Critical analysis of the European Standard EN 1536 concerning the stability of a pile borehole

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ABSTRACT

The European Standard EN 1536 aims to improve the co-operation and harmonization between design and execution, thus stablishing general principles for the correct execution of bored piles. In this dissertation, the principles related to unsupported excavation are analyzed with detail. According to EN 1536 this type of excavation should only be carried out in ground conditions where the collapse of ground material into the bore is not likely. However, the standard is not specific, becoming relevant to evaluate the stability of a borehole.

Therefore numerical analyses are developed for better understanding and analysis of this stability problem. In these analysis only undrained soils are considered. In a first approach studies are carried out on radial stability, in plane stress, with the plane perpendicular to the hole axis, verifying that no collapse mechanism is formed. However is set a criterion that limits the radial displacements to a fraction of the hole diameter. Then, a stability analysis of the bottom of the excavation is carried out, in axisymmetric state. It is then verified that at a certain depth a slip surface is formed, causing collapse. For this study it’s also verified that the presence of water inside the hole, has a significant stabilizer effect, allowing in some cases to double the critical excavation depth. Finally, the respective equations corresponding to the criterion of the radial displacements and the criterion of stability of the bottom of the excavation are presented, which after comparing them turns out that the first criterion is more conditioning.

1. INTRODUCTION

Given the lack of specificity of the principles recommended by EN 1536 “Execution of special geotechnical work – bored piles” it is pertinent to do a critical reflection about them. Since the subject matter covered by the standard is by itself too broad, this study will focus more on the excavation process of the pile boreholes. Concentrating more on unsupported boreholes, where the standard only states that they should be performed on soils where the collapse of the ground material into the bore is not likely. Therefore, in this work, numerical analyses will be developed for a better understanding and critical reflection of this problem.

2. EUROPEAN STANDARD EN 1536

Since Eurocode 7, essentially refers to design rules, there is a need to consult other normative references that help to improve the cooperation and harmonization between all parties involved and to assure their correct application for the safety and the last quality of the foundations. In 1999 the

European standard EN 1536 “Execution of special geotechnical work – Bored piles” was published, establishing general principles for the execution of bored piles.

Regarding the execution phase, the standard covers general principles about construction tolerances, excavation process, reinforcement, concreting and trimming. The principles about the excavation process are generally preventive measures to the formation of pile defects and anomalies that occur during this phase. The standard covers the following types of excavation: excavation supported by casings, excavation supported by fluids, excavation with continuous flight augers and unsupported excavations. This work will focus on unsupported excavations, which according to EN 1536, are only permissible in ground conditions which remain stable during excavation and where a collapse of ground material into the bore is not likely, giving no more detailed information. Besides, the standard also states that the stability of unsupported excavation shall be demonstrated by means of trial bored piles or by comparable experience before the commencement of the works.

However, sometimes is not possible to do trial bored piles, either because of execution deadlines or financial reasons. Therefore, it becomes pertinent to develop a detailed analysis about the stability of a pile borehole, with the purpose of studying its behavior in the collapse state, thus complementing EN 1536.

3. STABILITY OF A PILE BOREHOLE

In this stability analysis, only soils responding in undrained conditions will be studied, considering that the eventual rupture occurs before any dissipation of interstitial pressure. In a first approach to this stability problem, studies on the radial stability will be carried out in plane-stress state, with the plane perpendicular to the hole axis. This first analysis intends to study the collapse due to the reduction of radial stress caused by the excavation process itself. Then a stability analysis of the bottom of the excavation is carried out, through a numerical analysis in axisymmetric state. This second analysis intends to study the stability of the bottom of the hole as the excavation process proceed. In other words to study if the reduction of the vertical and radial stress in the bottom of the hole leads to a tridimensional collapse mechanism.

3.1 Radial stability analysis

It is considered the analytical solution for rectangular plates, in elastoplastic behavior, with a circular hole, presented in Rocha (1976). This analytical solution defines a stress state in the plane of the plate, around the opening. According to this analytical solution, excluding the hypothesis that the undrained shear strength, c_u, is zero, the borehole walls are always self-supporting regardless of the magnitude of the initial lateral stress, p_i. Since there will always be a finite plastic zone corresponding to the radial stresses imposed by the soil. In other words, as the initial stress at the boundary increases, the transition surface increases to a given finite value, never being formed a collapse mechanism.

