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# Numerical finite element modeling of creep behavior in clayey soil specimen under axial loading

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**ABSTRACT**

Study on clayey soil behavior under loading condition can help to understand the time-dependent creep phenomenon. The numerical and experimental procedures used by different scholars. The presented study used the numerical models to assessment of the clayey soil behavior regarding the creep. To this end, finite element method by Plaxis program was used for estimate the deformation and displacements analysis of clayey soils under creep event. According to the results, it has indicated the stress-strain-time behavior of unconsolidated soils under one-dimensional consolidation which it can be seen that the stress-strain applied during different time stages has different effects on the amount of creep. But the general nature of creepiness in unconfined soils is the same and one-way. This factor indicates the amount of all-out forces and residual stresses to improve the creep occurrence conditions. By plotting the axial stress-strain curve against the time logarithm, it can be stated that axial strains are the main cause of creep rupture in unreinforced soils.

## 1. Introduction

Clayey soils are one of the most complex behavioral materials under different stress-strain conditions which show variable physical, mechanical and chemical parameters. These types of soils can be called as the most favorable materials for the occurrence of creep (Taylor and Merchant, 2012). The creep led various damages on structures which build in or on clayey soils like houses, roads and facilities. Due to the special nature of clayey soils, it can be said that the creep is occurred regarding the stress - deformation changes in soil mass under different type of loadings (Braaten, 2003). Based on the technical observation of deformations on damages and subsidence on earth structures, it concluded that the creep in clayey soils is a time dependent phenomenon. Stressed soils are undergoing a series of deformations continuously over time. Creep refers to shear or volume deformation that depends on time under constant force or stress which usually occurs at a variable rate over-time. In laboratory studies, the specimen is loaded to a certain constant stress and its deformations are examined which can represents as creep (Budhu, 2010; Feda, 2016; Maslov, 2016). The relationship between variable strain and time in one-dimensional consolidation experiments is shown in Fig. 1. According to this figure, the first stage shows the elastic deformations of the soil particles. The second stage is called initial consolidation, in which a series of deformations occur due to the disappearance of pore water

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pressure during loading; so that the initial consolidation rate is controlled by the water exits through soil pores. The third stage is called secondary consolidation or creep, in which volumetric deformations occur under a constant stress. The rate of secondary consolidation is controlled by the viscous strength of the soil structure. As results, the relationship between creep and time logarithms may be linear with upward or downward curvature (Das, 2008).

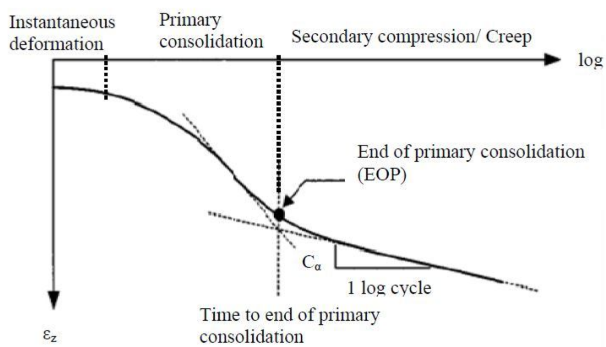


**Figure 1.** Strain-time relationship in one-dimensional consolidation test (Das, 2008)

The creep behavior of the soil reflects the deformations that occur over time under a constant stress, so it is necessary to study long-term deformations such as subsidence and ground/slope movements. When saturated soil is stressed, primary consolidation occurs with the depletion of pore water pressure. A significant number of sessions occur during the consolidation. After complete depletion of the pore water pressure (during the loading process), deformations occur over time, known as the secondary consolidation stage or creep. Long-term subsidence of soils can be attributed to soil creep behavior. Thus, it is important to predict the creep behavior of soils. Terzaghi and Peck (1967) formulated the one-dimensional consolidation theory to express volumetric changes during primary consolidation with several hypotheses that are widely used today. The primary consolidation rate depends on several factors such as permeability, sample thickness, drainage conditions and loading which is indicated by the consolidation coefficient ($C_v$). Depending on the drainage conditions (vertical, radial or bilateral), there are different methods for determining the $C_v$ coefficient (Berry and Wilkinson, 2007; Indraratna et al., 2010). In same thickness specimens, the primary consolidation rate under radial drainage conditions is higher than in vertical conditions (Robinson, 2015).

