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TWO-DIMENSIONAL NUMERICAL APPROACHES OF EXCAVATION SUPPORT SYSTEMS: A COMPREHENSIVE REVIEW OF KEY CONSIDERATIONS AND MODELLING TECHNIQUES

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ABSTRACT. This paper provides an overview of the characteristics of soil structure in excavation support systems and the use of numerical modeling to analyze these systems. It highlights the limitations of two-dimensional (2-D) analysis in capturing corner effects in deep excavations. The abstract emphasizes that soil is a complex material with time-dependent, nonlinear behavior affected by stress history. Various constitutive models are available to simulate soil behavior, including linear and nonlinear elastic models, elastic-perfectly plastic models, elasto-plastic models, and elastic-plastic models with kinetic hardening. The advantages and limitations of these constitutive models are discussed, with a focus on their ability to accurately represent soil behavior. The Mohr-Coulomb model is mentioned as a popular choice despite its limitations. Additionally, the availability of numerical software applications like PLAXIS, specifically PLAXIS 2-D, for geotechnical analysis is highlighted. Overall, this paper provides a concise summary of the characteristics of soil structure in excavation support systems, the importance of numerical modeling, and the different constitutive models used to simulate soil behavior in geotechnical analyses.

KEYWORDS: Excavation support systems; Numerical modeling; Two-dimensional analysis; Constitutive models; Mohr-Coulomb failure criterion.

1. INTRODUCTION

Excavation support systems are essential in construction projects involving deep excavations to ensure the stability and safety of surrounding structures and the excavation itself [1-2]. Numerical modeling plays a crucial role in understanding the behavior of soil structures within these support systems [3-4]. To accurately simulate soil behavior, various constitutive models are available. The choice of a suitable model depends on the specific geometry of the situation and the type of soil being analyzed [5]. The paper provides an overview of different constitutive models, including linear and nonlinear elastic models, elastic-perfectly plastic models, elasto-plastic models, and elastic-plastic models with kinetic hardening. Each model has its advantages and limitations in representing soil behavior under different loading circumstances. The advancement of numerical methods, supported by hardware and software developments, has facilitated the analysis of geotechnical challenges using numerical modeling [6]. Constitutive soil models have also evolved, allowing for more accurate simulations of soil behavior. The continuous development of these models has contributed to the progress and cost-effectiveness of complex numerical studies in geotechnical engineering [7].

The objective of this paper is to provide a comprehensive overview of the characteristics of soil structure in excavation support systems, the importance of numerical modeling in analyzing these systems, and the different constitutive models used to simulate soil behavior. By understanding these key considerations and modeling techniques, researchers and practitioners in geotechnical engineering can make informed decisions when analyzing excavation support systems and ensure the stability and safety of construction projects. In the subsequent sections of the paper, the authors delve into the specific details of numerical modeling techniques, limitations of 2-D analysis, constitutive models of soil behavior, and the availability of software applications like PLAXIS 2-D for geotechnical analysis. The paper aims to provide a comprehensive and informative resource for researchers, engineers, and practitioners involved in excavation support systems and geotechnical analysis.

2. Numerical Modeling

The characteristics of soil structure in excavation support systems could be investigated using numerical modeling, which also provides all the necessary data for the intended objectives [4, 8]. Depending on the specifics of the geotechnical issue, the numerical analysis might be performed in a two- or three-dimensional space. In most cases, excavation issues are evaluated using two-dimensional models, such as axisymmetric analysis and plain strain [9-11]. However, the limitations of 2-D analysis should be understood, and if necessary, totally 3-D investigations are needed [10]. In deep excavations, for example, the corner effects cannot be taken into account by 2-D modeling [2]. Consequently, in this case, the corner of the wall exhibits less noticeable deformation and ground movement compared to its center [1].

The soil found in nature is an anisotropic substance with many phases. Time, nonlinearity, route dependence, and stress history all play a role in its loading responses [10]. Deflections might dilate or compact and involve irreversible plastic strains. Theoretically, the ideal soil model would be able to forecast these soil responses under various loading circumstances. Typically, the soil model adopted depends on the actual situation geometry and soil type [5]. For addressing the various soil challenges, there are a number of constitutive models available. A basic description of these models will be given in the next section.

