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The current state of soil mechanics models

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Abstract: When designing geotechnical structures, there is a need to adopt a particular model to represent the soil we observe under a particular load. The parameters we set for this soil are sometimes incomplete, which is caused by various problems such as incomplete data from laboratory or field research. The choice of the correct model depends on how accurate the calculation we want to get, or how much risk plays a role for us and to what extent we can accept some solutions that do not give a true picture of soil behavior under load. That is why the proper model selection is the first step in solving geotechnical problems, on which all further work will depend.

Key words: constitutive model, soil mechanics, stresses, elasticity, plasticity

Trenutno stanje modela mehanike tla

Sažetak: Prilikom projektiranja geotehničkih građevina, postoji potreba za usvajanjem određenog modela kojim ćemo predstaviti tlo koje promatramo pod određenim opterećenjem. Parametri koje zadajemo tome tlu ponekad su nepotpuni, čemu su uzrok razni problemi poput nepotpunih podataka iz laboratorija ili terenskog ("in situ") istraživanja. Odabir pravilnog modela ovisi o tome koliku točnost izračuna želimo dobiti, odnosno koliku nam ulogu igra rizik i do koje granice možemo prihvatiti možda neka rješenja koja ne daju pravu sliku o ponašanju tla pod opterećenjem. Zato je pravilan odabir modela prvi korak pri rješavanju geotehničkih zadaća, o čemu će nam ovisiti sav daljnji rad.

Ključne riječi: konstitutivni model, mehanika tla, naprezanja, elastičnost, plastičnost

1. INTRODUCTION

A model is an attempt to represent a natural phenomenon, physical process and other events in nature in such a way that their behavior can be established and a solution to a specific problem from the real world can be found. If the definition of the model is limited to the field of soil mechanics, or to the constitutive model, its definition, according to Liu, 2005, is that the constitutive model describes the change in the stress state of the material element resulting from the loads acting on the element. The constitutive model provides information on the strength and deformation of the material in the infinitesimal element on which the stress acts. According to Nordal, 2008, the soil model is a mathematical relationship between stresses and strains, or between changes in stresses and strains. This relationship is often called the constitutive equation.

The actual soil behavior is very complex. There are different soil models that describe the relationships between stresses and strains and their failure behavior. In general, the criterion for the application of a soil model should always be a balance between the requirements of continuum mechanics, the requirement of realistic representation of soil behavior from the laboratory testing aspect, practicality in relation to changes in parameter values and simplicity of application within software packages (Ti et al., 2009).

In the paper of Roje-Bonacci et al., 2006, soil mechanics models are divided into elastic, plastic and elasto-plastic ones. According to Nordal, 2008, a division is made into linear-elastic models, elasto-plastic models, simple total stress models and simple effective stress models.

Soil models differ from each other in the number and type of parameters, their purpose, complexity and applicability, or in their description and possibilities (Lade, 2005). The development of constitutive models for soft soils over the past 30 years is shown in Figure 1, while the development of constitutive models for tunnels over the past 30 years is shown in Figure 2.

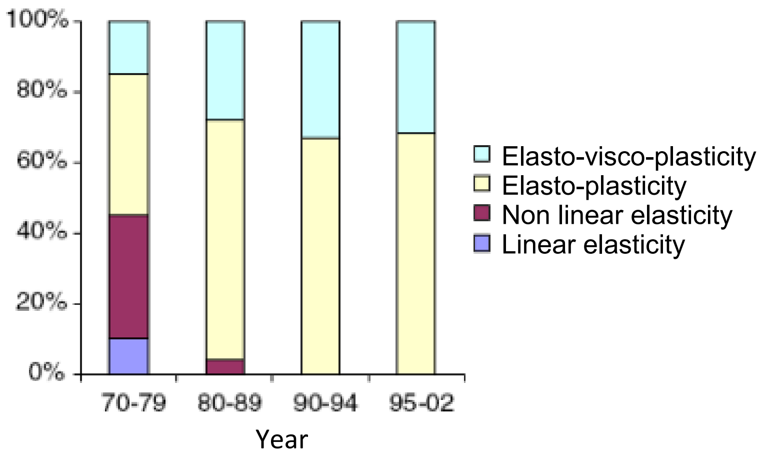

Figure 1. Development of constitutive soil models for soft soils over the past 30 years (Ti et al., 2009)

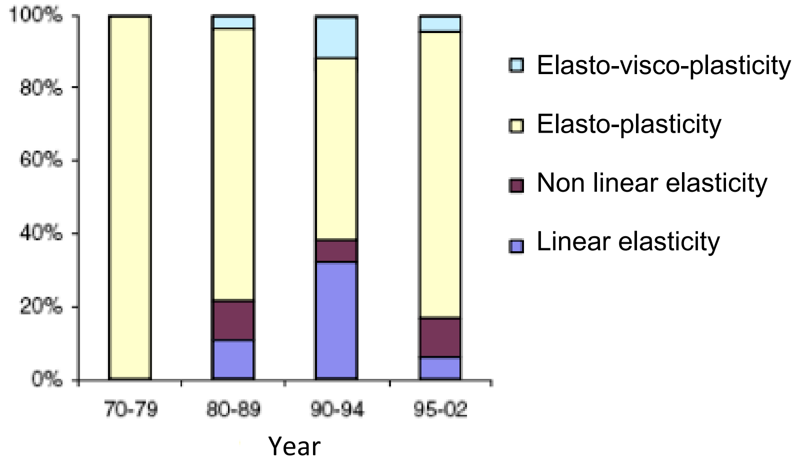

Figure 2. Development of constitutive soil models for tunnels over the past 30 years (Ti et al., 2009)

In the following, the paper presents models of soil mechanics according to the classification from the paper Roje-Bonacci et al., 2006. This classification includes models related to practical application.

