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POUZDANOST REZULTATA PRORAČUNA U FUNKCIJI ODABRANOG MODELA TLA

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Rezime:

Pouzdanost geotehničkih proračuna, opšte je poznato, u funkciji je kvaliteta ulaznih parametara, kako po pitanju opterećenja, tako i fizičko-mehaničkih svojstava tla. Pored navedenog na pouzdanost rezultata geotehničke analize utiče i odabir konstitutivnog modela tla. U radu se opisuju modeli tla i analizira pogodnost modela na realnom (izvedenom) geotehničkom projektu.

Ključne riječi:Model tla, geotehnički proračun, pouzdanost rezultata.

RELIABILITY OF THE RESULTS OF THE CALCULATIONS IN THE FUNCTION OF THE SELECTED SOIL MODEL

Abstract:

The reliability of geotechnical calculations is, as it is generally known, in function of the input parameters quality, both in terms of loads, as well as the physical and mechanical soil properties. In addition to above mentioned, reliability of the results of geotechnical analysis is also influenced by selection of constitutive soil model. This paper describes soil models and analyzes the suitability of the model on a real (implemented) geotechnical project.

Keywords:soil model, geotechnical calculation, reliability of the results

1. INTRODUCTION

The model is an attempt to show the natural phenomenon, physical process and other occurrences in nature in a manner that enables their behavior to be predicted. In soil mechanics, there is a great need for the development of a model that will describe soil behavior when changing the stress state. The calculation of deformations in the soil, which are caused by the external load or by internal forces, is an important task that needs to be solved because the safety of the building depends on the deformations that occur during its construction and duration. Classical soil mechanics distinguish two separate states of soil behavior under load:

- the state of minor deformations, which do not cause the fracture of the soil, is studied by using the theory of elasticity;
- the state of major deformations, which cause the fracture of the soil, where the stresses in the soil are such that their small increase causes major deformations at constant velocity, is studied by the method of boundary state of the plastic balance.

The elasticity theory is used in the study of stresses and deformations of soil at work level load, where the value of ground fracture has not been reached. Solutions are obtained by linear elasticity theories. The states of the progressive collapse is the middle between the elastic behavior and the boundary state. The progressive fracture theory studies the elastic-plastic transition from the initial, linear-elastic state to the boundary state of the collapse with plastic deformations. The basis for obtaining a solution in a progressive fracture is the stress - deformation relation, i.e. constitutive term for the soil. For practical applications, within the load sizes, the soil is neither linearly elastic nor completely plastic. The real behavior of the soil is non-linear, very complex and variable depending on the conditions to which it is exposed, and this has a great influence on the selection of soil parameters for geotechnical calculations (Atkinson, 2000). In the last thirty years, a scientific approach to the constitutive soil modeling has been developed. The concept of critical soil condition was developed at Cambridge University in the 1950s by the ideas of Roscoe and asc. (1958, 1968). Further developments were contributed by Schofield, Wroth and Palmer (according to Chen 1975). Although the behavior model is originally developed for normally consolidated clays and slightly over consolidated clay, it is believed that with some adjustments, it can be used to describe the mechanical behavior of all types of soil. Idealization is needed to obtain mathematically simple constitutive models for practical application. A time factor is excluded in order to apply the theory of elasticity and plasticity. For the fracture, a plastic model is applied, and for conditions well below the breaking level, the elastic model (Maksimović, 2014). The criterion for evaluating of the model should consider the balance between the demands from the continuum mechanics point of view (theoretically), demands from real presentation of soil behavior (based on field and laboratory testing - experimentally) and demands for the simplicity of applying the model (numerically). These are the three basic criteria for evaluating the model in soil mechanics. Constituent equations are necessary in all soil mechanics methods: planning and evaluating laboratory and field testing, analytical and numerical prediction, or reverse strain and deformation analysis within the soil itself. All materials, including soil, have limited strength that limits the range of possible stress states. The stress - deformation relation for soil behavior is extremely nonlinear, inelastic, depends on the previous stress and deformation history, and depends on the rate of deformation, boundary conditions, and other factors (Ishihara et al., 1975).

2. CONECTION BETWEEN TENSION AND DEFORMATION

The connection between tension and deformation represents the Model material or its constituent equation. Models of soil and rock materials are generally presented as a relation between infinitesimal changes in the value of the effective stress and the infinitesimal change of deformation values. This relation can be presented in the form (Tymoshenko, Gudier, 1962):

$$\dot{\vec{\sigma}}' = M \cdot \dot{\vec{\varepsilon}} \tag{1}$$

where M is the matrix of material stiffness.

3. SOIL MODELS AT STATIC TEST CONDITIONS

The elastoplastic model gives a more realistic picture of the deformations formed before the final plastic fracture (Figure 1.) The real ground approximately corresponds to the model of ideally elastic materials, only for the limited range of application of the main stresses. For the problems discussed so far, satisfactory solutions were provided by the theory of elasticity and solutions using the oedometric soil model.