3. Constitutive Models of Soil Behavior

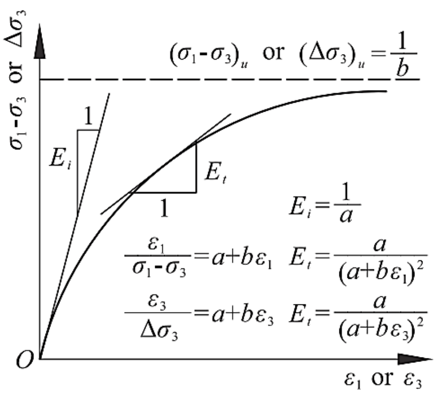
In recent years, the advancement in both hardware and software has enabled researchers to tackle numerous geotechnical challenges using numerical methods. This progress has made complex numerical studies more feasible and cost-effective. Additionally, the continuous development of constitutive soil models has played a significant role in these advancements. These models allow for the simulation of soil behavior using various approaches. Some models are specifically designed for a particular type of soil or a specific study, while others have broader applications and can be used for both cohesive and non-cohesive soils. To categorize the constitutive models with practical applications, several classifications have been proposed [8, 10, 12]:

1) Linear and non-linear elastic models: These models assume that soil behaves elastically within a certain range of stresses. They can capture the linear and non-linear response of soil under loading.
2) Linear elastic-perfectly plastic models: These models combine linear elasticity with perfect plasticity assumptions. They are suitable for studying soil behavior under cyclic loading conditions and can simulate the permanent deformation of the soil.
3) Elasto-plastic models: Elasto-plastic models consider both the elastic and plastic behavior of soil. They account for the progressive accumulation of plastic strains and are commonly used to analyze soil under monotonic loading.
4) Elastic-plastic models with kinetic hardening: These models incorporate additional features such as strain hardening or softening. They can capture the evolution of soil behavior over time, including the effects of cyclic loading and strain accumulation.

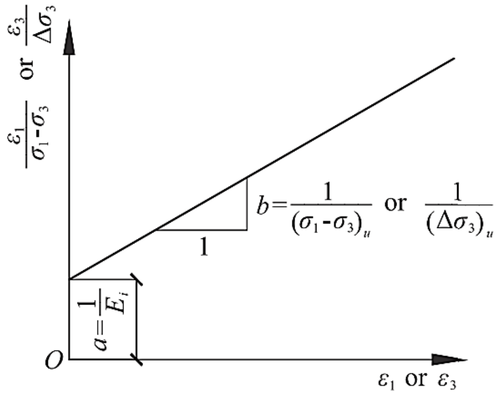
These constitutive models provide valuable tools for researchers and engineers to analyze and simulate the behavior of soil under different loading conditions. The choice of a specific model depends on the nature of the geotechnical problem at hand and the desired level of accuracy and complexity required for the analysis.

The first category of constitutive models, known as "isotropic elasticity," is based on Hooke's law and utilizes the principles of elasticity. This model characterizes the material by parameters such as the Poisson's ratio, bulk modulus, or shear modulus for soil skeletons. Classical soil mechanics has made extensive use of this method for analytically treating boundary value issues because of its simplicity. When the finite element approach was first developed, the elastic model was also used to represent soil. However, it has been found to be inadequate for accurately capturing the complex behaviors exhibited by real soils. Therefore, it is not suitable as a comprehensive soil model for most applications. Nevertheless, the elastic model can still be utilized to simulate rigid structures embedded in the soil, such as bored piles or concrete diaphragm walls. In such cases, the elastic model provides a reasonable approximation for the behavior of these rigid elements within the soil mass. It is important to note that when studying soil behavior, more sophisticated and realistic constitutive models are typically used to accurately capture the complexities observed in real soils. These models consider factors like soil non-linearity, plasticity, and other soil-specific characteristics, thereby providing more accurate representations of soil behavior under various loading conditions.

Unlike the linear model, the nonlinear elastic model takes into account the nonlinear relationship between shear stress and shear strain. Condner and Zelasko's hyperbolic model, which they presented in 1963, is a well-known example of this kind. As shown in Fig. 1a, the shear modulus in the hyperbolic model goes from a starting value to zero when the model fails. This shear behavior closely resembles the actual shear curves observed in loose sands and normally consolidated clays. The incorporation of the hyperbolic model into finite element software was first accomplished by Duncan and Chang in 1970 [13]. They combined the concept proposed by Ohde in 1939, which relates soil stiffness to stress dependence, with power law and Kondner's approach of approximating the stress-strain curve from drained triaxial compression tests using the hyperbolic technique. By doing so, Duncan and Chang developed a new theory [13]. The hyperbolic model, depicted in Fig. 1b, requires two parameters that can be determined from experimental results. In summary, the nonlinear elastic model, exemplified by the hyperbolic model, offers a more accurate representation of the nonlinear shear behavior of soils. Duncan and Chang's incorporation of this model into finite element software has provided a valuable tool for analyzing soil behavior, and the necessary model parameters can be determined through appropriate experimental testing.



