Stability analysis of road slopes in central Portugal

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Abstract: This paper aims to investigate slope stability related problems and the evaluation of stability in road slopes in central Portugal. The surveyed road slopes considered here are situated either along the highway IP3, the road access IC6, or the route EN234. The geological foundations of these roadcuts are the Beiras Schists and the Ordovician - Silurian Buçaco syncline.

Whenever road and railway networks are expanded, particularly when it is necessary to excavate, but also during their lifetimes, the existing conditions are modified and the stability risks may increase. The surveying of the existing rock slopes showed the occurrence of some instability phenomena such as wedge and translational failure, rock and mass fall and toppling. It should be remarked that the most frequent slope problems are related to wedge failure, toppling and translational landsliding. The rotational failures observed in soil and rock slopes are preferentially/mainly located in domains that are highly weathered or have densely spaced discontinuities with random orientations.

The evaluation of stability in the different sections of the herein envisaged slopes was based on deterministic and probabilistic analysis that took into account the values of the rock slope properties and the shear strengths of the surveyed discontinuities. For the determination of the values that characterise the safety factors and also for the evaluation of failure probability, the Rocscience (2001) software packages were used. The Monte Carlo simulation method applied to standard limiting equilibrium procedures was utilized because the main parameters that influence slope stability showed a considerable variation.

Résumé: Cet article porte sur l'étude des problèmes de stabilité des pentes de routes au Portugal central et leur évaluation. Les pentes examinées sont situées le long des voies de communication IP3, IC6 et EN234.

Les pentes des voies de communication sont constituées par des formations des schistes de Beiras et du synclinal Ordovicien-Silurien de Buçaco.

Toutes les fois que les réseaux routiers et ferroviaires sont augmentées, en particulier quand il est nécessaire d'excaver, mais également pendant leur durée de vie, les conditions préexistantes se modifient et les risques de stabilité augmentent. L'étude des pentes rocheuses a montré l'occurrence de certains phénomènes d'instabilité comme le glissement dièdre, le glissement plan, des chutes de blocs et sol et basculements. Il faut remarquer que les problèmes plus fréquents des pentes étudiées sont liés aux basculements et aux glissements plans et dièdres. Les glissements rotationnels observés dans les pentes rocheuses sont préférentiellement situés dans domaines qui sont fortement altérés ou où l'espacement des discontinuités avec des orientations aléatoires est très réduit.

L'évaluation de la stabilité dans les différentes sections des pentes étudiées s'est basée sur l'analyse déterministe et probabiliste qui a tenu compte des valeurs des propriétés des matériaux rocheux et des résistances au cisaillement des joints. Les programmes informatiques de Rocscience ont été utilisés pour définir les valeurs qui caractérisent les coefficients de sécurité et également pour évaluer la probabilité de rupture. La méthode de Monte Carlo appliquée aux procédures de l'équilibre-limit a été utilisée car les paramètres principaux qui déterminent la stabilité des pentes montrent une variation considérable.

Keywords: discontinuities, friction angle, metamorphic rock, slope stability, rock mechanics, safety.

INTRODUCTION

The authors carried out a study of rock slopes located in highways IP3, IC6 and EN234 in Central Portugal. The slopes are situated in the "Xisto-Grauváquico" Complex which has more recently been designated as "Dúrico-Beirão Supergroup" of Vendian-Cambric age. Slope 1 is situated at the junction of IP3 with IC6 and slopes 2, 3 and 4 are situated on the EN234.

The analysed slopes mainly consist of phyllites intercalated by meta-graywakes and more rarely by quartz veins. The analysis of stability problems was based on deterministic and probabilistic methods. For each section of the analysed slopes, the most frequent or the most critical situations of instability were adopted.

Rocscience (2001) software programs, such as RocPlane (version 2.0) and Swedge (version 4.0), were used to study planar failure and wedge failure.

In order to analyse rock slope stability it must first be established whether the orientation of the discontinuities can cause stability problems; in case the kinematics analysis determines the studied situation is potentially unstable, the slope analysis must be based on deterministic and probabilistic methods.

DETERMINISTIC ANALYSIS OF PLANAR FAILURE

The planar failure study of slopes 2, 3 and 4 was based on a deterministic analysis. The Factor of safety (FS) is usually established for evaluating the adequacy of design. It was obtained through the equilibrium limit method. The study consisted essentially in analyses of a rock mass limited by discontinuities without considering deformations. The observed slopes are structures located in the upper part of rock masses where deformability does not constitute a preponderant feature.

The FS values were obtained from equations of horizontal and vertical forces, according to the model developed by Hoek & Bray (1981). Norrish & Wyllie (1996) referred that the design of highway slopes should consider the following typical FS values (Table 1):

Table 1. FS values for highway slopes

Factor of safety	Slope type
1.5 - 2.0	Permanent structure
1.25 - 1.5	Temporary structure

The FS values were also classified in accordance with the values of Table 2.

