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Effect of consolidation on the behaviour of excavations in fine-grained soils

Fabiano Bertoldo[a,]*, Luigi Callisto[a]

[a]*Sapienza Università di Roma, via Eudossiana 18, Roma 00184, Italy*

Abstract

This study deals with the detrimental effects of consolidation on the behaviour of excavations carried out in fine-grained soils. With the aim of reproducing the main aspects of the mechanical behaviour of medium-soft clays, an advanced constitutive model was adopted, which is based on bounding surface plasticity and can take into account the damage to the soil microstructure induced by plastic strains. In a first stage of the work, this constitutive model was implemented into a finite element program, and its response was studied through a series of single-element tests, evidencing the effect of the different soil parameters and initial conditions. In the present paper, the constitutive model was used to simulate the behaviour of an idealised excavation, studying the effect of the progressive dissipation of the excess pore water pressures generated during the excavation stages. It was found that, under certain conditions, the initial degree of structure can be progressively lost during consolidation. This detrimental effect produces significant deformation increments and in some cases can drive the system to collapse.

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Keywords: Constitutive model; excavation; clay; consolidation; drainage conditions

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1. Introduction

The behaviour of excavations in fine-grained soils is affected by drainage conditions. During the excavation stages, which usually occur in nearly undrained conditions, negative excess pore water pressures develop in the soil;

* Corresponding author. Tel.: +39-0644585330.
E-mail address: fabiano.bertoldo@uniroma1.it

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the ensuing distribution of the pore pressures affects the effective stresses in the soil, and therefore has an influence on the behaviour of the structure. After the excavation, the negative excess pore pressures dissipate with time as an effect of consolidation, leading to a general reduction of the effective stresses in the soil. This effect can cause additional displacements of the retaining structures, especially in cases where excavations are poorly constrained in the horizontal direction. For example, a deleterious effect of consolidation on the lateral movement of a retaining wall placed in support of an excavation was observed in San Francisco [1], where the maximum horizontal displacement showed a significant increase after that the bottom of the excavation was reached.

In structured soils, the reduction of effective stresses associate to the consolidation produces plastic strains that in turn can lead to a deterioration of the mechanical characteristics of the soil; compared to the case of unstructured soil, this may have an unfavourable effect on the behaviour of the excavation. This paper evaluates for an idealised case the effect of consolidation on the behaviour of excavations carried out in structured soils, through finite element computations that employ an advanced constitutive model including the progressive loss of structure caused by plastic strains.

2. Description of the constitutive model

The behaviour of natural clays may diverge considerably from that of the reconstituted ones. For example, the typical relationship between void ratio and effective vertical stress of a soil formed by deposition in the laboratory usually has no direct counterpart in the site, where a series of natural phenomena interfere with the depositional process. Also, natural soils may exhibit a softening behaviour even along stress paths that do not lead to critical state. The RW constitutive model, obtained by rearranging the original constitutive model developed by Rouainia & Muir Wood [2] can take into account the above features of natural clays, as well as other key aspects of these materials. For example, this model can consider the dependence of the soil stiffness on the direction of the stress path and loading history: this aspect is fundamental in relation to excavation problems, in which soil elements located in front and behind the retaining walls experience very different stress paths.

The constitutive model was formulated according the bounding surface plasticity theory [3]; to allow for an appropriate evolution of the stiffness, the plastic modulus depends on the current distance between the yield surface and an outer bounding surface, called the structure surface. The yield surface, shown in Figure 1a, is centred in $\bar{\boldsymbol{\alpha}}$ and has a dimension that is related to the state variable p_C. In order to take into account the different strength shown in compression and extension triaxial tests (Fig. 1b), the shape of this surface depends on the Lode's angle through a parameter m. In the context of the bounding surface plasticity theory, an outer surface is defined (Fig. 1a), and a condition that the yield locus must be always internal to the structure surface is prescribed. The structure surface is centred on the isotropic axis (at $\hat{\boldsymbol{\alpha}}$) and has a size that is related to p_C via the state variable r, representing the current degree of structure.

