775	0.008	0.04	1.38

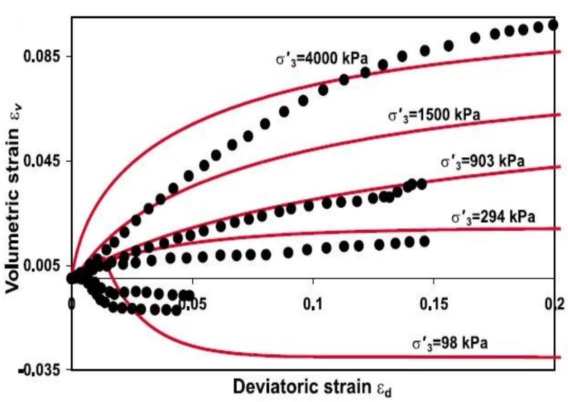

Fig. 3 Diagram of volumetric strain versus deviatoric strain for Marl soil with Corinthian structure (note: dotted line of experimental results and the red line of numerical modeling results) by Liu & Carter (2000)

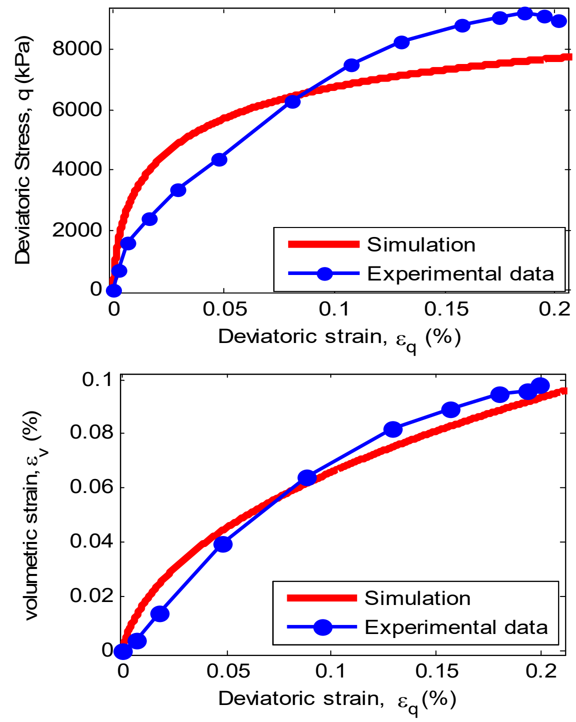

Fig. 4 Comparison of numerical modeling results and experimental results for Marl Corinth soil assuming a confinement pressure of 4000 kPa; laboratory results are extracted from Liu & Carter (2000)

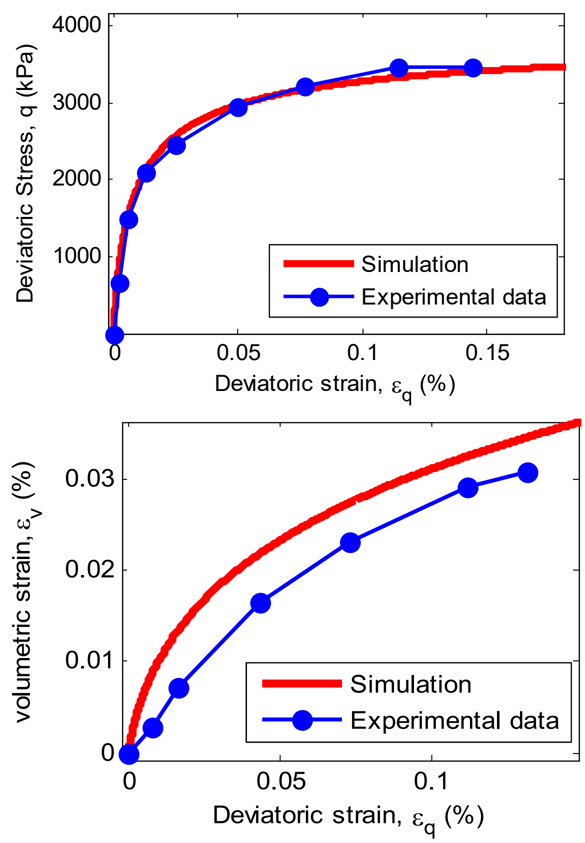

Fig. 5 Comparison of numerical modeling results and experimental results for Marl Corinth soil assuming a confinement pressure of 1500 kPa; laboratory results are extracted from Liu & Carter (2000)

