Short-term versus long-term stability problems in clay explained through classical figures and finite element simulations

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ABSTRACT: The construction of embankments on soft clay and the excavation of unsupported cuts in clay are classical reference problems, the discussion of which may be particularly fecund to clear up a number of paramount ideas on the behaviour of fine-grained soils. That discussion may involve issues such as: generation of positive *versus* negative excess pore pressures; undrained *versus* drained soil resistance; short-term *versus* long-term stability conditions; effective stress *versus* total stress analyses; and consolidation as a general phenomenon of dissipation of excess pore pressures whatever their sign. These two problems are the scope of two well-known figures by Bishop & Bjerrum (1960). The paper revisits these figures and provides an expanded interpretation. The usefulness of these figures, and of their detailed discussion, is emphasized and proposed as the closure of a course on Soil Mechanics, as the opening of a course on Soil Engineering, or even both. The discussion on the embankment on soft clay is enriched with results from a finite element simulation that gives emphasis on the change in time of the ground displacements, of the distribution of the excess pore pressures and of the safety factors against a general slip failure.

Keywords: Embankments, Cuts, Clayey masses, Consolidation, Short and long-term stability

1 Introduction

In a basic undergraduate course on Soil Mechanics the most complex issues are typically concentrated in the final part, particularly when the undrained loading of fine soils is discussed and the concept of undrained shear strength is introduced.

It is a real challenge for teachers to explain and for students to understand that, although the undrained shear strength, c_u, is to be used in total stress analyses, its value is totally controlled by the effective stresses, as well as by the excess pore pressures generated by the undrained loading.

An important issue to emphasize is that an undrained loading is always followed by a dissipation of the excess pore pressures, that is, by a consolidation process, which will proceed until equilibrium hydraulic conditions are re-established in the ground mass. And that this phenomenon is critical not only for the time-delayed soil volumetric deformations and for the resulting surface settlements, but also for the stability conditions of the geotechnical structures responsible for triggering the consolidation process.

The discussion of this matter is an excellent field to treat complex topics such as: generation of positive *versus* negative excess pore pressures; undrained *versus* drained soil mass resistance; short-term *versus* long-term stability conditions; effective stress *versus* total stress analyses. This discussion is strongly facilitated if based on two reference geotechnical problems: the construction of embankments on soft clayey soils and the excavation of cut slopes in clays (Lambe & Whitman, 1979; Matos Fernandes, 2020).

This paper is divided in two main parts. In the first part, these two problems are discussed on the basis of two classical figures by Bishop & Bjerrum (1960). In the second part, the problem of the embankment construction is simulated by utilizing a finite element code capable of performing soil consolidation coupled analyses (Borges, 1995). Some numerical results are presented that corroborate the considerations and conclusions of the theoretical discussion based on the first classical figure mentioned above.

2 Classical figures

2.1 Embankment construction on a soft clayey soil mass

Figure 1 contains the classical schematic illustrating the relevant phenomena involved in the construction of an embankment on a soft clayey deposit (Bishop & Bjerrum, 1960).

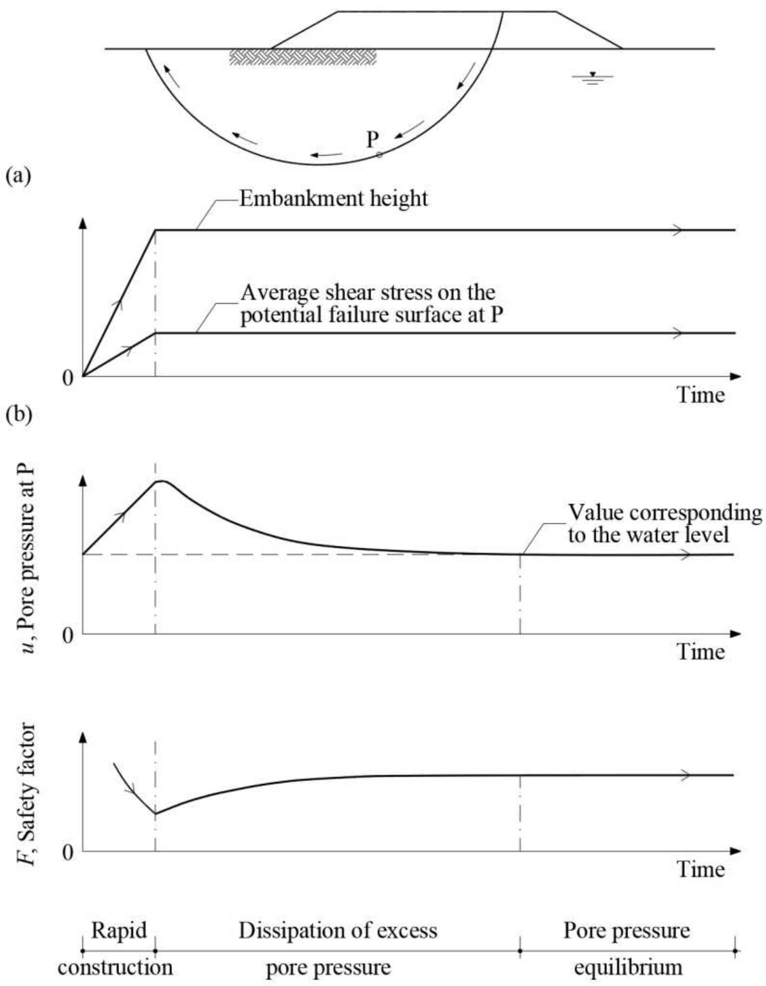

Figure 1. Embankment on soft clayey soil: a) layout and failure mechanism; b) change in time of the shear stress and pore water pressure at a generic point P and of the safety factor (adapted from Bishop & Bjerrum, 1960)

