

FINITE ELEMENT MODELING OF THE NON-LINEAR BEHAVIOR OF LANDSLIDES

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ABSTRACT:

Natural disasters called landslides have the potential to seriously harm people's safety, the environment, and structures, in addition to causing considerable damage to infrastructure. It is therefore important to be able to model and predict these phenomena. This paper focus on modeling non-linear processes using a finite element model behavior and assess the stability of a slope by calculating the safety coefficient. The characterization work was carried out on a landslide at the site located in Tizi-Ouzou (Algeria). It is characterized by steepness greater than 20%. Verification of the stability of this area, while taking into account the morphology and local geological context of the site, was carried out by finite element modeling using the Mohr-Coulomb criterion and adopting the Cam-Clay model in a Castem step-by-step non-linear calculation code, making it possible to extract the stress field, displacements and deformations and also to find the sliding surface corresponding to a minimum FOS safety coefficient. The results obtained highlight the areas of the slope undergoing plastic deformation, indicative of a high state of stress. The results make it possible to identify the weak points that show that the studied profile is exposed to a risk of landslide, in particular a deep landslide affecting soil layers with a safety factor FOS <1. Preventive actions are recommended to secure the site where supporting measures could be considered.

1. INTRODUCTION

Given that slope stability analysis an important consideration in geotechnical engineering, numerous methods have been proposed for quantitatively assessing the stability of a slope using the FOS safety factor (Yingren, Z. S. Z., 1999). These include the limit equilibrium method (LEM), the shear strength reduction method (SSRM), and the sliding surface stress method (SSSM) have been developed methods used in geotechnical analysis, which have been summarized by several researchers (Duncan, J. M., 1996), (DURVILLE, J.-L. and G. SEVE., 1996), (Khemissa, M., 2006), (Jibson, R. W., 2011).

The stability analysis then consists of searching for the most unfavorable failure surface, which corresponds to a minimum safety coefficient (FOS), this coefficient is used to determine the stability of the terrain. The slope can be considered stable if the safety coefficient is greater than 1. The standard description of the safety coefficient has been given by (Bishop, A. W., 1955). It was subsequently used by other authors such as (Morgenstern, N. u. and V. E. Price., 1965), (Spencer, E., 1967) and (Sarma, S. K., 1973).

The ratio of the shear strength to the shear stress at a given location on the sliding surface is known as the safety coefficient (Yingren, Z. S. Z., 1999), which corresponds to the following equation (Eq.1):

$$FOS = \frac{\tau_f}{\tau_m} \quad (1)$$

Where:

τ_f : Total shear force is available on the sliding surface,
 τ_m : Total shear stress that the collapsing soil mass has mobilized to preserve the equilibrium that is required.

These limit equilibrium theories are unable to take into account the soil's stress-strain behavior and necessitate hypotheses about the failure surface's geometry (Yingren, Z. S. Z., 1999). It is generally limited to the Mohr-Coulomb criterion.

$$\bar{\tau} = c + \sigma \tan \varphi \quad (2)$$

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Where c , ϕ and σ , are the cohesion, the angle of internal friction, and the normal stress, respectively. With the development of computer techniques and soil plasticity theory, FEM finite element programs have made great progress in the non-linear domain. Its pre-processing and post-processing are increasingly practical, and a significant indicator of a slope's stability is its safety factor. This method does not take into account a concept of the safety factor and proposes to evaluate the stresses and strains at the nodes of the elements. The use of the finite element method to solve geotechnical problems began in 1966. Several authors have carried out work in the field of finite elements, including the reference works by (Bouزيد, D. A., 2022), (He, B. and H. Zhang., 2012), the application of (Vengeon, J.-M., et al., 1999), (Sanavia, L., 2009), (Djerbal, L., 2013). The finite element method allows the vertical profile of a slope to be discretized into elements characterized by an elastoplastic behavior, with spatially variable physical properties. It can be used to analyze slope stability using method of reduction of the shear parameters (SSRM) (Liu, X. and M. Su., 2023), (Boulfoul, K., 2021), (Sarkar, S. and M. Chakraborty., 2021). This coefficient is determined by reducing the shear resistance characteristics (C and ϕ) until the calculation does not converge (He, Y., et al., 2022), in other words, until the slope breaks. This process involves several upper and lower bound analyses, each with different resistance parameters. From the Mohr-Coulomb equation and the definition of the safety coefficient, the formulae become:

$$\frac{\tau}{F_s} = \frac{c}{F_s} + \sigma \frac{\tan \phi}{F_s} = c_{crit} + \sigma \tan \phi_{crit} \quad (3)$$

