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# STABILITY FEM ANALYSIS OF ROCK MASSES MODELING PATTERN OF JOINTS

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**Key words:** FEM analysis, Rock Masses, Strength Reduction, Joints patterns.

**Abstract.** In the south of the city of Morelia, Mexico, there is a geological normal fault denominated "La Paloma". It has a height of 180 m and has limited the growth of the city. To improve the connectivity of the city, an urban road is building and it includes the digging of a tunnel that goes through this fault. Due to the presence of an ancient landslide in the exit tunnel, it is imperative to verify the stability of a slope in this zone. The geological structures founded "in situ" make complex the stability analyses, but the used of more realistic representation helps to understand the mechanism of failure. The data collected in geotechnical explorations helped to construct several models for slope stability analysis. Rocks and soils were identified in the interest area. In this way, an elastoplastic Finite Element Analysis (FEA) was carried out to verify the slope stability, considering a strength reduction by a safety factor. Stability was revised in static and seismic conditions. The rock structure is represented by using the Modified Hoek and Brown constitutive model and patterns of the joints with a Mohr-Coulomb constitutive model. The fragments of rock were emulated with joint patterns according to the geologic structure. The slope stability results show a stable slope considering static and pseudo-static FEA analysis. The failure mechanism could be appreciated with the slope stability analysis realized.

## 1 INTRODUCTION

Behavior of rock mass depends greatly on its origin, discontinuities and of the compressive strength of intact rock. The interaction between discontinuities and the intact rock in the mass, define the stability when the natural equilibrium is disturbed. If a modeling of the rock is closer to the reality, a proper stress-strain analysis can be done and it will be possible to understand the most probable failure mechanism [1].

Elasto-plastic Finite Element Analysis (FEA) is a powerful tool for slope stability analysis for soils and rocks. For rock mass, it is necessary to use joint patterns to have an adequate representation of real rock configurations. With both components, it is possible to study the slope failure mechanisms and the joint movements that can conduct to the generation of cracks for delimiting the mass of rock that can slide.

In this work, a study of the application of joint or discrete fracture networks to slope stability analysis, in a real case in the zone of an ancient landslide is presented. The zone is located in a populated area into the Morelia city, together with the construction of a tunnel in the zone such it is necessary to verify the stability of the site. With geotechnical information, a stratigraphy with joint patterns is proposed for advanced stability analysis. It was possible to verify the stability and the movement possibilities for the study area.

## 2 GEOPHISICAL AND GEOTHECNICAL EXPLORATIONS

Indirect methods were the basis for the geotechnical exploration. 15 Vertical Electrical Sounding (VES) with a depth of 250 m in the zone of the tunnel exit portal made by UNAM in 2012 [2] were used to perform the direct exploration. The indirect results show a zone of soil and zone of rock with different qualities. There are low resistivities values in the front of the tunnel and high resistivities in the back of the tunnel, this is an evidence of the location of the “La Paloma Fault” (Figure 1).

Two initial boreholes drilling, with a depth of 40 m, were programmed in the back of tunnel and one Standard Penetration Test (SPT) for soil in the front. The Figure 1 shows the localization of two boreholes. Figure 2 shows the two-borehole logs: the first one (s-1) in the upper part after vegetal layer, there is a stratum of a high plasticity clay mixed with andesite with a mean depth of 10 m, from this depth there is an alternation of andesitic and breccia rock of different thicknesses. The results in the second borehole are the same but with different thicknesses. The rock recovery was between 100 and 25% and the Rock Quality Designation (RQD) between 94 and 0%. It is clear in the stratigraphy that there are zones with rock blocks and zones with broken rock.

A geotechnical profile built with the studies mentioned about and information collected *in situ* is shown in Figure 1. There are four layers: andesite with clay, andesite, breccia and clay. Andesite with clay layer is the product of the weathering of the talus deposit. Andesite has different qualities, in the lowest part of the borehole has RQD of 14.6 and in the upper zone 94%. Breccia has less quality than the andesite and it could be more altered during the drilling. Different RQDs help to select the size of rock blocks for modeling. The transition zone between rock and clay helps to locate “La Paloma Fault”. The blue line is at the joint pattern limit of the landslide chosen for the proposed numerical model, which was selected in function of the UNAM studies. In accordance with the stratigraphy and observations in field, there is no clay layer that limits the slip of the rock mass in its natural state.