Figure 1 compares changes in quantities during the rapid construction of an embankment on a clay deposit and its post-construction phase, when the consolidation process (dissipation of excess pore water pressure) occurs. These quantities are: a) embankment height (i.e. load) and shear stress along a potential failure surface; b) pore water pressure and factor of safety for a generalized slip failure through the foundation soil and the embankment.

The main idea is that factor of safety varies with time: it increases with time after the completion of the surface loading. Consolidation, i.e., dissipation of excess pore pressure, results in an increase of effective stresses and then of the available soil resistance.

During construction, the load from the embankment is essentially applied under undrained conditions (rapid construction). The increase of the total mean stress and of the shear stress will induce positive excess pore pressure in the depicted soft soil, presumably a normally consolidated (NC) or lightly over-consolidated (OC) clay. This excess pore water pressure reaches a maximum at the end of construction, when the embankment reaches its maximum height.

Then, a two-dimensional (2D) consolidation process is triggered. As water starts gradually flowing out of the loaded soil, the pore pressure decreases, the effective stress increases and so do the shear strength and the factor of safety, which reaches its maximum value in the long-term. Hence, the critical situation for the evaluation of the minimum safety factor for this loading situation (that is, the situation to be considered in the stability analyses) is the end of construction. The conventional procedure to perform these analyses is the limit equilibrium method of slices, assuming undrained conditions and working in total stresses.

2.2 Cut excavation in a clayey soil mass

Figure 2 contains the classical schematic illustrating the relevant phenomena involved in the excavation of a sloped cut in a clay mass (Bishop & Bjerrum, 1960).

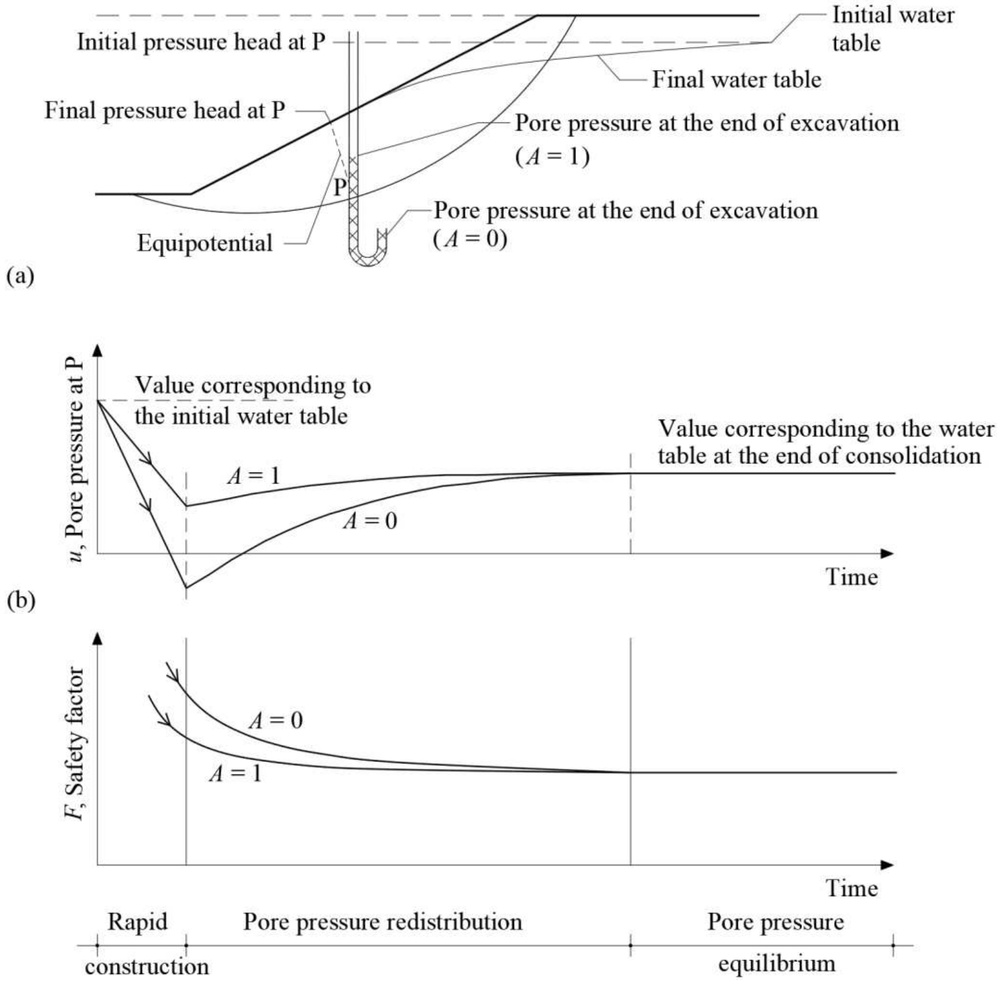

Figure 2. Sloped cut in clay: a) initial and final conditions of the water table and pore pressure at a generic point P at the end of the excavation; b) change in time of the pore water pressure at P and of the safety factor (adapted from Bishop & Bjerrum, 1960)

Figure 2 compares changes in quantities due to the excavation in clay deposit (a NC soil with A=1 or an OC soil with A=0, A being Skempton's pore pressure parameter) to form a cut slope, and the post-excavation phase when dissipation of excess pore water pressure takes place. These quantities, shown in Figure 2b, are the pore water pressure and factor of safety for a generalized slip failure of the excavated slope.