Table 2. Highways slope stability classification

Factor of safety	Slope stability
> 1.5	Stable
1.25 - 1.5	Intermediate stability
1 - 1.25	Precarious stability
< 1	Instable

The Mohr-Coulomb model was used to determine the FS for planar failure occurrence; the friction angle (ϕ) values for discontinuities were obtained from rock joint shear strength tests under low normal stresses (20 to 200 kPa) and are residual shear strength related (Muralha & Santarém Andrade 2000); the obtained values of the apparent cohesion were nonexistent or low and they were considered null. Rock joint shear strength tests under low normal stresses reproduce as closely as possible the features that occur in the upper parts of rock masses where the slopes are located.

The deterministic and probabilistic analyses must consider the effect of water on discontinuities.

Several situations were considered in the deterministic calculation of the FS: a) water absent on discontinuities; b) possible decrease of the value of ϕ, around 15% and 20%, due to the increase of the water content on discontinuities surfaces; c) presence of water pressure, a situation where the water height is located at 40% of the tension crack (TC) height and the water pressure peak is at the TC base.

In addition to establishing the FS based on the Mohr-Coulomb model, the authors also used the Barton & Bandis model (1990), which uses JRC (joint roughness coefficient) and JCS (joint compressive strength) values of the studied discontinuities to determine the FS.

The JRC and JCS values tend to diminish with the increase of the discontinuities persistence. The Barton & Bandis (1990) approach can be corrected in terms of scale effect on the FS establishment by the expressions defined by Bandis, Lumsden & Barton (1981).

The software program Rocplane (version 2.0) was used to determine planar failure. The discontinuities in slopes 2, 3 and 4 that define a planar failure were the TC (with dip greater than 70º and parallel to schistose cleavage) and the sliding planes with dip less than 45º.

For the different conditions of the influence of water on discontinuities (Tables 3, 4 and 5 and Figure 2), it is possible to verify a decrease of the FS values in results that considered the influence of water.

To analyse slope 2's FS, the ϕ values used were between 25º and 30º. The slope 2 FS values for the reductions of 15% and 20% of ϕ vary from 1.31 to 1.00. As a result, slope stability can be classified from intermediate to precarious, according to Table 2. The slope 2 FS values can be lower than 1. These last results define an unstable situation where the presence of water pressure on TC is considered.

To analyse slope 3's FS, the ϕ values used were between 27.2º and 33.1º. The slope 3 FS values that were defined according to the Mohr-coulomb model (Table 4) are lower than 1.5. FS values between 1.25 and 1.50 (intermedial stability) have been defined for discontinuities without water presence and with a maximum or mean ϕ. The slope 3 FS values are lower than 1 for discontinuities showing influence of water and a minimum ϕ.

To analyse slope 4's FS, the ϕ values used were between 24.2º and 28.4º. Almost all the FS values for slope 4 are below the unitary value and in this case they define an instability situation. The only FS value that is superior to 1 corresponds to a situation showing absence of water and a maximum friction angle. The FS results are reduced by the influence or presence of water on discontinuities.

The decrease of the slope 4 FS values comparatively to the slope 3 FS values can be explained by a higher dip of discontinuities and by the lower friction angle values.

The FS values have been determined according to the Barton-Bandis model without and with the scale effect (Tables 6, 7 and 8).

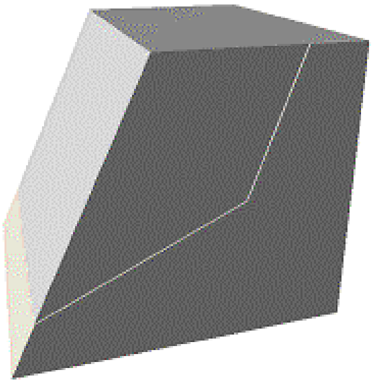

Figure 1. Simplified graphic representation of the possible slope 2 planar failure; the slope section height is about 3.0 meters

Table 3. FS values related with planar failure of slope 2

Slope 2 planar failure	Factor of safety		
	Maximum ϕ	**Minimum ϕ**	**Mean ϕ**
Water absent	1.59	1.28	1.54
ϕ reduction of 15%	1.31	1.07	1.27
ϕ reduction of 20%	1.22	1.00	1.19
40% of the tension crack filled with water	0.82	0.66	0.72

Table 4. FS values related with planar failure of slope 3

Slope 3 planar failure	Factor of safety		
	Maximum ϕ	**Minimum ϕ**	**Mean ϕ**
Water absent	1.40	1.10	1.31
ϕ reduction of 15%	1.15	0.92	1.08
ϕ reduction of 20%	1.07	0.86	1.00
40% of the tension crack filled with water	1.18	0.93	1.10