(a)



(b)

Fig. 1. *Duncan-Chang constitutive model (a) Hyperbolic form and (b) Linear form [9].*

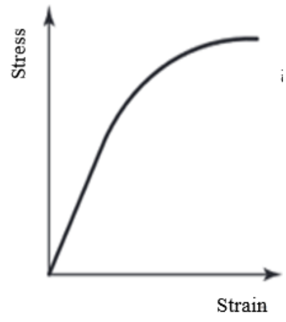
Nonlinear elastic models have the capability to accurately replicate the monotonic stress-strain curves observed in experimental tests like triaxial and oedometric tests, for certain loading patterns. However, these models face limitations when it comes to extrapolating beyond the range of calibration curves. They are unable to capture other important aspects of soil behavior, such as stress path dependence and volume change during shear [10]. Similar to linear elastic models, nonlinear elastic models have their drawbacks. For instance, they do not exhibit hysteretic behavior during cyclic loading. Additionally, unlike linear elastic models, nonlinear elastic models lack a strong theoretical foundation to support their assumptions and formulations [10]. It is important to acknowledge that while nonlinear elastic models can provide reasonable representations of certain soil behaviors, they have inherent limitations and may not fully capture the complexities and intricacies of soil response. To overcome these limitations and achieve a more comprehensive understanding of soil behavior, more advanced constitutive models that consider additional factors, such as plasticity, strain hardening, and stress history dependence, are often employed. These models aim to provide a more accurate representation of soil behavior under various loading conditions and are continuously evolving as research in geotechnical engineering progresses.

The elastic-perfectly plastic model, commonly referred to as the Mohr-Coulomb model, is a highly prevalent and extensively utilized constitutive model within the field of geotechnical engineering. It is a relatively simple model that combines Hooke's Law for elasticity with Coulomb's failure criterion in its generalized form. The elastic-perfectly plastic model is often considered adequate for various geotechnical problems, especially when utilized by knowledgeable and experienced users. For instance, this model can be applied to predict the deformation of a diaphragm wall resulting from excavation activities. Studies such as those conducted by Lim et al. [14] and Phienwej [15] have demonstrated the applicability of the elastic-perfectly plastic model in such scenarios. Nevertheless, extreme caution must be exercised when employing this model, specifically when handling delicate clay. The stress path predicted by the model may be misleading and can lead to an overestimation of the soil strength [10]. Therefore, it is crucial to carefully consider the specific characteristics of the soil and the limitations of the model when applying the elastic-perfectly plastic model in geotechnical analyses. While the elastic-perfectly plastic model has its advantages in terms of simplicity and ease of use, it is essential to recognize its limitations and potential pitfalls. Depending solely on this model without considering other factors and more advanced constitutive models may lead to inaccurate predictions and interpretations of soil behavior.

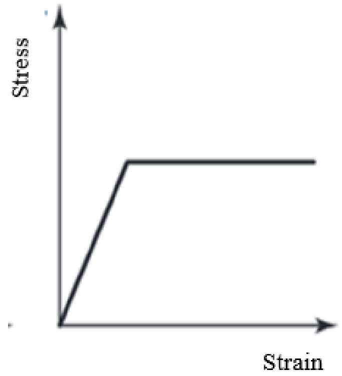
The isotropic hardening single surface model and the isotropic hardening double surface model are two elastoplastic models that geotechnical engineers use in their project planning and monitoring. These models comprise the third category of constitutive models. The isotropic hardening single surface plasticity model is the initial step in simulating the actual behavior of soils, and the Modified Cam-Clay (MCC) model serves as the fundamental soil model in this group [10]. This model distinguishes between elastic and plastic behavior using an elliptic yield surface. It is commonly used, particularly in the modeling of embankments in soft clay. Therefore, the soil's stress route usually stays within the yield surface when unloading is involved, such in an excavation. The anticipated deformations that occur during excavation are therefore determined by the elastic behavior [10]. A recognized instance of an isotropic hardening double surface plasticity model is the Hardening Soil (HS) model, which was derived from Vermeer's double hardening model [16]. This model provides more accurate displacement patterns under working stress conditions, particularly in the context of excavations. The Hardening Soil (HS) model has been updated to include more details about how soil behaves when subjected to minor stresses; this new version is called the Hardening Soil with minor Strain Stiffness (HSsmall) model [17]. These elasto-plastic models, including the MCC model and the Hardening Soil models, offer improved capabilities for capturing the nonlinear behavior of soils, including plasticity and hardening. They are particularly useful for analyzing geotechnical problems involving large deformations and complex stress paths. The modifications made to the Hardening Soil model to account for small strain stiffness allow for a more accurate representation of soil behavior under small strain conditions. It is important to note that while elastoplastic models provide a more realistic representation of soil behavior compared to elastic models, they still have limitations and may require calibration with experimental data to accurately capture specific soil characteristics.