The primary consolidation curve is the same varied under vertical and radial drainage conditions. This indicates that the $C_v$ coefficient is independent of the drainage conditions. So, to measure the compressibility of soils, the compressibility coefficient ($C_c$) is used (compressibility coefficient is the negative slope of the primary consolidation curve). Fig. 1 shows the strain-logarithmic time relationship of the saturated clay time under one-dimensional compression testing. The primary and secondary consolidation stages can be displayed. In the initial stage of deformation, the deformations are due to the disappearance of the pore water pressure and in the second stage, the deformations are affected by the viscous nature of the soil structure. The initial consolidation curve can be expressed by Eq. 1. The soil deformation at the end of initial consolidation can be calculated by Eq. 2.

The secondary consolidation coefficient ($C_\alpha$) is an important parameter for describing creep behavior and secondary consolidation and does not depend much on the experimental conditions (Mesri and Choi, 1979; Mesri, 2003). This coefficient can be determined in different ways. Eq. 3 is one of the widely used formulations that received a specific attention from professionals (Das, 2008). Walker and Raymond (2010) investigated the effect of primary consolidation on secondary consolidation of clay soils and concluded that the secondary consolidation coefficient has a linear relationship with the $C_c$. As experiment, the ratio of the $C_\alpha$ to the consolidation index was estimated to be about 0.25. The consolidation index ($C_c$) is expressed by Eq. 4.

$$e = e_0 - C_c \log \frac{\sigma}{\sigma_0} \quad (1)$$

$$e = e_{EOP} - C_\alpha \log \frac{t}{t_{100}} \quad (2)$$

$$C_\alpha = \frac{\Delta e}{\Delta \log t} \quad (3)$$

$$C_c = \frac{\Delta e}{\Delta \log \sigma'_z} \quad (4)$$

where, e porosity ratio, $e_0$ initial porosity, $\sigma$ stress and $\sigma_0$ initial stress, $e_{EOP}$ porosity at end of primary consolidation, $t_{100}$ time at end of primary consolidation, $C_\alpha$ is secondary consolidation coefficient, $\Delta e$ is porosity ratio variation during secondary consolidation and t is the time.

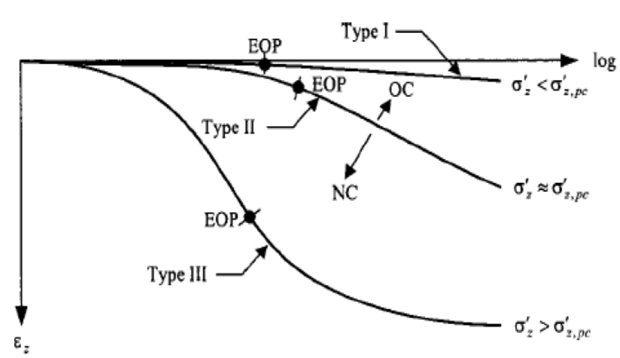


**Figure 2.** Strain-time relationship in one-dimensional consolidation test on clayey soils (Das, 2008)

Mesri and Godlewski (2001) expressed the $C_C$ ratio is 0.025±0.1 for natural clay. Mesri and Castro (2003) expressed a difference of 0.01±0.04 for non-organic soft clays and 0.01±0.05 for organic plastic clays. The scholar stated that a constant value of $C_\alpha/C_c$ indicates that if the compaction index is constant, the secondary consolidation coefficient of the clayey soils is independent of the effective vertical stresses. So, there is a nonlinear relationship between the vertical strains and the time logarithm ($\varepsilon_z$-log t) which is illustrated in Fig. 2. In this figure, curve 1 is for over-consolidation clays, curve 3 is for normal solidified clays, curve 2 is for slightly reinforced clay soils. Singh and Mitchell (2000) expressed that a nonlinearity relation for creep curves and introduced a parameter called m. Their relationship is as follows:

$$m = -\frac{\Delta \log \varepsilon_z}{\Delta \log t} \tag{5}$$

The value of m for normal consolidated clays is always less than one and greater than one for over consolidated clays. Of course, for clay soils that are slightly more consolidated, the m is initially greater than one, but over time during load is became less than one. Singh and Mitchell (2000) stated that the hypothetical A and B curves are used to determine the onset time of creep deformation. According to A curve, the thickness of the sample has no effect on the position of the initial consolidation curve in the e-σ' diagram. Curve A is suitable for experiments where the sample thickness is small. According to curve B, creep occurs throughout the duration of the deformation. This means that the deformations are not the same at the end of the primary consolidation. So, when the load is applied, it can be considered as a source. As results, the A and B curves provide the relationship between variable strain and time for different samples which is shown in Fig. 3. In fact, for low-thickness samples, the strain-time relationship is almost the same. Because the creep occurs during primary consolidation are low and can be ignored. According to B curve, the deformation's values of end of primary consolidation are larger for thicker specimens than thickness specimens. So, creep is significant during primary consolidation. Creep can lead to particle rearrangement and a more stable state, if stresses are isotropic; internal forces are vertical and tangential.

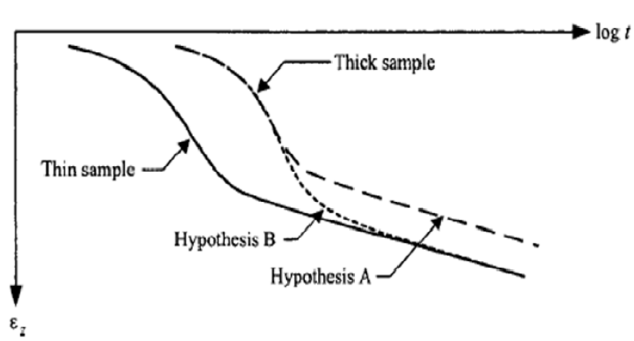


**Figure 3.** Effect of sample thickness on primary consolidation of normal consolidated clays (Singh and Mitchell, 2000)

## 2. Material and Methods

The first studies on stress-strain behavior were performed during one-dimensional consolidation by Carl Terzaghi. In 1923 he published his theory of one-dimensional consolidation, which became the basis for further studies. This theory is now known as the classical theory or the theory of consolidation. Terzaghi theory was developed to predict the long-term subsidence of structures and believes that the deformation of a saturated soil layer is determined only by the rate of extraction of porous water under the application of an external load. The soil skeleton is considered to be a linear malleable body with the characteristic of instantaneous deformation. Terzaghi's theory is based on the following hypotheses (Das, 2005; Azarafza and Asghari-Kaljahi, 2016):

- The soil is completely saturated.
- The soil-water system is homogeneous and isotropic.
- Water is assumed to be incompressible.
- Grain compressibility is ignored.
- There is a linear relationship between the effective vertical stress and the void ratio.
- Water flow is only in one direction.
- Darcy's law of linear leakage is valid.
- The permeability coefficient is assumed to be constant during consolidation.

Based on these hypotheses, and using an analogy between the theory of consolidation and the theory of heat transfer, Terzaghi proposed a differential equation based on the water pore pressure (Das, 2005):

$$\frac{\partial u}{\partial t} = C_V \cdot \frac{\partial^2 u}{\partial z^2} \tag{6}$$

where, u is pore water pressure, t is the time, z is depth and $C_v$ is consolidation coefficient that is related to the permeability as follows (Das, 2005):

$$C_V = \frac{k}{\gamma_w . m_v} \tag{7}$$

where, $m_v$ is volumetric compressibility coefficient were is defined as the change in volume per unit volume to increase the effective stress of the unit as (Das, 2005):

$$m_v = \frac{1}{1+e_0} . \frac{\Delta e}{\Delta \sigma'} \tag{8}$$

Solving the above equation for two two-way drainage conditions, or specific boundary conditions leads to the following equations (Das, 2005):

$$u = \sum_{n=0}^{\infty} [\frac{2.u_0}{N} . \sin(\frac{N.z}{H})] . e^{-(N^2.T_v)} \tag{9}$$