Assuming TBM drilling, the drilling operation in the written program is modeled in several stages. At each stage of modeling, it is assumed that 1.5 m of drilling will be performed. The analytical method presented by Sagaseta et al. (1998) has been used to validate the results. Eq. 4 represents the formula proposed by Sagaseta to estimate the amount of vertical settlement of the soil surface due to surface tunnel drilling (González & Sagaseta, 2001).

$$uy = \frac{V_s h}{\pi(h^2 + x^2)} \tag{14}$$

The depth of the center of the tunnel to the ground is indicated by the h, the horizontal distance from the center of the tunnel is indicated by the x, and the y also indicates the vertical direction. The reason for selecting this relationship to estimate the vertical settlement of the earth surface is that this relationship is basically for estimating the settlement of the soil surface resulting from the excavation of surface tunnels and not deep tunnels. In this relation, Vs indicates the percentage of soil volume loss due to drilling. The value of V_s depends on the type of soil, the drilling method, and the depth of the tunnel, and for hard clays (such as clay-based marl soils (Arifuzzaman et al., 2017)) a value close to one is usually considered by various researchers (e.g., for London marl soils (Loveland, 1984)).

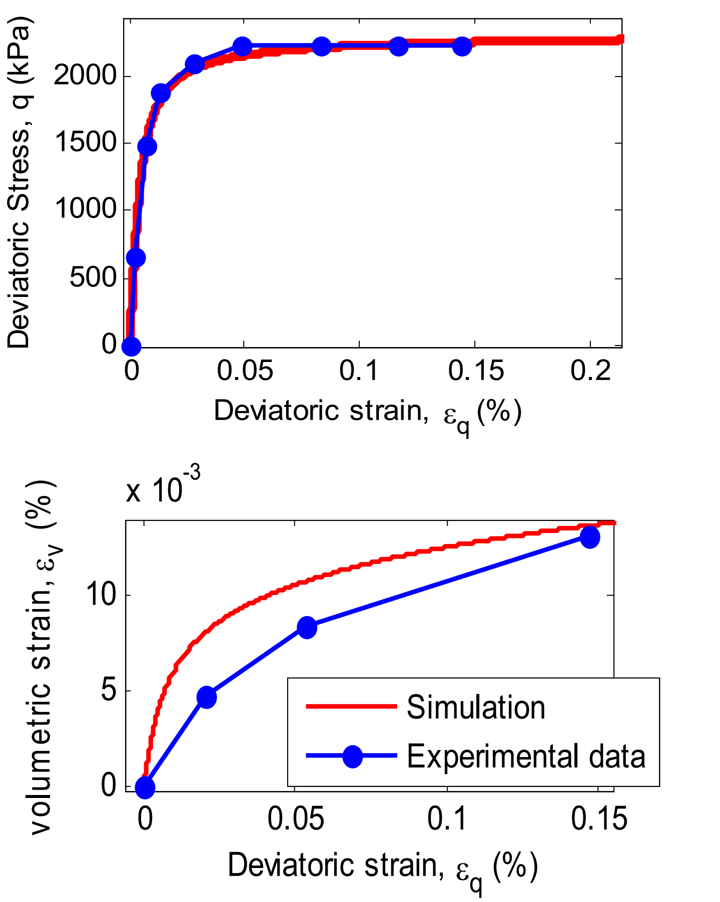

Fig. 6 Comparison of numerical modeling results and experimental results for Marl Corinth soil assuming a confinement pressure of 903 kPa; laboratory results are extracted from Liu & Carter (2000)