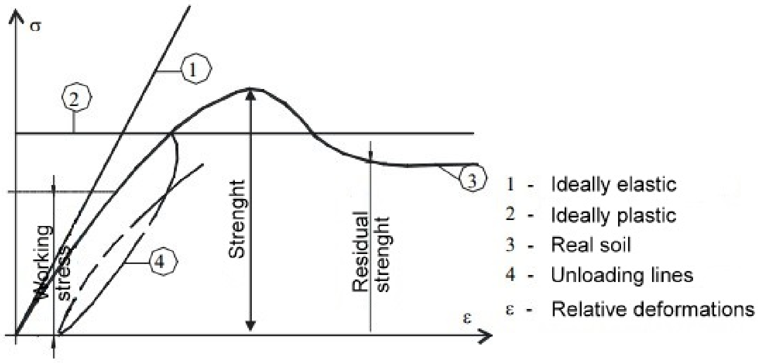


Figure 1. Stress - strain curve

When the ratio of the main stresses exceeds a certain range, deformations begin to increase much faster than the strain and eventually become very large. This is the boundary state of the plastic balance, in which a plastic flow with significant deformations begins. Soil behavior depends on the stress and deformation relation in the given conditions. The most commonly used, oedometric model, is a curing model as well as three-side models with lateral pressure. In shear tests with major deformations, models with softening to fracture occur. The most commonly used laboratory test to obtain the relation between stress and deformation is an oedometric test.

$$M_{k(\tan gent)} = \frac{d\sigma'_z}{d\varepsilon_z} \tag{2}$$

The strain of the deformation Δε_z, due to the increase in stress Δσ'_z, for the initial stress, p_o, is:

$$\Delta\varepsilon_z = \int_{p_0}^{p_0+\Delta\sigma'_z} \frac{d\sigma'_z}{M_k(\sigma'_z)} \tag{3}$$

The solution of integral depends on the shape of the functional link between the compression modules M_k and strain σ'_z. There is a great deal of attempts to improve the concept of the model, which increases the number of parameters, some of which can be determined, and some are assumed to get good agreement between a mathematically determined model and data obtained by tests. And when a model simulates the test well, which is a prerequisite for its acceptability; there are number of other tests that need to show that it will behave satisfactorily by arbitrary stress strains in various practical problems. The constituent equations in the continuum mechanics represent an analytical expression of the relation between the instantaneous stress state at a material point of the continuum and the history of deformation states through which the close environment of that point has passed. Through these equations, the mechanical properties of the material are included. For now, there is no constituent equation that describes the complexity of the mechanical behavior of the soil for the various conditions in which the soil can be found.

4. ELASTIC MODEL

These models include: Linear-elastic model, Duncan-Chang model (nonlinear hyperbolic elastic models) and anisotropic elastic model (cracked rock model).

4.1. Linear-elastic model

One of the simplest soil models is a linear elastic model in which stress is directly proportional to deformations. This model is most commonly used in the calculation of the settlement in soil mechanics because it corresponds to the assumption that the ground behaves in a linear resilience in minor deformations. The interpretation is shown in Figure 2 a).

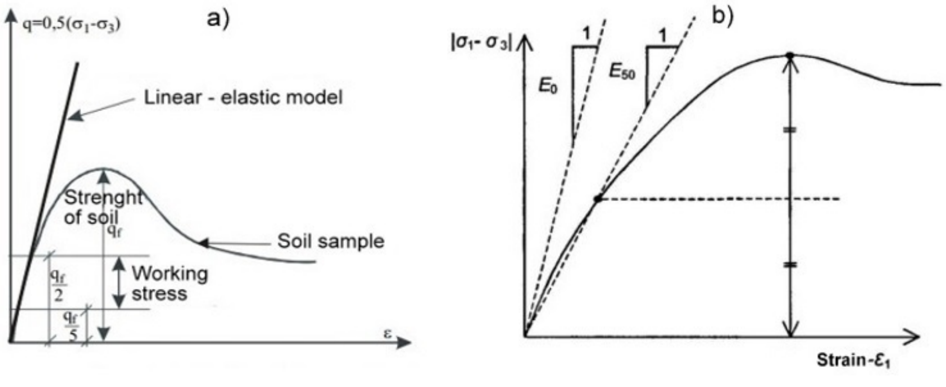


Figure 2. a) Explanation of the assumption of linear soil behavior; b) Definition of modules E0 and E50 for standard drained three-axis test (Yong, Townsend, 1980)