Table 5. FS values related with planar failure of slope 4

Slope 4 planar failure	Factor of safety		
	Maximum ϕ	**Minimum ϕ**	**Mean ϕ**
Water absent	1.02	0.84	0.96
ϕ reduction of 15%	0.84	0.71	0.80
ϕ reduction of 20%	0.79	0.66	0.74
40% of the tension crack filled with water	0.87	0.71	0.82

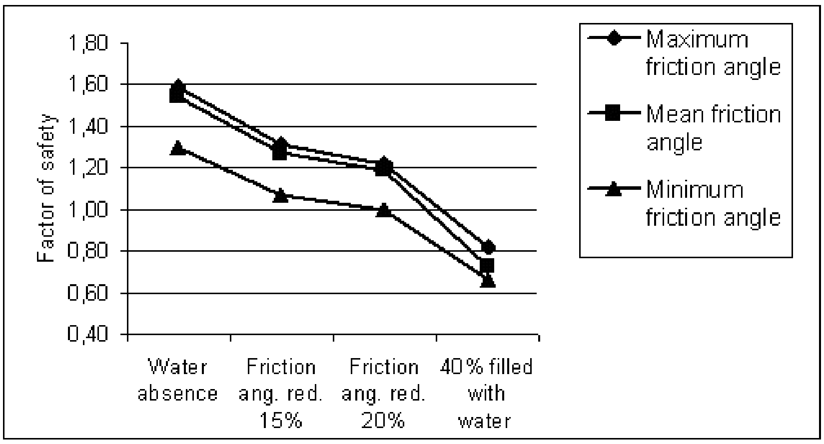

Figure 2. Variation of the FS values of slope 2 for different water conditions and friction angles

Table 6. FS values determined according to the Barton-Bandis model on slope 2

Slope 2 planar failure	**Factor of safety**		
	***JRC* maximum results (JRC=6)**	***JRC* minimum results (JRC=2)**	***JRC* average results (JRC=4)**
Water absent (without scale effect correction)	2.09	1.44	1.75
40% of the tension crack filled with water (without scale effect correction)	0.92	0.58	0.76
Water absent (with scale effect correction)	1.63	1.34	1.49
40% of the tension crack filled with water (with scale effect correction)	0.71	0.54	0.64

Table 7. FS values determined according to the Barton-Bandis model on slope 3

Slope 3 planar failure	**Factor of safety**		
	***JRC* maximum results (JRC=6)**	***JRC* minimum results (JRC=2)**	***JRC* average results (JRC=4)**
Water absent (without scale effect correction)	2.02	1.25	1.61
40% of the tension crack filled with water (without scale effect correction)	1.72	1.03	1.37
Water absent (with scale effect correction)	1.23	1.08	1.16
40% of the tension crack filled with water (with scale effect correction)	1.05	0.89	0.99

Table 8. FS values determined according to the Barton-Bandis model on slope 4

Slope 4 planar failure	**Factor of safety**		
	***JRC* maximum results (JRC=6)**	***JRC* minimum results (JRC=2)**	***JRC* average results (JRC=4)**
Water absence (without scale effect correction)	1.80	1.05	1.43
40% of the tension crack filled with water (without scale effect correction)	1.56	0.89	1.24
Water absence (with scale effect correction)	1.12	0.92	1.06
40% of the tension crack filled with water (with scale effect correction)	0.97	0.77	0.91

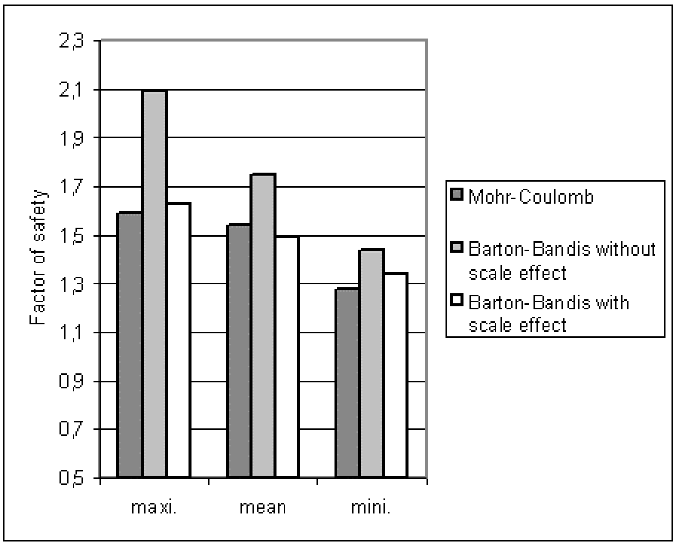

Figure 3. FS values of slope 3 obtained through the Mohr-Coulomb model and the Barton-Bandis (1990) model without and with scale effect

The discontinuities JRC values of slopes 2, 3 and 4 vary between 2 and 6. The JRC and JCS values were established by the geomechanical characterization of rock masses.

The FS values for slope 2, defined according to the Barton-Bandis method, are clearly inferior to 1 when 40% of the TC filled with water is considered.