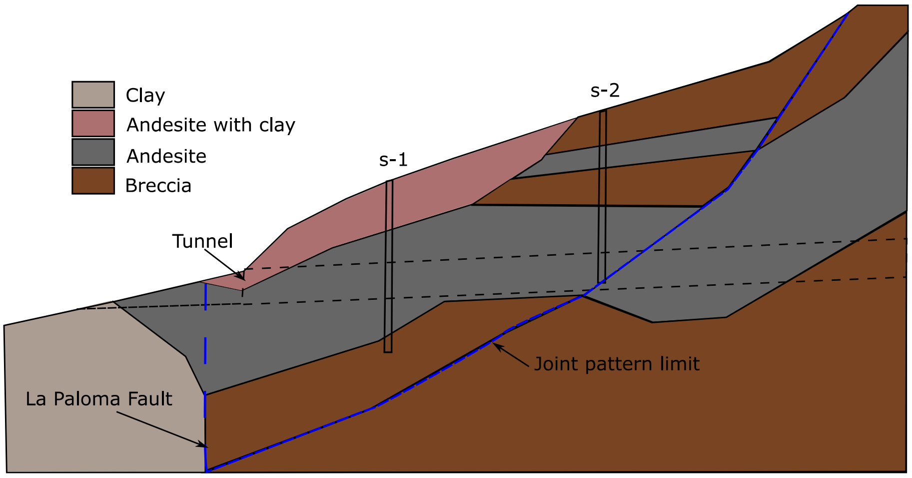



**Figure 1:** Ancient landslide stratigraphy

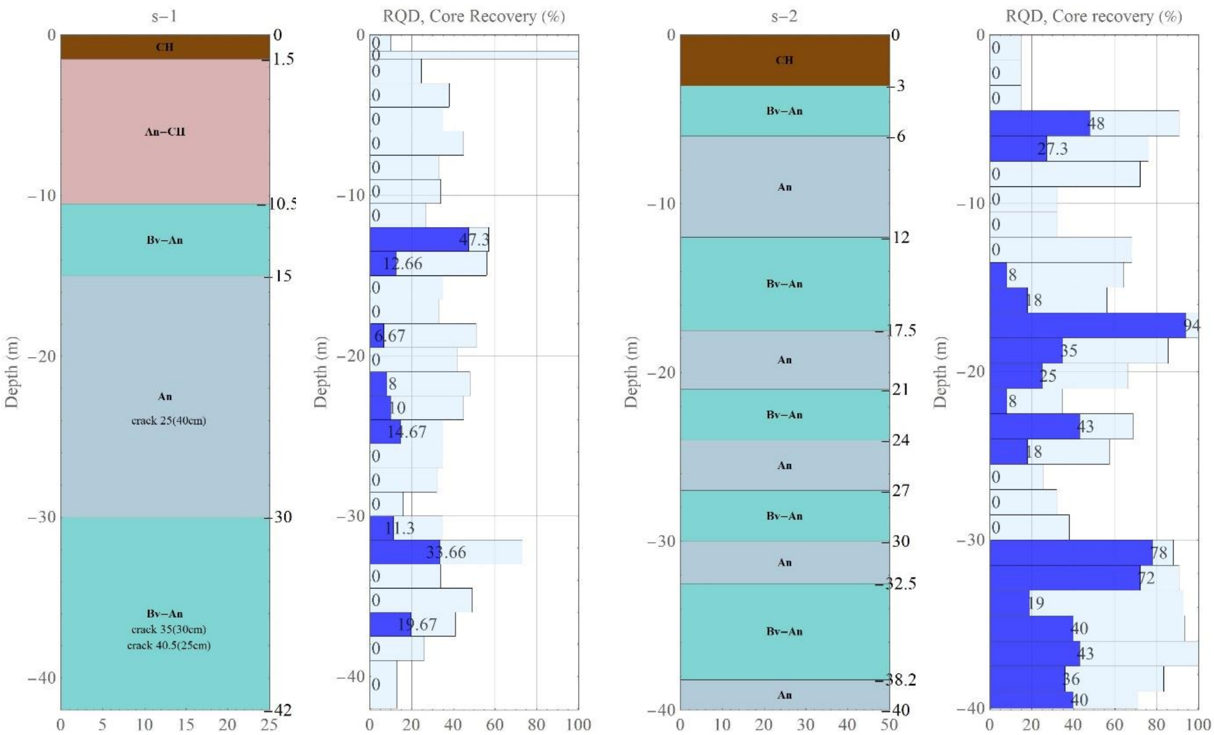


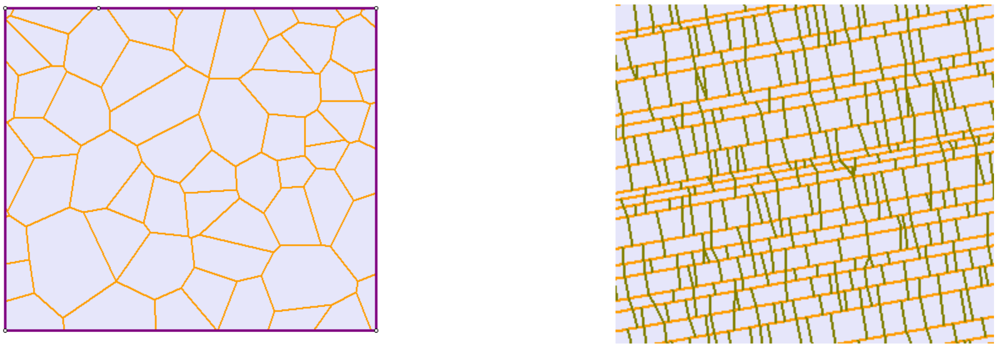

**Figure 2:** Log of the test boring for boreholes s-1 and s-2.

## 3 NUMERICAL MODELING

The geometric pattern of discontinuities was defined by the several discontinuities detected, their orientation, length and the distance between them. Furthermore, physical properties in discontinuities affect the mechanical behavior of the mass as friction, compressive strength, weathering, and filling. In the mass of slipped rock, the fragments are smaller and fragmentation is related to geologic origin of rock.

The rock with discontinuities is modeled with finite elements and joint patterns. The mechanical properties of joints are functions of the physical properties mentioned above. Finite element program RS2 of Rocsience© has the capabilities of performing numerical elastoplastic modeling with joint patterns. Shear Strength Reduction (SSR) technique is applied for performing rock slope stability analysis with Mohr-Coulomb and Hoek y Brown failure criteria.

A volcanic breccia is modeled with a Voronoi joint pattern (Figure 3a). In addition, the andesite has the tendency to be flat and therefore a cross jointed breaking pattern is adopted (Figure 3b). The size of the fragments was approximated according to the core recovery and field observations. The orientation of the rock fragments of andesite is erratic *in situ*, but for the analysis an unfavorable orientation in the movement direction was adopted. Finally, lower left stratum clay was considered continuous. Figure 4 shows the final joint pattern supposed for the slopes stability analysis of the ancient landslide.