$$N = \frac{\pi}{2}(2n+1), \text{ for n} = 1,2,3,... \tag{10}$$

$$\mathrm{T}_v = \frac{C_v.t}{H^2} \tag{11}$$

$$U_z = 1 - \frac{(\frac{1}{2H_{dr}}) . \int_0^{2H_{dr}} u_z dz}{u_0} \tag{12}$$

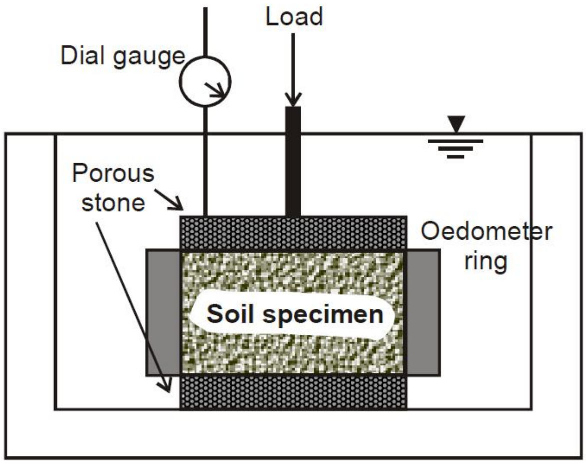


**Figure 4.** The shame of the Oedometer test (Das, 2005)

where, H is sample height, $u_0$ is initial pore pressure, $T_v$ is time factor, $U_z$ is degree of average consolidation. In 1910, the first equation was proposed for the one-dimensional consolidation test by Frontar in France. In 1919, the Swedish Geotechnical Commission conducted a one-dimensional consolidation test for clay by draining water from both sides of the sample through sand layers (Das, 2005; 2008). This method was later used by Terzaghi for consolidation analysis which known as Oedometer or uniaxial consolidation test as shown in Fig. 4.

With the advancement of technology and the entry of computers into the field of civil engineering and geotechnics, the analysis of various topics with the use of computer has become much broader and more complex problems have been solved by these approaches. Computer programs have made it much more accurate and easier to solve complex discrete and differential equations involving continuous environments such as soil. One of the most important issues that is comprehensively emphasized today and many researchers are trying to solve it, is the issues related to time-dependent creeps and consolidation, which has been raised as the goal of this research. In this regard, the numerical approach of finite element and Plaxis software has been used.

Plaxis able to use the creep model to analysis of consolidation behavior of the soil regarding time dependences loading condition. The presented study uses this advancement to modeling the one-dimensional consolidation on clayey soils under uniaxial loading. The models provide in several stages which can be categorized as geometric and boundary conditions, behavioral model, materials, water pore pressure status, and consolidation. The geometrical model is providing based on standard Oedometer tests and boundary conditions is defined based on default of Plaxis for the model. Table 1 is provided the information about the usage materials in the modeling which is regularly obtained from conducting different types of geotechnical test in laboratory. These tests are standards by american society for testing and materials (ASTM).

**Table 1.** The materials used in the modeling

| Parameter | Index | Unit | Value |
|---|---|---|---|
| Dry density | $\gamma_{unsat}$ | kN/m$^3$ | 17.20 |
| Saturated density | $\gamma_{sat}$ | kN/m$^3$ | 19.00 |
| Elastic modulus | $E_{ref}$ | kN/m$^3$ | 10000 |
| Poisson ratio | $\upsilon$ | - | 0.25 |
| Cohesion | $C_{ref}$ | kN/m$^3$ | 0.00 |
| Internal friction | $\phi$ | Degree | 35 |
| Dilation | $\psi$ | Degree | 0.00 |
| Creep soil compaction index | $\lambda^*$ | - | 0.105 |
| Creep soil swelling index | $\kappa^*$ | - | 0.016 |
| Modified creep index | $\mu^*$ | - | 0.004 |
| Consolidation pressure index | $k_0^{NC}$ | - | Default |