The fourth group of constitutive models in geotechnical engineering includes plasticity models with kinematic hardening and multiple surfaces. These models have the capability to represent soil softening, small strain behavior, and anisotropy, among other features. One such model is the Kinematic Hardening model, which includes the KH and 3-SKH surfaces [18-20]. By applying the flow rule at the yield surface, these models are adapted from the Cam-Clay model. They assume linear behavior within the elastic (recoverable) state. A more sophisticated soil model, the MIT-E3 Model [21] incorporates additional assumptions such non-associated flow rules and non-linear behavior in the recovered state (elasticity state). It is important to note that these models require a large number of complex input parameters. Traditional soil testing methods may not be able to provide all the necessary data for these models. Therefore, specialized testing techniques or parameter estimation methods, such as back-analysis, may be required to obtain the input parameters. These plasticity models with kinematic hardening and multiple surfaces offer more advanced capabilities for capturing the complex behavior of soils. They allow for the representation of phenomena such as soil softening, anisotropy, and small strain behavior. However, their complexity and data requirements should be carefully considered and appropriate methods should be employed to ensure accurate parameter estimation.

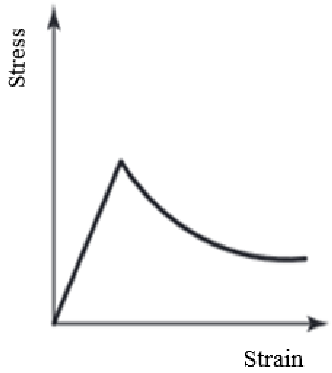
To summarize, one of the most accurate techniques for simulating soil behavior is to assume an initial elastic deformation up to the yield point, followed by plastic deformation until failure (Fig. 2). This approach recognizes that the soil can exhibit various responses beyond the yield point, such as strain hardening, strain softening, or perfectly plastic behavior [22]. By incorporating the concepts of elastic and plastic behavior, this technique allows for a more realistic representation of soil response. It acknowledges that soils can undergo both recoverable elastic deformations and irreversible plastic deformations, which are typically observed in geotechnical engineering applications. However, it is important to note that the accuracy of the simulation depends on the selection of an appropriate constitutive model and the availability of accurate input parameters. Different soil types and loading conditions may require specific models that can capture the particular behavior exhibited by the soil in question. Overall, adopting an approach that combines elastic and plastic deformation provides a more comprehensive understanding of soil behavior and enables more accurate predictions of soil response in geotechnical analyses.



(a) Strain Hardening



(b) Perfectly Plastic Hardening



(c) Strain Softening

Fig. 2. Plastic models [22].

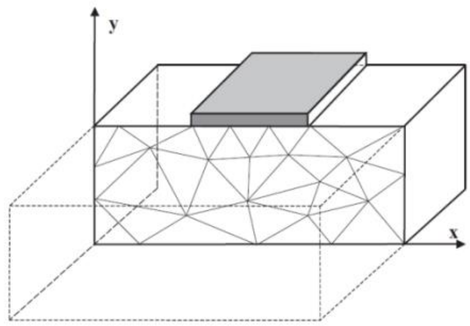
In spite of its apparent limitations, the Mohr-Coulomb (MC) model continues to be extensively utilized in traditional excavation design and research [23-25]. The primary rationale for this is the considerable convenience it provides, as the necessary soil parameters can be acquired via conventional in-situ or laboratory experiments, in addition to empirical correlations. Consequently, the MC model is considered user-friendly. Additionally, it proves to be highly advantageous when a simplified approach is preferred, particularly in cases where detailed laboratory or field assessments of the soil are lacking.