**a) Voronoi joint patterns for the breccia**

**b) Cross jointed pattern for the andesite**

**Figure 3.** Joint patterns employed for the modelling.

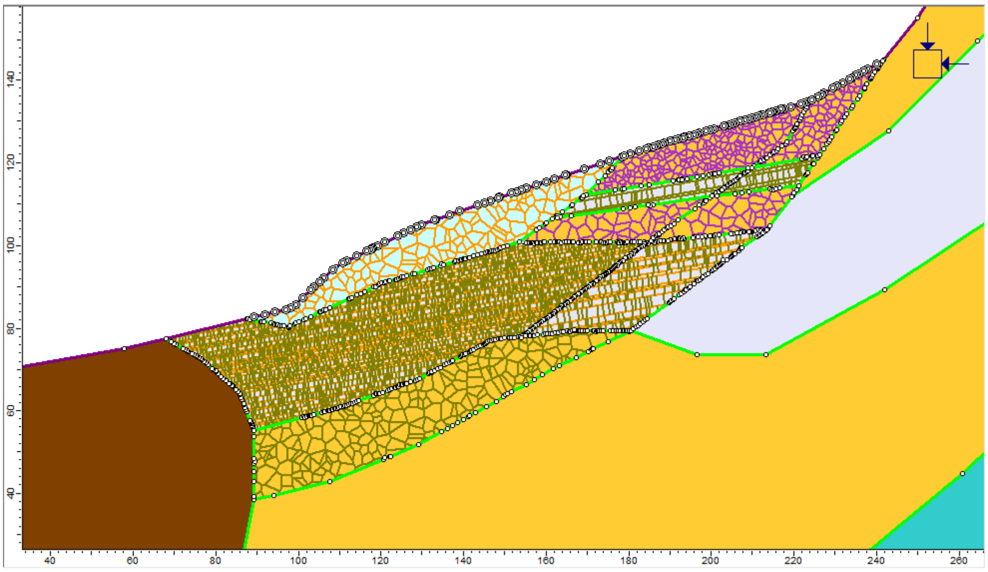



**Figure 4**. Discretization in RS2® of the domain showing the joint patterns

### 3.1 Mechanical properties of the rock and soil

Generalized Hoek and Brown Failure criterion [3] for intact rock was used and for the joints Mohr-Coulomb criteria. Hoek and Brown modified a nonlinear failure criterion of brittle intact rock for rock mass. The criterion has factors that reduce mechanical properties of the mass considering the characteristics of the joints in rock mass. Equation (1) defines failure criterion:

$$\sigma_1' = \sigma_3' + \sigma_{ci}\left(m_i\frac{\sigma_3'}{\sigma_{ci}} + s\right)^a \tag{1}$$

Where:

$m_b = m_i\, exp\left(\frac{GSI-100}{28-14D}\right)$, is a reduced value of the intact rock $m_i$, $s$ and $a$ are constants of the rock mass and are defined by:

$s = m_i\, exp\left(\frac{GSI-100}{9-3D}\right)$ and

$a = \frac{1}{2} + \frac{1}{6}\left(e^{-GSI/15} - e^{-20/3}\right)$

The Geological Strength Index (GSI) relates the geological field observations with the failure envelope, it can be estimated by graph presented by Hoek [3]. $D$ is a parameter related to the degree of disturbance to which rock mass has been subjected to blast damage and stress relaxation. Table 1 present the parameters used for stability analysis of the mass rock.

**Table 1.** Parameters of the constitutive models of the mass rock.

| Material | $\gamma_m$ (kN/m$^3$) | σci (MPa) | GSI | mi | $E_{RM}$ (MPa) | Disturbance factor. |
|---|---|---|---|---|---|---|
| Rock with clay | 22 | 67 | 55 | 19 | 26,800 | 0 |
| Andesite | 25 | 170 | 75 | 25 | 68,000 | 0 |
| Breccia-Andesite | 23 | 110 | 70 | 19 | 55,000 | 0 |

There are two important parameters that control the behavior of intact rock, the compressive strength, and GSI. In the case of the rocks found, andesite is the strongest one and rock with clay the weakest, due to the weathering to which it has been subjected. The GSI parameter represents a geologic structure of the intact rock and surface conditions, andesite has a good surface features, but the rock with clay has a fair surface condition.

Shear strength envelope for clay was estimated with an undisturbed sample obtained just above the rock with clay stratum. A consolidated undrained direct shear test has been carried out. The cohesion was of 29 kPa and the friction angle of 13.6°. Elastic parameters *E*= 15,000 kPa and Poisson ratio of 0.4 are derived also from the laboratory tests.

The crossing discontinuities are the weakest zones of the rock mass and determine its behavior, each material has independent properties. Roughness controls the shear strength of the discontinuities. The Barton and Choubey [4] criterium is used for the estimation of the rock joints strength. Joint Roughness Coefficient (JRC) and other parameters were measured directly from rock core samples of andesite and breccia. RS2® cannot perform the shear strength reduction stability analysis with Barton and Choubey criterium, so Mohr-Coulomb envelope was fitted for the range of normal stresses used. Joint stiffness (normal, *kn*, and shear, *ks*) depends on the infilling material. In the rock with clay case depends on the clay; in the case of the intact rock depends on the andesite and breccia surface roughness and strength. Table 2 shows the properties used for the analysis presented in this work.

**Table 2.** Parameters of the rock and clay joints

| **Material** | **Mohr-Coulomb** | | **kn (kPa/m)** | **ks** |
|---|---|---|---|---|
| | c (kPa) | $\phi$ (°) | | (kPa/m) |
| **Clay (CU)** | 29 | 17 | 150,000 | 55,000 |
| **Andesite Joint** | 200 | 57 | 5,440,350 | 2,014,940 |
| **Breccia-Andesite Joint** | 175 | 53 | 3,186,990 | 1,225,770 |

## 4 ANALYSIS AND RESULTS

Two cases are considered for the analysis: a static and pseudo-static seismic. The second case due to Morelia city is in a seismic zone, and all the designs are revised under this condition.