In this case:

$$c_{crit} = \frac{c}{F_s} \text{ et } \phi_{crit} = \arctan \left(\frac{\tan \phi}{F_s} \right) \quad (4)$$

According to the theory of slope stability based on resistance reduction and finite element analysis, the resistance reduction factor would be the safety factor of the slope, when the convergence condition is not satisfied (He, B. and H. Zhang., 2012). The FEM method has several advantages (Onyango, J. A. and C. Zhang., 2019):

- The stress-strain relationship is implied in addition to the equilibrium stress condition being satisfied.
- The crucial fracture surface is immediately identified (Mouyeaux, A., et al., 2015).
- It is used with non-linear elastic-plastic models, like Drucker-Prager, Von Mises cam-clay, and Mohr-Coulomb models.
- It is capable of observing incremental failure all the way to total shear failure.

The FEM method simply calculates the stress field, the displacement field, and the plastic zone. Several scientific research works described the capabilities of the finite element method, these studies compare the work's outcomes with those of other limit equilibrium methods and propose that the finite element approach is more useful in situations in reality (He, B. and H. Zhang., 2012), (Zaki, A., 1999) suggested that finite elements offer real advantages over limit equilibrium methods. Slope stability was evaluated by (Lane, P. and D. Griffiths., 2000), who used the finite element method to create operational diagrams for conditions that should be applicable to existing structures. The limit equilibrium approach and finite element analysis were implemented by (Kim, J., et al., 2002) to study of slopes with variable geometries and irregular soil profiles; the results show that the critical slip surface position and factor

for safety are similar. An initial approach to analyzing mine slope stability is described in the study (Díaz-Durán, F. A., 2023). Numerical simulations using finite elements were used to check the stability of slopes. The choice of a method adapted to the case study depends on several parameters: the means available, the general approach to the slope and the probability of obtaining the calculation parameters corresponding to the model. (Faure, 2000), (Yingren, Z. S. Z., 1999). The results of the work done by (LAFIFI, B., 2009) showed that varying geotechnical plasticity parameters within realistic value ranges, has a significant influence on slope stability.

The objective of this article is to define and describe the mechanisms of collapse of slopes by adopting a non-linear Cam Clay model using the finite element method with the corresponding model parameters, based on data collected from the Aiche Géo-Sol geotechnical analysis and study laboratory in Setif, Algeria. The mapping of the profile's safety coefficient and the calculation of the stress and strain fields were carried out using the Castem.2000 code, according to the stress deviator expansion method using the Mohr-Coulomb criterion, drawing on the work carried out by (Bouزيد, D. A., 2022). The idea behind the new finite element method is that changes in stress, not changes in soil shear characteristics, are what lead to failure, starting with the expansion of the Mohr circle of principal stress until failure (rupture) occurs.

2. SUMMARY OF THE METHOD

2.1 Stress deviator expansion using the Mohr-Coulomb

On the Mohr plane (σ , τ), we represent the stress state at a point by the Mohr circle (Figure 1), derived from the nonlinear calculation of the Cam-Clay model. We extend this circle to the Mohr-Coulomb line to obtain a state of limit stress (at failure).