2. ELASTIC SOIL MODELS

2.1 Linear-elastic models

In this model, the relationship between stress and strain is linear, or it is directly proportional to the strain (Roje-Bonacci et al., 2006; Brinkgreve, 2005). This relationship for the triaxial stress state and the triaxial strain state is expressed by equation (1), while the model for the uniaxial strain state and the plane stress state is shown in Figure 3.

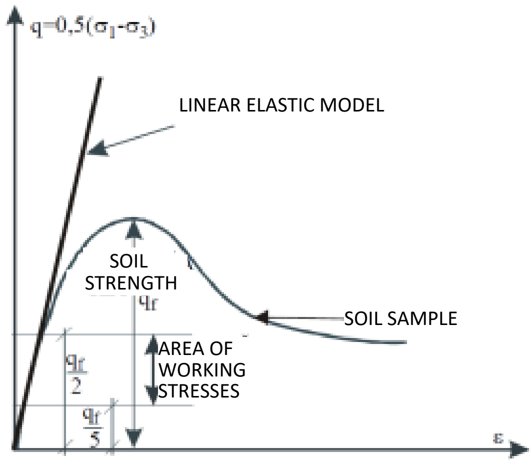

Figure 3. Linear-elastic soil model for the uniaxial strain state and plane stress state

$$\begin{Bmatrix} \sigma_x \\ \sigma_y \\ \sigma_z \\ \tau_{xy} \end{Bmatrix} = \frac{E}{(1+\nu)(1-2\nu)} \begin{bmatrix} 1-\nu & \nu & \nu & 0 \\ \nu & 1-\nu & \nu & 0 \\ \nu & \nu & 1-\nu & 0 \\ 0 & 0 & 0 & \frac{1-2\nu}{2} \end{bmatrix} \begin{Bmatrix} \varepsilon_x \\ \varepsilon_y \\ \varepsilon_z \\ \gamma_{xy} \end{Bmatrix} \tag{1}$$

The principal stresses are marked with σ_1 and σ_3 and are located on the principal stress directions of a body. The soil model for the uniaxial stress state from Figure 3 is defined by two parameters: Young's modulus or modulus of elasticity E and Poisson's ratio v. It should be noted that this model is also called Hooke's law, or linear isotropic elasticity (Brinkgreve, 2005). Using the known relationships from Šimić, 2002, this model can be further described using four parameters. So, in addition to the mentioned two parameters, there is also the shear modulus G and the volume deformation modulus K:

$$G = \frac{E}{2(1+\nu)}, K = \frac{E}{3(1-2\nu)} \tag{2}$$

Compared to other soil models, this model has the simplest relationship between stress and strain, but it is not very suitable for covering important features of soil behavior. Regardless of that, Hooke's law still plays an important role in more complex calculation (modeling) procedures, since it is often incorporated into the elastic part of more complex elastoplastic models. However, a modification of this model includes stiffness anisotropy, so that Young's modulus E and Poisson's ratio v are defined in two directions, and an additional shear modulus G, thus with five included parameters (Brinkgreve, 2005; Wood, 1994).

Through this modification with increase in parameters, this model can be used for stiff soils, thin concrete walls or slabs, or rocks, and for areas that do not have pronounced plasticity. This model is not generally suitable for soils (Brinkgreve, 2005). However, Lade, 2005, states that this model can be used for sand, clay and cemented soils. Wood, 1994, mentions the use of this model in the calculation of deformations of geotechnical structures loaded with working loads and in the calculation of stresses in corresponding laboratory tests.

2.2 Duncan-Chang model

This model is based on the stress-strain curve, first presented in 1970 by Duncan and Chang. It was obtained from a drained triaxial compression test, and can be described by a hyperbolic function:

$$\frac{\varepsilon}{\sigma_1 - \sigma_3} = a - b\varepsilon \tag{3}$$

a and b are determined by:

$$a = \frac{1}{E_i}, \qquad \frac{1}{b} = (\sigma_1 - \sigma_3)_f \tag{4}$$

In Figure 4, the above model is shown as a combination of presentations from Brinkgreve, 2005, Roje-Bonacci et al., 2006, and Ti et al., 2009.

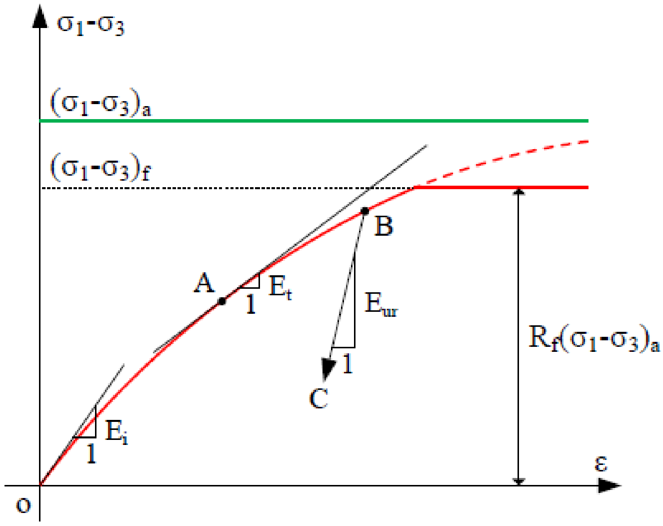

Figure 4. View of the Duncan-Chang model
(Brinkgreve, 2005; Roje-Bonacci et al., 2006; Ti et al., 2009)

This model consists of three moduli: initial modulus $\mathrm{E_i}$, tangent modulus $\mathrm{E_t}$ and unloading-reloading modulus $\mathrm{E_{ur}}$ (Mitchell and Gardner, 1971; Roje-Bonacci et al., 2006). $(\sigma_1 - \sigma_3)_f$ is the difference of principal stresses at failure, while $(\sigma_1 - \sigma_3)_a$ is the asymptotic difference of principal stresses with respect to the hyperbolic curve relating stress and strain. $\mathrm{R_f}$ is the failure parameter (ratio).

