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Evaluation of Geotechnical Properties and Parameters that most Influence the Safety Factor in Numerical Rock Slope Stability Analysis

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ABSTRACT: Most sensitivity studies of the safety factor in numerical slope stability analysis have been conducted with soil data. In the case of rock, due to the need for a greater amount of data, sensitivity analysis becomes more complex. Because of this, the objective of this study is to evaluate the geotechnical properties and parameters that most influence the safety factor in rock slope stability analysis using the numerical shear strength reduction method. The methodology used involved determining the geometric data of the slope, obtained on site, and defining the properties and parameters required for the stability analysis. In addition, the ranges of values for these geotechnical data were estimated based on the literature. Subsequently, the acquired data were utilized to conduct sensitivity simulations through geotechnical software. The results of the sensitivity analysis indicate that the uniaxial compressive strength, GSI, material constant, disturbance factor, and specific weight exert some influence on the numerical safety factor. This factor is sensitive to increases in value when conditions lead to improved slope quality and decreases in value when conditions lead to worse slope quality. With regard to the modulus of elasticity and Poisson's ratio, there was no impact on the numerical safety factor, which demonstrated a high degree of insensitivity to the variability of the intervals considered for these data. It can be reasonably concluded that the use of sensitivity analysis is of some importance as a complement to the numerical analysis of rock slope stability. This allows for a more reliable assessment of the actual state of rock slope stability.

KEYWORDS: Safety factor, numerical methods, geotechnical data, failure criterion, sensitivity analysis.

1 INTRODUCTION

Slope stability represents a pivotal domain within the field of engineering, with a primary focus on ensuring the safety of surfaces that are inclined towards the horizon. Such surfaces may be composed of soil, rock, or a combination of both, and may be either natural or artificial in origin. In recent decades, methodologies for slope stability analysis have undergone a significant evolution, progressing from the interpretation of simple graphs and tables to numerical simulations based on finite elements. The technological advance in computational capabilities has been a significant milestone in several areas, including geotechnics (Wyllie, 2017).

The prevailing deterministic approach considers the safety factor to be the ratio between resistive shear stresses and mobilizing shear stresses. However, methods such as the limit equilibrium slice method, which applies this deterministic approach, were deemed to be limited due to the necessity of assuming a potential failure surface and the difficulty of calculating various interactions in the search for the surface that converges to the lowest safety factor. This method allows for the possibility of failure to occur in a planar, wedge, circular, toppling, or mixed manner, and at a location above or below the toe of the slope. This technique is therefore particularly useful for assessing the stability of slopes subject to internal and external disturbances. With the advent of advanced computing, this limitation in the search for the surface has been effectively addressed through the implementation of an algorithm (Griffiths and Lane, 1999; Hammah *et al.*, 2005; Wyllie, 2017; Zhang, 2020).

For a considerable period of time, the slice method, developed by various authors, was a widely utilized approach in the analysis of slope stability in soil and rock. This method, based on limit equilibrium theory, had significant limitations. In order to apply it, it was necessary to assume a potential failure surface and consider or not consider the interaction between the forces acting on the slices. However, this often led to unrealistic results. To address these limitations, numerical analysis of slope stability based on finite elements emerged as a promising solution. This approach eliminates the restrictions imposed by the slice method. The model incorporates additional soil or rock properties and parameters, allowing the potential failure surface to be determined through the stress and strain analysis generated within the model. The safety factor is determined by the convergence or not of a reduction factor applied to the properties and parameters. Although the necessity for additional data is advantageous in terms of enhancing the accuracy of the safety factor, it also poses certain challenges. The incorporation of additional information may introduce uncertainties and variability due to potential inaccuracies in determining the necessary properties and parameters (Griffiths and Lane, 1999; Wyllie, 2017).

The selection of a failure criterion represents a pivotal aspect of the numerical method employed in slope stability analysis. Initially, the Mohr-Coulomb criterion was the most widely used, but its greater applicability to soils limited its use in studies involving rocks. In the case of slopes composed of rock masses, in which articulated blocks of intact rock are delimited by discontinuous surfaces, the Generalized Hoek-Brown criterion is more recommended. This criterion was developed and refined over four decades ago, with the specific aim of addressing the stability of rock masses (Griffiths and Lane, 1999; Hoek *et al*., 2002; Hammah *et al*., 2005; Wyllie, 2017; Zhang, 2020).