Along stress paths that remain inside the yield locus the response is hypo-elastic, depending on the parameters ν and κ^*. Plastic strains produce isotropic ($\dot{p}_C$) and kinematic ($\dot{\bar{\boldsymbol{\alpha}}}$) hardening of the yield surface. Specifically, the variation of p_C is a function of plastic volumetric strains, while the variation of $\bar{\boldsymbol{\alpha}}$ depends on all the components of the plastic strain increment via a kinematic hardening rule obtained by imposing that the two model surfaces cannot intersect (the main constitutive equations are given in the appendix A). The non-intersection condition implies the presence of the term $\dot{r}$ (which expresses the variation of the degree of structure) into the kinematic hardening rule. In the present constitutive model r can only decrease, as an effect of volumetric and deviatoric plastic strains, down to a minimum value of one, with a rapidity which is controlled by the parameter k.

In order to obtain a continuous and smooth evolution of the stiffness, a term h that depends on the distance between the two model surfaces was included in the kinematic hardening rule (and consequently in the plastic modulus) [4]. This distance, denoted by b in Figure 1c, represents the component of the stress tensor $\boldsymbol{\beta}$ (connecting the current stress state with its conjugate) along the normal to the yield surface. The term h depends on the quantities b_{max} and b_0: the first one represent the maximum value that b can assume by keeping unchanged the position of the structure surface (as shown in Fig. 1c), while b_0 denotes the value assumed by b in the previous elasto-plastic transition, and was introduced to obtain a continuous transition from elastic to elasto-plastic states, similarly to Kavvadas & Amorosi [5].

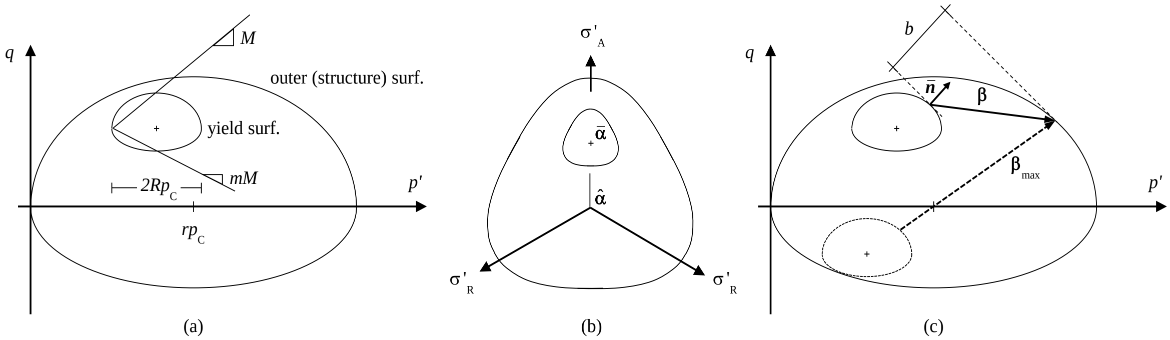

Fig. 1. Bounding and yield surfaces in triaxial conditions: (a), (c) in p'-q space; (b) in π plane.

Table 1. Soil properties and state parameters, including the factor of safety F against basal heave.

Case	ν	κ^*	λ^*	m	M	R	k	A_d	B	ψ	ψ_2	r_0	F
T1	0.22	0.025	0.15	0.88	1.0	0.1	2.5	0.5	5.0	2.0	0.0	1.0	1.50
T2	0.22	0.025	0.15	0.88	1.0	0.1	2.5	0.5	5.0	2.0	0.0	2.0	1.23
T3	0.22	0.025	0.15	0.88	1.0	0.1	4.0	0.5	5.0	2.0	0.0	2.0	1.19

The RW model was implemented in Abaqus through a UMAT subroutine written in Fortran language [4]. This subroutine performs the integration of the constitutive laws with the modified Euler explicit scheme [6], with an automatic subdivision of the strain increments taking into account the magnitude of nonlinearity in the response.

3. Analysis of excavations in fine-grained soils

3.1. Behaviour obtained during the excavation stages

An idealised problem was studied, consisting of a 7 m-deep and 6 m-large excavation supported by a 12 m-long retaining wall (Fig. 2a). The ground conditions comprise 3 metres of over-consolidated clay followed by a thick slightly over-consolidated clay deposit. Ground-water conditions were assumed hydrostatic with a water table positioned at 3 metres below the ground level.