SSR technique, in the case or Mohr-Coulomb failure criterion, consists of decreasing the values of $c$ and $\phi$ original parameters by a safety factor. The result is an increase of the plastic deformations that complicates the numerical convergence. When convergence is not achieved, the safety factor is found [5].

In the case of Modified Hoek and Brown failure criterium, a safety factor divides the equation of the shear failure criterion, to obtain a reduced failure envelope, $\tau^{red}$. Thereafter, a new set of parameters is calculated by modified the reduced envelope to fit the Hoek and Brown criterion. Finally, an elasto-plastic conventional finite element analysis is carried out to check the convergence [6]. The reported safety factor is the minimum value that causes the convergence failure.

Results for the static case are presented in Figure 5, the critical strength reduction factor or safety factor is 2.29. The figure shows the total displacements in meters against Shear Strength Reduction. For the critical safety factor, the largest displacements are in red and the minor ones in blue. It can be deduced that the mechanism of failure is a block with a displacement produced by the low strength of left lower clay. Midzone of the slope moves to the left and a crack is formed at the middle of the slope. The upper part of the movement is reduced but it generates cracks.

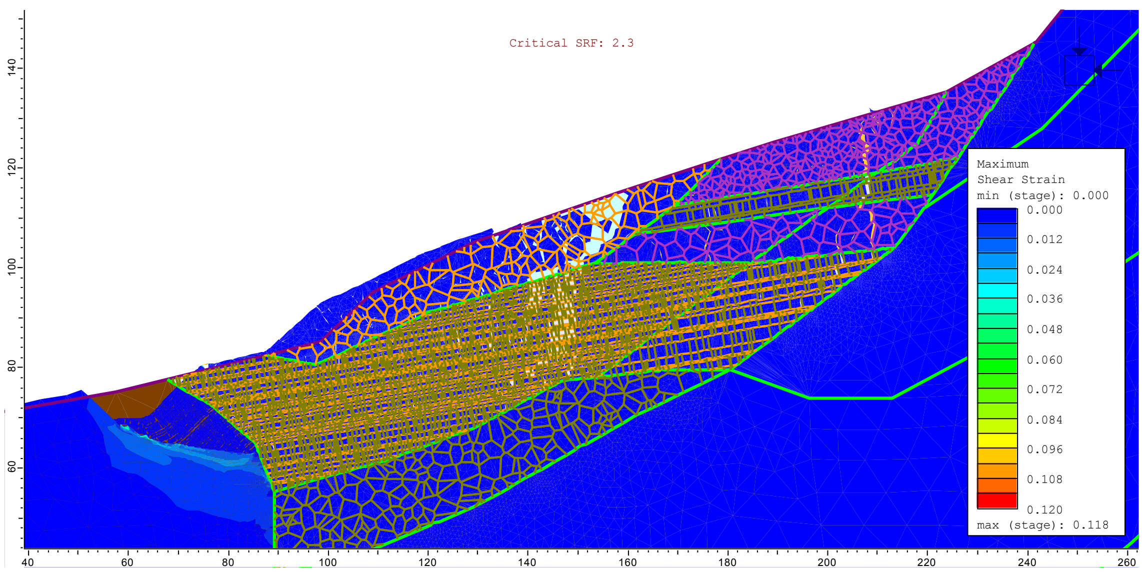


**Figure 5**. Displacement kinematics for the critical safety factor of the static analysis.

Figure 6 shows the strength reduction factor against the maximum displacement. Red triangles are values that failed to converge, and green ones do converge. Furthermore, the figure shows a very significant change of slope where the displacements are incremented rapidly. In the zone of the change the critical safety factor can be found. Therefore, this increase of the displacements gives rise to the failure mechanism under the modelized conditions.