The linear elastic model is based on Hooke's law. There are four parameters of material for an elastic model: Young's modulus of elasticity E, Poisson's coefficient v, coefficient of volume deformation K and shear modulus G, and only two are required for the full description of the material. The proportionality constants are Young's modulus of elasticity E and the effective Poisson coefficient v. Young's module is used as the basic stiffness module in the elastic soil model. Special attention is needed for the adoption of stiffness parameters in the calculation, because it is shown that the assumption of linear soil behavior in minor deformations is often incorrect. Namely, the materials demonstrate nonlinear behavior at the very beginning of the load. Typically, the initial inclination of the deformation curve is marked as E_0, and the value of the secant module at 50% strength is marked as E_50 (Figure 2 b)). For materials with a larger scope of linear elasticity, it is more realistic to use E0, but the E_50 is generally used for soil loading. Considering the problems of unloading, such as tunnel excavations, etc., it is necessary to use a parameter that can be determined at the return deformations, i.e. the release (Figure 1, line 4), E_ur instead of E_50. For the soil, both the E_ur release module and the load modulus E50 tend to grow with increasing pressure. Therefore, higher stiffness can be expected in deep layers of soil compared to shallow layers. The rigidity is much higher for relieving and reloading than for the primary load. When a model with a constant modulus of elasticity (compressibility) is used to represent the soil behavior, a value corresponding to the level of stress and the corresponding strain of stress must be selected. Poisson's coefficient is by definition the ratio of longitudinal and transverse deformation:

$$\nu = \frac{\varepsilon_{\text{transversal}}}{\varepsilon_{\text{longitudinally}}} \tag{4}$$

In the soil, this ratio is not nearly as simple as code, for example steel rod or concrete cube. When considering Poisson's soil coefficient, one should always keep in mind that only the pores are deformed in the soil, while the solid particles, according to the basic assumption, do not change their shape for the level of work stresses. Deformation occurs by mutual sliding and rolling of particles at the expense of pore loss. Since the lateral expansion is prevented in the oedometric test, this is the lateral deformation ε_b = 0, which leaves only an upright deformation ε_z, and from the equation 4 it follows that for such a Poisson model the coefficient is ν = 0. In the drained tri-axial test, the Poisson coefficient ν can be determined for each of the specially selected stress levels, i.e., the incremental stress Δσ_z as well as the secant and tangent module, according to the:

$$\nu = \frac{\Delta\varepsilon_z - \Delta\varepsilon_v}{2\Delta\varepsilon_z} \tag{5}$$

ε_z - upright, axial deformation;

ε_v - volume deformation.

The choice of the Poisson coefficient value is simple when elastic model or a Mohr-Coulomb model of soil is used, in contrast to other cases when it is much more complex. The relation between Young's module E and other stiffness modules, such as shear modules G, the compression module K, and the edo-modulus module E_oed, is given by the following equations (Hill, 1950):

$$G = \frac{E}{2(1+\nu)} \tag{6}$$

$$K = \frac{E}{3(1-2\nu)} \tag{7}$$

$$E_{oed} = \frac{(1-\nu)E}{(1-2\nu)(1+\nu)} \tag{8}$$

4.2. Nonlinear elastic soil model

A nonlinear elastic soil model was suggested by Duncan and Chang 1970, analyzing the deformations of embankments and dams. The model parameters can be obtained from the results of a three-axis test. The stress-deformation curve is hyperbolic, which connects the deviator stress (σ_1-σ_3) and the axial deformation ε according to the expression (Konder, 1963, Konder et al., 1963, 1965):

$$\frac{\varepsilon}{(\sigma_1-\sigma_3)} = a - b\varepsilon \tag{9}$$

Depending on the stress state and the deformation trace, the model contains three soil modules: the E_i starting module, the E_t tangent modulus and the load modulus - Eur listed in Table 1.

Table 1. Modulus of elasticity in non-linear elastic soil model

$\mathrm{E}_i = K_L p_a \left(\frac{\sigma_3}{p_a}\right)^n$	E_i - initial tangent module; K_L - modular load number; P_a - atmospheric pressure (p_a = 100 kPa, used as a reference parameter); σ_3 - minor main stress n, the exponent determining the impact of lateral pressure on the initial module. Where n and K are numbers (constants) derived from the results of the drained tri-axial laboratory tests.
$\mathrm{E}_t = \left[1 - \frac{R_f(\sigma_1-\sigma_3)(1-\sin\varphi)}{2c\cos\varphi + 2\sigma_3\sin\varphi}\right]^2 \mathrm{E}_i$	φ - angle of internal friction; c - cohesion; R_f - ratio of asymptote of hyperbole and shear strength (from 0.75 to 1.0), but can also be determined from the results of the drained three-axis test; σ_1 - maximum main stress; σ_3 - minimum main stress.
$\mathrm{E}_{ur} = K_{ur} . p_a \left(\frac{\sigma_3}{p_a}\right)^n$	Unlike the tangent module, the load-relief module is not dependent on the state of shifting stresses. This module can be calculated directly from the curve of the results of the drained three-axis test, based on unloading - reloading (line 4 in Figure 3).