The calculation was carried out with Abaqus/Standard and included the following stages: the initialisation of the in situ stress state and state parameters; a drained excavation to a depth of one meter; the placing of a horizontal constraint at the top of the wall; an undrained excavation down to the final depth; a consolidation stage to dissipate any excess pore water pressure. The retaining wall was described with beam elements and regarded as linearly elastic with a bending stiffness EI=0.312 GNm^2/m. The soil was modelled with 8-noded plane-strain porous elements with reduced integration, and was assigned the RW constitutive model (Table 1 summarizes the soil properties). In the initial conditions, the centre of the yield surface was assumed to coincide with the geostatic stress state, evaluated according with the profile of the earth coefficient at rest K_0 shown in Figure 2b. Three analysis cases were considered: a structureless one, and two cases with an initial degree of structure r_0=2, characterised by two different destructuration rates (k = 2.5 and k = 4). The initial dimension of the outer surface is the same for all cases, and varies with depth as shown in Figure 2c. For each case, the Bjerrum & Eide [7] factor of safety F against basal failure was calculated using values of the undrained shear strength obtained from the numerical simulation of consolidated-undrained triaxial tests, as reported in Table 1.

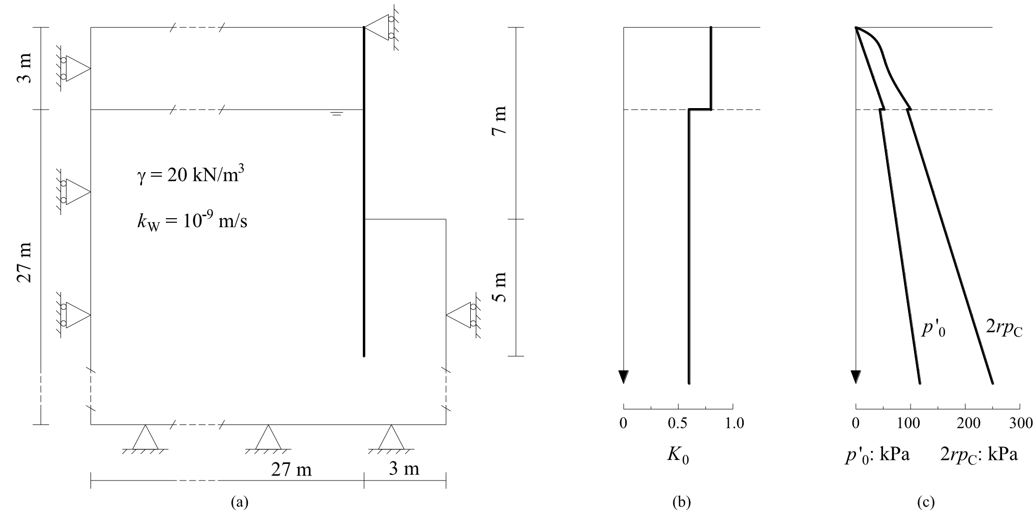

Fig. 2. Layout of the excavation and initial conditions.