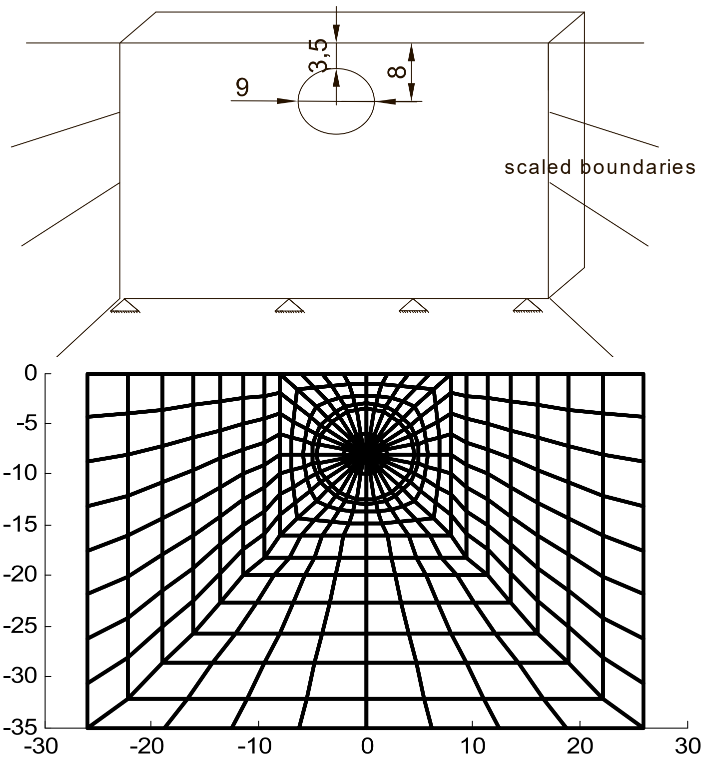

Fig. 7 Location of the tunnel at the site and a cross-section of the mesh used to solve the issue

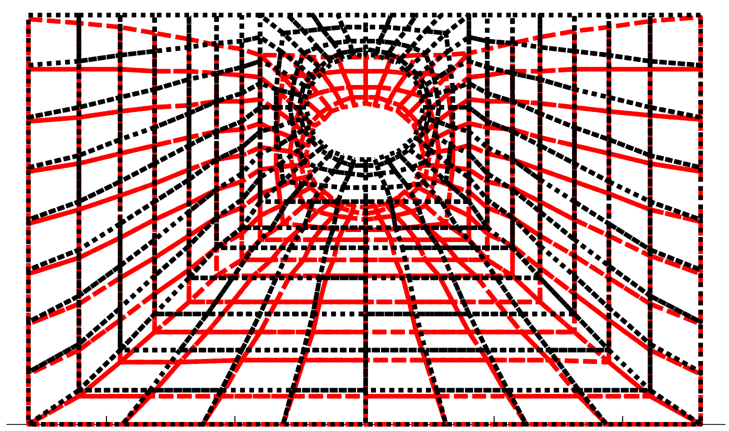

Fig. 8 How the system deforms due to surface tunnel drilling in Marl Corinth (structured) soil (deformations shown with 100x magnification)

The modeling process has been applied using the program written in MATLAB as described below. At first, the soil environment was modeled in 3D without any drilling and the values of soil residual stresses were calculated. In each drilling step, the number of forces applied to the system is calculated and the equilibrium equation is solved by using the numerical method of finite element-scaled boundary finite elements. It should be noted that although only gravitational forces are applied to the system after drilling and completion of the tunnel, during drilling and excavation part of the existing soil due to system imbalance, in addition to gravitational forces, unbalanced interaction force between soil excavation part and the rest of the soil should also be applied as part of the external force in the system (tunnel wall).

Finally, the values of the strains and displacements are calculated, by implicitly integrating the stress and modifying the stress values at each step. By performing tunnel drilling numerical modeling, the obtained results are explained in the following. Figure 8 shows how the system deforms due to tunnel drilling. Figure 9 displays the amount of deformation of the soil surface, which is obtained from both analytical and numerical methods. As this figure illustrates, the results of numerical modeling are closely related to the results of the analytical method and have been calculated with sufficient accuracy. Figure 10 shows the contour of the vertical deformation for a site section. The vertical deformation contour for the ground surface after completion of 6 m of drilling is also displayed in Figure 11. The values of vertical deformation in the tunnel wall after the assumed 6 m drilling are presented in Figure 12.

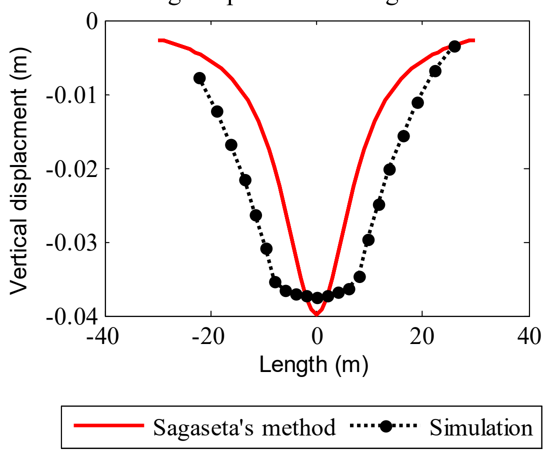

Fig. 9 Comparison of surface settlement values due to tunnel drilling in both numerical and analytical methods