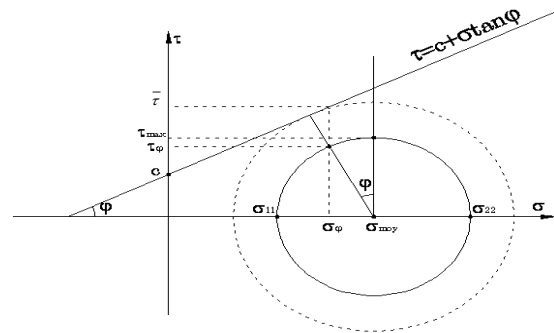


Figure 1. Stress state on the Mohr plane

On a facet whose normal vector makes an angle of $\left(\frac{\pi}{4} + \frac{\phi}{2}\right)$ about the major direction of the largest principal stress σ_{22} , we define the stresses (σ_ϕ, τ_ϕ) . We also define the limiting shear stress $\bar{\tau}$ such that:

$$\bar{\tau} = c + \sigma_\phi \tan \phi \quad (5)$$

The safety factor is ratio between the shear strength $\bar{\tau}$ and the shear stress τ_ϕ :

$$FOS = \frac{\tau_f}{\tau_m} = \frac{\bar{\tau}}{\tau_\phi} \quad (6)$$

The average stress σ_{moy} is defined by:

$$\sigma_{\text{moy}} = \frac{\sigma_{11} + \sigma_{22}}{2} \quad (7)$$

The maximum shear stress is noted τ_{max} equal to:

$$\tau_{\text{max}} = \frac{\sigma_{22} - \sigma_{11}}{2} \quad (8)$$

The perpendicular line to the Mohr-Coulomb line is drawn, and we deduce:

$$\tau_{\varphi} = \tau_{\text{max}} \cos \varphi \quad (9)$$

$$\sigma_{\varphi} = \sigma_{\text{moy}} - \tau_{\text{max}} \sin \varphi \quad (10)$$

2.2 Cam Clay's rupture model (Cambridge model)

The Cam-Clay model was developed by a team of researchers at Cambridge University. After establishing the fundamental concepts of the limit state and the critical state in 1958, these researchers spent a long time developing a behavioral model for isotropic reworked clays. In order to make the model quantitatively usable, they introduced two important assumptions that made it possible to define the limit state surface and the deformations developed along any state path:

- The first assumption is that the soil is a plastic material developing isotropic strain hardening,
- The second concerns the dissipated energy equation.

Historically, the name "Cam Clay" was attributed to a behavioral model which, on the basis of theoretical concepts such as the boundary state surface (BSS) and the "critical void index", generalized the behavior of saturated and moist clay observed in various laboratory tests. These authors analyzed the deformation evolution in terms of energy transfer. They first assumed that the flow law is applicable to wet clay, that wet clay behaves as an isotropic, elastic and non-linear medium within the boundary state surface (BSS) and that there are reversible and irreversible volumetric deformations while shear deformations are irreversible. They then made a further assumption that energy dissipation occurs by internal friction with a certain constant friction coefficient or critical slope M , and can be calculated in q - p ' space from the angle of friction φ (measured in a triaxial compression test) of the Mohr-Coulomb criterion as follows:

$$M = \frac{6 \sin \varphi}{(3 - \sin \varphi)} \quad (11)$$

$$M = \frac{p}{q} \quad (12)$$

p and q are given respectively by:

$$p = \frac{(\sigma_1 + \sigma_2 + \sigma_3)}{3} = \frac{I_1}{3} \quad (13)$$

$$q = \sqrt{\frac{[(\sigma_1 - \sigma_2)^2 + (\sigma_2 - \sigma_3)^2 + (\sigma_3 - \sigma_1)^2]}{2}} \quad (14)$$

$$q = \sqrt{3I'_2} \quad (15)$$

I'_2 is the second deviatoric invariant and can also be written as:

$$I'_2 = \frac{I_2^2}{3} - I_2 \quad (16)$$

Unlike the other models, they therefore introduce another representation space which is the (p' , q , e) space, with e representing the void index. The load function for this criterion is given in the (p' , q) plane by the relation:

$$F = \frac{q}{M_p} + \ln \left(\frac{2.7183p}{p'_c} \right) - 1 \quad (17)$$

With:

p'_c : is the pre-consolidation stress.