The majority of studies investigating the influence of properties and parameters have been conducted on soils using the Mohr-Coulomb failure criterion. In this context, the sensitivity of specific weight, cohesion, and friction angle was observed, while the modulus of elasticity and Poisson's ratio proved to be almost insensitive to the numerical safety factor. However, there is a paucity of studies on the sensitivity of rock mass properties and parameters, particularly when employing the Generalized Hoek-Brown failure criterion (Wyllie, 2017; Zhang, 2020). Given this gap, the aim of this work is to evaluate, by means of sensitivity analysis, the geotechnical properties and parameters that most influence the safety factor in rock slope stability analysis, using the numerical shear strength reduction method.

2 METHODOLOGY

The study was conducted in multiple stages. Initially, data was collected in the field in accordance with the guidelines recommended by Wyllie (2017) with the objective of establishing the geometry of the slope. Subsequently, the rock strength was determined based on the methodology proposed by Basu and Aydin (2004) and Aydin and Basu (2005). This methodology assesses the hardness of the rock by means of the rebound of the Schmidt hammer, taking into account corrections related to the orientation and type of hammer used. Subsequently, the density, uniaxial compressive strength, and modulus of elasticity were determined by correlating them with the rebound values, as well as their sensitivity intervals, in accordance with the methodologies proposed by Deere and Miller (1966), Aydin and Basu (2005) and Hoek and Diederichs (2006). Furthermore, the Poisson coefficient and its variability were determined based on the study proposed by Hoek and Brown (1997).

The GSI (Geological Strength Index) values, the material constant, and the disturbance factor adopted for the rock mass of the slope, as well as their sensitivity intervals and the parameters of the Generalized Hoek-Brown failure criterion, were obtained following the methodologies suggested by Marinos and Hoek (2000) and Hoek *et al*. (2002). Finally, a sensitivity analysis was conducted on all the data obtained for the slope under study, with regard to the uncertainty intervals for each property and parameter required for the analysis. This was done using the numerical shear strength reduction method. This method was proposed by Hammah *et al*. (2005) for rock masses using the Generalized Hoek-Brown failure criterion.

3 RESULTS

3.1 Location, geological and geomechanical description of the slope

The slope under study (see Figure 1) is located in the municipality of Boa Viagem, in the state of Ceará. The road cut is situated on the BR-020, a highway with a high traffic flow and which represents an area of geological risk due to the possible occurrence of mass movements in the rocks. Access to this area is possible from Fortaleza through the BR-020, which covers a distance of approximately 254 km. The dominant lithology on the slope is migmatite gneiss, which is characterized by complex banding. In accordance with the classification system of the Brazilian Geological Survey (CPRM), the rock is classified as a banded orthogneiss, with the presence of schist. The slope displays a multitude of discontinuities, which collectively define it as a rock mass. The prevalence of articulated blocks and the occurrence of various discontinuities, including fissures, joints, veins, faults, banding, and schistosity, are noteworthy characteristics (Forgiarini *et al*., 2021).

Figure 1. Target slope of the study.

3.2 Geometry and results of calculations of the geotechnical properties and parameters of the slope

The selected slope section exhibits an inclination of 87° and a length from the base to the crest of the slope of 6.69 m. By establishing a trigonometric relationship between these two data points, an approximate value of 6.68 m is obtained, assuming that the slope is practically vertical. The rock rebound was measured directly on the intact portions of the predominant rock present on the slope, at an inclination of +45°. Subsequently, the orientation and the N rebound value were corrected to L, taking into account the 50% highest values of the 20 data points collected. This yielded an average index value of 49.63. Based on the aforementioned average rebound, a natural apparent density of 2.51 g/cm^3 was determined by correlation, which is equivalent to a natural apparent specific weight of 0.0246 MN/m^3. For the uniaxial compressive strength and modulus of elasticity, which are dependent on both the rebound value and the bulk density, the calculations yielded a strength of 120.42 MPa and a modulus of elasticity of 51.54 GPa. With regard to the Poisson's ratio, a value of 0.25 was established for the slope condition with medium-hard rock and medium-quality mass. The data, calculated in a sequential manner, are presented in Table 1.