The Tables 3 to 8 and the Figure 3 show that the use of the Barton-Bandis method without scale effect correction supplies FS values clearly superior to the values defined by the Mohr-Coulomb method. By contrast, the FS results defined by the Barton-Bandis method with scale effect correction are similar to the FS values determined by the Mohr-Coulomb model.

DETERMINISTIC ANALYSIS OF WEDGE FAILURE

Rocscience's (2001) Swedge (version 4.0) software program incorporating an analytical program developed by Hoek & Bray (1981) was used to obtain FS on slopes where wedge failure might exist. The aim was to analyse all factors leading to wedge failure: the orientation of discontinuities, the ϕ and cohesion parameters, the weight of the rock wedge, the water pressure and the presence and influence of a TC.

The presence of water in discontinuities is a random parameter, depending on rainfall and time of year. As such, it is more difficult to determine or predict. In analysing the wedge failure of slope 1, apparent cohesion was taken to be zero, and we used the ϕ ascertained from the residual shear strength. The latter was determined by means of rock joint shear tests, using normal stresses of between 100 and 500 kPa.

To analyse wedge failure in slope 1 we looked at two discontinuity surfaces, with orientations of (80º, 135º) and (85º, 290º). To analyse slope 1's FS, the ϕ values used were between 34.0º and 26.9º.

Table 9. FS values related with wedge failure of the slope 1

Wedge failure of the slope 1	Factor of safety		
	Maximum ϕ	**Minimum ϕ**	**Mean ϕ**
Water absent	1.64	1.23	1.45
15% reduction in the ϕ	1.34	1.02	1.19
20% reduction in the ϕ	1.25	0.96	1.11
50% of the discontinuities filled with water	1.33	1.00	1.17
75% of the discontinuities filled with water	0.59	0.44	0.52

The FS of slope 1 obtained for the maximum and mean ϕ are greater than 1.1, except for the analysis that took account of discontinuities with about 75% of their height filled with water, for which the FS were <1.

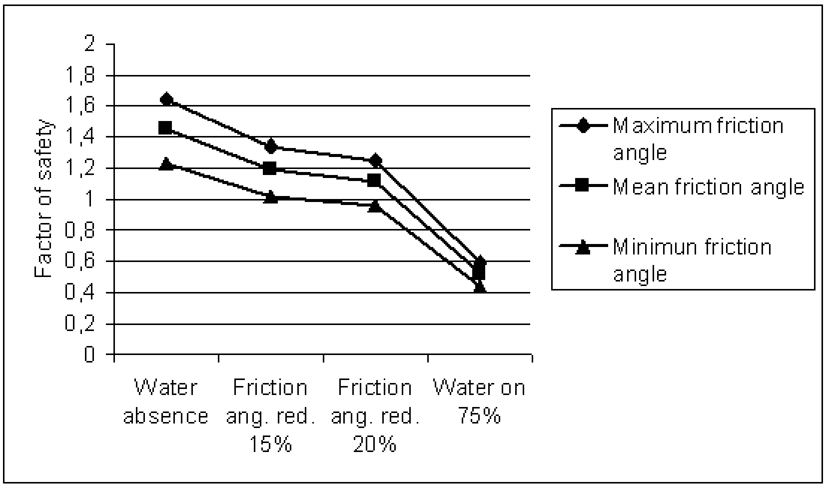

Figure 5. Variation of FS of the slope 1 for different conditions of water and friction angle

On slope 1, it was noted that FS figures lower than 1 relate to the development of water pressures, resulting in wedge failure. The 15% to 20% reduction in the ϕ did significantly reduce the FS results, but in the vast majority of cases this was not enough to bring the figures below 1.

PROBABILISTIC ANALYSIS OF PLANAR FAILURE

The rock material and discontinuity surfaces that appear on most of the slopes are highly variable, and possibly cannot be precisely understood, and so probabilistic methods are required (Mostyn & Small, 1987). Probabilistic study complements deterministic analysis. Probability is linked to events which, individually, are uncertain, but which are predictable in greater numbers.

In contrast to deterministic analysis, probabilistic analysis looks at the uncertainties and variations in the parameters being examined, given that most of them, which can be classed as geometric or mechanical, incorporate some degree of uncertainty and are not constant. For example, the orientations of discontinuities are usually variable, even on surfaces of the same family, and shear strength values, such as the ϕ, also fall within a variation range. The presence of water produce significant changes, and it is also worth bearing in mind the wide-ranging results of laboratory and “in-situ” tests. Variations can also occur depending on the analysis method employed (Park & West 2001).

The results of probabilistic methods are defined according to failure probability, whereas deterministic methods only take account of FS values.

As well as failure probability, safety can also be expressed using the reliability index (Christian, Ladd & Baecher 1994):

$$RI = (m(FS) - 1) / sd(FS)$$

where RI is the reliability index, m(FS) is the mean factor of safety and sd(FS) is the standard deviation of the factor of safety.