$$F = q^2 - M^2 p' p'_c + M^2 p'^2 \quad (18)$$

Cam-clay:

$$F = \frac{q}{M_p} + \ln \left(\frac{2.7183p'}{p'_c} \right) - 1 = 0 \quad (19)$$

The following formula gives the load function of the modified Cam Clay model (Liu, M. and J. Carter., 2002):

Cam-Clay modifies:

$$F = q^2 p' p'_c + M^2 p'^2 = 0 \quad (20)$$

The elastic slope $KAPPA(k)$ and the plastic slope $LAMDA(\lambda)$ in the e - $\log(p)$ diagram are defined by the following relationships:

$$K = \frac{C_s}{\ln 10} \quad (21)$$

$$\lambda = \frac{C_c}{\ln 10} \quad (22)$$

According to the oedometer diagram of the Cam-Clay models

$$\lambda = \frac{C_c}{2.303} \quad (23)$$

$$K = \frac{C_g}{2.303} \quad (24)$$

Where:

C_g : Swelling index

C_c : Coefficient of compressibility

The shear modulus is defined by the following relationship:

$$G = \frac{E}{2(1+\nu)} \quad (25)$$

E : Young's modulus (N/m²)

ν : Poisson's coefficient (without unit)

The soil's mechanical reaction is determined by the behavior laws that apply to it. It is therefore very important to choose a rheological model that can reproduce with optimum fidelity the response of the soil mass to the stresses imposed on it, according to the desired parameters. The soil mechanics characteristics used in Cambridge models are easily acquired. The behavior of a soil mass, according to these models, can be assimilated fairly successfully to that of an elastoplastic material, for which the fundamental stress-strain relationships and the equation for the boundary state surface are well defined.

3. PRESENTATION OF THE STUDY AREA

3.1 Localization of the site

Our study area is a part of the municipality of Tizi-Ouzou, (figure 2).



Figure 2 .Location of the wilaya of Tizi-Ouzou

According to the site plan (Figure 3), the study site is located in the north-west of the municipality of Tizi-Ouzou.



Figure 3. Site plan of the study area

3.2 Description of the site

3.2.1 Geology and lithology of the site: From a lithological point of view, the ground is almost entirely homogeneous, consisting of a surface layer of clayey silt of Quaternary age, no thicker than 1.2m, followed by an average 4m thick layer of marly clay, all resting on a marly bedrock that is weathered at the surface and compact at depth.

3.2.2 Geotechnical characteristics of the site: Table 1 gives the variation intervals for the geotechnical characteristics and the average values for the intrinsic characteristics of the soils covered by this study.

Layers	Marly clay	Friable marl	Compact marl
Natural Water Content (w_n %)	19 To 25.5	15 To 18	2 To 6
Degree Of Saturation (S_r %)	70 To 88	80 To 92	
Wet Density γ_h (t/m ³)	1.88 To 2.02	1.9 To 2.01	2.08 To 2.4
Dry Density γ_d (t/m ³)	1.53 To 1.62	1.58 To 1.65	2.0 To 2.25
Consolidation Pressure P_c (Bars)	1.7 To 1.95	1.9 To 2.2	
Coefficient Of Compressibility C_c (%)	19 To 22	20 To 24	

Swelling Index C_g (%)	<3	<4	
Density γ (Kn/m ³)	19	19.5	22
Cohesion C (Kpa)	35	43	100
Friction Angle ϕ (°)	11	8	0
Thickness (m)	4	5	82.3

Table 1. Geotechnical characteristics of the site

3.2.3 Seismic characteristics: Regarding the seismicity of the region, the municipality of Tizi-Ouzou is part of zone II_a according to the (RPA 2003), which is considered to be of medium intensity. We classified the site based on the lithological aspect of the soil extracted during core drilling, the compactness of the samples, and laboratory tests, in particular density, combined with geotechnical characteristics that are consolidation and intrinsic characteristics (C_u and ϕ_u). To this end, we considered it possible to classify the soils making up the study site as follows:

- Rocky site (compact Marl) classified in category S₁,
- Loose site (marly clay and weathered marl) classified in category S₃.