In addition, the Mohr-Coulomb model has been extensively used in many other types of study [26-29] to depict the drainage characteristics of granular soils. This underscores the extensive adoption and recognition of it within the geotechnical community. Furthermore, this version of the constitutive model is supported by the vast majority of finite element applications used in geotechnical investigations.

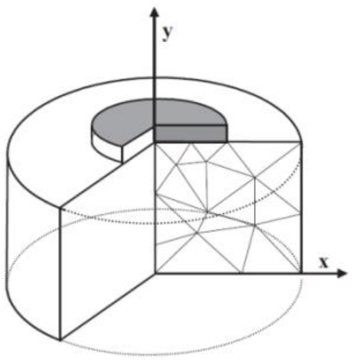
The field of geotechnical engineering has witnessed significant progress in computational methods, leading to the development of various numerical applications capable of solving complex problems in two or three dimensions. Notable applications in this domain include PLAXIS, FLAC, DIANA, Geo5, and ABACUS. According to Huat et al. [30], PLAXIS is widely considered to be one of the most important applications of finite element methods. Its reputation reflects its extensive use and effectiveness in tackling geotechnical challenges.

3.1. PLAXIS 2-D

Designed for use in geo-engineering tasks like tunneling, foundations, and excavation, the geotechnical finite element software PLAXIS allows for both two- and three-dimensional assessments of soil structure deformation and stability [31]. An accurate vertical soil cross-section might be used to create a geometry model and finite element mesh using the PLAXIS graphical user interface. An array of building stages and types of analyses may be modeled using PLAXIS's staged construction capabilities. The PLAXIS 2-D program allows users to create axisymmetric or flat strain models to depict any real-world scenario. Since stresses in the z-direction (namely, perpendicular to the cross-section) are assumed to be zero, a plane strain model might be used in geometrics with a uniform cross-section. However, the z-direction normal stresses are included by the model. An axisymmetric model is used when thinking about things that are circular, since stress and strain are thought to be the same in all the radial directions. PLAXIS [32] explains the axisymmetric and plane strain models in Fig. 3.



(a) Plane Strain



(b) Axisymmetric

Fig. 3. Example of PLAXIS Problems [32].

Through the utilization of PLAXIS, the user is exposed to a variety of seven unique soil models. This category comprises the following models: NGI ADP, Soft Soil (SS), Soft Soil Creep (SSC), Linear Elastic (LE), Mohr-Coulomb (MC), Hardening Soil (HS), and Hardening Soil with Small-Strain Stiffness (HSsmall).

4. Mohr-Coulomb Failure Criterion

The Mohr-Coulomb approach is widely recognized and commonly used in geotechnical engineering due to its effectiveness. However, it is important to acknowledge that this approach does have several significant drawbacks when compared to more complex soil models, as outlined by Choo [25]. One of the limitations of the Mohr-Coulomb model is its inability to accurately represent the behavior of contractive or loose soils. The failure criteria of the Mohr-Coulomb model may not align well with the characteristics and response of such soils. Consequently, when dealing with contractive soils, the predictions and analysis based on the Mohr-Coulomb model may not be as accurate or reliable. Another drawback of the Mohr-Coulomb model is its failure to capture plastic deformations under cyclic stress conditions. The model assumes that the soil remains perfectly elastic, which may not reflect the actual behavior of soils subjected to cyclic loading. This restriction can lead to limitations in predicting the response and performance of soils in cyclic stress conditions. However, it is important to note that in the investigation mentioned, these limitations of the Mohr-Coulomb model are not relevant. The study focuses on dense and medium-density sand subjected to monotonic static loads, which eliminates the concerns associated with contractive soils and cyclic stress. In this specific context, the Mohr-Coulomb model is considered appropriate and was utilized in the study. Overall, while the Mohr-Coulomb approach is widely used and effective in many cases, it is essential to be aware of its limitations and consider more complex soil models when dealing with contractive soils or cyclic stress conditions. In situations where the specific limitations of the Mohr-Coulomb model do not apply, it remains a valuable tool for analyzing and predicting the behavior of soils.