Poisson's coefficient of nonlinear elastic soil model can be considered as a constant, not depending on the state of stress; from the equation (5) or it can be calculated from the volume change module, which depends on lateral stress. The volume change module is given by the following expression:

$$B_m = K_m p_a \left(\frac{\sigma_3}{p_a} \right)^m \tag{10}$$

B_m - modulus of volume change;
K_m - Module number;
m - the exponent of the volume module.
The relation between the volume module and the Poisson coefficient can be determined by the theory of elasticity, so hence:

$$\nu = \frac{1}{2}\left(1 - \frac{E_t}{3B_m} \right) \tag{11}$$

Such module can be obtained by testing in the hydrostatic state of stress.

5. THE CONCEPT OF THE THEORY OF PLASTICIFITY

It consists of three basic links: the condition of relaxation, the law of relaxation and cure and the condition of fracture. Plastic constitutive models differ according to the assumed function of release. The stress and deformation ratio assumes that the material is acting linearly elastic before the relaxation in accordance with the elastic parameters E and ν determined in the model and perfectly plastic after release. The total deformation or ratio of deformation components is:

$$d\varepsilon = d\varepsilon_e + d\varepsilon_p \tag{12}$$

dε - total deformation;
dε_e - elastic deformation;
dε_p - plastic deformation.
Basic plastic models:

- Mohr-Coulom model,
- Drucker Prager model,
- Von Mises model,
- Tresca model,

of which the simplest and most extensively applied in geotechnics is Mohr -Coulomb model. Figure 3 gives graphical representations of the main stresses in the space for classical fracture theories.

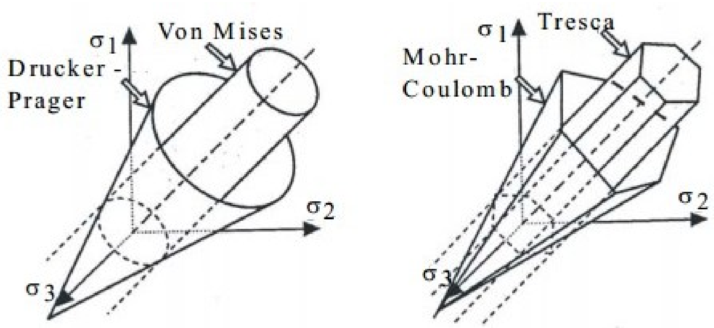
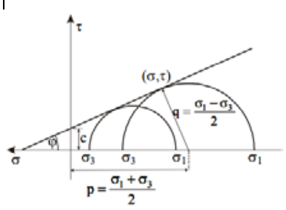
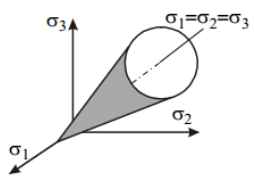
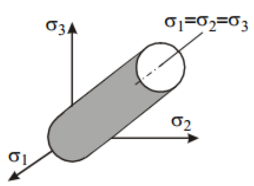
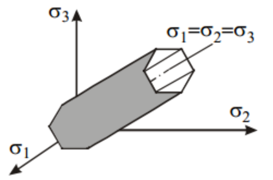


Figure 3. Classic fracture theories in the main stress area

Table 2. Overview of basic plastic models

Model	Graphic display	Fracture criterion	Description
Mohr - Coulomb's criterion for fracture for ɛ straight state of stress		$\tau = c + \sigma * tg\varphi$ A criterion written in the form of the maximum and minimum major stress, can be written for a general stress state in the form of three stress invariants.	τ - shift strain; σ - normal stress; c - cohesion material; φ - angle of friction.
Drucker - Prager model is given as a function of relaxation		$f = 3\sigma_m \sin\phi + J_2 - c = 0$	σ_m - main stress, J_2-second stress invariant, c, φ - maximum or peak cohesion and friction angle of the material
Von Mises model		$f(J_2) = \sqrt{3J_2} - \sigma_0 = 0$ li $f(J_2) = \sqrt{\sigma_x^2 - \sigma_y^2 - \sigma_x\sigma_y + 3\tau_{xy}^2} -$ $-\sigma_0 = 0$	σ_o -equivalent uni-axial stress determined by tests, J_2 - second deviator invariant of stress.

Tresca model		$f = 2q \cdot \cos\theta - k = 0$	f - function of relaxation q - second stress invariant k-maximum or peak uniform pressure compressive strength of the material.

6. ELASTIC - PLASTIC MODELS

This group of models includes:

- Ideally elastic - an ideal plastic model,
- Cam Clay and modified Cam Clay model,
- Deformation-softening model.