The main idea is that the factor of safety decreases progressively not only during but also after the completion of the excavation. During excavation, the loading is essentially applied under undrained conditions (rapid construction). The excess pore pressure within the soil mass close to the cut is the combined result of the drop in the mean total stress and the increase of the shear stress. The former induces negative excess pore pressure; the latter induces positive excess pore pressure in a NC clay and small positive or even negative excess pore pressure in a heavily OC clay. This explains why during the excavation the drop in the pore pressure is larger in the OC clay.

Then, the process of dissipation of the negative excess pore pressure starts. This is in fact a consolidation process, though involving changes in time of the variables opposite to those of the embankment construction.

Since pore water pressures in the vicinity of the cut are smaller than the ones in the mass far from the cut, the water starts gradually flowing towards the cut, the pore pressure increases, the effective stress decreases and so do the shear strength and the factor of safety, which reaches its minimum value in the long-term. Contrarily to the embankment case, the long-term pore water conditions are no longer hydrostatic, because the cut implies a change on the hydraulic boundary conditions. In the long-term, a steady-state flow of water towards the cut will be established.

Hence, the critical situation for the evaluation of the minimum safety factor for this loading situation (that is, the stability analyses) is the long-term, assuming drained conditions and working in effective stresses. The equilibrium pore pressures to be considered in the analyses are obtained from the corresponding steady-state flow net.

Table 1 contains a comparison between the two problems in terms of the change in stresses, induced strains, hydraulic conditions, ground surface movement, safety factors, stability analyses and other relevant issues. According to the experience of the first author, completing cell by cell the second and third columns of the table in the class is very stimulating and effective for the comprehension of this subject by the students.

Table 1. Comparison of features associated with embankments and cuts in clay (adapted from Matos Fernandes, 2020)

Item	Embankments	Cuts
Total mean stress increment	Increases	Decreases
Shear stress (under the embankment or behind the cut face)	Increases	Increases
Excess pore pressure, Δu	Positive	Negative
Soil type in which $\|\Delta u\|$ is maximum	Normally consolidated	Heavily over-consolidated
Change of u with time (after the end of construction/excavation)	Decreases	Increases
Flow during consolidation	Directed outwards	Directed towards the excavation
Change of water content and void ratio with time	Decrease	Increase
Change of volumetric deformation with time	Positive (compression)	Negative (expansion)
Ground surface displacement	Settlement	Heave
Change with time of the mean effective stress and of the shear strength (after the end of construction/excavation)	Increase	Decrease
Change with time of the safety factor	Increases (post construction)	Decreases
Phase in which the stability analyses should be performed	End of construction	End of consolidation
Conditions for the stability analyses	Undrained	Drained
Approach in terms of stress for the stability analyses by the method of slices	Total stress	Effective stress
Strength parameters involved in the analyses	c_u, considering its change with depth	Effective strength parameters, c' and ϕ'
Other data items required for the analyses	Unit weight	Steady-state flow net; unit weight

3 Numerical simulation of the construction of an embankment on soft clay

3.1 Brief description of the models employed

In the present paper, two computer codes are used: (i) a finite element (f.e.) program capable of performing soil consolidation coupled analysis (Borges, 1995); (ii) a stability analysis program that uses the f.e. results.

The f.e. program uses the following theoretical hypotheses: a) plane strain conditions; b) coupled formulation of the flow and equilibrium equations, with soil constitutive relations formulated in effective stresses (Biot consolidation theory), applied both during the construction and the post-construction period; c) utilisation of the *p-q-*θ critical state model, which is an extension of the Modified Cam-Clay model into the 3D stress space using the Mohr-Coulomb failure criterion, to simulate the soil behaviour (Lewis & Schrefler, 1987; Britto & Gunn, 1987; Borges,1995; Borges & Cardoso, 2001).

The stability is assessed, at any stage of the simulation, by the second computer program mentioned above, in which potential slip surfaces are generated and the respective overall safety factor is calculated, using the stress values provided by the f.e. analysis. Firstly, for each potential slip surface, the intersection points of the circle with the edges of the finite elements are determined. Then, the slip surface is divided into small line segments, each of them located inside of only one of the finite elements of the mesh. Afterwards, the average values of effective normal and shear stresses at each of those segments are computed, on the basis of the stress values in the Gauss points in the vicinity.