Table 1. Result of the calculation of the geotechnical properties and parameters of the slope.

Properties/Parameters	Data									
R_N (50% bigger)	61	61	60	60	59	58	54	54	54	53
R_N (+45° to 0°)	62.5	62.5	62	62	61	60	56	56	56	54
R_N to R_L	52.73	52.73	52.26	52.26	51.32	50.38	46.62	46.62	46.62	44.74
Average	49.63									
ρ (g/cm^3)	2.51									
γ (MN/m^3)	0.0246									
UCS (MPa)	120.42									
E (GPa)	51.54									
υ	0.25									

3.3 Result of the parameters of the Generalized Hoek-Brown failure criterion

The rock mass of the slope was classified as Block/Disturbed/Structured, with surface quality ranging from weak to very weak. The GSI was estimated to lie within the range of 15 to 30, with an average value of approximately 23. Given that the slope is predominantly composed of gneiss, a single material constant was assigned, with an average value of 28. The degree of slope disturbance was evaluated based on the impact of blasting operations in civil engineering projects, which resulted in significant disruption to the remaining mass. This factor was adopted as 1. The properties and parameters of the Generalized Hoek-Brown failure criterion were obtained using the RSData *software*. By inputting the requisite properties and parameters, including uniaxial compressive strength, GSI, material constant, disturbance factor, natural specific weight, and slope height, it was possible to generate the results depicted in Figure 2. It is important to note that the constants of the criterion and the properties and parameters related to the rock mass are of particular significance in this context.

Hoek Brown Classification	
UCS of intact rock (MPa)	120.42
GSI	23
mi	28
disturbance factor	1
Intact Modulus (MPa)	51540
Hoek Brown Criterion	
mb	0.114
s	2.67e-06
a	0.536

Rock Mass Parameters	
tensile strength (MPa)	0.003
uniaxial compressive strength (MPa)	0.124
global strength (MPa)	4.278
modulus of deformation (MPa)	1256.878
Failure Range Envelope	
application	Slopes
sig3max (MPa)	0.158
Mohr Coulomb Fit	
cohesion (MPa)	0.054
friction angle (°)	47.385

Figure 2. Results of the geotechnical parameters calculated in the RSData *software*.

3.4 Result of the analysis for the average data obtained

The numerical analysis was conducted using the shear strength reduction method, which necessitates the utilization of specific properties and parameters. These properties and parameters were included and utilized in the analysis, which was carried out using RS2 finite element *software*. The two-dimensional (2D) section of the slope was modeled in order to consider its height and slope, thus establishing an artificial finite representation of the horizontal and vertical extensions in the domain of influence of the problem area. As this is a rock slope, scale is one of the most relevant parameters for classification. Slopes can be classified as intact rock, discontinuous rock or rock mass. Although the slope under consideration is not particularly extensive, it exhibits more than four distinct types of discontinuities, which categorizes it as a rock mass. Given the focus on simulation, the 2D section of the slope was modeled as a continuous medium for simplification purposes, given the difficulty in acquiring data and modeling it as a discontinuous medium. It was therefore assumed that the discontinuities present are strongly articulated, with low persistence and spacing, allowing the Generalized Hoek-Brown failure criterion to be applied.

The finite element mesh model assumed a uniform distribution of elements, which were triangular with six nodes, resulting in approximately 2,500 elements. The properties and resistance parameters of the rock mass were applied to both the peak resistance and the residual resistance, using an elastic-plastic model. The analysis of the average data yielded a numerical safety factor of 1. This result indicates that the slope is on the verge of failure, with the influence of an internal or external parameter being sufficient to cause it to collapse. This is due to the fact that the mobilizing shear stresses of the potential failure surface are equal to the resistive shear stresses. Based on the results observed in Figure 3, it can be postulated that the potential failure zone, exhibiting lower values of resistive stresses, resembles a plane, non-circular surface. The potential failure surface emerges at the face and top of the slope, where the top characterizes a tensile zone and the base a compressive zone.