The advantage of the Duncan-Chang model is that it is widely used because its parameters (specified moduli) can be obtained from a standard triaxial test. The disadvantage of this model is that it is not suitable for computations where failure is calculated for soil that behaves fully plastically (Ti et al., 2009). According to Roje-Bonacci et al., 2006, this model poorly describes the decrease of shear modulus from the initial state to failure, depending on the shear strain.

2.3 Anisotropic-elastic model

Different properties of a material in different directions is called anisotropy. When incorporating anisotropy into a specific model, elastic anisotropy and plastic anisotropy are considered separately. Elastic anisotropy refers to the use of elastic stiffness parameters in different directions. Plastic anisotropy means the use of different stress/strain parameters in different directions, which is analyzed below in the Jointed Rock Model (Plaxis b.v., 2002) (Figure 5).

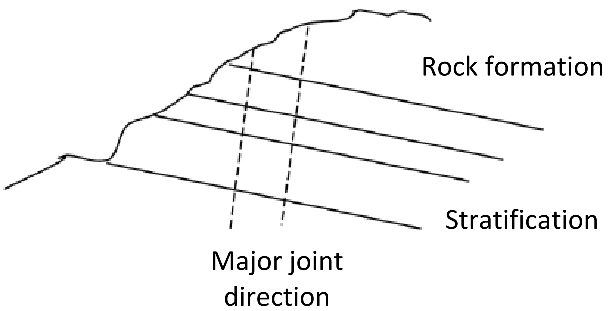

Figure 5. Concept of the Jointed Rock Model (Plaxis b.v., 2002)

The assumption is that this rock without discontinuities behaves as a transversely anisotropic elastic material, quantified by five parameters and with a direction of anisotropy. A maximum of three sliding directions/planes can be defined, of which the first plane is assumed to coincide with the direction of elastic anisotropy. Each plane may have different shear strengths.

The elastic material behavior in the Jointed Rock Model is described by an elastic material stiffness matrix, $\underline{\underline{D}}^*$. Matrix D* in the Jointed Rock Model is transversely anisotropic. Different stiffnesses can be used normal to and in a predefined direction ("plane 1"). This direction may correspond to the stratification direction or to any other direction with significantly different elastic stiffness properties. Consider a horizontal stratification, where "plane 1" is parallel to the x-z plane, and the following relations shown in equation (5) are set:

$$
\begin{aligned}
\dot{\varepsilon}_{xx} &= \frac{\dot{\sigma}_{xx}}{E_1} - \frac{\upsilon_2 \dot{\sigma}_{yy}}{E_2} - \frac{\upsilon_1 \dot{\sigma}_{zz}}{E_1} \\
\dot{\varepsilon}_{yy} &= -\frac{\upsilon_2 \dot{\sigma}_{xx}}{E_2} + \frac{\dot{\sigma}_{yy}}{E_2} - \frac{\upsilon_2 \dot{\sigma}_{zz}}{E_2} \\
\dot{\varepsilon}_{zz} &= -\frac{\upsilon_1 \dot{\sigma}_{xx}}{E_1} - \frac{\upsilon_1 \dot{\sigma}_{yy}}{E_2} + \frac{\dot{\sigma}_{zz}}{E_1} \\
\dot{\gamma}_{xy} &= \frac{\dot{\sigma}_{xy}}{G_2} \\
\dot{\gamma}_{yz} &= \frac{\dot{\sigma}_{yz}}{G_2} \\
\dot{\gamma}_{zx} &= \frac{2(1+\upsilon_1)\dot{\sigma}_{zx}}{E_1}
\end{aligned} \tag{5}
$$

A dot above the mark for a particular quantity indicates an infinitesimal value. The inverse of the anisotropic elastic material stiffness matrix, $\left(\underline{\underline{D}}^*\right)^{-1}$, is obtained from the relations determined in equation (5). The matrix D* can only be obtained by numerical inversion.