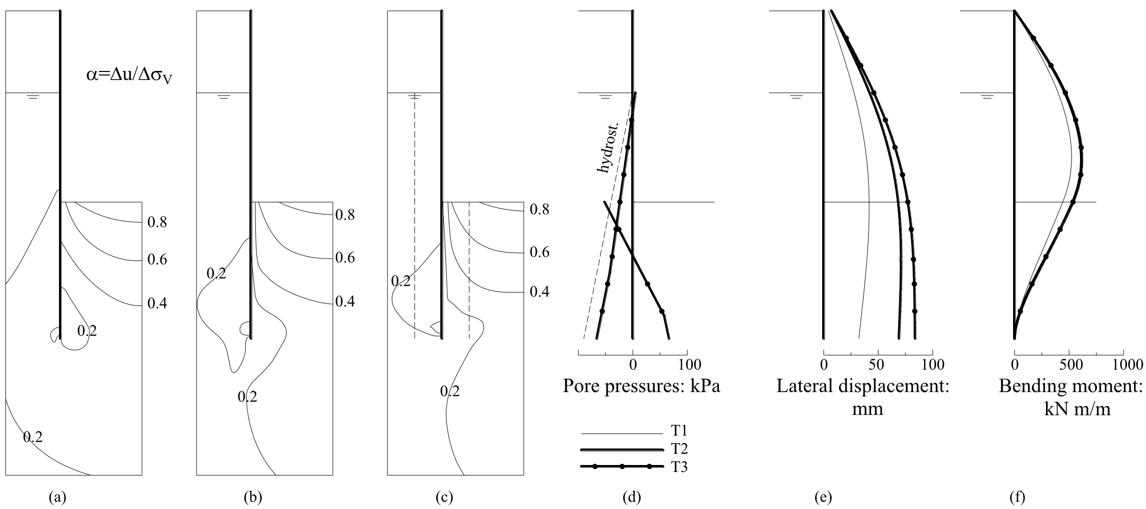

Fig. 3. Pore pressure reduction factors α in the final excavation stage for cases T1 (a), T2 (b) and T3 (c); (d) pore pressure distribution for case T3; (e) lateral displacements and (f) bending moments in the final excavation stage.

In the undrained excavation stages a reduction of the pore water pressures occurred as a response of the soil constitutive behaviour to the undrained conditions. In order to compare the results obtained in the different cases, it is useful to express the effect of the excavation through the ratio α of the excess pore water pressure Δu to the notional variation of the total vertical stresses $\Delta\sigma_V=\gamma\times H$ produced by the excavation [8]. Contours of α are plotted in Figures 3a-c with reference to the end of excavation: it can be observed that this ratio is larger at the bottom of the excavation and tends to decrease at grater depths. Large values of α are beneficial, implying substantial excess pore water pressures, corresponding to small decrements of the effective stresses. It can be seen from the contours of Figure 3 that the values of α are larger for case T1, characterised by the absence of structure. As the initial degree of structure becomes significant (case T2), and the velocity of the destructuration process increases (case T3), the

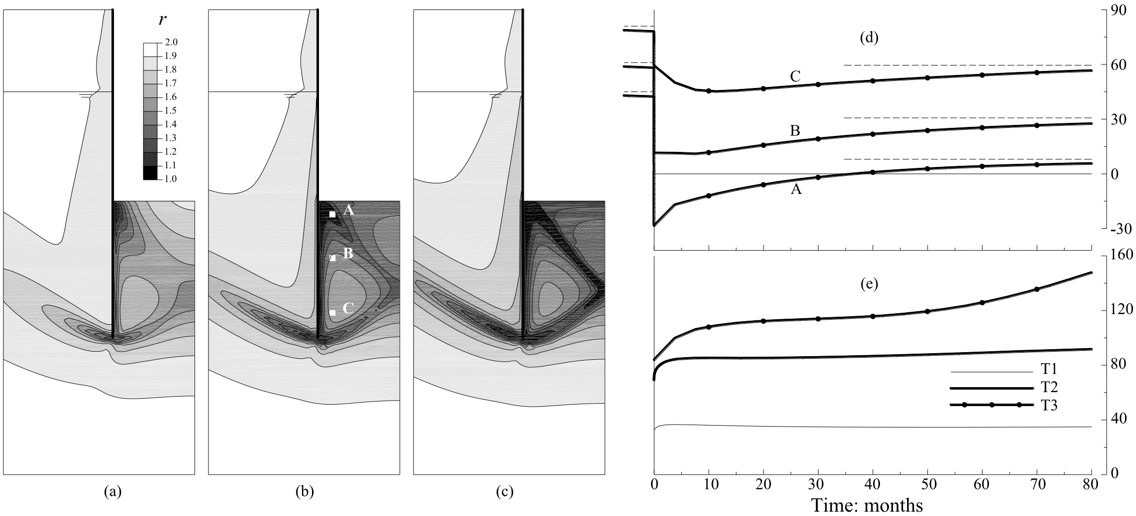

Fig. 4. Effect of consolidation: contours of the degree of structure for case T3 (a) at the beginning of the consolidation and after (b) 40 months and (c) 80 months; (d) change of pore water pressures case T3; (e) evolution of horizontal displacements of the wall toe.

values of α at the end of the excavation are smaller, as an effect of some loss of structure that has already occurred at this stage.