4 MODELING

Our case study is based on the modelling of a slope located in the town of Tizi-Ouzou. The terrain is composed of a 1.2m thick superficial layer of clayey silt (not shown on the profile, but its overload will be taken into account) resting on a 4m-thick layer of marly clay (layer 1), which in turn rests on a 5m-thick layer of Friable marl (layer 2). The whole rests on a marly bedrock. The considered thickness of the massif varies between 91.30m and 158.89m (figure 4). The stability of the embankment in question was modeled using the Castem 2000 code (version 2022). A software program Castem2000 (Le Fichoux 1998) is used to solve partial differential equations using the finite element method and, more generally, to calculate structures. It was created at the French Atomic Energy Commission's (CEA) Department of Mechanics and Technology (DMT). This code's primary characteristic is its extraordinary adaptability to the various applications that are unique to each user. We have developed programs in the Gibiane language, which we have coded using the new approach of increasing stress deviators.

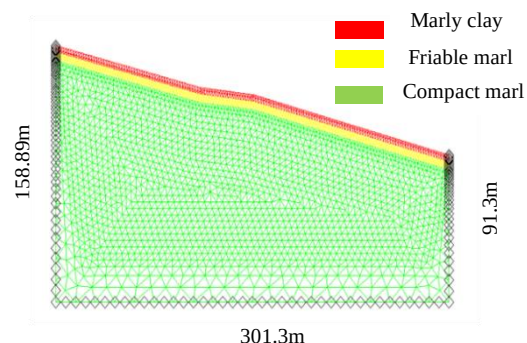


Figure 4. Soil profile

The first step in the modeling process is to create a 2D model in Castem. The selected mesh is composed of 10528 nodes and 5153 triangular elements with 6 nodes (TRI6). This is the way boundary conditions are defined: vertical geometric lines where a small value of the abscissa (x) is found (right) or biggest Abscissa(x) (left) of the model is blocked horizontally ($U_x = 0$), the dimensions of the geometric lines (y) is the same as the least ordinate. (y) of the model (at the base), are completely stopped in both directions ($U_x = U_y = 0$).

Concerning loading conditions, in the first instance, we have considered as the loading force the dead weight of the various layers that make up the massif, and in the second instance, we have taken into account the live load due to urban development, considered to be uniformly distributed at 80 kN/m². The numerical model must include the material properties defined in Table 2 the boundary conditions, the site geometry, and the numerical parameters required to solve the behavior equations for the ground materials. The model is then solved using the step-by-step method to produce stress and strain maps.

Layers	Marly Clay	Friable Marl	Compact Marl
Young's Modulus E (N/m ²)	3.00E+06	6.00E+06	2.00E+08
Poisson Coefficient ν	0.35	0.3	0.3
Initial Vacuum Index e_0	0.7981	0,70358	0.637
Friction Angle ϕ (°)	11	8	0
Coefficient Of Friction M	0.4075	0.2918	0
Cohesion C (Pa)	3.50E+04	4.30E+04	1.00E+05
Reconsolidation Pressure P_0 (N/m ²)	1845	2120	5000
Elastic Slope Kappa K	1,1697	1,194	0.013
Plastic Slope Lamda λ	9,00738	9,8046	0.4342
Shear Modulus G (N/m ²)	1.11E+06	2.31E+06	7.69E+07
Density Rho (Kg/m ³)	1900	1950	2200

Table 2. Soil parameters of the Cam-Clay model introduced in Castem

The applied incremental method requires the kinematic loading to be increased in successive constant steps until the CASTEM Step at Step procedure is broken down numerically. The CASTEM Step at Step procedure requires the input of a data table (which will also be used as an output table) containing the behavior model, the values of the parameters of the chosen model, the boundary conditions, the loading program, and a series of pseudo calculation times.