The reliability index considers the degree of uncertainty in determining the FS. Negative reliability-index figures indicate slope failures. Zero indicates the stability limit, and positive figures indicate stability.

To analyse the probability of planar failure and wedge failure we used the Monte Carlo simulation, which is one of the main probabilistic methods used in geotechnics, and which in this study is linked to the limit equilibrium. The Monte Carlo simulation is a numerical method which uses random sampling to analyse deterministic mathematical situations, such as the limit equilibrium method. Using the RocPlane (version 2.0) and Swedge (version 4.0) software programs, we introduced 1000 samples and obtained the same number of FS generated from the random values of the parameters considered. For each of these FS, we checked whether the equation representing the limit condition was satisfied or not. By conducting numerous iterations of the calculation, the probability of failure can be estimated, based on the proportion of results for which the FS are less than 1.

To classify the failure probability values obtained, we adopted the proposal of Santamarina, Altschaeffl & Chameau (1992), shown in Table 10.

The type of distribution obtained according to the Monte Carlo method was checked using the Kolmogorov-Smirnov (K-S) method.

Probabilistic analysis of planar failure was carried out using the RocPlane (version 2.0) program, for the same slope sections that underwent deterministic analysis. As such, the geometrical and mechanical parameters refer to the same rock materials and discontinuity surfaces.

Table 10. Criterion for slope probability of failure

Conditions	Criterion for probability of failure	Criterion for probability of failure (%)
Temporary Structures with low repair cost	0.10	10
Highway slopes	0.01	1
Acceptable in most cases except if lives may be lost	0.001	0.1
Acceptable for all slopes	0.0001	0.01

For the probabilistic analysis of planar failure on slopes 2, 3 and 4 we used the two-dimensional planar failure model developed by Hoek & Bray (1981), and to define the relationship between normal and shear stresses we used the Mohr-Coulomb model.

The parameters taken as variable in the probabilistic analysis were: the slope orientation, the failure surface orientation and the TC orientation, as well as the bulk density of the rock material, the ϕ of the discontinuities and the influence of water. We established the average values, the dispersal, and the statistical distribution of each of these parameters. The parameters used show normal statistical distribution, and constitute independent and random variables.

The percentage of water on cracks, and the consequent pressure, was assumed to be distributed normally, and also in the form of triangular distribution. The latter type of distribution is often used for water on discontinuities, as the values for such water are often unknown, highly variable, or even deemed to be random. In the water pressure model, it was considered that its peak value occurs at the base of the TC.

For shear strength figures, cohesion was taken as zero, which accords with the procedures followed for the deterministic analysis.

For the probabilistic analysis of planar failure on slopes 2, 3 and 4, the study was carried out for models of the mean ϕ values of discontinuities without water, for a 15% and 20% reduction in mean ϕ, and for a situation in which 40% of the TC's height is filled with water.

For the probabilistic analysis of planar failure in the area of slope 2 studied, it was deemed that the parameters that showed variation were the slope dip, the bulk density of the rock material, the failure surface dip, the mean ϕ of the discontinuities, and the TC dip. The standard deviation values of the parameters are connected to data obtained *in situ* and to values from laboratory tests.

TC with 40% of their height filled with water were established for two types of situation: normal statistical distribution with a standard deviation of 2%, and triangular statistical distribution with a minimum value of 20% and a maximum of 60%.

For the rock material bulk density a value of 21.73 kN/m^3 was obtained and used for the phyllites. These are the predominant rock materials in the area of slope 2. The degree of weathering of the rock material was also considered. The mean ϕ of discontinuities is 29.3º.

Five different types of simulations were made for discontinuities: a) without water, b) with a 15% reduction in the ϕ, c) with a 20% reduction in the ϕ, d) TC with 40% of its height filled with water for normal distribution, and e) TC with 40% of its height filled with water for triangular distribution. The only external force applied is water.

Table 11 showed that the FS obtained using the deterministic method are very similar to those for the probabilistic method, and sometimes identical.

The results of the different iterations of the Monte Carlo simulation and the resulting FS can be shown as histograms (Figure 6).

The reliability indices in Table 11 were positive where there was no water on the discontinuities, and where the mean ϕ of discontinuities was reduced by 15% and 20%, which matches the values for the probabilistic analysis of FS. For the model where the TC has 40% of its height filled with water, two statistical distribution situations were considered, and it was seen that the corresponding reliability indices are clearly negative. This implies instability in the area of slope 2 studied. The negative values of the reliability index for the model where the TC has 40% of its height filled with water, for normal and triangular distribution, are quite different, even though the probabilistic FS are similar. The reliability index results enable us to differentiate situations with different standard deviations, and produce an index closer to the probability of failure than FS.