For case T3, Figure 3d shows the distribution of the pore water pressure along the dashed lines of Figure 3c. It can be seen that the substantial decrease in pore water pressure computed in front of the wall corresponds to the development of significant suctions in the soil located below the excavation level. Figures 3e and 3f show the profiles of the horizontal displacements and bending moments in the retaining wall at the end of the undrained excavation. The analysis cases with an initial degree of structure show greater displacements and slightly greater internal forces; the largest wall deformation corresponds to case T3, for which the loss of structure with plastic strains proceeds more rapidly.

3.2. Effect of consolidation

The consolidation stage of the numerical analyses was carried out using a permeability coefficient of 10^{-9} m/s and assuming that the lateral and lower boundaries are fully impermeable, while water can flow in both directions through the bottom of the excavation. Figure 4d plots the pore water pressures obtained for case T3 in three elements situated near the wall at different depths (see Fig. 4b) as a function of time. It is seen that the pore water pressures drop from their initial value during the undrained excavation, while during the consolidation phase they tend asymptotically to the values associated to the final steady-state seepage condition. These increases in the pore water pressures produce a gradual reduction in the effective stresses in the soil, that in turn cause volumetric and deviatoric plastic strains. Figure 4e shows the temporal variation of the horizontal displacement computed for the toe of the wall. In the initial part of the consolidation the dissipation of the excess pore water pressure produces an increase of the wall displacements, and this is evident especially in cases of structured soil, where the decrease of the effective stresses is accompanied by a gradual degradation of the soil mechanical properties. For the analysis case with the larger destructuration rate (T3) that displacements appear initially to tend to a stationary value, but then they start to increase again, leading the system to collapse. This effect is due to a significant loss of structure occurring during consolidation, that can be appreciated looking at the contours of equal current degree of structure depicted in Figures 4a-c, that refer to the three successive time instants of 0, 40 and 80 months, corresponding to an average degree of consolidation of about 0%, 60% and 90% respectively. It is evident that in the latest time instant

(Fig. 4c) the degree of structure has vanished ($r = 1$) in a substantial portion of soil: in this destructured state, the soil is no longer able to support the excavation.

4. Conclusions

Consolidation has characteristically an unfavourable effect on the behaviour of an excavation, because it is associated to a gradual decrease in the mean effective stresses. For the typical deviatoric stress paths associated to an excavation, a decrease in the mean effective stresses may produce a significant damage to a soil characterised by an appreciable initial degree of structure, which is lost with a relative rapidity as an effect of plastic strains. For the idealised case presented in this paper, it would appear that the loss of structure occurring during consolidation can bring to collapse a structure that in the absence of this phenomenon would otherwise be stable. This result points to the necessity of considering the potential loss of soil structure in the prediction of long-term behaviour of excavations in low-permeability soils.

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Appendix A. Constitutive equations for the model.

$$\frac{\dot{p}_C}{p_C} = \frac{\dot{\varepsilon}_p{}^P}{\left(\lambda^* - \kappa^*\right)}$$

$$\dot{\bar{\boldsymbol{\alpha}}} = \dot{\hat{\boldsymbol{\alpha}}} + \left(\frac{\dot{p}_C}{p_C} + \frac{\dot{r}}{r - R}\right)\left(\bar{\boldsymbol{\alpha}} - \hat{\boldsymbol{\alpha}}\right) + \dot{\lambda}\frac{h}{b}\boldsymbol{\beta}$$

$$\frac{\dot{r}}{r-1} = -\frac{k}{\left(\lambda^* - \kappa^*\right)}\sqrt{\left(1 - A_d\right)\left(\dot{\varepsilon}_p{}^P\right)^2 + A_d\left(\dot{\varepsilon}_q{}^P\right)^2}$$

$$h = \frac{Bp_C}{R\left(\lambda^* - \kappa^*\right)}\left(\frac{b}{b_{max}}\right)^{\psi}\left(\frac{b_0}{b_0 - b}\right)^{\psi_2}$$