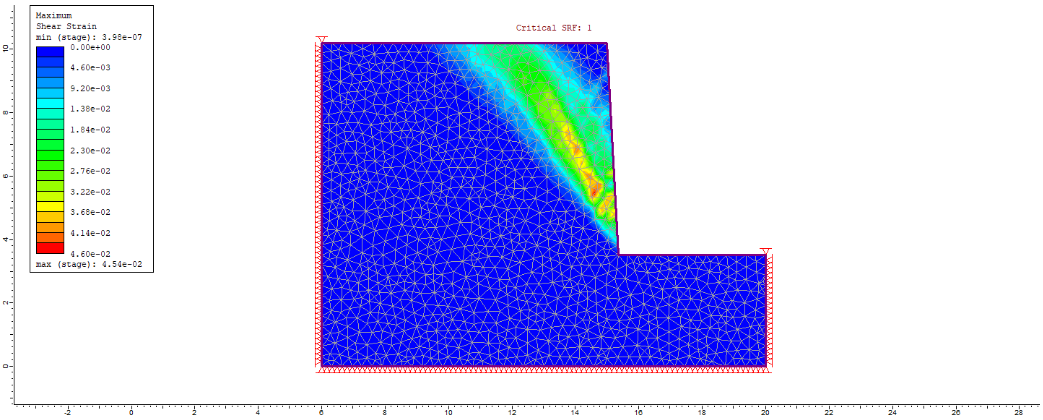

Figure 3. Result of the numerical analysis using the finite element method, based on the average data of the geotechnical properties and parameters.

3.5 Results of the sensitivity analysis for the uniaxial compressive strength

The uniaxial compressive strength of the rock on the slope is 120.42 MPa. According to the Deere and Miller abacus, the admissible variability for this data is ± 45 MPa. Consequently, for the average strength obtained, a sensitivity range of ±45 was assumed in relation to the value of 120, with increments of 15. Upon reducing the compressive strength of the rock to the lowest sensitivity value (75 MPa), the safety factor decreased to 0.97. Conversely, by increasing the resistance to the highest sensitivity value (165 MPa), the safety factor remained unchanged at the average value of 1. The results indicate that a reduction in the rock's compressive strength is accompanied by a reduction in the safety factor. However, when the strength is increased, the safety factor remains unchanged. This is likely due to the fact that the slope has high mechanical strength, which characterizes it as a competent mass. Furthermore, the potential failure surface converges on a wide range of higher strength values. In this case, the compressive strength exerts a minimal influence on the safety factor, with the numerical value of the safety factor exhibiting only a slight reduction as the rock strength value declines. Furthermore, the sensitivity interval adopted for the strength has a negligible impact on the numerical safety factor of the slope evaluated. Table 2 presents the detailed results of the sensitivity analysis for the intervals adopted.

Table 2. Results of the sensitivity analysis for uniaxial compressive strength.

UCS (MPa)	75	90	105	120	135	150	165
SF	0.97	0.98	0.99	0.98	1	1	1

3.6 Result of the sensitivity analysis for the GSI

The GSI for the rock mass present on the slope is approximately 23. Consequently, for this average GSI value, obtained within a range between 15 and 30, this same range was considered to be the sensitivity range for carrying out this analysis, with increments of 3. By reducing the GSI to the lowest sensitivity value (15), the safety factor decreased to 0.96. Conversely, an increase in the GSI to the highest sensitivity value (30) resulted in an increase in the safety factor to 1.02. The results indicate that a reduction in the GSI of the rock, which corresponds to a lower quality of the mass, also results in a decrease in the safety factor. Conversely, an increase in the GSI, which is indicative of a superior quality mass, also results in an elevated safety factor. In this instance, the GSI exerts an influence on the safety factor, which may either increase or decrease in accordance with the quality of the mass. The sensitivity interval adopted for the GSI has a relatively minor impact on the numerical safety factor of the slope evaluated. Table 3 presents the detailed results of the sensitivity analysis for the intervals adopted.