5 RESULTS AND INTERPRETATION

5.1 Mapping of slope stresses

The results of the horizontal, vertical and shear stresses are presented in the maps of Figure 5, figure 6 and figure 7. These are the horizontal stresses (Figure 5a and Figure 5b). They show that the maximum stresses exceeding 300 kPa (3 bars), shown

in dark red, are located in the lower half of the slope. This zone is subject to high tensile stresses. As you move up the slope, the stresses gradually decrease. The top of the slope is subject to compressive stresses of around 20 to 200 kPa (0.2 to 2 bars). These high horizontal stresses can lead to plastic deformation and creep, reducing stability. From the vertical stress mapping, it can be seen that vertical stresses are negligible at the surface. This zone supports the weight of the overlying soil (Figure 6a). This stress increases with depth. Figure 6b shows differentiated stresses in the overburden zone. These high vertical stresses at depth can lead to settlement and plasticity.

According to the shear stress map figure 7, the areas in red (at depth) and blue (under the overload) correspond to high shear stresses. Moderate values are found in the intermediate zones. Negative shear occurs at the interface of layers 2 and 3.

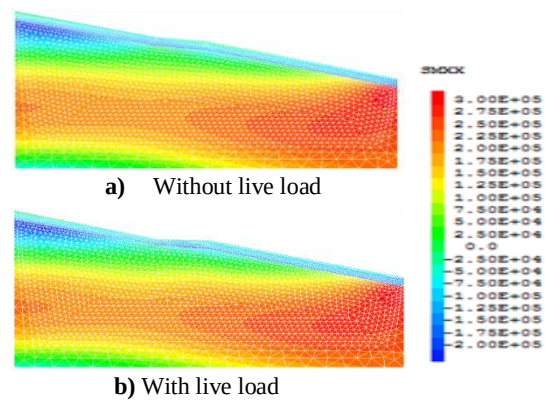


Figure 5. Horizontal stress SMXX (Pa)

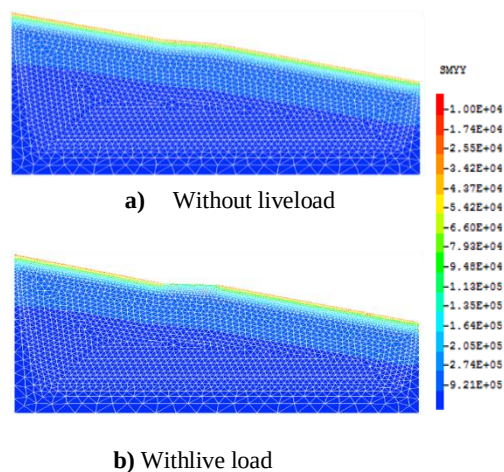


Figure 6. Vertical stress SMYY (Pa)

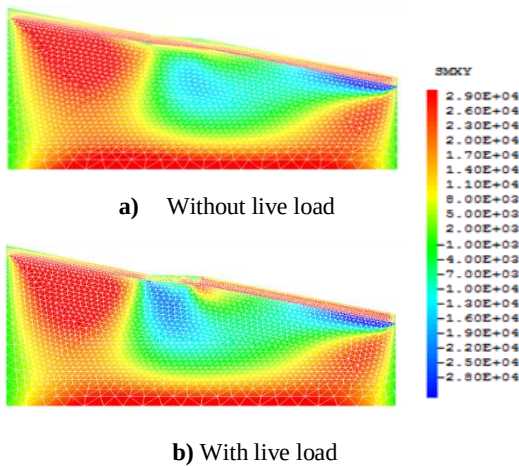


Figure 7. Shear stress SMXY (Pa)