3.1.1 Numerical analysis of radial stability

In this analysis, FLAC - *Fast Lagrangian Analysis Continua* software is used to verify the elastoplastic analytical solution presented in Rocha (1976) as well as to compare its conclusions about the radial stability of a borehole. This verification is carried out in plane-stress with the plane perpendicular to the axis of the hole, which is assumed that have the radius of 1.0 meter. Due to the symmetry of the problem, only a quarter of it needs to be analyzed. The model to analyzed in the FLAC software is shown in figure 3.1. The used finite difference mesh is shown in figure 3.2.

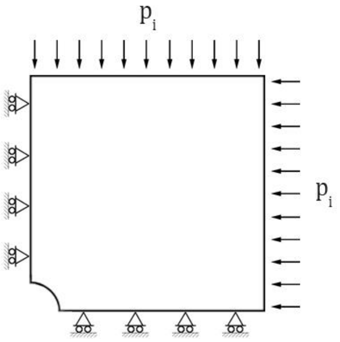

Figure 3.1 *- FLAC model for radial stability analysis of a borehole.*

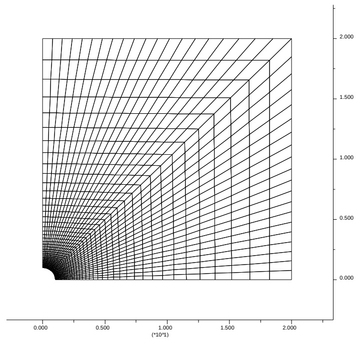

Figure 3.2 *- FLAC finite difference mesh of 20m x 20m with 1600 quadrilateral elements.*

According to Rocha (1976), when the pressure inside the hole is zero, the plastic zone begins to arise for $\frac{c_u}{p_i} = 1{,}0$ and increases as the value decreases. In this verification, it will be analyzed the distribution of the maximum and minimum principal stresses for the values of $\frac{c_u}{p_i}$ of 1.0; 0.4; 0.3 and 0.25, intending to check the distribution of the stress state as the plastic zone increases. In this way, the initial lateral pressure, p_i, remains fixed as the undrained shear strength varies. Table 3.1 shows the properties of the soil used in this numerical analysis.

Table 3.1 *- Soil properties for the numerical analysis of radial stability.*

Properties of the soil	E (MPa)	ν	γ (kN/m³)	c_u (kPa)	p_i (kPa)
	10	0,45	20	25 - 100	100

In figure 3.3 is presented the comparison between the results obtained through numerical analysis and the analytical solution for the distribution of radial and circumferential stresses along a radial line.

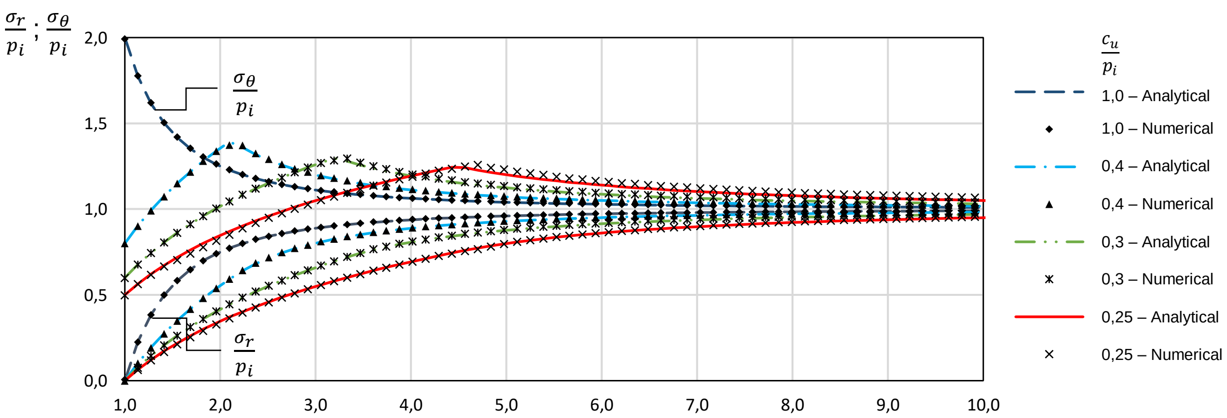

Figure 3.3 – Comparison between the stress distribution obtained through the numerical analysis and the analytical solution presented by Rocha (1976) along a radial line.

Since the values of the relative errors are quite low (less than 1.00%), it is possible to conclude that the numerical analysis is quite faithful to the analytical solution presented by Rocha (1976). The major difference between these two solutions lies in the maximum circumferential tension value, which is perfectly expectable since there are no nodal points at the exact position of the transition surface between the plastic and the elastic zone.