Thus, considering the slip circle divided into line segments, the overall safety factor, *F*, is computed through the equation:

$$F = \frac{\sum_{i=1}^{N} \tau_{fi} l_i}{\sum_{i=1}^{N} \tau_i l_i} \tag{1}$$

where τ_i is the acting shear stress at *i*-segment, τ_{fi} is the soil shear strength at *i*-segment, l_i is the *i*-segment length and *N* is the number of elements intersected by the slip surface. The soil shear strength, τ_{fi} is calculated by using the failure envelope equation, according to the *p-q-*θ critical state model. A large number of potential slip surfaces is analysed by the program, so that the most unfavourable slip circle is obtained, which corresponds to the smallest value of *F* calculated.

Note that, according to this procedure, the stability analysis is performed using effective stresses. This is possible because the stability computations are fed by the results of the f.e. simulations and, since mechanical-hydraulic coupled analyses are performed, effective stresses are obtained. Similar procedures were used by Borges & Cardoso (2001, 2002) to analyse the stability of geosynthetic-reinforced embankments on soft soils and by Borges (2024a, 2024b) to study the behaviour of embankments on soft soils reinforced with stone columns.

3.2 Presentation of the problem

The problem concerns a 2.0 m high symmetric embankment, with a very large longitudinal length, built on a 25 m thick saturated clay deposit lying on a rigid and impermeable stratum (Figure 3). The clay is moderately over-consolidated to the depth of 4 m (superficial OC crust) and normally consolidated from 4 to 25 m. The width of the embankment crest is 13.6 m. The water level is assumed at the ground surface.

Drained analysis is considered in the embankment material while a fully mechanical-hydraulic coupled analysis is adopted for the clay. The construction of the embankment is modelled by adding layers of elements at a uniform rate and is completed in 7 days.

The constitutive behaviour of the soils (clay and embankment fill material) is simulated by the *p-q-*θ critical state model whose parameters are shown in Table 2 (λ, slope of normal consolidation line and critical state line; *k*, slope of swelling and recompression line; *N*, specific volume of normally consolidated soil at mean normal effective stress equal to 1 kPa; Γ, specific volume of soil on the critical state line at mean normal effective stress equal to 1 kPa). Other geotechnical parameters are also indicated in Table 2: γ, unit weight; ϕ', angle of friction defined in effective stresses; ν', Poisson's ratio for drained loading; k_h and k_v, coefficients of permeability in horizontal and vertical directions. The undrained shear strength (c_u) of the clay, also presented in Table 3, is obtained from the other parameters of the model, applying equations of the Critical State Soil Mechanics, as explained in detail by Britto & Gunn (1987) and Borges (2024a, b).

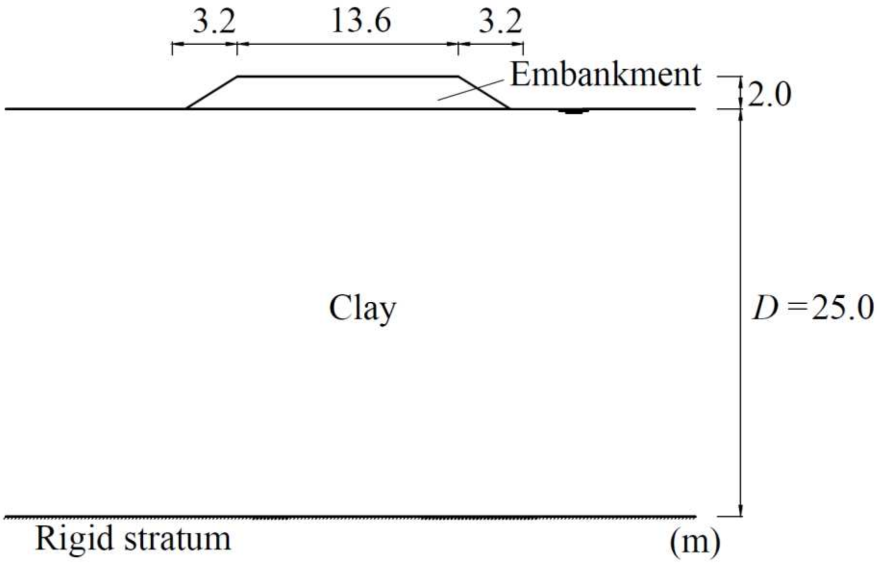

Figure 3. Cross section of the embankment on soft soil problem

The parameters of the clay are similar to those considered by Finno et al. (1991) of a soft ground in Chicago, USA. The values adopted for the embankment fill material are the same as those used by Borges & Cardoso (2001).

The assumed over-consolidation to the depth of 4 m intends to simulate a superficial crust, which is typical of alluvial clayey deposits (Lutenegger, 1995). This crust is formed by the seasonal cycles of the groundwater level, which is very close to the surface but not always at the same position. In the soil horizon above the water table, the capillarity develops negative pore pressures (suction). Since the soil is very fine, these negative pore pressures are high (in absolute values), which implies high interparticle forces. So, the variations of the groundwater level are equivalent, in terms of stresses, to loading-unloading cycles. This creates a thin OC superficial layer which is often called desiccated crust. The engineering relevance of this crust is paramount, since it makes feasible the construction of embankments and other structures founded at the surface of the deposit. According to Bjerrum (1973), the thickness of this crust may range from as little as 1 to 3 m to as much as 6 to 8 m, depending on the site conditions.