In general, the stratification plane will not be parallel to the global x-z plane. However, the relations from equation (5) will hold for a local n-s-t coordinate system, where the stratification plane is parallel to the s-t-plane. The orientation of this plane is defined by the dip angle and dip direction. As a consequence, the local material stiffness matrix has to be transformed from the local to the global coordinate system. A transformation of stresses and strains is considered first:

$$\underline{\sigma}_{nst} = \underline{\underline{R}}_{\sigma}\underline{\sigma}_{xyz}, \qquad \underline{\sigma}_{xyz} = \underline{\underline{R}}_{\sigma}^{-1}\underline{\sigma}_{nst} \tag{6}$$

$$\underline{\varepsilon}_{nst} = \underline{\underline{R}}_{\varepsilon}\underline{\varepsilon}_{xyz}, \qquad \underline{\varepsilon}_{xyz} = \underline{\underline{R}}_{\varepsilon}^{-1}\underline{\varepsilon}_{nst} \tag{7}$$

It holds that:

$$\underline{\underline{R}}^{T}_{\varepsilon} = \underline{\underline{R}}^{-1}_{\sigma}, \ \underline{\underline{R}}^{-T}_{\sigma} = \underline{\underline{R}}^{-1}_{\varepsilon} \tag{8}$$

Relationships can be transformed from the local n-s-t coordinate system to the global x-y-z coordinate system:

$$\left.\begin{array}{l} \underline{\sigma}_{nst} = \underline{\underline{D}}^{*}_{nst}\,\underline{\varepsilon}_{nst} \\ \underline{\sigma}_{nst} = \underline{\underline{R}}_{\sigma}\,\underline{\sigma}_{xyz} \\ \underline{\varepsilon}_{nst} = \underline{\underline{R}}_{\varepsilon}\,\underline{\varepsilon}_{xyz} \end{array}\right\} \Longrightarrow \underline{\underline{R}}_{\varepsilon}\,\underline{\sigma}_{xyz} = \underline{\underline{D}}^{*}_{nst}\,\underline{\underline{R}}_{\varepsilon}\,\underline{\varepsilon}_{xyz} \tag{9}$$

This model makes it possible to model the anisotropy of the rock mass. The big advantage of this model is that it uses the usual parameters for the rock mass (Equation (5)). The values of these parameters can also be determined for the direction that is intended to be analyzed (Hack et al., 2010). The same authors recommend the use of this model when modeling stresses and strains in tunnels.

3. PLASTIC SOIL MODELS

The concept of plasticity theory consists of three basic relationships: yield conditions, the law of yielding and hardening, and failure conditions. Plastic constitutive models differ in the assumed yield function.

According to Thakur and Nordal, 2005, the total soil deformation can generally be expressed as:

$$\varepsilon = \varepsilon^{e} + \varepsilon^{p} \tag{10}$$

The following relation (Roje-Bonacci et al., 2006) also applies:

$$d\varepsilon = d\varepsilon^{e} + d\varepsilon^{p} \tag{11}$$

where ε is the total relative deformation, ε^{e} is the elastic relative deformation, while ε^{p} is the plastic deformation.

The Mohr-Coulomb model, the Drucker-Prager model, the von-Mises model and the Tresca model will be described below.

3.1 Mohr-Coulomb soil model

The model is simple and applicable to the 3D stress state, where only two strength parameters (cohesion c and internal friction angle φ) are needed to describe the plastic behavior (Ti et al., 2009).

The theory is based on the fact that failure is controlled by maximum shear stresses that depend on normal stresses, which is shown by Mohr's circle (Figure 6) for the stress state at failure for the maximum and minimum principal stress.

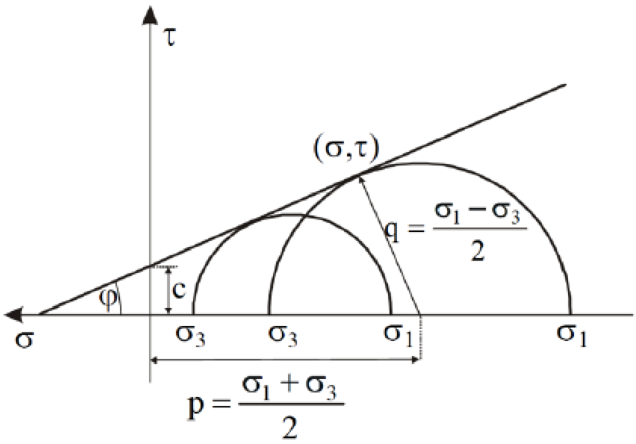

Figure 6. Mohr-Coulomb failure criterion for plane stress

According to Figure 6, Mohr-Coulomb's law is represented by:

$$\tau = c + \sigma \cdot \mathrm{tg}\varphi \tag{12}$$

From the same figure, it is evident that:

$$\tau = q \cdot \cos\varphi \text{ and } \sigma = p - q \cdot \sin\varphi \tag{13}$$

If the expressions for τ and σ from equation (13) are substituted in equation (12), the Mohr-Coulomb criterion can be written in the form:

$$q - p \cdot \sin\varphi - c \cdot \cos\varphi = 0 \tag{14}$$

where:

$$q = (\sigma_1 - \sigma_3)/2 \text{ and } p = (\sigma_1 + \sigma_3)/2 \tag{15}$$

The soil behavior at failure is well covered by this model (Brinkgreve, 2005). It is established that the combination of stresses that causes failure in soil samples fits the shape of the failure surface, which is in the form of a regular hexagonal prism (Roje-Bonacci et al., 2006; Ti et al., 2009). Figure 7 shows the yield surface for the Mohr-Coulomb model.

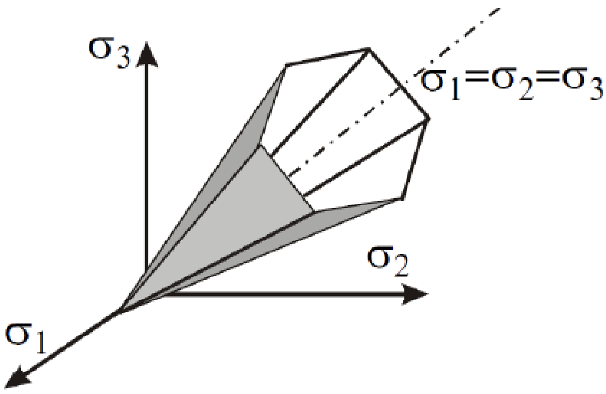

Figure 7. Yield surface for the Mohr-Coulomb model for the case when cohesion is c=0 kPa

In Ti et al., 2009, the authors state that the soil behavior is well described by this model for the drained condition, but that it is preferable to use undrained parameters (c and φ) for undrained analysis. Also, for perfectly plastic behavior, the model does not include the effect of hardening or softening of the soil.