Concerning the radial stability, in figure 3.4 it is possible to observe the history of the radial displacements at a point located on the wall of the borehole with 1 meter of radius, for the various values of $\frac{c_u}{p_i}$.

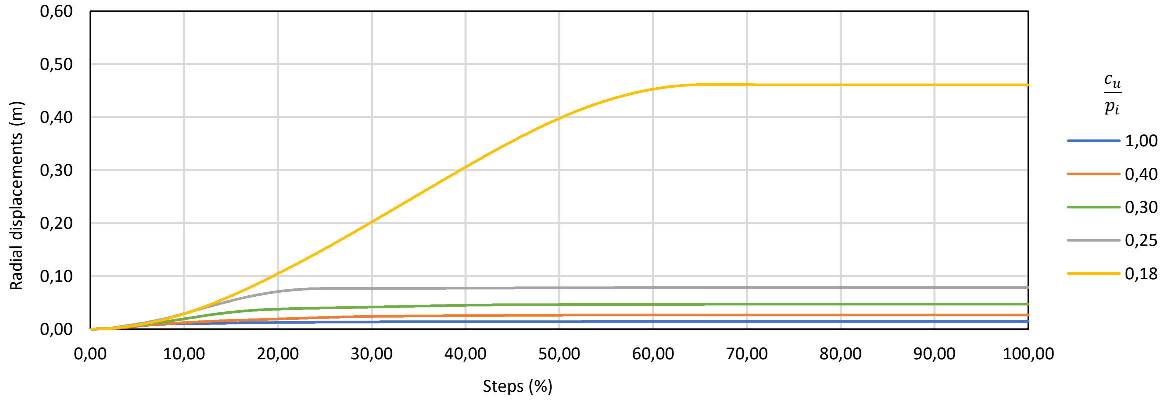

Figure 3.4 *– Radial displacement history in a point located at the borehole wall for the various parameters of* $\frac{c_u}{p_i}$.

Figures 3.5, 3.6 and 3.7 show the plastic points for some of the parameters $\frac{c_u}{p_i}$ studied above.

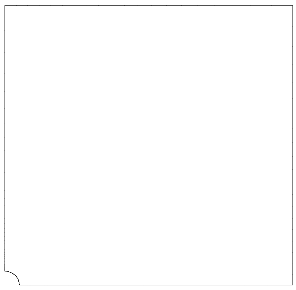

Figure 3.5 *- Plastic points* $\frac{c_u}{p_i} = 1{,}0$

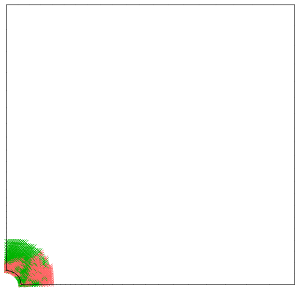

Figure 3.6 *- Plastic points* $\frac{c_u}{p_i} = 0{,}3$

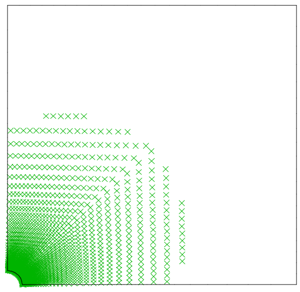

Figure 3.7 *- Plastic points* $\frac{c_u}{p_i} = 0{,}18$

As can be seen in figure 3.4, the history of the radial displacements tends towards a given finite value, thus verifying that an equilibrium state is reached, even for values as low as $\frac{c_u}{p_i} = 0{,}18$. The figures 3.5, 3.6 and 3.7 shows that for each equilibrium state a plastic zone is formed, which becomes larger with decreasing values of $\frac{c_u}{p_i}$. Following this analysis it was verified that there is no radial collapse of the borehole wall, as the analytical solution had already concluded. When the undrained shear strength (c_u) decreases in relation to the initial lateral stress (p_i), the plastic zone consequently increases, and no collapse mechanism is formed.

3.1.2 Radial displacements criterion

Although there is no radial collapse of the borehole wall, excessive radial displacements can compromise the proper execution of bored piles, leading to the reduction of the excavation section and making it impossible to execute. Thus, it is necessary to define a criterion that indicates the values of radial displacements admissible to the good execution of a bored pile.