Table 2. Geotechnical parameters of the clay and of the embankment fill

	γ (kN/m^3)	ν' (-)	ϕ' (º)	c' (kPa)	k_h (m/s)	k_v (m/s)	p-q-θ critical state model			
							λ	k	N	Γ
Clay	16	0.25	26	0	10^{-9}	10^{-9}	0.18	0.025	3.158	3.050
Embankment	20	0.30	35	0	-	-	0.03	0.005	1.817	1.800

Table 3. Undrained shear strength (c_u) of the clay

Depth, z (m)	c_u (kPa)
0	13.0
$0 \le z \le 4$	13.0 - 6.53 (*)
4	6.53
$4 \le z$	$1.633z$

*Linear variation

3.3 Discussion of the results

A finite element analysis provides a large amount of results concerning displacements, stresses, pore pressures and other quantities. Just a small portion of that information is presented herein.

Figure 4 illustrates the surface vertical displacements at the completion of the embankment construction and at the end of consolidation. Since the construction is completed in a very short period, the volumetric strains of the foundation soil are practically null. Then, at the end of construction the settlement under the embankment must be compensated by the upward vertical displacements in the vicinity. The ensuing consolidation process produces positive volumetric (compression) strains, inducing surface settlements.

For the central point of the base of the embankment the consolidation settlement is about twice the immediate settlement. The change in time of the settlement of this point is depicted in Figure 5. The progressive reduction of the settlement rate during the consolidation reveals a pattern similar to that of the classical theory of consolidation of Terzaghi.

In theoretical terms, the consolidation process in never concluded. In practice, it can be seen that, say, beyond 600 months (50 years) the change in the surface settlements is negligible. This very large time period is a consequence, beyond the very low permeability of the clay soil, of the large thickness of the layer (25 m) and of the assumption of an impermeable lower boundary.

Figure 6 presents the horizontal displacements under the embankment toe (x =10 m), at the end of construction and at the end of consolidation. It can be seen that the incremental displacements during consolidation are quite small, in comparison with the vertical displacements that occur in the same time period. That is, during consolidation the foundation soil is loaded in conditions that are close to a confined loading, similar to the conditions prevailing in an oedometer test. This fact is the base of the well-known simplified method of Skempton & Bjerrum (1957) for the calculation, in similar cases, of the consolidation settlement using the compressibility parameters provided by the oedometer test.

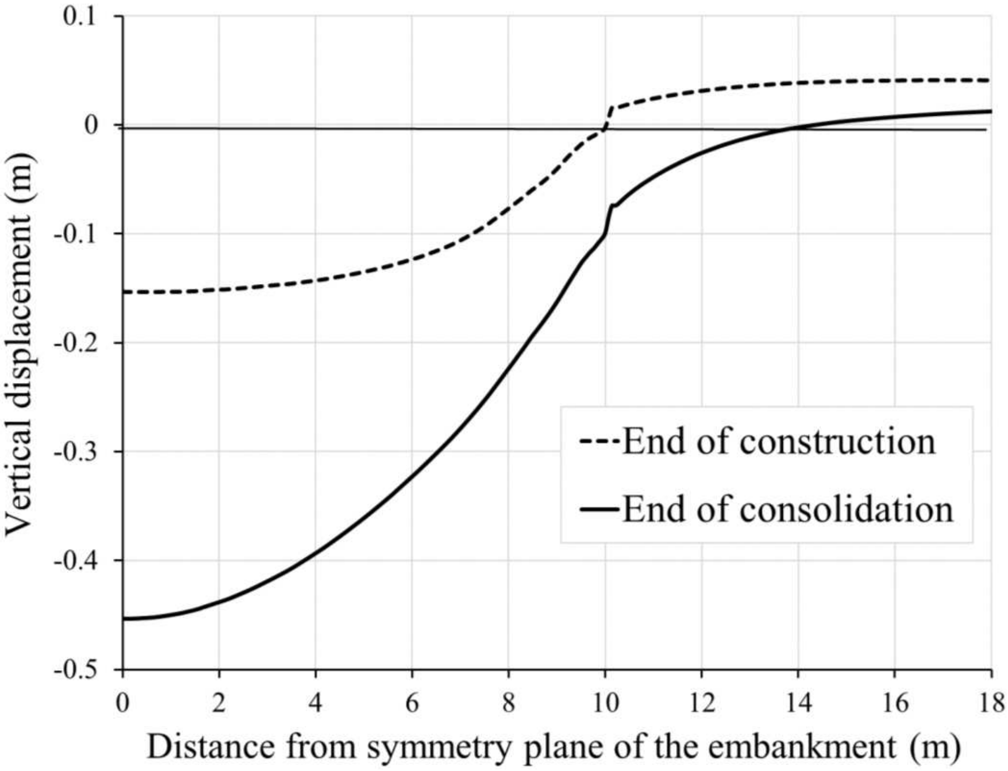

Figure 4. Vertical displacements of ground surface at the end of construction (7th day) and at the end of consolidation