6.1. Ideally elastic - an ideal plastic model

Typical stress-deformation curve of linearly elastic - ideally plastic models is shown in Figure 4. The stresses are directly proportional to the deformations until the point of delivery is reached, and after the point of release the stress-strain curve is horizontal.

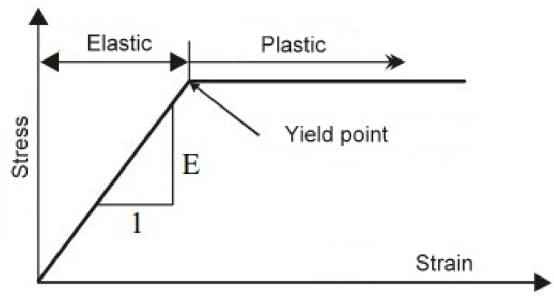


Figure 4. Elastic - an ideal plastic model

The elastoplastic theory, which describes the behavior of Figure 4, consists of the following elements:

Relative deformation is disassembled, on elastic and plastic component:

$$\{d\varepsilon\} = \{d\varepsilon_e\} + \{d\varepsilon_p\} \tag{12}$$

The elastic component of deformation can cause changes in stress. Elastic constituent equation has the form:

$$\{d\sigma\} = \{C_e\} + \{d\varepsilon_e\} \tag{13}$$

The relaxation function is defined by the form:

$$f = f(\sigma_x, \sigma_y, \sigma_z, \tau_{xy}) \tag{14}$$

or in matrix form:

$$df = \left[\frac{\delta f}{\delta\sigma}\right]\{d\sigma\} \tag{15}$$

If f < 0, the function describes the elastic property of the material. When f = 0, it describes strength or plasticity law. The plastic potential function takes the form:

$$g = g(\sigma_x, \sigma_y, \sigma_z, \tau_{xy}) \tag{16}$$

The direction of the plastic deformation increment is determined by the law of flow:

$$\{d\varepsilon_p\} = \lambda\left\{\frac{\delta g}{\delta\sigma}\right\} \tag{17}$$

Where:

g - the function of the plastic potential

λ - plastic scalar factor

The dλ must always have a positive value, and it is obtained from the condition that the function of relaxation is constant when fracture.

In geotechnics, Mohr-Coulomb's Law of Strength is most often used for the law of release:

$$f = \frac{\sigma_1 + \sigma_3}{2}\sin\varphi - \frac{\sigma_1 - \sigma_3}{2}\cos\varphi + c\cdot\cos\varphi = 0 \tag{18}$$

Usually the Mohr-Coulomb function of release is used for the plastic potential function, where the angle of internal friction φ is replaced by the angle of dilation ψ. Tangens of dilation angle is the ratio of increment of plastic volume deformation and plastic increment shear deformations (GeoSlope).

6.2. Cam Clay and a modified Cam Clay model

Cam Clay and modified Cam Clay models are in the group of elastoplastic models with curing (Atkinson and Bransby, 1978 and Britto and Gunn, 1987). Figure 3 explains the procedure of choosing an edometric test or isotropic compression test as one that can provide satisfactory data for modeling the Cam Clay model. Figure 5 (a) shows the ratio of the effective stress p' and the volume change ν, where ν = 1 + e, a, e, is the porosity of the sample. Figure 5 (b) shows the same pressure ratio shown on the natural logarithm scale, lnp'. Further simplification can be done when the hysteresis loop that occurs when unloading and reloading is replaced by the direction. The approximation is sufficiently accurate for the needs of further calculations. This view describes the actual state of the soil during over-consolidation, unloading and reloading. The direction of over-consolidation and the direction of normal consolidation show the properties of an elastic - plastic hardening curve of stress - deformation relation. The direction of over-consolidation corresponds to the linear-elastic part and the direction of normal consolidation to the plastic-curing part.

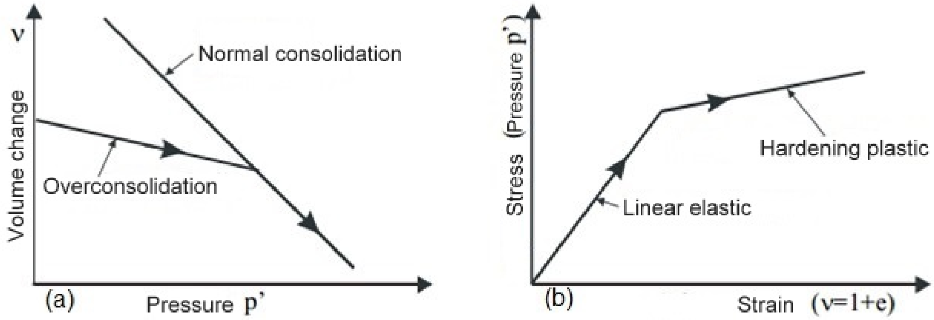


Figure 5. Consolidation and stress ratio

6.2.1. Release function in the Cam Clay model

Plastic hardening means the possibility of expanding the surface of the release. The description of the expansion of the release surface is achieved by introducing a variable for the plastic deformation into a function of release. In Cam Clay models this is a variable volume deformation ε_pv so that now the function of release has the form:

$$f = f(\sigma_x, \sigma_y, \sigma_z, \tau_{xy}, \varepsilon_{pv}) \tag{19}$$

In both models, the associated law of flow is assumed. In the Cam Clay model, the release function is given by the expression:

$$f = \frac{q}{M \cdot p'} + \ln\left(\frac{p'}{p'_x}\right) - 1 \tag{20}$$

while in the modified Cam Clay model it is given by the term:

$$f = \frac{q^2}{p'} + M^2 p' - 2M^2 p'_x \tag{21}$$

where:

p' - average effective stress;

$$p' = \frac{1}{3}\left(\sigma'_x + \sigma'_y + \sigma'_z\right) \tag{22}$$

q – stress deviator;

$$q = \sqrt{\frac{1}{2}\left[\left(\sigma_x - \sigma_y\right)^2 + \left(\sigma_y - \sigma_z\right)^2 + \left(\sigma_z - \sigma_x\right)^2\right] + \tau_{xy}} \tag{23}$$

M – Parameter of the material as a function of the angle of internal friction φ or direction gradient of the critical state when this is displayed in p'- q coordinates.

$$p'_x = \exp\left(\frac{\Gamma - \nu - k \ln p'}{\lambda - k}\right) \tag{24}$$

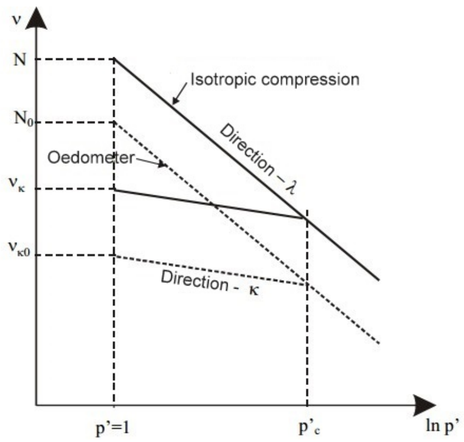


Figure 6. The results of laboratory tests of isotropic compression and uniaxial compression (edometric test) shown in the diagram ν - specific volume / ln p'- pressure

Parameters from the picture:

ν - the specific volume, defined by the pore-coefficient e, ν = 1 + e;

λ - the direction slope that represents the relation of the specific volume ν and the natural logarithm of average effective stress ln p', during isotropic compression of normally consolidated soil and

κ - the direction slope which represents the ratio between ν and ln p' in the elastic region

N and N_0 in Figure 6 are specific volumes of normally consolidated soil at the pressure p'= 1 kPa, and ν_k i ν_k_0 are of the specific volume of the over-consolidated samples when p' = 1 kPa. The position of the possible κ-forms is not unambiguous, but depends on the stress of the over-consolidation p'_c. Figure 7 a) and b) show Cam Clay models in the coordinate system p'- q.

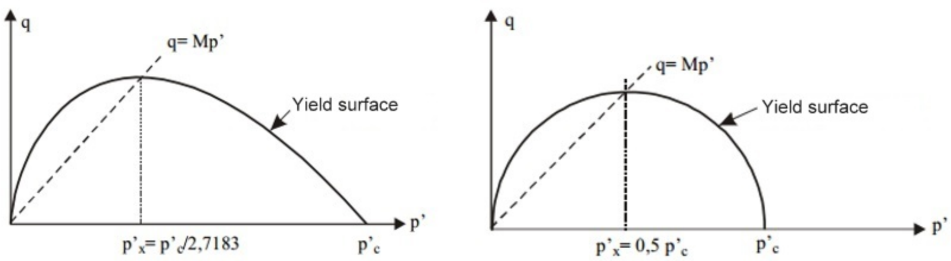


Figure 7. a) Cam Clay model b) Modified Cam Clay model

If pc' denotes the over-consolidation stress, i.e. the intersection of the direction slope κ and the direction slope λ, then in the Cam Clay model, the dimensions p'_x and p'_y are related by the expression:

$$\ln p'_x = \ln p'_c - 1 \tag{25}$$

and in the modified model:

$$p'_x = 0.5 p'_c \tag{26}$$

The size Γ denotes the specific soil volume on the line of critical states, for p'= 1 kPa, or ln p' = 0 (corresponds to the values of N in Figure 6). The line of critical states is given by the expression:

$$\nu = \Gamma - \lambda \ln p' \tag{27}$$

and represents the state of the material with expansion angle of $\psi = 0$.