For this case of study, the assumed displacement criterion is , $\frac{\Delta d}{d} \leq 0{,}05$, where d is the hole diameter. In other words, it is assumed that a volume variation of 10% of the total volume of the hole is allowed. Therefore, since the radius of the bore is 1 meter, the permissible radial displacements are 0,05 m or less. From figure 4, it can be seen that for $\frac{c_u}{p_i} = 0{,}30$, the radial displacements obtained by the numerical analysis are 0.047 m, approximately 0.05 m. It is thus defined, according to the above criterion, that it is only possible to ensure a good execution of bored piles for values of $\frac{c_u}{p_i}$ equal or greater than 0,3.

3.2 Stability analysis of the bottom of the excavation

Since no analytical solutions to this problem were found in the literature, this stability analysis will go straightly to a numerical analysis in axisymmetric state. The analysis of this problem can be put as the determination of the critical excavation depth, h_{crit}, in which the collapse occurs by the bottom of excavation. The excavation process will be simulated through a staged excavation of 2 by 2 meters, while the total displacements are monitored at a control point to check in every stage if an equilibrium or a collapse state is reached.

This study is carried out for two cases. The first one consists of the analysis of the critical excavation depth of a hole without any kind of support, and the second one intends to analyze the same hole but fully filled with water. Therefore, the model for the numerical analysis in axisymmetric state is presented in the figure 3.8 and the finite difference mesh is presented the figure 3.9.

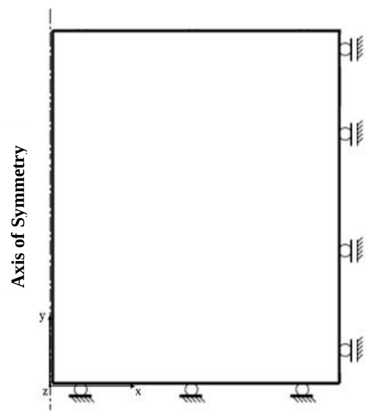

__Figure 3.8__ - FLAC model for the stability analysis of the bottom of the excavation.

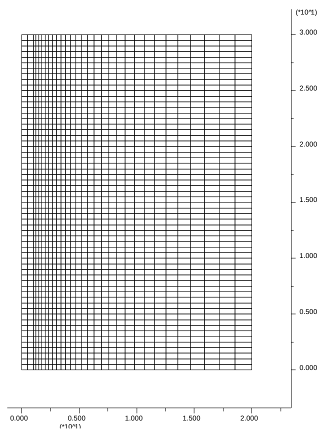

__Figure 3.9__ - FLAC finite difference mesh of 20m x 30m with 1800 quadrilateral elements.

This analysis will be performed according to the parameters of $\frac{c_u}{\gamma}$, where γ is the volumetric weight. Table 3.2 shows the properties of a soil analyzed for $\frac{c_u}{\gamma} = 1{,}0\ m$.

***Table 3.2** – Properties of the soil studied for the stability of the bottom of the excavation with* $\frac{c_u}{\gamma} = 1{,}0m$.

Soil Properties	E (MPa)	ν	γ (kN/m^3)	c_u (kPa)
	10	0,45	20	20

In order to carry out this analysis it is necessary to define in advance the initial stress state in the FLAC model. The vertical stresses σ_v, along the y axis, are defined by the linear variation $\sigma_v = \gamma h$, where h is the depth of each element in the model. The horizontal stresses σ_h, along the x and z axis, are defined by multiplying a factor K by σ_v. If this analysis were performed in effective stresses this factor K would be equal to the at-rest earth pressure coefficient, K_0, however this numerical analysis is performed for undrained conditions, and therefore done in total stresses. Once assumed that the water table is on the surface level, K can be represented by the following expression:

$$K = \frac{K_0(\gamma - \gamma_w) + \gamma_w}{\gamma} \quad (3 - 1)$$

Assuming that in this study $K_0 = 1$, through Eq. 3-1, $K = 1$. As such the horizontal stresses σ_h, are defined by the linear variation of $\sigma_h = 1.\gamma.h$.

Since the numerical analysis of this problem is carried out without any reference to analytical solutions, it is pertinent to verify beforehand the reliability of the adopted finite difference mesh. The mesh verification was carried out through the verification of the classic analytical solution referring to the ultimate bearing capacity of the base of a bored pile:

$$q_b = 9c_u + \gamma D \quad (3 - 2)$$

Where: q_b is the ultimate bearing capacity, γ the volumetric weight and D the pile length.