Table 11. Slope 2 deterministic safety factors and probabilistic results

Discontinuities models	Deterministic safety factors	Probabilistic safety factors	Standard deviation of the probabilistic safety factor	Probability of failure (%)	Reliability index
Water absence	1.54	1.56	0.22	0.00 %	2.55
Mean ϕ reduction of 15%	1.27	1.29	0.19	4.97%	1.53
Mean ϕ reduction of 20%	1.19	1.20	0.18	11.56%	1.11
40% of the tension crack filled with water (normal distribution)	0.72	0.72	0.12	98.39%	-2.33
40% of the tension crack filled with water (triangular distribution)	-	0.74	0.23	86.00%	-1.13

The K-S method showed that the various iterations of the Monte Carlo method for the model for discontinuities with a 20% reduction in the mean ϕ have lognormal distribution, with a probability of 95%.

Distribution adjustment using the K-S method also enabled us to assume lognormal distribution for the results of the probabilistic analysis for discontinuities without water, with a 15% reduction in mean ϕ, and with the TC having 40% its height filled with water (normal distribution), whereas for the model where the TC has 40% of its height filled with water (triangular distribution) Beta distribution was assumed.

Table 11 and Figure 6 revealed that the probabilities of planar failure differ for the situations examined. It was clear to see that the probability of failure increases as the influence of water on discontinuities increases.

The probabilistic FS where there is no water on the discontinuities is 1.56, and its failure probability is zero. Where TC have 40% of their height filled with water (normal distribution), the FS falls to about 0.72, and failure probability is extremely high: about 98.39%.

The probability of failure for TC with 40% of their height filled with water (triangular distribution) is lower than that noted for TC with 40% of their height filled with water (normal distribution) (Figure 6), the former model showing higher dispersal results.

It was observed that the probability of failure identified for the model in which there was a 15% reduction in the mean ϕ of discontinuities is greater than the values recommended for highways such as the EN234, whereas the values for the probability of failure where there is a 20% reduction in the mean ϕ of discontinuities, and in TC with 40% of their height filled with water (normal and triangular distribution) are higher than the recommended values for temporary structures.

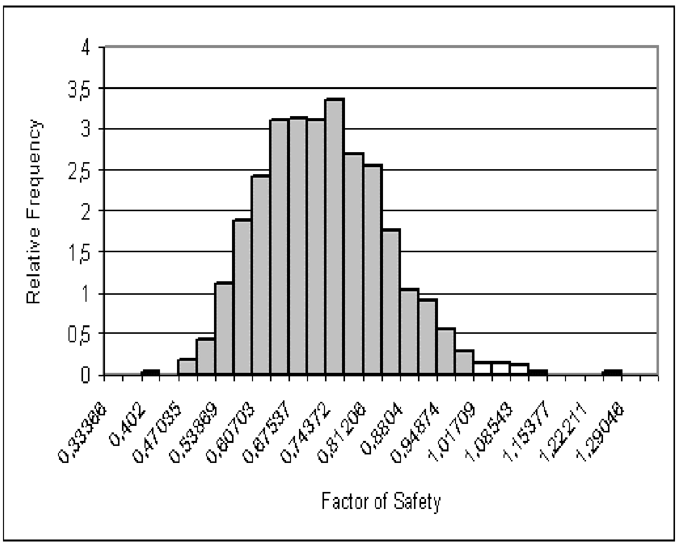

Figure 6. Distribution of the slope 2 factor of safety histogram. The factor of safety was determined from the mean ϕ and 40% of the TC filled with water (normal distribution). The grey bars at the left of the distribution represent situations with factor of safety less than 1.0

In the probabilistic analysis of planar failure of the area of slope 3 studied, the variable parameters considered were: the mean ϕ of discontinuities, the rock material bulk density and the dip of the slope, the failure surface dip and the TC dip. The distribution of these parameters was assumed to be normal.

For the bulk density of the phyllites, which are the predominant rock materials in the area of slope 3 studied, we obtained a value of 23.5 kN/m^3. The mean ϕ of discontinuities is 31.3º.

It was assumed that TC have 40% of their height filled with water, and that there are two types of statistical distribution for their values: Gauss distribution where the standard deviation is 2%, and triangular distribution, with a minimum value of 0% and a maximum of 100%.

Table 12 shows that the FS obtained using the deterministic method are similar to the FS using the probabilistic method, which is linked to the use of limit equilibrium equations in both methods.

For models where TC have 40% of their height filled with water (normal and triangular distribution), the results for FS using probabilistic analysis are virtually the same, which is not the case with reliability indices and failure probabilities. The difference is linked to the spread of the results of the probabilistic analyses carried out.