3.2 Drucker-Prager soil model

This model represents a simplified Mohr-Coulomb model, in the way that the yield surface is replaced by a cone shape (Brinkgreve, 2005) (Figure 8).

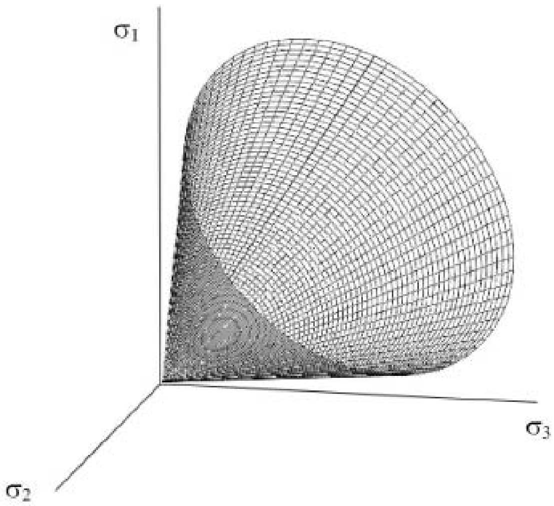

Figure 8. Yield surface for the Drucker-Prager model (Brinkgreve, 2005)

The model is defined through the expression for the yield function f:

$$f = \sqrt{J_2} - \alpha I_1 - k = 0 \tag{16}$$

where f is the yield function, J_2 and I_1 are the corresponding stress invariants, while α and k are the material parameters (Table 1). These parameters are shown for three possible cases from Figure 9. (Nordal, 2008).

Table 1. Value of parameters α and k for three possible cases (Nordal, 2008)

Possible cases	α	k
Triaxial pressure ($\sigma_2 = \sigma_3$)	$\frac{2\sin\varphi}{\sqrt{3}(3-\sin\varphi)}$	$\frac{6c\cos\varphi}{\sqrt{3}(3-\sin\varphi)}$
Triaxial tension ($\sigma_2 = \sigma_1$)	$\frac{2\sin\varphi}{\sqrt{3}(3+\sin\varphi)}$	$\frac{6c\cos\varphi}{\sqrt{3}(3+\sin\varphi)}$
Internal tangent circle	$\frac{\sin\varphi}{\sqrt{3}(3+\sin^2\varphi)}$	$\frac{6c\cos\varphi}{\sqrt{3}(3+\sin^2\varphi)}$

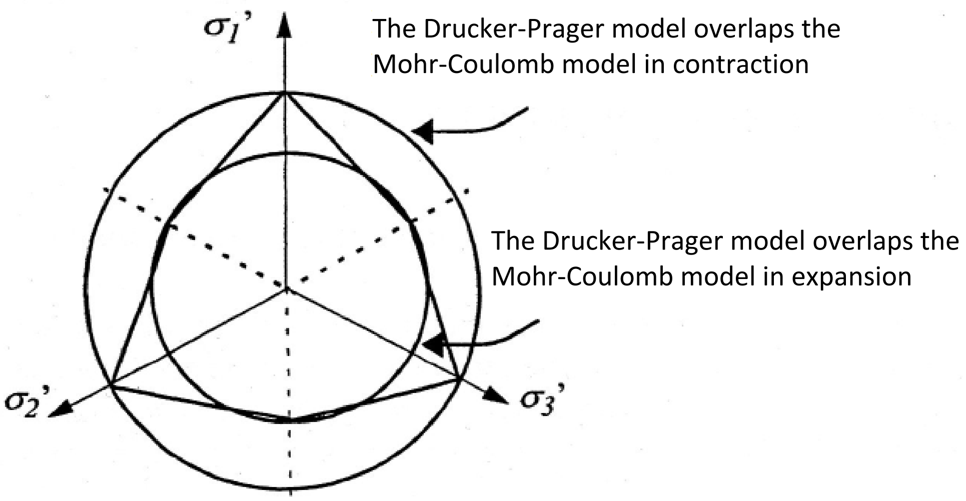

Figure 9. Comparison of Drucker-Prager yield surface with Mohr-Coulomb yield surface in plane view (Nordal, 2008)

Because of its simplicity, the Drucker-Prager model is often used in geotechnical engineering. However, it was found that the circular shape of the Drucker-Prager yield surface in the deviatoric stress plane does not match well with experimental data. For the above reason, caution is needed when the Drucker-Prager model is used in geotechnical engineering (Yu, 2006).

3.3 Tresca model

This model is considered a particular case of the Mohr-Coulomb failure criterion. The yield surface is a regular hexagonal prism (Figure 10) (Taiebat and Carter, 2008).