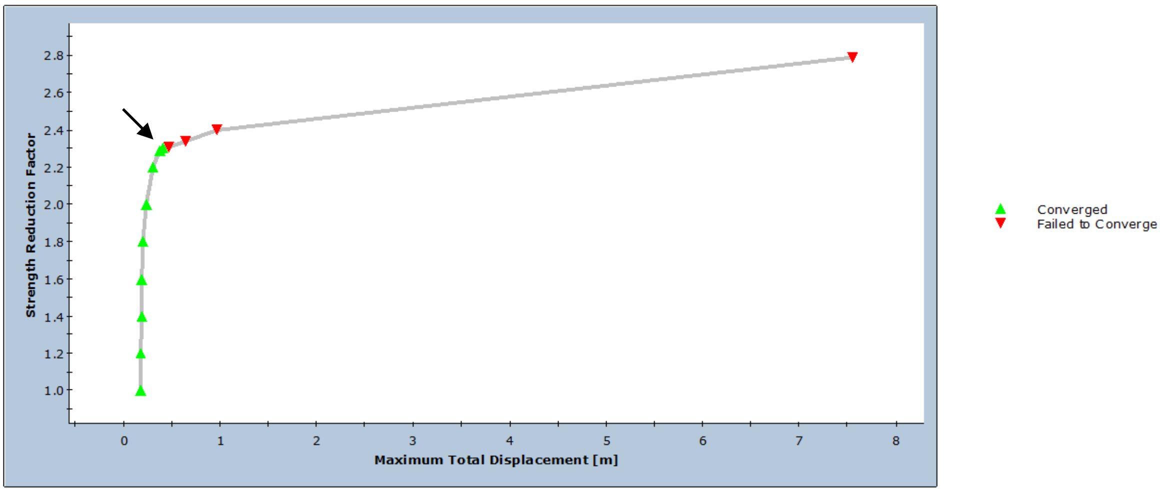



**Figure 6**. Shear strength reduction versus displacement.

Morelia is located on the central portion of the Mexican volcanic belt. The interaction between the tectonic plates of Rivera, Cocos and North America generates a great number of earthquakes in the region. The displacement between the Cocos and North America plates produces superficial earthquakes, between 15 and 20 km of depth, as in the earthquake of 1985 with magnitude of 8.1 in the coasts of Michoacán, Mexico. An interaction between Cocos and Rivera plates produces earthquakes within the continent, which are less frequent. Cruz in 2015 [7] made a deterministic study to get the seismic coefficient for the design of works of the urban road in the south of Morelia. From the study, it was concluded that the seismic coefficient for pseudo-static slope stability analysis is of 0.145.

For the same previous analysis, a horizontal force proportional was added to the mass and the SSR slope analysis was performed, the results are shown in Figure 7. As consequence, the factor of safety was reduced to 1.24 but it still greater than 1.0. Maximum deformation is incremented from 0.12 to 0.4 meters, so the cracks are more open especially in the middle zone. In general, the mechanism of failure is the same already presented. Increased deformation causes the safety factor to decrease, this is reflected on non-convergence of the calculus. Figure 8 shows a graph of the shear strength reduction *against* maximum displacement. As in the previous analysis, there is a change of slope in the critical strength reduction factor.

A concern of the inhabitants of the zone is the magnitude of the landslide that could be presented. The factor of safety is greater than one so there is not risk of failure. It could exist a risk if the lower clay is removed without an adequate reinforcement. In that case mass of rock could move downward and generate a cracking in the upper zone. It means that rock blocks can stand on their own with certain angles. A physical corroboration of this state was located aside the study zone, were a cut of 20 m height and 70 with degrees was excavated for a construction of a house and is in stable conditions.