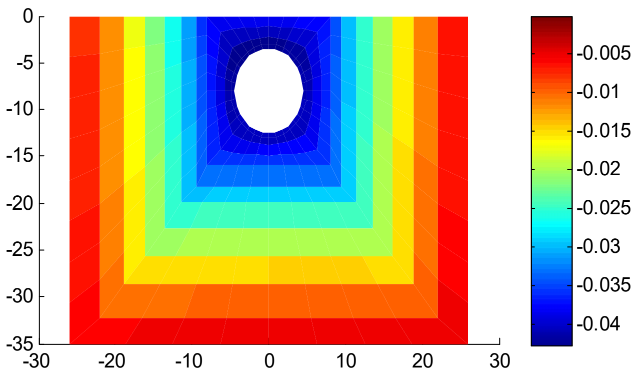

Fig. 10 Vertical deformation contour due to tunnel drilling

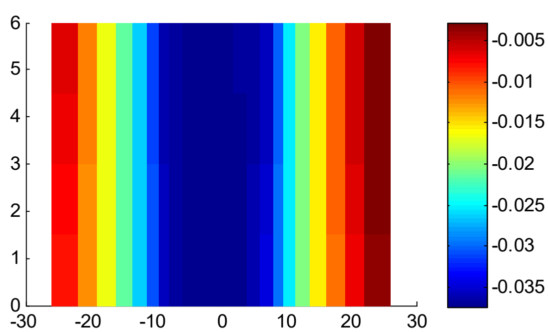

Fig. 11 Vertical deformation contour at the ground surface after 6 m completion drilling

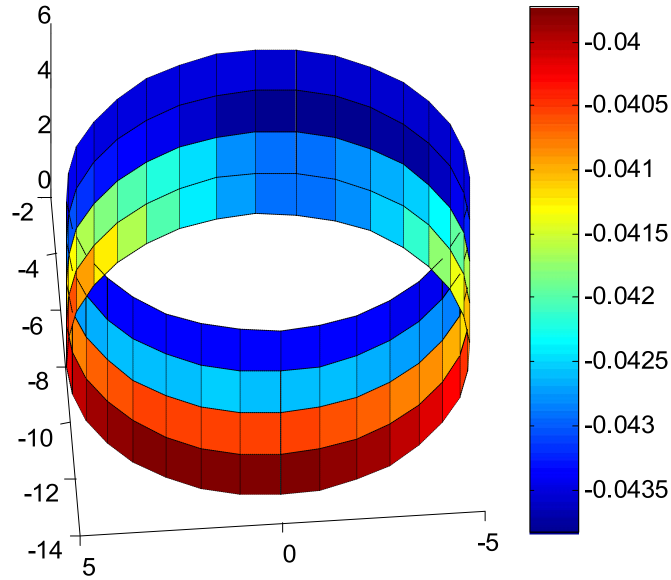

Fig. 12 Vertical deformation contour in the tunnel wall after 6 m completion drilling

V. Conclusion

Due to the unacceptable amount of error in the modified cam-clay model for structured soil in calculating volumetric strain values, it is necessary to modify this model to improve performance. In this paper, a modification coefficient for the hardening or softening part of the plastic stress-strain interface matrix was proposed in a modified Cam-clay constitutive model based on structured soil parameters. Comparison of numerical modeling and the results of the triaxial experiment on marl corinth soil (very strong structured soil) showed that the performance of the modified constitutive model is much improved compared to the initial model for measuring stress values and for measuring volumetric strain values. Such a modification that does not increase the parameters of a constitutive model and greatly improves results has not been proposed before. The surface tunnel drilling operation in structured clay (marl corinth soil) was modeled in 3D by a finite element - boundary finite element program, after validating the constitutive model with the proposed corrections. The combined method of finite element-scaled finite element in the 3D model for tunnel drilling has not been used in papers and research. Comparison of ground settlement values due to tunnel drilling between numerical and analytical methods showed that the proposed model is well able to simulate the issue of tunnel drilling in structured soil. It should be noted that the simulation of applied geotechnical issues in such structures has been very rare in the technical literature, due to the lack of appropriate constitutive models for structured soils in commercial software.

Acknowledgment

We extend our thanks to the reviewers for their meticulous attention to detail and constructive suggestions that greatly improved the quality of this manuscript. Your contributions have been instrumental in shaping this work.

Authors' contributions

Negar Salehi Alamdari conducted the main data analysis, contributed to the data collection, preprocessing, and interpretation, and was responsible for drafting the initial manuscript. Hamid Reza Azizi Pestehbaglo assisted in the development of the methodology and performed validation checks, provided supervision, conceptual guidance, and critical revision of the manuscript. All authors read and approved the final manuscript.

Conflict of interest

The authors have not disclosed any competing interests.

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