6.3. Deformation-softening model

This model is elastic - softening - plastic and consists of three linear parts. The linear part grows to maximum shear strength, a softening part, in which the shear strength decreases from maximum to residual strength and the part in which the strength does not change (residual strength, Figure 8).

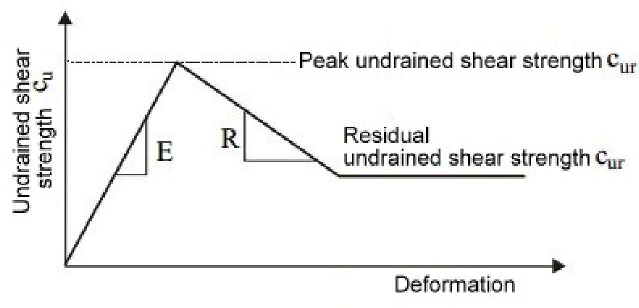


Figure 8. Deformation - softening model

The release function for this model is given through the positional stresses q and the undrained strength of c_u:

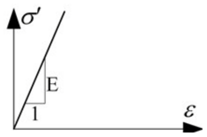
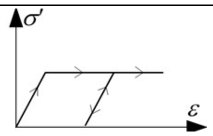
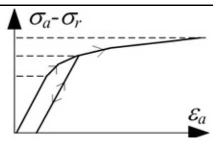
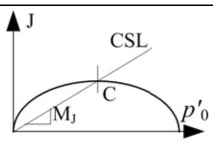
$$f = f\left(\sigma, \varepsilon_p\right) = q - \sqrt{3}c_u \tag{28}$$

The fracture at the shear strength c_u is equal to (σ_1 - σ_3) / 2. Shear stress q can be expressed over another invariant of stress J_2, q = 3J_2.

Each of these models has advantages and disadvantages, which are substantially limiting their usability for numerical simulation of structure and soil interactions. Basic characteristics of constitutional soil models available in commercial computer programs for the calculation of geotechnical structures are shown in Table 3.

Table 3. Basic characteristics of constitutional soil models for calculation of geotechnical structures

Model	The basic curve	Elements of soil behavior	Deficiency	Applica tion

Linear - elastic		- linear elastic behavior	- unlimited strength; - no dilatation occurs; - Unique stiffness for load /unloading - dependence between volume and shear stiffness	GSU (monolithic rocks)
Ideally elastic - plastic		- linear elastic behavior to fracture; - a unique fracture surface; - unique dilatation in case of fracture	- Unique stiffness for load / unloading -unique stiffness depending on the state of stress and deformation	Robust load control
Hardening		- shearing curing; - stiffness reduction depending on the state of deformation - dependency of starting stiffness from the stress condition	- lack of peak strength reduction - lack of critical condition - additional rules for unrelated behavior	Sand gravel rigid clay
A critical state model		- critical state; - isotropic cure; - reduction of peak strength	- critical condition	Soft clay

7. PHYSICAL MODEL

The reliability of the results in the function of the selected constitutional soil model was analyzed on the physical model. In the natural slopes after the excavations, the soil mass movement has occurred, which was observed over time by using an inclinometer, on the inclinometer borehole IBK 11 (Figure 9). The determined soil parameters for all layers on the engineering-geological profile (Figure 10) are shown in Table 4.