Table 12. Slope 3 deterministic FS and probabilistic results

Discontinuities models	Deterministic safety factors	Probabilistic safety factors	Standard deviation of the probabilistic safety factor	Probability of failure (%)	Reliability index
Water absent	1.31	1.31	0.16	0.8	1.94
Mean ϕ reduction of 15%	1.08	1.08	0.14	28.7	0.57
Mean ϕ reduction of 20%	1.00	1.01	0.13	49.9	0.08
40% of the tension crack filled with water (normal distribution)	1.10	1.11	0.13	21.5	0.85
40% of the tension crack filled with water (triangular distribution)	-	1.10	0.19	31.4	0.53

The Monte Carlo simulation results for discontinuity models without water, with a 15% and 20% reduction in the mean ϕ of discontinuities, show lognormal distribution for 95% probability, which was verified via the adjustment effected using the K-S method.

With the K-S method, the various results of the Monte Carlo simulation for TC with 40% of their height filled with water, with normal distribution, were better adapted to Beta distribution, whereas the results of TC with 40% of their height filled with water with triangular distribution correspond to normal distribution.

Probabilistic analysis of planar failure in the area of slope 3 studied shows considerable result variability. The situations with the greatest possibility of failure were for the model where the mean ϕ of discontinuities was reduced by about 20%, in which the probability of failure was about 49.9%.

In the probabilistic analysis carried out of discontinuities without water, the probability of failure is virtually zero (0.8%). In line with the classification in Table 8, the results for all situations studied, except for the discontinuities model without water, are not good enough for temporary slopes, and even less for permanent slopes on highways such as the EN234.

For the probabilistic analysis of planar failure in the area of slope 4 studied, it was deemed that the parameters that showed variation were the slope dip, the bulk density of the rock material, the failure surface dip, the mean ϕ of the discontinuities, and the TC dip.

For the area studied of the slope 4, the bulk density of the rock material comprising phyllites is 22.3 kN/m^3 and the mean ϕ of discontinuities is 27º. It was assumed that TC had 40% of their height filled with water. For the statistical distribution of their values, two situations were looked at: normal distribution with a standard deviation of 2%, and triangular distribution with a minimum value of 0% and a maximum of 100%.

Table 13. Slope 4 deterministic safety factors and probabilistic results

Discontinuities models	Deterministic safety factors	Probabilistic safety factors	Standard deviation of the probabilistic safety factor	Probability of failure (%)	Reliability index
Water absent	0.96	0.96	0.11	67.1	-0.36
Mean ϕ reduction of 15%	0.80	0.80	0.10	97.0	-2.00
Mean ϕ reduction of 20%	0.74	0.75	0.09	99.1	-2.78
40% of the tension crack filled with water (normal distribution)	0.82	0.83	0.09	96.0	-1.89
40% of the tension crack filled with water (triangular distribution)	-	0.81	0.14	91.5	-1.36

The results of the FS established for deterministic analysis are virtually the same as the FS obtained via the probabilistic method, and produce figures of less than 1 (Table 13), with indications of instability, which is confirmed by extremely high failure probability values, of over 91%, for discontinuity models with the presence or influence of water. Reliability indices were negative for all of the situations examined, confirming the possibility of instability in the area of slope 4 studied. For TC with 40% of their height filled with water (normal and triangular distribution), the FS values were very close (0.83 and 0.81). The respective values for the reliability index and probability of failure were further apart, which is linked to the standard deviation of the results of the Monte Carlo simulation.

The results of the Monte Carlo simulation for the model in which TC have 40% of their height filled with water (normal distribution) show lognormal distribution for a probability of 95%, according to the K-S method. For probabilistic study of TC with 40% of their height filled with water, with triangular distribution, it was assumed, using the K-S method, that the FS results obtained using the Monte Carlo simulation are better adapted to normal distribution. For the other models, it was assumed, in line with K-S distribution, that the results of the probabilistic analysis were better adapted to lognormal distribution.

The probabilities of planar failure in the area of slope 4 studied are high, even where there is no water on the discontinuities. Using the Santamarina, Altschaeffl & Chameau classification (1992) (Table 10), the results meant that from a safety viewpoint we could not consider the probability of failure to be sufficient for a highway such as the EN234. However, it should be borne in mind that the rock material involved in the planar failure in the area of slope 4 studied does not exceed 3 m^3.

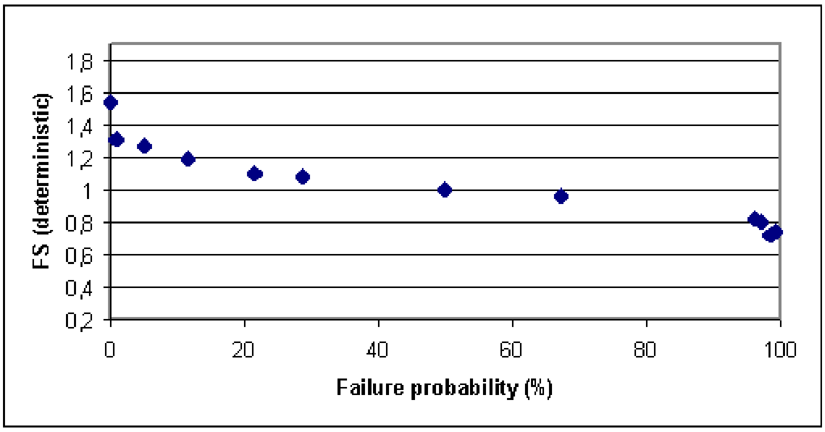

Figure 7. Relation between the FS (deterministic analysis) and the failure probability (%)

Figure 7 shows the relationship between the FS ascertained using deterministic analysis and the failure probabilities obtained using probabilistic analysis.