When comparing the results obtained through the numerical analysis with those obtained by the analytical solution, a relative error of only 0.22% was found. It can be then assumed that the mesh is also capable of producing very assertive results when analyzing the critical excavation depth of a pile borehole.

3.2.1 Numerical Analysis of the stability of the bottom of the hole

Initially the process of excavation of a borehole with 1 meter of radius is simulated for the value of $\frac{c_u}{\gamma} = 1{,}0\ m$. As previously stated this excavation process is simulated by means of a staged excavation of 2 by 2 meters, while the total displacements are being monitored at a control point on the ground surface near the borehole wall. Figure 3.10 shows the history of the total displacements along the simulation of the various phases of excavation.

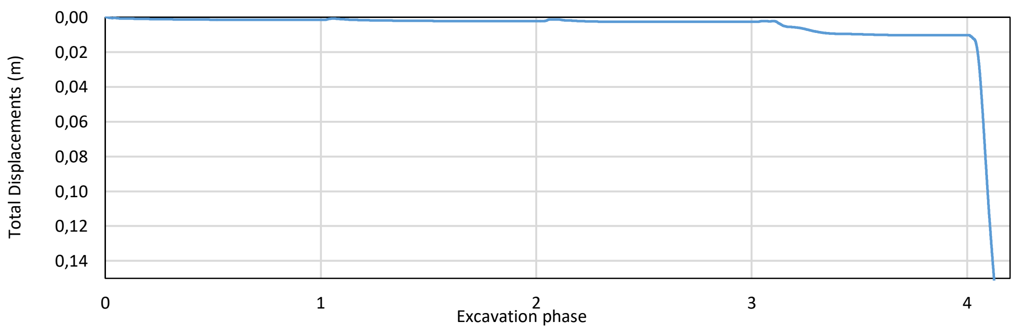

***Figure 3.10** – History of the total displacements obtained at a control point, located on the ground surface near the wall of the borehole, along the various excavation phases for* $\frac{c_u}{\gamma} = 1{,}0$

It is verified that in the fifth excavation phase the total displacements at the control point began to increase infinitely, verifying a collapse state. Thus, it is conclude that the critical excavation depth, h_{crit}, for $\frac{c_u}{\gamma} = 1{,}0\ m$, is between 8 and 10 meters. By optimizing the depth of excavation of the fifth phase, it is verified that the state of collapse occurs from an excavation depth of 9,5 meters, so $h_{crit} = 9{,}5\ m$.

Figures 3.11 and 3.12 show, respectively, the plastic points and the field of total displacements that occur at the imminence of the collapse.