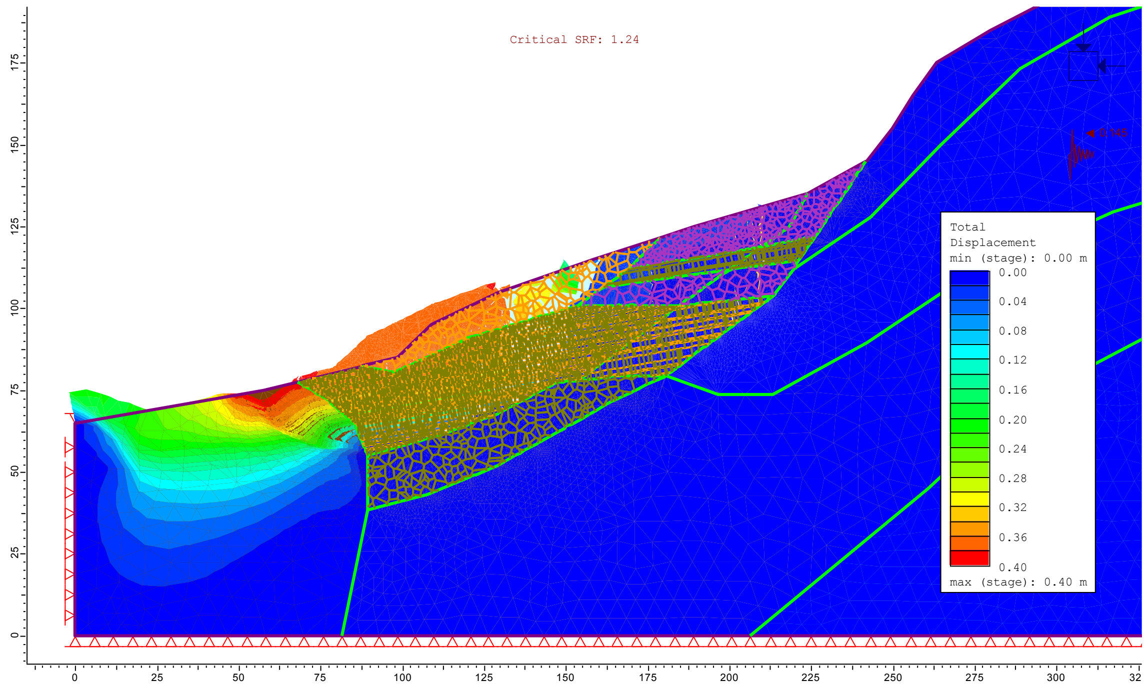



**Figure 7.** Kinematics displacement of the Pseudo-static analysis obtained from the SSR stability analysis.

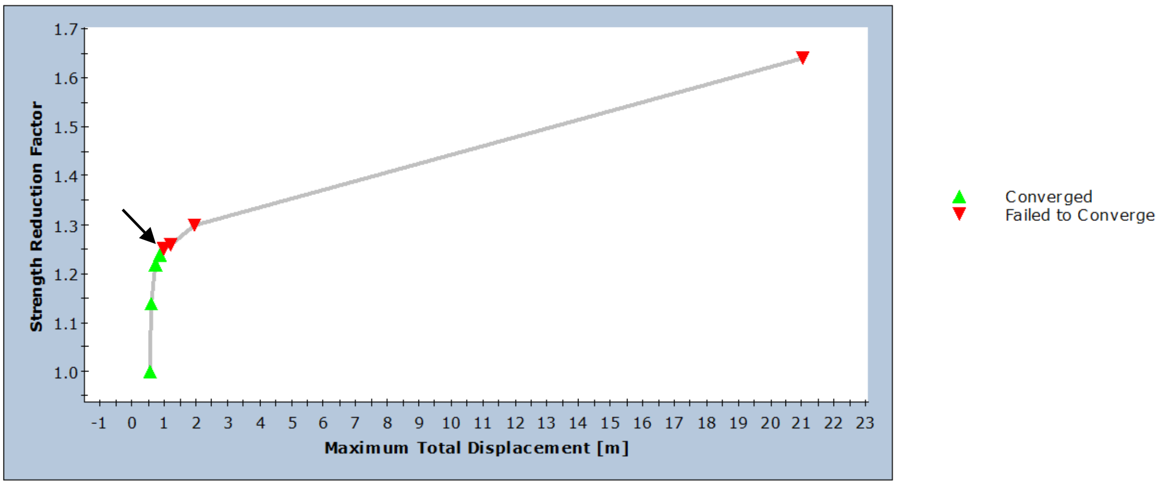



**Figure 8**. Shear strength reduction versus displacement for pseudo-static analysis.

## 5 CONCLUSIONS

The use of the finite elements with networks of joints helps to perform a stability analysis of slopes in rock masses more accurately. It is necessary to have field information of the fracturing patterns of the rock, for making a suitable approximation by constructing a dominium with joint networks. With this, it is possible to capture and understand the failure mechanism of rock mass in slope stability analysis in a realistic way. The strength reduction technique within a framework of elasto-plasticity allows to magnify the deformations and failure mechanism emerges in a natural way.