The Mohr-Coulomb model is widely utilized as an initial approximation for describing and analyzing soil behavior. It provides a simplified stress-strain relationship that follows a linear pattern in the elastic range under normal loading conditions. This relationship is governed by two parameters: Young's modulus (E) and Poisson's ratio (ν), which are derived from Hooke's law and characterize the material's elasticity. In addition to the stress-strain relationship, the Mohr-Coulomb model incorporates a failure criterion defined by two parameters: the friction angle (φ) and the cohesion (c). These parameters determine the soil's resistance to shear and its ability to sustain stress without failure. The friction angle represents the internal friction of the soil, while the cohesion accounts for any cohesive forces present. The flow rule in the Mohr-Coulomb model is described by the dilatancy angle (ψ). This angle represents the change in volume caused by shearing and accounts for the irreversible deformation of the soil during the shearing process. Including the dilatancy angle allows the model to simulate the realistic behavior of soils in terms of volume change. References such as Ti et al. [33] and Rahman [10] further discuss and provide insights into the application and parameters of the Mohr-Coulomb model in soil mechanics and geotechnical engineering.

According to Briaud [22], the failure criteria of the model depicted in Fig. 4 can be expressed using the following equation:

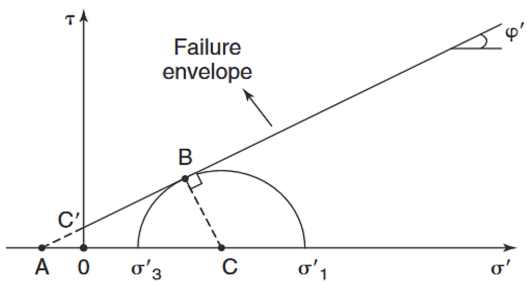
$$\tau_f = \sigma'_{nf} \tan\emptyset' + c' \qquad (1)$$

In this equation, τ_f represents the shear stress at failure, σ'_{nf} is the effective normal stress at failure, φ' is the effective friction angle, and c' is the effective cohesion. This equation describes the relationship between shear stress, effective normal stress, friction angle, and cohesion in the model, allowing for the determination of failure conditions in the analyzed materials.

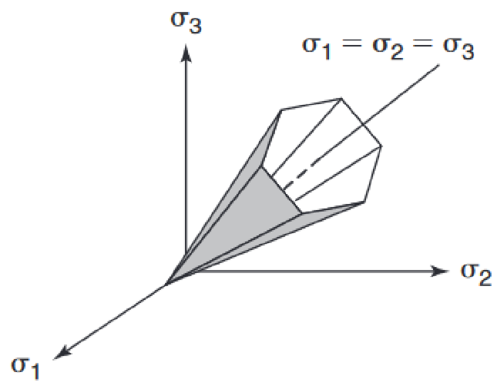
According to Briaud [22], when expressed in terms of effective main stress, the Mohr-Coulomb yield function takes on the following form:

$$\tau_f = 0.5(\sigma'_1 - \sigma'1) + (\sigma'1 + \sigma'1)\sin\emptyset' + c'\cos\emptyset' \qquad (2)$$

In this context, σ′1 represents the main effective principal stress while σ′3 stands for the minor effective principal stress.



(a) Failure envelope



(b) Mohr-Coulomb yield surface

Fig. 4. Mohr-Coulomb yield criterion [22].

***Table 1:** Inputs description of the Mohr-Coulomb approach.*

Var.	Description	Analysis of Variables	Ref.
$\emptyset'$	Interfriction angle	Failure line slope estimated using the MC failure criteria	[5]
c'	Cohesion	Based on the MC criteria, the y-intercept of the failure line	[5]
ψ	Dilatancy angle	Function of axial strain (ε_a) and volumetric strain (ε_v)	[10]
E'	Reference secant stiffness from drained triaxial test or plate loading test	Secant modulus at 30:50% strength, can be calculated as shown in Fig. 1.	[22]
υ	Poisson's ratio	0.3-0.4(drained), 0.495(undrained), 0.15-0.25(unloading)	[10]
K_o	Lateral earth pressure coefficient at rest	$1 - \sin\emptyset'$ (basic configuration)	[34]

4.1. YOUNG'S MODULUS (E)

One way to quantify a material's resistance to deformation is by evaluating its modulus of elasticity, which is also called its shear modulus or Young's modulus. An essential stiffness modulus in soil mechanics, Young's modulus establishes a relationship between strain and stress. In other words, it's the ratio of a material's normal strain to its normal stress. As a measure of the material's stiffness under shear loading circumstances, the shear modulus is often used in soil mechanics. It is determined by dividing the shear stress by the shear strain. For soil loading conditions, it is often suitable to use the secant modulus at 30% to 50% of the material's strength. This secant modulus provides an average measure of stiffness at a specified level of loading. As seen in Fig. 5, the modulus of subgrade reaction, abbreviated "ks," may be determined by using the procedures outlined in Lin et al. [35] and ECP-202 [36]. The specific equations and procedures for determining the value of "k_s" are provided in these references. One of the most crucial parameters in soil-structure interaction study is the modulus of subgrade reaction, which is used to determine how foundations and pavements respond to loads.