Table 3. Results of the sensitivity analysis for the GSI.

GSI	15	18	21	24	27	30
SF	0.96	0.98	0.99	1	1.01	1.02

3.7 Result of sensitivity analysis for the material constant

The material constant for the rock present on the slope is 28. Consequently, for this average value of the material constant, obtained within a range between 23 and 33, this same range was considered to be the sensitivity range for carrying out this analysis, with increments of 2. By reducing the value of the material constant to the lowest sensitivity value (23), the safety factor remained the same as the factor with average data, which is 1. Conversely, by increasing the constant to the highest sensitivity value (33), the safety factor decreased to 0.99. The results demonstrate that a reduction in the material constant does not affect the safety factor. Conversely, an increase in the constant resulted in a reduction in the safety factor, which remained constant. Moreover, these results suggest that the slope's high mechanical resistance, which characterizes it as a competent mass, may be responsible for the observed phenomenon. The potential failure surface appears to converge to a wide range of smaller and larger values of the constant. In this instance, the material constant exerts a minimal influence on the numerical safety factor of the slope under evaluation. Table 4 presents the detailed results of the sensitivity analysis for the intervals adopted.

Table 4. Results of the sensitivity analysis for the material constant.

m_i	23	25	27	29	31	33
SF	1	0.99	0.99	0.99	0.99	0.99

3.8 Result of the sensitivity analysis for the disturbance factor

The disturbance factor for the rock mass present on the slope is 1. Therefore, for this average value of the disturbance factor, a range between 0.7 and 1 was assumed for the purposes of this analysis, encompassing a mass condition with minimal disturbance and a highly disturbed mass. The range was divided into increments of 0.1. Upon reducing the value of the disturbance factor to its lowest sensitivity value (0.7), the safety factor increased to 1.03. Conversely, when the disturbance factor was increased to its highest sensitivity value (1), the safety factor remained equal to the average value, which is 1. These results indicate that when the disturbance factor is decreased, which corresponds to a low disturbance condition, the safety factor increases. Conversely, when the disturbance factor was increased, which is equivalent to a disturbance condition, the safety factor remained unchanged. In this instance, the disturbance factor exerts a relatively minor influence on the numerical safety factor of the slope under evaluation. Table 5 presents the detailed results of the sensitivity analysis for the intervals adopted.

Table 5. Results of the sensitivity analysis for the disturbance factor.

D	0.7	0.8	0.9	1
SF	1.03	1.01	1	1

3.9 Result of the sensitivity analysis for the modulus of elasticity

The modulus of elasticity of the rock on the slope is 51.54 GPa. According to the Deere and Miller abacus, the admissible variability for this data is ±15 GPa. Consequently, for the average modulus of elasticity obtained, a sensitivity range of ±15 was assumed in relation to the value of 50, with increments of 5. By reducing the value of the modulus of elasticity to the lowest sensitivity value (35 GPa), the safety factor remained equal to the factor with average data, which is 1. Conversely, by increasing the value of the modulus of elasticity to the highest sensitivity value (65 GPa), the safety factor also remained equal to 1. These results indicate that by varying the modulus of elasticity of the rock, the safety factor remains unchanged. In this instance, the modulus of elasticity exerts no influence on the numerical safety factor of the slope under evaluation. Table 6 presents the detailed results of the sensitivity analysis for the intervals adopted.

Table 6. Results of the sensitivity analysis for the modulus of elasticity.

E (GPa)	35	40	45	50	55	60	65
SF	1	1	1	1	1	1	1

3.10 Result for sensitivity analysis of Poisson's ratio

The Poisson's ratio of the rock mass on the slope is 0.25. For this coefficient, a sensitivity range was assumed, varying from high hardness and high mass quality (0.20) to soft hardness and very poor mass quality (0.30), with increments of 0.02. By reducing the value of the Poisson's ratio to the lowest sensitivity value (0.20), the safety factor remained equal to the factor with average data, which is 1. Conversely, by increasing the value of the Poisson's ratio to the highest sensitivity value (0.30), the safety factor also remained equal to 1. These results indicate that by varying the Poisson's ratio of the rock mass, the safety factor does not change. In this instance, the Poisson's ratio exerts no influence on the numerical safety factor of the slope under evaluation. Table 7 presents the detailed results of the sensitivity analysis for the intervals adopted.