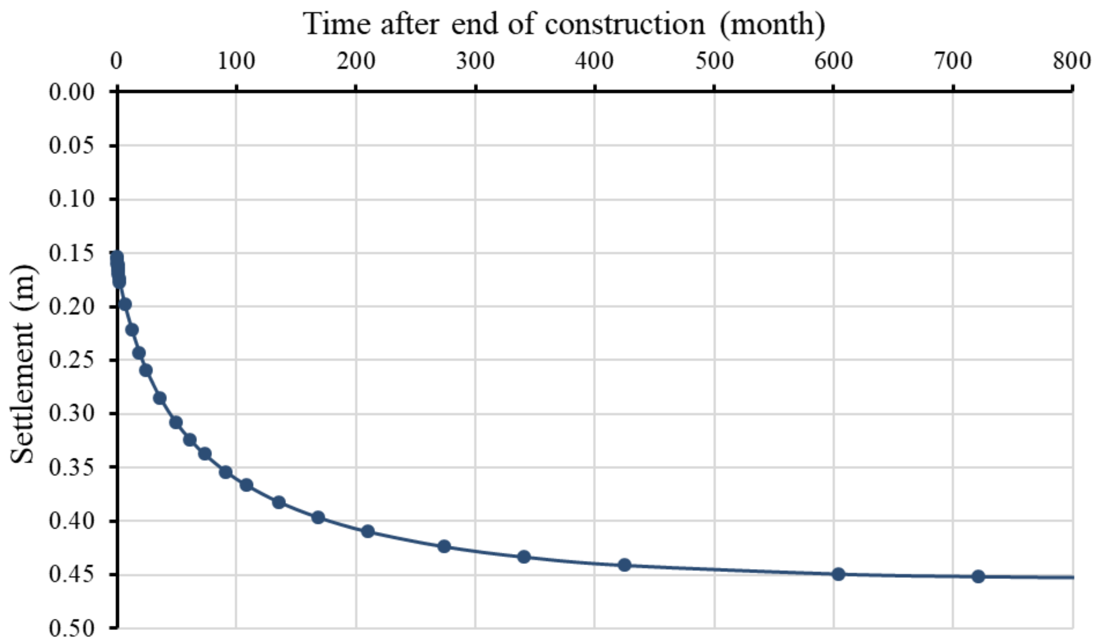

Figure 5. Settlement versus time at midpoint of the embankment's base

A more detailed observation of Figure 6 reveals that during consolidation the horizontal displacements diminish in the OC superficial crust and increase in the lower NC soil mass. The superficial OC crust is (elastically) reloaded, and experiences volumetric positive strains (compression), which explain the volume reduction, and therefore inward horizontal displacements. On the other hand, the lower NC mass is loaded in virgin compression, exhibiting an elastoplastic behaviour. In addition to the volumetric strains, which induce inward horizontal displacements, some plastic deviatoric strains arise, provoking outward horizontal displacements. When this effect is stronger than the former, like in this case, the resulting horizontal displacements increase during the consolidation process.

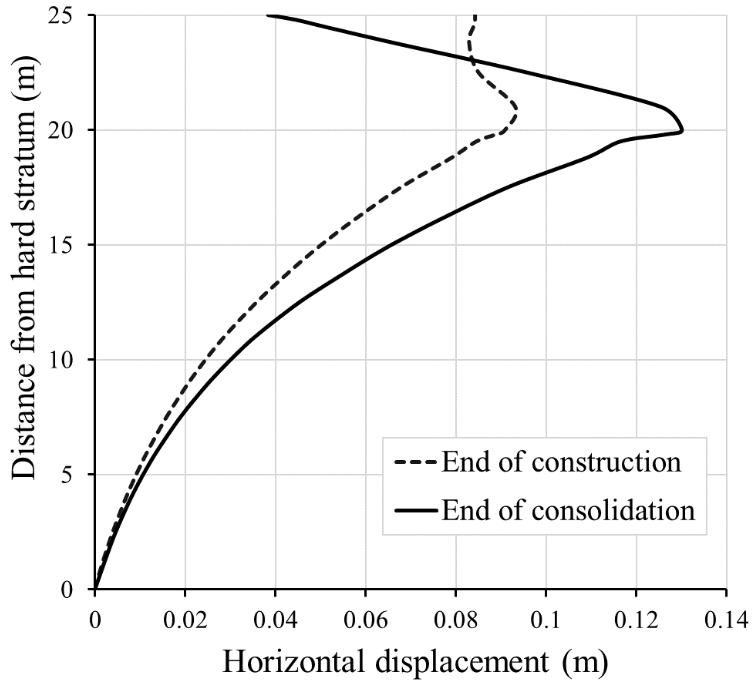

Figure 6. Horizontal displacements under the embankment toe (x =10 m), at the end of construction (7th day) and at the end of consolidation

The distribution of the excess pore pressure in the foundation soil at the end of the construction and for three stages of the consolidation process, can be observed in Figure 7. The distribution for the end of construction reveals a more complex pattern, resulting from the influence of the over-consolidated crust. This effect vanishes during consolidation. As might be expected, the excess pore pressure reduces with the horizontal and vertical distances from the central point of the embankment base. Besides, since the clay is assumed isotropic in terms of permeability, the isolines of Δu are perpendicular to the impermeable lower boundary of the clay deposit. It should be noted that the isolines of Δu coincide with the isolines of total head (equipotential lines), since the initial value of total head (before construction) is constant in the ground, i.e. it has the same value at all points of the saturated clay (hydrostatic condition).