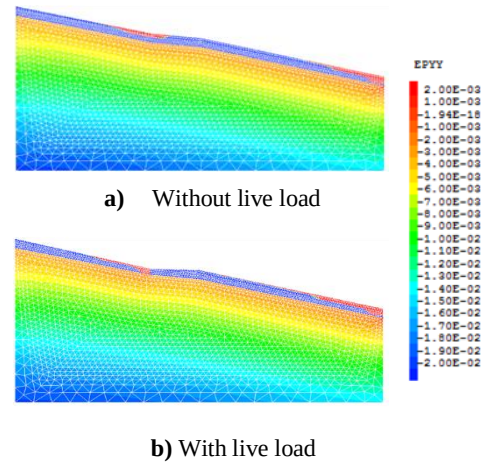


Figure 9. Total vertical strains EPYY

5.2 Mapping of slope total strains

The maps (figure 8, figure 9 and figure 10) show maximum strains of the upper layers and discontinuities at the interface of layers 2 and 3. This is due to the weaker mechanical and rheological characteristics of the upper layers. These variations in geotechnical characteristics can significantly influence the overall behavior of the soil. These maps (figure 8) show the distribution of horizontal strains in the embankment under the effect of its deadweight, and under the overload due to urban development, revealing the areas subject to the greatest stress. This can help detect points of structural weakness. Strains are greatest at the foot of the slope and at the surface. The mapping of total vertical deformations (figure 9) shows a significant deformation in layers 1 and 2 despite low stresses. This can be explained by the weaker mechanical characteristics of layers 1 and 2 compared with layer 3. By analyzing the mapping of total shear strains, Figure 10 shows clear discontinuities at the interface of layers 2 and 3. This can be explained by the weaker mechanical properties of layers 1 and 2 compared with layer 3.

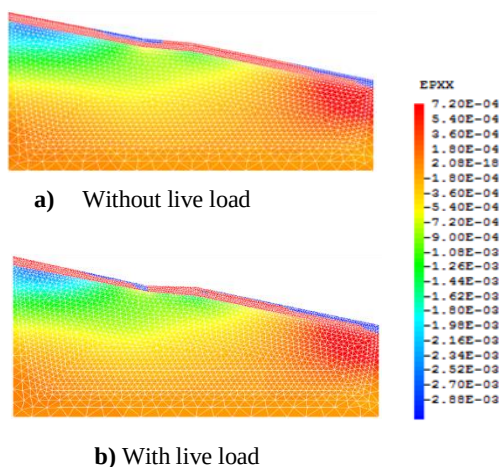


Figure 8. Total horizontal strains

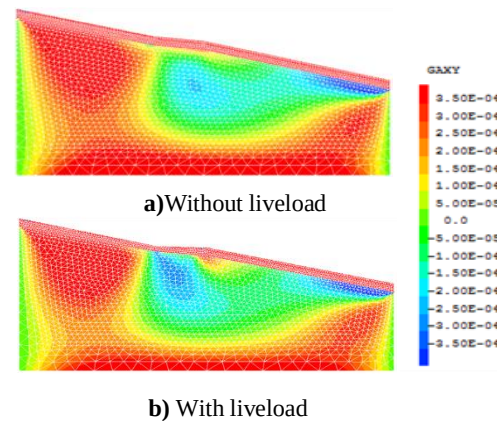


Figure 10. Total shear strains GAXY

5.3 Mapping of slope Plastic strains

The mapping of horizontal plastic (Figure 11) strains shows maximum values on the first two layers and discontinuities at the interface of layers 2 and 3. These maps (Figure 12) highlight areas of the slope undergoing plastic deformation, indicative of a high state of stress. They can be used to identify weak points where reinforcement measures may be required.