$$k_s = \frac{q_a}{\delta_a} \quad (3)$$

$$q_a = \frac{q_u}{f.s} \quad (4)$$

In this formula, q_a stands for the permissible bearing capacity, δ_a for the allowable settlement that corresponds to q_a, q_u for the ultimate bearing capacity, and f.s for the safety factor. The factor of safety is typically chosen to ensure a sufficient margin of safety in the design, and a commonly used value is 3.0. By substituting the values of q_a, δ_a, q_u, and *f.s* into the equation, the modulus of subgrade reaction, k_s, can be determined. The modulus of subgrade reaction is a parameter used to characterize the stiffness of the soil in foundation design and analysis, particularly in relation to the settlement and bearing capacity of the soil.

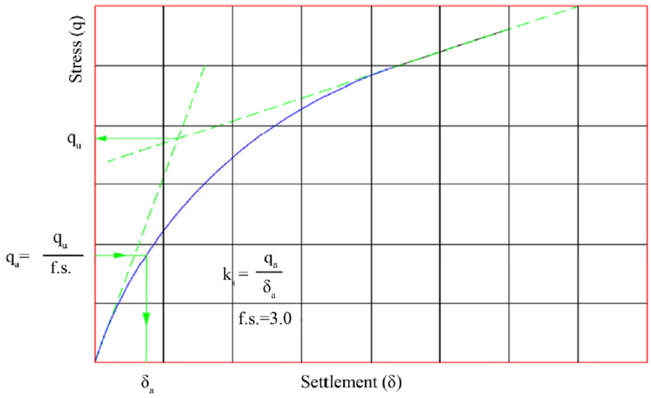


Fig. 5. Determination of subgrade reaction "k_s" [37].

The above equation can be used to get the modulus of subgrade response, abbreviated as ks, and it is derived from the theory of elasticity for a rigid plate applied to a semi-infinite elastic soil subjected to a concentrated force [38-39]. The modulus of subgrade reaction, ks, can be calculated using the expression 1.13 times the Young's modulus, E, divided by the quantity (1 minus the Poisson's ratio, ν, squared), multiplied by the reciprocal of the square root of the area, A. This equation provides a mathematical relationship for estimating the ks value, which is a crucial parameter in analyzing the behavior of soil and its interaction with rigid plates subjected to concentrated loads. From Equations 3, 4 and 5, Young's modulus can be calculated.

$$k_s = 1.13 \times \frac{E}{1-\upsilon^2} \times \frac{1}{\sqrt{A}} \quad (5)$$

4.2. POISSON'S RATIO (υ)

There is a narrow range of 0.3 to 0.4 for the drained Poisson's ratio of soils (**υ**) under loading conditions [40]. While being unloaded, the values range from 0.15 to 0.25. Poisson's ratio for an undrained state is 0.5. It is numerically challenging to use an undrained Poisson's ratio of 0.5, thus = 0.495 is recommended instead. According to Bishop and Hight [41], obtaining accurate readings of strain and/or the calibration relationship is one of the primary challenges in determining Poisson's ratio.

4.3. COHESION (C') AND FRICTION ANGLE (∅′)

Cohesion is given in stress units, denoted by the symbol "c'". To reduce computational mistakes while using the PLAXIS program, it is advised to use a low cohesion value (c' > 0.2 kN/m²) even for cohesionless materials (c' = 0) [42, 10]. As shown in Fig. 4, the Mohr-Coulomb failure criteria may be used to compute the friction angle (Ø′) from a shear stress vs. normal stress plot.. This angle of friction is measured in degrees.