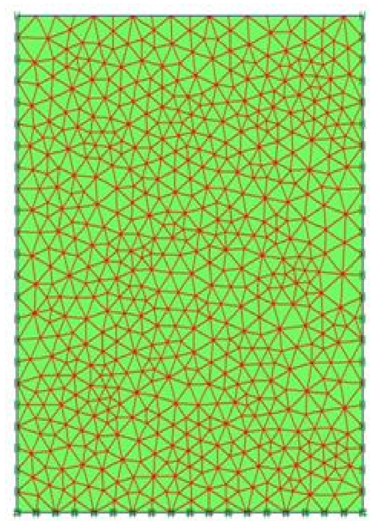

**Figure 5.** The geometrical model of the test with boundary conditions

Fig. 5 shows a geometrical model with boundary conditions. The prepared model is considered to be saturated. During the consolidation phase, the model was subjected to consolidation loading for 1 day (24 hours) and the behavioral changes of the model were measured.

## 3. Results and Discussions

By considering the various stages of the one-dimensional consolidations in creep analysis by finite element modeling, it can be said that the executive model related to the creep model has been implemented in a completely favorable way for uniaxial loading. Figs. 6 to 11 are presents the results one-dimensional consolidations by Plaxis software. The modeling contains the displacements, stress field, shear stress conditions and active pore pressure status. The models indicated that the uniaxial loading in time duration can provide the complex creep status which lead to plastic failure in clayey soil specimen.