In the graph in Figure 7, there is an inversely proportional connection between the FS and the failure probabilities, and some concentration of points projected for the pairs of values for failure probabilities of between 95% and 100%, and of FS (deterministic analysis) with values of 0.82 or less. The Spearman's correlation coefficient is -0.994 and is significant at the 0.01 level. The graph in Figure 7 only serves as a reference for the situations studied, and does not serve for extrapolations between the deterministic FS and the probability of planar failure for other slopes comprising rock materials, with different discontinuity shear strength and geometric parameters.

PROBABILISTIC ANALYSIS OF WEDGE FAILURE

Probabilistic methods were used to study wedge failure for slope 1.

The shear strength of discontinuities, the orientations of discontinuities, and the orientations of slope 1 were used as probabilistic parameters, because the values obtained were variable.

The Swedge program (version 4.0) was used for probabilistic analysis of wedge failure. Four models were established: discontinuities with no water; a 15% and 20% reduction in the mean ϕ of discontinuities, and discontinuities with 50% to 100% of the height filled with water. Probabilistic analysis was carried out using the Monte Carlo simulation, as it was with the planar failure probabilistic study.

The Swedge program (version 4.0) does not allow probabilistic analysis in which water pressure is a variable, unlike the RocPlane program (version 2.0).

Geometric and shear strength parameters were deemed to be variables and have normal distribution. The bulk density of phyllites in the area of slope 1 examined is 21.30 kN/m^3. The mean ϕ of failure surfaces is 30.8º.

The FS results of the probabilistic analysis were between 1.1 and 1.5, which can be classed as intermediate to precarious stability.

The results of the probabilistic FS were analysed using the K-S method. For models involving a 15% or 20% reduction in the mean ϕ of discontinuities, it was established that the FS results showed lognormal distribution for a probability of 95%. The best adaptation of the probabilistic analysis results using the K-S method for dry discontinuities corresponds to lognormal distribution. Where discontinuities are filled with water to a height of 50%, it was assumed, again using the K-S method, that the results of the probabilistic FS are related to Beta distribution.

Table 14. Slope 1 deterministic FS and probabilistic results

Discontinuities models	Deterministic safety factors	Probabilistic safety factors	Standard deviation of the Probabilistic safety factor	Probability of failure (%)	Reliability index
Water absent	1.45	1.47	0.36	8.02	1.33
Mean ϕ reduction of 15%	1.19	1.21	0.31	25.50	0.68
Mean ϕ reduction of 20%	1.11	1.13	0.28	35.07	0.46
50% of the discontinuities filled with water	1.17	1.17	0.43	34.23	0.40

According to Table 14, the probability of failure increases with the reduction in the mean ϕ of discontinuities and with the rise in water pressure on the discontinuities. The failure probability values were not acceptable for slopes of highways such as the IP3 and IC6. In a more favourable stability situation, the failure probability was about 8.02%, which may be acceptable for temporary structures, but not for slopes of major highways.

CONCLUSIONS

Deterministic analysis of planar failure and wedge failure in the slopes studied showed that potential instability is made worse by water action, with the possible reduction in ϕ values and by the increase in water pressure on the surfaces of discontinuities.

The FS results obtained using the Barton-Bandis model, without correcting for the scale effect, were higher than those obtained using the Mohr-Coulomb model. The latter were close to those obtained using the Barton-Bandis model with scale effect correction of the JRC and JCS values. Although the values were similar, we do not propose that the determination of FS found using the Mohr-Coulomb model should be replaced by those found using the Barton-Bandis model. Very often the JRC and JCS values are somewhat imprecisely established, and sometimes established qualitatively. However, a preliminary indication of the FS can be gained using the Barton-Bandis model, when corrected for scale.

The parameters used in deterministic methods are often variable. As such, we conducted probabilistic studies of the stability of rock slopes using the Monte Carlo simulation method.

As with the deterministic analysis, when conducting the probabilistic study of planar and wedge failures, we used different models for the presence of water on discontinuities. We noted an increased probability of failure and a reduction in the reliability index in situations involving the influence and presence of water. As such, in some sections of the slopes studied, the probability of failure is extremely high, and the reliability indices are negative. There was an inversely proportional relationship between the values for deterministic FS and the failure probability results, with a Spearman's correlation coefficient of -0.994. The value used as a reference for the probabilistic analysis was the probability of failure. Values for the reliability indices were closer to the probability of failure than the FS values.

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