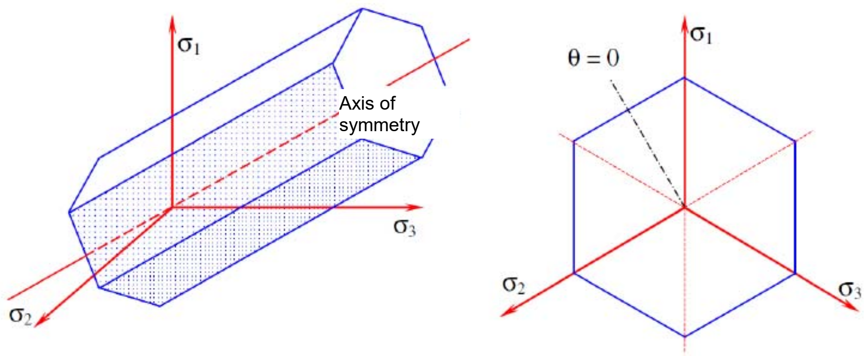

Figure 10. Yield surface for the Tresca model (Taiebat and Carter, 2008)

The model is defined through the expression for the yield function f:

$$f = \sqrt{J_2} \cdot \cos\theta - s_u = 0 \tag{17}$$

where J_2 is the second stress invariant, θ is the Lode angle, which determines the orientation of the stress plane with respect to the direction of principal stresses, s_u is the undrained shear strength of the soil (Taiebat and Carter, 2008, Yu, 2006).

3.4 Von-Mises model

The von-Mises model is primarily conceived as an approximation and mathematically more suitable simplification of the Tresca model (Nordal 2008). Furthermore, it holds that yielding occurs when the second stress invariant reaches a critical value (Yu, 2006), i.e. when the yield function f is equal to 0:

$$f = \sqrt{J_2} \cdot \cos\theta - k = 0 \tag{18}$$

where J_2 is the second stress invariant, and k is the undrained shear strength of the soil in pure shear. The yield surface for the von-Mises model is a cylinder (Figure 11)

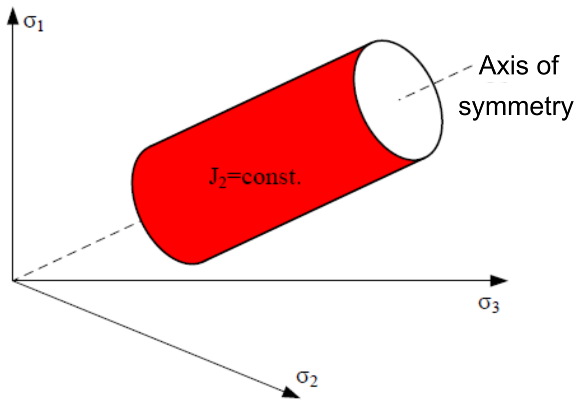

Figure 11. Yield surface for the von-Mises model (Nordal, 2008)

Figure 12 shows the yield surface for the Tresca model and the yield surface for the von-Mises model in a plane view.

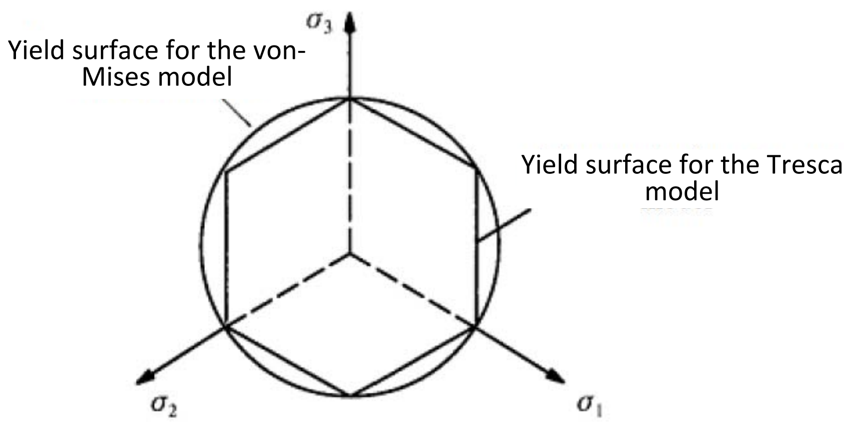

Figure 12. Yield surface for Tresca model and yield surface for von-Mises model in plane view (Yu, 2006)

If the undrained shear strength of the soil in pure shear, k, is suitably chosen so that the circle representing the failure surface of the von-Mises model passes through the corners of the hexagons representing the failure surface of the Tresca model (Figure 12), the following relation holds:

$$k = \frac{s_u}{\cos\theta} \tag{19}$$

According to equation (19) and Figure 12, i.e. by comparing the functions of the von-Mises model and the Tresca model, it is evident that the von-Mises criterion generally implies a slightly higher undrained shear strength. The difference depends on the Lode angle θ (Yu, 2006). Yu, 2006, also recommends using this model in cohesive soils.

4. ELASTO-PLASTIC SOIL MODELS

4.1 Linear elastic-perfectly plastic model

Figure 13 shows the linear elastic-perfectly plastic behavior of the soil.

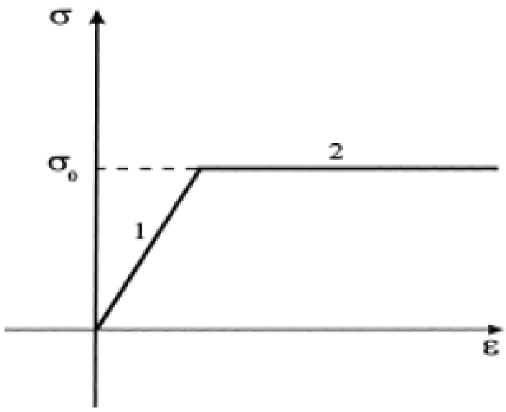

Figure 13. Linear elastic-perfectly plastic soil behavior (Mouazen and Neményi, 1998)

The model that describes this soil behavior consists of two parts. In the first part, marked with 1 in Figure 13, the behavior of the soil is linear elastic, while in part 2 the behavior is perfectly plastic (Mouazen and Neményi, 1998).