In the case of stability analysis of “La Paloma”, antique landslide, it was possible to verify the stability conditions. Factors of safety of 2.3 and 1.24 were derived from static and seismic stability analysis. Two joint patterns were adopted to represent the rock block as observed in the field. Giving to the options available in the RS2®, the Voronoi pattern was selected for breccia and cross jointed for the andesite. This representation is simplified and shows the most unfavorable conditions for stability of the mass rock. For a more accurately representation, more information is needed and can be directly obtained from the tunnel excavation.

More research is required in the use of joint patterns for the analysis of slopes stability. The success depends on adequate pattern representation, geological conditions are complex in this case. Another issue is the constitutive models used for the content between rock, in this work it is used Mohr-Coulomb criterion, but there are some others like Barton and Choubey [4] that can be incorporated in the stress reduction analysis.

## REFERENCES

[1] Elmo, Davide, and Doug Stead. "An integrated numerical modelling–discrete fracture network approach applied to the characterization of rock mass strength of naturally fractured pillars." Rock Mechanics and Rock Engineering 43.1 (2010): 3-19.

[2] Universidad Nacional Autónoma de México. Estudio Geológico-Geotécnico del ramal camelinas del Libramiento Sur de Morelia. México, DF., (2012).

[3] Hoek, E., Carranza-Torres, C.T., and Corkum, B. (2002), Hoek-Brown failure criterion – 2002 edition. Proc. North American Rock Mechanics Society meeting in Toronto in July 2002.

[4] Barton, N., and Choubey, V. The shear strength of rock joints in theory and practice. Rock Mechanics and Rock Engineering 10.1 (1977): 1-54.

[5] Abramson, Lee W. Slope stability and stabilization methods. John Wiley & Sons, (2002).

[6] Hammah, R. E., Yacoub, T. E., Corkum, B. C., & Curran, J. H. The shear strength reduction method for the generalized Hoek-Brown criterion. In Alaska Rocks 2005, The 40th US Symposium on Rock Mechanics (USRMS). American Rock Mechanics Association. (2005).

[7] Chávez, C., Arreygue, J. E., Cruz J. I. and Alarcon J. Informe Final de los Estudios Geotécnicos Complementarios al Proyecto Geotécnico del Ramal Camelinas del Libramiento Sur de Morelia. SCT, Mexico, (2015).

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# CHARACTERIZATION OF EVOLVING PLASTIC ANISOTROPY AND ASYMMETRY OF A RARE-EARTH MAGNESIUM ALLOY SHEET BY MEANS OF A NON-ASSOCIATED FLOW RULE

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**Key words:** Anisotropic Plasticity, Non-associated Flow Rule, Magnesium Alloy

**Abstract.** The superior ductility of rare-earth magnesium alloys over conventional magnesium sheets makes them promising candidates for light-weight structural alloys. However, these alloys possess severe evolving anisotropy and tension-compression asymmetry as a result of activation of different deformation mechanisms (slip or twinning) that is extremely challenging to model numerically. In this study, the constitutive plastic behaviour of a rare-earth magnesium alloy sheet, ZEK100 (O-temper), was considered at room temperature, under quasi-static conditions. A CPB06 yield criterion for hcp materials was employed along with a non-associated flow rule where the yield function and plastic potential were calibrated at different plastic deformation levels to account for evolving anisotropy in proportional loading. The constitutive model was implemented as a user material subroutine (UMAT) into the commercial finite element package, LS-DYNA, along with an interpolation technique to consider the evolving anisotropy of the material. Finally, predictions of the model were compared with the experimental results in terms of flow stresses and plastic flow directions under various proportional loading conditions and along different test directions. It was shown that the predictions of the model were in good agreement with experimental data.

## 1 INTRODUCTION

Most commercial alloys used in sheet metal forming and vehicle crashworthiness applications exhibit some degree of orientation-dependent plastic response, and depending on the severity of plastic anisotropy, isotropic yield functions might not be suitable candidates for modelling the behaviour of the materials. To overcome this issue, a large number of anisotropic yield functions have been proposed in the literature with the largest contributions from the Barlat family of yield criteria [1,2] in which linear transformations are applied on the stress tensor to account for anisotropy. However, these models were intended for bcc and fcc cubic materials with slip-dominated deformation mechanisms while magnesium alloys have