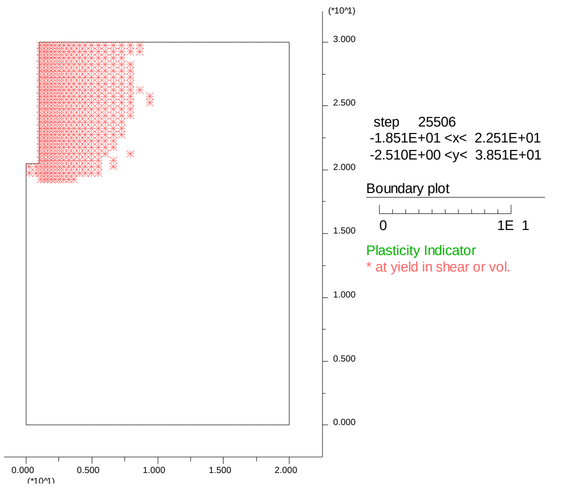

***Figure 3.11** – Plastic points at the imminence of the collapse ($h_{crit} = 9{,}5\ m$).*

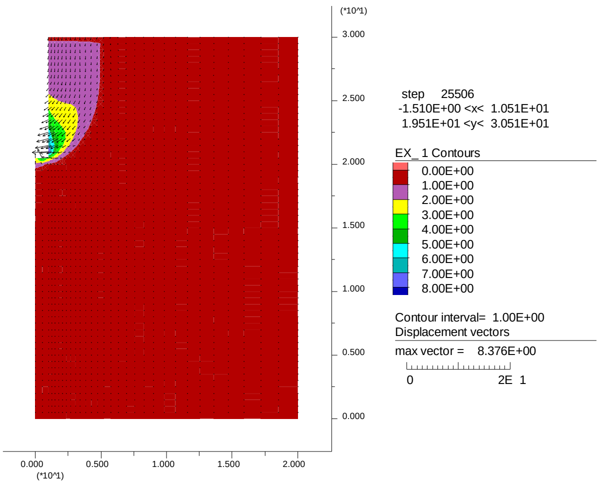

***Figure 3.12** - Total displacements field at the imminence of the collapse ($h_{crit} = 9{,}5\ m$).*

At the imminence of collapse it can be seen that a plastic zone is formed and takes a tridimensional collapse mechanism in the bottom of the excavation. In other words, at the same time that the bore walls are displaced radially into the bore the bottom of the excavation presents an upward movement. After verified the collapse and analyzed the critical excavation depth for the value of $\frac{c_u}{\gamma} = 1{,}0\ m$, it was analized h_{crit} for other values of $\frac{c_u}{\gamma}$. Figure 3.13 shows the critical excavation depths determined for the different values of $\frac{c_u}{\gamma}$.

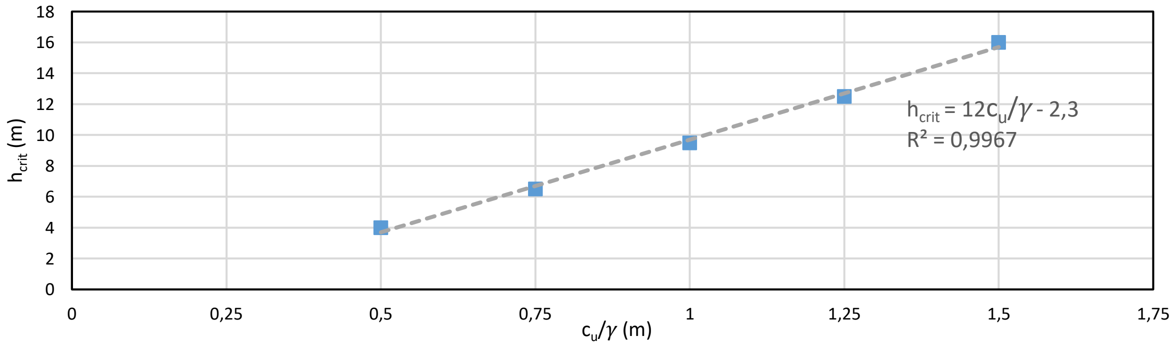

***Figure 3.13** – Critical excavation depth as function of $\frac{c_u}{\gamma}$.*

It is then possible to do a linear regression line, defining a very approximate expression of the critical excavation depth of a pile borehole as function of the parameter of $\frac{c_u}{\gamma}$

$$h_{crit} = 12\frac{c_u}{\gamma} - 2{,}3 \tag{3-3}$$

For values of $\frac{c_u}{\gamma}$ less than 0,5 m, the collapse mechanism differs from the previously observed mechanism, so Eq. 3-3 is only valid from $\frac{c_u}{\gamma} = 0{,}5$, since it is from this value that the collapse always occurs in the same way.

The values of the critical excavation depths obtained by Eq. 3-3 for soils with low undrained shear strength, $c_u \leq 20kPa$, should be taken in account with some reservation. There is other effects not covered in this numerical simulation (i.e. fissures, degradation of mechanical properties of the soil, etc.) that can lead to critical excavation depths quite low than the ones determined by de Eq. 3-3.

3.2.2 Numerical Analysis of the stability of the bottom of a hole full filled with water

Since the presence of water has a stabilizing effect it is necessary to use a more extensive mesh. In this study the mesh used is 50 x 60 meters with 4800 quadrilateral elements. For this case the excavation process is again simulated by means of a staged excavation of 2 by 2 meters. The presence of water is simulated at each excavation stage by introducing a hydrostatic pressure with $\gamma_w = 10\ kN/m^3$. As in the previous case, during the excavation process the total displacements are monitored at a control point on the ground surface near the hole.

For $\frac{c_u}{\gamma - \gamma_w} = 2{,}0\ m$, it was verified that in the twelfth excavation phase the total displacements at the control point began to increase infinitely, verifying a collapse state. Thus, for $\frac{c_u}{\gamma - \gamma_w} = 2{,}0\ m$, the critical excavation depth of a hole completely filled by water is between 22 and 24 meters. Once the excavation depth of the twelfth stage has been optimized, it is verified that the collapse state occurs from 23 meters of excavation, so $h_{crit} = 23\ m$. Figures 3.14 and 3.15 show, respectively, the plastic points and the total displacement field for $\frac{c_u}{\gamma - \gamma_w} = 2{,}0\ m$.