Table 7. Result of the sensitivity analysis for Poisson's ratio.

υ	0.20	0.22	0.24	0.26	0.28	0.30
SF	1	1	1	1	1	1

3.11 Result of sensitivity analysis for specific weight

The specific weight of the rock on the slope is 0.0246 MN/m^3. For this average specific weight value, a sensitivity range was assumed that covers the typical specific weight range of most rocks, varying between 0.020 MN/m^3 and 0.035 MN/m^3, with increments of 0.003. By reducing the specific weight value to the lowest sensitivity value (0.020 MN/m^3), the safety factor remained unchanged at 1, as it did when using the average data. Conversely, by increasing the specific weight to the highest sensitivity value (0.035 MN/m^3), the safety factor decreased to 0.98. These results demonstrate that a reduction in the specific weight does not affect the safety factor. Conversely, when the specific weight was increased, the safety factor exhibited a minimal decline. These outcomes suggest that the slope's high mechanical strength, which characterizes it as a competent mass, may be responsible for this phenomenon. The potential failure surface appears to converge on a wide range of higher specific weight values. In this instance, the specific weight exerts a negligible influence on the numerical safety factor of the slope under evaluation. Table 8 presents the detailed results of the sensitivity analysis for the intervals adopted.

Table 8. Results of the sensitivity analysis for specific weight.

γ (MN/m^3)	0.020	0.023	0.026	0.029	0.032	0.035
SF	1	1	0.99	0.99	0.98	0.98

4 CONCLUSIONS

Geotechnical variability is a complex attribute that arises from a multitude of sources of uncertainty pertaining to the properties and parameters of materials, including soils and rocks. However, slope stability analysis enables the determination of the safe height, slope, and/or depth for an excavation. The slope in question is on the verge of collapse, with a numerical safety factor equal to 1 for the average data of the properties and parameters of the rock mass, considering the simplification of the model as a continuous medium. The potential failure zone of the slope is represented by a plane, non-circular surface, with the probability of failure occurring in this area being high.

The results of the sensitivity analysis indicate that the uniaxial compressive strength, GSI, material constant, disturbance factor, and specific weight all exert some influence on the numerical safety factor. This factor is sensitive to increases in value when conditions lead to improved slope quality and decreases in value when conditions lead to worse slope quality. Among the parameters examined, only the GSI demonstrated a significant influence on the variation of the factor, while the others exhibited either minimal or no impact on the numerical safety factor for the sensitivity intervals adopted. These unexpected results may be related to the properties and parameters of the rock mass, which exhibits high mechanical strength and is therefore classified as a competent mass. The potential failure surface formed is converging on a wide range of values in these data, resulting in a low sensitivity of the numerical safety factor within the ranges considered.

With regard to the elastic modulus and Poisson's ratio, there was no impact on the numerical safety factor, which was found to be relatively insensitive to the variability of the intervals considered for these data. This result was anticipated, given that these two data sets are employed to model the stress-strain behavior in the slope section, with a significant impact on the scale of tensile and compressive areas in the potential failure zone. Despite the specific characteristics of the study, it was possible to identify the properties and parameters that influence the sensitivity or insensitivity of the numerical safety factor.

The selected slope exhibited high mechanical competence, resulting in a negligible or insignificant impact on the sensitivity of the numerical safety factor. An application to low-competence slopes could expand the comprehension of the impact of these sensitive and insensitive properties and parameters, as demonstrated in this study. Consequently, it can be posited that the incorporation of sensitivity analysis represents a valuable addition to the numerical analysis of rock slope stability. The safety factor is susceptible to alterations in the values of the principal properties and parameters of the rock mass. By employing this methodology, it is possible to ascertain the actual stability conditions with greater precision, thereby enabling the certification of the true state of stability of the rock slope with greater reliability.

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