4.4. DILATANCY ANGLE (ψ)

When granular materials are subjected to shear deformations, a volume change known as dilatancy is observed [43]. A compacted dense granular material, in contrast to most other solid materials, tends to dilate (expand in volume) upon shear deformations (Fig. 6). As a result of their interlocking patterns, grains in a compacted condition are unable to freely move through or in between one another. In this case, stress causes a lever motion between adjacent grains, leading to an expansion in bulk volume. However, when shear stress is applied to granular materials that are initially in a highly loose condition, the material may constantly shrink. A material sample's shear reaction may be characterized as dilative if its volume increases with shear, or contractive if its volume decreases with shear [44]. The dilatancy effect causes the angle of friction to grow with increasing confinement up to a maximum value (Fig. 6). The angle of friction drops precipitously after the soil's maximum strength has been mobilized. Consequently, geotechnical engineering in these types of soils has to take into account the possibility of a weakening after the soil strength hits this maximum value [45]. Dilatancy angles are given in degrees, denoted by the symbol "ψ". The following formulae may be used as a general guideline when estimating the dilatancy angle for quartz sands [46-48].

$$\psi = \emptyset' - 30^{o} \quad (6)$$

$$\psi = -2 + \frac{12.5D_r}{100} \quad (7)$$

Where, ψ is the dilatancy angle (°), $\emptyset'$ is the internal friction angle (°), and D_r is the relative density (%). Clayey soils typically have a minimal dilatancy for cohesive materials, with the exception of significantly over-consolidated layers. In this situation, the value of ψ= 0 would be reasonable to employ [10].

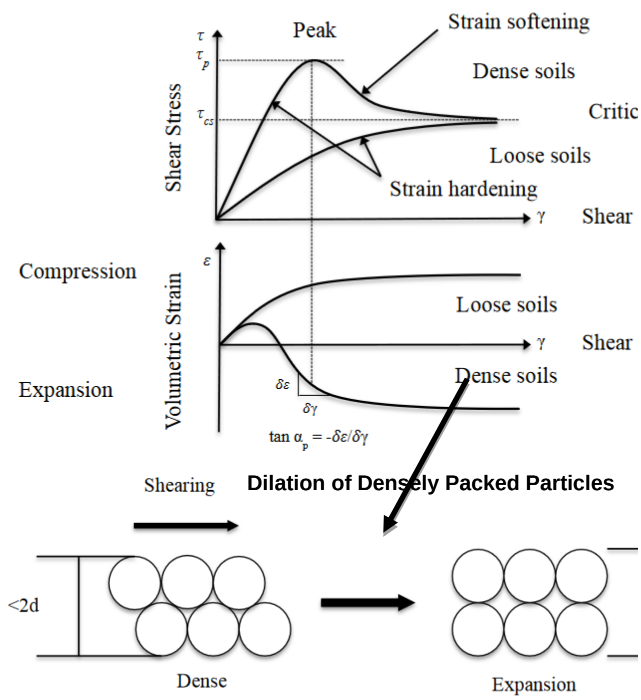


Fig. 6. Shearing Behavior of granular soil [47].

5. CONCLUSIONS

Taking into consideration the review that has been provided in this article, the following conclusions can be drawn:

1. Numerical modeling is a valuable tool for investigating the characteristics of soil structure in excavation support systems.
2. Two-dimensional (2-D) analysis is commonly used for evaluating excavation problems, but the limitations of 2-D analysis should be considered, and three-dimensional (3-D) investigations may be necessary in certain cases, such as deep excavations.
3. Soil is a multi-phase and anisotropic material, and its behavior is time-dependent, nonlinear, and influenced by stress history. Deflections, dilations, and irreversible plastic strains can occur.
4. Linear elastic models have simplicity but fail to capture significant aspects of real soil behavior. Nonlinear elastic models can capture the nonlinear relationship between shear stress and strain but have limitations in extrapolating beyond calibration curves.
5. Elastic-perfectly plastic models are widely used and suitable for many geotechnical issues but caution is needed in soft clay situations to avoid overestimating soil strength.
6. Elasto-plastic models, such as the Modified Cam-Clay (MCC) model and the Hardening Soil (HS) model, provide more accurate predictions of soil behavior, especially during

excavation and unloading.

7. Due to its simplicity and user-friendliness, the Mohr-Coulomb model continues to be extensively used in conventional excavation design and research, despite its limitations.
8. The ratio of axial strain to lateral strain is defined by Poisson's ratio. From 0.3 to 0.4 (drained) and 0.15 to 0.25 (unloading), their values are dependent on the soil's loaded and unloading circumstances.
9. Soil shear strength, measured in stress units, is known as cohesiveness. It is advised to choose a low cohesion value even for materials without cohesion in order to reduce computational mistakes.
10. Dilatancy is significant in compacted dense granular materials, where shear deformations cause an expansion in bulk volume. However, cohesive materials typically exhibit minimal dilatancy.

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