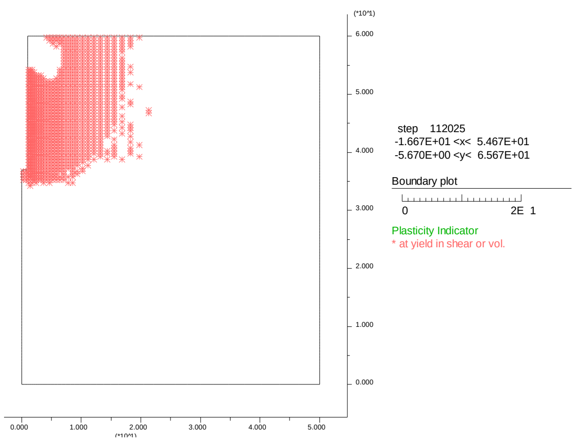

Figure 3.14 *– Plastic points at the imminence of the collapse for pile borehole ($h_{crit} = 23{,}0\ m$).*

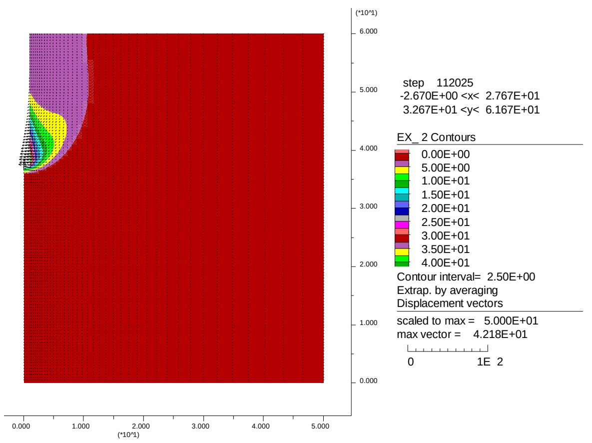

Figure 3.15 *- Total displacements field at the imminence of the collapse for a pile borehole ($h_{crit} = 23{,}0\ m$).*

Just like in the first case, at the imminence of the collapse, the plastic zone presents itself as a tridimensional collapse mechanism at the bottom of the excavation. Figure 3.16 shows the critical excavation depths obtained for other values of the parameter $\frac{c_u}{\gamma - \gamma_w}$.

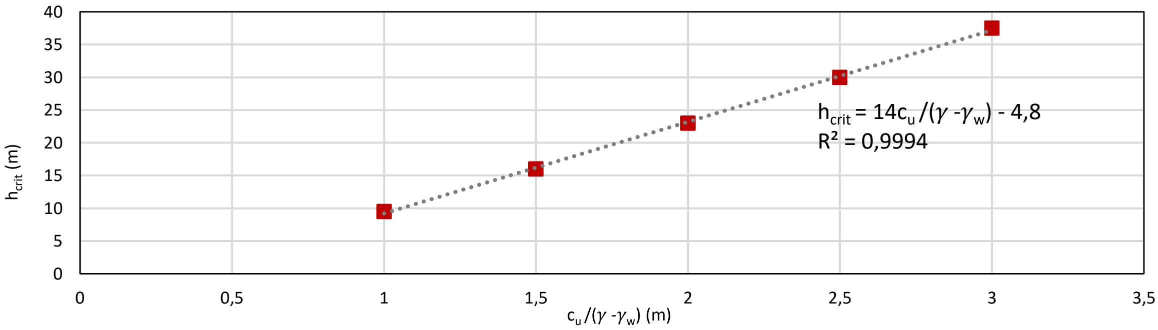

Figure 3.16 *– Critical excavation depth as function of $\frac{c_u}{\gamma-\gamma_w}$ for a borehole fully filled with water.*

By analyzing h_{crit} for other values of $\frac{c_u}{\gamma-\gamma_w}$ it is possible to do a linear regression line, obtaining a fairly approximate expression for the critical excavation depth of a borehole completely filled with water as function of the parameter $\frac{c_u}{\gamma-\gamma_w}$

$$h_{crit} = 14\frac{c_u}{\gamma-\gamma_w} - 4{,}8 \tag{3-4}$$

As in the first case of study, the critical excavation depth expression Eq. 3-4 does not pass through the origin. This is because, for values of $\frac{c_u}{\gamma-\gamma_w}$ less than 1,0 m, the collapse mechanism begins to be different from the previously observed. In this way, Eq. 3-4 is only valid for values of $\frac{c_u}{\gamma-\gamma_w}$ greater than 1.0 m.