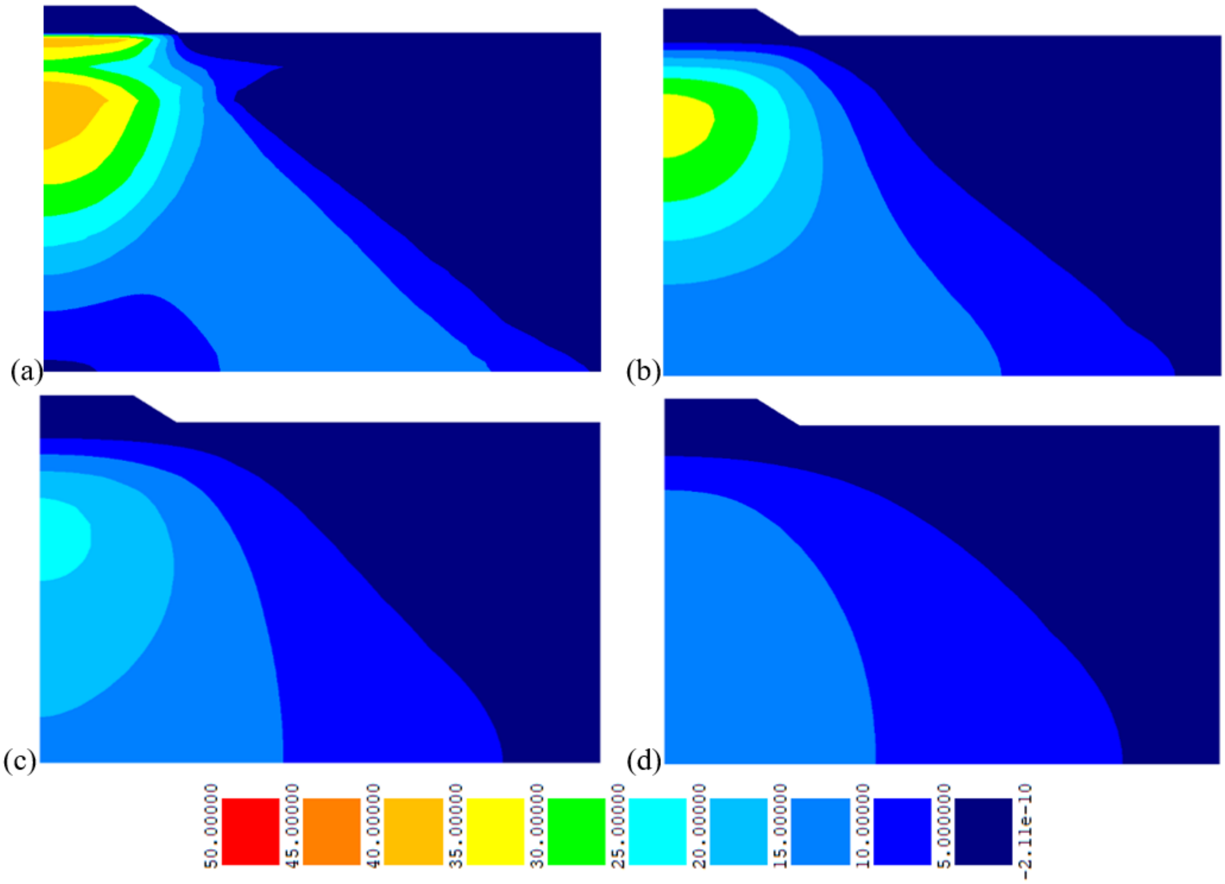

Figure 7. Distribution of excess pore pressure (Δu) in the clay deposit: a) end of construction (7th day) (Δu_{max} = 39.7 kPa); b) 6 months after end of construction (Δu_{max} = 32.1 kPa); c) 2 years after end of construction (Δu_{max} = 21.1 kPa); d) 5 years after end of construction (Δu_{max} = 13.6 kPa)

On the basis of the distribution of the pore pressure and of the total and effective stresses for each time after the construction, and by applying the procedure described in Section 3.1 above, the safety factor against a general failure was computed. The results, presented in Figure 8, confirm that the critical situation in terms of stability is the end of construction and that a substantial increase of the safety factor occurs during consolidation.