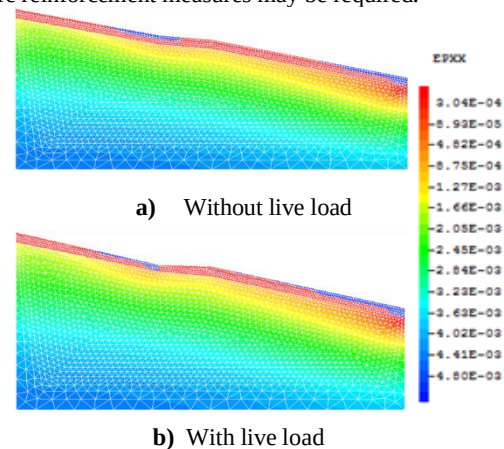


Figure 11. Horizontal plastic strains EPXX

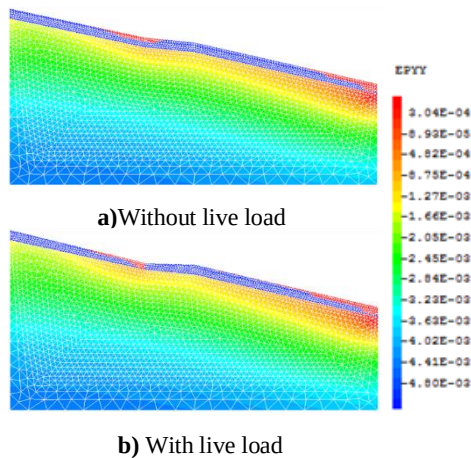


Figure 12. Vertical plastic strains EPYY

5.4 Mapping of slope Sliding surface

Maps (Figure 13a and Figure 13b) show the critical sliding surface and allow a quantitative assessment of the overall stability via the safety factor. The results obtained gave an average FOS value of 1.21 for the slope without overloading on the first 2 layers and an average value of 1.19 with overloading. The calculation of the overall FOS coefficient for the profile as a whole (figure 13a) is 0.297 without overloading, and 0.28 for the slope with urbanization overloading (figure 13b), which is much lower than 1, this shows that the profile is exposed to the risk of landslide.

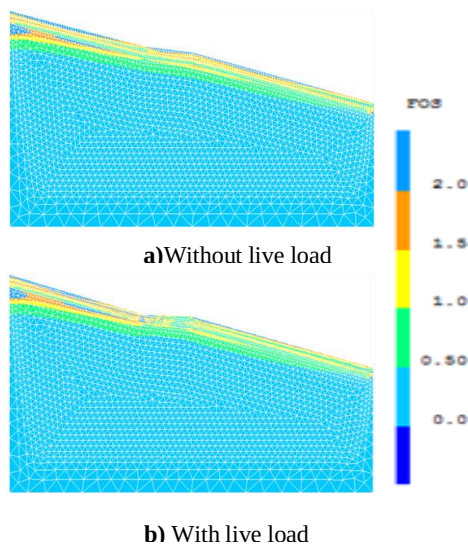


Figure 13. Safety factor mapping FOS

6 CONCLUSION

The studies undertaken in this article consisted in simulating the behavior of a slope located in the town of TIZI-OUZOU, using Castem software and relying on the results of previous studies for the choice of input parameter values. The results obtained enable us to visualize the surface area within which the ground could give way and cause a landslide. This surface is

determined by analyzing the stresses and strains acting on the ground, and thus determining an overall value for the safety coefficient. By examining the mapping of the safety factor values, it is possible to identify potentially unstable risk zones. The choice of the Cam-Clay's non-linear behavior model is based on taking into account the complex geometry of slopes, including variations in cohesion, the angle of internal friction, and other geotechnical parameters. The results obtained show that the masses subjected to the action of their deadweight are subject to considerable plastic deformation depending on the boundary conditions considered.

The method of increasing the stress deviator is considered to be rigorous for several reasons. For a number of reasons, the stress deviator method is seen to be rigorous. Initially, the definition of the Factor of Safety as the relation between the soil's shears strength and mobilized shear stress.

Maintaining this relationship ensures an accurate assessment of the soil's ability to withstand the applied loads. In addition, with this technique, the shear stress that results from a rise in main stress on the same plane as the shear resistance can build gradually. In other words, it considers how the stresses in the soil change as the load does, thus providing a dynamic view of the soil response to the applied stresses. By combining these aspects, the Stress Deviator Augmentation method offers a comprehensive and accurate approach to assessing soil stability and ensuring safe and durable designs in a variety of geotechnical engineering contexts. Based on the results, appropriate measures are recommended to reinforce ground stability, such as installing support structures, modifying slopes or draining groundwater to reduce the mobilizing forces. The mapping of safety factors can help guide engineering actions to ensure the safety of both infrastructures and populations.

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