For instance, according to Eq. 3-3, the critical excavation depth of a borehole in a soil with volumetric weight, γ, of 20 kN/m^3 and undrained shear strength, c_u, of 20 kPa, is $h_{crit} = 9{,}7\ m$. If this excavation is fully filled with water, the critical excavation depth according to the expression Eq. 3-4 is $h_{crit} = 23{,}2\ m$. It is thus concluded that the presence of water inside the hole, in this example, is quite influential, allowing an excavation depth of more than twice of the excavation without presence of water.

However, for practical applications, the critical excavation values determined for soils with low undrained shear strength, $c_u \leq 20kPa$, should be taken in account, once more, with some reservation. As the first case, in this numerical simulation it was not covered other effects, like degradation of mechanical properties due the presence of water, draining layers, etc. that can lead to critical excavation depths quite low than the ones determined by Eq. 3-4.

3.3 Comparison between both stability analyses

Although the collapse only occurs at the bottom of the excavation, radial displacements can compromise a good execution of the bored piles, leading to the formation of a smaller section. Therefore, it is necessary to analyze both criteria in order to understand which one determines the critical excavation depth. Since p_i represents the initial lateral stress it is possible to define it by multiplying the coefficient K by σ_v, so, $p_i = K.\gamma.h$. Since $K = 1$ is assumed, it is possible to define the criterion of radial displacements by the equation $h_{crit} = 3{,}33\frac{c_u}{\gamma}$. For the other case of study, where the borehole is completely filled with water, the water exerts a radial pressure inside the excavation, p. Since the radial stability analysis is performed with a hydrostatic initial stress state, the criterion of radial displacements for borehole fully filled with water can be represented by $h_{crit} = 3{,}33\frac{c_u}{\gamma-\gamma_w}$.

Figure 3.17 presents a comparison between the criterion of stability of the bottom of the excavation defined in Eq. 3-3 and the radial displacement criterion. Figure 3.18 presents the same criteria comparison but for the case of a borehole fully filled with water.

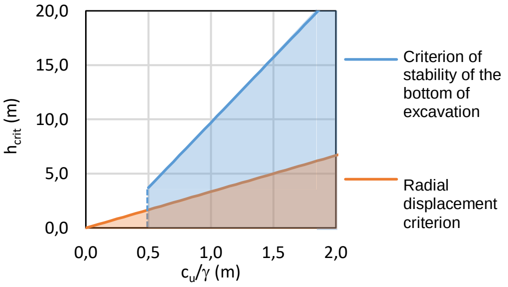

Figure 3.17 *- Comparison between the Radial displacement criterion and the Criterion of stability of the bottom of excavation.*

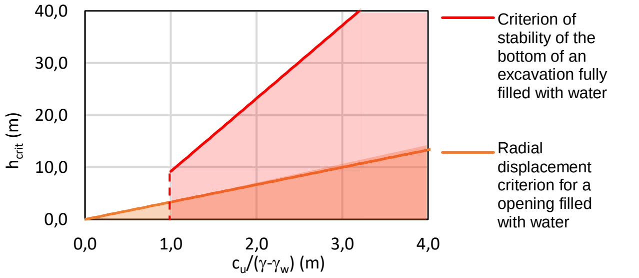

Figure 3.18 *- Comparison between the Radial displacement criterion and the Criterion of stability of the bottom of an excavation fully filled with water.*

For both cases of study is verified that the radial displacements criterion is more conditioning to the depth of excavation.

However, it should be noted that in this study only the value of the undrained resistance, c_u, was varied while all other parameters remained constant. Therefore, the validity of these expressions need to be confirmed by a more extensive parametric study. In addition, in practice there are other factors not contemplated in this study that may lead to much smaller critical excavation depths such as: fissure occurrence, degradation of the mechanical properties of the soils due to the constructive process, occurrence of sandy intercalations, or draining layers, etc.

4. CONCLUSIONS

Concerning the principle referred by EN 1536 about unsupported excavations, it is concluded that for soils responding in undrained conditions the collapse of the excavation only occurs at the bottom of the excavation. However, although EN 1536 suggests that an unsupported excavation is only permitted in ground conditions where the collapse is not likely to occur, the displacements caused by stress relief may compromise the execution of bored piles, leading to the reduction of the borehole section and making impossible to be execute. In fact, after comparing the two criteria, it was concluded that for both cases studied, excavation fully filled with water or not, the radial displacements are the conditioning factor to the critical excavation depth of a pile borehole.

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