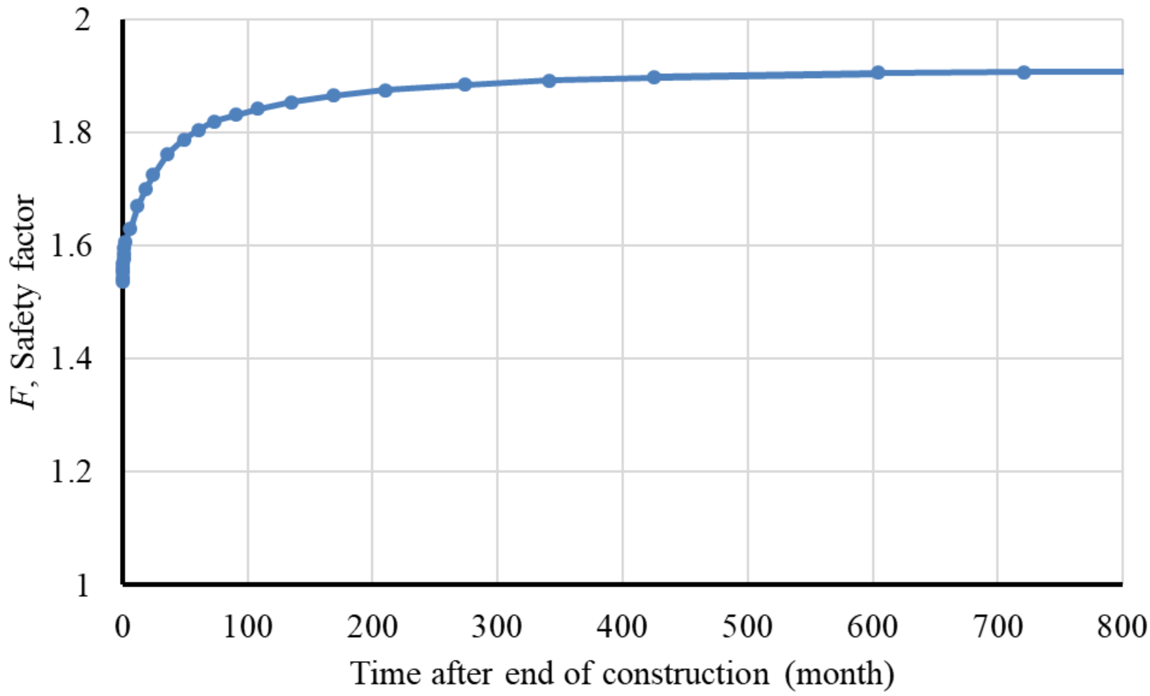

Figure 8. Safety factor (*F*) versus time after the end of construction

Finally, the critical slip surfaces (minimum safety factor) for the end of construction and for the end of consolidation are shown in Figure 9. It can be seen that these slip surfaces are not coincident, with the one for the end of consolidation encompassing a larger volume of the foundation soil further away from the embankment. This is reasonable because consolidation improves more the resistance in the soil mass underneath the loaded area compared to the soil further away.

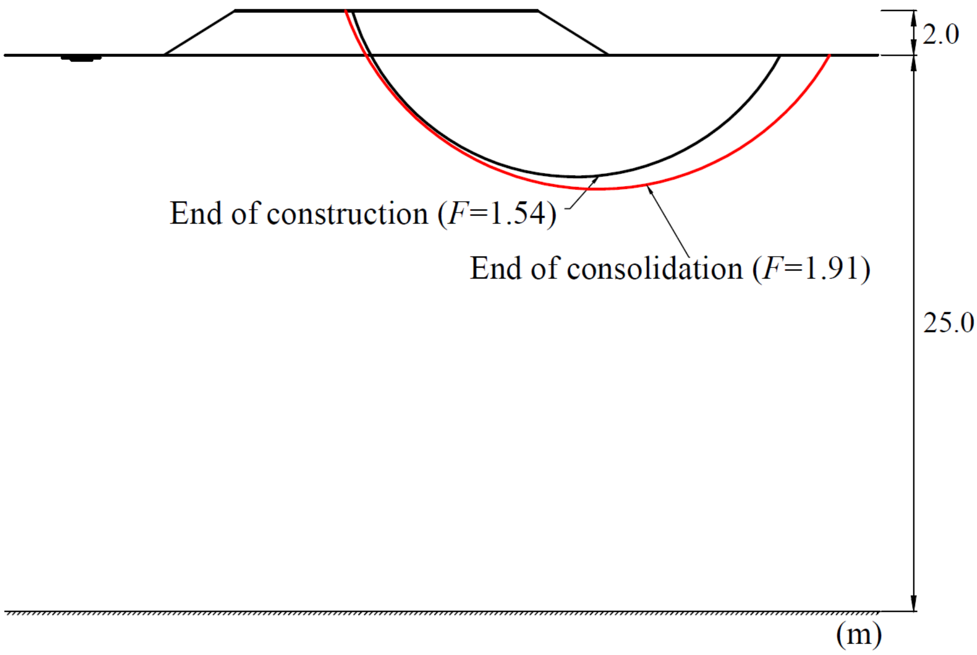

Figure 9. Critical slip surfaces and respective overall safety factor values (*F*) at the end of construction and at the end of consolidation

4 Concluding remarks

The paper revisits two classical figures (Bishop & Bjerrum, 1960), describing two common and relevant geotechnical works: the construction of embankments on soft clay (loading) and the excavation of unsupported sloped cuts in clays (unloading). These conceptual figures are elaborated upon with in-depth commentary and a table summarizing and contrasting trends for the mechanical and hydraulic phenomena triggered by the construction of these two works, thus touching on most key issues of soil behaviour. The loading case is supplemented with numerical simulation results (immediate and consolidation surface settlements and horizontal displacements in depth, excess pore pressures and their change in time, and safety factor against a general failure and its change in time) that confirm the conceptual stability trends at a point and offer insights into the behaviour of the entire soil mass.

Since the discussion is not anymore restricted to shear strength parameters in laboratory-controlled conditions (fully drained or fully undrained), but extends to the resistance and stability of a large soil mass over time, it can be said that this subject is particularly suitable not only as an epilogue of a basic course on soil mechanics (Soil Mechanics or Soil Mechanics 1), but also as an inaugural class of a course on soil engineering (Soil Mechanics 2 or Geotechnical Engineering).

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