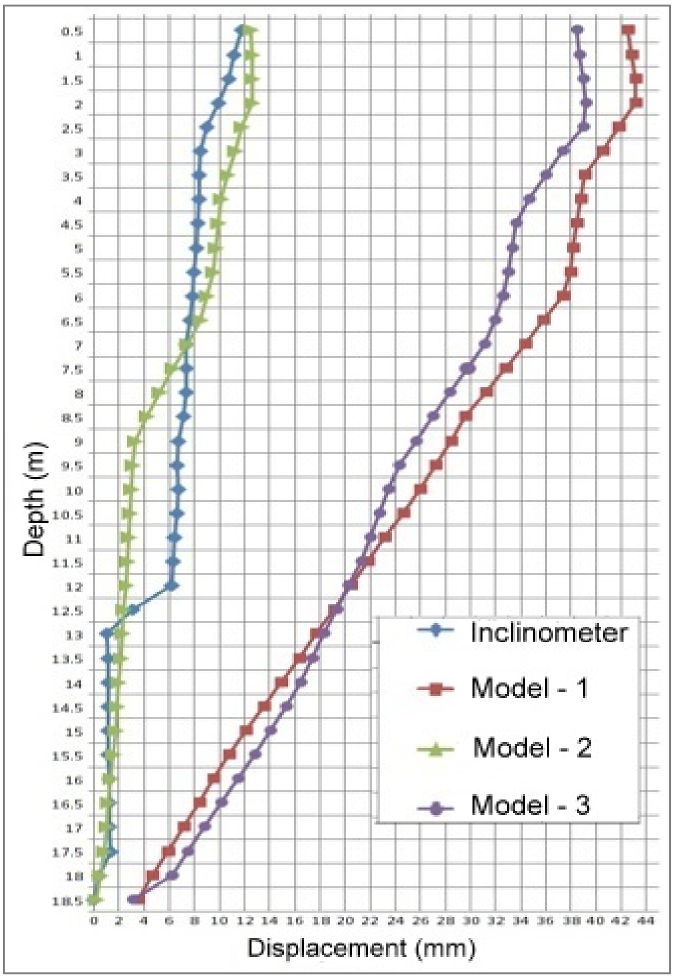


Figure 9. Movements on inclinometer IBK 11

Soil model number 1 represents Mohr-Coulomb model, isotropic stiffness, Mohr-Coulomb model, orthotropic stiffness, is represented by soil model number 2 and modified Cam-Clay model, isotropic stiffness, is represented by soil model number 3 in the Table 4.

The results of the movement were obtained using a software package Rocscience Phase 2 v.6.0., based on the finite elementh methode (FEM).

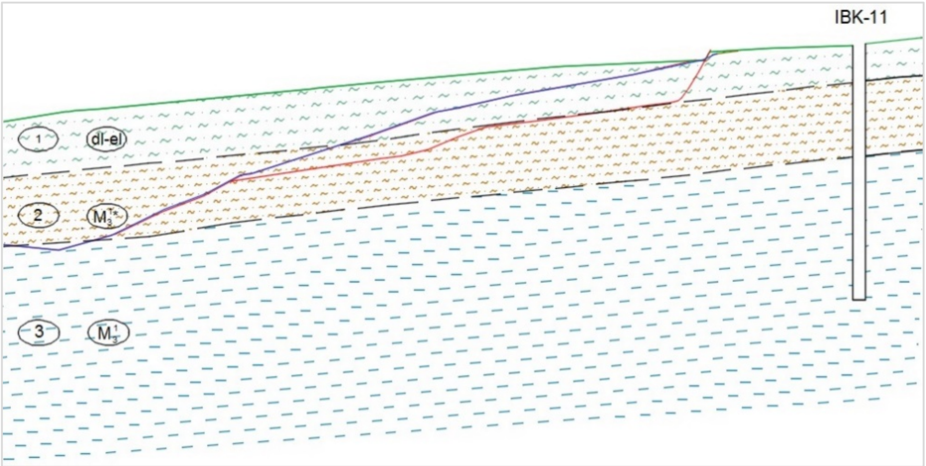


Figure 10. The engineering-geological profile

Table 4. Input parameters for soil models obtained by testing

Soil models		1			2			3		
Soil		1	2	3	1	2	3	1	2	3
γ	[kN/m3]	16.5	19.5	20.0	16.5	19.5	20.0	16.5	19.5	20.0
ϕ	[□]	16 (10)	16 (11)	18 (12)	16 (10)	16 (11)	18 (12)			
c	[kN/m2]	15 (3)	25 (5)	40 (8)	15 (3)	25 (5)	40 (8)			
M_v	[MPa]	5.4	13.1	22.8				5.4	13.1	22.8
ν	[-]	0.40	0.36	0.34				0.40	0.36	0.34
E	[MPa]	2.52	7.79	14.81				2.52	7.79	14.81
G_12	[MPa]				2.00	4.41	44.78			
E_1	[MPa]				4.00	8.30	87.00			
E_2	[MPa]				6.00	12.5	130.0			
E_z	[MPa]				5.00	12.0	120.0			
α	[□]				3	3	3			
N_12 =ν_2 =ν_2z	[-]				0.40	0.36	0.34			
M	[-]							0.59	0.37	1.21
λ	[-]							0.1	0.052	0.096
κ	[-]							0.03	0.01	0.02
ν	[-]							0.40	0.36	0.34
OCR	[-]							1	1	1

8. CONCLUSION

While solving geotechnical problems, we are able, by using computers, to apply some of the listed soil models included in the programs. Most computer programs, using the finite element methode, are able to determine the state of deformation and displacement, which is not possible by the limit balance method. Determination of the state of deformation and displacement is of crucial importance in most geotechnical engineering problems. Using the FEM (Finite Element Method), a more realistic estimation of the stress, deformation and shift distribution is possible, as well as finding the local fracture zone during the construction of the building. In addition, it is possible to more realistically look at and describe the geotechnical environment as heterogeneous, anisotropic, discontinuous, and changes in the conditions of equilibrium of the excavations. The reliability of the established geotechnical parameters has the greatest influence on the success of the geotechnical calculation. But the applicability of the soil model (Figure 9) also has a major impact on the results of the geotechnical calculation, and this should be given special attention.

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