According to the equations, the relative deformation can be divided into elastic and perfectly plastic components. The elastic component has its constitutive equation where:

$$\sigma = C_E \cdot \varepsilon \tag{20}$$

where C_E is the elastic constant of the system (Roje-Bonacci, 2003).

The yield function, f, is determined by:

$$f = f\,(\sigma_x, \sigma_y, \sigma_z, \tau_{xy}) \tag{21}$$

Thus, for $f < 0$, the stress is in the area of elastic deformations, and for $f = 0$ the yield function describes the strength law. This model is used when considering shear failure in soil due to exceeding the shear strength (Roje-Bonacci, 2003).

4.2 Cam-Clay (original) model and modified Cam-Clay model of soil

Cam-Clay is a model integrated into the Plaxis 8.0 software package, an illustration of the Cam-Clay model is given in Figure 14 in the $p' - q$ coordinate system. p' is the mean effective stress, q is the stress deviator, p_c is the overconsolidation stress, M is the critical state parameter.

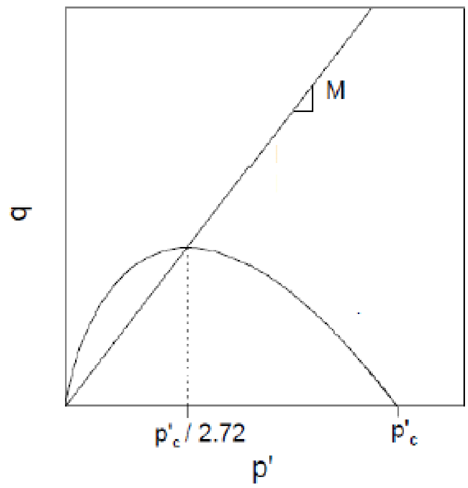

Figure 14. Cam-Clay (original) model in the $p' - q$ coordinate system (Baxter, 2000)

In order to describe this model, the presented Figure 15 in the ln $p' - v$ coordinate system describes the laboratory results of oedometer tests and isotropic compression tests, performed in order to obtain some parameters (data) to be used for modeling the Cam-Clay model. v is the specific volume, determined using the pore coefficients e, where $v = 1 + e$ (Baxter, 2000).

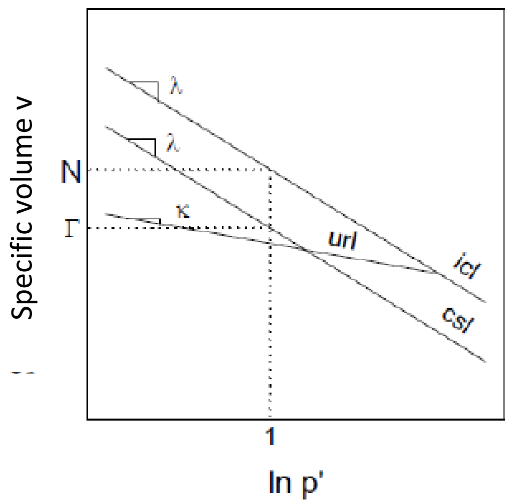

Figure 15. Representation of the results of the oedometer test and isotropic compression test (Baxter, 2000)

The "url" line (unloading-reloading line) and the "icl" line (isotropic compression line) describe the compressibility of the soil and are assumed to be linear. The parameter κ is the gradient of the "url" line, while the parameter λ is the gradient of the "icl" line. The "csl" line represents the critical state line and is parallel to the "icl" line. With this curve, plastic shear deformation occurs without the formation of plastic volumetric stresses. The parameter N indicates the position of the "icl" line in the ln $p' - \nu$ coordinate system. The parameter N is also the value of the specific volume ν on the "icl" line with the value $p' = 1$ (Baxter, 2000).

For the Cam-Clay (original) model, the changes (increments) of relative elastic volumetric strain $\delta\varepsilon_p{}^e$ and relative elastic shear strain $\delta\varepsilon_q{}^e$ can be expressed as:

$$\delta\varepsilon_p{}^e = \frac{\kappa}{\nu p'}\delta p' \text{ and } \delta\varepsilon_q{}^e = 0 \tag{22}$$

The modified Cam-Clay model is shown in Figure 16.

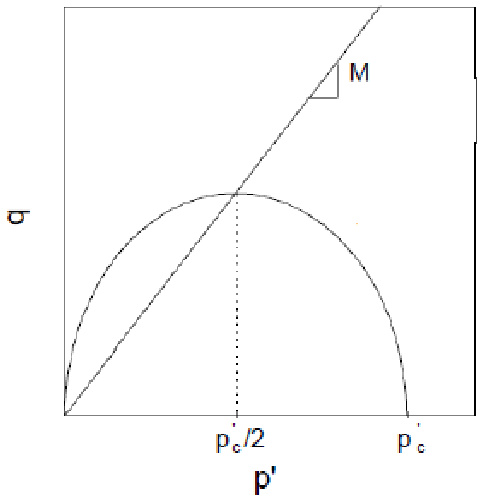

Figure 16. Modified Cam-Clay model (Baxter, 2000)