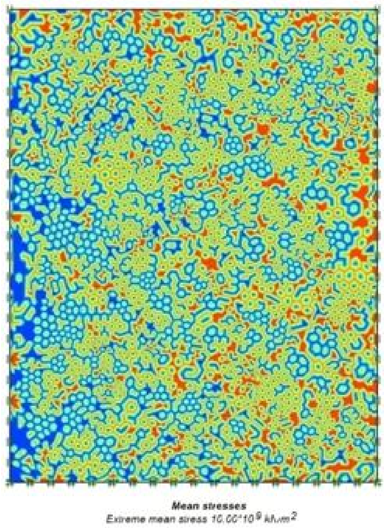


**Figure 6.** The stress field condition during consolidation stages

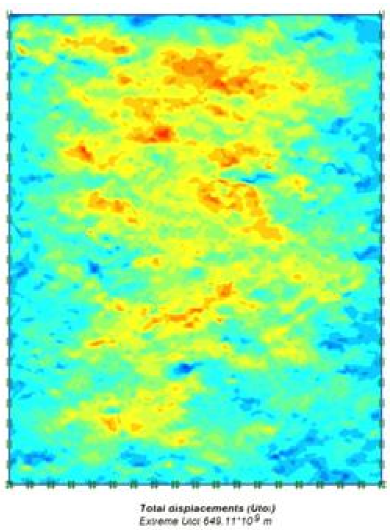


**Figure 7.** The total displacements during consolidation stages

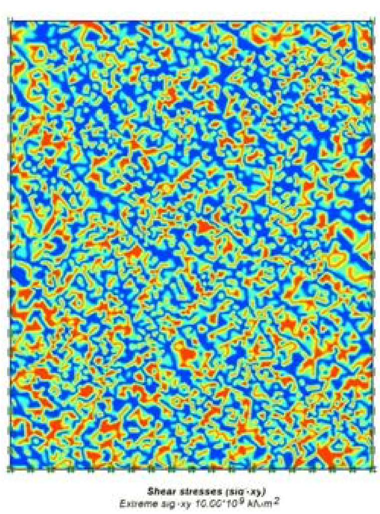


**Figure 8.** The main shear stress condition during consolidation stages

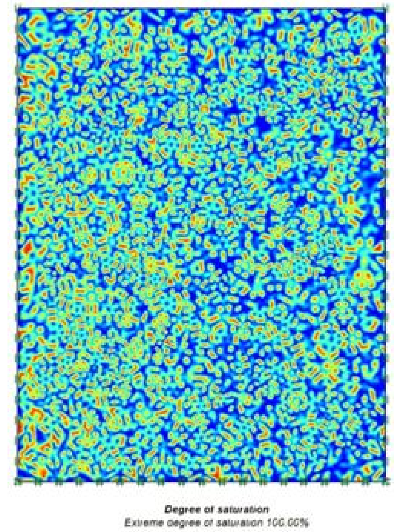


**Figure 9.** The saturation degree during creep occurrence

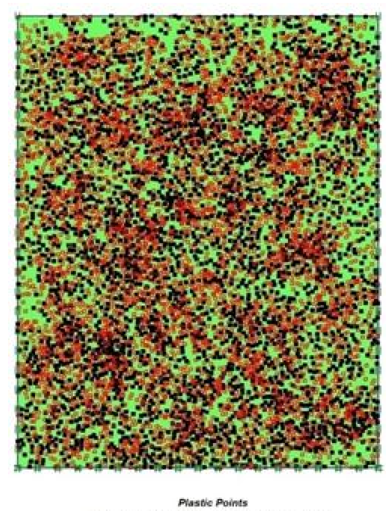


**Figure 10.** The plastic points distribution during creep occurrence

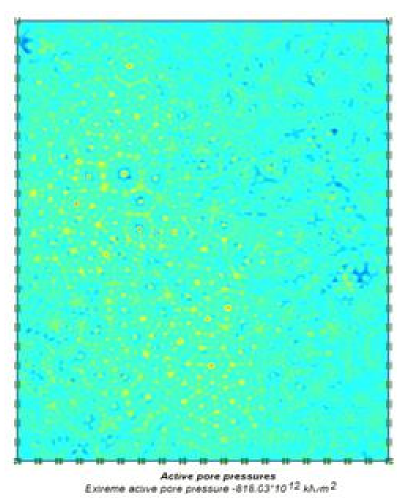


**Figure 11.** The active pore pressure status during creep occurrence

Based on the results of numerical modeling related to the creep model by Plaxis, it can be stated that the amount of creep occurring in the uniaxial loading is affected by the axial loading condition which can be justified by the application controlled loading. As shown in Fig. 12, the amount of creep's strain that related to one-dimensional consolidation lead to low to high loading condition with deformation in sample in the body of unconfined soil. This event has also been confirmed by laboratory experiments.

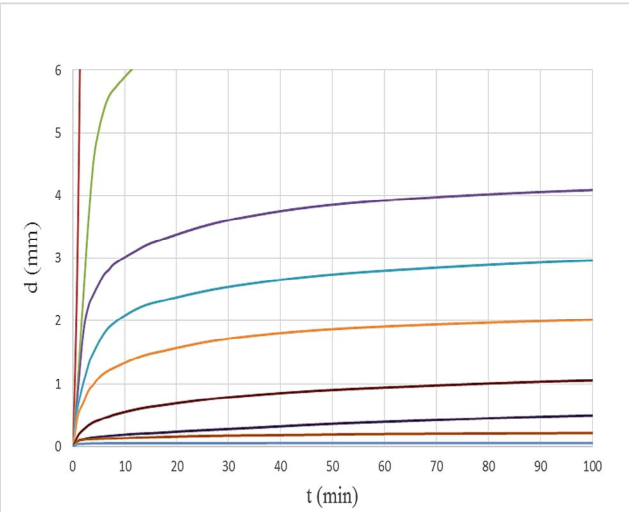


**Figure 12.** The time-strain diagram related to the creep in various axial loading

## 4. Conclusion

Creep behavior is one of the most common phenomena observed in unconsolidated soils. The study is considered the clayey soils behavior regarding to the creep under the uniaxial loadings. The consolidation process during one-dimensional consolidation test as same as Oedometer test is used to estimate the creep condition on studied sample. The numerical model by using Plaxis software was use to demonstrate the creep behavior during the axial loading. In this regard, the models provide in several stages which can be classified in geometric and boundary conditions, behavioral model, materials, water pore pressure status, and consolidation. According to the results, it has indicated the stress-strain-time behavior of unconsolidated soils under one-dimensional consolidation which it can be seen that the stress-strain applied during different time stages has different effects on the amount of creep. But the general nature of creepiness in unconfined soils is the same and one-way. This factor indicates the amount of all-out forces and residual stresses to improve the creep occurrence conditions. By plotting the axial stress-strain curve against the time logarithm, it can be stated that axial strains are the main cause of creep rupture in unreinforced soils.

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