For the modified Cam-Clay model, the changes (increments) of the relative elastic volumetric strain $\delta\varepsilon_p{}^e$ and the relative elastic shear strain $\delta\varepsilon_q{}^e$ can be expressed as:

$$\delta\varepsilon_p{}^e = \frac{\kappa}{\nu p'} \quad \delta p' \text{ and } \delta\varepsilon_q{}^e = \frac{1}{3G}\delta q \tag{23}$$

Relative volumetric strains can also be expressed through the volumetric strain modulus, K:

$$K = \frac{\delta p'}{\delta\varepsilon_p{}^e} \tag{24}$$

For the Cam-Clay (original) model and for the modified Cam-Clay model, the modulus K is equal to:

$$K = \frac{\nu p'}{\kappa} \tag{25}$$

The yield function f defines the boundary between elastic and elasto-plastic deformation. At stresses below the yield surface, defined by the yield function f, only elastic deformations occur. Above the yield surface, elastic and plastic deformations occur (Baxter, 2000).

The yield function for the Cam-Clay (original) model is given by the expression:

$$f = \frac{q}{Mp'} + \ln\frac{p'}{p'_x} - 1 \tag{26}$$

The parameter p' defines the size of the yield surface.

The shape of the yield surface for the modified Cam-Clay model is an ellipse. For the Cam-Clay model, the yield surface intersects the M line at $p'_c/2{,}72$, and for the modified Cam-Clay model at $p'_c/2$.

In Baxter, 2000, the Cam-Clay (original) model and the modified Cam-Clay model were used to model the behavior of bentonite clays used for seepage control diaphragms and it was concluded that their application was justified. According to Brinkgreve, 2005, the modified Cam-Clay model is most suitable for soft soils such as normally consolidated clays.

4.3 Deformation-softening model

In this model, the concept of deformation softening means the formation of plastic deformations with reduction of stresses that cause the material to yield. This model is shown in Figure 17.

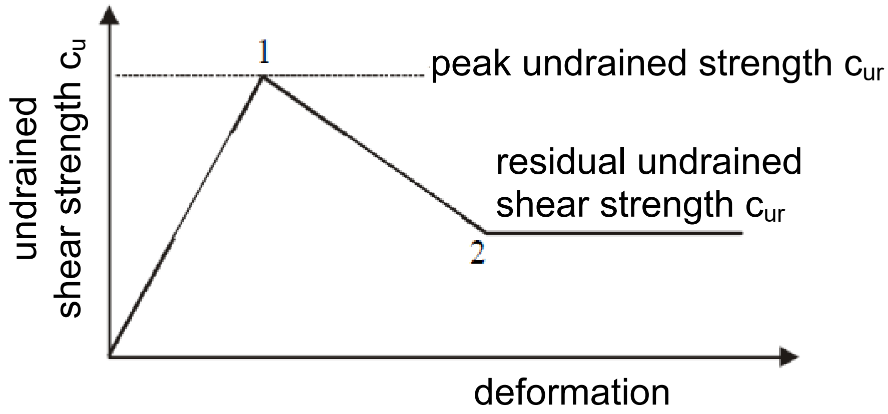

Figure 17. Deformation-softening soil model

This model consists of three linear parts. The first is the linear part that increases to the highest shear strength (point 1), the second part is the softening part in which the shear strength decreases to the highest residual strength (point 2), while the third part is from point 2 onwards, where the shear strength c_{ur} does not change. Consequently, this model is elastic-softening-plastic (Roje-Bonacci et al., 2006).

The yield function f for this model is given by shear stresses q and undrained shear strength c_u:

$$f = q - \sqrt{3c_u} \tag{31}$$

Failure at shear strength, c_u, is equal to:

$$C_u = \frac{\sigma_1 - \sigma_3}{2} \quad (32)$$

Figure 18 also shows an example of an extreme softening situation for sensitive clays, which can occur, shown in the $\sigma - \varepsilon$ coordinate system.

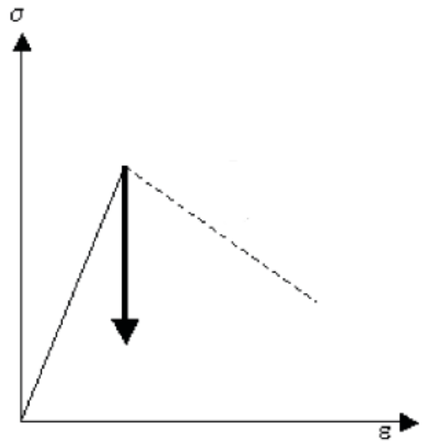

Figure 18. Extreme situation of softening of sensitive clay in the deformation-softening soil model (Thakur and Nordal, 2005)

5. CONCLUSION

Soil mechanics models have the role of representing the stress-strain relationship of different soil types for different load cases. The purpose of these models is to present the actual soil behavior as faithfully and realistically as possible, based on data obtained from field and laboratory tests. A model of soil mechanics should simulate the real behavior of the soil and it should have such properties that the parameters needed for its description and definition can be obtained from the simplest possible tests, be it laboratory tests or field tests (Lade, 2005).

It is appropriate to choose a soil model that allows fitting (adjustment) to the data obtained in laboratory tests. It is very important to include calculations from as many experimental measurements as possible in order to obtain the highest degree of reliability of the model, which ultimately allows a greater applicability of that model. Furthermore, such a model provides the "right" answer to the problem being solved, despite the fact that such an answer involves some degree of assumptions with a certain level of accuracy (Ti et al., 2009).

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